SCHOOL OF CIVIL ENGINEERING

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-80/6

ADDITIVES TO CONTROL SLAKING IN COMPACTED SHALES

M. Surendra

PURDUE UNIVERSITY
NDIANA STATE HIGHWAY COMMISSION

80-6

Interim Report

ADDITIVES TO CONTROL SLAKING IN COMPACTED SHALES

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

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

May 14, 1980

Project: C-36-5L

File: 6-6-12

Attached is an Interim Report on the HPR Part II Study titled "Design and Construction Guidelines for Shale Embankments". This is Interim Report No. 8 and is entitled "Additives to Control Slaking in Compacted Shales". It is authored by M. Surendra of our staff.

The research determined the effects of various salts and of lime on the durability of several Indiana shales. A number of the salts significantly changed the slake durability of some shales. Lime consistently improved the durability of tested shales.

The Report is submitted for review, comment and acceptance as partial fulfillment of the objectives of the referenced HPR study.

Respectfully submitted,

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Interim Report

ADDITIVES TO CONTROL SLAKING IN COMPACTED SHALES

by

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Project No.: C-36-5L

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in cooperation with the

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and the

U.S. Department of Transportation
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16. Abstract

The conventional manner of stabilization of nondurable shales is through degradation and compaction. However, if the shale is hard, this is extremely difficult to accomplish. This research tested the ability of certain inexpensive salts to modify the durability of selected Indiana shales. If the salt decreased the durability, it would be added prior to shale compaction; if the effect were vice versa, the salt could be added at the time of compaction.

Mostly, the salts tried did not significantly affect the short term durability, although calcium sulfate, aluminum sulfate, ferrous sulfate, ferric chloride and sodium chloride were sometimes successful.

Given the limited success of the salts, lime was tested for effect on the durability. 3% lime effectively increased the durability in the short term. Lime reactiveness increased with the exchangeable sodium percentage of the shale.

No long term effects were studied, nor were the potential problems of mixing the additives with the shales in the field resolved. Further research will be required with respect to these questions.

17. Key Words

Shale, Slake Durability, Chemical Stabilization, Lime, Classification, Pore Size Distribution

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LIST OF ABBREVIATIONS AND SYMBOLS

Å	Angstroms (10^{-10}m)
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
cc	cubic centimeter
cm	centimeter
ESP	Exchangeable Sodium Percentage
ft	feet
g	gram
in	inch
I_d	Slake Durability Index (from one cycle, 200 revolution test)
I_{dd}	Slake Durability Index of dry samples (500 revolutions)
I_{ds}	Slake Durability Index of soaked samples (500 revolutions)
I_{d2}	Slake Durability Index from the second cycle of testing (200 revolutions)
I_s (50)	Point Load Strength Index corrected for 50mm core diameter
ISHC	Indiana State Highway Commission
kN	kilonewton
kg/m^3	kilogram per cubic meter
lb	pound
m	meter

min	minute
ml	milliliter
mm	millimeter
MN	meganewton (10^6 newton)
MPa	Mega Pascal (10^3 kN/m^2)
μ	micron (10^{-3}mm)
μ	micrometer (10^{-6}m)
pcf	pounds per cubic foot
PLS	Point Load Strength
psi	pounds per square inch
R^2	coefficient of multiple determination
SAR	Sodium Absorption Ratio
Sat.	Saturated
S.I.	Slaking Index from simple and five cycle slaking test
Wt.	weight

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

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

Divide	By	To Obtain
millimeters	25.4	inches
meters	0.3048	feet
milliters (cc)	3785	U.S. Gallon
kilograms	0.4536	pounds (mass)
kilograms per cubic meters	16.02	pounds (mass)
kilonewton per square meter	6.895	pounds (force) per cubic foot
cubic meter	0.02832	cubic feet
square meter	645.2×10^{-6}	square inch

HIGHLIGHT SUMMARY

Many combinations of durability and strength can be expected when dealing with shales to be used in compacted highway embankments. The classification systems used to group shales into durable and non-durable categories are based on their slaking properties and often do not consider the hardness, degradability or the physico-chemical properties. Hard and durable shales can be placed as a rock fill, while the soft and non-durable shales must be thoroughly degraded and placed in thin lifts as a soil fill. The hard and non-durable shales are difficult to stabilize by mechanical means, such as increased compactive effort. However, due to their non-durable nature, they often develop excessive settlements during the service life of the embankment and even cause slope failures.

In this study chemical additives were investigated that could either: (a) help break down the shales during placement or (b) reduce the deterioration caused by slaking during the service life of the compacted shale embankment. The Slake Durability Test was used to evaluate the change in durability effected by various chemical additives in the slaking fluid, as well as lime in the compaction water. Chemical composition and pore size distribution of the shales were also investigated to determine their effects on durability. The principal conclusions from this investigation on selected Indiana shales were:

1. Slaking of non-durable shales cannot be attributed solely to the "air-breakage" phenomenon.
2. The slake durability index of the New Providence (a hard and non-durable) shale was increased by using 0.1N solutions of sodium chloride, calcium sulfate, and ferrous sulfate as the slaking fluid. Aluminum sulfate reduced the slake durability index when used at the same concentration.
3. The slake durability of Mansfield (a soft and non-durable) shale was increased by using calcium sulfate, aluminum sulfate, and ferrous sulfate at a concentration level of 0.1N. Ferric chloride decreased the slake durability index.
4. The sodium absorption ratio (SAR) from the saturation extract test can be used to estimate the durability of shales.
5. The parameters (cumulative porosity, median diameter and spread factor) of the pore size distribution of shale aggregates can be used to predict the durability.
6. Lime mixed with the compaction water increased the slake durability index of the compacted New Providence (a hard and non-durable) shale. Sixty days of curing period and 3% lime were found to be sufficient.

7. Lime did not effectively increase the durability of compacted Osgood (a hard and non-durable) shale.
8. Increased durability of compacted shales with lime is accompanied by increases in dry and soaked unconfined compressive strengths.

CHAPTER 1 INTRODUCTION

1.1 Background

Shale is the most commonly occurring sedimentary rock and is often exposed on the surface. Construction of highway embankments require economical use of material from adjacent cuts or nearby borrow sources. As shale formations crop out in most parts of southern Indiana, they are in highway cuts. Use of excavated shale from cuts and borrow areas in the compacted embankments as a rock fill [in lift thicknesses of about 1m (3 ft)] led to various problems (viz., excessive settlement and slope failures).

This initiated the research program at Purdue University through the Joint Highway Research Project to study the use of Indiana shale in compacted highway embankments. Deo (1972) investigated different shales of Indiana and proposed a classification system (based on laboratory tests to predict the field behavior) which is currently being used by the Indiana State Highway Commission. Chapman (1975) investigated several laboratory tests to evaluate the shale behavior to be used during classification. Bailey (1976) investigated the factors relating to degradation of shales during the compaction process. van Zyl (1976) prepared a statistical analysis of the data provided by the Indiana State Highway Commission for the shales tested in their laboratory. Abeyesekera (1978) investigated the stress-deformation and strength characteristics of compacted New Providence shale. Witsman (1979) investigated the effect of compacted prestress on compressibility of compacted New Providence shale. Hale (1979) investigated different compaction methods to develop a standard for evaluating degradation during the compaction process for shales.

Table 1.1.1 gives a list of shales studied and the properties investigated by the above investigators followed by the shales studied and properties examined during this investigation. The detailed information for shales studied is presented in Chapter 3.

1.2 Statement of Objectives

The behavior of a shale during construction and in service of an embankment depends on its degradation and slaking properties. The classification systems used to group the shales into durable and non-durable categories are based on the slaking properties of the shales evaluated in the laboratory, correlated with the field behavior. These systems do not consider the hardness, degradability or the physico-chemical properties of the shales. The hard and durable shales can be placed as a rock fill and the soft and non-durable shales as a soil fill. The hard and non-durable shales pose particular problems in embankments, often developing excessive settlement and even slope failures. These hard and non-durable shales are difficult to stabilize by mechanical means, such as increased compactive effort.

The objectives of this research are to:

1. identify the predominant minerals present in the shale and their effect on the durability of shale;

Table 1.1.1 Summary of Shale Properties Investigated at Purdue University

INVESTIGATED BY	SHALES USED	PROPERTIES INVESTIGATED
DEO (1972)	1. KLONDIKE 2. ATTICA 3. PAOLI X, Y, 3, 5 4. SCOTTSBURG 5. LYNNVILLE 6. CANNELTON 7. I-65 8. I-75 9. State Road 37 A, B 10. State Road 67 A, B	1. Slaking in water (Slake durability index) 2. Soundness 3. Abrasion characteristics 4. X-Ray diffraction 5. Activity, Atterberg limits 6. Compaction & CBR 7. Absorption 8. Bulk unit weight 9. Breaking characteristics
CHAPMAN (1975)	1. HARDINSBURG 2. NEW ALBANY 3. MANSFIELD 4. PALESTINE 5. KOPE 6. KLONDIKE	1. Slaking 2. Soundness 3. Atterberg limits 4. Rate of slaking 5. Los Angeles abrasion test 6. Ultrasonic cavitation 7. Schmidt hardness test 8. Washington degradation test 9. Ethylene Glycol soaking test 10. Mineralogy
BAILEY (1976)	1. BIG CLIFTY 2. BORDEN 3. CLORE 4. HANEY 5. HARDINSBURG 6. KOPE 7. NEW ALBANY 8. NEW PROVIDENCE 9. MANSFIELD 10. PALESTINE 11. WALTERSBURG 12. DILLSBORO	1. Scleroscope hardness 2. Point load strength 3. Degradation due to long term soaking 4. Absorption due to long term soaking 5. Shale degradation and compacted density from static, dynamic, kneading, and gyratory types of compaction
ABEYESEKERA (1978)	1. NEW PROVIDENCE	1. Strength characteristics of compacted shales
WITSMAN (1979)	1. NEW PROVIDENCE	1. Effects of prestressing in compaction 2. Effective embankment loadings with surcharges

Table 1.1.1 Continued

INVESTIGATED BY	SHALES USED	PROPERTIES INVESTIGATED
HALE (1979)	1. OSGOOD 2. NEW PROVIDENCE 3. PALESTINE	1. Point Load Strength 2. Degradation due to Compaction (static and dynamic)
THIS INVESTIGATION	1. NEW PROVIDENCE 2. MANSFIELD 3. NEW ALBANY 4. OSGOOD 5. ATTICA 6. PALESTINE 7. HARDINSBURG 8. KLONDIKE	1. Slaking characteristics with different salt solutions for shales 1, 2, and 3. 2. Lime as an additive in the compacted state for shales 1 and 4. 3. X-ray diffraction for estimation of clay minerals for shales 1, 2 and 3 4. Point Load Strength tests for shales 1, 2, 3, 5, 6 and 7 5. Pore size distribution studies for 1, 2, 3, 4, 5, 6, 7 and 8

2. study the type of (chemical) additives that could possibly be used during excavation or placement stages of construction and thereby reduce the amount of slaking of the shales during service; and
3. study the control of lime as an additive when used with different types of compacted shales.

CHAPTER 2 LITERATURE REVIEW

2.1 Shale and Its Classification

2.1.1 Description of Shale

Shale is defined as follows in the dictionary of geological terms (American Geological Institute, 1976):

> "Shale includes the indurated, laminated, or fissile claystones and siltstones. The cleavage is that of bedding and such other secondary cleavage or fissility that is approximately parallel to bedding. The secondary cleavage has been produced by the presence of overlying sediments and plastic flow."

The term shale is used by some to designate all argillaceous sediments, this includes claystones, siltstones, mudstone and marl. Others (See Figure 2.1.1 after Twenhofel, 1937; Grabow, 1920) designate the largest group as the mudstone or mud rock group and classify shale as a member of this group. Additionally, sedimentary rocks can be classified as soluble or insoluble. (See Figure 2.1.2 after Underwood, 1967). Sandstone, siltstone, claystone, shale and coal are the insoluble rocks.

Shale can be considered as a fine grained clastic sedimentary rock and is one of the most commonly occurring soil or rock materials on the earth's surface. It is encountered very often in embankment construction. Shale properties pertinent to embankment construction include its degradation and slaking behavior. Degradation is defined as the reduction in aggregation size due to construction operation, while slaking is

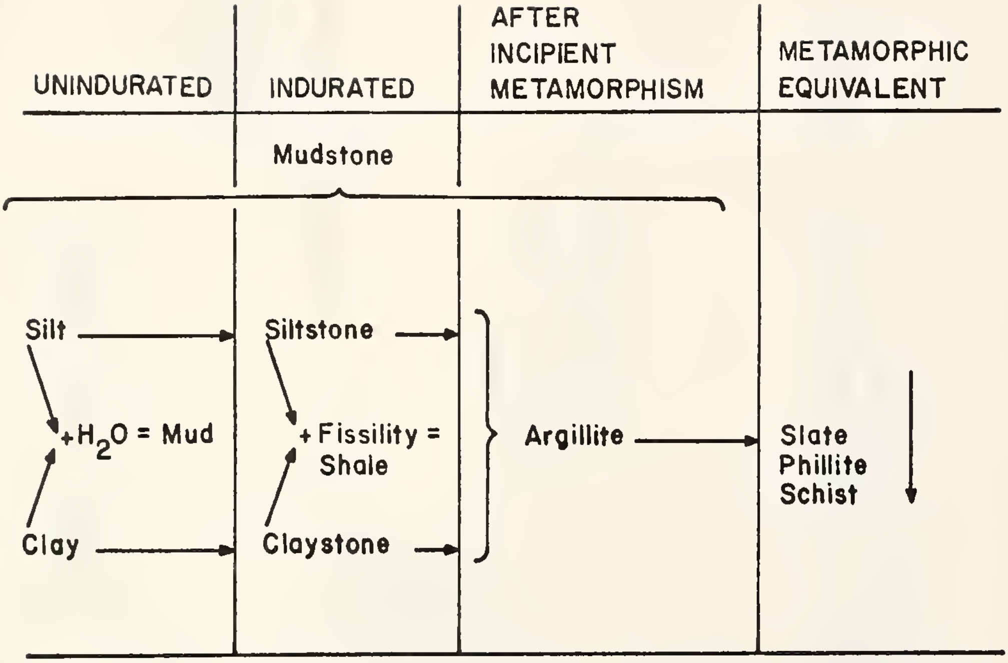


FIGURE 2.1.1 CLASSIFICATION OF SHALE AND RELATED ROCKS (AFTER TWENHOFEL, 1937)

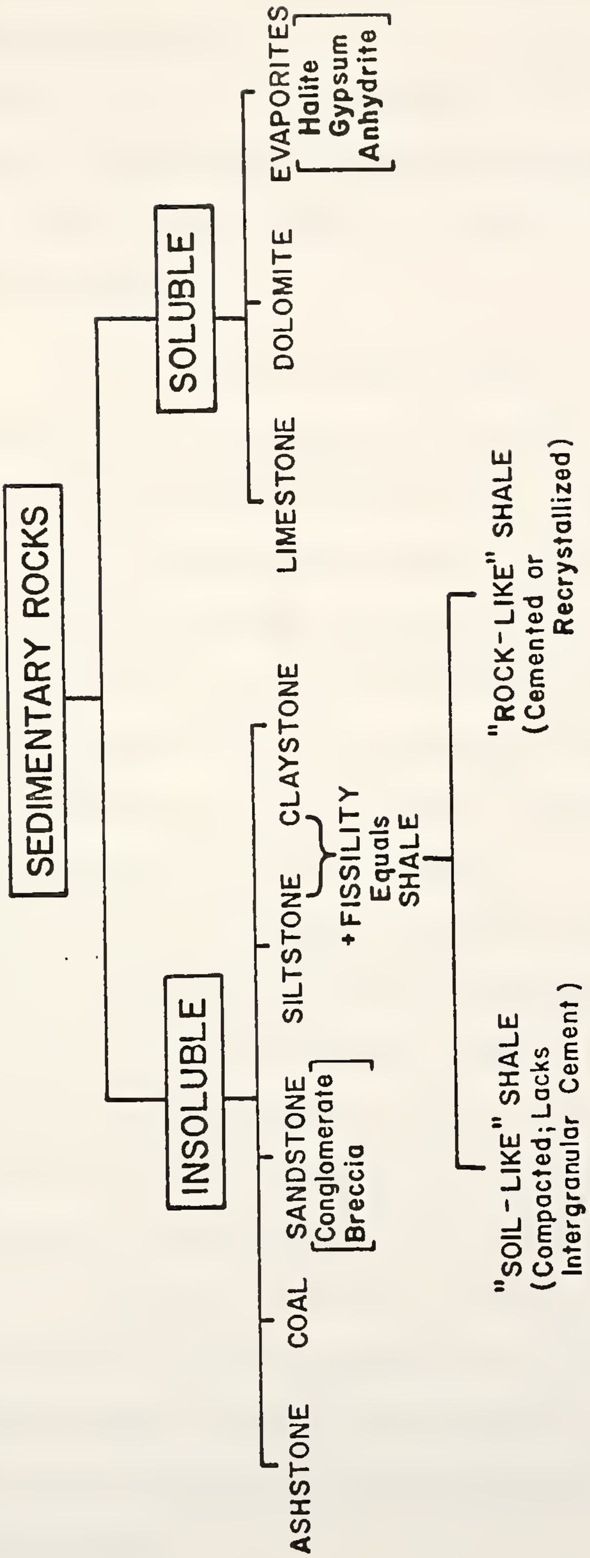


FIGURE 2.1.2 CLASSIFICATION OF SEDIMENTARY ROCKS
(AFTER UNDERWOOD, 1967)

disintegration due to certain environmental factors that are present within the embankment.

Table 2.1.1 (after Underwood, 1967) lists physical properties considered important for the engineering evaluation of shales and shows their probable in situ behavior. Table 2.1.2 gives physical properties of typical shales.

2.1.2 Classification of Shales

Shales can be classified in many ways. A classification system (Figure 2.1.3) proposed by Mead (1938) divides shale into two broad groups: (1) compaction ("soil like") shales which are compacted sediments under the overburden and which usually lack significant amounts of cementing material; and (2) cemented ("rock like") shales in which the cementing materials may be calcareous, siliceous, feruginous, gypsiferous, phosphatic, etc., or in some cases the bonding may be due to recrystallization of thin clay minerals.

The process of compaction referred to here can be defined as a decrease in thickness of a layer of compressible material under its own weight or as a result of surface loads. Materials become denser and stiffer due to the decrease in volume. On the other hand, the process of consolidation or cementation is one process that changes loose agglomeration of particles into rock. It may be due to chemical cementation. Usually the "rock like" shales are sufficiently consolidated and lithified so that they maintain their shape even when subjected to alternate cycles of wetting and drying. The compaction or "soil like" shales tend to slake rapidly when subjected to the same cycles of wetting and drying.

TABLE 2.1.1 AN ENGINEERING EVALUATION OF SHALES (AFTER UNDERWOOD, 1967)

PHYSICAL PROPERTIES			PROBABLE IN-SITU BEHAVIOR						
LABORATORY TESTS AND IN-SITU OBSERVATIONS	AVERAGE RANGE OF VALUES		High Pore Pressure	Low Bearing Capacity	Tendency To Rebound	Slope Stability Problems	Rapid Slaking	Rapid Erosion	Tunnel Support Problems
	UNFAVORABLE	FAVORABLE							
Compressive Strength	50-300 psi	300-5000 psi	✓	✓					
Modulus of Elasticity	20,000-200,000 psi	200,000-2x10^6 psi		✓					✓
Cohesive Strength	5-100 psi	100 psi to > 1500 psi			✓	✓			
Angle of Internal Friction	10-20°	20-65°			✓	✓			
Dry Density	70-110 pcf	110-160 pcf	✓					✓	
Potential Swell	3-15%	1-3 %			✓	✓		✓	✓
Natural Moisture Content	20-35%	5-15%	✓			✓			
Coefficient of Permeability	10^{-6}-10^{-10}cm/sec	> 10^{-6} cm/sec	✓			✓	✓		
Predominant Clay Minerals	Montmorillonite, Illite	Kaolinite, Chlorite	✓			✓			
Activity Ratio = $\frac{\text{P.I.}}{\text{\% Clay}}$	0.75 to > 2.0	0.35 to 0.75				✓			
Wetting and Drying Cycles	Reduces to grain sizes	Reduces to flakes					✓	✓	
Spacing of Rock Defects	Closely Spaced	Widely Spaced		✓		✓		✓	✓
Orientation of Rock Defects	Adversely Oriented	Favorably Oriented		✓		✓			✓
State of Stress	> Existing Overburden Load	≅ Overburden Load			✓	✓			✓

TABLE 2.1.2 SOME PHYSICAL PROPERTIES OF TYPICAL SHALES (AFTER UNDERWOOD,1967)

NAME OF SHALE Formation, Age, Locality		Compress strength psi	Modulus Elasticity psi	Cohesion psi	Angle of Internal Friction	Dry Density lbs/cu ft	Potential Swell %	Natural Moisture %	Predominant Clay Minerals	Activity Ratio	SOURCE OF DATA
BEARPAW, Cret., Canada	Weathered Unweathered	7 to 84 156 - 406	7500 18 000	3 to 6 22	6° to 20 30°	85, 95 95, 108	0.5% - 2% 5% 20%	29 to 36 18 to 27	Illite Montmorillonite Mixed - layer	0.30 to >1.5	Bingham (1964), Morton (1965) Peterson (1954), 1958), Laboratory and field slope analyses
PIERRE, Cret., So Dakota		20-1400	20,000-140,000	2 to 30	8°-25°	95-110	3% to 5%	18 to 27	do	0.3 to >2	Corps of Engineers, Missouri River Division
FT UNION Tert., No Dakota		20-1050	11,200 58 000	10	20°	95-115	2% (?)	18-24	Illite	—	
PEPPER Cret., Texas		28-70	—	2.6	7° 18°	110	—	20%	Illite Mont Kaolinite	1.2	Corps of Engineers, South West Division
DEL RIO Cret., Texas		56-154	—	1.8	19°-28	119	—	12%	Illite, Kaolinite, Mont	1.0	
TRINITY Cret., Texas		30-170	2 400 33 000	0.7	28	115-135	—	11-17%	—	—	
TAYLOR Cret., Texas		250-550	6,000 20,000	1.5-25	8°-30°	112-118	—	15%-18%	—	—	
Composite CYCLOTHEM of Pennsylvanian Shales (Eastern Ohio and Western Penn.)	Silty Clayey	210	1 000,000	58	23	138	—	9.1	—	—	Corps of Engineers Pittsburg District
	Carbonaceous	8165	—	1562	16°	—	—	—	—	—	
	Clay Banded	2084	—	931	7°	—	—	—	—	—	
	Clayey Ferruginous	1661	—	688	29°	—	—	—	—	—	
	Sandy	3674	826 500	1800	9°	—	—	—	—	—	
NIOBRARA, Calc Sh Cret Colo									Illite Bed		U S Geological Survey, Denver, Colorado
MOWRY, Cret Colo									Kaolinite, Chlorite Illite		
GRANEROS Cret., Colo									Ben., Ill Mx-layer		
MORRISON Jura Colo									Mont., Illite		
CARLILE, Cret Colo									Illite, Bed		
MAUV, Calc Shale, Cam., Utah		5270	2.3×10^{6}	1,160	88°	188	—	2%	—	—	U S Bureau of Reclamation, Denver (1953)
Quartzose Sh., Cam Utah		17,770	2.3×10	3,190	45°	165	—	4%	—	—	

1 Ton sq ft approx 14 lbs per inch

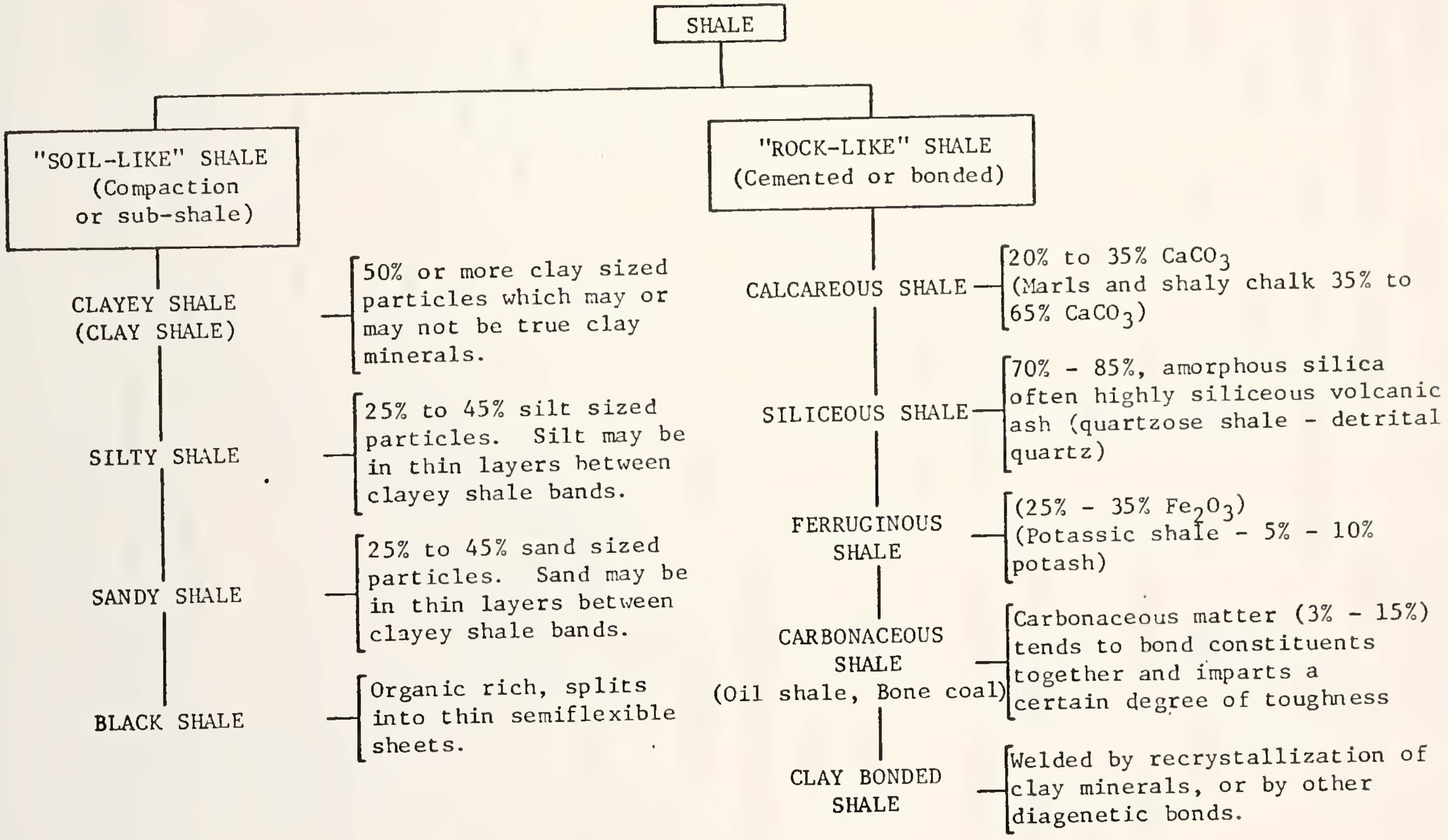


FIGURE 2.1.3 CLASSIFICATION SYSTEM OF SHALES (AFTER MEAD, 1938)

Siliceous shales can contain up to 85% SiO_2 and calcareous shales from 25% to 35% $CaCO_3$, but there are exceptions (Mead, 1938). Because the average chemical composition of all shales is so similar, chemical composition cannot be used for classification or identification.

Gamble (1971) studied shales from different parts of the United States and proposed a classification system shown in Table 2.1.3. This classification system is based on the grain size and breaking characteristic of the shales. Gamble (1971) also proposed an engineering classification of shales and other argillaceous rocks shown in Figure 2.1.4. This engineering classification is based on the Slake Durability (2 cycle) Test and the Atterberg limits.

Fissility (a property of splitting easily along closely spaced parallel planes) could be used to determine the character of a shale, but it has its limitations. Fissility observed in weathered shale at the outcrop may be entirely different from the fissility observed in a fresh exposure in an excavation. However, it helps in determining long time stability of slopes or the erodability of shale exposed in an unlined spillway chute or other similar structure (Grice, 1960).

Attempts have been made to classify shale by use of "common-word" modifiers. Terms such as "immature" shale, "heavy" shale, "light" shale, "popcorn" shale, "gumbo-shale", "firm" shale, etc., have been used. These terms may be significant to those who use them but are of doubtful value to most engineers.

An additional classification scheme (Figure 2.1.5) for argillaceous material was reported by Morgenstern and Eigenbrod (1974). A primary

TABLE 2.1.3 CLASSIFICATION FOR ARGILLACEOUS ROCKS (AFTER GAMBLE, 1971)

Unindurated	Indurated Group		After Incipient Metamorphism	Metamorphic Equivalents
	Mudrocks(Shales or Mudstones)			
	Breaking Characteristics			
	Massive	Fissile or Shaly		
Silt →	Siltstone	Silty Shale	Argillite →	Slate, Phyllite, or Schist
Mud (mixture or undetermined amount of silt and clay, with minor amount of sand)	Mudstone	Shale		
Clay →	Claystone	Clayey Shale		

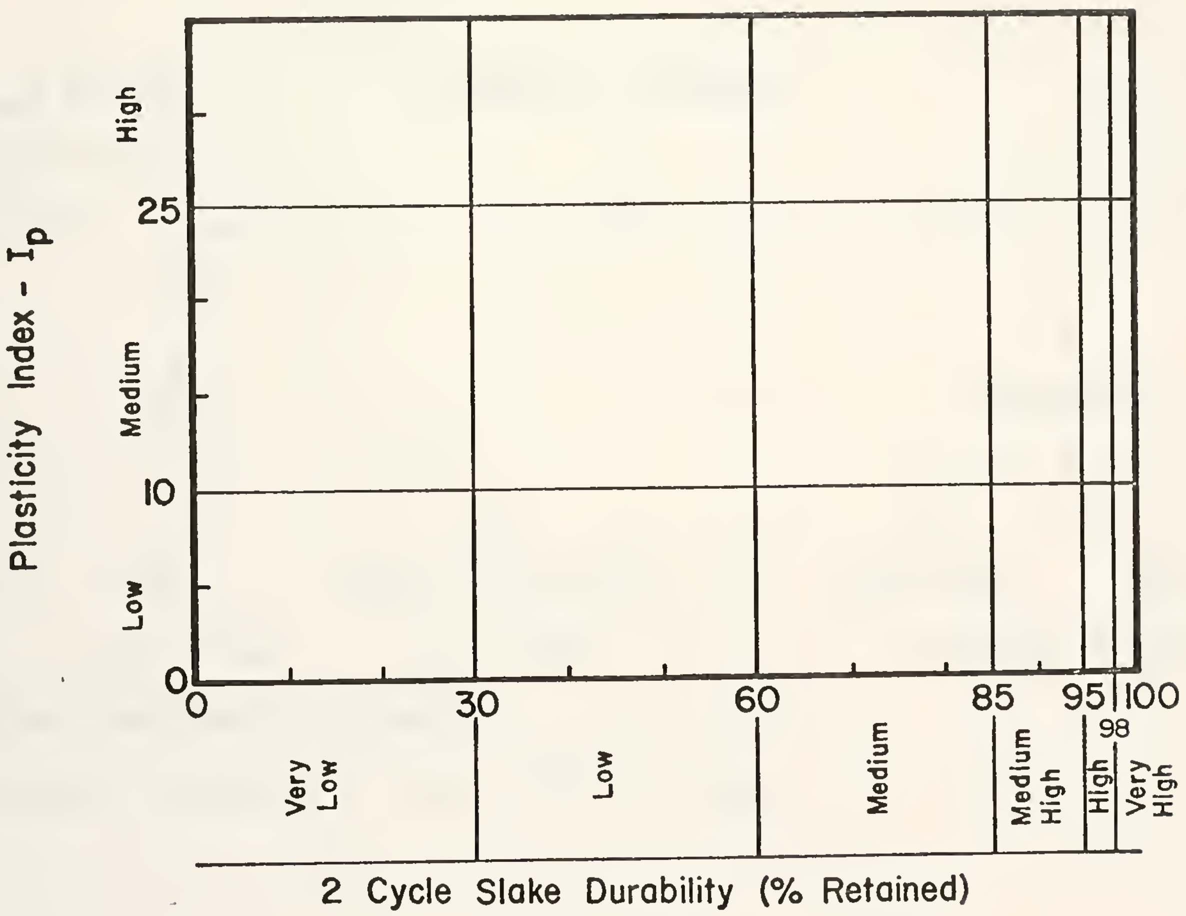


FIGURE 2.1.4 GAMBLE'S DURABILITY-PLASTICITY CLASSIFICATION FOR SHALES & OTHER ARGILLACEOUS ROCKS (AFTER GAMBLE, 1971)

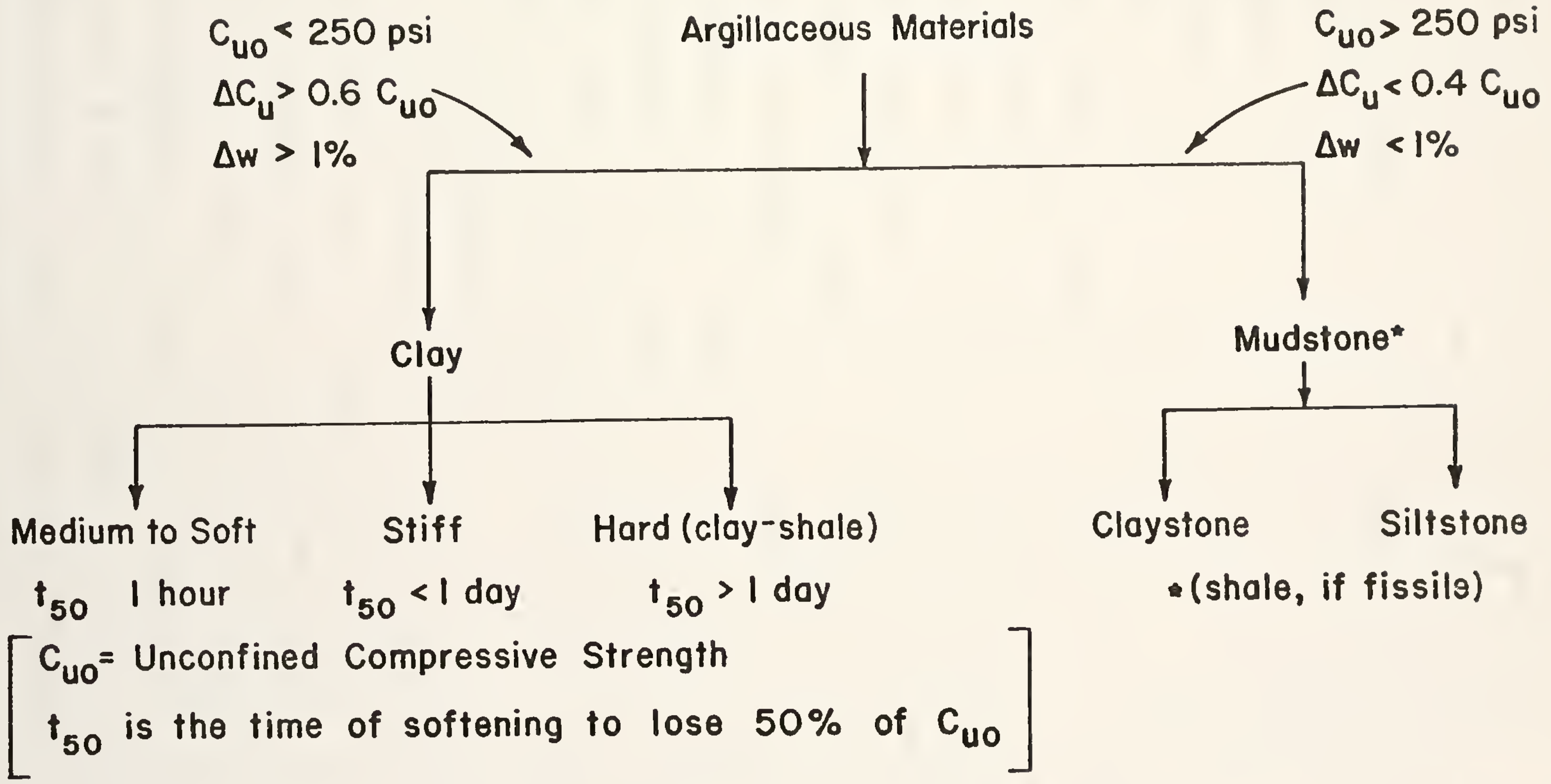


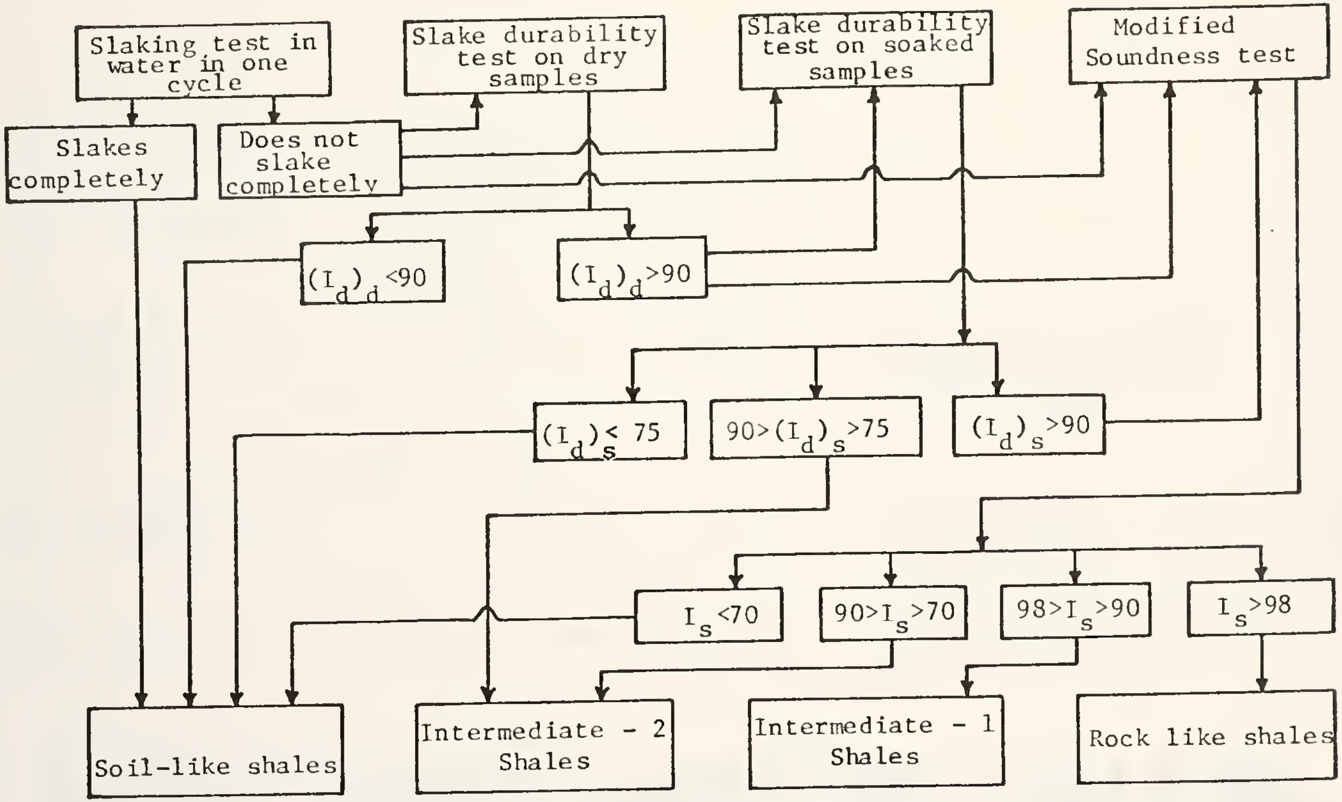
FIGURE 2.1.5 ENGINEERING CLASSIFICATION OF ARGILLACEOUS MATERIALS (AFTER MORGENSTERN & EIGENBROD, 1974)

feature is the use of variations in strength and water deterioration characteristics of shales to produce a classification.

A classification system developed by Deo (1972), is currently being used by the Indiana State Highway Commission (Figure 2.1.6). This classification system is based on four simple tests, and divides shales into "soil like" shales, "rock like" shales, and "Intermediate 1 or 2". The tests used are a 5 cycle simple slaking test, 500 revolution slake durability test on dry and soaked samples and a sodium sulfate soundness test, modified by using 50% solution rather than a saturated one.

Chapman (1975) concluded from his study and the results from the tests conducted by the Indiana State Highway Commission that none of the Indiana shales tested could be classified as "Intermediate 1 or 2". It is suggested that drying out of stored samples in the laboratory gave higher slake durability indices, causing the shale to fall under the "Intermediate 1 or 2" category instead of "soil like". However, an "Intermediate 2" type shale has been recently identified by the ISHC (Rahn, 1980). This shale was so classified by its modified soundness value, as the slake durability index for a soaked sample (500 revolutions) was greater than 90.

Hudec (1978) in his study of Ontario shales proposed a classification system based on the reuslts from a 5 cycle slake durability test (200 revolutions each). This classification system shown in Figure 2.1.7 separates shales into four categories, viz., Rock like, Low Loss, Intermediate Loss and High Loss, and suggests that Rock like should be placed as a rock fill and the others as soil fill. Increased compactive effort and water content is required for increasing durability of the shales to be able to place it as a soil fill.



$(I_d)_d$ = Slake durability index from test on dry samples (1 cycle, 500 revolutions)

$(I_d)_s$ = Slake durability index from test on soaked samples (1 cycle, 500 revolutions)

I_s = Index from Modified sodium sulphate soundness test (5 cycles)

FIGURE 2.1.6 PROPOSED CLASSIFICATION OF INDIANA SHALES FOR EMBANKMENT CONSTRUCTION (AFTER DEO, 1972)

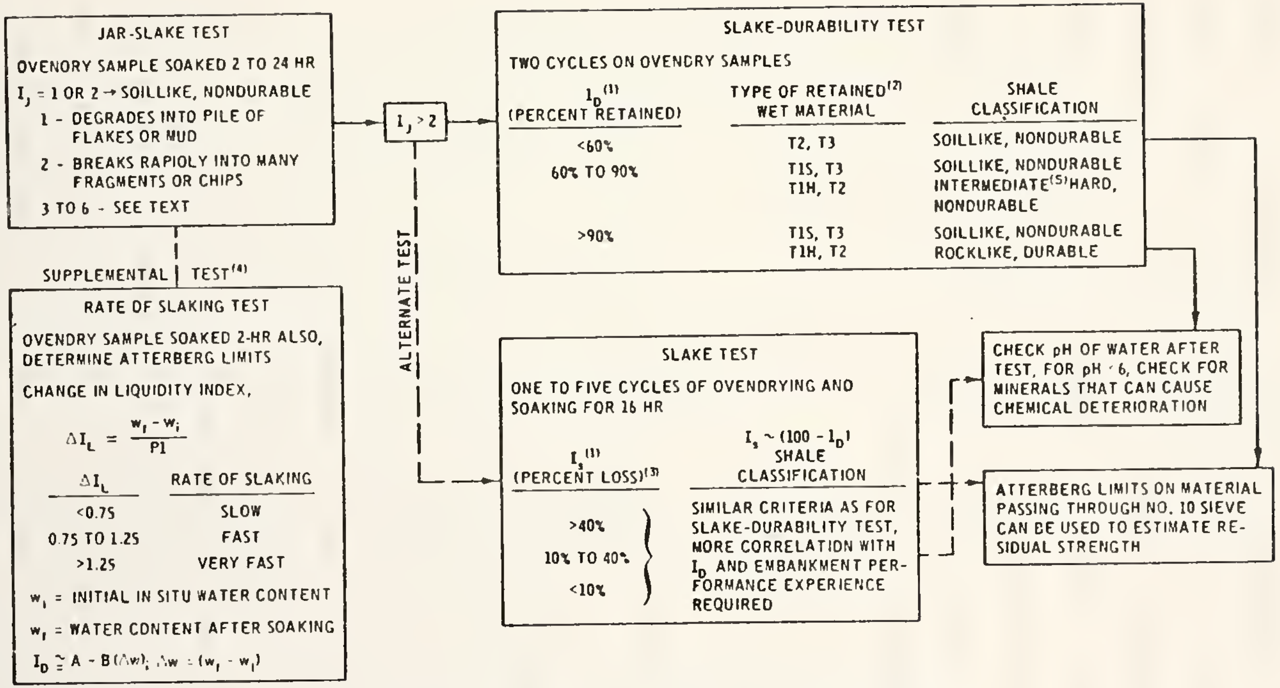
Classification	Number of Samples	Limits (% Loss)	Slake Durability Index	Nature of Loss
Rock-like	20	0-5	95-100	No discernible effect.
Low Loss	6	5-25	75-95	Minor spalling along bedding planes.
Intermediate Loss	3	25-60	40-75	Spalling and Disintegration.
High Loss	14	60-100	0-40	Disintegration into mud-like consistency.

FIGURE 2.1.7 DURABILITY CLASSIFICATION (5-CYCLE SLAKE DURABILITY TEST ON 13mm (+1/2 INCH) SAMPLE SIZE) (AFTER HUDEC, 1978)

The above classification systems (Deo, 1972; Hudec, 1978) do not consider the strength of shales in the dry state to be an indication of the resistance to breakdown which controls the amount of compactive effort to be used in the field.

Strohm et al., (1978) suggested a classification system given in Figure 2.1.8 which considers test results from Jar-Slake Test, Rate of Slaking Test, Slake Durability Test (2 cycles, 200 revolutions each) and Slake Test (5 cycles). The categories in this system are: Soil like, Non-Durable; Intermediate Hard, Non-Durable; and Rock like, Durable. This system is advantageous over the other classification systems cited in this section as it takes into consideration the hardness of the shale in addition to the durability properties. "Soft" is used to describe a shale that can be broken apart with the fingers; "hard" means that this cannot be done. Thus, hardness is not defined by an index obtained from a test and is subject to different interpretations.

A shale is classified as Soil like, Non-Durable, even though the slake durability index is greater than 90, when the sample has a T1S,T3 type of fragmentation. The major drawback in this instance is that at the end of a standard test, the index is ignored and the sample is classified according to an empirical rating system. It is suggested that the retained material be further sieved on a larger sieve to give a meaningful solution to this problem (Chandra, 1970). The larger sieve opening to be used may be a function of type of shale tested. This system also calls for checking the pH of water after the Slake Durability and Slaking Test, and obtaining Atterberg limits on the material passing 2mm (#10 sieve) to estimate the residual strength. These two



NOTE [1] DIFFERENT LIMITING VALUES MAY BE JUSTIFIED ON BASIS OF LOCAL EMBANKMENT PERFORMANCE EXPERIENCE.

[2] TYPE T1 - NO SIGNIFICANT BREAKDOWN OF ORIGINAL PIECES.
TYPE T1S - SOFT, CAN BE BROKEN APART OR REMOLDED WITH FINGERS.
TYPE T1H - HARD, CANNOT BE BROKEN APART.
TYPE T2 - RETAINED PARTICLES CONSIST OF LARGE AND SMALL HARD PIECES.
TYPE T3 - RETAINED PARTICLES ARE ALL SMALL FRAGMENTS.

[3] USING NO. 10 SIEVE.

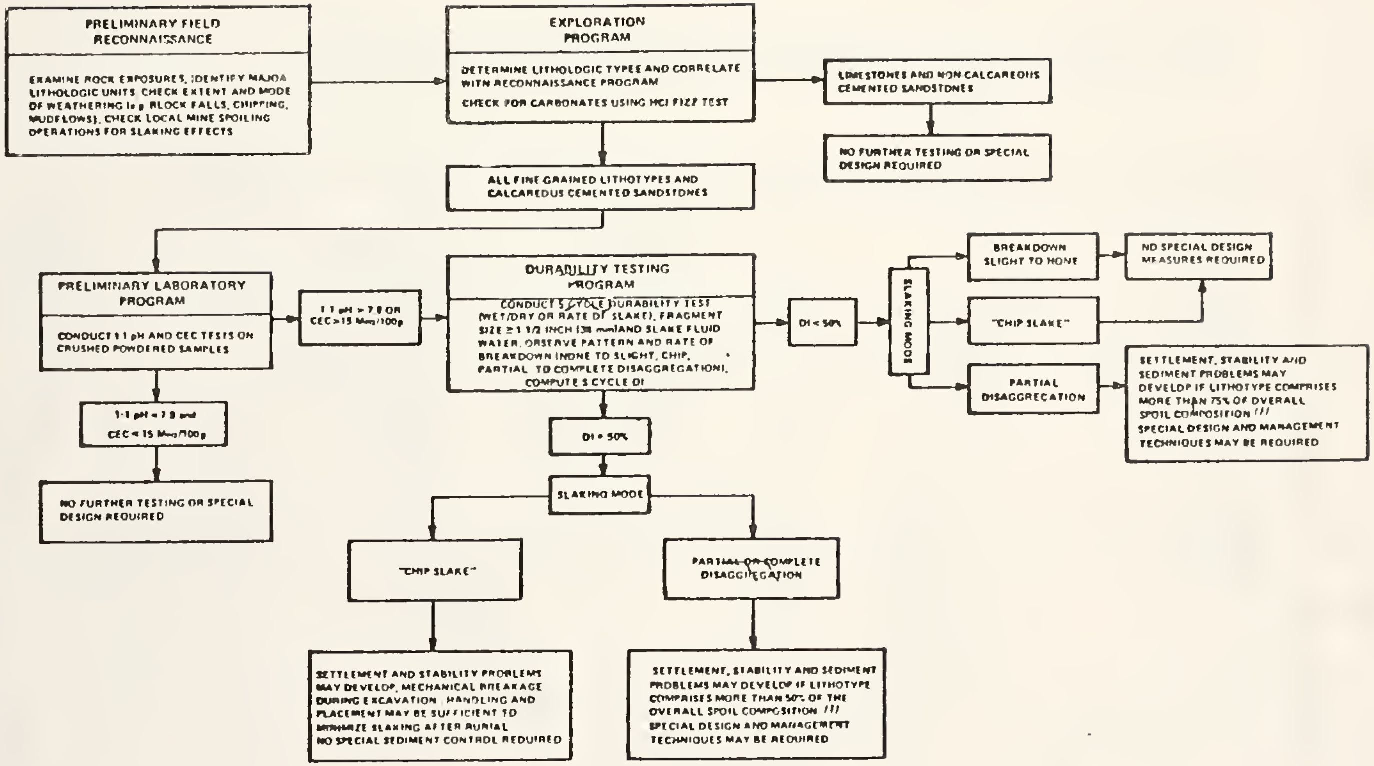
[4] CAN BE PERFORMED ON JAR-SLAKE TEST SAMPLES IF IN SITU NATURAL WATER CONTENT IS KNOWN. PI SENSITIVE TO DEGREE OF PULVERIZATION.

[5] REQUIRES SPECIAL PROCEDURES TO ASSURE GOOD DRAINAGE AND ADEQUATE COMPACTION (95% T-99) FOR LOOSE LIFT THICKNESS UP TO 24-IN. MAXIMUM.

FIGURE 2.1.8 CLASSIFICATION CRITERIA FOR SHALES USED IN HIGHWAY EMBANKMENTS (AFTER STROHM et al.,1978) factors help in describing the shale with respect to its physico-chemical and mineralogical nature.

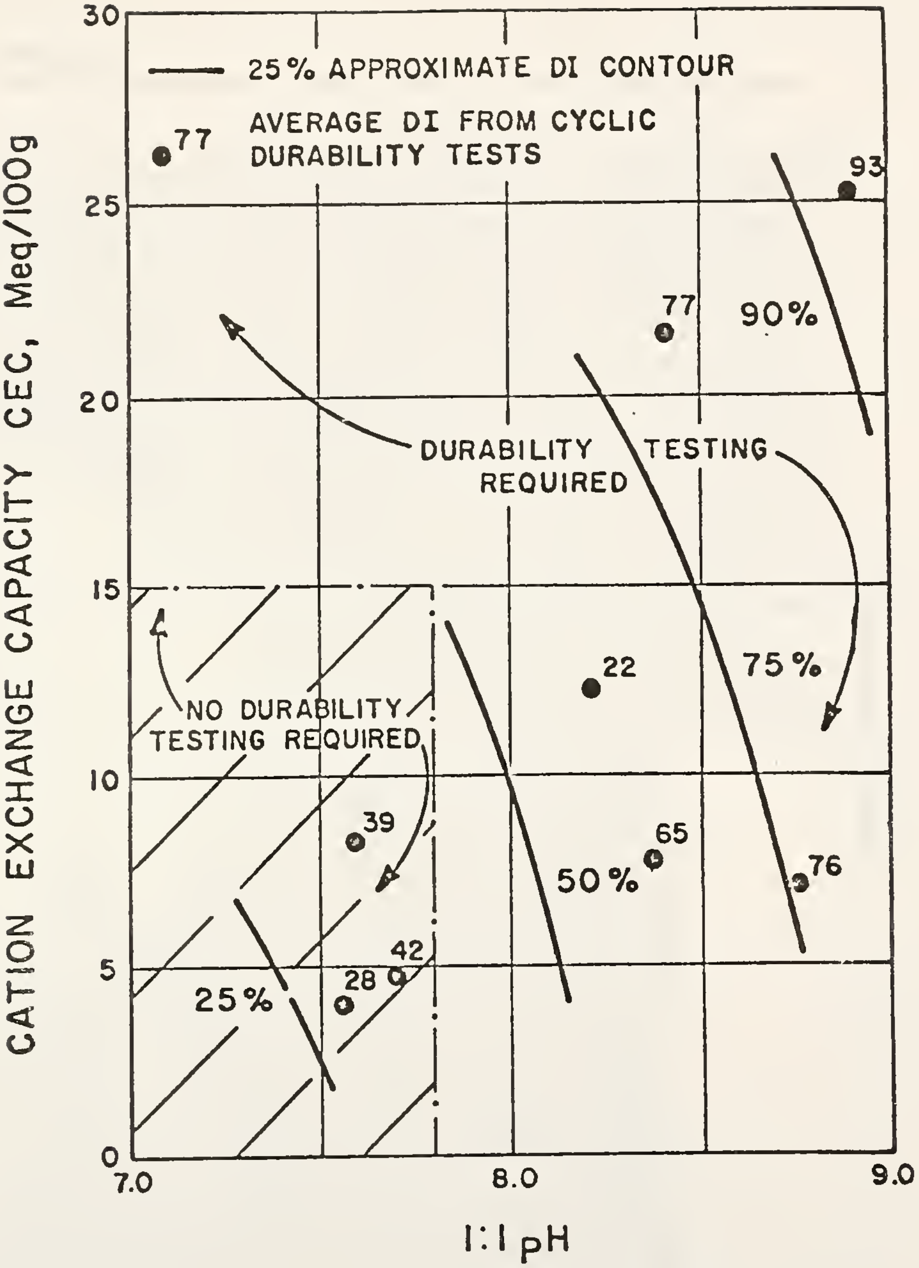
Andrews et al., (1979) of D'Appolonia Consulting Engineers in their study of "Environmental effects of slaking of surface mine spoils in eastern and central United States" for the United States Department of the Interior, Bureau of Mines, have presented a classification system shown in Figures 2.1.9 and 2.1.10. This system includes field observation of the specimens before the laboratory testing, to distinguish the limestone and cemented sandstones that might be present with the shales. Further, the samples are subjected to durability testing depending on the pH of powdered samples in water at 1:1 concentration, and on the cation exchange capacity. From Figure 2.1.9 it is determined whether durability tests are required or not. The durability testing program consists of 5 cycle wet/dry or rate of slaking test, with observation of the type of fragmentation resulting from the test. The Degradation Index (D.I.) which gives a measure of breakdown of shales in the above tests uses a weighting factor dependent on the mean equivalent mesh size of the sieves used for sieving before and after the durability tests.

Franklin (1979) proposed a shale rating chart presented in Figure 2.1.11. This chart classifies shales by the slake durability index (I_{d_2}) and the point load strength index I_s (50) (corrected for sample size of 50mm core diameter) for those which have a slake durability index (from 2 cycle, 200 revolutions test) greater than 80, or the plasticity index (I_p) if the slake durability index is less than 80. The shales are rated on a scale of 1 to 9; 9 being the hard and durable and 1 being the soft and non-durable. Figure 2.1.12 gives correlations between the shale rating thus obtained and the field performance of the



(1) BASED ON [illegible] MIXING OF LITHOTYPES DURING PLACEMENT

FIGURE 2.1.9 PROPOSED CLASSIFICATION SYSTEM (AFTER ANDREWS, et al., 1979)



DI = Degradation Index
(from 5 Cycle Wet/Dry or Rate of Slaking Test)

FIGURE 2.1.10 PRELIMINARY LABORATORY CLASSIFICATION
(AFTER ANDREWS et al., 1979)

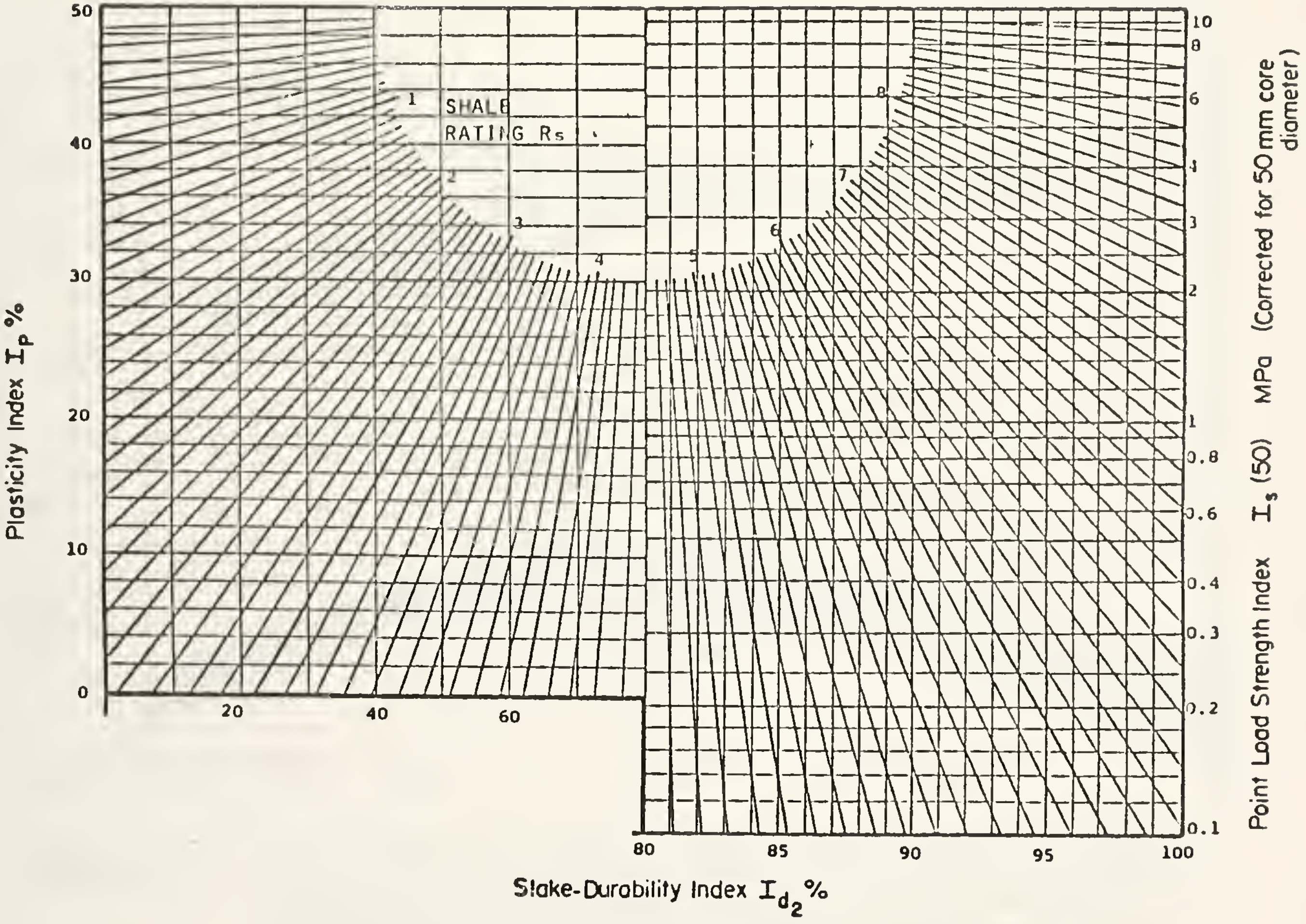


FIGURE 2.1.11 SHALE RATING CHART (AFTER FRANKLIN, 1979)

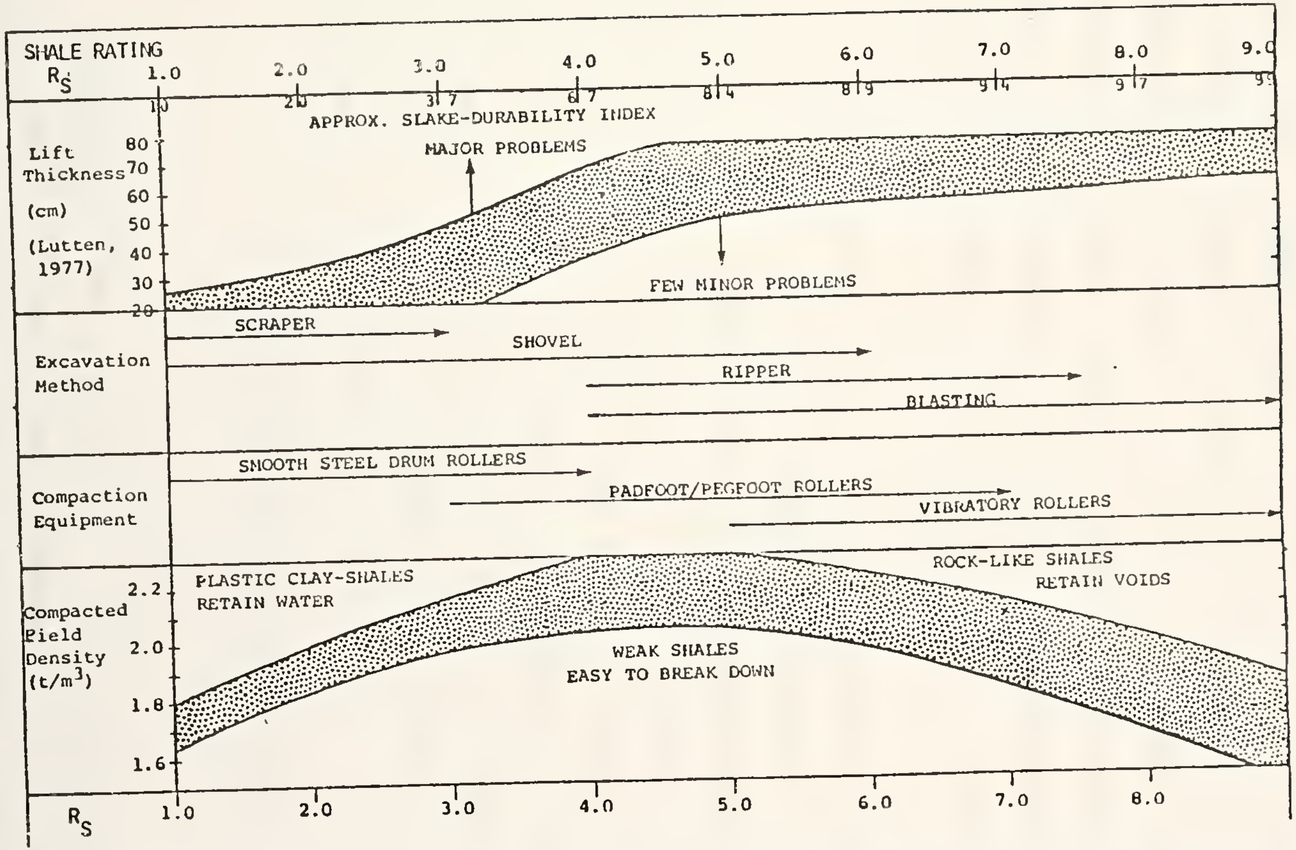


FIGURE 2.1.12 TENTATIVE CORRELATIONS BETWEEN THE TYPE OF SHALE EXPRESSED AS THE SHALE RATING R_s, AND BEHAVIOR OF THE SHALES IN EMBANKMENT CONSTRUCTION (AFTER FRANKLIN, 1979)

shales. This classification system was derived in a study of Ontario, Canada shales which do not have appreciable amounts of expandable clays (Franklin, 1979). In the absence of core samples it is suggested that the point load strength (PLS) tests be run on irregular pieces of shale. Hale (1979) has shown that the PLS is dependent on the size of the specimen tested. Therefore, the size of the specimen needs to be specified before evaluating the PLS to be used in this rating system if irregular pieces of shale are used. This system also needs to be verified for shales from other regions where the amounts of expandable clays are greater.

A classification system using such common-word modifiers, viz., Rock like, Soil like, etc., should be limited to the region of development. In general a classification system needs to take into consideration both durability properties and measures of the strength. Even so, the indices obtained from these tests depend on the moisture content of the specimen being tested. The tests used in these classification systems are on discrete pieces of shale or intact samples. The behavior of the shales in the compacted state should be evaluated from field experience or by building test pads.

2.2 Sedimentation and Weathering Process

2.2.1 Structure

Composition. The composition of a shale depends on several factors such as source and environment of sedimentation and diagenesis of the sediments. The environment and source of the sediment controls the original type of minerals, and the changes are caused by diagenesis of these minerals. The two groups of minerals present most commonly are detrital minerals and authigenic minerals. Table 2.2.1 gives the common constituents of mudrocks. The detrital minerals consist mainly of quartz and the clay minerals. The ability of clay minerals to imbibe water depends upon the type of clay mineral, which in turn controls some of the engineering properties. The authigenic minerals consist of cements that are formed after deposition. The contribution of mineral type to the engineering behavior depends on the cementing minerals binding the detrital minerals together, and once the cementing effects are broken, upon the amount and type of clay minerals.

Clay minerals are hydrous aluminosilicates, platy in shape and are generally less than two micrometers in equivalent diameter. The attractive or repulsive forces on the individual clay minerals are the result of electrical charge deficiencies. The charge deficiencies are induced by the ionic substitution within the lattice and perhaps by broken bonds on lattice edges.

Clay minerals are classified based on the structural configuration which results from the arrangement of the silica and alumina sheets. The silica sheet consists of silicon atoms tetrahedrally coordinated with oxygen, and the alumina sheet has aluminum or magnesium atoms octahedrally coordinated with the hydroxyl radical. The chemical composition and ionic substitutions within the basic structures leads to further classification. The clay mineral classification shown in Table 2.2.2 (after Shamburger et al., 1975) is as follows:

a. Two-Layer Clays. These consist of one silicon tetrahedral layer bonded to one aluminum octahedral layer. Kaolinite is the common mineral when the octahedral layer contains

Table 2.2.1 Common Mineral Constituents of Mudrocks (after Shamburger et al., 1975)

Detrital			Authigenic	
Sand size	Silt size	Clay size	Noncementing	Cements
Quartz	Quartz	Quartz	Clay minerals	Calcite
Feldspars	Feldspars	Feldspars	Zeolite	Dolomite
Micas	Micas	Micas	Feldspars	Siderite
Heavy minerals	Chlorite* Heavy minerals**	Clay minerals	Pyrite	Gypsum
(Rock fragments)	Clay minerals	Kaolinite Halloysite Illite/mica† Smectite Chlorite* Vermiculite Mixed-layer types		Quartz/silica
(Volcanic glass)	Kaolinite			Iron oxides and hydroxides
(Shells)				Aluminum oxides and hydroxides
(Organic matter)				
	(Rock fragments)	(Organic matter)		Pyrite
	(Volcanic glass)			
	(Shells)	(Allophane)		
	(Organic matter)			

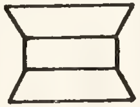
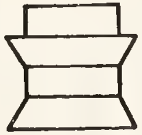
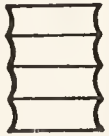
( ) detrital rock fragments; amorphous, nonmineral glass, and biological constituents.

* The properties and composition of silt-size chlorite are significantly different from clay-size chlorite.

** Heavy minerals may also occur as clay size

† Micaceous minerals exhibiting 10 Å d-spacings and clay size are also called clay-mica.

TABLE 2.2.2 STRUCTURAL CLASSIFICATION OF CLAY MINERALS (FROM SHAMBURGER et al., 1975)

Category	Thickness	Configuration		Example
2-layer clay minerals	7 Å		Octahedral Tetrahedral	Kaolinite
3-layer clay minerals	10 to 15 Å		Tetrahedral Octahedral Tetrahedral	Illite Vermiculite Montmorillonite
Mixed-layer clay minerals:				
Regular	14 Å		Octahedral Tetrahedral Octahedral Tetrahedral	Chlorite
	26 to 29 Å		Montmorillonite Chlorite Montmorillonite Chlorite	Interlayered montmorillonite and chlorite
Random	Variable		Montmorillonite Chlorite Chlorite Montmorillonite Montmorillonite	Mixed-layer montmorillonite and chlorite

mainly aluminum; serpentine consists of a magnesium-rich octahedral layer.

b. Three-Layer Clays. These clays have one octahedral layer bonded between two tetrahedral layers; examples of this type are illite, vermiculite, and montomorillonite. These minerals may occur dioctahedrally or trioctahedrally.

c. Mixed-Layer Clays. These clays consist of an interstratification of tetrahedral, octahedral, and two and three-layer combinations. The mixing may be regular or random. An example of a regular-mixed layer clay is chlorite, a three-layer plus octahedral layer repetition; another common one is montmorillonite-chlorite. The randomly mixed layer clays consist of many possible combinations.

When the cementing bonds are broken by the process of degradation due to the compaction process during contruction, or by slaking due to weathering or other mechanisms, the shale loses its original properties and behaves as a soil. The resulting engineering properties such as swelling, strength, etc., depend on the type and amount of both the clay and non-clay minerals present. Table 2.2.3 gives the relation between clay minerals and engineering behavior. However, Table 2.2.3 is only for monomineralic materials, whereas the strength of a mixture of clay and non-clay components depends on the type and amount of the non-clay constituents present and the sedimentation history (Grim, 1962).

Fabric. The "fabric" of sedimentary rocks is a term used to describe the distribution of the particulate materials including clay minerals or groups of clay minerals and the cementation. A so-called "dispersed" arrangement or the face-to-face contact of individual or

Table 2.2.3 Engineering Properties of Clay Minerals
(from Shamburger et al., 1975)

Clay	Plastic Limit	Liquid Limit	Plasticity Index	Average Free Swell, Percent	Range, Percent
Sodium-montmorillonite	91	441	350	1500	1400 to 1600
Calcium-montmorillonite	68	155	87	102	65 to 145
Illite	39	86	47	89	60 to 120
Kaolinite	31	54	23	28	5 to 60

Clay	Unconfined Compressive Strength, psi	
	wet specimen	Air-dried specimen
Sodium montmorillonite	56	195
Kaolinite	100	66

groups of clay platelets occurs frequently in shales, and tends to induce fissility. Flocculated arrangements may form under different environments and involve partly edge-to-face contact of platelets. This kind of orientation is commonly seen in mudstones, siltstones and claystones (Shamburger et al., 1975). The development of any one kind of orientation reflects the sedimentation environment, the diagenesis and the loading history.

The factors causing fissility are (Ingram, 1953),

> "(1) Rotation of flaky clay particles into positions perpendicular to the force exerted by the overlying material with concurrent flattening of clay aggregates;
> (2) Settling of clay particles in water with the platy surface parallel to the surface of deposition;
> (3) Orientation of clay particles by weak currents;
> (4) Growth of clay minerals perpendicular to bedding;
> (5) Presence of illite: illite tends to be more easily dispersed and also, it is the most common constituent of the older shales;
> (6) Expansion due to the unloading caused by erosion of the overburden;
> (7) Low carbonate content;
> (8) High sulfur and carbon content;
> (9) Weathering process; and
> (10) Small grain size of non-clay minerals."

The interlayer cation and the ions in the pore water influence the properties of clay minerals in shales. Ionic substitutions generally result in negative charge imbalance, whereas either positive or negative charge deficiences result from broken bonds. Ions are also attracted at the surface of the octahedral layer of kaolinite by hydrogen bonds. The electrical balance on the surface is satisfied by a layer of ions surrounding the clay minerals. These ions significantly affect the hydration characteristics of the clay minerals.

2.2.2 Factors Contributing to Material Breakdown

The breakdown of shale in an embankment is due to several factors including its geologic history and environment, placement in the embankment, and drainage conditions. In this section the geologic and environmental factors that govern the response of shale in embankments are discussed.

Geologic Age. The geologic age of a rock influences its engineering properties through the depth of burial, diagenesis, introduction of cements and weathering processes. The older rocks are more durable compared to the younger ones but are not independent of factors such as the environment of deposition. An example is the transformation of mixed layer minerals into illite which may take place under marine environments, or metamorphism.

Authigenesis and Diagenesis. Authigenesis is the process in which new minerals are formed in place after the deposition of the sediments. The formation of new minerals may reduce the void ratio and hence decrease the permeability. Diagenesis is the conversion of minerals of one type to another when sediments are lithified into sedimentary rocks. Diagenesis has been considered to be incipient metamorphism. Expansive clay minerals like montmorillonite are transformed into illite through progressive removal of water layers under pressure (Mitchell, 1976).

Cementation. Cementation commonly controls the stability of soil aggregates and cemented shales. The presence of cementing materials in the shales can be recognized but the amounts are difficult to quantify. Slaking processes in shales may be predominantly due to dissolution of cementing material.

Leaching, Ion Exchange, and Differential Solution. Marine clay deposits, uplifted above sea level due to change in geological events, are subjected to leaching by the percolating ground water. Leaching may cause a change in interparticle forces between colloids due to ion exchange and removal of dissolved salts. Leaching has been shown to be a major cause in the formation of quick clays (Lambe, 1953; Bjerrum, 1954; Mitchell, 1956).

Shales may be subjected to differential solution followed by leaching which results in the removal of materials, including cementing agents, from the sediments. Channels, sinkholes, etc., are the end products of such solution activities in calcareous sediments.

Jointing and Fissuring of Clay Soils. Joints in clays deposited in flood plains are due to cyclic expansion and contraction caused by wetting and drying. Joints and fissures in preconsolidated clays result from unloading or shrinkage induced during drying. Closely spaced joints cause slides in excavations by softening of the clay in these joints when water is admitted into the joint system. The other processes which induce fissures in clays are synaeresis (separation of fluid from a gel), weathering, and organic agents such as tree roots, etc.

Weathering. Weathering is a process where the rock is broken down to various sizes, shapes and composition. Reiche (1950) defines weathering as "the response of materials within the lithosphere to conditions at or near its contact with the atmosphere ..."

The physical processes causing weathering are unloading, thermal expansion and contraction, crystal growth (including post action), colloid plucking, and organic activities (Reiche, 1950). The removal of overburden pressure induces the rock mass to expand by opening of cracks and joints. Surface peeling and spalling is called exfoliation. Thermal expansion and contraction accents the planes of weakness. The growth of crystals in a rock due to concentration of solutions at a local point may exert repulsive forces disrupting the successive layers. Calcium sulfate and magnesium sulfate crystals have been shown to undergo the above mentioned disruptive forces. Colloidal materials shrink on drying and may remove flakes from the rock mass with which they are in contact. Organic matter, if mineralized, can produce both soluble and insoluble active compounds. The soluble compounds disperse on clays whereas the insoluble ones fix the clays.

It has been recognized (Reiche, 1950) that crystal growth and unloading are the predominant factors involved in the physical process of weathering. Physical weathering essentially breaks down a rock mass into particles of reduced size increasing the surface area available for chemical weathering processes.

The chemical processes of weathering include hydrolysis, chelation, cation exchange, oxidation, and carbonation. Hydrolysis is the process where ions of the minerals react with the hydrogen and hydroxyl ions of water forming hydroxides of metal ions. Dissolvable materials formed by hydrolysis are removed by leaching of flowing water. The pH of the water controls the solubility of the oxides of aluminum, silica, and the clay minerals that are formed as a result of hydrolysis. Chelation is the complexing and removal of metal ions and is effective in driving hydrolysis reactions. Organic compounds that have ring structures hold the metal ions within the rings by covalent bonding. Cation exchange is an important process of weathering because of the activation of the silicate minerals due to hydrolysis by flowing ground water solution. Oxidation is defined as the loss of electrons by cations, while reduction is the gain of electrons (Mitchell, 1976). Oxidation processes depend on dissolved oxygen in the water. Weathering processes involving oxidation of pyrite are given by the following equations:

$$2FeS_2 + 2H_2O + 7O_2 \longrightarrow 2FeSO_4 + 2H_2SO_4$$

$$FeSO_4 + 2H_2O \longrightarrow Fe(OH)_2 + H_2SO_4$$

The H_2SO_4 formed in these reactions serves to rejuvenate the process, drive the hydrolysis of silicates, and react with limestone to produce gypsum and carbonic acid. Reactions which cause reduction, influence the bacterial action. Plants, on weathering, store energy that may be used in later stages of weathering. Carbonation is the process in which carbonate or bicarbonate combines with rocks in the presence of atmospheric CO_2.

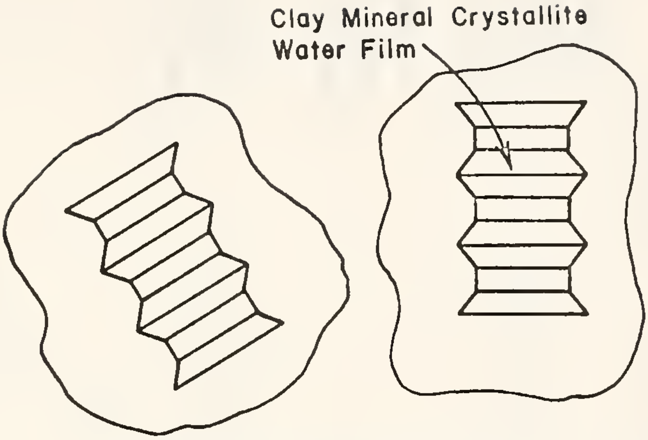
2.2.3 Breakdown of Shales

The geological and mineralogical conditions which cause deterioration of shales in embankments are reviewed in this section. These may occur simultaneously or may occur as a consequence of each other.

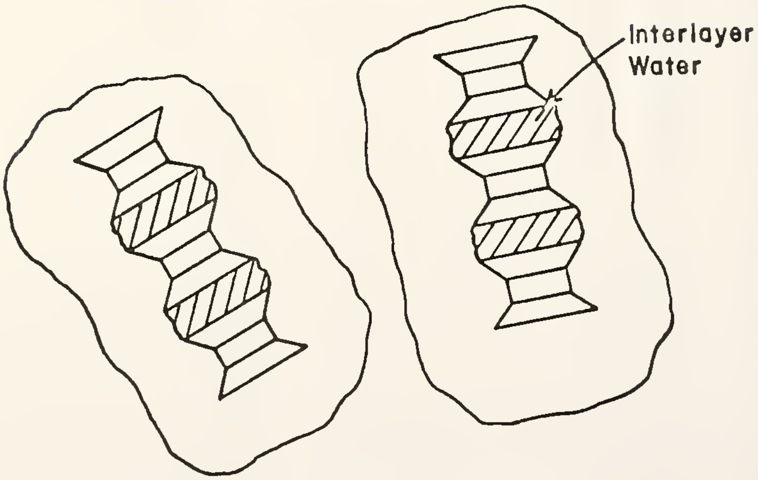
Expansive clay minerals. Water in the clay-mineral system of a shale is the main cause of deterioration. Water molecules surround either the individual or aggregate clay crystallites. The water molecules tend to develop an orientation around the clay particles; this attempts to reduce the Van der Waal's attraction and increase the repulsive force between particles causing swelling, thus increasing the volume of the clay. This phenomenon can occur in poorly cemented shales in the presence of water. Additionally, some clay minerals like montmorillonite, vermiculite, and chlorite admit water into the interlayers of the clay minerals causing swelling. The magnitude of this swelling is much larger than the swelling of the aggregates due to water between them. A schematic representation of these two types of swelling presented by Shamburger et al., (1975) is shown in Figure 2.2.1.

Dispersive clay minerals. Dispersion of clay materials is caused by the swelling and disaggregation of clay minerals, followed by erosion and removal of the dispersed material. The exchangeable cations of the clay are responsible for producing dispersion. Sherard et al., (1972) in the study of piping of earthen dams reported that good correlation exists between the dissolved salts in the saturation extract and exchangeable sodium percentage (ESP).

Figure 2.2.2 from Sherard et al., (1972) presents the performance of earth dams with respect to chemical environment. [ESP is defined as the ratio of Na concentration in meq/100 gm to CEC (total cation exchange capacity) in meq/100 gm, and percent sodium as the ratio of Na concentration in meq/100 gm to $Ca^{++} + Mg^{++} + Na^{+} + K^{+}$ concentration in meq/100 gm of the saturation extract]. Dispersion is also related to



a. CLAY MINERAL CRYSTALLITE WITH PERIPHERAL WATER ENVELOPE



b. CLAY MINERAL CRYSTALLITE WITH PERIPHERAL WATER ENVELOPE AND INTERLAYER WATER

FIGURE 2.2.1 RELATIONSHIPS BETWEEN CLAY MINERALS AND SORBED WATER (FROM SHAMBURGER et al., 1975)

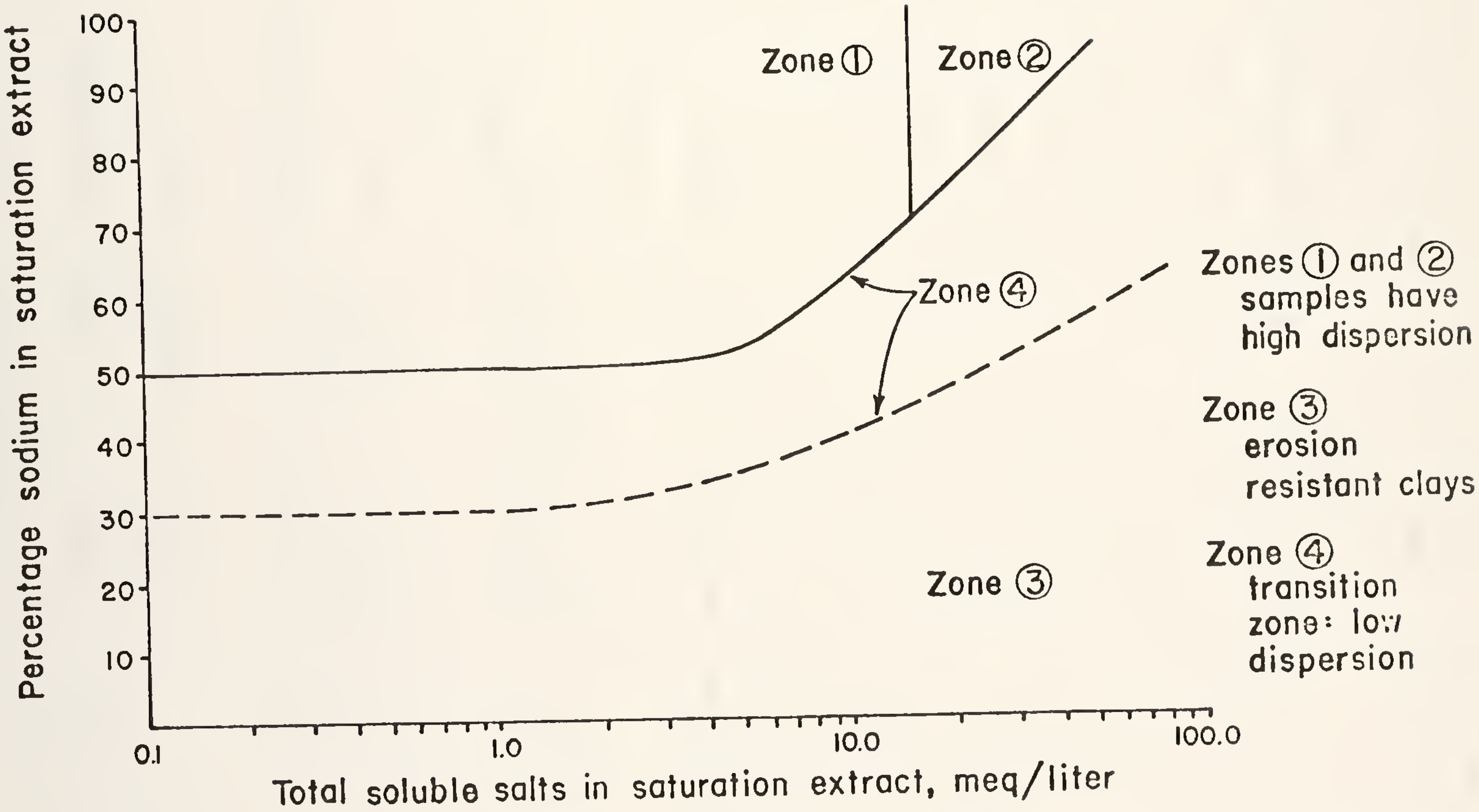


FIGURE 2.2.2 SUMMARY OF CORRELATION BETWEEN TEST RESULTS AND DAM EXPERIENCE (AFTER SHERARD et al., 1972)

the relative permeability of the embankment. Greater dispersion in embankments can be expected if they are highly permeable, provided that they have dispersive clay minerals present.

Clay mineral weathering. Deterioration of shales in embankments can be caused by the mineralogical chnges that take place in clay minerals due to the infiltration of water into the embankments. These changes include hydration of clay crystallites and exchange of cations. Clays which have tendencies for swelling are affected the most. As this process requires movement of water in the system, permeability of the compacted embankment controls the rate of deterioration to some extent, i.e., shales placed as rock fills deteriorate faster than the ones placed as soil fills. The water introduced into the interlayer clay crystallites causes the clay mineral to be more plastic and reduces the strength. Deterioration of illite in acidic environments is given below (from Shamburger et al., 1975):

illite + iron sulfate + water + oxygen

$$4KAl_3Si_3O_{10}(OH)_2 + 12FeSO_4 + 54H_2O + 3O_2 \longrightarrow$$

jarosite + aluminum hydroxide + silica hydroxide + sulfuric acid

$$4KFe_3(OH)_6(SO_4)_2 + 12Al(OH)_3 + 12Si(OH)_4 + 4H_2SO4$$

illite + sulfuric acid + water

$$KAl_3Si_3O_{10}(OH)_2 + 2H_2SO4 + 6H_2O \longrightarrow$$

Alunite + silica hydroxide

$$KAl_3(OH)_6(SO4)_2 + 3Si(OH)_4$$

Both jarosite and alunite are soluble in water and are easily removed from the embankment by solution.

Cement Weathering. The cementing agent which binds the clay minerals, silt and sand size particles in the shale is an important factor in weathering. The problems that are caused by the removal of cement in the shales have been identified (Shamburger et al., 1975) as: (1) admission of water which results in dispersion and weathering of clay minerals; (2) loss of strength between grain-to-grain contacts; and (3) crystallization forces of the newly formed clay minerals. The stability of cementing minerals in the oxidizing and chemical environment as defined by Eh (oxidation and reduction potential) and pH (H^+ ion concentration) respectively, is given in Table 2.2.4 (from Shamburger et al., 1975). Calcite, dolomite and pyrite are unstable, whereas quartz, silica, oxides, and hydroxides of iron and aluminum are stable. Calcite, dolomite and gypsum are attacked by the solution action whereas pyrite and other sulfides undergo an oxidation process causing weathering. The reaction of carbonates in an acid environment is given below.

calcite + carbonic acid

$$CaCO_3 \quad + \quad H_2CO_3 \qquad\qquad Ca^{++} \quad + \quad 2(HCO_3)^-$$

calcite + sulfuric acid + water gypsum + carbonic acid

$$CaCO_3 \quad + \quad H_2SO_4 \quad + \quad 2H_2O \qquad\qquad CaCO_4 \cdot 2H_2O + H_2CO_3$$

Pyrite reacts with water in the presence of oxygen to produce iron sulfate and sulfuric acid, and this sulfuric acid may react in turn with

Table 2.2.4 Mineral Cements and their Stable Environments (from Shamburger et al., 1975)

Mineral Cement	pH	Eh	Stable in Embankment
Calcite	≥ 7.8	Variable	No
Dolomite	≥ 7.8	Variable	No
Siderite	Variable	Variable	Variable
Gypsum	≥ 7.8	Variable	Variable
Quartz	More or less Eh and pH	independent of	Yes
Silica (amorphous)	More or less Eh and pH	independent of	Yes
Pyrite	4 to 8	Reducing	No
Iron oxides & hydroxides	≥ 7.8	Oxidizing	Yes
Aluminum oxides and hydroxides	More or less pH and Eh	independent of	Yes

the carbonates as shown above. Hydration accompanies their reaction causing swelling because of increased volume of hydrated sulfates.

Crystallization pressures. Crystallization pressures are developed during processes such as the formation of gypsum by the reaction of calcite with sulfuric acid. Surface disruptions on the fracture plane have been reported due to the crystallization of gypsum. The formation of new minerals causes a change in volume and an increase in internal pressure. Table 2.2.5 gives the increase in volume of crystalline solids due to weathering reaction.

Unloading. The release of overburden and confining pressure upon excavation of the rock causes the material to increase in void ratio. Well cemented, older rocks which have undergone changes in mineralogy tend to exhibit less rebound compared to the young, uncemented rocks. The state of stress in the existing rock formation commonly controls the behavior of the rocks excavated and placed in embankments. Stress relief also promotes entry of water into the joints, fractures and defects; entry of water enhances the weathering processes.

Biological effects. The effects of micro-organisms does not seem to be applicable to embankment materials whereas it might have some influence in weathering of intact rock. Micro-organisms have been reported to alter the chemical equilibrium of the system. These organisms tend to oxidize iron and sulfur compounds causing the weathering of these cementing agents. De-icing agents used on pavements have been reported to cause weathering of shales in embankments (Shamburger et al., 1975). The commonly used de-icing agents [sodium chloride, calcium chloride, urea $(Ca(NH_2)_2)$, ethylene and propylene glycol, etc.] enter

Table 2.2.5 Volume Increases of the Crystalline Solid Phases of Selected Chemical Weathering Reaction. (from Shamburger et al., 1975)

Weathering Reaction		Volume Increase of Crystalline Solids, Percent
Original Mineral	New Mineral	
Illite	Alunite	8
Illite	Jarosite	10
Calcite	Gypsum	60
Pyrite	Melanterite	536

the embankment in the form of a solution and tend to cause weathering by ion exchange and dispersion phenomena.

2.3 Slaking and Degradation

The deterioration of shale in the embankment may be caused principally by two processes: slaking and degradation. These two processes are not mutually exclusive, but may be complimentary.

2.3.1 Slaking Tests

Slaking is defined in the dictionary of geological terms (American Geological Institute, 1976) as "loosely, the crumbling and disintegration of earth materials when exposed to air or moisture. More specifically, the breaking up of dried clay when saturated with water, due either to compression of entrapped air by inwardly migrating capillary water, or to the progressive swelling and sloughing off of the outer layers." Slaking is measured in the laboratory by the percentage of weight retained or lost through a given sieve as a result of soaking in water. A number of tests following this concept have been developed by various investigators. Slaking of soil aggregates is studied by placing a sample on a screen and subjecting it to slaking by the impact of a water drop (McCalla, 1944). Slaking is evaluated in the classification test proposed by Deo (1972) by: the 5 cycle slaking test, the 500 revolution dry Slake Durability Test, and the modified sodium sulfate soundness test. The 5 cycle slaking test involves repeated wetting and drying of shale material retained on a #10 sieve. The percentage dry weight of material lost at the end of the fifth cycle gives the slaking index. The Slake Durability Test, developed by Franklin (1970), uses a rotating drum made of 2mm mesh, and the percentage by weight of material retained at the end of a number of revolutions in water gives the Slake Durability Index. The modified sulfate soundness test uses a 50% solution of sodium sulfate and involves five cycles of soaking the shale specimen in this solution for 16 to 18 hours followed by oven drying. The percentage weight of material retained on the 12.5mm (1/2 in) and 8.0mm (5/16 in) sieves gives the sodium soundness index. (Refer to Deo, 1972 for test details.)

Chapman (1975) reviewed several tests (rate of slaking test by Morgenstern and Eigenbrod, ethylene glycol test used by Pennsylvania Department of Transportation, Washington Degradation test, ultrasonic cavitation and Los Angeles Abrasion test) and reported that slake index, slake durability, and rate of slaking tests were most practical in identifying "problem shales" for embankment construction.

Noble (1977) used a 25% solution of sulfuric acid as the slaking fluid and identified shales from Virginia which were non-durable. Both the slaking test and the Slake Durability Test were unable to identify these shales as non-durable. In his tests, the sulfides in the shale reacted with water producing sulfuric acid which deteriorated the shales further by solution action and hydrolysis of weathering process. Therefore, this test simulates the field weathering process for shales which are rich in sulfides.

2.3.2 Slaking Mechanisms

Terzaghi and Peck (1967) attributed the slaking phenomenon to the compression of entrapped air in the pores as water entered these pores. This entrapped air in the pores exerts tension on the solid skeleton, causing the material to fail in tension. The behavior can be recognized in the case of soil aggregates and poorly cemented (i.e., compacted) shales and mudstones. Moriwaki (1975) found that slaking of compacted kaolinite can be attributed to this mechanism. There have been cases (Badger et al., 1956; Nakano, 1967) where this mechanism did not satisfactorily explain the observation.

Clay surface hydration by ion adsorption has been suggested as the second mechanism causing slaking through swelling of illite, chlorite and montmorillonitic clays (Chenevert, 1970). Differential swelling due to hydration or osmotic swelling, is reported to be the main cause of slaking in expansive materials (Moriwaki, 1975). Tschebotarioff (1973) defines slaking as a surface phenomenon in the following way "...the clay layer at the exposed surface swells first and therefore expands more than the adjoining inner layers, the induced relative displacements are liable to detach the surface layer and cause it to disintegrate and slough away. The process can then be repeated and gradually progress from the surface inward."

Removal of cementing agents in the case of shales, siltstones, and mudstones by the dissolving action of the moving ground water is considered as the third mechanism causing slaking (Badger et al., 1956; Moriwaki, 1975). The pH of the percolating ground water, the presence of oxygen, carbon dioxide, and other minerals present in the shales control the slaking due to this mechanism.

No single mechanism can be considered as the dominant cause for slaking of shales. A combination of the above mentioned mechanisms by either one triggering the other or each occurring independently is the most likely. The composition and the environment in which the shale is placed determines the principal mechanism causing the failure.

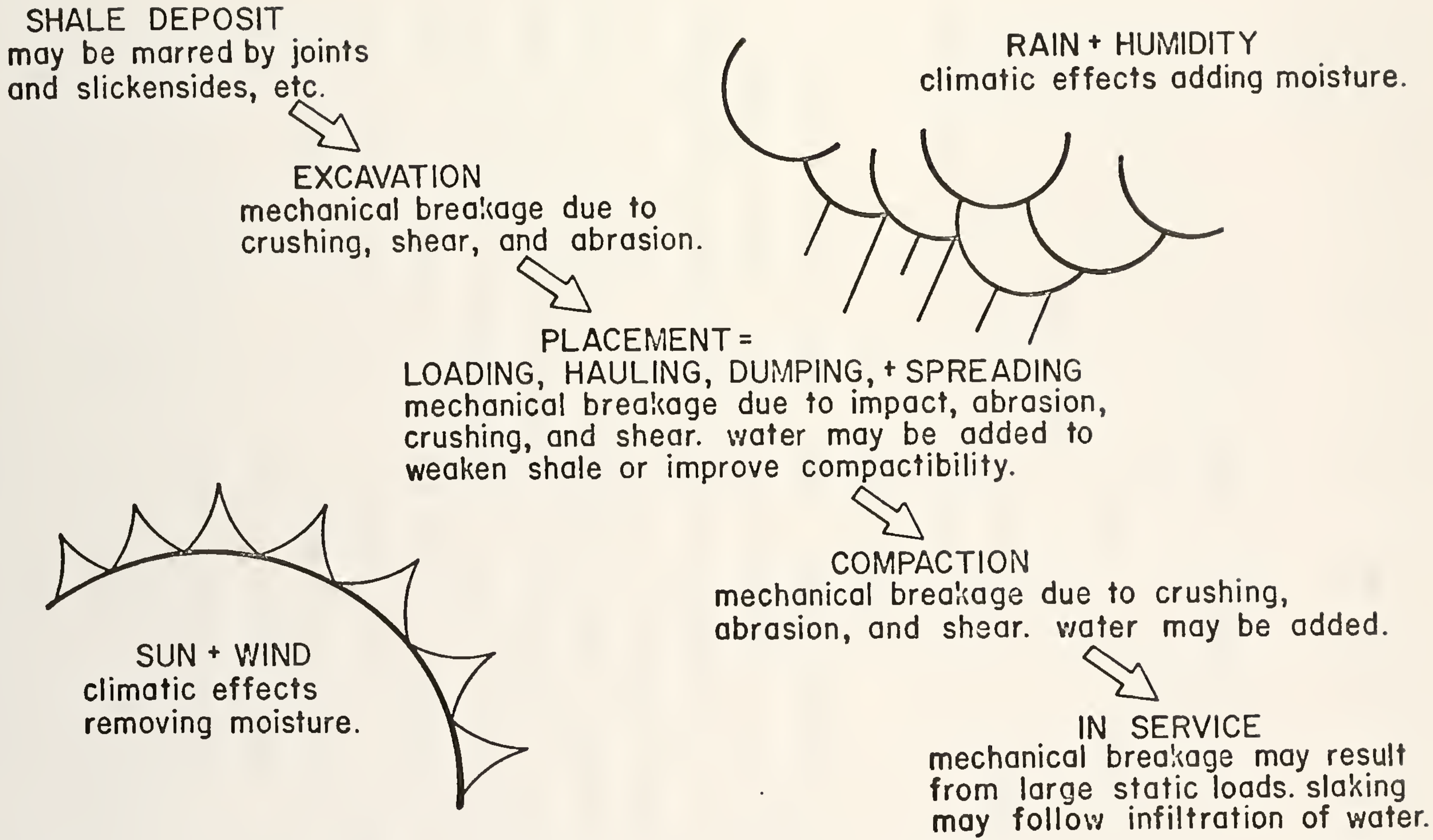
2.3.3 Degradation

Degradation is defined as the reduction in aggregation size due to construction operations and post-construction loadings. Figure 2.2.3 taken from Bailey (1976) describes various stages at which a shale in the embankment undergoes degradation, viz., mechanical breakdown due to impact, abrasion, crushing, and shear during excavation, placement, compaction and during service. Slaking may also take place during compaction and in service and thereby accelerate shale breakdown.

Bailey (1976) investigated four compaction techniques in the laboratory and concluded that the static compaction test was preferred as a means to evaluate the degradation caused by compaction. The index used to evaluate degradation was the index of crushing, which was sensitive and gave better correlations with the compaction effort. Scleroscope hardness and point load strength tests were also performed on different shales, and it was found that the point load strength had good correlation with other properties and was sensitive to moisture content. The point load strength test could be used as a relative degradation test where enough test data are available.

Hale (1979) investigated the effects of initial gradation, maximum aggregate size, and moisture on shale degradation and compacted density. Point load strength test results on non-durable shales were correlated with degradation values from the standard impact compaction test.

Abeyesekera (1978) studied one-dimensional compression and collapse on wetting of New Providence shale samples, and concluded that the ratio of long term to immediate collapse could be used as an indicator of compaction degradation. It was also noted that the ratio of immediate collapse on wetting to the deformation prior to wetting gives an index



SHALE DEPOSIT
may be marred by joints and slickensides, etc.

EXCAVATION
mechanical breakage due to crushing, shear, and abrasion.

PLACEMENT =
LOADING, HAULING, DUMPING, + SPREADING
mechanical breakage due to impact, abrasion, crushing, and shear. water may be added to weaken shale or improve compactibility.

COMPACTION
mechanical breakage due to crushing, abrasion, and shear. water may be added.

IN SERVICE
mechanical breakage may result from large static loads. slaking may follow infiltration of water.

RAIN + HUMIDITY
climatic effects adding moisture.

SUN + WIND
climatic effects removing moisture.

FIGURE 2.2.3 FACTORS INFLUENCING DEGRADATION OF SHALE USED TO CONSTRUCT EMBANKMENTS (FROM BAILEY, 1976) of rapid slaking under load upon wetting. More test data on different shales are required before this test could be used as a degradation index test.

2.4 Additives Used in Stabilization

Various additives have been used to improve soil properties. Additives have been used to improve strength, reduce permeability, control settlement, prevent erosion, and in some cases hasten a chemical reaction. Some of the additives that are discussed in this section include inorganic salts, lime, grouts, and chemical solutions.

2.4.1 Inorganic Salts

Many salts ($NaCl$, $CaCl_2$, $NaNO_3$, $NaHCO_3$, NaH_2PO_4, $BaCl_2$, $MgCl_2$, KCl, $KMnO_4$, Na_2SiO_3) have been studied in the laboratory as potential stabilizers, but economics have restricted use in the field to only a few of these. $NaCl$ and $CaCl_2$ are the common salts that are cited in the literature as stabilizing agents (Thornburn and Mura, 1969). The salt solution increases the concentration of electrolyte in the pore water, and substitutes higher valence ions for those of lower valence. The increase in the electrolyte concentration causes the double layer to depress and the repulsive forces to decrease. The effect of $NaCl$ on the plastic and liquid limits have been found to be dependent on the soil type (Thornburn and Mura, 1969). There is a strength increase observed with soils stabilized with $NaCl$. It has also been observed that $NaCl$ reduces or eliminates frost heave by lowering the freezing point of water and decreasing the permeability. No cementation is said to occur between the $NaCl$ and the soil, however, an increase in compressive strength suggests the possibility of cementation between particles due to salt addition. Use of NaCl requires fine grained material and freedom from organic matter to react most efficiently.

$CaCl_2$ has also been used successfully in stabilization (Thornburn and Mura, 1969). Addition of $CaCl_2$ in excess of 1% has been shown to decrease the density, as the calcium ions increase the repulsive forces by changing the charge on the particle from negative to positive. $CaCl_2$ has been used for frost heave protection.

Inorganic salts have been used as trace amounts in lime, lime-fly ash and cement stabilization. The postulated mechanisms involved are: (a) acceleration of the pozzolanic reaction, (b) production of secondary cementitious products, and (c) combination with the primary, or pozzolanic, cementitious products (Mateos and Davidson, 1961).

The type of soil is an important factor in salt stabilization: sodium salts seem effective with calcareous soils.

2.4.2 Lime

Lime stabilization is used to hasten construction operations, modify the subgrade, and improve strength and durability (resistance to freeze-thaw action) of fine grained soils. The most common types of lime used in stabilization are: hydrated high calcium lime, $Ca(OH)_2$; monohydrated dolomitic lime, $Ca(OH)_2 \cdot MgO$; calcitic quick lime, CaO; and dolomitic quick lime, $CaO \cdot MgO$. Hydrated lime is used more frequently than quick lime.

The reactions taking place in soils treated with lime are complex. However, several explanations have been presented in the literature. Cation exchange and flocculation agglomoration are said to occur rapidly, resulting in immediate changes in soil plasticity, workability and the immediate uncured strength. Depending on the soil type and temperature, pozzolanic reaction will occur which forms various cementing agents resulting in increased strength and durability of the soil-lime mixture. Pozzolanic reactions are time dependent and hence there are increases in strength with time.

Flocculation and agglomeration produce an apparent change in texture with the clay particles "clumping" together into larger sized "aggregates" (Transportation Research Circular, 1976). The mechanism of flocculation and agglomeration is explained by Herzog and Mitchell (1963) as an ion exchange phenomenon where the electrolyte concentration of the pore fluid is increased by the exchange of calcium ions. Diamond and Kinter (1965) suggest that the rapid formation of cementing material of calcium aluminate hydrate develops flocculation and agglomeration tendencies in soil-lime mixtures.

Soil-lime pozzolanic reaction is between lime, water and sources of soil silica and alumina. The sources of silica and alumina in soil are quartz, fledspars, micas, and other alumino-silicate minerals.

Eades and Grim (1966) suggested that elevation of the pH level of the soil- lime mixture causes the silica in the clay minerals to be dissolved out of the structure and to combine with the calcium to form calcium silicate. This process continues as long as $Ca(OH)_2$ and silica are available to react in the soil-lime mixture. Diamond et al., (1964) concluded that in a highly alkaline soil-lime system, the reaction involves a dissolution at the edges of the silicate particles followed by the precipitation of the reaction products. It is also suggested (Diamond and Kinter, 1965; Ormsby and Bolz, 1966) that surface chemical reactions can occur and new phases may be formed on the surface of clay particles.

The degree to which the soil-lime pozzolanic reaction can take place depends on the natural soil properties. Thompson (1966) has termed those soils that react with lime to produce a substantial strength increase [greater than 344.75kN/m^2 (50 psi) following 28 day curing at 22.8^{o}C] as reactive and those that display limited pozzolanic reactivity [less than 344.75kN/m^2 (50 psi) strength increase] as nonreactive. Some of the major soil properties and characteristics which influence the lime-reactivity of a soil are (Transportation Research Circular, 1976): soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquioxide ratio, and silica-alumina ratio.

Lime has been successfully used by mixing with pulverized shale and compacting to form an erosion resistant lining for the "Black Thunder Slot" storage in Wyoming (Jones et al., 1978).

2.4.3 Grouts

Grouting is a method of injecting suitable mixtures of cement and water or other admixtures into the rock mass to prevent excessive settlement and to reduce the seepage of water. In the case of shale embankments grouting is employed to reduce excessive settlements caused by deterioration of non-durable shales, by reducing the permeability of the compacted material. The common grouts used in the stabilization of shale embankments are discussed in this section (Bragg and Zeigler, 1975).

Cement Grouts. Cement grout has been used very successfully in various instances, especially in railroad embankments (Smith and Peck, 1955) and in the construction of a seepage barrier for the foundations of dams (Krynine and Judd, 1957). Cement grout has also been used by the Ohio Department of Transportation as a remedial measure to stabilize a shale bridge approach fill that experienced excessive settlement [approximately 0.45m (1.5 ft), Bragg and Zeigler, 1975]. Cement grout is injected into the fill through bore holes and the spacing of these holes depends on the size of the voids present in the shale embankment mass. Even though cement grouts are more efficient in more permeable soils such as sands and gravels, they have also been used successfully in stabilizing slides in clay soils. The grout does not penetrate the voids of the clay or fissures in the clay, but penetrates along the failure surface, and forms a solid layer of hardened cement paste after it sets up. The grout entering the failure plane may also displace the water which is the main cause for deterioration.

Cement grout can be used to treat two extreme problem conditions; first, for controlling excessive settlement due to the presence of large voids in the shale mass and second for treating shear failures in deteriorated shales where the grout is concentrated on the failure surface. When grout is applied to sidehill fills, supplementary drainage measures are essential to prevent new failures from developing below the treated area.

Chemical Grouts. The main advantage of using chemical grouts is that they have a very low viscosity, hence there is an increased penetration compared to the cement grouts. Further, the setting time can be controlled by using appropriate catalysts. The common types of chemical grouts are: (a) sodium silicates, (b) chrome-lignins, and (c) organic monomers (Bragg and Zeigler, 1975). Chemical grouts are used to fill the voids to reduce settlement and decrease the permeability. There is also an increase in strength that results from chemical stabilization.

Chemical grouts may be beneficial in instances where cement grouts are unsatisfactory because of the low permeability in clayey soils or lack of well developed failure surfaces. However, chemical grouting is very expensive compared to cement grouting. An estimate of cost reported in Bragg and Zeigler (1975) after Robnett et al., (1971) is: $30 to $125 for chemical grouting and $10 to $20 for cement grouting per cubic meter of treated material. Chemical grouts require precise mixing, and it is suggested (Bragg and Zeigler, 1975) that specialists be consulted.

Drill-hole Lime. Lime has been successfully used as a stabilizing agent to reduce plasticity and increase the workability and strength of clayey soils. Lime slurry placed in drill holes is used as a remedial measure to stabilize embankments undergoing distress, subgrades in expansive clays, and landslides in soft clay soils. The main disadvantage of this method is that both lime migration and strength increases are slow, time-dependent processes. The reaction of lime with soil is the same as described in soil-lime stabilization: initial strength increase due to base exchange and flocculation, and long-term, time-dependent pozzolanic reactions. Drill holes (of minimum diameter 30cm) are spaced at 1 to 1.5m centers as the penetration of lime is limited to a distance of 5 to 8cm. Bragg and Zeigler (1975) gives a brief review of the cases where drill-hole lime has been successfully used to stablize distress in embankments and subbases.

Lime Slurry Pressure Injection. This method was developed to improve the penetration of lime in clay soils. Lime slurry is pumped into a soil layer under a high pressure (1 400 kN/m^2) through hollow rods with a special injection nozzle at the tip. Lime slurry is introduced at successive intervals until the desired depth is reached. A state-of-the-art assessment of this technique is presented by Robnett et al., (1971) and Thompson and Robnett (1975). The penetration of lime is in nearly horizontal seams in the clay soils, and the strength increase may not be achieved immediately after the injection. There may even be a temporary reduction in strength due to swelling of the clays on introduction of large volumes of water. Potential problems include difficulties involved in pushing the injection rod through shale fills containing large chunks of material and the introduction of large volumes of water with the lime into the fill (Bragg and Zeigler, 1975). Field trials are necessary if this method is to be used successfully in shale embankments.

Ion Exchange. Ion Exchange is a technique patented by Ion Tech, Inc., of Daly City, California. This method has been used successfully in treating landslides (Arora and Scott, 1974). The technique consists of treating clay minerals with concentrated chemical solutions (selected by Ion Tech, Inc., based on laboratory tests on the material), and is applied to the soil through cracks and/or drill holes. Successful treatment by this method requires a clearly defined surface or zone of failure, saturation of clay and cracks, or boring for the introduction of the chemicals (Mearns et al., 1973). Bragg and Zeigler (1975) give a review of the cases where this technique has been used to treat landslides. This method may be used to treat shale embankments; however, field tests are necessary.

CHAPTER 3 DESCRIPTION OF MATERIALS STUDIED

3.1 Shales of Indiana

In this section a brief review of the shales encountered in the construction of highway embankments in the State of Indiana is presented. Shales in Indiana are commonly encountered in highway construction as they often crop out or lie buried under a shallow overburden. Montmorillonite clay is usually associated with recent sediment and as such is not found in any of the Indiana shales which were deposited in the Paleozoic Era.

Shale descriptions are taken principally from Harrison and Murray (1964).

3.1.1 Shales of Ordovician Age

This is the oldest geologic rock system found in Indiana (Deo, 1972). The Dillsboro Formation is from this geologic system. Table 3.1.1 gives the description of these shales and references to the studies made where further information on these shales can be obtained.

3.1.2 Shales of the Silurian Age

The rocks of the Silurian geologic system consist primarily of a succession of limestones and dolomites. Only one shale unit in the Silurian in Indiana has been assigned the rank of formation. That is the Waldron Shale which was not studied in this investigation. The shale unit which was selected is the Osgood member of the Salamonie Dolomite. It is classified as being of member status due to its relatively small lateral extent. The shales of this member are generally calcareous or dolomitic. See outcrop area on page 68.

Table 3.1.1 Indiana Shales Studied at Purdue University

Investigated by	Shale #	Geologic Group Formation Member	System/ Series	From	Description	Classification	Comments
Dec. 1972	Klondike	Borden Gr.	Mississippian/Osage*			Intermediate 1	
	Attica	" "	"			" 1	
	67A	" "	"			" 2	
	67B	" "	"			" 2	
	37A	" "	"			" 1	
	37B	" "	"			" 1	
	Paoli 3	" "	"			" 2	
	Paoli 5	" "	"			" 2	
	Paoli X	" "	"			Soil-like	
	Paoli Y	" "	"			" "	
	Cannelton	Buffalo Wallow Gr.	Mississippian/ Chester	Sm. Dam Emb.		" "	
	I-65	New Albany	Devonian	Hwy. Emb.		Intermediate 2	
	Scottsburg	" "	"			" 2	
	Lynnville	Carbondale Group	Pennsylvanian/ Allegheney			" 2	
	I-74	Dillsboro	Ordovician/ Cincinnatian	Hwy. Emb.		Soil-like	
Chapman, 1975	1	Hardinsburg (lower)	Mississippian/ Chester	I-64 at Sulphur	Gray, soft	Soil-like	Excavated by ripping
	2	New Albany	Mississippian/ Kinderhookian	I-265 at New Albany	Black, very hard	Rock-like	Blasting
	3	Mansfield	Pennsylvanian/ Pottsville	I-64 at St. Croix	Med. Gray, Med. hard	Soil-like	Ripping
	4	Palestine	Mississippian/ Chester	I-64 at St. Croix	Gray, Med. hard	Soil-like	
	5	Kope	Ordovician/ Cincinnatian	I-74 near Dearborn Co.	Gray-green, soft	Soil-like	

*The Meramec and Osage Series have been recently combined into the Valmeyeran Series by the Indiana Geologic Survey, but the continued use of the older terms is still acceptable.

Table 3.1.1 Continued

Investigated by	Shale #	Geologic Group Formation Member	System/ Series	From	Description	Classification	Comments
	6	Borden	Mississippian/ Osage*	Near Klondike	Gray-Lt. Tan, Hard	Rock-like	
Bailey, 1976	1 2	Mansfield	Pennsylvanian/ Pottsville	Perry Co.	Gray, flaggy	Soil-like	
	3	Clore	Mississippian/ Chester	Perry Co.	Dk. Gray, flaggy	Soil-like	
	4 5 6	Palestine	Mississippian/ Chester	Perry Co.	Dk. Gray, flaggy	Soil-like	
	7	Waltersburg	Mississippian/ Chester	Crawford Co.	Gray, soft	Soil-like	
	8	Hardinsburg (lower)	Mississippian/ Chester	Perry Co.	Gray, flaggy	Soil-like	
	9	Big Clifty	Mississippian/ Chester	Orange Co.	Gray, flaggy	Soil-like	Rock Core Boring
	10	Haney	Mississippian/ Chester	Orange Co.	Yellow-tan, flaggy	Soil-like	
	11 12	Borden Gr.	Mississippian/ Valmeyeran	Attica Fountain Co.	Dk. Gray, flaggy, fossiforous	Soil-like	Rubber tired front end tractors
	13	Borden	Mississippian/ Valmeyeran	Tippecanoe Co.	Dk. Brown, Tan fairly massive	Rock-like	
	14 15	New Providence	Mississippian/ Valmeyeran	Floyd Co.	Dk. Gray, massive	Soil-like	Bulldozer
	16	New Albany	Devonian	Floyd Co.	Dk. Gray, black, massive	Rock-like	May contain crystals of Pyrite
	17	New Albany	Devonian/ Senecan	Marion Co.	Green-gray, flaky	Soil-like	
	18 19	New Albany (Morgan Trail Mbr)	Devonian/ Senecan	Marion Co.	Gray, massive	Rock-like	
	20	Kope	Ordovician/ Cincinnatian	Dearborn Co.	Gray-green, flaggy	Soil-like	
	21	Dillsboro	Ordovician/ Cincinnatian	Dearborn Co.	Gray	Soil-like	

Table 3.1.1 Continued

Investigated by	Shale #	Geologic Group Formation Member	System/ Series	From	Description	Classification	Comments
	22	Whitewater	Ordovician/ Cincinnatian	Union Co.	Gray, flaggy	Soil-like	
Abeyesekera, 1978		New Providence	Mississippian/ Valmeyeran	I-265 in Floyd Co.	Light, Gray, Medium, massive	Soil-like	Ripping
Witsman, 1979		New Providence	Mississippian/ Valmeyeran	" "	"	" "	"
Hale, 1979	1.	New Providence	Mississippian/ Valmeyeran	I-265 in Floyd Co.	Gray, Medium flaggy	Soil-like	
	2.	Osgood Mbr.	Silurian/Niagaran	S.R. 107	Blue-gray	Soil-like	
	3.	Palestine	Mississippian/ Chester	I-64	Brown-gray	Soil-like	
This Investigation	1	New Providence	Mississippian/ Valmeyeran	I-265 in Floyd Co.	Light Gray, Medium massive, hard	Soil-like	
	2	Mansfield	Pennsylvanian/ Pottsville	St. Croix	Med. Hard	Soil-like	
	3	New Albany	Mississippian/ Kinderhookian	I-265 at New Albany	Black, Very hard	Rock-like	Blasting
	4	Osgood Mbr.	Silurian/Niagaran	S.R. 107	Blue-gray	Soil-like	
	5	Borden Gr.	Mississippian/ Valmeyeran	Attica, Fountain Co.	Dk. Gray, flaggy	Soil-like	
	6	Palestine	Mississippian/ Chester	I-64	Brown-gray	Soil-like	
	7	Hardinsburg (lower)	Mississippian/ Chester	I-64 at Sulphur, Perry Co.	Gray, soft	Soil-like	
	8	Borden Gr.	Mississippian/ Osage	Near Klondike, Tippecanoe Co.	Gray, hard	Rock-like	

3.1.3 Shales of the Devonian Age

As with the Silurian system, the Devonian system is composed mainly of limestones and dolomites. The main shale formation which is commonly encountered in construction projects is the New Albany Shale. This formation is predominantly Devonian, but has the distinction of overlapping two geologic systems, the Mississippian and Devonian. The clay minerals are generally illite and chlorite, with kaolinite present in some areas. Quartz is the most abundant non-clay mineral. Pyrite is also commonly found. See page 68 for outcrop area.

3.1.4 Shales of Mississippian Age

Shales are very common in this system, especially in the Chester Series. However, the only shale unit which has been given the rank of formation is the lower Mississippian New Providence Shale. The shales of the Borden Group are also within the Mississippian System. The samples from the Klondike and Attica areas could not be accurately given formation names due to the absence of distinct marker beds to define the formational boundaries. The Borden Group New Providence Shale, however, is lithologically distinct enough to be readily recognizable. The shales of the Chester Series are variable in physical properties and minerology, particularly in lateral directions. In many places a shale becomes a sandstone or limestone in less than a mile and in some places much shorter distances. For this reason it is difficult to even assign a member status to the shales. See page 68 for outcrop locations.

3.1.5 Shales of Pennsylvanian Age

Pennsylvanian formations are also stratigraphically complex because of common changes from one rock type to another over relatively short distances. Pennsylvanian formations display little difference lithologically and it is therefore very difficult to accurately define formational boundaries unless one is familiar with the coal members and beds assigned to each. Two general types of shales are found in Indiana's Pennsylvanian formations: dark gray to black fine-grained thin bedded, organically rich shale and light gray silty relatively thick bedded shale. The Mansfield shale tested in this investigation is closer to the latter type. See page 68 for outcrop belt locations.

3.2 Shale Sampling Locations

The shales used in this study are New Providence (Shale #1), Mansfield (Shale #2), and New Albany (Shale #3). The New Providence Shale was sampled in May 1975 for study by Abeyesekera (1978). This shale was sampled at an elevation of 575 ft along a road excavation on I-265 in Floyd County, south central Indiana. The shale lies at the base of the Borden group, which crops out in a narrow band about 12 to 15 miles wide, from New Albany to about Lafayette. The Mansfield and New Albany Shales were sampled in spring 1974 for study by Chapman (1975). The Mansfield Shale was sampled near St. Croix, Indiana, on an I-64 contract, and was excavated by ripping. It belongs to the Mansfield Formation of the Pennsylvanian Pottsville Series and was found above the Mississippian- Pennsylvanian contact. The New Albany Shale was sampled at the base of a cut for I-265 near New Albany, Indiana, and was excavated by blasting. New Albany Shale forms a transition between Devonian and Mississippian rocks in Indiana, with some portions being a part of the Mississippian Kinderhookian Series. The Osgood Shale (shale #4) was sampled for study by Hale (1979) near the intersection of Indiana S.R. 107 and U.S. 421 in Madison, Indiana, and was excavated with a backhoe. Osgood Shale, a blue-gray, hard and flaggy shale, is a member of the Salomonie Dolomite and lies at the base of the Niagaran Series in the Silurian System. The other shales used in this study were also sampled during the previous investigations at Purdue University. Table 3.2.1 gives a brief description of the nature of these shales and the investigation during which these shales were sampled. The exact location of shale sampling sites can be obtained from the ISHC laboratory data sheets presented in Tables E-1.1 to E-1.10 in Appendix E-1.

Table 3.2.1 Description of the Shales Studied

ISHC Laboratory No.	Shale Type	Deo's Classification	Physical Nature	Sampled by
75-55731 79-55198	#1 New Providence	Soil-like	Hard and Non-durable	Abeyesekera (1978) Hale (1979)
74-54684	#2 Mansfield	Soil-like	Soft and Non-durable	Chapman (1975)
74-54621	#3 New Albany	Rock-like	Hard and Durable	Chapman (1975)
79-55199	#4 Osgood	Soil-like	Hard and Non-durable	Hale (1979)
75-55564	#5 Attica	Soil-like	Hard and Non-durable	Bailey (1976)
74-54716 79-55204	#6 Palestine	Soil-like	Soft and Non-durable	Chapman (1975) Hale (1979)
73-51703	#7 Lower Hardinsburg	Soil-like	Soft and Non-durable	Chapman (1975)
75-55315	#8 Klondike	Rock-like	Hard and Durable	Chapman (1975)

The sampling location of these shales are shown in Figure 3.2.1 on a map of Bedrock Geology of Indiana (after Wayne, 1956). All samples were stored in 27 gallon metal garbage cans with double plastic liners.

3.3 Description of Shales Studied

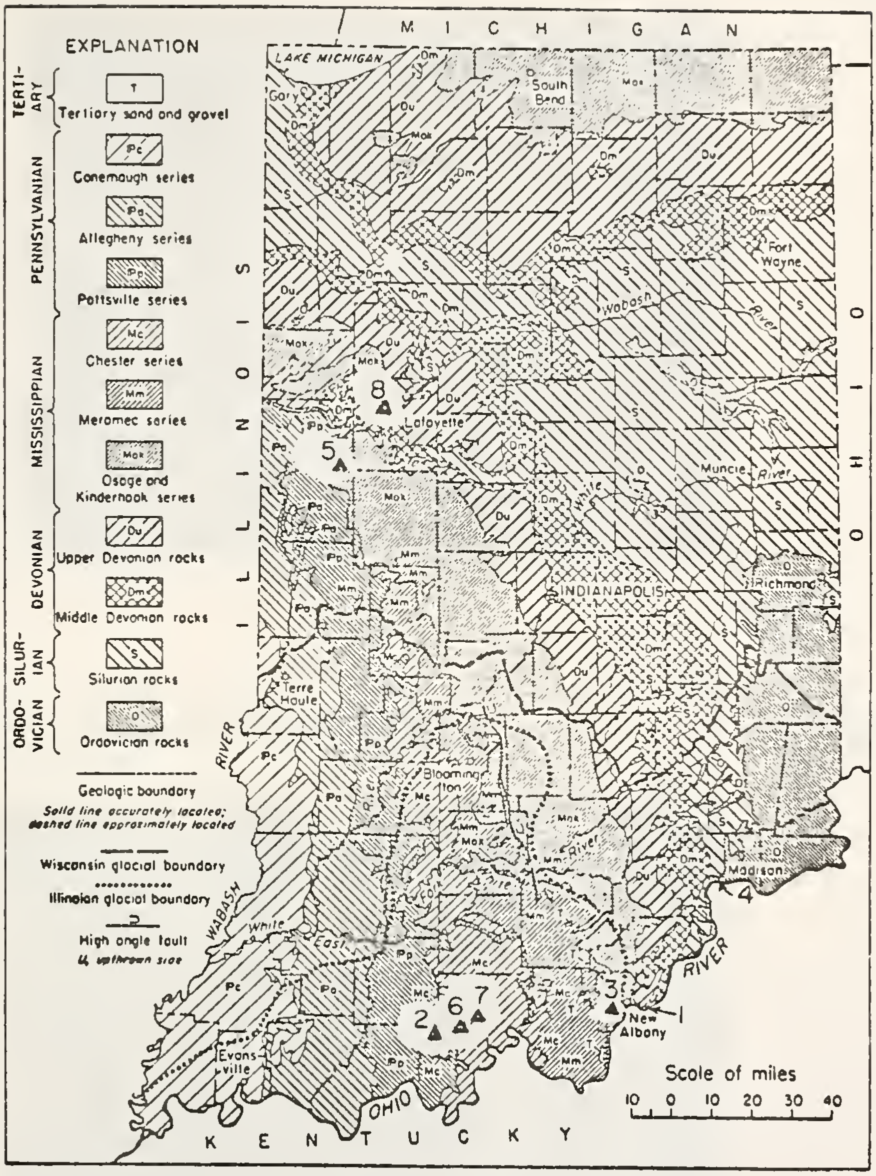
3.3.1 Megascopic and Microscopic Features

Shale #1: New Providence Shale. This shale is sandy to silty in texture and ranges in color from blue-gray to brown. It is massive to blocky on fresh surfaces, but on weathered surfaces it displays definite partings and breaks out into small pieces. The finer grained shales are softer than the coarse grained ones in the formation.

Petrographic study of thin sections showed angular quartz grains with some iron oxide flakes in a clay matrix. In a cross section along the bedding plane (Figure 3.3.1), the clay was present in the form of domains which were unevenly distributed over the section. On the section perpendicular to the bedding planes (Figures 3.3.1 and 3.3.4) alternate layers of clay groups were observed with filled channels and clay matrix with coarser silty layers which were predominantly quartz.

Shale #2: The Mansfield Shale. This shale is medium gray, silty and medium hard with a low moisture content. It is massive and does not show definite cleavage planes in hand specimens.

The thin section oriented along the bedding plane (Figure 3.3.2) showed clay uniformly distributed over the cross section with some pores that could be due to sample preparation. On a cross section perpendicular to the bedding, the bedding planes were visible under lower magnification (40x) (Figure 3.3.4). Clay and quartz were distributed

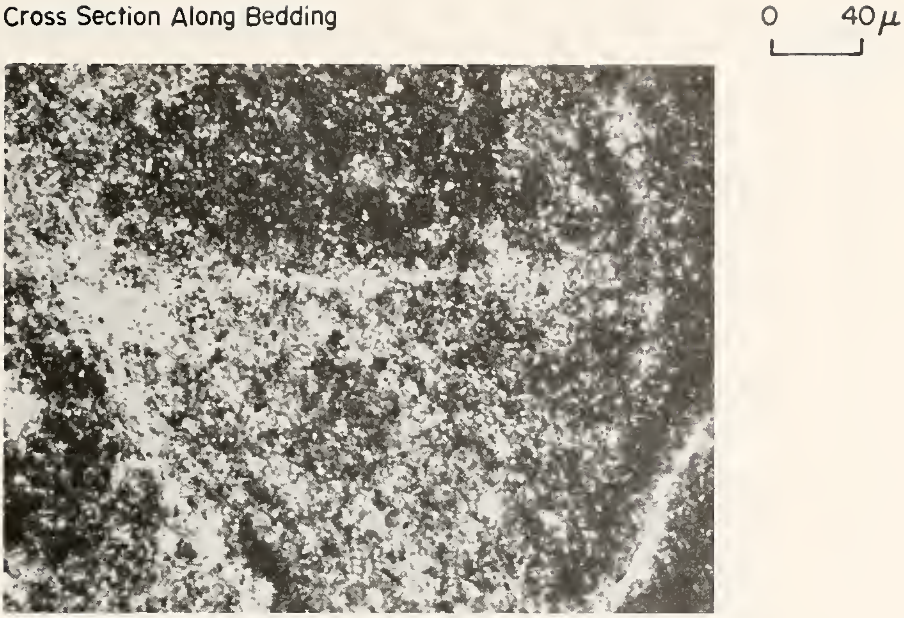


LEGEND

1. New Providence
2. Mansfield
3. New Albany
4. Osgood
5. Attica
6. Palestine
7. Lower Hardinsburg
8. Klondike

FIGURE 3.2.1 BEDROCK GEOLOGY OF INDIANA AND SHALE SAMPLING LOCATIONS

Cross Section Along Bedding



Cross Section Perpendicular to Bedding

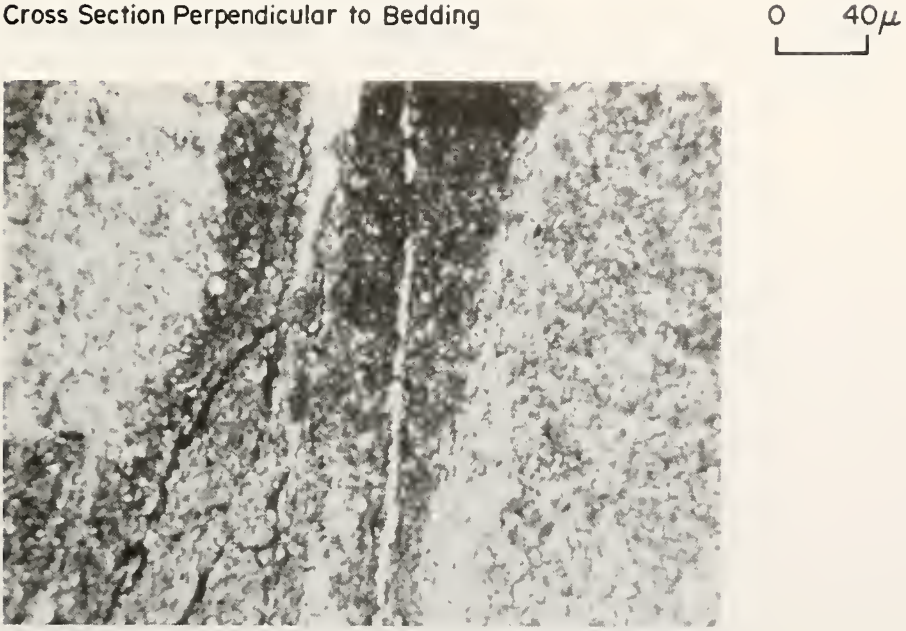
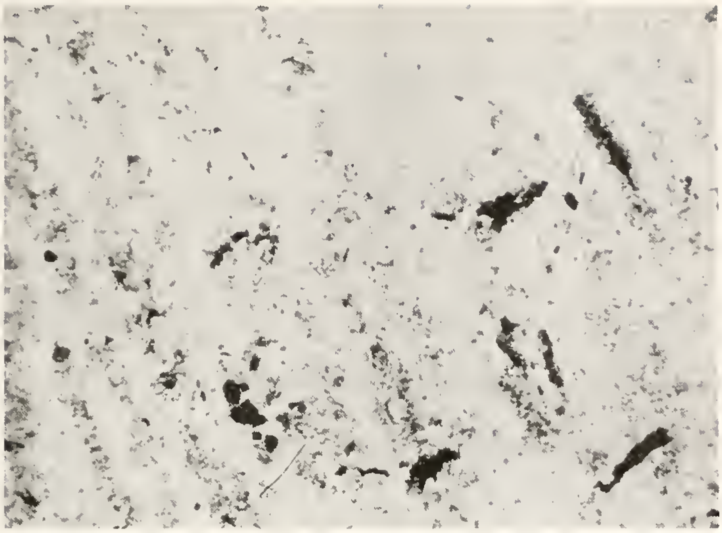


FIGURE 3.3.1 THIN SECTION OF NEW PROVIDENCE SHALE (MAGNIFICATION: 300x)

Cross Section Along Bedding



Cross Section Perpendicular to Bedding

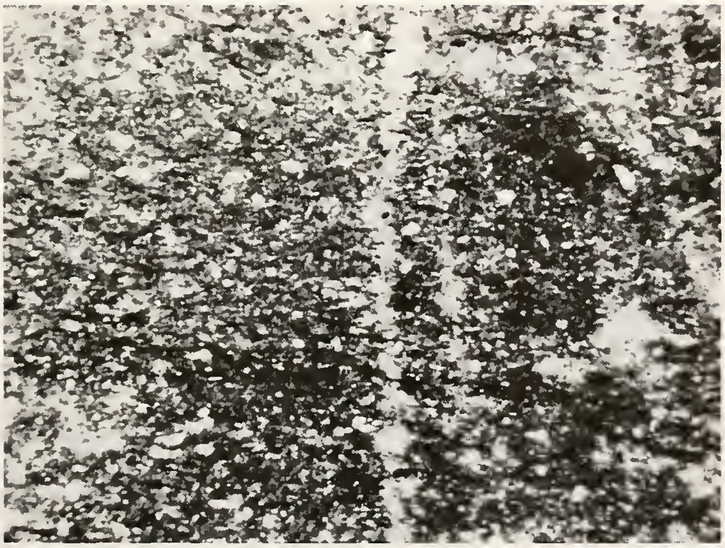


FIGURE 3.3.2 THIN SECTION OF MANSFIELD SHALE
(MAGNIFICATION : 300x)

throughout. Some of the pores present were also well distributed throughout the section.

Shale #3: The New Albany Shale. This shale is black in color, very hard, fine grained and had very little moisture when it was sampled. It is massive in nature and some pyrite seams and nodules were found throughout the shale.

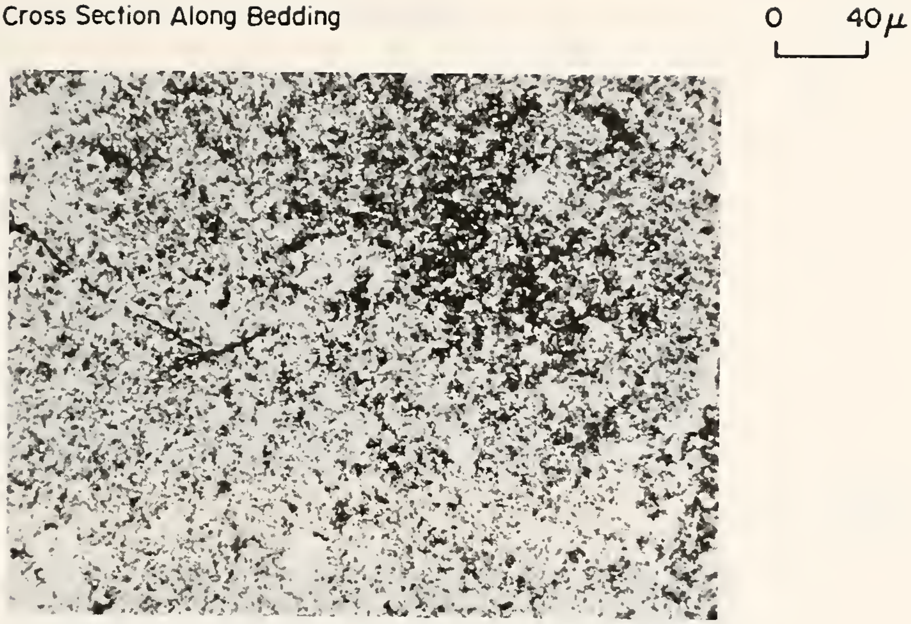
A thin section along the bedding plane (Figure 3.3.3) when viewed through a petrographic microscope showed a small amount of quartz which was uniformly distributed. Fine grained, close bands were visible at lower magnification (40x) (Figure 3.3.4) on a cross section perpendicular to the bedding. A higher magnification (300x) of the same cross section (Figure 3.3.3) showed bands of dark brown clay minerals (illite) which were preferentially oriented, along with appreciable amounts of organic material.

3.3.2 X-Ray Analysis

The X-ray diffraction analysis was conducted on powdered shale samples to identify the principal clay and non-clay fractions. An attempt was made to quantify the percentages of clay minerals present in different shales by examining the clay (less than two micrometer) size fraction.

Shale samples were powdered using the Proctor hammer in a compaction mold. The powdered samples were prepared in an aluminum mount by the modified McCreery (1942) method as described in Deo (1972). The samples for the clay mineral analysis were prepared according to the procedure outlined by Kinter and Diamond (1956).

Cross Section Along Bedding



Cross Section Perpendicular to Bedding

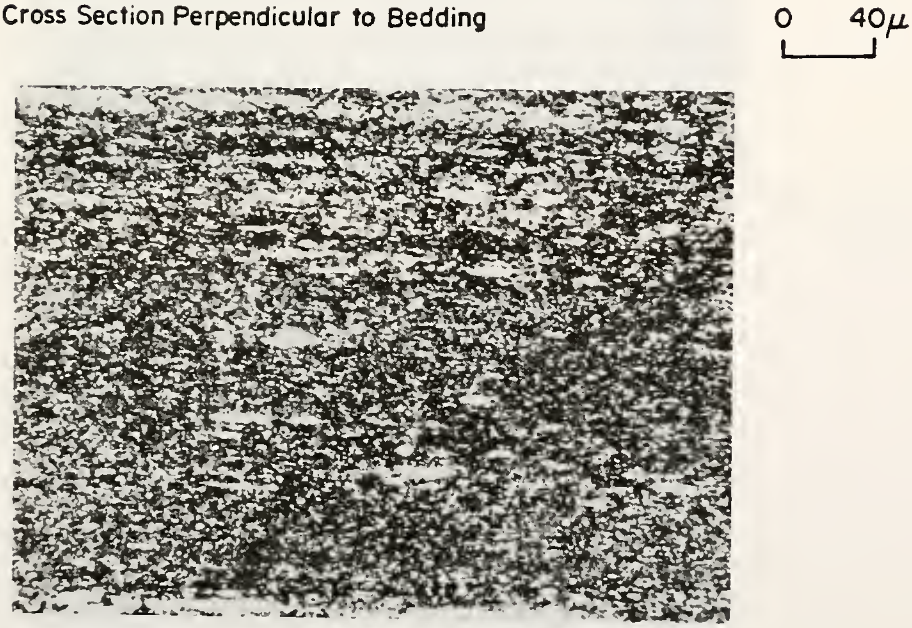
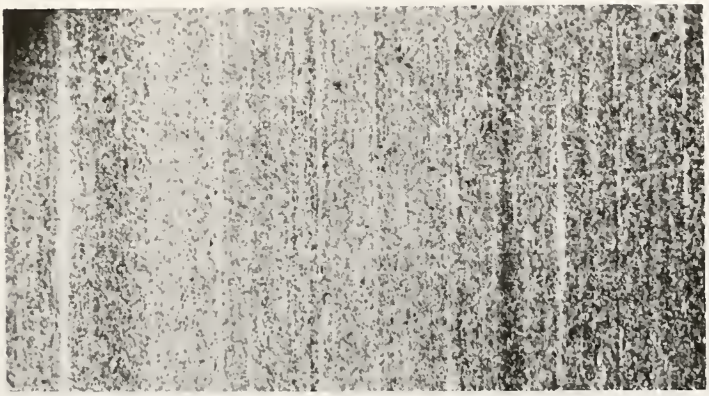


FIGURE 3.3.3 THIN SECTION OF NEW ALBANY SHALE (MAGNIFICATION: 300x)

Cross Section Perpendicular to Bedding

New Providence

Mansfield



New Albany

FIGURE 3.3.4 THIN SECTION OF SHALES STUDIED (MAGNIFICATION: 40x)

Shale #1: The New Providence Shale. This shale was found to contain quartz, illite, chlorite, traces of kaolinite, plagioclase and halite from the X-ray diffraction pattern for the powdered shale (Appendix A-1). The relative abundance of the clay minerals was estimated as described in Appendix A-2 and is presented in Table 3.3.1. The New Providence shale has illite as the predominant clay mineral (about 93%) followed by kaolinite (about 5%) and chlorite (about 3%), respectively (Appendix A-2). The mineral composition of the shale (Table 3.3.2) in this formation has been reported (Shaffer, 1979) to consist of silica (about 60%) and alumina (about 16%) and trace amounts of other oxides. The percentage clay, defined as less than five micrometers and greater than 1 micrometer, is 17-23% and percentage colloid, defined as less than one micrometer is 7-16% as reported from laboratory tests conducted by the Indiana State Highway Commission.

Shale #2: The Mansfield Shale. Analysis of the Mansfield shale based on the X-ray diffraction pattern for the powdered shale suggested that quartz, illite, kaolinite, chlorite and K-feldspar were present (Appendix A-1). The relative percentages of the clay minerals as shown in Table 3.3.1 indicate that illite is the predominant clay mineral (about 71%) with kaolinite being approximately 27% of the total percentage of clay, and chlorite in smaller percentages [less than 3%] (Appendix A-2). The mineral composition of the shale in this formation is silica (55-60%) and alumina [16-22%] (Table 3.3.2), and other oxides in trace amounts (Shaffer, 1979). The percentage of clay (of less than five micrometer and greater than one micrometer) size is 13-28% and percentage colloid (of less than one micrometer) size is 18-29% as reported from laboratory tests conducted by the Indiana State Highway Commission.

Shale #3: The New Albany Shale. The New Albany shale was found to consist of quartz, illite, kaolinite, K-feldspar and pyrite. From the X-ray diffraction pattern of tiles with clay (less than two micrometer) size deposits (Appendix A-2) examination of the X-ray diffraction pattern for the powdered shale it was determined that the clay minerals consist of predominately (Table 3.3.1) illite (about 93%) and a smaller percentage of kaolinite (about 5%), and traces of chlorite (less than 3%). The mineral composition of the shale is silica (50-60%), alumina (12-17%) as presented in Table 3.3.2, and traces of other oxides (Murray, 1955). The percentage of clay (less than five micrometer and greater than one micrometer) size is 12-27% and percentage colloid (less than one micrometer) size is 7-17% as reported from laboratory tests conducted by the Indiana State Highway Commission.

3.3.3 Chemical Analysis

Several chemical tests were run on the shales studied in this research. The tests were performed by the Soil Testing Laboratory of the Agronomy Department at Purdue University. The test procedures were those used routinely on agricultural soils. Tests were performed on ground shale samples and on extracts of diluted pore water.

The percentage organic matter, total soluble salts and ion concentrations were determined from the ground sample, and the ion concentrations of the diluted pore water were determined by extracting the pore water by means of a filter and vacuum system. Some of the procedures used in the Agronomy laboratory for these tests are given in Appendix A-4. The results of the tests are presented in Chapter 5.

Table 3.3.1 Percentage of Clay Minerals in Different Shales

Shale	Kaolinite %	Illite %	Chlorite %
New Providence	4 5 } 4.5	94 91 } 92.5	2 4 } 3
Mansfield	25 28 } 26.5	71 70 } 70.5	4 2 } 3
New Albany	1 9 } 5	99 87 } 93	- 5 } 2.5

Table 3.3.2 Chemical Analyses of the Shales from the Same formation as that of test shales

	S_iO_2	H_2O_3	Fe_2O_3	FeO	MgO	M_nO	CaO	Na_2O	K_2O	T_iO_2	P_2O_5
New Providence‡	60.2	16.3	1.98	4.23	1.98	0.06	1.78	1.11	3.78	0.87	0.11
	66.1	15.6	5.73		0.81	0.06	0.25	0.16	3.71	2.22	0.011
Mansfield‡	55.1	19.6	0.11	6.65	1.83	0.18	0.55	0.17	3.31	0.86	0.17
	59.1	16.6	0.18	5.78	1.87	0.16	2.02	0.52	2.97	0.67	0.17
	53.9	21.7	2.65	4.30	1.49	0.11	0.26	0.10	3.20	0.81	0.11
New Albany*	56.7	12.5	4.04		2.23	0.098	2.42	0.53	3.44	0.67	0.061
	62.9	15.6	3.70		2.18	0.10	2.15	0.49	4.55	0.81	0.038
	57.5	16.5	4.29		2.48	0.11	3.49	0.37	3.9	0.65	0.18
	50.3	13.5	3.57		2.42	0.089	5.72	0.19	3.2	0.5	0.086

‡ Results from N.R. Shaffer

* Results from H. Murray

ESP (exchangeable sodium percentage, the relative amount of sodium in the exchange complex), SAR (sodium absorption ratio) expressed in meq/l of the saturation extract, percent sodium (arithmetic percentage of sodium in the saturation extract), and ESR (exchangeable sodium ratio) were computed from the results of chemical tests and are presented in Chapter 5.

Other tests such as pH and loss on ignition (LOI) were conducted in the Materials Laboratory of the School of Civil Engineering and these results are also reported in Chapter 5. The pH of the different shales did not vary appreciably and were slightly acidic. The loss on ignition provides a a rough idea of the organic matter present in these samples of shale.

3.4 Soil Properties

The soil index values provided by the ISHC of the shales studied are described in this section. Table 3.4.1 gives the Atterberg limits for the shales studied. These shales are classified according to the Unified Soil Classification, AASHTO classification, and Textural classification systems in Table 3.4.1, and the Atterberg limits are plotted on the plasticity chart in Figure 3.4.1. Textural classification gives an indication of whether the shale is clayey or silty. The other properties, viz., natural density and moisture, are given in Tables E-1.1 to E-1.10 in Appendix E-1.

Table 3.4.1 Soil Index Values*

Shale	Atterberg Limits w_L	w_P	I_p	Unified Soil Classification	AASHTO Classification	Textural Classification
1. New Providence	33.6	22.6	11.0	CL	A - 4(10)	Silty Clay (Shale)
2. Mansfield	32.3	21.7	10.6	CL	A - 6(17)	Silty Clay (Shale)
3. New Albany	34	28.4	5.6	ML & OL	A - 4(7)	Silty Clay (Shale)
4. Osgood	25.6	17.9	7.7	CL	A - 4(4)	Silty Loam (Siltstone)
5. Attica	32.6	21.4	11.2	CL	A - 6(11)	Silty Clay (Shale)
6. Palestine	28.0	22.8	5.2	ML & OL	A - 4(2)	Clay Loam (Shale)
7. Lower Hardinsburg	42.3	27.3	15.0	ML & OL	A - 7-6(25)	Clay (Shale)
8. Klondike	26.5	21.0	5.5	CL - ML	A - 4(0)	Sandy Clay Loam (Siltstone)

* From ISHC Laboratory Test Results.

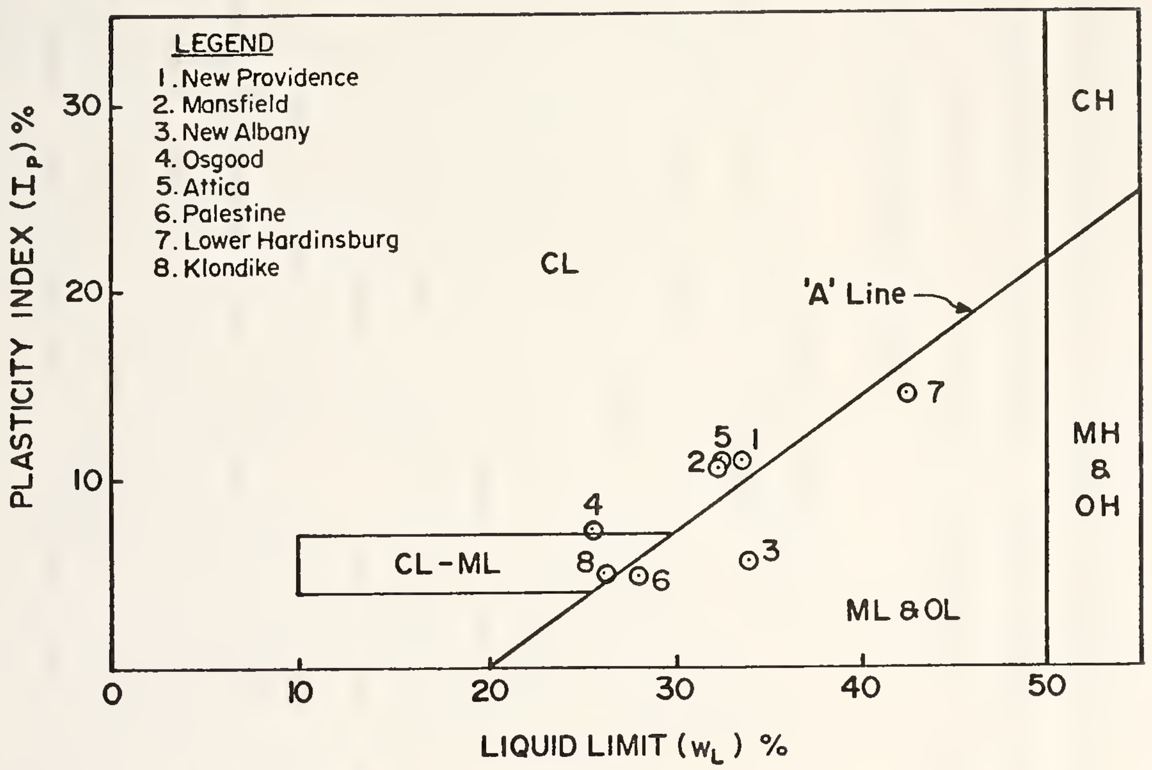


FIGURE 3.4.1 ATTERBERG LIMITS OF TEST SHALES PLOTTED ON PLASTICITY CHART

CHAPTER 4 LABORATORY TESTING

Laboratory tests conducted on the shale samples during this investigation are reported in this section. Some of the test procedures are detailed in Appendix B. The results obtained from these tests are described in the following chapter.

4.1 Slaking Index Test

The Slaking Index (S.I) was determined according to the test procedure (from ISHC, "Method of Test for Determining the Slaking Index of Shale, Test Method No. Ind. 502-73") described in Appendix B-1. In this test, six pieces of shale weighing approximately 150 gms were selected and oven dried to constant weight at about 105°C. Each piece was soaked in a 600 ml beaker containing the slaking fluid (usually distilled water) for 24 hours, with the water level over the shale sample by at least 13.0mm (1/2 in). Water was then drained from the shale sample and washed over a 2.0mm (#10) sieve and the retained material oven dried at 105 ± 5°C to constant weight (for approximately 24 hours). After five cycles of this procedure the Slaking Index was calculated as follows and expressed as a percentage:

$$\text{S.I.} = \frac{\text{oven dry wt. of material lost at the end of 5 cycles}}{\text{oven dry wt. of the sample before the test}} \times 100$$

Five tests were performed on each shale and the average reported. Table 4.1.1 gives the Slaking Index for the shales studied, with water as the slaking fluid.

The only change from the above procedures was that a 2mm (#10) screen was used to suspend the piece of shale in the beaker. The screen was used for the following reasons (Bouyoucos, 1929):

1. The screen prevented the potential "shielding effect of the slaked portion on the unslaked portion.
2. The rate of slaking was thereby considerably hastened.
3. It shows definitely when the material is completely slaked.

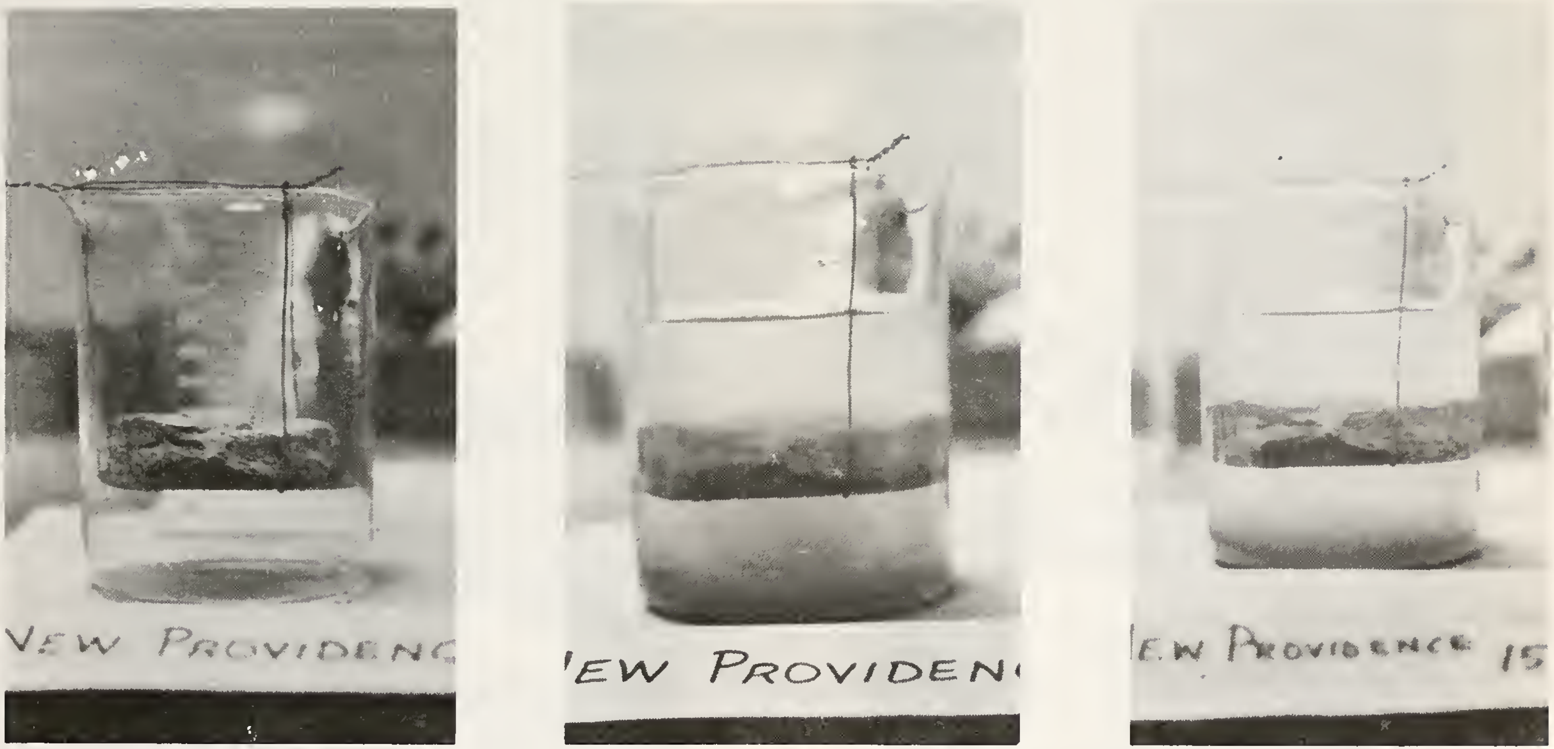
Figures 4.1.1 and 4.1.2 show the slaking of shales #1 and #2 at different intervals. After the first cycle of wetting, the retained material consisted of large and small fragments in the case of shales #1 and #2, however shale #3 remained unchanged. By the end of the fifth cycle, shales #1 and #2 were reduced to small fragments, while shale #3 remained virtually unchanged.

Slaking of compacted samples was accomplished in a similar manner by immersing the sample in a 2000 ml beaker containing the slaking fluid (water). In this case the disaggregation of the compacted specimen was visually examined and described and the shape of the specimen at the end of the test was also noted.

Slaking of these shales was also studied under a vacuum. The oven dried shale samples were kept in an air tight chamber with lucite cylindrical walls, allowing visual observation of the sample during slaking. A vacuum was applied at the top of the chamber via a pump and the

Table 4.1.1 Durability and Point Load Strength Properties

Shale	Slaking Index (S.I.)	Slake Durability Index (Id_2)	Deo's Classification	Point Load Strength (PLS)	
				PSI	MPa
1. New Providence	50.81	58	soil-like	2347.1	16.18
2. Mansfield	40.78	66	soil-like	1352.1	9.32
3. New Albany	0.14	99.1	rock-like	3234.5	23.68

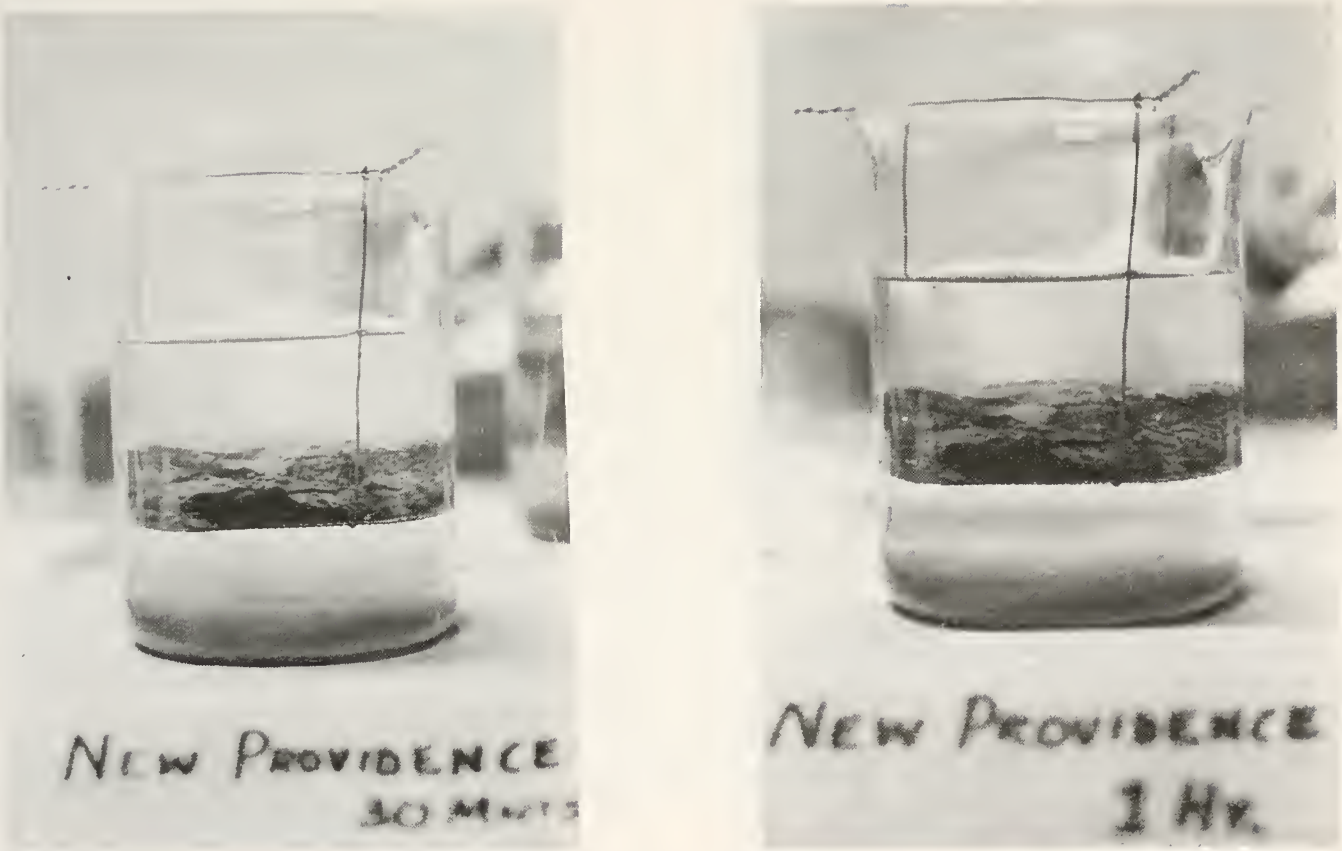


Sample Before Slaking

Sample at the Instant when Water is Added

Duration of Slaking : 15 Minutes

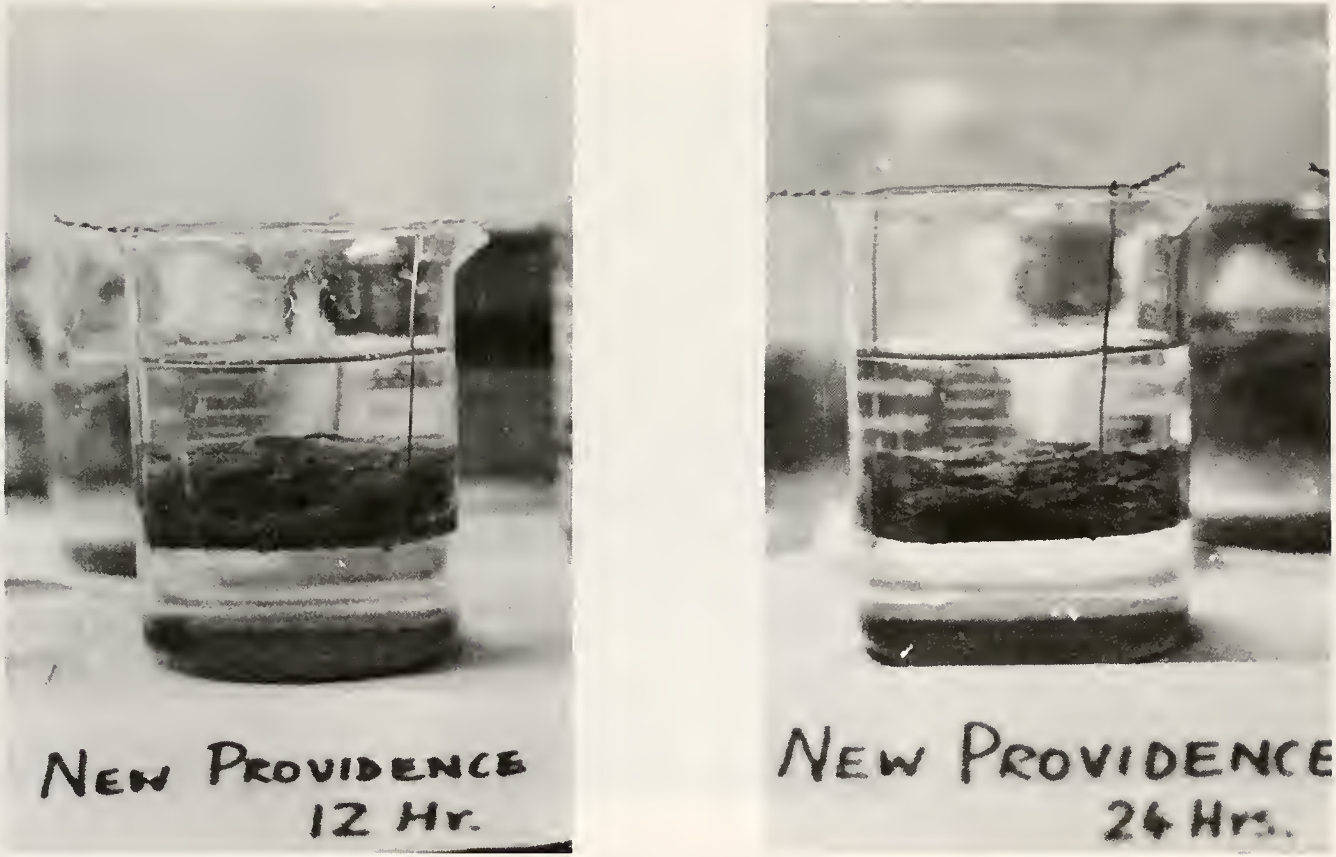
FIGURE 4 . 1.1 SLAKING OF NEW PROVIDENCE SHALE IN WATER



Duration of Slaking : 30 Minutes

Duration of Slaking : 1 Hour

FIGURE 4.1.1 CONTINUED



Duration of Slaking: 12 Hours

Duration of Slaking: 24 Hours

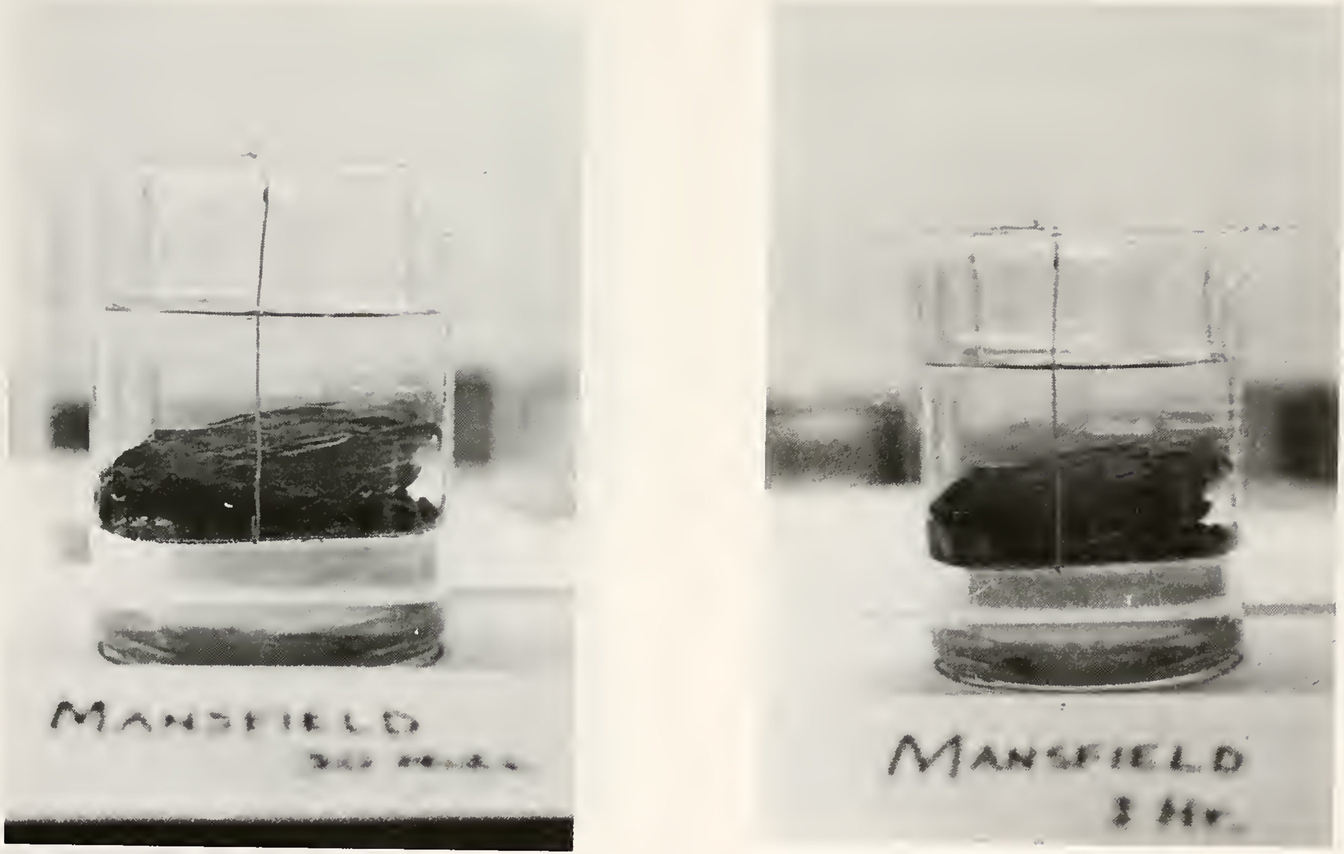
FIGURE 4.1.1 CONTINUED

Sample Before Slaking

Sample at the Instant when Water is Added

Duration of Slaking : 15 Minutes

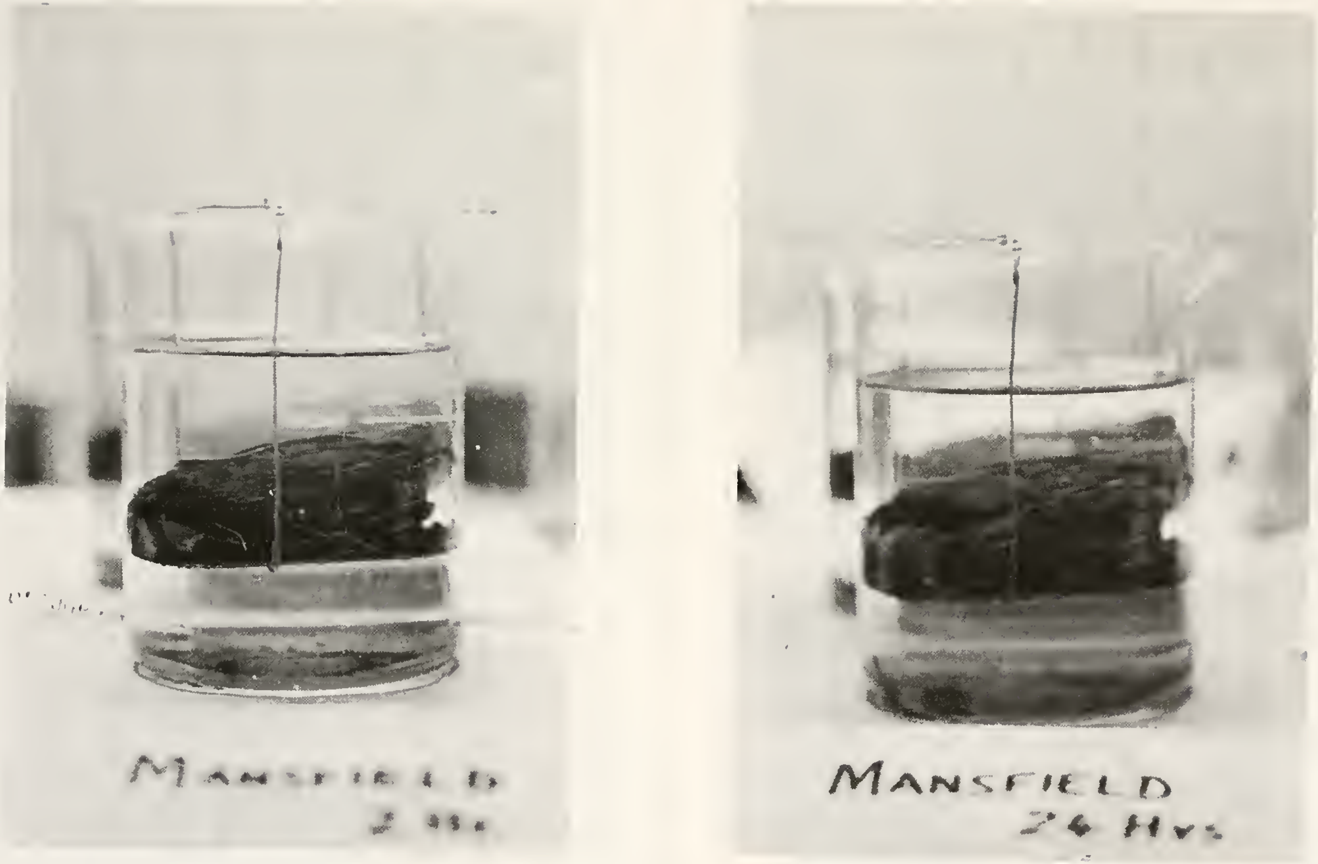
FIGURE 4.1.2 SLAKING OF MANSFIELD SHALE IN WATER



Duration of Slaking: 30 Minutes

Duration of Slaking: I Hour

FIGURE 4.1.2 CONTINUED



Duration of Slaking: 2 Hours

Duration of Slaking: 24 Hours

FIGURE 4.1.2 CONTINUED

pressure in the chamber was measured using a manometer and a McLeod gage. After about six hours of applying vacuum, the pressure in the chamber was measured. When the pressure reached 0.1mm of mercury in the McLeod gage, water was allowed to enter the chamber through the bottom and the vacuum was shut off. After the level of water in the chamber reached about one inch above the sample, the water inlet at the bottom was disconnected and the top was exposed to the atmosphere. The sample was allowed to slake in the chamber for 24 hours and wet sieved over a 2mm (#10) sieve. The Slaking Index was computed as before.

4.2 Slake Durability Test

The slake durability index was determined according to the International Society for Rock Mechanics procedure described in Appendix B-2. The slake durability apparatus (developed from Franklin, 1970) shown in Figure 4.2.1, consists of a drum with a screen opening of 2mm (#10). The drum is rotated by an electric motor in a bath of slaking fluid (usually water) at a constant rate (20 rpm). The slaking sample consists of ten equidimensional pieces of shale each weighing between 40 and 60 grams. The pieces are oven dried to constant weight at 110± 5°C, cooled to room temperature, and placed in the drum of the apparatus. The drum is immersed in the tub containing the slaking fluid and is rotated for 200 revolutions. At the end of the test, the material retained in the drum was oven dried and weighed. The retained material was subjected to another cycle of slaking in the rotating drum. The slake durability index (I_{d2}) was calculated at the end of the second cycle as follows:

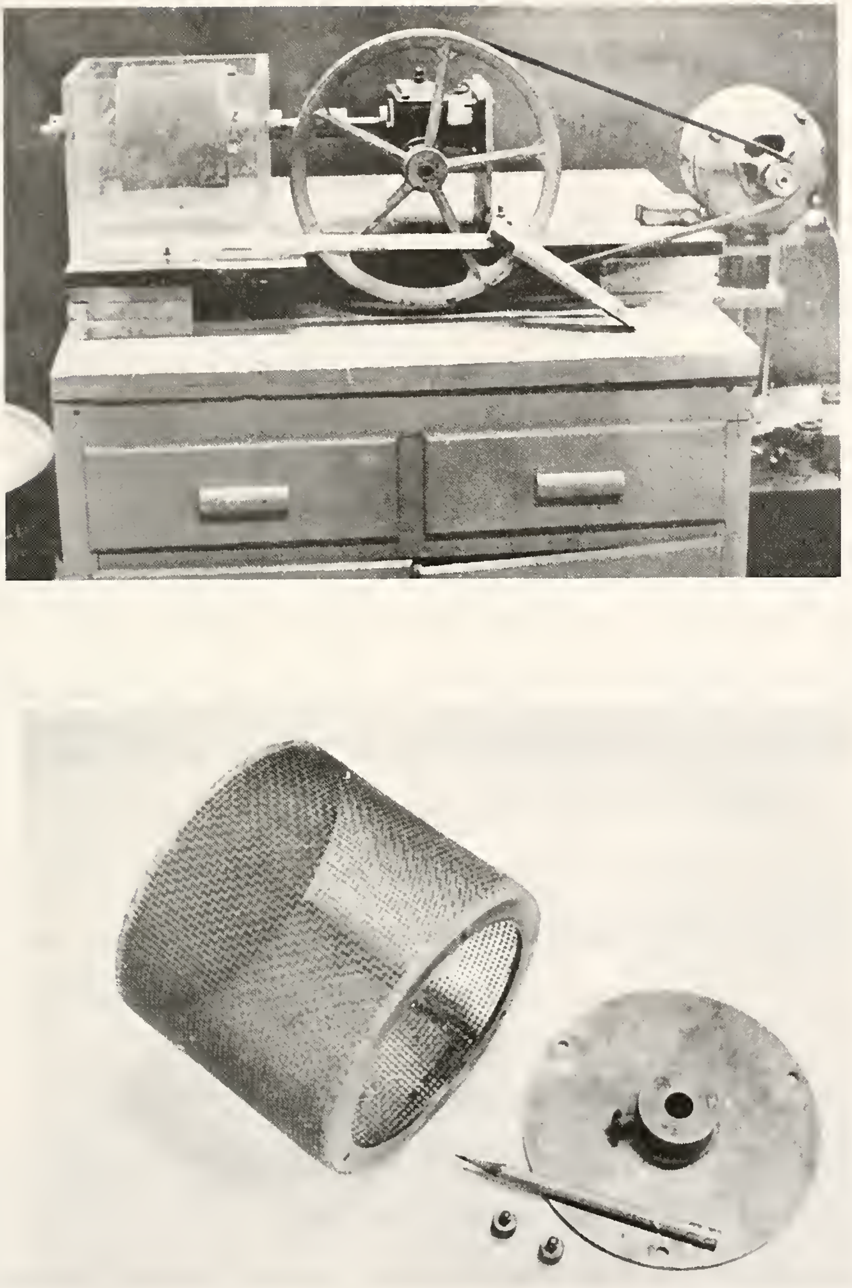


FIGURE 4.2.1 SLAKE DURABILITY APPARATUS

$$I_{d2} = \frac{\text{oven dry wt. of material retained at the end of 2nd cycle}}{\text{oven dry wt. of the sample before the test}} \times 100$$

At least four tests were run for each shale and average values are reported. The slake durability indices, presented in Table 4.1.1 are an average of six tests.

Slake durability of the compacted specimens was determined in the same manner as explained above. The compacted specimens, 101.6mm (4 in) in diameter and approximately 56.3mm (2.22 in) high,were used in the slake durability apparatus and allowed to slake while undergoing tumbling. The test specimens were obtained by cutting a sample of Proctor mold size [101.6mm (4 in) in diameter and 115.82 (4.56 in) high] into two pieces using a high speed band saw. The sizes and shapes of the specimens at the end of the test were observed, in addition to calculating the percentage of the original weight retained. In cases where no additive was used, the compacted specimen usually disaggregated to discrete pieces of shale after the first cycle. The effect of additives and curing on the slaking characteristics of the compacted shale is discussed in the next chapter.

4.3 Point Load Strength (PLS) Test

A diagram of the PLS test apparatus is shown in the Figure 4.3.1 (from Bailey, 1976). The load was applied by an electrically driven compression testing machine at a constant rate of deformation of 0.254mm (0.01 in) per minute. The load was monitored through a 5000 pound (22.24 kN) capacity, SR4 type, load cell. The initial dial gage reading indicates the guide plate height, and from this the sample thickness

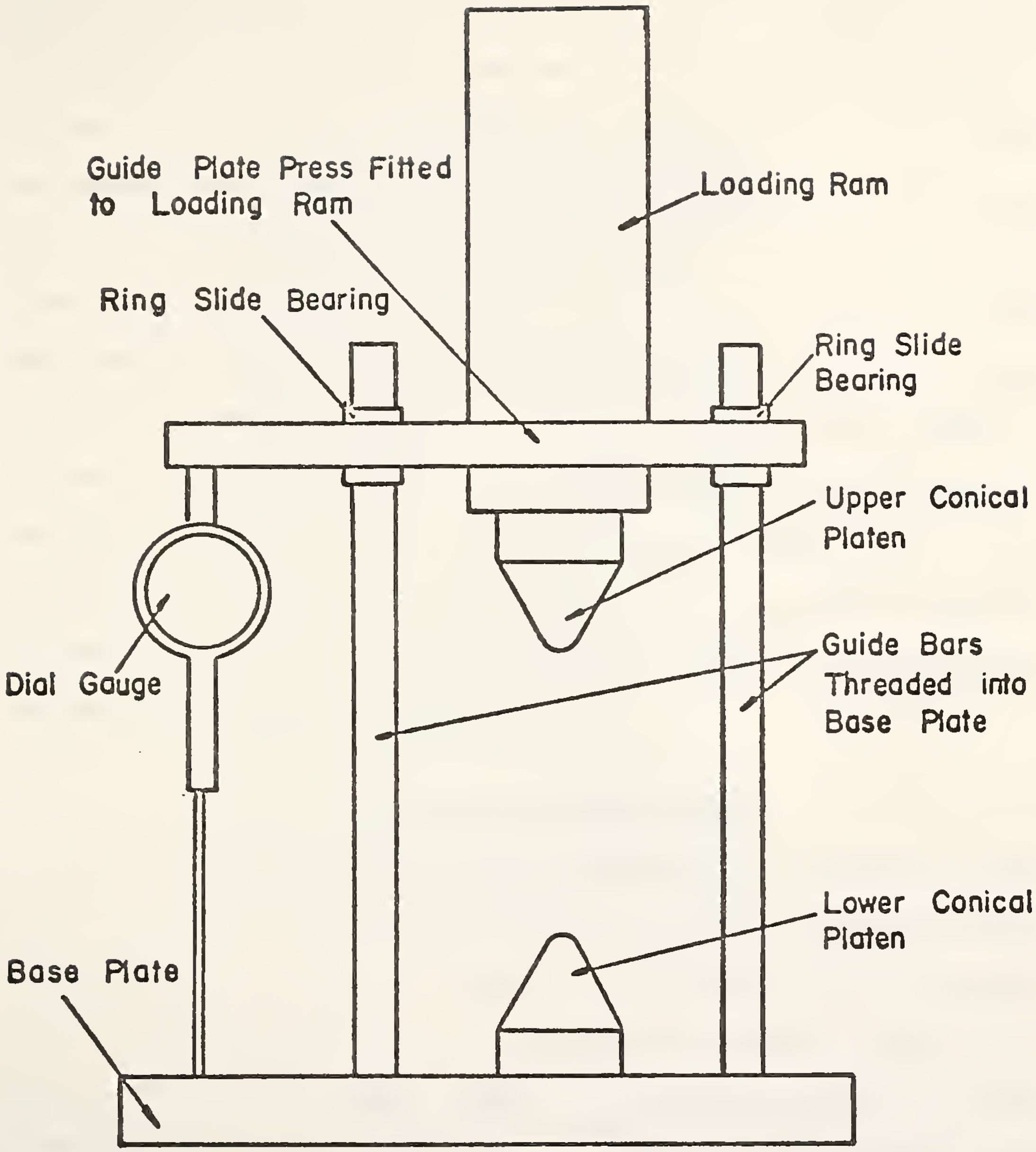


FIGURE 4.3.1 SIDE VIEW OF POINT LOAD TEST APPARATUS (FROM BAILEY, 1976)

(D) can be obtained. The sample of shale consisted of platy pieces approximately equidimensional for the plan area. The sample was loaded perpendicular to the bedding planes and the initial sample thickness of the shales tested varied from 2.56mm to 13.41mm (0.10 in to 0.53 in). The samples were oven dried to constant weight at 110± 5°C and cooled to room temperature before testing (i.e., all the samples were tested at near zero moisture content). Point load strength index (PLS) was computed by taking the ratio of maximum compressive load (P) to the square of the initial sample thickness (D). The point load strengths of the New Providence, the Mansfield and the New Albany shale samples are presented in Table 4.1.1 and these are averages of several tests. The point load strength index is represented as PLS in all cases where D/L was less than one (where 2L is the width of the sample). The results of test data are presented in Appendix D.

4.4 One Dimensional Collapse Test

The one dimensional collapse test (developed by Abeyesekera, 1978) is a test in which sudden densification due to soaking takes place under a surcharge. The apparatus consists of a 112.52mm (4.43 in) diameter consolidation cell, loaded in the consolidation frame. Figure 4.4.1 shows a test setup. Discrete pieces of oven dried shale of a known gradation were hand placed in the cell and the initial height of the specimen was determined under a nominal surcharge load (10 kg). The specimen was loaded in increments of 10 kg up to a load of 140 kg. The deformation was monitored by means of a dial gage. Slaking fluid (in this case, water) was introduced into the specimen through the base and the immediate collapse was recorded. The deformation under this load in

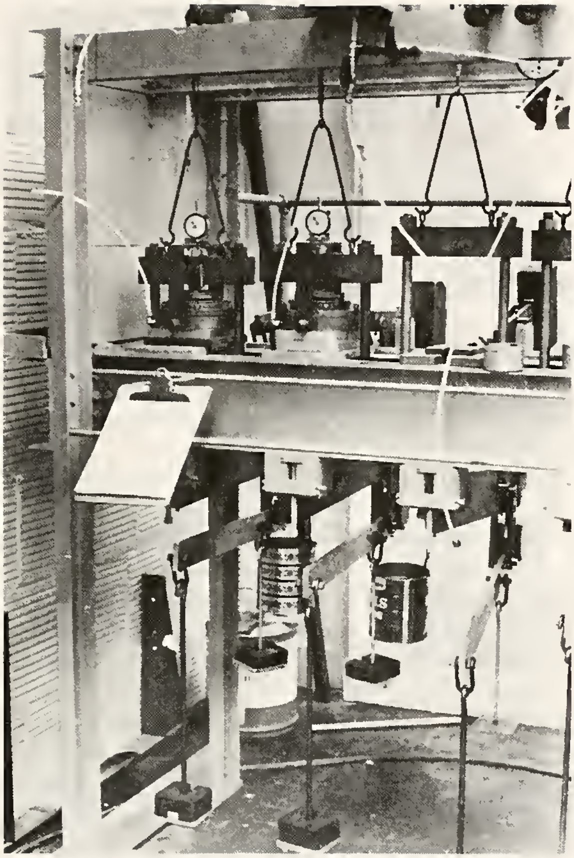


FIGURE 4.4.1 APPARATUS USED FOR ONE-DIMENSIONAL COLLAPSE TEST

the soaked condition was monitored with time for at least 24 hours after soaking. The material was wet sieved through the required sieve or sieves and the change in gradation was recorded. This gives a measure of breakdown due to soaking under load. Slaking of the shale in this test was computed as the ratio of dry weight of the material retained on 4.75mm sieve to dry weight of the initial sample.

4.5 Pore Size Distribution

Pore size distribution studies have been successfully applied in several branches of engineering, especially in geotechnical engineering. Garcia-Bengochea (1978) gives a brief review of the application of this method in geotechnical engineering.

Pore size distribution determinations for this study were performed with mercury intrusion. The mercury intrusion equipment used in the study had a pressuring capacity of 413 700 kN/m^2 (60,000 psi) and was capable of intruding pores from 16 microns to 0.0027 microns in size. The mercury intrusion technique employs the principle that the surface tension of a non-wetting liquid will oppose the entry of the liquid into a small pore of a solid. Washburn (1921) determined that the non-wetting liquid (in this case, mercury) can be forced into the pores with the help of external force, and this external force was found to be inversely proportional to the pore diameter. Assuming a cylindrical pore, Washburn (1921) calculated the relation to be:

$$P = -\frac{4\ T_s \cos \theta}{d}$$

where P = the absolute pressure required for instrusion

T_s = the surface tension of the intruding liquid

θ = the contact angle between the solid and the liquid, and

d = the limiting pore diameter.

The procedure used in this test, the assumptions made in the study, and the appropriate precautions and corrections are described in Appendix B-4. The determination of pore size distribution is given briefly below.

The oven dried sample of shale (sometimes consisting of 2-3 discrete pieces) is initially evacuated and surrounded by mercury, the pressure is raised in small increments, and the volume of mercury entering the sample after each increment is recorded. With each pressure increment, the mercury is forced into the accessible pores in the sample, of a diameter larger than or equal to that calculated by the Washburn equation. The volume of pore space between pressure increments is recorded and from this the limiting pore diameter is computed and the pore size distribution generated.

The pore size distribution is presented in the form of differential distribution and cumulative distribution curves for this study. Figure 4.5.1 provides these curves for the three shales studied. The individual curves for each shale along with the curves for other shales are supplied in Appendix B-4.

4.6 Compaction

Shales from the New Providence and Osgood formations were used for the study of stabilization with lime in the compacted state. Both of

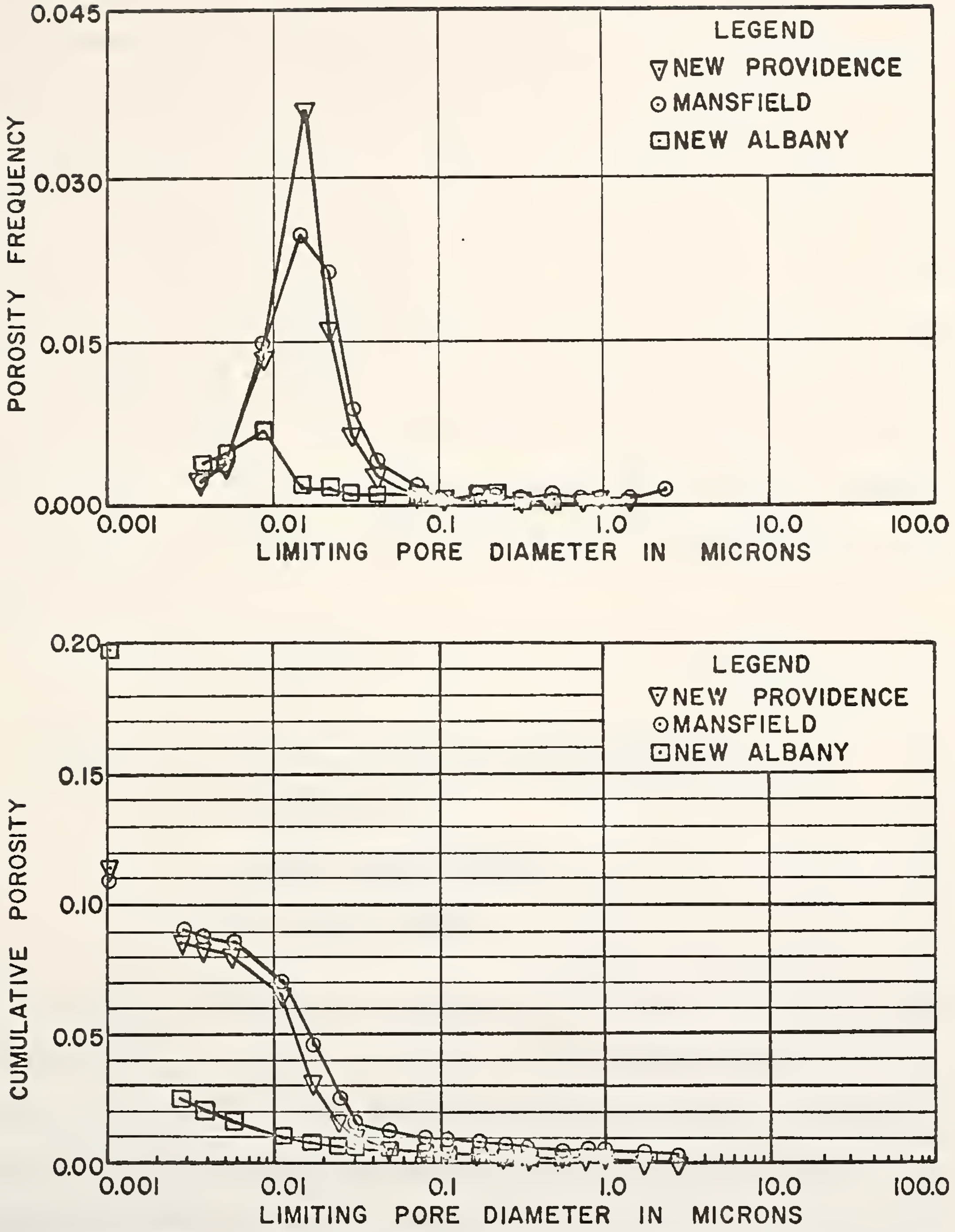


FIGURE 4.5.1 DIFFERENTIAL AND CUMULATIVE PORE SIZE DISTRIBUTION FOR MANSFIELD, NEW PROVIDENCE, AND NEW ALBANY SHALES

these shales belong to the same category, viz., hard and non-durable, and they have been used for other studies at Purdue University (Bailey, 1976; Abeyesekera, 1978; Witsman, 1979; Hale, 1979). The New Providence shale was stored in metal cans and the Osgood shale in a metal container (see Hale, 1979 for details). The stored shale samples were broken into pieces of about 100mm (4 in) size and passed through a jaw crusher which was set to yield pieces less than 12.5mm (1/2 in) in size. The crushed shale was sieved and separated using a nest of 12.5mm, 9.5mm, 4.75mm, 2.36mm, 1.00mm, and 75 μm (1/2", 3/8", #4, #8, #16, and #200) sieves in a Gilson sieve shaker. The gradation used for compaction is shown in Figure 4.6.1. This gradation matches closely the exponential gradation of n equal to 1 (Witsman, 1979) in the Talbot and Richart (1923) equation:

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

where P = percentage by weight finer than size d

d = any diameter

D = maximum grain diameter

n = an abstract number

This gradation was selected by Witsman (1979) when a batch of crushed New Providence shale gave a gradation closely matching the gradation of n value equal to 1. This gradation also falls within the range of gradations investigated by Abeyesekera (1978) and makes efficient use of the shale for the compaction purposes.

The sieved, crushed shale was prepared to provide batches of 2.27 kg (5 lbs) at the $n = 1$ gradation. Each batch was placed in small

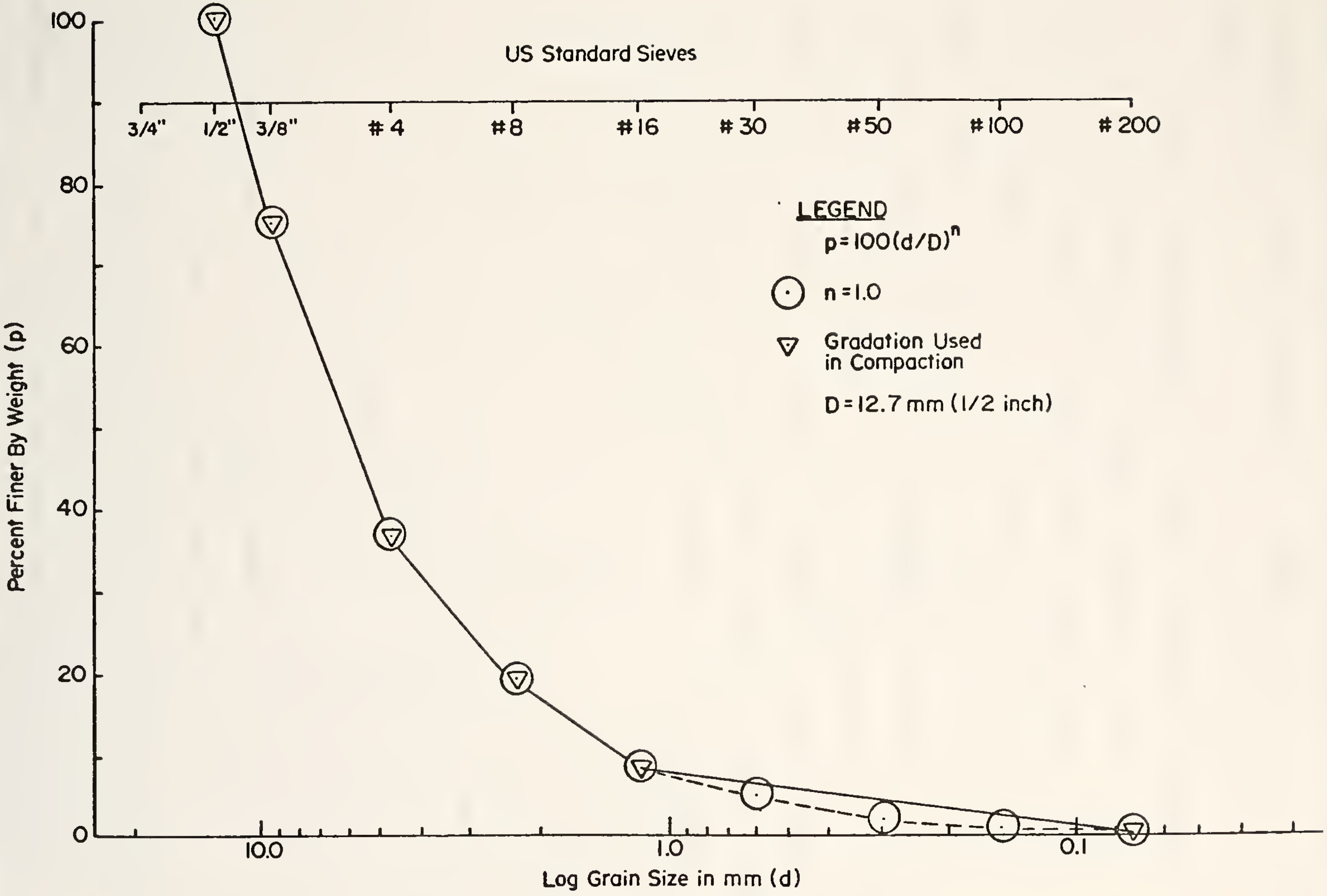


FIGURE 4.6.1 GRADATION USED IN COMPACTION

double polythene bags and tied. The bags of shale were kept at room temperature and relative humidity in the laboratory (for less than one month).

At first, distilled water was added to the prepared gradation in a pan and mixed thoroughly for a short time with a large spoon. The sample was then compacted to the standard Proctor density. However, it was not possible to get a well defined compaction curve with this method. Hence the sample was cured in the plastic bags for 12 - 24 hours after adding water and then subjected to compaction. With this procedure, the shale samples do slake to a limited extent, but a well defined Proctor compaction curve is obtained. The test results agreed closely with the compaction curve obtained earlier by Witsman (1979) for the New Providence shale. [Witsman (1979) also cured the samples for 12 - 24 hours prior to compaction.] Figures 4.6.2 and 4.6.3 give the moisture density curves for the New Providence and Osgood shales with various percentages of lime for the standard Proctor compactive effort.

Lime can be added in the dry state and mixed with the aggregates of shale prior to the addition of water or can be added in the form of a water slurry to the dry aggregates of shale. Both of these approaches were tried. It was found that when lime was used in the dry state it was not possible to get uniform distribution of the lime in the sample and lime had a tendency to form small lumps even after thorough mixing. Lime when added in the form of a slurry, on the other hand, gave a very uniform distribution of lime and reproducible densities. Different percentages (3, 5 and 7%) of lime were used and the moisture density curves were obtained for each percentage in the standard Proctor compaction test, as shown in Figure 4.6.2. Laboratory reagent grade $Ca(OH)_2$ was

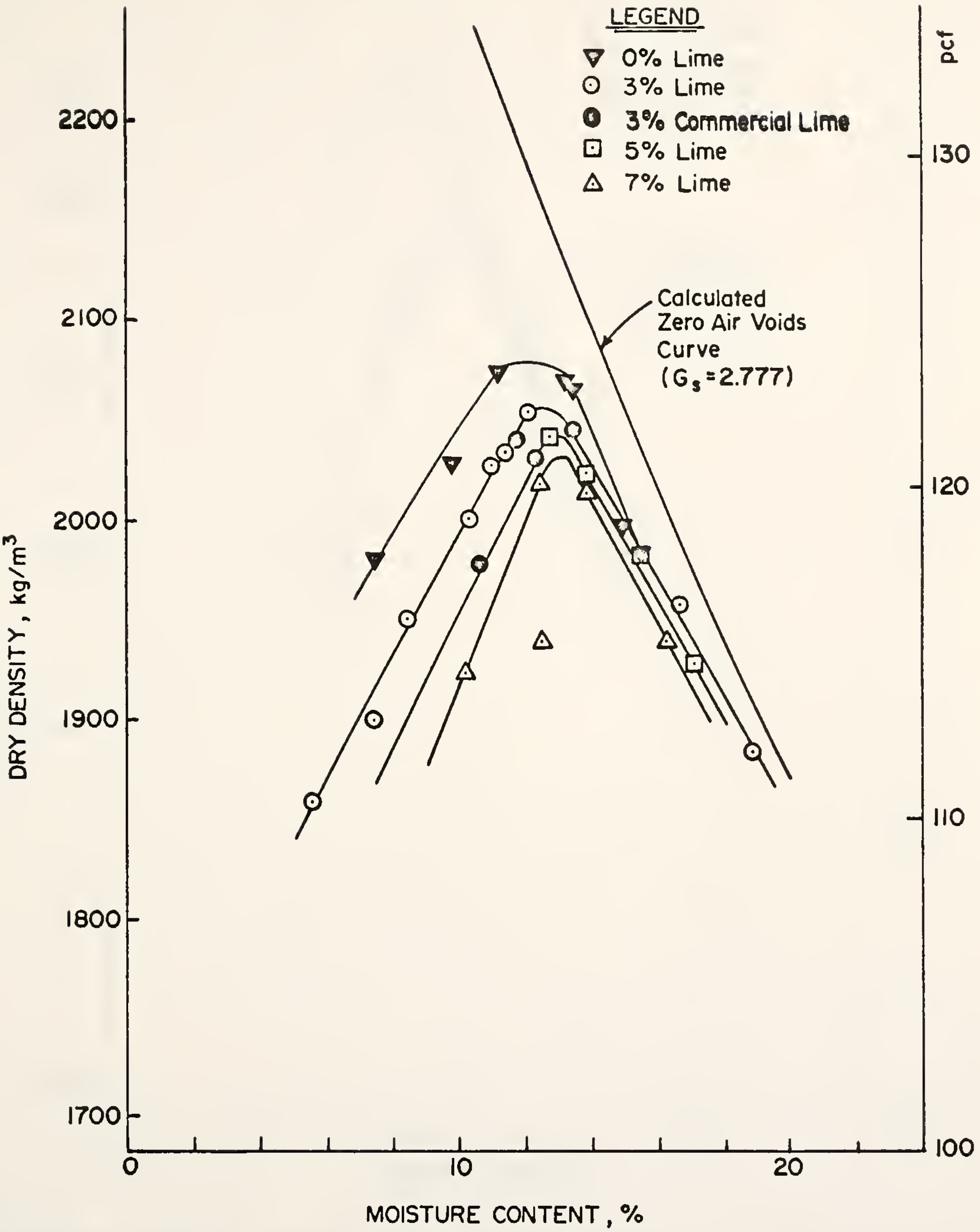


FIGURE 4.6.2 MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION TESTS ON NEW PROVIDENCE SHALE WITH VARIOUS PERCENTAGES OF LIME

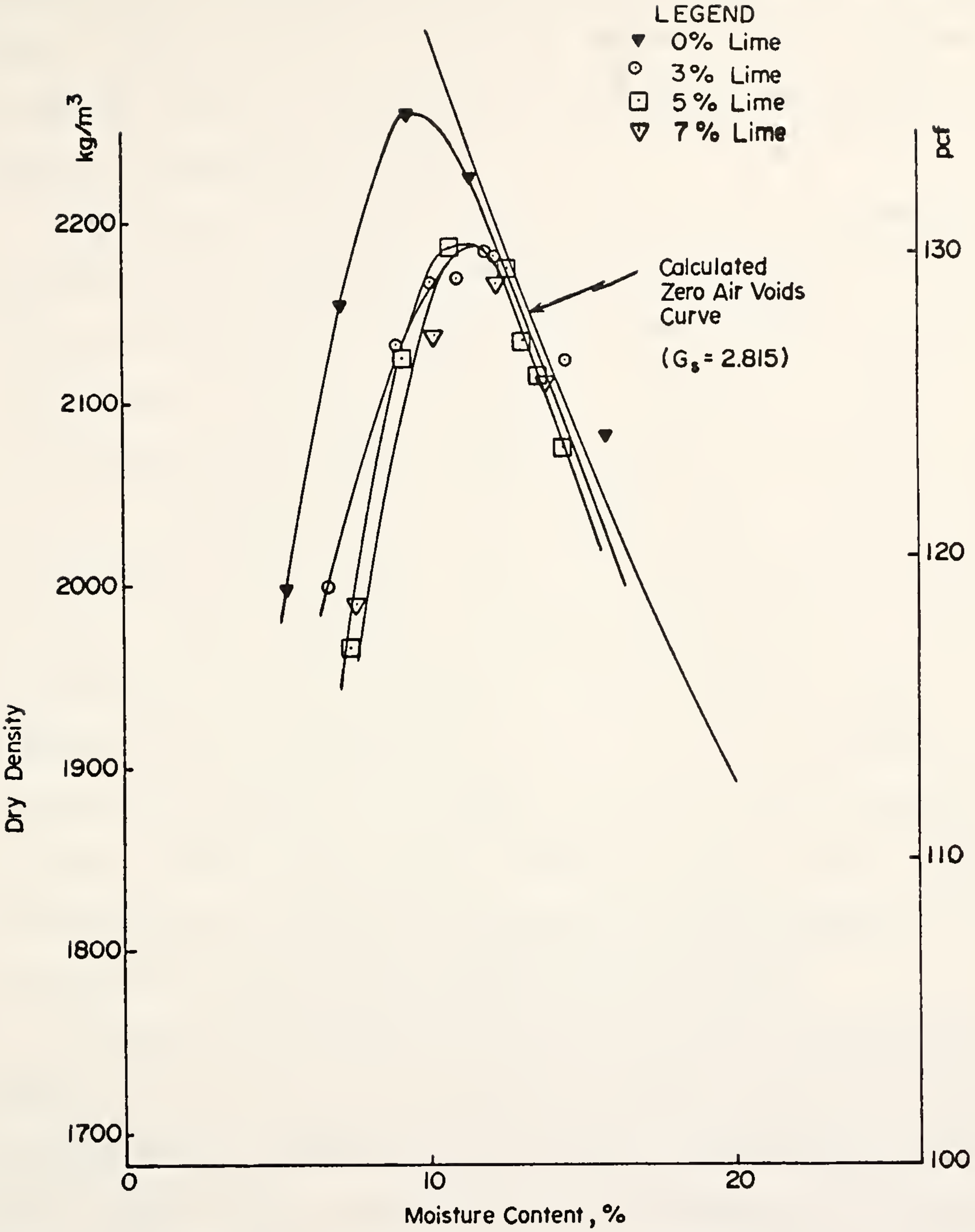


FIGURE 4.6.3 MOISTURE-DENSITY RESULTS FROM PROCTOR COMPACTION TESTS ON OSGOOD SHALE WITH VARIOUS PERCENTAGES OF LIME

the lime used in defining the moisture density relations. The required amount of lime was added to the water and agitated for 2 - 3 minutes in a mechanical stirrer to form the slurry. The mixing and curing of the samples was accomplished as before.

It can be seen from Figures 4.6.2 and 4.6.3 that the optimum moisture content for the different percentages of lime (namely 0, 3, 5 and 7%) did not vary much. The maximum dry density was reduced from 2070.25 to 2040.75 kg/m^3 (123.8 to 121 pcf), i.e., a reduction of 2.3% for the New Providence shale and from 2261.54 to 2186.36 kg/m^3 (134.5 to 130 pcf), i.e., a reduction of 3.5% for the Osgood shale. It can be seen that for Osgood shale (Figure 4.6.3) two of the data points lie beyond the zero air voids curve which is theoretically impossible to obtain. The computation of water content for the data points were done on the trimmings and the excess material that was left after compaction, by oven drying for 24 hours. It is expected that the error is in computing the water content which should actually be lower than reported. Decreased moisture content will move these data points towards the left of the zero air voids curve and increase the dry density.

The samples were compacted at a moisture content of about 2% wet of optimum (i.e., at 14% for the New Providence shale and at 12 - 13% for the Osgood shale), to ensure that there would be enough water available for the curing of lime. After the sample was compacted in the standard 0.00094 m^3 (1/30 ft^3) Proctor compaction mold, the collar of the mold was removed and the top of the sample was trimmed using a metal straight edge. The sample was weighed and then extruded from the mold using a hydraulic jack. The extruded sample was sealed in a polythene bag using a bag sealer and sealed again in another polythene bag. The samples were stored at the laboratory temperature until the day of testing.

The lime used in this study was from a single batch (50 lb bags) of commercial slaked lime [$Ca(OH)_2$]. The moisture-density curve using the commercial lime was essentially the same as the one obtained by using the laboratory reagent lime. X-ray diffraction patterns of both the limes used are given in Appendix B-3.

4.7 Strength Test

The strength of compacted samples with additives was evaluated by the unconfined compression test. The apparatus, shown in Figure 4.7.1, consisted of a Model 56 Wykeham Farrance loading frame with an 8.9 kN (1 ton) capacity. The load was applied by an electrically driven motor and monitored with a 11.12 kN (2500 lb) capacity Karol-Warner proving ring. The displacement was measured using a dial gage of ±0.0254mm (±0.001 in) accuracy. The rate of loading was 1.4707mm/min (0.0579 in/min).

The sample extruded from the compaction mold, 101.6mm (4 in) in diameter and 116.33mm (4.58 in) high, was used for the test specimen. The height to diameter ratio was 1.145. This size of the test specimen was used in order to avoid problems of trimming to a more favorable length-diameter ratio.

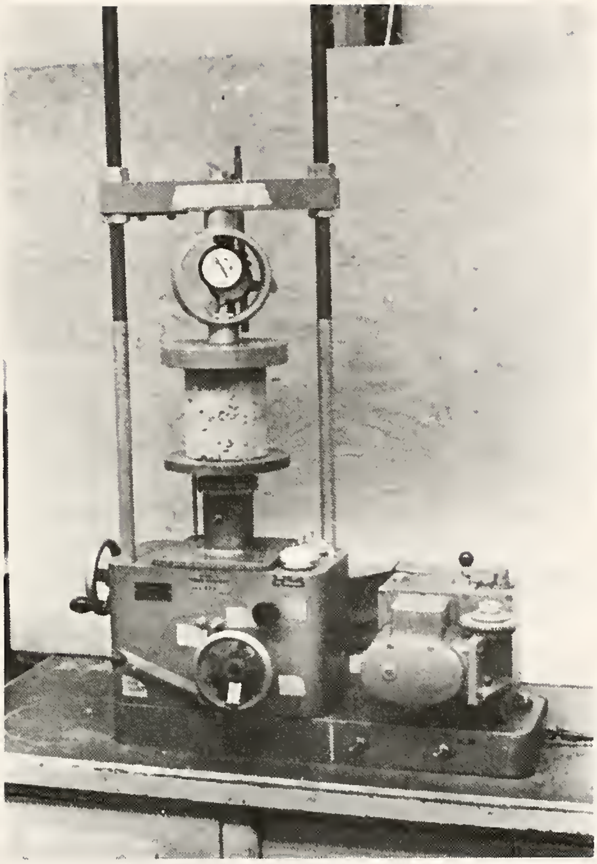


FIGURE 4.7.1 UNCONFINED COMPRESSIVE STRENGTH TESTING OF COMPACTED SHALE SAMPLES

CHAPTER 5 RESULTS AND DISCUSSION

The mechanisms controlling the process of slaking, as mentioned earlier in the literature review, are: (1) the air-breakage phenomenon; (2) differential swelling; and (3) dissolution of cementing agents. Experiments were performed in the laboratory to investigate these mechanisms and to identify the mechanisms that control slaking in compacted shales. In the case of compacted Indiana shales, it is proposed that no single mechanism, of the three mentioned above, controls the slaking properties. On the other hand, a combination of the three mechanisms, either one triggering the other or all of them occuring at the same time, is suggested.

5.1 Slaking of Shales in Atmosphere and Under Vacuum

Tests were conducted in the laboratory to see if the air-breakage phenomenon (proposed by Terzaghi and Peck, 1967) was the principal mechanism controlling the slaking properties of Indiana shales. Before presenting the test results, the development of the air-breakage phenomenon by Van Eeekhout (1976) is given in the following paragraph.

Expansion (or swelling) of rock which does not contain expandable clays is attributed to the disruption of capillary tension. Upon exposure of a rock formation to the atmosphere (air), the capillary tension that exists in the pores or at any discontinuity (joint, bedding and crack), controls the amount of moisture present and is a function of the vapor pressure in equilibrium with the capillary water. Figure 5.1.1 shows a crack tip of radius r_c and a curved water meniscus of radius r_w. The relation between the meniscus radius (r_w) and its vapor pressure as governed by the Kelvin equation is:

$$\ln \frac{P_r}{P_o} = \frac{-2\Gamma V}{r_w RT} \cos\theta$$

where, p_o = water vapor pressure over water with no curvature

p_r = water vapor pressure over water of curvature r_w

Γ = surface tension of water (0.0072 J/m^2)

V = molar volume of water (0.018 $m^3/kg/m$-mole)

R = gas constant

T = absolute temperature

θ = contact angle between the vapor and the pores

From the above expression it can be seen that if p_o is greater than p_r, the water pocket will grow until equilibrium is reached. The equilibrium meniscus radii for various relative humidities are given in Table 5.1.1.

If a shale which is fully saturated in situ, is subjected to a relative humidity of less than 50% (i.e., subjected to drying), and then saturated again, the shale can undergo contraction-expansion with either no air allowed to enter, or with air being allowed to enter the pores.

Air is not allowed to enter during the contraction process, in the case of rocks which have very low porosity and continuous pores which control the permeability. Chenevert (1970) found in his testing of shales that shales which did not have air in the pores were stronger

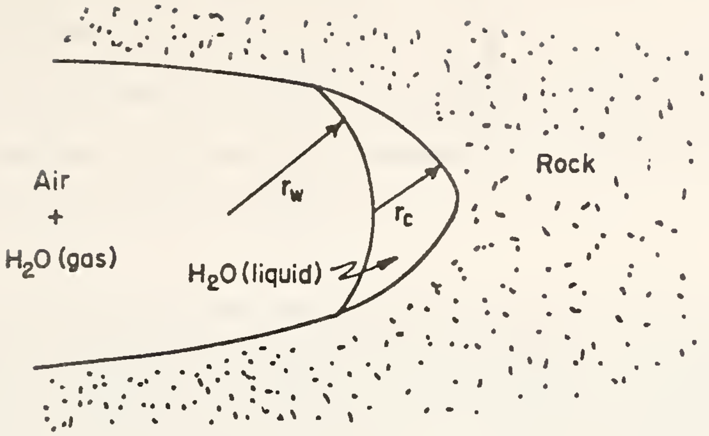


FIGURE 5.1.1 SCHEMATIC DIAGRAM OF WATER POCKET AT A CRACK TIP (AFTER WIEDERHORN, 1967; FROM VAN EEKHOUT, 1976)

Table 5.1.1 Dependence of Meniscus Radius on Relative Humidity (after Wiederhorn, 1967; from Van Eekhout, 1976)

100 x p_r/p_o (relative humidity)	Meniscus Radius (m) Pure Water
100	∞
99	1.0×10^{-7}
90	1.0×10^{-8}
75	3.7×10^{-9}
30	8.8×10^{-10}
10	3.4×10^{-10}
1	2.3×10^{-10}
0.2	1.7×10^{-10}
0.0017	1.2×10^{-10}

because of the overall confining pressure that is caused by the absence of air in the pores.

If air is allowed to enter the pores during contraction, the process of slaking of shales upon immersion in water is explained by Taylor and Spears (1970) as follows:

> "During dry periods evaporation from the surfaces of rock fragments promotes high suctions, which in turn result in increased shearing resistance (of individual fragments) by virtue of high contact pressures. With extreme desiccation the bulk of the voids will be filled with air, which, on rapid immersion in water, becomes pressurized by the capillary pressures developed in the outer pores. Failure of the mineral skeleton along the weakest plane ensues and an increased surface area is then exposed to a further sequence of events."

Slaking of New Providence, Mansfield and New Albany shale were observed in the laboratory under vacuum and also in the atmosphere as shown in Figure 5.1.2. The results are presented in Table 5.1.2. It can be concluded from Table 5.1.2 that about 35% to 45% of the slaking could be attributed to the air-breakage phenomenon of Terzaghi and Peck (1967). The slaking that occurs as a result of the other two mechanisms therefore constitutes a majority of the slaking. This will occur after the skeleton of the shale structure is weakened by the above air-breakage phenomenon.

5.2 Slaking of Shales in Different Slaking Fluids

The slaking fluid used in the slake durability and slaking tests was changed by adding different inorganic salts to the water, and the effects were studied. Three shales, New Providence, Mansfield and New Albany, were analyzed chemically by the Soil Testing Laboratory in the

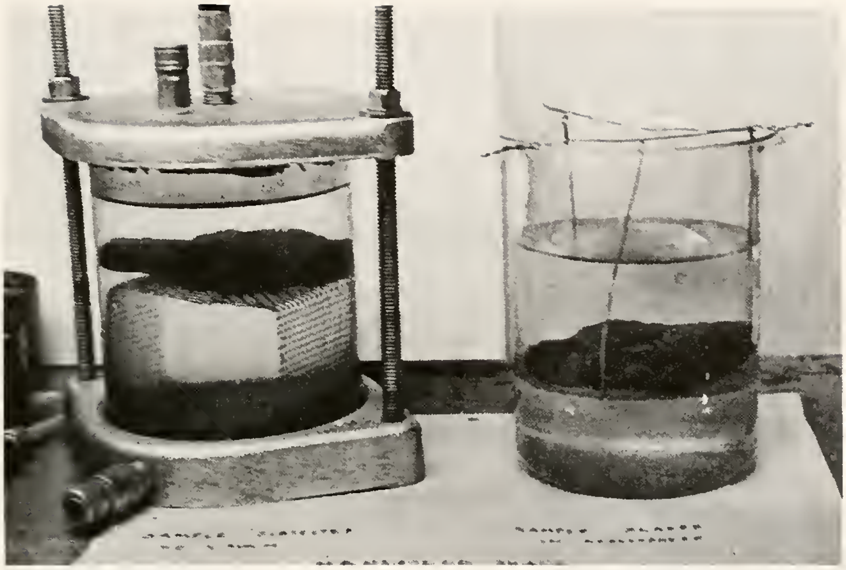


FIGURE 5.1.2 SLAKING OF A PIECE OF SHALE SUBJECTED TO VACUUM AND IN ATMOSPHERE

Table 5.1.2 Slaking Under Vacuum and in Atmospheric Pressure

Slaking / Shale Type	Slaking Index in Water*	Slaking Index in Water After Evacuating the Pores	Slaking Index Due to Compression of Entrapped Air	Percentage of Slaking Due to Entrapped Air	Percentage of Slaking Not Due to Entrapped Air
New Providence	1.91	1.07	0.84	44%	56%
Mansfield	1.37	0.87	0.52	37.4%	62.6%

* Results from One Cycle Slaking Test

Note: Slaking Index is the percentage of material lost through No. 10 sieve. There was no measurable slaking for the New Albany shale.

Agronomy department at Purdue University. The X-ray diffraction analyses were performed in the Materials Laboratory of the School of Civil Engineering at Purdue University.

5.2.1 Chemical Properties of Shale

The second mechanism causing slaking as mentioned in the beginning of Chapter 5, namely the mechanism of differential swelling, was studied in this research and is described in this section. Ion adsorption on clay particle surfaces may cause swelling of illite, chlorite or montmorillonitic minerals in a shale which leads to slaking (Chenevert, 1970; Tschebotarioff, 1973; Moriwaki, 1975). The clay minerals present in these shales were identified by X-ray diffraction. An estimation of the relative percentages of clay minerals in the clay size fraction was also accomplished by the X-ray diffraction technique and is presented in detail in Appendices A-1 and A-2. Clay minerals in the Indiana shales consisted predominantly of illite with some chlorite and kaolinite. There were some traces of mixed-layer clay minerals found in some samples. Montmorillonitic clay minerals were not found in any of the shales studied, as these shales belong to relatively older geologic formations.

Sherard et al., (1972) in the study of piping of earth dams recognized that some clays were prone to erosion under certain chemical environments. They presented data, shown here as Figure 5.2.1, in which the expected performance of clay in an embankment dam with respect to erosion is correlated with the chemical environment. Zone 1 and Zone 2 in Figure 5.2.1 include nearly all the samples from dams which failed by breaching in Oklahoma and Mississippi, viz., highly prone to dispersion.

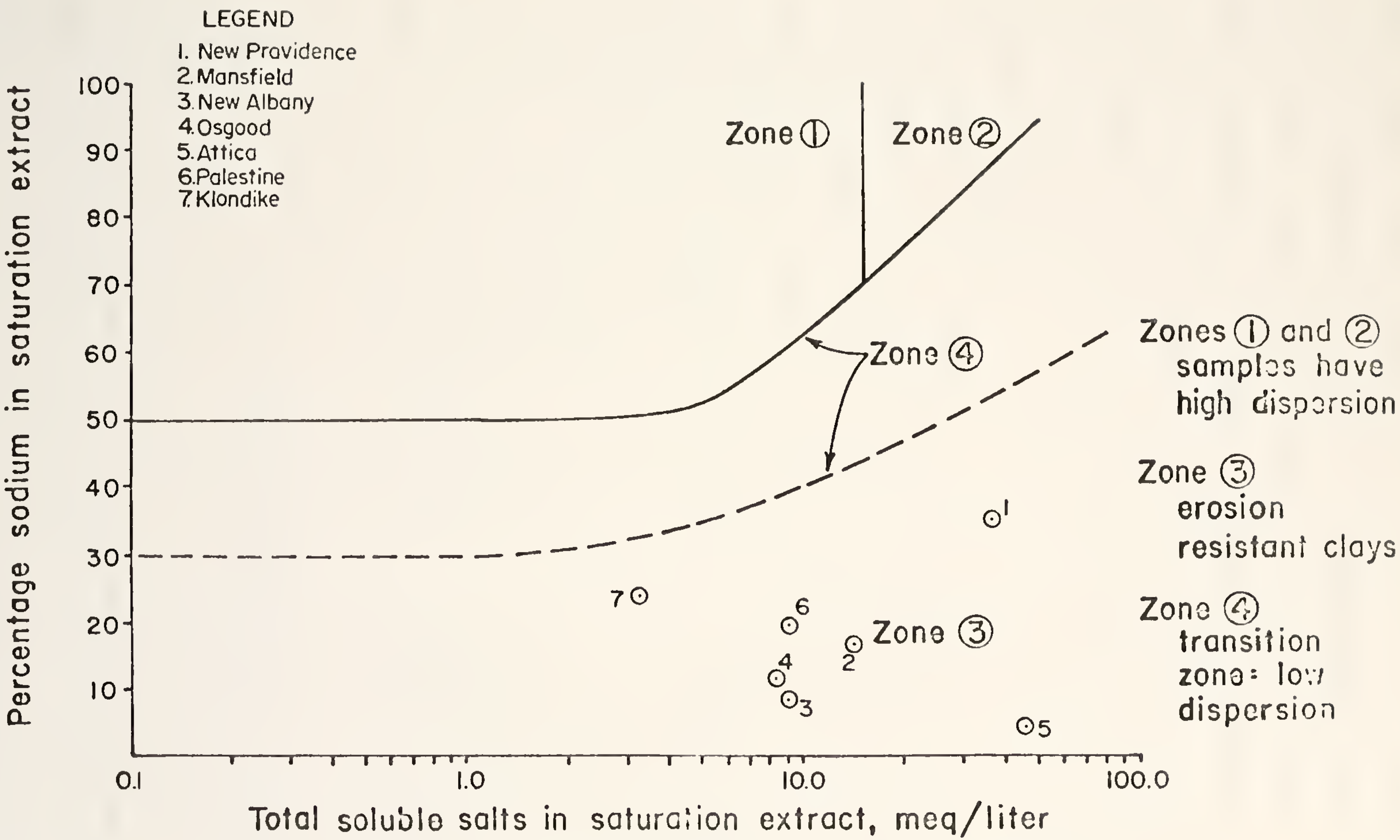


FIGURE 5.2.1 SEVEN INDIANA SHALES PLOTTED ON THE SUMMARY OF CORRELATION BETWEEN TEST RESULTS AND DAM EXPERIENCE (AFTER SHERARD et al, 1972)

Zone 3 includes erosion resistant clays. Zone 4, also called the transition zone, contains clays which have low dispersion characteristics, and the lower boundary of this zone is not well established. Results from the chemical tests on seven Indiana shales are presented in Tables 5.2.1 and 5.2.2. The results of these tests, when plotted in Figure 5.2.1, suggest that these shales are erosion resistant. Therefore, it can be concluded that any breakdown of these shales would not be of a dispersive clay nature.

The results from the chemical tests in Tables 5.2.1 and 5.2.2 and the percentages of clay minerals were used in linear regression against the slake durability of seven Indiana shales and the correlation coefficients of these variables are given in Table 5.2.3. It can be seen from Table 5.2.3 that Na in meq/liter from the saturation extract, Mg from the CEC, percentages of chlorite and kaolinite clay minerals, and Loss On Ignition (LOI), can be used to predict the slake durabiliity of the shales. Sodium Absorption Ratio (SAR) [which is a function of Na, Ca and Mg] can also be used to predict the slake durability as it has a fairly good correlation (Figure 5.2.2). SAR gave an R^2 value of 0.6097 when used in regression relative to the slake durability index for the seven shales.

The pH of the shales studied (Table 5.2.1) varies from 5.2 to 7.8 and does not show any definite trend with the slaking characteristic of the shales. New Albany shale which has a very high percentage of organic matter (about 5%) compared to the other shales is the most durable, and the hardest shale among those studied.

Table 5.2.1 Chemical Analyses Results

Chemical Analysis of Saturation Extract:

SHALE TYPE	Ca meq/liter	Mg meq/liter	K meq/liter	Na meq/liter	Total Soluble Salts meq/liter
1. New Providence	13.0	9.17	2.73	13.96	38.86
2. Mansfield	5.1	6.33	1.77	2.22	15.42
3. New Albany	5.2	2.75	0.74	0.77	9.46
4. Osgood	5.17	1.28	1.34	1.04	8.83
5. Attica	9.58	36.2	1.39	2.09	49.26
6. Palestine	2.34	5.12	0.16	1.87	9.49
7. Klondike	0.9	1.15	0.28	0.74	3.42

Chemical Analysis of Shale Samples:

SHALE TYPE	Ca meq/100gm	Mg meq/100gm	K meq/100gm	Na meq/100gm
1. New Providence	5.94	4.2	1.24	3.02
2. Mansfield	4.24	5.0	1.18	0.59
3. New Albany	4.63	1.71	0.76	0.22
4. Osgood	6.06	4.8	0.4	0.04
5. Attica	16.19	4.3	0.23	0.11
6. Palestine	4.08	4.93	0.09	0.21
7. Klondike	3.04	2.11	0.26	0.09

SHALE TYPE	Organic Matter %	Soluble Salts µmho/cm	Loss On Ignition (LOI) %	pH
1. New Providence	1.10	3.2	4.87	5.58
2. Mansfield	1.59	1.39	7.11	5.18
3. New Albany	5.36	0.89	20.71	6.45
4. Osgood	0.58	128	20.6	7.8
5. Attica	0.59	275	3.8	4.9
6. Palestine	1.01	22	4.8	7.1
7. Klondike	0.18	8	2.2	6.3

Table 5.2.2 Results from the Chemical Tests

SHALE TYPE	New Providence	Mansfield	New Albany	Osgood	Attica	Palestine	Klondike
CEC meq/100gm	16.61	16.36	7.92	11.6	36.7	11.1	7.3
$ESP = \frac{Na}{CEC}$ %	18.2	3.6	2.8	0.34	0.3	1.89	1.23
$SAR* = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}}$	4.19	0.93	0.39	0.58	0.44	0.96	0.73
$\% \text{ Sodium}* = \frac{Na}{(Ca + Mg + Na + K)}$	36	14	8	11.78	4.24	19.58	24.1
$ESR = \frac{Na}{CEC - Na}$ %	22.2	3.74	2.86	0.35	0.30	1.93	1.25

* The ratios are from saturation extract.

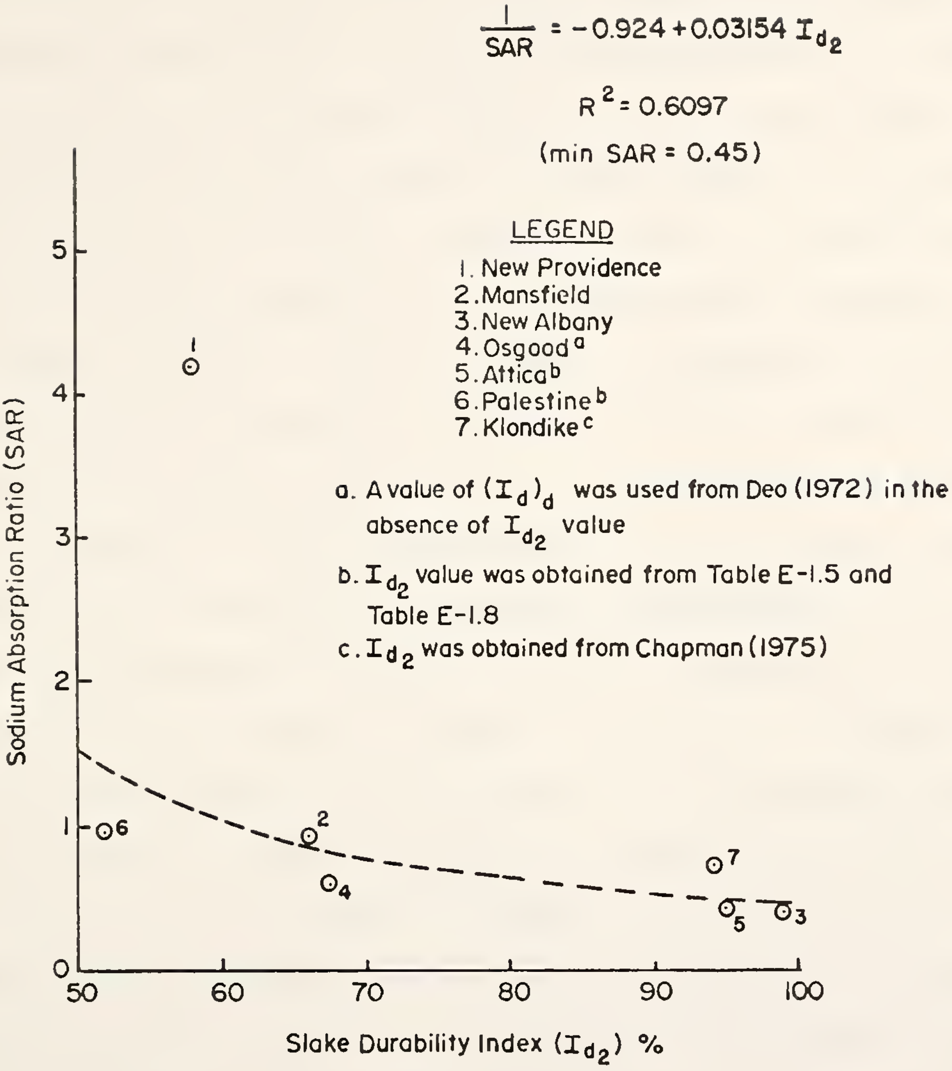
TABLE 5.2.3 COEFFICIENT OF CORRELATION FOR THE VARIABLES USED IN THE REGRESSION EQUATION TO PREDICT THE SLAKE DURABILITY INDEX OF SEVEN INDIANA SHALES

X2	.51549														
X3	.87598	.24919													
X4	.78723	.08550	.78299												
X5	.34677	.95182	.25581	.02132											
X6	.25498	.23676	.38715	.20709	.23198										
X7	.53599	-.18468	.78091	.61690	-.24248	.06085									
X8	.74562	-.01059	.78619	.98950	-.08292	.15313	.70739								
X9	.00007	-.23190	-.09682	-.13679	-.20867	-.53645	.32748	-.05384							
X10	.31691	.82557	.07925	-.20718	.92022	.29072	-.46751	-.32180	-.35621						
X11	-.01746	-.35395	.23611	-.03340	-.16468	.56491	.04292	-.04478	-.28589	.09312					
X12	-.51559	-.65659	-.47754	-.32580	-.48578	-.02187	-.43477	-.33223	.02341	-.19941	.59823				
X13	.59697	.97614	.39070	.14016	.94904	.39680	-.07118	.04964	-.30354	.82816	-.19169	-.66517			
X14	.71029	-.06874	.74505	.97741	-.14216	.08528	.71362	.99571	.01564	-.38595	-.07564	-.29882	-.02019		
X15	.68150	-.05155	.72011	.98728	-.12131	.18961	.60893	.98687	-.16483	-.32952	-.00542	-.24247	.00465	.98093	
X16	.29749	-.37867	.32818	.75773	-.48088	.02437	.36170	.77416	-.12221	-.58153	-.03525	.00491	-.35791	.78520	.84645
X17	.71975	-.05585	.74871	.98231	-.12499	.09065	.69870	.99675	-.00372	-.36456	-.06696	-.29326	-.00746	.99952	.98441
X18	.45500	-.21647	.53578	.50463	-.27096	-.25518	.88058	.60205	.69879	-.53698	-.71272	-.34193	-.18999	.64370	.68727
X19	.03362	-.10134	.39768	.05652	-.22084	.36201	.66705	.14560	.04953	-.31666	.07106	-.47131	.04124	.12488	.06911
X20	.28102	-.08382	.62735	.38581	-.21733	.40214	.81943	.46509	-.02229	-.36999	.08876	-.53122	.06881	.44141	.39708
Y	-.28078	.02745	-.38342	-.46282	.06355	-.88340	-.17608	-.42653	.40195	.06362	-.48589	-.12207	-.10291	-.38833	-.48358
	X1	X2	X3	X4	X5	X6	X7	X8	X9	X10	X11	X12	X13	X14	X15

X17	.78686				
X13	.23576	.62484			
X19	-.00434	.10420	.42143		
X20	.26596	.42431	.54421	.94214	
Y	-.37519	-.39019	.04482	-.24600	-.18008
	X16	X17	X18	X19	X20

LEGEND

x_1 Ca from Saturation Extract
x_2 Mg from Saturation Extract
x_3 K from Saturation Extract
x_4 Na from Saturation Extract
x_5 Ca from CEC
x_6 Mg from CEC
x_7 K from CEC
x_8 Na from CEC
x_9 % Organic Matter
x_{10} Soluble Salts
x_{11} Loss on Ignition
x_{12} pH
x_{13} Total CEC
x_{14} ESP
x_{15} SAR
x_{16} % Na
x_{17} ESR
x_{18} Illite (%)
x_{19} Kaolinite (%)
x_{20} Chlorite (%)
y Slake Durability Index



$$\frac{1}{SAR} = -0.924 + 0.03154\, I_{d_2}$$

$R^2 = 0.6097$

(min SAR = 0.45)

a. A value of $(I_d)_d$ was used from Deo (1972) in the absence of I_{d_2} value

b. I_{d_2} value was obtained from Table E-1.5 and Table E-1.8

c. I_{d_2} was obtained from Chapman (1975)

FIGURE 5.2.2 SLAKE DURABILITY INDEX VS SODIUM ABSORPTION RATIO FOR SEVEN INDIANA SHALES

5.2.2 Effect of Chemical Additives on Slaking

Slaking of shales was studied in the simple (one cycle) slaking and slake durability tests. The slaking fluid used in these tests consisted of salt solutions at one level of concentration. The preliminary test results on Mansfield shale are given in Table 5.2.4. The concentration of the slaking fluid was kept constant at one relatively low level, viz., 0.1 N solution. Table 5.2.5 gives the weights of salts to be used to obtain different concentrations of the salt solutions. It is evident from this table that higher concentrations of the salt solutions are not economically practical. The salts used were those commonly used in civil engineering practice either as chemical additives or as catalysts with lime stabilization. Sodium chloride, calcium chloride, ferric chloride, calcium sulfate, aluminum sulfate and ferrous sulfate were selected. The results when these solutions were used as a slaking fluid are presented in Tables 5.2.6 and 5.2.7. Figures E-3.1 and E-3.2 in Appendix E-3 show the behavior of New Providence and Mansfield shales in the above slaking fluids. Tables 5.2.6 and 5.2.7 give the mean value of the Index, the change in mean value of the Index compared to the Index when water was used as a slaking fluid, percentage change of the above mean, standard deviation, and coefficient of variation. By examining the coefficient of variation, it can be seen that the inherent variation in the result of a treatment can be very appreciable. To facilitate the comparisons of the different treatments with water, a 't' distribution was used in the statistical hypothesis testing. The null hypothesis was that there is no difference between the mean value of the index obtained for any one treatment when compared with the control slaking (i.e., when water was used as a slaking fluid).

Table 5.2.4 Slake Durability Index of Mansfield Shale (Preliminary Tests) with NaCl in the Slaking Fluid

Concentration	0.1 N	2N
Slake Durability Index	55.33 67.39	81.22 74.32
Mean	61.41	77.77

Table 5.2.5 Weights of Salts Used in the Slaking Fluid

Concentration / Salts	0.1 N gm/1000cc	1 N gm/1000cc	2 N gm/1000cc	0.1 N gm/3300cc	1 N gm/3300cc	2 N gm/3300cc
NaCl	5.84	58.4	116.8	19.27	192.7	385.4
$CaCl_2$	5.55	55.5	110.0	18.32	183.2	366.4
$FeCl_3$	9.01	90.1	180.2	29.73	297.3	594.6
$CaSO_4$	8.61	86.1	172.2	28.41	284.1	568.2
$AL_2\ (SO_4)_3$	11.11	111.1	222.2	36.66	366.6	733.2
$FeSO_4$	8.50	85.0	170.0	28.05	280.5	561.0

Note: Volume of the slaking fluid in the slake durability bath was 3300cc.

TABLE 5.2.6 SLAKE DURABILITY TEST RESULTS

SLAKING FLUID / SHALE TYPE		WATER	0.1 N NaCl	0.1 N $CaCl_2$	0.1 N $FeCl_3$	0.1 N $CaSO_4$	0.1 N $Al_2(SO_4)_3$	0.1 N $FeSO_4$
NEW PROVIDENCE	MEAN	58	69	60.5	57.4	69.3	34.5	71.3
	Δ MEAN		+11	+2.5	-0.6	+11.3	-23.5	+13.3
	% Δ MEAN		19	4.3	-1	19.5	-40.5	22.9
	Std. Dev.	5.85	8.12	8.15	1.04	3.52	7.06	3.95
	t		3.32	0.751	-1.41	7.86	-8.15	8.27
	Coeff. of Var.	10.08	11.77	13.47	1.81	5.08	20.46	5.54
MANSFIELD	MEAN	66	64.8	66.2	56.1	77.2*	88.5*	78.6
	Δ MEAN		-1.2	+0.2	-9.9	+11.2	+16.5	+12.6
	% Δ MEAN		-1.8	0.3	-15	17	25	19.1
	Std. Dev.	2.55	0.79	0.66	0.58	3.52	1.22	2.96
	t		-3.40	0.75	-41.7	6.36	27.05	9.52
	Coeff. of Var.	3.86	1.22	1.00	1.03	4.56	1.48	3.77

NOTE: There was no measurable difference in the Slaking Durability Index for the New Albany Shale

+ve sign indicates increased Slaking Durability Index

All the indices are mean of six tests

*Mean of four tests

TABLE 5.2.7 ONE CYCLE SLAKING TEST RESULTS

SLAKING FLUID / SHALE TYPE		WATER	0.1 N NaCl	0.1 N $CaCl_2$	0.1 N $FeCl_3$	0.1 N $CaSO_4$	0.1 N $Al_2(SO_4)_3$	0.1 N $FeSO_4$
NEW PROVIDENCE	MEAN	3.16*	0.84	0.47	1.06	0.79	1.52	0.65
	Δ MEAN		+2.32	+2.69	+2.10	+2.37	+1.64	+2.51
	% Δ MEAN		73.42	85.13	66.46	75	51.9	79.43
	Std. Dev.	1.48	0.19	0.064	0.11	0.099	0.92	0.36
	t		17.18	59.44	27.94	33.86	2.52	9.54
	Coeff. of Var.	46.84	22.74	13.62	10.03	12.53	60.47	55.49
MANSFIELD	MEAN	5.41*	3.32	5.75	5.72	2.64	1.24	2.20
	Δ MEAN		+2.09	-0.34	-0.31	+2.77	+4.17	+3.21
	% Δ MEAN		38.63	-6.28	-5.73	51.2	77.08	59.33
	Std. Dev.	1.23	0.4596	3.2881	5.332	2.375	0.24	0.45
	t		6.43	-0.15	-0.08	1.65	24.57	9.46
	Coeff. of Var.	23.97	13.84	57.18	93.22	89.96	19.35	21.82

NOTE: There are no measurable differences in the Slaking Index for the New Albany Shale

+ve sign indicates increased Slaking

All the indices are mean of two tests

*Mean of five tests

The treatments were compared at 95% and 98% levels of confidence as shown in Figure 5.2.3.

The slaking test was not very good in discriminating between the treatments, as the values of the 't' statistics were very high. However, results from the Slake Durability Test using the 't' distribution [$t_{0.02,5} = 3.365$, $t_{0.02,3} = 4.541$, $t_{0.02,1} = 31.821$, $t_{0.05,5} = 2.571$, $t_{0.05,3} = 3.182$, $t_{0.05,1} = 12.706$] for comparing the mean values of the index obtained from each treatment with the control were very satisfactory.

In the Slake Durability Test, sodium chloride and calcium chloride have no effect at the 98% level of confidence. Sodium chloride does have an effect at the 95% confidence level. Ferric chloride increases slaking of Mansfield shale but not of New Providence shale at a 98% level. Calcium sulfate and ferrous sulfate reduce slaking at a 98% level of both shales. The slaking of Mansfield shale with the addition of aluminum sulfate is different from that of New Providence shale. Aluminum sulfate produced increased durability, as measured by both slake durability and one cycle slaking tests, at a 98% level of confidence for Mansfield shale, which is rich in kaolinite content compared to the New Providence shale. Sodium chloride and calcium sulfate increased the durability in both slake durability and one cycle slaking tests for New Providence shale, which has a high percentage of sodium in the saturation extract.

New Albany shale, a durable shale, was unaffected by different slaking fluids. This shale is well cemented and has a high percentage (about 5%) of organic matter compared to the other shales.

Hypothesis $H_0: \mu_0 = \mu_1$

Alternative Hypothesis $H_1: \mu_0 \neq \mu_1$

μ_0 = mean value of slake durability index with water as the slaking fluid

μ_1 = mean value of slake durability index with salt solution as the slaking fluid

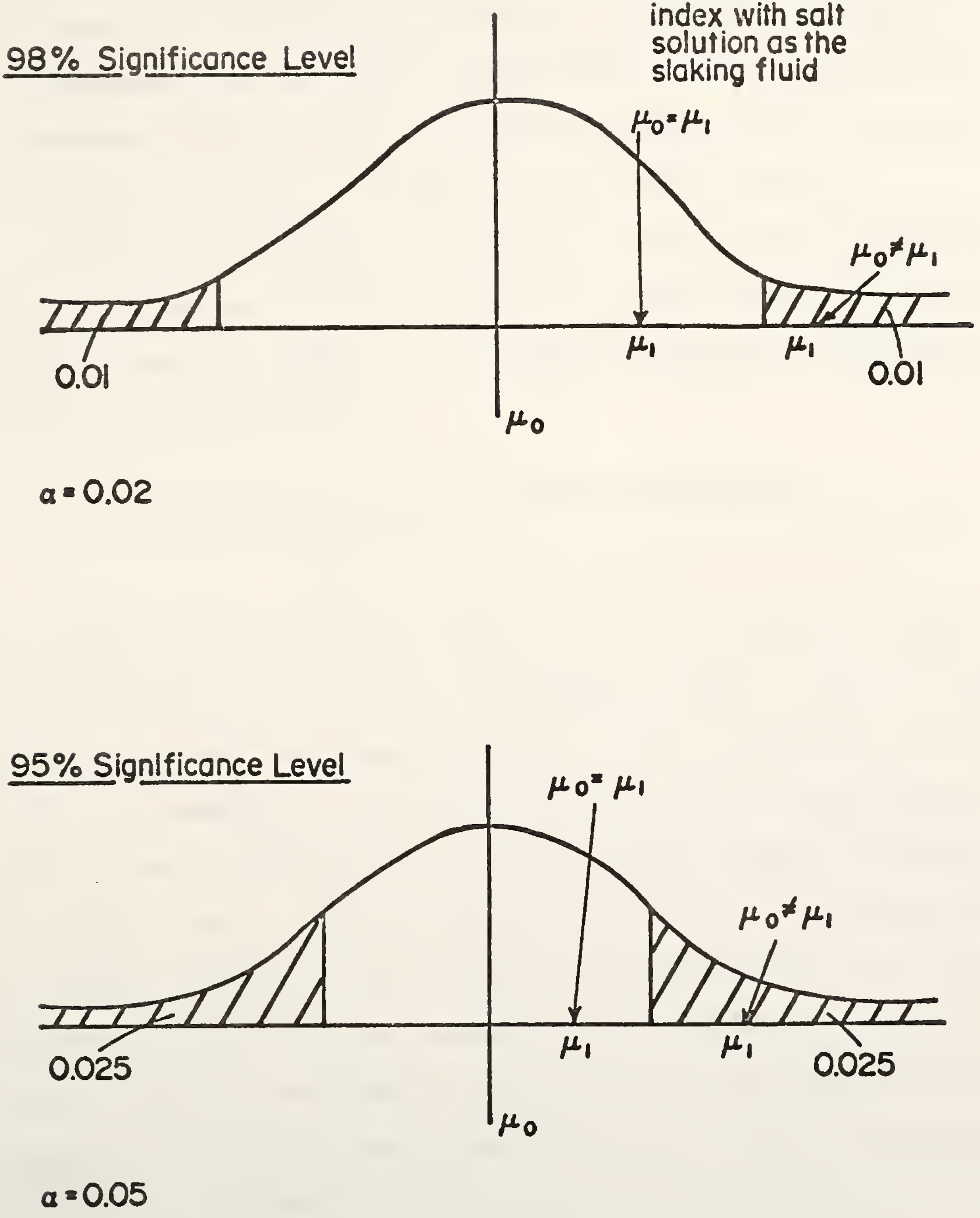


FIGURE 5.2.3 HYPOTHESIS TESTING

The results of the Slake Durability Tests with different slaking fluids is represented graphically in Figure 5.2.4, with the 95% and 98% confidence levels of the mean, and the range of the Index.

These results are sufficiently positive to indicate that the slaking properties of shales can be altered by altering the slaking fluid for a particular type of shale. Such chemicals can be incorporated in an embankment during placement through water addition prior to the compaction process. Water is essential to the chemical reaction.

Alteration of durability in the long term has not been investigated and will require both laboratory testing and field verification. Other chemicals not investigated in this study may be more effective for shales of different composition.

5.3 Slaking of Compacted Shales with Additives

5.3.1 Collapse on Soaking

Slaking tests and slake durability tests are performed on discrete pieces of shales. The various classification systems that have been developed use the above tests and other tests on discrete pieces of shale or in some cases on disaggregated samples, viz., Atterberg limits and pH. These classification systems are used to predict the behavior of a shale from a certain formation when used in a compacted shale embankment. The behavior of the shales in a compacted state is a little different from that of the discrete pieces. Various factors, viz., permeability of the embankment, ground water flow and pH, effect of prestress due to compaction forces, and gravity forces, affect the behavior of shales in the compacted state.

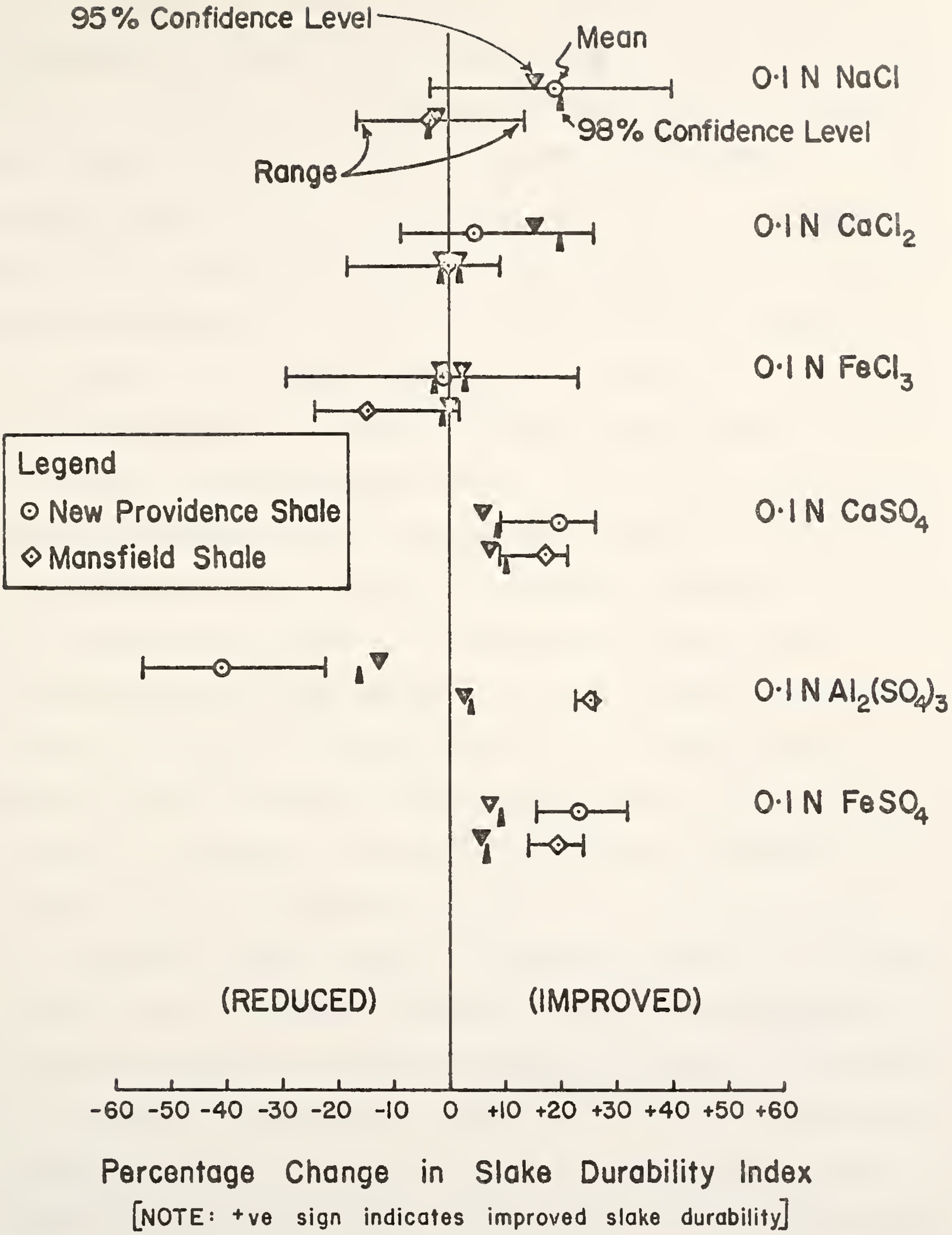


FIGURE 5.2.4 PERCENTAGE CHANGE IN SLAKE DURABILITY INDEX OF SHALE WITH DIFFERENT SLAKING FLUIDS

It has been shown (Hilf, 1948) that a cohesive soil compacted dry of optimum will collapse when saturated under a constant load. Mishu (1963) in his study of one-dimensional compressibility of compacted clays showed that there is a critical degree of saturation for a given compactive effort, below which collapse can occur. As the compactive effort is increased, the amount of collapse and the load required to cause the collapse also increases. Abeyesekera (1978) suggested that the weight of the specimen is enough to induce anisotropy in an isotropically confined specimen which could cause collapse on saturation in a triaxial specimen. Collapse of compacted shale on saturation in a laboratory oedometer test may result from the breakdown of the structure (i.e., weakening of clay binding the individual aggregates) and with time, breakdown of aggregates by the process of slaking. Bailey (1976) has shown that point load strength of discrete pieces of shales varies inversely as the moisture content and has the highest value when the shale is dry. Hence, shale pieces being loaded at various points of contact in a compacted mass will be subjected to breakdown as the strength reduces on saturation.

Abeyesekera (1978) devised a simple test called the "one-dimensional collapse test" which provides an insight into the compression and breakdown of shale under load. To exaggerate the effect of the particulate nature of shales, the test involves a loose hand placement of the discrete pieces of shale in a 10cm (4 in) consolidation cell. The sample thus prepared is loaded to a load equivalent of an average embankment height (6-7m or 18-21 ft) and water is allowed to enter the cell to cause collapse. Deformations are monitored throughout the loading increments and with time after soaking. The initial deformation during the loading increment gives an indication of the dry strength (or resistance to breakdown) and the collapse on wetting gives an indication of amount of softening and readjustment of the aggregates due to lubrication that takes place. The long term deformation after the soaking gives an indication of the collapse that might occur due to slaking as slaking is a time dependent phenomenon. The factors that were measured in this test are:

1) Initial gradation
2) Final gradation, which gives an index of crushing under load and soaking
3) Immediate collapse on soaking;
4) Final collapse.

The factors that affect this test are:

1) Initial gradation;
2) Initial state of packing;
3) Load at soaking.

An attempt was made to standardize this test. A total amount of 500 gm of shale was used in which 250 gm of material passed the 19.0mm (3/4 in) and was retained on 9.5mm (3/8 in) and the remaining 250 gm of material passed the 9.5mm (3/8 in) and was retained on a 4.75mm (#4) sieve. This sample was packed in the oedometer by hand with very little tamping. The load at soaking was chosen to be 140 kg which was derived from Abeyesekera's (1978) work corresponding to an average embankment height (6-7m or 18-21 ft). This test was performed on some of the shales with varying hardness (as defined by point load strength) and slaking (as defined by slake durability index) properties. The results are presented in Figures 5.3.1 to 5.3.4 and in Table 5.3.1.

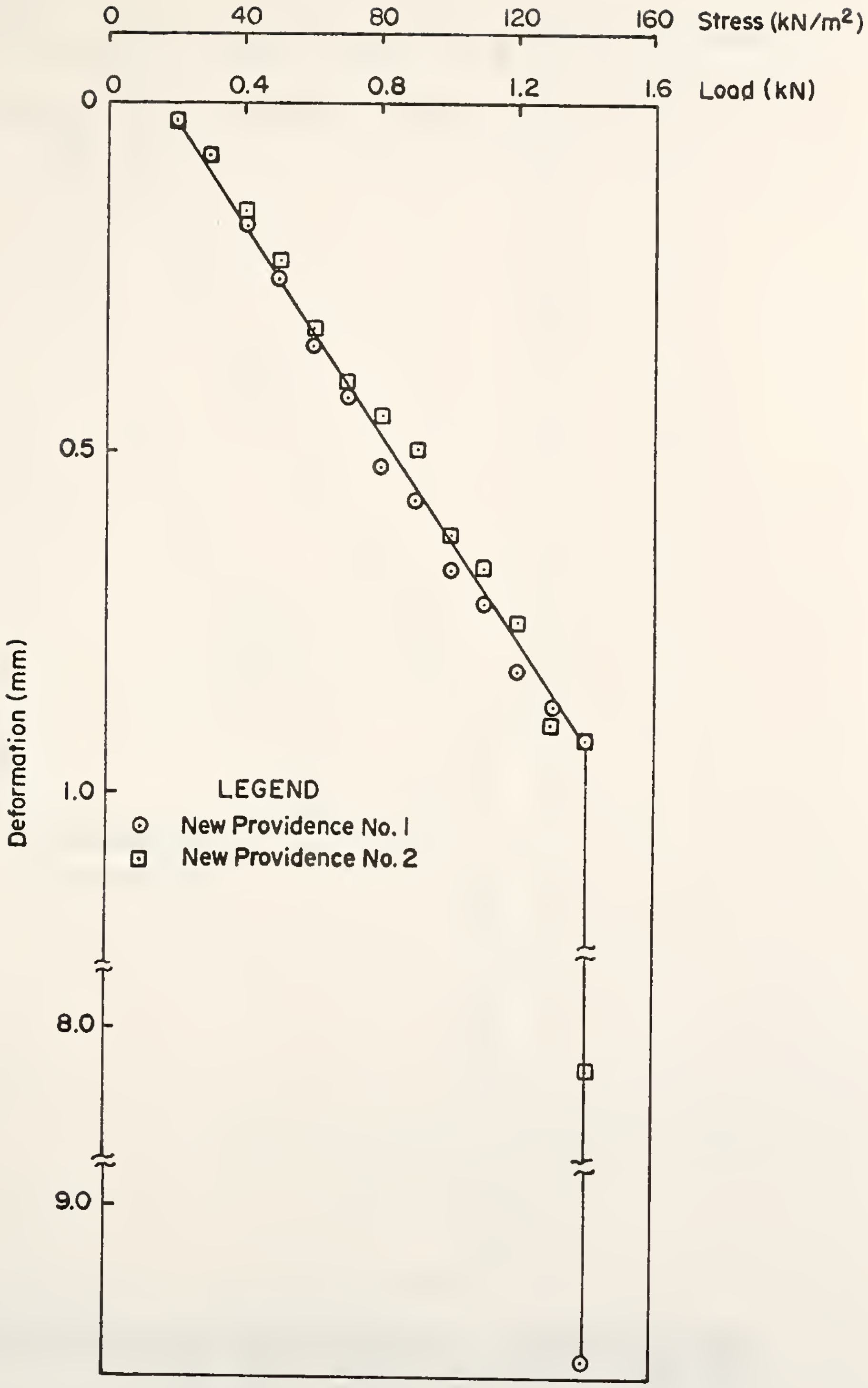


FIGURE 5.3.1 ONE DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING OF NEW PROVIDENCE SHALE

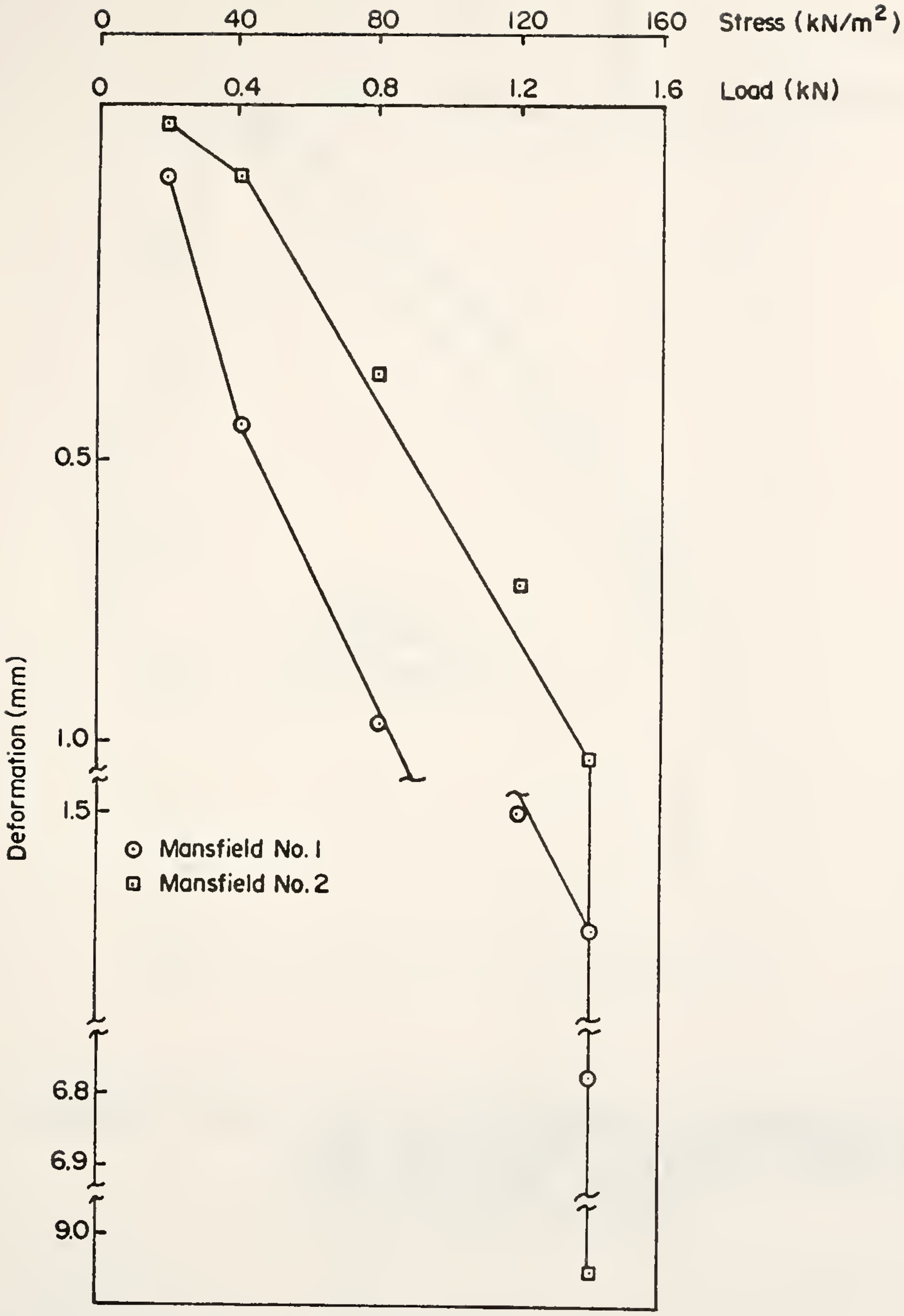


FIGURE 5.3.2 ONE DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING OF MANSFIELD SHALE

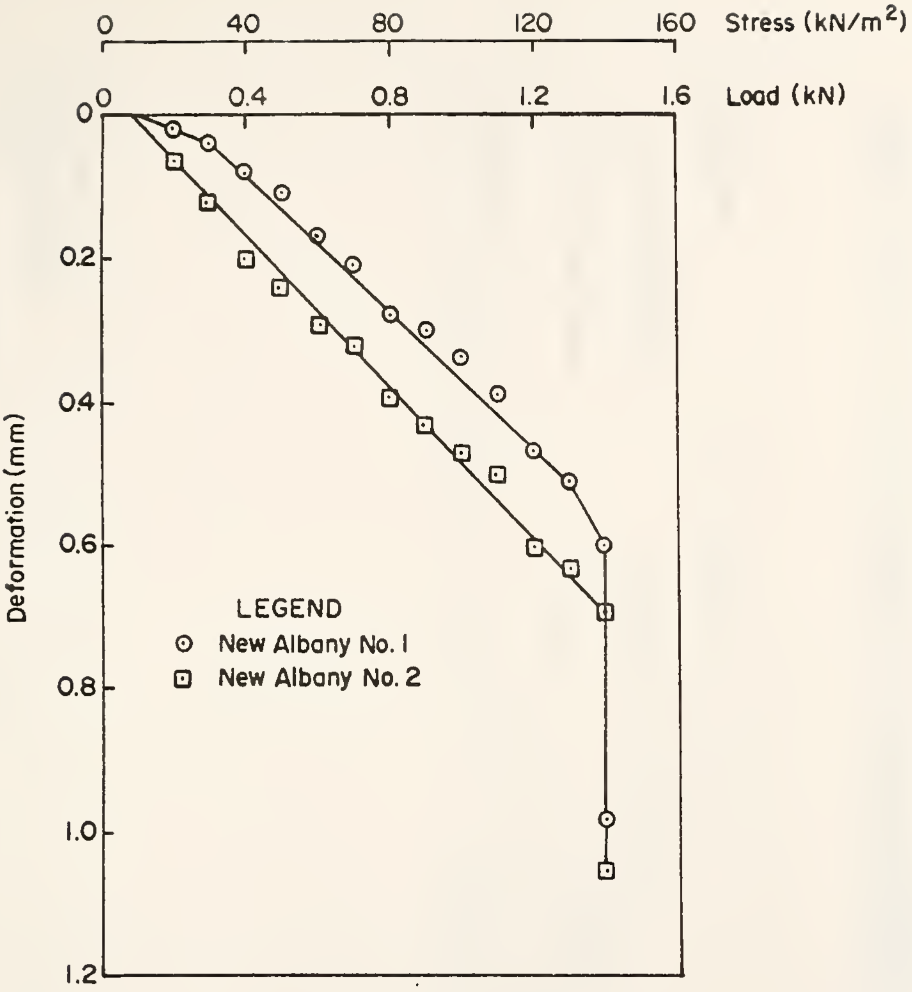


FIGURE 5.3.3 ONE DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING OF NEW ALBANY SHALE

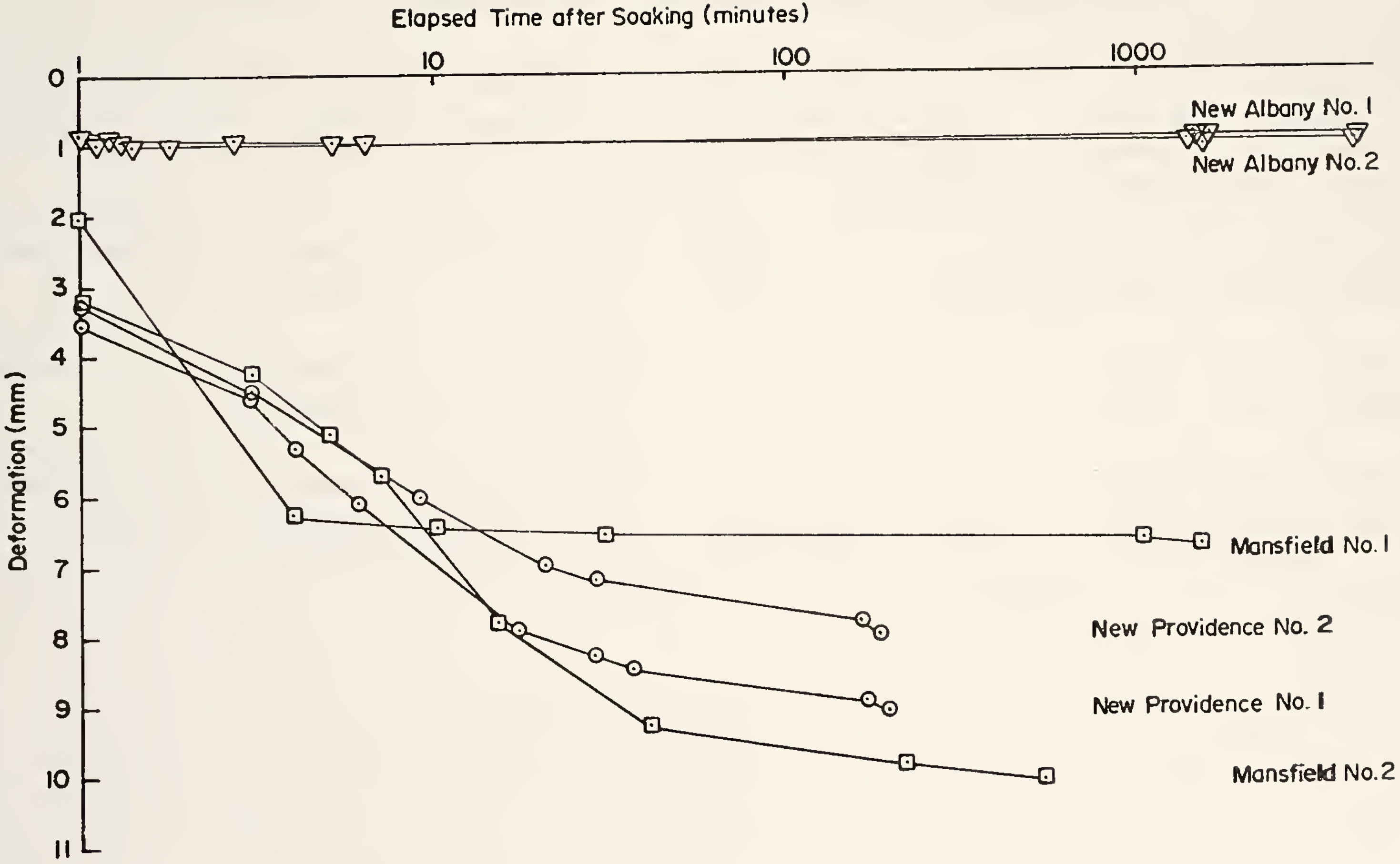


FIGURE 5.3.4 LONG-TERM DEFORMATION OF SHALES IN THE COLLAPSE ON WETTING TEST

Table 5.3.1 Results of One Dimensional Collapse on Wetting Test

Shale Type	Test #	Slaking Fluid	Initial Compression %	Collapse due to Soaking (after 5 min) %	Final Collapse after Slaking %	% wt. Loss through #4 sieve	Initial Density	
							lb/ft^3	kg/m^3
New Providence	1	Water	2.47	12.53	21.47	40.66	82.21	1384
	2	Water	2.47	11.53	18.60	39.48	82.21	1384
Mansfield	1	Water	5.07	11.20	12.47	4.27	77.73	1308
	2	Water	2.40	10.93	23.80	9.08	83.06	1398.3
New Albany	1	Water	1.57	0.21	2.57	1.04	82.21	1384
	2	Water	1.55	0.52	2.76	1.02	82.21	1384

5.3.2 Slaking of Compacted Shales

As mentioned previously in this section, slaking behavior of a shale is evaluated in the laboratory by slaking and slake durability tests. The values that are obtained from these tests are used as a guide to the behavior of the compacted shale in the field. In the case of the slaking test, a piece of shale is immersed in a slaking fluid (usually water) for a period of time and the amount of breakdown is measured as the percentage weight of material that is lost through a certain sieve. The mode of breakdown in this test depends on the type (i.e., composition, formation, age, etc.) of shale. Some shales tend to break down completely and the end product is a clay-like material, while others do not break down at all. There is an intermediate state of breakdown wherein the shale piece may reduce in size by the breakdown of the edges and in some cases by separation on the fissures. These types of shales, which have an intermediate type of breakdown, are most difficult to predict in their behavior in a compacted state. If the discrete piece of shale is loaded perpendicular to its fissures in the compacted state, it will break down to a lesser extent than when it is loaded parallel to its fissures. Hence, the breakdown of a shale due to water under load depends on how well cemented it is along its fissures. The slaked portion of a shale remains intact with its parent material unless displaced by an external force. This external force which removes the slaked portion from the parent piece of shale can be gravity (as in the case of a slaking test); induced mechanical energy (as in the case of slake durability) due to the tumbling action; and compactive forces (in the field during compaction and loads under the pavement in an embankment); as well as percolating water in or under an embankment.

Durability of compacted shale specimens were evaluated in the laboratory in the Slake Durability Test. The standard Slake Durability Test procedure was used except that the sample consisted of compacted shale instead of a charge of discrete pieces of shale.

As the size of the compacted sample used in the Slake Durability Test was much larger than the ones used in a standard slake durability test, no attempt was made to correlate indices obtained from these tests. As the sample size increases, the slake durability index decreases, and vice versa, as shown in Figure 5.3.5. When the compacted sample disaggregates into discrete pieces, the size of these discrete pieces are much smaller compared to the sample in the standard test and are expected to give a lower value of the slake durability index.

The sample size was not a factor when comparisons were made for different percentages of lime cured for different periods of time. The indices obtained from these tests were used to compare the relative effect of percentage of lime and curing period on the different types of shale.

In addition to obtaining the slake durability index, the type of breakdown of the sample was visually observed. In the case of compacted specimens subjected to a slake durability test, the specimen can undergo three types of fragmentation as shown in Figure 5.3.6 and represented schematically in Figure 5.3.7. The first type defined as 'Type I' is a complete disaggregation of the individual aggregates which constituted the compacted specimen at the beginning of the test. 'Type II' is one where the compacted specimen is reduced to less than half its initial volume with the remainder disaggregated. In this case the aggregates start to disaggregate at the boundary and progress inward. The last

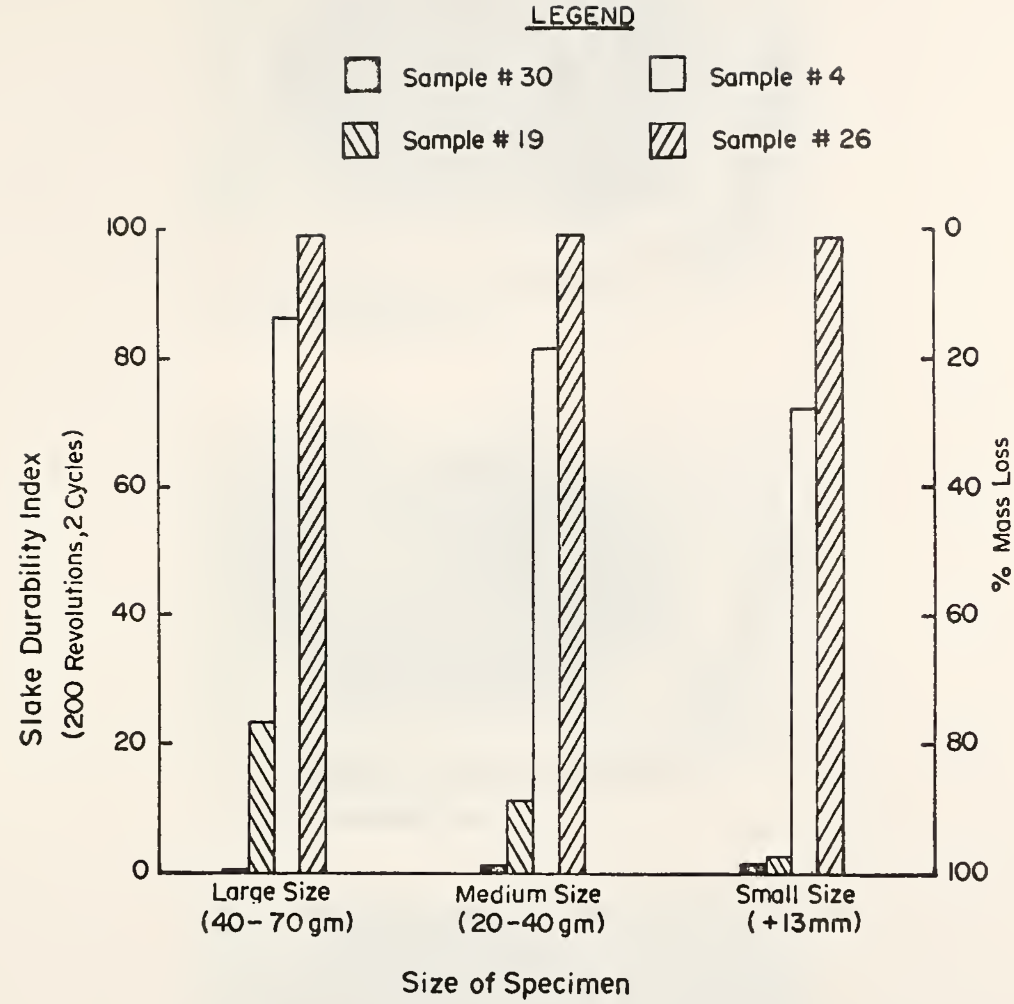
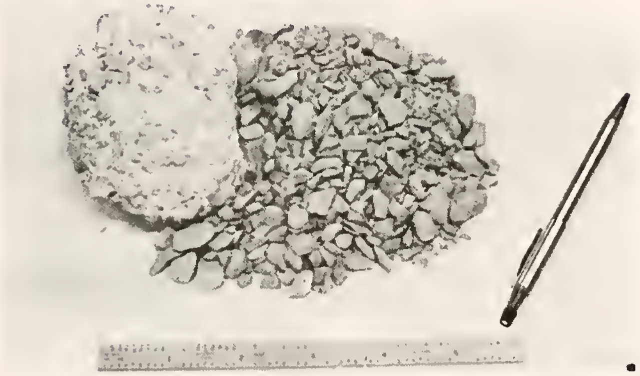


FIGURE 5.3.5 EFFECT OF SIZE OF SPECIMEN ON SLAKE DURABILITY INDEX (DATA FROM HUDEC, 1978)

Type I Fragmentation



Type II Fragmentation

Type III Fragmentation

FIGURE 5.3.6 TYPES OF FRAGMENTATION DURING SLAKE DURABILITY TEST OF COMPACTED SHALE

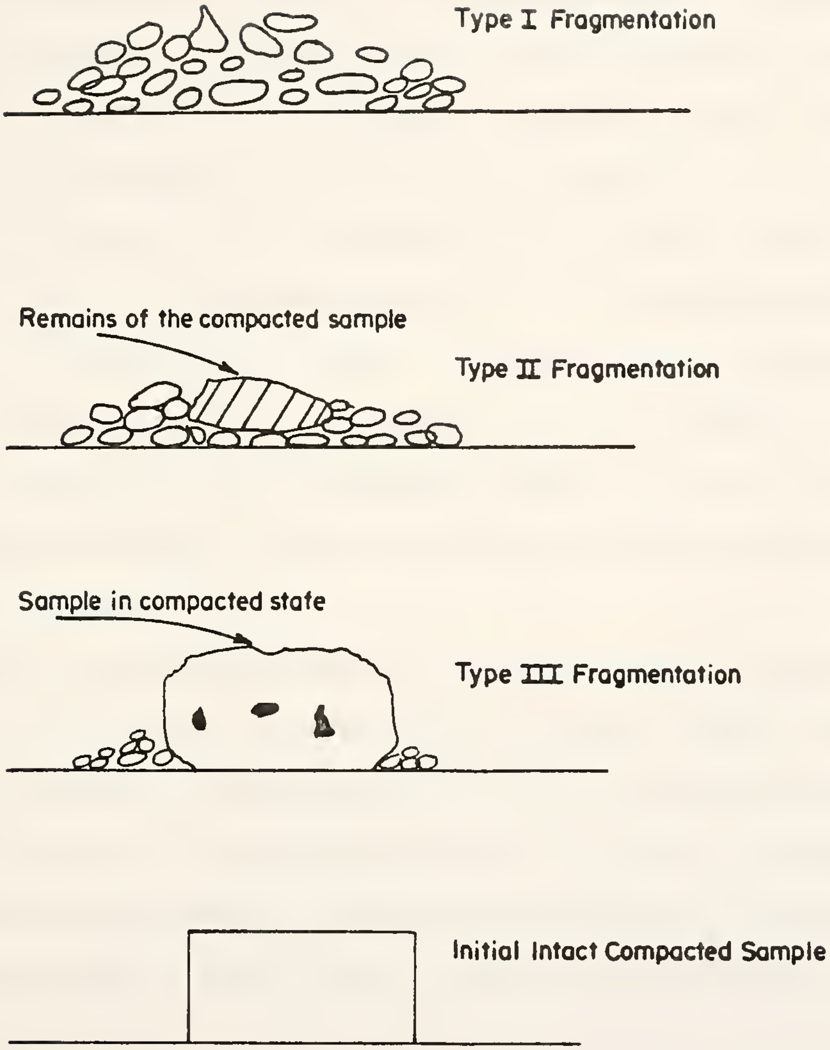


FIGURE 5.3.7 SCHEMATIC REPRESENTATION OF THE TYPES OF FRAGMENTATION DURING SLAKE DURABILITY TEST OF COMPACTED SHALE

type of breakdown is 'Type III', wherein the sample is little affected by the slaking process and more or less retains its intial shape and volume. The breakdown of shales in the compacted state can occur at two levels. The first level of slaking is disaggregation which is described by the Type I fragment. The second level is further breaking down of the aggregations once they have separated from the compacted state. There may be an intermediate state where the aggregates tend to break down while they are still in the compacted mass, but this can occur only on the edge or face of the compacted specimen. This intermediate state of breakdown is represented by Type II fragments. The intermediate state may ultimately result in a breakdown to Type I fragments. The reduction of Type II into Type I fragments depends on the type of shale and gradation, molding water content, and compactive effort used during the process of compaction.

An additive used during compaction can limit slaking of shales in two ways: (1) the additive can react with the discrete pieces of shale in reducing the amount of breakdown, or (2) act as a cementing agent in binding the aggregates of the compacted mass to maintain its compacted state. If the compacted sample is disaggregated, then the slaking properties of the individual shale particles control further slaking of the shale.

5.3.3 Lime Stabilization

Lime was tested as a stabilizing agent to reduce the slaking of New Providence and Osgood shales in the compacted state. Compacted samples were prepared as described in Section 4.6 with various percentages of lime, namely 3%, 5% and 7%. The shale samples were compacted with lime at about 2% wet of optimum to insure the availability of water for the reaction with lime during the curing period. Eight specimens were prepared for each shale type, percentage of lime and curing period. Samples for specific tests were selected from these specimens by using random numbers. Of the eight specimens, three were used for the Slake Durability Test (six samples were obtained by cutting them in half), four specimens were used for unconfined compressive strength tests (two for dry and two for soaked samples), and one specimen was used for simple slaking tests (two samples were obtained by cutting it in half).

At the end of the curing period the compacted samples were cut in half to give two samples of approximate size 101.6mm (4 in) in diameter and 55.9mm (2.2 in) in height. These samples were tested for durability in a slake durability apparatus. The slaking fluid was de-ionized water.

The results of slake durability testing of compacted samples with lime as the additive are presented in Table 5.3.2 for New Providence shale and in Table 5.3.3 for Osgood shale. The results are also presented graphically in Figures 5.3.8 and 5.3.9 for New Providence and Osdgood shales, respectively. It can be seen from Figure 5.3.8 that the durability of compacted New Providence shale does not improve until after 28 days of curing. The durability of the shale increases rapidly at 60 days curing period and further increases in curing period, viz., 90 days, produce very little improvement. The samples cured for 7 and 14 days had Type I fragments at the end of the test, while samples cured for 28 days had Type II fragments, with a chunk of about 50mm x 30mm of compacted sample left at the end of test. The rest of the sample was disaggregated. The samples tested at the end of 60 day and 90 day

Table 5.3.2 Slake Durability Index of Compacted New Providence Shale with Various Percentages of Lime

CURING PERIOD LIME %	7 days	14 days	28 days	60 days	90 days
0%	22.9*		n.a.	n.a.	n.a.
3%	30.28	32.7	51.1	76.22	80.61
5%	40.57	41.40	47.9	74.67	81.90
7%	39.90	42.77	42.3	78.62	73.72

* Tested between 7 and 14 days.

n.a. not applicable

Table 5.3.3 Slake Durability Index of Compacted Osgood Shale with Various Percentages of Lime

CURING PERIOD LIME %	7 days	14 days	28 days
0%	39.51*		n.a.
3%	50.42	55.68	55.43
5%	51.01	52.32	52.46
7%	50.83	54.37	52.26

* Tested between 7 and 14 days

n.a. not applicable

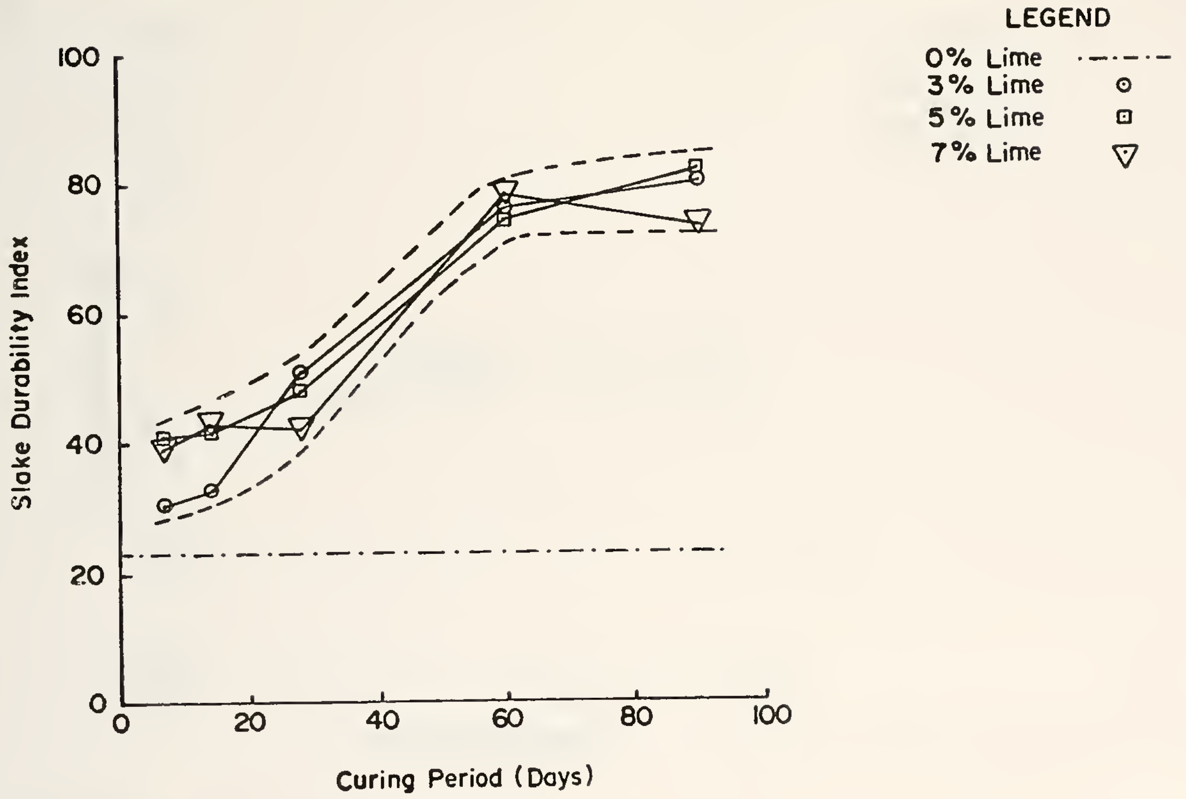


FIGURE 5.3.8 RELATIONSHIP OF SLAKE DURABILITY INDEX OF COMPACTED NEW PROVIDENCE SHALE WITH CURING PERIOD

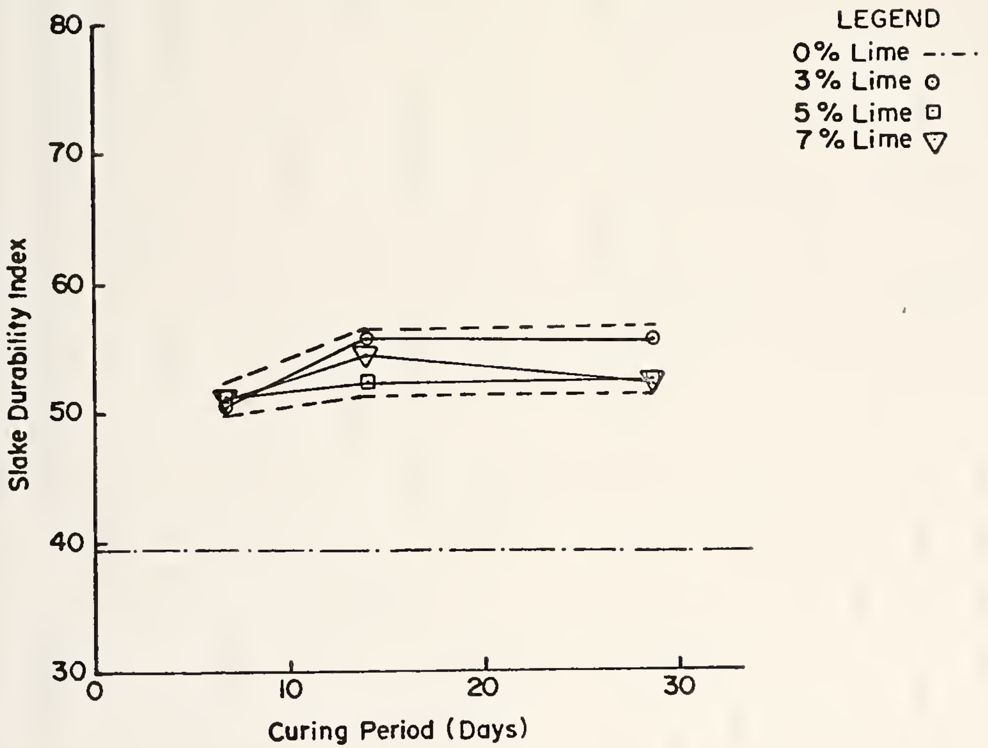


FIGURE 5.3.9 RELATIONSHIP OF SLAKE DURABILITY INDEX OF COMPACTED OSGOOD SHALE WITH CURING PERIOD

curing periods had Type III fragments with very little disaggregation. Osgood shale had Type I fragments at the end of all the tests that were performed for all curing periods and percentages of lime.

The durability of compacted shale should depend on the density to which it is compacted. If the density is high, the sample is more durable and will result in Type II and Type III fragments compared to a sample of lower density which will result in Type I fragments for a given shale type, percentage of lime and curing period. It has been shown (Gau and Olson, 1971) that the density of compacted specimens, as indicated by the penetration resistance (Leonards, 1955) for impact compaction, varies within the sample from top to bottom. Gau and Olson presented evidence that for samples compacted dry of optimum, the density at the top is higher than that at bottom and for the samples compacted near optimum, the density at the bottom was higher than at the top.

The approximate wet density (since the shape of the sample was not exactly cylindrical, with some damage caused during cutting of the sample, the volume of the sample was taken to be $0.000472 m^3$ (1/60 ft^3) in all cases) for all the samples tested is reported in Table E-2.1 and Table E-2.2 for New Providence and Osgood shales respectively, in Appendix E-2. The samples were not oven dried prior to the Slake Durability Test and water content at the end of curing period could not be determined, hence the dry densities of the samples are not reported. As the shale samples used for the Slake Durability Tests were obtained from cutting a compacted sample into two, it was found that the top half of the sample had higher density than the lower half. This difference in density of the samples produced variation in the modes of fragmentation as well as in the slake durability indices for the same shale type, percentage of lime and curing period. The first and second slake durability index of all the compacted samples of New Providence and Osgood shales is presented in Table E-2.3 and E-2.4 respectively in Appendix E-2. It can be seen from Table E-2.3 and E-2.4 that the differences in the slake durability indices caused by the differences in the compacted densities were reduced at the end of the second cycle, and the differences of fragmentation were pronounced only at the lower percentages of lime and shorter curing periods. As the curing period was increased to 60 and 90 days, the difference in the type of fragmentation between two specimens of the same compacted sample were reduced.

Unconfined compressive strength tests were performed on samples at the end of each curing period for all the percentages of lime as described in Chapter 4. Samples were also soaked in a bath of water for a period of seven days and the reduction in strength due to soaking was measured by performing the strength tests on these soaked specimens. Some of the compacted samples of New Providence shale that were cured for less than 28 days disaggregated when soaked in water and all the samples of Osgood shale cured 28 days or less, disaggregated.

The results of the compressive strength tests are given in Table 5.3.4 and Table 5.3.5 for the New Providence and Osgood shales. The above results are represented graphically in Figures 5.3.10 and 5.3.11. It can be seen that both the dry (at the end of the curing period at its natural moisture content) and soaked strength for both the shales has an increasing tendency with respect to the curing period for a given percentage of lime. The dry strength of New Providence shale cured for 90 days is not presented as it exceeded the capacity of the testing

Table 5.3.4 Unconfined Compressive Strength (in kN/m^2) of Compacted New Providence Shale with Various Percentages of Lime and Curing Periods

CURING PERIOD / LIME PERCENTAGE	7 days		14 days		28 days		60 days		90 days	
	DRY	SOAKED	DRY	SOAKED	DRY	SOAKED	DRY	SOAKED	DRY	SOAKED
0		505.33 500.03	502.68*	D			n.a.		n.a.	
3	475.00 485.27	D	360.82 385.29 367.16	D	514.57 459.62 486.03	234.43 166.58	632.62 785.34	471.55 394.95		409.56 453.00
	480.13		371.09		486.74	200.51	708.98	433.25		431.28
5	462.86 445.07	D	547.39 448.73	212.92 246.29	566.15 482.79 566.70	X	838.85 979.09	376.95 378.67		270.40 433.70
	453.97		498.06	229.60	538.55		908.97	377.81		402.05
7	388.33 409.49	D	492.03 504.44	D	X	X	752.66 802.58			328.82 293.38
	398.91		498.23				777.62	382.19		311.10

* Tested less than 28 days, Air dried samples.

D Sample was disintegrated during soaking.

X Could not be tested because of sample damage.

n.a. not applicable

Table 5.3.5 Unconfined Compressive Strength (in kN/m^3) of Compacted Osgood Shale with Various Percentages of Lime and Curing Periods

CURING PERIOD / LIME PERCENTAGE	7 days		14 days		28 days	
	DRY	SOAKED	DRY	SOAKED	DRY	SOAKED
0	787.62 738.80 710.05	745.49*	D		n.a.	
3	147.69 151.07 149.38	D	38.34 38.82 38.58	D	88.88 76.60 82.74	D
5	151.55 157.41 154.58	D	142.04 157.41 149.72	D	203.68 218.88 211.78	D
7	254.56 222.71 238.64	D	258.18 284.14 268.66	D	278.97 400.39 339.68	D

* Tested less than 28 days, Air dried sample

D Sample was disintegrated during soaking

n.a. not applicable

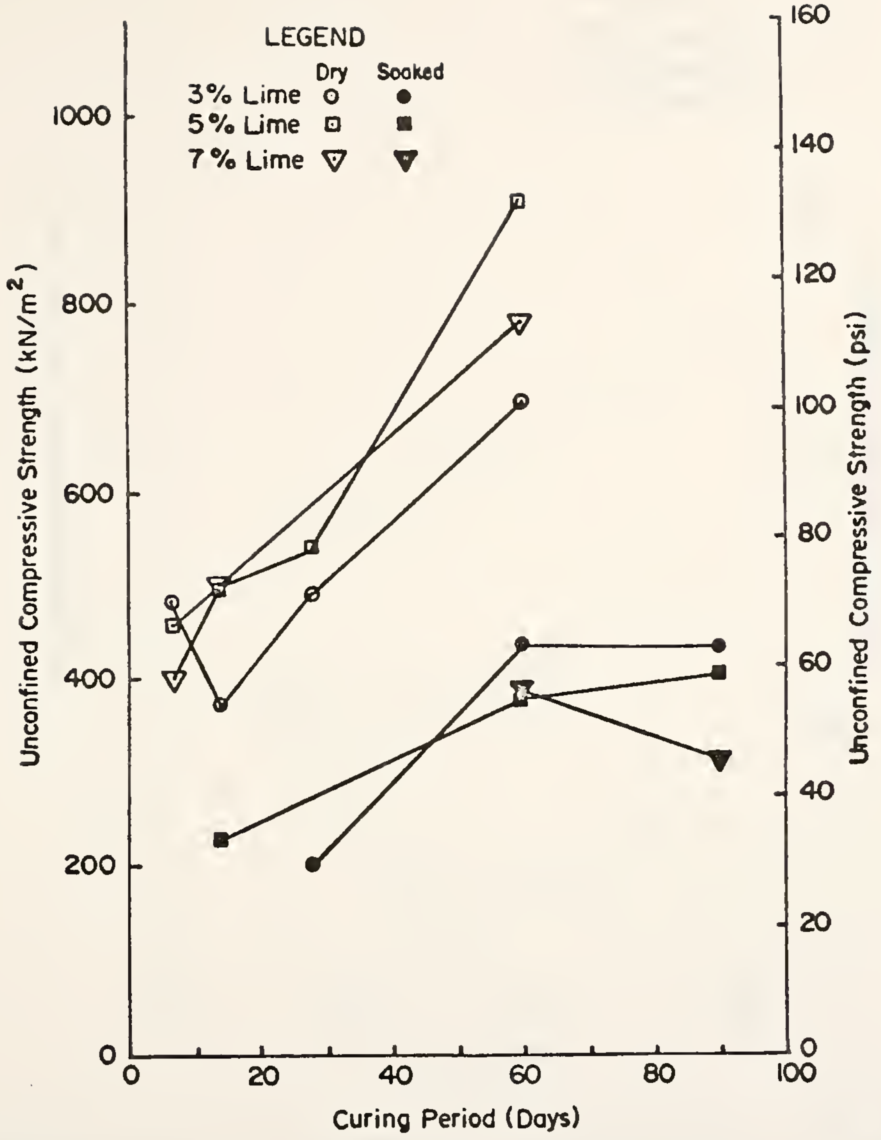


FIGURE 5.3.10 UNCONFINED COMPRESSIVE STRENGTH OF COMPACTED NEW PROVIDENCE SHALE WITH VARIOUS PERCENTAGES OF LIME AND CURING PERIOD

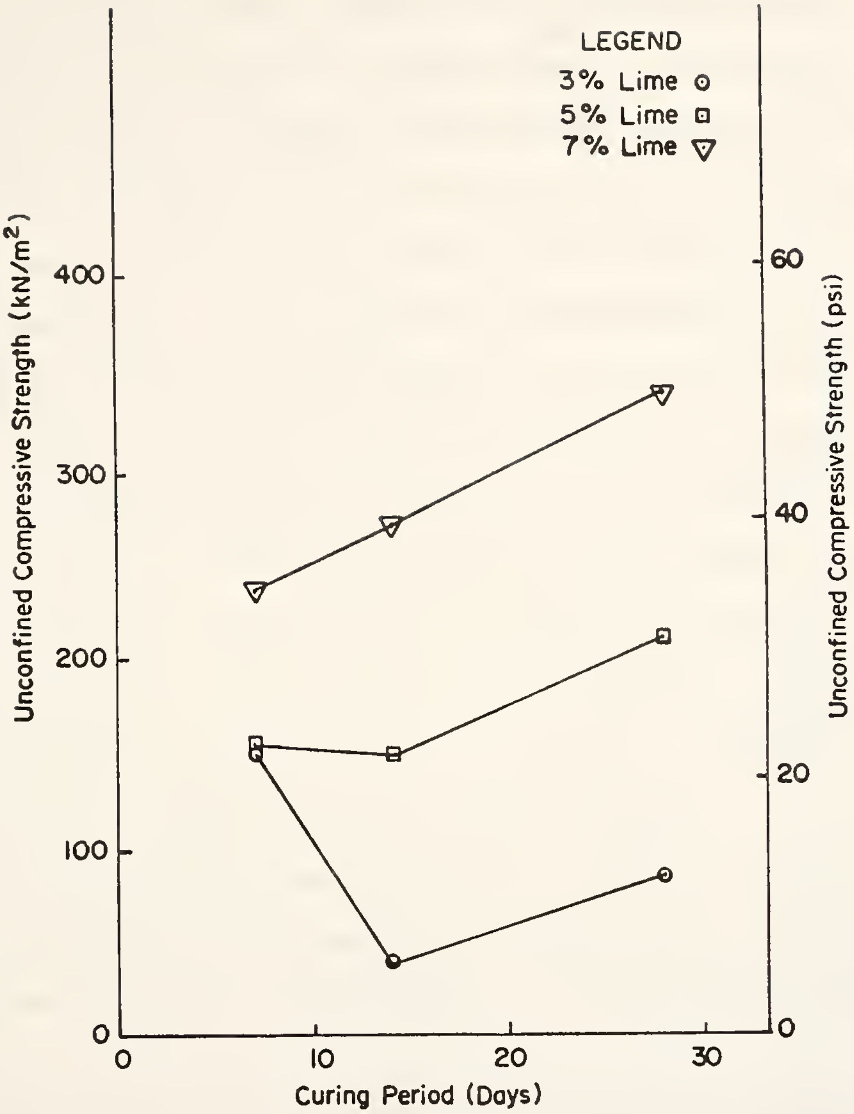


FIGURE 5.3.11 UNCONFINED COMPRESSIVE STRENGTH OF COMPACTED OSGOOD SHALE WITH VARIOUS PERCENTAGES OF LIME AND CURING PERIOD

machine. In Figures 5.3.10 and 5.3.11, the dry strengths of New Providence and Osgood shales compacted with 3% lime are higher at the end of seven days than after 14 days of curing. The reason for this discrepancy is that the specimen cured for seven days had a lower moisture content, compared to the ones cured for 14 days, by about 2.6% and 2.9% at the end of the curing period for New Providence and Osgood shales, respectively. From the effective stress principle, drier specimens will have higher strengths compared to the ones which have higher moisture content. Dry density and water content of the shale samples tested for unconfined compressive strength of New Providence and Osgood shales are presented in Tables E-2.5 and E-2.6 respectively, in Appendix E-2. The strength of the specimens also depends on the density to which they are compacted. It can be seen that the strengths of the compacted shales with additives are subjected to discrepancy in trends as a result of the variation in densities of the compactd specimens.

The variation of moisture content with curing period was much less, as shown in Figure 5.3.12. Some of the samples underwent drying during the process of testing which resulted in some variation of moisture contents.

The main intent of the study was to ivestigate the durability properties of compacted shales with lime as an additive. The strength tests were performed to show the corresponding effects on stability.

Simple (one cycle) slaking tests were also performed on the compacted specimens. The sample was prepared by cutting a compacted specimen into two and then soaking it in a beaker containing the slaking fluid (i.e., water). A 2mm (#10) mesh was used to support the sample in the beaker. Figure E-3.7 in Appendix E-3 gives the slaking of compacted

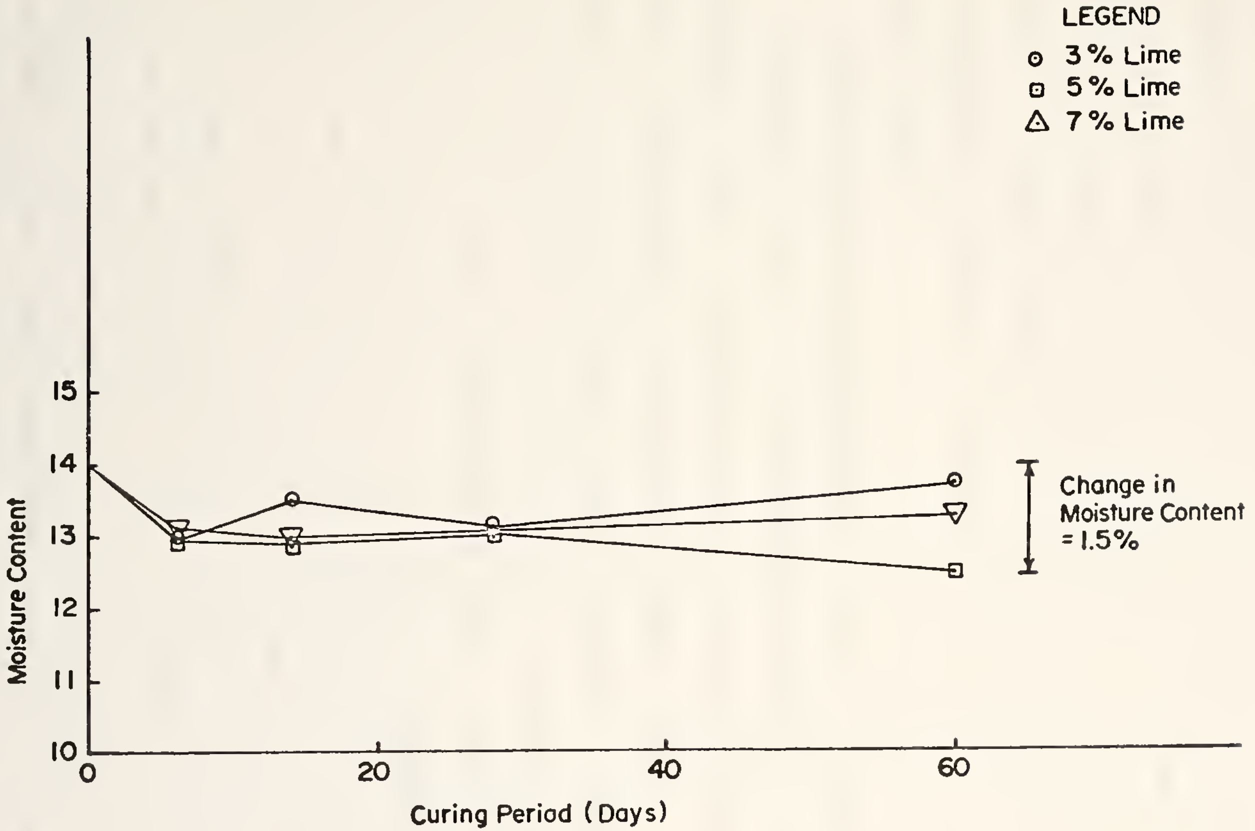


FIGURE 5.3.12 VARIATION OF MOISTURE CONTENT WITH CURING PERIOD OF COMPACTED NEW PROVIDENCE SHALE WITH VARIOUS PERCENTAGES OF LIME

New Providence and Osgood shales in water. The results of these tests are presented in Tables 5.3.6 and 5.3.7 for New Providence and Osgood shales, respectively. It was seen from these tests that it was difficult to get a meaningful result, as it was very difficult to get the weight of the amount of material retained on the screen. The amount of material retained on the 2mm sieve is obtained by wet sieving; and during this process, the shale particles started disaggregating progressively. Because of this practical difficulty, testing of the samples of the other curing periods were abandoned. The results of these tests were not used to assess the suitability of lime as an additive for compacted shales.

Summary of Lime Stabilization. Lime can be used to control slaking of compacted shales. This has been experimentally verified with the New Providence shale in the laboratory by conducting slake durability tests which satisfactorily discriminate between durable and non-durable shales. It can also be seen from the results in Table 5.3.2 and Figure 5.3.3 that lime was of little help in improving slake durability of Osgood shale. Exchangeable Sodium Percentage (ESP) gives an indication of lime reactivity with soil (Transportation Research Circular, 1967), and in case of the shales studied, the New Providence shale had a much higher ESP (ESP = 18.2%) compared to that of Osgood (ESP = 0.34%).

Thompson (1966) terms soils as "reactive" or "non-reactive" based on the increase in strength after the addition of lime and 28 days of curing. By this definition both the shales studied can be classified as non-reactive. Thus, unconfined compressive strength results cannot be used alone to determine the suitability of lime for increasing compacted shale durability.

Table 5.3.6 Slaking Index of Compacted New Providence Shale

CURING PERIOD % LIME	7 days	14 days
0	*	
3	11.8 12.04	18.54 21.24
5	17.13 19.51	18.48 17.80
7	17.03 16.54	15.58 17.38

Table 5.3.7 Slaking Index of Compacted Osgood Shale

CURING PERIOD % LIME	7 days	14 days
0	*	
3	17.12 16.52	17.17
5	16.55 13.38	11.95 14.10
7	12.43 12.07	12.02 11.05

* It was not possible to compute the Slaking Index because of the practical difficulty described in the text.

The type of breakdown, indicated by the mode of fragmentation, demonstrates the extent to which lime is effective. As mentioned earlier, Type II fragmentation is an intermediate stage of disaggregation, and may result in Type I fragmentation upon subsequent slaking. Hence when investigating the slaking characteristics of compacted shales, changes in type of fragmentation should be examined, as well as changes in slake durability indices.

5.4 Pore Size Distribution

Pore size studies were made on eight Indiana shales as described in detail in Appendix B-4. The intrusion constant used in this study was taken from Kaneuji (1978) as 160 [units: $(\mu)(\text{psi})$] or 1103.2 [units: $(\mu)(\text{kN/m}^2)$], and the same value was used for all the shales. Two tests were run on each shale and the differential and cumulative pore size distribution curves are presented in Figures B-4.2 to B-4.9 in Appendix B-4. The parameters from these distributions used for correlation with the slake durability index were:

(1) Cumulative porosity [Ratio of intruded pore volume to the volume of the sample];

(2) Median Diameter [Diameter of the pore corresponding 50-percentile of the intruded volume];

(3) Spread Factor [Ratio of pore diameter corresponding to 25-percentile of the intruded volume to the median diameter].

The above parameters from the cumulative pore size distribution curves for the shales studied are given in Table 5.4.1. The three parameters from the pore size distribution study (except for Osgood and Klondike shales) were used in linear regression to correlate with the slake durability index, and a value of $R^2 = 0.8146$ was obtained. Figure 5.4.1 gives a plot of estimated slake durability index against measured slake durability index.

It can be seen from Figure 5.4.1 that the slake durabilty index can be estimated with a good reliability from the pore size parameters.

5.5 Point Load Strength

Point load strengths were determined in the laboratory as described in Section 4.3 for six Indiana shales. All the samples tested were oven dried prior to testing. The thickness of the samples tested varied from 2.56mm (0.10 in) to 13.41mm (0.53 in) and the average was 6.59mm (0.25 in). The results of this test along with the proposed correction for the sample thickness are presented in Appendix D.

Table 5.4.1 Slake Durability Index and Pore Size Parameters for the Shales Studied

Shale	Slake Durability Index	$\frac{1}{\text{Cumulative Porosity}}$	$\frac{1}{\text{Median Diameter}}$	Spread Factor
New Providence	58*	11.83 11.30	71.43 71.43	2.00 2.14
Mansfield	66*	11.09 11.16	62.5 52.63	1.56 1.42
New Albany	99.1*	42.55 55.09	114.94 100.00	2.64 3.00
Osgood	67.3†	14.86 14.94	50.00 83.33	10.25 2.58
Attica	82.1X	9.29 9.06	37.04 38.46	1.52 1.54
Palestine	51.7†	8.12 5.57	3.57 8.33	2.93 4.00
Lower Hardinsburg	9.8‡	12.67 12.41	125.00 108.70	1.75 1.96
Klondike	98.3‡	7.69 6.33	22.20 27.03	1.78 2.38

* Results from this Investigation
† Results from ISHC Laboratory tests (Table E-1.5 and E-1.8)
‡ Result from Chapman (1975)
X Result from Abeyesekera (1980)

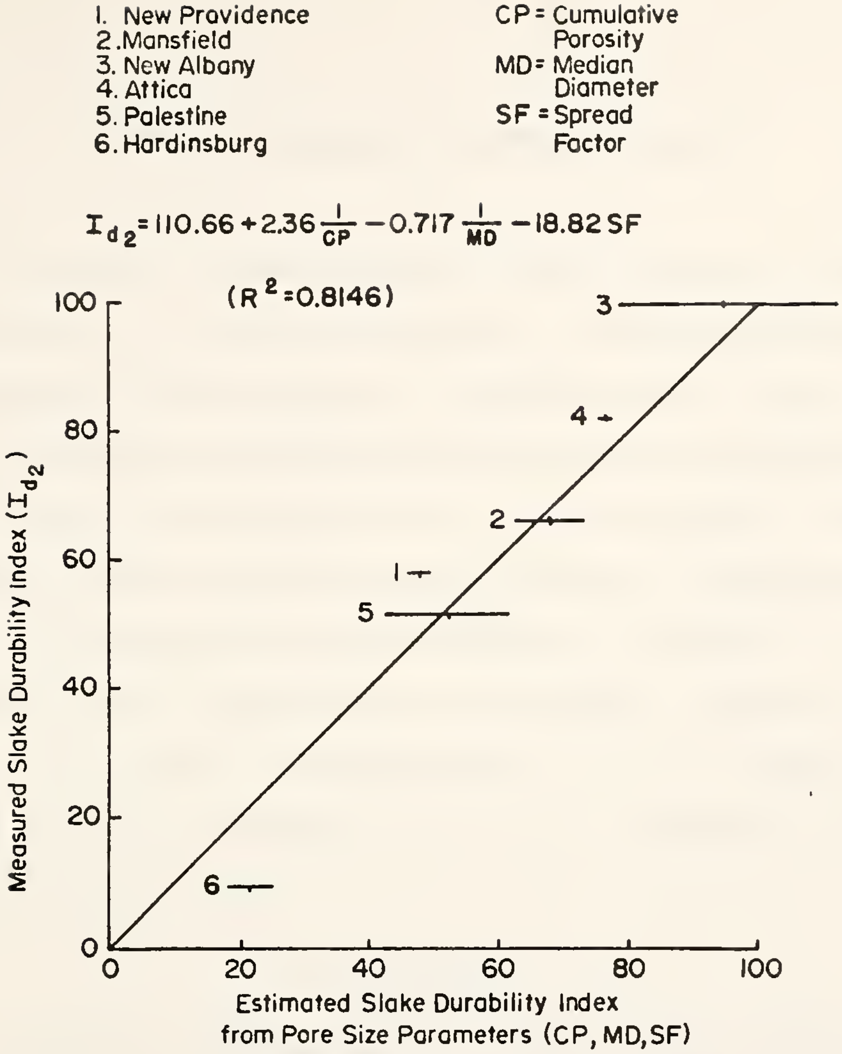


FIGURE 5.4.1 PLOT OF DURABILITY ESTIMATED FROM PORE SIZE PARAMETERS AGAINST MEASURED VALUES

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

The factors that control slaking of shales in service in compacted embankments are: (1) shale type (formation, age, mineralogy) and the mode of placement (lift thickness, compactive effort, height of the embankment), and (2) other environmental factors (e.g., ground water and pH). The objective of this research was to study the effect of chemical additives that could be applied during excavation or placement stages of construction to change the amount of slaking of the shale in service. The effect of lime on both durability and compacted strength was also studied. The conclusions reached from this study and recommendations for future study and implementation of the use of additives in the field are presented in this chapter.

6.1 Conclusions

The conclusions presented here are based on the results of the laboratory investigations conducted during this research on Indiana shales. Laboratory tests were performed on those shales which were sampled from a particular location in a shale formation. As the properties of these shales often vary laterally and with depth within the same formation, a variation in the results is expected for samples from different locations of the same formation.

1. Slaking of non-durable shales cannot be attributed solely to the "air-breakage" phenomenon. However,

this phenomenon weakens the shale and allows the other two mechanisms, viz., differential swelling and dissolution of cementing agents, to operate more effectively.

2. The slaking properties of shales can be changed (i.e., either increased or decreased as shown in Figure 5.2.4) by altering the slaking fluid. The type and concentration of the slaking fluid which will effectively alter the slaking property of a shale depends on the type of shale, viz., geological formation, age, chemical composition, clay minerals.
    a) The slake durability index of discrete pieces of New Providence (a hard and non-durable) shale was increased by adding sodium chloride, calcium sulfate, and ferrous sulfate to the slaking fluid at a concentration level of 0.1 N. Aluminum sulfate reduced the slake durability index when used in the slaking fluid at the same concentration.
    b) The slake durability of discrete pieces of Mansfield (a soft and non-durable) shale was increased when 0.1 N concentration of calcium sulfate, aluminum sulfate and ferrous sulfate was used in the slaking fluid. Ferric chloride at the same concentration level in the slaking fluid decreased the slake durability of Mansfield shale.

c) There was no measurable difference in the slake durability index of the New Albany (a hard and durable) shale when the chemical additives were used in the slaking fluid.

d) Shales of origins and chemical compositions similar to the ones tested can be expected to display similar changes in the durability characteristics. Other chemicals may be more effective in changing the durability of shales of different composition.

3. The sodium absorption rate (SAR) from the saturation extract test can be used to estimate the durability of shales as shown in Figure 5.2.2.

4. The parameters from the study of the pore size distribution of shales--namely, cumulative porosity, median diameter and spread factor--can be used to predict the durability. The pore size parameters along with the point load strength index can be used to classify the shales in regard to durability and strength, respectively (Figure C-7).

5. The one-dimensional collapse test can be used to classify a shale with respect to its strength and durability. The immediate compression may be used to evaluate the strength whereas long term deformation after soaking can be used to estimate the degradation due to slaking under load.

6. Lime mixed with the compaction water increased the slake durability characteristics of the compacted New Providence shale. The durability increased with increasing curing period and percentage of lime. There was a gradual increase in the durability up to 28 days and a sudden increase in the durability after 60 days of curing. Further lengthening of the curing period resulted in very little increase in the durability. An increase in percentage of lime had very little influence on the increase in the durability for curing periods after 60 days. Three percent lime was found to be a sufficient percentage.

   There is a tendency for increase in the unconfined compressive strength associated with increased curing period. However, the increase in strength at 28 days of curing was low and less than 344.75 kN/m^2 (50 psi) and hence the shale cannot be termed as "reactive" with respect to lime (Thompson, 1966).

7. Lime was not effective in increasing the durability of compacted Osgood (a hard and non-durable) shale, even though there was a slight increase in the slake durability index with the addition of lime.

   The unconfined compressive strength also increased slightly with the increased curing period, but the shale cannot be classified as "reactive" as defined by Thompson (1966).

8. Exchangeable Sodium Percentage (ESP) can be used to determine the reactiveness of a soil with lime (TRC, 1967). It seems that ESP can also be used as an indicator of reactiveness of a shale with lime. Lime was effective in increasing the durability of compacted New Providence shale which had an ESP of 18.2%; whereas Osgood shale with an ESP of 0.34% did not show any appreciable increase in durability with the addition of lime.

9. The Slake Durability Test can be used successfully to discriminate among lime-stabilized compacted shales in the laboratory.

   The mode of fragmentation during slaking of a compacted shale is instructive. A Type III fragment indicates that lime is effective in improving the durability as it binds the aggregates and retains the compacted state. A Type II fragment may lead to Type I fragment upon further slaking, hence that shale should be evaluated carefully even though it may have a high value of the slake durability index.

10. Increased durability of compacted shales with lime is accompanied by increase in dry and soaked unconfined compressive strengths.

6.2 Recommendations

6.2.1 Recommendations for Future Research

1. Mineralogical analysis by X-ray diffraction technique should be used on the shales tested during future inves-

tigations to identify the clay minerals present and establish correlation between the durability and type and amount of clay minerals.

2. Pore size distribution tests should be performed on all shales in the future, since parameters of these distributions have promise for predicting durability of shales.
3. Chemical analyses, described in Section 3.3.3, should be performed during future investigations on shale, since these also have potential in the prediction of slaking.
4. Alteration of durability by chemicals in the long term has not been investigated in this study and will require both laboratory testing and field verification.
5. The one-dimensional collapse test may be correlated with strength and durability properties. This single test could possibly replace other classification tests, provided sufficient correlation with field behavior is established.
6. The Slake Durability Test can be used to evaluate the performance of a lime stabilized compacted shale sample in the laboratory. The material that is retained at the end of the second cycle may be further sieved on a larger sieve opening (25.0 - 37.5mm or 1 - 1 1/2 in) and an index can be computed, based on the percentage by weight of material retained on this sieve, to get a better representation of the type of fragmentation of a compacted sample.

7. Laboratory investigation should be performed to determine if the change in pH with the addition of lime could be correlated with durability in order to determine the optimum percentage of lime to be used in compacted shales.

6.2.2 Recommendations for Field Application

1. If the results from additional laboratory tests show that the salt solutions can be used, they should be field tested and their performance monitored for both short and long term effects, particularly with regard to possible leaching of the salt solution.
2. Lime can be used in compacted shale embankments near bridge abutments to reduce the possible settlement of the approach. Lime can also be incorporated in the construction of an embankment by lime modified encasement of an untreated core.
3. Field investigations of lime stabilization are necessary to evaluate the effect of shale aggregate size.

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APPENDICES

The following Appendices are not included in this copy of this report:

Appendix	Title	Pages
A	X-Ray Diffraction and Chemical Test Procedures	176-221
B	Laboratory Test Procedures	222-255
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West Lafayette, Indiana 47907

APPENDIX C

CLASSIFICATION OF SHALES

APPENDIX C

CLASSIFICATION OF SHALES

Earlier classification systems were based on visual observation of physical features--viz., fissility, breaking characteristics, etc.--of shales in-situ and hand specimens in the laboratory (Twenhofel, 1937; Underwood, 1967; Gamble, 1971). These classification systems have been discussed in Chapter 2. The more recent classification systems take into consideration the results from durability tests and observed field behavior (Gamble, 1971; Deo, 1972; Hudec, 1978; Strohm, et al., 1978; Andrews, et al., 1979; Franklin, 1979). These classification systems may also employ test results in the form of Atterberg limits, pH's, rate of slaking values, and point load strengths. A classification system proposed by Morgenstern and Eigenbrod (1974) used the unconfined compressive strength of soaked specimens to classify the shales. A summary of these classification systems is presented in Table C-1.

The principal features needed in a shale classification system are:

1. a measure of durability (i.e., resistance to degradation during service); and
2. a measure of strength--or hardness--(i.e., resistance to breakdown in the field which determines the ease with which it can be placed in an embankment).

Table C-1 Summary of Classification Tests

Classification System	Durability Test	Strength Test	Other Tests	Comments
Gamble (1971)	200 revolutions, 2 cycle slake durability test	-	Atterberg Limits (I_p).	
Deo (1972)	a)500 revolutions 1 cycle, wet & dry slake durability b)slaking test c)modified soundness test	-	-	
Morgenstern & Eigenbrod (1974)	-	unconfined compressive strength	-	Loss of strength on soaking
Hudec (1978)	200 revolutions 5 cycle slake durability test	-	-	
Strohm, et al. (1978)	a)200 revolutions 2 cycle slake durability test b)slaking test c)rate of slaking test	-	a) pH b) Atterberg Limits (I_p)	Visual observation of the fragmented material in the slake durability test
Andrews, et al. (1979)	200 revolutions, 2 cycles, slake durability test	-	a) pH b) CEC	Preliminary classification based on pH and CEC determines the need for any durability test
Franklin (1979)	200 revolutions, 2 cycles, slake durability test (I_{d_2})	point load strength (PLS)	Atterberg Limits (I_p)	For $I_{d_2} > 80$, PLS is needed; For $I_{d_2} < 80$, I_p is needed.

Classification systems thus developed use measured properties in the laboratory to predict field behavior. The Slake Durability Test is normally used to distinguish durable and non-durable shales. Additional tests for durability are sometimes necessary (Noble, 1977; Strohm, et al., 1978; Franklin, 1979). Visual observation of the material at the end of the Slake Durability Test is also useful (Chapman, 1975; Strohm, et al., 1978).

The Indiana State Highway Commission uses the classification system developed by Deo (1972). Indiana shales may be placed in three categories, viz., hard and durable, hard and non-durable, and soft and non-durable, as shown in Figure C-1. In this Figure, the durability is rated from the Slake Durability Test and the "hardness" from the point load strength test. It can be seen that increasing durability is generally associated with increased point load strength. The properties of shales represented in this Figure are given in Table C-2.

In the Slake Durability Test, the specimens (discrete pieces of shale) are subjected to a joint effect of abrasion caused by the tumbling action of the rotating drum and softening by water. The shale may be cycled in this test by removing the retained material from the drum, oven drying it, and reinserting it in the drum. The breakdown is increased by increasing the number of cycles, as shown in Figure C-2. Drying the sample prior to the Slake Durability Test does increase the degradation. Varying reasons for this are supplied by Nakano (1967) and Bailey (1976). Degradation is also a function of the number of revolutions of the drum as shown in Figure C-3.

Point load strength indexes corrected for sample thickness (from Appendix D), and at zero moisture content are plotted on the Shale

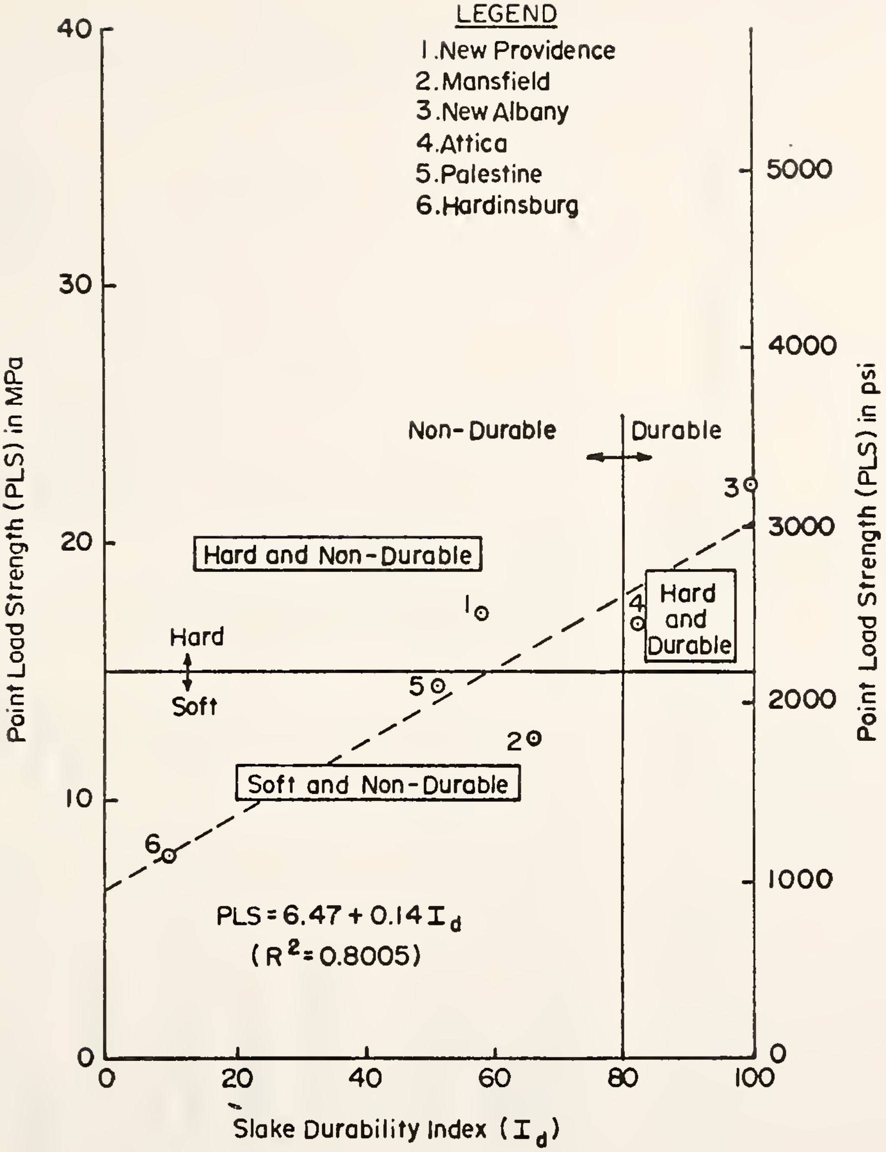


FIGURE C-1 VARIATION OF POINT LOAD STRENGTH WITH SLAKE DURABILITY INDEX

Table C-2 Properties of Shales Studied

Shale		Slake Durability Index I_d	Point Load Strength* (PLS) MPa	Plasticity Index I_p
No.	Formation			
1	New Providence	58.0	17.25	11
2	Mansfield	66.0	12.4	10.6
3	New Albany	99.1	22.3	5.6
4	Attica	82.1†	16.8	5
5	Palestine	51.7	14.5	5.2
6	Lower Hardinsburg	9.8	8.30	15

*Values corrected for sample thickness from Appendix D.

†Second cycle slake durability index was estimated (from Abeyesekera, 1980)

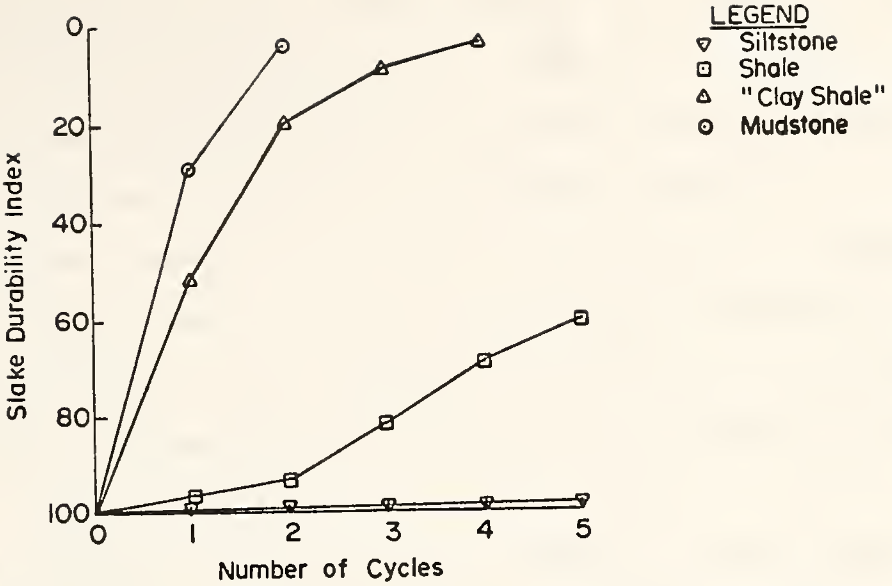


FIGURE C-2 INFLUENCE OF NUMBER OF CYCLES ON THE SLAKE DURABILITY INDEX (DATA FROM GAMBLE, 1971)

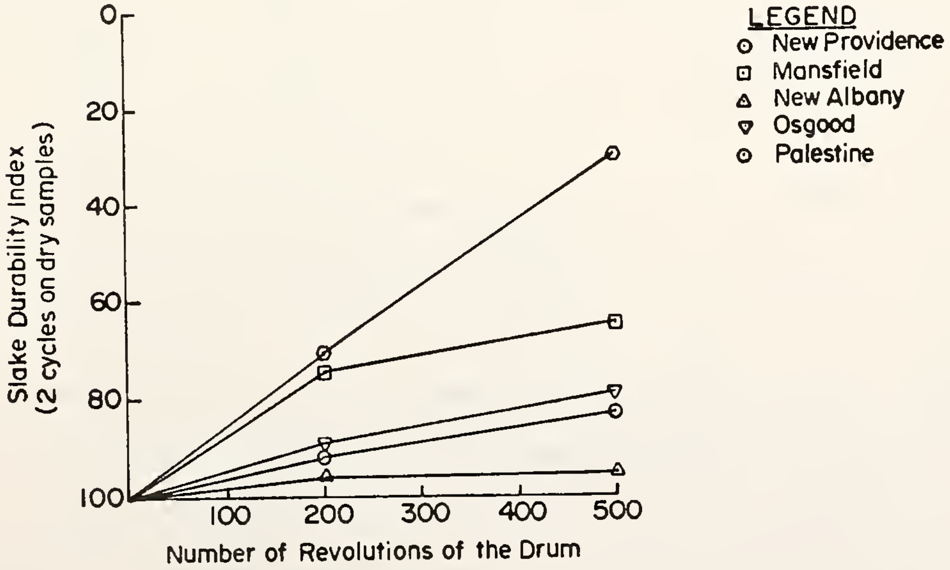


FIGURE C-3 INFLUENCE OF NUMBER OF REVOLUTIONS OF THE DRUM ON SLAKE DURABILITY INDEX (DATA FROM INDIANA HIGHWAY COMMISSION)

Rating Chart (after Franklin, 1979) in Figure C-4. Note that the New Albany and Attica shales plot outside the limits of the chart. The point load strength values plotted in this chart were not corrected for the size effect as proposed by Broch and Franklin (1972).

Parameters from the pore size distribution study, viz., cumulative porosity, median diameter and spread factor, were correlated with the slake durability by linear regression. The R^2 value for six shales was 0.8146. The results of this regression are presented in Figure C-5.

When the predicted slake durability index was plotted against the point load strength index, as shown in Figure C-6, it was found that most of the shales plotted in a narrow range of strengths with varying durabilities. The implication is that durability is more sensitive to pore size distribution than is strength. An explanation for this is:

1. 'Matrix' (the skeleton) of a shale is depicted by the pore size parameters.
2. Two shales having the same pore size parameters will have the same durability as predicted by the regression equations.
3. The durability and strength (as indicated by the point load strength) of a shale depends on its cemented nature.
4. It is hypothesized that point load strength as predicted by the pore size parameters when compared with its measured value, makes it possible to classify shales into durable or non-durable categories.

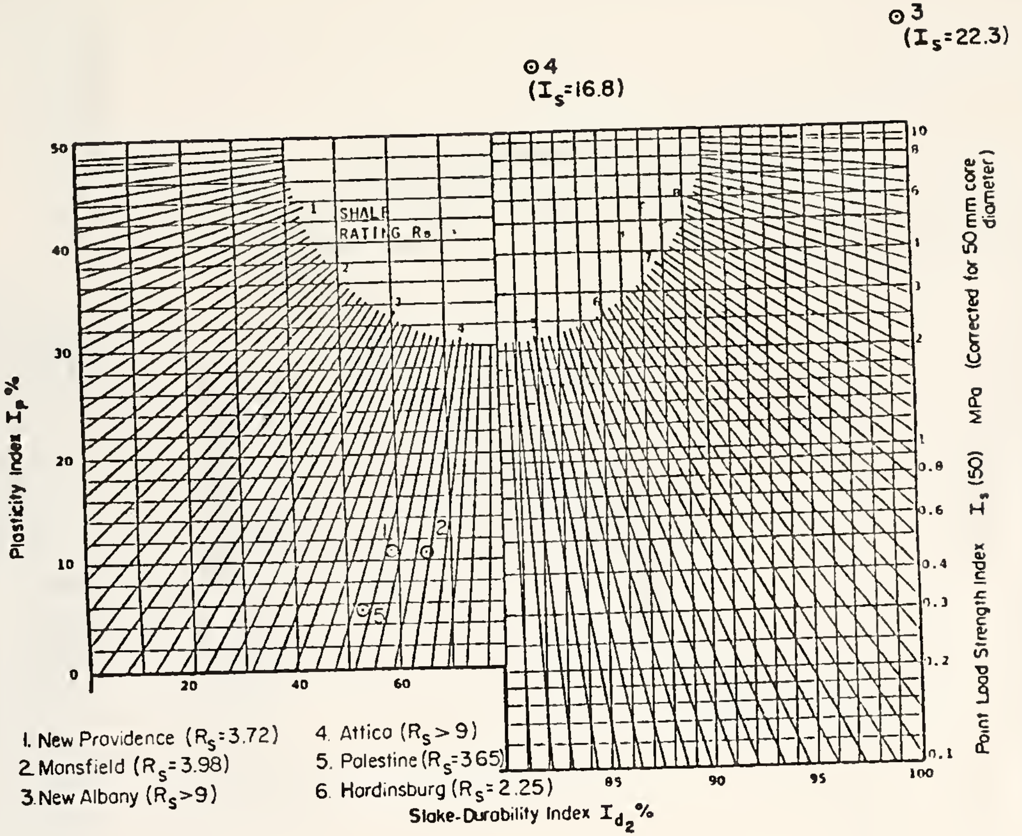
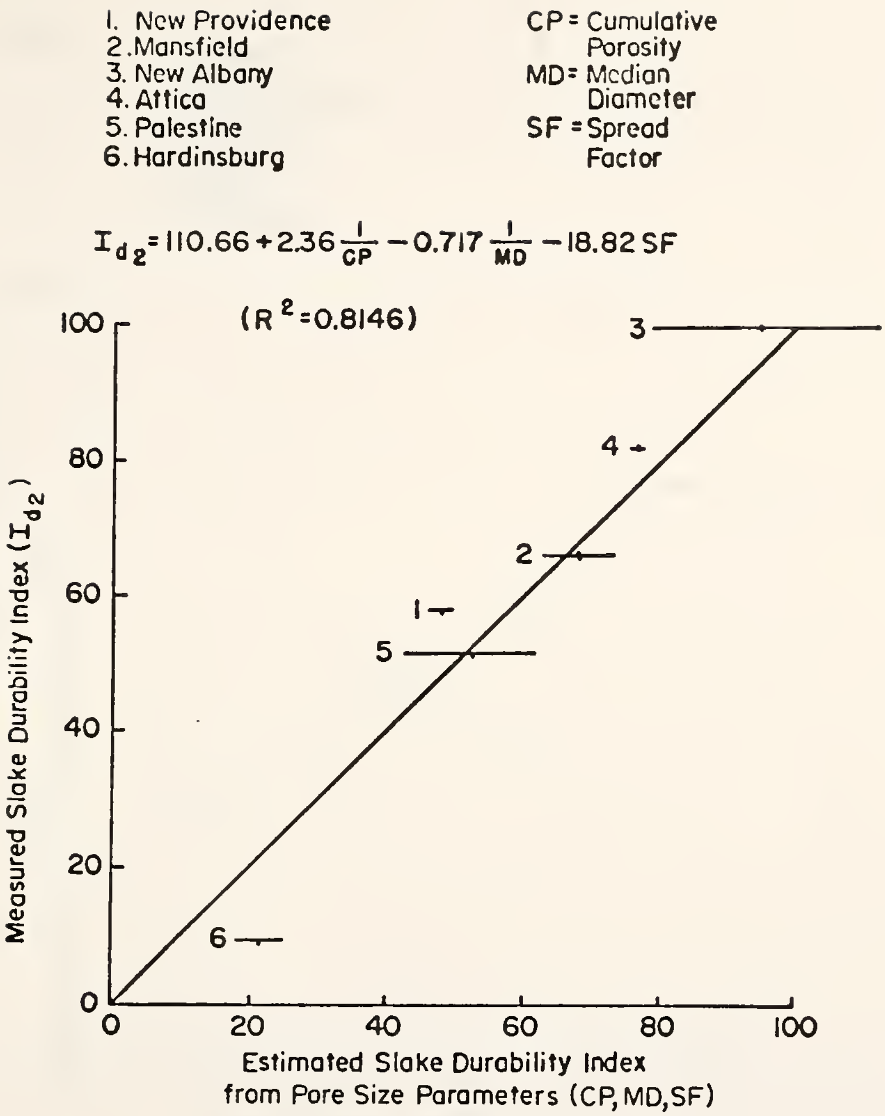


FIGURE C-4 TEST SHALES PLOTTED ON SHALE RATING CHART (AFTER FRANKLIN, 1979)



$$I_{d_2} = 110.66 + 2.36\frac{1}{CP} - 0.717\frac{1}{MD} - 18.82\,SF$$

$(R^2 = 0.8146)$

FIGURE C-5 PLOT OF ESTIMATED SLAKE DURABILITY INDEX (I_{d_2}) FROM PORE SIZE PARAMETERS AGAINST THE MEASURED VALUES

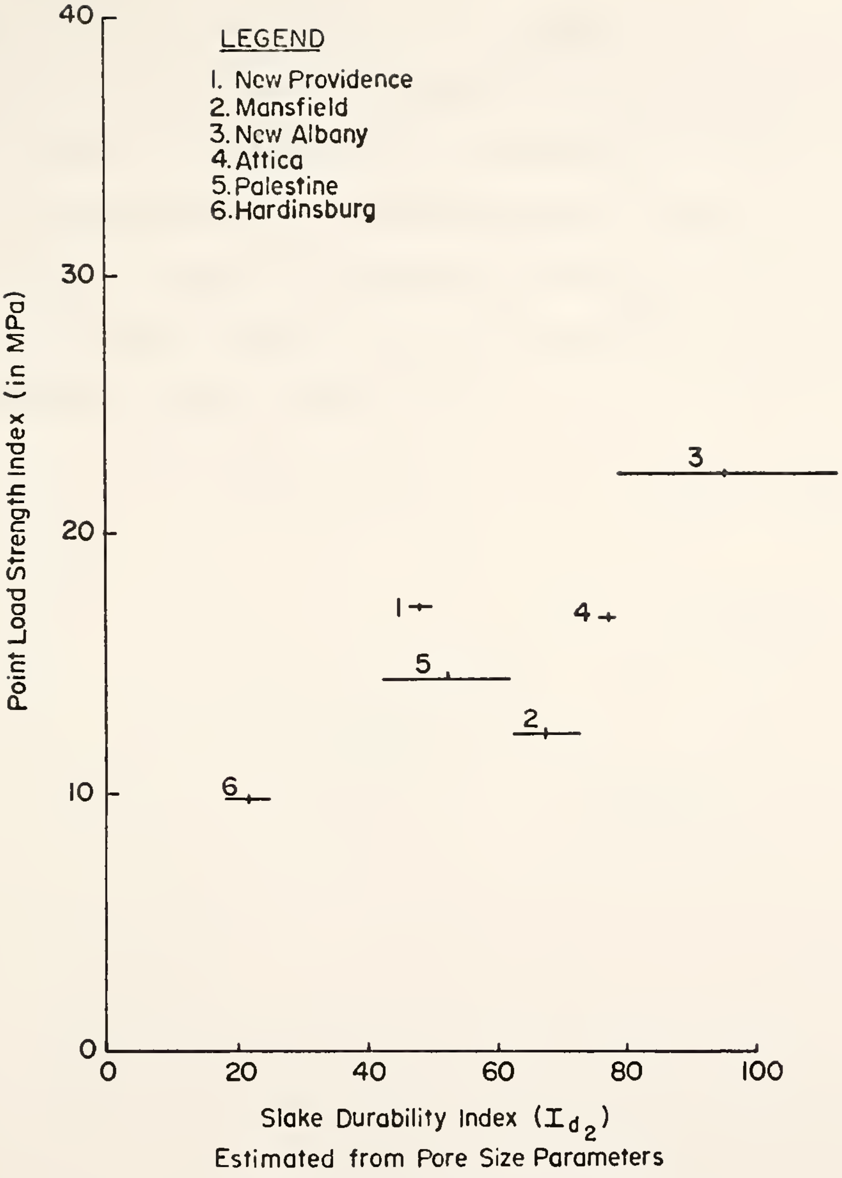
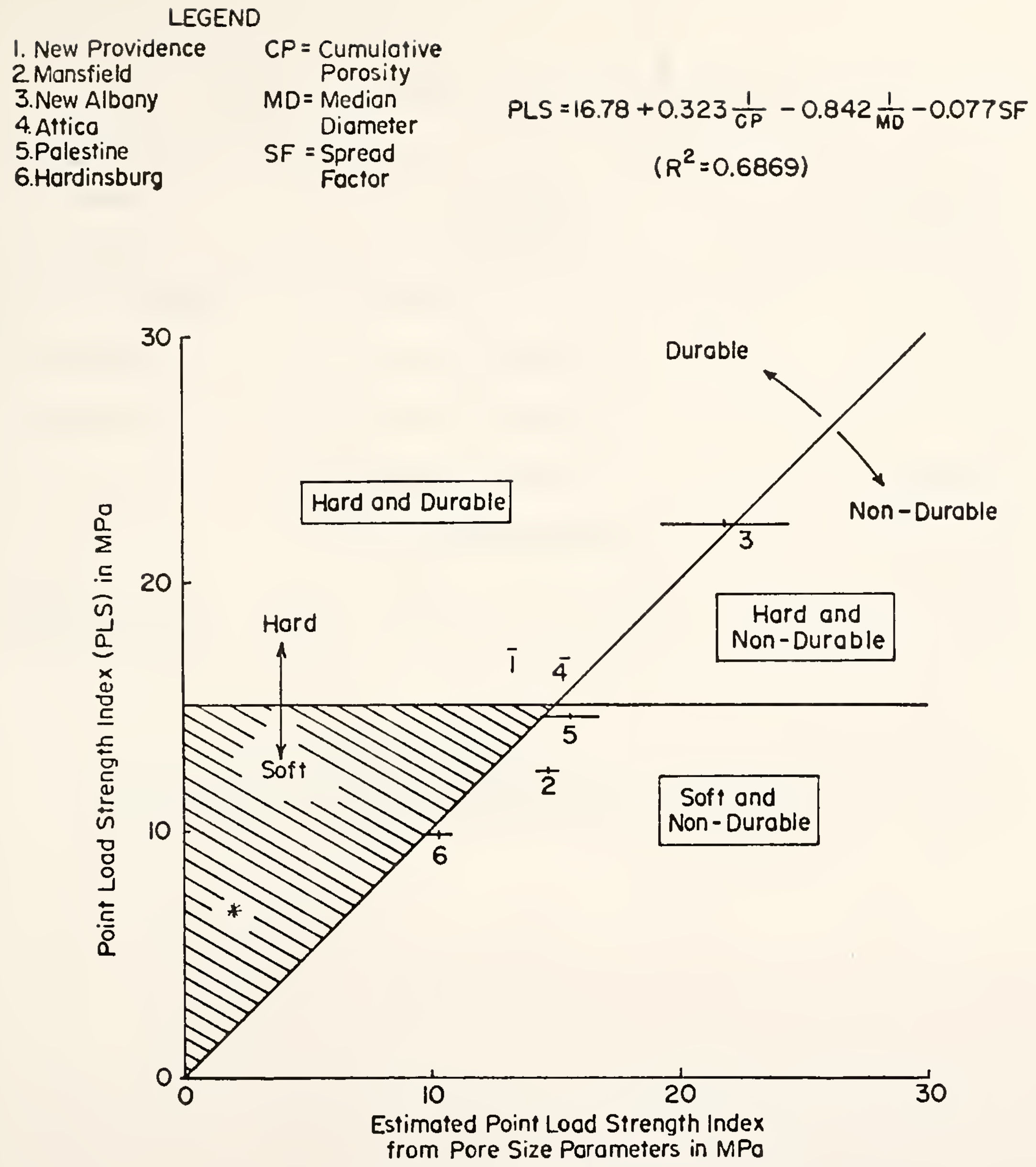


FIGURE C-6 PLOT OF ESTIMATED SLAKE DURABILITY INDEX FROM PORE SIZE PARAMETERS AGAINST POINT LOAD STRENGTH INDEX FOR SIX INDIANA SHALES

Point load strength indexes (corrected for size effects from Appendix D) are plotted against the measured values in Figure C-7. Durable shales plot above the diagonal line and non-durable shales fall below it except in the case of New Providence shale. The R^2 value is low at approximately 0.7. An arbitrary strength (indicated by the field behavior) can be selected to designate the shale as hard or soft. In this case a value of 15 MPa (2175.5 psi) is used. The results from this classification are given in Table C-3.



* Soft and Durable Shales are not Encountered

FIGURE C-7 PLOT OF ESTIMATED POINT LOAD STRENGTH INDEX FROM PORE SIZE PARAMETERS AGAINST MEASURED POINT LOAD STRENGTH INDEX

Table C-3 Classification of Test Shales

Shale	This Classification	Deo's Classification
New Providence*	Hard and Durable	soil-like
Mansfield	Soft and Non-Durable	soil-like
New Albany	Hard and Durable	rock-like
Attica	Hard and Durable	soil-like
Palestine	Soft and Non-Durable	soil-like
Lower Hardinsburg	Soft and Non-Durable	soil-like

*This is a non-durable shale as it has a Slake Durability Index from a 2 cycle test of less than 80.

APPENDIX D

POINT LOAD STRENGTH TEST

The point load strength test can be used for classification of shales along with other durability tests as mentioned in Appendix C. The point load strength of a given shale normal to the bedding depends on: (1) moisture content (Bailey, 1976), and (2) size of specimen (Broch and Franklin, 1972; Brook, 1977; Hale, 1979). Figure D-1 from Bailey (1976) gives the variation of point load strength with moisture content.

All the specimens tested in this study were oven dried to achieve a standardized initial condition even though overdrying induces hairline cracks in the specimens.

Correction for the sample size proposed by Broch and Franklin (1972) is given in Figure D-2. Point load strength corresponding to a core diameter of 50mm is taken as the representative value for the rock type. The 50mm standard was chosen as it lies within the range of cores obtained in the field. Results of point load test on four rock types (from Bieniawski, 1975) are plotted in this Figure and it can be concluded that each curve is representative of a rock type.

The point load test can be run on irregular lumps (of size D/L = 1.0 to 1.4) in the absence of core samples (Appendix B-3). This test (with irregular lumps) was used in this research, but in all cases D/L

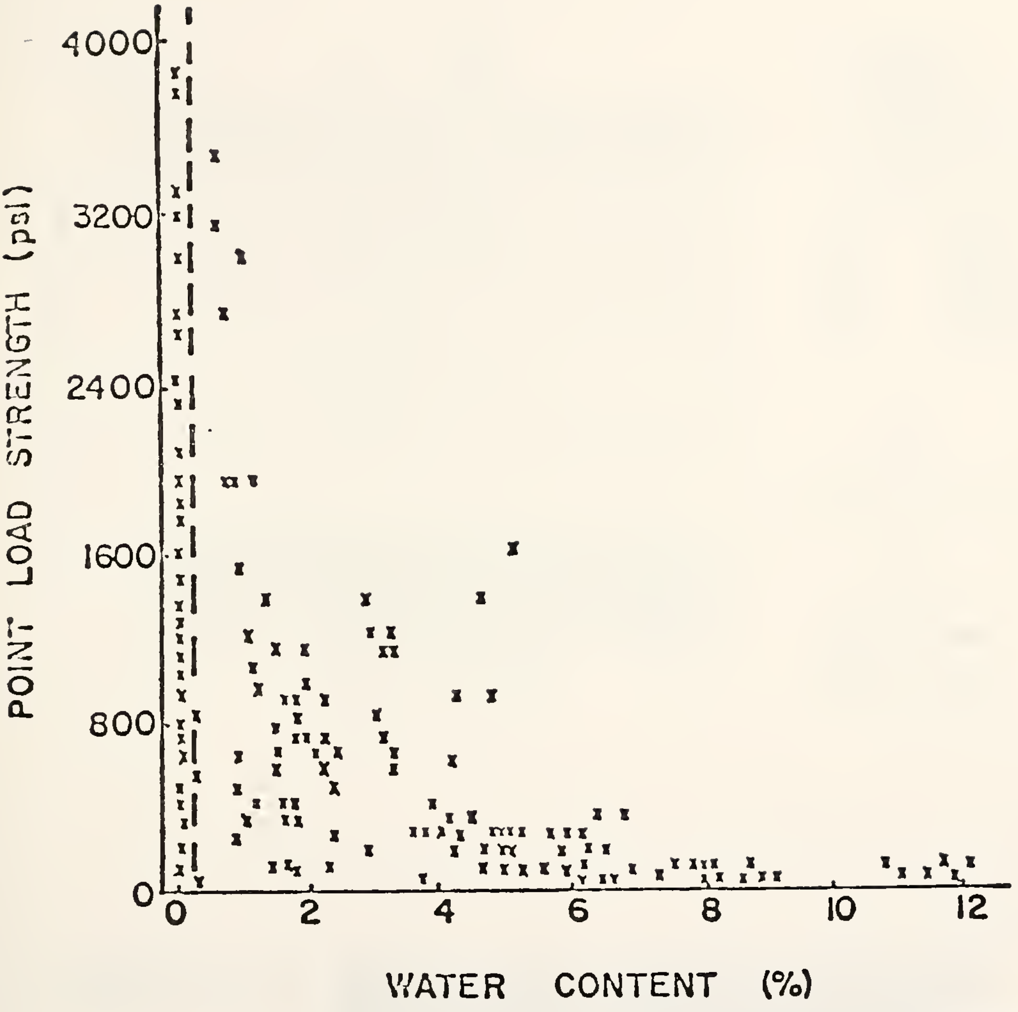


FIGURE D-1 VARIATION IN POINT LOAD STRENGTH WITH WATER CONTENT FOR ALL SHALES TESTED
(AFTER BAILEY, 1976)

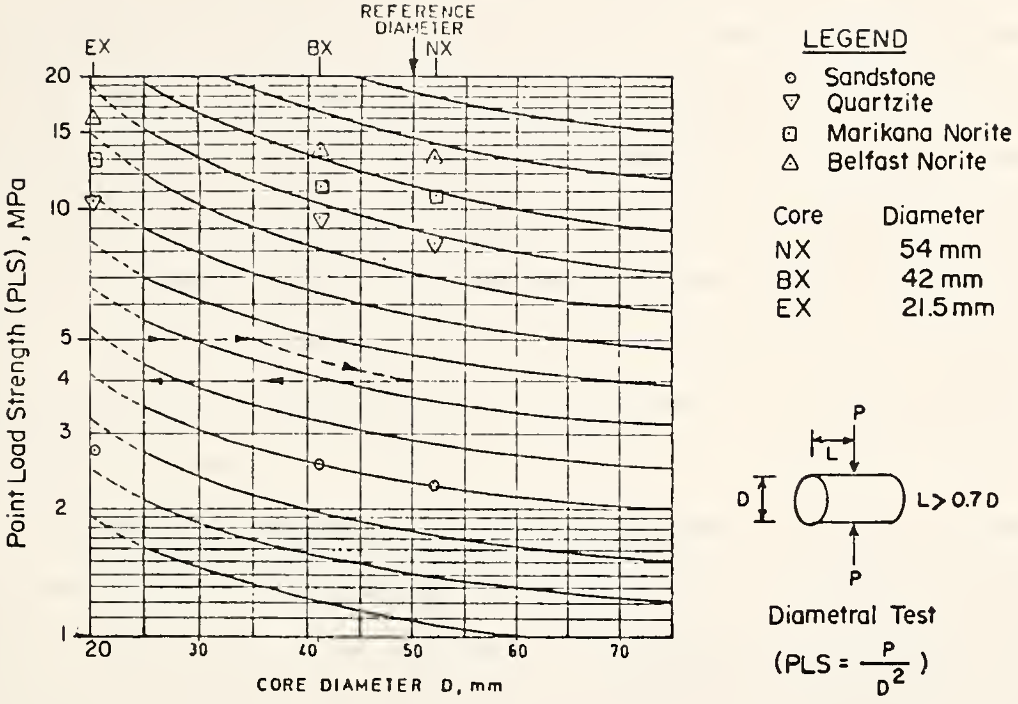


FIGURE D-2 SIZE CORRELATION CHART FOR POINT LOAD STRENGTH INDEX (AFTER BROCH AND FRANKLIN, 1972) was less than one due to the fissile nature of shale. It is proposed that the point load strength test be conducted in the laboratory, as all the other tests used in a classification system are laboratory tests.

Point load strength tests were conducted in the laboratory on six Indiana shales and a total of 56 samples were tested. The thickness of samples varied from 2.56mm (0.10 in) to 13.41mm (0.53 in) and the average was 6.59mm (0.26 in). Point load strength values for these samples are given in Table D-1 and in Figure D-3. Linear regression was performed by transforming D (the sample thickness) into $1/\sqrt{D}$ and a R^2 value of 0.6086 was obtained. The regression line is also plotted in Figure D-3 with the data points, and this line is the mean response of all shales tested.

As mentioned earlier, this relation depends on the type of shales and the regression line given in Figure D-3 is for all the shales. Hence, relationships were developed for different shales using linear regression and these are presented in Table D-2. The variation of point load strength with sample thickness for different shale types given by the statistical relationship is plotted in Figure D-4 along with the mean response of six shales. It is evident from this Figure that the variation is dependent on the shale type.

It is proposed that point load strength be evaluated at a sample thickness of 6.6mm (0.26 in) to be used in the classification system, viz., the one proposed in Appendix C. The point load strength evaluated in such a manner for the six shales studied is presented in Table D-3. It is recommended that similar relationships be developed for other shale types from laboratory test results and the corresponding value of point load strength at a sample thickness of 6.6mm be chosen for

Table D-1 Point Load Strength (PLS) for Different Sample Thickness (D)

Shale Type	PLS (MPa)	D (mm)
New Albany	24.73	4.60
	23.00	5.56
	23.29	5.56
Mansfield	14.26	8.03
	15.96	4.95
	7.56	7.52
	9.25	9.04
	6.49	11.70
New Providence	3.86	11.25
	31.64	4.04
	18.73	5.77
	16.53	7.52
	15.84	7.01
	17.18	6.88
	7.92	10.52
	15.60	6.45
Palestine	15.37	5.49
	2.89	9.68
	14.52	5.79
	11.03	7.39
	27.70	5.44
	13.66	8.56
	8.28	7.37
	15.94	6.35
	6.31	7.42
	10.93	9.53
	30.37	4.65
	16.02	5.06
Attica	10.08	9.32
	14.30	6.81
	25.02	4.83
	24.00	5.92
	20.08	5.28
	24.45	4.83
	13.68	8.46
	15.52	7.16
	26.30	3.63
	22.42	5.16
	8.80	8.38
	17.76	6.38
	17.22	7.37
Lower Hardinsburg	10.79	5.11
	2.26	13.41
	9.36	6.30
	2.94	10.97
	3.85	11.53
	5.90	7.65
	4.07	7.49
	5.17	7.21
	7.49	6.63
	18.2	3.63
	5.73	6.48
	21.18	4.67
	3.74	8.59
	10.44	5.28
	8.78	2.57

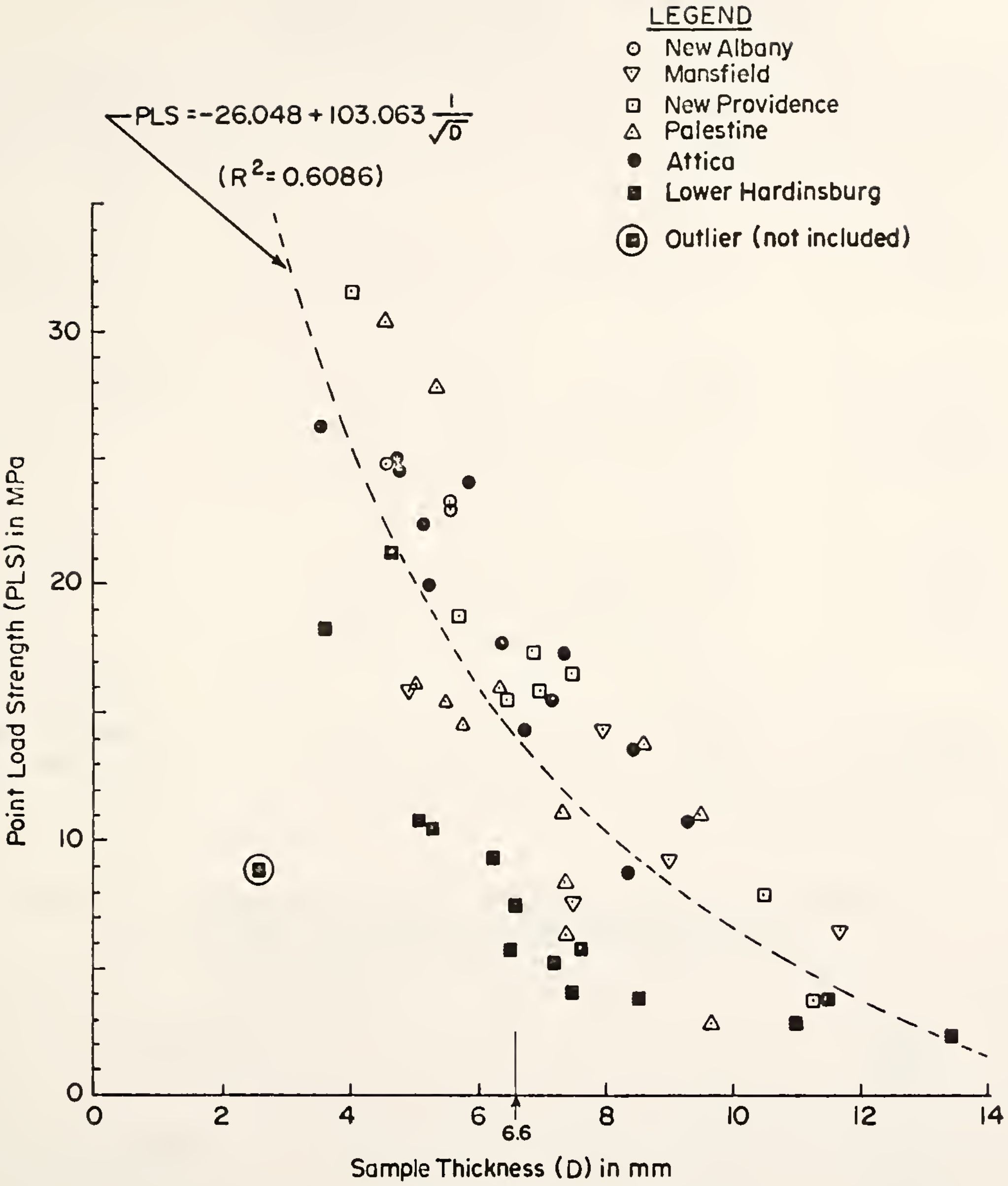


FIGURE D-3 POINT LOAD STRENGTH FOR DIFFERENT SAMPLE THICKNESS OF SIX INDIANA SHALES

Table D-2 Statistical Relationship of Point Load Strength (PLS) and Sample Thickness (D) for Different Shale

Shale Type	Statistical Relationship	R^2
New Albany	$PLS = 11.27 + 28.07 \frac{1}{\sqrt{D}}$	0.8505
Mansfield	$PLS = -9.52 + 56.41 \frac{1}{\sqrt{D}}$	0.6082
New Providence	$PLS = -32.56 + 127.66 \frac{1}{\sqrt{D}}$	0.9573
Attica	$PLS = -19.55 + 93.88 \frac{1}{\sqrt{D}}$	0.8238
Palestine	$PLS = -36.87 + 131.86 \frac{1}{\sqrt{D}}$	0.6203
Lower Hardinsburg	$PLS = -19.84 + 72.62 \frac{1}{\sqrt{D}}$	0.7795
Mean of above six shales	$PLS = -26.05 + 103.06 \frac{1}{\sqrt{D}}$	0.6086

Table D-3 Point Load Strength (PLS) Evaluated at a Sample Thickness of 6.6mm Using the Statistical Relationships

Shale Type	PLS (MPa)
New Albany	22.30
Mansfield	12.40
New Providence	17.25
Attica	16.80
Palestine	14.50
Lower Hardinsburg	8.30

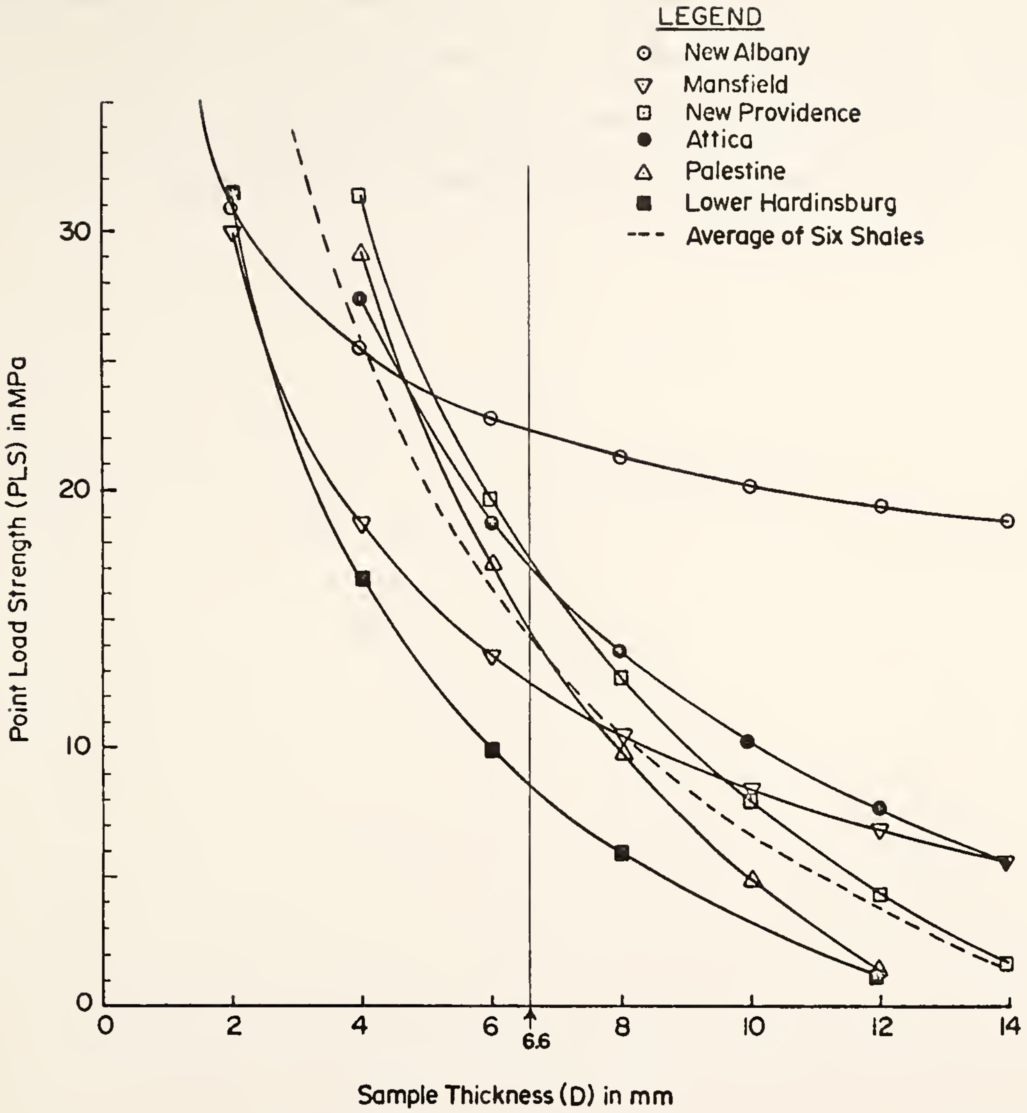


FIGURE D-4 REGRESSION LINES REPRESENTING VARIATION OF POINT LOAD STRENGTH WITH SAMPLE THICKNESS FOR DIFFERENT INDIANA SHALES classification. In the absence of a relationship for a specific shale type, the relationship for the mean can be used for preliminary evaluation by tracing a path parallel to the mean curve and evaluating the strength that corresponds to a sample thickness of 6.6mm (0.26 in).

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