FINAL REPORT

For the Florida Department of Transportation

PB99-133993

An Evaluation of Engineering Properties of Problematic Soils in Highway Construction Phase I Study

Research Report No. FL/DOT/RMC/0702 (1)-9192

State Job No.: 99700-7606-119

WPI No.: 0510702

FSU Project No.: 6120-518-39

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16. Abstract

This study presents the research effort to evaluate problematic A-2-4 soils in highway construction. Typical problems encountered by contractors in Florida during construction are that the soils retain excess moisture or are difficult to dry and compact. One prevalent theory was that the contruction problems were directly related to the presence of expansive clays in the soils. An experimental program was conducted to evaluate six problem soils. The soils were chosen by the Florida Department of Transportation (FDOT) because of past problems experienced during construction. A set of laboratory tests was performed to determine the engineering properties of each soil. A review of the current understanding of expansive soils in highway design was also conducted. The results of the experimental program showed that certain engineering properties give a possible indicator of potential construction problems and can be used to screen A-2-4 soils before use in highway projects. The results also showed that the presence of expansive clay is not a conclusive indicator of a problem A-2-4 soil.

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METRIC CONVERSIONS

inches = 25.4 millimeters

feet = 0.305 meters

square inches = 645.1 millimeters squared

square feet = 0.093 meters squared

cubic feet = 0.028 meters cubed

pounds = 0.454 kilograms

poundforce = 4.45 newtons

poundforce per square inch = 6.89 kilopascals

pound per cubic inch = 16.02 kilograms per meters cubed

DISCLAIMER

"The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Department of Transportation or the U.S. Department of Transportation. This publication is prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation."

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Highway contractors in the State of Florida occasionally encounter soils which are troublesome or problematic during construction. Common problems include excessive expansion, hard to compact, and slow drying. The Florida Department of Transportation (FDOT) has attempted to avoid such problems by limiting the type of soils which can be used in roadway embankment.

FDOT Design Standards (FDOT, 1992) limit 'select' soil materials for embankment to A-1, A-3, and A-2-4 based on the AASHTO Classification System. A-1 soils consist mostly of gravel and sand size particles and some percentage of silt. A-3 soils are more of a fine sand and small amount of silt. For both soils, the amount of clay is very low. A-1 and A-3 are relatively problem free when used in embankments.

A-2-4 soils are silty or clayey gravel and sand. They may have a maximum of thirty-five (35) percent passing the No. 200 sieve and a maximum plasticity index of ten (10). Some types of A-2-4 soils can contain significant amounts of clay and, thus, can experience problems during construction. The design standards even note that certain types can "retain excess moisture and may be difficult to dry and compact."[1] The standards recommend that such soils be used in the embankment above the existing water level at the time of construction. However, the standards do not outline any method to identify or screen out these certain types of soils.

1.2 Scope of Study

Guidelines or specifications to identify and screen out potentially problematic A-2-4 soils before their use during construction would be very beneficial. Such a system would reap benefits from both a time and financial standpoint to the FDOT and contractors. A study has been undertaken in cooperation with the FDOT to address this issue.

The purpose of this study was to evaluate the engineering properties of potentially problematic soils. The evaluated soils for this study were selected by the FDOT in conjunction with the researchers. Six different soils from around the State of Florida were evaluated.

The primary goal was to perform an array of tests on each of the soils. The tests performed included: Atterberg limits, grain-size analysis, compaction test, bearing ratio,

[1] Florida Department of Transportation (1992). "Roadway and Traffic Design Standards." Topic No. 625-010-003-6, Item No. 505. Tallahassee, FL.

expansion index, permeability, x-ray diffraction, and scanning electron microscope. The results of these tests would possibly allow the following two objectives to be evaluated: (1) Determine if expansive clay exists in enough quantity in the soil to be the major factor in the construction problems, (2) Provide a method to identify problematic soils based on their engineering properties.

The second purpose of this study was to provide a review of the current understanding of expansive soils in highway design. A review of the mechanisms and influencing factors of volume change in expansive soils was conducted. The current AASHTO specifications which account for swelling from expansive soils in both flexible and rigid pavement design were also reviewed along with several other maintenance and construction procedures which can reduce swelling effects.

1.3 Report Organization

This report summarizes the research conducted on expansive and problematic soils in Florida highway construction. The results from a laboratory study to determine the engineering properties of problematic soils are presented in detail in the report. A review of the effects of expansive soil in highway design along with design methods to compensate for these effects are also provided in Appendix A.

Chapter 1 presents the purpose and objectives of this research and experimental study. A literature review of methods to identify expansive soils and swell potential is presented in Chapter 2. The collection and evaluation program of problematic soils is outlined along with the test procedures in Chapter 3. Results from the evaluation program are provided in Chapter 4. The conclusions and recommendations based on this research are then presented in Chapter 5.

Concurrently with the evaluation program, a separate literature review was undertaken as to the causes of volume change in expansive soils along with current pavement design and preventive methods. An in-depth review of this subject is presented in Appendix A.

CHAPTER 2

LITERATURE REVIEW

Early identification of problematic and/or expansive soils before their use in highway projects can save considerable time and money. Experience has shown that certain soils that fit the AASHTO classification for A-2-4 materials can experience construction problems (slow drying or hard to compact) when used. Some of this experience came at a costly price tag to the FDOT when problem soils were encountered.

Unfortunately, because of the fast paced nature of the highway construction business, almost no follow-up research has been conducted on encountered problem soils. Problem soils are bypassed, replaced, or treated so that construction can continue without delay. No central testing program or identification data base exists at the State level, and possibly only in the mind of the District Geotechnical Engineer at the local level.

2.1 Background

A study was conducted in the early 1970s (University of

Florida, 1972) for the FDOT on the engineering properties of A-2-4 soils. The purpose of the study was to establish: (1) test characteristics that would alert soils and materials engineers to a possible troublesome soil; (2) procedure to define with certainty those soils that could be expected to be difficult to handle; and (3) suggestions for possible construction procedure to permit utilization of the troublesome A-2-4 materials within the roadbed or elsewhere within the rights-of-way.

The study consisted of analyzing six A-2-4 soils, three with a good history and three with a poor history of construction performance. The soils were selected by FDOT engineers and the testing performed at the University in Gainesville. The testing program included grain-size analysis, Atterberg limits, compaction, permeability, specific gravity, and X-ray diffraction.

The study recommended that all soils identified as A-2-4 which are greasy and sticky to the touch and have natural moisture contents much greater than the optimum moisture content be further tested to establish permeability values. Soils with permeability values down to 10^{-4} cm/sec ($3x10^{-5}$ ft/sec) can be considered as non-problematic. If the permeability is lower then 10^{-4} cm/sec, X-ray diffraction should be conducted. If the soil then shows the presence of montmorillonite and/or attapulgite clay particles, then the soil can be considered to be problematic. Unfortunately, probably due to the availability and cost of these tests at the time of the report, these recommendations were never implemented.

The current research project is a revival and extension of the aforementioned study by UF. A greater number of soils were tested and a few additional engineering and material properties measured (Limerock Bearing Ratio, Expansion Index, SEM). The results of this program will provide further insight in dealing with the problematic soils.

From the UF report, it appears that some relationship between the existence of expansive clays in the soil and construction problems may exist. The amount of expansive clay will also dictate if the soil is just problematic or expansive since actual swelling is a function of quantity as well as other factors described in Appendix A. The mere presence of expansive clay does not make a soil expansive. Some threshold percentage of expansive clay in the soil must exist in order for the soil to have a high swelling potential.

2.2 Location of Expansive Soils in Florida

The first step in identifying potentially problematic soils is to identify the location of expansive soils and soils with expansive clays. The U.S. Department of Transportation developed occurrence and distribution maps of potentially expansive soils for the entire United States (Federal Highway

Administration, 1979). The maps categorize soils in terms of volume change and expected frequency of occurrence. The four categories used were high, medium, low, and nonexpansive.

Most of Florida is classified nonexpansive due to the prevalence of limestone and marl. Some argillaceous materials do occur locally, and higher classification is required. Figure 2.1 shows a blow-up of the distribution map for the State of Florida. The Ocala formation extends along the northern part of the state with a finger extending down to the city of Ocala. This soil is classified as low expansion potential. The groups of soil around the Tampa area and along part of the west coast (Tampa Group) are classified as medium. The medium expansive material located in south Florida is due to the fine-grained materials occurring in the swamps and bogs.

The identification process can be further narrowed or refined by consulting other sources. U.S. and State Geological Survey maps complemented with information from the U.S. Department of Agriculture (USDA) soil surveys are often good indicators. Survey maps are available on a scale that is more workable for road and bridge projects. Other sources that are available include USDA Soil Conservation Service (SCS) County Soil Surveys or U.S. and State Geological Survey Natural Hazard Maps. One last source which is available, but often harder to locate, is from local contractors and engineers.

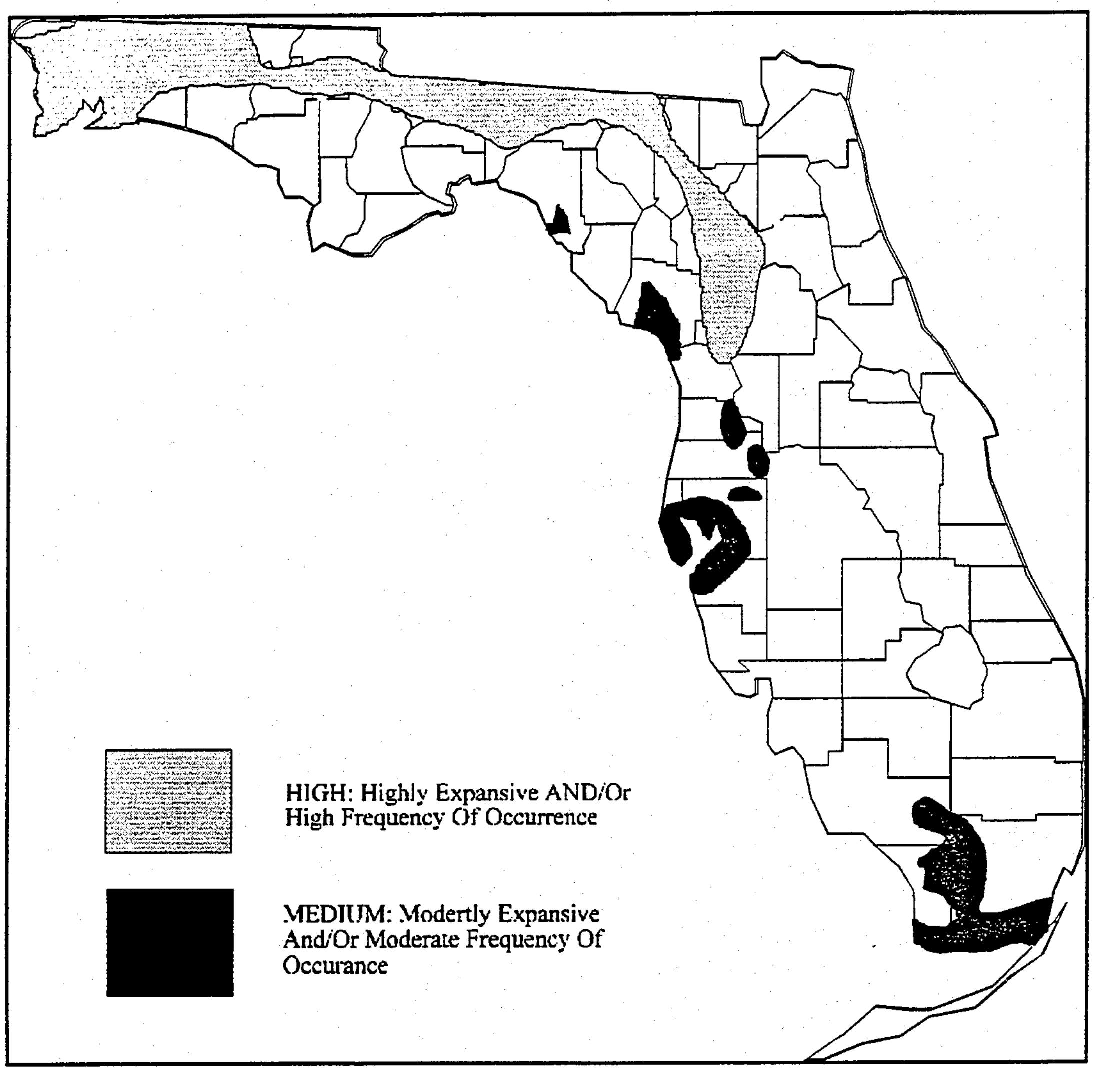


Figure 2.1: Location Map of Expansive Soils in Florida

Soils which are suspected to be expansive or problematic from any of the above sources should be subjected to field exploration and sampling. Extra borings should be taken, and depth of boring be at least 1.5 meters (5 feet) below the active zone. For cut sections, the borings should extend at a minimum 1.5 meters below the final grade. The borings should also extend 1.5 meters below the foundation surface on shallow fills.

Once soil samples have been collected, they must be tested and their swell potential estimated. Many methods, both empirical and theoretical, have been proposed over the years to identify these values. The next section will review a few of the methods presented.

2.3 Classification Methods

Numerous methods to identify or classify expansive soils have been proposed over the years. Their accuracy varies, often by geographical location. A method providing good results in one area may perform poorly in another due to different soil characteristics. A few of the more popular methods are presented for review.

2.3.1 Activity Method

A. W. Skempton proposed a method for identification based on the colloidal activity (Skempton, 1953). Skempton defined activity (A_c) by Equation 2.1.

$$A_c = \frac{P.I.}{C} \tag{2.1}$$

P.I. = Plasticity Index
C = Percent clay particles smaller than 0.002mm

A clay or soil can then be classified according to its activity based on the criteria shown in Table 2.1.

Table 2.1: Classification of Clay by Activity

Activity	Classification
< 0.75	Inactive Clay
0.75-1.25	Normal Clay
> 1.25	Active Clay

The activity for each clay type is constant. Montmorillonite is defined as active, Illite as normal, and Kaolinite as inactive. The proportion of shear strength due to cohesion increases as activity increases. In the same manner, the bearing capacity increases in the same proportion on the condition that the clay does not swell. This method is generally a good indicator of expansive soils though it is inapplicable to some soils such as Israeli and Australian clays (Kassiff and Holland, 1966).

2.3.2 U.S.B.R. Classification

The U.S. Bureau of Reclamation (U.S.B.R.) Classification method (Holtz and Gibbs, 1957) defines the degree of swell of a soil in relationship to the percentage swell of an air-dry sample in the oedometer under a vertical pressure of 6.895 kN/m^2 (1.0 psi). Four soil properties are used to classify the soil: shrinkage limit, plasticity index, percent smaller than 1μ, and percent free swell.

The percent free swell test is performed by slowly pouring 10 cc of air-dried soil passing the No. 40 sieve into a 100 cc graduate filled with water. The free swell (F.S.) is then calculated by noting the swelled volume of the soil at the bottom of the graduate and Equation 2.2.

$$F.S. = \frac{V - V_0}{V_0} \, x \, 100\% \qquad (2.2)$$

V = soil volume after swelling (cc)
V_0 = soil volume before swelling (10cc)

The soil is then classified according to the degree of swell using Table 2.2. Soils with free swell above 100% are considered to have high swelling potential. There is a lot of overlap in the categories as can be seen in Figure 2.2.

2.3.3 Seed's Method

The next classification system is Seed's method (Seed et al., 1962). The method classifies clayey soil according to its swelling potential which is defined as the percent vertical swell under a pressure of 6.895 kN/m^2 (1.0 psi) for a sample compacted at optimum moisture content and maximum dry density in a consolidometer.

Based on experiments, Seed et al. proposed the relationship in Equation 2.3 for the swelling potential (S). The researchers were also able to determine that the factor K was a function of the activity only.

$$S = K\,C^{x} \qquad (2.3)$$

C = percentage of colloids smaller than 2μ
x = constant depending on the clay type
K = factor calculated from the intercept of the straight line relating log S to log C and dependant on the type of clay minerals

Using empirical relationships developed for different clay minerals and Eq. 2.1, Seed et al. concluded that the swelling potential of clay could be established from the plasticity index only. Table 2.3 shows the proposed

Table 2.2: U.S.B.R. Classification Method

Degree of Swell	Swell in oedometer under a pressure of 1.0 psi, %	S.L. %	P.I. %	Percent smaller than 1μ	F.S. %
Very high	>30	<10	>32	>27	>100
High	20-30	6-12	23-45	18-37	>100
Medium	10-20	8-18	12-34	12-27	50-100
Low	<10	>13	<20	<17	<50

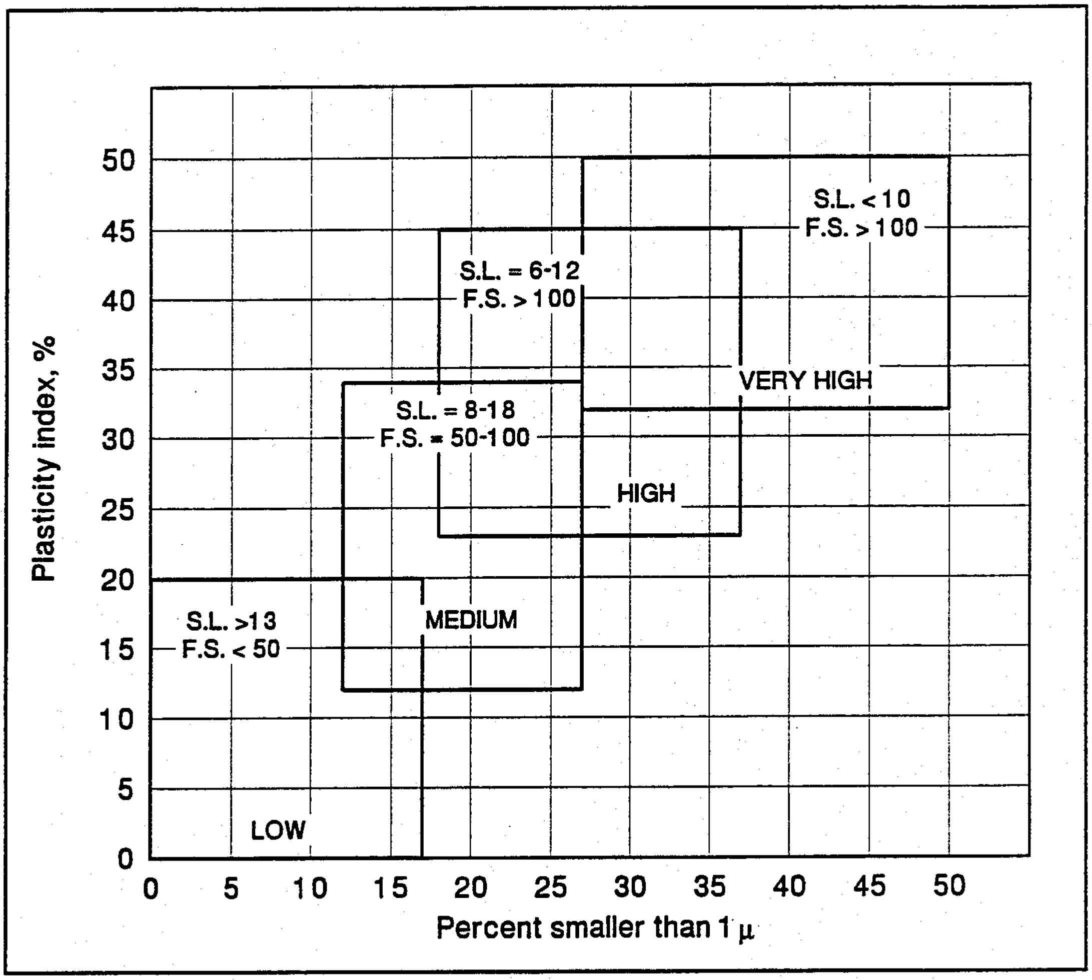


Figure 2.2: Overlay of U.S.B.R. classification categories

relationship between the plasticity index and swelling potential. Table 2.4 shows the U.S.B.R. categories for swell as a function of swelling potential as proposed by Seed et al. The clay soils tested in the study were artificial in that they were prepared in the laboratory by mixing percentages of different clay minerals. There is no certainty that normally occurring clay soils will behave in the same manner.

2.3.4 Lambe's Method

T.W. Lambe proposed the use of a swell index to classify swelling clays in subgrades (Lambe, 1960). Using a specially designed apparatus, the swelling pressure developed in a compacted sample within two hours of soaking is measured. The measured swelling pressure in lb/ft^2 is the swell index for the soil.

Lambe also developed a grading method called the P.V.C. (potential volume change) to classify the soil. The P.V.C. is obtained by using Figure 2.3 and the measured swell index. With the P.V.C., Table 2.5 is then used to classify the soil.

The big advantage of Lambe's method and apparatus was speed of test performance and the possibility of standardization so as to compare results. The big disadvantage associated with the method was technical difficulties encountered when using the equipment to measure swell pressure.

Table 2.3: Relationship Between P.I. and Swelling Potential

P.I., %	Swelling Potential, %
10	0.4 - 1.5
20	2.2 - 3.8
30	5.7 - 12.2
40	11.8 - 25.0
50	20.1 - 42.6

Table 2.4: Seed's Classification Method for Clayey Soil

Degree of Swell	Swelling potential, %
Low	0 - 1.5
Medium	1.5 - 5.0
High	5 - 25
Very High	25

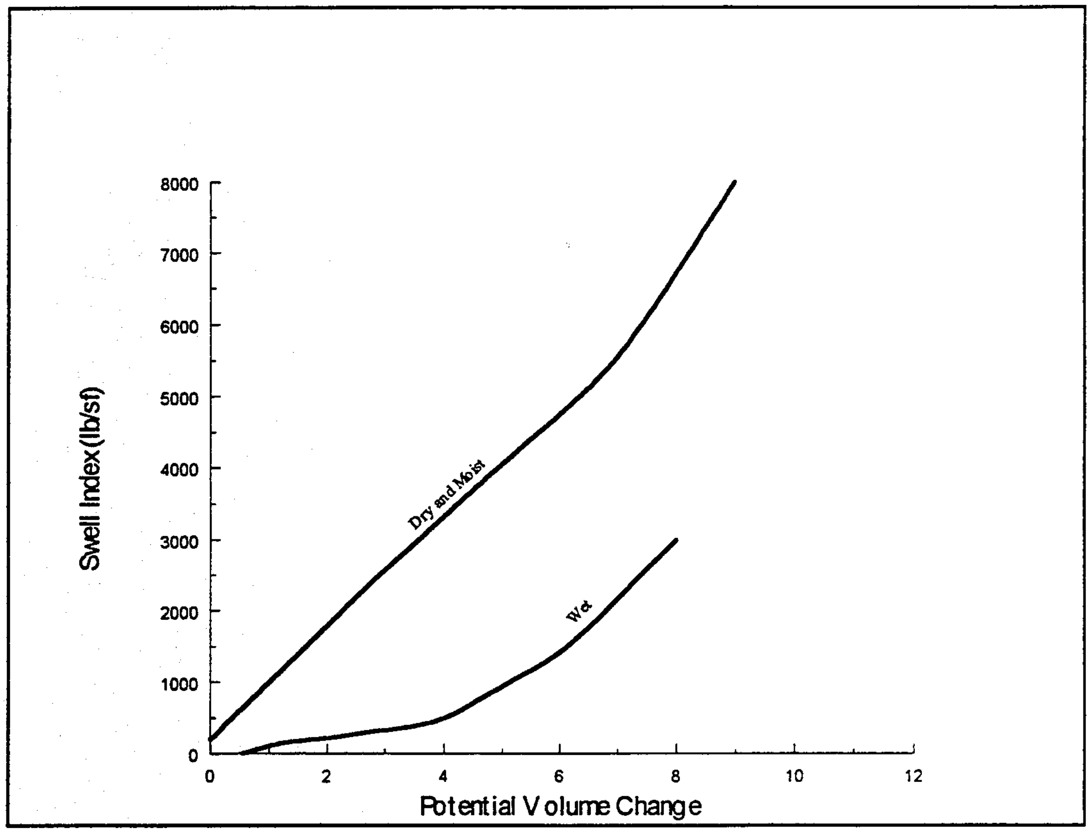


Figure 2.3: Potential Volume Change Versus Swell Index (Lambe, 1960)

Table 2.5: P.V.C. Grading Classification Method

P.V.C. Grading	Clay Category
<2	Noncritical
2-4	Marginal
4-6	Critical
>6	Very Critical

2.3.5 WES Method

The U.S. Army Engineer Waterways Experiment Station (WES) conducted a four (4) year study of expansive clays in highway subgrades. A result of the study was the development of an expedient methodology for identification of potentially expansive soils (Federal Highway Administration, 1979). The WES classification system of potential swell is shown in Table 2.6.

Three soil properties are required for the WES method: liquid limit, plasticity index, and natural soil suction (τ_{nat}). The soil suction is the force exerted by a soil mass responsible for soil-water retention. More simply, it is the pulling force exerted on soil-water by the soil mass. The most convenient way of measuring the soil suction is with thermocouple psychrometers.

The Federal Highway Administration made recommendations

Table 2.6: WES Classification of Potential Swell

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

1 tsf = 95.76 kN/m^2

for using the WES classification system with pavement design. Soils exhibiting a low classification of potential swell can be used with confidence that expansion problems will be minimal. Soils with a marginal classification should be further sampled, tested, and classified at the specific site. High potential swell soils should also be tested further and estimates of anticipated volume change made.

2.3.6 Expansion Index Method

More recently in 1988, the American Society for Testing and Materials issued the test method for determining the Expansion Index of soils. The method provides an index to the swell potential of compacted soils when inundated with distilled water (Anderson and Lade, 1981).

The test method, which is described in detail in Section 3.3.6, standardizes the classification of swell potential in soils in a very simple experiment. The soil is compacted into a metal ring at a degree of saturation between 40 and 60% and then placed in a consolidometer. After applying a vertical confining pressure of 6.895 kN/m^2 (1 lbf/in^2), the specimen is inundated with distilled water. The deformation of the sample is then measured over a twenty-four (24) hour period.

The Expansion Index is the ratio of deformation to original specimen height multiplied by 1000. The classification of the soil's expansion potential is based on Table 2.7. This classification system was used in the research study on problematic soils documented in this study.

Table 2.7: Classification System of Expansion Potential of a Soil

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

2.3.7 Summary

Each of the methods presented are simple and based on easily measurable parameters. The most common parameters used are plasticity index, liquid limit, percent of clay particles, and free swell. Use of mineralogical, chemical, and pedological properties of the soil are rarely used in a rational approach to classify the soils. Therefore, no one classification method presented satisfies all situations. The use of the plasticity index and liquid limit alone should not be used to identify problem soils because it can be misleading. It should always be supplemented with other tests such as free swell at a minimum.

CHAPTER 3

SOIL EVALUATION

A test program was initiated to evaluate troublesome or problematic soils. The soils were selected by Florida DOT personnel based on past or present construction problems. The purpose of the program was to test and evaluate the engineering properties of the soils and possibly determine indicators for future identification of problem soils.

3.1 Experimental Program

Six problem soils were evaluated in the test program. Two soils each from FDOT District 2, 3, and 5 were used. The State FDOT Geotechnical Engineer at each district selected two soils from either past or present FDOT projects. The criteria for selection was that the soils be A-2-4 and have experienced problems during construction.

The engineering properties of each soil were evaluated during the program. The following tests were performed on each soil:

1. Moisture Content
2. Liquid Limit
3. Plastic Limit
4. Grain Size Analysis (Sieve and Hydrometer)

5. AASHTO Classification
6. Compaction Test
7. Expansion Index
8. X-Ray Diffraction
9. Scanning Electron Microscope
10. Permeability
11. Lime Rock Bearing Ratio (2 and 4 day soaking)

Each test was performed three times, except for the Limerock Bearing Ratio (LBR) test which was done twice and the permeability which was done two to three times.

The procedure used for each test conforms to the American Society for Testing and Materials (ASTM) Standards and/or Manual of Florida Sampling and Testing Methods as appropriate. The method used and the appropriate reference for each test are listed in Section 3.3.

3.2 Soil Sample Collection

Each FDOT District involved in the study was requested to identify two problem soils for evaluation. Preliminary testing of each soil was conducted at the individual District Soils Laboratory. The results were reviewed and soils either accepted or rejected by project personnel. If accepted, the soils were collected and shipped by District personnel to the laboratory in Tallahassee. Soils arrived between February and September 1995.

With the exception of the District 5 soils which arrived in soil bags, all soils were shipped in sealed containers.

The soils were stored in 55 gallon sealed containers at an environmentally controlled building. Each container was marked with the soil's name which was based on its location of origin.

Table 3.1 lists each of the soils selected for evaluation. The name used to identify each soil during the testing program is shown in bold. Also included is the AASHTO classification, soil description, and soil color. A site location map is provided in Figure 3.1.

As can be seen from the Table 3.1, all soils except the Clay County soil are A-2-4. The preliminary testing of this soil had classified it as A-2-4. During the evaluation

Table 3.1: Soils Selected for Evaluation Program

District	Location	AASHTO Class	Description	Color
2	**Clay County** SR 100 and C-21B	A-2-6	Clayey Sand	Orange
2	**Madison County** SR 14	A-2-4	Silty Sand	Brown
3	Jackson County US 231, **Alford City**	A-2-4	Silty Sand	Brown
3	Jackson County US 231, **Jacobs Road**	A-2-4	Silty Sand	Reddish Brown
5	**Brevard County** SR 520	A-2-4	Silty Sand	Black
5	**Marion County** U.S. I-75	A-2-4	Clayey Sand	Dark Brown

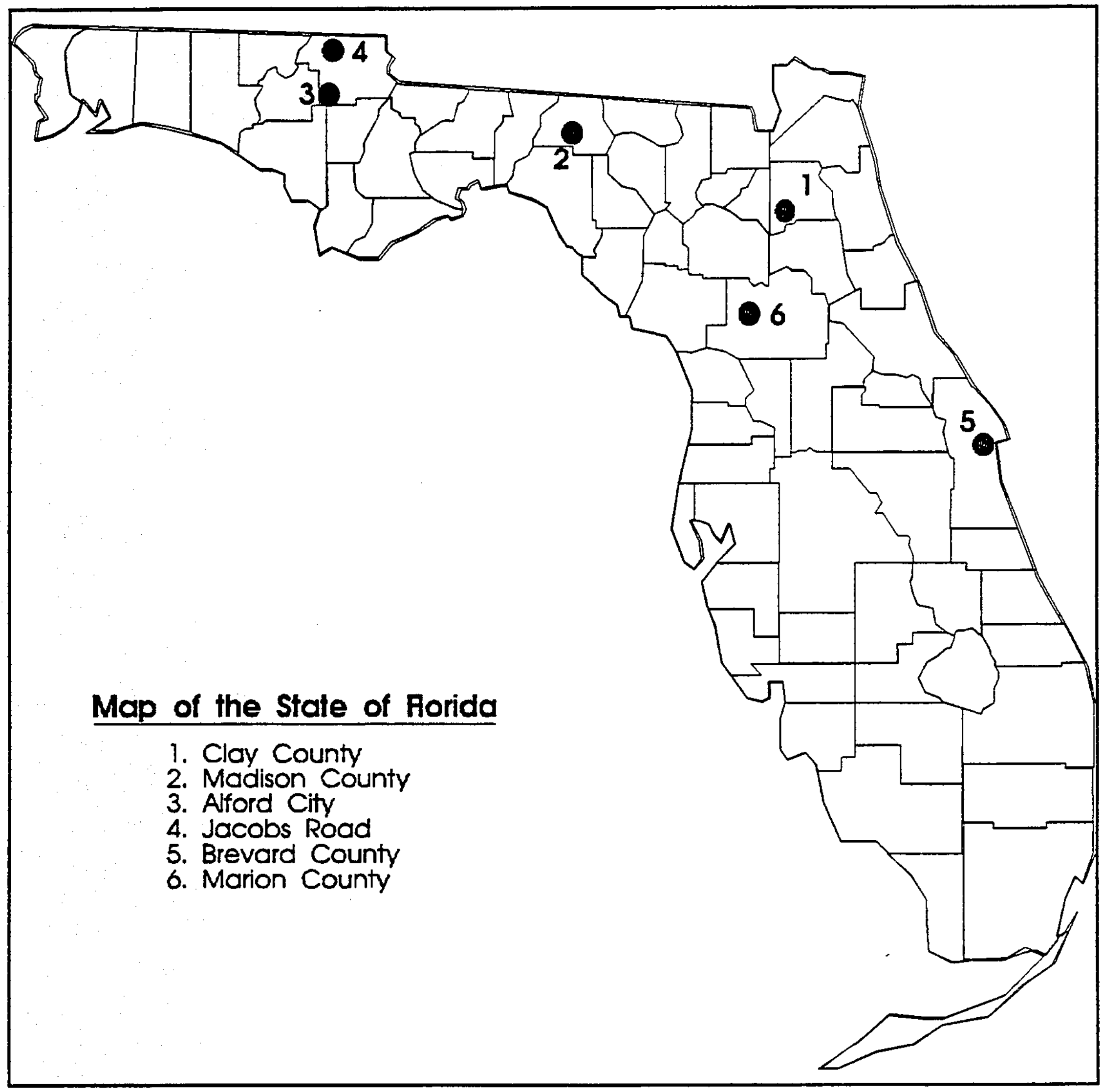


Figure 3.1: Site Location Map of Evaluated Soils

program, it was found that the soil that had actually been collected after the preliminary testing was an A-2-6. The location from which the soil had been collected had a history of the soil type varying greatly within a short distance. It is most likely that the soil collected was a short distance from the location where the preliminary sample had been taken. This soil was kept in the evaluation program despite its classification since it had been identified as a problematic soil by the FDOT engineers.

3.3 Test Procedures

3.3.1 Moisture Content

The first soil property evaluated upon arrival at the laboratory was the moisture or water content. Three soil samples from varying depths within the container were obtained and used for this test. Metal tins were used to dry the soil in a drying oven at 110° C (230° F) for twenty-four hours. The procedure and calculations used conformed to the Standard Method ASTM D 2216.

3.3.2 Atterberg Limits

The Atterberg limits in current geotechnical engineering refer to the liquid limit (LL), plastic limit (PL), shrinkage limit (SL), and plasticity index (PI). The limits represent the transition points between the four basic behavior states of soil. These states, which are moisture content dependant, are solid, semi-solid, plastic, and viscous liquid. Figure 3.2 graphically shows the relationship between each of the limits. The plastic index is the arithmetic difference between the LL and PL.

The liquid limit, plastic limit, and plasticity index were determined for the soils in this test program. These properties were used in the classification of the soil and gave an indication of their engineering properties. The procedure followed for determining the LL was ASTM D 423 and for PL and PI was ASTM D 424.

3.3.3 Grain Size Analysis

A grain-size analysis is the determination of the size range of particles present in a soil expressed as a percentage of the total dry weight. The distribution of grain sizes or gradation is used for classification purposes and an indication of other engineering properties.

The two generally used methods to determine particle size distribution in soil are: (1) Sieve analysis for particle sizes larger than 0.075 mm (.003 in), and (2) Hydrometer analysis for particles smaller than 0.075 mm. Both methods were used in the test program. A combination of ASTM D 421

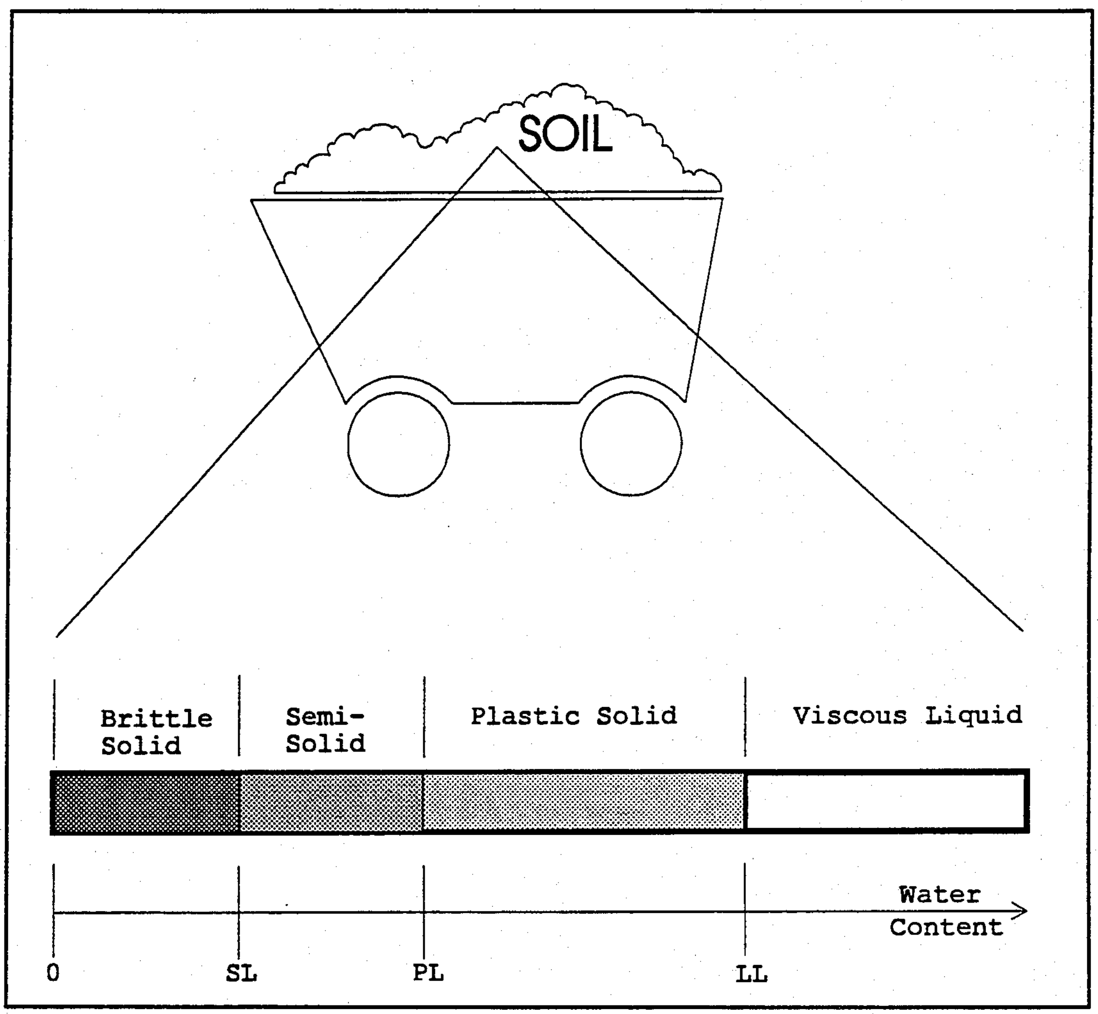


Figure 3.2: Graphic Representation of the Atterberg Limits

and ASTM D 1140 Standard Methods were used.

For the sieve analysis, a dried soil sample of known weight was soaked in water and then poured through a No. 40 and No. 200 sieve. The soil in the sieves was then washed until the rinse water was clear. The remaining soil in the sieves was then dried and poured through the eight (8) nested sieves of the sizes shown in Table 3.2 and placed in a mechanical shaker for ten minutes. The percent passing was then calculated and plotted on semi-log paper versus the particle diameter.

Table 3.2: Sieve Sizes used for Grain-Size Analysis

Sieve Number	Diameter(mm)
4	4.750
10	2.000
25	0.710
40	0.425
60	0.250
100	0.150
140	0.106
200	0.075

1 mm = 0.0394 inch

Hydrometer analysis is used to determine the distribution of soil particle sizes for clays and silts. For this project, it is particularly useful in identifying the percentage of clay in each soil. The analysis is based on the principle that soils dispersed in water will settle at different velocities. The velocity of the particle depends on shape, size, weight, and viscosity of the water. The diameter of a particle and the corresponding percent passing can be determined for a range of particle sizes by assuming that the soil particles are spheres and the particle velocity can be expressed by Stokes' law.

3.3.4 Compaction Test

At low initial water contents in the compaction test, the weight of soil solids per unit volume increases as the water content is increased for the same compactive effort. This is because the water acts as a softener and lubricant that allows the soil particles to slide across each other into denser configurations. Eventually though, the water content reaches a level in which the water begins to occupy spaces that could have been filled by solids. Any further increase in water content causes a reduction in the dry unit weight.

The transition point or maximum dry density corresponds to a water content defined as the optimum moisture content. Determination of the maximum dry density and optimum moisture content are very important for compaction control in the field and are the intended results of the compaction test. The compaction test used in this study was the modified Proctor test. The modified Proctor test better represents field conditions since the development of heavier rollers. The procedures used conform to Standard Method ASTM D 1557.

The soil was compacted in a standard 101.6 mm (4 in.) Proctor mold in five equal layers. Each layer was compacted by twenty-five blows by a 4.54 kg (10 lb) rammer from a height of 457 mm (18 in.). Several soil samples were tested at increasing water contents.

A Rainhart Mechanical Compactor was used for all compaction tests. The calibration of the device was checked several times during the testing period to assure compliance with the specified test procedures. The compaction process was repeated until the wet unit weight of the compacted soil decreased. Determination of the dry unit weight of the soil was done for each compacted sample and then plotted versus the water content. The optimum water content and maximum dry density were found from the peak of the curve.

3.3.5 Bearing Ratio

The FDOT flexible pavement design procedure (FDOT, 1990) is currently based on the 1972 AASHTO design model. The model represents the subgrade strength by a Soil Support Value (SSV). In Florida, the SSV is generally computed by determining the Limerock Bearing Ratio (LBR) of the subgrade. The LBR value is then related to the SSV by the chart shown in Figure 3.3. While the 1986 and 1993 AASHTO design guides

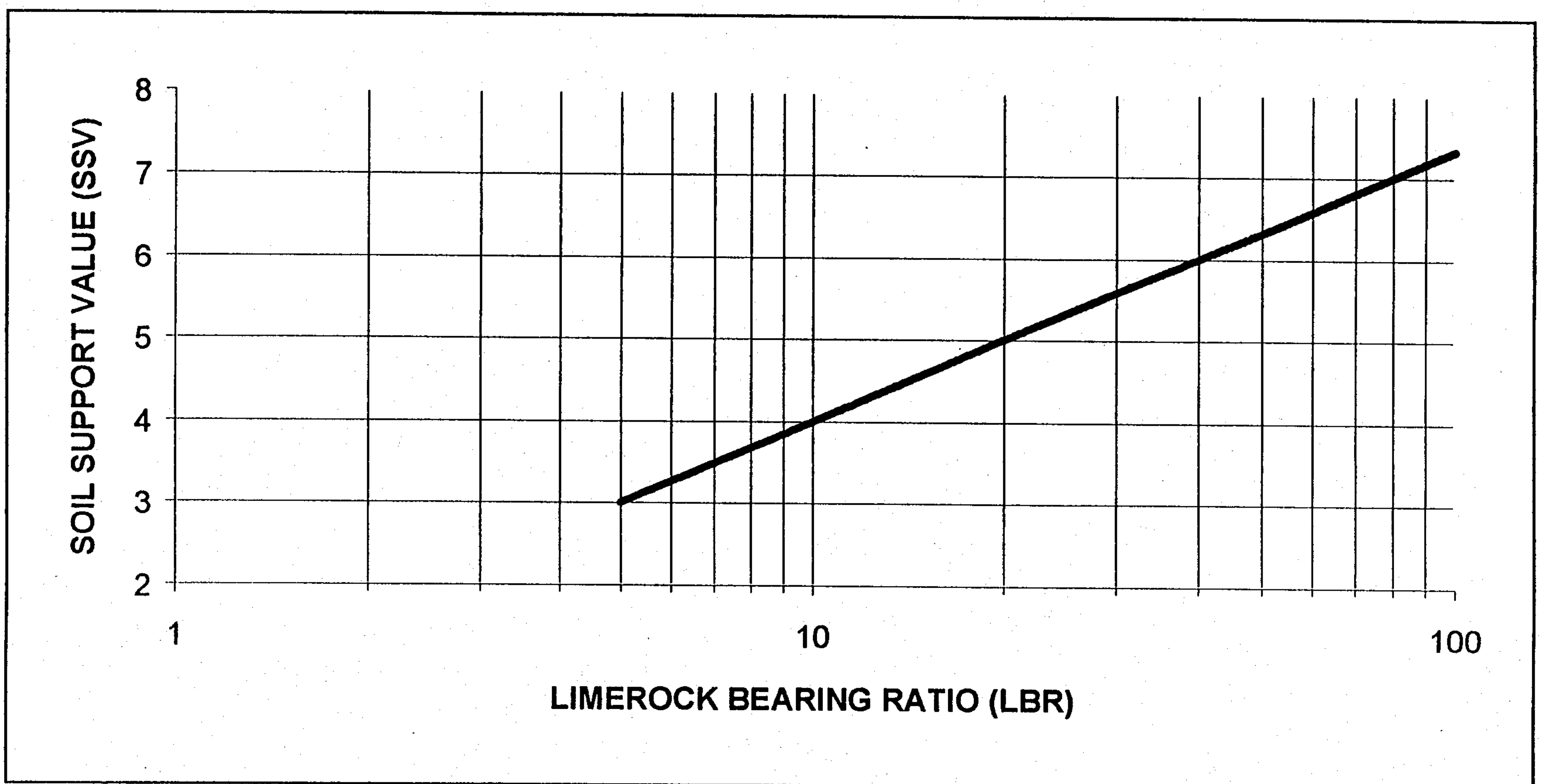


Figure 3.3: Relationship Between SSV and LBR Values

recommend the use of the resilient modulus (M_R) to characterize the strength of the roadbed soil, these methods have not been adopted by FDOT due to the unavailability of a consistent and convenient method for the determination of M_R in the field.

The LBR test is a modification of the California Bearing Ratio (CBR) test and is a measure of the bearing capacity of a soil. The test is a measure of the load required to cause a standard circular piston to penetrate a soil specimen at a specified rate. The LBR test was adopted in the early 1960's after a study initiated by FDOT showed that a modified CBR test would best represent the soils encountered in Florida. The three main modifications that were incorporated into the LBR test were the elimination of the swell test, using a two-day soaking period instead of four-day, and standardizing the strength standard to that of typical crushed Florida limerock which is 5.5 MPa (800 psi).

As mentioned above, the LBR soaking period is two-days as compared to four-days for the CBR. In this study, both two-day and four-day soaking periods were used in performing the LBR test. The purpose for using both was to determine if there was a significant change in values due to the possible presence of expansive clay and also because of the low permeability of the problem soils. The CBR swell test was also incorporated into the test procedure so as to determine the extent of any swelling. The test method used conformed to

Florida Method FM 5-515 for the LBR test and ASTM Standard D 1883 for the swell test.

Approximately 5.44 kg (12 lb) of soil passing the No. 4 sieve was air dried and the moisture content determined. Water was added in sufficient amount to bring the moisture content close to the optimum moisture content as determined from the compaction test. After standing for at least twelve hours, the soil was compacted into a 152.4 mm (6 in.) diameter mold (Figure 3.4) in five equal layers. Each layer was compacted by fifty-six blows from a 4.54 kg (10 lb) hammer dropped from a height of 457 mm (18 in.). The Rainhart Compaction machine mentioned in the last section was used to compact all samples.

The mold was placed in a soaking tank with a swell plate placed on top of the soil. A tripod with dial gage was then set on top of the swell plate stem. The specimen remained in the soak tank for either 48 hours or 96 hours ± 4 hours. At that time, the final amount of swell was recorded and the mold removed from the soaking tank.

A surcharge of 9.08 kg (20 lb) for embankment specimens was added to the soil. The mold was then placed into a Rainhart automatic compression machine with automatic recording device and a load applied through the piston at a constant rate of approximately 1.27 mm per minute (0.05 in. per minute) (Figure 3.5). The recording device produced a plot of load (pounds) versus deflection.

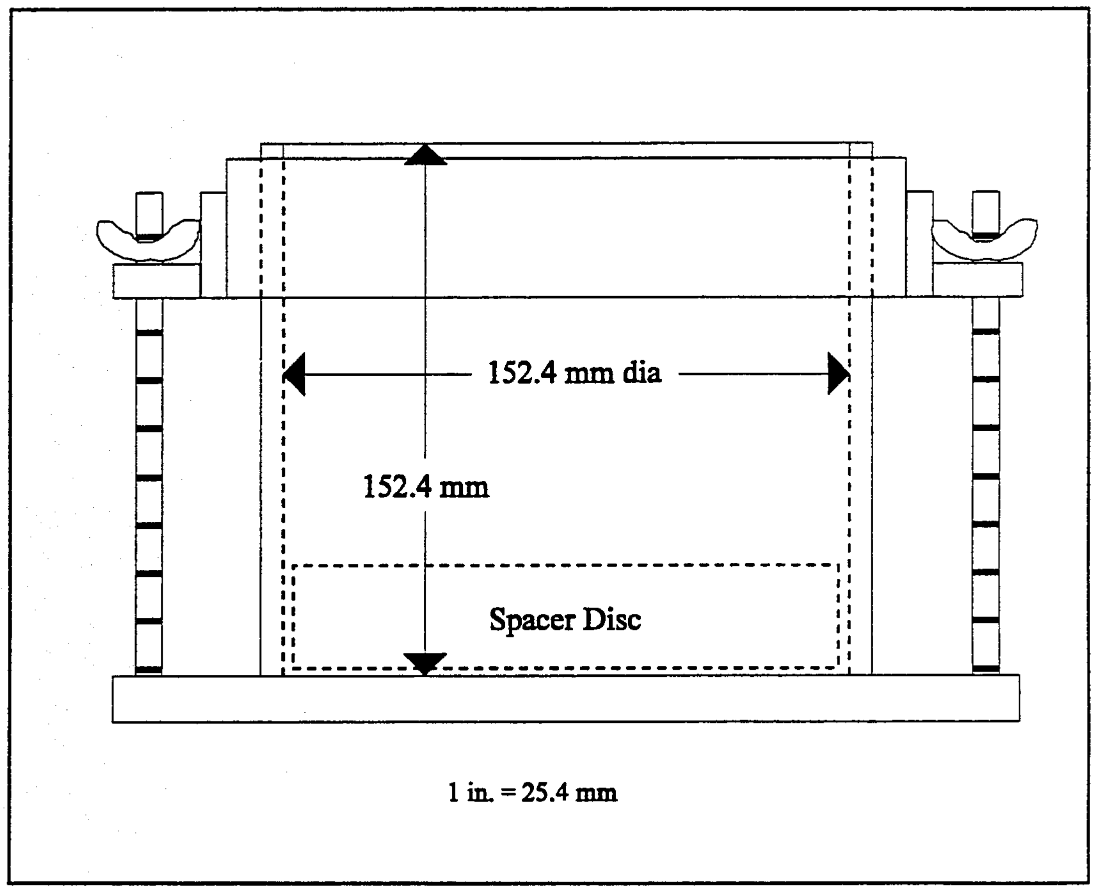


Figure 3.4: LBR Mold

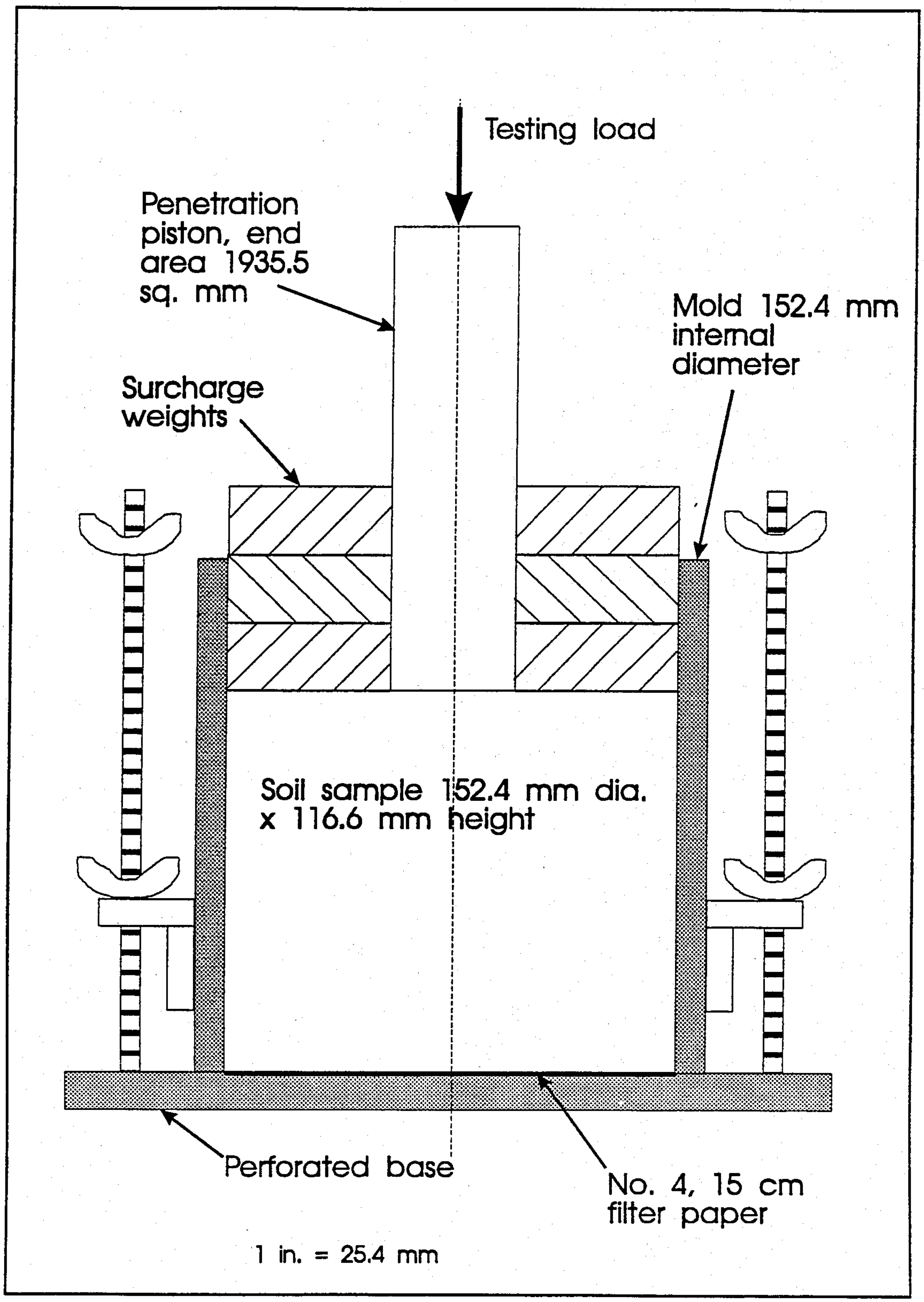


Figure 3.5: LBR Mold with Penetration Load

The plot was corrected for surface irregularities and the corrected unit load was taken at a penetration of 2.54 mm (0.1 in.) from the intersection of the corrected curve and the x-axis. The corrected unit load was than divided by the piston cross-sectional area to give the load in units of MPa (psi). The LBR value was than calculated using Equation 3.1.

$$LBR\ (\%) = \frac{Corrected\ Unit\ Load\ (\frac{lb}{inch^2})}{800\ (\frac{lb}{inch^2})} \times 100 \qquad (3.1)$$

3.3.6 Expansion Index

In 1988, after recognizing a need for a consistent method for measuring the expansion potential of soils, ASTM issued specification D 4829-88 (Standard Test Method for Expansion Index of Soils). This specification is based on California's Uniform Building Code Standard No. 29-2 and outlines the method for determining the expansion index of soils.

The expansion index, EI, is considered to be a basic index property of soil and therefore comparable to other indices such as plastic limit, liquid limit, and plasticity index. The index is not designed to duplicate any field conditions such as soil density, water content, or soil water chemistry. Instead, the procedure keeps test conditions constant which allows direct correlation of data between different organizations. The soil can be classified as to its expansion potential using the criteria shown in Table 3.3.

Table 3.3: Classification System of Expansion Potential of a Soil

Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

A specimen was formed by compacting the soil at the optimum moisture content in the mold in two equal layers for a total depth of roughly two inches. Each layer was compacted by fifteen uniformly distributed blows from a height of 305 mm (12 in.) by a 2.5 kg (5.5 pound) tamper. The sample was then trimmed and placed in the mold as shown in Figure 3.6. The procedure was repeated if the degree of saturation of the sample did not fall between forty (40) and sixty (60) percent.

A load of 6.9 kPa (1 lbf/in^2) was placed on the specimen and a dial indicator attached. The sample was submerged in distilled water for a period of 24 hours or until the rate of expansion became less than 0.005 mm/h (0.0002 in./h). The expansion index was then calculated using Equation 3.2.

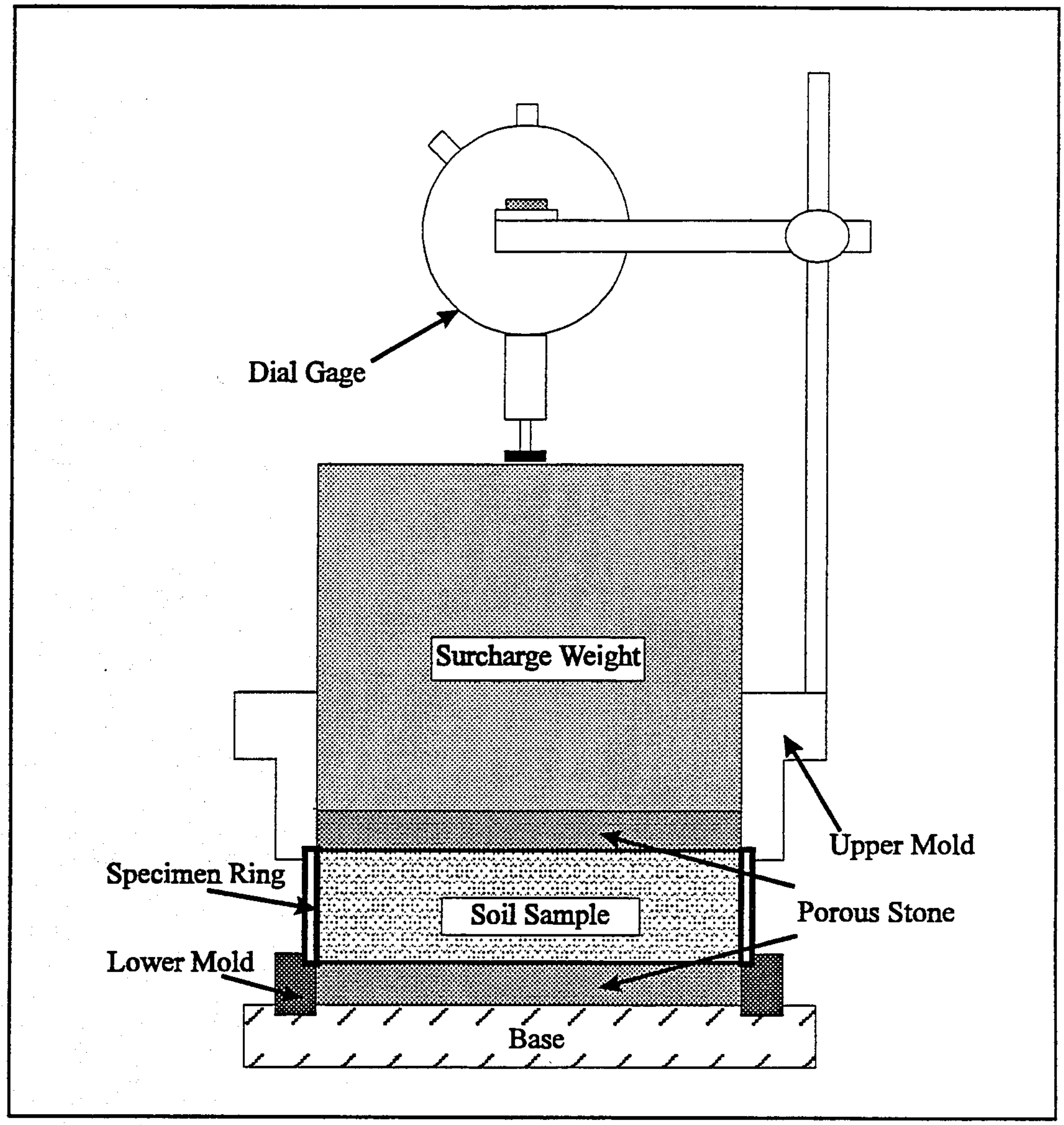


Figure 3.6: Expansion Index Mold

$$EI_{meas} = \frac{D_2 - D_1}{H_1} * 1000 \qquad (3.2)$$

where

H_1 = Initial height of soil specimen
D_2 = Final dial indicator reading
D_1 = Initial dial indicator reading

3.3.7 X-Ray Diffraction and Scanning Electron Microscope

A knowledge of the mineralogical and noncrystalline chemical composition of a soil can be of great value for understanding engineering properties. Several qualitative methods exist which can be used in the determination of the composition of soils. Two such methods, X-ray diffraction and the Scanning Electron Microscope (SEM), were used in this study.

X-ray diffraction is the most widely used method for identification of fine-grained soil minerals and the study of their crystal structure. Analysis of soil by this method is based on Bragg's law which is given in Equation 3.3. If the wavelength of the X-ray used is known, and since no two minerals have the same spacing of inter-atomic planes in three dimensions, the angle at which the x-ray is reflected can be used to identify the material.

$$n \lambda = 2\, d \sin\theta \qquad (3.3)$$

where

λ	=	wavelength of X-ray
d	=	distance between atomic planes
θ	=	angle between X-ray beam and atomic plane
n	=	integral number (1,2,etc.)

The clay fraction of a soil sample was obtained by first mixing a sample of soil with a dispersing agent of sodium tripoly. After allowing the sample to settle for 3.5 hours, the liquid was withdrawn and the clay fraction taken from the first 50 mm (1.7 in.) of settlement. The clay fraction was put on a glass slide and allowed to dry at room temperature. The glass slide was then placed in a collimated beam of parallel x-rays. The X-ray diffractometer chart that was produced from the recording is a function of radiation intensity and 2θ. The peaks on the diffractometer chart can be compared directly with patterns for known materials. ASTM maintains an extensive index of patterns based on the strongest lines in the pattern.

X-ray diffraction is particularly suited for identification of clay minerals because the strongest reflection is characteristic of the clay mineral group. X-ray diffraction is not suited for a quantitative determination of the amounts of different minerals in a soil due to the difference in mass absorption coefficients of different minerals, particle orientations, sample weights, surface texture, hydration, and crystallinity of the mineral.

The other method mentioned above is the Scanning Electron

Microscope. SEM can reveal particles and particle arrangements directly since it can resolve distances to less than 100 Å. This allows the possibility of studying small clay particles.

The SEM has a magnification range of x20 to x150,000 and a depth of field which is 300 times greater than that of the light microscope. SEM allows the viewing of secondary electrons emitted from a sample surface in what appears as three dimensional images. It is from these images that the individual particles can be observed in their surrounding arrangements.

The sample was prepared by applying a carbon adhesive to a small flat circular disk and then dipping it into a container of the dried soil. The sample was then placed in an evacuated sample chamber which could be viewed by the microscope. Photographic images of the soil particle structure were taken after the microscope was focused and the images resolved.

3.3.8 Permeability

Permeability is the property of a porous material which permits the passage or seepage of fluids through its interconnecting voids. Darcy's law is an empirical relationship that relates the flow velocity, V_h, of a liquid through soil to the hydraulic gradient, i_h. Darcy's law is shown in Equation 3.4 where k_h is termed the coefficient of permeability by geotechnical engineers. Virtually all steady-state and transient flow analysis in soils are based on Darcy's law.

$$V_h = k_h \, i_h \tag{3.4}$$

The coefficient of permeability of soils is dependent on several factors: degree of soil saturation, pore-size distribution, grain-size shape and distribution, void ratio, degree of packing, and fluid viscosity. For clayey soils, the soil structure, ionic concentration, and the water layer thickness around the clay particles also play a factor in the coefficient of permeability. No other soil property is likely to show the great range of values between coarse and fine-grained soils or vary in a given deposit as does the coefficient of permeability.

Permeability testing of the soil in this project was performed at the FDOT State Materials Office in Gainesville. One of two methods was used for testing depending on the soil. The Florida Method FM 5-513 was used when possible. If the soil was difficult to saturate completely, then ASTM Standard D 5084 was followed and the coefficient of permeability found by using a Flexible Wall Permeameter (FWP).

The Florida Method FM 5-513 outlines the determination of the coefficient of permeability by using a commercial

Compaction Permeameter from SoilTest in a falling head mode. The unit consists of a standard compaction mold and collar, clamped between a cast aluminum base and top assembly as shown in Figure 3.7. Soil at the optimum moisture content was compacted in the mold until the maximum dry density.

If the soil sample was difficult to saturate, than a flexible wall permeameter was used. The FWP is best used when the coefficient of permeability is less than $1x10^{-3}$ cm/sec ($3.28x10^{-5}$ ft/sec). The soil was compacted at its optimum moisture content in a mold to a density close to the maximum dry density. After allowing the soil sample to saturate and consolidate, the falling-head increasing tailwater level method was used to determine the coefficient of permeability. Since the coefficients of permeability were expected to be low, hydraulic gradients between 69 and 138 kPa (10 and 20 psi) were used.

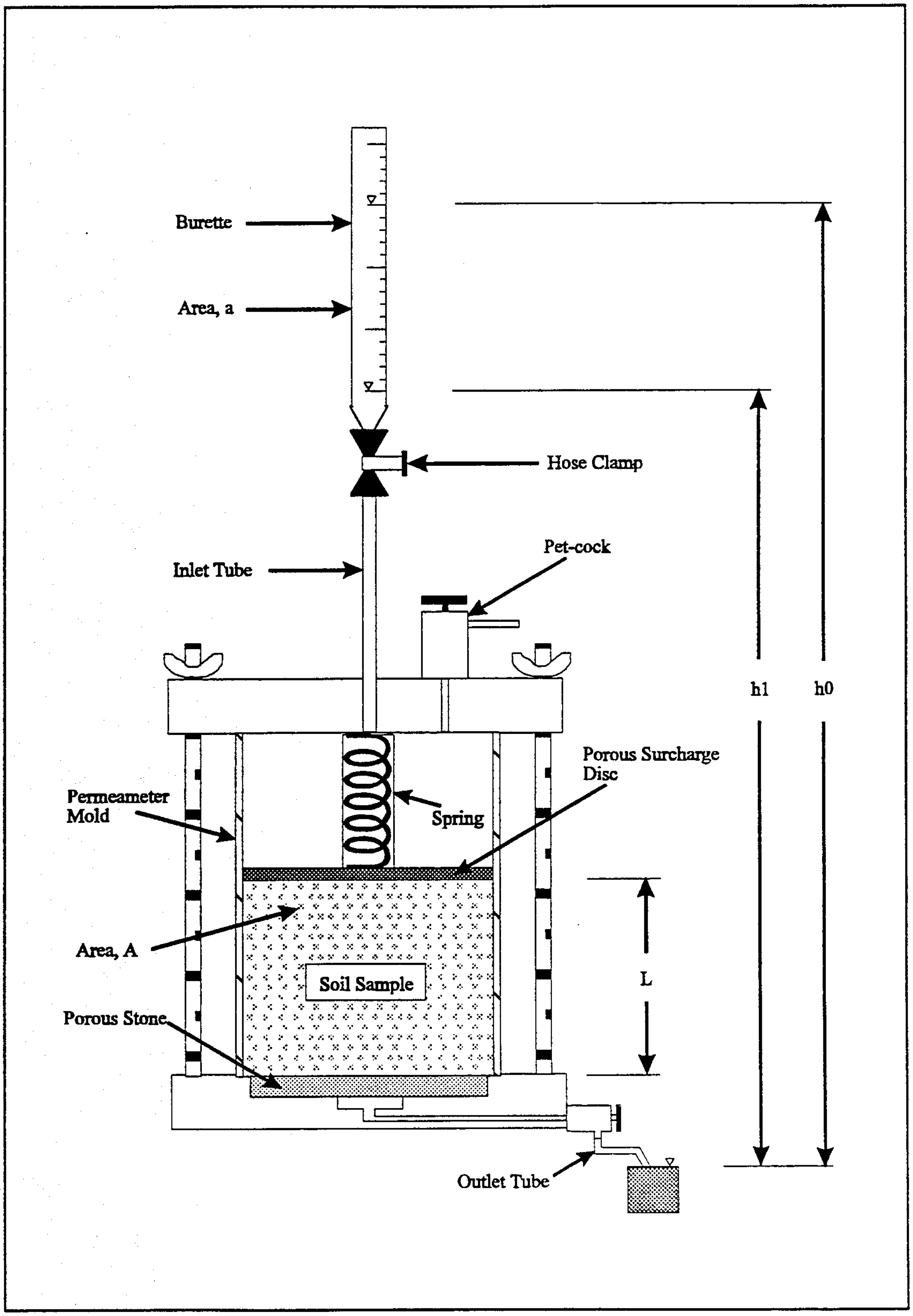


Figure 3.7: Permeameter Setup for Falling Head

CHAPTER 4

SUMMARY AND ANALYSIS OF EXPERIMENTAL TEST RESULTS

This chapter summarizes the test results obtained from the experimental program. The results are presented in the same order of presentation as the test methods in Chapter 3. A summary of all the test results are shown in Table 4.1. Analysis based on the results is presented in the final section in this chapter.

4.1 Moisture Content

The primary purpose for obtaining the moisture content upon arrival at the laboratory was to ensure that a constant level was maintained while the soils were stored during the testing program. Periodic checking showed that the storage of the soils in 55 gallon drums was satisfactory for maintaining moisture control.

The results of the test could also be used as an indication of the in-situ moisture content of the soil. This was a possibility for the soils from District 2 and 3. The values from the District 5 soils were probably less than the in-situ values because they were shipped in soil bags. This

Table 4.1: Summary of Experimental Program Results

	District 2		District 3		District 5	
Soil Properties	**Clay County**	**Madison County**	**Alford City**	**Jacobs Road**	**Brevard County**	**Marion County**
Soil Color	Orange	Light Brown	Light Brown	Reddish-Brown	Black	Dark Brown
Visual Description	Clayey Sand	Silty Sand	Silty Sand	Silty Sand	Silty Sand	Clayey Sand
AASHTO Classification	A-2-6	A-2-4	A-2-4	A-2-4	A-2-4	A-2-4
Unified Classification	SC	SM	SM	SM	SM	SC
Moisture Content, %	11.91	12.99	13.66	13.05	2.22	8.72
Liquid Limit	27	18	NP	15	19	23
Plastic Limit	14	NP	NP	NP	NP	14
Plastic Index	13	NP	NP	NP	NP	9
Passing 200 Sieve, %	27.5	25.1	17.6	20	16.8	19.9
Clay, %	24	12	4	8	10	12
Maximum Dry Density, kN/m	20.5	20.6	20.4	20.5	19.8	20.4
Optimum Moisture Content,	9.1	8.5	7.6	7.9	9.25	9.3
LBR (2 Day Soaking)	30,31	85,89	88,100	63,78	91,94	88,90
LBR (4 Day Soaking)	48,51	88,89	96,108	77,78	93,94	83,94
Permeability, cm/sec	$6x10^{-5}$ - $1x10^{-6}$	$3x10^{-6}$ - $3x10^{-7}$	2 - $5x10^{-6}$	2 - $9.5x10^{-5}$	2 - $8x10^{-5}$	$3x10^{-5}$ - $2x10^{-6}$
Expansion Index	2(Very Low)	1(Very Low)	0(Very Low)	0(Very Low)	0(Very Low)	1(Very Low)

is especially true for the Brevard County soil which was shipped inside a tractor trailer during a dry period in the summer.

Between three and four tests were completed on each soil. The results are shown in Table 4.2.

Table 4.2: Moisture Content Results

		Moisture Content, %				
District	**Location**	**Test 1**	**Test 2**	**Test 3**	**Test 4**	**Avg**
2	Clay County	11.96	12.16	11.81	11.71	11.91
2	Madison County	12.90	12.72	12.73	13.61	12.99
3	Alford City	13.88	12.89	14.20	----	13.66
3	Jacobs Road	13.17	12.68	13.32	----	13.05
5	Brevard County	2.22	2.17	2.22	2.26	2.22
5	Marion County	8.71	8.90	8.77	8.49	8.72

4.2 Atterberg Limits

The three Atterberg properties measured in the experimental program were the Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI). The plasticity index, which is the numerical difference between the liquid limit and plastic limit, is often used as an indicator of possible problematic or expansive soil as was noted in Chapter 2.

Each test was performed three to four times on each soil. The results were then averaged. A summary of the averaged results is given in Table 4.3.

Table 4.3: Atterberg Limits Results

District	Location	LL	PL	PI
2	Clay County	27	14	13
2	Madison County	18	NP	NP
3	Alford City	NP	NP	NP
3	Jacobs Road	15	NP	NP
5	Brevard County	19	NP	NP
5	Marion County	23	14	9

4.3 Grain Size Analysis and Soil Classification

Three engineering properties are required to classify soils by either the AASHTO or Unified Classification Systems. These properties are liquid limit, plastic limit, and grain size distribution. Soil classification is always the first step when evaluating a soil for use in highway projects. The soil classification is often a good indicator as to how it will perform both during and after construction. FDOT only allows soils classified as A-1, A-3, or A-2-4 under the AASHTO System for use as embankment in pavement projects.

Grain size analysis was performed on all soils in the project by sieve analysis and hydrometer testing. Three tests were done for both the sieve analysis and hydrometer testing for each soil. From the sieve analysis, percentage passing for each grain size was averaged and the result plotted versus grain size. The hydrometer test results were also plotted on the graph and a best-fit curve drawn.

Figures 4.1 through 4.6 show the percentage passing versus grain size for each soil. From the plots, the corresponding percentage of gravel, sand, silt, and clay for each soil was determined and is shown in Table 4.4. Finally, the soils are classified using the AASHTO and Unified Systems in Table 4.5.

4.4 Compaction Test

A modified Proctor compaction test was performed on all soils in the program. The primary purpose of the test was to determine the maximum dry density and optimum water content. Besides being very important for compaction control in the

Table 4.4: Percentage of Major Soil Constituents

District	Location	Gravel,%	Sand,%	Silt,%	Clay,%
2	Clay County	0	72.5	3.5	24
2	Madison County	1.4	73.5	13.1	12
3	Alford City	2	80.4	13.6	4
3	Jacobs Road	1.8	78.2	12	8
5	Brevard County	2.4	80.8	6.8	10
5	Marion County	1.3	78.8	7.9	12

Table 4.5: Soil Classification by AASHTO and Unified Systems

District	Location	Passing No.40	Passing No.200	LL, %	PI, %	AASHTO	Unified
2	Clay County	89.5	27.5	27	13	A-2-6	SC
2	Madison County	91.0	25.1	18	NP	A-2-4	SM
3	Alford City	71.9	17.6	NP	NP	A-2-4	SM
3	Jacobs Road	78.3	20	15	NP	A-2-4	SM
5	Brevard County	89.3	16.8	19	NP	A-2-4	SM
5	Marion County	78.2	19.9	23	9	A-2-4	SC

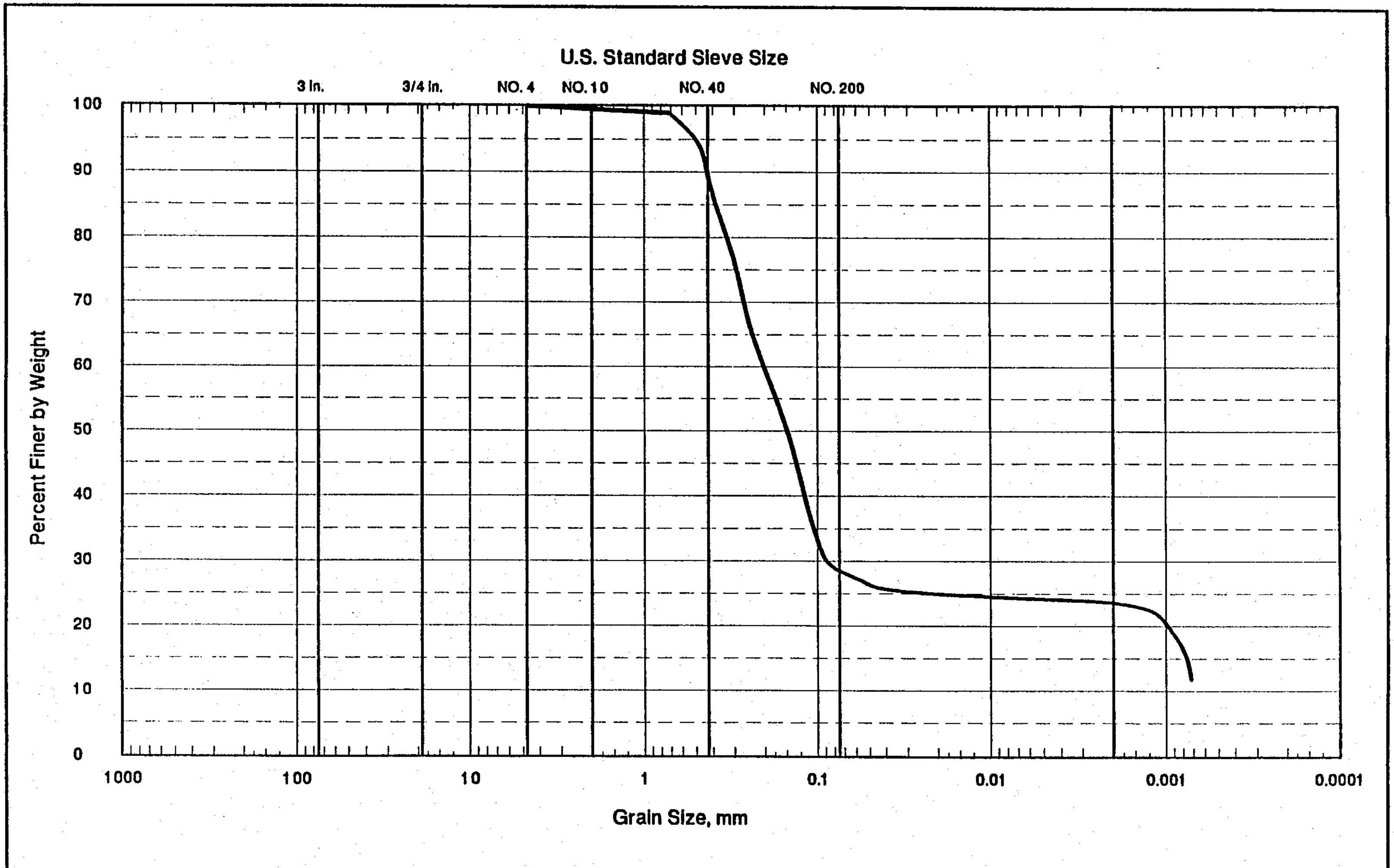


Figure 4.1: Grain Size Distribution Plot for Clay County Soil

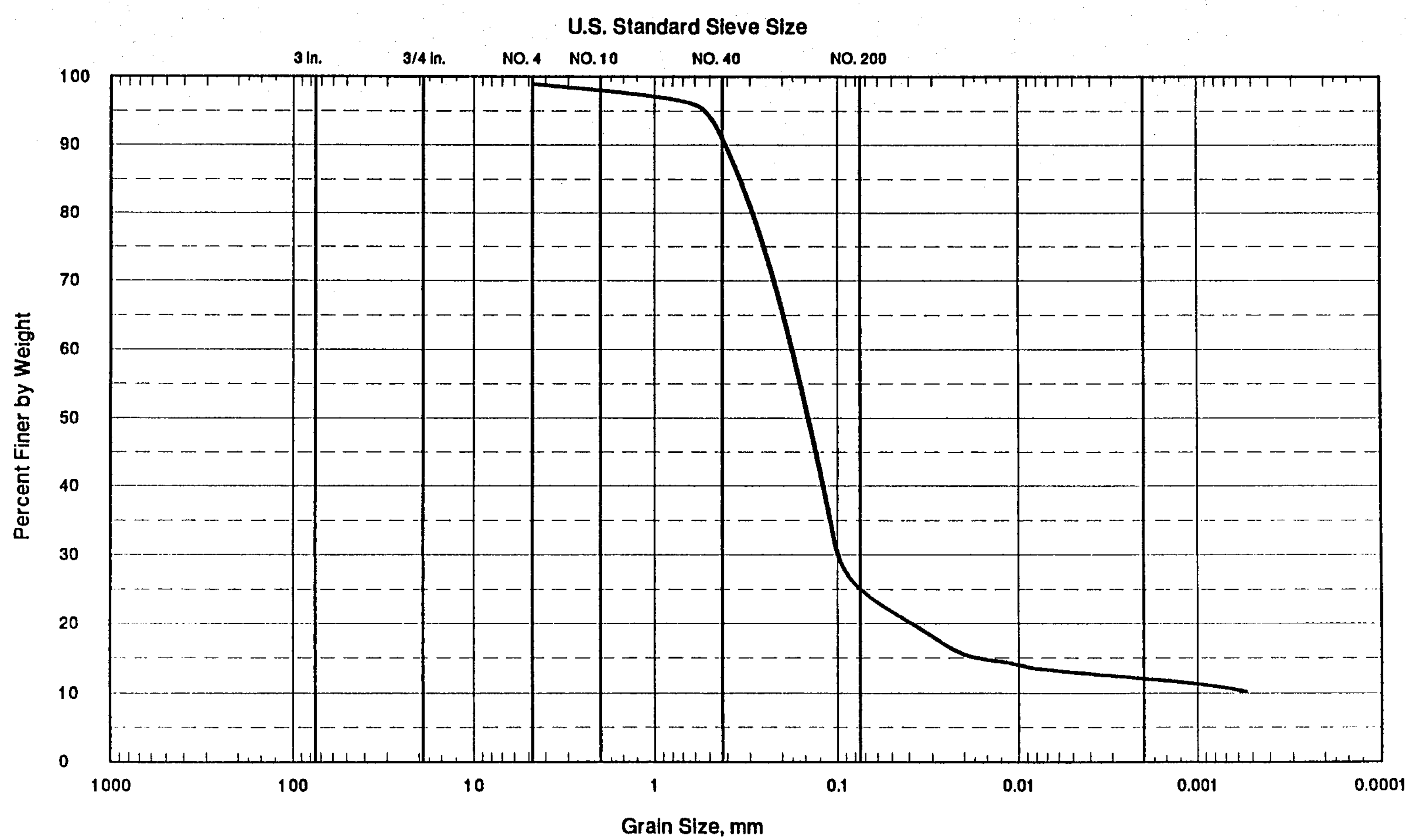


Figure 4.2: Grain Size Distribution Plot for Madison County Soil

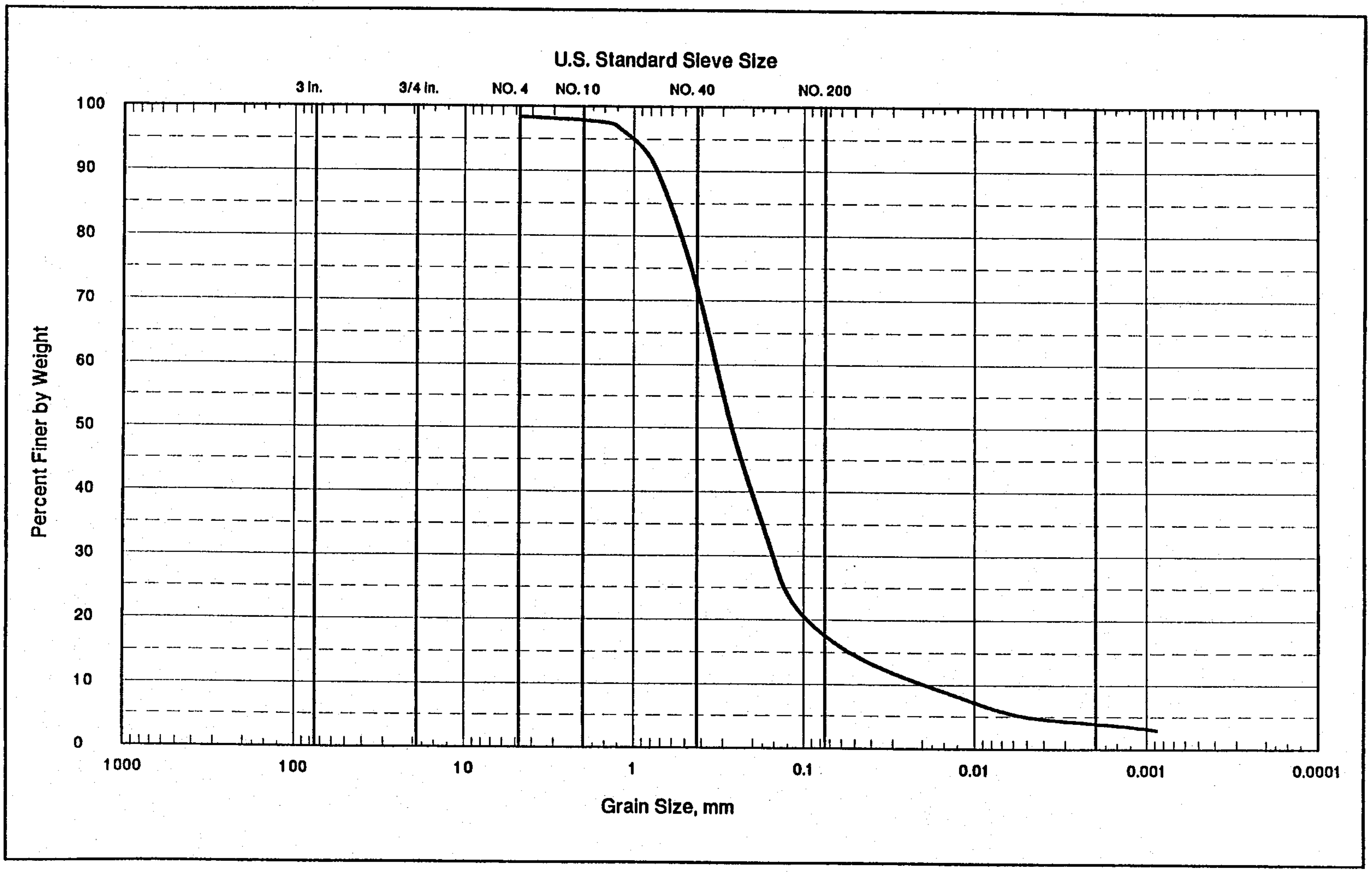


Figure 4.3: Grain Size Distribution Plot for Alford City Soil

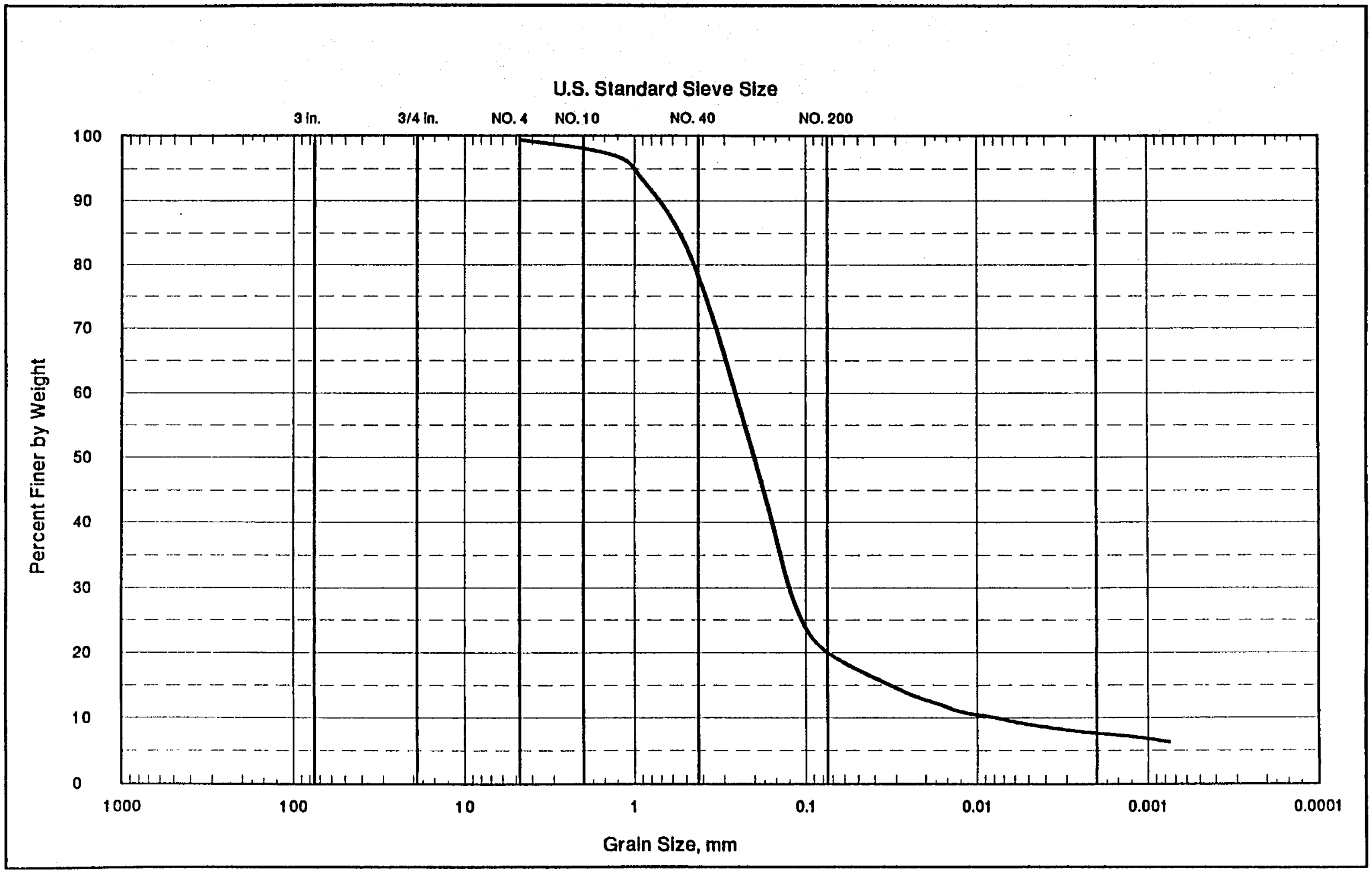


Figure 4.4: Grain Size Distribution Plot for Jacobs Road Soil

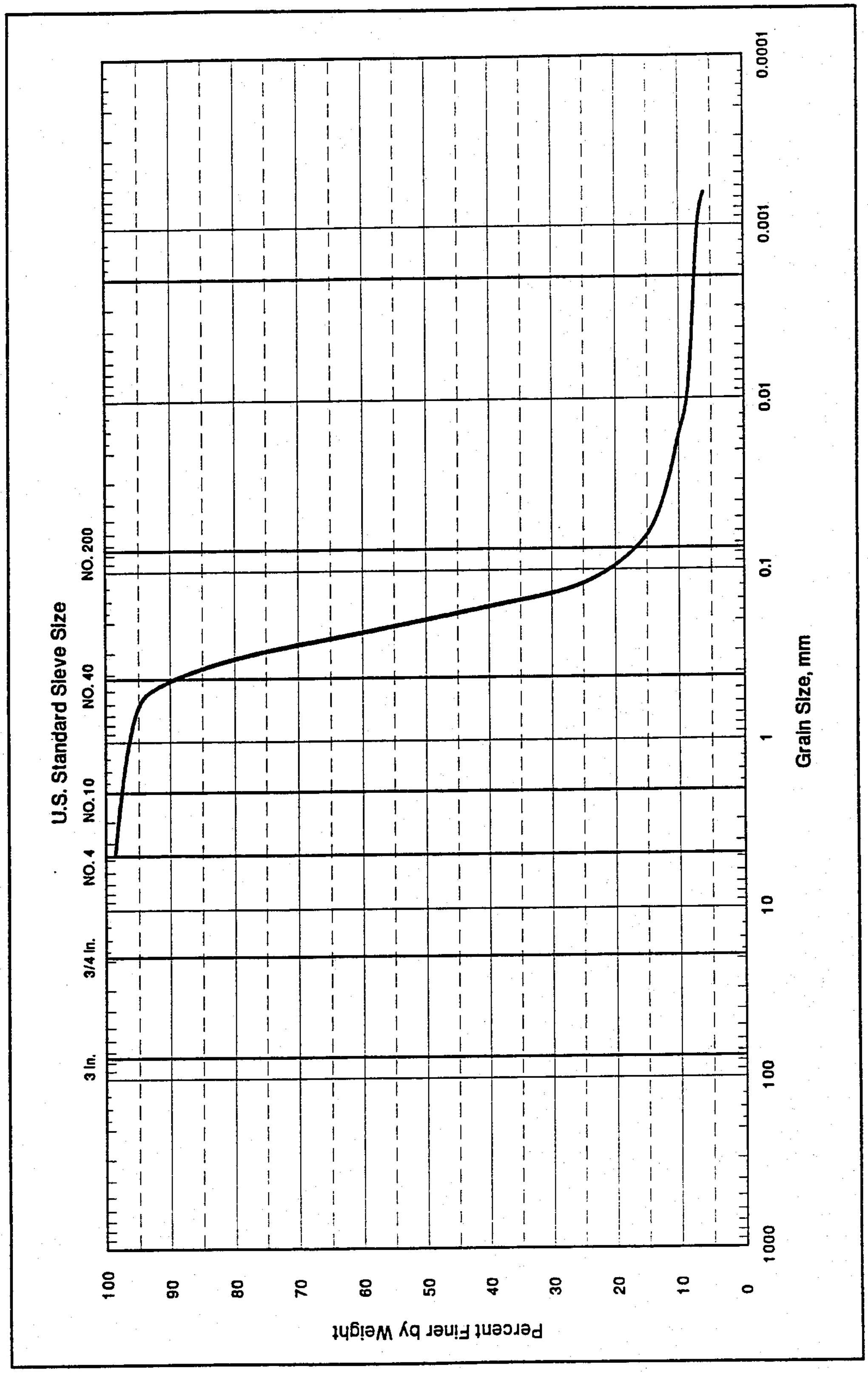


Figure 4.5: Grain Size Distribution Plot for Brevard County Soil

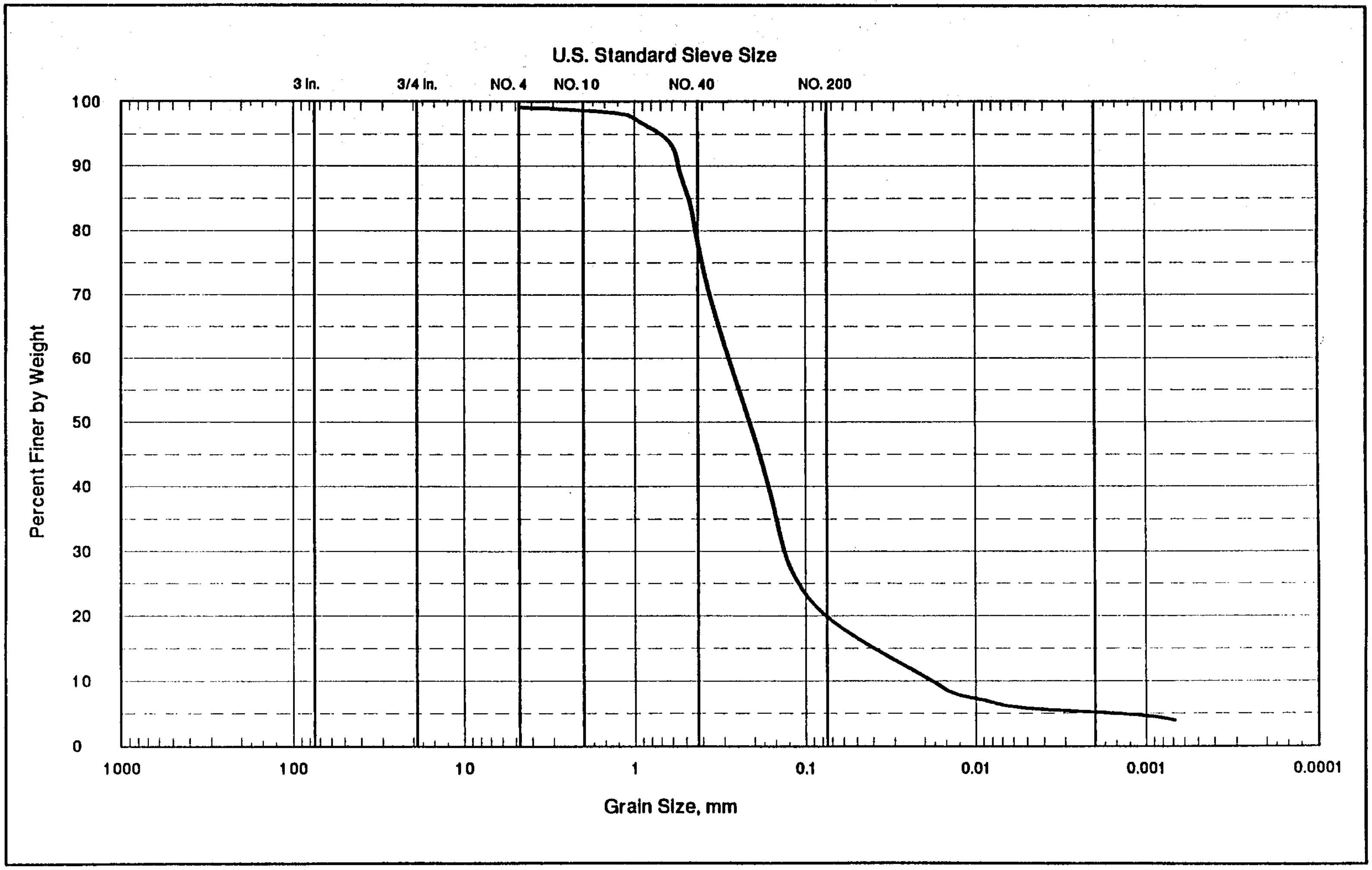


Figure 4.6: Grain Size Distribution Plot for Marion County Soil

field, it is also required before such testing as the Limerock Bearing Ratio, Permeability, and Expansion Index can be performed.

Each soil was evaluated three times with between four and six data points for each test. From the test, the dry density of the soil at its corresponding water content could be determined. Graphs of the dry density versus the water content are shown in Figures 4.7 through 4.12.

Also shown on each plot is an average regression line of all the data points. The maximum dry density and the corresponding optimum water content can be determined from the regression line. Table 4.6 lists the values for each of the soils evaluated.

The results from the compaction test for the maximum dry density are consistent with normal values obtained for clayey sand and silty sand soils. The values are slightly higher than what would be obtained using the standard Proctor compaction test because of the greater compactive effort applied to the soil.

4.5 Bearing Ratio

The Limerock Bearing Ratio (LBR) is the primary soil strength parameter used for pavement design in the State of Florida. The LBR test has been used since it was adopted by FDOT in the 1960's. Extensive research and decades of

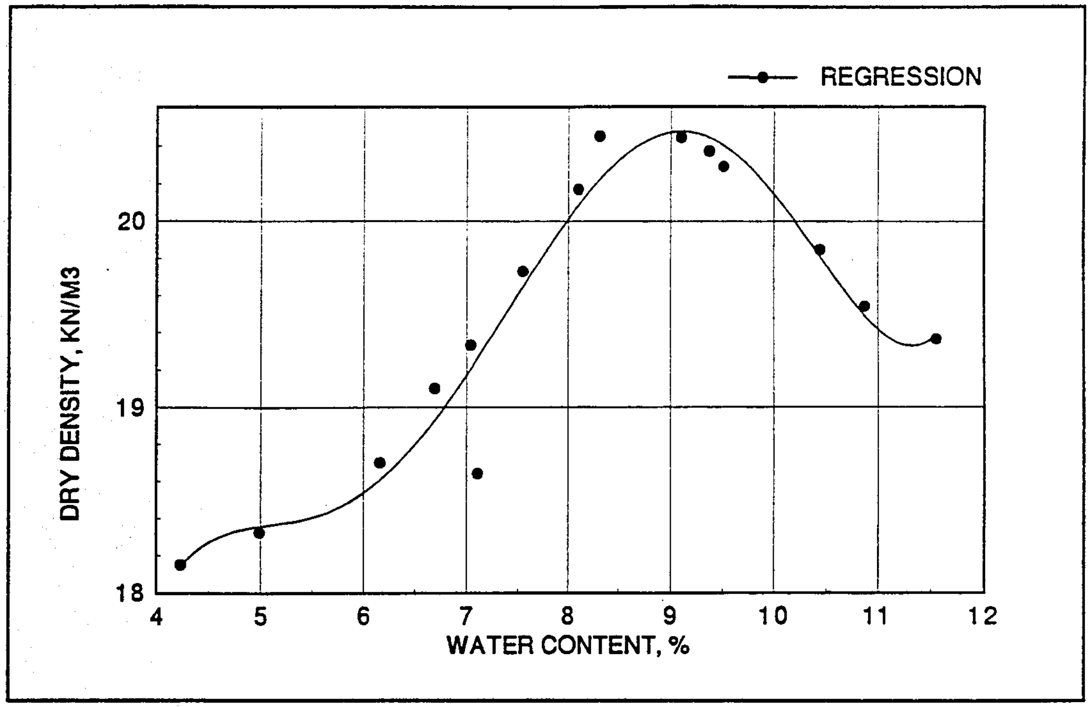


Figure 4.7: Plot of Compaction Test for Clay County

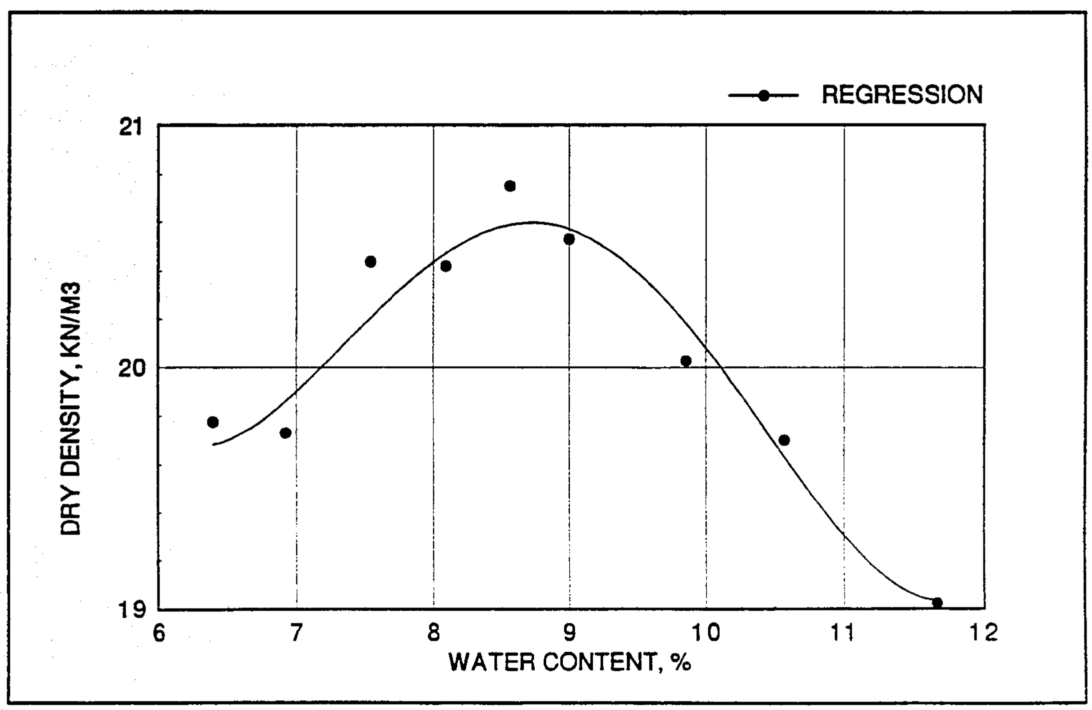


Figure 4.8: Plot of Compaction Test for Madison County

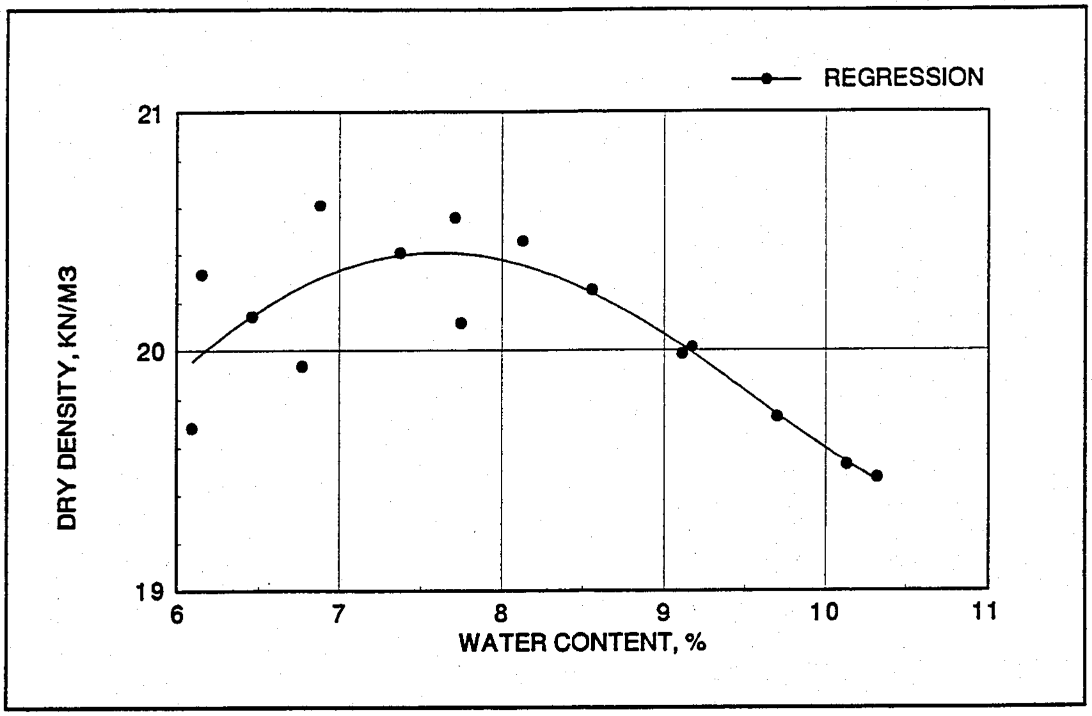


Figure 4.9: Plot of Compaction Test for Alford City

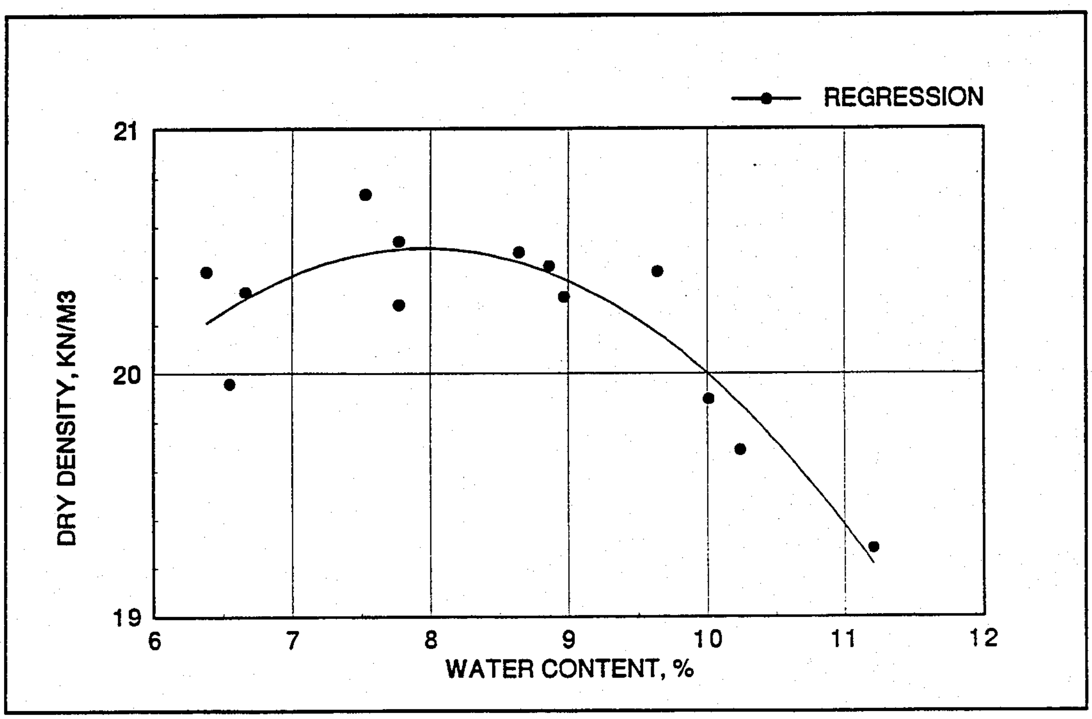


Figure 4.10: Plot of Compaction Test for Jacobs Road

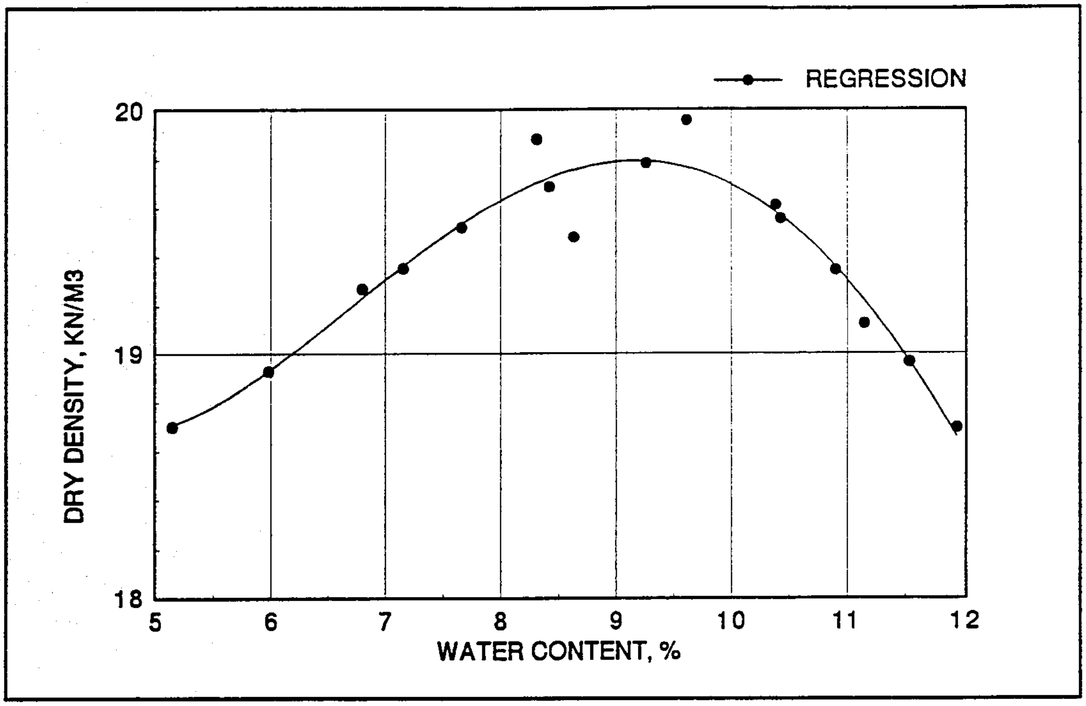


Figure 4.11: Plot of Compaction Test for Brevard County

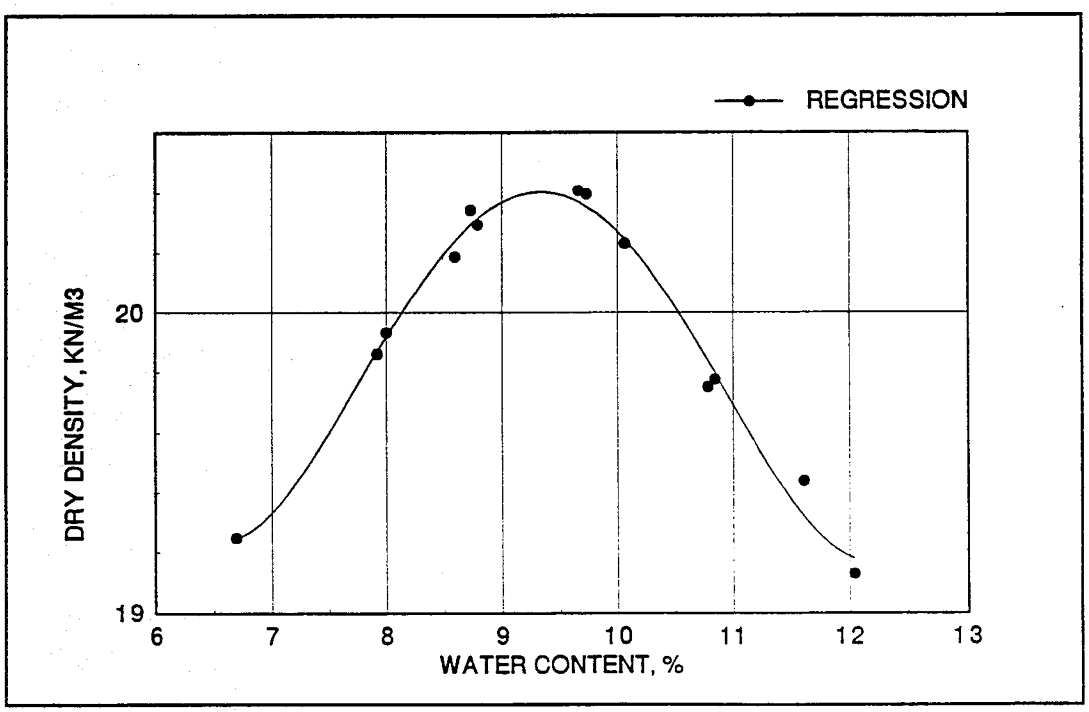


Figure 4.12: Plot of Compaction Test for Marion County

Table 4.6: Results from the Modified Proctor Compaction Test

District	Location	Max Dry ρ, kN/m^3	Optimum W_c, %
2	Clay County	20.5	9.1
2	Madison County	20.6	8.5
3	Alford City	20.4	7.6
3	Jacobs Road	20.5	7.9
5	Brevard County	19.8	9.25
5	Marion County	20.4	9.3

kN/m^3 = 6.36 lb/cf

experience have been accumulated in the use of the LBR in Florida. From this experience, minimum LBR values of soils have been set by the FDOT for use in pavement construction. Determination of the LBR value is therefore very important in any evaluation of a soil's potential use in highway construction.

Each sample was compacted and tested near the soil's optimum moisture content as determined from the modified Proctor test. Each soil was tested twice for both two-day and four-day soaking. The LBR value and percent swell of each test was then determined. The results of the two-day and four-day soaking tests for the soils in the experimental program are shown in Tables 4.7 and 4.8 respectively.

The testing showed no significant swelling for either the two-day or four-day soaking. There was an increase in the amount of swelling between the two soaking periods for all the A-2-4 soils but no discernable difference in LBR values. For the one A-2-6 soil, Clay County, there was no difference in swelling percent between the two soaking periods though there was an increase in the LBR value for the four-day soaking.

4.6 Expansion Index

The expansion index is used to classify soils according to their potential for expansion. The soil, which is compacted to a degree of saturation (S_{meas}) of 50%±1, is saturated and allowed to swell against a surcharge pressure. The difference between the initial and final height of the soil after saturation divided by the initial height and multiplied by a scale constant of one thousand is the index value, EI_{50}.

Achieving a degree of saturation of 50%±1 turned out to be very difficult. In just about every case, an empirical relationship recommended in the ASTM Standard Test Method was used to calculate EI_{50} for S_{meas} within forty (40) and sixty (60) percent saturation[2]. Equation 4.1 shows the recommended relationship. The results of the testing program are shown in

[2]American Society for Testing and Materials. "Standard Test Method for Expansion Index of Soils", *Annual Book of ASTM Standards: Soil and Rock*. D 4829-88.

Table 4.7: Limerock Bearing Ratio Values for 2-day Soaking

District	Location	W_c, (%)	ρ_{dry}, kN/m^3	LBR	Swell,%
2	Clay County	9.06 9.10	19.4 19.3	30 31	.41 .13
2	Madison County	8.50 8.50	20.1 20.4	85 89	.02 .02
3	Alford City	7.75 7.75	20.0 20.3	88 100	.00 .02
3	Jacobs Road	7.93 7.93	20.2 20.2	63 78	.07 .13
5	Brevard County	9.25 9.25	19.5 19.4	91 94	.00 .04
5	Marion County	9.27 9.27	20.1 20.1	90 88	.11 .02

Table 4.8: Limerock Bearing Ratio Values for 4-day Soaking

District	Location	W_c, (%)	ρ_{dry}, kN/m^3	LBR	Swell,%
2	Clay County	9.39 9.39	19.9 19.9	51 44	.15 .41
2	Madison County	8.50 8.50	20.0 20.1	88 89	.09 .00
3	Alford City	7.76 7.76	20.0 20.2	96 108	.16 .11
3	Jacobs Road	7.93 7.93	20.2 20.2	77 78	.42 .29
5	Brevard County	9.25 9.25	19.4 19.4	94 93	.07 .07
5	Marion County	9.27 9.27	20.2 20.2	94 83	.04 .00

Table 4.9.

$$EI_{50} = EI_{meas} - (50 - S_{meas}) \frac{65 + EI_{meas}}{220 - S_{meas}} \qquad (4.1)$$

Bold values in Table 4.9 indicate that S_{meas} was within ±1% of 50% saturation and therefore Equation 4.1 was not used. All of the soils fell into a classification of very low as described in Table 3.3. A value of fifty (50) or greater would have been required for the soil to be classified at a level where potential problems in pavement design would become significant.

4.7 X-Ray Diffraction and Scanning Electron Microscope

X-ray Diffraction was performed on all of the soils in the experimental program. The testing was performed by Mr. Robert E. Goddard, a Professional Geologist with the Florida Department of Transportation. All the soils were tested at the FDOT State Materials Office in Gainesville, Florida.

The X-ray diffraction was performed on the clay fraction (<.002 mm) of the soil to determine the minerals present. A Phillips Automatic Powder Diffractometer Model APD 35-20 was used to produce the X-ray diffractometer charts for the samples. The results of the analysis is shown in Table 4.10.

As discussed in Appendix A, the ability of a clay mineral

Table 4.9: Expansion Index Values from Experimental Program

District	Location	Test 1	Test 2	Test 3	Avg	Potential Expansion
2	Clay County	5	2	0	2	Very Low
2	Madison County	0	3	0	1	Very Low
3	Alford City	0	0	0	0	Very Low
3	Jacobs Road	0	0	0	0	Very Low
5	Brevard County	0	0	0	0	Very Low
5	Marion County	4	0	0	1	Very Low

0-20:Very Low; 21-50:Low; 51-90:Medium; 91-130:High; >130:Very High

to attract a double-layer of water is dependent on the cation exchange capacity and specific surface area of the mineral. Table A.2 shows that Montmorillonite has large values for both and is therefore normally considered expansive. Kaolinite and Illite have values several magnitudes less than Montmorillonite with Illite being higher by a factor of four (4). In Table 4.10, Smectite is the group name for water sensitive expansive layered clay minerals which include Montmorillonite. Chlorite has a double-layer attraction potential comparable to Illite.

All of the soils showed Kaolinite as the primary mineral present. Only the Brevard County soil showed any presence of Smectite or Illite. Alford City and Madison County did show

Table 4.10: X-ray Diffraction Results

District	Location	Minerals Present
2	Clay County	Kaolinite
2	Madison County	Kaolinite, minor quartz, trace chlorite
3	Alford City	Kaolinite, minor quartz, trace chlorite
3	Jacobs Road	Kaolinite, trace quartz
5	Brevard County	Kaolinite, Smectite, Illite, minor quartz
5	Marion County	Kaolinite

trace amounts of chlorite.

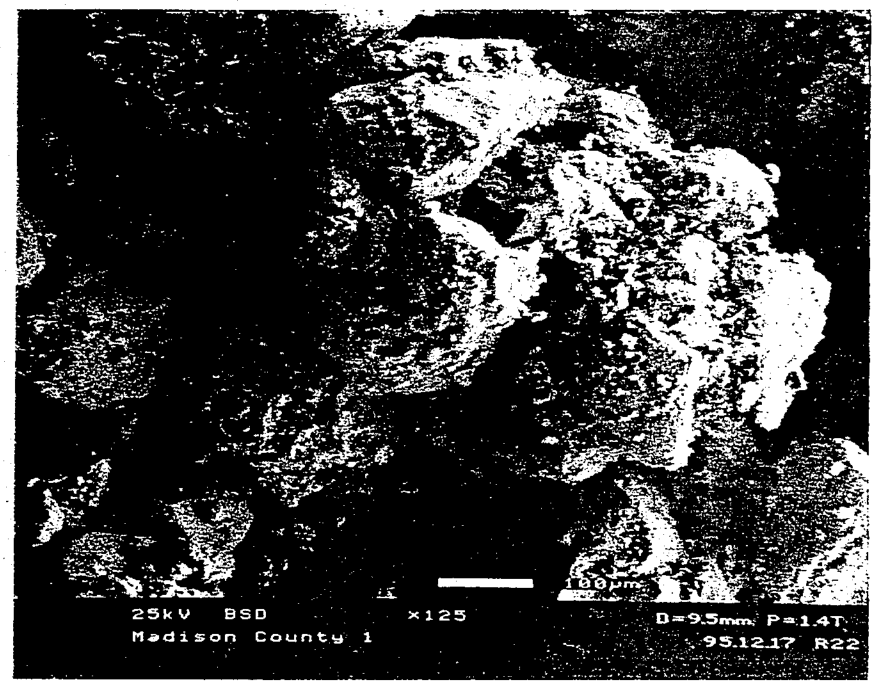
The Scanning Electron Microscope analysis was performed at the National High Magnetic Field Laboratory in Tallahassee, Florida by Mr. Farhad Booeshaghi. Two samples per soil were analyzed to produce pictures of different magnifications. An Environmental Scanning Electron Microscope (ESEM) Model E-3 by ElectroScan Corporation was used to take the pictures. The main difference between the ESEM and traditional SEM is its independent control of specimen chamber pressure and the range which that pressure can occupy during operation. This allows it to operate at pressures where water is maintained as a liquid, gas, or solid. The operating voltage for the ESEM was either 15 or 25 keV. The working distance varied from 8 to 10 mm for observations and the usual chamber pressure was 186.7 N/m^2 (1.4 torr).

Figure 4.13 and 4.14 show ESEM pictures for Madison County and Marion County. Appendix B contains ESEM pictures for all the soils in the evaluation program. In Figure 4.13, a cluster of sand and silt particles is shown. The foremost particle in the center of the picture is sand with a diameter of roughly 220 μm which is comparable to passing a No. 60 sieve. Attached to the sand are silt particles ranging in diameter from 6-13 μm. Individual clay particles are too small to distinguish at this magnification.

Figure 4.14 shows a clump of sand, silt, and clay particles. The cracks along the surface are from shrinkage during the drying process of the soil. This picture shows that a normal sieve analysis may not give an accurate size distribution due to attractive forces which hold the particles together. Either a mechanical breaking of the soil or a wet sieve is required as was used in this test program to separate the clay and silt particles from the sands.

4.8 Permeability

Permeability testing measures the rate at which water flows through a soil structure. Coarse grained soils naturally have a higher permeability than fine grained soils. Typical values of permeability for sands are between 1 and


100μm
25kV BSD ×125
Madison County 1
D=9.5mm P=1.4T
95.12.17 R22

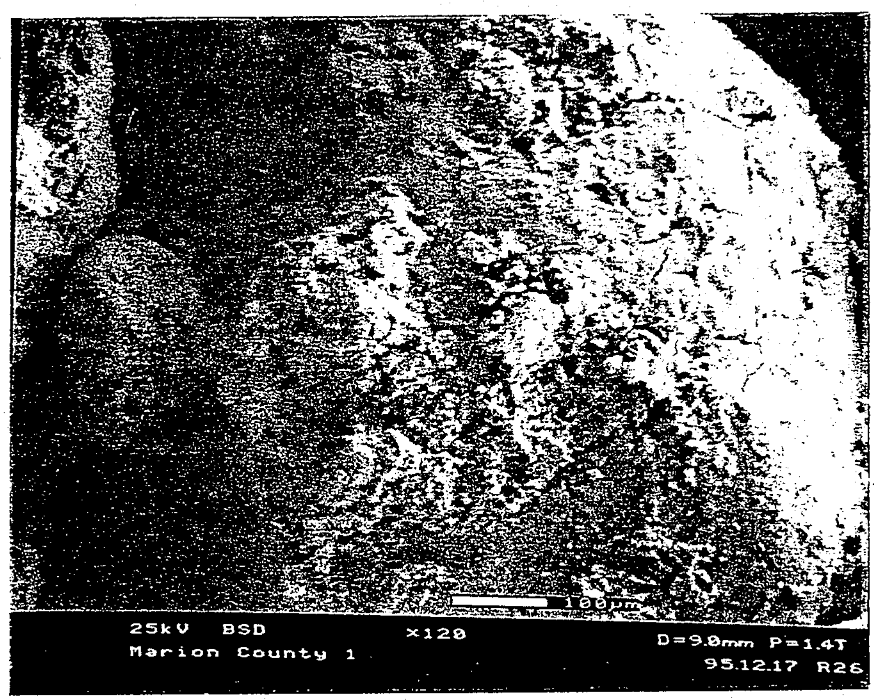


Figure 4.14: ESEM Picture of Marion County Soil

$1x10^{-3}$ cm/sec.

Each soil in the experimental program was tested to determine the permeability. The testing was done by Mr. David J. Horhota and his staff at the FDOT State Materials Office in Gainesville, FL. The testing was performed during the late summer and early fall of 1995.

As mentioned in Chapter 3, one of two test methods was used depending on the difficulty in saturating the sample. The Compaction Permeameter from SoilTest was used on easily saturated soils while a Flexible Wall Permeameter was used on difficult soils. During the test program, only the Alford City soil was able to be used in the Compaction Permeameter. The results of the permeability testing is shown in Table 4.11.

Tests were run at different confining pressures. The confining pressure was used during the saturation and consolidation phases of the test and varied from 48 to 241 kPa (7 to 35 psi). The greater the confining pressure, the longer it takes to saturate the sample though a denser sample is formed during the consolidation phase. Therefore, a greater confining pressure equates to a lower permeability. Confining pressures of 69 and 138 kPa (10 and 20 psi) were used, first to check that the permeability did decrease with an increase in pressure. Secondly, and probably more importantly, the typical permeability value assumed in pavement subbase using this test is between these two pressures, so an average is

Table 4.11: Permeability Results from Experimental Program

District	Location	Dry Density (kN/m^3)	Water Content (%)	Permeability (cm/sec)			
				Compaction Permeameter (SoilTest)	Flexible Wall Permeameter Confining Pressure		
					69 kPa	138 kPa	Average
2	Clay County	19.3	9.8		1.85E-06	1.23E-06	1.54E-06
		18.5	9.0		5.74E-05	5.65E-05	5.70E-05
2	Madison County	19.6	9.8		4.11E-07	2.03E-07	3.07E-07
		19.7	9.6		1.20E-06	5.65E-07	8.83E-07
		19.6	9.8		3.13E-06	2.05E-06	2.59E-06
3	Alford City	19.3	7.1	4.93E-06			
		19.9	7.2	4.23E-06			
		19.7	7.3	2.32E-06			
3	Jacobs Road	19.6	8.6		2.79E-05	1.84E-05	2.32E-05
		19.4	7.6		6.10E-05	5.54E-05	5.82E-05
		19.3	7.3		1.10E-04	7.73E-05	9.37E-05
5	Brevard County	18.6	10.2		9.38E-05	5.36E-05	7.37E-05
		18.7	10.1		3.63E-05	1.92E-05	2.78E-05
5	Marion County	19.5	9.6		3.98E-06	1.29E-06	2.64E-06
		19.1	9.6		2.49E-05	2.13E-05	2.31E-05

1 kN/m^3 = 6.36 lb/cf

usually taken.

At the beginning of the permeability testing, each soil was to be tested with three samples. This turned out to be very time consuming and labor intensive due to the saturation and consolidation periods required. The number of samples tested per soil was reduced to two half-way through the testing program. Two samples happens to be the current FDOT standard for determining permeability for state projects.

As was mentioned in Chapter 3, it is not uncommon for permeability values to show a great range of values between coarse and fine-grained soils or vary in a given deposit. This can be seen in the average values listed for each soil. Therefore, a range of permeability values will be specified for each soil as was done in Table 4.1 instead of one specific value.

4.9 Analysis of Results

From the results presented above, several observations may be made. These observations begin to paint a picture that describes the nature and performance of these soils when viewed together.

The primary complaint about the soils from the contractors was that they were hard to compact and slow drying. In most specifications for earth work, the contractor is required to achieve a compacted field dry unit weight of

95% of the maximum dry unit weight as determined by the Proctor compaction test. Referring to Table 4.6, the maximum dry density for each soil along with the corresponding optimum moisture content is shown.

Since the contractor wants to compact the soil at a moisture content near the optimum to achieve the most economical compactive effort, the initial moisture contents of the soil shown in Table 4.2 should be compared with the optimum values. Excluding the soil from Brevard County because of the circumstance noted in Section 4.1, all of the soils arrived with a moisture content near or greater than the optimum as shown in Figure 4.15. This gives an indication that the in-situ moisture content was also greater. The study done at UF (University of Florida, 1972) also found this relation.

If the soil is slow drying, the water is not leaving at a fast rate. As mentioned before, permeability measures the rate at which water flows through a soil structure. Many factors effect the flow of water including grain-size shape and distribution, pore-size distribution, void ratio, and degree of packing. Figures 4.1 through 4.6 show the grain size distribution in the soils. All of the soils have a good distribution of particle sizes for the sand range. This and the fact that all the soils have between seventeen and twenty-eight percent passing the No. 200 sieve as shown in Figure 4.16 allow for tight packing of the particles during

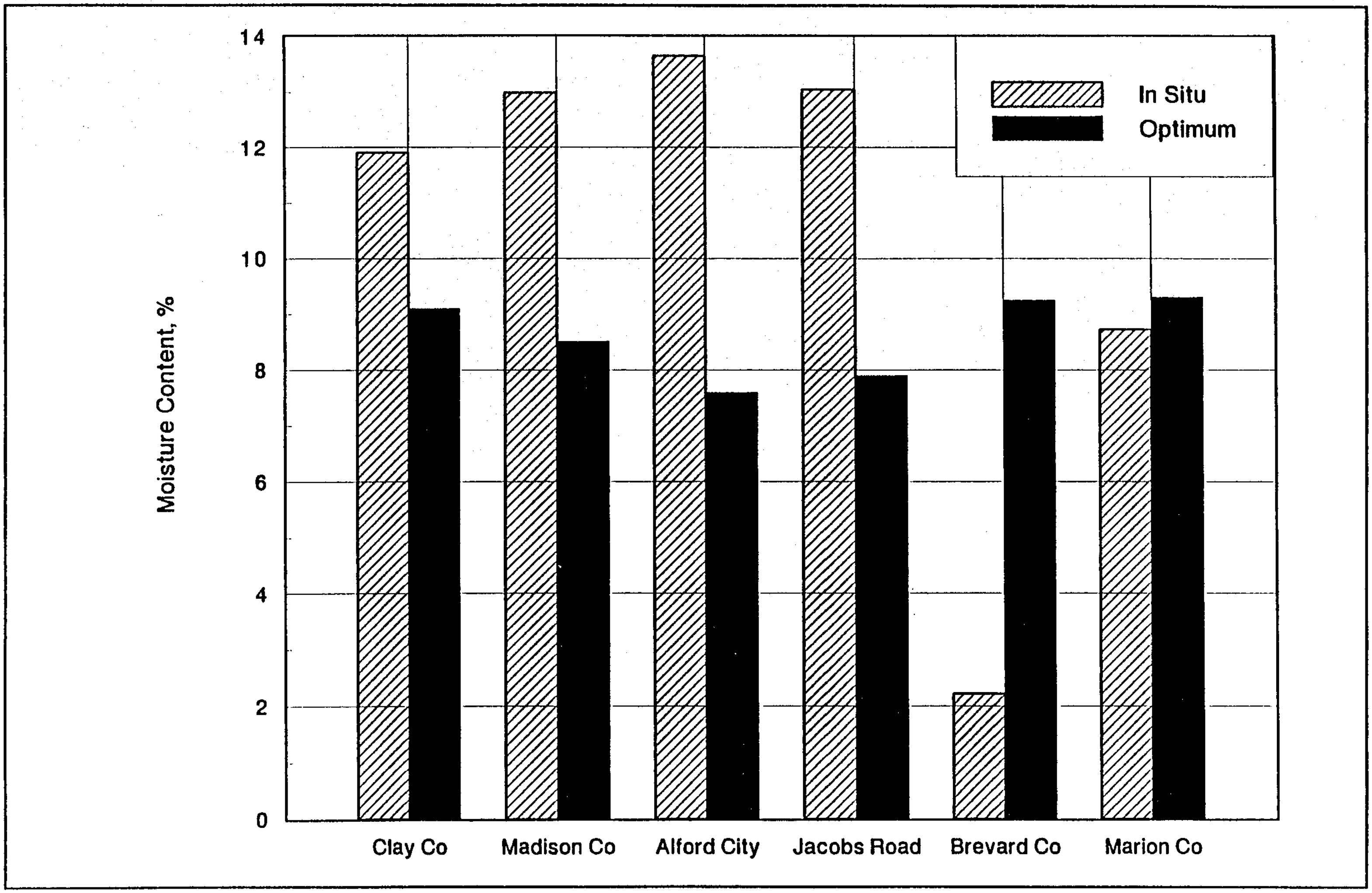


Figure 4.15: Comparison of Optimum and In-situ Moisture Content

compaction. This can be confirmed by observing the maximum dry density values in Table 4.6 which range between 19.8 and 20.6 kN/m^3 (126 and 131 lb/cf).

With a tight packing and good grain-size distribution, a low value of permeability would not be uncommon. Figure 4.17 shows that the permeability values for the problem soils run in the range of 10^{-5} to 10^{-7} cm/sec. Again, this coincides with the results from the UF study.

Permeability rates can also be affected by certain clay particles in soils. Factors such as clay structure, ionic concentration, and water layer thickness around the clay particles can influence the permeability. As discussed in Appendix A, certain types of clays such as Montmorillonite have a large cation exchange capacity and specific surface which directly affect ion concentration and the double-layer of water around clay particles. The presence of Montmorillonite or similar type clay in the soil could definitely be a factor in the low permeability values.

Table 4.10 lists the mineral composition of the clays as determined from X-ray Diffraction. The major constituent in all the soils was Kaolinite. Brevard County did show the presence of Montmorillonite and Illite. Kaolinite has a low cation exchange capacity and specific surface, so its influence on the permeability would obviously be much less than for Montmorillonite.

Another concern was that swelling of the soil from

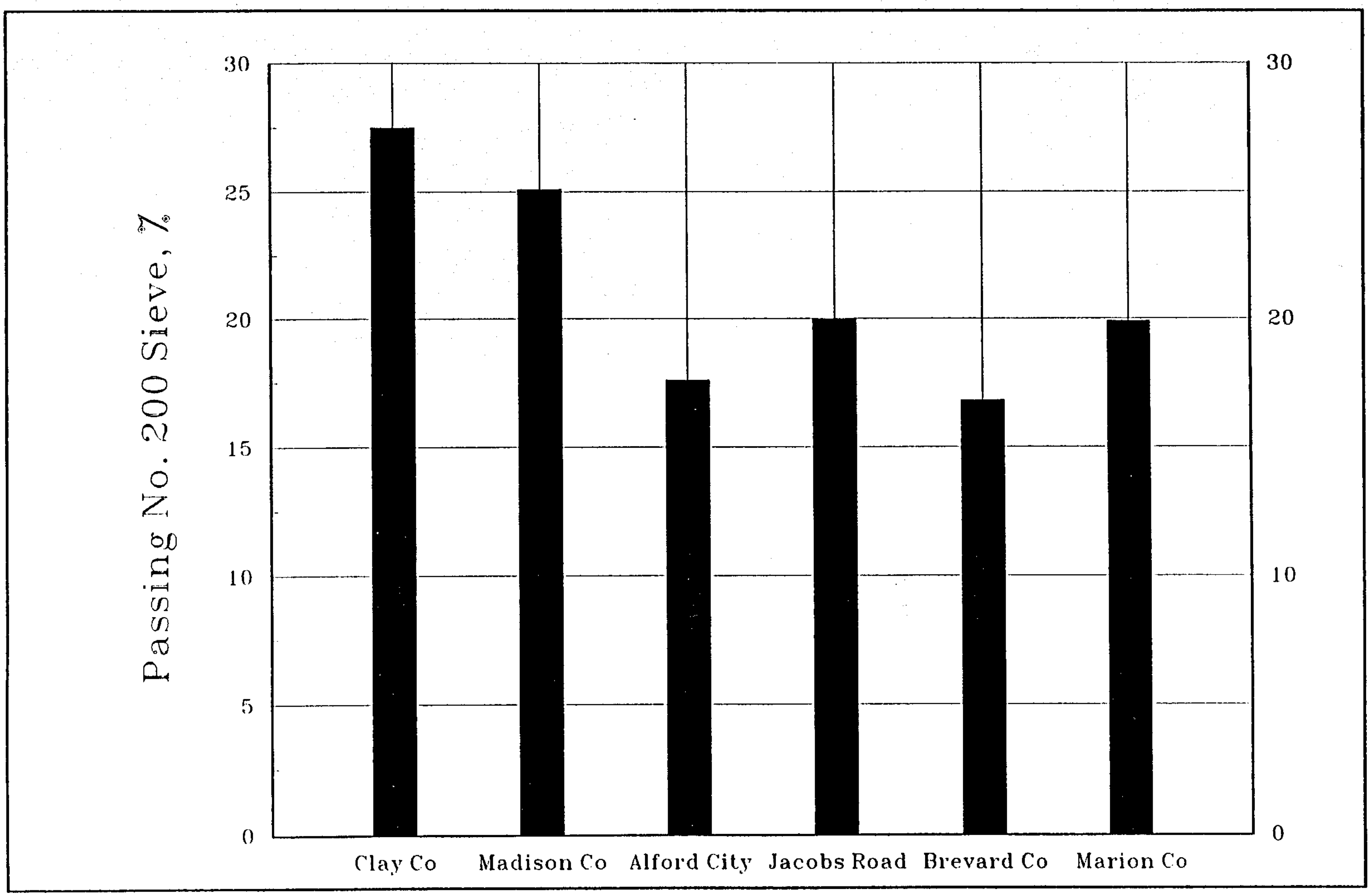


Figure 4.16: Comparison of Passing No. 200 Sieve Percentage

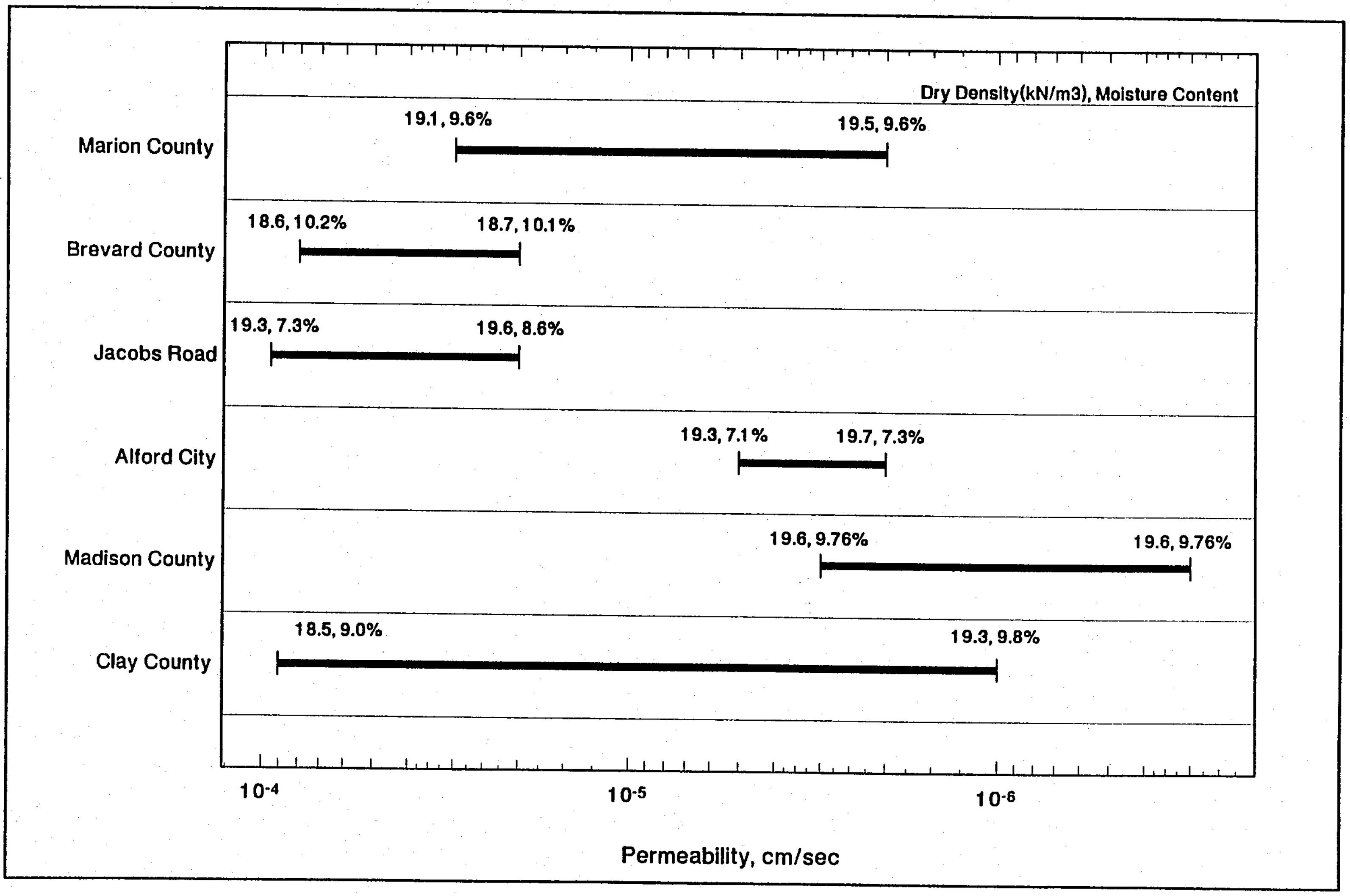


Figure 4.17: Comparison of Permeability Ranges

expansive clays could contribute to the problems. In particular, it was postulated that LBR values could be affected by only soaking the sample for two days as compared to four days for a standard CBR test. The LBR value of each soil was tested for both two-day and four-day soaking. Swell measurements were also taken on each sample.

The results of the tests, which are listed in Table 4.7 and Table 4.8, show that no significant swelling of the sample occurred. The results do show that there was a difference in swelling for the A-2-4 soils between the two-day and four-day soaking, but the amounts were still insignificant.

From Figure 4.18 it can also be seen that there was no change in LBR values regardless if it was soaked for two or four days for the A-2-4 soils. Usually it is expected that LBR values decrease with longer soaking times. In this case, the low permeability of the soils probably did not allow for substantial saturation of the sample and, therefore, no effect on the LBR value occurred. The increase in LBR for Clay County can most likely be attributed to the greater initial dry density of the soil for the four day soaking than the two day soaking.

All of the A-2-4 soils had LBR values greater than the FDOT minimum standard of forty (40). The A-2-6 soil (Clay County) did not reach the minimum until after four days of soaking. Regardless of the LBR value, the A-2-6 soil would not be eligible for use because only A-1, A-3, and A-2-4 soils

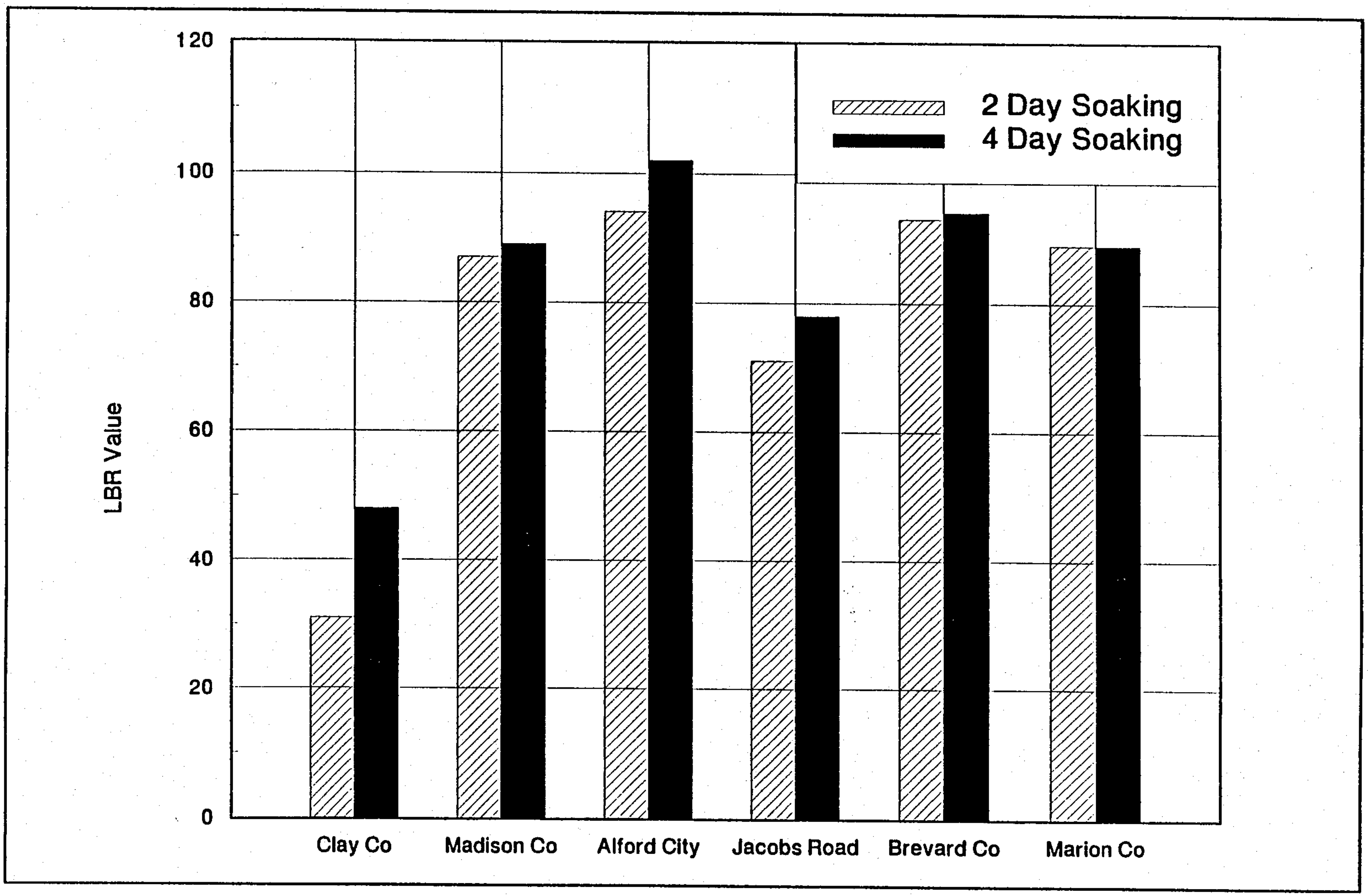


Figure 4.18: Comparison of LBR Values

are identified as 'select' for embankments.

The Expansion Index test was performed on all the soils to classify them as to their expansion potential. As seen in Table 4.9, all of the soils classified as Very Low for expansion potential. Since the Activity value which was discussed in Chapter 2 has been used for a long time as an indicator of expansion potential, values for each soil were calculated and are shown in Table 4.12. Each soil is classified as Inactive and therefore reaffirms the results from the Expansion Index test.

The last observation to be made is in the use of the Unified Classification System. Table 4.5 showed the AASHTO and Unified classification of each soil. The soils in this study were either SM or SC. A comparison of the AASHTO and Unified Systems (Liu, 1967) showed that the most likely soils in the Unified System to fall in the A-2-4 classification are GM, SM, GC, and SC. All of the soils in this test program and the UF study identified as having construction problems were either SM or SC. It may then be deduced that A-2-4 soils that classify as GM or GC do not experience construction problems and may be used without further testing.

Table 4.12: Activity Values for Soils in Experimental Study

District	Location	Activity	Classification
2	Clay County	.54	Inactive
2	Madison County	0	Inactive
3	Alford City	0	Inactive
3	Jacobs Road	0	Inactive
5	Brevard County	0	Inactive
5	Marion County	.67	Inactive

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

The conclusions based on the data and analysis from this study are summarized below.

1. Each of the soils evaluated had been identified by FDOT as problematic during construction. In all cases, the predominant complaint was that the soil was slow drying and hard to compact in order to meet the specified percent compaction. One prevalent theory at FDOT was that the construction problems were directly related to the presence of expansive clays in the soils.

2. All but one of the problem soils had an in-situ moisture content above or close to the optimum moisture content.

3. The Plastic Index is often used as an indicator of potentially expansive soil when high. All of the

soils had a low PI value.

4. The percentage of clay in the A-2-4 soils ranged from four (4) to twelve (12) percent. The predominant clay type in all the soils was Kaolinite which is considered non-expansive. Only the Brevard County soil showed a presence of Montmorillonite.

5. All of the soils in the program were silty sand or clayey sand and were identified as SM or SC under the Unified Classification System. None of the soils identified as problematic by FDOT in this test program or the one done by the University of Florida were classified as GM or GC.

6. No significant indication of expansion or swelling was found from the LBR swell test, Expansion Index test, or Activity value. In fact, all soils were classified as Very Low and Inactive in regards to expansion potential. No significant difference was found between LBR values for two-day soaking and four-day soaking.

7. Permeability values for all the soils were low and ranged from 10^{-5} to 10^{-7} cm/sec. The soils were very difficult to saturate during the permeability

testing. Low permeability and high in-situ moisture content may be the main causes for the difficulty of drying and compacting the problem soils during construction.

8. A similar study done by UF (University of Florida, 1972) on problematic soils found similar characteristics and engineering properties except that expansive Montmorillonite clay was found in a majority of the samples. The study concluded that the presence of Montmorillonite clay in a soil could be used as an indicator of problematic soils. The current study showed that soils can experience the same problems and characteristics without the presence of Montmorillonite and therefore should not be used as an indicator.

5.2 Recommendations

The following recommendations are based on the results and observations from identified problem soils.

1. A-2-4 soils experiencing construction problems do not fall into the Unified Classification of GM or GC. A-2-4 soils classified as SM or SC should be identified for further testing.

2. Soils identified for further testing should have their in-situ and optimum moisture content determined. If the in-situ moisture content is greater than the optimum and the soil is greasy and sticky to the touch, the soils may be considered potentially troublesome.

3. Permeability values should be determined on all potentially troublesome soils. In most cases, a Flexible Wall Permeameter may be required and the samples should be compacted near the maximum dry density. If the permeability value is 10^{-5} cm/sec or lower, the likelihood of problems during construction may be considered high.

4. If the potential for problems is high for a soil, it should be substituted with another soil. If this is not possible, adequate time should be allocated for drying and compacting the soil in the construction schedule. Mechanical and chemical alterations may be utilized to stabilize the soil by established construction techniques.

APPENDIX A

EXPANSIVE SOILS

APPENDIX A

EXPANSIVE SOILS

Thirty-six states in the U.S. and its territories have expansive soils. The Southwest, Southeast, Western Mountains, and Central Plains have the highest concentrations. These expansive soils can cause construction problems in highways and foundations. Compounding the problem is the lack of an exact identification method for these soils. Over the years different models and equations have been developed to identify expansive soils based on different tests and properties, but none have held up over time in every situation.

The purpose of this appendix is to examine highway construction and pavement design in relationship to expansive soils. A review of the mechanisms and influencing factors of volume change in expansive soils will be provided. The current AASHTO specifications which account for swelling from expansive soils in both flexible and rigid pavement design will be included along with several other procedures which can reduce swelling affects. In addition, some of the more successful maintenance and construction methods that have been used will be presented.

A.1 Volume Changes in Expansive Soils

"Expansive soils", "troublesome soils", "problem soils", or "active soils" are all terms that have been used to describe soils which experience volume changes when their moisture content varies. These volume changes can cause serious and expensive damage to roads and foundations. Methods for identifying the expansive soils, predicting their swelling potential, and modifying the soil structure have been presented over the years, but to date a consistent method for dealing with these soils has not been developed.

For highways, the first indication of damage from expansive soil subgrades is a reduction in riding quality. If the problem is not tackled immediately, a loss of pavement integrity will soon follow. The first signs that become apparent is an unevenness along a section of roadway without any significant cracking. Longitudinal cracking and local deformation will than become visible. Finally, localized failure of the pavement will occur.

It is important to identify expansive soils and have a qualitative indication of the extent of the potential swell problem as early in the design and construction sequence as possible. Table A.1 lists three methods available for minimizing volume change and pavement damage. The first method, while seeming quite simple, is often not an option. Route selection is often completed before the geologic materials transversed are considered. Instead, local, social, economic, environmental, and/or political considerations drive the selection process. This leaves methods two and three which will be discussed later in Section A.3.

Table A.1: Methods for Minimizing Volume Change

1. Select route to avoid the expansive material. 2. Alter the expansive material by mechanical or chemical means to reduce its potential volume change. 3. Maintain in situ moisture content or increase the moisture content to an equilibrium condition.

Once a route has been selected, distribution maps should be consulted to identify any categories of expansive soils that may be transversed by the road. Maps such as the U.S. Geological Survey, U.S. Department of Agriculture soil surveys, or the U.S. Federal Highway Administration's distribution survey manual[1] can be used to estimate the likelihood that expansive soils will be encountered. If such maps indicate that expansive soils may be present along the route, then further research is warranted. Past experience from existing highways and structures within the area should

[1] Federal Highway Administration (1976). *An Occurrence and Distribution Survey of Expansive Materials in the United States by Physiographic Areas*, FHWA-RD-76-82, January 1976.

be reviewed for indications of expansive problems. A field exploration program should also be initiated to identify and classify expansive materials. Once the expansive soils have been identified, measures can be taken to offset their effect.

Mechanisms and factors which influence volume change in expansive soils will be discussed further in this section. Methods for minimizing problems in pavements during design, construction, and maintenance stages will be covered in subsequent sections.

A.1.1 Mechanisms of Volume Change

The component of expansive soil that is most responsible for the volume changes is clay. Clay is a fine crystalline material with particle diameters of 0.002 mm or less. Most clay particles exist as thin flakes, tubes, of fibers depending on molecular geometry and crystal lattice. The three most common clay minerals are Kaolinite, Montmorillonite, and Illite.

Table A.2 lists typical properties of the three most common clay minerals. It should be noted from the table that the specific surface and cation exchange capacity of Montmorillonite is far greater than the other clay minerals by several factors. As will be seen shortly, this will be very important in regards to volume change.

Table A.2: Properties of Clay Minerals[2]

Mineral	Kaolinite	Illite	Montmorillonite
Particle Thickness	0.5 to 2 microns	0.003 to 0.1 microns	≥ 9.5 A°
Particle Diameter	0.5 to 4 microns	0.5 to 10 microns	0.05 to 10 microns
Specific Surface (m^2/gram)	10 -20	65-180	50-840
Cation Exchange Capacity (millequivalents) / (100 grams)	3-15	10-40	80-150

There are three basic microscale mechanisms which cause volume change in expansive clays: 1) Clay mineral surface attraction; 2) Cation hydration; 3) Osmotic repulsion. There is also one macroscale mechanism, that of elastic rebound. When summed, these four mechanisms account for the volume change experienced by expansive clays.

Clay minerals, because of their natural physicochemical properties, possess a net negative electrical charge imbalance. This negative imbalance attracts cations and the positive pole of dipolar water molecules to create an electrically neutral system called the clay "micelle". Because of this, a double-layer of water molecules is "built" around the clay mineral and volume change occurs.

[2] Woodward-Clyde & Associates (1967). *A Review Paper on Expansive Clay Soils, Volume 1*, pg 9.

Referring back to Table A.2, it should now be obvious that Montmorillonite is far more expansive than Illite or Kaolinite. With a greater cation exchange capacity and larger surface area, the amount of water attracted to the double-layer will be several magnitudes larger for Montmorillonite than for the other two clay minerals. Experience in the field has confirmed this by finding that soils with Montmorillonite clay have far greater expansion potential.

The attached cations further contribute to the double-layer by combining with water through the process of hydration. The last microscale mechanism that influences the volume change in expansive soils is osmotic repulsion. Osmotic repulsion occurs when the concentration of ions differ between the double-layer of water and the pore water. When this occurs, the pore water will move toward the higher ion concentration and attempt to reduce it, thus adding to the thickness of the double-layer.

Elastic rebound denotes all causes of volume changes due to the deformation or rearrangement of the solid component in the clay-water-air system. While in most situations this mechanism is insignificant, it does become important during cut/fill operations when clays are reworked and overburden pressures are changed. Quantifying the exact volume change caused by this mechanism is difficult because cut/fill operations often include other factors such as moisture content and density changes.

A.1.2 Influencing Factors of Volume Change

There are several factors that can influence the swell and swell potential of a soil. Swelling potential is defined as the ability and degree that a soil might swell if its environment is changed, and is thus a property of the soil. Swell is a measure of the actual increase in volume of a soil mass when the soil's environment is changed, and is thus a measure of the degree to which the swelling potential is realized under given conditions.

Swell is determined by the initial conditions of the soil, the surcharge load, and other environmental factors. The swelling potential is determined solely by the type and amount of clay mineral in the soil. So drawing from the previous section, clays containing high percentages of Montmorillonite will generally have high swelling potentials. And of course, swell potential for a soil increases as the percentage of clay increases.

The initial conditions of a soil consist of the geotechnical properties of density, water content, soil structure, and permeability. Each of these properties can have a significant effect on the swell of an expansive clay. High dry density, for example, means that more clay particles are contained per unit of volume. The higher density results in a higher energy level from the force fields surrounding the clay particles, and therefore a greater potential for expansion.

The lower the initial water content of the soil, the greater the swell that will occur when the water content is increased. Preferably, the water content should be near the optimum moisture content or above. Swelling will not occur without variations in the subgrade moisture content, so if a expansive subgrade soil can be isolated from any moisture changes, then the swell can be reduced or minimized. All of the construction and maintenance methods which are covered in Section A.3 are based on this principal of reducing moisture variations.

The soil structure of the material is also important. It has been shown that increased dispersion of the soil particles results in reduced swelling. Higher water contents produce a more dispersed structure, so it is apparent that water content and soil structure are closely related. Along the same lines, it has been found that compaction methods which produce increased shearing strains result in a more dispersed soil structure.

It should be intuitive that an increase in applied surcharge load will reduce the magnitude of swell. As a surcharge load is increased on a soil, so does the effective stresses which cause the amount of swelling to decrease. On the other hand, as the load is reduced, so are the effective stresses which allows the soil to swell.

Environmental effects which are important include depth of the active zone, surface drainage, depth to and fluctuations of the groundwater table, vegetation, temperature, and climate. All of these deal with avenues for changes in the moisture content which has been shown above to be the driving force behind any volume change in expansive soil. For example, a change in temperature of 1°C has been shown from experiments conducted at Princeton to be equivalent to a hydrostatic head of 3 ft in its ability to cause moisture migration.[3] All of these factors presented influence the amount of swell.

A.1.3 Identification of Expansive Soils

Many different methods have been proposed over the years to identify expansive soils. Most methods have proven to be applicable to select soils or locations. Recently, the American Society for Testing Materials has released a new specification for standardizing the classification of soil expansion which was reviewed in Chapter 2 and is used in the experimental study.

There are certain visual indicators which can be used to signify the presence of expansive soils. Five such indicators

[3] Winterkorn, H. F. (1951). Discussion of Paper by F. L. D. Wooltorton on "Movements in the Desiccated Alkaline Soils of Burma", *Transactions*, ASCE, Vol. 116.

are listed in Table A.3[4]. These indicators can be used when performing the field exploration program and can help select soil sample locations.

Table A.3: Visual Indicators of Expansive Soils

1. Exposed surfaces of expansive soils, when dry, exhibit an irregular or pebbly texture resembling popcorn. Also, on dried, exposed surfaces, desiccation cracks will be evident particularly during dry times of the year. The more frequent and deeper the desiccation cracks, the greater the potential swell. The dry strength of the exposed surface material is generally very high.
2. When moist, the plasticity of expansive soils is evident by attempting to roll a small piece of the soil into a thread. The easier it is to roll the thread, the higher the plasticity and generally the higher the potential swell.
3. Fissures and slickenslides are abundant in freshly exposed surfaces of most expansive soils.
4. When wet, expansive soils have a very slick, cohesive texture and will adhere to shoes or tires of vehicles.
5. Distortions or tell-tale damage to adjacent structures will be evident.

A.2 Pavement Design Methods

Design of highway pavements on expansive soils requires special attention and considerations. The design still needs to meet the general requirements of all pavements, namely that they will not fail, there will be minimal deformation, and the cost is not inhibitive. The design must also account for

[4] U.S. Department of Transportation (1980). *EXPANSIVE SOILS in Highway Subgrades: Summary*, Federal Highway Administration, FHWA-TS-80-236,April 1980.

expected volume changes in the pavement subgrade which will occur during the analysis period. This volume change can be accounted for in the design layer thickness, or by applying one of the methods listed in Section A.3.

Various methods have been developed for pavement design, each based on a different set of factors. Most of the previous design methods are based on the C.B.R. design criteria. The reference by Kassiff gives an excellent review of past design procedures from around the world for pavements on expansive soils. In the United States, most pavement designs are based on the methods suggested by the American Association of State Highway and Transportation Officials (AASHTO). Each state modifies the design procedures to account for local conditions and preferences.

The 1993 *AASHTO Guide for Design of Pavement Structures* specifies design procedures to account for pavements which will be built on potentially expansive soils. The method outlined is simple and straight forward, and requires each state department to calibrate the Swell Rate Constant Nomograph based on experience from local conditions. The design procedures recommended by AASHTO will be reviewed in the following section followed by a section on a few additional features that can be incorporated into pavement designs to reduce swelling by expansive soil subgrades.

A.2.1 AASHTO Pavement Design

The 1993 AASHTO Guide recommends design procedures for flexible and rigid pavements. As part of each procedure, the designer has the option of accounting for subgrade soils which are expansive. The method, which is basically the same for rigid or flexible, is very simple to use.

In the AASHTO Guide, swelling and frost heave are treated together because both are considered to lead to a significant loss in serviceability and ride quality. The loss of serviceability from roadbed swelling, frost heave, and cumulative axle loads are cumulative and are used to determine the length of the performance period for each stage of construction. For both pavement types, the expected length of the performance period is found through a iterative process at which the combined serviceability loss due to traffic and environment reaches the terminal level.

The most important part in accounting for the serviceability loss from swelling in the design procedure is the development of the serviceability loss versus time plot. The development of this plot is identical for both flexible and rigid pavement design, and will be demonstrated in this section. The loss of serviceability from swelling should not be used in the design process if one of the methods outlined in Section A.3 is to be used and the potential for differential swelling then eliminated.

The procedure for developing the serviceability loss versus time plot (swelling curve) is outlined in Appendix G of the AASHTO Guide. Three variables which affect the rate and magnitude of the serviceability loss need to be estimated: 1) Swell Probability; 2) Swell Rate Constant; 3) Potential Vertical Rise. The swell probability is a percentage that represents the likelihood of swelling at a given location. If it is determined, from laboratory and field testing, that the subgrade soil under a road has a high probability of expanding, than the swell probability for the entire road is 100%. If the proposed road design transverses both expansive and non-expansive soils, the road can be separated into swelling and nonswelling sections with the swelling sections having a 100% swell probability or the entire road can be assigned an average value which would be the percentage of the total area subject to swell.

The swell rate constant estimates the rate at which swelling will occur in the subgrade. It is a function of moisture supply and soil fabric. Values range from 0.04 to 0.2. Figure A.1[5] shows a chart that can be used to estimate the constant. AASHTO recommends that agencies calibrate this chart based on local conditions and experience. A high swell rate constant indicates a greater accessibility of water to the subgrade soil, and thus a greater chance of swelling.

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[5] American Association of State Highway and Transportation Officials (1993). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C, Figure G.2.

The last variable which must be estimated is the potential vertical rise (V_R), and it represents the amount of vertical expansion that is likely under extreme swell conditions. Preferably, designers should use laboratory results or local experience to estimate this variable. Figure A.2[6] presents a chart provided in the AASHTO Guide for estimating V_R based on the soil's plasticity index, moisture condition, and layer thickness. This chart should only be used when data on subgrade soil is unavailable and every attempt has been made to obtain it.

The data for predicting the swell chart can be tabulated per road section as shown in the example in Figure A.3[7]. Weighted averages based on section length should be calculated for the swell rate constant, swell probability, and potential vertical rise. Only values greater than 0.2 inches should be used in calculating the vertical rise. Using these three weighted values and the chart shown in Figure A.4[8], a swelling curve can be plotted. Taking several time periods, the corresponding serviceability loss can be found from the chart

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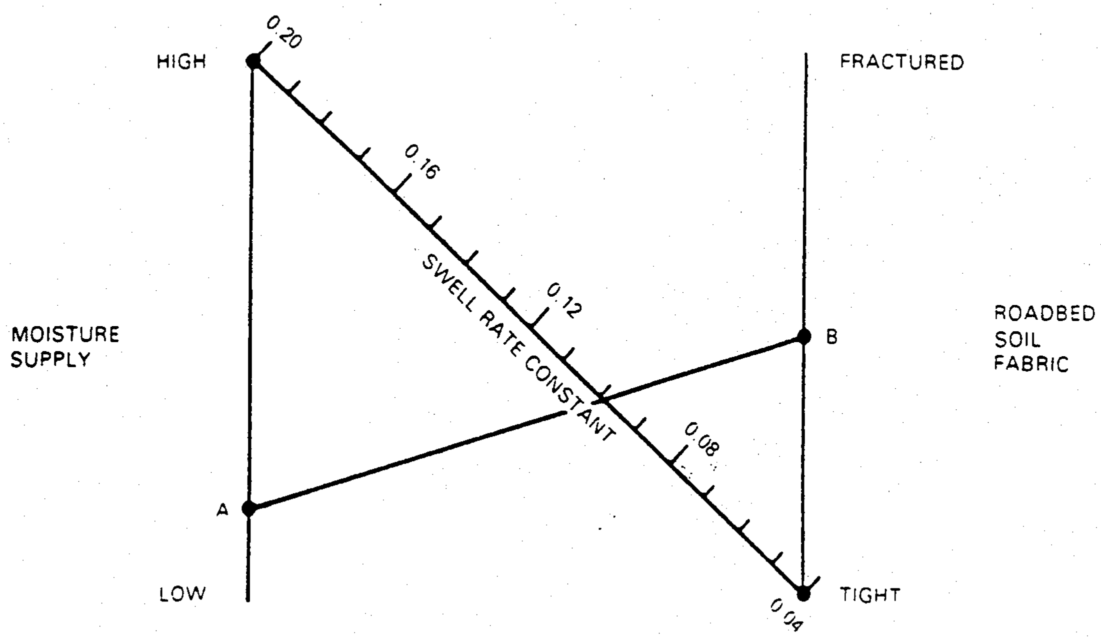
[6] American Association of State Highway and Transportation Officials (1993). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C, Figure G.3.

[7] American Association of State Highway and Transportation Officials (1993). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C, Table H.1.

[8] American Association of State Highway and Transportation Officials (1993). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C, Figure G.4.



NOTES:

a) LOW MOISTURE SUPPLY:

Low rainfall
Good drainage

b) HIGH MOISTURE SUPPLY

High rainfall
Poor drainage
Vicinity of culverts, bridge abutments, inlet leads

c) SOIL FABRIC CONDITIONS (self explanatory)

d) USE OF THE NONOGRAPH

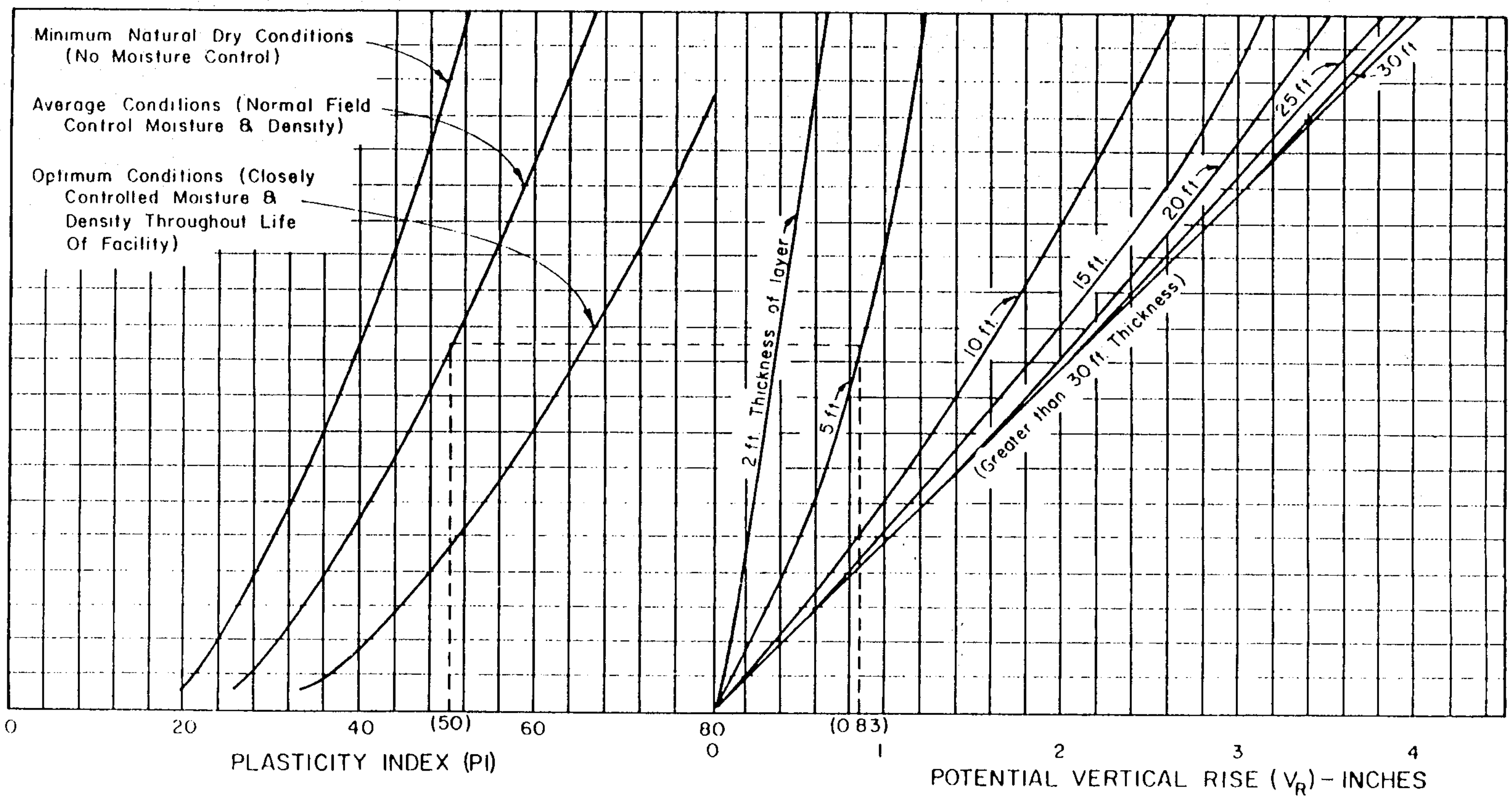
1) Select the appropriate moisture supply condition which may be somewhere between low and high (such as A).

2) Select the appropriate soil fabric (such as B). This scale must be developed by each individual agency.

3) Draw a straight line between the selected points (A to B)

4) Read swell rate constant from the diagonal axis (read 0.10)

Figure A.1: Nomograph for Estimating Swell Rate Constant



NOTES:

1. This figure is predicated upon the following assumptions.
   a. The subgrade soils for the thickness shown all are passing the No. 40 mesh sieve.
   b. The subgrade soil has a uniform moisture content and plasticity index throughout the layer thickness for the conditions shown.
   c. A surcharge pressure from 20 inches of overburden ($\pm$ 10 inches will have no material effect)
2. Calculations are required to determine V_R for other surcharge pressures.

Figure A.2: Chart for Estimating the Approximate Potential Vertical Rise

(1) Bore Hole Number	(2) Section Length (ft)	(3) Roadbed Thickness (ft)	(4) Soil Plasticity Index (PI)	(5) Moisture Condition	(6) Potential Vertical Rise (in.)	(7) Soil Fabric	(8) Swell Rate Constant
1	900	> 30	48	Optimum	0.82	rel. tight	0.07
2	1,200	> 30	56	"	1.34	"	"
3	800	> 30	67	"	2.20	"	"
4	1,000	> 30	15	"	0.00	"	0.10
5	1,000	> 30	46	"	0.70	"	0.07
6	1,100	> 30	62	"	1.86	"	"
7	1,000	> 30	65	"	2.00	"	"
8	900	> 30	71	"	2.60	"	"
9	1,200	> 30	38	"	0.28	"	"
10	800	> 30	60	"	1.80	"	"
11	900	> 30	19	"	0.00	"	0.10
12	1,200	> 30	51	"	1.04	"	0.07
13							
14							
15							
16							
Total	12,000						

Figure A.3: Table for Estimating Swell Parameters for Pavement Design

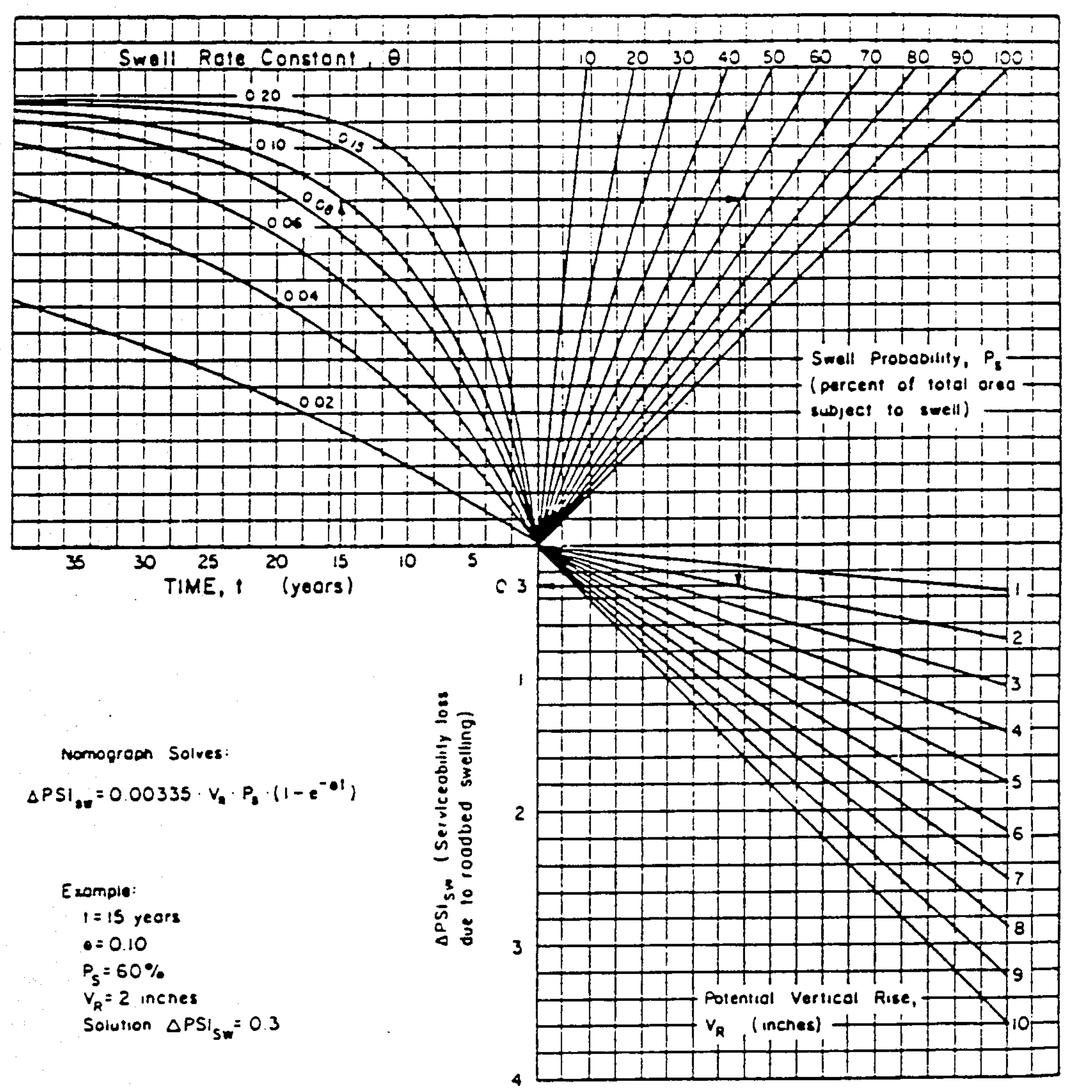


Figure A.4: Chart for Estimating Serviceability Loss Due to Roadbed Swelling and plotted as shown in Figure A.5[9].

With the swelling curve completed, the design of the pavement using either the flexible or rigid method can continue. The loss of serviceability due to swelling is used to determine the length of the performance period for each stage of construction.

A.2.2 Additional Pavement Design Features

When designing roads which will traverse expansive soils, there are a few design tips that can sometimes be incorporated which will reduce the swelling potential. The first is to avoid cut sections, particularly deep cuts, whenever possible. This will reduce the necessity of establishing new subgrade moisture equilibrium conditions beneath the new grade lines. It will also reduce the elastic rebound component of volume change.

Use of paved shoulders is also very important. A paved shoulder can help in reducing the infiltration of moisture from the sides. Under normal conditions AASHTO recommends four (4) feet left shoulders and ten (10) feet right shoulders. When expansive soils are involved, the left shoulder should be increased to between six (6) and eight (8) feet.

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[9] American Association of State Highway and Transportation Officials (1993). *AASHTO Guide for Design of Pavement Structures*, Washington, D.C, Figure 2.2.

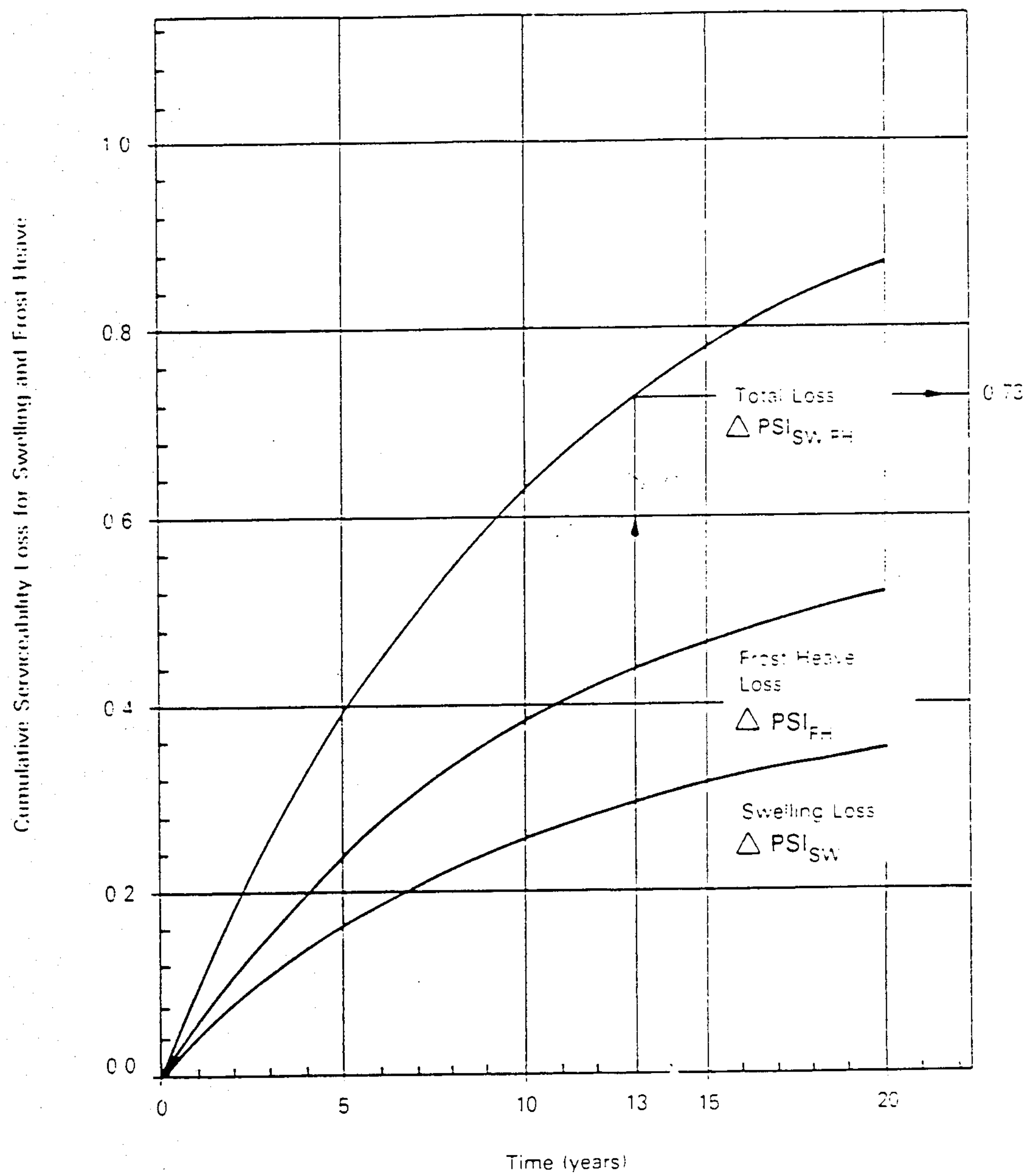

Figure A.5: Plot of Serviceability Loss Due to Swelling and Frost Heave

Minimization of the difference in physical characteristics (moisture content, density, and soil structure) are important at discontinuities such as cut/fill transitions, culverts, utility trenches, and pipeline crossing. Differences in these physical characteristics can lead to uneven volume changes along the length of the road which will cause localized distress of the pavement.

And finally, when using membranes for horizontal barriers, care should be taken to ensure that the membrane is continuous around culverts and overpasses. If guardrails, traffic signs, and reflector posts are used, special care should be taken so that the membrane is not punctured and thus provide an avenue for water to infiltrate into the subgrade.

A.3 Preventive Methods

Back in Table A.1, three alternatives were listed for minimizing volume change. While the first one usually cannot be influenced by the engineer, the last two can. Many different methods have been proposed and attempted. A few have proven useful in reducing the volume change in soils. While the individual methods may vary, the ultimate goal is to control the moisture content in the soil. And as mentioned before, without a change in moisture, there can be no change in volume.

The major sources of water that can cause the moisture content of the soil to change are listed in Table A.4.[10] In order to prevent the soil from swelling, the engineer can develop measures to prevent the water from reaching the pavement subgrade or alter the soil so that it is insensitive to the moisture change.

Table A.4: Sources of Water for Moisture Changes in Soil

1. Surface infiltration of precipitation through the pavement and/or verge slopes.
2. Upward vertical movement from the groundwater.
3. Lateral seepage from up-grade sources.
4. Condensation of water in porous base materials by temperature fluctuations.

The different preventive methods that have been developed can be broken into several categories. These methods can be accomplished either during and/or after construction, and as an addition to or as integral part of the pavement design. The soils can be mechanically or chemically altered, barriers built to restrict water movement and thus maintain in situ moisture content, or maintenance performed regularly. Table A.5 lists several of the more popular and successful methods

[10] U.S. Department of Transportation (1980). *EXPANSIVE SOILS in Highway Subgrades: Summary*, Federal Highway Administration, FHWA-TS-80-236, April 1980, pg. 3.

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that have been used, and a few will be discussed further in the sections below.

The choice of methods to be used depends on many factors. The swell potential of the soil and the type of highway to be designed will often dictate which method can be used. In addition, the allowable expense, time, and site conditions must also be considered.

Table A.5: Methods for Preventing Swell in Pavement Subgrades

Mechanical	Maintain Moisture Content
Ripping	Prewetting
Scarifying	Ponding
Subexcavation	Surface Drainage
Replacement	Subsurface Drainage
Surcharge Loading	Vertical Membranes
	Horizontal Membranes
Chemical	Maintenance
Lime Stabilization	Isolated Overlays
Organic Chemical Stabilization	Section Removal and Mudjacking
Electroosmosis and Base Exchange	Crack and Joint Sealing
	Proper Drainage Replacement

A.3.1 Mechanical Alterations

Mechanical alteration of soil consists of physically working the material so as to disturb the structure and then remolding it while maintaining water and density control.

Methods which fall into this category include ripping, scarifying, subexcavation, and surcharge loading. Each method has advantages and disadvantages, and their use is dictated by the swell potential and type of road.

Ripping or scarifying is the process of mixing or turning the top layer of soil. It can be accomplished with bulldozers with rear teeth attachments, graders, or similar equipment. The depth of mixing is usually two feet or less, so it is considered a minimal effort treatment. The soil can than be recompacted, usually maintaining between 92-95% of the maximum dry density and optimum moisture content. This method is best suited for secondary highways which can tolerate larger deformations or for primary highways where the soil has low swell potential.

Subexcavation and replacement require the removal and replacement of expansive subgrade soils. This method requires more effort, and has an average influence range of between four to six feet. The excavation is than backfilled with a non-expansive soil. The excavated soil can be used for replacement if it is mechanically or chemically altered. This method is best suited for high potential swell materials that have a initial moisture content that is less than the optimum moisture content by at least ten percent. Costs for this method can be high depending on the quantity of expansive soil that must be excavated and replaced.

When expansive soils are involved, granular soils should never be used as backfill material. Granular soil allow surface water access to the in situ materials and thus defeat the purpose. This applies not only to subexcavation, but also to fill used around conduits, pipes, culverts, and trenches. Instead, cohesive and preferably nonswelling soils should be used.

The last mechanical alteration to be mentioned here is surcharge loading. By applying a load to the soil material, the expected volume change can be counteracted. This method is best suited for secondary highways or low swell potential soils.

A.3.2 Chemical Alterations

Chemical alteration is the modification of the clay mineral or clay-water combination by the addition of chemical compounds. The modification can be caused by base exchange, increase in ionic concentration in the free water, chemical modification of the clay mineral, or any other physical-chemical reaction. The result of the alteration is a reduction in the swell potential of the soil. Chemical alteration is dependent on several factors including grain size distribution, amount of clay material, method of adding the chemical, amount and cure time of the chemical, and the water content of the soil-chemical mixture.

Lime is the most widely used and effective additive for modification of expansive clays. The major limitation to using lime is applying the chemical to sufficient depths. Generally, mix-in-place techniques are limited to 8-12 inches in depth, with specialized equipment allowing depths of two feet to be reached.

Lime modification of soil is well suited for preconstruction fill and backfill operations. During fill operations, the lime can be applied and mixed in the borrow area. For subexcavation, the lime can be applied to the backfill stockpiles and mixed before replacement. Very little additional effort and time are required for these methods.

Two postconstruction treatments of expansive soil by lime have been used for highways. The two methods, drill-hole lime and lime slurry pressure injection (LSPI), are both controversial as to their mechanisms and success. The two types of lime most used for soil treatment are quicklime (CaO) and hydrated lime ($Ca(OH)_2$).

For the drill-hole lime technique, holes are drilled through the pavement. The holes are than backfilled with either lime or a lime slurry-sand mixture. The lime than diffuses away from the holes into the expansive soil and reacts with the clay minerals by ion exchange.

LSPI requires the pumping of a lime slurry into the expansive soil. Hollow injection rods are used to pump the lime slurry into the soil at pressures up to 200 psi. The original idea behind this was that the pressure would cause the lime to diffuse through the soil at a greater rate than by regular diffusion. In fact, the lime slurry moves through the soil by cracks and fissures and very little diffusion actually occurs.

A.3.3 Maintaining Moisture Content

As shown in Section A.1, soils that are near or above the optimum water content have less swelling potential. So increasing the moisture content to the optimum moisture content before construction and maintaining the moisture content during and after construction is very important. There are several methods that can be used on highways to maintain the desired moisture content.

Prewetting or ponding of expansive soil is the process of saturating the subgrade with water before construction begins. The purpose is to increase the in situ moisture content and thus dissipate the potential swell prior to construction of the pavement. Logistics such as construction scheduling, construction of retaining dikes, water supply, drainage after completion, and lime treating the saturated surface are all major points of consideration for the selection and use of this method. It is just as important that the in situ moisture content be maintained after construction and does not go down, or shrinkage will occur which is just as damaging to pavements as swelling.

Membranes or barriers can be used to successfully minimize subgrade moisture variations and the associated volume change of expansive soils in highway subgrades. Membranes can be horizontal, vertical, or both, and they perform best where the major influence on subgrade moisture is from surface infiltration. Table A.6[11] lists four preconstruction applications of waterproofing membranes that can be used.

Table A.6: Preconstruction Applications of Waterproofing Membranes

1. Continuous horizontal membrane over the entire subgrade and ditches.
2. Full-depth asphalt pavement with a sprayed asphalt or synthetic fabric membrane beneath the ditch.
3. Full-depth asphalt pavement with paved ditches in cut sections.
4. Vertical synthetic fabric membrane cutoffs.

Best performance of horizontal membranes occurs when applied over the entire subgrade section, down the verge slopes, and up the back slope a specified distance. On interstate and primary roads, a continuous membrane (back slope to back slope) should be applied for all cut sections on

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[11] U.S. Department of Transportation (1980). *EXPANSIVE SOILS in Highway Subgrades: Summary*, Federal Highway Administration, FHWA-TS-80-236, April 1980, pg. 21.

expansive soils. When filling is required, the membrane should extend down the fill slope to assure that moisture cannot reach the compacted subgrade material. When possible, the median in divided four-lane highways should also be included in the membrane application.

Use of vertical membrane cutoffs has been limited mainly because of construction problems and availability of a strong enough fabric to withstand placement and service requirements. The ideal depth of the vertical membrane cutoff should be to the bottom of the active zone. It has been found that membrane cutoffs of 2 to 3 ft will generally not provide enough positive influence in stopping water infiltration to justify their use.

Drains, both surface and subsurface, can be used to maintain moisture content by diverting water away from the subgrade. Surface drains mainly consist of ditches along the shoulders of the roadway. Proper slope design of the pavement and shoulders is important to prevent water from pooling. It is widely known that moisture variations near the shoulders are significant and should be avoided because they lead to deterioration of the pavement.

Subsurface drains can be used in conjunction with vertical membranes, when the soil is fractured or fissured, or the groundwater table is near the active zone. When used with vertical membranes, drains can be used just below and to the outside of the membrane to collect the diverted water.

Special attention needs to be taken when designing subsurface drains to prevent them from clogging with clay and silt particles.

A.3.4 Maintenance

Costs for maintenance of roads on expansive soil subgrades run roughly ten times greater than for the same type of pavement constructed on a suitable, nonexpansive subgrades. But the consequence of not performing these routine repairs will result in higher replacement costs down the line. Probably the most significant factor leading to volume change of expansive soils in subgrades is the lack of maintaining proper drainage. Keeping drainage ditches clear and maintaining proper slopes on the pavement and shoulders will go a long way towards limiting expansion of the subgrade soil. Maintenance workers should be observant for signs of inadequate drainage such as ponded water in the ditch, soft spots in the ditch or verge slopes, or the presence of plants or trees that grow best in saturated or submerged environments like willows and cattails.

Deep rooted vegetation near roads should be avoided when possible. Root systems of some bushes and trees can cause differential changes to the moisture content, thus causing localized swelling or shrinking to the subgrade soil. Instead, the shoulders should be sloped and covered with grass or preferably an impervious barrier such as asphalt.

Sealing of cracks and joints in the pavement should be an integral part of any highway maintenance program. This simple measure can help minimize the infiltration of moisture into the subgrade through the pavement. Many compounds are available that can be used to seal the cracks and prevent further infiltration.

Remedial maintenance can be performed once signs of distress from swelling begin to show. These techniques do not solve the swelling problem, they only improve the riding quality of the road and are therefore just cosmetic repairs. Isolated overlays can be used to level distortions while removal and replacement of sections of pavement and subgrade can be used to remove distortions in flexible pavements. Mudjacking can be used to level distortions in concrete slabs.

APPENDIX B

ESEM PICTURES OF SOILS IN EVALUATION PROGRAM

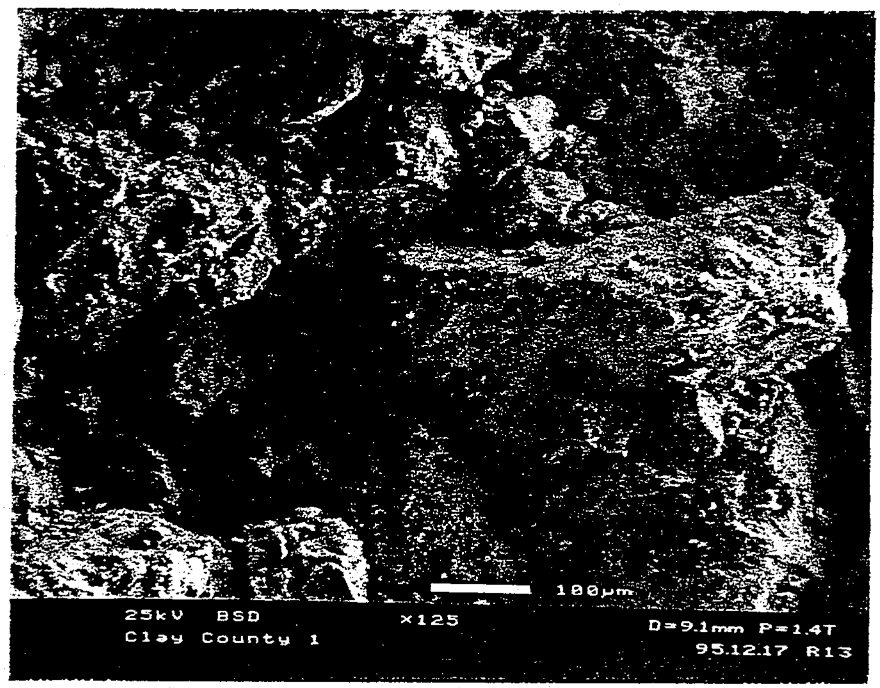

Figure B.1: ESEM Picture of Clay County Soil

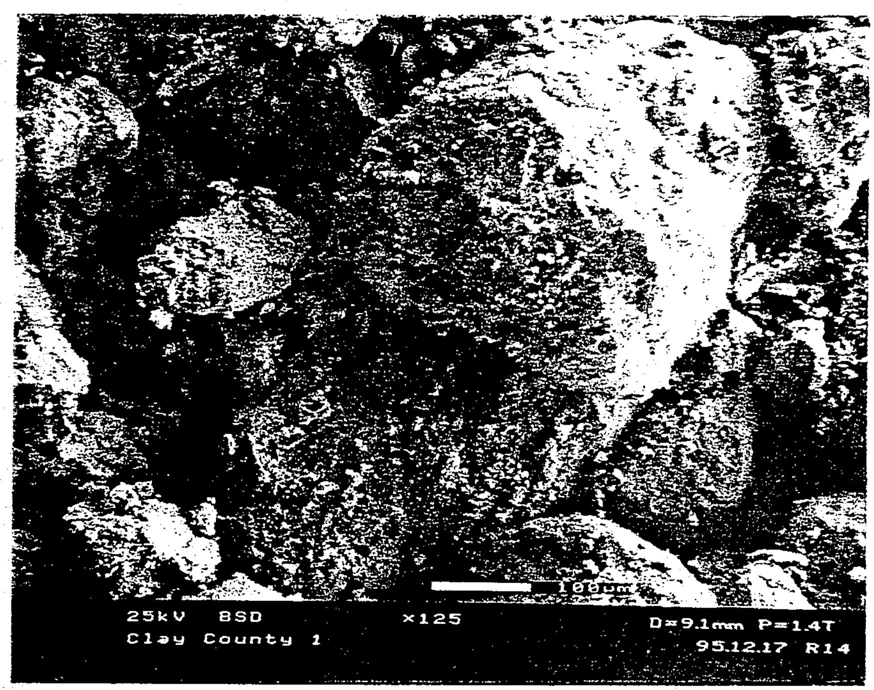

Figure B.2: ESEM Picture of Clay County Soil

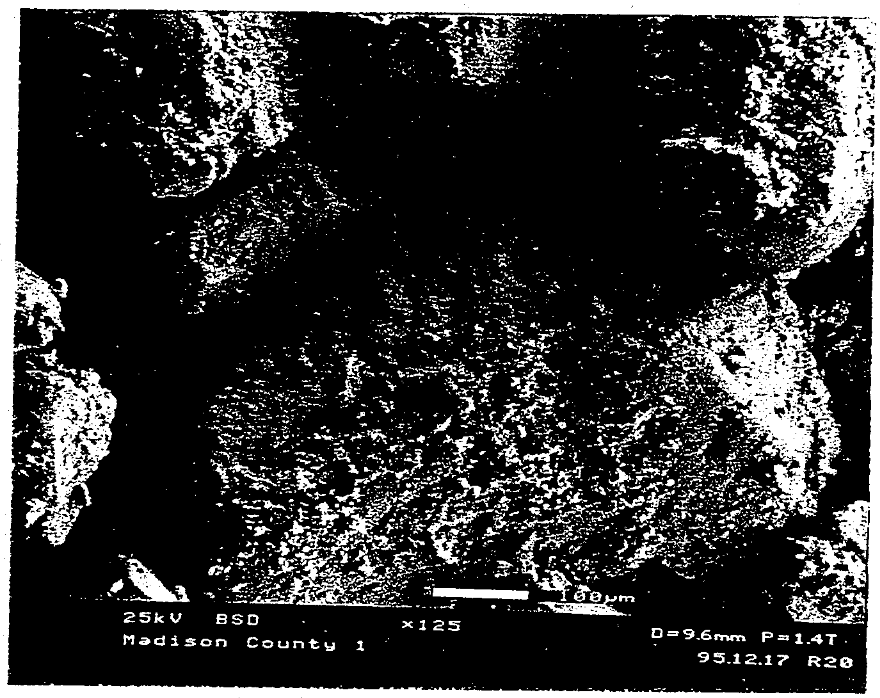

Figure B.3: ESEM Picture of Madison County Soil

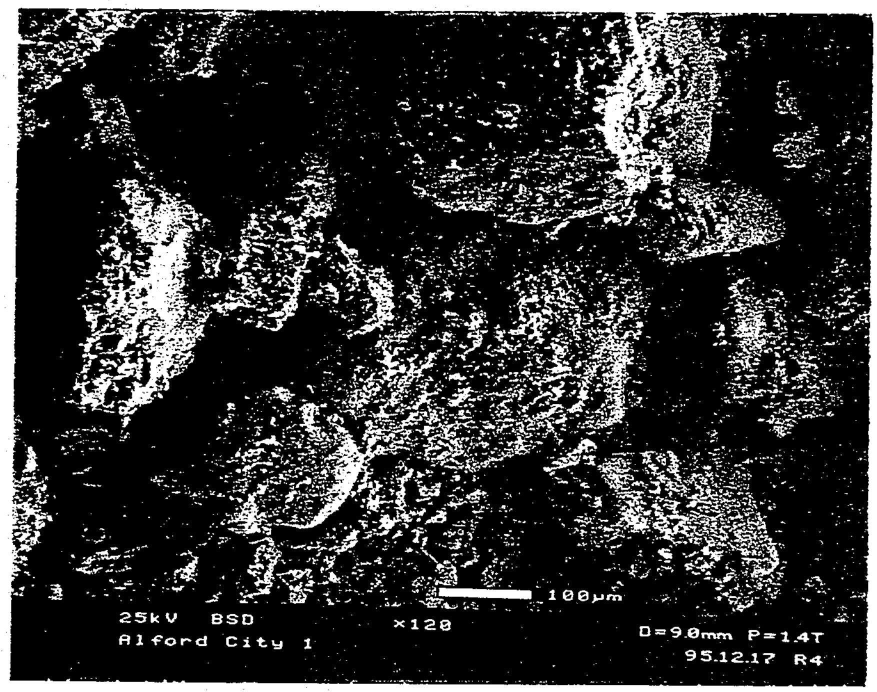

Figure B.4: ESEM Picture of Alford City Soil

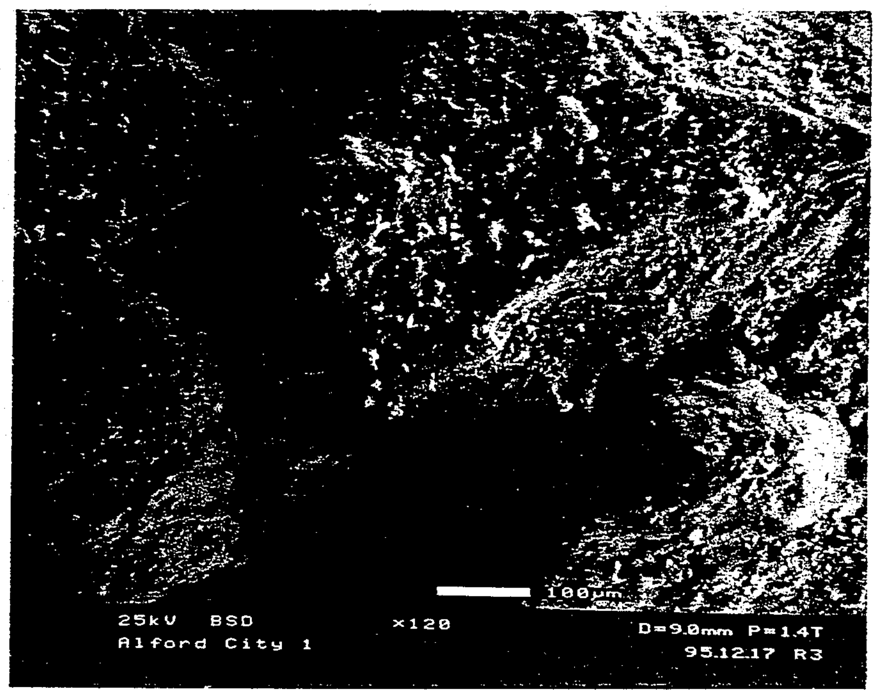

Figure B.5: ESEM Picture of Alford City Soil

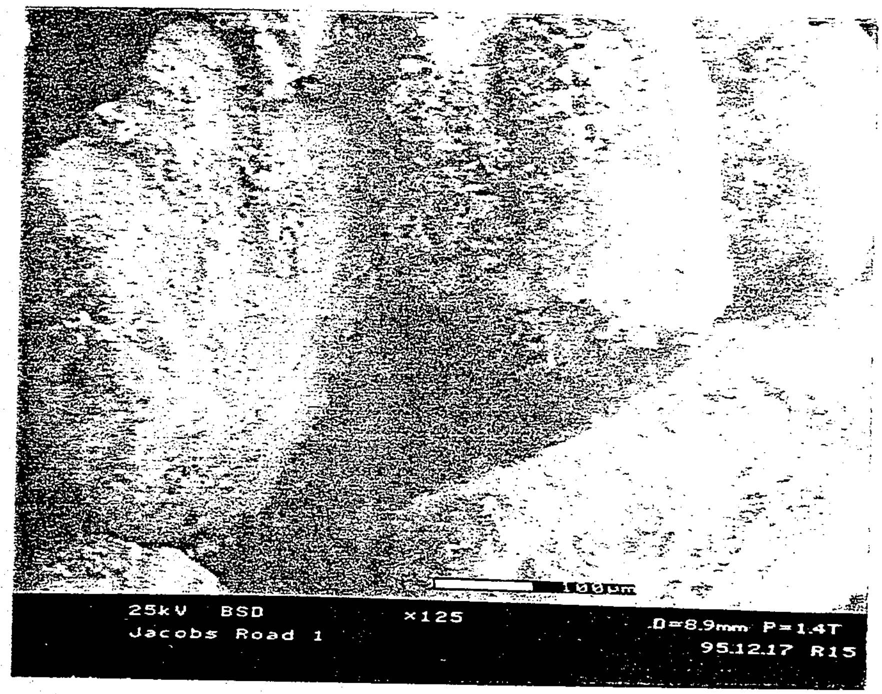

Figure B.6: ESEM Picture of Jacobs Road Soil

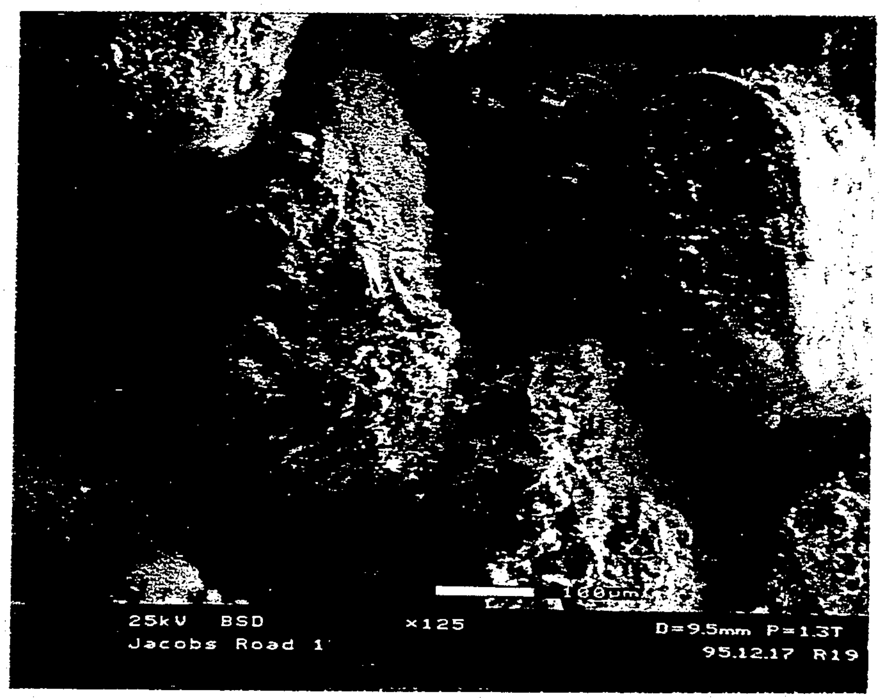

Figure B.7: ESEM Picture of Jacobs Road Soil

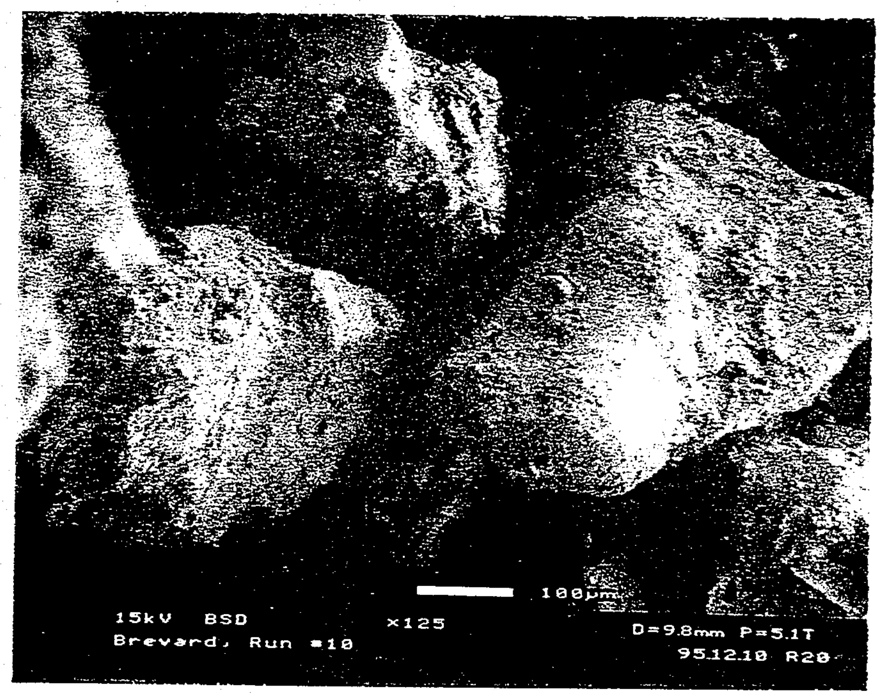

Figure B.8: ESEM Picture of Brevard County Soil

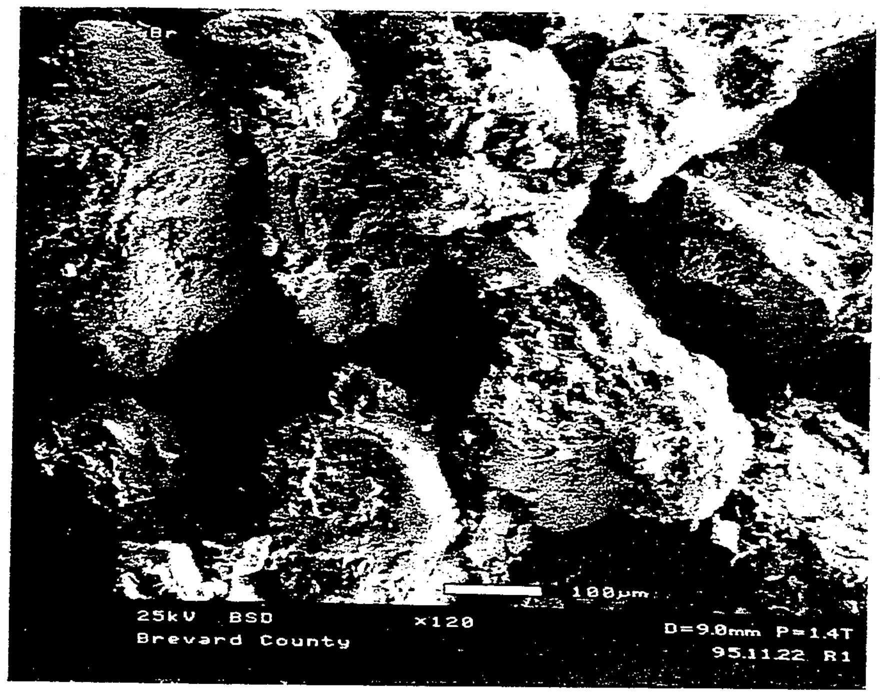

Figure B.9: ESEM Picture of Brevard County Soil

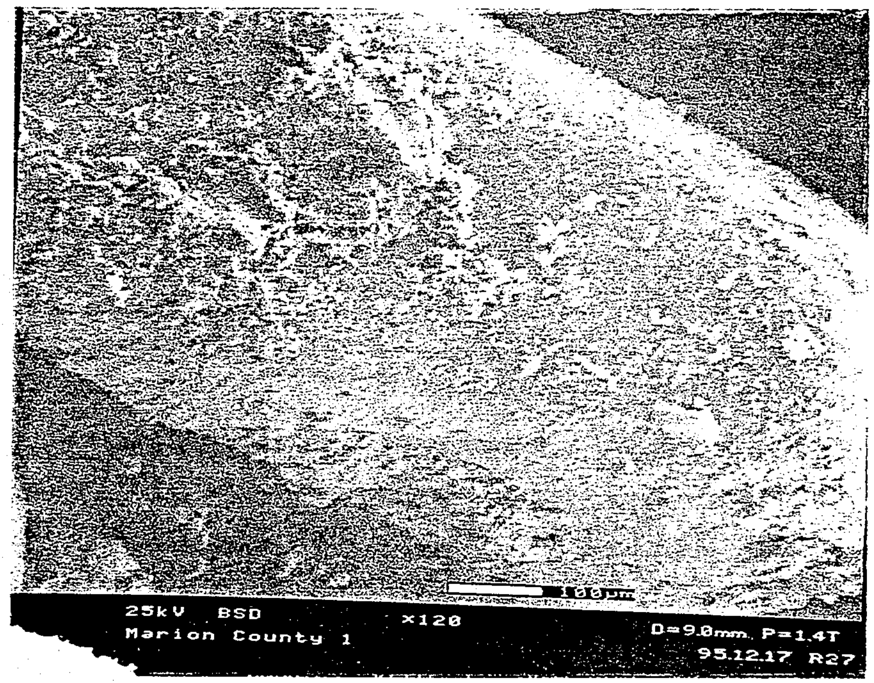

Figure B.10: ESEM Picture of Marion County Soil

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