FINAL REPORT

For the Florida Department of Transportation

Design Highwater Clearances For Highway Pavements

Volume I

FDOT Research Contract No.: BD-543-13
FSU Project No.: OMNI 017032

by

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August 2008

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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

pounds per cubic inch = 16.02 kilograms per meters cubed

Technical Report Documentation Page

1. Report No.
FL/DOT/RMC/BD-543-13

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
Design Highwater Clearances For Highway Pavements

5. Report Date
August 2008

6. Performing Organization Code

7. Author(s)
W. V. Ping and Ching-Chin Ling

8. Performing Organization Report No.
FSU C&G No. OMNI 017032

9. Performing Organization Name and Address
Department of Civil & Environmental Engineering
FAMU-FSU College of Engineering
2525 Pottsdamer Street
Tallahassee, Florida 32310-6046

10. Work Unit No. (TRAIS)

11. Contract or Grant No.
FDOT BD453-13

12. Sponsoring Agency Name and Address
Florida Department of Transportation
Research Center, MS30
605 Suwannee Street
Tallahassee, Florida 32399-0450

13. Type of Report and Period Covered
Final Report
October 2005 - April 2008

14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the Federal Highway Administration, U.S. Department of Transportation

16. Abstract

High groundwater table exerts detrimental effects on the roadway base and the whole pavement. Base clearance guidelines have been developed to prevent water from entering the pavement system in order to reduce its detrimental effects. In these guidelines a minimum height, the clearance, between a groundwater level and a particular elevation within the pavement system is specified. This report presents an experimental study to evaluate the effects of high groundwater and moisture on determining pavement base clearance for granular subgrades. Full-scale in-lab test-pit tests were conducted to simulate pavement profile and vehicle dynamic impact on the pavement. Eleven types of subgrade were tested for this study. From the test, using layer theory, the results of the resilient modulus for each layer (layer resilient modulus) can be compared with the resilient modulus results from laboratory test. The dominant factor or factors of the effect of moisture to resilient modulus will be discussed.

The results showed that a 36-in. base clearance was considered adequate for the base protection of most of the A-3 and A-2-4 subgrades against high groundwater tables. The lab resilient modulus and layer resilient modulus had the same trend for each soil according to the moisture content change. The percent of fines or the percent of clays of subgrade soil was not a good indicator to measure the influence of moisture effect on the resilient modulus. The coefficient of uniformity and coefficient of curvature of the subgrade gradations, which better represent the whole shape of the gradation curve, are better indicators of the effect of moisture on modulus. The SR70 A-2-4 (14% fines), A-2-4 (30% fines), Oolite A-1, Branch A-2-4 (23% fines) soils were extremely susceptible to the change of high groundwater table; the equivalent modulus reduction rates were more than 50% for lowering the base clearance from 2 ft. to 0 ft. For the Levy A-3 (4%), SR70 A-3 (8%), A-2-4 (12%), A-2-4 (24%), and Spring Cemetery A-2-4 (15%) soils, the reduction rates were also very significant for the base clearance from 2 ft. to 0 ft. with the equivalent modulus reduction rates in the range of 21% to 45%.

17. Key Words
pavement, subgrade, resilient modulus, database, soil laboratory

18. Distribution Statement
This document is available to the public through the National Technical Information Service, Springfield, Virginia, 22161

19. Security Classif. (of this report)
Unclassified

20. Security Classif. (of this page)
Unclassified

21. No. of Pages
473

22. Price

Form DOT F 1700.7 (8-72)
PF V2.1, 12/13/93

Reproduction of completed page authorized

ACKNOWLEDGEMENTS

Funding for this research was provided by the Florida Department of Transportation (FDOT) and Federal Highway Administration (FHWA) through the Research Center of the FDOT. This research was initiated by FDOT District 4 Drainage Design Office. Bruce Dietrich and Rick Renna were the FDOT project managers. The research study was coordinated by the pavement design, drainage design, and geotechnical design offices of FDOT Central Office in Tallahassee.

The field monitoring program was conducted by Ron Lewis and Rick Venick of the FDOT State Materials Office. The field logistics were provided by the District 4 Materials and Maintenance Offices.

The laboratory testing program was coordinated by David Horhota with the Geotehnical Materials Section of the State Materials Office. Rick Venick and Ron Lewis conducted the test-pit study program.

The FDOT Research Center, through the assistance of Richard Long and his staff, provided financial and contractual support to the project.

EXECUTIVE SUMMARY

High groundwater table exerts detrimental effects on the roadway base and the whole pavement. Base clearance guidelines have been developed to prevent water from entering the pavement system in order to reduce its detrimental effects. In these guidelines a minimum height, the clearance, between a groundwater level and a particular elevation within the pavement system is specified. This report presents an experimental study to evaluate the effects of high groundwater and moisture on determining pavement base clearance for granular subgrades. Full-scale in-lab test-pit tests were conducted to simulate pavement profile and vehicle dynamic impact on the pavement. Eleven types of subgrade were tested for this study. From the test, using layer theory, the results of the resilient modulus for each layer (layer resilient modulus) can be compared with the resilient modulus results from laboratory test. The dominant factor or factors of the effect of moisture to resilient modulus will be discussed.

TEST SUBGRADE MATERIALS

The soils under investigation in this research were the typical A-3 and A-2-4 subgrade materials in use in the State of Florida. A total of eleven types of soil were investigated.

The materials were further divided into three groups according to the test schedule as follows:

(I) Phase I: (From Dec. 1999 to Feb. 2000)

1. Levy A-3 soil – 4% fines
2. SR70 A-3 soil – 8% fines
3. SR70 A-2-4 soil – 14% fines

(II) Phase II: (From Jun. 2000 to Mar. 2001)

4. A-2-4 soil - 12% fines
5. A-2-4 soil - 20% fines
6. A-2-4 soil - 24% fines
7. A-2-4 soil - 30% fines
8. Miami Oolite A-1 soil

(III) Phase III: (From Jul. 2005 to Apr. 2007)

9. Spring Cemetery A-2-4 soil - 15% fines
10. Branch A-2-4 soil - 23% fines
11. Iron Bridge A-2-6/A-2-4 soil - 31% fines

The Iron Bridge soil is a borderline soil between A-2-4 and A-2-6, and is classified as an A-2-6 soil throughout the report.

LABORATORY RESILIENT MODULUS TESTS

The tests were performed using the AASHTO T292-91I test standard for the Phase I and II soils, with both middle-half and full-length LVDT position measurements, while the tests for the Phase III soils were conducted using the AASHTO T307-99 test standard with only full-length LVDT position measurement. The resilient modulus tests were performed at the dry, optimum, and soaked conditions for the Phase I and II soils, while only at the optimum water content for the Phase III soils. As for the compaction effort, the 100% Modified Proctor was used for the Phases I and Phase II soils, while the 100% Standard Proctor was used for the Phase III soils. The resilient modulus data obtained from the bulk stress of 75.8 kPa (11 psi), which was three times the confining pressure of 13.8 kPa (2 psi) plus one deviator stress of 34.5 kPa (5 psi), were used for analysis.

TEST-PIT EQUIVALENT MODULUS TESTS

A full-scale simulation was conducted to evaluate the effect of a high groundwater level on the modulus of the subgrade soil in the test-pit experimental program. With adjustment of the groundwater level in the subgrade, the dynamic plate load tests were performed to measure the flexible deformations; from this, the equivalent moduli of the materials in the test pit were derived.

The equivalent moduli were, however, measured for the composite layers of subgrade and embankment under the plate loading, with or without an additional limerock base layer. A layer system using KENLAYER was setup to estimate the resilient modulus value for the individual subgrade layer under the high groundwater level.

CONCLUSIONS

1. Based on laboratory resilient modulus test, the resilient modulus value of each subgrade soil decreased with an increase in moisture content. However, the rates of reduction for these soils were not at the same level. The SR70 A-2-4 (14%) and Oolite A-1 soils were very sensitive to the change of moisture content from the optimum to soaked conditions. These two soils had the reduction rates of 26% and 31%. The other soil types were not as sensitive to the moisture content change (with reduction rates lower than 20%) as those two soils.
2. The moisture content in subgrade soil was a major factor affecting the resilient modulus. In addition, the test results showed that other factors including dry unit weight, LBR, percent of clay, coefficient of uniformity (C_u) and coefficient of curvature (C_c) also significantly affected the resilient modulus. The C_u and C_c were considered as two good indicators for correlating the moisture sensitivity of granular soils.
3. No relationship existed between the reduction rate and the percentage of fines in soil. The percentage of fines was not a good indicator for categorizing the soils in terms of the sensitivity of resilient modulus to moisture effect. However, the percentage of fines was a good indicator to predict the permeability properties of soil. The permeability value under

saturated condition decreased with an increase in percentage of fines.

4. Based on the test-pit test results, the A-2-4 (24%) soil was very sensitive to the change of high groundwater level from +0.0 in. to +12.0 in. above the embankment (i.e., lowering base clearance from 3 ft. to 2 ft.), the plate load equivalent modulus values were reduced 28%.
5. The SR70 A-2-4 (14%), A-2-4 (30%), Oolite A-1, Branch A-2-4 (23%), and Iron Bridge A-2-6 (31%) soils were extremely sensitive to the change of high groundwater level from +12.0 in. to +36.0 in. above the embankment (i.e., lowering base clearance from 2 ft. to 0 ft.). The plate load equivalent modulus reduction rates were more than 50%. For the Levy A-3 (4%), SR70 A-3 (8%), A-2-4 (12%,) A-2-4 (24%), and Spring Cemetery A-2-4 (15%) soils, the reduction rates were also very significant for the base clearance from 2 ft. to 0 ft. with the plate load equivalent modulus reduction rates in the range of 21% to 45%. The A-2-4 (20%) soil was the least sensitive soil in response to the change of high groundwater level with the plate load equivalent modulus reduction rate about 17%.
6. Adding a 5-in. limerock base layer was very beneficial to the pavement resistance, and the equivalent modulus values were almost doubled. The added limerock base layer certainly improved the dynamic performance of the pavement.

7. Comparing the laboratory resilient modulus results with the subgrade layer modulus values from test-pit, the modulus values were generally within the same range from the same type of soil. The laboratory resilient modulus value at optimum condition was lower than the layer modulus (about 50% to 70%) for the same type of soil tested in the test-pit with a base clearance of two feet (24 in.), except that the SR70 A-2-4 (14%) soil had about the same modulus for both tests.
8. When a pavement design is prepared, pavement designers and geotechnical engineers typically do not know the exact soil that will be used for the embankment. Due to the lack of a direct relationship between percent fines and modulus reduction and the high variability of the moduli reductions, cautions should be exercised when reducing base clearance below three feet. It was evident in this research that when base clearances were reduced to two feet, the plate load equivalent modulus reductions were up to 28%. When base clearances were further reduced to one foot, the plate load equivalent modulus reductions were up to 43%. Furthermore, with base clearances at zero foot, the modulus reductions were up to 81%.
9. The results of the case study indicated that for some sensitive soils, such as SR70 A-2-4 (14%) and A-2-4 (30%) soils, an increase of high groundwater table would demand a significant

increase of the required thickness of asphalt concrete layer in order to have the same quality of pavement performance. The most severe condition was for base clearance reduced from two feet to zero foot. The other subgrade soils also required some increase of asphalt concrete layer thickness.

10. In areas with high groundwater levels, adequate base clearance should be maintained to minimize the moisture damage and to achieve quality performance of the pavement.

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CHAPTER 1
INTRODUCTION

1.1 PROBLEM STATEMENT

A high groundwater table exerts detrimental effects on the roadway base and the whole pavement. The determination of design high groundwater elevation is one of the most important steps towards setting up grade lines in a roadway design. The pavement system must be designed in such a way that water is prevented from entering the places where it can cause damage. The Florida Department of Transportation (FDOT) has developed high groundwater clearance guidelines to prevent water from entering the pavement system in order to reduce its detrimental effects. In these guidelines a minimum height, the clearance, between a groundwater level and a particular elevation within the pavement system is specified. The guidelines are intended to satisfy two concerns: 1) to prevent potential damages to the roadway base due to groundwater saturation or high moisture content from capillary suction; 2) to achieve the required compaction and stability during construction operations.

Despite the focus on these concerns, the prevailing guidelines (AASHTO, 1993) neglect the fact that each roadway is built with a different type of subgrade material. Subgrade materials in construction are required to be the selected materials (such as A-3, A-2-4 soils and Oolite in Florida), which cover a wide range of soils. There can be different geotechnical properties associated with different subgrade soils such as permeability and suction in unsaturated state, which are critical for capillary behavior.

In addition, these guidelines do not take into account the effect of dynamic loadings and some of the design criteria such as the resilient modulus of the subgrade materials. As a result, the prevailing guidelines could be overly conservative in some cases, while in other cases the specified minimum base clearance could be inadequate. In view of this, it is important to evaluate the effects of high groundwater level on pavement performance and the minimum base clearances for establishing the roadway grade lines. In addition, experimental data are needed to justify the design guidelines.

1.2 SCOPE OF STUDY

The primary objective of this study was to evaluate the effect of high groundwater level on pavement subgrade performance. Eleven typical subgrade soils used for pavement construction in Florida (including A-3, A-2-4 soils and Oolite) were obtained for evaluation. A full-scale laboratory evaluation of the subgrade performance was conducted in a test-pit facility. The subgrade and base layer profile of a full-scale flexible pavement system was simulated in the test-pit facility. Moisture conditions were manipulated by raising and lowering the groundwater level in the test-pit. The subgrade materials were tested under various moisture conditions that simulated different field conditions. The effect of the dynamic loadings was evaluated using the repeated plate load in the test-pit test.

In conjunction with the full-scale test-pit program, a laboratory triaxial testing program was carried out to evaluate the resilient modulus of subgrade materials. The effect of moisture on the resilient properties of subgrade materials was evaluated using soil specimens under dry or soaked conditions for the resilient modulus tests. In addition, a limited field monitoring program was also conducted at SR70 (near Fort Pierce, Florida) to evaluate the moisture profile of subgrade soils under the influence of the seasonal variation of precipitation and air temperature in the field.

1.3 REPORT ORGANIZATION

This report summarizes the experimental program, test results, and analyses of the study to evaluate the effect of high groundwater level on the pavement performance of eleven typical Florida subgrade soils. The background and objectives of this research study are presented in this chapter. A literature review of the concepts and research related to the design high groundwater level clearance is summarized in Chapter 2. The experimental program, including a description of test equipment, test setup and test procedure for full-scale test-pit and laboratory triaxial tests, is presented in Chapter 3. Chapter 4 provides the results of laboratory resilient modulus test, suction test and permeability test. The experimental results of the test-pit test are summarized in Chapter 5. The analysis of laboratory resilient modulus test results is established and discussed in Chapter 6. The analysis of test-pit experimental results is discussed in Chapter 7. Chapter 8 presents the case study of SR70 and the field monitoring program. The analysis of the effect of high groundwater level is summarized in Chapter 9. Finally, conclusions and recommendations of this research study are presented in Chapter 10.

CHAPTER 2
LITERATURE REVIEW

2.1 SOURCES OF WATER IN PAVEMENT

There are many sources of the water that reaches the pavement structure and its immediate vicinity. To evaluate the various sources, the pavement designer should consider the entire profile and cross section of the highway as well as the surface and subsurface drainage systems that are to be used for the operation and structural integrity of the overall facility. The pavement structure designer, who may not be directly involved with all aspects of the facility, cannot predict the possible sources and amounts of water without knowledge of the surface and subsurface drainage geometry.

Free water enters the structural section and the adjacent area from many sources. Cedergren et al. (1972) state that the most abundant and often overlooked source is undoubtedly atmospheric precipitation, by which surface water is supplied from rain (usually the largest amount), snow, hail, condensing mist, dew and melting ice. This water reaches the structural section in several ways:

1. Cracks in the pavement - New pavements can be constructed so that they are virtually impermeable, but they cannot be constructed without joints or without cracks forming well before the desired life of the pavement structure is attained.

2. Infiltration through the shoulders.

3. Infiltration from the side ditches.

4. Melting of an ice layer from a frost area during the thawing cycle.

5. Free water from pavement base - If the base is not properly drained, it may act as a source of free water for the subbase and subgrade.

6. High groundwater table.

7. Condensation of water vapor (small amounts).

The first five sources can be particularly significant if the surface drainage is not properly designed or maintained.

Any free-water surface can act as a source of capillary water, which will move from the free-water surface when a capillary potential exists. The distance it moves depends primarily on the pore-size distribution in the soil. Capillary water can become free water and vice versa. These changes may be affected by fluctuations in temperature and the pore-size distribution of the soil.

Free-water surfaces and capillary fringe water are both sources for water vapor. Under shifting temperature and pressure conditions, water vapor can change back to either free water or capillary water.

2.2 ESTABLISHING FREE-WATER SURFACE IN SUBGRADE

By using the basic data of the original groundwater profile and the proposed highway geometry, surface drainage facilities, and subsurface drainage facilities, the free-water surface in the vicinity of the pavement can be predicted. Techniques for making these predictions are available. The location of the seasonal free-water surface is important because it affects the equilibrium moisture content, the bearing capacity and the frost susceptibility of the subgrade, and the rate at which the infiltrated water can be drained from the base and subbase materials.

Recommendations on the minimum depth from the pavement surface to the free-water surface vary. Typical criteria are: Massachusetts-7 ft (2.1 m); Michigan and Minnesota-5 ft(1.5 m); Saskatchewan-8 ft to 12 ft(2.4 m to 3.7 m) and Nebraska-3 ft to 4 ft(0.9 m to 1.2 m) in granular materials and 7 ft(2.1 m) in cohesive soils.

Investigators in Germany concluded that a critical depth is 2 m (6.6 ft) below the pavement surface. Researchers in Sweden found significant reduction in bearing capacity when the water table is raised to within 70 cm (27 in.) of the surface, and further reduction when it is raised to within 30 cm (11 in.) of the surface. This research in Sweden is particularly significant because it shows the effect of the groundwater table on subgrade strength independent of its relationship with frost-heave problems. This study was conducted using both a gravel base and a crushed-stone base on a frost-susceptible silt subgrade. No details on gradation or permeability were given.

Although no specific criteria regarding these variables were found, the critical depth to the water table is probably a function of subgrade strength, subgrade permeability, subgrade capillarity and the ratio of the design vertical live load stress to the live load plus dead load vertical stress. These items are important because the strength of the subgrade must be assessed at the effective stress level (i.e., total stress minus pore pressure), whereas the driving force to cause failure is at the total stress level.

2.3 RESILIENT MODULUS OF SOILS AND AFFECTING FACTORS

The resilient modulus is defined as the deviator dynamic stress (due to moving vehicular traffic) divided by the resilient axial (recoverable) strain. This concept is derived from the fact that the major component of deformation induced into a pavement structure under the traffic loading is not associated with plastic deformation or permanent deformation, but with elastic or resilient deformation. Thus, the resilient modulus is considered to be a necessary variable for determining the stress-strain characteristics of pavement structures subjected to traffic loading.

The resilient modulus of unstabilized granular base and subgrade soils is highly dependent upon the stress state to which the material is subjected within the pavement in addition to other variables. As a result, constitutive models including the effect of stress state must be used to present laboratory resilient modulus test results, in a form suitable for use in pavement design. The resilient modulus depends on deviator stress and confining stress. Two popular and simple regression models are presented as follows:

1. When modulus is dependent on bulk stress:

$$M_r = k_1 \theta^{k_2} \tag{2-1}$$

2. When modulus is dependent on confining pressure:

$$M_r = k_3 \sigma_3^{k_4} \tag{2-2}$$

Where,

θ = bulk stress, sum of the principal stresses, $(\sigma_1 + \sigma_2 + \sigma_3)$

σ_3 = confining pressure or minor principal stress

k_1, k_2, k_3, k_4 = regression constants

Many factors influence the resilient modulus of soils. A brief review of the significant factors is discussed in this chapter. Moisture is one of the factors affecting the modulus of soils. A thorough review of the literature concerning the effect of moisture is provided accordingly.

The factors that influence the resilient modulus of soils include the following: soil type, soil properties, dry unit weight, water content, stain level, test procedures and size effect. A brief review of the significant observations with regard to these factors is discussed in the following sections.

2.3.1 Soil Types

The resilient modulus is significantly influenced by the type of pavement soils. For instance, Chen et al. (1994) investigated the variability of resilient moduli due to aggregate type. The AASHTO T 292-91I test procedure was used to conduct tests on six selected aggregate types of soils. Conclusions show that for a given gradation, the differences in M_R values due to aggregate sources were between 20 to 50%.

2.3.2 Soil Properties

The resilient modulus is also significantly correlated with such soil properties as the liquid limit, plastic limit and grain size distribution. Thompson and Robnett (1989) concluded that properties that tend to contribute to low resilient modulus values are low plasticity, high silt content, low clay content and low specific gravity. From the study, regression equations were developed for predicting M_R based on soil properties.

2.3.3 Dry Density

Variations in the density of the laboratory test specimen with the same water content produce variable effects on the resilient response of subgrade soils. Theoretically, Young's modulus of a soil is proportional to its density. Trollope et al. (1962) reported that the resilient modulus of dense sand might be 50% higher than that of loose sand.

2.3.4 Water Content

The effect of the water content on the resilient response of soils was noticed a long time ago. A general relationship between dry density, water content and resilient modulus for subgrade soils is shown in Figure 2.1 (Monismith, 1989). The effect of moisture on the resilient modulus is the focus of this study.

2.3.5 Strain Amplitude

The strain level also has a significant effect on the resilient modulus. As the strain amplitude increases, the modulus of the soil decreases. Kim et al. (1991) identified the relationship of the strain amplitude versus the modulus of the compacted subgrade soils, as shown in Figure 2.2. Figure 2.2 shows that the resilient modulus decreases with increasing strain amplitude.

2.3.6 Test Procedure

T 292-91I and T 294-92 are two of the most extensively used test procedures in recent years. Because of the differences in confining pressure and test sequence, the two procedures normally produce different results. Zaman et al. (1994) found that the T 294-92 test procedure gave higher resilient modulus values than those obtained by using the T 292-91I test procedure. Ping and Hoang (1996) had similar results. This phenomenon was attributed to the stress sequence, which had a stiffening and strengthening effect on the specimen structure as the stress level increased.

2.3.7 Size Effect

Specimen size has an influence on the resilient modulus of soils. The diameters of the specimen could be as small as 2.0 in., however, the most common sizes are 4.0 and 6.0 in. in diameter. The ratio of height to diameter is usually 2.0.

The testing of materials composed of large particles demands larger specimens. T 292-92I specifies that a minimum 90% by material weight used to prepare the compacted specimen in the laboratory should have a maximum particle size finer than 1/6 the specimen diameter. The maximum particle size of the remaining material shall be no larger than 1/4 of the specimen diameter.

Zaman et al (1994) conducted a series of resilient modulus tests on six of the most commonly encountered aggregates that are used as the base/subbase of roadway in Oklahoma. The testing materials consisted of three limestones, one sandstone, one granit, and one rhyolite. The specimens were prepared at three different levels of gradation. The maximum particle sizes varied from 0.75 in. to 1.5 in..

Vibration and compaction methods were employed in preparing specimens. The specimens were 4 in. and 6 in. in diameter. The test results of the 4-in. and the 6-in. samples were analyzed. In all cases, the resilient moduli for the 4-in. specimens were 20 to 50% higher than those for the 6-in. specimens.

2.4 EFFECT OF MOISTURE

2.4.1 Detrimental Effect of Water

Experts recognized the detrimental effect of water on a pavement system. The detrimental effects of water, when entrapped in the pavement structure, can be summarized as follows:

1. It reduces the strength of unbounded granular material and subgrade soils.
2. It causes pumping of concrete pavements with subsequent faulting, cracking and general shoulder deterioration.

With the high hydrodynamic pressure generated by moving traffic, pumping of fines in the base course of flexible pavements may also occur with a resulting loss of support.

3. In northern climates with a depth of frost penetration greater than the pavement thickness, a high water table causes frost heave and the reduction of load-carrying capacity during the frost melting period.
4. Water causes differential heaving over swelling soils.
5. Continuous contact with water causes stripping of asphalt mixture and durability or “D” cracking of concrete.

This study is focused on the first issue, the effect of water on the strength of granular material and subgrade soil.

2.4.2 Effect of Moisture on Resilient Modulus

Since the introduction of resilient modulus, the effect of moisture content is considered a main factor which may change the value of resilient modulus.

Seed et al. (1962) noted a rapid increase in resilient deformations for specimens of the AASHO Road Test subgrade soils compacted with a water content above the optimum level (Seed et al., 1962). For specimens compacted below optimum water content, resilient deformations were characteristically low.

Hicks and Monismith (1971) analyzed the factors that may affect the resilient modulus of granular material. They used two aggregates for the investigation: one was the well-graded, subangular, partially crushed gravel and the other was the well-graded crushed rock. They found that the following factors may have a significant influence on the stress-deformation characteristics under short-duration repeated loads: (1) stress level (confining pressure), (2) degree of saturation, (3) dry density (or void ratio), (4) fines content (percent passing No.200 sieve), and (5) load frequency and duration.

As for the effect of degree of saturation, the following is what Hicks and Monismith found:

k_1 decreased from the dry to partially saturated test series where the comparisons were made on the basis of total stresses. For the dry test series, the cell pressure was approximately equal to the total stress and in this case only, the effective stress. For the partially saturated test series, the cell pressure was equal to the total stress and not the same as the effective stress. They did not attempt to measure the pore pressure; hence, effective stresses could not be properly defined in the tests for partially saturated materials. Figure 2.3 provides an indication of this effect for each aggregate at two levels of grading: coarse and fine.

When the data were plotted in the conventional manner in Figure 2.4, the modulus associated with the partially saturated test series was the lowest. The reason for this could be the manner in which the data were compared; data for the dry and partially saturated specimens were compared on the basis of total stresses, whereas data for the dry and saturated specimens were compared using effective stresses.

It appears that, if all results were defined in terms of total stresses, the value of k_1 would steadily decrease with increasing degree of saturation (or water content), as shown in Figure 2.3. Although there were inherent differences in the dry density (mean value of 126.6 pcf for water content of 2.4% and 132.2 pcf for water content of 6.3%) for their tests, the reduction in k_1 with increasing water content was apparent (Hicks et al., 1971).

Thompson and Robnett (1976) summarized the effect of an AASHO road test on subgrade soil in 1976. A typical effect of moisture on resilient modulus is shown in Figure 2.5. The resilient modulus decreases as moisture increases (Thompson et al., 1976).

In "Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition," published in 1982, the Asphalt Institute suggested that "in order to retain a given value for the resilient modulus (M_r) the dry density must increase as the molding water content increases." See Figure 2.1 for the general relationship between dry density, water content and resilient modulus for subgrade soils.

Pumphery and Lentz (1986) used repeated laboratory repeated load triaxial tests to estimate the effects of highway traffic on the permanent and resilient deformation of the subgrade sand commonly used as a foundation for flexible highway pavement structure in Florida in 1986. Combinations of confining stress and cyclic principal stress difference (test variables) and of dry unit weight and moisture content (sample variables) were used for each sample and loaded to 10,000 cycles. Confining stress, cyclic principal stress difference and dry unit weight were correlated with permanent strain and resilient modulus and thus affected deformation properties of these soils. However, moisture content correlated with neither permanent strain nor resilient modulus.

In this test, Pumphery and Lentz used a type of uniform, fine sand from a borrowed pit as a sample in Leon County, Tallahassee, Florida. It was classified A-3 according to AASHTO classification. Standard (AASHTO T-99) and Modified (AASHTO T-180) compaction tests were conducted to determine maximum dry unit weight and optimum moisture content.

Several of the test and sample variables, such as confining stress, cyclic principal stress difference, dry unit weight and moisture content were selected for study. Various combinations of these factors were tested in cyclic triaxial tests. A cyclic principal stress difference was set at different percentages of the peak static soil strength determined from samples tested at similar dry unit weight, moisture content and confining stress combinations.

An inverted haversine wave form of a 0.1-second duration was used for all repeated load tests. This period is roughly equivalent to the time in which a vehicle traveling 30 mph affects a point in the top of the subgrade of a flexible pavement structure. The 0.1-second was followed by a 0.9-second rest period to allow proper damping of the load before the following load was applied. Therefore, a frequency of one load per second resulted. All cyclic tests were continued to 10,250 cycles.

Tests were conducted at two different moisture content levels of the sand: 3% below optimum, and at optimum. Preliminary plans included testing samples at 3% above optimum. However, samples could not be compacted to the required density using the tamping method, so this moisture condition was eliminated from the program.

The effect of moisture content on resilient modulus has been a particularly elusive characteristic for researchers to examine. Through analysis, no definite trend has emerged for all materials in this area. Figure 2.6 contains comparisons of highway subgrade sand samples tested cyclically at different levels of moisture content in the sand. Because of the scatter in the points, no satisfactory relationships were found between moisture content and resilient modulus.

Thadkamalla and George (1995) studied the effect of saturation on the resilient modulus. Three modes of saturating (wetting) were investigated: (1) capillary saturating, (2) vacuum saturating and (3) molding at wet of optimum moisture content. Results showed that the degree of saturation above optimum moisture content had a nominal effect (20%) on the resilient modulus of coarse-grain soils, whereas it had a severe effect (50 to 75% decrease) on the resilient modulus of fine-grain soils. Another finding was that both degree of saturation and saturating mode affected the resilient modulus of fine-grain soil. Vacuum saturation caused drastic decreases in resilient modulus.

In this study, Thadkamalla and George used two coarse-grain and two fine-grain soils. The two coarse-grain soils were the A-2-4, 26% finer than a No. 200 sieve, and A-2, 23% finer than a No. 200 sieve. The two fine-grain soils were the A-7-5, 97% finer than a No. 200 sieve, and A-4, 51% finer than a No. 200 sieve.

The percentage reduction of resilient modulus with degree of saturation for typical coarse-grain and fine-grain soils is shown in Figure 2.7. As expected, resilient modulus decreased with saturation, resulting in the following observations:

1. The resilient modulus of coarse-grain soil was not significantly affected by the amount and manner of saturation; the reduction was approximately 20%.
2. The resilient modulus of fine-grain soils was drastically reduced by saturation, the reduction being 50 to 75% depending on the degree of saturation and the saturating method used.

In the case of fine-grain soils, the saturating method used had a varying effect on the resilient modulus of the specimens tested. The resilient modulus value of the vacuum-saturated specimen decreased exponentially with increasing degrees of saturation, where it decreased linearly with the capillary saturating and also with specimens molded at wet of optimum moisture content.

In the case of fine-grain soils, the decrease in resilient modulus for both capillary saturated specimens and those molded at wet of optimum moisture content was nearly identical (Thadkamalla et al, 1995).

Barksdale, Alba, Khosla, Kim, Lambe and Rahman (1996) prepared a report about the laboratory determination of resilient modulus for flexible pavement design. This report discussed the moisture sensitivity of resilient modulus. They found that achieving a saturated sample required the use of good equipment maintained by a meticulous laboratory technician. A much more practical approach was to simply initially prepare the specimen at the desired moisture content. Specimens could not be successfully prepared at moisture contents greater than 3 to 4% above optimum and achieve satisfactory dry densities. A moisture content of 3 to 4% above optimum, however, was sufficient to show moisture sensitivity.

Figures 2.8, 2.9, and 2.10 show the important reduction in resilient modulus that occurred as specimens soaked under a back pressure of 10 psi applied at the base and the corresponding increase in modulus after the water was partially drained from the specimen. The more cohesive clayey sand (Figure 2.8) subgrade soil was clearly much more moisture-susceptible than the silty sand (Figures 2.9 and 2.10) with the average retained resilient modulus upon soaking being about 40% and 75%, respectively, of the impact compacted specimens at optimum moisture content. Soaking the silty sand for up to 10 days only increased the degree of saturation from 84% to 92% for the kneading compacted specimen. Achievement of a higher degree of saturation would have resulted in a larger reduction in resilient modulus (Barksdale et al., 1997).

Fredlund et al. (1997) also examined the effect of variations in deviator stress on the resilient modulus for specimens prepared at both dry and wet of optimum water contents. For wet of optimum specimens, the resilient modulus was shown to vary more with variations in deviator stress than those tested dry of optimum. Typical behavior showed a significant decrease in the resilient modulus with increasing deviator stress for wet of optimum test specimens. For those tested dry of optimum, the resilient modulus also decreased with increasing deviator stress, but to a much lesser extent than those tested wet of optimum.

In Florida, the State Department of Transportation (FDOT) has been using the repetitive rigid plate test to evaluate the characteristics of Florida pavement for more than 20 years. Ping, Yang and Ho (1998) made a summary of these tests. Figures 2.11, 2.12 and 2.13 illustrate the moisture effect on resilient modulus. The five typical subgrade soils were all granular materials (sands). A test-pit facility was used to simulate the subgrade and base components of a flexible pavement system. By rising and lowering the water table, the moisture of the pavement was changed; these were called the soaked and drained tests. The soaked and drained test conditions were under somewhat lower and higher moisture levels than the optimum test conditions, respectively. As for the moisture effect on resilient modulus, a summary follows:

The resilient modulus and permanent deformation under various moisture conditions were compared in order to examine the effect of moisture. The resilient modulus and permanent deformation versus the moisture content are shown in Figures 2.11 and 2.13, respectively. As can be seen from the figures, the moisture has a significant effect on the resilient modulus and permanent deformation. As shown in Figure 2.11, an increase in moisture has a strong detrimental effect on the resilient modulus for all of the five subgrade soils. For Crawfordville sand, Ocala sand and Brooksville sand, the modulus values were not changed much with a change in moisture. For Alachua sand and Panama City sand, however, the moisture had a significant effect on the resilient modulus. The resilient modulus values were increased almost five times with the change in moisture from the soaked condition to the drained condition.

In addition, the figures also show that under the drained condition major differences existed in the moduli among five subgrades, whereas under the soaked condition relatively small differences were observed. Some subgrades (such as Alachua sand) are very sensitive to the changes in moisture content.

The reduction in resilient modulus due to an increase in the degree of saturation is most significant for Alachua sand. One factor contributing to this effect may be the higher degree of saturation (75%) at the optimum condition for Alachua sand. The Crawfordville and Brooksville sands have lower degrees of saturation (59%) at the optimum condition, and the detrimental effect on the resilient modulus due to the higher degree of saturation is found to be much less than that which occurs for the Alachua sand. Therefore, a degree of saturation of about 80 to 90% for a granular material (depending on its optimum degree of saturation) may be sufficient when taking the most critical moisture condition into consideration for determining the resilient modulus in laboratory. Based on the experience with this study, preparation of a laboratory specimen may not be possible without backpressure saturation or vacuum saturation when the degree of saturation is beyond 80 to 85% for a granular material (Ping et al., 1998).

Drumm, Reeves, Madgett and Trolinger (1997) summarized their tests of the effect of saturation on resilient modulus. A series of resilient modulus tests were designed to investigate the variation in resilient modulus due to post-compaction increases in water content. Triplicate specimens were prepared for eleven soils throughout Tennessee, with each specimen having target values of optimum water content and maximum dry density. One specimen was tested at optimum and the other two were tested at increasing levels of saturation. All soils exhibited a decrease in resilient modulus with an increase in saturation, but the magnitude of the decrease in resilient modulus was found to depend on the soil type. The soils with the highest resilient modulus for optimum conditions were found to experience the greatest decrease with saturation.

They realized that varying the moisture content at the time of saturation may not represent the actual variation in properties under field conditions. The moisture content at compaction affects the strength and stiffness properties of the soil due to the influence of particle orientations during compaction. These soil structure effects are known as important factors governing resilient response. Therefore, to accurately predict how subgrade soil will react with seasonal moisture changes, the specimens should first be compacted to near field conditions (such as optimum moisture content and maximum dry density), and then the water content should be increased before resilient modulus testing. Samples were selected from 11 active construction projects in Tennessee. These soils were representative of materials commonly found in pavement subgrades. Since the majority of the subgrade soils in Tennessee are fine-grained, the research was restricted to those with more than 50% passing the No. 200 sieve.

Drumm et al. did the cyclic triaxial testing in general accordance with the Strategic Highway Research Program (SHRP) Protocol P-46 (1989). The conditioning was 200 load repetitions. For each combination of cell pressure and deviator stress, they applied 100 load repetitions. The load duration was 0.1 seconds and the cycle duration was 1.0 seconds. Table 2.1 is a summary of specimen conditioning and loading scheme (Strategic Highway

Research Program 1989). The typical effects of post compaction saturation on resilient modulus are shown in Figures 2.14, 2.15, 2.16 and 2.17.

Figure 2.14 shows a typical reduction in resilient modulus with an increase in the degree of saturation. The upper curve represents triaxial results for a specimen compacted near the optimum moisture content of 29.4% and a degree of saturation of 91.9%. The middle curve represents a specimen saturated to a moisture content of 30.1% and 93.4% saturation. The lower curve represents a specimen saturated to a moisture content of 30.7% and 95.4% saturation. Since the effect of confining stress on the resilient modulus of these fine-grained soils was small, smooth curves have been fitted to the data points; the resulting curve is the average of the results at confining pressures of 41 kPa (6 psi), 28 kPa (4 psi), and 14 kPa (2 psi). Figure 2.14 shows that as the moisture content and degree of saturation increased, there was a corresponding decrease in resilient modulus values. This was observed for all 11 soils tested.

To illustrate the magnitude of these changes by showing values from the triaxial curves at similar deviator stresses and confining pressures, Figure 2.15 shows a plot of M_r versus moisture content at these stress conditions for the Knox County Station 4000 specimens. Figure 2.16 shows a plot of M_r versus degree of saturation. Figure 2.17 summarizes the variation in resilient modulus with degrees of saturation for eleven subgrade soils. In general, the resilient modulus decreased with the increase of degree of Saturation. The A-7-6 and A-7-5 soils have the highest resilient modulus at optimum water content and maximum dry density and they are most susceptible to changes in M_r due to changes in water content or degree of saturation. The lower resilient modulus A-4 and A-6 soils were less susceptible to decreases in M_r with increases in water content (Drumm et al., 1997).

Andrew, Drumm and Jackson (1998) measured the seasonal variation in subgrade resilient modulus by Falling Weight Deflectometer. They found the resilient modulus of fine-grained soil was dependent on moisture content. They suggested an effective roadbed soil resilient modulus, which incorporated moisture variations and the corresponding resilient modulus. This effective modulus is equivalent to the combined effect of all the seasonal modulus values.

From the above research, the following key points can be summarized:

1. The effect of moisture on resilient modulus varies with soil type. Usually, it has a significant effect on fine-grained soil but not as much of an effect on coarse-grain or granular soil and/or sand. Moisture may have no

significant effect on some sand, such as A-3 in Leon County, Florida.

2. Moisture has an effect on resilient modulus and, in turn, the resilient modulus depends on deviator stress and confining pressure.

3. Moisture imbibition methods have an influence on the resilient modulus. The vacuum saturating severely affects air-water interface in the soil. It cannot simulate moisture imbibition akin to field conditions. Capillary saturation and molding at wet may be more suitable (Andrew et al., 1998).

2.4.3 Explanation of Moisture Effect on Resilient Modulus

Edil and Motan (1979) studied the relationship between the resilient behavior of subgrade soil and soil-water potential (or soil suction), which gave some explanation of the moisture effect on resilient modulus.

Soil suction causes an increase in effective stress in a subgrade or base as the material dries out. The increase in effective stress can cause a significant increase in resilient modulus. Soil suction decreases as the degree of saturation increases and is not present when the soil is saturated.

The researchers found that the energy of a soil-water system could be expressed as a function of its characteristic water retention curve, or the relationship between the free energy of water in the soil and that of pure water in a free surface condition.

Total soil-water potential or soil suction is defined as the work required to remove the infinitesimal quantity of water from the soil and provides a measure of the combined effects of the forces holding the water in the soil. With the exception of cementation bonds, it implicitly includes the effects of the fundamental interaction forces that influence the deformation characteristics of the soil. The total soil-water potential of a soil varies with its water content, mineralogy, solutes present in the pore water and soil fabric, among other parameters.

The soil suction concept provides a fundamental soil parameter that reflects mechanical behavior. The few existing investigations that relate the mechanical response under repetitive loading conditions to soil suction indicate that soil suction is an important moisture variable for describing resilient behavior and relating it to the soil environment.

Edil et al. studied the relationship between the resilient modulus, residual strain, post-repetitive loading strength and moisture regime of two fine-grained soils and drew the following conclusions:

1. Characteristic water retention curves were useful for reflecting the susceptibility of compacted soils to moisture changes.
2. The resilient modulus and strength strongly depended on compaction moisture content on the dry side of optimum with insignificant dependency on the wet side (with the range of ±2% of optimum), whereas the residual strain exhibited the opposite behavior.
3. The moisture regime subsequent to compaction was expressed most suitably in terms of soil suction. It was an intrinsic parameter of the moisture equilibrium and reflected the effects of soil type and fabric, climate and position of groundwater table on the mechanical response better than moisture content or degree of saturation alone.
4. Resilient modulus and post-repetitive loading strength were primarily related to soil suction. For silt loam soils investigated, variations in these properties were small for suction values less than 100 kPa. This suction corresponded roughly to 2% dry-of-optimum moisture content. For suction greater than this, however, significant increases in mechanical properties (on the order of three- to six- fold) were reached.

5. The opposite behavior was seen in the residual strain.
6. Resilient modulus increased monotonically for soil suctions from 100 kPa to a critical suction beyond which it decreased. This critical suction appeared to be about 800 kPa (116 psi) (corresponding moisture content was 2% dry of optimum) for the soil tested.
7. The number of loading cycles resulted in significant increases in resilient modulus and residual strain and some increase in compressive strength (Edil et al, 1979).

Table 2.1 Summary of Specimen Conditioning and Loading Scheme (Strategic Highway Research Program 1989)

Cyclic loading (1)	Cell Pressure	Deviator Stress (3)	Number of load repetitions (4)
	σ_c [kPa (psi)] (2)	σ_d [kPa (psi)] (3)	
Conditioning	41 (6)	28 (4)	200
Testing	41 (6)	7 (1)	100
	41 (6)	14 (2)	100
	41 (6)	28 (4)	100
	41 (6)	41 (6)	100
	41 (6)	55 (8)	100
	41 (6)	69 (10)	100
	28 (4)	7 (1)	100
	28 (4)	14 (2)	100
	28 (4)	28 (4)	100
	28 (4)	41 (6)	100
	28 (4)	55 (8)	100
	28 (4)	69 (10)	100
	14 (2)	7 (1)	100
	14 (2)	14 (2)	100
	14 (2)	28 (4)	100
	14 (2)	41 (2)	100
	14 (2)	55 (8)	100
	14 (2)	69 (10)	100

Note: Load duration=0.1 s; cycle duration =1 s.

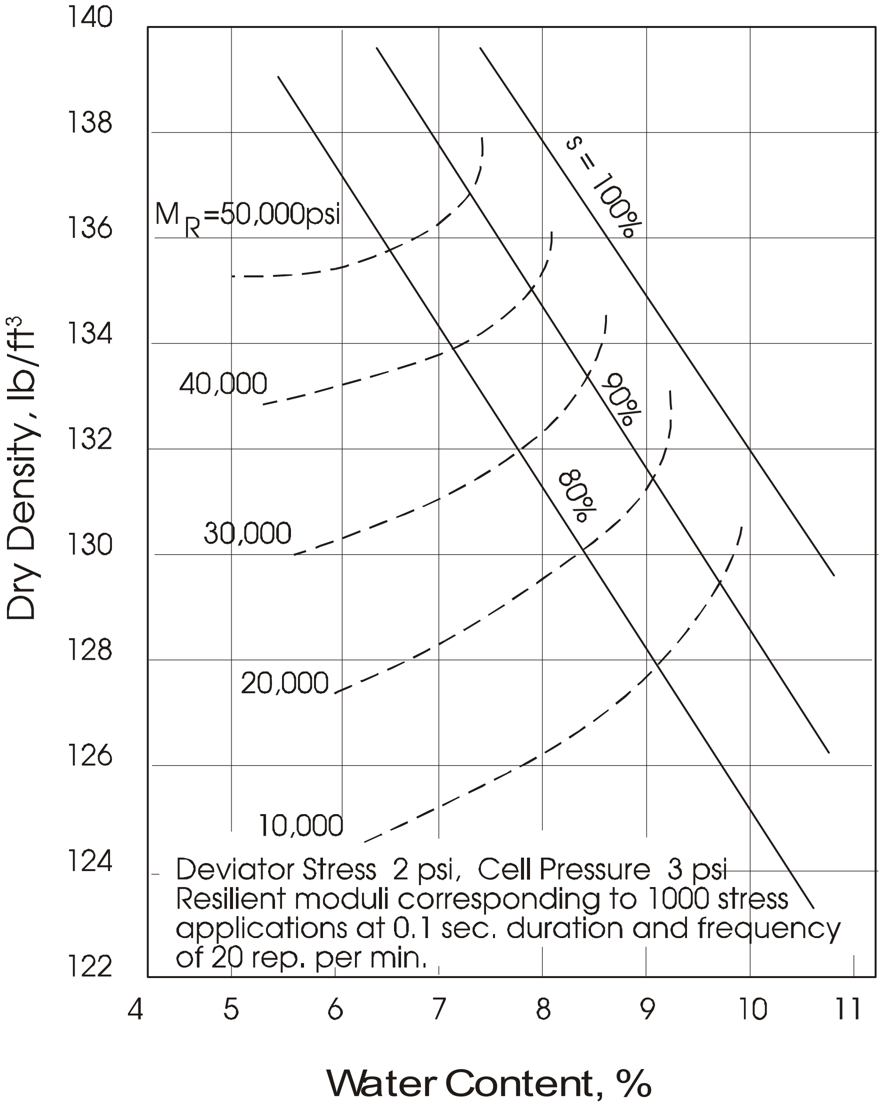


Figure 2.1 Water Content-Dry Density-Resilient Modulus Relationship for Subgrade Soil (After Monismith 1989)

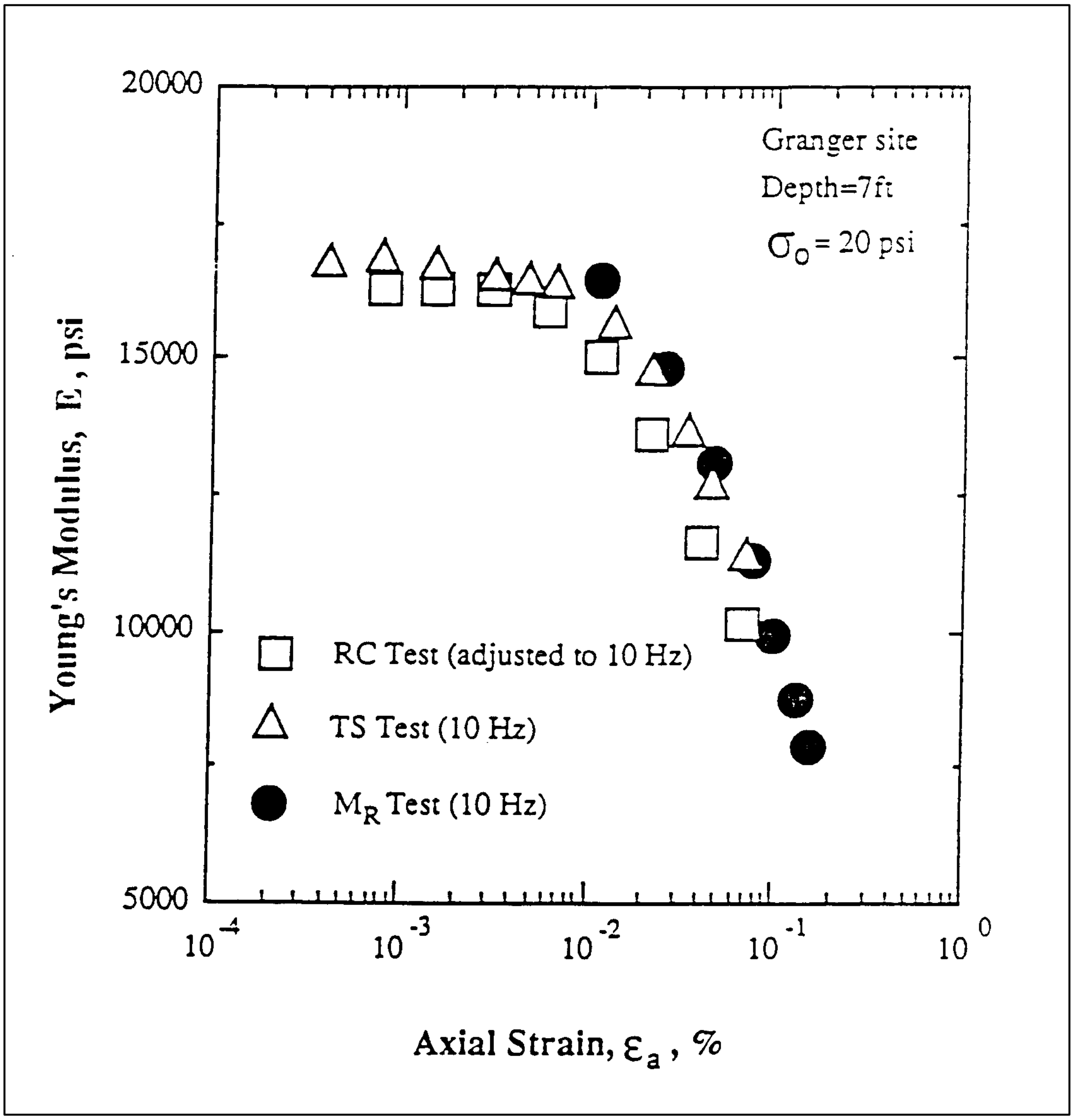


Figure 2.2 Comparisons of M_r Values of Undisturbed Compacted Subgrade Soils Determined by RC, TC and M_r Tests (Kim and Stokoe 1991)

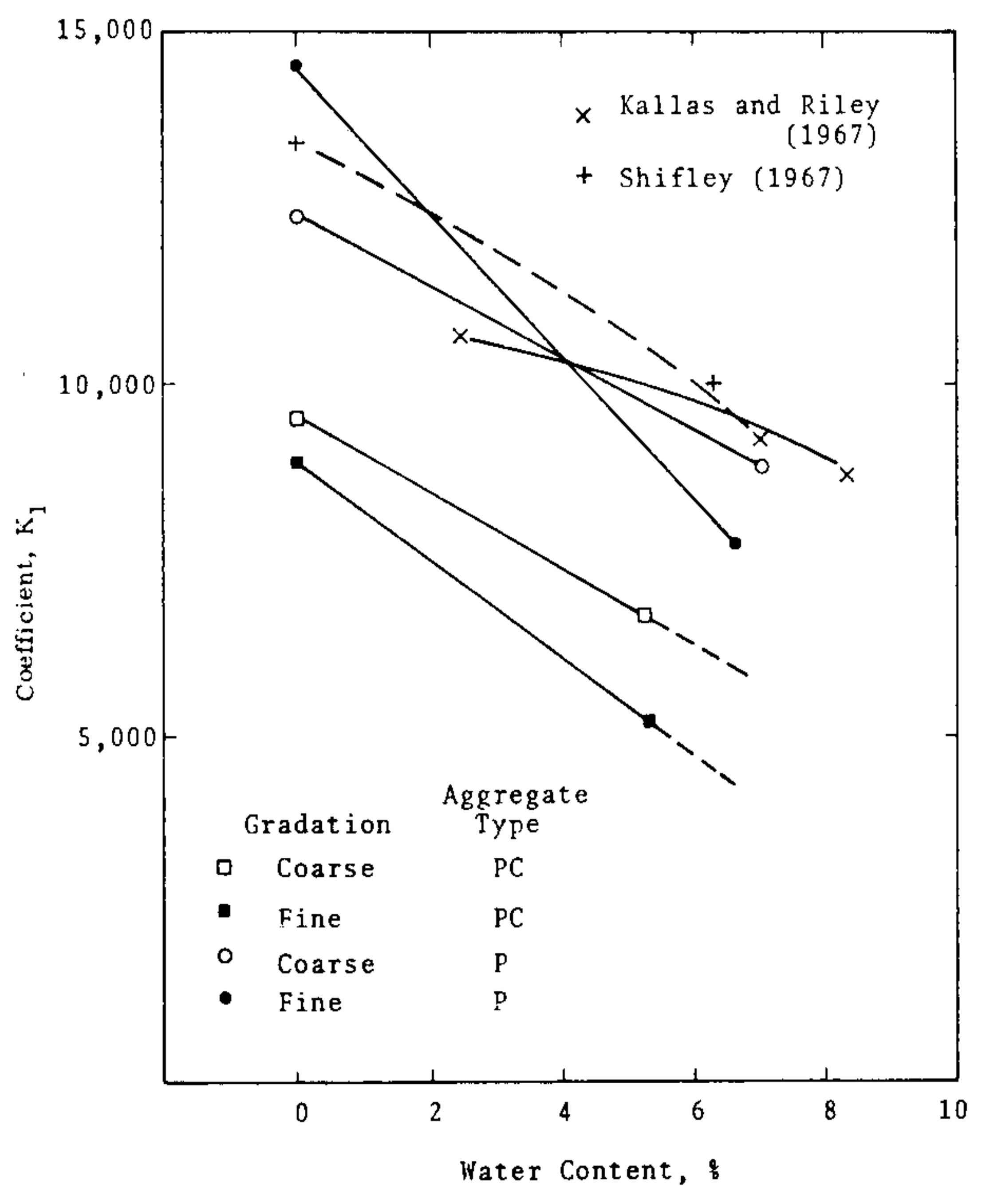


Figure 2.3 Variation in Regression Constant k_1, with Water Content in Relationship, $M_r=k_1\sigma_3^{k2}$

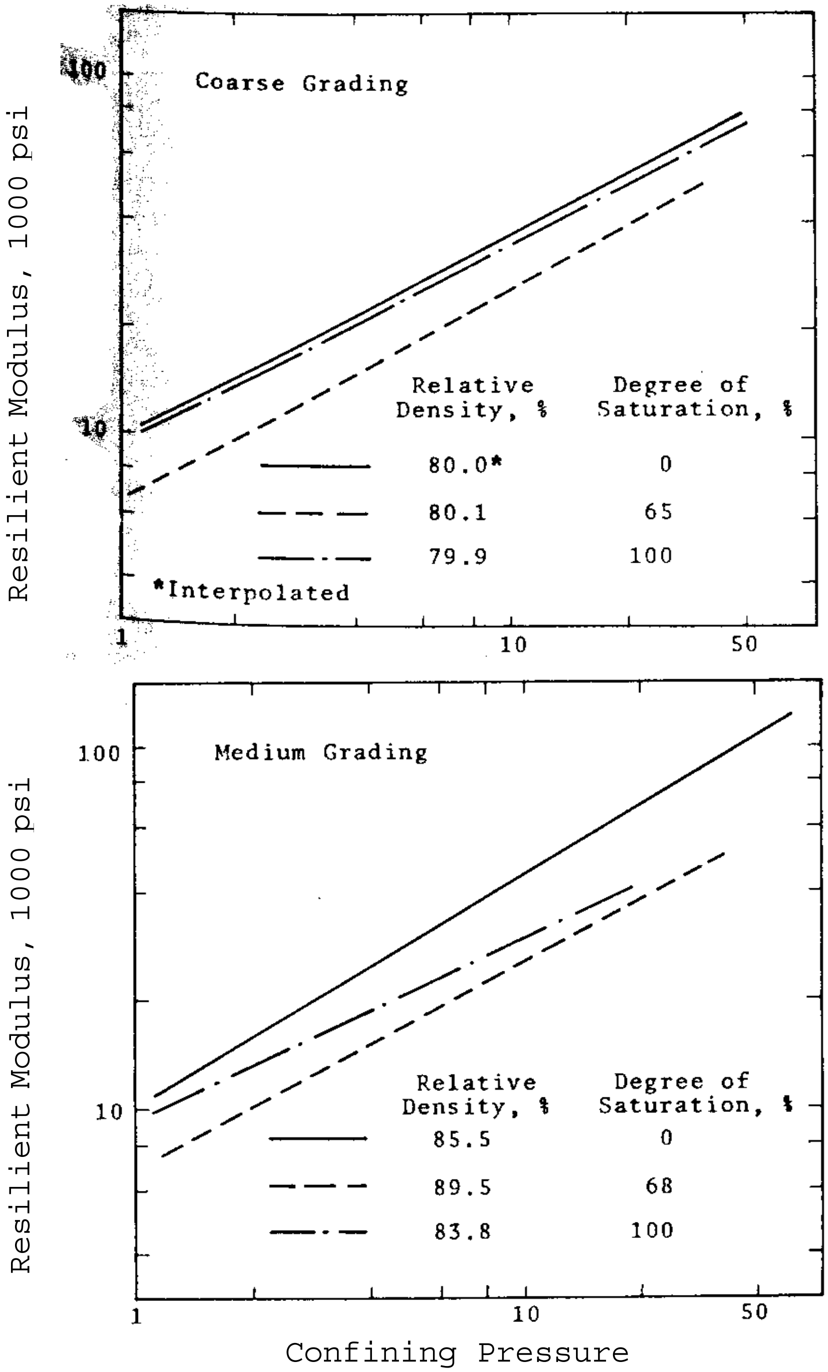


Figure 2.4 Effect of Degree of Saturation on the Relationship between Modulus and Confining Pressure (Partially Crushed Aggregate)

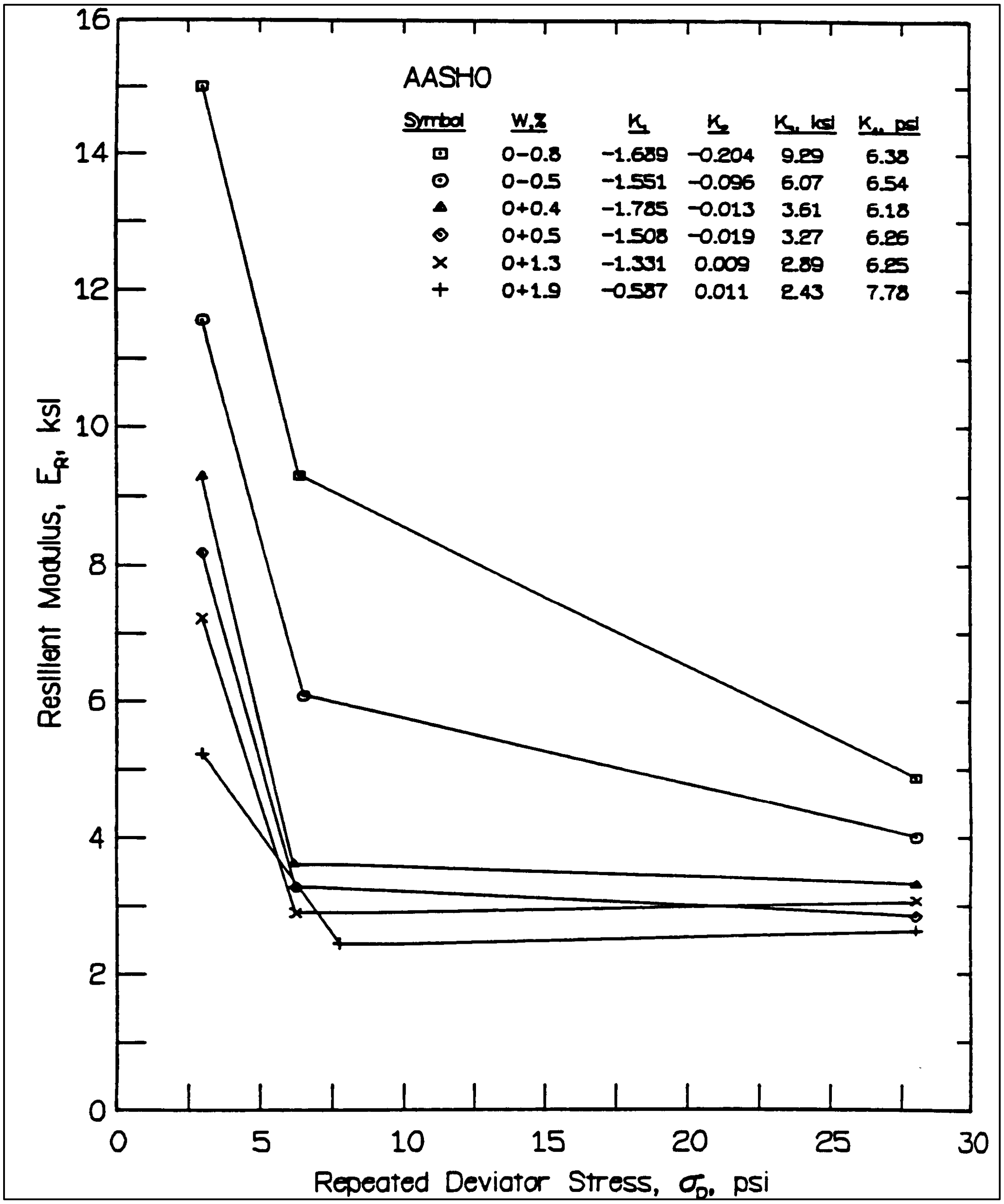


Figure 2.5 Typical AASHO Road Test Subgrade Resilient Modulus

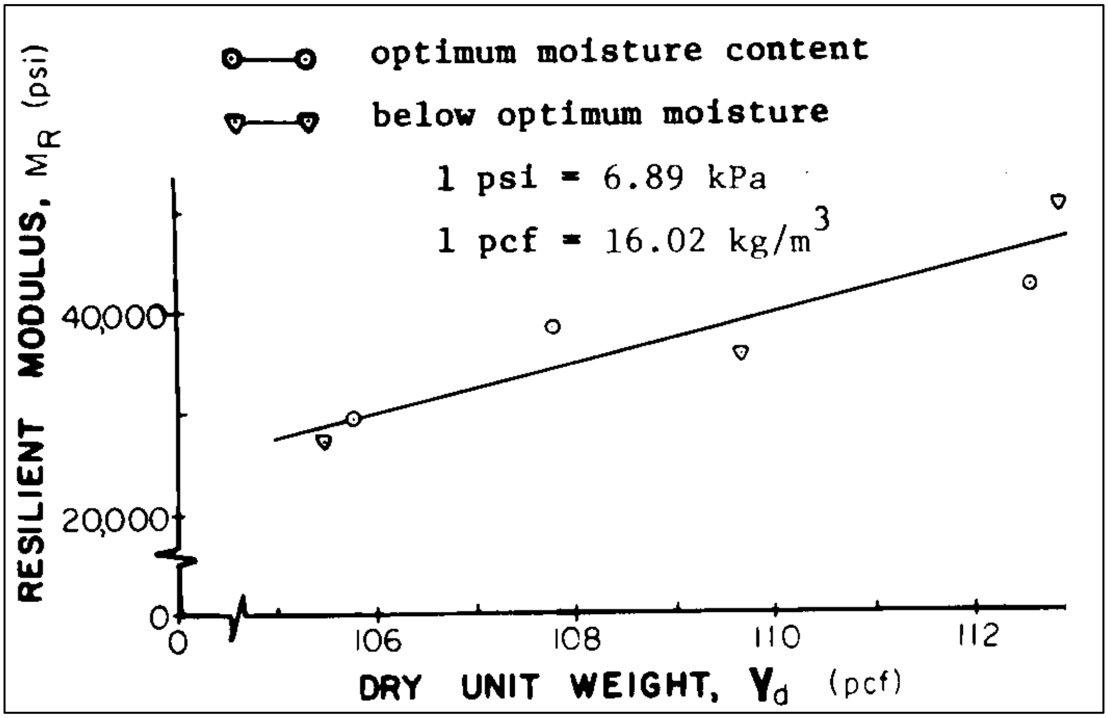


Figure 2.6 Effect of Dry Unit Weight and Moisture Content on Resilient Modulus at N=10,000

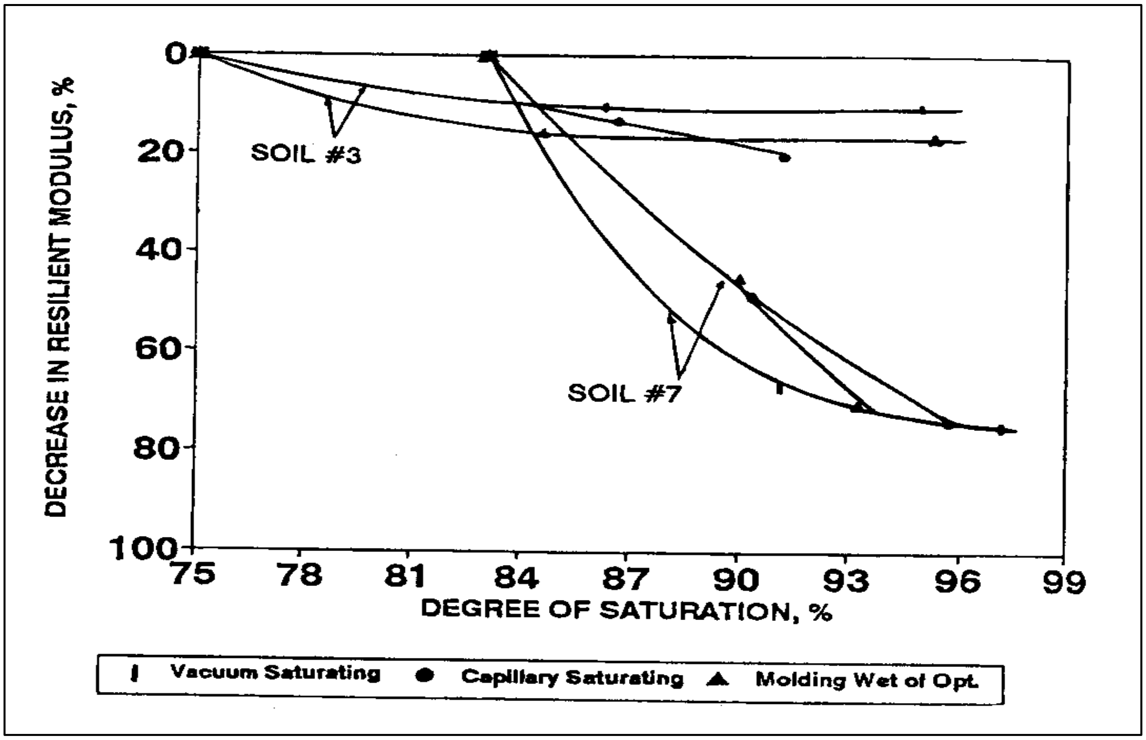


Figure 2.7 Reduction in Resilient Modulus with Degree of Saturation for Coarse- and Fine-Grain Soils

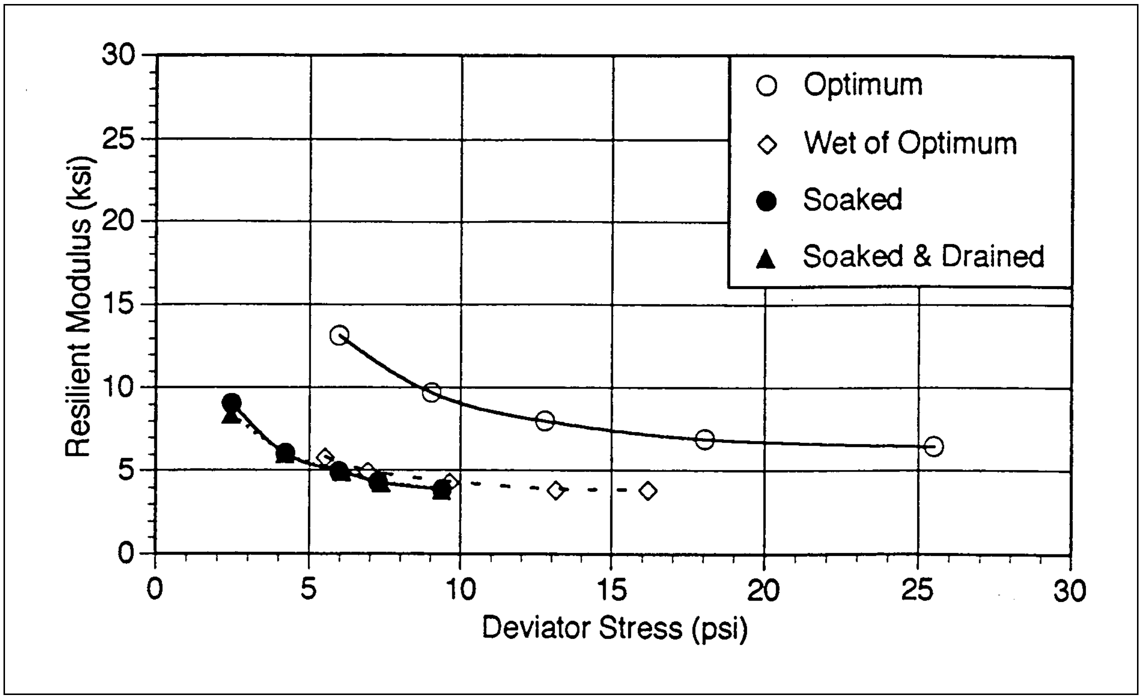


Figure 2.8 Effect of Moisture Conditioning on Resilient Moduli of Clayey Sand (A-6) Cohesive Soil Impact Compacted

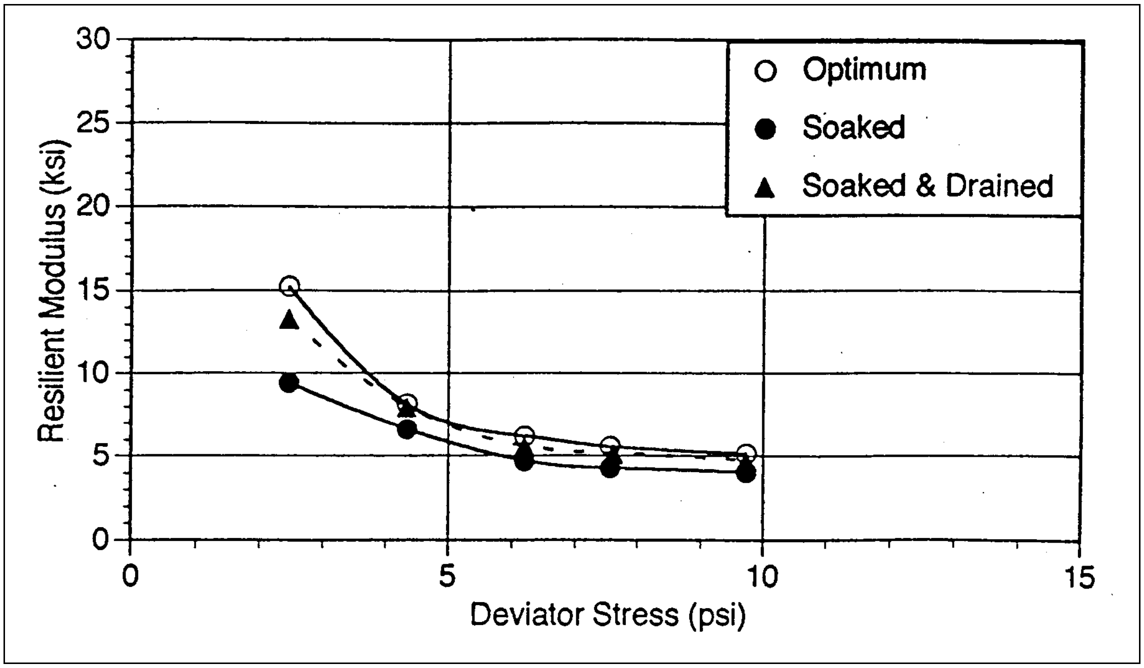


Figure 2.9 Effect of Moisture Conditioning on Resilient Moduli of Silty Sand (A-5) Cohesive Soil (Compacted Using Impact Method)

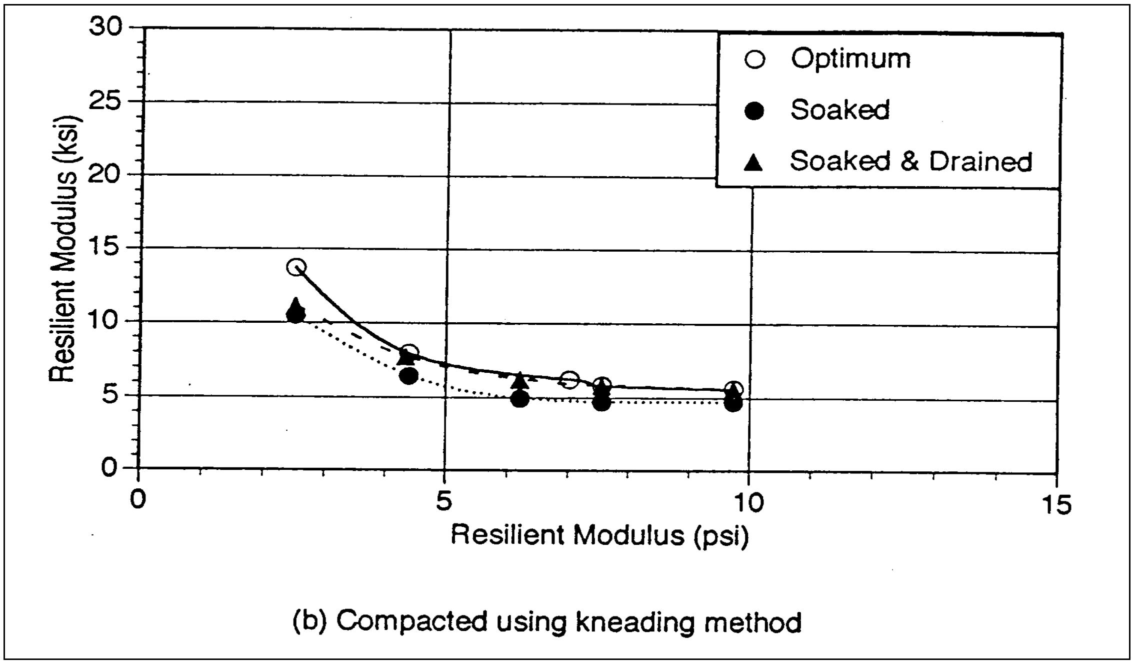


Figure 2.10 Effect of Moisture Conditioning on Resilient Moduli of Silty Sand (A-5) Cohesive Soil

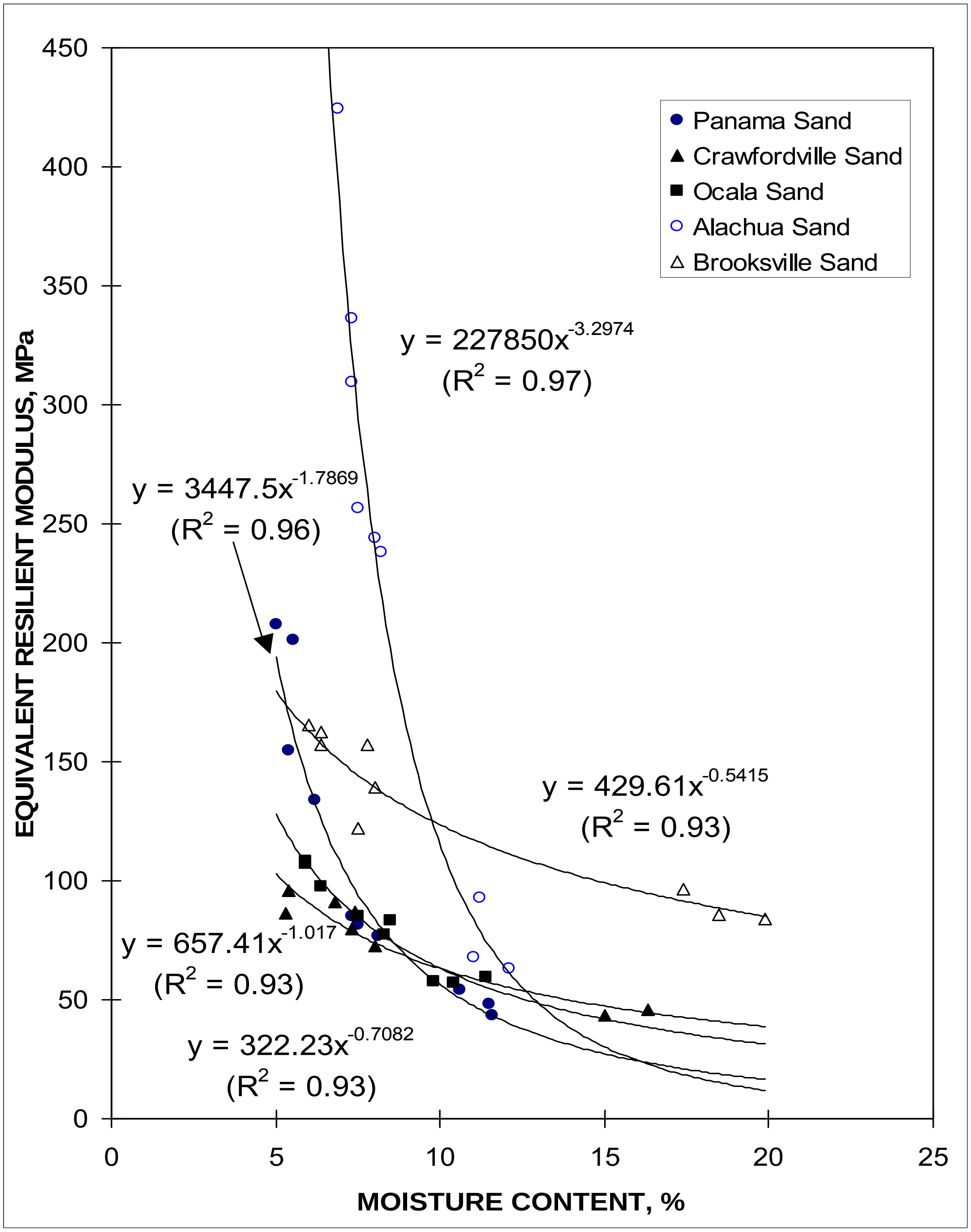


Figure 2.11 Resilient Modulus versus Moisture Content

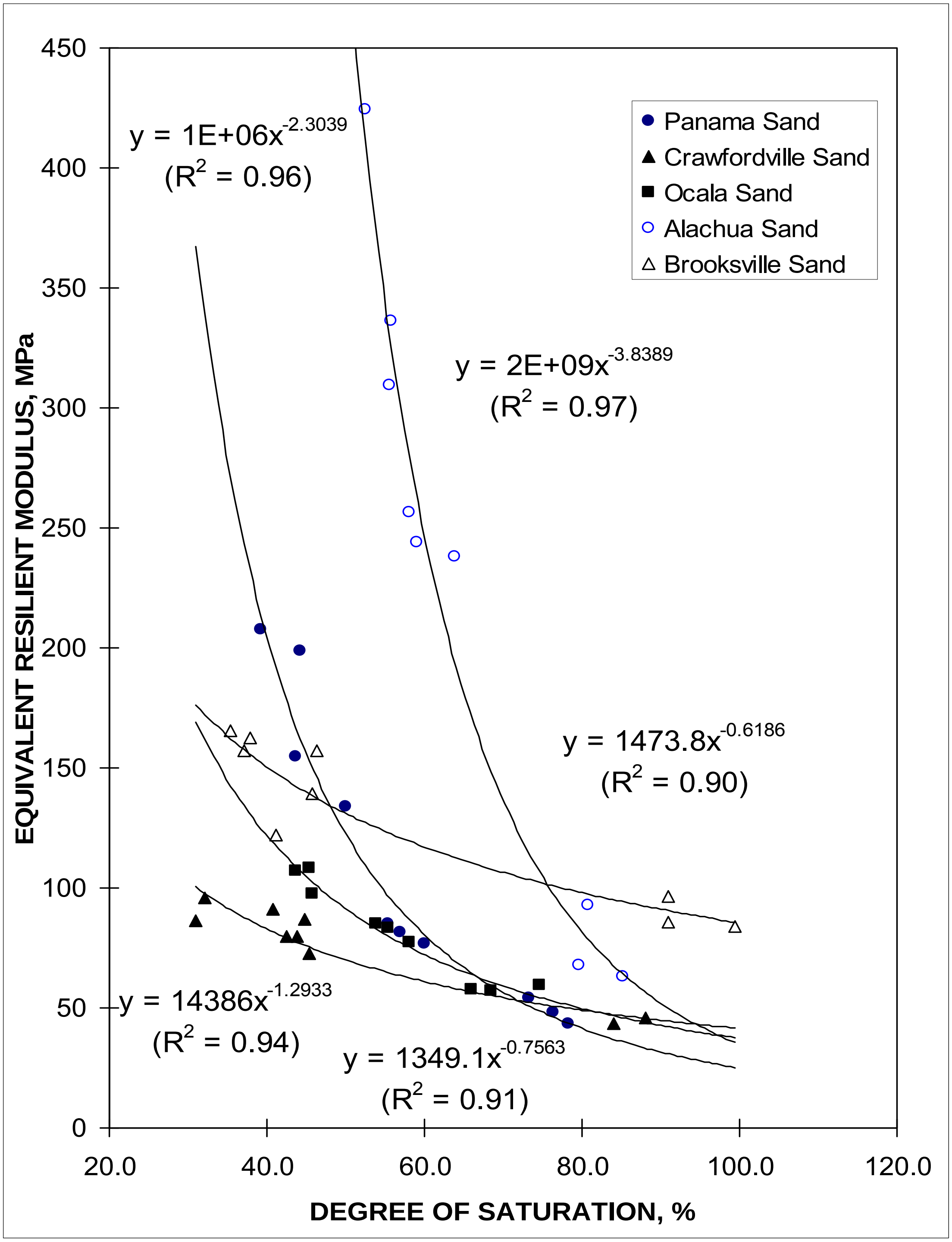
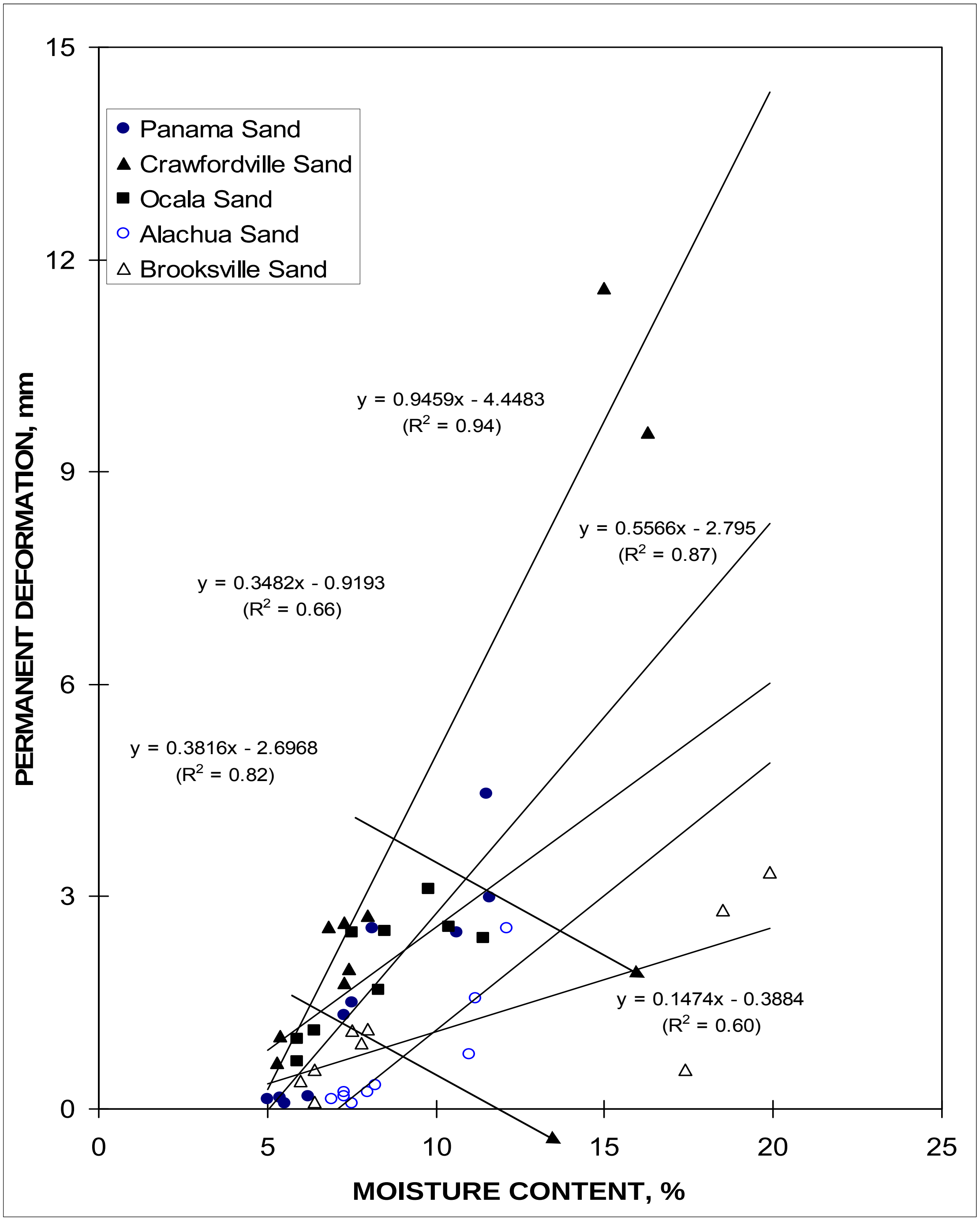


Figure 2.12 Resilient Modulus versus Degree of Saturation



15
12
9
6
3
0
0
5
10
15
20
25
PERMANENT DEFORMATION, mm
MOISTURE CONTENT, %
● Panama Sand
▲ Crawfordville Sand
■ Ocala Sand
○ Alachua Sand
△ Brooksville Sand
y = 0.9459x - 4.4483
(R^2 = 0.94)
y = 0.5566x - 2.795
(R^2 = 0.87)
y = 0.3482x - 0.9193
(R^2 = 0.66)
y = 0.3816x - 2.6968
(R^2 = 0.82)
y = 0.1474x - 0.3884
(R^2 = 0.60)

Figure 2.13 Permanent Deformation versus Moisture Content

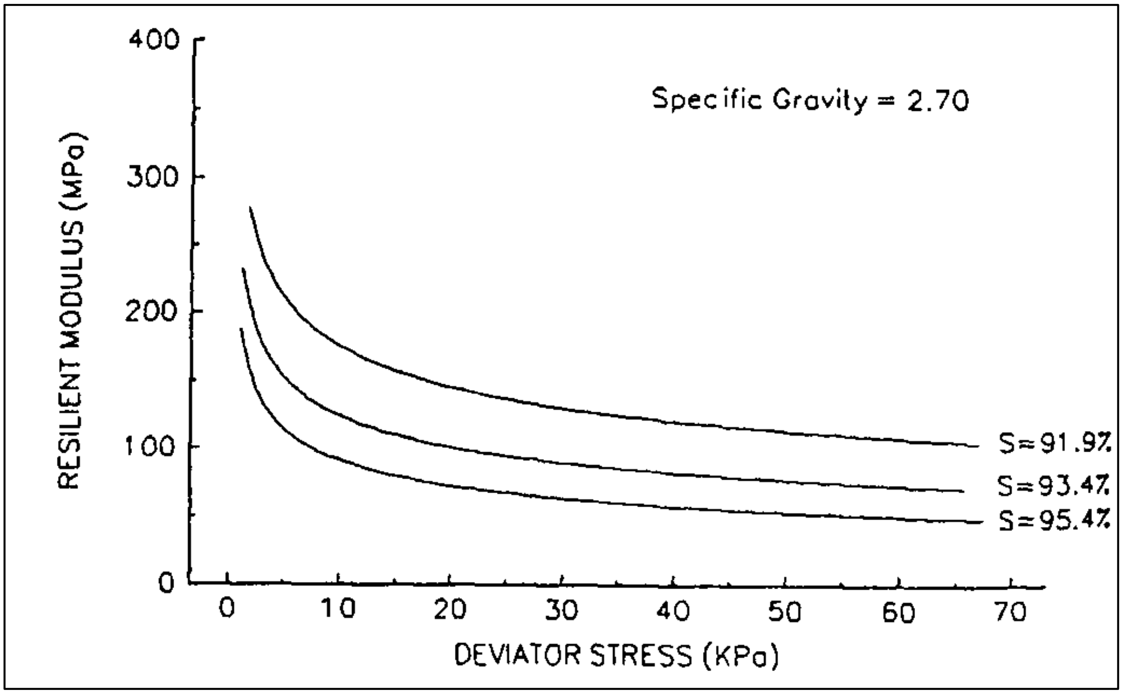


Figure 2.14 Typical Effect of Postcompaction Saturation on Resilient Modulus

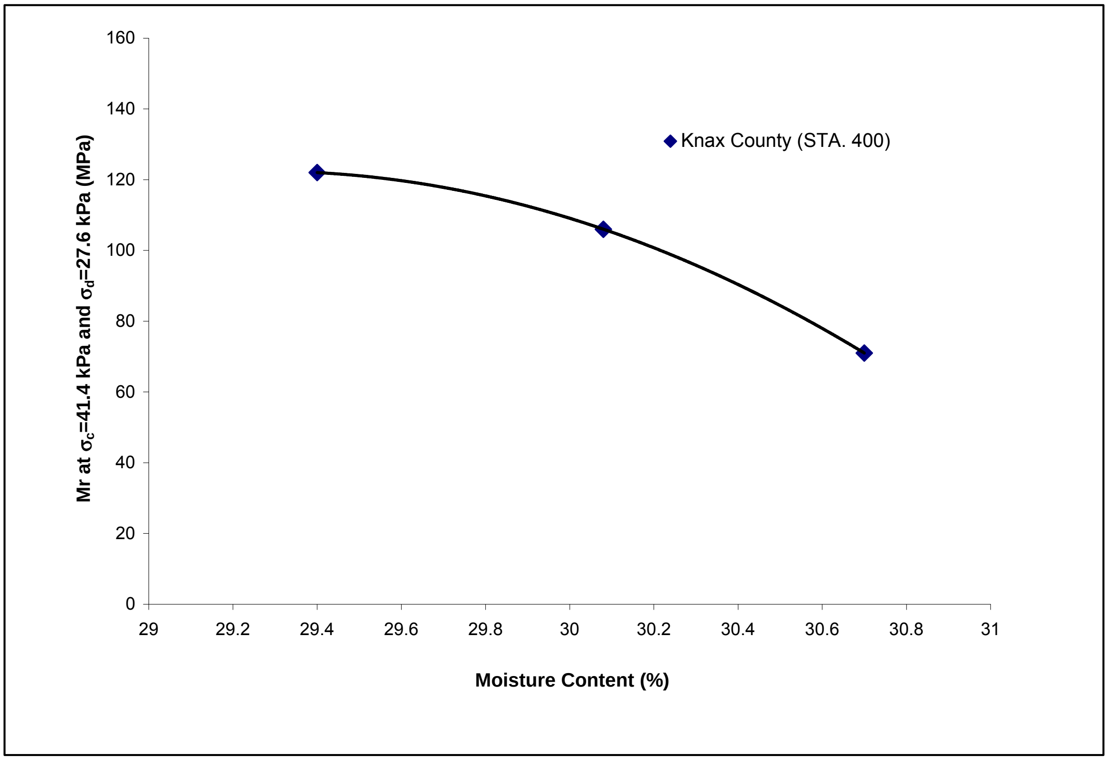


Figure 2.15 Typical Effect of Postcompaction Moisture Increase on Resilient Modulus

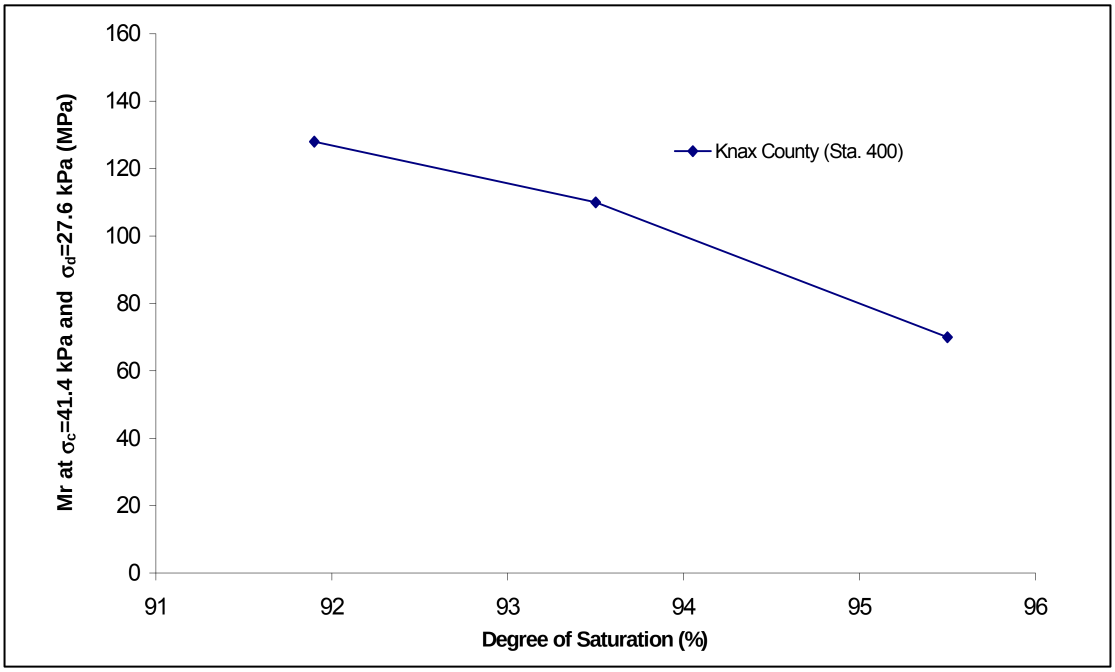


Figure 2.16 Typical Effect of Post Compaction Saturation on Resilient Modulus (1)

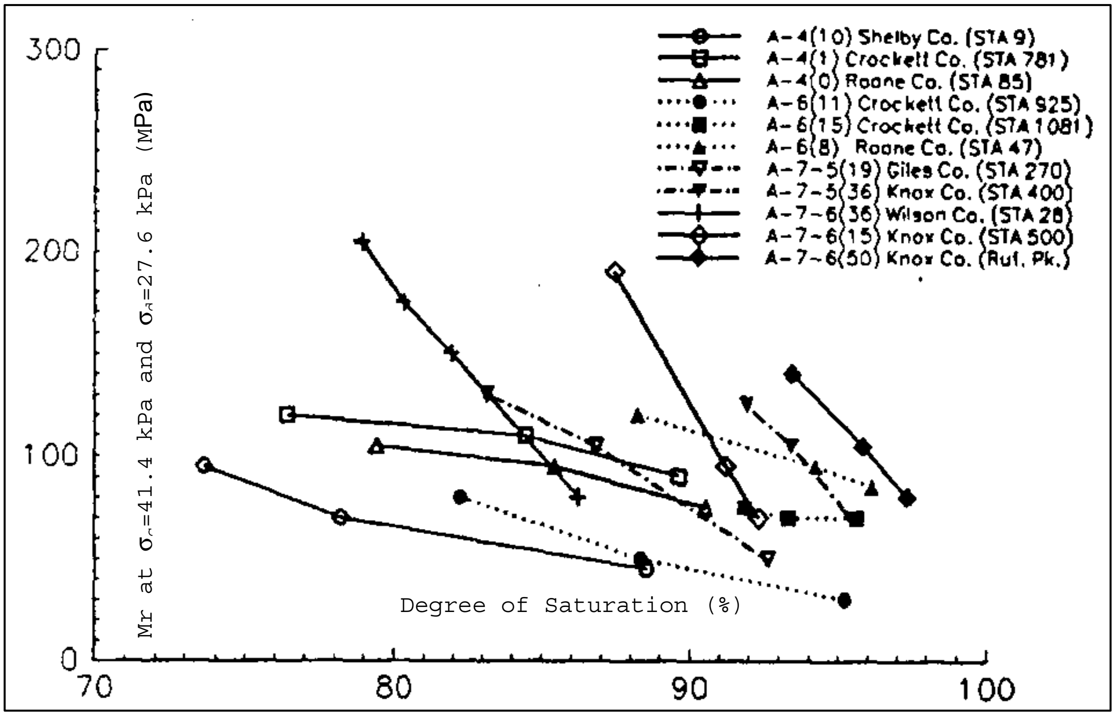


Figure 2.17 Typical Effect of Post Compaction Saturation on Resilient Modulus (2)

CHAPTER 3
EXPERIMENTAL PROGRAMS

3.1 GENERAL

An experimental program was conducted to evaluate the effect of high groundwater level on pavement subgrade performance. The subgrade soils were selected or mixed in conjunction with Florida DOT personnel and were believed to be representative of typical Florida subgrade soils. The experimental program included a laboratory resilient modulus test program, a suction test program, a permeability test program, a full-scale test-pit test program, and a field monitoring program. The purpose of the experimental program was to test and evaluate the subgrade soils under different moisture conditions for determination of high groundwater effect. Three phases of the experimental program were conducted from 1999 to 2007. In this chapter, Phases I and II of the experimental program are described in Sections 3.3 through 3.7, and Phase III is presented in Section 3.8.

3.2 SUBGRADE MATERIALS

The soils under evaluation in this study were the typical A-3 and A-2-4 subgrade materials used in Florida. To represent the typical Florida pavement soils, a total of eleven types of soil collected from various regions of Florida were evaluated under three stages of testing (Phase I, Phase II, and Phase III). The percent of fines passing the No.200 sieve of these materials ranged from 4% to 31%. The Phase I materials included Levy County A-3 (4% passing No.200), SR70 A-3 (8% passing No.200) and SR70 A-2-4 (14% passing No.200). The Phase II materials included: A-2-4 (12% passing No.200), A-2-4 (20% passing No.200), A-2-4 (24% passing No.200), A-2-4 (30% passing No.200) and Oolite. The Phase III materials included: Spring Cemetery A-2-4 (15% passing No.200), Branch A-2-4 (23% passing No.200) and Iron Bridge A-2-6 (31% passing No.200). Pertinent characteristics of the eleven subgrade soils are summarized in Table 3.1.

3.3 LABORATORY RESILIENT MODULUS TEST PROGRAM

3.3.1 Soil Moisture Condition

When a road is constructed, the subgrade is compacted at optimum moisture content. After construction, the moisture content will change due to rainfall, groundwater, capillary rise and so on. An increase in moisture will have a detrimental effect on the resilient modulus of subgrade material.

In the laboratory, back-pressuring de-air water flushing is generally used by researchers to saturate soil samples for testing. In some cases, a high back pressure, which could be more than 100 psi, is used. However, in actual field conditions, the surrounding confining pressure of a typical subgrade pavement layer is around 2 psi.

In order to simulate the actual field conditions, a laboratory test procedure using soil specimens with four- to six-day soaking was used to evaluate the soil resilient modulus due to an increase in moisture. In addition, the Limerock Bearing Ratio (LBR) (Florida test method designation: FM 5-515) required the specimen to be soaked for two days before testing. Furthermore, the design high groundwater criteria required that the standing water duration should exceed 24 hours for traditional frequencies. Accordingly, a laboratory experimental program was undertaken to evaluate the effect of moisture on the resilient properties of subgrade materials.

3.3.2 Specimen Preparation

The primary objective of this laboratory test program was to evaluate the effect of moisture on the resilient modulus of granular subgrade soils. All soil materials were compacted in the laboratory to their optimum moisture and density conditions and then dried or soaked for the resilient modulus test. The equipment and procedure for specimen preparation are described as follows.

3.3.2.1 Equipment Preparation

Split mold - A 10.2 cm in diameter by 20.3 cm in height (4-in. by 8-in.) split mold was chosen to prepare the laboratory test specimen. The mold assembly has a steel cylindrical split mold, a base, and a collar at the top. Threaded rods are used to hold the collar, mold and base together.

Compaction machine - A mechanical soil compactor manufactured by Rainhart Company was used to compact the soil specimens. Different compaction energy levels can be achieved. The machine is designed to perform test methods AASHTO Designation *T 99* and *T 180*.

Sample extruder - A sample extruder was also provided by Rainhart Company. This hydraulic extruder with a long travel length worked well with the split mold.

Miscellaneous apparatus - Apparatus used for specimen preparation also included rubber membranes for encasing the specimen, balances, ovens, a microwave oven, straight edges, a No. 4 sieve, filter papers, porous stones, mixing tools, and miscellaneous tools.

3.3.2.2 Specimen Compaction

The making of specimens followed the AASHTO Designation T180, Modified Proctor Compaction. The air-dried material (except the Oolite from Miami) was sieved through a No. 4 sieve (4.76 mm) and mixed with enough water to provide the optimum water content previously determined in accordance with AASHTO T-180. The Oolite from Miami was crushed and then was sieved through a sieve of 3/8-in. opening. The mix was then compacted to give a resilient modulus specimen size of 4-in. in diameter and 8-in. in height, according to its optimum moisture and density conditions obtained from the Modified Proctor procedure. Because the specimen for the Modified Proctor is 4 in. in diameter and 4.586 in. in height, which is different in size from the resilient modulus specimen, a conversion was made to achieve the same compaction effort for the resilient modulus specimen. An equivalent compaction effect, with 8 layers at 27 blows for each layer, was applied to prepare for the resilient modulus specimen. The weight of the compactor and the height of the falling weight were kept the same as the Modified Proctor. These specimens, at optimum compacted conditions, represent the actual subgrade layer in the field immediately after construction.

During the compaction, the following value was used for calculation of dry density:

$$\gamma_{d\max} = \frac{W_t - W_m}{V_m \cdot (1 + w/100)} \tag{3-1}$$

where,

$\gamma_{d\max}$ = Maximum dry density

W_m = Weight of mold

W_t = Total weight of specimen and mold

w = Moisture content, in percent

V_m = Volume of mold

3.3.3 Soaking and Drying

After compaction, specimens were subjected to soaking and drying to reach the desired moisture content for testing.

3.3.3.1 Soaking Procedure

The specimens were soaked in water with the mold. The soaking process is illustrated in Figure 3.1. In order to prevent soil particles from falling or leaking into the water, the following measures were implemented (see Figure 3.2):

1. Two circular filter papers, which are larger in diameter than the mold, were placed on the top and bottom of the specimen, respectively. The outer edge of the filter paper was folded up around the end of the mold. A rubber band was placed outside the outer edge of the filter paper around the end of the mold, which tightly secured the filter paper

and the mold. Some sealant was also placed at the joint of the split mold to prevent leakage.

2. Two circular porous stones were placed on both ends of the mold. During the entire soaking time, a surcharge, which is made of a heavy steel ring with approximately the same outside diameter of the mold, was placed on the top of the mold. It exerted a force on the porous stones to prevent possible separation of the porous stone and the mold assembly. The whole mold assembly (Figure 3.1) was placed into a bucket of water for soaking. A porous cylinder stone was placed on the bottom of the mold assembly for a better flow of water to the specimen.

A test trial was carried out to find the suitable soaking time. The specimen with the mold was taken out of the water bucket and weighed every day. It was found that the weight of A-3 specimen stopped increasing after two days of soaking. For the A-2-4 specimen, the weight stopped increasing after four days. Therefore, the soaking time was set to be longer than four days.

The moisture content after soaking may be calculated using the following procedure:

1. The weight of soaked specimen may be calculated using the following equation:

$$W_s = W_{st} - W_m \tag{3-2}$$

where,

W_s = Weight of soaked sample

W_{st} = Weight of the soaked sample with mold

W_m = Weight of the mold

2. The unit weight of the soaked sample, $\boldsymbol{\gamma_s}$, may be calculated as the following:

$$\gamma_s = \frac{W_s}{V_m} \tag{3-3}$$

Where,

V_m = Volume of mold

3. The moisture content after soaking, $\boldsymbol{w_s}$, is obtained:

$$w_s = \frac{\gamma_s - \gamma_d}{\gamma_d} \tag{3-4}$$

Where,

γ_d = Dry unit weight of sample

4. The degree of saturation, S, can be calculated as:

$$S = \frac{w \cdot G_s \cdot \gamma_d}{G_s \cdot \gamma_w - \gamma_d} \tag{3-5}$$

Where,

G_s = Specific gravity of soil solids

w = Moisture content

γ_w = Unit weight of water

γ_d = Dry unit weight of sample

3.3.3.2 *Drying Procedure*

The specimens were exposed to the air inside the laboratory room for drying (see Figure 3.3). The equations for calculating the moisture content are similar as the above for soaked sample except the variables of the soaked specimen would be replaced by the corresponding ones of the dried specimen. Therefore, it is not repeated hereafter.

3.3.4 Resilient Modulus Test Procedure

The resilient modulus test method adopted for Phase I and Phase II of this project was AASHTO T 292-91I, "Resilient Modulus of Subgrade Soils and Untreated Base/Sub-base Materials." This method was considered an improvement to the AASHTO T 274-82 Method. The T292-91 method covered procedures for preparing and testing untreated subgrade and untreated base/sub-base materials for determination of resilient modulus under conditions representing a simulation of the physical conditions and stress states of materials beneath flexible pavements subjected to moving wheel loads. For the Phase III of this project, the resilient modulus test procedures basically followed the test method from AASHTO T307-99. A deviation from the test procedure was made by using the internally-mounted LVDTs for the vertical full-length measurements instead of external LVDTs illustrated in AASHTO Designation T307-99. All subgrade materials were compacted in the laboratory to 100% of optimum moisture and maximum density using AASHTO Designation T-99 (Standard Proctor Compaction) according to the FDOT requirements. A 4-in. by 8-in. split mold was used to prepare the test specimen. The blow number was modified to achieve the energy condition specified in the AASHTO T-99. Two specimens were compacted at the same time to keep them in a duplicate condition. The AASHTO T307-99 Designation specifies that the wet density of the laboratory-compacted specimen shall not vary by more than +/-3.0 percent of the target wet density and the moisture content of the laboratory-compacted specimen shall not vary more than +/-1 percent for Type 1 materials or +/-0.5 percent for Type 2 materials from the target moisture content. The resilient modulus testing equipment and procedures are described as follows.

3.3.4.1 Test Equipment

An MTS model 810 closed-loop servo-hydraulic testing system and a resilient modulus triaxial testing system were used in this study. The major components of these systems were: loading system, digital controller, workstation computer, triaxial cell, and linear variable differential transducer (LVDT) deformation measurements system. The resilient modulus testing equipment is schematically shown in Figure 3.4. The following sections describe some of the most noteworthy equipment with respect to loading system, triaxial cell, deformation measurement devices, and data acquisition and control systems.

Loading System

An MTS series 318 load unit consisting of a load frame, and a hydraulic actuator was provided by MTS System Corporation. An MTS TestStar System was used to control the loading system from a workstation computer (Figure 3.12). A repeated dynamic load was programmed by a function generator in the TestStar software from the computer. In this study, a haversine waveform of load shape was used. The loading pulse duration and the rest period were set at 0.1 and 0.9 seconds, respectively.

Triaxial Cell

The triaxial cell (Figure 3.5) and transducers were provided by Research Engineering Inc. The external chamber is made of cast acrylic and can resist the maximum confining pressure of 689 kPa (100 psi). The confining fluid is limited by the air only. The cell is fitted with a safety release valve that is set to release at approximately 758 kPa (110 psi).

The cell came equipped with two pore pressure lines to the cap and two to the base. These four lines are connected to the pore pressure transducer stand through a 3.2 mm (1/8") tube from the valves on the cell to the fittings. The pore pressure to the cap or the base or both at same time can be monitored during the test from the transducer panel meter. This pressure is the inside pore pressure of the test sample. There are two valves attached to the transducer stand to release the air in the testing sample.

The cell or chamber pressure is provided by an air compressor and is adjusted by a pressure regulator. There is also another pore pressure line connected from the valve on the bottom of the cell to the fitting on the pore pressure transducer stand. The cell pressure can be monitored by both a conventional gauge and a pressure transducer.

Deformation Measurement Devices

In this study, for the *T 292-91I* procedure, four LVDTs were mounted inside the triaxial cell. Two of them were positioned in the middle half length of the specimen (10.2-cm) by using diametrically-opposed clamps around the specimen's axis (Figure 3.5). The other two diametrically-opposed LVDTs were attached to the top platen of the test specimen and rested on the top of the cell. All four LVDTs were adjustable and arranged around the specimen evenly. Calibrations were made periodically during the laboratory testing program. This setup was used to compare the resilient modulus measurements obtained from the LVDTs at different locations. A ten-channel signal conditioner was used to condition, amplify, filter, and transmit the signal from the LVDTs to the TestStar data recording system.

Data Acquisition and Control System

The TestStar control system is designed in such way that signal functioning, data acquisition, function generation, closed-loop servo-control, and hydraulic-pressure control are all provided within a single unit; thus, the user interacts with the control console entirely through the keyboard of a personal computer. A personal computer was used to control closed-loop servo feedback systems. The computer can be programmed to scan analog input channels, digitize the signal data, and compare the most recent data to the most current value of intended signal in a fracture of a millisecond.

There are three data modes to define how data is collected: 1) peak/valley levels of each cycle; 2) data at a specified time interval; and 3) data at each time an input channel signal changes a specified amount. Each of these modes can be used to acquire certain data. The mode of the peak and valley levels of each cycle was used in this study. The output of the data acquisition system included a graphic display of sampled dynamic load and displacement waveforms and a data file. The data file format was selected for use with spreadsheet programs (Excel). The collected data were further processed by analyzing, plotting, or implementing a word processing program.

3.3.4.2 Resilient Modulus Test Procedures

The resilient modulus test procedures were basically followed from AASHTO T 292-91I and AASHTO T307-99 (Table 3.2). A deviation from the test procedure was made by using the two additional internally-mounted LVDTs for the full length measurements. The AASHTO T292-91I test procedures are described in the following sections.

Test Setups (T 292-91I)

Prior to testing, the compacted soil specimen was removed from the mold using an extruder. Using a vacuum membrane expander, the membrane was pulled over the specimen and perforated stones. The membrane-enclosed soil specimen with the perforated stones on the top and the bottom was placed onto the bottom platen in the triaxial chamber. The top platen was fitted in place, and the specimen membrane ends were folded over the platens and secured with an O-ring. Two LVDT clamps were affixed to the upper and lower quarter points of the specimen (for 10.2-cm measurements). It was ensured that the LVDT clamps lay in horizontal planes. Then, two LVDTs were installed to the clamps. The other two LVDTs were mounted on the top platen to measure the resilient deformation of the entire 20.3-cm- (8-in.-) long specimen. These four LVDTs were adjusted to the appropriate positions to permit enough travel distance during the testing. The assembly of the triaxial cell was completed by closing the triaxial chamber (Figure 3.5). The drainage valve to the specimen was left open.

Specimen Conditioning

Specimen conditioning was applied to simulate the stress history that exists in field conditions. The procedures for specimen conditioning are described as follows:

a. Load the MTS load frame to the triaxial load cell, and be sure that the load frame is firmly contacted with the triaxial load cell.

b. Turn on the air compressor machine to produce a confining chamber pressure of 103.4 kPa (15 psi) for granular subgrade and embankment soils (T292-91I).

c. Zero the load reading from the control panel. Open a programmed template from Testware program according to the test material. The programmed templates enable the loading device to produce a haversine wave with the fixed load duration of 0.1 seconds with a 0.9-second period of relaxation.

d. Begin the conditioning by applying 1,000 repetitions of a corresponding deviator stress. Monitor the permanent axial deformation occurring during conditioning.

e. After completion of the specimen conditioning phase, monitor the permanent axial deformation occurring to the specimen throughout the remainder of the test. If the permanent axial strain exceeds 5%, the test should be terminated.

Confining Pressure and Loading Sequences

AASHTO T292-91I specifies that after the specimen conditioning phase is completed, the testing phase should begin immediately. However, the resilient modulus values are very much affected by the deviator stresses in some cases, especially when a lower deviator stress follows a much higher one. Therefore, for this study, a 15- to 20-minute rest period was taken prior to the testing phase as suggested by Ping and Ge (1996).

Since the laboratory resilient modulus simulates the conditions in the pavement subgrade, the stress-state should be selected to cover the expected in-service range. Resilient properties of granular specimens should be tested over the range of confining pressures expected within the subgrade layer. A template was created in the TestStar software program to monitor the test sequence. In the test sequence (Table 3.3), the confining pressures decrease while the deviator stresses increase during each confining pressure stage.

Cyclic Loading Procedures

After 15 to 20 minutes of rest period after the specimen conditioning phase, the test phase was completed using the following procedures:

1. Open a template and apply 50 repetitions (T292-91I*)* of smallest deviator stress at the highest confining pressure (T292-91I). The average recoverable deformation of each repetition is recorded automatically.
2. Apply the same repetitions of each of the remaining deviator stresses to be used at the present confining pressure.
3. Decrease (T292-91I) the confining pressure to the next desired level and adjust the deviator stress to the smallest value to be applied at this confining pressure. Prior to applying 50 repetitions (T292-91I) to the specimen, a 15- to 20-minute rest period was used.
4. Increase the deviator stress to the next desired level and continue the process of Steps 2 and 3 until testing has been completed for all desired stress states.
5. Disassemble the triaxial chamber and remove all apparatus from the specimen.

3.3.5 Determination of Resilient Modulus

During the resilient modulus test, after finishing the specimen conditioning stage, a series of tests with different deviator stresses at different confining pressures were performed and the data were recorded for every cycle of each test. However, only the last five cycles of each test were used for analyses following the AASHTO T292-91I procedure.

The resilient modulus (M_r) was calculated from the load and deformation using the following equation:

$$M_r = \frac{\sigma_d}{\varepsilon_R} \tag{3-6}$$

Where σ_d is the deviator stress and ε_R is the resilient or recoverable strain.

3.3.6 Regression Analysis

The test results are reported in a tabular form and in plots of logarithmic graphs that show the variation of the M_R versus the bulk stress (θ). In some cases, the plots required are logarithmic graphs showing the variation of the M_r versus the confining pressure. The regression models are presented as follows:

1. Modulus dependent on bulk stress:

$$M_r = k_1\theta^{k_2} \tag{3-7}$$

2. Modulus dependent on confining pressure:

$$M_r = k_3\sigma_3^{k_4} \tag{3-8}$$

Where,

θ = Bulk stress, sum of the principal stresses, (σ_1 + σ_2 + σ_3)

σ_3 = Confining pressure or minor principal stress

k_1, k_2, k_3, k_4 = Regression constants

3.3.7 Testing Program

The Phase I and Phase II laboratory resilient modulus testing program is summarized in Table 3.4. Eight types of pavement soils obtained from across the state of Florida were tested in the laboratory. Two replicate resilient modulus tests were conducted for each moisture condition of the soils. The soil specimens were tested at the optimum, dried, and soaked conditions.

3.4 SUCTION TEST PROGRAM

The soil suction test was followed from the AASHTO Designation T273-86 to determine the soil suction value at different moisture contents for all eight soil types.

3.4.1 Methodology

The suction test (T273-86) method utilizes thermocouple psychrometers of the Spanner type for determining the total soil suction force.

The thermocouple psychrometer measures relative humidity in soil through a technique called Peltier cooling. If a current is caused to flow through a single thermocouple junction in the proper direction, that particular junction will cool, causing water to condense on it when the dew point is reached. The voltage developed between the thermocouple and reference junction is proportional to the temperature difference and is measured by a microvoltmeter. Because relative humidity is a function of the dew point and ambient temperature, the voltage output can be related to relative humidity or soil suction by a calibration curve.

Laboratory measurements to evaluate total soil suction by thermocouple psychrometer may be made with the apparatus shown in Figure 3.6.

3.4.2 Test Devices

Thermocouple Psychrometer

A total of nine thermocouple psychrometers (PST-55-15-SF) of Spanner type with a known cooling coefficient (Πv) produced by Wescor Inc. were used in this test of water potential measurement. This psychrometer consisted of a sensing thermocouple junction, a chromel-constantan thermocouple, and two reference junctions of copper-constantan and copper-chromel. A PST-55-15-SF Psychrometer was specified as a psychrometer that was covered with a Dutch weave stainless thermocouple shield. SF is the connector with which the connection process can be completed by plugging this connector into the SUREFAST receptacle on the front panel of a microvoltmeter.

To be accurate in water potential measurement, the psychrometer must be kept from contamination to achieve the right output of the evaporation rate. A contaminated junction will result in a reduction of accurate data readings.

Sample Chamber

This part of the equipment was comprised of a sample container, which was a one-pint metal can with wax coated interior to prevent corrosion and sealed by a rubber stopper, and a polystyrene thermal container, which was an insulated box with 1.5 inch thickness of foamed polystyrene and wide enough to accommodate nine sample containers. In the suction test, the thermocouple psychrometer was inserted into a well-sealed sample container within which the soil specimen or calibration solution was placed. Then the whole sample chamber (an insulated box containing nine sample containers) was put into an environmental chamber to achieve the desired equilibrium for output recording.

Microvoltmeter

A microvoltmeter is also defined as a monitoring system. The type used here is WESCOR HR-33T dew point microvoltmeter. It is a self-contained electronic system specifically designed for the measurement of water potential force with thermocouple transducers. It can automatically maintain the temperature of the thermocouple junction at a dew point temperature when operating in dew point mode. The HR-33T shows the water potential information in either the dew point mode or psychrometric mode. In this research, the dew point mode was selected to obtain more accuracy in the water potential measurement.

3.4.3 Calibration

The calibration of the thermocouple psychrometer can be conveniently accomplished using known molalities of a salt solution (sodium chlorides) to correlate with outputs from the thermocouple. This process is conducted by suspending the psychrometer over a salt solution with a known osmotic suction under a constant temperature (isothermal). It requires the same set of apparatus as is illustrated in Figure 3.6 except that the soil specimen was substituted by one piece of filter paper (5.5 cm in diameter) saturated with a 2 ml sodium chloride solution of known water potential. Salt solutions with specified concentration were sealed within sample containers. These cans were subsequently enclosed in an insulated box within an environmental chamber waiting for the humidity in the psychrometers in equilibrium with the relative humidity of the salt solution before the data collection began. Upon using an HR-33T microvoltmeter for data collection, eight amps of cooling current were applied for 30 seconds. The output of the psychrometer was approximately 0.75 microvolts per bar in dew point; these HR-33T readings (E_{τ}) should be corrected to 25°C:

$$E_{25} = \frac{E_\tau}{0.325 + 0.027T} \tag{3-9}$$

These microvoltmeter outputs, which are related to the humidity inside the cans, were recorded at least three times a day after equilibrium was achieved. The last three stabilized readings were averaged as the final output (E_τ). The calibration curve of each psychrometer was expressed by a linear equation:

$$\tau^{o} = A\ E_{25} - B \tag{3-10}$$

Where: τ^{o} = Total soil suction, kPa

A, B = Calibration constant

E_{25} = Psychrometric microvoltmeter readings corrected to 25°C, μV

The standard osmolality of 290, 1000, 1800 mOs/kg with a known sodium chloride concentration in the salt solution were introduced as a calibration standard. The calibration result is demonstrated from Figure 3.7 to Figure 3.13 and in Table 3.5. The concentration of sodium chloride for standard osmolality and their related suction values under a certain temperature is shown in Table 3.6.

3.4.4 Sample Preparation

In a sample preparation for a suction test, sample soils were compacted under optimum moisture contents within a mold that had a height of 8 in. and a 4-in. diameter. The required energy was achieved by using a 10-pound hammer dropped from a height of 18 in. with 25 blows for each layer of eight equal layers. Dry density and optimum moisture were measured immediately after compaction of the sample. Nine 1.5-cubic-in. samples of the specimen were cut from the compacted soil for suction measurement. Of those nine cubic specimens, two were directly sealed into sample containers representing the natural condition of soil; four were wetted by 1, 2, 3, and 4 ml of distilled water respectively right after these samples were cut, and then placed into sample containers; three were dried at room temperature for 1, 3, and 4.5 hours (5.5 hours for the Levy County A-3 soil) respectively before they were sealed into sample containers. Nine cans of specimens were enclosed within the insulated box before they were stored in an environmental chamber for relative humidity equilibrium. Thus, a wide range of water content levels on specific soil was established for the water potential evaluation.

3.4.5 Test Procedure

The temperature equilibrium was attained within a few hours after closing the thermal container (insulated box). Equilibrium of the relative humidity of air measured by the psychrometer and the relative humidity in the soil specimen was acquired within two or three days. Upon using HR-33T for the psychrometer output recording, the $^{\circ}C/\mu V$ button was switched to $^{\circ}C$ and the RANGE button to $30^{\circ}C$ to record temperature output (T) between $0^{\circ}C$ to $30^{\circ}C$. The switch was then changed to μv (psychrometer), the meter was set to zero, and cooling current (8mA) was applied for 30 seconds (identical to calibration); then, the psychrometer output (E_{τ}) was recorded in microvolts. The above process was repeated for every psychrometer in the equipment setup. The last three stabilized readings were averaged as the final output (the same procedure used in calibration). After the readings were completed, the specimen was removed from the containers. The water content was determined in each specimen using a microwave and electronic balance. The E_{τ} value was converted to E_{25} by Equation (3-7), and the soil suction τ of each soil specimen was determined by entering the respective calibration curve with E_{25}.

For the accurate measurement of soil suction, results show that enough power supply should be secured. The RANGE switch was moved to +BATT for a battery check. Batteries were replaced when the voltage reading fell below 16 volts.

3.5 PERMEABILITY TEST PROGRAM

For SR70 A-2-4 soil, the ASTM Designation D5084-90 Flexible Wall Permeameter (FWP) method was performed. This method was proved to be adequate for determining the hydraulic conductivity of compacted porous material like the SR70 A-2-4 soil. The applicable permeability range for this test is less than or equal to 1.0 X 10^{-5}m/s.

For SR70 and Levy County A-3 soils, the ASTM Designation D2434-68 Constant Head method was adopted. This method was proved to be suitable for the establishment of the coefficient of permeability in disturbed granular subrades, like an A-3 soil, having a permeability value higher than 1.0 X 10^{-5}m/s and less than 10% fines passing the No.200 sieve.

For A-2-4 (12%), A-2-4 (20%), A-2-4 (24%), A-2-4 (30%) and Crushed Miami Oolite A-1, the ASTM Designation D5084-90 was adopted. The test equipment was HUMBOLDT Triaxial/Hydraulic Conductivity Testing Equipment. The constant head and flexible wall methods were used for permeability measurements.

3.6 TEST-PIT EXPERIMENTAL PROGRAM

3.6.1 Introduction of Test-pit Test

The Florida DOT test-pit test facility has been adopted to determine the strengths and performance of Florida flexible pavement materials. The test-pit facility re-constructs and simulates the subgrade and base components of a flexible pavement system on a full-scale basis. The major concerns of test-pit test programs are the deformation and equivalent resilient modulus of a layered system under the static loading and cyclic dynamic loading, which is used in modeling the impact of moving vehicles on the pavement. The cyclic loading of a circular plate is activated with a one-second interval within which the loading and resting periods would be 0.1 and 0.9 seconds respectively. For the evaluation of moisture influence on the performance of pavement material, the water table is adjusted within the pit while conducting a plate load test. The research program of DHW requires the ground water table to be adjusted from a drained to flooded condition with four stops. The TDR probes (the principle of which will be addressed in Appendix A) would be deployed within the test-pit for the monitor of moisture profile of pavement material.

The purpose of test-pit experimental program was to evaluate the capillary behavior and resilient modulus of the subgrade materials with changing groundwater levels. The test-pit evaluation of subgrade soils served the following advantages:

(A) The test-pit can be used to simulate the different material components of a pavement system on a full-scale basis.

(B) The test-pit can facilitate the change of water level so as to simulate the different moisture conditions in a practical situation.

(C) Together with a loading system, the test can be used to investigate the deformation characteristics of subgrade materials under the influence of static and dynamic loads.

The capillary action and resilient deformation of the materials under investigation were evaluated with three levels of groundwater elevation: flooded, intermediate levels between the embankment-subgrade interface, and 12 in. above the embankment. To offset the loss due to capillary rise and evaporation, extra water had to be added within the pit to keep the water table constant at each designated elevation prior to the moisture equilibrium and plate load test.

3.6.2 Test-pit Setup

The complete setup of test-pit experiment is mainly comprised of two parts -- full-scale test-pit and loading system.

Test-pit

The FDOT test-pit for the research of design highwater clearance is shaped like a rectangular reinforced concrete vessel that is 24 ft. long, 8 ft. wide and 7 ft. deep. Below the subgrade material (3 ft. in thickness) was the standard embankment that was composed of three layers of different materials. The bottom layer was composed of a bed of 12-in. (305 mm) river gravel that facilitated the upward percolation of ground water. A builder's sand layer that was 12-in. (305 mm) thick rested upon the river gravel and was kept separated with gravel by a permeable filter fabric. The third layer was a 12-in. (305 mm) depth of standard A-3 soil (embankment) that was used as the top layer of simulated embankment.

Loading System

A hydraulic loading device was attached to an over-hanging 24 WF beam which facilitated the transverse movement of the loading device, while the 24 WF beam itself traveled longitudinally above the test pit, thus providing a two-dimensional selection of loading location. A standard 12-in. diameter rigid plate was used to simulate the single wheel load upon the tested soil. Vertical deformations of the soil were measured through linear variable displacement transducer (LVDT). To best simulate the dynamic impact of moving vehicles on the subgrade, the plate loads were conducted in a cyclic manner, one second per cycle with loading periods of 0.1 and 0.9 seconds for the rebound of tested materials. This was consistent with the loading frequency used in laboratory triaxial resilient modulus tests. In order to achieve a certain deformation curve with respect to the number of load cycles, 30,000 load cycles were conducted.

The loading system together with the cross sectional view of a test-pit is illustrated in Figure 3.14.

3.6.3 Method of Analysis

The resilient modulus obtained from the plate load tests on subgrade is based on Boussinesq's theory of deflections at the center of a circular plate. Burmister has extended this theory to a two-layer elastic system. The layers are assumed to be homogeneous, isotropic, and elastic solid with a continuous interface with the bottom layer being infinite in depth. Under these circumstances, the equivalent single-layer resilient modulus under the cyclic loading on a two-layer system (base and subgrade layers) can be derived from the theory of elasticity:

$$E_{eR} = \frac{\pi p a}{\Delta_R}(1-\upsilon^2) \tag{3-11}$$

where:

E_{eR} = Equivalent resilient modulus of a two-layer system

Δ_R = Resilient deflection of the two-layer system at N (number of cyclic load)

p = Surcharge pressure from the circular plate

a = Radius of the circular plate

υ = Poisson's ratio

If υ=0.35 and 0.5, Equation (3-9) will be as follow:

$$E_{eR} = \frac{1.38pa}{\Delta_R} \qquad (\upsilon = 0.35) \tag{3-12}$$

$$E_{eR} = \frac{1.18pa}{\Delta_R} \qquad (\upsilon = 0.50) \tag{3-13}$$

The equivalent modulus is an excellent criterion for the evaluation of the strength of pavement materials. With the decrease of equivalent modulus, deformation increases after the repeated loading. The magnitude of deformation does affect the potential rutting of the pavement. Thus, E_{eR} is a good index for the evaluation of potential pavement rutting. Design consideration of a minimum E_{eR} value can control potential excessive rutting of the pavement.

3.6.4 Time Domain Reflectometry (TDR)

The research of design highwater clearances can only be conducted under a full awareness of seasonal water content under pavement. Time domain reflectometry (TDR) now serves as one of the most reliable nondestructive methods for monitoring both in situ and in lab soil moisture content. Measuring the time period for an electric signal traveling through the guide-rod of a TDR probe as a mediate parameter, direct access to volumetric moisture content of subgrade soils was gained using a Campbell Scientific CS615 Water Content Reflectometer. TDR technique determines the changing moisture content of subgrade soil by measuring the proportionally changing conductivity profile (dielectric constant) within subgrade soil mixture. The basic concept for a TDR probe was described in Appendix A.

The alternative equipment for the collection of moisture data is a moisture cell. It was used in the test-pit to justify the proper operation of CS615 probes and may not be used as a prime access to moisture data because of its insensitivity to the moisture ranging from 4% to 16%. Use of a Time Domain Reflectometer is a relatively dependable approach for measuring the moisture content of granular soil.

Description of the Equipment

Manufactured by Campbell Scientific, the CS615 TDR probe (Figure 3.15) is also known as the Water Content Reflectometer. Its output is a square wave and can be connected to Campbell Scientific datalogger CR10X, CR10.

High-speed electronic components on the circuit board were configured as a bistable multivibrator. The output of the multivibrator was connected to the probe rod, which acted as a wave guide. The oscillation frequency of the multivibrator was dependent on dielectric constant of the soil measured. The dielectric constant was predominantly dependent on the water content. Digital circuitry scaled the multivibrator output to an appropriate frequency for measurement with a datalogger. The CS615 output was essentially a square wave with an amplitude swing of 0.25 VDC. The period of the square wave output ranged from 0.7 to 1.6 milliseconds and was used for the calibration to water content. The measured period can be coverted to moisture content using calibration value.

Two soil properties which can affect the response of the CS615 to changes in water content: high clay content (30% or above) and high electrical conductivity (more than dsm-1, salted soil e.g.). In these cases, the required calibration must be generated for the specific soil.

Conversion to Universal Model Form

Instead of detecting the moisture content of soil through measuring apparent length L_a, CS615 TDR uses time period t as a standard access to volumetric water content. A conversion deduction to universal model using parameter t (travel time of the square wave along the CS615 TDR probe guide rod) helped to establish a better understanding of this equipment.

Refer to the Equation (A-1) in Appendix A, $K_a = (L_a/L_pV_p)^2$, where L_a: apparent length; L_p: actual length of CS615 TDR probe guide rod (0.3M); in this case, the travel distance should be two times the TDR length. For V_p, the ratio of propagation velocity to the speed of light, usually 0.99 is used for maximum resolution. Here 1.0 is used for approximation, thus:

$$L_p\sqrt{K_a} = L_a \qquad (3\text{-}14)$$

$$t = \frac{L_a}{C} = \frac{0.6\sqrt{K_a}}{C} \qquad (3\text{-}15)$$

Where,

C = Speed of light ($3\text{x}10^8$ m/s);

t = Travel time on the rod. Also:

$$V_w(\%) = 0.125\sqrt{K_a} - 0.125 - \frac{\gamma_d}{8G_s\gamma_w} \qquad (3\text{-}16)$$

$$V_w(\%) = 0.125 \times C \times t \div 0.6 - 0.125 - 0.08 = 0.208Ct - 0.205 \qquad (3\text{-}17)$$

It is evident that the above equation is the universal model for volumetric moisture content through use of the CS615 TDR probe.

The Calibration of the CS615 TDR Probe

As mentioned before, the sample soil for calibration represented the model form of all soil types of granular soils without losing accuracy. A standard equation was generated then for measuring each type of soil by using a specific CS615 TDR probe. Thus, the calibration process becomes one of calibrating each individual TDR probe. The following is the calibration data for each of the six TDR probes used in the test-pit test.

$$V_w(\%) = C_0 + C_1 \times t + C_2 \times t^2 \tag{3-18}$$

Where,

t = Time period for the square wave traveling through the guide rod of TDR probe

C_o, C_1, C_2 = Constant for mathematics modeling

The calibration data and calibration curves were presented in Table 3.6 and Figure 3.16.

Note: 1) Since the equipment cannot locate the time value that was in the order of magnitude about 10-9 second, all these period values were amplified at the unit of millisecond. Here Campbell Scientific took 256x128 as the time amplification factor. 2) The apparent length between two inflection points on the trace and TDR travel period were basically identical; the only difference rests upon different interpretation (Campbell Scientific Inc., 1998).

3.6.5 Test Arrangement

The test-pit evaluation was performed at the FDOT State Materials Office. The experimental programs of the Phases I and

II soils are chronologically described in the following sections:

Phase I Program

(A) Levy County A-3 subgrade (4% passing No.200) was compacted and experimented in one half of the test-pit (8 ft. by 6 ft.) from the date of Dec/9/1998 to Apr/8/1999.

(B) SR70 A-3 (8% passing No.200) and A-2-4 (14% passing No.200) subgrades were compacted and experimented upon in one test-pit (8 ft. by 12 ft.) from the date of Apr/13/1999 to Feb/14/2000. Separated by wooden partitions, each of these subgrades accounted for one half of the test-pit area.

Phase II Program

(A) A-2-4 (12%), A-2-4 (20%) and A-2-4 (24%) subgrades were compacted and experimented in one test-pit from the date of Jun/20/2000 to Jan/8/2001, separated by wooden partitions.

(B) A-2-4 (30%) and Oolite were compacted and experimented in one test-pit from Jun/20/2000 to Dec/21/2000.

During the test, three feet of subgrade material was compacted within the test-pit under its optimum moisture condition. The subgrade materials were compacted into seven layers. With the exception of the first and last lifts three inches thick, each lift was six inches in thickness. The CS615 probe was embedded in each of these layers respectively staggering one another, whereas six moisture cells were placed vertically at six inches apart. The circular rigid loading plate was positioned on the mid-point between two columns of vertically arranged CS615 probes.

The compaction data and procedure for the tested soils are presented in Tables 3.7, 3.8, 3.9 and 3.10. The CS615 probe installation and test layout for the first test phase are illustrated in Figures 3.17, 3.18, and 3.19.

The actual views of the test-pit loading system and compaction equipment are illustrated in Figure 3.20 and Figure 3.21.

3.6.6 Test Procedure

Phase I(A): Levy County A-3 soil

Sequence of the plate load test:

- Water table 20 in. below embankment with a 20-psi plate load (without limerock base)
- Water table on the surface of embankment with a 20-psi plate load (without limerock base)
- Water table 12 in. above the embankment with a 20-psi plate load (both with and without 5-in. limerock base) and a 50-psi plate load (with 5-in. limerock Base)

- Water table all the way up to the surface of subgrade (flooded case) with a 20-psi plate load and a 50-psi plate load (with 5-in. limerock base)

The chronological record of the test procedure is summarized in Table 3.7.

Phase I(B): SR70 A-3 & SR70 A-2-4 soil

Sequence of the plate load test:

- Water table at the top of embankment with a 20-psi plate load (without limerock base)
- Water table at 12 in. above the embankment with a 20-psi plate load (without limerock base)
- Water table at 12 in. above the embankment with a 50-psi plate load (with 5-in. limerock base)
- Water table at 36 in. above the embankment with a 50-psi plate load (with 5-in. limerock base)
- Water table at 24 in. below the embankment with a 50-psi plate load (with 5-in. limerock base), two sets of data recorded with one week apart (drained condition)
- Water table back to 36 in. above the embankment with a 50-psi plate load (with 5-in. limerock base)

The chronological record of the test procedure is summarized in Table 3.8.

Phase II(A): A-2-4 12%, A-2-4 20%, A-2-4 24%

Sequence of the plate load test:

- Water table on the surface of embankment with a 20-psi plate load (without limerock base)
- Water table 12 in. above the embankment with a 20-psi plate load (both with and without 5-in. limerock base) and a 50-psi plate load (with 5-in. limerock Base);
- Water table all the way up to the surface of subgrade (flooded case) with a 50-psi plate load (with 5-in. limerock base)

The chronological record of the test procedure is summarized in Table 3.9.

Phase II(B): A-2-4 30%

Sequence of the plate load test:

- Water table on the surface of embankment with a 20-psi plate load (without limerock base)
- Water table 12 in. above the embankment with a 20-psi plate load (without 5-in. limerock base) and a 50-psi plate load (with 5-in. limerock Base)
- Water table all the way up to the surface of subgrade (flooded case) with a 50-psi plate load (with 5-in. limerock base)

The chronological record of the test procedure is summarized in Table 3.10.

Phase II(B): Oolite A-1

Sequence of the plate load test:

- Water table 12 in. above the embankment with a 50-psi plate load (without 5-in. limerock base);
- Water table all the way up to the surface of subgrade (flooded case) with a 50-psi plate load (with 5-in. limerock base)

The chronological record of the test procedure is summarized in Table 3.10.

3.7 PHASE III TEST-PIT TEST PROGRAM

3.7.1 Three Additional Test Materials

Three additional weak subgrade materials under evaluation in this supplemental research study were Spring Cemetery (A-2-4, 15% passing No.200), Branch (A-2-4, 23% passing No.200), and Iron Bridge (A-2-6/A-2-4, 31% passing No.200). The Iron Bridge soil is a borderline soil between A-2-4 and A-2-6, and some of the sample tests by the State Materials Office (SMO) showed it to be an A-2-4 soil. To make it noticeable different from the other soils, the Iron Bridge soil is designated as an A-2-6 soil in this report. The basic characteristics of the three subgrade soils are appended in Table 3.1. The resilient modulus was measured using AASHTO T307-99 with the full-length LVDT position inside the triaxial cell. The permeability was obtained at 7 psi effective stress using ASTM D5084-9. The samples were compacted to approximately 100% of the Standard Proctor maximum unit weight.

3.7.2 Test-pit Test

3.7.2.1 Test-pit setup

The test-pit setup basically followed the format used for the Phase I and Phase II tests of the eight soils. Some modifications are noted in the following sections.

Dimension. The dimension of the test-pit for Phase III program was twenty four feet long, nine feet wide and six feet deep. Each of the three soils was compacted and experimented upon simultaneously in one-third of the new test pit (24 ft. by 9 ft.). A 12-in. layer of stabilized subgrade and a 24-in. subgrade layer were constructed on top of a 24-in. existing A-2-4 soil layer. Beneath the existing A-2-4 soil layer were the 9-in. builder’s sand layer and 9-in. river-gravel that facilitated the upward percolation of groundwater. The cross sectional view of the new test-pit is illustrated in Figure 3.22.

Compaction Techniques. The 12-in. top layer of the tested material was compacted into 98% of Modified Proctor maximum unit weight (AASHTO T-180), while the bottom 24-in. subgrade layer was compacted into 100% of Standard Proctor maximum unit weight (AASHO T-99). The compaction data are presented in Table 3.11.

Moisture Content Measurement. Two types of moisture content measurement devices were used in this study: the nuclear gauge and the TDR probes. A new TDR-based apparatus from ESI (Environmental Sensors Inc.) called "Moisture Point" was used to monitor the moisture profile of tested materials. The modified H probe, which is at 6 in. below the surface of the subgrade and has cables attached to it, was used to measure the moisture profile of the bottom five segments, and a regular K probe was used to get the top six inches of subgrade and the surface (limerock) moisture profile. The nuclear gauge measured the moisture content of the top-layer material and the moisture data was used for the plate load test analysis. The Backscatter mode of the nuclear gauge was adopted in this study. A picture of the measuring devices is shown in Figure 3.23. The installation and test layout of these measuring devices are illustrated in Figure 3.24.

3.7.2.2 Test Procedure

Nine levels of groundwater table condition were designated to cover various conditions of the water content under the pavement. Figure 3.22 illustrates the various levels of water table condition for the plate load test with and without a limerock base layer. The plate load tests under nine different water levels were further described as follows:

Test Condition A: water level at the interface of the subgrade and embankment with a 20-psi plate load (without 5-in. limerock base)

Test Condition B: water level at 12 in. above the surface of the embankment A-2-4 soil with a 20-psi plate load (without 5-in. limerock base)

Test Condition C: water level at 24 in. above the surface of the embankment A-2-4 soil with a 20-psi plate load (without 5-in. limerock base)

Test Condition D: water level at the surface (interface) of the embankment A-2-4 soil with a 50-psi plate load (with 5-in. limerock base)

Test Condition E: water level at 12 in. above the surface of the embankment A-2-4 soil with a 50-psi plate load (with 5-in. limerock base)

Test Condition F: water level at 24 in. above the surface of the embankment A-2-4 soil with a 50-psi plate load (with 5-in. limerock base)

Test Condition G: water level all the way up to the surface of stabilized subgrade layer (flooded case) with a 50-psi plate load (with 5-in. limerock base)

Test Condition H: water level drained down to 24 in. above the surface of the embankment A-2-4 soil with a 50-psi plate load (with 5-in. limerock base)

Test Condition I: water level further drained down to 12 in. above the surface of the embankment A-2-4 soil with a 50-psi plate load (with 5-in. limerock base)

A chronological record of the Phase III test procedure is summarized in Table 3.12. Three replicate plate load tests were conducted on each soil material after the establishment of moisture equilibrium at each water level. The designated plate load test numbers and their corresponding loading conditions are further described in Chapter 5.

Table 3.1 Characteristics of Eleven Subgrade Materials

Material		Passing No.200 Sieve	Percent of Clay	Max. Dry Density (modified)		Opt. Moisture Content (Modified)	LBR*	CBR
		(%)	(%)	(kN/M3)	(pcf)	(%)		
Phase I	Levy Co. A-3	4	N/A	16.7	106.5	10	22	18
	SR-70 A-3	8	6	17.6	112	11.5	45	36
	SR-70 A-2-4	14	10	19.2	122	10.5	124	99
Phase II	A-2-4 (12%)	12	3	17.3	110.6	12.1	30	24
	A-2-4 (20%)	20	8	19.5	124.4	10	146	117
	A-2-4 (24%)	24	5	18.2	116.3	10.7	69	55
	A-2-4 (30%)	30	N/A	18.2	116	12	72	58
	Oolite	N/A	N/A	20.8	132.6	7.6	194	155

Material		Passing No.200 Sieve	Percent of Clay	Max. Dry Density (Modified)		Opt. Moisture Content (Modified)	Max. Dry Density (Standard)		Opt. Moisture Content (Standard)	LBR	CBR
		(%)	(%)	(kN/M^3)	(pcf)	(%)	(kN/M^3)	(pcf)	(%)		
Phase III	Spring Cemetery A-2-4	15	4	18.6	118.4	9.3	18.6	118.2	9.2	83	66
	Branch A-2-4	23	6	21.1	134.7	7.2	20.1	128.4	8.8	132	106
	Iron Bridge A-2-6	31	16	20.8	132.4	8.2	19.4	123.3	10.3	127	102

* CBR (California Bearing Ratio) = 0.8 * LBR (Limerock Bearing Ratio)

Table 3.2 Comparison of Resilient Modulus Test Procedures for Granular Soils

Test method		AASHTO T 292-91I			AASHTO T307-99		
Procedure		Confining Pressure	Deviator Stress	Load Number	Confining Pressure	Deviator Stress	Load Number
Unit		psi	psi		psi	psi	
Conditioning		15	12	1000	6	4	500
Testing Sequences	1	15	7	50	6	2	100
	2	15	10	50	6	4	100
	3	15	15	50	6	6	100
	4	10	5	50	6	8	100
	5	10	7	50	6	10	100
	6	10	10	50	4	2	100
	7	10	15	50	4	4	100
	8	5	3	50	4	6	100
	9	5	5	50	4	8	100
	10	5	7	50	4	10	100
	11	5	10	50	2	2	100
	12	2	3	50	2	4	100
	13	2	5	50	2	6	100
	14	2	7	50	2	8	100
	15				2	10	100

Table 3.3 Raw Data and Calculation Procedure

MATERIAL: PANAMA SAND		LOCATION: PANAMA CITY BEACH, FLORIDA	
OPT. MOISTURE:	8.5%	DRY DENSITY:	19.37 kN/m^3
MAX. DRY DEN.:	19.64 kN/m^3	MOISTURE:	7.63%
LBR:	88.00	TEST DATE:	5/9/95

Confining Pressure	Raw Data							Calculation Results		
	Axial Load	LVDT (10.2-cm)	LVDT (10.2-cm)	LVDT (20.3-cm)	LVDT (20.3-cm)	Axial Segments	Load Cycles	Axial Strain (10.2-cm)	Axial Strain (20.3-cm)	Deviator Stress
kPa	kN	mm	mm	mm	mm	segments	Cycles			kPa
103.35	-0.3433	0.902544	0.811618	6.466652	6.903526	1				
103.35	-0.0015	0.889247	0.798318	6.499898	6.935017	2	1	0.000131	0.000159	42.188
103.35	-0.3515	0.902894	0.812143	6.466214	6.902651	4				
103.35	-0.0015	0.889247	0.797968	6.500335	6.934580	5	2	0.000137	0.000163	43.191
103.35	-0.356	0.902894	0.812143	6.465339	6.902651	7				
103.35	-0.0032	0.889597	0.798318	6.499898	6.934143	8	3	0.000133	0.000163	43.541
103.35	-0.3589	0.902894	0.812318	6.465339	6.902213	10				
103.35	-0.0025	0.889422	0.798668	6.499898	6.934580	11	4	0.000133	0.000165	43.985
103.35	-0.3645	0.903244	0.812843	6.465339	6.901339	13				
103.35	-0.0019	0.889247	0.798318	6.499460	6.935017	14	5	0.000140	0.000167	44.751
103.35	-0.3718	0.903594	0.813018	6.463589	6.900464	16				
103.35	-0.003	0.889597	0.798318	6.499898	6.935017	17	6	0.000141	0.000174	45.517
103.35	-0.3753	0.903768	0.813193	6.464027	6.900027	19				
103.35	-0.0026	0.889422	0.798493	6.499460	6.935454	20	7	0.000143	0.000174	45.999
103.35	-0.3737	0.903768	0.813193	6.463589	6.901339	22				
103.35	-0.0016	0.889422	0.798318	6.499460	6.934580	23	8	0.000144	0.000170	45.914
103.35	-0.3794	0.903594	0.813368	6.463589	6.900027	25				
103.35	-0.0042	0.889772	0.798318	6.499023	6.933705	26	9	0.000142	0.000170	46.293
103.35	-0.3818	0.904293	0.813543	6.463152	6.900027	28				
103.35	-0.0017	0.889422	0.798493	6.499898	6.934143	29	10	0.000147	0.000174	46.907
103.35	-0.3904	0.904293	0.813893	6.462277	6.899152	31				
103.35	-0.005	0.889772	0.798668	6.499460	6.934580	32	11	0.000146	0.000179	47.560
103.35	-0.382	0.904468	0.813543	6.461840	6.899152	34				
103.35	-0.0042	0.889597	0.798318	6.499023	6.933705	35	12	0.000148	0.000177	46.624
103.35	-0.3899	0.904643	0.813718	6.462277	6.898714	37				
103.35	-0.0056	0.889772	0.798493	6.498148	6.934143	38	13	0.000148	0.000175	47.427
103.35	-0.3847	0.904293	0.813893	6.462715	6.900027	40				
103.35	-0.0028	0.889422	0.798318	6.499023	6.934580	41	14	0.000150	0.000174	47.134
103.35	-0.3862	0.904118	0.813718	6.463152	6.899152	43				
103.35	-0.0039	0.889422	0.798143	6.498585	6.933705	44	15	0.000149	0.000172	47.172
103.35	-0.3909	0.904293	0.813893	6.461402	6.898714	46				
103.35	-0.0026	0.889422	0.798493	6.499023	6.934143	47	16	0.000149	0.000180	47.919
103.35	-0.3904	0.904293	0.813718	6.461840	6.899152	49				
103.35	-0.0026	0.889597	0.798318	6.499460	6.934143	50	17	0.000148	0.000179	47.853
103.35	-0.3895	0.904293	0.814068	6.460965	6.898714	52				
103.35	-0.005	0.889597	0.798493	6.498585	6.934143	54	18	0.000149	0.000180	47.456
103.35	-0.3889	0.904468	0.814068	6.461840	6.898714	55				
103.35	-0.0064	0.889772	0.798843	6.499023	6.934143	56	19	0.000147	0.000179	47.210
103.35	-0.3917	0.904468	0.814068	6.461402	6.898277	58				
103.35	-0.0045	0.889422	0.798318	6.498585	6.933268	59	20	0.000152	0.000178	47.777
103.35	-0.3935	0.904818	0.813893	6.460965	6.897403	61				
103.35	0.0008	0.889597	0.798318	6.499023	6.934580	62	21	0.000152	0.000185	48.657
103.35	-0.3961	0.904993	0.814593	6.461402	6.897840	64				
103.35	0.0005	0.889597	0.798493	6.499460	6.935017	66	22	0.000155	0.000185	48.950
103.35	-0.3918	0.904468	0.814068	6.461402	6.898714	67				
103.35	-0.0039	0.889597	0.798143	6.498585	6.933268	68	23	0.000152	0.000177	47.863
103.35	-0.3892	0.904643	0.814243	6.460965	6.899152	70				
103.35	-0.0036	0.889772	0.798318	6.498585	6.934580	71	24	0.000152	0.000180	47.579
103.35	-0.3925	0.904293	0.814243	6.461402	6.898714	73				
103.35	-0.004	0.889772	0.798493	6.499023	6.933705	74	25	0.000149	0.000179	47.938

Table 3.3-continued

Confining Pressure	Raw Data							Caculation Results		
	Axial Load	LVDT (10.2-cm)	LVDT (10.2-cm)	LVDT (20.3-cm)	LVDT (20.3-cm)	Axial Segments	Load Cycles	Axial Strain (10.2-cm)	Axial Strain (20.3-cm)	Deviator Stress
kPa	kN	mm	mm	mm	mm	segments	Cycles			kPa
103.35	-0.3884	0.904468	0.813893	6.461402	6.899152	76				
103.35	-0.0035	0.889597	0.798493	6.499023	6.933705	77	26	0.000149	0.000178	47.494
103.35	-0.3944	0.904468	0.814243	6.460965	6.898714	79				
103.35	-0.0046	0.889772	0.798143	6.498585	6.933705	80	27	0.000152	0.000179	48.099
103.35	-0.3902	0.904643	0.813893	6.460965	6.898714	82				
103.35	-0.0042	0.889772	0.798493	6.498148	6.934143	83	28	0.000149	0.000179	47.626
103.35	-0.3971	0.904818	0.814418	6.460965	6.897403	85				
103.35	-0.0058	0.889947	0.798493	6.498148	6.933705	87	29	0.000152	0.000181	48.288
103.35	-0.3919	0.904468	0.814068	6.461402	6.898277	88				
103.35	-0.0055	0.889772	0.798318	6.498585	6.934143	89	30	0.000150	0.000180	47.683
103.35	-0.3848	0.904468	0.813718	6.460965	6.898714	91				
103.35	-0.0039	0.889247	0.798143	6.498585	6.934143	92	31	0.000152	0.000180	47.002
103.35	-0.3991	0.904993	0.814068	6.460527	6.897403	94				
103.35	-0.0068	0.889772	0.798493	6.498148	6.933268	95	32	0.000152	0.000181	48.411
103.35	-0.3968	0.904468	0.814243	6.460965	6.897403	97				
103.35	-0.0085	0.889772	0.798493	6.498585	6.933705	98	33	0.000150	0.000182	47.919
103.35	-0.3906	0.904468	0.814068	6.461840	6.898277	100				
103.35	-0.0019	0.889597	0.798318	6.498585	6.934143	101	34	0.000151	0.000179	47.976
103.35	-0.3919	0.904468	0.814243	6.460965	6.898714	103				
103.35	-0.0065	0.889772	0.798493	6.499023	6.933705	104	35	0.000150	0.000180	47.569
103.35	-0.4046	0.905343	0.814593	6.460090	6.897403	106				
103.35	-0.0048	0.889772	0.798318	6.499023	6.933705	107	36	0.000157	0.000185	49.338
103.35	-0.3916	0.904818	0.814068	6.461840	6.898277	109				
103.35	-0.0055	0.889772	0.798318	6.498585	6.934143	110	37	0.000152	0.000179	47.645
103.35	-0.3882	0.904643	0.813893	6.460965	6.899152	112				
103.35	-0.0012	0.889597	0.798143	6.498585	6.934143	113	38	0.000152	0.000179	47.758
103.35	-0.3876	0.904643	0.813718	6.462277	6.898714	115				
103.35	-0.0066	0.889947	0.798843	6.497711	6.932830	116	39	0.000146	0.000171	47.011
103.35	-0.3925	0.904468	0.814068	6.461402	6.897840	118				
103.35	-0.0039	0.889947	0.798493	6.499023	6.933705	119	40	0.000148	0.000181	47.957
103.35	-0.3898	0.904643	0.813893	6.461840	6.898277	121				
103.35	-0.0029	0.889597	0.798318	6.499023	6.933705	122	41	0.000151	0.000179	47.740
103.35	-0.3851	0.904468	0.813718	6.461840	6.898277	124				
103.35	-0.0063	0.890122	0.798668	6.499023	6.933705	125	42	0.000145	0.000179	46.747
103.35	-0.3942	0.904468	0.814418	6.461402	6.898714	127				
103.35	-0.0065	0.889772	0.798493	6.498148	6.934143	129	43	0.000151	0.000178	47.844
103.35	-0.3964	0.904643	0.814243	6.460965	6.898714	130				
103.35	-0.0075	0.889772	0.798668	6.498148	6.933268	131	44	0.000150	0.000177	47.995
103.35	-0.3914	0.904818	0.814243	6.462277	6.898277	133				
103.35	-0.0052	0.889772	0.798668	6.498585	6.933268	134	45	0.000151	0.000175	47.664
103.35	-0.3932	0.904643	0.814418	6.461402	6.897840	136				
103.35	-0.0006	0.889597	0.798143	6.499460	6.933268	137	46	0.000154	0.000181	48.458
103.35	-0.396	0.904643	0.813893	6.461840	6.898277	139				
103.35	-0.0044	0.889597	0.798493	6.498585	6.932830	140	47	0.000150	0.000175	48.326
103.35	-0.3907	0.904643	0.814243	6.461840	6.898277	142				
103.35	-0.0031	0.889247	0.797968	6.498585	6.933705	143	48	0.000156	0.000178	47.834
103.35	-0.3945	0.904818	0.813893	6.460965	6.898277	145				
103.35	-0.0087	0.889772	0.798493	6.497711	6.932393	146	49	0.000150	0.000174	47.607
Average of the last five cycles										
103.35	-0.3932	0.904713	0.814138	6.461665	6.898189					
103.35	-0.0044	0.889597	0.798353	6.498585	6.933093			0.000152	0.000177	47.978
Resilient modulus from 10.2-cm measurement = 47.977/0.000152 = 315493.29 kPa = 315.20 MPa										
Resilient modulus from 20.3-cm measurement = 47.977/0.000176 = 271473.26 kPa = 271.22 MPa										

Table 3.4 Summary of Laboratory Resilient Modulus Tests

Soils	Condition	Sample No.	Dry Density (pcf)	Moisture Content(%) @ Compaction	Moisture Content(%) before Test
A-3 Levy County	Dried	A3LEVYD1	106	9.50	8.08
		A3LEVYD2	105.8	9.60	4.30
	Optimum	A3LEVYO1	105.6	9.50	9.50
		A3LEVYO2	105.8	9.60	9.60
	Soaked	A3LEVYS1	105.8	9.50	13.47
		A3LEVYS2	105.27	9.50	15.77
		A3LEVYS3	105.4	9.50	15
		A3LEVYS4	105.1	9.60	15.27
SR70 A-3	Dried	A3SR70D1	111.6	11.40	7.82
		A3SR70D2	110.7	11.40	5.31
		A3SR70D3	108.8	11.40	4.48
		A3SR70D4	110.63	11.40	4.00
	Optimum	A3SR70O1	111	11.40	11.40
		A3SR70O2	110.8	11.40	11.40
	Soaked	A3SR70S1	109.7	11.40	13.41
		A3SR70S2	109.7	11.40	13.69
A-2-4 12%	Dried	A2412%D1	110.6	12.10	7.10
		A2412%D2	110.7	12.10	7.04
	Optimum	A2412%O1	109.3	12.10	12.10
		A2412%O2	109.8	12.10	12.10
	Soaked	A2412%S1	109.6	12.10	14.60
		A2412%S2	109.6	12.10	14.60
SR70 A-2-4	Dried	A24SR70D1	120.3	10.60	8.41
		A24SR70D2	120.6	10.60	7.76
		A24SR70D3	120.9	10.60	3.12
	Optimum	A24SR70O1	120.4	10.80	10.80
		A24SR70O2	119.8	10.39	10.39
	Soaked	A24SR70S1	121.4	10.60	11.23
		A24SR70S2	120	10.60	11.70

Table 3.4-continued

Soil	Condition	Sample No.	Dry Density (pcf)	Moisture Content(%) @ Compaction	Moisture Content(%) before test
A-2-4 20%	Dried	A2420%D1	117.3	10.00	8.26
		A2420%D2	117.9	10.00	7.32
	Optimum	A2420%O1	117.9	10.00	10.00
		A2420%O2	118.9	10.00	10.00
	Soaked	A2420%S1	119	10.00	11.57
		A2420%S2	118	10.00	12.27
A-2-4 24%	Dried	A2424%D1	114	10.70	7.65
		A2424%D2	116	10.70	7.72
	Optimum	A2424%O1	115.1	10.70	10.70
		A2424%O2	115.1	10.70	10.70
	Soaked	A2424%S1	116.9	10.70	12.00
		A2424%S2	116.9	10.70	11.45
A-2-4 30%	Dried	A2430%D1	116.1	12.00	7.00
		A2430%D2	115.12	12.00	6.30
	Optimum	A2430%O1	115.8	12.00	12.00
		A2430%O2	115.1	12.00	12.30
	Soaked	A2430%S1	116.4	12.00	13.40
		A2430%S2	116	12.00	13.20
Oolite	Dried	OOLITED1	131.35	7.80	5.60
		OOLITED2	131.3	7.80	4.40
	Optimum	OOLITEO1	131.08	7.80	7.80
		OOLITEO2	131.22	7.80	7.80
	Soaked	OOLITES1	131.52	7.80	8.20
		OOLITES2	131.2	7.80	8.09

Table 3.5 Equations of Calibration Line for Seven Psychrometers

Psychrometer Number	Cooling Coefficient	Calibration Equation	R^2
1	52	$y= 262.74E_{25}+359.83$	0.9995
2	53	$y= 152.68E_{25}-138.37$	0.9973
3	54	$y= 191.31E_{25}+307.75$	1
4	55	$y= 152.29E_{25}-212.7$	0.9984
5	56	$y= 146.72E_{25}-17.414$	0.9998
6	56(B)	$y= 160.51E_{25}-267.39$	1
7	58	$y= 146.08E_{25}-210.7$	0.9995

Nacl/100g solution (gram)	0.9094	3.115	5.463
Osmolality (mM/kg)	290	1000	1800
Suction at 25°C (kPa)	727.6	2509	4516

Table 3.6 Calibration Data for CS615 Probes

Probe No.	CS615 water content	CS615 Period	Gravimetric Water Content	Bulk Density (pcf)	Volumetric Water Content	C_0	C_1	C_2
1	0.422	1.297	0.122	113.1	0.22	-0.214	0.222	0.087
2	0.361	1.227	0.11	116.8	0.21	-0.214	0.195	0.123
3	0.442	1.319	0.09	117.9	0.17	-0.214	0.282	0.007
4	0.315	1.172	0.092	117.8	0.17	-0.214	0.215	0.096
5	0.277	1.123	0.092	115.6	0.17	-0.214	0.177	0.147
6	0.149	0.945	0.073	109.7	0.128	-0.214	0.009	0.373

Table 3.7 Test-pit Test Procedure for Levy County A-3 Soil

Date	Test Procedure
12/9/99 -12/15/99	Levy County sand (A-3) was compacted in test pit with moisture sensor put in place.
12/15/98 -12/22/98	Water in subgrade was allowed to drain and stabilize with elevation 20 in. below embankment
12/30/1998	First plate load was performed when moisture condition at each level came to their steady state.
1/5/99--1/26/99	Water table was raised gradually to the surface of embankment
2/5/1999	Moisture content stabilized, second plate load test was conducted.
2/5/99--2/26/99	Water table was raised to 12 in. above embankment and moisture at each level reached to its steady state on 2/26/99, then the third plate load test was conducted.
2/26/99--3/3/99	5-inch thickness of lime rock base was built on the top of subgrade soil on 3/3/99,
3/23/1999	Moisture condition stabilized. plate load test with loading pressure 20 psi was conducted
3/24/1999	Plate load tests with loading pressure 50 psi was conducted
3/24/99--3/31/99	Water table was raised to the top of subgrade. With moisture equilibrium achieved at each level
3/31/1999	Plate load tests were conducted under 50 psi
4/1/1999	Plate load tests were conducted under 20 psi
4/5/1999	Water was drained down to 20 in. below embankment
4/8/1999	Test pit was excavated

Table 3.8 Test-Pit Test Procedures for SR70 A-3 & A-2-4 Soil

Date	Test Procedure
4/13/99--4/29/99	SR-70 A-3 and A-2-4 soils were compacted in test-pit with CS615 probes put in place
4/29/99--5/17/99	Water in subgrade soil was allowed to drain and stabilized with elevation 24 in. below the embankment
5/17/99--6/10/99	Water table was raised to 12 in. below the embankment. Moisture content was stabilized
6/10/99--7/22/99	Water table was raised to the top of embankment. Moisture content was stabilized
7/19/1999	Plate load tests were conducted for A-3 (20 psi)
7/20/1999	Plate load tests were conducted for A-2-4 (20 psi)
7/22/99--9/3/99	Water table in subgrade was raised to 12 in. above the embankment
8/24/1999	Plate load tests were conducted for A-2-4 (20 psi)
8/25/1999	Plate load tests were conducted for A-3 (20 psi)
9/1/1999	5-inch thickness of limerock base was built on top of the subgrade soil
9/2/1999	Plate load tests were conducted for A-2-4 (50 psi)
9/3/1999	Plate load tests were conducted for A-3 (50 psi)
9/03/99--10/11/99	Water table was raised to the surface of subgrade.
9/29/1999	Plate load test was conducted for A-3 (50 psi)
9/30/1999	Plate load test was conducted for A-2-4 (50 psi)
10/5/1999	Another plate load test was conducted for A-3 (50 psi)
10/11/99--1/06/00	Water table dropped all the way down to 24 in. below the embankment
12/28/1999	Plate load tests were conducted for A-2-4 (50 psi)
12/29/1999	Plate load tests were conducted for A-3 (50 psi)
1/4/1999	Plate load tests were conducted A-3 (50 psi)
1/5/1999	Plate load tests were conducted A-2-4 (50 psi)
1/06/00-- 2/14/00	Water table moved back to the surface of subgrade soil
2/1/2000	Plate load tests were conducted for A-2-4 (50 psi)
2/2/2000	Plate load tests were conducted for A-3 (50 psi)

Table 3.9 Test-Pit Test Procedures for A-2-4 (12%, 20% & 24%) Soil

Date	Test Procedure
6/20/2000	Water table dropped all the way down to 24 in. below the embankment
8/4/2000	Raise Water Table to top of embankment
9/19/00~9/26/00	Plate load tests were conducted for A-2-4 (12%, 20%, 24%) (20 psi)
9/26/2000	Raise Water Table to 12" above embankment
11/1/00~11/14/00	Plate load tests were conducted for A-2-4 (12%, 20%, 24%) (20 psi)
12/1/2000	Limerock cap placed
12/11/00~12/21/00	Plate load tests were conducted for A-2-4 (12%, 20%, 24%) (50 psi)
12/21/2000	Raise Water Table to bottom of limerock
2/26/01~3/8/01	Plate load tests were conducted for A-2-4 (12%, 20%, 24%) (50 psi)
3/12/2001	Lower Water table to 12" above embankment

Table 3.10 Test-Pit Test Procedure for A-2-4 (30%) & Oolite

Date	Test Procedure
6/20/2000	Water table dropped all the way down to 24 in. below the embankment
7/7/2000	Raise Water Table to top of embankment
8/16/2000	Raise Water Table to 12" above embankment
10/5/2000	Plate load tests were conducted for A-2-4 (30%) (20 psi)
10/6/00~10/17/00	Plate load tests were conducted for A-2-4 (30%) (20 psi) and Oolite (50psi)
11/28/2000	Limerock cap placed
12/11/00~12/18/00	Plate load tests were conducted for A-2-4 (30%) and Oolite (50 psi)
12/21/2000	Raise Water Table to bottom of limerock
2/7/01~2/12/01	Plate load tests were conducted for A-2-4 (30%) and Oolite (50 psi)
3/12/2001	Lower Water Table to 12" above embankment

Table 3.11 Test-Pit Subgrade and Embankment Compaction Data

Soil	**Laboratory Density**				**Test-Pit Density**													
	MOD		STD		Lift #1		Lift #2		Lift #3		Lift #4		Lift #5		Lift #6		Limerock	
	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)	OMC (%)	MDD (pcf)
Spring Cemetery	9	120	10	116	7.3	115.8	8.5	115.9	7.8	116.6	9.2	116.1	9.5	117.5	8	117.3	10.4	115.1
Branch	7	134	8.5	127	8.6	126.3	8	126.4	7.8	126.6	8.6	127.6	5.1	130.8	6.6	131.1	10.2	114.2
Iron Bridge	8	132	10	124	10.1	124.5	7.4	126.4	8.1	124.1	8	123.8	7.8	129.5	7.3	131.1	10.6	113.5

Note : 1. MOD : Modified Proctor
2. STD : Standard Proctor
3. OMC : Optimum Moisture Content
4. MDD : Maximum Dry Density

Table 3.12 Test-Pit Phase III Test Procedure for Three Additional Soils

Date	Test Procedure
7/11/2005	Material placing completed
7/19/2005	Testing began all materials at -3.0' WT
8/4/2005	Testing finished all materials at -3.0' WT and the Water Table raised to -2.0 WT
8/11/2005	Noticed Capillary rise had stabilized on Spring Cemetery material
8/12/2005	Started testing on Spring Cemetery material only 8/12/2005
8/16/2005	Finished testing on Spring Cemetery material only 8/16/2005
9/13/2005	Started testing on Branch & Iron Bridge materials
9/29/2005	Finished testing on Branch & Iron Bridge materials
10/2/2005	Water table raised to -1.0' WT
12/6/2005	Testing began all materials at -1.0' WT
12/22/2005	Testing finished all materials at -1.0' WT
1/3/2006	Water table lowered to -3.0'
1/9/2006	5" of Limerock placed on
3/6/2006	Testing began all materials at -3.0' WT w/Limerock
3/17/2006	Testing finished all materials at -3.0' WT w/Limerock
3/20/2006	Water table raised to -2.0' WT
5/15/2006	Testing began all materials at -2.0' WT w/Limerock
5/25/2006	Testing finished all materials at -2.0' WT w/Limerock
5/31/2006	Water table raised to -1.0 WT
7/31/2006	Next phase of testing to begin on
8/9/2006	Testing finished all materials at -1.0' WT w/Limerock
8/17/2006	Water table raised to bottom of Limerock
10/16/2006	Testing to begin all materials with WT at bottom Limerock
10/26/2006	Testing finished all materials at WT at bottom Limerock
10/31/2006	Water table lowered to -1.0 WT
1/2/2007	Next phase of testing to begin on
1/17/2007	Testing finished all materials at -1.0' Drawdown WT w/Limerock
1/18/07	Water table lowered to -2.0 WT
3/19/2007	Next phase of testing to begin on
4/14/2007	Testing finished all materials at -2.0' Drawdown WT w/Limerock

Figure 3.1 Samples under Soaking

Figure 3.2 Sample in Mold before Soaking

Figure 3.3 Samples under Drying

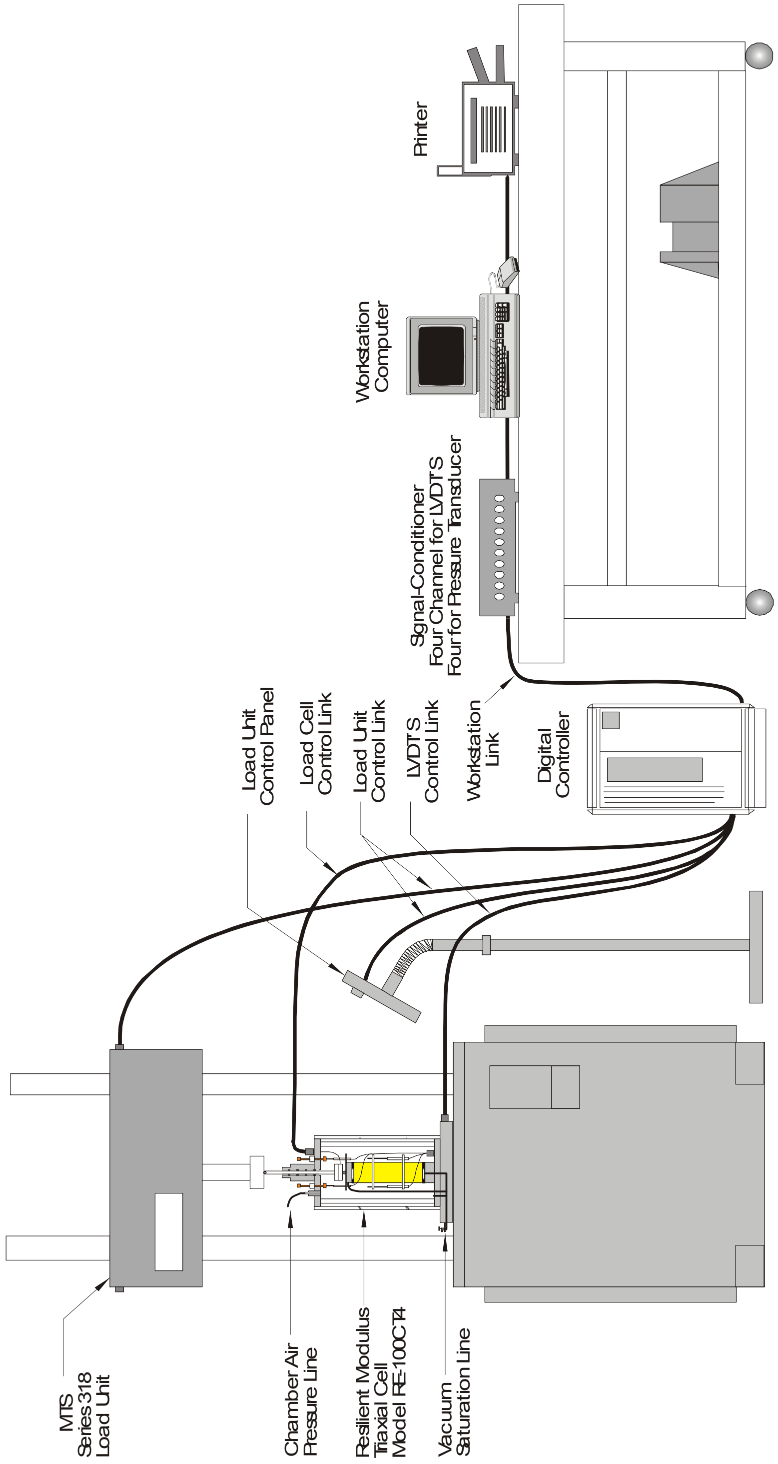


Figure 3.4 Sketch of the Resilient Modulus Testing Equipment

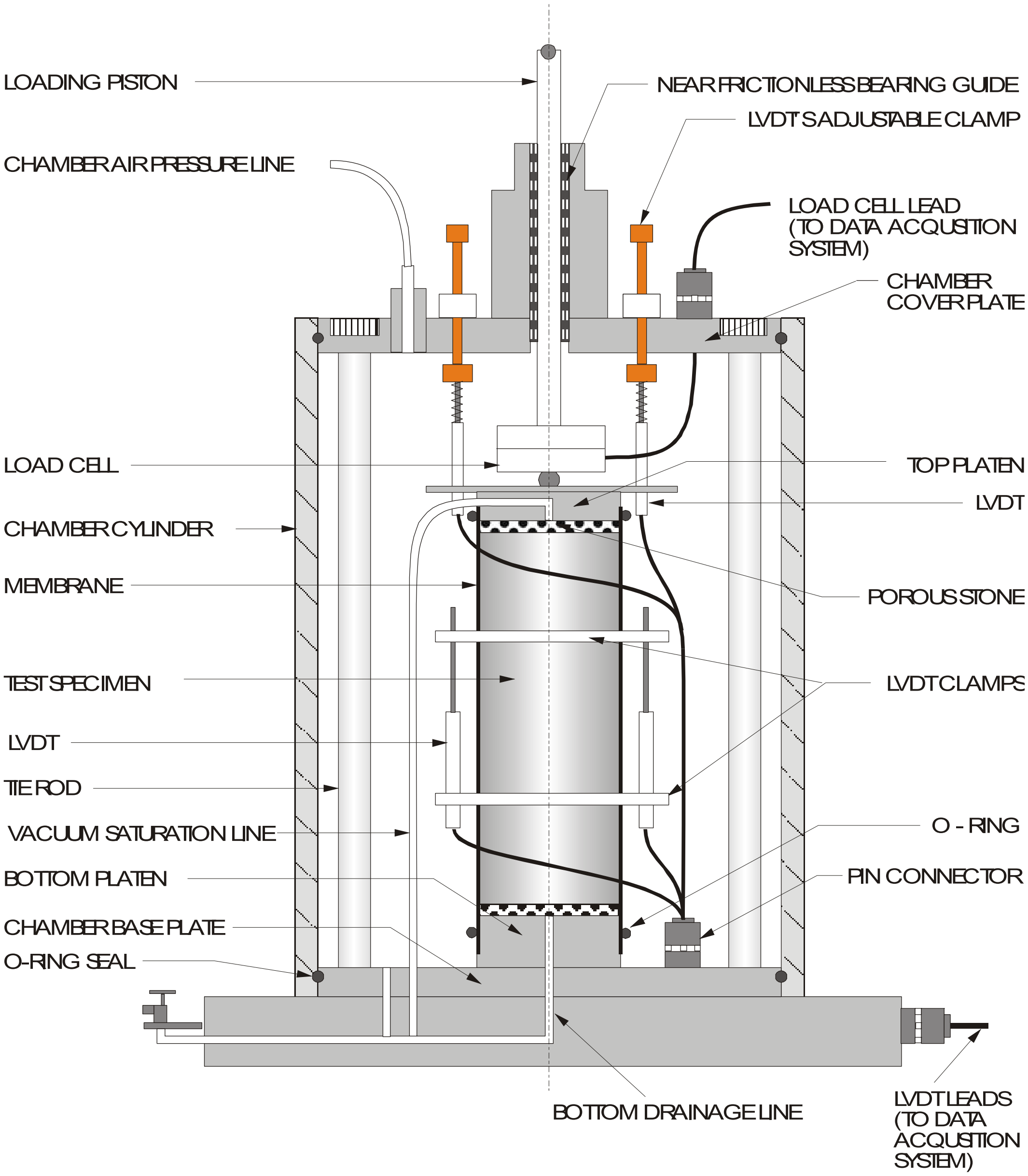


Figure 3.5 Triaxial Chamber with Internal LVDTs and Load Cell

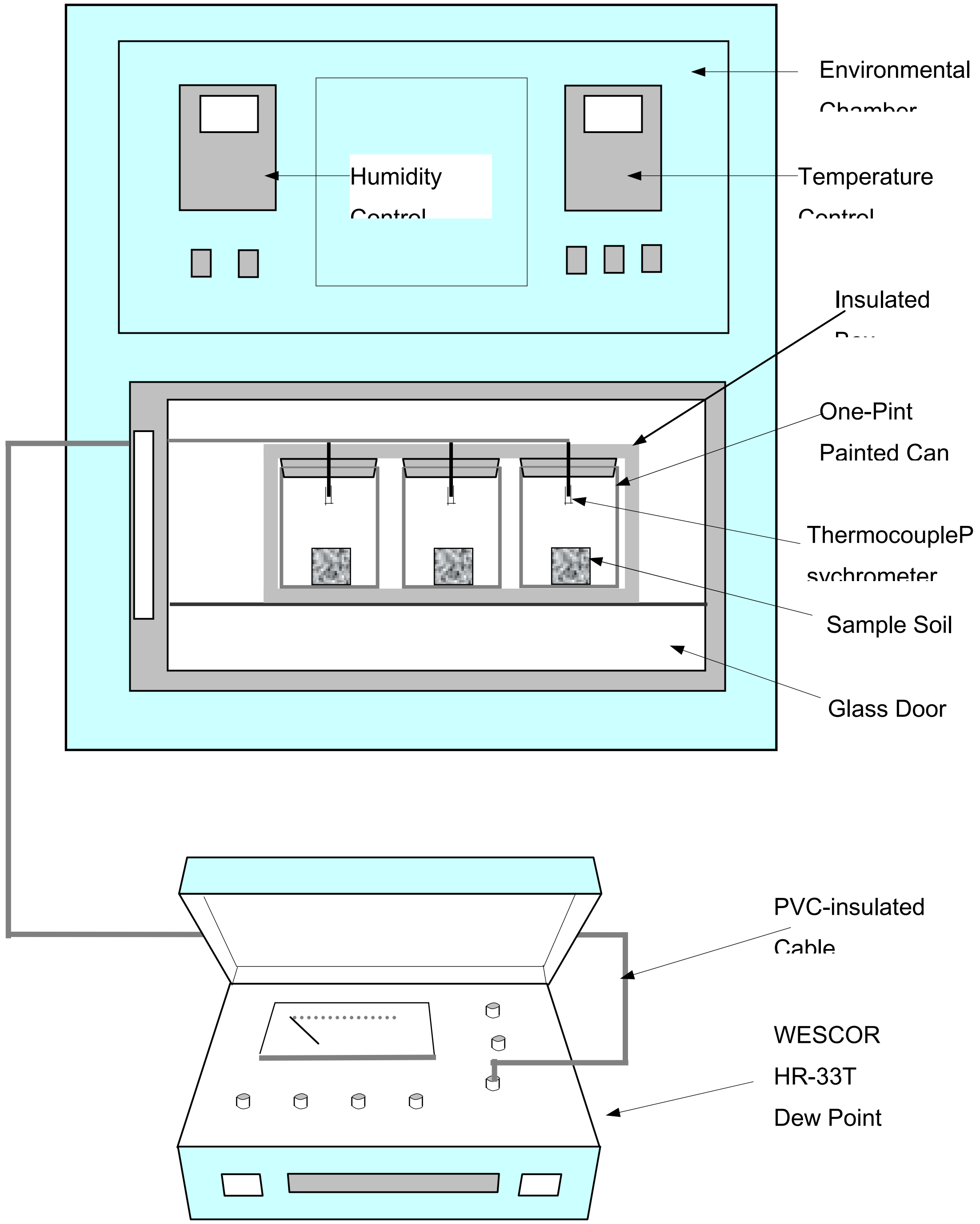


Figure 3.6 A Schematic Illustration of T273-86 Soil Suction Test Setup

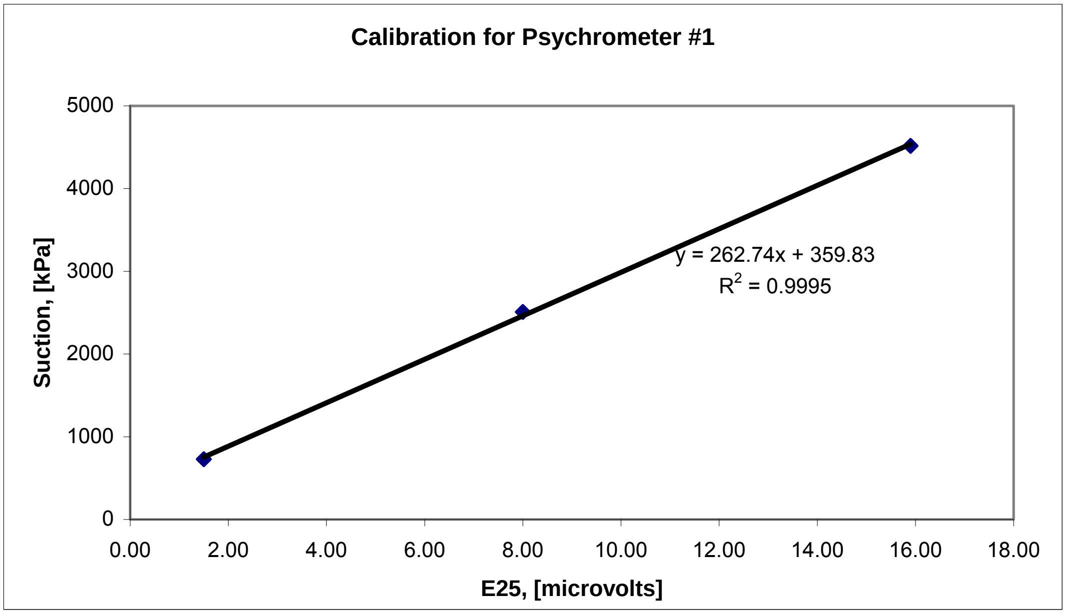


Figure 3.7 Calibration Line for Psychrometer No.1

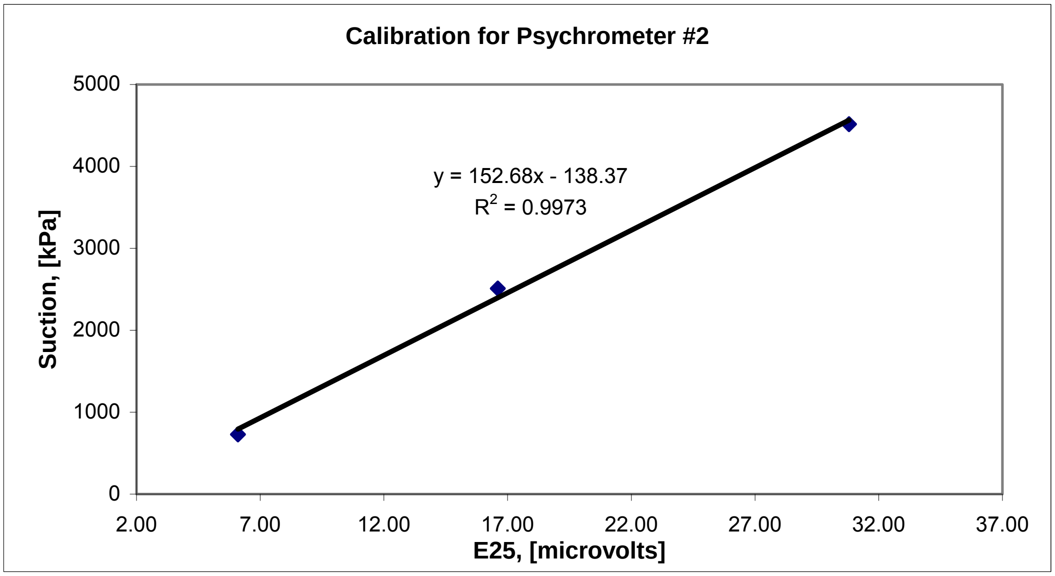


Figure 3.8 Calibration Line for Psychrometer No.2

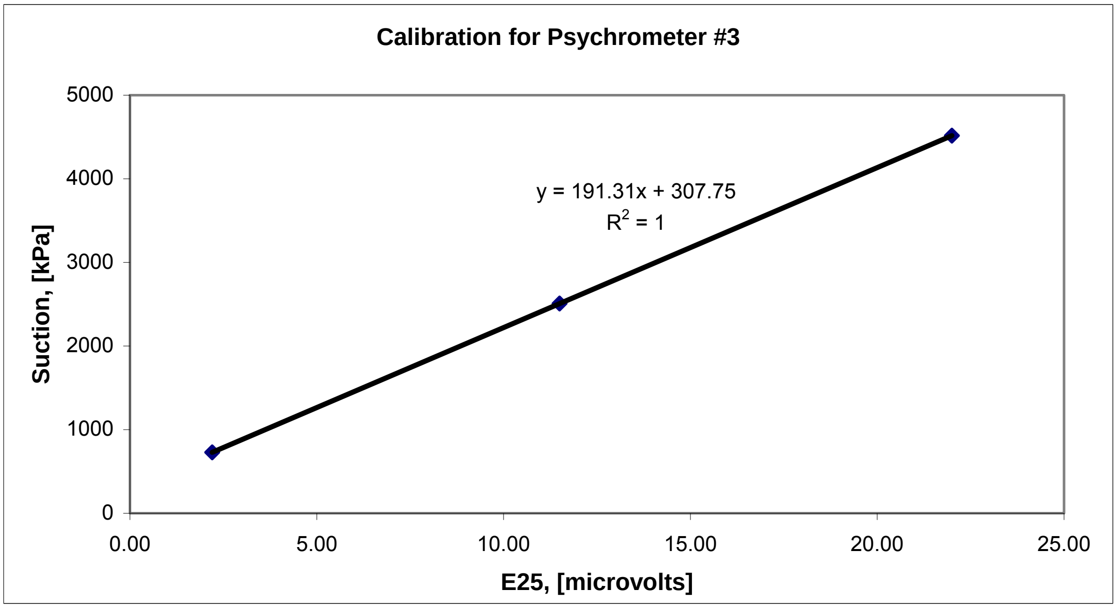


Figure 3.9 Calibration Line for Psychrometer No.3

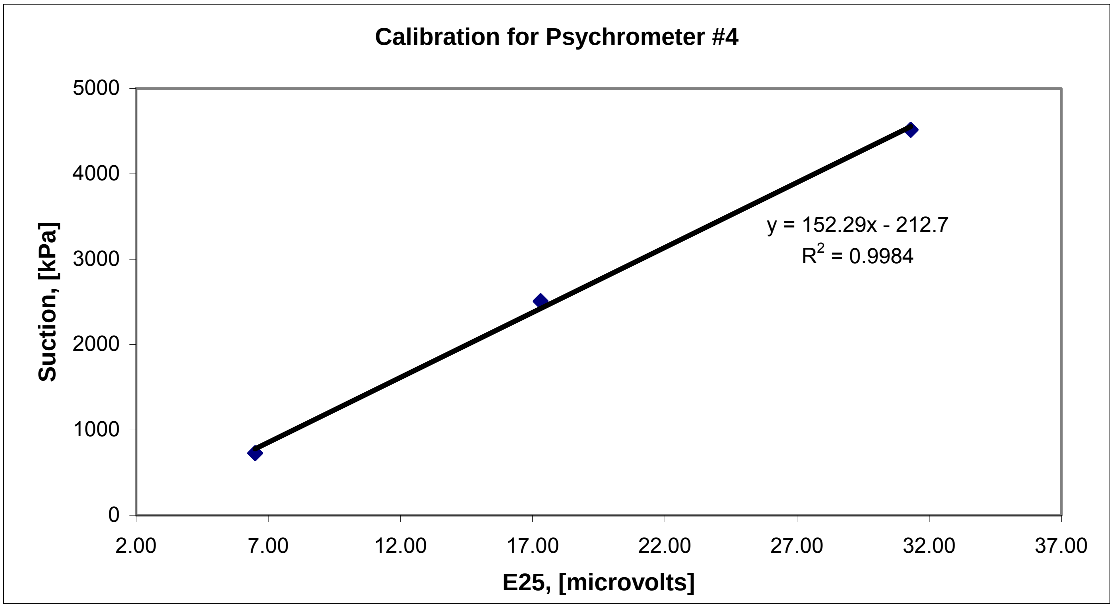


Figure 3.10 Calibration Line for Psychrometer No.4

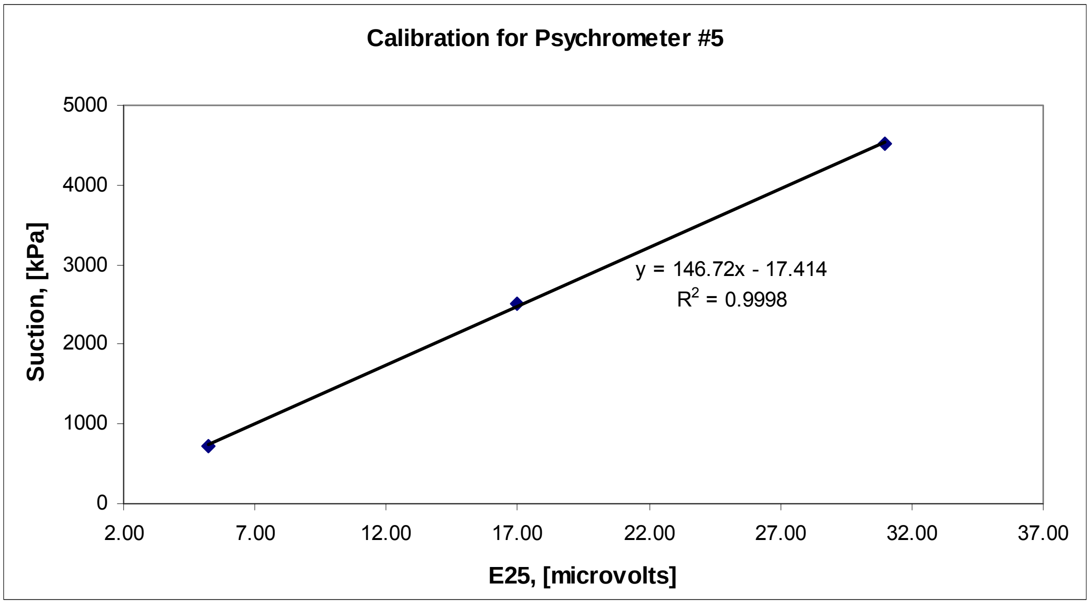


Figure 3.11 Calibration Line for Psychrometer No.5

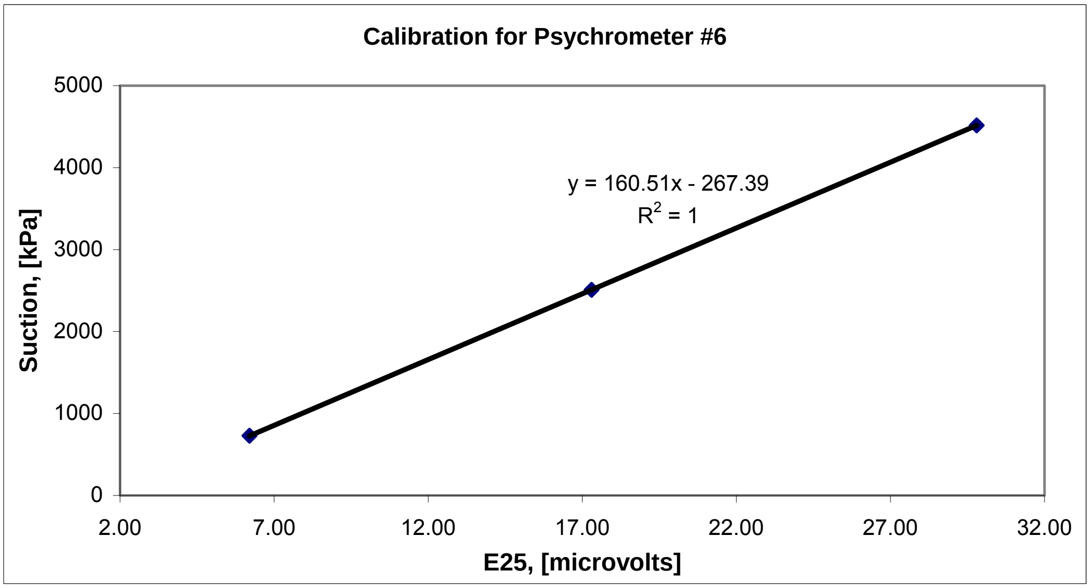


Figure 3.12 Calibration Line for Psychrometer No.6

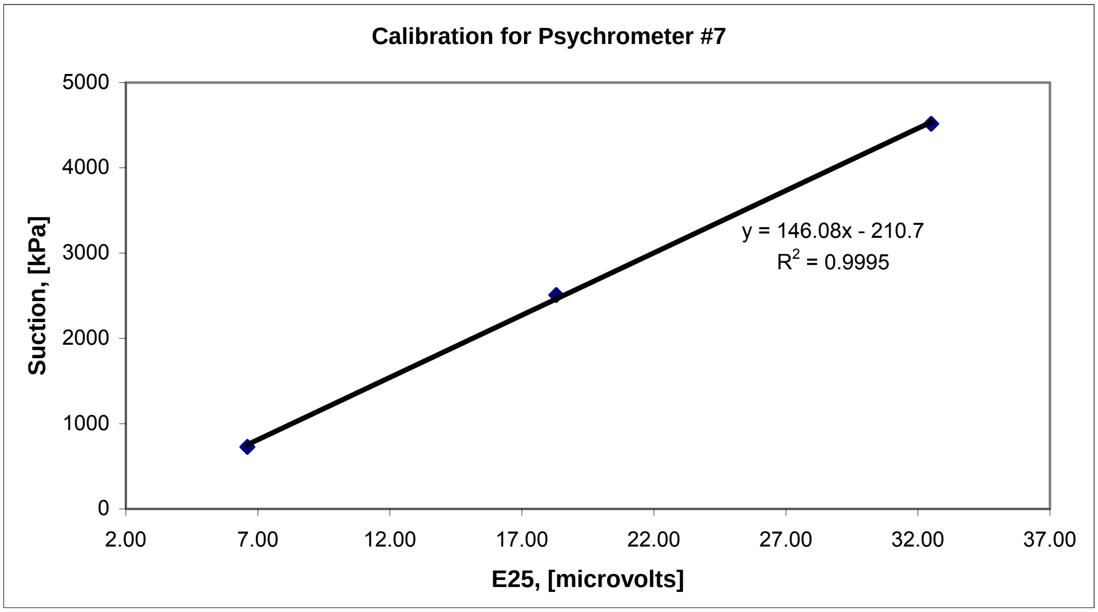
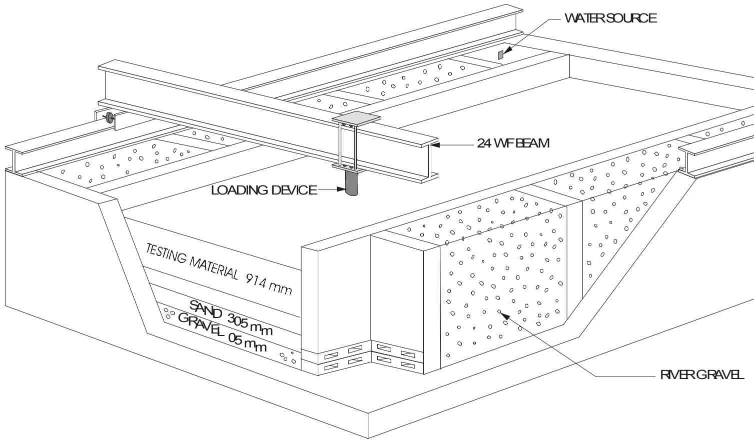


Figure 3.13 Calibration Line for Psychrometer No.7


WATER SOURCE
24 WF BEAM
LOADING DEVICE
TESTING MATERIAL 914 mm
SAND 305 mm
GRAVEL 05 mm
RIVER GRAVEL

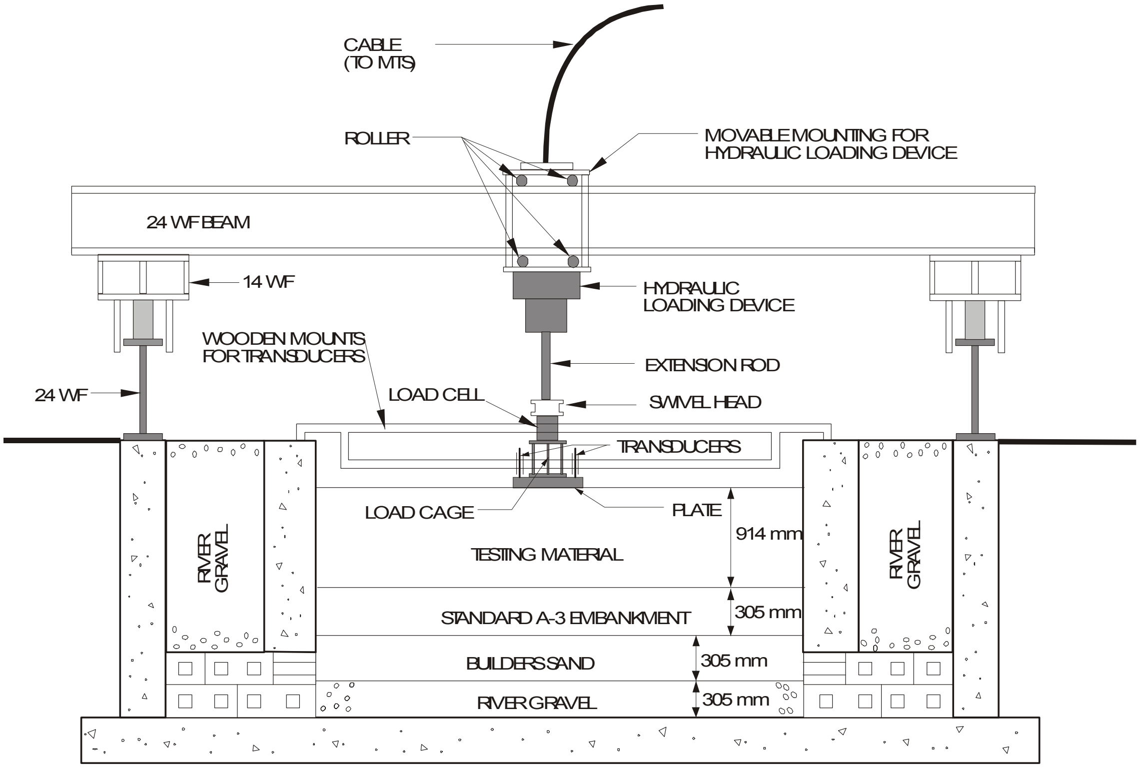

CABLE
(TO MTS)
ROLLER
MOVABLE MOUNTING FOR
HYDRAULIC LOADING DEVICE
24 WF BEAM
14 WF
HYDRAULIC
LOADING DEVICE
WOODEN MOUNTS
FOR TRANSDUCERS
EXTENSION ROD
24 WF
LOAD CELL
SWIVEL HEAD
TRANSDUCERS
LOAD CAGE
PLATE
914 mm
RIVER
GRAVEL
TESTING MATERIAL
RIVER
GRAVEL
STANDARD A-3 EMBANKMENT
305 mm
BUILDERS SAND
305 mm
RIVER GRAVEL
305 mm

Figure 3.14 Schematic Diagram of Loading System & Cross Sectional View of Test-Pit

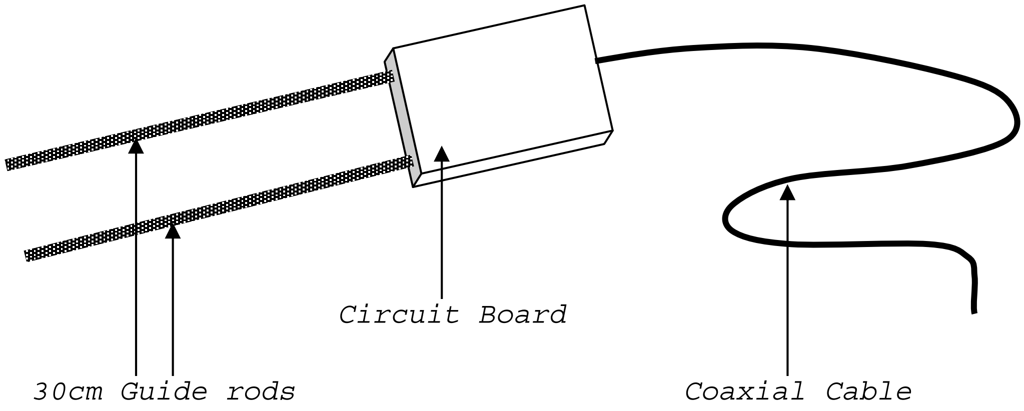


Figure 3.15 An Actual View of CS615 Probe

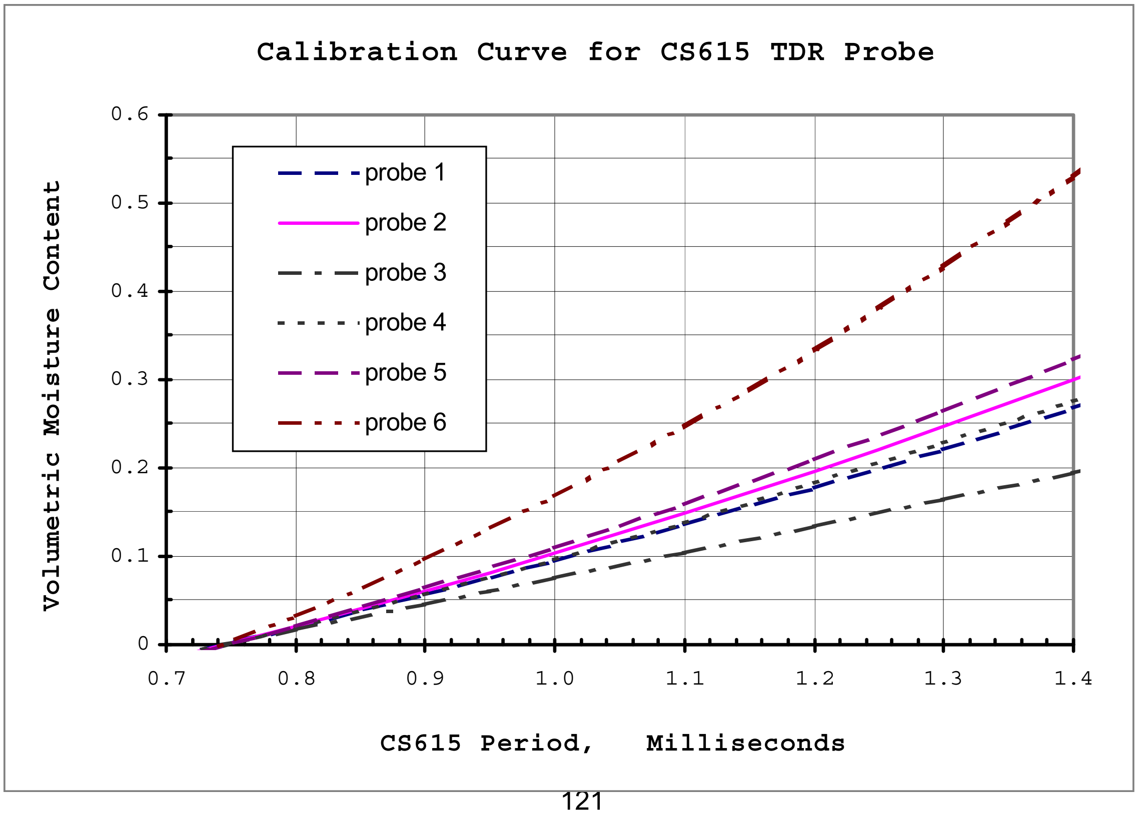


Figure 3.16 Calibration Curve for CS615 TDR Probe

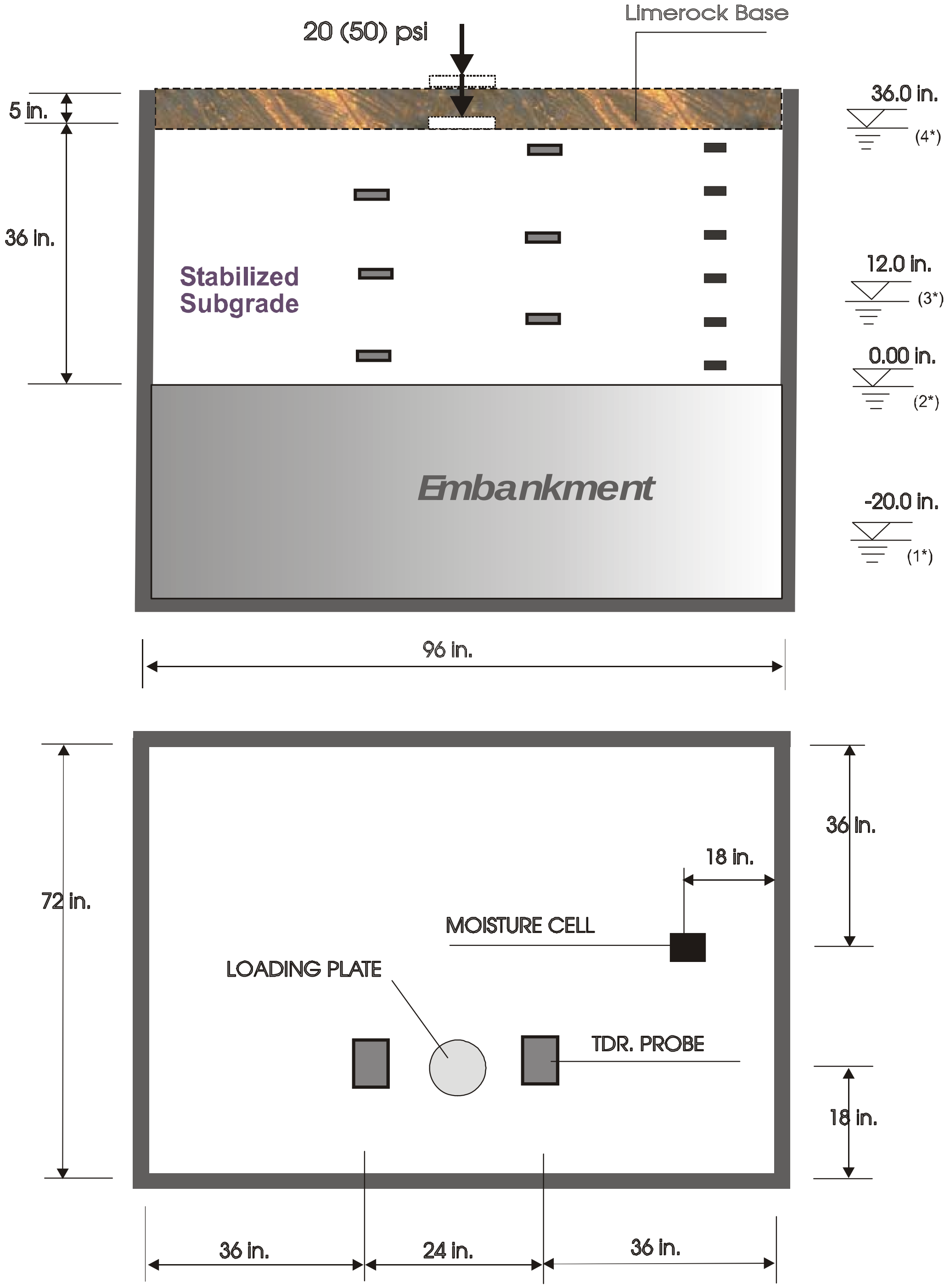


Figure 3.17 Test-Pit Setup for Levy County A-3 Subgrade
(* Sequence of Water Table Adjustment)

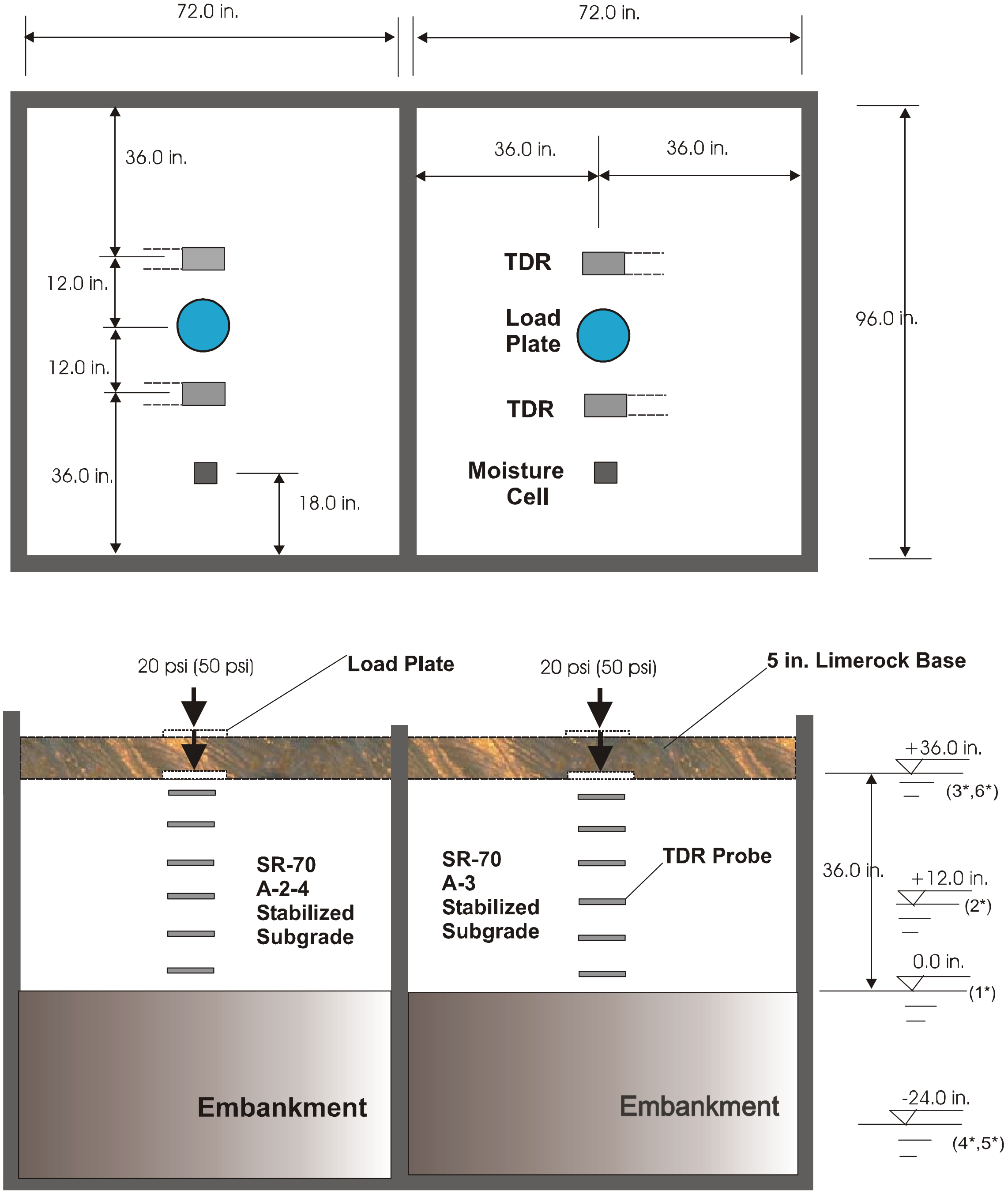


Figure 3.18 Test-pit Setup for SR70 A-3 & A-2-4 Subgrades
(* Sequence of Water Table Adjustment)

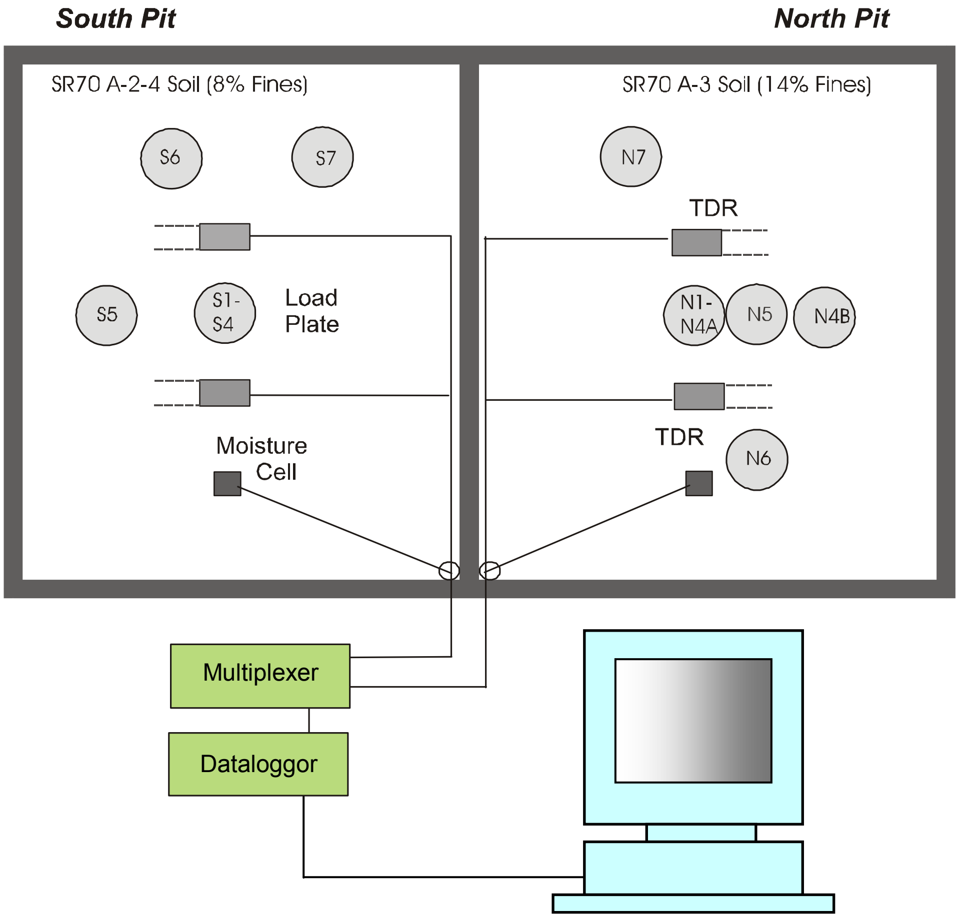


Figure 3.19 Plate Load Test Loading Position (SR70 A-3 and A-2-4) and Connection of Data Readout

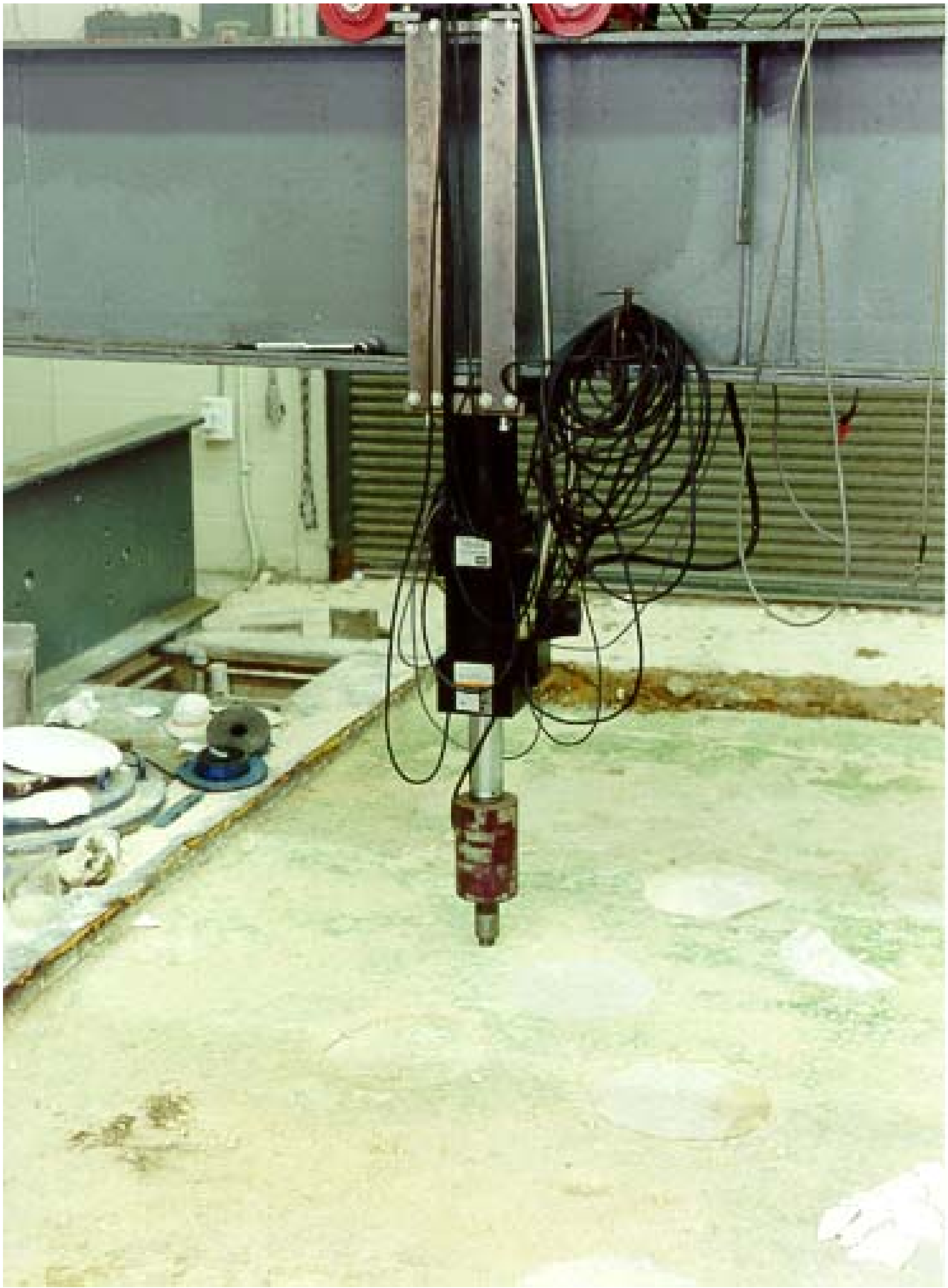
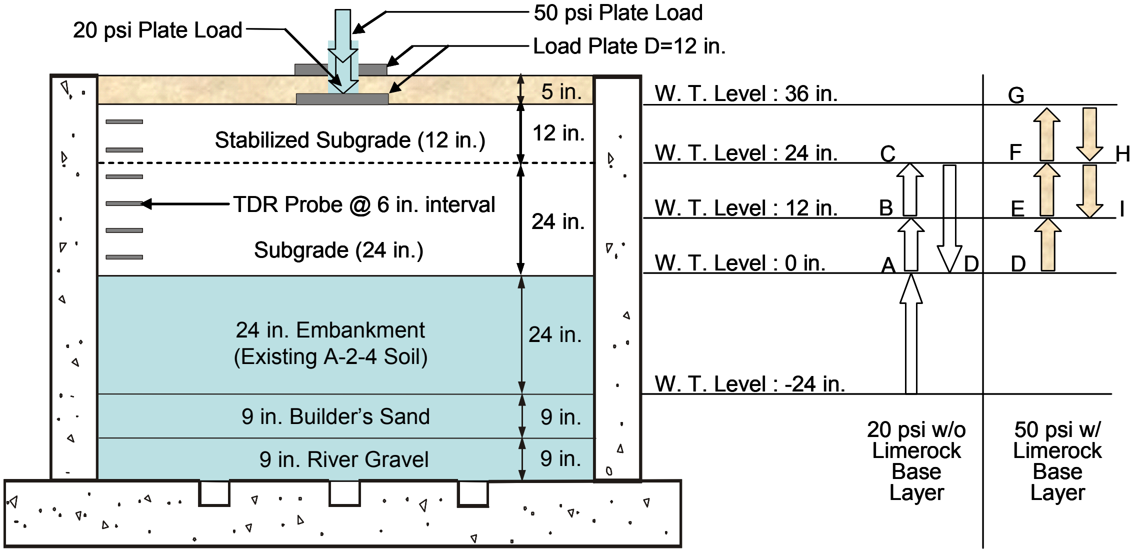


Figure 3.20 An Actual View of Test-Pit Loading System

Figure 3.21 An Actual View of Test-Pit and Compaction Equipment



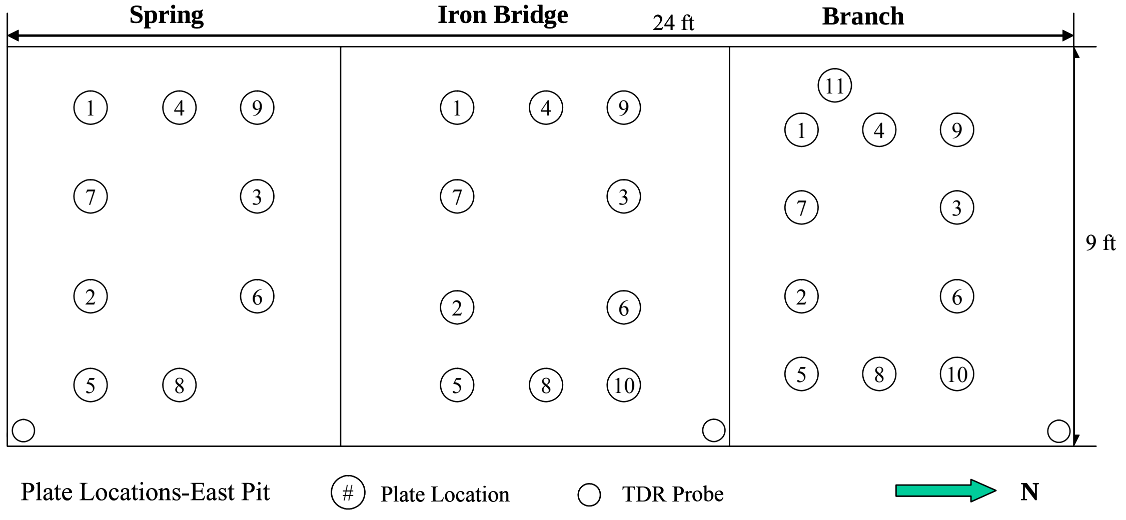
(a) Pavement Profile

(b) Water Level Variations

Figure 3.22 Cross Sectional View of Phase III Test Pit Experimental Program

Figure 3.23 TDR and Nuclear Gauge used for additional three soils

(A) Without Limerock Base Layer



(B) With Limerock Base Layer

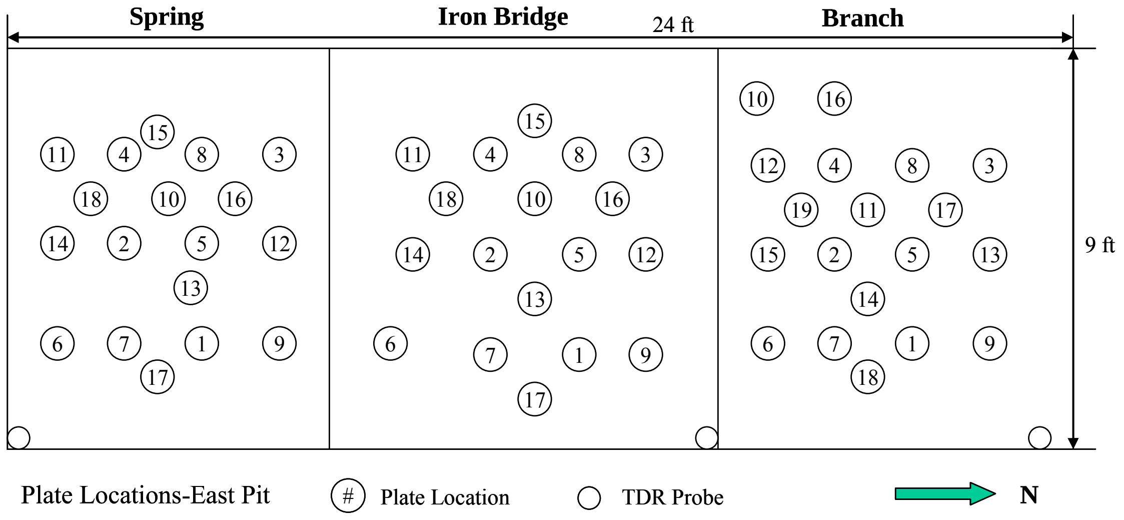


Figure 3.24 Layouts of Phase III Test Pit Experimental Program

CHAPTER 4
PRESENTATION OF LABORATORY TEST RESULTS

4.1 LABORATORY RESILIENT MODULUS

The AASHTO T292-91I test method was used for the original eight soils, while the AASHTO T307-99 test method was followed for the additional three soils. During the resilient modulus test, specimen conditioning was conducted first. Then, a series of tests at different deviator stresses and confining pressures were performed, and the data were recorded for every cycle of the test. However, only the last five cycles were used for computation of resilient modulus. The resilient modulus (M_r) was calculated from the deviator stress and resilient strain using Equation (3-6).

Generally two resilient modulus tests were conducted for each moisture condition. The resilient modulus test results were reported in a tabular form including the deviator stress, axial strain, confining pressure, and bulk stress. A regression model was used to get the regression equation of M_r from the confining pressure and bulk stress.

$$M_r = k_1 \theta^{k_2} \tag{4-1}$$

$$M_r = k_3 \sigma_3^{k_4} \tag{4-2}$$

A summary of typical resilient modulus test results is presented in Table 4.1. The results included the confining pressure, deviator stress, bulk stress, axial strain, and their corresponding resilient modulus values. Typical regression models for the resilient modulus versus bulk stress and confining pressure are shown in Figures 4.1(A) and 4.1(B). The resilient modulus test results using T292-91I for the original eight types of soil are summarized and presented in Appendices D.1 to D.8. The resilient modulus test results using T307-99 for the additional three soils are summarized and presented in Appendices D.9, D10, and D11.

In all of the regression equations in this study, the resilient modulus M_r is in units of MPa while the bulk stress θ and the confining pressure σ_3 are in units of kPa.

4.1.1 Phase I and Phase II Resilient Modulus Results

Levy County A-3 soil with 4% fines

The individual test results of the Levy County A-3 soil with 4% fines are presented in detail in Appendix D.1. Seven samples were tested for resilient modulus. A summary of the regression models of M_r versus bulk stress is presented in Table 4.2(A) and the regression relations are shown in Figure 4.2(A). A summary of the regression models of M_r versus confining pressure is presented in Table 4.2(B) and the regression relations are illustrated in Figure 4.2(B). The effect of moisture on the resilient modulus was not significant.

SR70 A-3 soil with 8% fines

The individual test results of the SR70 A-3 soil are presented in Appendix D.2. Eight samples were tested for resilient modulus. A summary of the regression models of M_r versus bulk stress is presented in Table 4.3(A), and the regression relations are demonstrated in Figure 4.3(A). A summary of the regression models of M_r versus confining pressure at different moisture content levels is presented in Table 4.3(B), and the regression relations are illustrated in Figure 4.3(B). The moisture had a minor effect on the resilient modulus of SR70 A-3 soil.

A-2-4 soil with 12% fines

The individual test results of the A-2-4 soil with 12% fines are presented in Appendix D.3. Six samples were tested for resilient modulus. A summary of the regression models of M_r versus bulk stress at different moisture content levels is presented in Table 4.4(A), and the regression relations are illustrated in Figure 4.4(A). A summary of the regression models of M_r versus confining pressure at different moisture content is presented in Table 4.4(B), and the regression relations are shown in Figure 4.4(B). The effect of moisture on the resilient modulus of A-2-4 soil with 12% fines was not very significant.

SR70 A-2-4 soil with 14% fines

The test results of the SR70 A-2-4 soil with 14% fines are presented in Appendix D.4. Six samples were tested for resilient modulus. A summary of the regression models of M_r versus bulk stress is presented in Table 4.5(A), and a summary of the regression models of M_r versus confining pressure is presented in Table 4.5(B). Figure 4.5(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.5(B) shows the M_r versus confining pressure at different moisture content levels. The moisture had a significant effect on the resilient modulus of SR70 A-2-4 soil with 14% fines.

A-2-4 soil with 20% fines

The test results of the A-2-4 soil with 20% fines are presented in Appendix D.5. Six samples were tested. Table 4.6(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.6(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.6(A) shows the M_r versus bulk stress at different moisture content. Figure 4.6(B) shows the M_r versus confining pressure at different moisture content levels. The moisture has some effect on the resilient modulus of the A-2-4 soil with 20% fines.

A-2-4 soil with 24% fines

The test results of the A-2-4 soil with 24% fines are presented in Appendix D.6. Six samples were tested. Table 4.7(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.7(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.7(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.7(B) shows the M_r versus confining pressure at different moisture content levels. The moisture had some effect on the resilient modulus of the A-2-4 soil with 24% fines.

A-2-4 soil with 30% fines

The test results of the A-2-4 soil with 30% fines are presented in Appendix D.7. Six samples were tested. Table 4.8(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.8(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.8(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.8(B) shows the M_r versus confining pressure at different moisture content levels. The effect of moisture on the resilient modulus was very significant.

Miami Oolite A-1 soil

Oolite from Miami was crushed in order to meet the laboratory requirement for resilient modulus test. The test results of Miami Oolite (A-1) soil are presented in Appendix D.8. Six samples were tested. Table 4.9(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.9(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.9(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.9(B) shows the M_r versus confining pressure at different moisture content levels. The effect of moisture on the resilient modulus was very significant for the A-1 soil.

4.1.2 Phase III Resilient Modulus Results

The resilient modulus results of the additional three soils were obtained using AASHTO T307-99 test method. The resilient moduli were obtained based on the full-length, internal LVDT measurements. At least four resilient modulus tests were performed for each soil at the optimum compacted condition (compacted to 100% of Standard Proctor maximum unit weight). No data were available under both soaked and dried conditions. The average resilient modulus values at 2 psi confining pressure and 11 psi bulk stress from the test results were then presented for each soil. The resilient modulus test results using T307-99 are summarized and presented in Appendices D.9, D.10, and D.11, for the additional three types of soil.

Spring Cemetery A-2-4 soil with 15% fines

The individual test results of the Spring Cemetery A-2-4 soil are presented in Appendix D.9. Four samples were tested for resilient modulus. Table 4.10(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.10(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.10(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.10(B) shows the M_r versus confining pressure at different moisture content levels. The effect of moisture on the resilient modulus was very significant.

Branch A-2-4 soil with 23% fines

The individual test results of the Branch A-2-4 soil are presented in Appendix D.10. A total of six samples were tested. Table 4.11(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.11(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.11(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.11(B) shows the M_r versus confining pressure at different moisture content levels. The effect of moisture on the resilient modulus was very significant.

Iron Bridge A-2-6 soil with 31% fines

The individual test results of the Iron Bridge A-2-6 soil are presented in Appendix D.11. Four samples were tested. Table 4.12(A) presents a summary of the regression models of M_r versus bulk stress, and Table 4.12(B) presents a summary of the regression models of M_r versus confining pressure. Figure 4.12(A) shows the M_r versus bulk stress at different moisture content levels. Figure 4.12(B) shows the M_r versus confining pressure at different moisture content levels. The effect of moisture on the resilient modulus was very significant.

4.2 SOIL SUCTION TEST RESULTS

The suction test results are summarized in Table 4.13 and shown in Figure 4.13 for the eight soil types for different water content levels. As shown in Figure 4.13, the suction value generally decreases with an increase in moisture content. The trend is in general agreement with the Soil-Water Characteristic Curve (SWCC), which defines the soil’s ability to store and release water. Suction data were not available for the three additional soils.

For the two A-3 soils (Levy A-3 and SR70 A-3), the suction values, in range of 2 kPa to 60 kPa, were lower than that of the A-2-4 soils. The range of suction value for the A-2-4 soils was from 30 kPa to 600 kPa at around the optimum moisture content. The A-3 soils had only a small amount of fines, so there were larger pores in the soil. The A-2-4 soils had more fines with smaller pores. The soil with smaller pores would contain and suck more water than the soil with larger pores.

As shown in Figure 4.13, the suction values are not much different among the eight soils at around the optimum moisture content. The psychrometer test may not be accurate enough on measuring suction value of sandy materials. Other test methods such as the filter paper test may achieve more accuracy.

4.3 PERMEABILITY TEST RESULTS

The permeability test results are summarized in Table 4.14 and shown in Figure 4.14. As shown in Figure 4.14, a general trend exists that the measured permeability decreases with an increase in the percent of fines. The permeability results indicated that the percent of fines was a good indicator of the soil permeability.

Table 4.1 Typical Resilient Modulus Test Results

Summary of Resilient Modulus Test Results							
Type: T292-91I				**Soil Identification**			
Sample No.	A2430%S2			A-2-4, 30% fine			
Lab. Moist.	13.20%			Opt. Moist.	12.00%		
Lab. Den.	116	pcf		Opt. Den.	115.70	pcf	
				Conditioning Information			
				Load Type: Dynamic			
				Dev. Stress: 82.74 kPa			
				Conf. Stress: 103.42 kPa			
				No. Reps.: 1000			
Confining Pressure	**Axial Load**	**Dev. Stress**	**Bulk Stress**	**Middle Strain**	**Full Length Strain**	**Middle Modulus**	**Full Length Modulus**
kPa	**kN**	**kPa**	**kPa**			**MPa**	**Mpa**
103.42	0.373	45.990	356.250	0.000033	0.000199		231.51
103.42	0.541	66.779	377.039	0.000156	0.000290	427.66	229.93
103.42	0.821	101.265	411.525	0.000324	0.000440	312.53	230.39
68.95	0.261	32.237	239.087	0.000020	0.000193		166.99
68.95	0.374	46.141	252.991	0.000116	0.000281	398.38	164.48
68.95	0.541	66.690	273.540	0.000245	0.000397	271.73	168.04
68.95	0.820	101.199	308.049	0.000449	0.000594	225.14	170.43
34.47	0.150	18.511	121.921	0.000007	0.000172		107.39
34.47	0.261	32.224	135.634	0.000132	0.000314	243.45	102.72
34.47	0.374	46.141	149.551	0.000245	0.000440	188.53	104.98
34.47	0.541	66.687	170.097	0.000413	0.000612	161.35	108.98
13.79	0.150	18.477	59.847	0.000105	0.000276	176.33	66.89
13.79	0.262	32.298	73.668	0.000263	0.000477	122.64	67.77
13.79	0.374	46.071	87.441	0.000454	0.000660	101.56	69.79

Table 4.2(A) Regression Model of Resilient Modulus versus Bulk Stress for Levy County A-3

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula ($y=Mr$, $x=\theta$)	R^2
8.08%	A3LEVYD1	31.2400	0.3925	$y=31.24x^{0.3925}$	0.9836
4.30%	A3LEVYD2	25.1780	0.4316	$y=25.178x^{0.4316}$	0.9877
9.50%	A3LEVYO1	20.7880	0.4454	$y=20.788x^{0.4454}$	0.9856
9.60%	A3LEVYO2	18.4610	0.481	$y=18.461x^{0.481}$	0.9895
13.47%	A3LEVYS1	33.7400	0.4451	$y=33.74x^{0.4451}$	0.5326
15.00%	A3LEVYS2	12.2240	0.5512	$y=12.224x^{0.5512}$	0.9864
15.27%	A3LEVYS3	23.988	0.4472	$y=23.988x^{04472}$	0.9022
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula ($y=Mr$, $x=\theta$)	R2
8.08%	A3LEVYD1	13.3500	0.5215	$y=13.35x^{0.5215}$	0.994
4.30%	A3LEVYD2	7.1728	0.6227	$y=7.1728x^{0.6227}$	0.9967
9.50%	A3LEVYO1	14.1630	0.4911	$y=14.163x^{0.4911}$	0.9854
9.60%	A3LEVYO2	4.7729	0.6972	$y=4.7729x^{0.6972}$	0.9952
13.47%	A3LEVYS1	15.0760	0.5043	$y=15.076x^{0.5043}$	0.9819
15.00%	A3LEVYS2	4.6188	0.6708	$y=4.6188x^{0.6708}$	0.9954
15.27%	A3LEVYS3	3.7170	0.7073	$y=3.717x^{0.7073}$	0.9965

Table 4.2(B) Regression Model of Resilient Modulus versus Confining Pressure for Levy County A-3

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
8.08%	A3LEVYD1	70.5960	0.3262	$y=70.596x^{0.3262}$	0.9921
4.30%	A3LEVYD2	62.2770	0.3566	$y=62.277x^{0.3566}$	0.9853
9.50%	A3LEVYO1	52.6030	0.3696	$y=52.6030x^{0.3696}$	0.9934
9.60%	A3LEVYO2	50.8130	0.3966	$y=18.461x^{0.3966}$	0.9877
13.47%	A3LEVYS1	68.9590	0.4246	$y=68.959x^{0.4246}$	0.9932
15.00%	A3LEVYS2	37.5900	0.4631	$y=37.59x^{0.4631}$	0.9985
15.27%	A3LEVYS3	56.9750	0.3884	$y=56.975x^{0.3884}$	0.9978
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
8.08%	A3LEVYD1	39.4880	0.4331	$y=39.488x^{0.4331}$	0.9921
4.30%	A3LEVYD2	27.1880	0.5074	$y=27.188x^{0.5074}$	0.9958
9.50%	A3LEVYO1	38.9780	0.4089	$y=38.978x^{0.4989}$	0.9975
9.60%	A3LEVYO2	20.8960	0.5717	$y=20.896x^{0.5717}$	0.9949
13.47%	A3LEVYS1	41.7490	0.4261	$y=41.749x^{0.4261}$	0.997
15.00%	A3LEVYS2	18.5980	0.5571	$y=18.598x^{0.5571}$	0.9986
15.27%	A3LEVYS3	16.2710	0.5859	$y=16.271x^{0.3854}$	0.9946

Table 4.3(A) Regression Model of Resilient Modulus versus Bulk Stress for SR70 A-3

Moisture Content	Sample No.	Middle Half			
		k1	k2	Formula (y=Mr, x=θ)	R2
7.8%	A3SR70D1	22.1260	0.4192	$y=22.126x^{0.4192}$	0.9881
5.3%	A3SR70D2	30.2110	0.4057	$y=30.211x^{0.4057}$	0.9652
4.5%	A3SR70D3	61.9200	0.3097	$y=61.92x^{0.3097}$	0.8779
4.0%	A3SR70D4	69.8830	0.3252	$y=69.883x^{0.3252}$	0.4158
11.4%	A3SR70O1	24.3600	0.4302	$y=24.36x^{0.4302}$	0.9864
11.4%	A3SR70O2	23.9860	0.4364	$y=23.986x^{0.4364}$	0.9785
13.4%	A3SR7S1	12.2840	0.5942	$y=12.284x^{0.5942}$	0.7924
13.7%	A3SR70S2	9.2997	0.5803	$y=9.2997x^{05803}$	0.9553
Moisture Content	Sample No.	Full Length			
		k'1	k'2	Formula (y=Mr, x=θ)	R2
7.8%	A3SR70D1	10.2870	0.5320	$y=10.287x^{0.532}$	0.9853
5.3%	A3SR70D2	13.6220	0.5195	$y=13.622x^{0.5195}$	0.9654
4.5%	A3SR70D3	6.3577	0.6873	$y=6.3577x^{0.6873}$	0.9482
4.0%	A3SR70D4	11.0630	0.5585	$y=11.063x^{0.5585}$	0.9829
11.4%	A3SR70O1	10.4820	0.5520	$y=10.482x^{0.5520}$	0.9872
11.4%	A3SR70O2	12.6830	0.5254	$y=12.683x^{0.5254}$	0.9804
13.4%	A3SR7S1	7.6672	0.5929	$y=7.6672x^{0.5929}$	0.9937
13.7%	A3SR70S2	3.8317	0.7132	$y=3.8317x^{0.7132}$	0.9773

Table 4.3(B) Regression Model of Resilient Modulus versus Confining Pressure for SR70 A-3

Moisture Content	Sample No.	Middle Half			
		k3	k4	Formula (y=Mr, x=σ_3)	R2
7.8%	A24SR70D1	53.1320	0.3471	$y=53.132x^{0.3471}$	0.9910
5.3%	A24SR70D2	68.2200	0.3446	$y=68.22x^{0.3446}$	0.9925
4.5%	A24SR70D3	110.3000	0.2744	$y=110.3x^{0.2744}$	0.9988
4.0%	A24SR70D4	69.8830	0.3252	$y=69.883x^{0.3252}$	0.4158
11.4%	A24SR70O1	59.7990	0.3568	$y=59.799x^{0.3568}$	0.9855
11.4%	A24SR70O2	58.4120	0.3670	$y=58.412x^{0.367}$	0.9952
13.4%	A24SR70S1	36.9340	0.5320	$y=36.934x^{0.532}$	0.9798
13.7%	A24SR70S2	40.1160	0.4262	$y=40.116x^{0.4262}$	0.9661
Moisture Content	Sample No.	Full Length			
		k'3	k'4	Formula (y=Mr, x=σ_3)	R2
7.8%	A24SR70D1	31.0580	0.4426	$y=31.058x^{0.4426}$	0.9877
5.3%	A24SR70D2	38.1300	0.4440	$y=38.13x^{0.444}$	0.9998
4.5%	A24SR70D3	24.3960	0.5898	$y=24.396x^{0.5898}$	0.9852
4.0%	A24SR70D4	34.4330	0.4705	$y=34.433x^{0.4705}$	0.9941
11.4%	A24SR70O1	32.8830	0.4601	$y=32.883x^{0.4601}$	0.9890
11.4%	A24SR70O2	37.1680	0.4406	$y=37.168x^{0.4406}$	0.9916
13.4%	A24SR70S1	26.2990	0.4923	$y=26.299x^{0.4923}$	0.9957
13.7%	A24SR70S2	23.2210	0.5221	$y=23.221x^{0.5221}$	0.9746

Table 4.4(A) Regression Model of Resilient Modulus versus Bulk Stress for A-2-4 12% Soil

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula (y=Mr, x=θ)	R^2
7.1%	A2412%D1	19.4130	0.4562	$y=19.413x^{0.4562}$	0.9686
7.0%	A2412%D2	15.1390	0.5161	$y=15.139x^{0.5161}$	0.9859
12.1%	A2412%O1	12.0340	0.5303	$y=12.034x^{0.5303}$	0.9875
12.1%	A2412%O2	9.0054	0.5911	$y=9.0054x^{0.5911}$	0.9407
14.6%	A2412%S1	9.2350	0.5557	$y=9.235x^{0.5557}$	0.9893
13.6%	A2412%S2	11.8290	0.5211	$y=11.829x^{0.5211}$	0.9869
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula (y=Mr, x=θ)	R^2
7.1%	A2412%D1	7.0555	0.6066	$y=7.0555x^{0.6066}$	0.9924
7.0%	A2412%D2	5.3978	0.6638	$y=5.3978x^{0.6638}$	0.994
12.1%	A2412%O1	6.0838	0.6311	$y=6.0838x^{0.6311}$	0.9852
12.1%	A2412%O2	6.6733	0.6202	$y=6.6733x^{0.6202}$	0.9700
14.6%	A2412%S1	7.4721	0.5740	$y=7.4721x^{0.5740}$	0.9931
13.6%	A2412%S2	6.7001	0.6053	$y=6.7001x^{0.6053}$	0.9961

Table 4.4(B) Regression Model of Resilient Modulus versus Confining Pressure for A-2-4 12%

Moisture Content	Sample No.	Middle Half			
		k_3	K_4	Formula (y=Mr, x=σ_3)	R^2
7.1%	A2412%D1	49.1900	0.3839	$y=49.19x^{0.3839}$	0.9892
7.0%	A2412%D2	43.8560	0.4312	$y=43.856x^{0.4312}$	0.9886
12.1%	A2412%O1	35.5150	0.4454	$y=35.516x^{0.4454}$	0.998
12.1%	A2412%O2	30.0830	0.4987	$y=30.0830x^{0.4987}$	0.9700
14.6%	A2412%S1	29.2350	0.4624	$y=29.235x^{0.4624}$	0.9891
13.6%	A2412%S2	34.795	0.4344	$y=34.795x^{0.4344}$	0.9893
Moisture Content	Sample No.	Full Length			
		k'_3	K'_4	Formula (y=Mr, x=σ_3)	R^2
7.1%	A2412%D1	26.0570	0.4928	$y=26.057x^{0.4928}$	0.9841
7.0%	A2412%D2	21.594	0.5499	$y=21.594x^{0.5499}$	0.9904
12.1%	A2412%O1	21.7620	0.5328	y=21.762x0.5328	0.9992
12.1%	A2412%O2	21.1340	0.5271	$y=21.134x^{0.5271}$	0.9916
14.6%	A2412%S1	24.5660	0.4776	$y=24.566x^{0.4776}$	0.9928
13.6%	A2412%S2	23.6990	0.5013	$y=23.699x^{0.5013}$	0.9949

Table 4.5(A) Regression Model of Resilient Modulus versus Bulk Stress for SR70 A-2-4

Moisture Content	Sample No.	Middle Half			
		k_1	K_2	Formula ($y=Mr$, $x=\theta$)	R^2
8.41%	A24SR70D1	427.8900	0.0540	$y=427.89x^{0.054}$	0.1721
7.76%	A24SR70D2	1211.8000	-0.1032	$y=1211.8x^{-0.1032}$	0.1929
10.80%	A24SR70O1	132.9400	0.1698	$y=132.94x^{0.1698}$	0.3393
10.39%	A24SR70O2	65.1530	0.2769	$y=65.153x^{0.2769}$	0.6244
11.23%	A24SR70S1	24.8220	0.4310	$y=24.822x^{0.431}$	0.877
11.70%	A24SR70S2	31.5620	0.4321	$y=31.562x^{0.4321}$	0.4503
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula ($y=Mr$, $x=\theta$)	R^2
8.41%	A24SR70D1	57.4670	0.3861	$y=57.467x^{0.3861}$	0.9121
7.76%	A24SR70D2	1.9460	0.9431	$y=1.946x^{0.9431}$	0.9922
10.80%	A24SR70O1	42.3150	0.3332	$y=19.852x^{0.4414}$	0.8241
10.39%	A24SR70O2	24.9300	0.4210	$y=24.9300x^{0.4210}$	0.8487
11.23%	A24SR70S1	9.6913	0.5608	$y=9.6913x^{0.5608}$	0.932
11.70%	A24SR70S2	5.4516	0.6335	$y=5.4052x^{0.6335}$	0.9716

Table 4.5(B) Regression Model of Resilient Modulus versus Confining Pressure for SR70 A-2-4

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
8.41%	A24SR70D1	451.9800	0.0607	$y=451.98x^{0.0607}$	0.9849
7.76%	A24SR70D2	884.4900	-0.0571	$y=884.49x^{-0.0571}$	0.8558
10.80%	A24SR70O1	171.5100	0.1698	$y=39.914x^{0.439}$	0.9311
10.39%	A24SR70O2	104.8700	0.2578	$y=104.87x^{0.2578}$	0.9937
11.23%	A24SR70S1	55.6000	0.3807	$y=55.6x^{0.3807}$	0.9994
11.70%	A24SR70S2	63.7580	0.4172	$y=63.758x^{0.4172}$	0.9996
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
8.41%	A24SR70D1	120.0400	0.3369	$y=120.04x^{0.3369}$	0.9978
7.76%	A24SR70D2	14.031	0.7789	$y=14.031x^{0.7789}$	0.9964
10.80%	A24SR70O1	78.6520	0.2962	$y=45.686x^{0.3888}$	0.9969
10.39%	A24SR70O2	54.9130	0.3727	$y=54.913x^{0.3727}$	0.9959
11.23%	A24SR70S1	28.5950	0.4870	$y=28.595x^{0.487}$	0.9998
11.70%	A24SR70S2	19.5280	0.5366	$y=19.528x^{0.5366}$	0.9936

Table 4.6(A) Regression Model of Resilient Modulus versus Bulk Stress for A-2-4 20%

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula (y=Mr, x=θ)	R^2
8.3%	A2420%D1	54.1420	0.3077	$y=54.142x^{0.3077}$	0.9395
7.3%	A2420%D2	45.1320	0.3647	$y=45.132x^{0.3647}$	0.8629
10.0%	A2420%O1	12.2060	0.5459	$y=12.206x^{0.5459}$	0.9793
10.0%	A2420%O2	10.397	0.5586	$y=10.397x^{0.5586}$	0.9830
11.6%	A2420%S1	13.8160	0.5163	$y=13.816x^{0.5163}$	0.9749
12.3%	A2420%S2	10.0030	0.5621	$y=10.003x^{0.5621}$	0.9845
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula (y=Mr, x=θ)	R^2
8.3%	A2420%D1	11.0210	0.5807	$y=11.021x^{0.5807}$	0.9771
7.3%	A2420%D2	11.5170	0.5914	$y=11.517x^{0.5914}$	0.9612
10.0%	A2420%O1	7.7395	0.6080	$y=7.7395x^{0.6080}$	0.9881
10.0%	A2420%O2	6.5854	0.6529	$y=6.5854x^{0.6529}$	0.9867
11.6%	A2420%S1	7.8573	0.5907	$y=7.8573x^{0.5907}$	0.9825
12.3%	A2420%S2	6.9320	0.6137	$y=6.9320x^{0.6137}$	0.9893

Table 4.6(B) Regression Model of Resilient Modulus versus Confining Pressure for A-2-4 20%

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula (y=Mr, x=σ_3)	R^2
8.3%	A2420%D1	99.0040	0.2652	$y=99.0040x^{0.2652}$	0.9907
7.3%	A2420%D2	89.7970	0.3218	$y=89.797x^{0.3218}$	0.9896
10.0%	A2420%O1	38.2730	0.4519	$y=38.2730x^{0.4519}$	0.9881
10.0%	A2420%O2	34.2940	0.4928	$y=34.294x^{0.4928}$	0.9961
11.6%	A2420%S1	39.2840	0.4361	$y=39.284x^{0.4361}$	0.9915
12.3%	A2420%S2	31.8970	0.4694	$y=31.897x^{0.4694}$	0.9932
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula (y=Mr, x=σ_3)	R^2
8.3%	A2420%D1	35.5060	0.4906	$y=35.506x^{0.4906}$	0.9988
7.3%	A2420%D2	37.5780	0.5086	$y=37.5780x^{0.5086}$	0.9994
10.0%	A2420%O1	27.1730	0.5073	$y=27.1730x^{0.5073}$	0.9899
10.0%	A2420%O2	25.0700	0.5470	$y=25.07x^{0.547}$	0.9956
11.6%	A2420%S1	26.3510	0.4953	$y=26.351x^{0.4953}$	0.99
12.3%	A2420%S2	24.6500	0.5116	$y=24.65x^{0.5116}$	0.9924

Table 4.7(A) Regression Model of Resilient Modulus versus Bulk Stress for A-2-4 24%

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula ($y=Mr$, $x=\theta$)	R^2
7.72%	A2424%D1	16.7250	0.4530	$y=16.725x^{0.4530}$	0.9517
7.65%	A2424%D2	18.7130	0.4522	$y=18.713x^{0.4522}$	0.9580
10.70%	A2424%O1	21.5070	0.3889	$y=21.507x^{0.3889}$	0.9734
10.70%	A2424%O2	15.5120	0.4671	$y=15.512x^{0.4671}$	0.9976
12.00%	A2424%S1	4.9754	0.6255	$y=4.9754x^{0.6255}$	0.9744
11.40%	A2424%S2	8.2687	0.6080	$y=8.2687x^{0.608}$	0.8505
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula ($y=Mr$, $x=\theta$)	R^2
7.72%	A2424%D1	10.5540	0.5262	$y=10.554x^{0.5262}$	0.9844
7.65%	A2424%D2	13.5780	0.4954	$y=13.578x^{0.4954}$	0.9771
10.70%	A2424%O1	8.5045	0.5469	$y=8.5045x^{0.5469}$	0.9916
10.70%	A2424%O2	6.9937	0.5987	$y=6.9937x^{0.5987}$	0.9757
12.00%	A2424%S1	3.6325	0.6465	$y=3.6325x^{0.6465}$	0.9979
11.40%	A2424%S2	3.7207	0.7035	$y=3.7207x^{0.7035}$	0.9769

Table 4.7(B) Regression Model of Resilient Modulus versus Confining Pressure for A-2-4 24%

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
7.72%	A2424%D1	41.4400	0.3859	$y=41.44x^{0.3859}$	0.981
7.65%	A2424%D2	45.9930	0.3865	$y=45.993x^{0.3865}$	0.9913
10.70%	A2424%O1	47.0930	0.3291	$y=47.093x^{0.3291}$	0.9984
10.70%	A2424%O2	38.8840	0.4017	$y=38.884x^{0.4017}$	0.993
12.00%	A2424%S1	17.5770	0.5290	$y=17.577x^{0.529}$	0.9986
11.40%	A2424%S2	25.5980	0.5399	$y=25.598x^{0.5399}$	0.9982
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
7.72%	A2424%D1	31.0020	0.4415	$y=31.002x^{0.4415}$	0.9901
7.65%	A2424%D2	36.8590	0.4196	$y=36.859x^{0.4196}$	0.9935
10.70%	A2424%O1	26.6810	0.4528	y=26.681x0.4528	0.997
10.70%	A2424%O2	23.7020	0.5039	$y=23.702x^{0.5039}$	0.9859
12.00%	A2424%S1	14.2170	0.5319	$y=14.217x^{0.5319}$	0.994
11.40%	A2424%S2	15.9130	0.5977	$y=15.913x^{0.5977}$	0.9804

Table 4.8(A) Regression Model of Resilient Modulus versus Bulk Stress for A-2-4 30%

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula (y=Mr, x=θ)	R^2
6.30%	A2430%D1	596.33	0.0647	$y=596.33x^{0.0647}$	0.0379
7.00%	A2430%D2	540.26	0.0793	$y=540.26x^{0.0793}$	0.0564
12.00%	A2430%O1	10.877	0.5973	$y=10.877x^{0.5973}$	0.3938
12.30%	A2430%O2	9.9673	0.5778	$y=9.9673x^{0.5778}$	0.4464
13.40%	A2430%S1	12.556	0.5469	$y=12.556x^{0.5469}$	0.712
13.20%	A2430%S2	13.122	0.5448	$y=13.122x^{0.5448}$	0.6345
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula (y=Mr, x=θ)	R^2
6.30%	A2430%D1	19.176	0.6226	$y=19.176x^{0.6226}$	0.9343
7.00%	A2430%D2	21.326	0.6001	$y=21.326x^{0.6001}$	0.9418
12.00%	A2430%O1	3.3241	0.7184	$y=3.3241x^{0.7184}$	0.9353
12.30%	A2430%O2	3.2058	0.7096	$y=3.2058x^{0.7096}$	0.9459
13.40%	A2430%S1	2.7408	0.7601	$y=2.7408x^{0.7601}$	0.9590
13.20%	A2430%S2	3.2634	0.7073	$y=3.2634x^{0.7073}$	0.9681

Table 4.8(B) Regression Model of Resilient Modulus versus Confining Pressure for A-2-4 30%

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula (y=Mr, x=σ_3)	R^2
6.30%	A2430%D1	578.4	0.1012	$y=578.4x^{0.1012}$	0.8096
7.00%	A2430%D2	537.35	0.1142	$y=537.35x^{0.1142}$	0.8753
12.00%	A2430%O1	27.255	0.6002	$y=27.255x^{0.6002}$	0.9982
12.30%	A2430%O2	25.023	0.5683	$y=25.023x^{0.5583}$	0.9932
13.40%	A2430%S1	33.301	0.5059	$y=33.301x^{0.5059}$	0.993
13.20%	A2430%S2	34.09	0.5106	$y=34.09x^{0.5106}$	0.9944
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula (y=Mr, x=σ_3)	R^2
6.30%	A2430%D1	64.499	0.5363	$y=64.499x^{0.5363}$	0.9942
7.00%	A2430%D2	68.457	0.5188	$y=68.457x^{0.5188}$	0.9996
12.00%	A2430%O1	13.503	0.6208	$y=13.503x^{0.6208}$	0.9943
12.30%	A2430%O2	12.879	0.6112	$y=12.879x^{0.6112}$	0.9965
13.40%	A2430%S1	12.42	0.6492	$y=12.42x^{0.6492}$	0.9941
13.20%	A2430%S2	13.528	0.6004	$y=13.528x^{0.6004}$	0.989

Table 4.9(A) Regression Model of Resilient Modulus versus Bulk Stress for A-1 Oolite

Moisture Content	Sample No.	Middle Half			
		k_1	k_2	Formula (y=Mr, x=θ)	R^2
5.60%	OOLITED1	24.9270	0.5514	$y=24.927x^{0.5514}$	0.9343
4.40%	OOLITED2	34.2920	0.5020	$y=34.292x^{0.502}$	0.8909
7.80%	OOLITEO1	5.0349	0.7568	$y=5.0349x^{0.7568}$	0.9182
7.80%	OOLITEO2	5.9633	0.7204	$y=5.9633x^{0.7204}$	0.9199
8.20%	OOLITES1	1.6414	0.8946	$y=1.6414x^{0.8946}$	0.9429
8%	OOLITES2	3.8590	0.7655	$y=3.859x^{0.7655}$	0.9588
Moisture Content	Sample No.	Full Length			
		k'_1	k'_2	Formula (y=Mr, x=θ)	R^2
5.60%	OOLITED1	4.9032	0.7596	$y=4.9032x^{0.7596}$	0.9807
4.40%	OOLITED2	9.9158	0.5774	$y=9.9158x^{0.5774}$	0.9637
7.80%	OOLITEO1	3.0146	0.8194	$y=3.0146x^{0.8194}$	0.9552
7.80%	OOLITEO2	3.5722	0.7921	$y=3.5722x^{0.7921}$	0.9545
8.20%	OOLITES1	0.9275	0.9888	$y=0.9275x^{0.9888}$	0.9728
8%	OOLITES2	2.4621	0.8330	$y=2.4621x^{0.833}$	0.9593

Table 4.9(B) Regression Model of Resilient Modulus versus Confining Pressure for A-1 Oolite

Moisture Content	Sample No.	Middle Half			
		k_3	k_4	Formula (y=Mr, x=σ_3)	R^2
5.60%	OOLITED1	75.8540	0.4682	$y=75.854x^{0.4682}$	0.9653
4.40%	OOLITED2	90.4880	0.4371	$y=90.488x^{0.4371}$	0.9739
7.80%	OOLITEO1	24.0410	0.6205	$y=24.041x^{0.6205}$	0.9254
7.80%	OOLITEO2	27.3050	0.5951	$y=27.3050x^{0.5951}$	0.9269
8.20%	OOLITES1	10.9870	0.7349	$y=678.53x^{0.0469}$	0.9429
8%	OOLITES2	18.3600	0.6454	$y=18.36x^{0.6454}$	0.9759
Moisture Content	Sample No.	Full Length			
		k'_3	k'_4	Formula (y=Mr, x=σ_3)	R^2
5.60%	OOLITED1	23.4790	0.6349	$y=23.479x^{0.6349}$	0.9872
4.40%	OOLITED2	38.8570	0.5740	$y=38.857x^{0.574}$	0.9870
7.80%	OOLITEO1	16.4290	0.6842	$y=16.429x^{0.6842}$	0.9642
7.80%	OOLITEO2	18.6200	0.6585	$y=18.62x^{0.6585}$	0.9603
8.20%	OOLITES1	7.3804	0.8180	$y=7.3804x^{0.818}$	0.971
8%	OOLITES2	13.3310	0.7043	$y=13.331x^{0.7043}$	0.9784

Table 4.10(A) Regression Model of Resilient Modulus versus Bulk Stress for Spring Cemetery A-2-4

Moisture Content	Sample No.	Full Length			
		K_1	K_2	Formula ($y=Mr$, $x=\theta$)	R^2
9.20%	SC001C1	6.5289	0.5608	$y=6.5289x^{0.5608}$	0.9495
9.20%	SC001D1	5.2654	0.5801	$y=5.2654x^{0.5801}$	0.9739
9.30%	SC001E1	5.6704	0.5511	$y=5.6704x^{0.5511}$	0.9832
9.30%	SC001F1	5.1598	0.5944	$y=5.1598x^{0.5944}$	0.9453

Table 4.10(B) Regression Model of Resilient Modulus versus Confining Pressure for Spring Cemetery A-2-4

Moisture Content	Sample No.	Full Length			
		K_3	K_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
9.20%	SC001C1	21.539	0.4691	$y=21.539x^{0.4691}$	0.9204
9.20%	SC001D1	18.509	0.4811	$y=18.509x^{0.4811}$	0.9149
9.30%	SC001E1	15.422	0.4996	$y=15.422x^{0.4996}$	0.9332
9.30%	SC001F1	15.812	0.5393	$y=15.812x^{0.5393}$	0.9612

Table 4.11(A) Regression Model of Resilient Modulus versus Bulk Stress for Branch A-2-4

Moisture Content	Sample No.	Full Length			
		K_1	K_2	Formula ($y=Mr$, $x=\theta$)	R^2
8.70%	BH001C1	31.985	0.3848	$y=31.985x^{0.3848}$	0.5244
8.70%	BH001D2	13.277	0.6256	$y=13.277x^{0.6256}$	0.7156
8.90%	BH001E1	46.913	0.3171	$y=46.913x^{0.3171}$	0.4444
8.90%	BH001F1	39.755	0.3637	$y=39.755x^{0.3637}$	0.4793
9.30%	BH001G1	8.1296	0.577	$y=8.1296x^{0.577}$	0.7335
9.30%	BH001H1	13.914	0.4985	$y=13.914x^{0.4985}$	0.6085

Table 4.11(B) Regression Model of Resilient Modulus versus Confining Pressure for Branch A-2-4

Moisture Content	Sample No.	Full Length			
		K_3	K_4	Formula ($y=Mr$, $x=\sigma_3$)	R^2
8.70%	BH001C1	61.913	0.3702	$y=61.913x^{03702}$	0.9127
8.70%	BH001D2	48.637	0.5285	$y=48.637x^{0.5285}$	0.9407
8.90%	BH001E1	70.958	0.3401	$y=70.958x^{0.3401}$	0.883
8.90%	BH001F1	66.206	0.3792	$y=66.206x^{0.3792}$	0.8918
9.30%	BH001G1	25.442	0.4991	$y=25.441x^{0.4991}$	0.9417
9.30%	BH001H1	32.379	0.475	$y=32.379x^{0.475}$	0.9591

Table 4.12(A) Regression Model of Resilient Modulus versus Bulk Stress for Iron Bridge A-2-6

Moisture Content	Sample No.	Full Length			
		K_1	K_2	Formula (y=Mr, x=θ)	R^2
10.30%	IB001C1	13.211	0.3678	$y=13.211x^{0.3678}$	0.7348
10.30%	IB001D1	11.443	0.3946	$y=11.443x^{0.3946}$	0.7027
10.40%	IB001E1	12.603	0.3762	$y=12.603x^{0.3762}$	0.6927
10.40%	IB001F1	15.269	0.3269	$y=15.269x^{0.3269}$	0.6658

Table 4.12(B) Regression Model of Resilient Modulus versus Confining Pressure for Iron Bridge A-2-6

Moisture Content	Sample No.	Full Length			
		K_3	K_4	Formula (y=Mr, x=σ_3)	R^2
10.30%	IB001C1	43.418	0.1751	$y=43.418x^{0.1751}$	0.285
10.30%	IB001D1	39.218	0.2051	$y=39.218x^{0.2051}$	0.3062
10.40%	IB001E1	43.411	0.1746	$y=43.411x^{0.1746}$	0.2499
10.40%	IB001F1	43.379	0.158	$y=43.379x^{0.158}$	0.2355

Table 4.13 Suction Values for the Eight Soils

Soil	Suction - Water Content Regression (Suction = $A(e)^{B(\text{Water Content})}$)			Optimum Water Content	Suction
	A	B	R-Square	%	kPa
Levy	35.108	-0.0728	0.0151	10	17
SR70 A-3	457.66	-0.3039	0.6394	11.5	14
A-2-4 (12%)	209.72	0.0613	0.8523	12.1	440
SR70 A-2-4	12094	-0.4762	0.5145	10.5	81
A-2-4 (20%)	16837	-0.381	0.9951	10	373
A-2-4 (24%)	5892.2	-0.2728	0.8162	10.7	318
A-2-4 (30%)	594.19	-0.0517	0.132	12	320
Miami Oolite A-1	474.73	-0.1109	0.6174	7.6	204

Table 4.14 Permeability Test Results

Soil Type	Percentage of Passing No. 200 sieve (%)	Permeability (cm/s)
Levy A-3	4	$5.52*10^{-3}$
SR70 A-3	8	$2.06*10^{-3}$
A-2-4 (12%)	12	$3.05*10^{-4}$
SR70 A-2-4	14	$2.5*10^{-4}$
Spring Cemetery A-2-4	15	$2.76*10^{-4}$
A-2-4 (20%)	20	$1.04*10^{-4}$
Branch A-2-4	23	$7.42*10^{-7}$
A-2-4 (24%)	24	$6.50*10^{-5}$
A-2-4 (30%)	30	$2.01*10^{-5}$
Iron Bridge A-2-6	31	$5.6*10^{-7}$

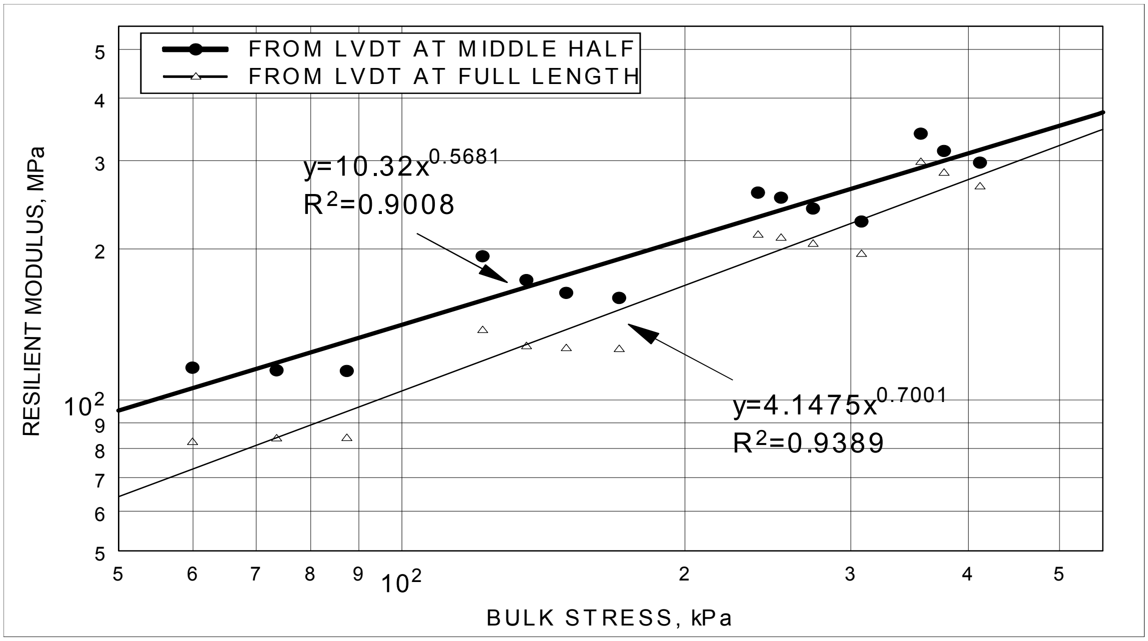


Figure 4.1(A) Typical Regression Model of Resilient Modulus versus Bulk Stress for A-2-4 30% after Soaking (Sample # A2430%S2)

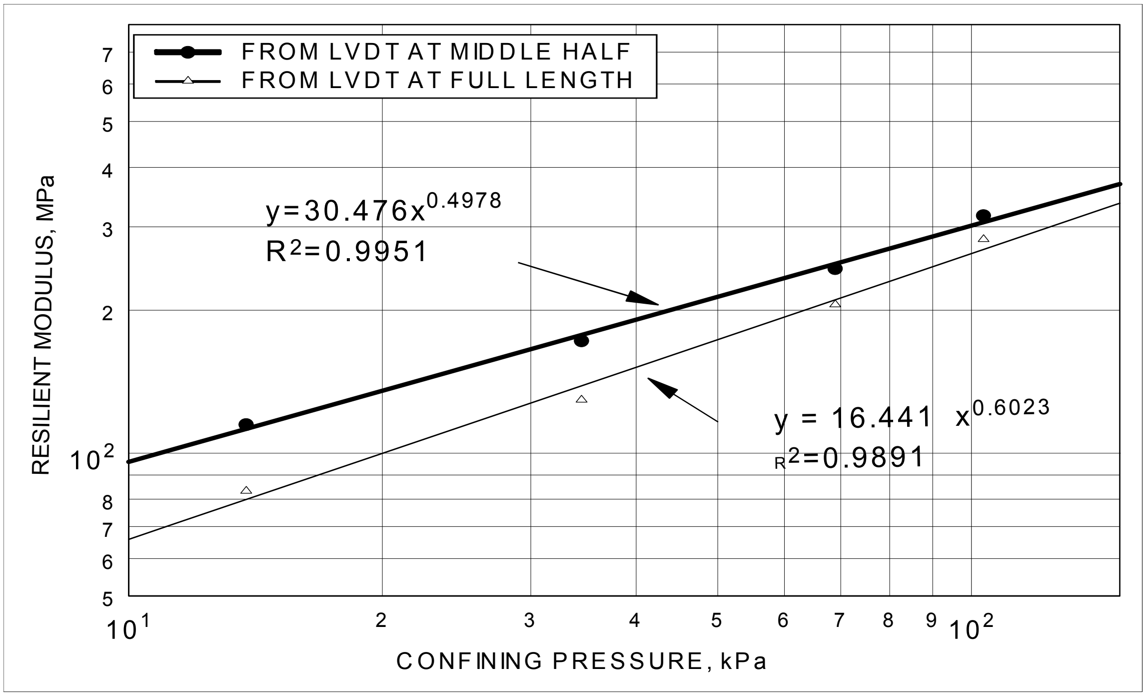


Figure 4.1(B) Typical Regression Model of Resilient Modulus versus Confining Pressure for A-2-4 30% after Soaking (Sample # A2430%S2)

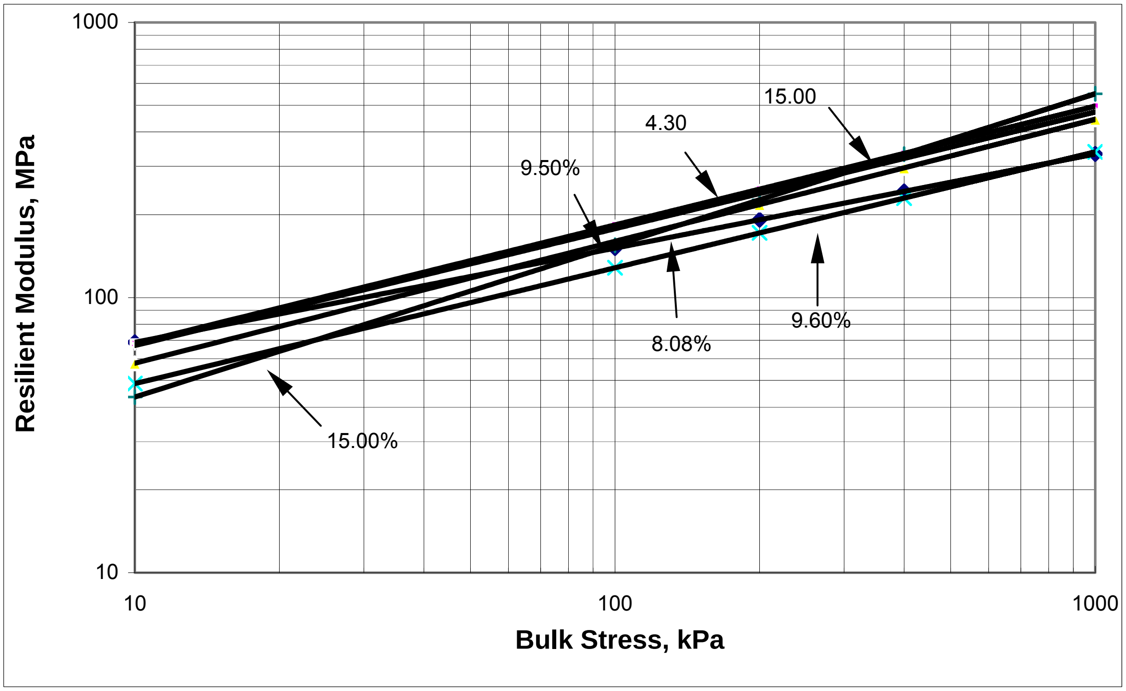


Figure 4.2(A) Resilient Modulus vs. Bulk Stress for Levy County A-3 at Different Moisture Contents

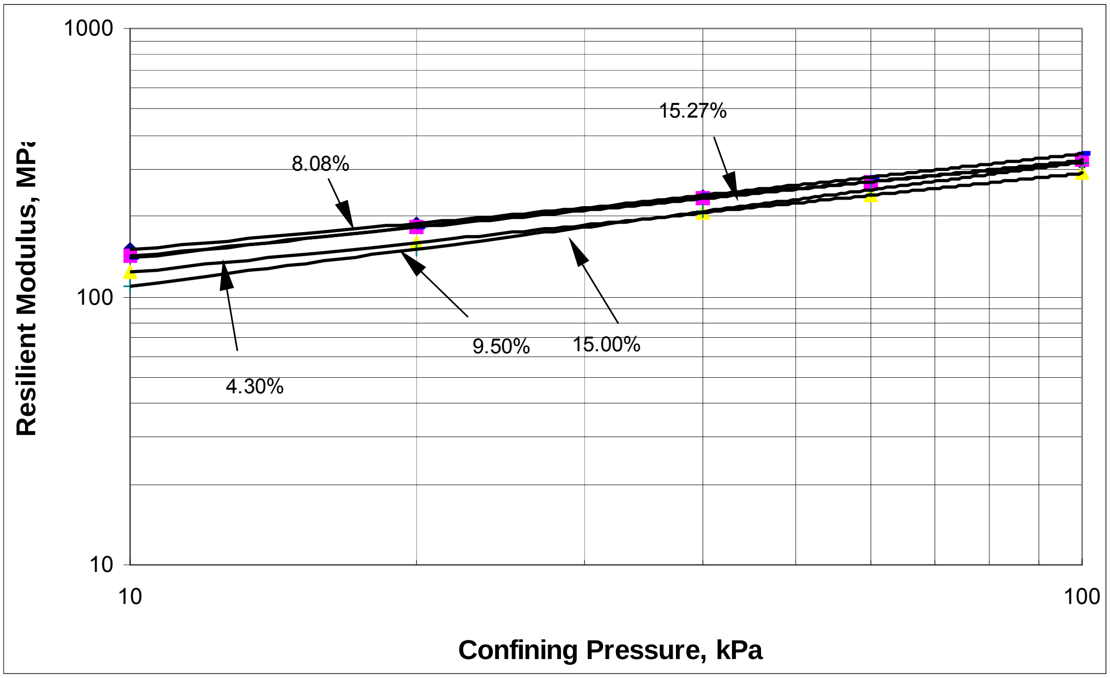


Figure 4.2(B) Resilient Modulus vs. Confining Pressure of Levy County A-3 at Different Moisture Contents

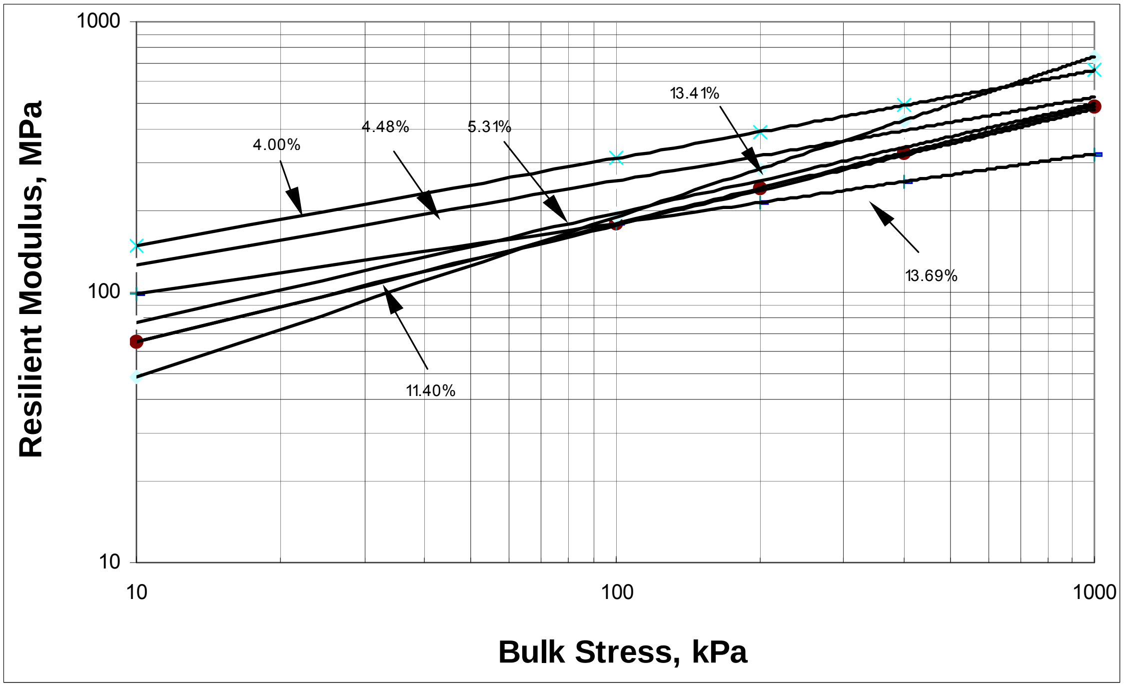


Figure 4.3(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of SR70 A-3

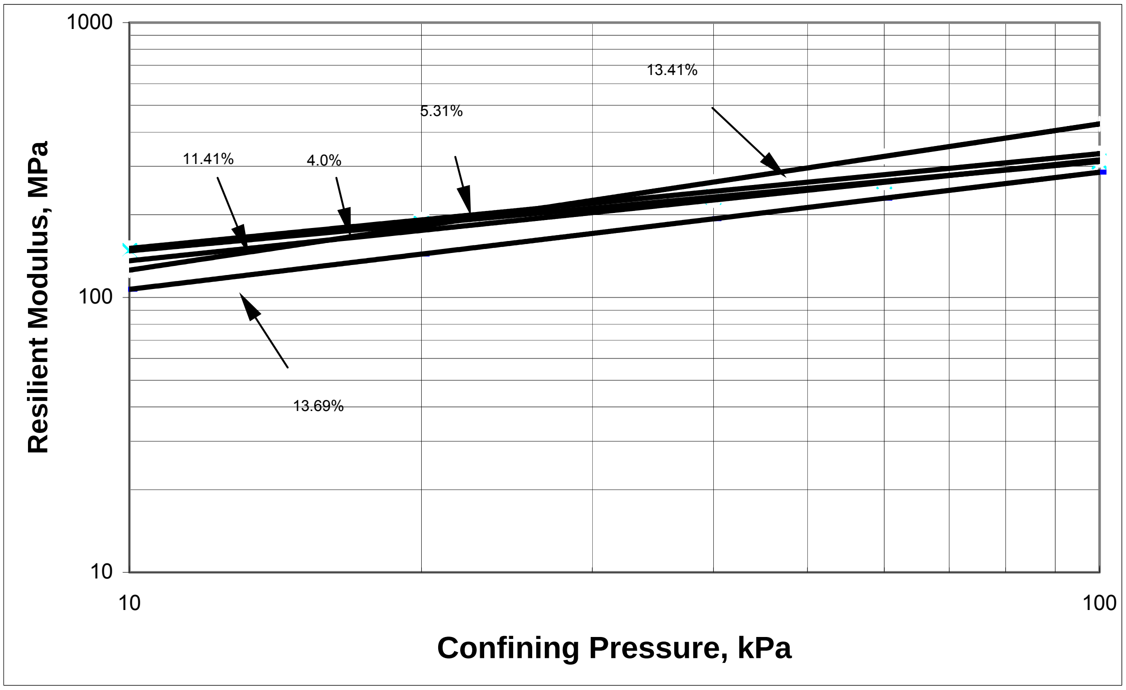


Figure 4.3(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of SR70 A-3

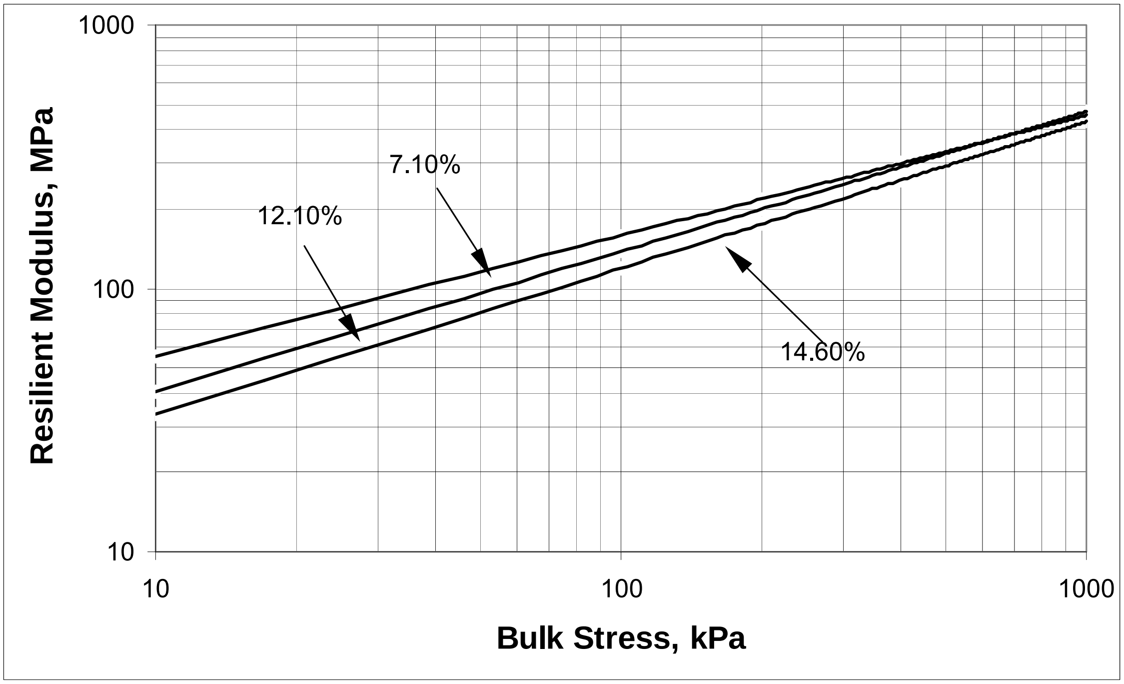


Figure 4.4(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of A-2-4, 12%

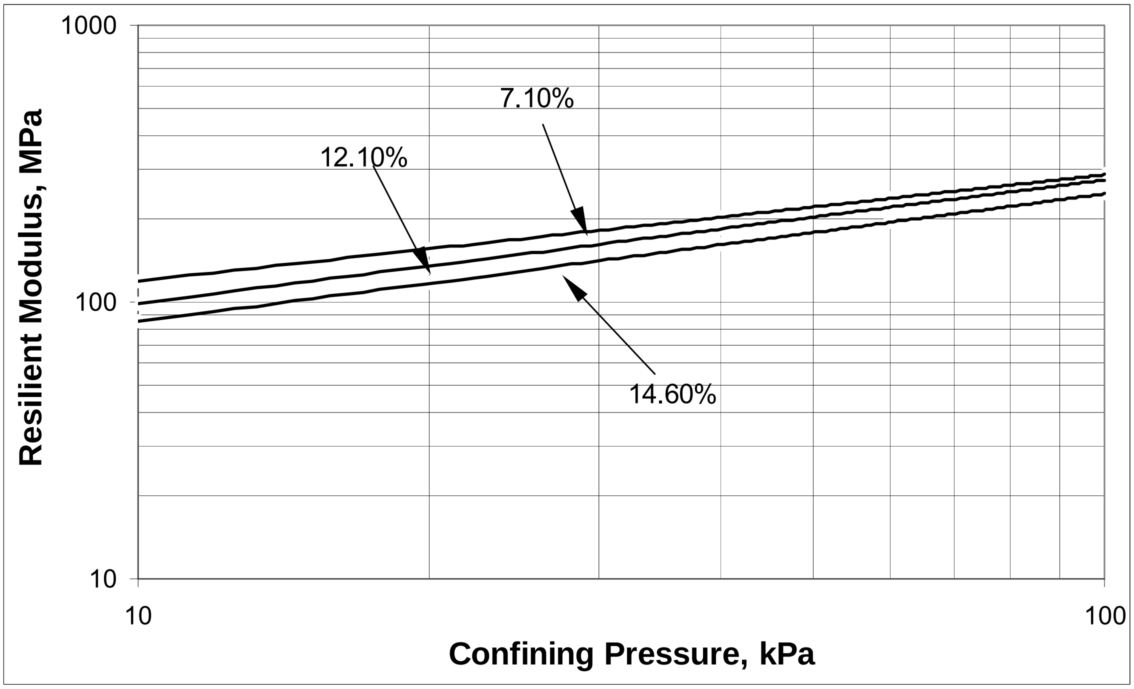


Figure 4.4(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of A-2-4, 12%

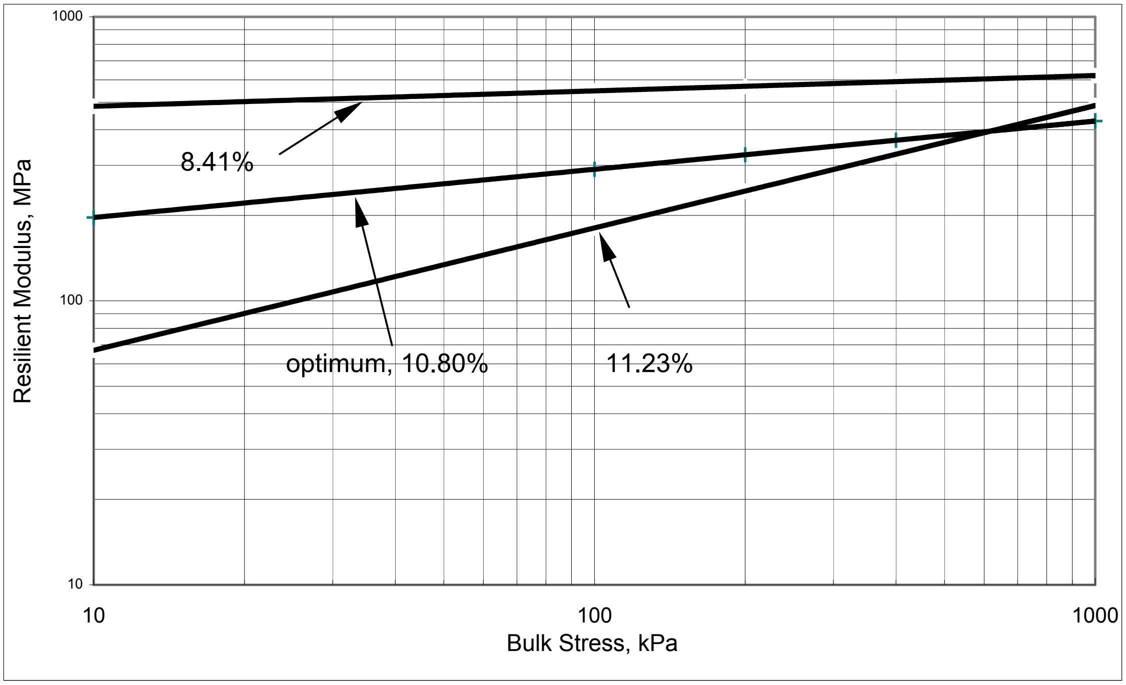


Figure 4.5(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of SR70 A-2-4

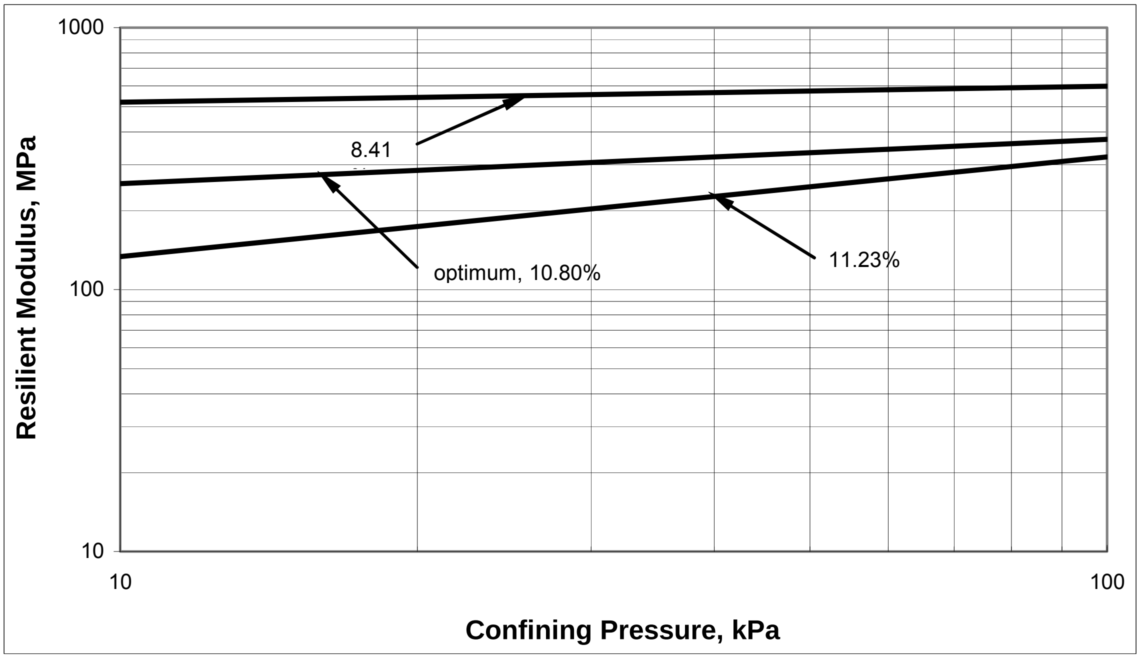


Figure 4.5(B) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of SR70 A-2-4

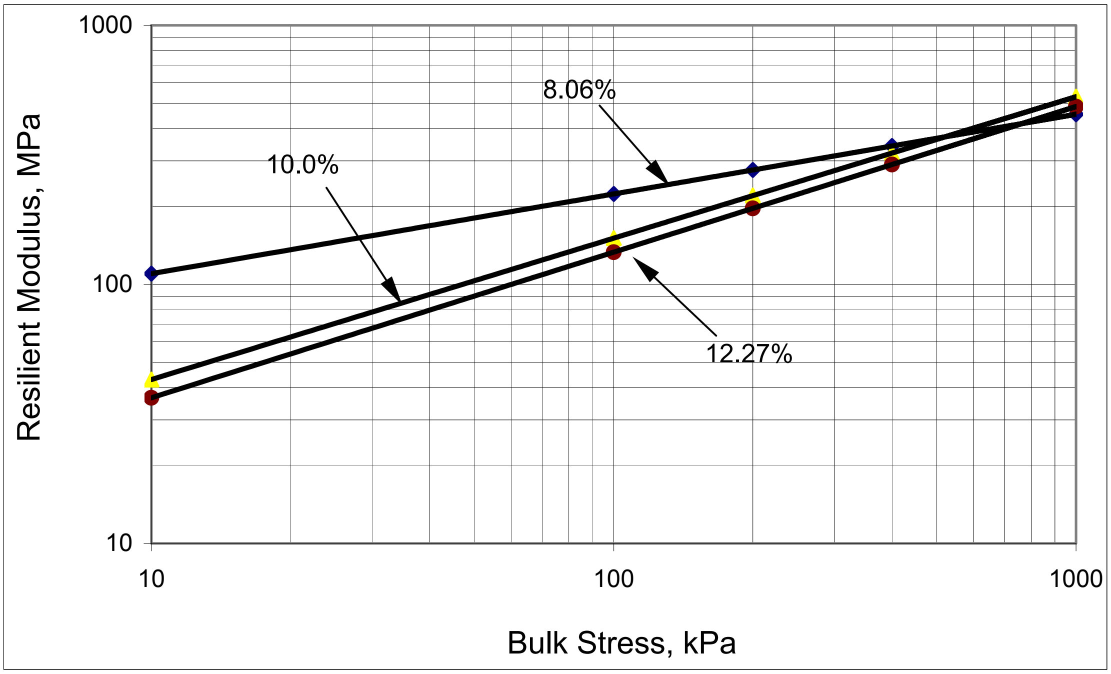


Figure 4.6(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of A-2-4 20%

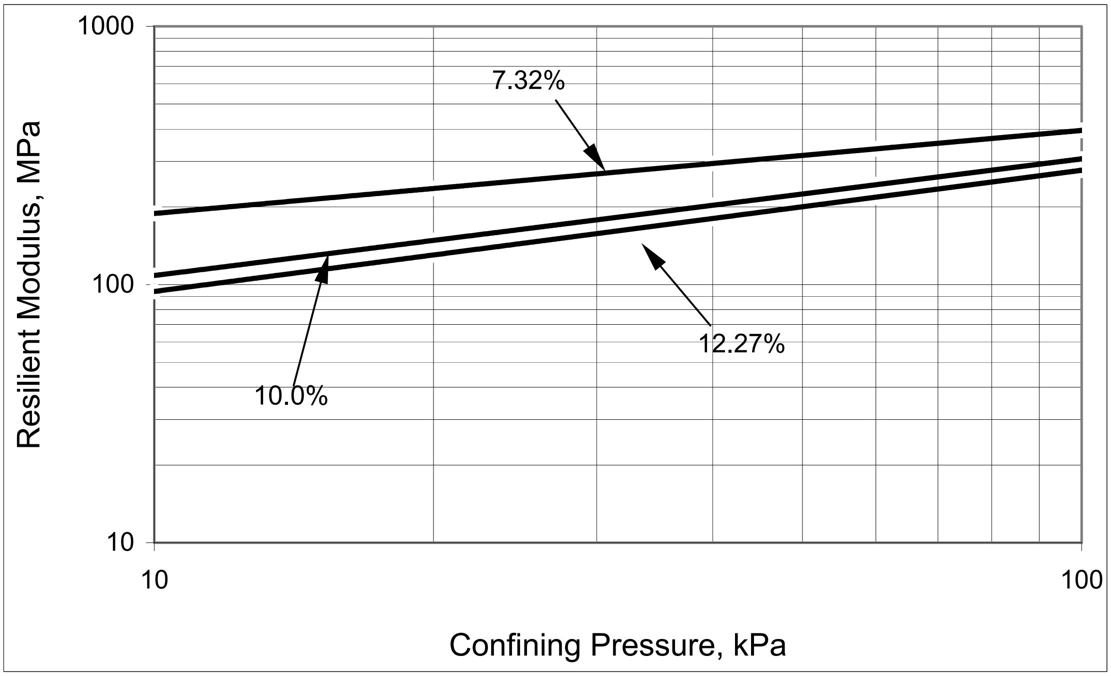


Figure 4.6(B) Resilient Modulus vs. Confining Stress at Different Moisture Contents of A-2-4 20%

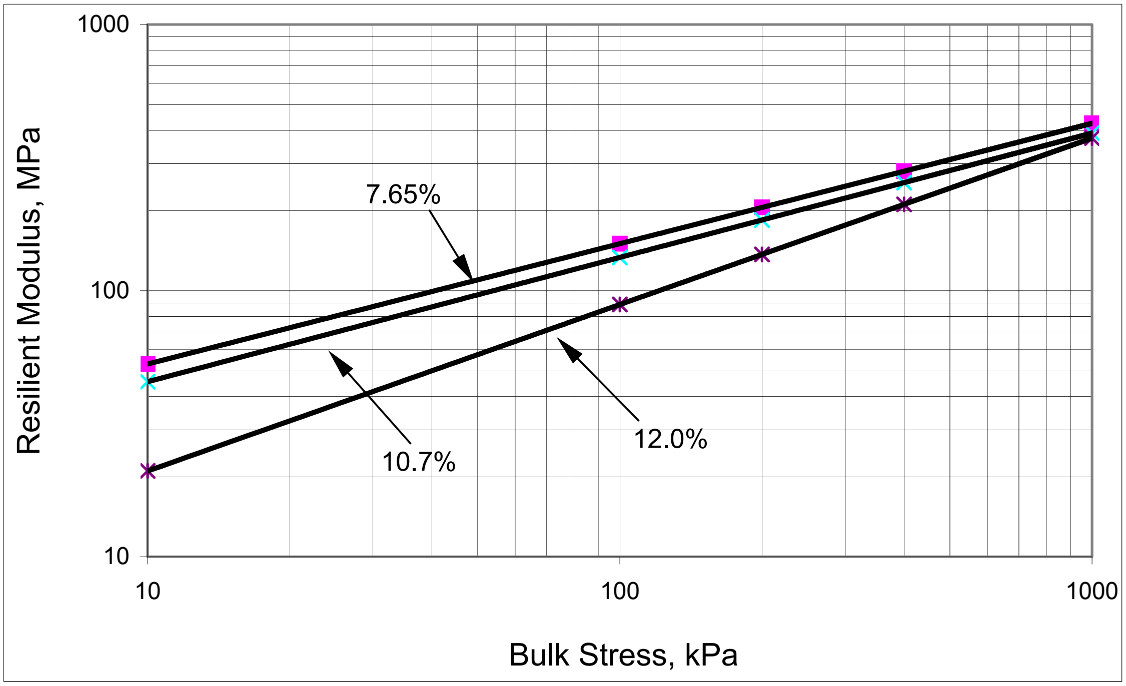


Figure 4.7(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of A-2-4 24%

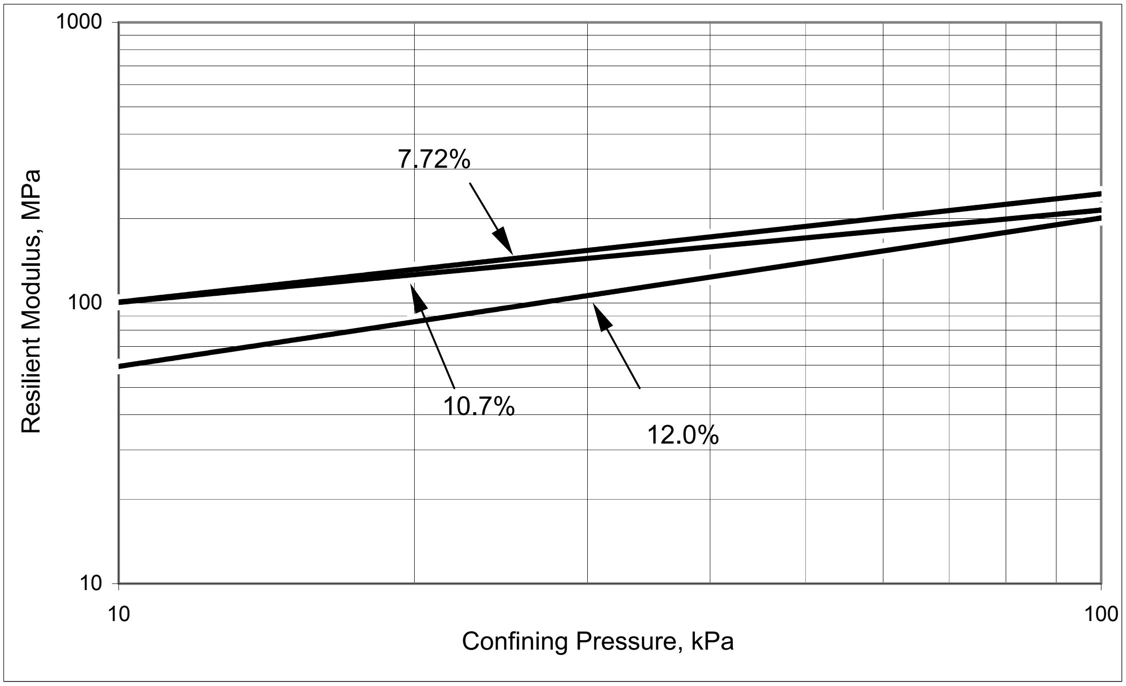


Figure 4.7(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of A-2-4 24%

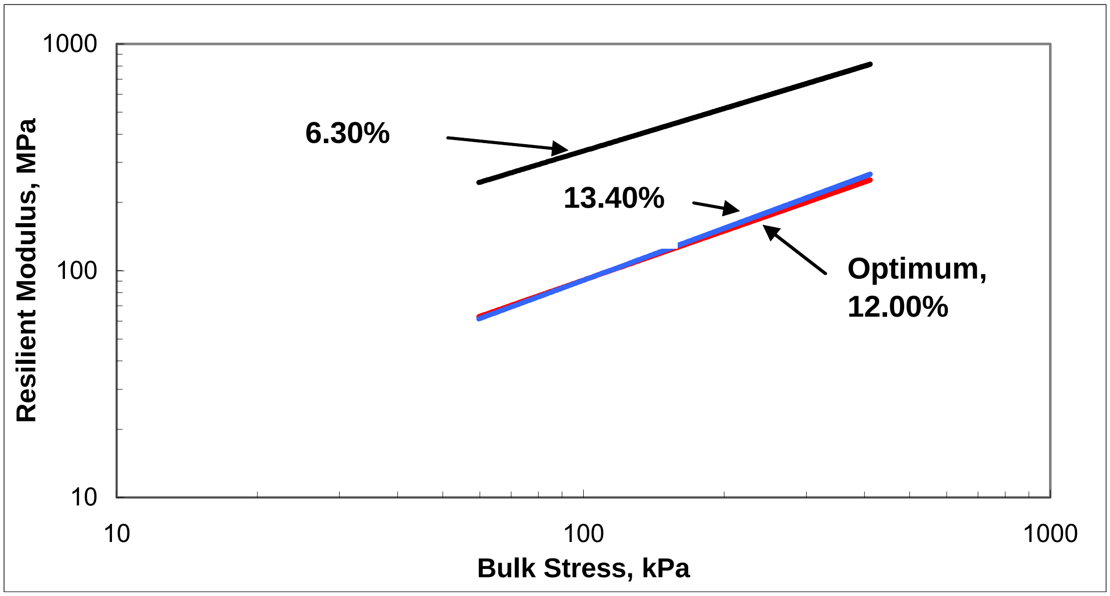


Figure 4.8(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of A-2-4 30%

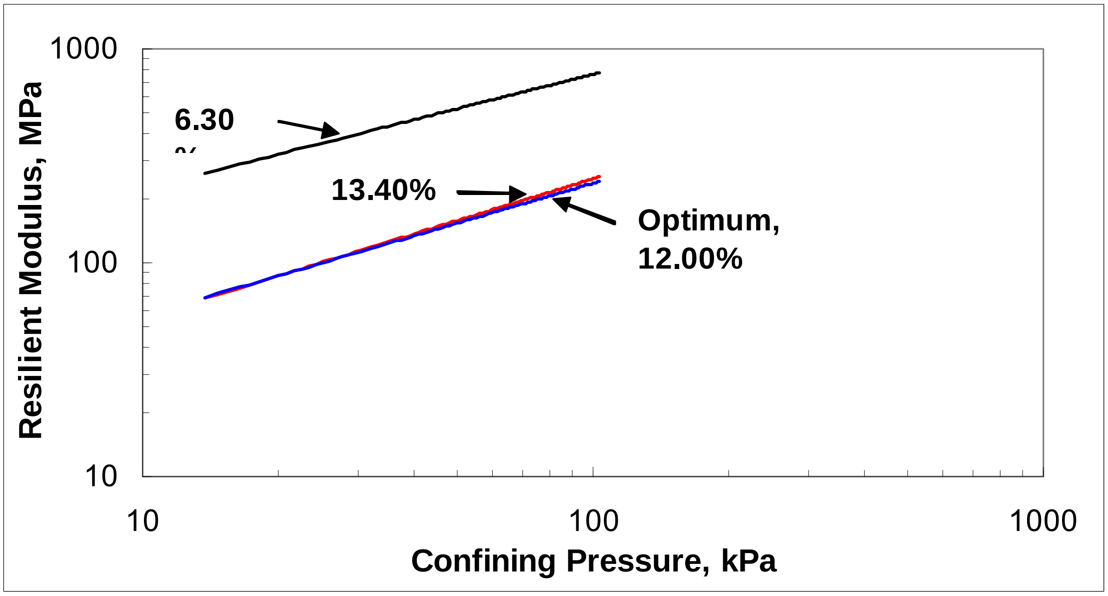


Figure 4.8(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of A-2-4 30%

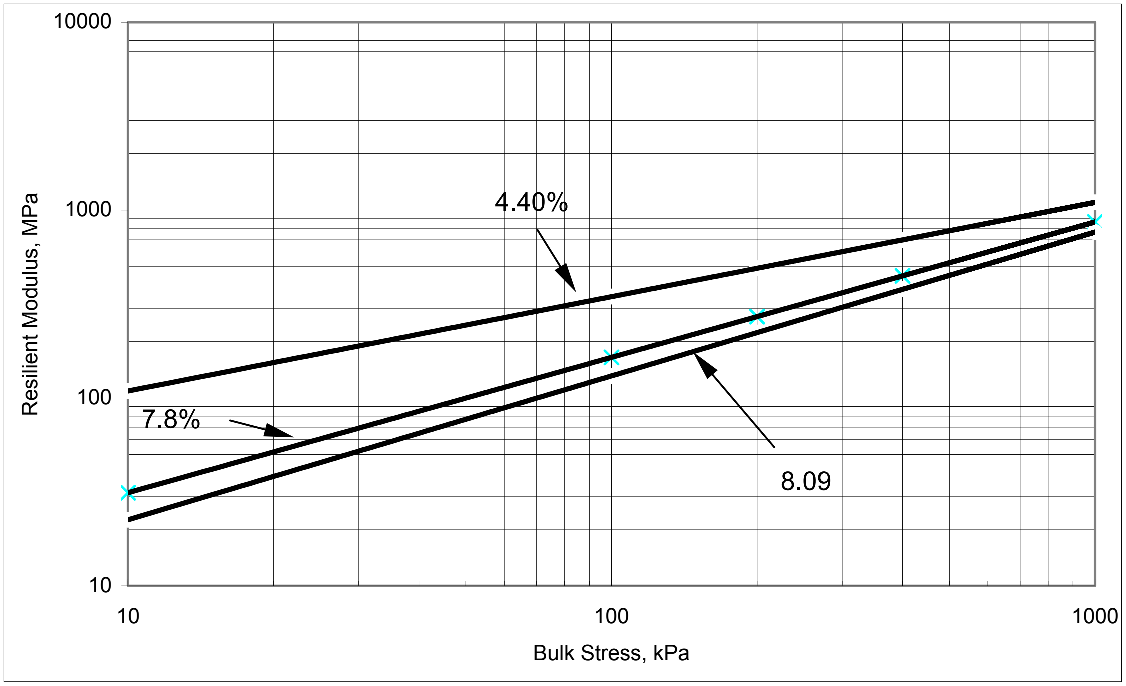


Figure 4.9(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of Oolite, Miami

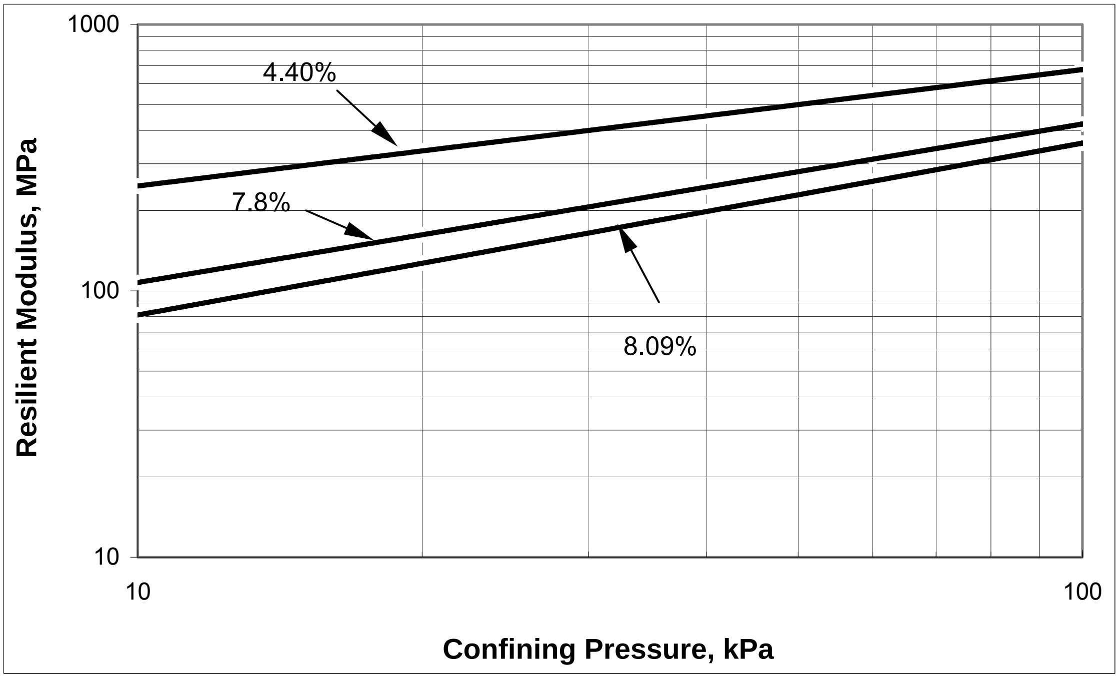


Figure 4.9(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of Oolite, Miami

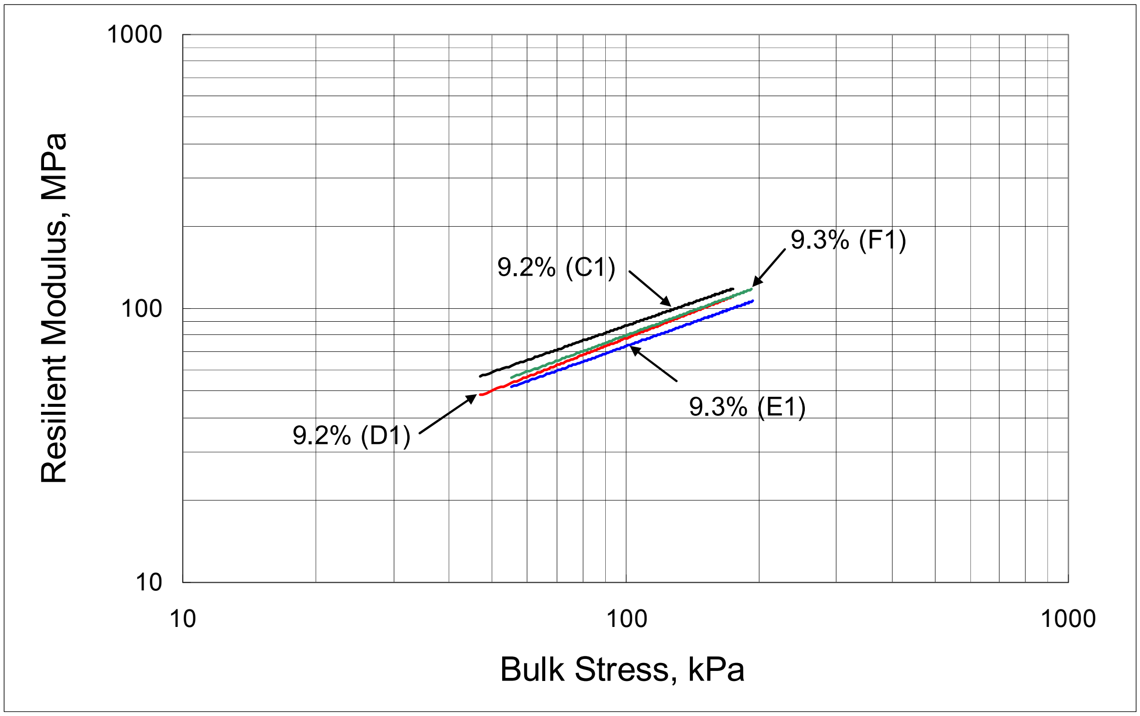


Figure 4.10(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of Spring Cemetery A-2-4 Soil

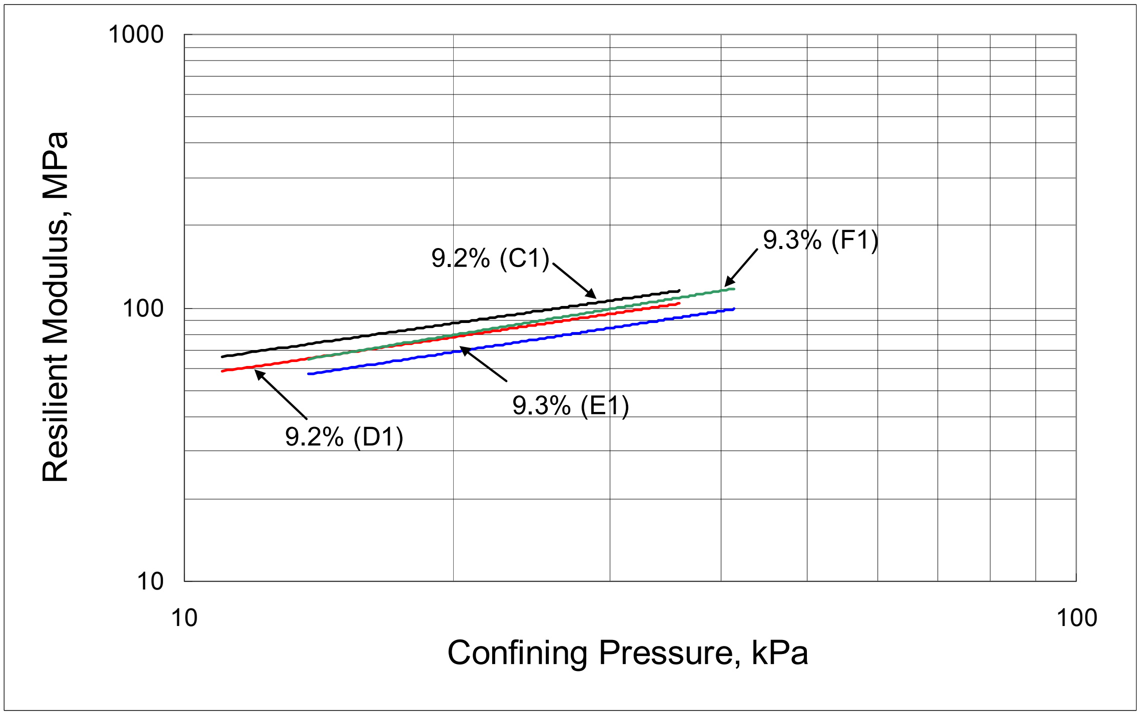


Figure 4.10(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of Spring Cemetery A-2-4 Soil

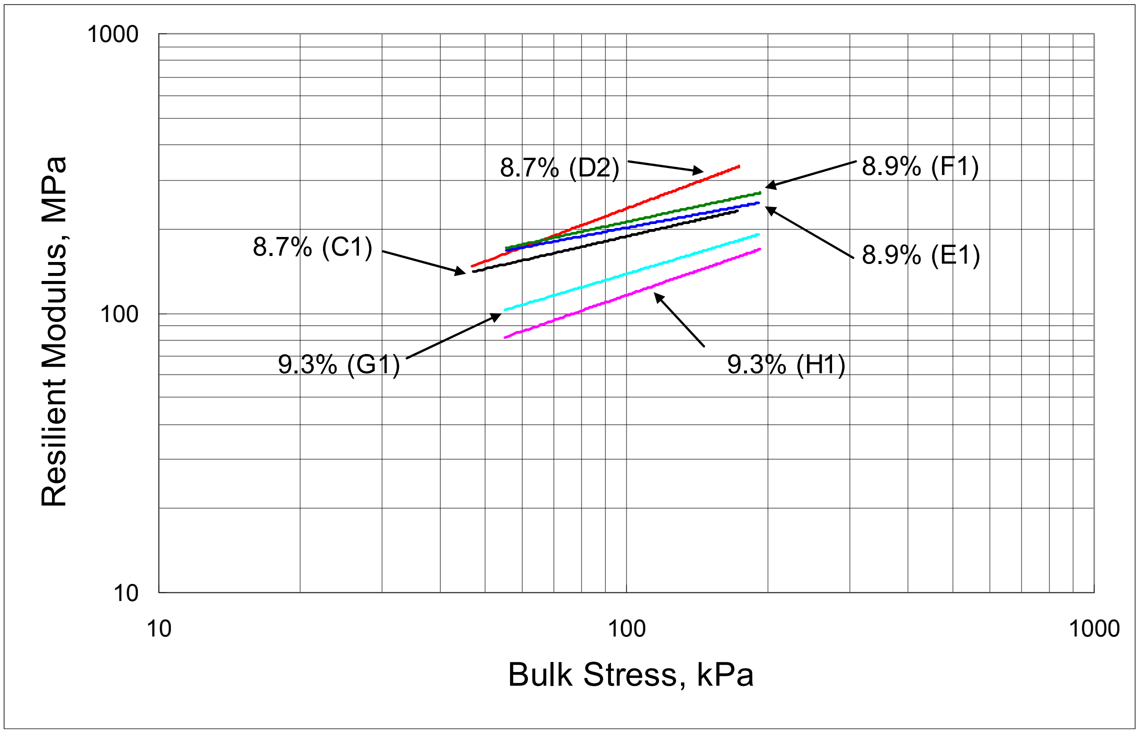


Figure 4.11(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of Branch A-2-4 Soil

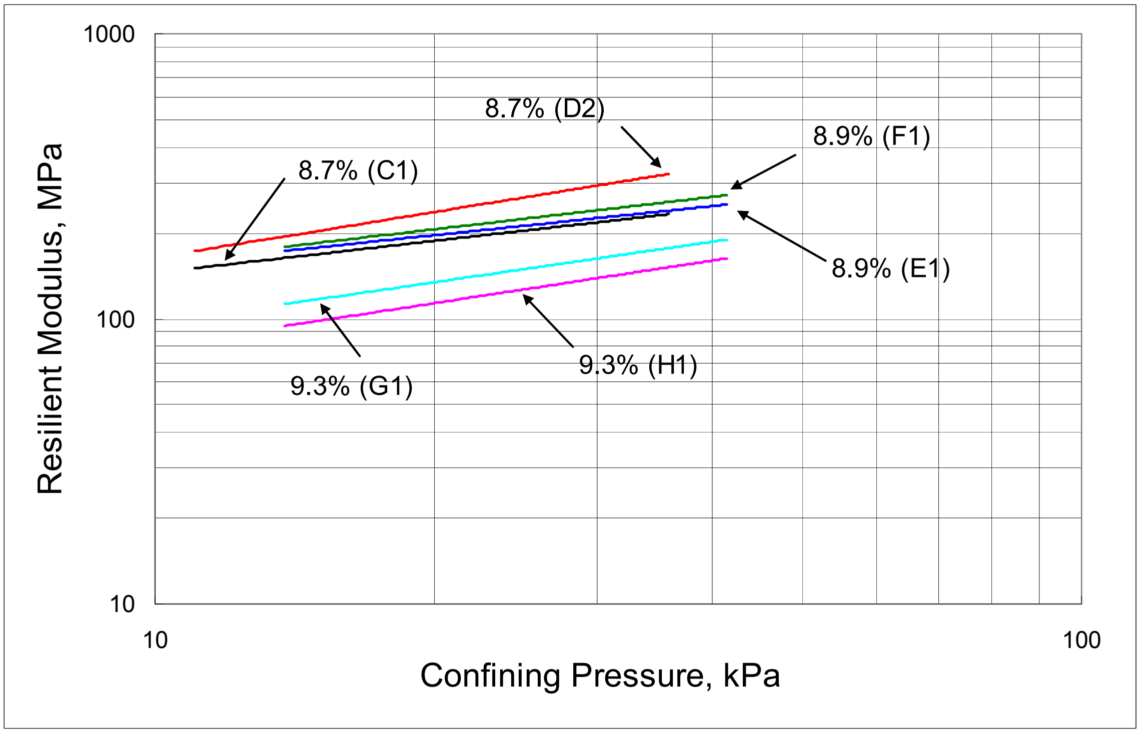


Figure 4.11(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of Branch A-2-4 Soil

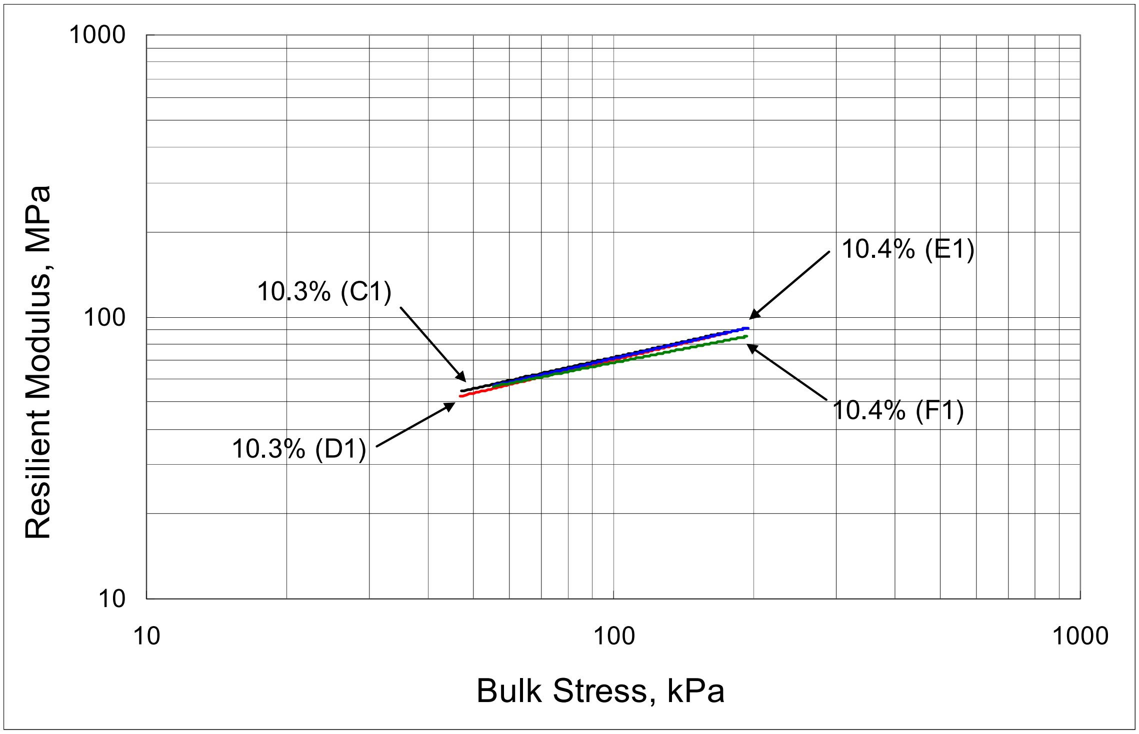


Figure 4.12(A) Resilient Modulus vs. Bulk Stress at Different Moisture Contents of Iron Bridge A-2-6 Soil

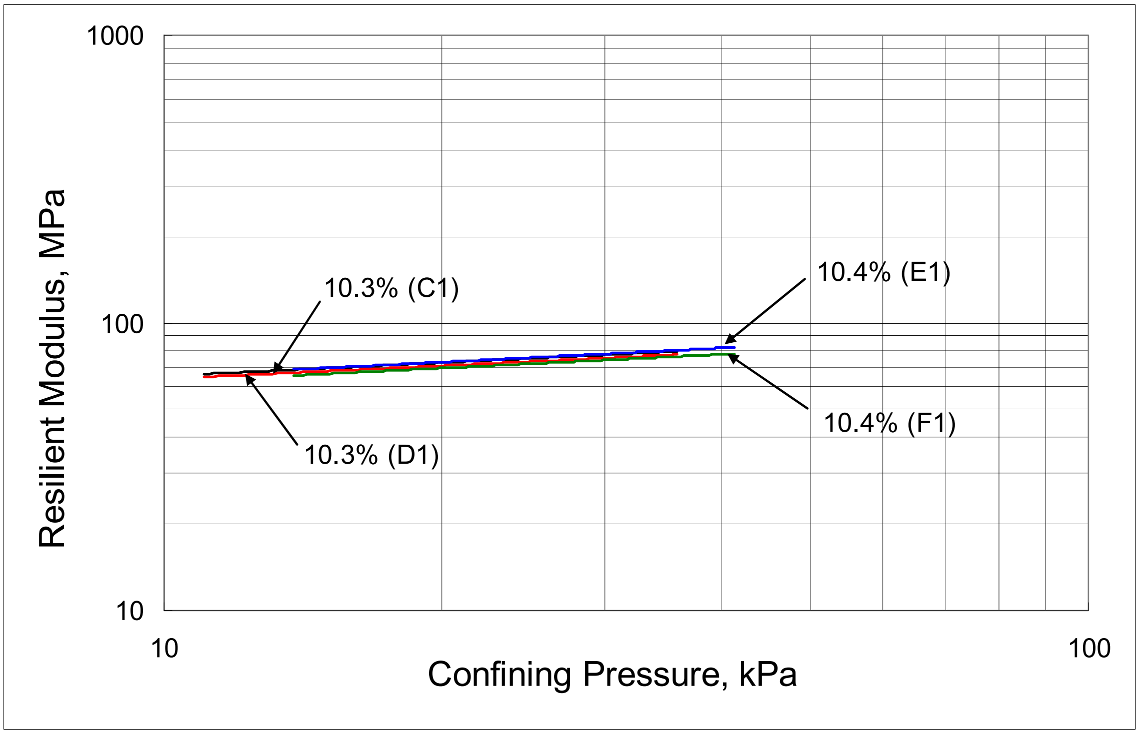


Figure 4.12(B) Resilient Modulus vs. Confining Pressure at Different Moisture Contents of Iron Bridge A-2-6 Soil

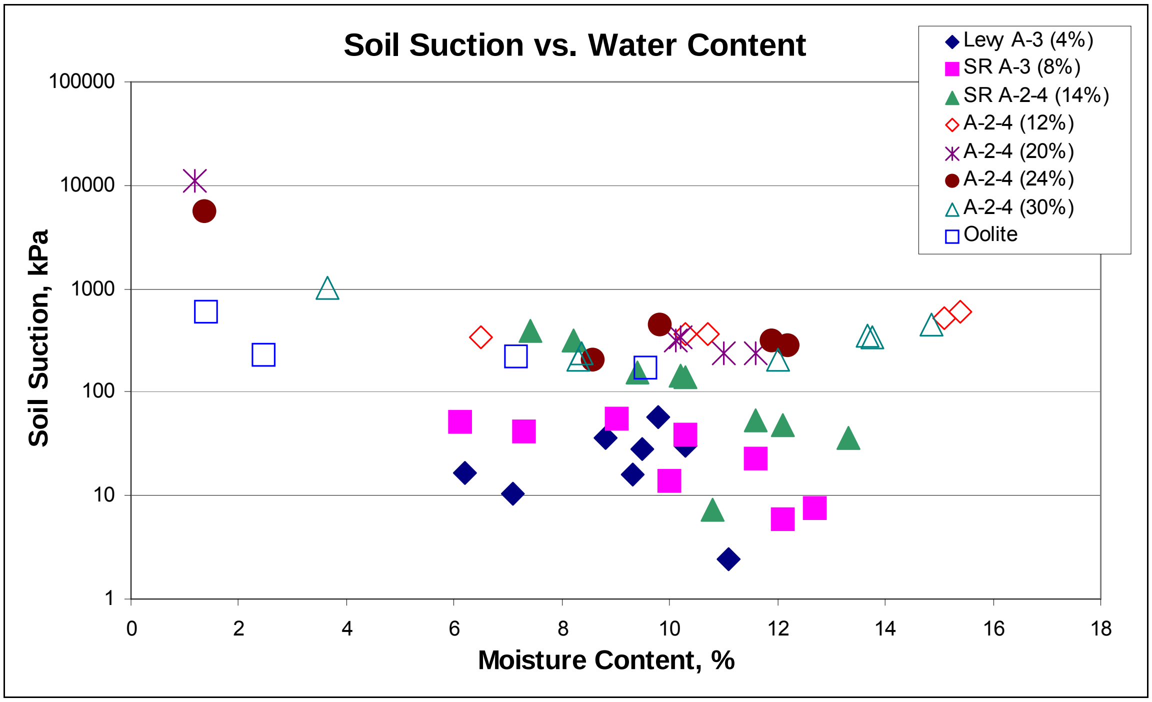


Figure 4.13 Suction Value for Each Soil at Different Moisture Content Levels

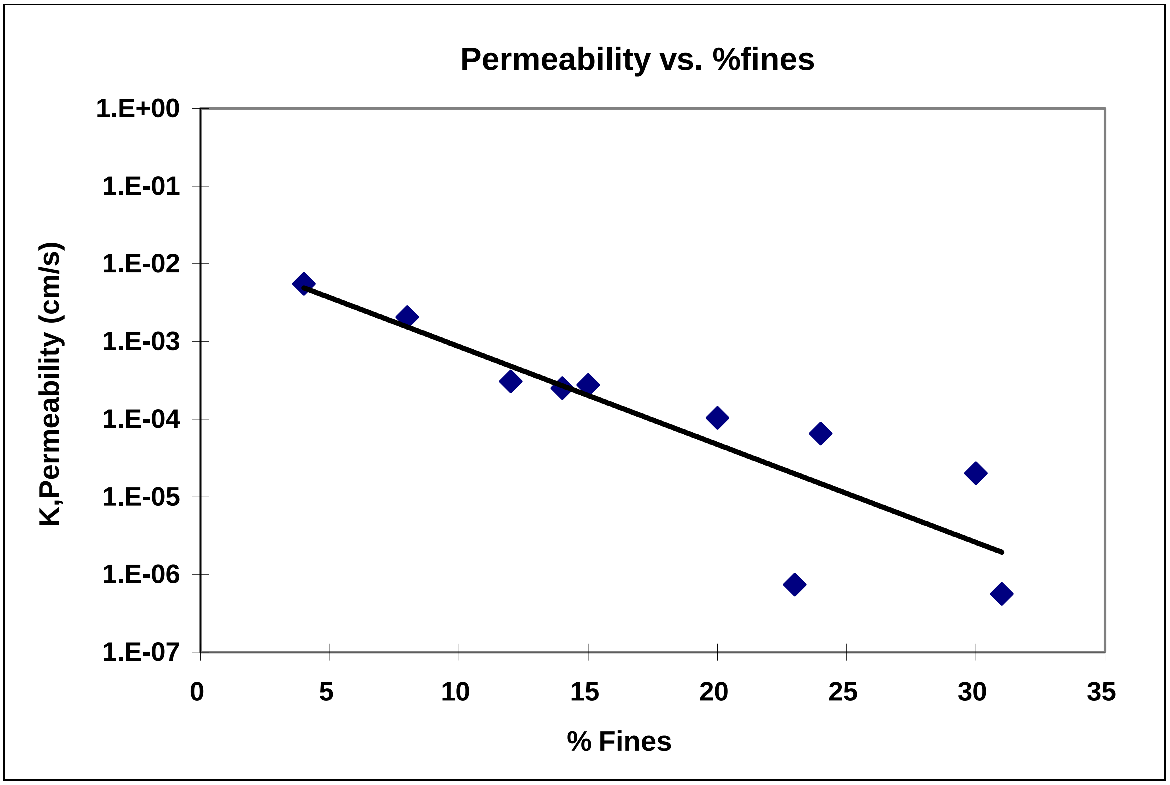


Figure 4.14 Permeability vs. Percent of Fines for Eight Soils

CHAPTER 5
PRESENTATION OF TEST-PIT TEST RESULTS

5.1 GENERAL

Eleven types of soil representing typical Florida subgrade materials were tested in the test-pit program. For each soil, static and cyclic (up to 30,000 cycles for simulation of the dynamic effect) plate load tests were conducted under different levels of groundwater table. Since the resilient behavior of subgrade soil under the dynamic loading was influenced by the soil properties as well as the moisture conditions, a detailed evaluation was made of the moisture profile for various groundwater levels. The test-pit experimental results are presented in reference to the various levels of groundwater table in the appendices.

5.2 TEST NUMBER AND LOAD CONDITIONS

A series of plate load tests were conducted at each time when the moisture equilibrium was achieved after adjusting the groundwater level. The designated test numbers and their corresponding loading conditions for each soil are listed in

Table 5.1 through Table 5.11. The relative elevation 0.0 in. is set at the interface between the subgrade and embankment.

5.3 MOISTURE PROFILE RESULTS

5.3.1 Moisture Profile in Equilibrium

To monitor the water movement in soils, the water content at each water level (6-in. intervals) was recorded on a daily basis for each material after the material was placed and compacted in place until the water content became stable. The moisture profiles in an equilibrium state under different water levels are summarized in Tables 5.12(A) through 5.22(A). To correlate the moisture profile as a result of water level adjustment with the resilient behavior of the tested subgrade soil, an accurate moisture profile was obtained of the subgrade soil under the plate load test. Tables 5.12(B) through 5.22(B) present the moisture profiles at the time of plate load test. Since the TDR probes used to measure the water content of the three additional soils were not calibrated properly using the tested materials, the moisture profile data for the additional three soils should be carefully checked and only used for a reference.

All volumetric water content measured through the TDR probe was converted into gravimetric water content according to

Equation B-1 (refer to Appendix B). The dry unit weight (γ_d) of each layer of the subgrade soil was measured when the soil was initially compacted in the test pit. Thus, the dry unit weight (γ_d) of the subgrade at a corresponding elevation for the TDR probe was approximated during the experiment. A linear interpolation was used to indicate the water content at each increment level of the subgrade within a specific test.

For the Levy County A-3 soil, the moisture profiles in an equilibrium state after the adjustment of water levels are presented in Table 5.12(A) and shown in Figure 5.1. The moisture profiles at the time of plate load test are summarized in Table 5.12(B).

For the SR70 A-3 soil, the moisture profiles in an equilibrium state after the adjustment of water levels are presented in Table 5.13(A) and shown in Figure 5.2. The moisture profiles at the time of plate load test are summarized in Table 5.13(B).

For the SR70 A-2-4 soil, the moisture profiles under different water levels are presented in Table 5.3 and shown in Figure 5.14(A). The moisture profiles at the time of plate load test are summarized in Table 5.14(B).

For the A-2-4 (12%) soil, the moisture profiles under different water levels are presented in Table 5.15(A) and shown in Figure 5.4. The moisture profiles at the time of plate load test are summarized in Table 5.15(B).

For the A-2-4 (20%) soil, the moisture profiles under different water levels are presented in Table 5.16(A) and shown in Figure 5.5. The moisture profiles at the time of plate load test are summarized in Table 5.16(B).

For the A-2-4 (24%) soil, the moisture profiles under different water levels are presented in Table 5.17(A) and shown in Figure 5.6. The moisture profiles at the time of plate load test are summarized in Table 5.17(B).

For the A-2-4 (30%) soil, the moisture profiles under different water levels are presented in Table 5.18(A) and shown in Figure 5.7. The moisture profiles at the time of plate load test are summarized in Table 5.18(B).

For the Oolite soil, the moisture profiles under different water levels are presented in Table 5.19(A) and shown in Figure 5.8. The moisture profiles at the time of plate load test are summarized in Table 5.19(B).

For the Spring Cemetery A-2-4 (15%) soil, the moisture profiles under different water levels are presented in Table 5.20(A) and shown in Figure 5.9. The moisture profiles at the time of plate load test are summarized in Table 5.20(B).

For the Branch A-2-4 (23%) soil, the moisture profiles under different water levels are presented in Table 5.21(A) and shown in Figure 5.10. The moisture profiles at the time of plate load test are summarized in Table 5.21(B).

For the Iron Bridge A-2-6 (31%) soil, the moisture profiles under different water levels are presented in Table 5.22(A) and shown in Figure 5.11. The moisture profiles at the time of plate load test are summarized in Table 5.22(B).

The moisture profiles for the eleven subgrade soils are combined together and presented in Figures 5.12 and 5.13 for the water level at -24.0 in., Figures 5.14 and 5.15 for the water level at 0.0 in., Figures 5.16 and 5.17 for the water level at +12.0 in., Figure 5.18 for the water level at +24.0 in., and Figures 5.19 and 5.20 for the water level at +36 in..

5.3.2 Moisture Profile with Time

The daily moisture variations for each level of water elevation were recorded with the elapsed time until the water level changed. These moisture-time relationships were plotted in figures that help to find the trend of the drainage and capillary rise effect. The figures are presented in Appendix E for reference.

5.4 PLATE LOAD TEST RESULTS

The plate equivalent modulus values under various loading conditions and the number of load cycles are presented in Tables 5.23, 5.24, 5.25, 5.26, 5.27, 5.28, 5.29, 5.30, 5.31, 5.32 and 5.33 for the Levy County A-3, SR70 A-3, SR70 A-2-4, A-2-4 (12%), A-2-4 (20%), A-2-4 (24%), A-2-4 (30%), Oolite, Spring Cemetery A-2-4, Branch A-2-4, and Iron Bridge A-2-6 soils, respectively. For each plate load test, two figures are grouped together to represent a specific set of plate load test results. The figure series “A” represents the equivalent modulus versus the number of load cycles, while figure series “B” represents the moisture profiles on the condition of this plate load test.

Levy County A-3 Soil

The plate load test results are presented in Figures 5.21, 5.22, 5.23 for the Levy County A-3 soil, representing the three cases of 20-psi test load without limerock base layer, 20-psi test load with limerock base layer, and 50-psi test load with limerock base layer, respectively.

SR70 A-3 Soil

The plate load test results are presented in Figures 5.24, 5.25, 5.26, and 5.27 for the SR70 A-3 soil. The data are grouped into four cases: a) 20-psi test load without limerock base layer, b) 50-psi test load with limerock base layer under different water levels, c) 50-psi plate load with limerock base layer under drained conditions, and d) 50-psi test load with limerock base layer under flooded conditions.

SR70 A-2-4 Soil

The plate load test results are presented in Figures 5.28, 5.29, 5.30, and 5.31 for the SR70 A-2-4 soil. The data are grouped into four cases: a) 20-psi test load without limerock base layer, b) 50-psi test load with limerock base layer under different water levels, c) 50-psi plate load with limerock base layer under drained conditions, and d) 50-psi test load with limerock base layer under flooded conditions.

A-2-4 (12%) Soil

The plate load test results are presented in Figures 5.32 and 5.33 for the A-2-4 (12%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

A-2-4 (20%) Soil

The plate load test results are presented in Figures 5.34 and 5.35 for the A-2-4 (20%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50 -psi test load with limerock base layer under different water levels.

A-2-4 (24%) Soil

The plate load test results are presented in Figures 5.36 and 5.37 for the A-2-4 (24%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

A-2-4 (30%) Soil

The plate load test results are presented in Figures 5.38 and 5.39 for the A-2-4 (30%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

Oolite A-1 Soil

The plate load test results are presented in Figure 5.40 for the Oolite soil. The data are only grouped into one case: 50-psi test load with limerock base layer under different water table levels.

Spring Cemetery A-2-4 (15%) soil

The plate load test results are presented in Figures 5.41 and 5.42 for the Spring Cemetery A-2-4 (15%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

Branch A-2-4 (23%) soil

The plate load test results are presented in Figures 5.43 and 5.44 for the Branch A-2-4 (23%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

Iron Bridge A-2-6 (31%) soil

The plate load test results are presented in Figures 5.45 and 5.46 for the Iron Bridge A-2-6 (31%) soil. The data are grouped into two cases: a) 20-psi test load without limerock base layer, and b) 50-psi test load with limerock base layer under different water levels.

Table 5.1 Plate Load Test Number and Corresponding Loading Conditions for Levy County A-3

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
1-1	-20	N-1	20	No	12/30/1998
1-2	0	N-2	20	No	2/5/1999
1-3	+12	N-3	20	No	2/26/1999
1-4	+12	N-4	20	Yes	3/23/1999
1-5	+12	N-5	50	Yes	3/24/1999
1-6	+36	N-6	50	Yes	3/31/1999
1-7	+36	N-7	20	Yes	4/1/1999

Table 5.2 Plate Load Test Number and Corresponding Loading Conditions for SR70 A-3

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
2-1	0	N-1	20	No	7/19/1999
2-2	+12	N-2	20	No	8/25/1999
2-3	+12	N-3	50	Yes	9/3/1999
2-4	+36	N-4	50	Yes	9/29/1999
2-5	+36	N-5	50	Yes	10/5/1999
2-6	-24	N-6	50	Yes	12/29/1999
2-7	-24	N-7	50	Yes	1/4/2000
2-8	+36	N-8	50	Yes	2/2/2000

Table 5.3 Plate Load Test Number and Corresponding Loading Conditions for SR70 A-2-4

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
3-1	0	S-1	20	No	7/20/1999
3-2	+12	S-2	20	No	8/24/1999
3-3	+12	S-3	50	Yes	9/2/1999
3-4	+36	S-4	50	Yes	9/30/1999
3-5	-24	S-5	50	Yes	12/28/1999
3-6	-24	S-6	50	Yes	1/5/2000
3-7	+36	S-6	50	Yes	2/1/2000

Table 5.4 Plate Load Test Number and Corresponding Loading Conditions for A-2-4 (12%)

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
4-1	-24	1	20	No	8/3/2000
4-2	-24	2	20	No	8/4/2000
4-3	0	3	20	No	9/19/2000
4-4	0	4	20	No	9/19/2000
4-5	+12	5	20	No	11/1/2000
4-6	+12	6	20	No	11/1/2000
4-7	+12	1 LR	50	Yes	12/14/2000
4-8	+12	2 LR	50	Yes	12/21/2000
4-9	+36	3 LR	50	Yes	2/26/2001
4-10	+36	4 LR	50	Yes	2/28/2001

Table 5.5 Plate Load Test Number and Corresponding Loading Conditions for A-2-4 (20%)

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
5-1	-24	1	20	No	8/1/2000
5-2	-24	2	20	No	8/2/2000
5-3	0	3	20	No	9/21/2000
5-4	0	4	20	No	9/22/2000
5-5	+12	5	20	No	11/6/2000
5-6	+12	6	20	No	11/8/2000
5-7	+12	1 LR	50	Yes	12/11/2000
5-8	+12	2 LR	50	Yes	12/13/2000
5-9	+36	3 LR	50	Yes	3/1/2001
5-10	+36	4 LR	50	Yes	3/5/2001

Table 5.6 Plate Load Test Number and Corresponding Loading Conditions for A-2-4 (24%)

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
6-1	-24	1	20	No	7/27/2000
6-2	-24	2	20	No	7/28/2000
6-3	0	3	20	No	9/25/2000
6-4	0	4	20	No	9/26/2000
6-5	+12	5	20	No	11/9/2000
6-6	+12	6	20	No	11/14/2000
6-7	+12	1 LR	50	Yes	12/18/2000
6-8	+12	2 LR	50	Yes	12/20/2000
6-9	+36	3 LR	50	Yes	3/6/2001
6-10	+36	4 LR	50	Yes	3/8/2001

Table 5.7 Plate Load Test Number and Corresponding Loading Conditions for A-2-4 (30%)

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
7-1	-24	1	20	No	7/5/2000
7-2	0	2	20	No	8/14/2000
7-3	0	3	20	No	8/15/2000
7-4	+12	4	20	No	10/5/2000
7-5	+12	5	20	No	10/6/2000
7-6	+12	6	20	No	10/19/2000
7-7	+12	1 LR	50	Yes	12/13/2000
7-8	+12	2 LR	50	Yes	12/14/2000
7-9	+36	3 LR	50	Yes	2/11/2001
7-10	+36	4 LR	50	Yes	2/14/2001

Table 5.8 Plate Load Test Number and Corresponding Loading Conditions for Oolite

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
8-1	-24	1	50	No	7/6/2000
8-2	0	2	50	No	8/10/2000
8-3	0	3	50	No	8/11/2000
8-4	+12	4	50	No	10/4/2000
8-5	+12	5	50	No	10/9/2000
8-6	+12	6	50	No	10/10/2000
8-7	+12	7	50	No	10/17/2000
8-8	+12	1 LR	50	Yes	12/11/2000
8-9	+12	2 LR	50	Yes	12/18/2000
8-10	+36	3 LR	50	Yes	2/7/2001
8-11	+36	4 LR	50	Yes	2/9/2001

Table 5.9 Plate Load Test Number and Corresponding Loading Conditions for Spring Cemetery A-2-4

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
9-1	0	1	20	No	7/19/2005
9-2	0	2	20	No	7/20/2005
9-3	0	3	20	No	7/21/2005
9-4	+12	4	20	No	8/12/2005
9-5	+12	5	20	No	8/15/2005
9-6	+12	6	20	No	8/16/2005
9-7	+24	7	20	No	12/20/2005
9-8	+24	8	20	No	12/21/2005
9-9	+24	9	20	No	12/22/2005
9-10	0	1 LR	50	Yes	3/15/2006
9-11	0	2 LR	50	Yes	3/16/2006
9-12	0	3 LR	50	Yes	3/17/2006
9-13	+12	4 LR	50	Yes	5/23/2006
9-14	+12	5 LR	50	Yes	5/24/2006
9-15	+12	6 LR	50	Yes	5/25/2006
9-16	+24	7 LR	50	Yes	8/4/2006
9-17	+24	8 LR	50	Yes	8/7/2006
9-18	+24	9 LR	50	Yes	8/9/2006
9-19	+36	10 LR	50	Yes	10/24/2006
9-20	+36	11 LR	50	Yes	10/25/2006
9-21	+36	12 LR	50	Yes	10/26/2006
9-22	+24	13 LR	50	Yes	1/12/2007
9-23	+24	14 LR	50	Yes	1/16/2007
9-24	+24	15 LR	50	Yes	1/17/2007
9-25	+12	16 LR	50	Yes	4/5/2007
9-26	+12	17 LR	50	Yes	4/6/2007
9-27	+12	18 LR	50	Yes	4/9/2007

Table 5.10 Plate Load Test Number and Corresponding Loading Conditions for Branch A-2-4

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
10-1	0	1	20	No	7/28/2005
10-2	0	2	20	No	7/29/2005
10-3	0	3	20	No	8/1/2005
10-4	0	4	20	No	8/3/2005
10-5	+12	5	20	No	9/13/2005
10-6	+12	6	20	No	9/14/2005
10-7	+12	7	20	No	9/15/2005
10-8	+12	8	20	No	9/29/2005
10-9	+24	9	20	No	12/6/2005
10-10	+24	10	20	No	12/7/2005
10-11	+24	11	20	No	12/8/2005
10-12	0	1 LR	50	Yes	3/6/2006
10-13	0	2 LR	50	Yes	3/7/2006
10-14	0	3 LR	50	Yes	3/8/2006
10-15	+12	4 LR	50	Yes	5/15/2006
10-16	+12	5 LR	50	Yes	5/16/2006
10-17	+12	6 LR	50	Yes	5/17/2006
10-18	+24	7 LR	50	Yes	7/27/2006
10-19	+24	8 LR	50	Yes	7/28/2006
10-20	+24	9 LR	50	Yes	7/31/2006
10-21	+24	10 LR	50	Yes	8/15/2006
10-22	+36	11 LR	50	Yes	10/16/2006
10-23	+36	12 LR	50	Yes	10/17/2006
10-24	+36	13 LR	50	Yes	10/18/2006
10-25	+24	14 LR	50	Yes	1/3/2007
10-26	+24	15 LR	50	Yes	1/4/2007
10-27	+24	16 LR	50	Yes	1/8/2007
10-28	+12	17 LR	50	Yes	3/20/2007
10-29	+12	18 LR	50	Yes	3/21/2007
10-30	+12	19 LR	50	Yes	3/30/2007

Table 5.11 Plate Load Test Number and Corresponding Loading Conditions for Iron Bridge A-2-6

Test Number	Water Table (in.)	Test Location	Plate Load (psi)	5-in Limerock Base Layer	Test Date
11-1	0	1	20	No	7/25/2005
11-2	0	2	20	No	7/26/2005
11-3	0	3	20	No	7/27/2005
11-4	0	4	20	No	8/4/2005
11-5	+12	5	20	No	9/16/2005
11-6	+12	6	20	No	9/22/2005
11-7	+12	7	20	No	9/23/2005
11-8	+24	8	20	No	12/9/2005
11-9	+24	9	20	No	12/13/2005
11-10	+24	10	20	No	12/14/2005
11-11	0	1 LR	50	Yes	3/9/2006
11-12	0	2 LR	50	Yes	3/10/2006
11-13	0	3 LR	50	Yes	3/13/2006
11-14	+12	4 LR	50	Yes	5/18/2006
11-15	+12	5 LR	50	Yes	5/19/2006
11-16	+12	6 LR	50	Yes	5/22/2006
11-17	+24	7 LR	50	Yes	8/1/2006
11-18	+24	8 LR	50	Yes	8/2/2006
11-19	+24	9 LR	50	Yes	8/3/2006
11-20	+36	10 LR	50	Yes	10/19/2006
11-21	+36	11 LR	50	Yes	10/20/2006
11-22	+36	12 LR	50	Yes	10/23/2006
11-23	+24	13 LR	50	Yes	1/9/2007
11-24	+24	14 LR	50	Yes	1/10/2007
11-25	+24	15 LR	50	Yes	1/11/2007
11-26	+12	16 LR	50	Yes	4/2/2007
11-27	+12	17 LR	50	Yes	4/3/2007
11-28	+12	18 LR	50	Yes	4/4/2007

Table 5.12(A) Moisture Profile of Levy County A-3 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-20	12.5	9.6	8.3	7.2	7	5	1/5/1999
0	15.1	14.5	10.9	8	6.8	4.5	2/5/1999
+12	16.5	15.7	15.7	14	9.6	6.2	3/24/1999
+36	16.6	16.4	16.4	14.8	14.7	15.1	4/5/1999
Drain	15.6	12.3	10	9	8.3	6.3	4/8/1999

Table 5.12(B) Moisture Profile of Levy County A-3 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)					
				3	9	15	21	27	33
1-1	-20	20	No	12.9	9.9	8.6	7.4	7.2	5.2
1-2	0	20	No	15.1	14.5	10.9	8	6.8	4.5
1-3	+12	20	No	15.8	15.2	14.9	13.2	9.1	5.5
1-4	+12	20	Yes	16.3	15.7	15.7	14	9.5	6.2
1-5	+12	50	Yes	16.3	15.7	15.7	14	9.5	6.2
1-6	+36	50	Yes	16.7	16.3	16.3	14.8	14.7	15
1-7	+36	20	Yes	16.7	16.4	16.4	14.8	14.7	15

Table 5.13(A) Moisture Profile of SR70 A-3 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	8.6	10.6	10.8	10.8	9.7	7.2	5/17/1999
-12	9.5	10.7	10.7	10.6	9.3	6.9	6/10/1999
0	17.2	13.4	11.2	10.6	8.9	6.7	7/22/1999
+12	16.7	20.4	19.3	13.8	10.7	8	9/3/1999
+36	16.8	19.9	20.4	21.4	19.6	17.1	10/8/1999
Drained	8.6	10.5	10.6	10	8.3	6.1	1/6/2000

Table 5.13(B) Moisture Profile of SR70 A-3 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)					
				3	9	15	21	27	33
2-1	0	20	No	17.2	13.4	11.2	10.6	8.9	6.7
2-2	+12	20	No	16.7	20.6	18.8	13.4	9.7	6.8
2-3	+12	50	Yes	16.7	20.4	19.3	13.8	10.7	8
2-4	+36	50	Yes	16.9	20.2	20.6	21.6	19.8	17.1
2-5	+36	50	Yes	16.9	20	20.5	21.5	19.7	17.2
2-6	-24	50	Yes	8.6	10.6	10.7	10.1	8.4	6.2
2-7	-24	50	Yes	8.6	10.5	10.6	10	8.3	6.1
2-8	+36	50	Yes	15.4	17.9	15.6	16.1	15.4	14.5

Table 5.14(A) Moisture Profile of SR70 A-2-4 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	12.1	11.1	8.9	8.8	9	7.4	5/17/1999
-12	12.4	11.4	9.1	9	9.2	7.6	6/10/1999
0	14.5	12.2	9.2	9.1	9.3	8.3	7/22/1999
+12	14.6	18.3	10.8	9.7	9.6	11.5	9/3/1999
+36	14.6	18	11.1	15.9	19.3	32.7	9/29/1999
Drain	14	14.6	9.8	11.2	13.3	13.7	1/6/2000
Drain	14.4	17.7	11.1	15.7	17.7	30.4	10/14/1999

Table 5.14(B) Moisture Profile of SR70 A-2-4 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)					
				3	9	15	21	27	33
3-1	0	20	No	14.4	12.1	9.2	9.1	9.4	8.1
3-2	+12	20	No	14.6	18.3	10.5	9.3	9.3	8.1
3-3	+12	50	Yes	14.6	18.3	10.8	9.7	9.6	11.4
3-4	+36	50	Yes	14.6	18	11.1	16	19.5	33.2
3-5	-24	50	Yes	13.9	14.6	9.8	11.1	13.4	14.6
3-6	-24	50	Yes	14	14.6	9.9	11.2	13.4	13.9
3-7	+36	50	Yes	14	15.8	10	12.8	16.4	22.7

Table 5.15(A) Moisture Profile of A-2-4 (12%) Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	5.09	5.06	4.6	4.1	2.5	2.99	8/3/2000
0	10.38	12.05	11.53	7.69	3.88	4.6	9/25/2000
+12	11.15	12.95	13.28	11.17	6.46	7.72	12/20/2000
+36	11.36	13.26	13.6	11.55	7.16	10.99	12/29/2000
+41	11.41	13.33	13.73	11.74	7.3	11.25	1/5/2001

Table 5.15(B) Moisture Profile of A-2-4 (12%) Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
				3	9	15	21	27	33	
4-1	-24	20	No	5.09	5.06	4.60	4.10	2.50	2.99	8/3/2000
4-2	-24	20	No	5.08	5.06	4.60	4.12	2.50	2.99	8/4/2000
4-3, 4-4	0	20	No	10.38	12.02	11.73	7.9	3.97	4.75	9/19/2000
4-5,4-6	+12	20	No	10.77	12.68	12.92	11.47	5.6	6.39	11/1/2000
4-7	+12	50	Yes	11.11	12.93	13.19	11.06	6.44	7.69	12/14/2000
4-8	+12	50	Yes	11.15	12.95	13.27	11.19	6.44	7.68	12/21/2000
4-9	+36	50	Yes	11.3	13.16	13.49	11.79	7.45	11.05	2/26/2001
4-10	+36	50	Yes	11.26	13.12	13.44	11.83	7.46	11.22	2/28/2001

Table 5.16(A) Moisture Profile of A-2-4 (20%) Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	9.51	9.20	9.31	10.40	4.21	3.39	8/3/2000
0	9.55	9.32	9.39	10.36	3.93	3.26	9/25/2000
+12	9.58	9.43	9.58	10.52	4.08	3.48	12/20/2000
+36	9.64	9.66	9.87	11.09	9.30	8.05	12/29/2000
+41	9.67	9.70	9.92	11.12	9.38	8.17	1/5/2001

Table 5.16(B) Moisture Profile of A-2-4 (20%) Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
				3	9	15	21	27	33	
5-1	-24	20	No	9.48	9.17	9.32	10.44	4.39	4.21	8/1/2000
5-2	-24	20	No	9.49	9.18	9.33	10.44	4.39	4.03	8/2/2000
5-3	0	20	No	9.55	9.32	9.38	10.37	3.97	3.42	9/21/2000
5-4	0	20	No	9.55	9.32	9.38	10.36	3.96	3.33	9/22/2000
5-5	+12	20	No	9.58	9.45	9.69	10.69	4.64	3.48	11/6/2000
5-6	+12	20	No	9.58	9.45	9.7	10.7	4.69	3.28	11/8/2000
5-7	+12	50	Yes	9.6	9.51	9.78	10.85	5.65	3.73	12/11/2000
5-8	+12	50	Yes	9.6	9.52	9.77	10.85	5.68	3.76	12/13/2000
5-9	+36	50	Yes	9.62	9.66	9.86	11.15	9.37	7.98	3/1/2001
5-10	+36	50	Yes	9.61	9.64	9.85	11.14	9.37	8.02	3/5/2001

Table 5.17(A) Moisture Profile of A-2-4 (24%) Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	7.04	6.84	6.67	7.12	5.49	5.23	8/3/2000
0	7.21	8.54	10.18	8.45	6.05	5.36	9/25/2000
+12	7.39	8.62	11.38	12.07	7.02	6.06	12/20/2000
+36	7.56	8.80	11.51	12.24	9.54	8.97	12/29/2000
+41	7.58	8.85	11.60	12.37	9.83	13.39	1/5/2001

Table 5.17(B) Moisture Profile of A-2-4 (24%) Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
				3	9	15	21	27	33	
6-1	-24	20	No	7.05	6.84	6.71	7.22	5.56	5.37	7/27/2000
6-2	-24	20	No	6.99	6.80	6.62	7.21	5.53	5.16	7/28/2000
6-3	0	20	No	7.21	8.54	10.18	8.45	6.05	5.36	9/25/2000
6-4	0	20	No	7.22	8.55	10.2	8.43	6.00	5.25	9/26/2000
6-5	+12	20	No	7.29	8.56	11.34	12.07	7.00	5.98	11/9/2000
6-6	+12	20	No	7.33	8.6	11.39	12.17	6.95	5.66	11/14/2000
6-7	+12	50	Yes	7.36	8.6	11.33	12.05	7.04	6.05	12/18/2000
6-8	+12	50	Yes	7.39	8.62	11.38	12.07	7.02	6.06	12/20/2000
6-9	+36	50	Yes	7.55	8.73	11.36	12.06	9.67	13.07	3/6/2001
6-10	+36	50	Yes	7.58	8.76	11.44	12.1	9.73	13.05	3/8/2001

Table 5.18(A) Moisture Profile of A-2-4 (30%) Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	11.93	12.17	11.73	12.48	10.82	7.87	7/6/2000
0	16.01	13.90	12.57	13.02	10.90	8.04	8/14/2000
+12	15.55	16.80	16.16	14.04	11.24	8.06	12/20/2000
+36	15.66	16.97	16.43	14.92	11.46	8.55	1/19/2001

Table 5.18(B) Moisture Profile of A-2-4 (30%) Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
				3	9	15	21	27	33	
7-1	-24	20	No	11.96	11.99	11.60	12.51	10.95	8.05	7/5/2000
7-2	0	20	No	16.01	13.90	12.57	13.02	10.90	8.04	8/14/2000
7-4	+12	20	No	15.75	16.76	16.3	13.83	11.23	8.04	10/5/2000
7-5	+12	20	No	15.76	16.77	16.29	13.82	11.22	8.02	10/6/2000
7-6	+12	20	No	15.68	16.82	16.24	13.86	11.2	7.92	10/19/2000
7-7	+12	50	Yes	15.53	16.75	16.19	14.01	11.28	8.1	12/13/2000
7-8	+12	50	Yes	15.54	16.76	16.19	13.99	11.29	8.09	12/14/2000
7-9	+36	50	Yes	15.66	16.95	16.43	14.92	11.47	8.64	2/11/2001
7-10	+36	50	Yes	15.64	16.92	16.39	14.89	11.45	8.64	2/14/2001

Table 5.19(A) Moisture Profile of Oolite Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
	3	9	15	21	27	33	
-24	2.94	2.53	2.49	5.96	4.76	4.02	7/6/2000
0	3.18	3.05	2.48	5.88	4.52	3.70	8/14/2000
+12	3.17	3.24	3.53	6.05	4.36	3.50	12/20/2000
+36	3.18	3.25	3.72	6.42	4.51	4.26	1/19/2001

Table 5.19(B) Moisture Profile of Oolite Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	Limerock	Moisture Content (%) @ Each Elevation (in. above Embankment)						Date Recorded
				3	9	15	21	27	33	
8-1	-24	50	No	2.98	2.53	2.50	6.04	4.76	4.02	7/6/2000
8-2	0	50	No	3.18	3.06	2.54	5.99	4.61	3.94	8/10/2000
8-3	0	50	No	3.18	3.06	2.51	5.94	4.55	3.82	8/11/2000
8-4	+12	50	No	3.17	3.23	3.70	6.04	4.46	3.56	10/4/2000
8-5	+12	50	No	3.18	3.23	3.73	6.01	4.46	3.53	10/9/2000
8-6	+12	50	No	3.18	3.23	3.73	6.02	4.46	3.49	10/10/2000
8-7	+12	50	No	3.18	3.23	3.75	5.99	4.43	3.47	10/17/2000
8-8	+12	50	Yes	3.18	3.24	2.82	5.92	4.34	3.51	12/11/2000
8-9	+12	50	Yes	3.16	3.22	3.52	6.02	4.34	3.52	12/18/2000
8-10	+36	50	Yes	3.18	3.25	3.79	6.55	4.57	4.38	2/7/2001
8-11	+36	50	Yes	3.18	3.25	3.80	6.55	4.56	4.38	2/9/2001

Table 5.20(A) Moisture Profile of Spring Cemetery A-2-4 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Date Recorded
	3	9	15	21	27	33	Base	
Drained	5.5	5.8	7.3	5.7	4.1			7/18/2005
0	13	11.4	8.5	5	4			8/4/2005
+12	13.5	12.8	12	8.7	4.7			9/29/2005
+24	14	13.4	13	13.8	11.9	6.4*		1/3/2006
0	13	11.2	9.1	6.9	6.8	4.6	7.7	3/17/2006
+12	13.4	12.6	11.4	11.6	8.7	4.9	8.2	5/30/2006
+24	13.1	12.9	12	11.6	12.9	12.1	12.5	8/17/2006
+36	13.7	13.2	12.5	12.5	13.1	12.9	18.4	10/27/2006
+24	13.5	12.9	12.7	13	13.5	12.9	13.6	1/17/2007
+12	13.4	13.8	13.2	11.7	9.9	5.9	10.5	4/9/2007

* Moisture content was measured using backscatter moistures with the Nuclear Density Gauge

Table 5.20(B) Moisture Profile of Spring Cemetery A-2-4 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	5-in Base Layer (Lime-rock)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Test Date
				3	9	15	21	27	33	Base	
9-1	0	20	No	12.2	9.2	7	5.1	4			7/19/2005
9-2	0	20	No	12.2	10.2	7.5	5.1	4.3			7/20/2005
9-3	0	20	No	12.5	9.2	7.8	4.8	4.3			7/21/2005
9-4	+12	20	No	13.1	12.5	11.2	8	4.6			8/12/2005
9-5	+12	20	No	13.3	12.3	11.8	8.5	4.9			8/15/2005
9-6	+12	20	No	13.3	12.5	11.7	7.9	5.1			8/16/2005
9-7	+24	20	No	14.1	13.5	12.8	13.9	10.5	7.4*		12/20/2005
9-8	+24	20	No	14.3	13.5	12.7	13.6	10.4	7.4*		12/21/2005
9-9	+24	20	No	14.3	13.6	13.1	14.2	10.3	6.8*		12/22/2005
9-10	0	50	Yes	13	10.1	8.8	6.6	6.3	4.6	8.2	3/15/2006
9-11	0	50	Yes	13.1	10.8	9.1	6.8	6.4	4.3	7.7	3/16/2006
9-12	0	50	Yes	13	11.2	9.1	6.9	6.8	4.6	7.7	3/17/2006
9-13	+12	50	Yes	13.4	12.8	11.5	11.7	8.7	4.9	8.2	5/23/2006
9-14	+12	50	Yes	13.5	12.6	11.4	11.6	8.7	4.7	8.2	5/24/2006
9-15	+12	50	Yes	13.4	12.6	11.7	11.7	8.9	4.7	8.3	5/25/2006
9-16	+24	50	Yes	13.4	12.9	12	11.7	12.6	10.7	12.1	8/4/2006
9-17	+24	50	Yes	13.1	12.6	11.8	11.6	12.6	10	11.6	8/7/2006
9-18	+24	50	Yes	12.8	12.5	11.8	11.6	12.4	10.2	11.6	8/9/2006
9-19	+36	50	Yes	13.5	13.5	12.5	12.5	13	12.7	12.8	10/24/2006
9-20	+36	50	Yes	13.4	13.1	12.3	12.5	13.1	13.1	17.5	10/25/2006
9-21	+36	50	Yes	14	13.2	14.4	13	13.8	13.2	17.9	10/26/2006
9-22	+24	50	Yes	13.5	13.4	12.3	12.5	13.5	12.9	13.6	1/12/2007
9-23	+24	50	Yes	13.7	13.2	12.5	12.4	13.5	12.9	13.6	1/16/2007
9-24	+24	50	Yes	13.5	12.9	12.7	13	13.5	12.9	13.6	1/17/2007
9-25	+12	50	Yes	13.4	14	13.2	12.2	99	6.0	10.3	4/5/2007
9-26	+12	50	Yes								4/6/2007
9-27	+12	50	Yes	13.4	13.8	13.2	11.7	9.9	5.9	10.5	4/9/2007

* Moisture content was measured using backscatter moistures with the Nuclear Density Gauge

Table 5.21(A) Moisture Profile of Branch A-2-4 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Date Recorded
	3	9	15	21	27	33	Base	
Drained	9.4	7.6	6.9	6.8	5.2			7/18/2005
0	10.2	9.4	8.7	6.8	5.2			8/4/2005
+12	10.5	10.9	11.1	9.3	10.1			9/29/2005
+24	10.5	10.2	10.9	9.6	12.4	3.4*		1/3/2006
0	10.2	10.5	10.9	9.0	8.8	5.8	4.5	3/17/2006
+12	10.3	10.5	10.5	10.0	12.0	6.3	4.7	5/30/2006
+24	10.5	10.9	10.3	9.3	12.9	8.1	6.2	8/17/2006
+36	10.6	11.0	10.9	9.6	13.4	10.2	10.0	10/27/2006
+24	10.5	10.8	10.5	10.0	13.2	9.2	8.2	1/17/2007
+12	10.5	11.2	11.7	9.8	12.6	8.3	5.1	4/9/2007

Table 5.21(B) Moisture Profile of Branch A-2-4 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Test Load (psi)	5-in Base Layer (Lime-rock)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Test Date
				3	9	15	21	27	33	Base	
0	10-1	20	No	10.2	9.3	9.1	6.8	5.0			7/28/2005
0	10-2	20	No	10.2	9.1	8.7	6.9	5.2			7/29/2005
0	10-3	20	No	10.3	9.4	8.5	6.8	5.2			8/1/2005
0	10-4	20	No	10.2	9.3	8.7	6.9	5.1			8/3/2005
+12	10-5	20	No	10.2	10.9	10.6	9.0	9.6			9/13/2005
+12	10-6	20	No	10.2	10.9	10.8	9.2	9.6			9/14/2005
+12	10-7	20	No	10.2	10.9	10.9	9.2	9.8			9/15/2005
+12	10-8	20	No	10.5	10.9	11.0	9.3	10.1			9/29/2005
+24	10-9	20	No	10.3	10.2	10.3	9.7	12.9	3.9*		12/6/2005
+24	10-10	20	No	10.5	10.1	10.5	9.7	12.7	3.3*		12/7/2005
+24	10-11	20	No	10.2	10.2	10.3	9.7	12.7	3.7*		12/8/2005
0	10-12	50	Yes	10.1	9.9	10.3	9.2	9.2	5.7	4.7	3/6/2006
0	10-13	50	Yes	10.1	10.1	10.5	9.3	9.1	5.6	4.5	3/7/2006
0	10-14	50	Yes	9.9	10.1	10.2	9.4	9.1	6.1	4.5	3/8/2006
+12	10-15	50	Yes	10.3	10.5	10.9	9.8	12.3	6.0	4.7	5/15/2006
+12	10-16	50	Yes	10.2	10.8	11.0	9.6	12.1	6.4	4.8	5/16/2006
+12	10-17	50	Yes	10.3	10.9	10.8	9.7	12.1	6.3	4.7	5/17/2006
+24	10-18	50	Yes	10.3	10.9	10.8	9.7	12.9	8.0	5.9	7/27/2006
+24	10-19	50	Yes	10.5	10.5	10.8	9.6	13.4	8.6	6.2	7/28/2006
+24	10-20	50	Yes	10.1	10.8	10.8	9.4	12.7	8.3	6.2	7/31/2006
+24	10-21	50	Yes	9.3	10.9	10.9	9.4	12.6	8.4	6.7	8/15/2006
+36	10-22	50	Yes	10.3	10.8	11.1	9.8	13.2	9.6	9.8	10/16/2006
+36	10-23	50	Yes	10.2	11	10.9	9.8	13.4	9.7	9.8	10/17/2006
+36	10-24	50	Yes	10.5	10.6	10.5	10.0	13.2	9.6	10.1	10/18/2006
+24	10-25	50	Yes	10.5	11.4	12.2	9.8	12.8	9.0	8.7	1/3/2007
+24	10-26	50	Yes	10.5	11.0	10.8	9.8	13.2	9.3	8.7	1/4/2007
+24	10-27	50	Yes	10.3	11.0	10.8	9.8	13.2	9.7	8.9	1/8/2007
+12	10-28	50	Yes	9.6	11.3	11.5	9.4	13.1	8	5.1	3/20/2007
+12	10-29	50	Yes	10.3	11.2	10.8	9.3	13.5	7.9	5.3	3/21/2007
+12	10-30	50	Yes	9.7	11.3	12.0	10.4	12.0	8.2	5.1	3/30/2007

* Moisture content was measured using backscatter moistures with the Nuclear Density Gauge

Table 5.22(A) Moisture Profile of Iron Bridge A-2-6 Soil

Water Table (in.)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Date Recorded
	3	9	15	21	27	33	Base	
Drained	5.1	7.3	6.2	7.1	5.7			7/18/2005
0	10.5	10.4	9.2	7.2	6			8/4/2005
+12	12.4	10.8	12.3	10.7	10.7			9/29/2005
+24	12	11.1	12.9	11	12.4	8.7*		1/3/2006
0	11.6	11	11.4	11	9.3	8.5	6.2	3/17/2006
+12	11.9	10.8	12.5	11	11	9.3	6.9	5/30/2006
+24	12	11	12.5	11.2	11.7	10.4	7.7	8/17/2006
+36	12.4	11.4	12.5	11.6	13.1	13.4	12.8	10/27/2006
+24	12.3	11.4	12.3	11.6	11.9	11.3	8.8	1/17/2007
+12	11.5	11.0	12.5	11.8	11.6	9.8	6.4	4/9/2007

* Moisture content was measured using backscatter moistures with the Nuclear Density Gauge

Table 5.22(B) Moisture Profile of Iron Bridge A-2-6 Soil (During Plate Load Test)

Test No.	Water Table (in.)	Plate Load (psi)	5-in Base Layer (Lime-rock)	Moisture Content (%) @ Each Elevation (in. above Embankment)							Test Date
				3	9	15	21	27	33	Base	
0	11-1	20	No	8.8	9.2	5.4	7.4	5.9			7/25/2005
0	11-2	20	No	9.4	9.9	5.9	7.4	5.9			7/26/2005
0	11-3	20	No	9.7	9.6	5.7	7.4	6			7/27/2005
0	11-4	20	No	10.5	10.4	9.2	7.2	6			8/4/2005
+12	11-5	20	No	12.8	11	12.5	10.5	10.6			9/16/2005
+12	11-6	20	No	12.3	11.3	12.5	10.9	10.6			9/22/2005
+12	11-7	20	No	12.6	11	12.5	10.7	10.8			9/23/2005
+24	11-8	20	No	11.9	11	12.5	11	12.4	8.7*		12/9/2005
+24	11-9	20	No	11.9	11.3	12.9	10.7	12.1	7.9*		12/13/2005
+24	11-10	20	No	11.9	11.4	12.9	10.7	12.2	8.8*		12/14/2005
0	11-11	50	Yes	11.9	11.3	11.7	11	9.5	8.7	6.2	3/9/2006
0	11-12	50	Yes	11.3	11.4	11.4	10.9	8.9	8.5	6.2	3/10/2006
0	11-13	50	Yes	10.9	10.7	11.3	10.9	9.7	8.5	6.2	3/13/2006
+12	11-14	50	Yes	12.2	11	12.3	11	11	9.3	6.9	5/18/2006
+12	11-15	50	Yes	12	11	12.5	11.2	11	9.4	6.9	5/19/2006
+12	11-16	50	Yes	12	10.8	12.5	11	11	9.1	6.9	5/22/2006
+24	11-17	50	Yes								8/1/2006
+24	11-18	50	Yes	12.2	11.3	12.6	11.3	11.6	10.5	7.8	8/2/2006
+24	11-19	50	Yes	11.9	11.1	12.6	11.3	11.7	10.1	8	8/3/2006
+36	11-20	50	Yes	11.9	11.3	12.9	11.3	13.6	13.7	12.5	10/19/2006
+36	11-21	50	Yes	12.3	11.4	12.5	11.2	13.3	13.5	13.2	10/20/2006
+36	11-22	50	Yes	12	11.6	12.9	11.2	12.2	13.3	12.1	10/23/2006
+24	11-23	50	Yes	12	11.5	12.8	11.5	11.8	11.4	8.9	1/9/2007
+24	11-24	50	Yes	12.2	11.3	12.8	11.3	12.1	11.4	8.8	1/10/2007
+24	11-25	50	Yes	12.3	11.5	12.5	11.8	11.9	11.1	9.3	1/11/2007
+12	11-26	50	Yes	11.8	11.1	12.5	11.3	11.4	10.3	6.6	4/2/2007
+12	11-27	50	Yes								4/3/2007
+12	11-28	50	Yes	11.7	11.3	12.8	11.3	11.6	10.3	6.6	4/4/2007

* Moisture content was measured using backscatter moistures with the Nuclear Density Gauge

Table 5.23 Equivalent Modulus of Levy County A-3 Soil

EQ Modulus (MPa): 1.38 pa/(Resilient Deformation)								
Test No.		1-1	1-2	1-3	1-4	1-5	1-6	1-7
Load (psi)		20	20	20	20	50	50	20
Limerock		NO	NO	NO	YES	YES	YES	YES
Test Date		12/30/1998	2/5/1999	2/26/1999	3/23/1999	3/24/1999	3/31/1999	4/1/1999
Water Table (in.)		-20	0	12	12	12	36	36
No. of Plate Load Cycles	1	131	158	118	165	185	201	128
	4	138	192	131		258	204	183
	5	145	197	130		237	207	169
	10	153	237	130		230	216	154
	25	159	229	133		238	213	151
	50	163	223	136		234	203	152
	100	165	158	135		226	205	143
	200	165	145	135	207	240	194	142
	500	165		133	200	243	198	147
	1000	168	142	130	203	238	179	149
	2000	170	133	128	200	235	185	150
	5000	167		129	200	241	177	153
	10000	173	139		222	251	185	160
	15000	177			229	267	197	166
	20000	179			224	257	198	173
	25000	179	150		229	265	199	176
	30000	182		132	227	280	203	176
Average from 10,000 Cycles		178	145	132	226	264	196	170

Table 5.24 Equivalent Modulus of SR70 A-3 Soil

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)

Test No.		2-1	2-2	2-3	2-4	2-5	2-6	2-7	2-8
Loads		20 psi	20 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Test Date		7/19/99	8/25/99	9/3/99	9/29/99	10/5/99	12/29/99	1/4/00	2/2/00
Limerock		NO	NO	YES	YES	YES	YES	YES	YES
Water Table (in.)		0	12	12	36	36	-24	-24	36
No. of Plate Load Cycles	1	148	88	208	240	159	317	419	193
	4	131	127	267	301	350	877	564	240
	5	127	121	265	293	332	801	499	224
	10	136	128	259	286	299	782	493	223
	25	150	146	262	292	286	813	499	230
	50	159	168	261	283	283	817	443	224
	100	165	188	257	282	207	821	485	217
	200	158	177	266	274	211	609	485	219
	500	164	182	267	267	211	576	472	217
	1000	163	176	264	253	218	566	464	212
	2000	170	167	265	243	219	544	450	208
	5000	174	174	268	247	213	516	431	209
	10000	187	170	281	236	214	510	418	209
	15000	207	171	292	231	228	515	410	206
	20000	217	175	300	229	213	496	421	213
	25000	205	178	310	227	220	495	399	204
	30000		179	314	225	223	479	393	208
Average from 10,000 Cycles		204	174	300	230	220	499	408	208

Table 5.25 Equivalent Modulus of SR70 A-2-4 Soil

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)								
Test No.		3-1	3-2	3-3	3-4	3-5	3-6	3-7
Loads		20 psi	20 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Test Date		7/20/99	8/24/99	9/2/99	9/30/99	12/28/99	1/5/00	2/1/00
Limerock		NO	NO	YES	YES	YES	YES	YES
Water Table (in.)		0	12	12	36	-24	-24	36
No. of Plate Load Cycles	1	91	83	183	55	149	176	65
	4	178	150	258	0	304	393	79
	5	182	157	274	0	304	351	82
	10	185	165	255	0	296	351	90
	25	181	164	255	0	316	368	95
	50	185	160	247	0	350	356	100
	100	178	160	251	0	326	360	112
	200	175	156	252	92	215	355	111
	500	177	160	246	91	218	357	105
	1000	177	161	233	91	227	474	97
	2000	177	159	232	90	226	411	87
	5000	175	153	226	94	230	421	85
	10000	173	152	218	100	231	386	61
	15000	183	151	218	108	233	396	53
	20000	188	155	226	111	235	401	68
	25000	188	156	238	105	233	367	0
	30000	182	153	233	105	233	367	0
Average from 10,000 Cycles		183	154	227	106	233	383	61

Table 5.26 Equivalent Modulus of A-2-4 (12%)

EQ Modulus (MPa) : 1.38 pa/(Resilient Deformation)											
Loads		Plate Load 20 psi						Plate Load 50 psi			
Test No.		4-1	4-2	4-3	4-4	4-5	4-6	4-7	4-8	4-9	4-10
Limerock		NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Test Date		8/3/00	8/4/00	9/19/00	9/19/00	11/1/00	11/1/00	12/14/00	12/21/00	2/26/01	2/28/01
Water Table (in.)		-24.0	-24.0	0.0	0	12	12	12	12	36	36
No.of Plate Load Cycles	1	135	106	120	82	98	104	157	166	135	142
	4			145	145	133	136	215	202	167	181
	5				0	129	135	220	197	159	179
	10			134	134	126	133	176	207	161	175
	25	175	134	139	115	131	137	180	201	158	169
	50	172	135	139	117	128	128	183	205	162	164
	100	172	137	137	117	129	127	201	208	161	171
	200	173	140	136	120	130	125	203	209	165	173
	500	168	148	132	123	141	133	209	215	169	177
	1000	169	154	127	118	149	128	213	217	164	178
	2000	167	153	123	122	143	124	220	221	177	179
	5000	163	165	122	122	134	123	227	224	168	181
	10000	181	167	123	124	129	124	233	222	173	182
	15000	182	167	122	126	131	124	255	228	177	176
	20000	175	172	122	130	120	126	259	232	170	175
	25000	180	169	123	130	128	127	261	226	173	172
	30000	173	175	124	131	127	126	262	235	173	170
Average from 10,000 Cycles		178	170	123	128	127	125	254	229	173	175

Table 5.27 Equivalent Modulus of A-2-4 (20%)

EQ Modulus (MPa) : 1.38 pa/(Resilient Deformation)											
Loads					Plate Load 20 psi			Plate Load 50 psi			
Test No.		5-1	5-2	5-3	5-4	5-5	5-6	5-7	5-8	5-9	5-10
Limerock		NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Test Date		8/1/2000	8/2/2000	9/21/2000	9/22/2000	11/6/2000	11/8/2000	12/11/2000	12/13/2000	3/1/2001	3/5/2001
Water Table (in.)		-24	-24	0	0	12	12	12	12	36	36
No.of Plate Load Cycles	1	218	133	149	141	170	143	157	190	138	135
	4	231	163	195	206	245	211	203	270	195	182
	5	231	187	181	213	208	0	193	227	194	180
	10	231	173	186	208	217	200	199	245	192	178
	25	224	183	184	197	219	196	175	240	199	179
	50	225	190	183	198	212	211	188	235	188	181
	100	187	176	178	184	197	181	185	238	185	195
	200	199	190	179	189	189	179	205	245	186	200
	500	200	155	178	184	187	162	204	253	190	210
	1000	200	159	176	185	184	154	209	251	215	217
	2000	203	162	175	185	186	154	203	255	209	226
	5000	200	166	172	182	172	158	204	265	181	227
	10000	205	170	175	189	178	164	217	269	168	239
	15000	205	173	175	194	180	162	217	277	171	248
	20000	222	180	178	196	183	173	224	286	173	240
	25000	222	177	182	177	180	173	225	275	172	255
	30000	222	179	180	179	184	171	228	280	168	251
Average from 10,000 Cycles		215	176	178	187	181	169	222	277	170	246

Table 5.28 Equivalent Modulus of A-2-4 (24%)

EQ Modulus (MPa) : 1.38 pa / (Resilient Deformation)											
Loads				Plate Load 20 psi				Plate Load 50 psi			
Test No.		6-1	6-2	6-3	6-4	6-5	6-6	6-7	6-8	6-9	6-10
Limerock		NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Test Date		7/27/2000	7/28/2000	9/25/2000	9/26/2000	11/9/2000	11/14/2000	12/18/2000	12/20/2000	3/6/2001	3/8/2001
Water Table (in.)		-24	-24	0	0	12	12	12	12	36	36
No.of Plate Load Cycles	1	114	280	146	313	107	97	151	156	124	113
	4		201	206	592	151	133	212	225	172	151
	5		206	213	364	140	133	202	224	155	145
	10		201	208	338	140	133	196	212	153	146
	25		165	197	333	148	132	189	200	150	142
	50	146	165	198	330	136	130	191	197	148	139
	100	146	165	184	300	137	129	193	197	186	130
	200	146	166	189	287	139	128	203	199	173	129
	500	146	154	184	248	137	126	205	205	188	130
	1000	146	160	185	217	134	126	213	205	197	134
	2000	151	165	185	223	139	126	216	211	202	130
	5000	166	162	182	214	137	127	208	208	191	176
	10000	173	164	170	201	141	128	211	206	182	171
	15000	165	166	173	196	141	129	216	211	184	168
	20000	171	169	180	201	143	130	218	208	178	165
	25000	176	167	177	199	139	131	224	210	170	162
	30000	185	167	179	192	139	130	219	213	165	143
Average from 10,000 Cycles		174	167	176	198	141	130	218	210	176	162

Table 5.29 Equivalent Modulus of A-2-4 (30%)

EQ Modulus (MPa) : 1.38 pa/(Resilient Deformation)											
Loads		Plate Load 20 psi						Plate Load 50 psi			
Test No.		7-1	7-2	7-3	7-4	7-5	7-6	7-7	7-8	7-9	7-10
Limerock		NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Test Date		7/5/2000	8/14/2000	8/15/2000	10/5/2000	10/6/2000	10/19/2000	12/13/2000	12/14/2000	2/11/2001	2/14/2001
Water Table (in.)		-24	0	0	12	12	12	12	12	36	36
No.of Plate Load Cycles	1	137	136	143	156	126	250	140	202	97	91
	4	195	244		174	200	149	176	254	139	132
	5	190	224		171	192	140	177	256	138	128
	10	180	200	156	165	199	140	187	257	127	122
	25	179	189	148	177	187	135	173	240	120	118
	50	177	179	147	174	165	134	173	230	114	110
	100	175	174	144	200	164	126	177	242	112	108
	200	170	173	148	197	152	125	178	232	110	107
	500	172	174	149	175	157	132	188	238	108	106
	1000	173	172	151	176	171	129	196	241	106	105
	2000	180	176	156	172	156	124	204	246	102	105
	5000	186	183	160	177	161	124	210	256	100	105
	10000	198	187	162	188	168	129	216	261	99	104
	15000	201	196	170	186	187	135	224	269	97	104
	20000	214	194	176	182	183	131	243	283	93	106
	25000	214	202	183	186	182	139	252	292	91	106
	30000	217	203	181	186	182	129	256	302	87	102
Average from 10,000 Cycles		209	196	175	186	180	133	238	281	93	104

Table 5.30 Equivalent Modulus of Miami Oolite A-1

EQ Modulus (MPa) : 1.38 pa/(Resilient Deformation)												
Loads		Plate Load 50 psi										
Test No.		8-1	8-2	8-3	8-4	8-5	8-6	8-7	8-8	8-9	8-10	8-11
Limerock		NO	NO	NO	NO	NO	NO	NO	YES	YES	YES	YES
Test Date		7/6/00	8/10/00	8/11/00	10/4/00	10/9/00	10/10/00	10/17/00	12/11/00	12/18/00	2/7/01	2/9/01
Water table (in.)		-24	0	0	12	12	12	12	12	12	36	36
No.of Plate Load Cycles	1	398	310	254	145799	378	532	544	289	404	183	188
	4	479	358		27615	609	568	699	369	467	177	230
	5	423	345		24940	565	556	660	331	449	175	229
	10	388	337	299	20711	490	568	637	335	456	183	213
	25	366	312	276	23670	471	545	618	317	457	181	192
	50	435	298	273	27615	450	549	616	335	521	188	193
	100	425	319	278	15062	455	538	579	356	511	186	179
	200	419	279	281	18410	449	531	603	343	586	203	176
	500	318	291	288	11835	451	528	577	366	685	180	180
	1000	336	284	301	33137	471	518	575	367	696	179	187
	2000	375	310	307	18410	490	515	590	349	645	294	196
	5000	376	346	327	41424	534	515	569	393	603	304	200
	10000	382	378	346	20711	573	528	547	406	628	289	197
	15000	402	410	363	16569	652	560	558	390	605	290	201
	20000	431	411	367	33138	654	564	542	388	614	299	200
	25000	438	415	377	42121	641	568	528	398	650	298	195
	30000	467	421	378	33583	642	568	536	394	652	301	191
Average from 10,000 Cycles		424	407	366	29224	633	557	542	395	630	295	197

Table 5.31(A) Equivalent Modulus of Spring Cemetery A-2-4 Soil

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)										
Test No.		9-1	9-2	9-3	9-4	9-5	9-6	9-7	9-8	9-9
Plate Load		20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi
Limerock		No	No	No	No	No	No	No	No	No
Water Table (in.)		0	0	0	12	12	12	24	24	24
No. of Plate Load Cycle	1	92	100	104	93	99	109	64	78	61
	4		80	114	102	106	117	75	89	67
	5		73	119	101	110	114	74	89	67
	10		83	137	99	102	114	75	89	67
	25		107	108	100	105	115	76	89	68
	50		106	111	101	106	115	77	89	69
	100	110	106	113	101	106	115	78	89	71
	200	111	107	113	102	107	116	79	88	72
	500	113	107	114	103	108	116	79	89	74
	1000	113	107	115	104	109	116	79	90	76
	2000	114	107	115	104	109	115	80	89	77
	5000	114	108	115	103	110	114	79	89	79
	10000	114	109	116	104	109	114	79	86	80
	15000	114	110	117	104	109	113	79	86	81
	20000	115	111	116	104	109	113	78	87	82
	25000	116	111	117	104	110	113	79	86	82
	30000	119	111	117	104	110	113	79	87	82
Average from 10,000 Cycles		115	110	117	104	110	113	79	86	81

Table 5.31(B) Equivalent Modulus of Spring Cemetery A-2-4 Soil (Cont'd)

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)																			
Test No.		9-10	9-11	9-12	9-13	9-14	9-15	9-16	9-17	9-18	9-19	9-20	9-21	9-22	9-23	9-24	9-25	9-26	9-27
Plate Load		50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Limerock		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Water Table (in.)		0	0	0	12	12	12	24	24	24	36	36	36	24	24	24	12	12	12
No. of Plate Load Cycle	1	92	93	92	68	70	72	57	52	49	41	47	45	54	53	58	64	70	65
	4	256	252	251	183	196	198	120	135	134	115	124	117	147	150	158	170	194	174
	5	256	260	258	189	193	189	124	142	131	115	129	123	147	148	159	178	192	181
	10	257	260	258	194	188	193	129	140	127	115	127	122	145	144	155	177	192	180
	25	252	256	258	187	185	191	149	139	119	114	124	120	142	143	152	176	190	179
	50	251	256	260	192	182	191	147	138	120	115	123	119	140	142	150	176	190	180
	100	250	255	259	193	184	193	146	139	124	115	122	119	139	141	149	177	190	181
	200	248	256	261	194	184	194	146	139	125	115	121	118	138	141	148	178	190	181
	500	250	255	264	198	191	198	147	141	130	117	122	117	137	139	148	179	190	182
	1000	251	255	266	199	192	199	147	142	132	117	121	115	136	139	147	180	190	182
	2000	252	253	263	199	190	199	147	143	134	117	120	110	134	137	145	177	188	181
	5000	248	249	260	199	188	198	147	144	136	114	116	100	132	136	142	179	191	179
	10000	246	246	256	198	184	198	149	145	137	111	113	98	131	135	139	179	191	177
	15000	245	244	254	198	192	199	149	146	137	109	112	103	131	134	140	177	189	177
	20000	244	243	252	195	193	199	150	146	137	108	112	106	130	134	139	176	193	177
	25000	242	242	251	197	193	200	151	147	136	108	113	108	131	133	139	176	193	177
	30000	244	242	251	198	192	200	147	147	136	108	114	106	131	133	141	178	194	177
Average from 10,000 Cycles		244	243	253	197	191	200	149	146	137	109	113	104	131	134	139	177	192	177

Table 5.32(A) Equivalent Modulus of Branch A-2-4 Soil

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)												
Test No.		10-1	10-2	10-3	10-4	10-5	10-6	10-7	10-8	10-9	10-10	10-11
Plate Load		20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi
Limerock		No	No	No	No	No	No	No	No	No	No	No
Water Table (in.)		0	0	0	0	12	12	12	12	24	24	24
No. of Plate Load Cycle	1	247	346	270	248	248	169	197	366	175	172	117
	4	292	420	305	291		248	232	439	225	213	128
	5	285	405	296	281	302	251	231	430	222	213	127
	10	345	399	303	275	307	262	232	427	222	193	133
	25	297	392	305	269	307	254	234	427	220	204	148
	50	286	390	304	267	307	255	233	425	219	202	148
	100	288	392	307	264	308	254	231	424	218	200	149
	200	289	396	309	264	308	257	232	428	219	199	152
	500	286	402	314	266	306	265	232	443	219	197	154
	1000	289	406	317	268	307	264	232	448	223	198	158
	2000	294	414	318	271	311	274	239	456	228	197	161
	5000	295	429	325	274	311	305	246	468	233	199	163
	10000	306	433	329	287	316	309	241	460	247	197	165
	15000	312	438	326	292	320	316	242	455	251	198	166
	20000	315	439	326	293	323	322	242	451	251	201	167
	25000	318	441	327	296	322	327	243	460	251	200	167
	30000	320	445	330	296	325	329	246	464	250	202	167
Average from 10,000 Cycles		314	439	328	293	321	321	243	458	250	200	166

Table 5.32(B) Equivalent Modulus of Branch A-2-4 Soil (Cont'd)

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)																				
Test No.		10-12	10-13	10-14	10-15	10-16	10-17	10-18	10-19	10-20	10-21	10-22	10-23	10-24	10-25	10-26	10-27	10-28	10-29	10-30
Plate Load		50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Lime Rock		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Water Table (in.)		0	0	0	12	12	12	24	24	24	24	36	36	36	24	24	24	12	12	12
No. of Plate Load Cycle	1	214	252	190	182	234	156	156	157	81	148	57	54	65	70	62	70	152	135	163
	4	677	746	565	510	705	456	456	462	243	457	182	161	189	213	189	215	470	420	539
	5	666	737	598	510	682	461	461	462	227	440	182	163	194	214	188	213	480	416	551
	10	623	741	583	511	672	443	443	451	239	430	180	161	190	214	189	211	451	412	483
	25	614	740	541	514	670	434	434	446	240	429	178	158	187	213	190	209	450	410	477
	50	612	749	549	519	673	433	433	446	242	431	178	157	187	214	190	210	448	403	476
	100	616	757	520	523	679	432	432	449	243	434	180	155	185	215	190	209	448	401	476
	200	615	760	556	525	680	433	433	450	244	435	180	155	187	215	191	208	445	395	477
	500	620	767	613	534	684	427	427	449	244	436	180	151	185	215	190	208	441	387	477
	1000	623	771	577	541	690	422	422	439	241	435	179	153	184	212	189	207	434	380	485
	2000	636	776	593	548	694	416	416	441	238	432	177	150	181	212	186	206	426	373	491
	5000	619	784	606	558	698	554	408	439	234	424	175	145	178	206	182	204	416	357	496
	10000	590	785	607	561	709	556	405	435	228	418	173	139	174	201	179	202	393	350	501
	15000	607	780	621	563	715	555	398	434	224	415	171	135	172	198	177	203	384	347	508
	20000	605	790	631	570	714	551	394	434	223	409	170	131	173	198	176	202	390	344	506
	25000	595	796	638	574	687	546	392	432	223	407	168	122	169	197	175	198	387	345	505
	30000	570	798	650	573	692	545	390	430	219	404	164	126	168	196	174	200	395	344	510
Average from 10,000 Cycles		593	790	629	568	704	551	396	433	223	410	169	131	171	198	176	201	390	346	506

Table 5.33(A) Equivalent Modulus of Iron Bridge A-2-6 Soil

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)											
Test No.		11-1	11-2	11-3	11-4	11-5	11-6	11-7	11-8	11-9	11-10
Plate Load		20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi	20 psi
Lime Rock		No	No	No	No	No	No	No	No	No	No
Water Table (in.)		0	0	0	0	12	12	12	24	24	24
No. of Plate Load Cycle	1	76	129	184	128	142	106	157	100	70	87
	4	87	150	206	155	175	145	200	134	99	110
	5	83	145	200	151	173	144	194	128	98	107
	10	81	143	199	151	170	145	192	128	97	106
	25	84	139	197	151	168	147	191	128	99	103
	50	83	139	196	152	168	149	192	129	100	104
	100	82	139	197	152	168	150	193	130	101	104
	200	82	139	199	151	168	151	195	131	102	105
	500	82	141	199	150	171	153	198	133	102	107
	1000	83	143	200	150	172	154	202	134	103	108
	2000	84	145	204	150	172	155	206	136	103	108
	5000	84	149	209	153	175	159	215	138	103	109
	10000	82	154	215	152	180	163	219	140	104	109
	15000	77	155	220	153	183	166	224	141	105	110
	20000	80	156	224	154	186	169	229	142	105	110
	25000	82	157	228	155	192	171	232	143	105	109
	30000		156	229	156	196	172	242	143	106	109
Average from 10,000 Cycles		80	155	223	154	187	168	229	142	105	109

Table 5.33(B) Equivalent Modulus of Iron Bridge A-2-6 Soil (Cont'd)

EQ Modulus (MPa) : 1.38 pa/ (Resilient Deformation)																			
Test No.		11-11	11-12	11-13	11-14	11-15	11-16	11-17	11-18	11-19	11-20	11-21	11-22	11-23	11-24	11-25	11-26	11-27	11-28
Plate Load		50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Lime Rock		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Water Table (in.)		0	0	0	12	12	12	24	24	24	36	36	36	24	24	24	12	12	12
No. of Plate Load Cycle	1	158	199	178	181	177	166	121	120	102	65	58	66	90	75	85	120	125	124
	4	498	607	549	540	534	486	363	369	293	189	166	187	260	218	250	389	385	394
	5	500	592	544	547	508	483	360	357	298	192	165	190	265	221	252	384	376	376
	10	489	590	544	543	501	479	355	362	293	186	158	183	258	211	250	391	364	373
	25	483	587	515	540	499	480	354	350	287	176	150	175	252	203	245	390	361	370
	50	487	591	514	542	500	480	354	355	288	171	146	170	253	200	242	391	361	372
	100	493	600	518	541	508	480	358	356	288	165	142	166	251	198	240	390	360	372
	200	496	604	519	540	509	479	359	356	289	162	140	164	250	197	238	391	359	374
	500	507	612	528	541	520	480	363	364	289	154	135	159	246	198	232	390	358	376
	1000	515	621	532	544	522	480	364	359	288	147	131	155	244	194	232	390	354	377
	2000	523	624	537	548	528	484	366	359	286	138	124	152	242	194	225	391	351	378
	5000	529	633	548	558	539	488	369	353	284	122	101	148	239	195	219	386	351	381
	10000	536	642	538	571	544	491	370	354	285	110	100	143	238	198	219	387	351	386
	15000	542	655	563	578	550	488	371	352	283	104	96	141	238	202	218	390	353	387
	20000	550	662	566	585	554	491	371	358	283	102	94	138	238	200	221	392	353	391
	25000	563	663	573	588	556	491	370	344	285	100	92	136	239	202	221	394	355	393
	30000	575	653	585	593	556	493	370	347	284	99	91	134	236	202	221	397	348	395
Average from 10,000 Cycles		553	655	565	583	552	491	370	351	284	103	95	138	238	201	220	392	352	390

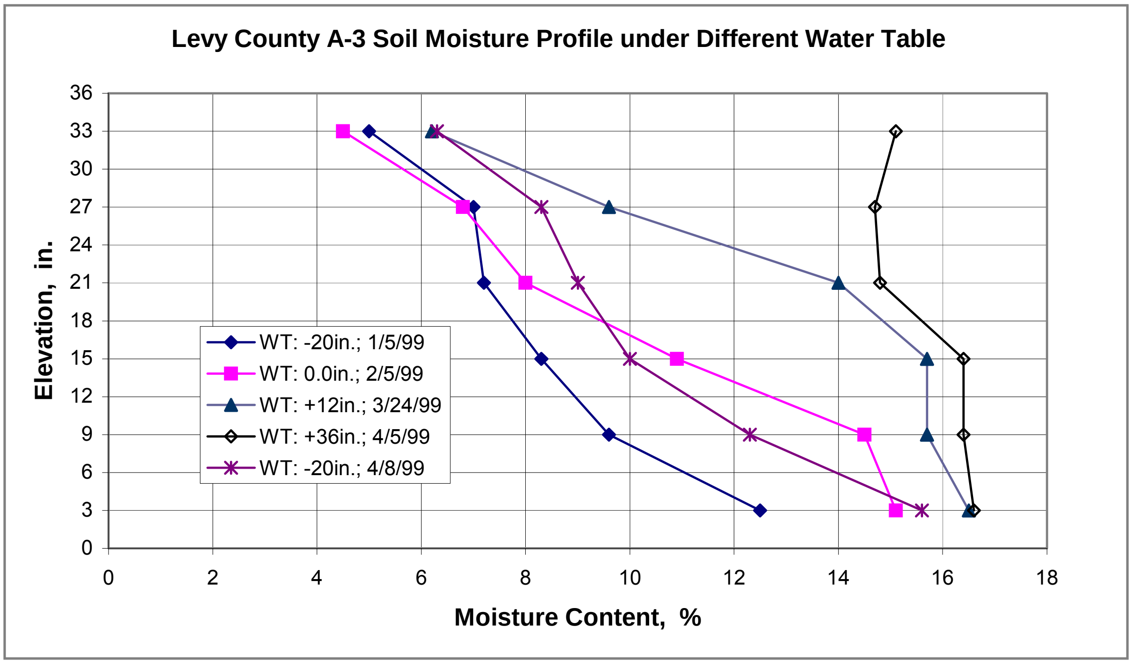


Figure 5.1 Levy County A-3 Soil Moisture Profile under Different Water Table Levels

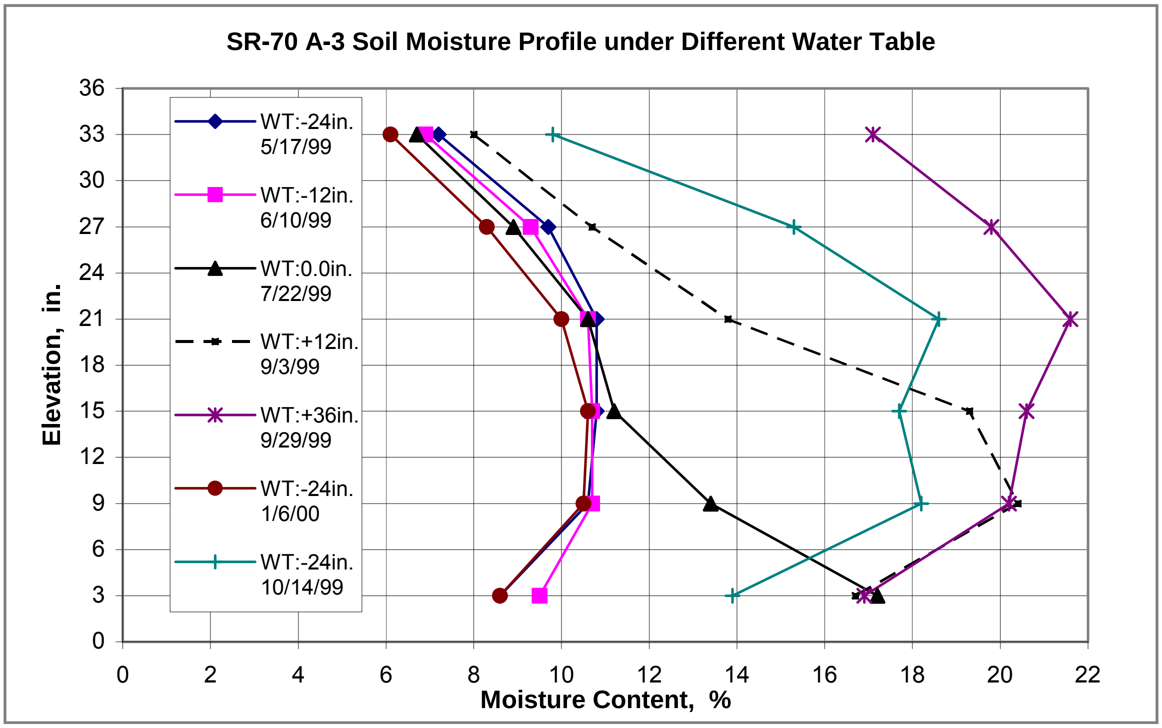


Figure 5.2 SR70 A-3 Soil Moisture Profile under Different Water Table Levels (Moisture nearly stabilized from 9/29/99 to 10/11/99)

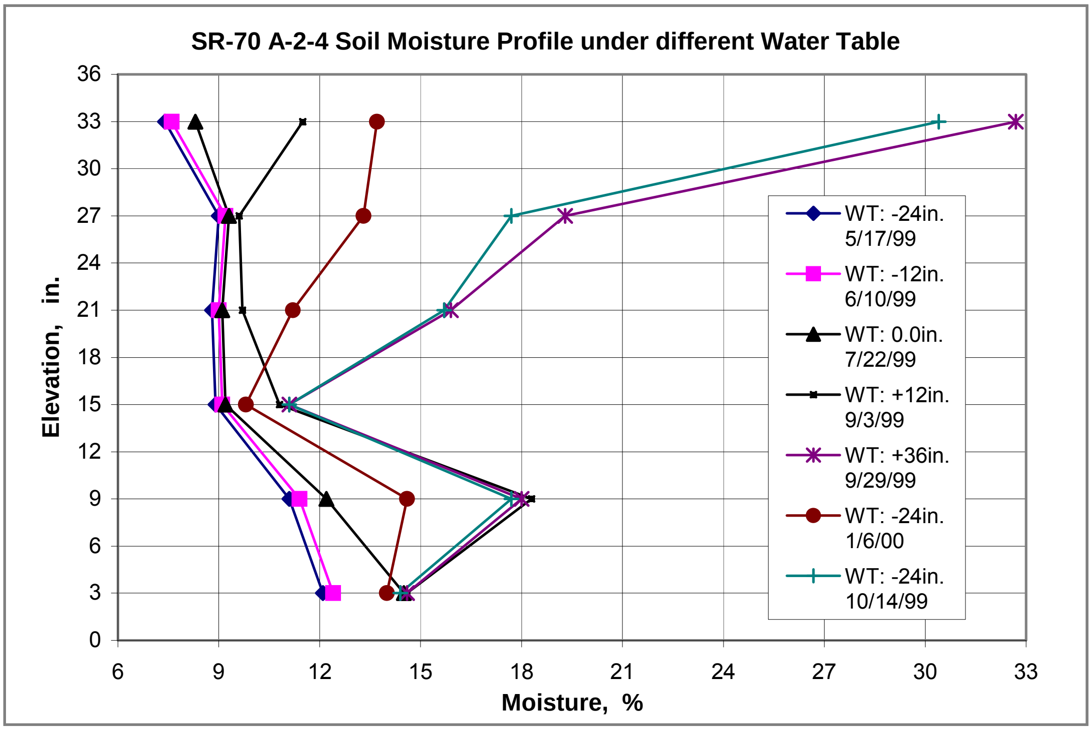


Figure 5.3 SR70 A-2-4 Soil Moisture profile under Different Water Table Levels (Moisture nearly stabilized from 9/29/99 to 10/11/99)

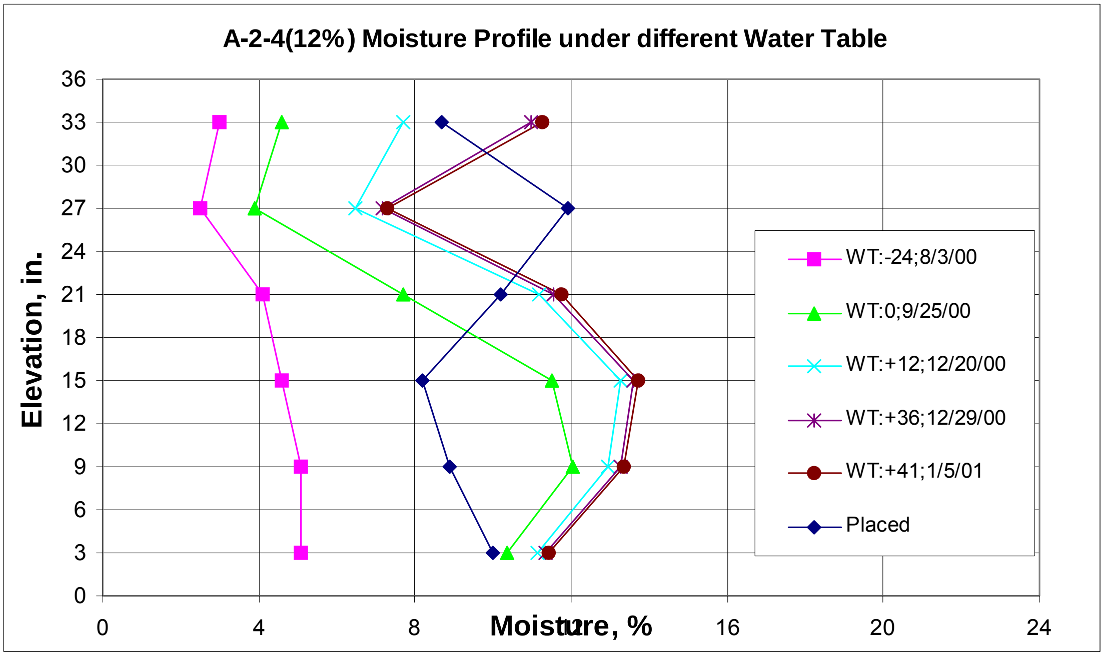


Figure 5.4 A-2-4 (12%)Soil Moisture profile under Different Water Table Levels

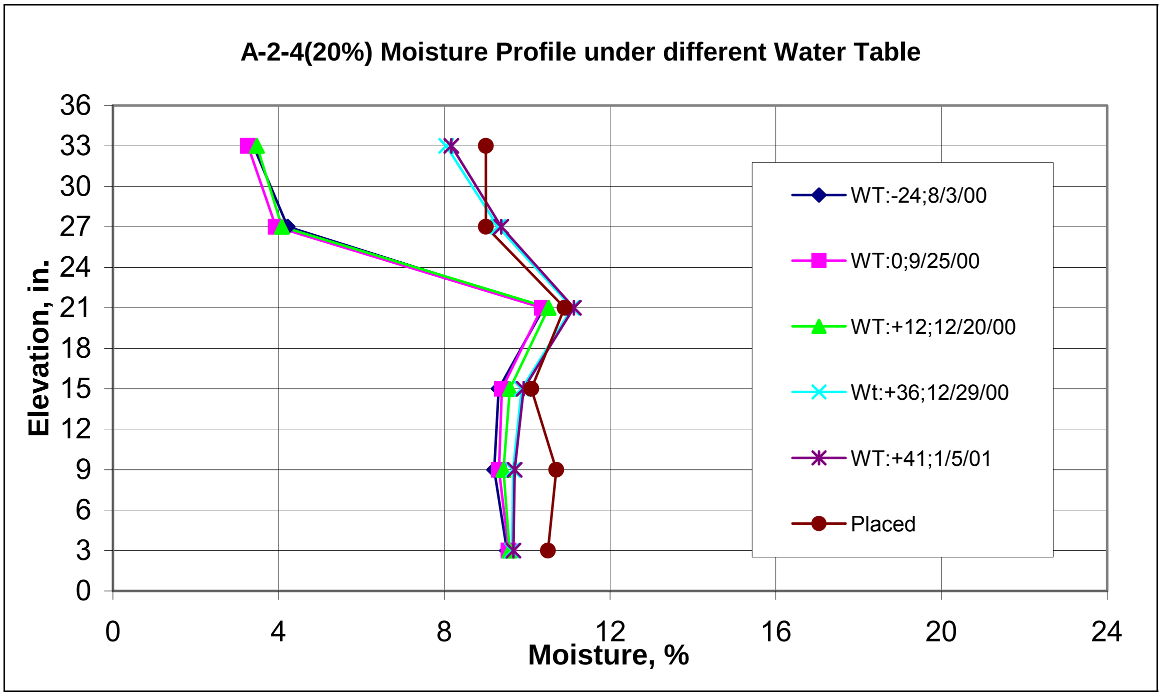


Figure 5.5 A-2-4 (20%) Soil Moisture profile under Different Water Table Levels

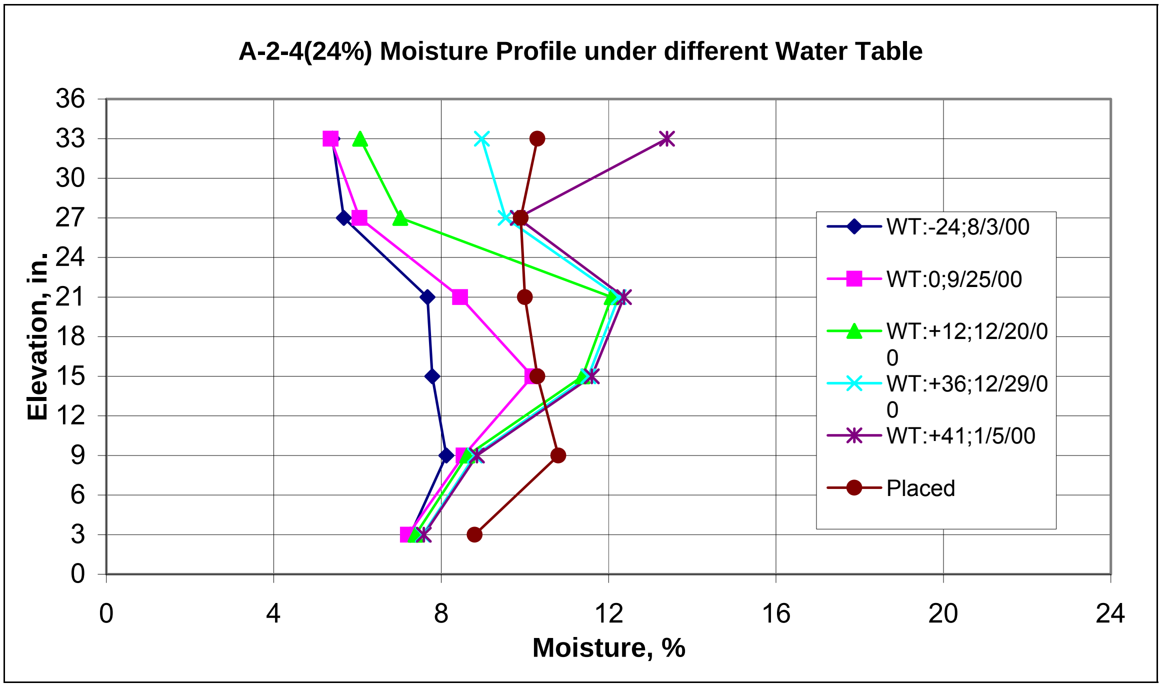


Figure 5.6 A-2-4(24%) Soil Moisture profile under Different Water Table Levels

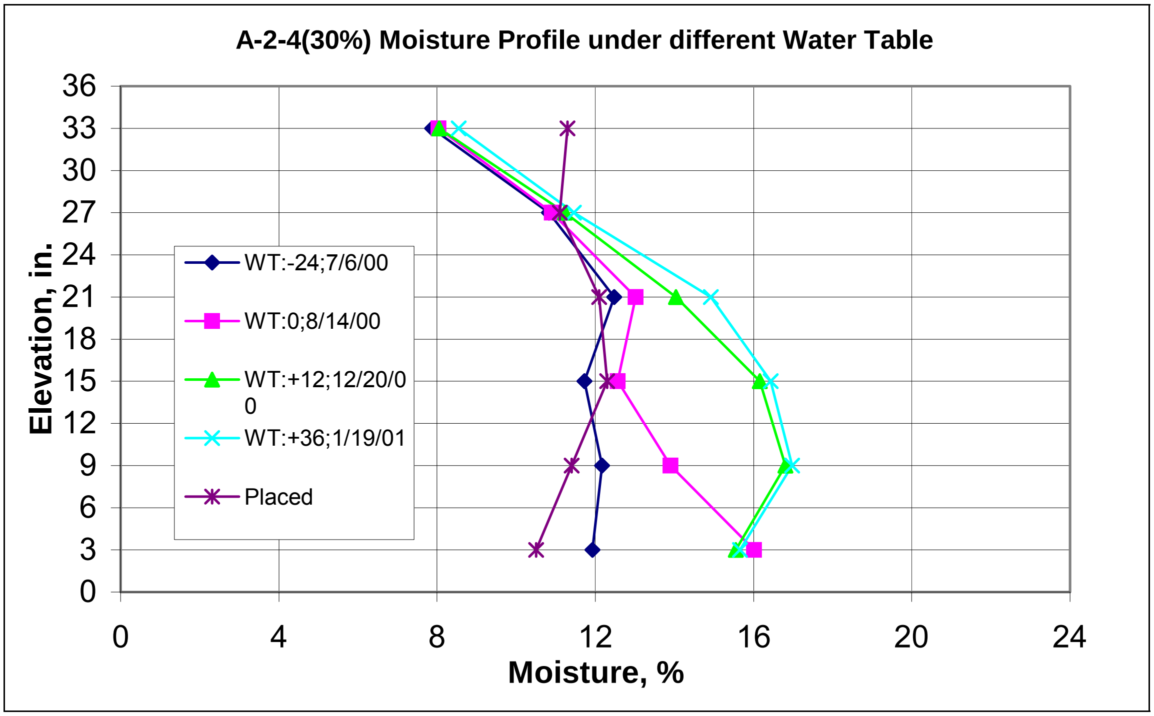


Figure 5.7 A-2-4(30%) Soil Moisture profile under Different Water Table Levels

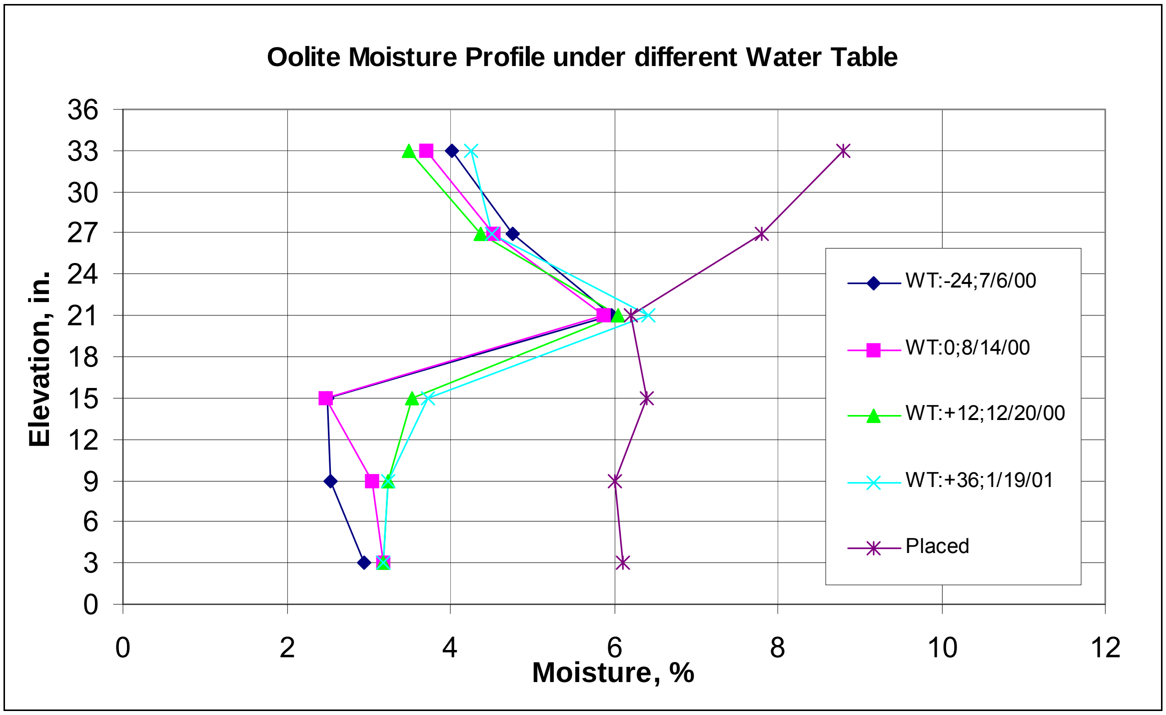


Figure 5.8 Miami Oolite A-1 Soil Moisture profile under Different Water Table Levels

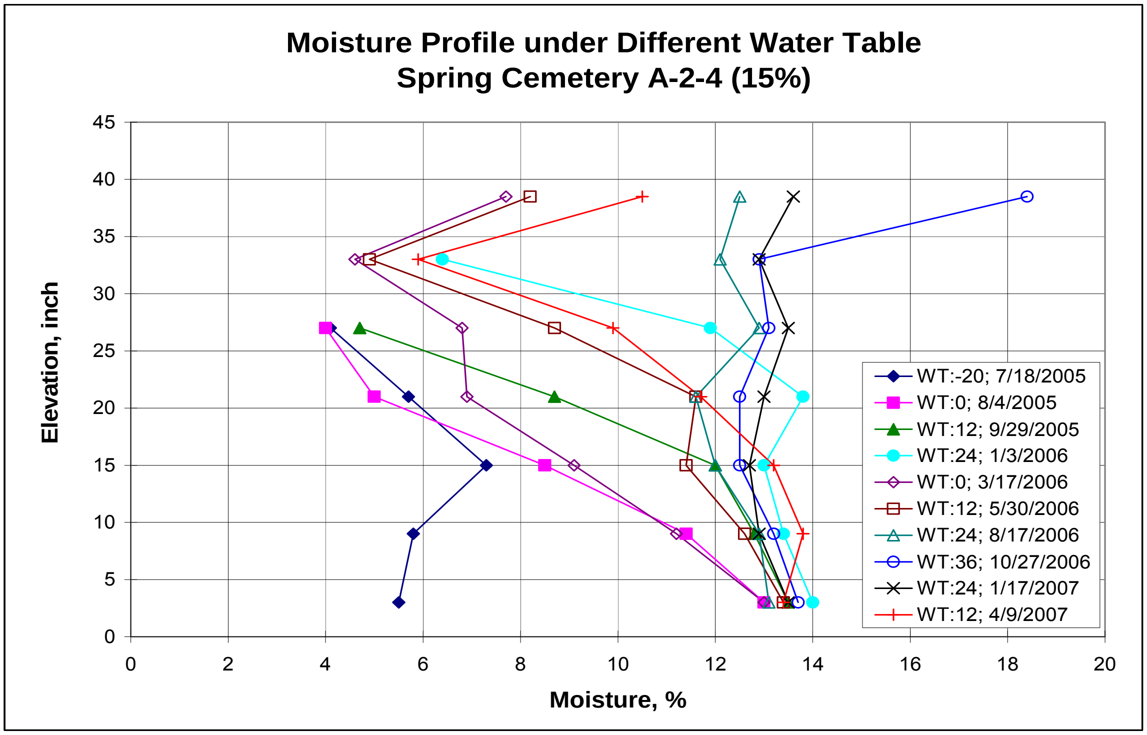


Figure 5.9 Spring Cemetery A-2-4(15%) Soil Moisture profile under Different Water Table Levels

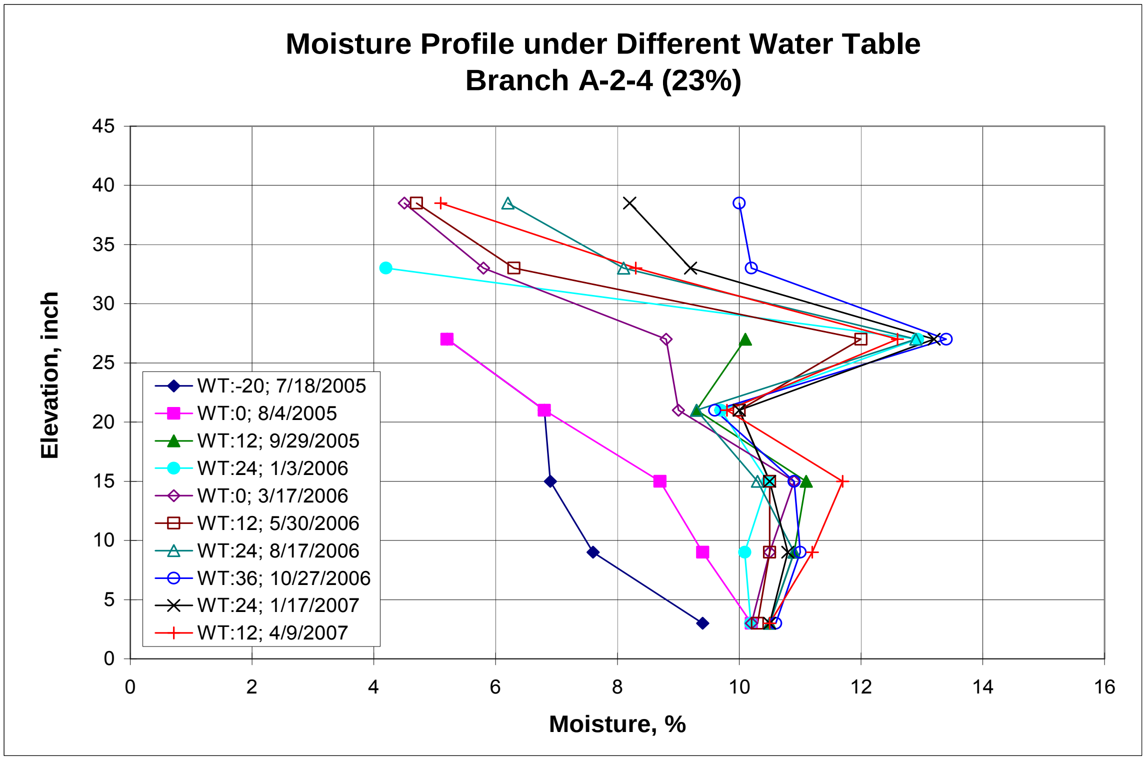


Figure 5.10 Branch A-2-4(23%) Soil Moisture profile under Different Water Table Levels

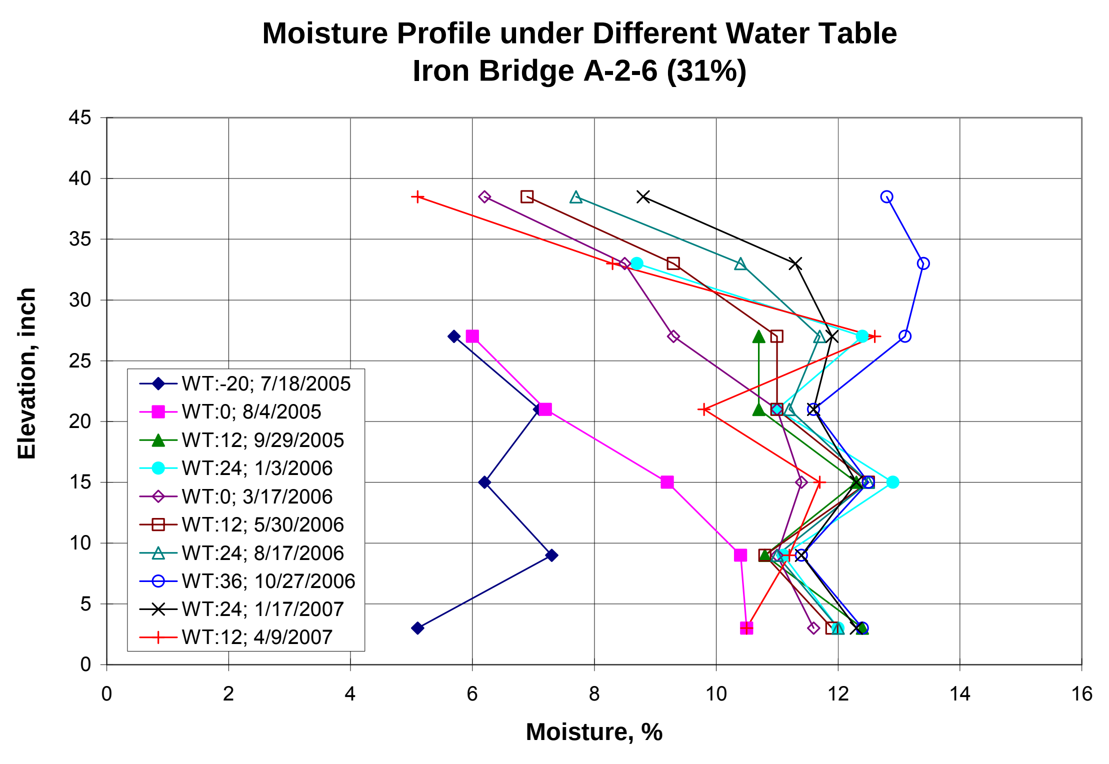


Figure 5.11 Iron Bridge A-2-6(31%) Soil Moisture profile under Different Water Table Levels

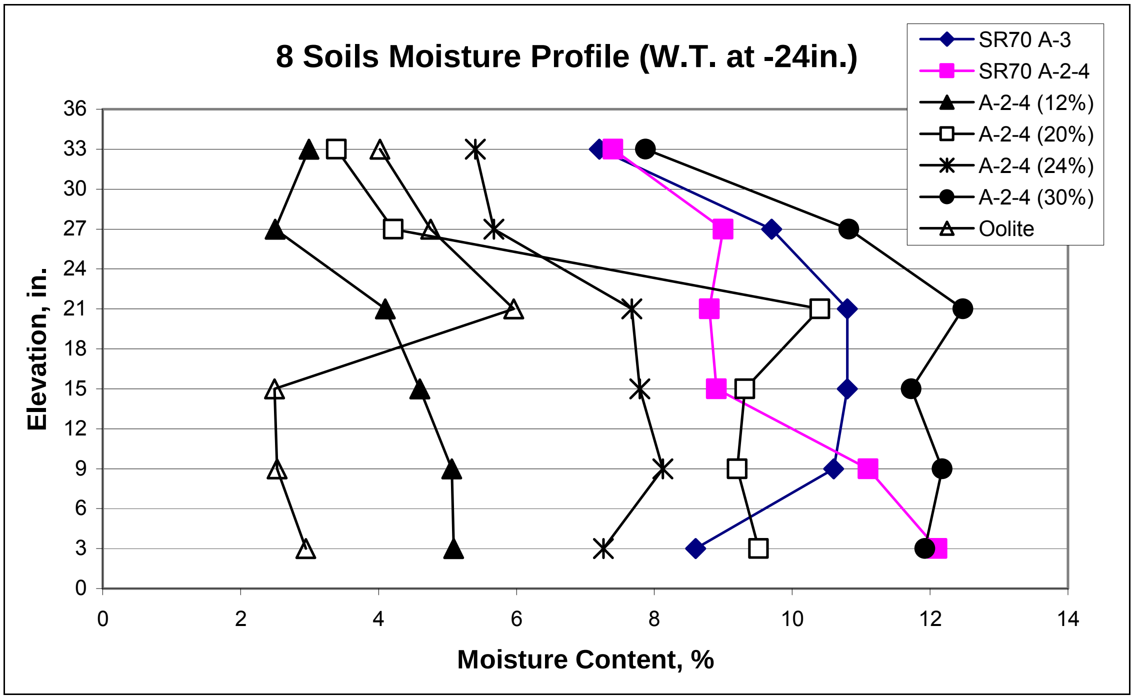


Figure 5.12 8 Soils Moisture Profiles (Water Table at -24 in., Drained Condition)

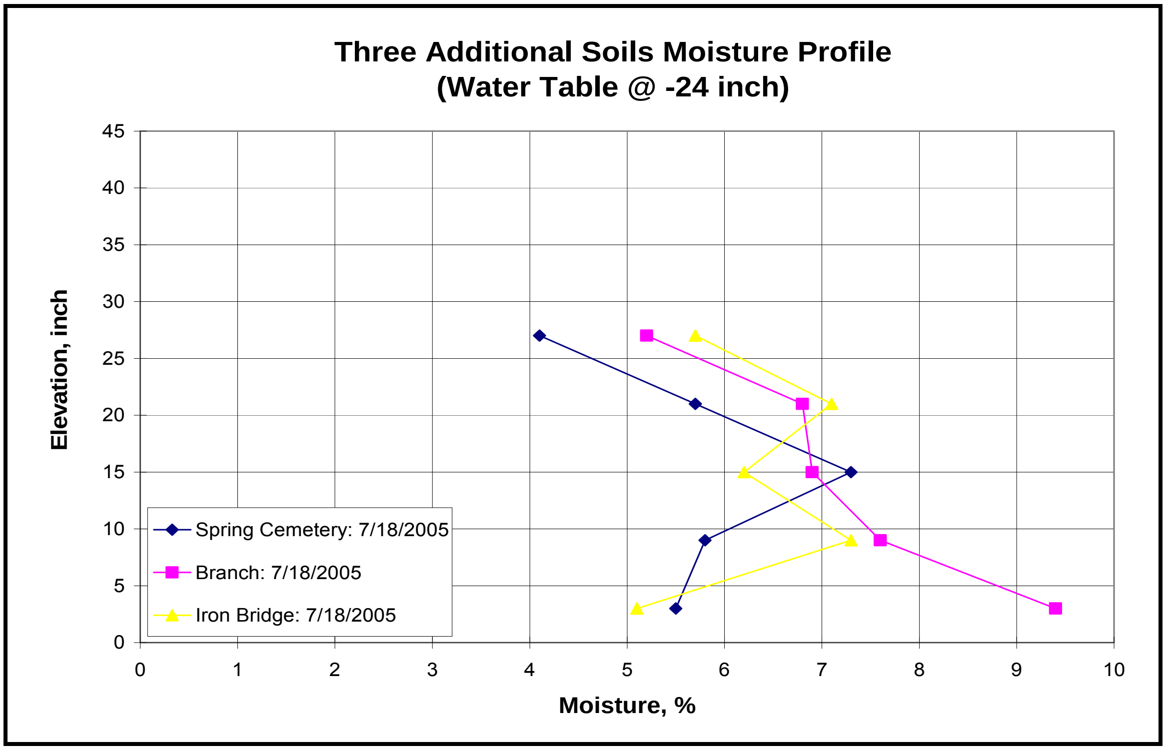


Figure 5.13 Three Additional Soils Moisture Profiles (Water Table at -24 in., Drained Condition)

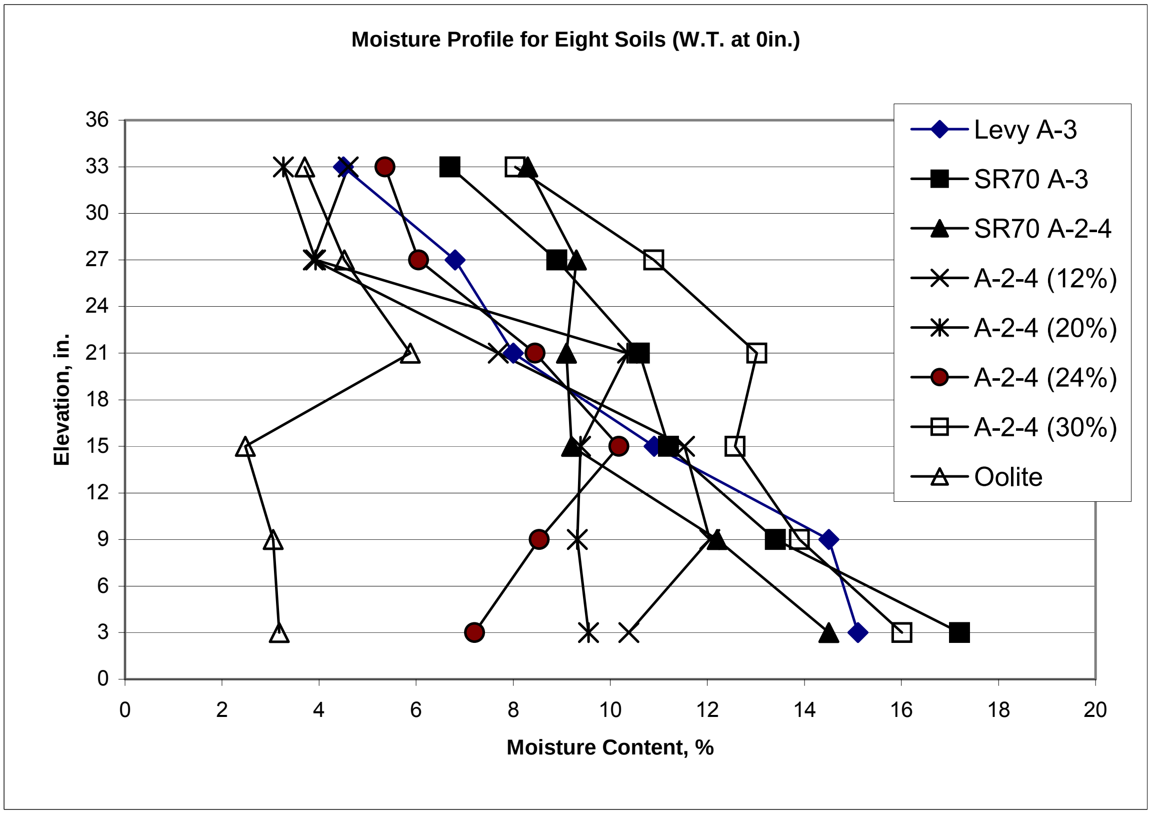


Figure 5.14 Eight Soils Moisture Profiles (Water Table at 0.0 in.)

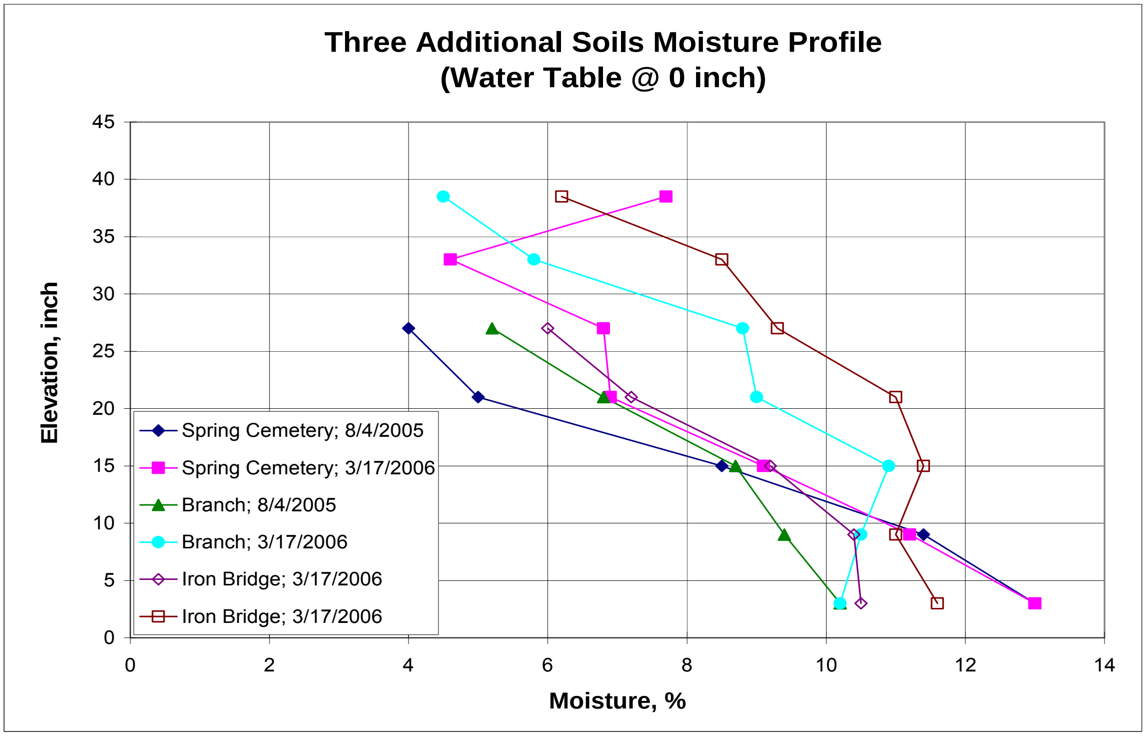


Figure 5.15 Three Additional Soils Moisture Profiles (Water Table at 0.0 in.)

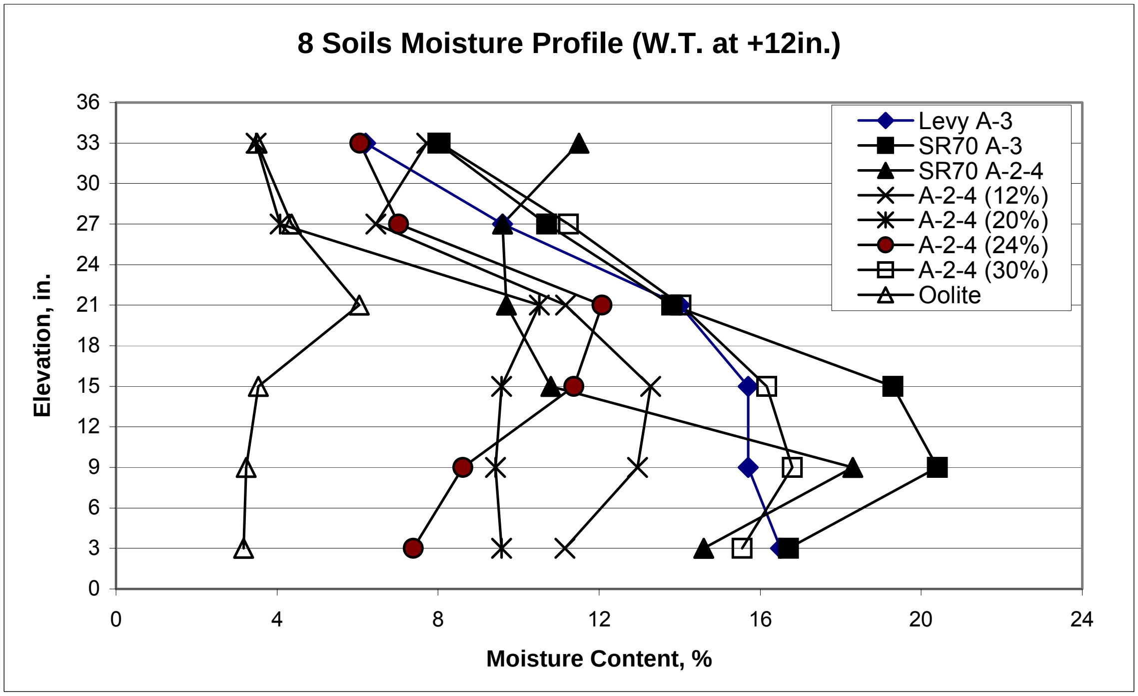


Figure 5.16 Eight Soils Moisture Profiles (Water Table at +12.0 in.)

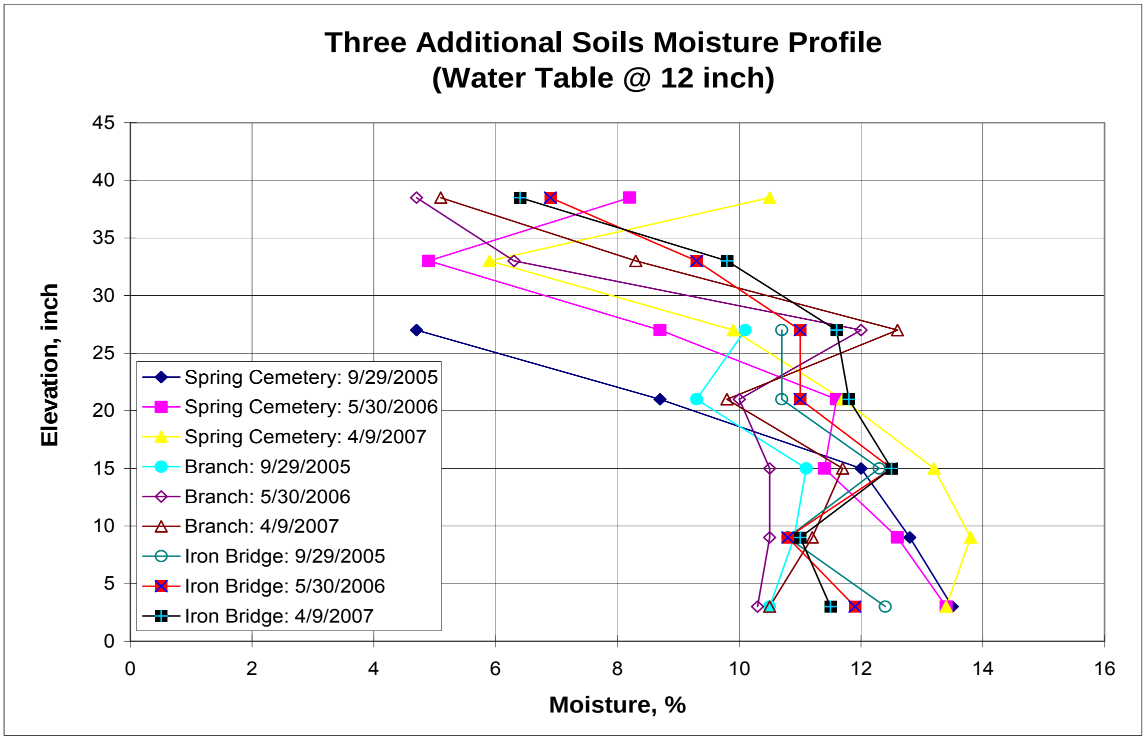


Figure 5.17 Three Additional Soils Moisture Profiles (Water Table at +12.0 in.)

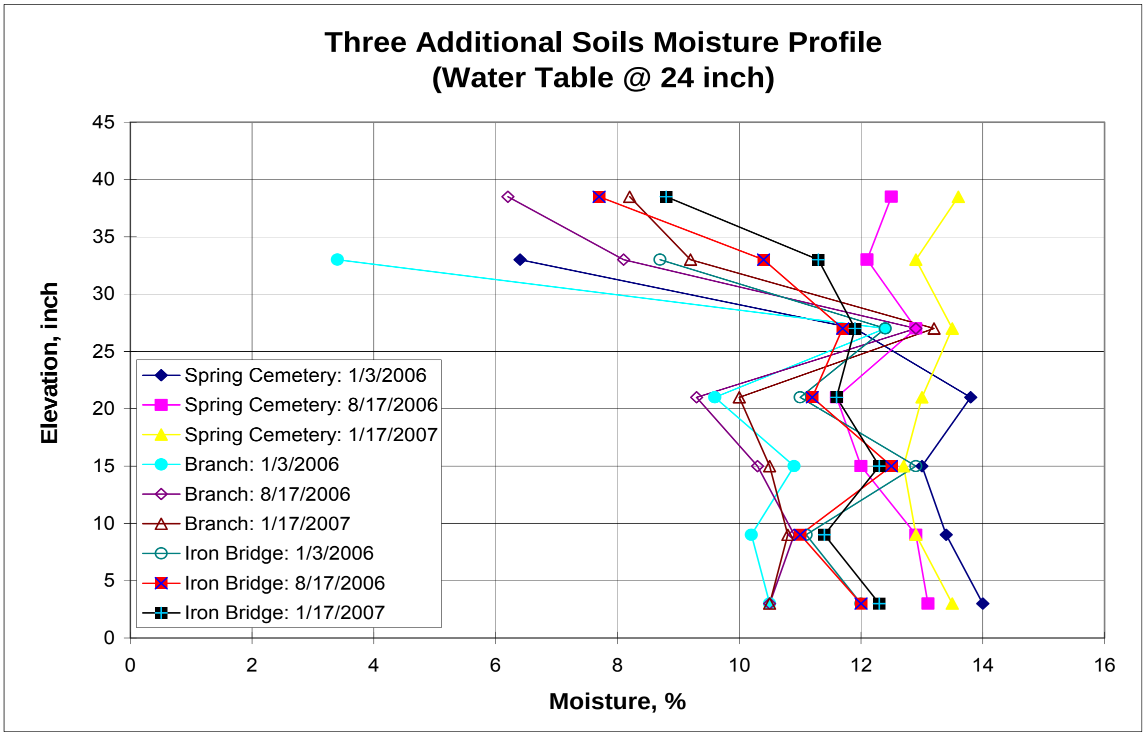


Figure 5.18 Three Additional Soils Moisture Profiles (Water Table at +24.0 in.)

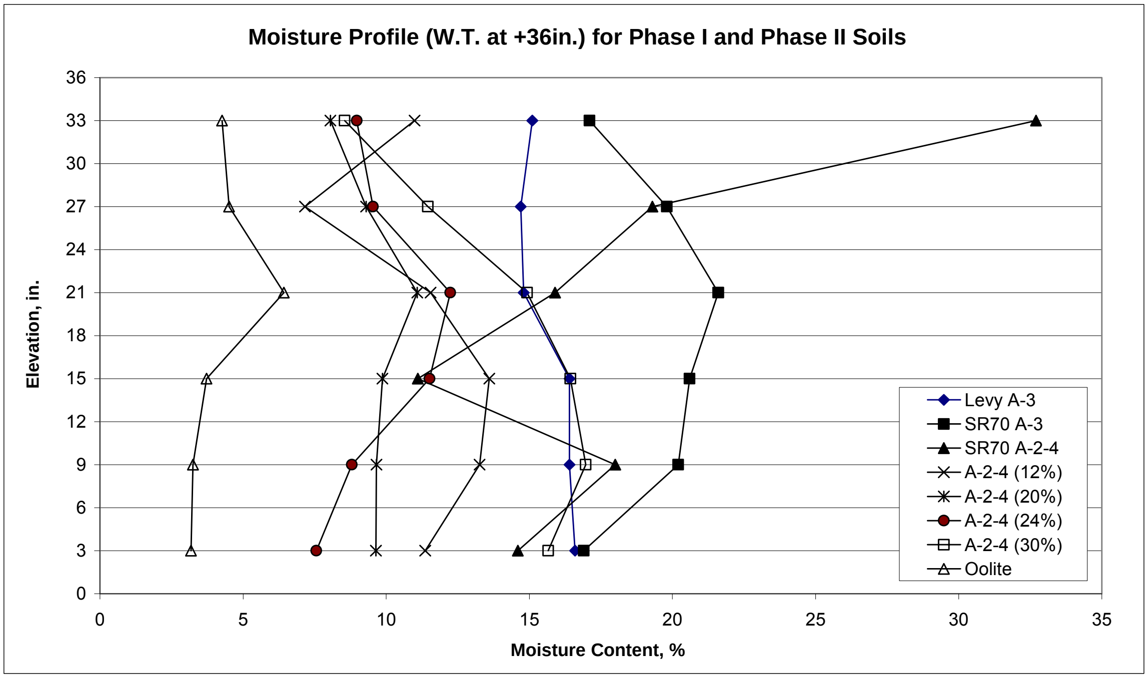


Figure 5.19 Eight Soils Moisture Profiles (Water Table at +36.0 in., Saturated Condition)

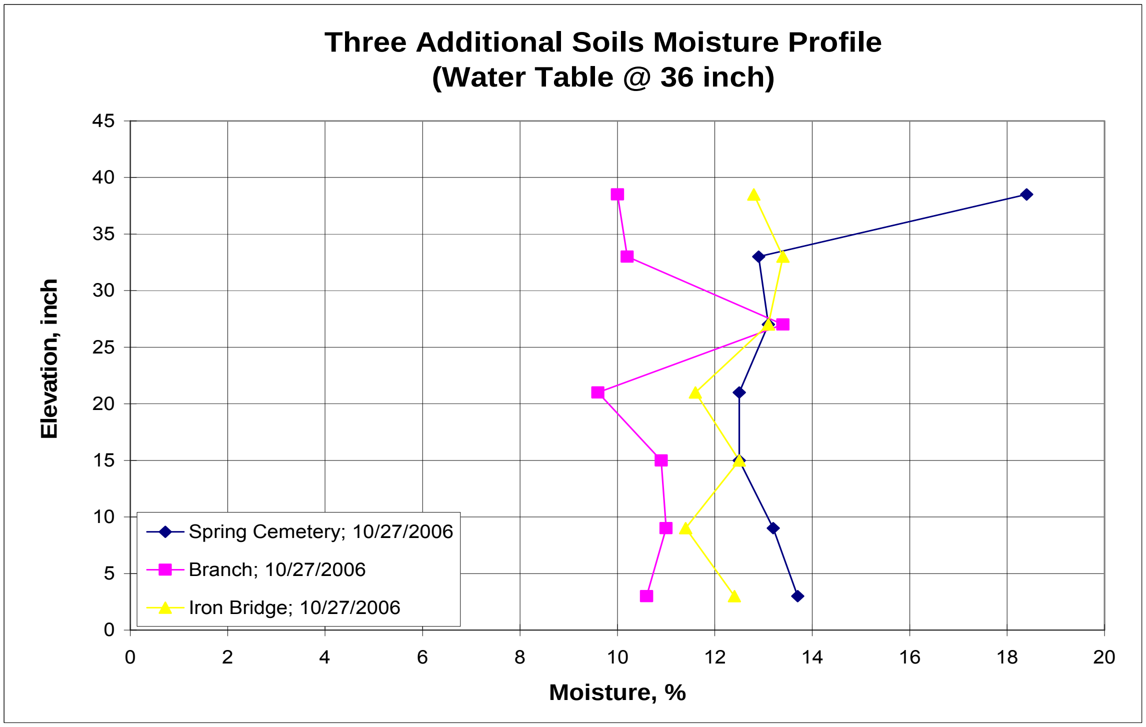


Figure 5.20 Three Additional Soils Moisture Profiles (Water Table at +36.0 in., Saturated Condition)

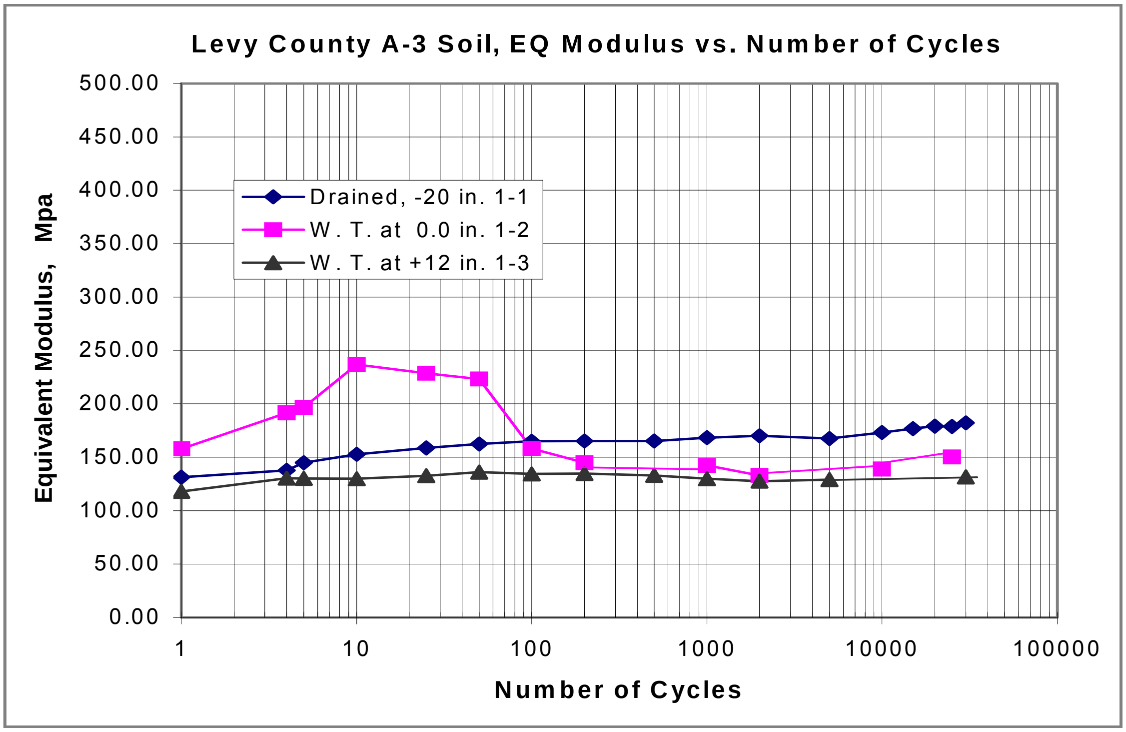


Figure 5.21(A) Levy County A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

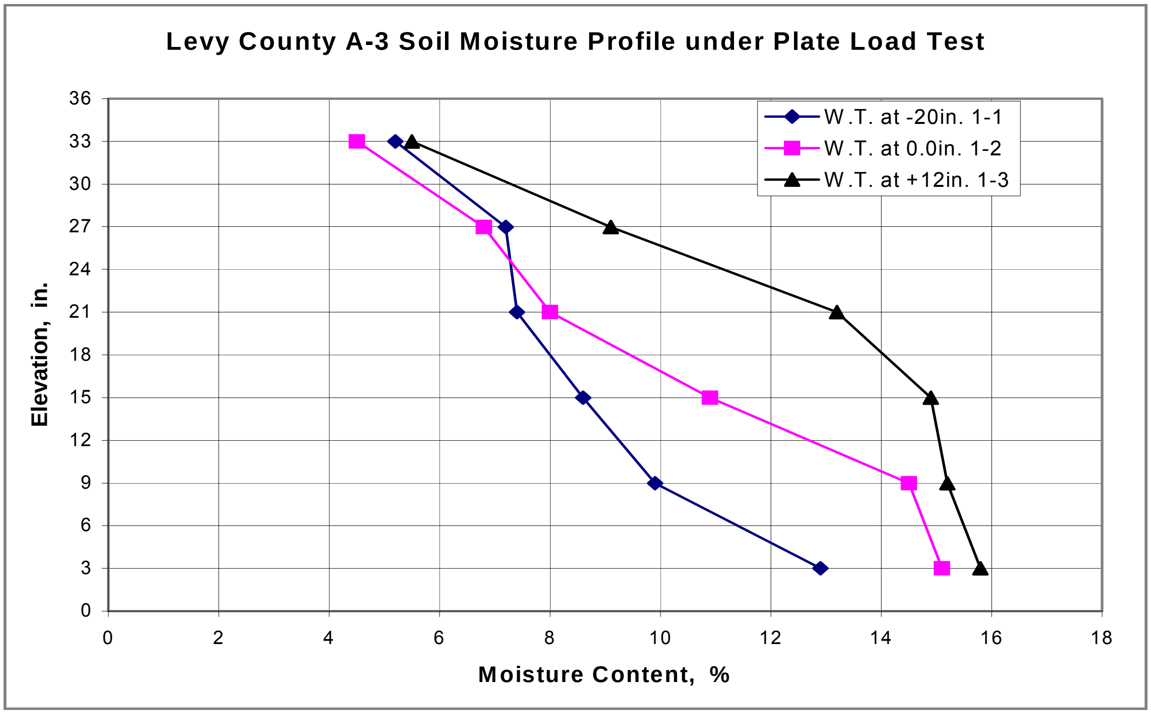


Figure 5.21(B) Levy County A-3 Soil Moisture Profile under Plate Load Test (20 psi without Limerock)

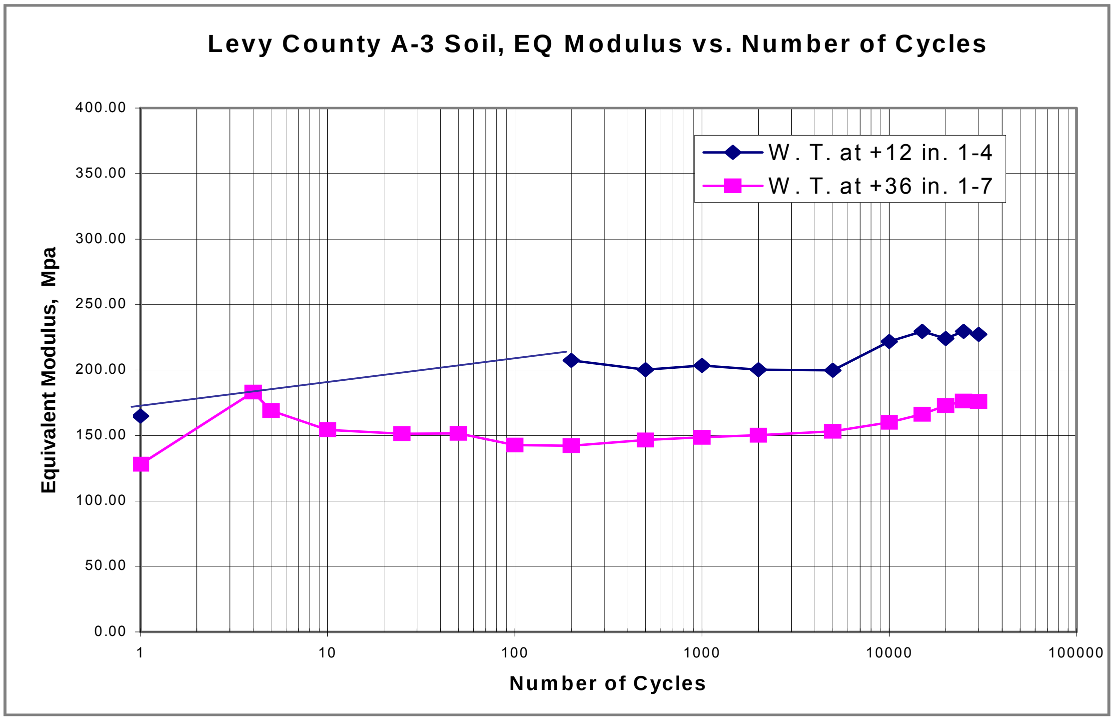


Figure 5.22(A) Levy County A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi with Limerock)

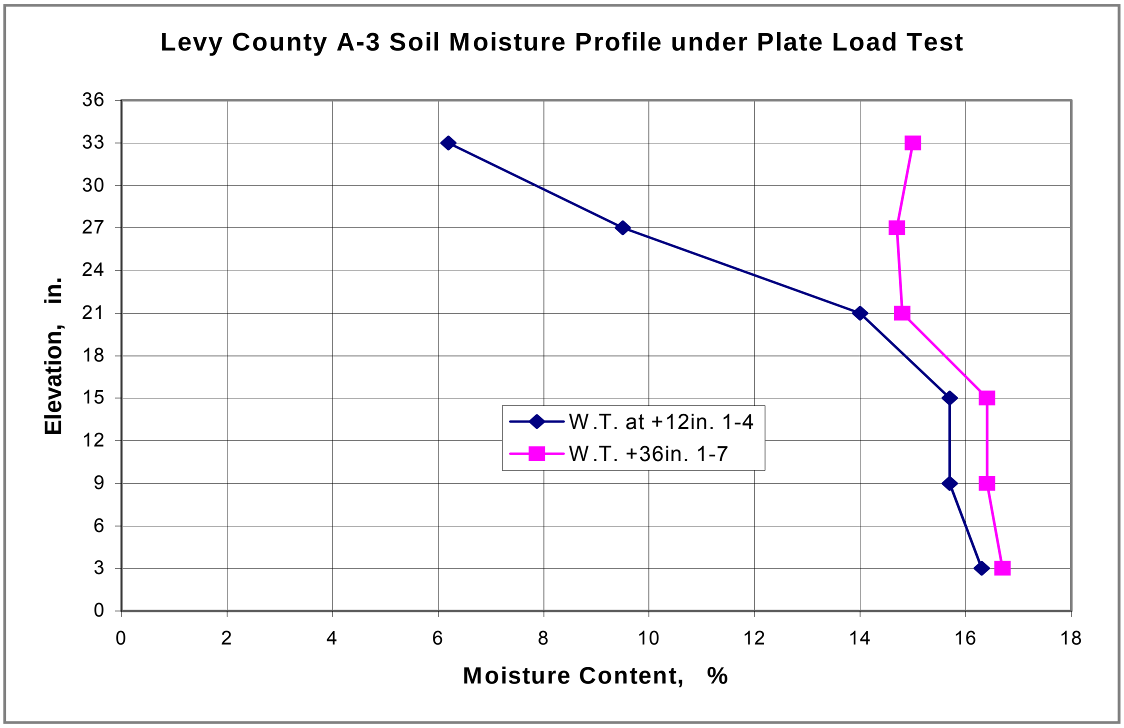


Figure 5.22(B) Levy County A-3 Soil Moisture Profile under Plate Load Test (20 psi with Limerock)

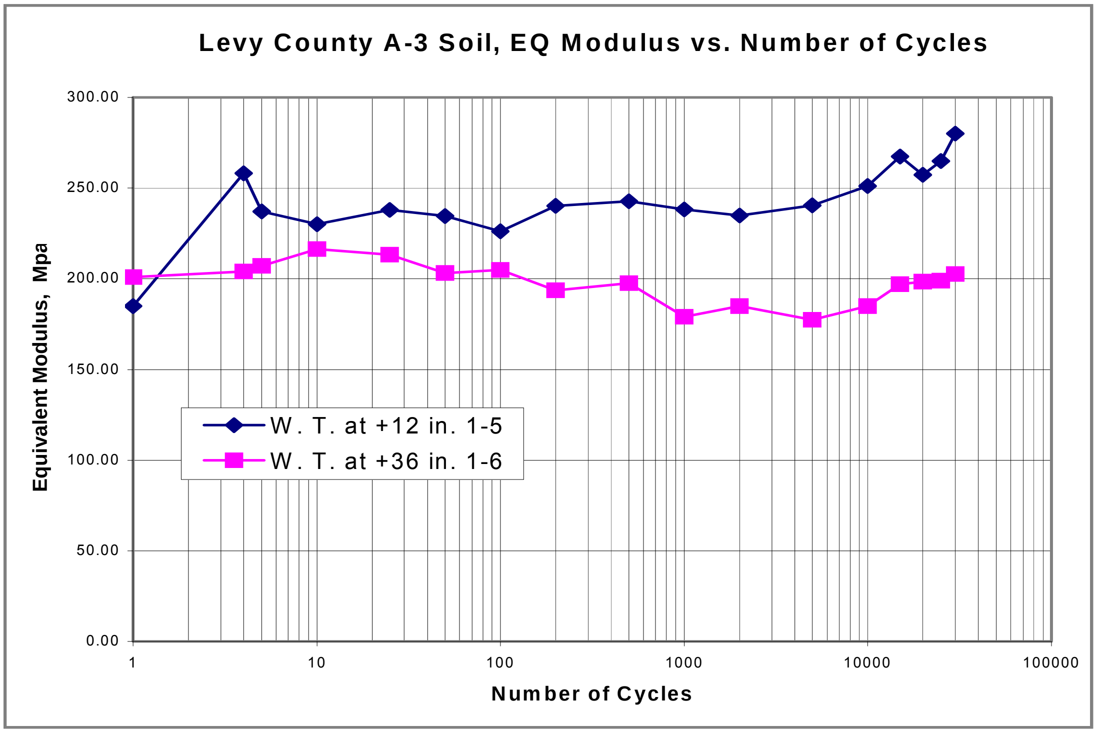


Figure 5.23(A) Levy County A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

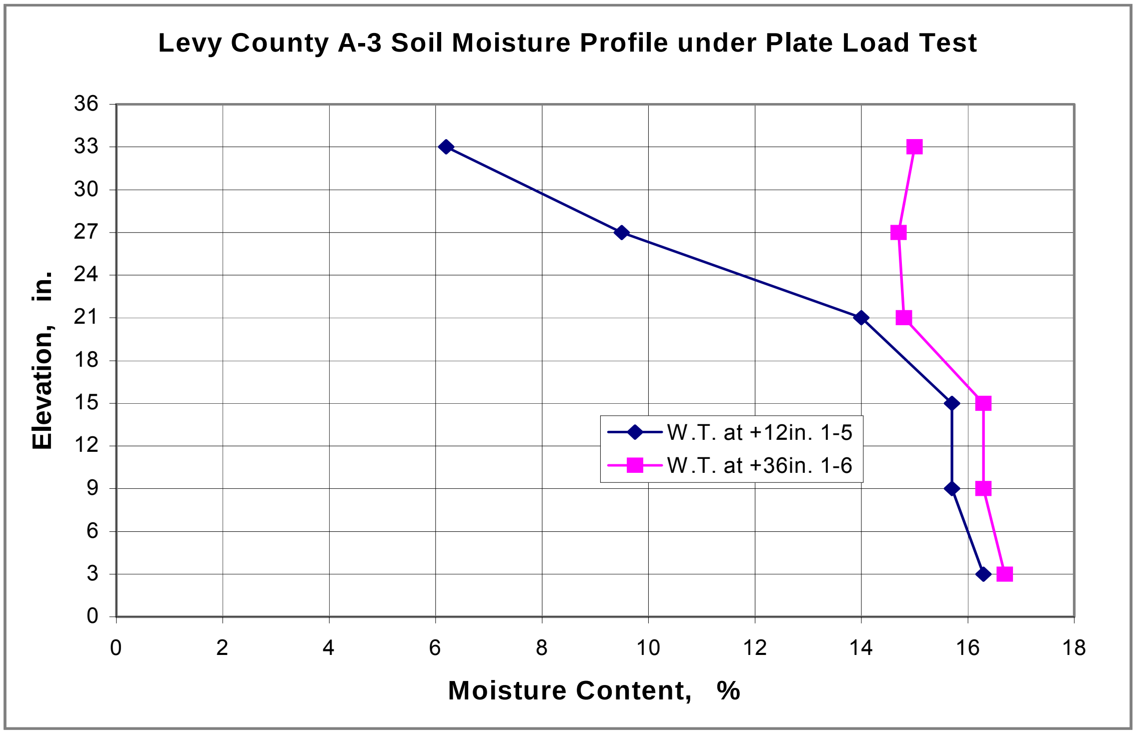


Figure 5.23(B) Levy County A-3 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

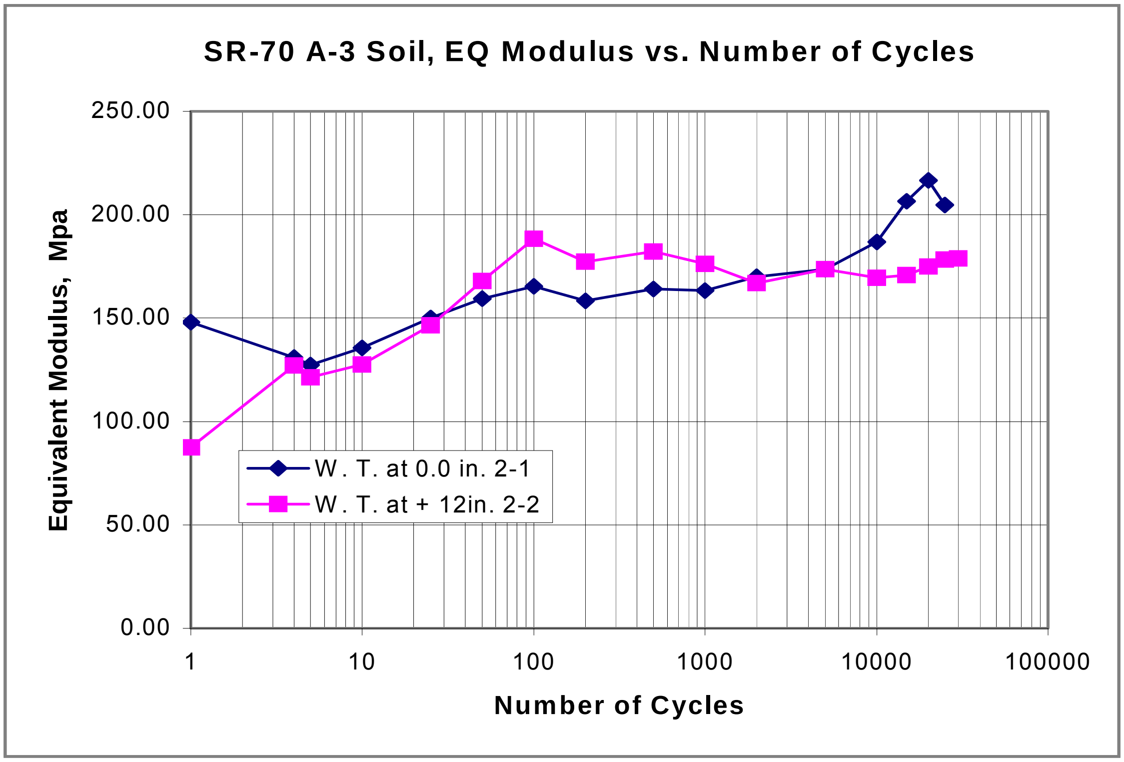


Figure 5.24(A) SR70 A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

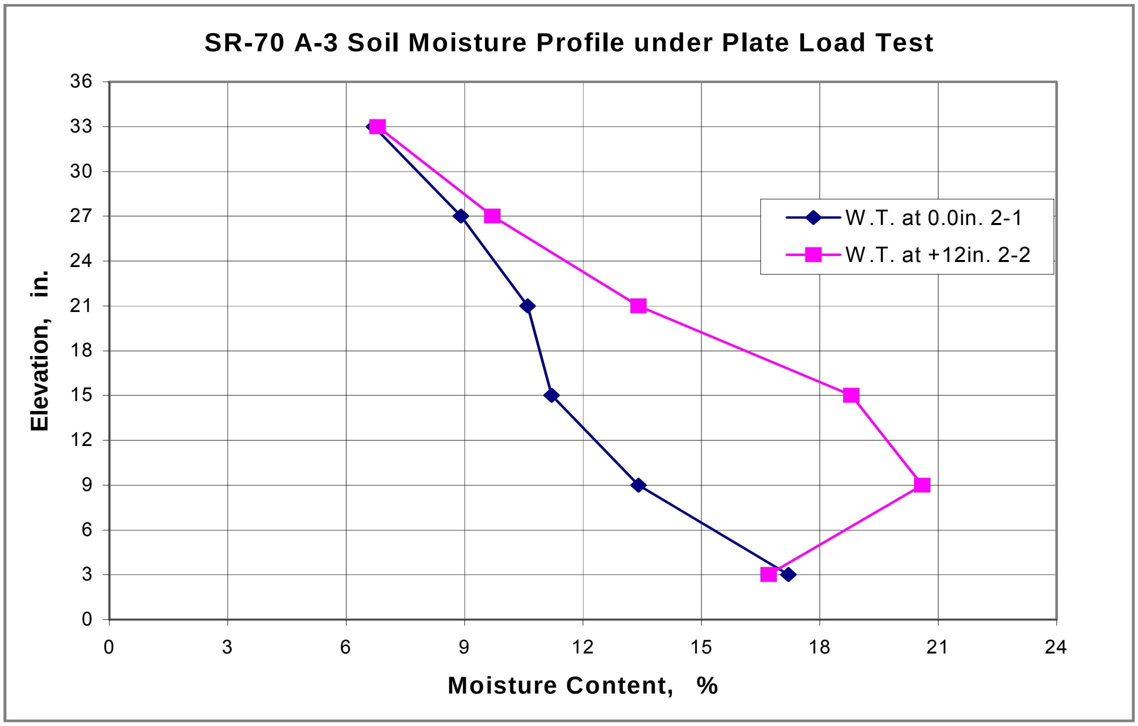


Figure 5.24(B) SR70 A-3 Soil Moisture Profile under Plate Load Test (20 psi without Limerock)

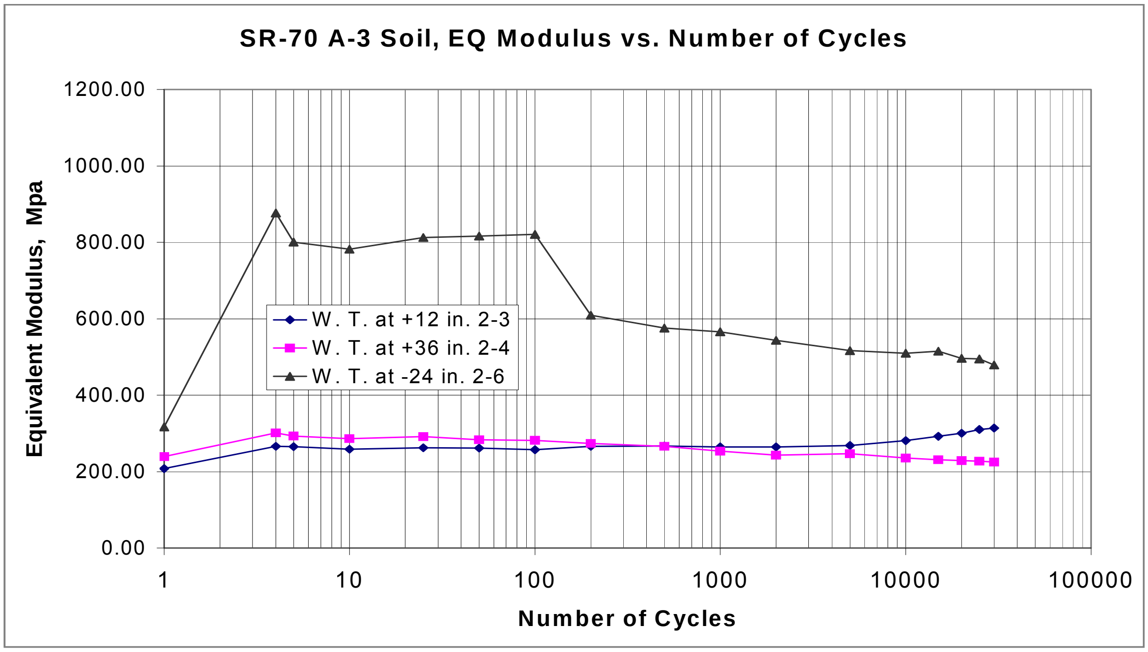


Figure 5.25(A) SR70 A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

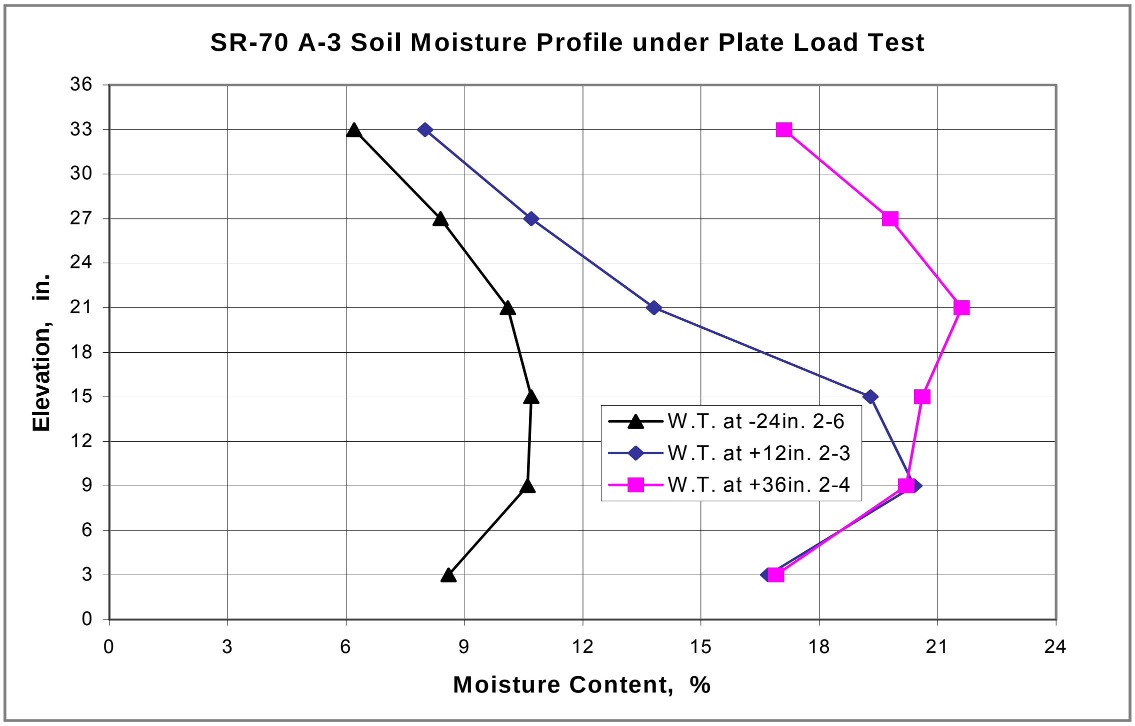


Figure 5.25(B) SR70 A-3 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

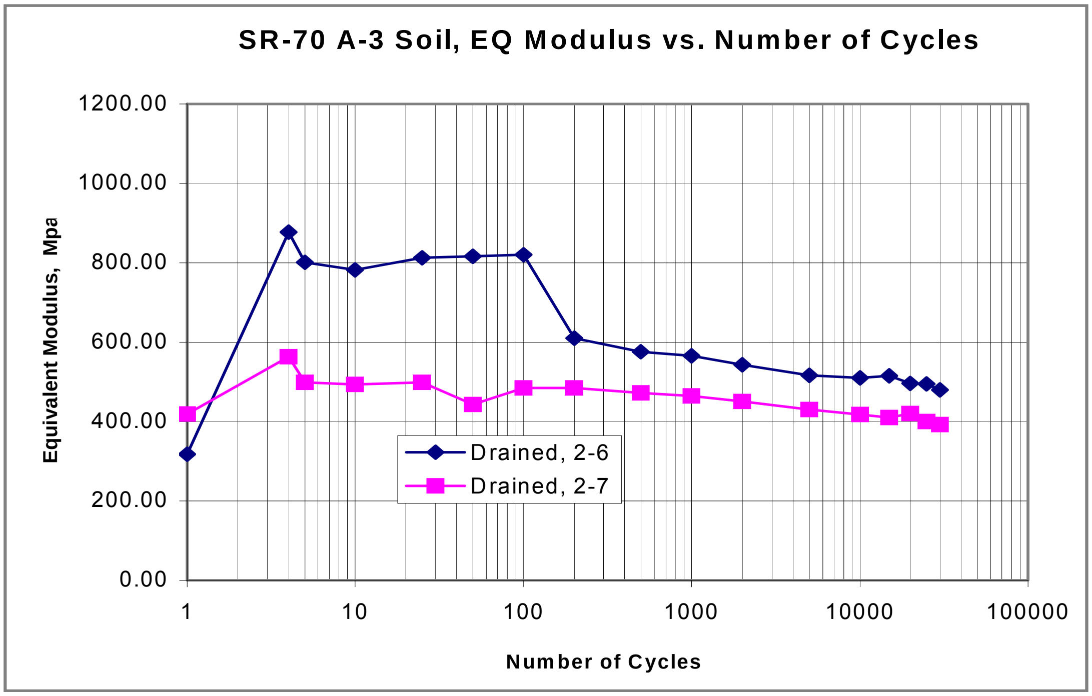


Figure 5.26(A) SR70 A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

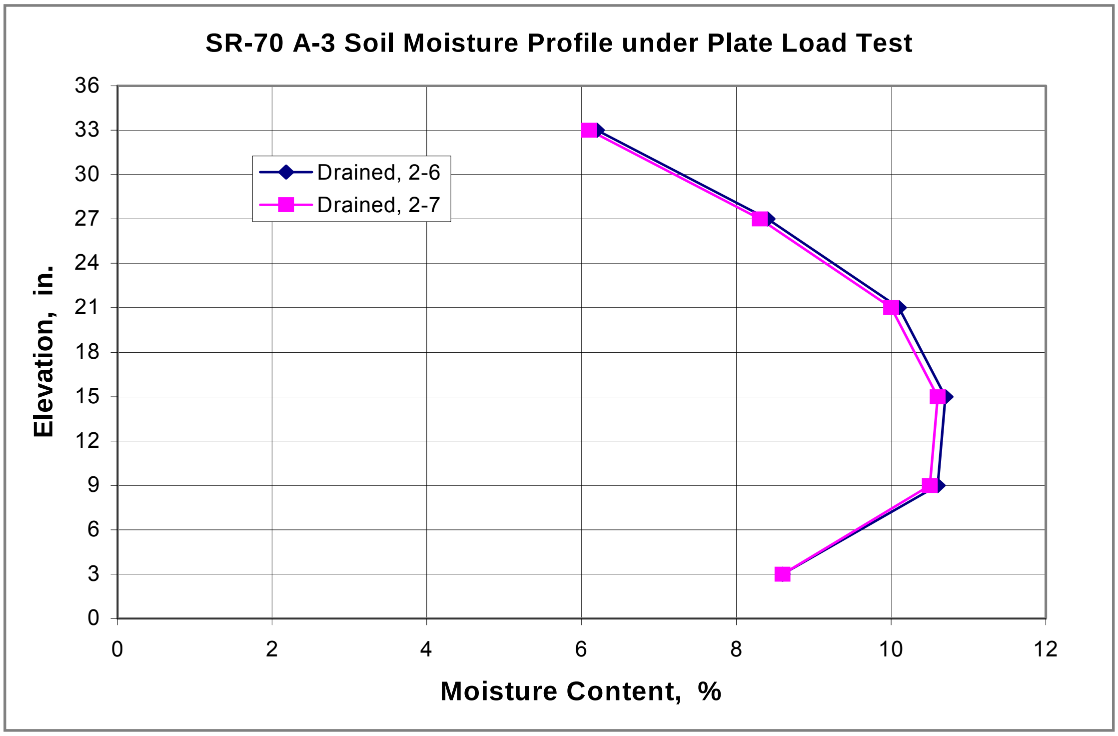


Figure 5.26(B) SR70 A-3 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

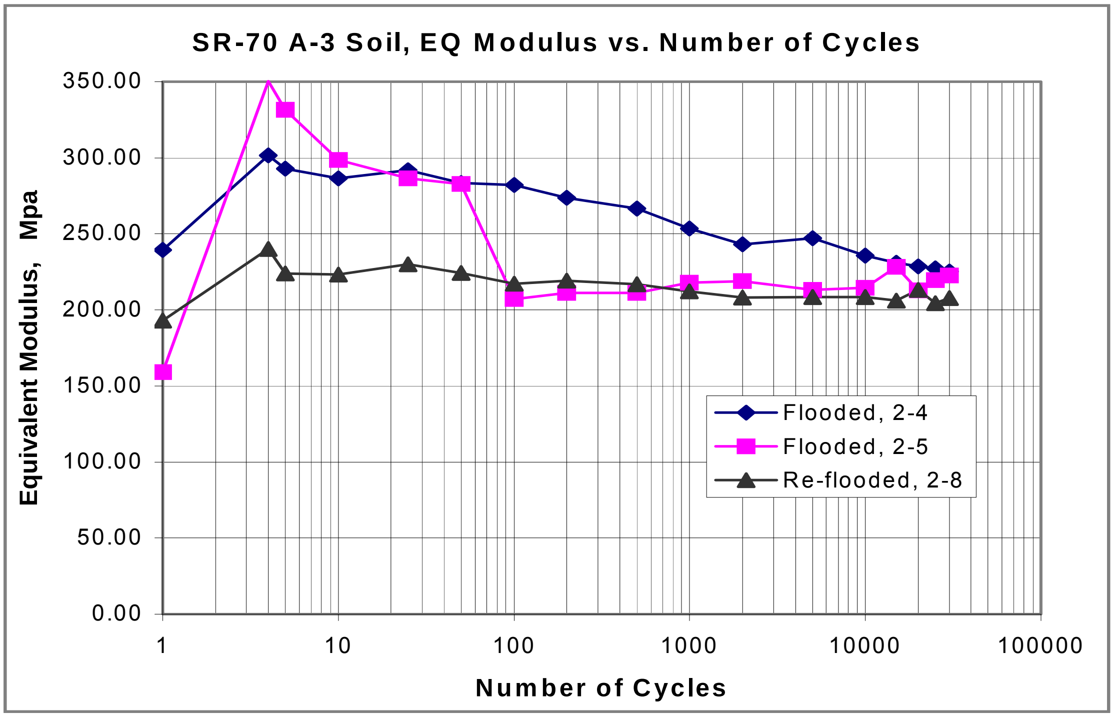


Figure 5.27(A) SR70 A-3 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

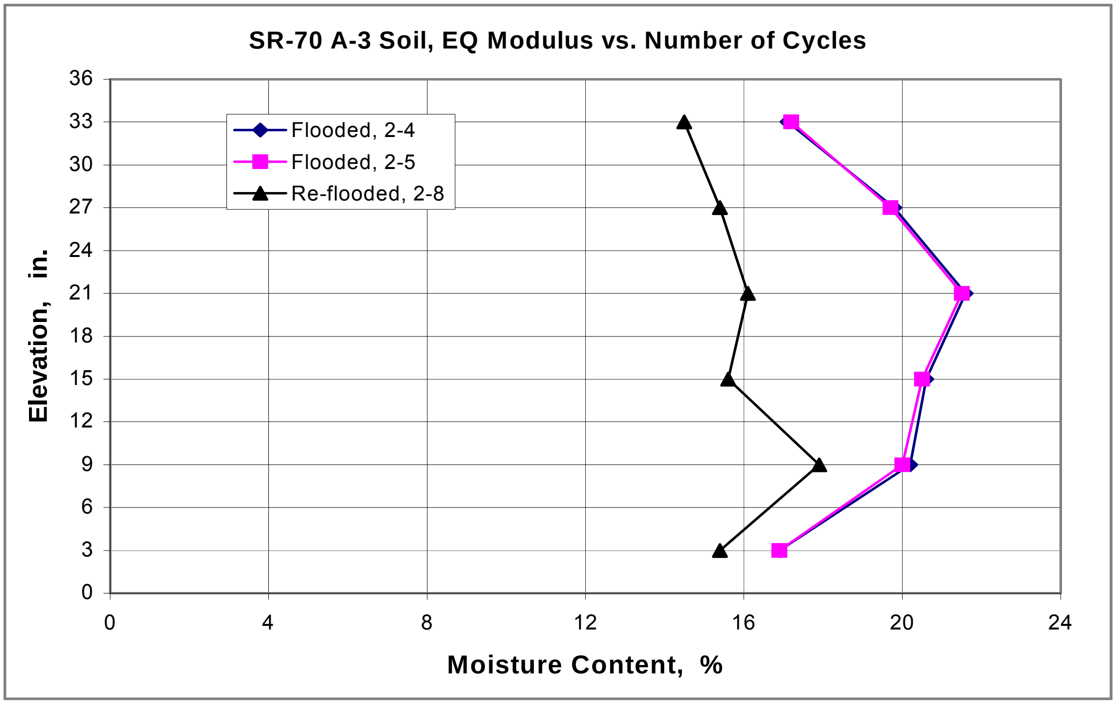


Figure 5.27(B) SR70 A-3 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

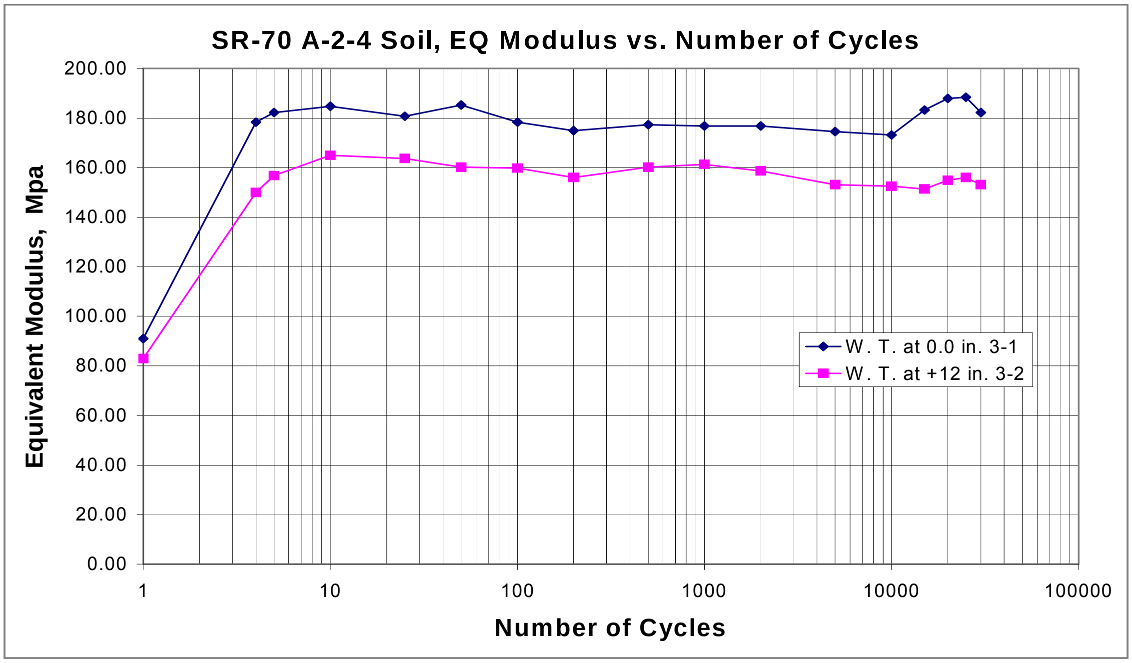


Figure 5.28(A) SR70 A-2-4 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

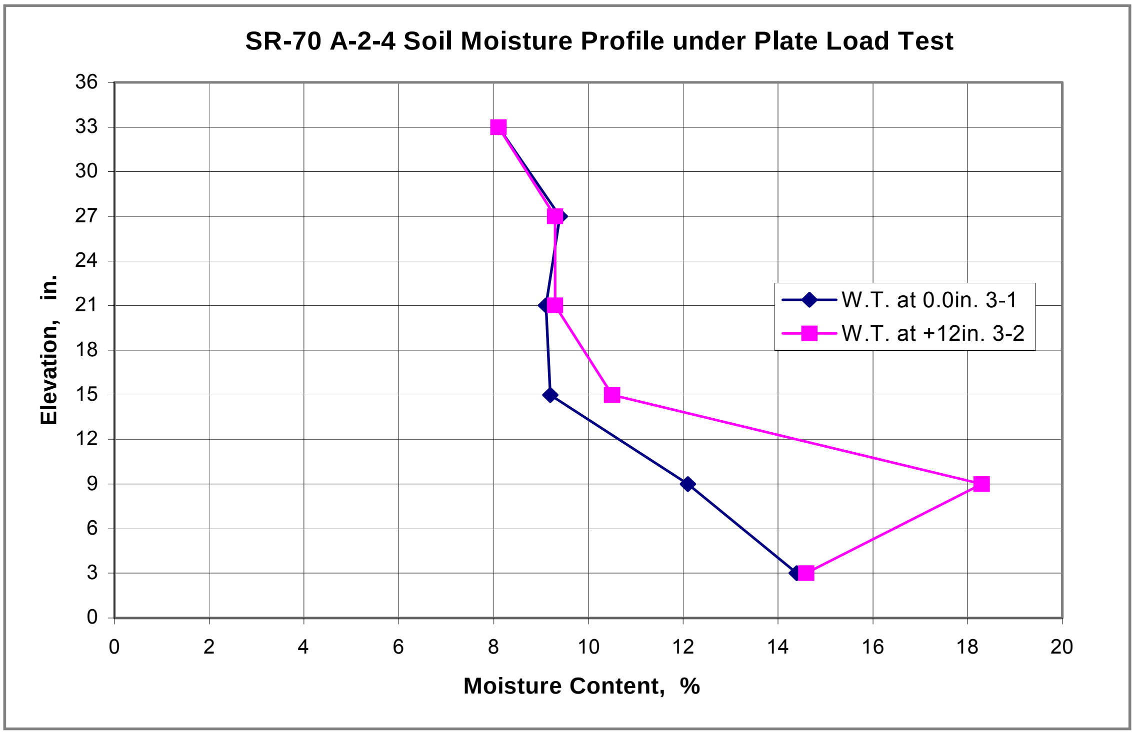


Figure 5.28(B) SR70 A-2-4 Soil Moisture Profile under Plate Load Test (20 psi without Limerock)

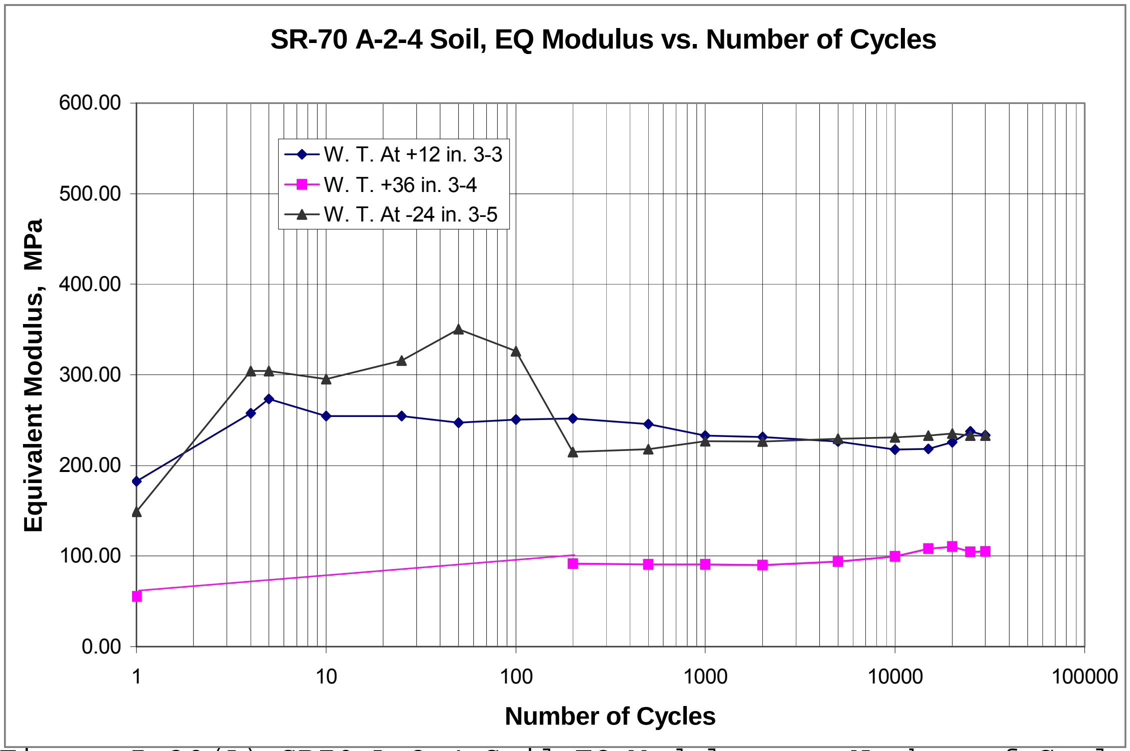


Figure 5.29(A) SR70 A-2-4 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

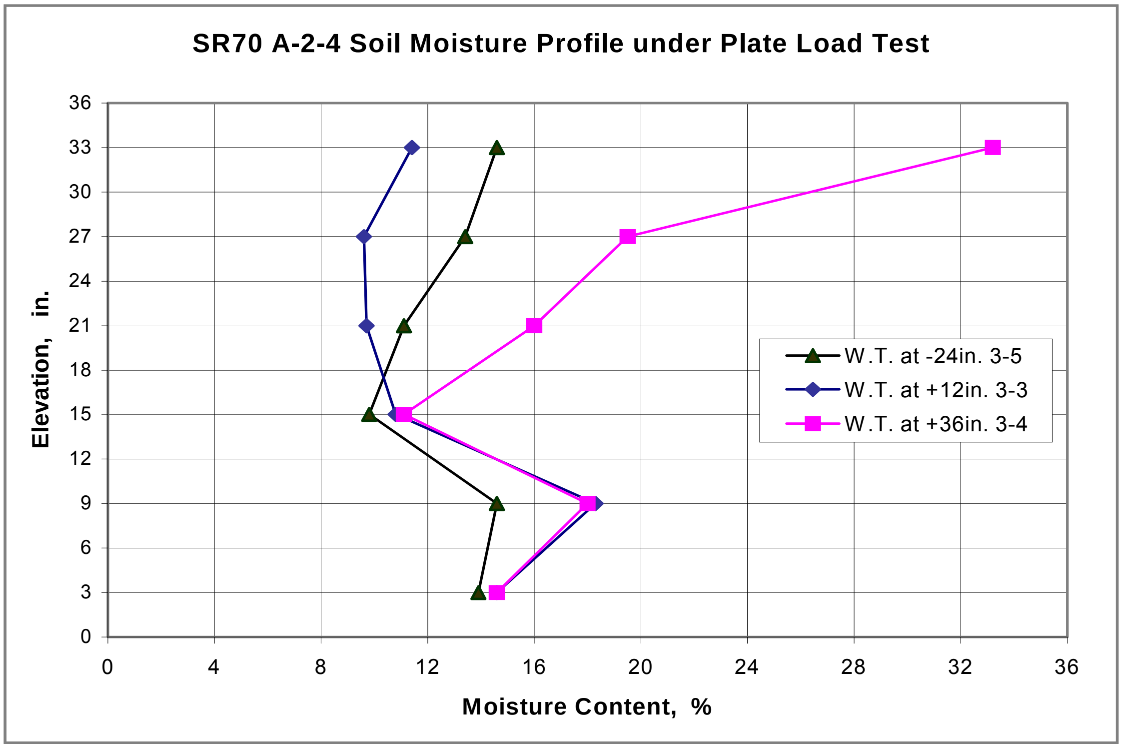


Figure 5.29(B) SR70 A-2-4 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

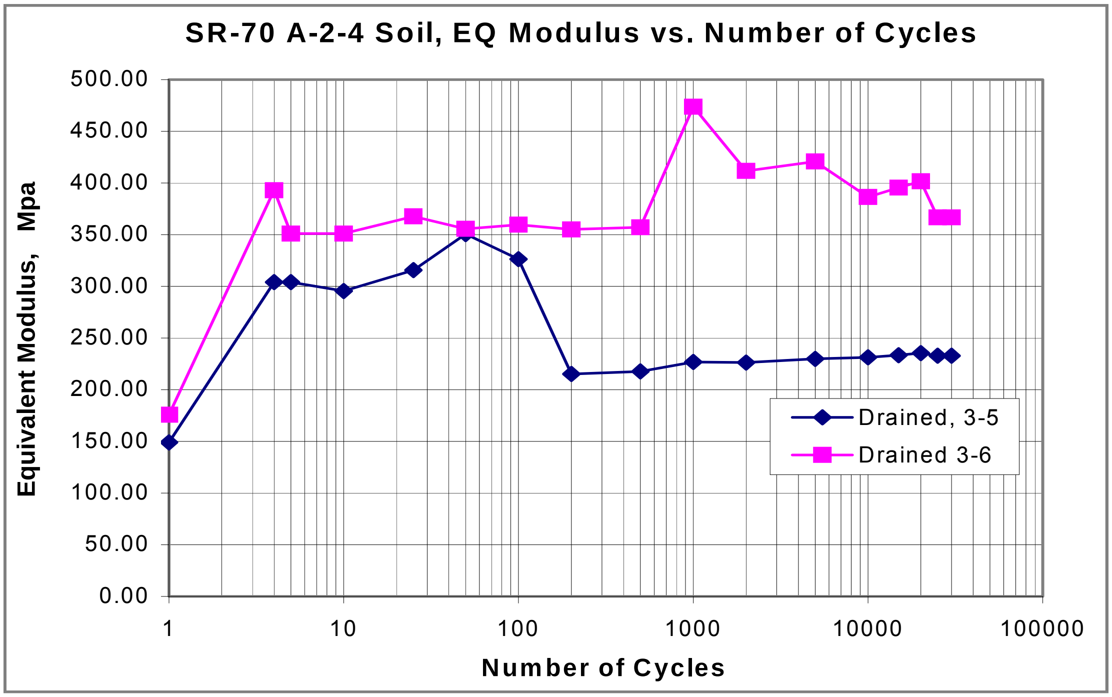


Figure 5.30(A) SR70 A-2-4 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

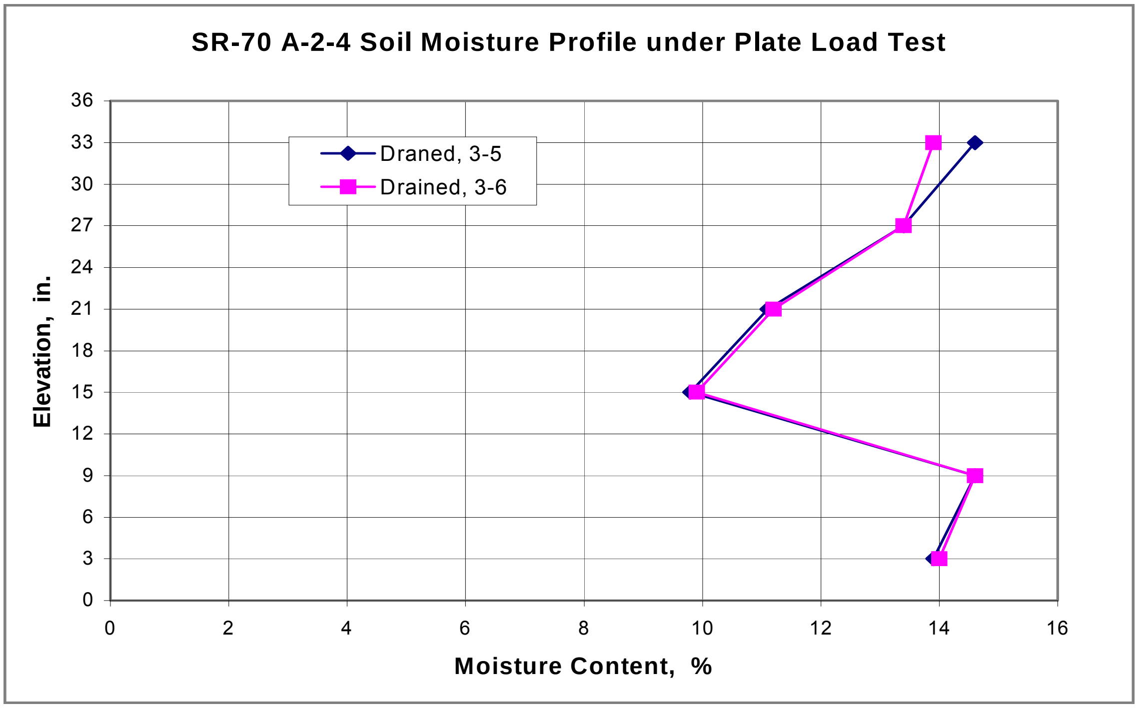


Figure 5.30(B) SR70 A-2-4 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

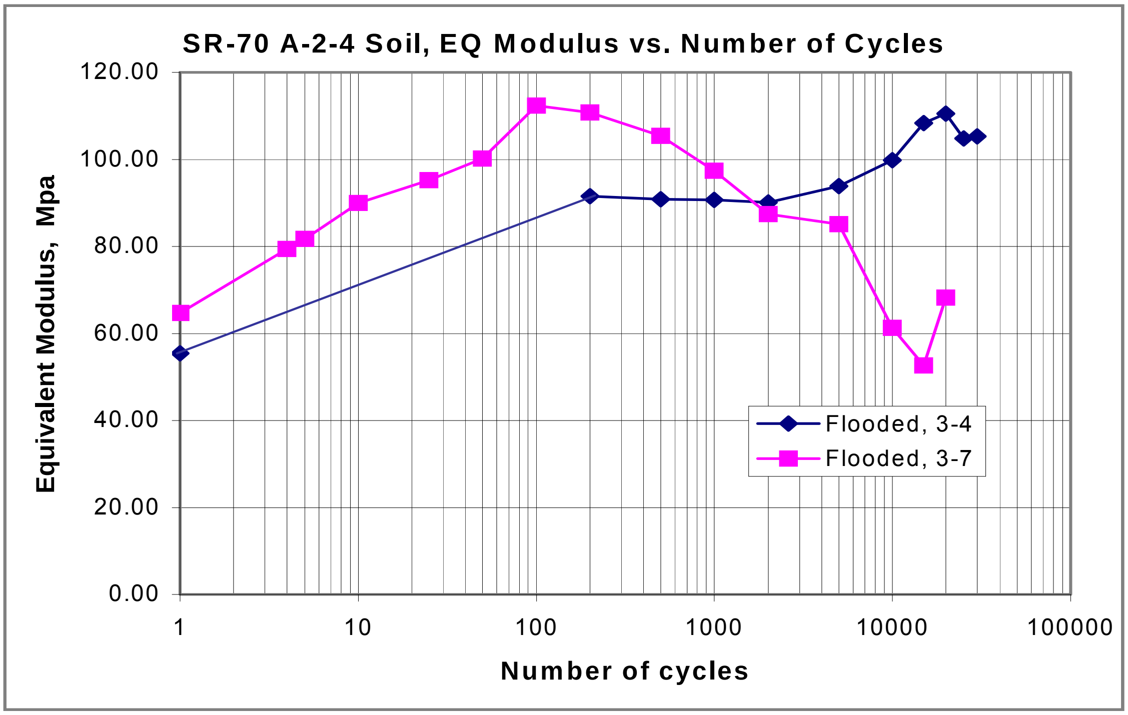


Figure 5.31(A) SR70 A-2-4 Soil EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

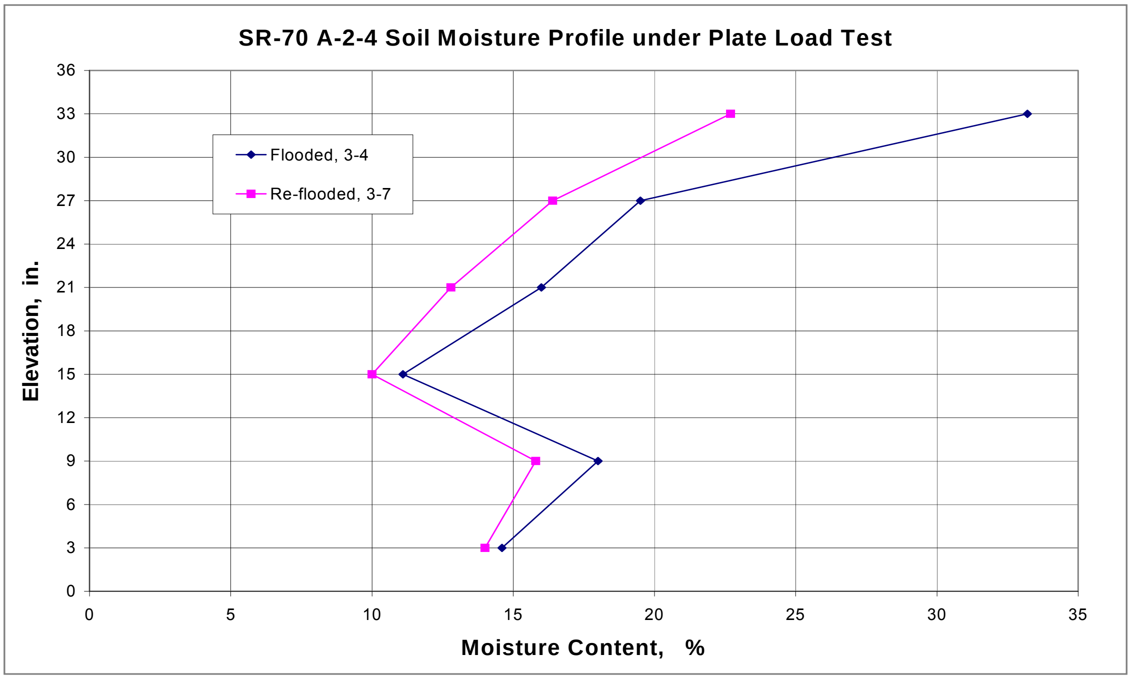


Figure 5.31(B) SR70 A-2-4 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

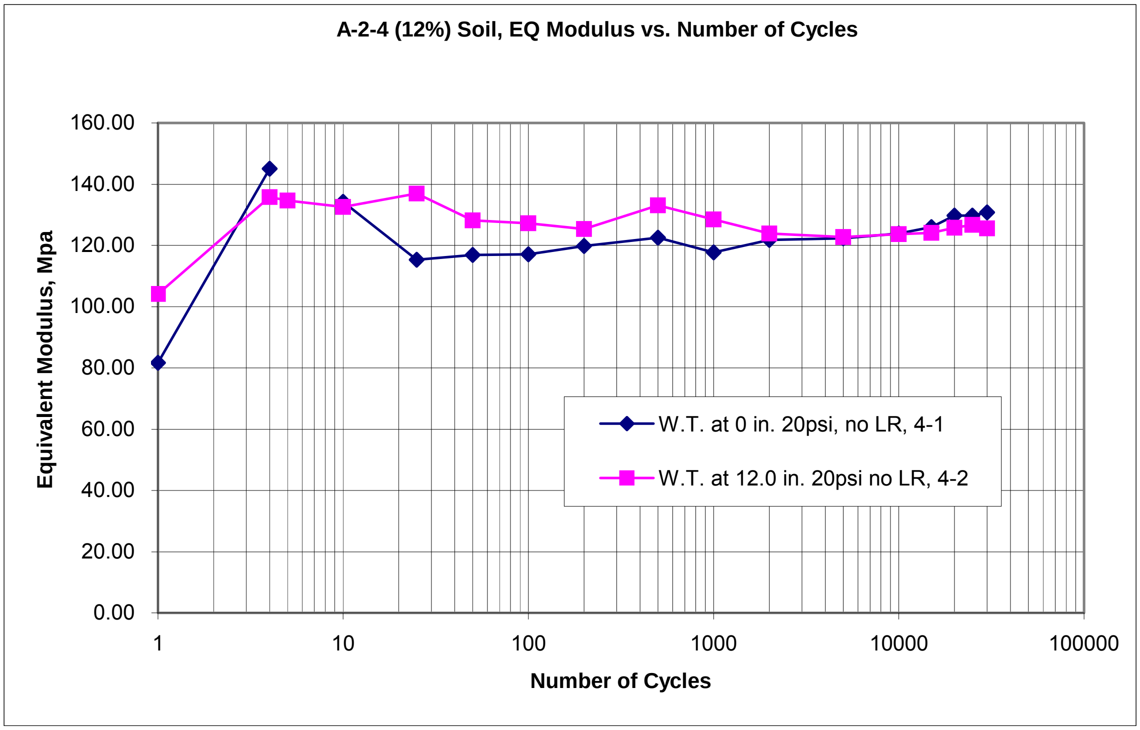


Figure 5.32(A) A-2-4 (12%) EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

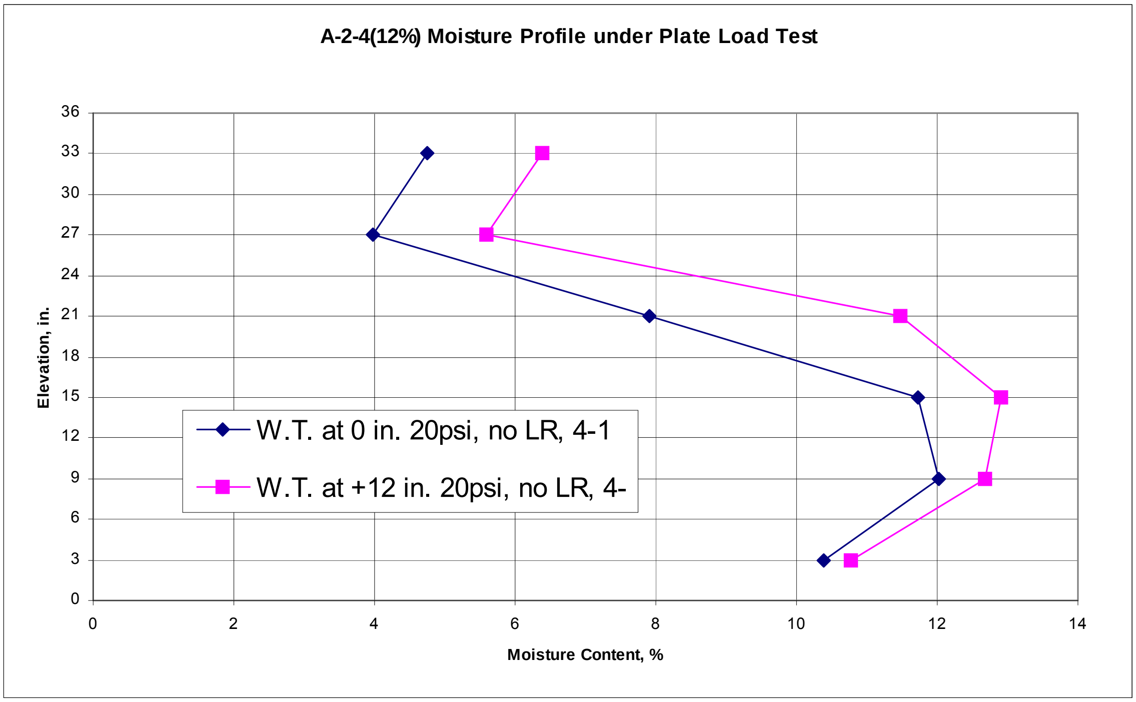


Figure 5.32(B) A-2-4 (12%) Moisture Profile under Plate Load Test (20 psi without Limerock)

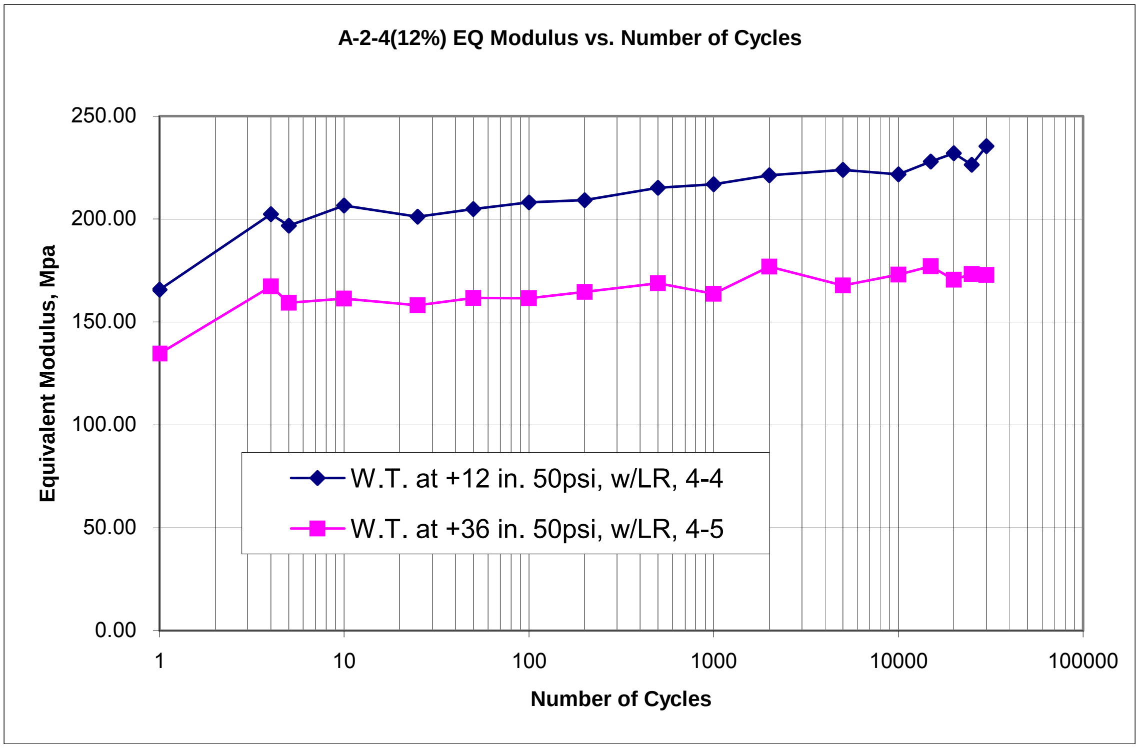


Figure 5.33(A) A-2-4 (12%) EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

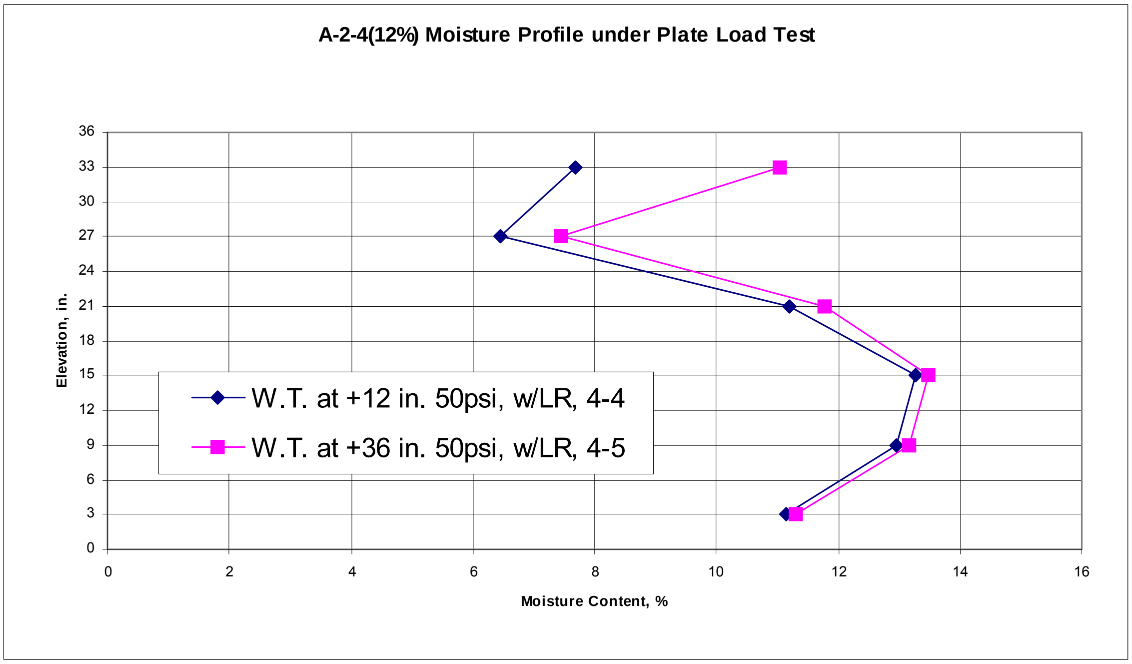


Figure 5.33(B) A-2-4 (12%) Moisture Profile under Plate Load Test (50 psi with Limerock)

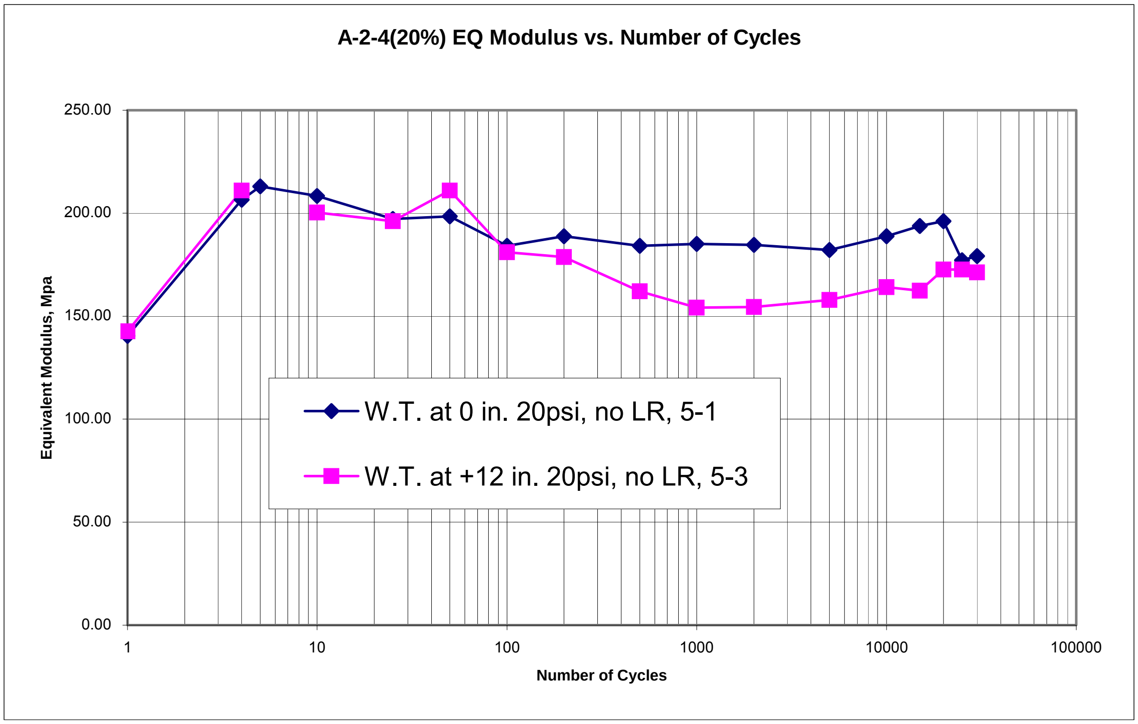


Figure 5.34(A) A-2-4 (20%) EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

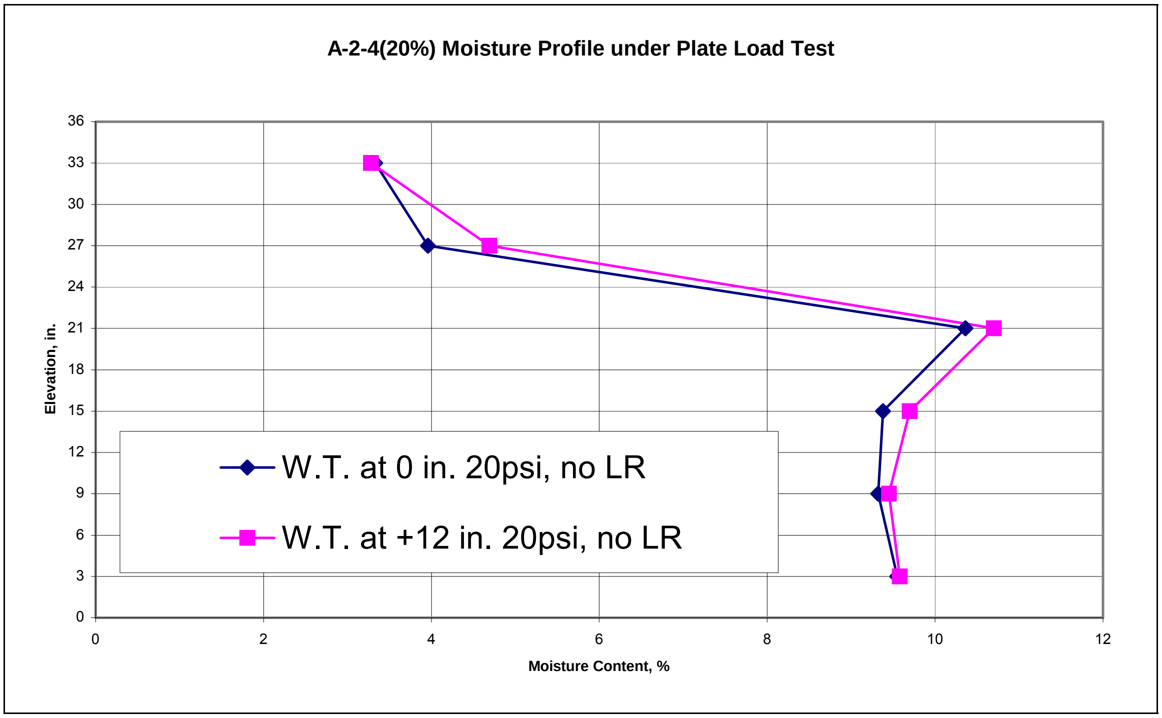


Figure 5.34(B) A-2-4 (20%) Moisture Profile under Plate Load Test (20 psi without Limerock)

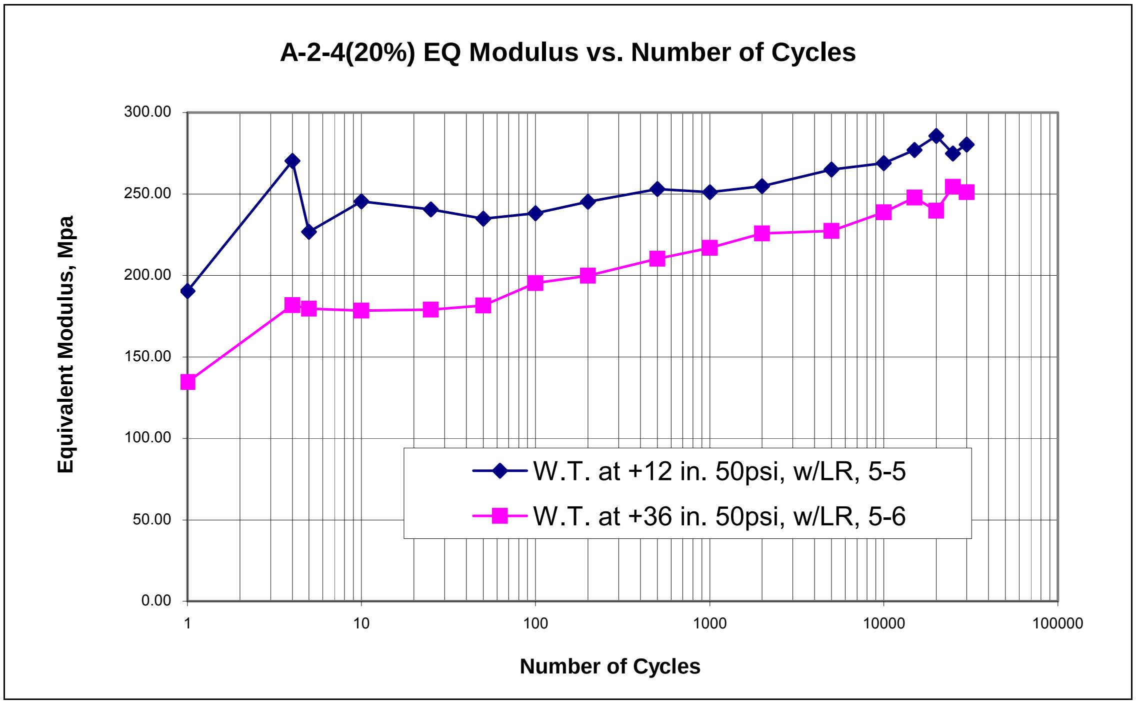


Figure 5.35(A) A-2-4 (20%) EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

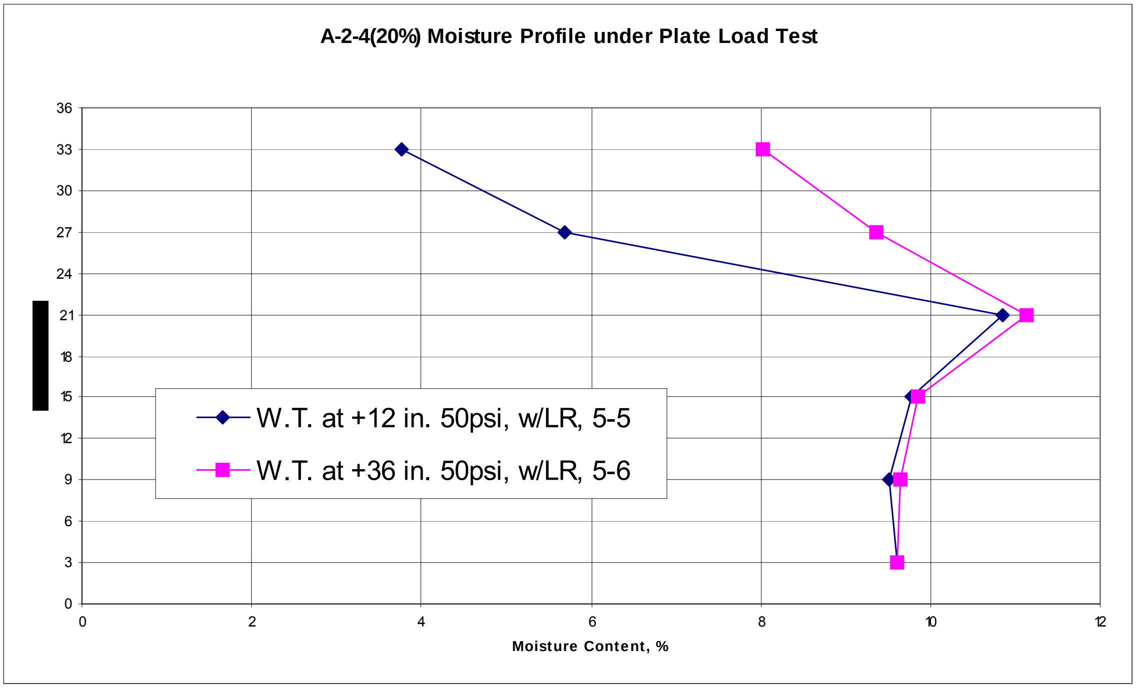


Figure 5.35(B) A-2-4 (20%) Moisture Profile under Plate Load Test (50 psi with Limerock)

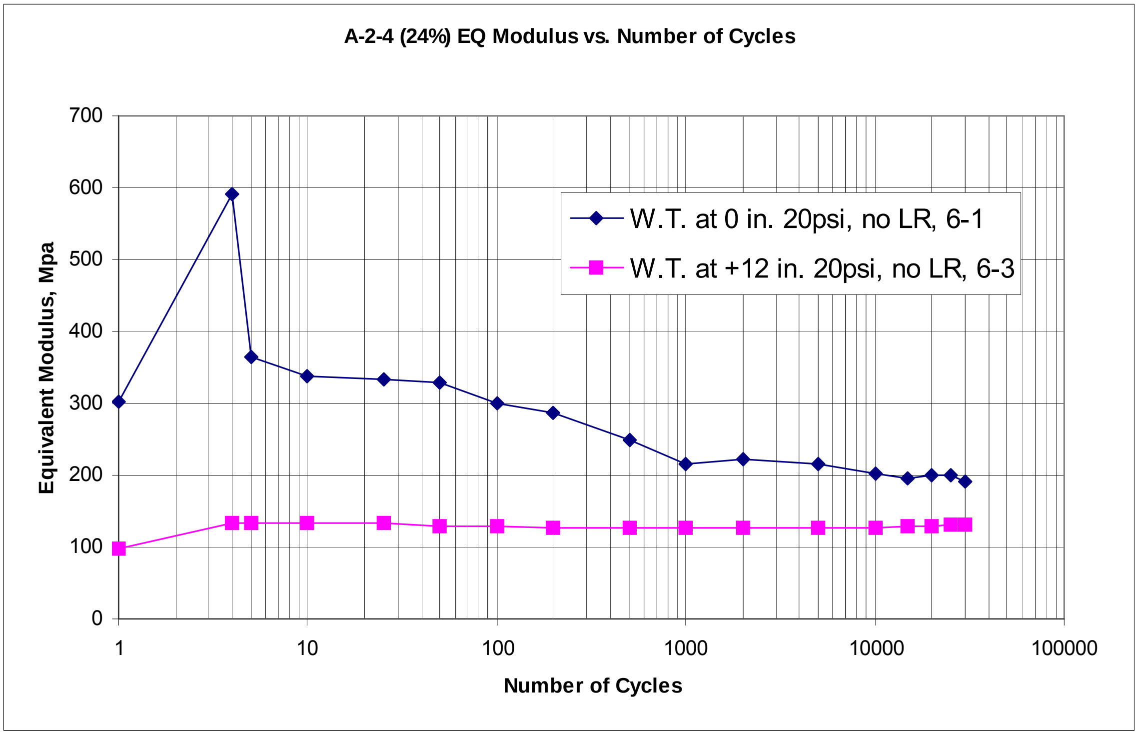


Figure 5.36(A) A-2-4 (24%) EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

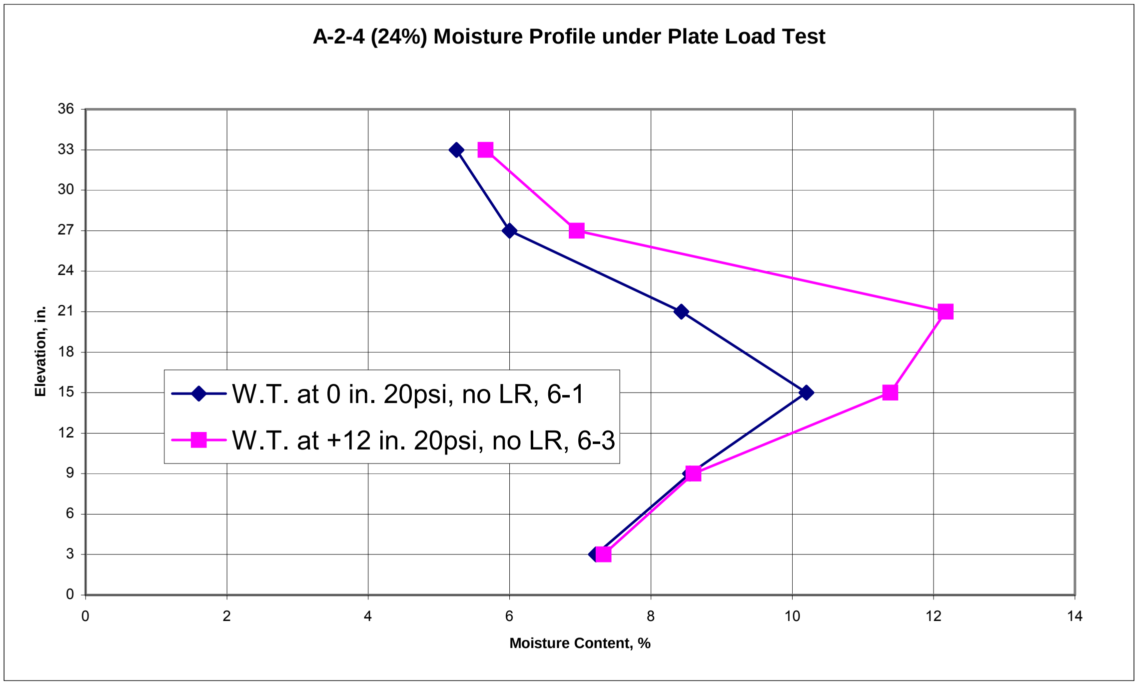


Figure 5.36(B) A-2-4 (24%) Moisture Profile under Plate Load Test (20 psi without Limerock)

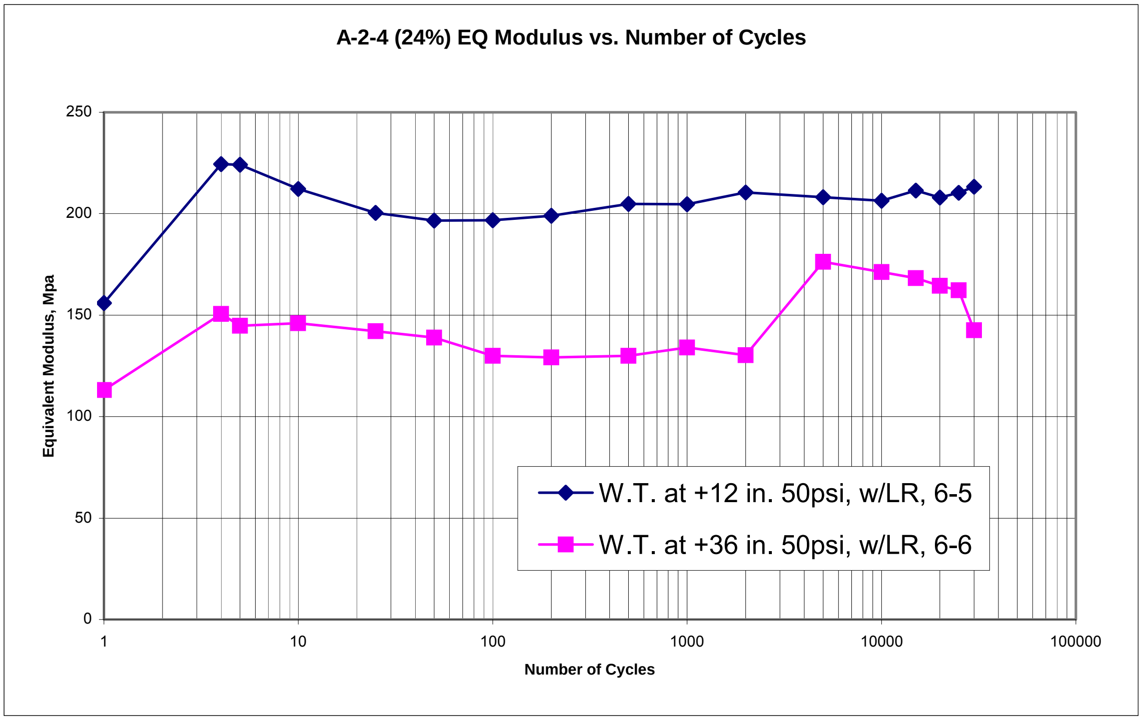


Figure 5.37(A) A-2-4 (24%) EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

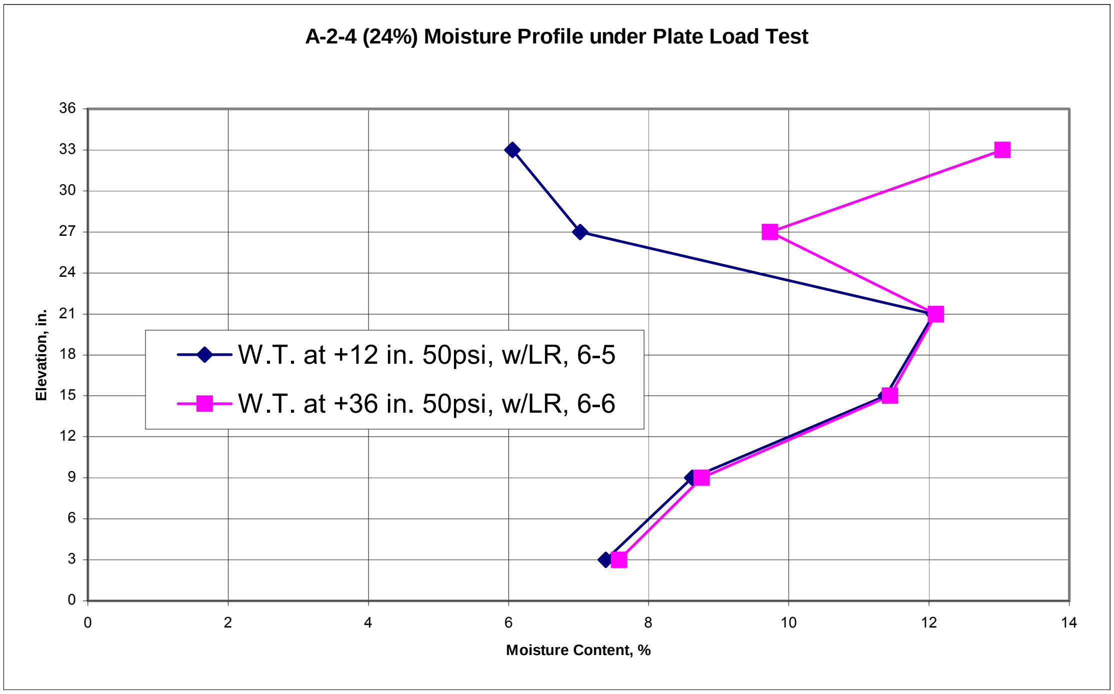


Figure 5.37(B) A-2-4 (24%) Moisture Profile under Plate Load Test (50 psi with Limerock)

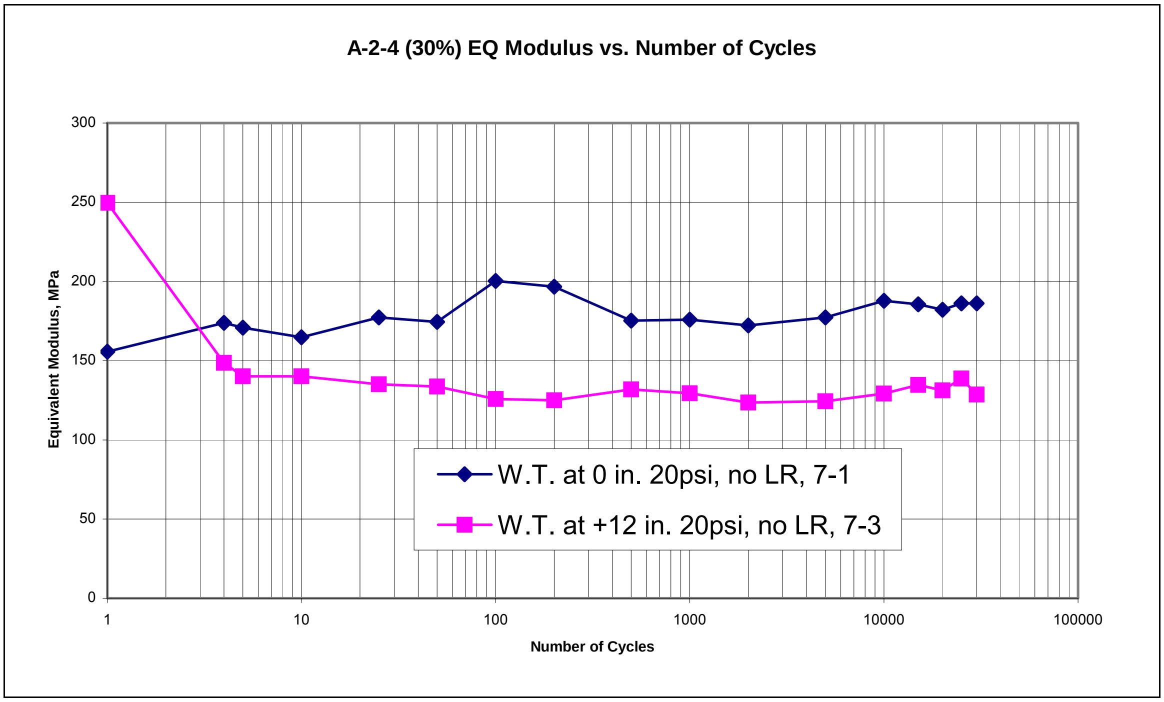


Figure 5.38(A) A-2-4 (30%) EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

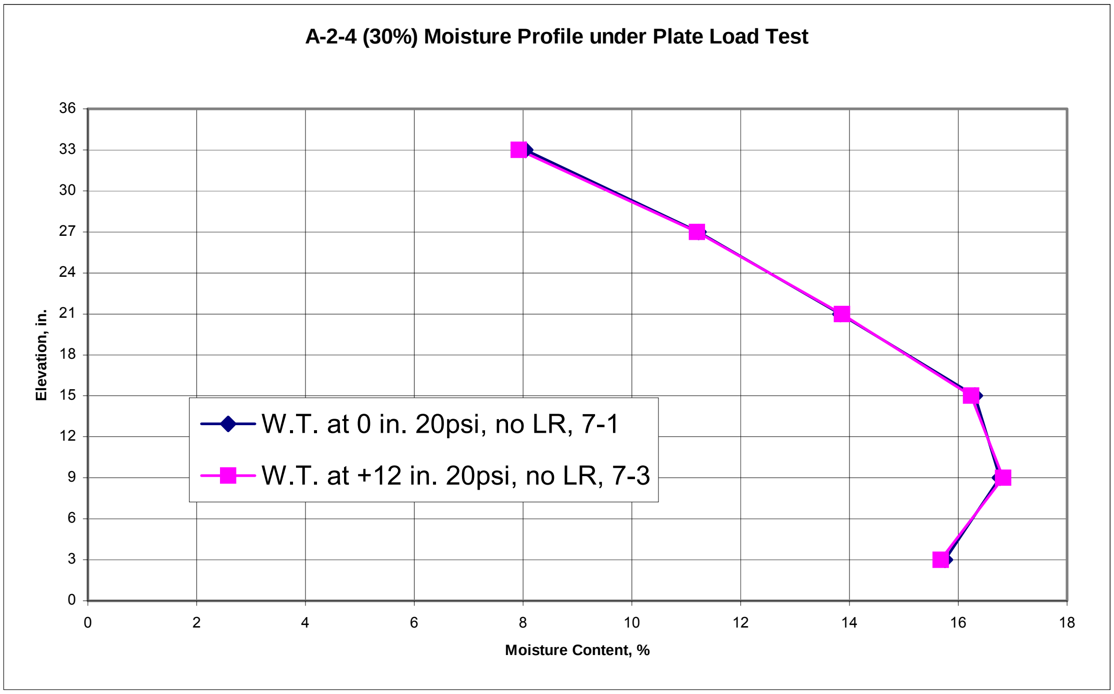


Figure 5.38(B) A-2-4 (30%) Moisture Profile under Plate Load Test (20 psi without Limerock)

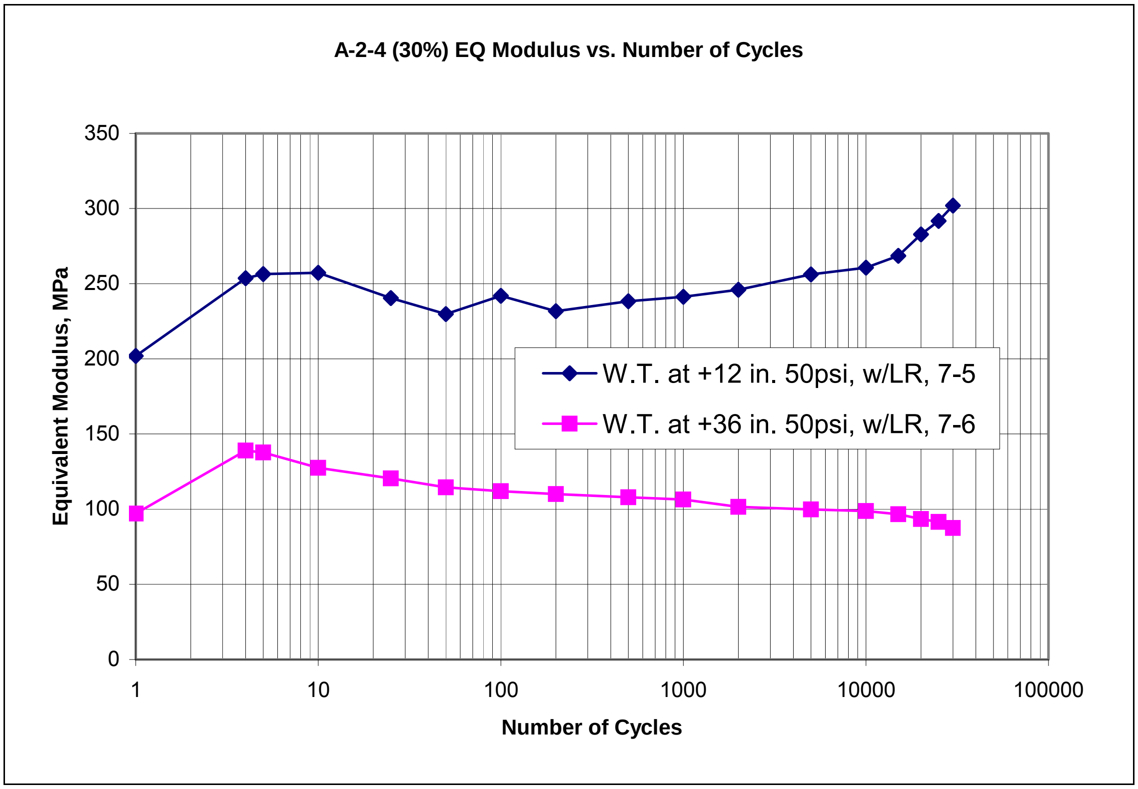


Figure 5.39(A) A-2-4 (30%) EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

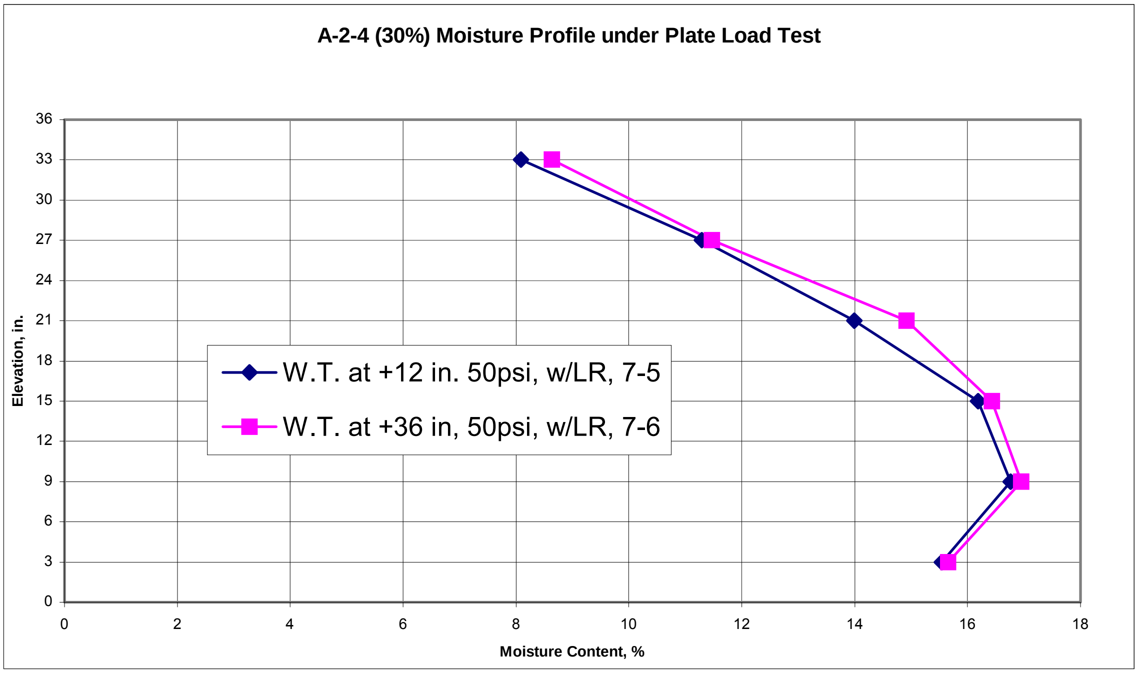


Figure 5.39(B) A-2-4 (30%) Moisture Profile under Plate Load Test (50 psi with Limerock)

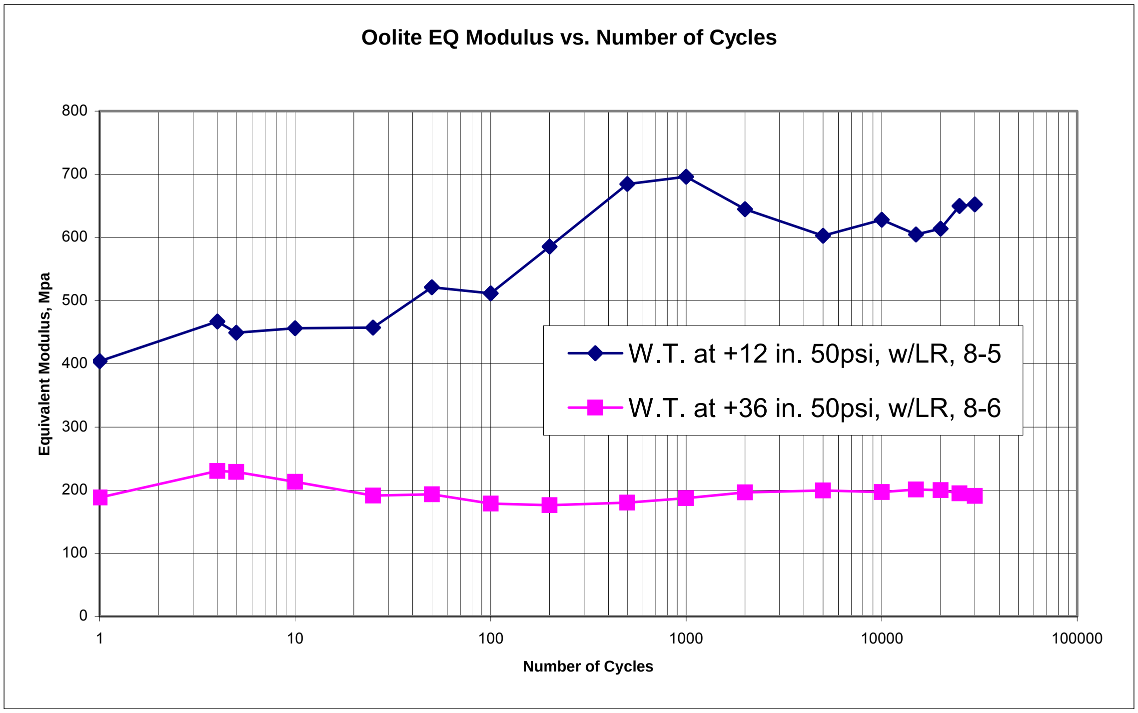


Figure 5.40(A) Oolite EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

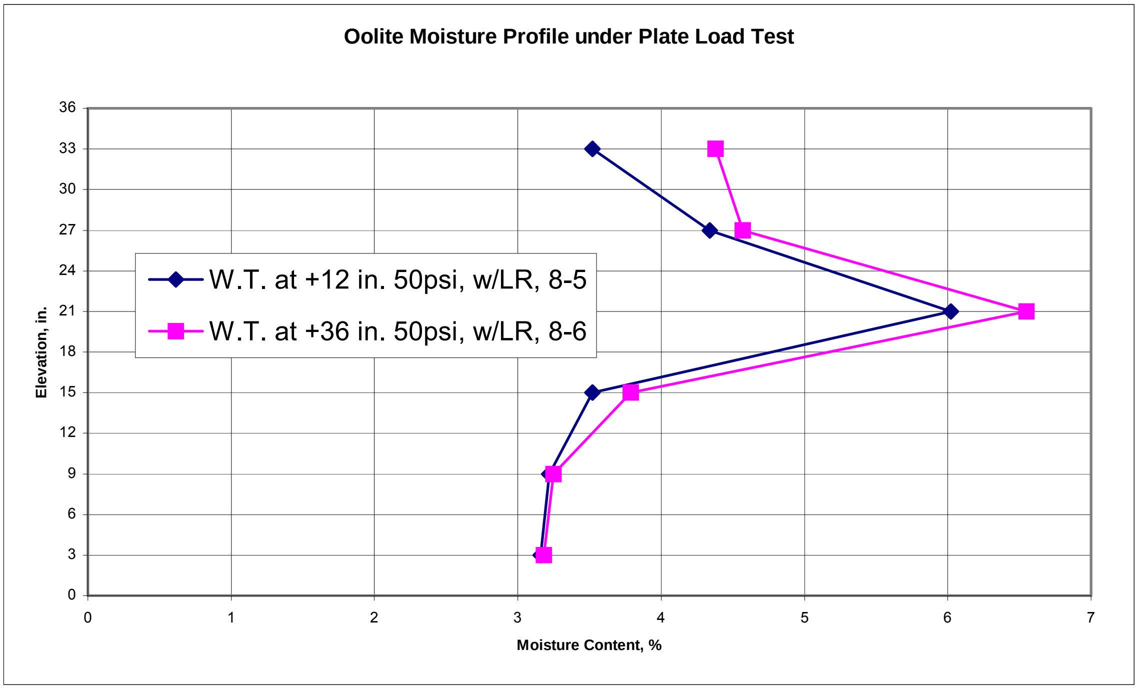


Figure 5.40(B) Miami Oolite A-1 Soil Moisture Profile under Plate Load Test (50 psi with Limerock)

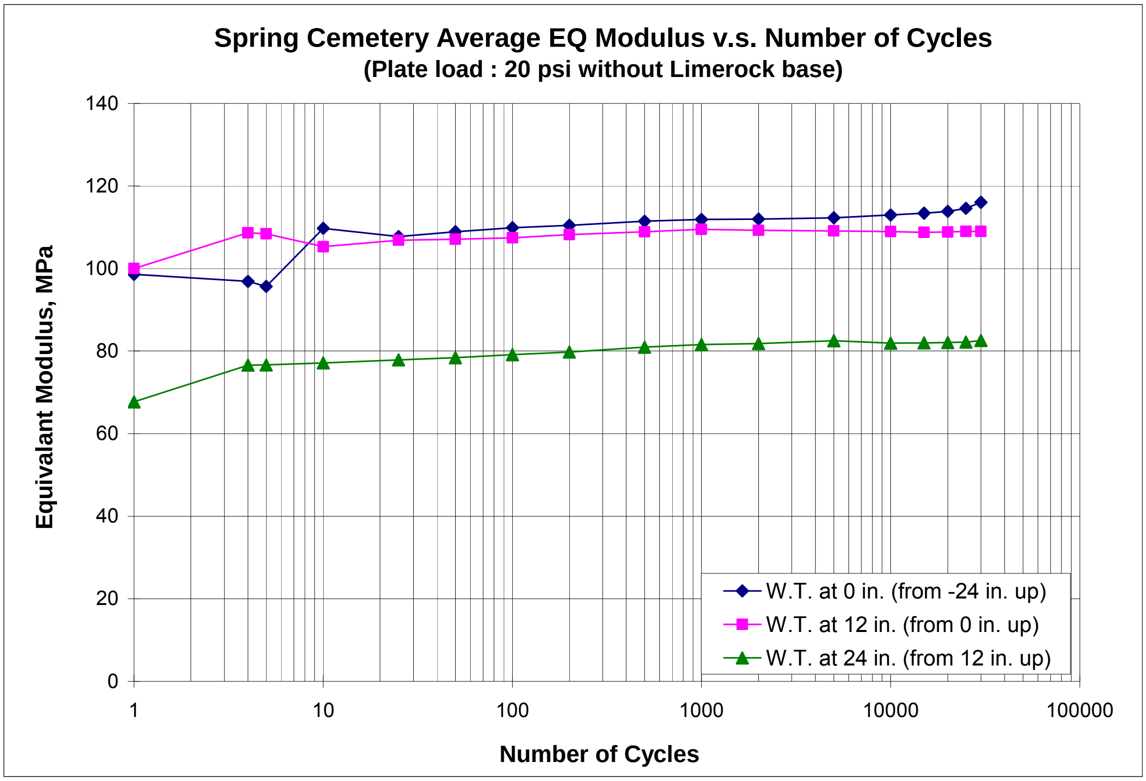


Figure 5.41(A) Spring Cemetery EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

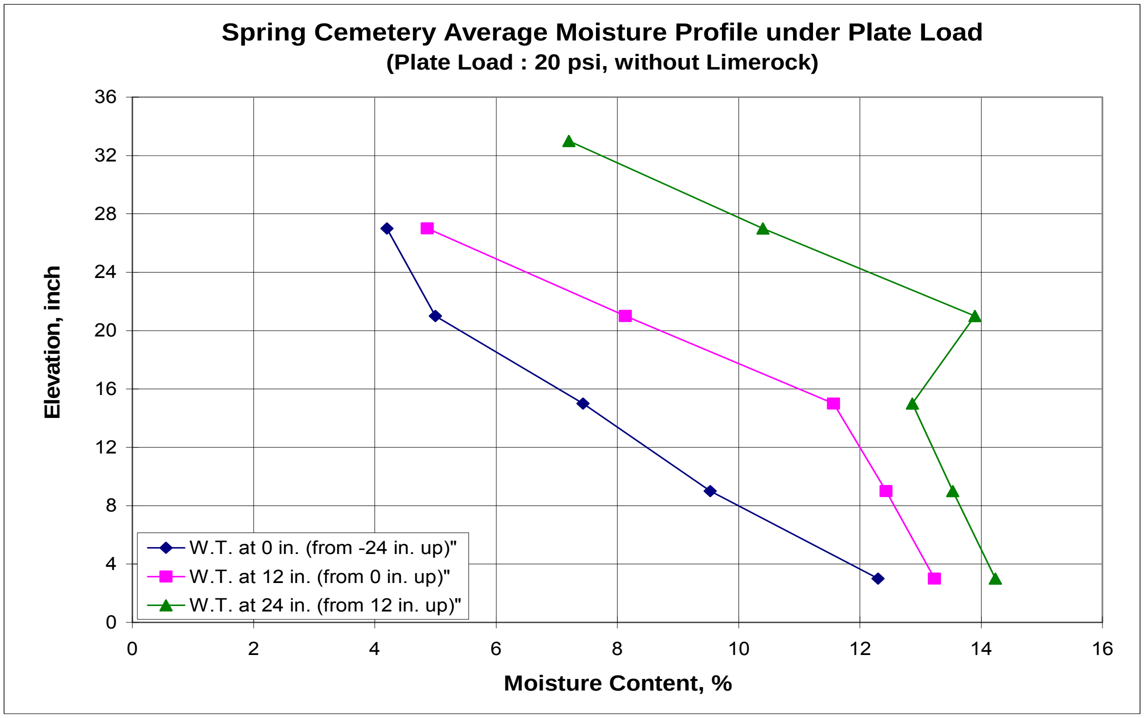


Figure 5.41(B) Spring Cemetery Moisture Profile under Plate Load Test (20 psi without Limerock)

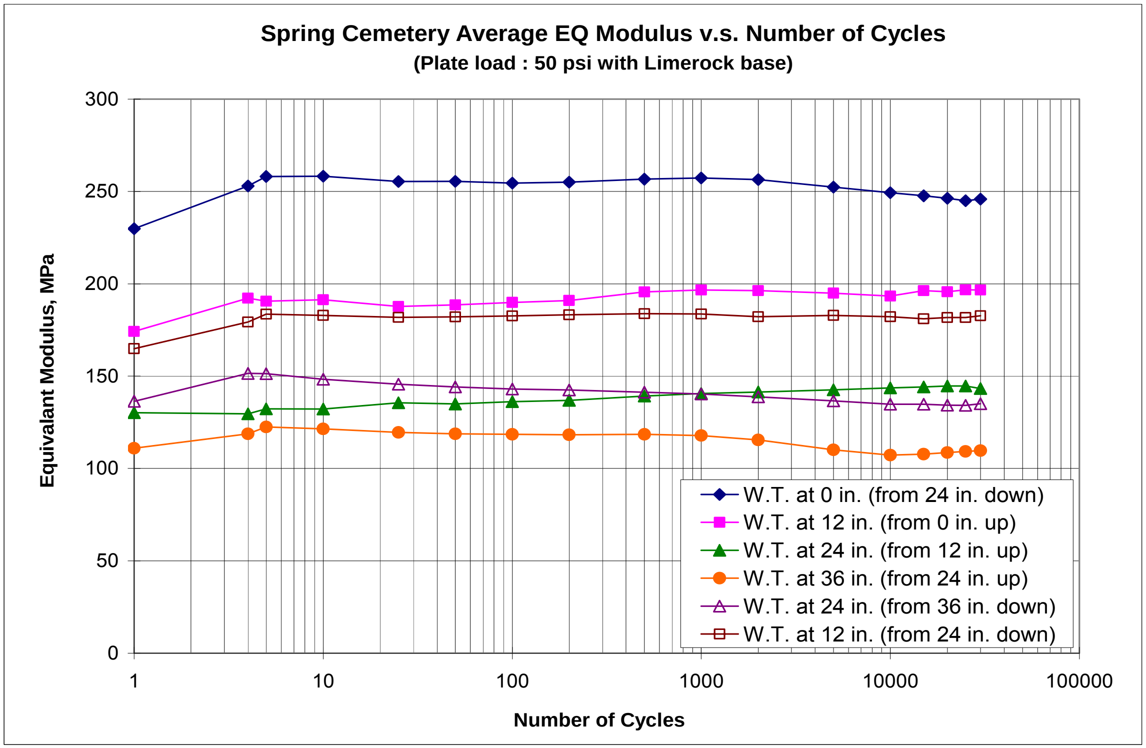


Figure 5.42(A) Spring Cemetery EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

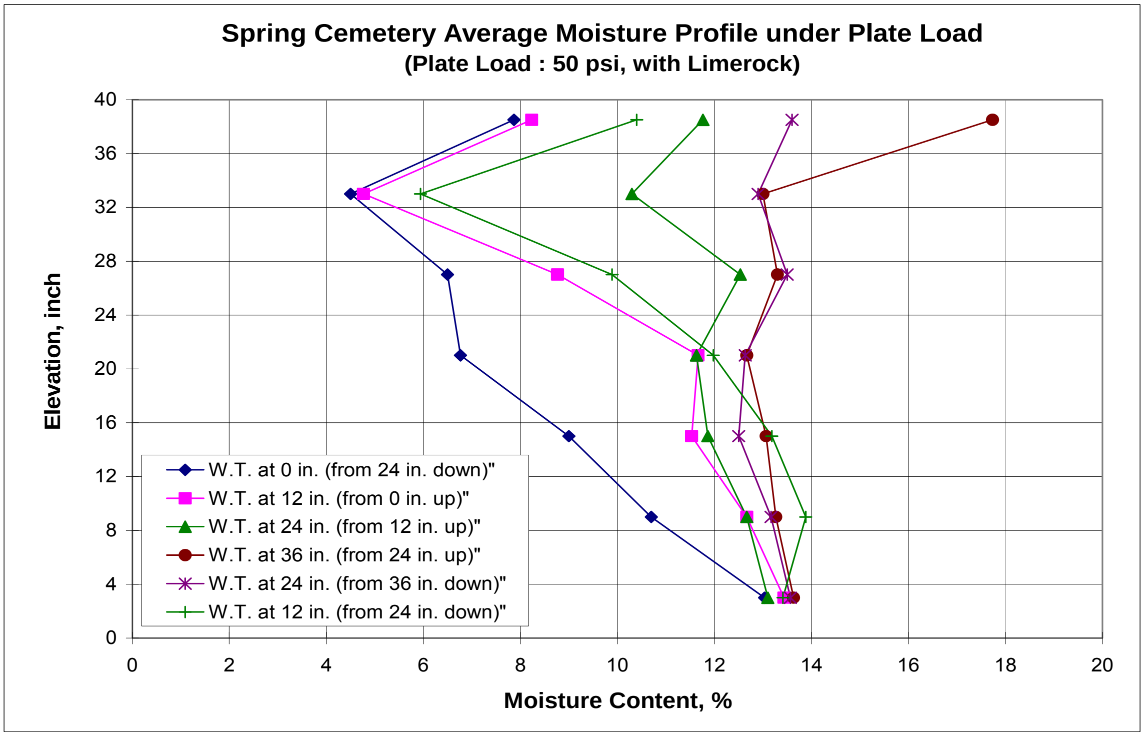


Figure 5.42(B) Spring Cemetery Moisture Profile under Plate Load Test (50 psi with Limerock)

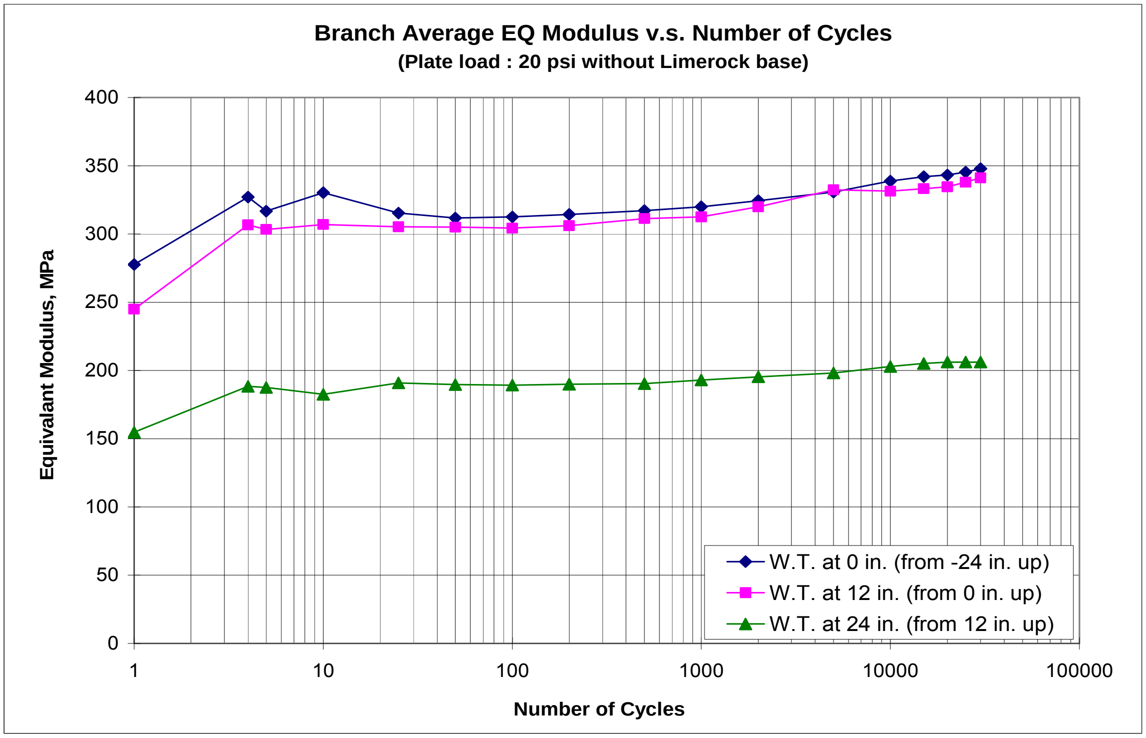


Figure 5.43(A) Branch EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

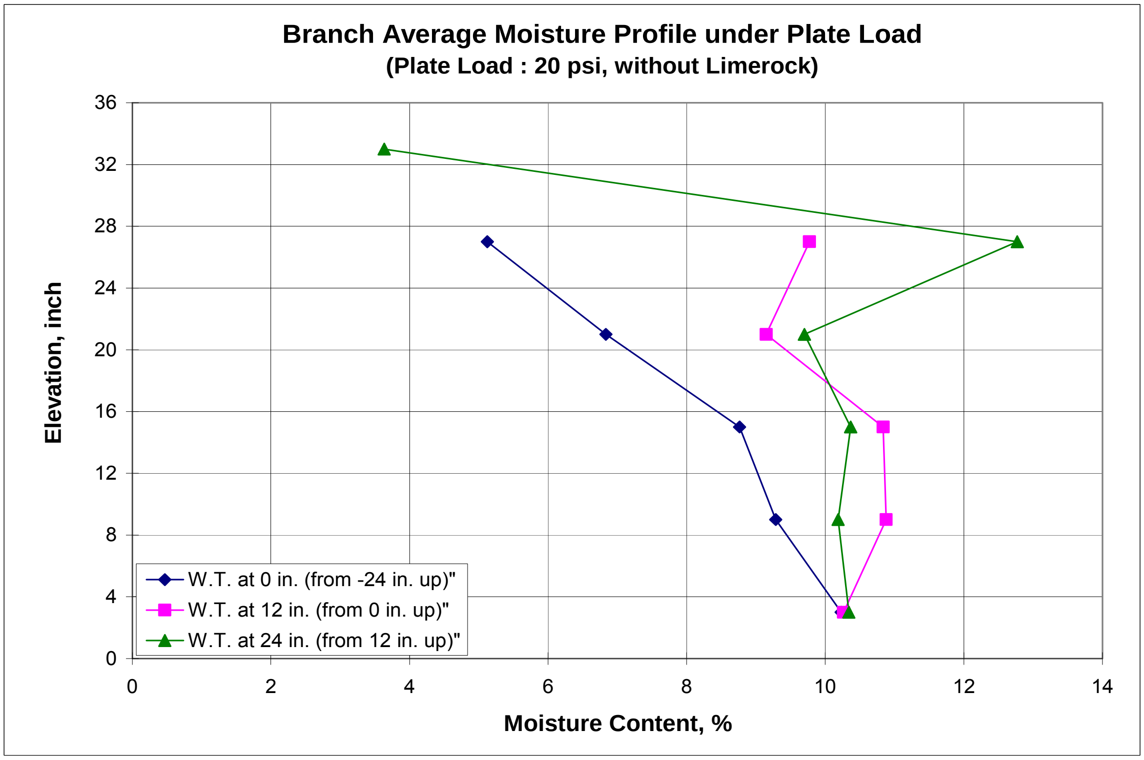


Figure 5.43(B) Branch Moisture Profile under Plate Load Test (20 psi without Limerock)

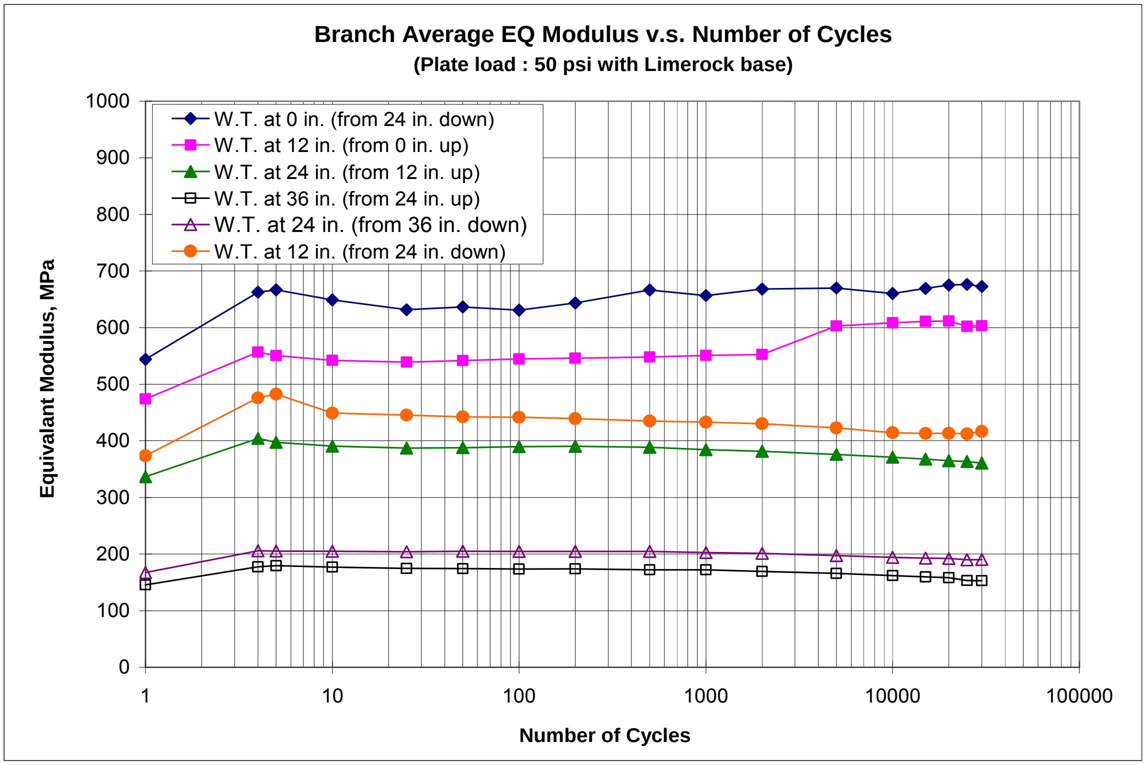


Figure 5.44(A) Branch EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

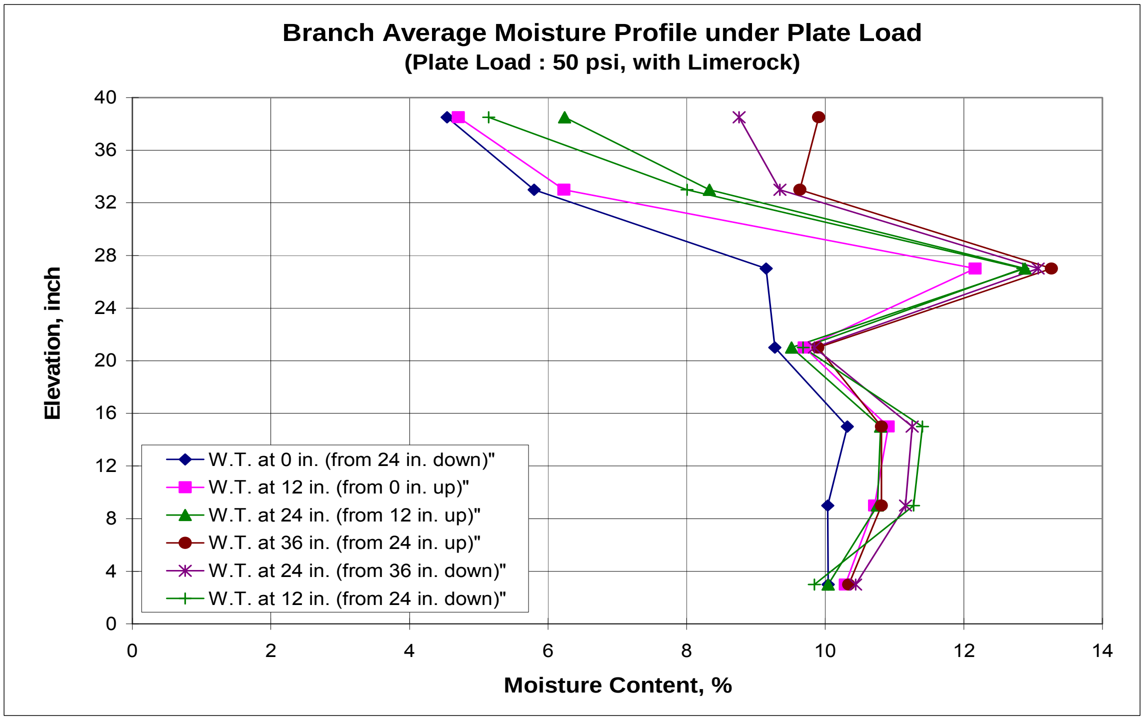


Figure 5.44(B) Branch Moisture Profile under Plate Load Test (50 psi with Limerock)

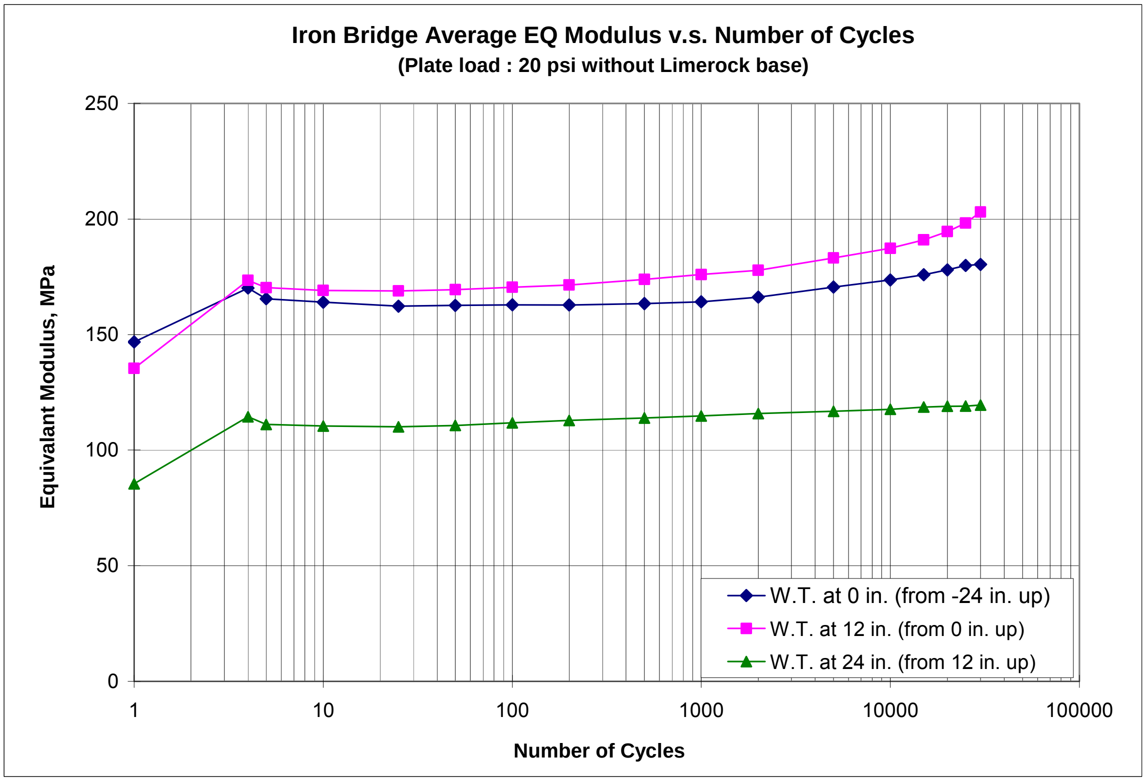


Figure 5.45(A) Iron Bridge EQ Modulus vs. Number of Cycles under Different Water Tables (20 psi without Limerock)

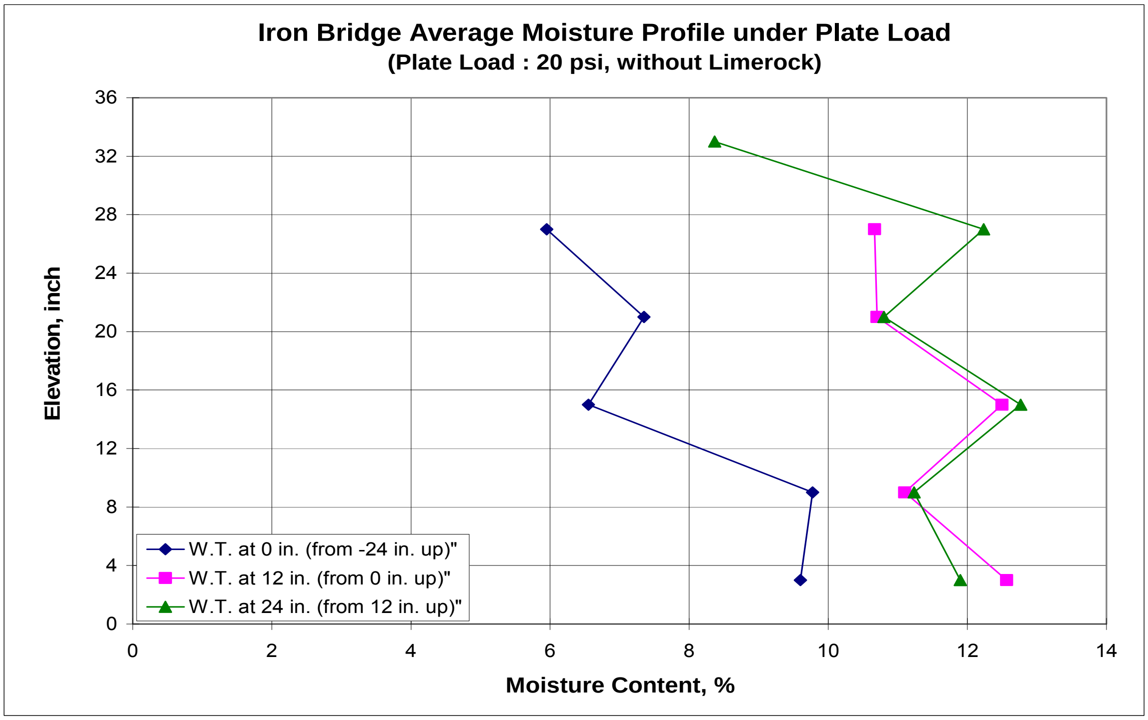


Figure 5.45(B) Iron Bridge Moisture Profile under Plate Load Test (20 psi without Limerock)

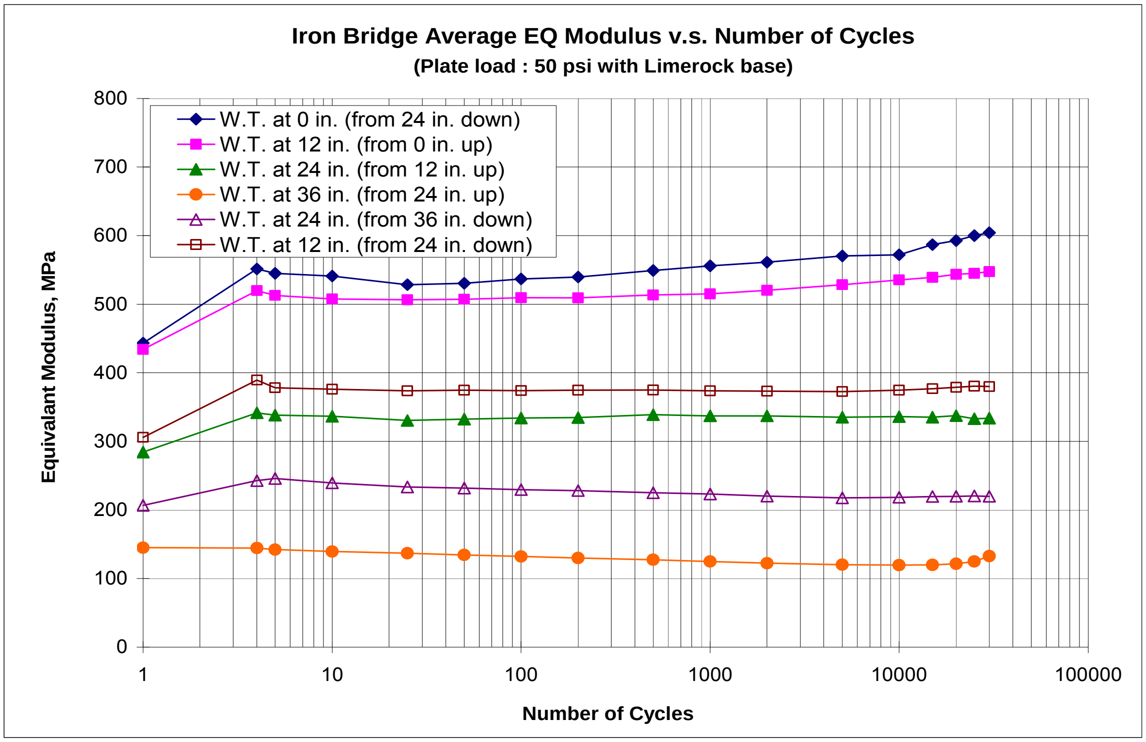


Figure 5.46(A) Iron Bridge EQ Modulus vs. Number of Cycles under Different Water Tables (50 psi with Limerock)

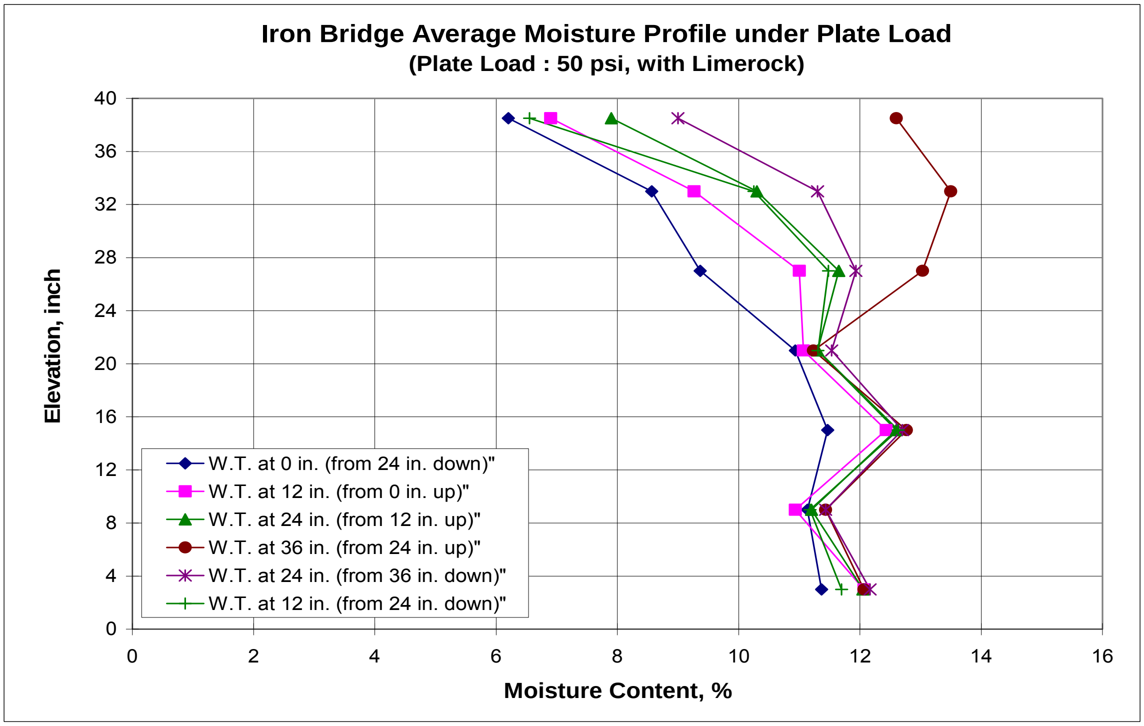


Figure 5.46(B) Iron Bridge Moisture Profile under Plate Load Test (50 psi with Limerock)

CHAPTER 6
ANALYSIS OF LABORATORY TEST RESULTS

6.1 LABORATORY RESILIENT MODULUS

The results of the laboratory tests are further analyzed in this chapter. A significant difference existed between the resilient modulus values computed from deformations measured by middle-half LVDTs and full-length LVDTs. The discrepancy was mainly caused by the end effect and friction. Previous research from Hoang (1996) concluded that, for the T292-91I test procedure, the average ratio of the resilient modulus values between the middle-half and full-length LVDT position measurement ranged from about 1.3 at lower confining pressures to about 1.15 at higher confining pressures. Zhang (2004) indicated that the resilient modulus values measured by using the full-length LVDTs were not representative of the actual resilient modulus due to end effect caused by uneven contact between the end platens and specimen. Therefore, the original analysis in this report was mainly based on the data from the middle-half LVDT position measurement. However, since three additional materials were tested only using T307-99 with the full-length LVDT position measurement, the resilient modulus data from the full-length

LVDTs were used when the analysis was made comparing all eleven subgrade materials. Both the middle-half and full-length measurement data were presented in pairs in order to provide a complete comparative analysis.

It should be noted that the Phase III soils were not included due to a lack of test data at the dry and soaked conditions. Among these soils, the A-2-4 (30%) soil was reconstituted from other soils. The characteristics of this soil were not clear and should be examined.

6.2 MOISTURE EFFECT ON RESILIENT MODULUS

In the analysis, the effect of moisture on resilient modulus was mainly evaluated according to the following aspects:

1. To compare the regression curves of resilient modulus versus bulk stress and confining pressure at different moisture content
2. To evaluate the effect of moisture on the coefficient constant of the regression model
3. To compare the resilient modulus versus moisture content at different confining pressures and deviator stresses
4. To evaluate the effect of moisture on the resilient modulus and the reduction in resilient modulus due to soaking at the confining pressure of 13.8 kPa (2 psi) and deviator stress of 34.5 kPa (5 psi).

The analyses of the moisture effect on the resilient modulus for each of the eleven soils are presented in the following sections.

6.2.1 Levy County A-3 (4%) Soil

Two regression models for the resilient modulus of granular soils were presented; one was dependent on bulk stress (Equation 2-1) and the other was dependent on confining pressure (Equation 2-2). Four regression coefficient constants (k_1, k_2, k_3, and k_4) from the middle-half and full-length LVDT position measurements are presented in Tables 4.2(A) and 4.2(B) for the Levy County A-3 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.2(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.2(B). The results showed that the moisture had a limited effect on the resilient modulus.

Figure 6.1 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.2(A) and 4.2(B), respectively. The data showed that the k_1 and k_3 values decreased as moisture content increased, but the effect is considered not to be significant when compared with other soils. Figure 6.2 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figure 4.2(A) and 4.2(B). The k_2 and k_4 values had a slight increase when moisture content increased.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.3 and 6.4, respectively. The data showed that the resilient modulus increased with an increase in both confining pressure and deviator stress at the same moisture condition. Both figures show that the resilient modulus decreased when moisture content increased.

In actual field conditions, the confining pressure at subgrade layers was found to be approximately 13.8 kPa (2 psi). In a laboratory resilient modulus test, the resilient modulus value obtained at a deviator stress of 34.5 kPa (5 psi) under the confining pressure 13.8 kPa (2 psi) was considered representative of the in-situ subgrade modulus. The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are summarized and presented at various moisture conditions for the Levy County A-3 soil in Table 6.1. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.5. The data showed that moisture content had some effect on the resilient modulus but the effect was not significant. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.6. The reduction of resilient modulus due to soaking was not significant.

In a summary, moisture had a slight effect on the resilient modulus of the Levy County A-3 soil. The resilient modulus increased with an increase in confining pressure for the A-3 soil.

6.2.2 SR70 A-3 (8%) Soil

Four regression constants (k_1, k_2, k_3, and k_4) from middle-half and full-length LVDT position measurements are presented in Tables 4.3(A) and 4.3(B) for the SR70 A-3 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.3(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.3(B). The results showed that moisture had a slight effect on the resilient modulus.

Figure 6.7 shows the moisture effect on the constants k_1 and k_3 of the regression models. Regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.3(A) and 4.3(B). The data showed that the k_1 and k_3 values decreased as the moisture content increased. Figure 6.8 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figures 4.3(A) and 4.3(B). The k_2 and k_4 values had a slight increase when moisture content increased.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.9 and 6.10, respectively. The resilient modulus increased with an increase in confining pressure. Figure 6.10 shows that, at the dry side, the resilient modulus decreased as deviator stress increased. This is a different result from that which was obtained when the resilient modulus of the soil was tested at the optimum and soaked conditions, where the effect of deviator stress was not significant.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are summarized and presented in Table 6.2 at various moisture conditions for the SR70 A-3 soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.11. The data showed that moisture content had some effect on the resilient modulus from middle-half LVDT measurements, but had no effect on the resilient modulus from full-length LVDT measurements. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.12. The reduction rate of resilient modulus due to soaking was 12.7%.

In a summary, the moisture had some effect on the resilient modulus of the SR70 A-3 soil. The drying process caused some increase in the resilient modulus of the A-3 soil. The soaking process decreased the resilient modulus by about 12.7%. The effect of moisture was not very significant.

6.2.3 A-2-4 (12%) Soil

Four regression constants are presented in Tables 4.4(A) and 4.4(B) for the A-2-4 12% soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.4(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.4(B). The results showed that the moisture had a slight effect on the resilient modulus.

Figure 6.13 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.4(A) and 4.4(B). The data showed that the moisture had some effect on the k_1 and k_3. Figure 6.14 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figure 4.4(A) and Figure 4.4(B). The data showed that the moisture had some effect on the constants.

The effects of moisture on the resilient modulus at different confining pressures and different deviator stresses are demonstrated in Figures 6.15 and 6.16, respectively. The data showed that the resilient modulus increased with an increase in both confining pressure and deviator stress at the same moisture condition. Both figures show that the resilient modulus decreased when moisture content increased.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented for various moisture conditions for the A-2-4 12% soil in Table 6.3. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.17. The data showed that moisture content had some effect on the resilient modulus. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.18. The reduction rate of resilient modulus due to soaking was 8.7%.

In a summary, the moisture has some effect on the resilient modulus of the A-2-4 soil with 12% fines. The drying caused an increase in the resilient modulus, while the soaking decreased the resilient modulus by 8.7%.

6.2.4 SR70 A-2-4 (14%) Soil

Four regression constants are presented in Tables 4.5(A) and 4.5(B) for the SR70 A-2-4 soil with 14% fines. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.5(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.5(B). The results showed that moisture had a significant effect on the resilient modulus.

Figure 6.19 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.5(A) and 4.5(B). The data showed that the moisture had a significant effect on the k_1 and k_3. Figure 6.20 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figures 4.5(A) and 4.5(B). The data showed that moisture had a significant effect on the constants.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.21 and 6.22, respectively. The resilient modulus increased with an increase in confining pressure. The effect was more pronounced in the soaked condition. The resilient modulus decreased as deviator stress increased. Different from the confining pressure, the effect of deviator stress was more significant at the dry side than that at the wet side.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented in Table 6.4 for various moisture conditions for the SR70 A-2-4 14% soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.23. The data showed that moisture content had a significant effect on the resilient modulus. The resilient modulus values under the optimum and soaked conditions are compared and illustrated in Figure 6.24. The reduction rate of resilient modulus due to soaking was 26%.

In summary, the moisture had a significant effect on the resilient modulus of the SR70 A-2-4 soil with 14% fines. The drying process caused a significant increase in the resilient modulus. The soaking decreased the resilient modulus by 26%.

6.2.5 A-2-4 (20%) Soil

Four regression constants are presented in Tables 4.6(A) and 4.6(B) for the A-2-4 20% soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.6(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.6(B). The results showed that the moisture had some effect on the resilient modulus. The effect at the dry side is more significant than that at the wet side.

Figure 6.25 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.6(A) and 4.6(B). Figure 6.26 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figure 4.6(A) and Figure 4.6(B). The data showed that the moisture had some effect on k_1, k_2 k_3, and k_4.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.27 and 6.28, respectively. The resilient modulus increased with an increase in confining pressure at about same rate at both the dry and wet side. The resilient modulus did not vary much as deviator stress changed. The effect of deviator stress on the resilient modulus was low for the A-2-4 20% soil.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented in Table 6.5 for various moisture conditions for the A-2-4 20% soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.29. The data showed that moisture content had some effect on the resilient modulus. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.30. The reduction rate of the resilient modulus due to soaking was 1.3%.

In summary, the moisture had some effect on the resilient modulus of the A-2-4 soil with 20% fines. The drying caused an increase in the resilient modulus, while the soaking decreased the resilient modulus by 1.3%.

6.2.6 A-2-4 (24%) Soil

Four regression constants are presented in Table 4.7(A) and 4.7(B) for the A-2-4 24% soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure

4.7(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.7(B). The results showed that moisture had some effect on the resilient modulus. The effect at the wet side was more significant than that at the dry side.

Figure 6.31 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.7(A) and 4.7(B). Figure 6.32 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figures 4.7(A) and 4.7(B). The data showed that moisture had some effect on the constants.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.33 and 6.34, respectively. The resilient modulus increased with an increase in confining pressure at about same rate at both the dry and wet sides. The resilient modulus did not vary much as deviator stress changed. The effect of deviator stress on the resilient modulus was low for the A-2-4 24% soil.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented in Table 6.6 for various moisture conditions for the A-2-4 24% soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.35. The data showed that moisture content had some effect on the resilient modulus. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.36. The reduction rate of the resilient modulus due to soaking was 18.8%.

In summary, the moisture had a significant effect on the resilient modulus of the A-2-4 soil with 24% fines. Drying caused a significant increase in the resilient modulus, while soaking decreased the resilient modulus by 18.8%.

6.2.7 A-2-4 (30%) Soil

Four regression constants are presented in Tables 4.8(A) and 4.8(B) for the A-2-4 30% soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.8(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.8(B). The results showed that the moisture had a significant effect on the resilient modulus when tested at the dry side of optimum. The difference in the resilient modulus between the conditions of optimum and soaked was insignificant.

Figure 6.37 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.8(A) and 4.8(B). Figure 6.38 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figures 4.8(A) and 4.8(B).

The moisture effect on the regression constants was found to be different from the results for the other soils and was significant at the dry side of optimum.

The effects of moisture on the resilient modulus at different confining pressures and different deviator stresses are demonstrated in Figures 6.39 and 6.40, respectively. The resilient modulus increased with an increase in confining pressure, but decreased as deviator stress increased. The effect of deviator stress was more significant when tested at the dry side of optimum.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented in Table 6.7 for various moisture conditions for the A-2-4 30% soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.41. The data showed that the moisture content had a significant effect on the resilient modulus. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.42. The data showed that the resilient modulus was slightly higher when tested in the soaked condition. This is an exception to the water-resilient modulus relationship.

In summary, moisture had a very significant effect on the resilient modulus of the A-2-4 soil with 30% fines. The decrease in moisture content due to drying caused a great increase in the resilient modulus. The increase of moisture due to soaking did not affect the resilient modulus.

6.2.8 Miami Oolite A-1 Soil

Four regression constants are presented in Tables 4.9(A) and 4.9(B) for the Miami Oolite A-1 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.9(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.9(B). The results showed that the moisture had a significant effect on the resilient modulus at the dry side of optimum.

Figure 6.43 shows the moisture effect on the constants k_1 and k_3 of the regression models. The regression constants k_1 and k_3 are the y-intercept of the resilient modulus in Figures 4.9(A) and 4.9(B). Figure 6.44 shows the moisture effect on the constants k_2 and k_4. The regression constants k_2 and k_4 are the slopes of regression equations in Figures 4.9(A) and 4.9(B). The data showed that moisture had a significant effect on the regression constants.

The effects of moisture on the resilient modulus at different confining pressures and deviator stresses are demonstrated in Figures 6.45 and 6.46, respectively. The resilient modulus increased with an increase in confining pressure. The effect was more significant at the higher confining pressure. The resilient modulus did not vary much as deviator stress changed.

The effect of deviator stress on the resilient modulus was low for the Miami Oolite A-1 soil.

The resilient modulus values under the condition of confining pressure 13.8 kPa (2 psi) and deviator stress 34.5 kPa (5 psi) are presented in Table 6.8 for various moisture conditions for the Miami Oolite A-1 soil. The effect of moisture on the resilient modulus under these conditions is shown in Figure 6.47. The data showed that moisture content had a significant effect on the resilient modulus. The resilient modulus values at the optimum and soaked conditions are compared and illustrated in Figure 6.48. The reduction rate of the resilient modulus due to soaking was 31%.

In summary, the effect of moisture was significant on the resilient modulus of the Miami Oolite A-1 soil. The decrease in moisture content due to drying caused a significant increase in the resilient modulus. The increase in moisture due to soaking reduced the resilient modulus by 31%.

6.2.9 Spring Cemetery A-2-4 (15%) Soil

Four regression constants are presented in Tables 4.10(A) and 4.10(B) for the Spring Cemetery A-2-4 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.10(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.10(B). Since the resilient modulus data were only obtained from the full-length LVDT position measurement under the optimum compacted condition, the effect of moisture content on the resilient modulus was not available for this soil. The resilient modulus values obtained at a deviator stress of 41.4 kPa (6 psi) under the confining pressure 13.8 kPa (2 psi) for each test are listed in Table 6.9.

6.2.10 Branch A-2-4 (23%) Soil

Four regression constants are presented in Tables 4.11(A) and 4.11(B) for the Branch A-2-4 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.11(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.11(B). Since the resilient modulus data were only obtained from the full-length LVDT position measurement under the optimum compacted condition, the moisture content effect on the resilient modulus was not available for this soil. The resilient modulus values obtained at a deviator stress of 41.4 kPa (6 psi) under the confining pressure 13.8 kPa (2 psi) for each test are listed in Table 6.10.

6.2.11 Iron Bridge A-2-6 (31%) Soil

Four regression constants are presented in Tables 4.12(A) and 4.12(B) for the Iron Bridge A-2-6 soil. The resilient modulus versus bulk stress at different moisture content is presented in Figure 4.12(A), while the resilient modulus versus confining pressure at different moisture content is presented in Figure 4.12(B). Since the resilient modulus data were only obtained from the full-length LVDT position measurement under the optimum compacted condition, the effect of moisture content on the resilient modulus was not available for this soil. The resilient modulus values obtained at a deviator stress of 41.4 kPa (6 psi) under the confining pressure 13.8 kPa (2 psi) for each test are listed in Table 6.11.

6.3 DISCUSSION OF THE EFFECT OF MOISTURE AND STRESS ON RESILIENT MODULUS

The average resilient moduli (at 11 psi bulk stress from the regression models) of each soil for different water conditions are summarized in Table 6.12 and illustrated in Figures 6.49 and 6.50. The resilient modulus values ranged from 116 MPa (16824 psi) to 158 MPa (22916 psi) with the middle-half position at optimum condition, except that SR70 A-2-4 (14%) soil had a relatively higher resilient modulus of 247 MPa (35824 psi). The SR70 A-2-4 (14%) and A-2-4 (30%) had extremely high resilient modulus values with 541 MPa (78465 psi) and 775 MPa (112404 psi) respectively under dry conditions with the middle-half measurement. With the full-length position measurement at the optimum condition, the average resilient modulus values ranged from 64 MPa (9282 psi) to 119 MPa (17259 psi) except that SR70 A-2-4 and Branch A-2-4 had relatively higher values of 167 MPa

(24221 psi) and 186 MPa (26977 psi). A similar result occurred for the middle-half measurement; the SR70 A-2-4 and A-2-4 (30%) soils had higher resilient moduli of 306 MPa (44382 psi) and 285 MPa (41336 psi) under dry conditions with the full-length measurement, but the values were much lower than those measured with middle-half position. The ratio of the middle-half to full-length LVDT position measurements was from 1.13 to 1.85 with an average ratio of 1.36, which conforms to the findings from Hoang (1996). The laboratory resilient moduli are compared with the layer moduli in Chapter 9.

6.3.1 Moisture Effect on Resilient Modulus

Table 6.13 summarizes the resilient modulus for dry, optimum, and soaked conditions, with degree of saturation. The SR70 A-2-4 (14%) soil had a relatively higher resilient modulus with the higher degree of saturation among the eleven soils, except that A-2-4 (30%) had the highest resilient modulus for dry conditions and Branch had the highest resilient modulus for the optimum condition with full-length measurement. For the two A-3 soils, Levy County and SR70, the degree of saturation was the lowest, and about 71% at the soaked condition.

The reduction in resilient modulus was calculated from dry to optimum and optimum to soaked water conditions based on the resilient modulus calculated from the regressions at 11 psi (75.8 kPa) bulk stress. The results are summarized in Tables 6.14(A) and 6.14(B). The reduction in resilient modulus versus increased percent of water content are analyzed and presented in the figures from Figures 6.51 through 6.58.

Figure 6.51 shows that, with middle-half measurement, the SR70 A-2-4 (14%) and A-2-4 (30%) soils had the higher total loss in resilient modulus on the water condition from dry to optimum. The loss was more significant on the dry side for all eight soils with the exception of, the A-2-4 (24%) soil, which had more loss on the wet side. In terms of percent of reduction in resilient modulus, the SR70 A-2-4 (14%), A-2-4 (30%), and Miami Oolite A-1 soils had higher reduction rates compared to other soils, as shown in Figure 6.52. The same situation applied to the data with full-length measurement. As shown in Figures 6.53 and 6.54, SR70 A-2-4 and A-2-4 (30%) had higher total resilient modulus loss and reduction rates. When considering the effect per 1% increase in moisture content on the resilient modulus, SR70 A-2-4, A-2-4 (30%), and Miami Oolite A-1 had the higher loss among the others. It is observed that the A-2-4 (24%) soil had a relatively higher reduction in resilient modulus per 1% increased moisture content at its wet side. The comparison can be seen in Figures 6.55, 6.56, 6.57 and 6.58.

According to the reduction rates, the eight soils were further classified into four categories based on their susceptibility to moisture. The classification of the moisture effect by the rate of reduction in resilient modulus is summarized in Table 6.15(A) for the middle-half and Table 6.15(B) for the full-length measurement. From the tables, the moisture effect from the optimum to soaked condition of two subgrade soils, SR70 A-2-4 and Miami Oolite A-1 are considered very severe. The A-2-4 (24%) is considered severe compared to other A-2-4 soils. The reduction rate of resilient modulus versus increased level of moisture content for eight soils are presented in Figures 6.59 and 6.60.

From the above analysis, the results showed that moisture had a detrimental effect on the resilient modulus of subgrade soils. Figure 6.61 demonstrates the moisture effect on the resilient modulus. In general, an increase in moisture caused a reduction in the resilient modulus. The degree of reduction was different among various types of soils. The degree of reduction for A-2-4 soils was more apparent than that of A-3 soils.

6.3.2 Stress Effect on Resilient Modulus

The laboratory resilient modulus is stress-dependent. The resilient modulus increased with an increase in confining pressure for granular soils. The test results showed the significant effect of the confining pressure on the resilient modulus of the eleven materials. The effect of the deviator varied for different soils. Most of the test results showed that the resilient modulus increased with increasing deviator stress, while the resilient modulus of the SR70 A-2-4 (14%), A-2-4 (30%), and Branch soils decreased with increasing deviator stress. The inverse proportion of the resilient modulus to deviator stress occurred when the sample was hard, especially at the higher confining pressure for fine-grained materials. The effects of deviator stress for the A-2-4 (20%) and Miami Oolite A-1 soils were not significant.

The regression constants k_1 and k_3 are dependent on moisture content, which can change with the seasons. k_2 and k_4 are related to soil types, either coarse-grained or fine-grained soils. The increase of the constants k_2 and k_4 with increasing moisture content indicated that the resilient modulus became more sensitive to confining pressure and bulk stress with an increase in moisture content. The increase of moisture reduced the rigidity of the soil structure and made it more sensitive to the surrounding pressure. The increase of moisture could also increase the Poisson's ratio of the soil.

6.4 EFFECT OF SOIL PROPERTIES ON RESILIENT MODULUS

This section will discuss how basic engineering properties affect the resilient modulus in this study. Table 6.16 summarizes the tested material characteristics for the eleven soils.

6.4.1 Percent of Fines

Generally, the percentage of fines passing sieve No. 200 of a subgrade soil can significantly influence the effect of moisture on its resilient modulus. The A-2-4 soils with a relatively high percentage of fines are more susceptible to an increase in moisture than the A-3 soils, as can be seen in Figure 6.61. However, as shown in Figure 6.62, with different percentages of fines for the A-2-4 and A-3 soils, the percentage of fines passing sieve No. 200 may not be a dominant factor in predicting the resilient modulus.

The reduction rates of the resilient modulus due to drying and soaking versus the percentages of fines passing sieve No. 200 are illustrated in Figures 6.63 and 6.64 for the seven soils excluding the Miami Oolite A-1 soil. Apparently, the A-3 soils with higher fines had a higher reduction rate in resilient modulus. However, there is no trend for the A-2-4 soils. Among the A-2-4 soils, the SR70 A-2-4 (14%) had the highest reduction rate in resilient modulus, while the A-2-4 (12%) had the lowest one, with the exception of the A-2-4 (30%) soil. The A-2-4 (30%) showed an extremely high reduction rate at the dry side, but a low reduction rate at the wet side. The data showed that the percentage of fines had a certain level of contribution to the susceptibility of the soil to the water content change, and can be further investigated.

6.4.2 Limerock Bearing Ratio

The LBR values versus the percentages of fines passing sieve No. 200 for the eleven soils are also presented in Figure 6.65. The A-2-4 soils had a relatively higher LBR than the A-3 soils, with the exception of the A-2-4 (12%) soil. Comparing the data shown in Figures 6.66 and 6.67, the A-1 soil had a higher reduction rate with a higher LBR value, while the A-3 and A-2-4 soils with lower fines had a lower reduction rate with lower LBR values. The reduction rates are proportional to the LBR values with the exception of the A-2-4 soils with 20% and 30% fines for moisture conditions from optimum to soaked with middle-half measurement. For the moisture conditions from dry to optimum, the reduction rates for the A-2-4 soils decreased with an increasing LBR, with the exception of the A-2-4 with 24% fines.

6.4.3 Maximum Dry Unit Weight

Figure 6.68 shows the relationship of the resilient modulus to the dry unit weight of the eleven soils. The figure shows there was no correlation between the two. From the Figure 6.69, which shows a strong correlation between LBR and maximum dry unit weight, the reduction in resilient modulus is increasing with an increasing maximum dry unit weight. The findings for the effect of LBR on the reduction rate of the resilient modulus from 6.4.2 can be applied here. Figures 6.70 and 6.71

demonstrate the trend of the effect of maximum dry unit weight on the reduction rate in the resilient modulus.

6.4.4 Gradation

Figure 6.72 presents the gradation curves of the eight soils. The other characteristics can be found in Table 6.16. Both SR70 A-2-4 and Iron Bridge A-2-6 Soils had the higher percent of clay content. This can contribute to a higher resilient modulus, as shown in Figure 6.73. But the resilient modulus of Iron Bridge A-2-6 soil can be reduced by the presence of plasticity. In general, the soils with a higher percentage of clay have a higher resilient modulus. However, they also have higher reduction rates in resilient modulus and are more sensitive to a change in moisture content level, as shown in Figures 6.74 and 6.75. The Branch A-2-4 Soil had the highest resilient modulus (full-length measurement). This may be attributed to the higher LBR value and the well-graded characteristic.

As found in both studies from Zhang (2004) and Ling (2007), the resilient modulus increases with an increasing coefficient of uniformity (C_u), but with a decreasing coefficient of curvature (C_c). The data showed that SR70 A-2-4 had an extremely high coefficient of uniformity (C_u) and coefficient of curvature (C_c), with a 10% clay content. This may lead to its high resilient modulus. So did the Branch A-2-4 soil. With plasticity

Index 5 and 12, Both Branch A-2-4 and Iron Bridge A-2-6 soils had a very high reduction rate in Test-Pit tests when the soils were soaked.

Table 6.1 M_r vs. Moisture Content, Levy County A-3 Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
4.3	13.79	0.26	32.19	73.56	160.58	104.48
8.1	13.79	0.26	32.22	73.59	166.03	124.2
9.5	13.79	0.27	32.67	74.04	142.11	115.78
9.6	13.79	0.26	32.29	73.66	150.25	97.20
13.5	13.79	0.26	32.29	73.66	191.5	128.53
15.0	13.79	0.26	32.22	73.59	134.78	83.88
15.3	13.79	0.26	32.35	73.72	156.29	86.58

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.2 M_r vs. Moisture Content, SR70 A-3 Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
4.0	13.79	0.26	32.28	73.65	243.52	121.11
4.5	13.79	0.26	32.27	73.64	220.51	105.24
5.3	13.79	0.26	32.22	73.59	166.77	120.13
7.8	13.79	0.26	32.31	73.68	131.8	100.3
11.4	13.79	0.26	32.59	73.96	154.71	111.97
11.4	13.79	0.26	32.36	73.73	153.5	118.83
13.4	13.79	0.26	32.53	73.9	154.8	98.31
13.7	13.79	0.34	41.4	82.78	133.97	98.6

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.3 M_r vs. Moisture Content, A-2-4 12% Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
7.1	13.79	0.26	32.24	73.61	136.64	98.45
7.0	13.79	0.26	32.25	73.62	138.2	94.08
12.1	13.79	0.26	32.31	73.68	121.64	90.44
12.1	13.79	0.26	32.32	73.69	120.49	96.38
14.6	13.79	0.26	32.32	73.69	101.14	88.5
13.6	13.79	0.26	32.25	73.62	111.12	90.98

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.4 M_r vs. Moisture Content, SR70 A-2-4 14% Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
3.1	13.79	0.26	32.27	73.64	4681.48	188.41
7.8	13.79	0.26	32.34	73.71	697.43	110.52
8.4	13.79	0.26	32.24	73.61	511.14	290.58
10.4	13.79	0.27	32.71	74.08	198.69	144.73
10.8	13.79	0.27	32.84	74.21	271.18	172.86
11.2	13.79	0.26	32.26	73.63	144.0	100.43
11.7	13.79	0.26	32.2	73.57	152.24	81.35

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.5 M_r vs. Moisture Content, A-2-4 20% Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
8.3	13.79	0.26	32.26	73.63	199.1	127.75
7.3	13.79	0.26	32.25	73.62	211.21	140.96
10.0	13.79	0.26	32.4	73.77	130.15	105.15
10.0	13.79	0.26	32.28	73.65	125.88	107.18
11.6	13.79	0.26	32.22	73.59	124.96	98.38
12.3	13.79	0.26	32.32	73.69	109.79	96.16

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.6 M_r vs. Moisture Content, A-2-4 24% Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
7.7	13.79	0.26	32.32	73.69	114.68	100.26
7.7	13.79	0.26	32.24	73.61	129.71	112.84
10.7	13.79	0.26	32.33	73.7	110.74	90.24
10.7	13.79	0.26	32.28	73.65	112.97	91.87
12.0	13.79	0.26	32.31	73.68	72.08	59.70
11.4	13.79	0.26	32.32	73.69	99.33	78.14

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.7 M_r vs. Moisture Content, A-2-4 30% Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
6.3	13.79	0.26	32.24	73.61	690.15	255.33
7.0	13.79	0.26	32.16	73.53	671.12	266.52
12.0	13.79	0.26	32.24	73.61	102.9	70.32
12.3	13.79	0.26	32.26	73.63	90.15	65.26
13.4	13.79	0.26	32.36	73.73	120.35	70.44
13.2	13.79	0.26	32.3	73.67	122.64	67.77

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.8 M_r vs. Moisture Content, Miami Oolite Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Middle Modulus	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa	MPa
4.4	13.79	0.26	32.18	73.55	289.02	181.92
5.6	13.79	0.26	32.32	73.69	267.45	129.41
7.8	13.79	0.26	32.23	73.61	141.04	110.25
7.8	13.79	0.26	32.29	73.66	149.98	116.89
8.1	13.79	0.26	32.23	73.6	109.93	91.99
8.2	13.79	0.26	32.25	73.62	86.90	70.67

* Data is selected at a deviator stress of 5 psi and a confining pressure of 2 psi.

Table 6.9 M_r vs. Moisture Content, Spring Cemetery Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa
9.2	11.03	0.33	40.98	74.07	71.53
9.2	11.03	0.33	41.18	74.27	63.7
9.2	13.79	0.37	41.32	82.69	63.26
9.2	13.79	0.37	41.37	82.74	68.75

* Only optimum conditions are available.

** Sample was compacted to 100% Standard Proctor.

*** Data is selected at a deviator stress of 6 psi and a confining pressure of 2 psi.

Table 6.10 Mr vs. Moisture Content, Branch Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa
8.7	11.03	0.33	40.33	74.23	150.93
8.7	11.03	0.32	39.62	73.52	178.23
8.9	13.79	0.33	40.16	81.53	170.93
8.9	13.79	0.32	39.79	81.16	180.32
9.3	13.79	0.33	41.05	82.42	95.02
9.3	13.79	0.33	41.46	82.83	110.74

** Sample was compacted to 100% Standard Proctor.
** Data is selected at a deviator stress of 6 psi and a confining pressure of 2 psi.

Table 6.11 M_r vs. Moisture Content, Iron Bridge Soil

Moisture Content	Confining Pressure	Axial Load	Dev. Stress	Bulk Stress	Full Length Modulus
%	kPa	kN	kPa	kPa	MPa
10.3	11.03	0.33	41.09	74.18	76.23
10.3	11.03	0.33	41.09	74.18	76.23
10.4	13.79	0.33	41.3	82.67	70.62
10.4	13.79	0.34	41.44	82.81	72.31

* Only optimum conditions are available.
** Sample was compacted to 100% Standard Proctor.
*** Data is selected at a deviator stress of 6 psi and a confining pressure of 2 psi.

Table 6.12 Summary of Laboratory Resilient Moduli at 11 psi Bulk Stress for 11 Soils

Soil	Sample No.	Moisture Content (%)	Resilient Modulus (Mpa) @ 11 psi (75.84 kPa) Bulk Stress		Avg. Resilient Modulus (Mpa) @ 11 psi (75.84 kPa) Bulk Stress	
			Middle Half	Full Length	Middle Half	Full Length
Levy	A3LEVYD1	8.08	170.83	127.60	166.95	116.92
	A3LEVYD2	4.30	163.07	106.24		
	A3LEVYO1	9.50	142.93	118.68	145.50	108.14
	A3LEVYO2	9.60	148.08	97.60		
	A3LEVYS2	15.00	132.87	84.25	149.54	81.83
	A3LEVYS3	15.27	166.22	79.40		
SR70-A3	A3SR70D1	7.80	135.82	102.90	208.23	120.16
	A3SR70D2	5.30	174.92	129.08		
	A3SR70D3	4.50	236.61	124.55		
	A3SR70D4	4.00	285.57	124.11		
	A3SR70O1	11.40	156.82	114.33	157.72	118.81
	A3SR70O2	11.40	158.62	123.29		
	A3SR70S1	13.40	160.83	99.82	137.74	91.90
	A3SR70S2	13.70	114.65	83.97		
A24-12%	A2412%D1	7.10	139.86	97.47	140.61	96.50
	A2412%D2	7.00	141.36	95.52		
	A2412%O1	12.10	119.49	93.45	117.91	95.62
	A2412%O2	12.10	116.34	97.78		
	A2412%S1	14.60	102.35	89.64	107.61	90.84
	A2412%S2	13.60	112.87	92.04		
SR70-A24	A24SR70D1	8.41	540.56	305.67	540.56	305.67
	A24SR70O1	10.80	277.24	179.01	246.63	166.62
	A24SR70O2	10.39	216.01	154.23		
	A24SR70S1	11.23	160.35	109.81	182.61	97.21
	A24SR70S2	11.70	204.87	84.61		
A24-20%	A2420%D1	8.30	205.11	136.11	211.96	142.54
	A2420%D2	7.30	218.82	148.97		
	A2420%O1	10.00	129.66	107.57	123.17	109.37
	A2420%O2	10.00	116.69	111.16		
	A2420%S1	11.60	129.11	101.33	121.55	100.04
	A2420%S2	12.30	113.98	98.75		

Table 6.12 - Continued

Soil	Sample No.	Moisture Content (%)	Resilient Modulus (Mpa) @ 11 psi (75.84 kPa) Bulk Stress		Avg. Resilient Modulus (Mpa) @ 11 psi (75.84 kPa) Bulk Stress	
			Middle Half	Full Length	Middle Half	Full Length
A24-24%	A2424%D1	7.72	118.84	102.95	125.67	109.43
	A2424%D2	7.65	132.51	115.91		
	A2424%O1	10.70	115.79	90.73	116.47	92.05
	A2424%O2	10.70	117.16	93.37		
	A2424%S1	12.00	74.59	59.64	94.76	68.91
	A2424%S2	11.40	114.93	78.19		
A24-30%	A2430%D1	6.30	789.07	283.91	775.29	285.18
	A2430%D2	7.00	761.52	286.44		
	A2430%O1	12.00	144.34	74.51	132.95	71.84
	A2430%O2	12.30	121.56	69.17		
	A2430%S1	13.40	133.96	73.58	136.34	71.65
	A2430%S2	13.20	138.73	69.71		
Oolite	OOLITED1	5.60	271.17	131.36	286.20	126.04
	OOLITED2	4.40	301.23	120.72		
	OOLITEO1	7.80	133.26	104.62	134.04	107.39
	OOLITEO2	7.80	134.82	110.15		
	OOLITES1	8.20	78.88	67.01	92.47	78.82
	OOLITES2	8.00	106.06	90.63		
Spring Cemetery	SC001C1	9.20		73.98		67.01
	SC001D1	9.20		64.86		
	SC001E1	9.30		61.61		
	SC001F1	9.30		67.62		
Branch	BH001C1	8.70		169.17		186.33
	BH001D2	8.70		199.14		
	BH001E1	8.90		185.10		
	BH001F1	8.90		191.92		
Iron Bridge	IB001C1	10.30		64.92		63.79
	IB001D1	10.30		63.15		
	IB001E1	10.40		64.22		
	IB001F1	10.40		62.86		

Table 6.13 Summary of Average Resilient Moduli at Different Moisture Conditions

Soil	DRY CONDITION				OPTIMUM CONDITION				SOAKED CONDITION			
	Moisture Content (%)	Degree of Saturation (%)	Mr @ 11 psi Bulk Stress (Mpa)		Moisture Content (%)	Degree of Saturation (%)	Mr @ 11 psi Bulk Stress (Mpa)		Moisture Content (%)	Degree of Saturation (%)	Mr @ 11 psi Bulk Stress (Mpa)	
			Middle Half	Full Length			Middle Half	Full Length			Middle Half	Full Length
Levy A-3 (4%)	6.20	29.76	166.95	116.92	9.55	45.60	145.50	108.14	15.15	71.49	149.54	81.83
SR 70 A-3 (8%)	5.40	29.25	208.23	120.16	11.40	62.55	157.72	118.81	13.55	71.94	137.74	91.90
A-2-4 (12%)	7.05	38.47	140.61	96.50	12.10	64.00	117.91	95.62	14.10	75.39	107.61	90.84
SR 70 A-2-4 (14%)	8.41	60.75	540.56	305.67	10.60	75.76	246.63	166.62	11.45	85.95	182.61	97.21
A-2-4 (20%)	7.80	51.73	211.96	142.54	10.00	67.91	123.17	109.37	11.95	81.40	121.55	100.04
A-2-4 (24%)	7.70	47.37	125.67	109.43	10.70	66.58	116.47	92.05	11.70	76.03	94.76	68.91
A-2-4 (30%)	6.65	41.65	775.29	285.18	12.15	75.71	132.95	71.84	13.30	84.69	136.34	71.65
Oolite	5.00	51.93	286.20	126.04	7.80	80.47	134.04	107.39	8.15	84.74	92.47	78.82
Spring Cemetery					9.25	57.39		67.01				
Branch					8.80	63.41		186.33				
Iron Bridge					10.35	80.37		63.79				

*Use Specific Gravity = 2.65 for the first eight soils

Table 6.14(A) Summary of Reduction in Resilient Modulus from Dry to Optimum Condition

From Dry to Optimum Condition														
Soil	Moisture Content Change (%)	Degree of Saturation Change (%)	Resilent Modulus Change (MPa)		% Reduction in Resilient Modulus (%)		Resilent Modulus Change per 1% Moisture Content (MPa)		% Reduction in Resilent Modulus per 1% Moisture Content (%)		Resilent Modulus Change per 1% Degree of Saturation (MPa)		% Reduction in Resilent Modulus per 1% Degree of Saturation (%)	
			Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length
A-3, 4%, Levy County	3.35	15.84	-21.45	-8.78	12.85	7.51	-6.40	-2.62	3.84	2.24	-1.35	-0.55	0.81	0.47
A-3, 8%, SR-70	6.00	33.30	-50.51	-1.35	24.26	1.12	-8.42	-0.22	4.04	0.19	-1.52	-0.04	0.73	0.03
A-2-4,12%	5.05	25.53	-22.70	-0.88	16.14	0.91	-4.50	-0.17	3.20	0.18	-0.89	-0.03	0.63	0.04
A-2-4, 14%, SR70	2.19	15.01	-293.93	-139.05	54.38	45.49	-134.21	-63.49	24.83	20.77	-19.59	-9.27	3.62	3.03
A-2-4, 20%	2.20	16.18	-88.79	-33.17	41.89	23.27	-40.36	-15.08	19.04	10.58	-5.49	-2.05	2.59	1.44
A-2-4, 24%	3.00	19.21	-9.20	-17.38	7.32	15.88	-3.07	-5.79	2.44	5.29	-0.48	-0.90	0.38	0.83
A-2-4, 30%	5.50	34.06	-642.34	-213.34	82.85	74.81	-116.79	-38.79	15.06	13.60	-18.86	-6.26	2.43	2.20
Oolite	2.80	28.54	-152.16	-18.65	53.17	14.80	-54.34	-6.66	18.99	5.28	-5.33	-0.65	1.86	0.52

* Only Phase I and Phase II data are available.

Table 6.14(B) Summary of Reduction in Resilient Modulus from Optimum to Soaked Conditions

From Optimum to Soaked Condition																
Soil	Moisture Content Change (%)	Degree of Saturation Change (%)	Resilent Modulus Change (MPa)		% Reduction in Resilient Modulus (%)		Resilent Modulus Change per 1% Moisture Content (MPa)		% Reduction in Resilent Modulus per 1% Moisture Content (%)		Resilent Modulus Change per 1% Degree of Saturation (MPa)		% Reduction in Resilent Modulus per 1% Degree of Saturation (%)	
			Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length	Middle Half	Full Length
A-3, 4%, Levy County	5.60	25.89	4.04	-26.31	-2.78	24.33	0.72	-4.70	-0.50	0.94	0.16	-1.02	-0.11	0.94
A-3, 8%, SR-70	2.15	9.39	-19.98	-26.91	12.67	22.65	-9.29	-12.52	5.89	2.41	-2.13	-2.87	1.35	2.41
A-2-4,12%	2.00	11.39	-10.30	-4.78	8.74	5.00	-5.15	-2.39	4.37	0.44	-0.90	-0.42	0.77	0.44
A-2-4, 14%, SR70	0.85	10.20	-64.02	-69.41	25.96	41.66	-75.32	-81.66	30.54	4.09	-6.28	-6.81	2.55	4.09
A-2-4, 20%	1.95	13.48	-1.62	-9.33	1.32	8.53	-0.83	-4.78	0.67	0.63	-0.12	-0.69	0.10	0.63
A-2-4, 24%	1.00	9.45	-21.71	-23.14	18.64	25.14	-21.71	-23.14	18.64	2.66	-2.30	-2.45	1.97	2.66
A-2-4, 30%	1.15	8.97	3.39	-0.19	-2.55	0.26	2.95	-0.17	-2.22	0.03	0.38	-0.02	-0.28	0.03
Oolite	0.35	4.26	-41.57	-28.57	31.01	26.60	-118.77	-81.63	88.61	6.24	-9.75	-6.70	7.27	6.24

* Only Phase I and Phase II data are available.

Table 6.15(A) Classification of Moisture Effect by Rate of Reduction (Middle Half, Optimum to Soaked)

Reduction Rate	Moisture Effect	Soil Type	Mr Reduction (%)	Maximum Dry Density (kN/M^3)	LBR
<5%	Very Minor	A-3 Soil with 4% fines	-2.8	16.73	22
		A-2-4 soil with 30% fines	-2.5	18.22	72
		A-2-4 soil with 20% fines	1.3	19.54	146
5-15%	Minor	A-2-4 Soil with 12% fines	8.7	17.37	30
		A-3 soil with 8% fines	12.7	17.59	45
15-30%	Severe	A-2-4 Soil with 24% fines	18.7	18.27	69
		SR 70 A-2-4 soil with 14% fines	25.9	19.16	124
>30%	Very Severe	Miami Oolite A-1 soil	31	20.83	194

Table 6.15(B) Classification of Moisture Effect by Rate of Reduction (Full Length, Optimum to Soaked)

Reduction Rate	Moisture Effect	Soil Type	Mr Reduction (%)	Maximum Dry Density (kN/M^3)	LBR
<5%	Very Minor	A-2-4 soil with 30% fines	0.2	18.22	72
5-15%	Minor	A-2-4 Soil with 12% fines	5	17.37	30
		A-2-4 soil with 20% fines	8.6	19.54	146
15-30%	Severe	A-3 soil with 8% fines	22.6	17.59	45
		A-3 Soil with 4% fines	24.3	16.73	22
		A-2-4 Soil with 24% fines	25.1	18.27	69
		Miami Oolite A-1 soil	26.6	20.83	194
>30%	Very Severe	SR 70 A-2-4 soil with 14% fines	41.7	19.16	124

* Only Phase I and Phase II data is available.

Table 6.16 Summary of Tested Materials Characteristics

Soil		Classif-ication	Passing Sieve No. 200	CC	CU	Clay Content	Plastic Index	Optimum Moisture Content	Max. Dry Density		LBR	Permeability	Suction @ OMC
			%			%		%	pcf	kN/m^3		cm/sec	kPa
Phase I	Levy	A-3	4	1.8	3.3	0	-	10	106.5	16.7	22	5.5E-03	17
	SR70-A3	A-3	8	0.9	3.8	6	-	11.5	112	17.6	45	2.1E-03	14
	SR70-A24	A-2-4	14	58.3	290.0	10	-	10.5	122	19.2	124	2.5E-04	81
Phase II	A24-12%	A-2-4	12	1.3	3.0	3	-	12.1	110.6	17.4	30	3.1E-04	440
	A24-20%	A-2-4	20	1.6	3.8	8	-	10	124.4	19.5	146	1.0E-04	373
	A24-24%	A-2-4	24	5.1	14.7	5	-	10.7	116.3	18.3	69	6.5E-05	318
	A24-30%	A-2-4	30	5.7	38.2	6	-	12	116	18.2	72	2.0E-05	320
	Oolite	A-1	-	-	-	-	-	7.6	132.6	20.8	194	-	204
Phase III	Spring Cemetery	A-2-4	15	2.0	7.3	4		9.2/9.3	118.2/118.4	18.6/18.6	83	2.8E-04	-
	Branch	A-2-4	23	4.2	37.5	6	5	8.8/7.2	128.4/134.7	20.2/21.2	132	7.4E-07	-
	Iron Bridge	A-2-6	31	2730*	15000*	16	12	10.3/8.2	123.3/132.4	19.4/20.8	127	5.6E-07	-

Note:

1. Phases I and II soils were compacted to 98% modified, while Phase III soils were compacted to 100% Standard for laboratory resilient modulus tests.
2. All soils were compacted to 98% modified for permeability tests.
3. “*”: Estimated values, due to extremely small grain size diameter at D_{10}.

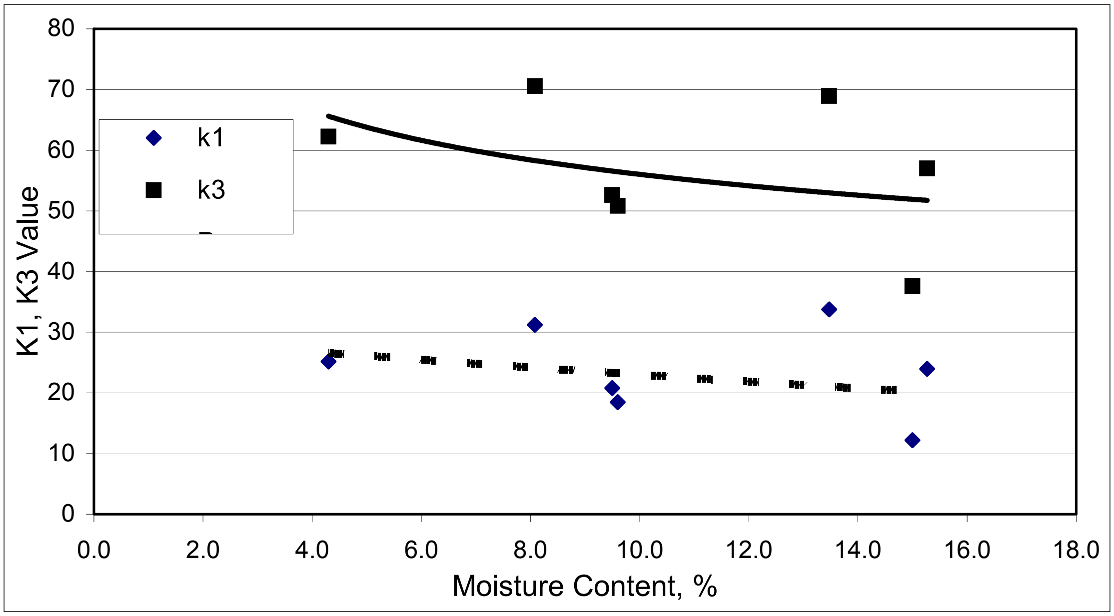


Figure 6.1 k_1, k_3 vs. Moisture Content for Levy County A-3 Soil

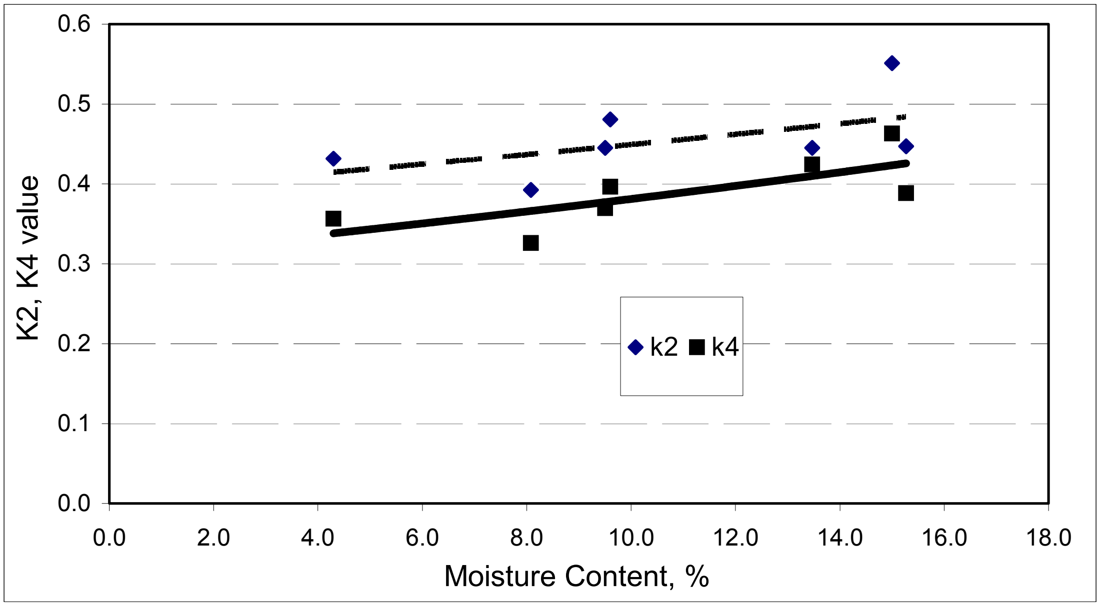


Figure 6.2 k2, k4 vs. Moisture Content for Levy County A-3 Soil

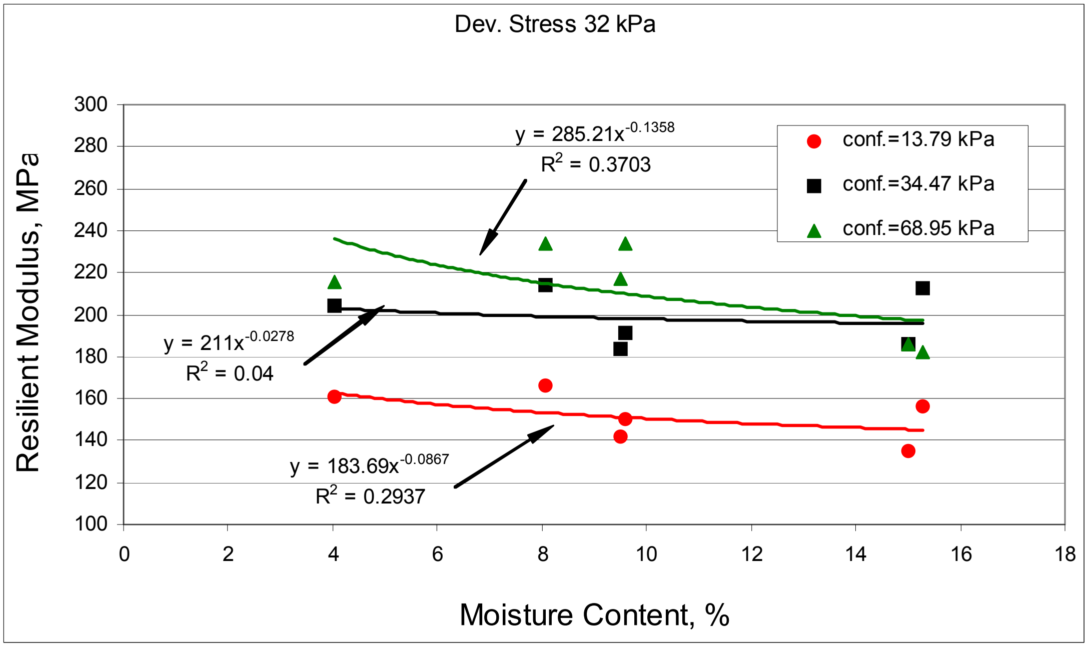


Figure 6.3 Mr vs. Moisture Content at Different Confining Pressures for Levy County A-3 Soil

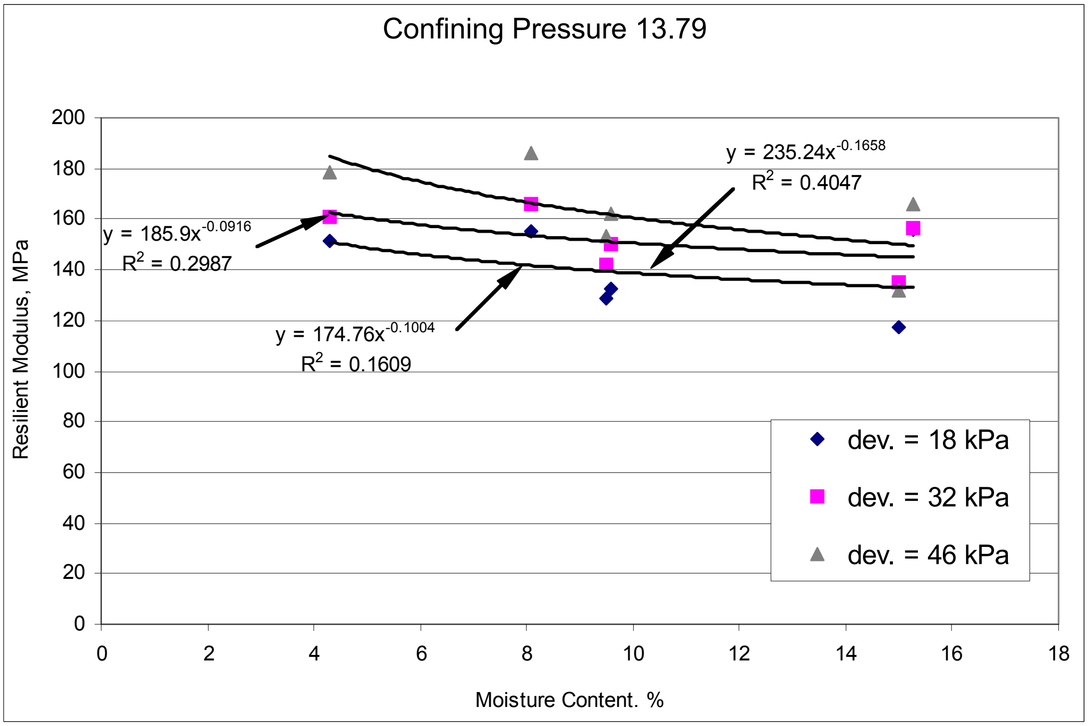


Figure 6.4 Mr vs. Moisture Content at Different Dev. Stresses for Levy County A-3 Soil

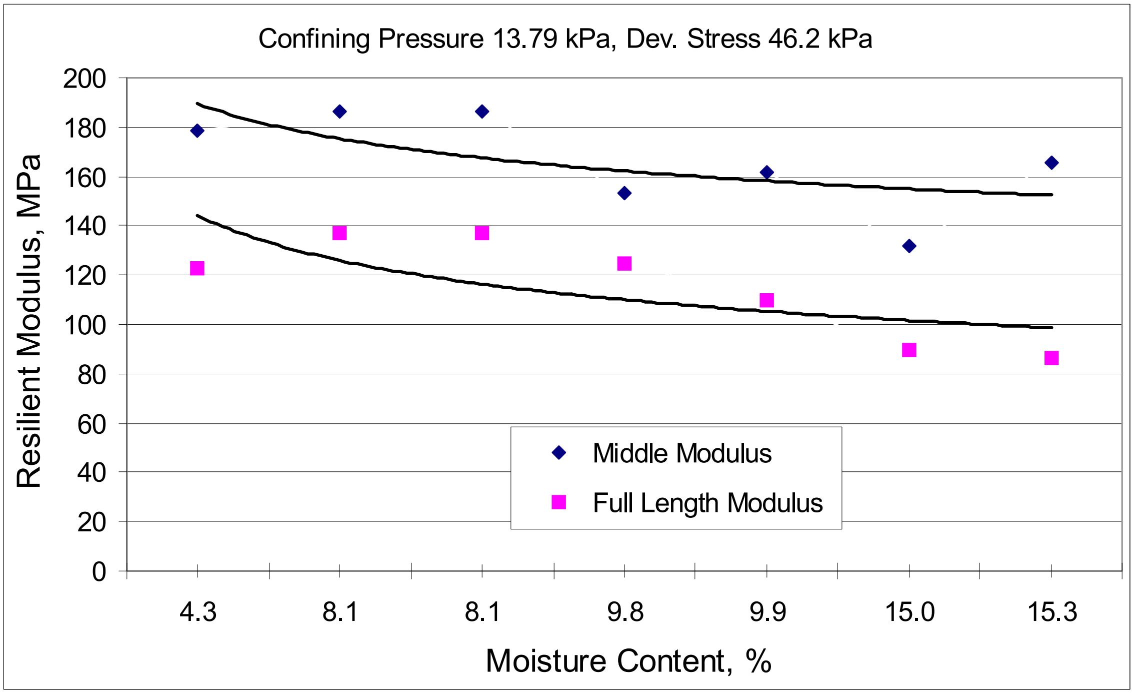


Figure 6.5 Mr vs. Moisture Content for Levy County A-3 Soil

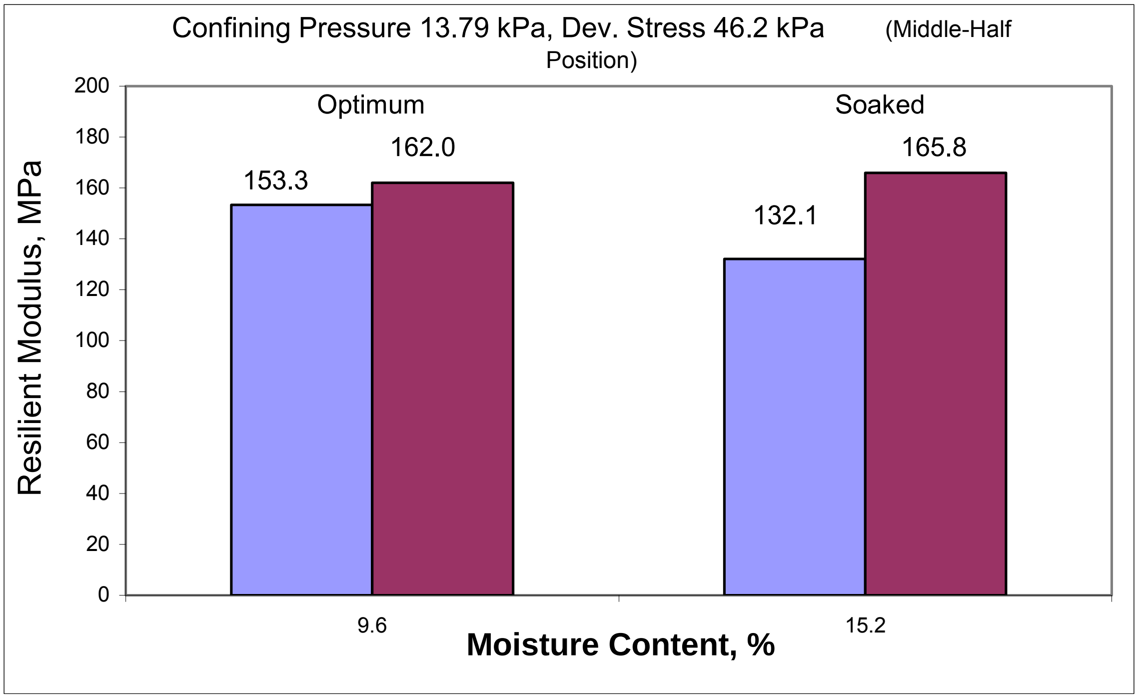


Figure 6.6 Mr vs. Moisture Content at Optimum and Soaked Conditions for Levy County A-3 Soil

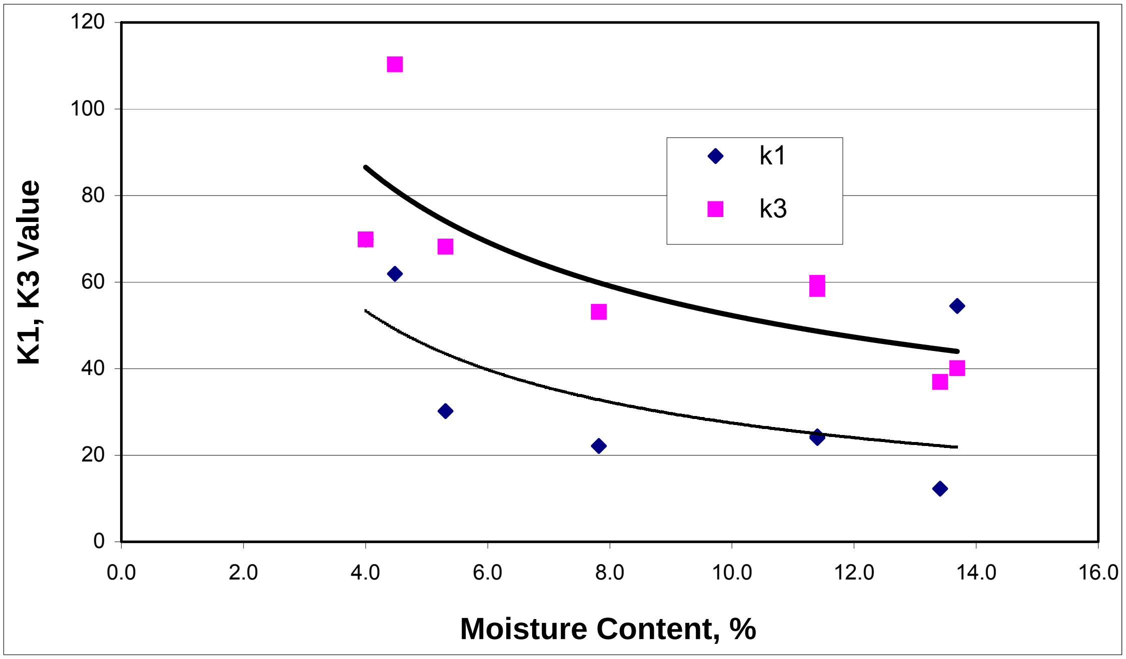


Figure 6.7 k_1, k_3 vs. Moisture Content for SR70 A-3 Soil

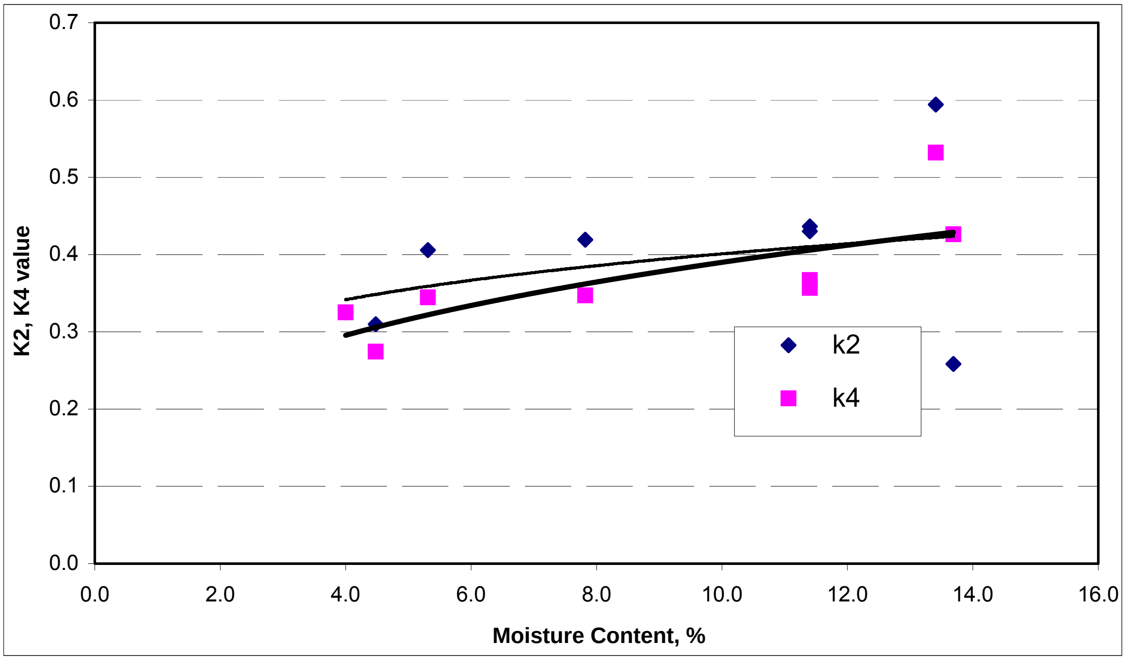


Figure 6.8 k2, k4 vs. Moisture Content for SR70 A-3 Soil

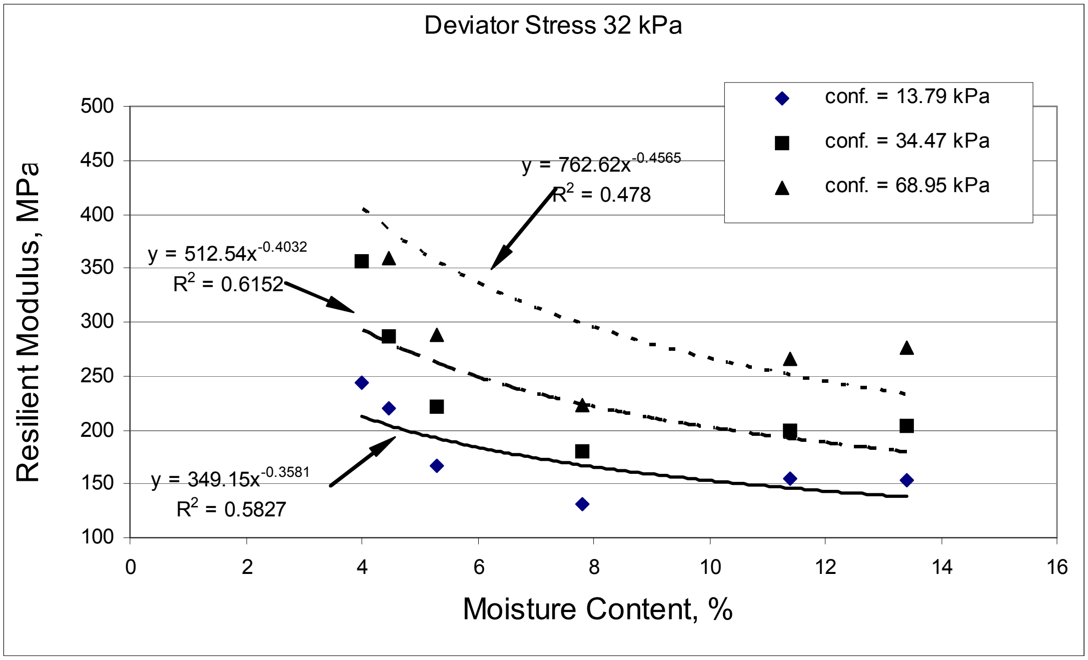


Figure 6.9 Mr vs. Moisture Content at Different Confining Pressures for SR70 A-3 Soil

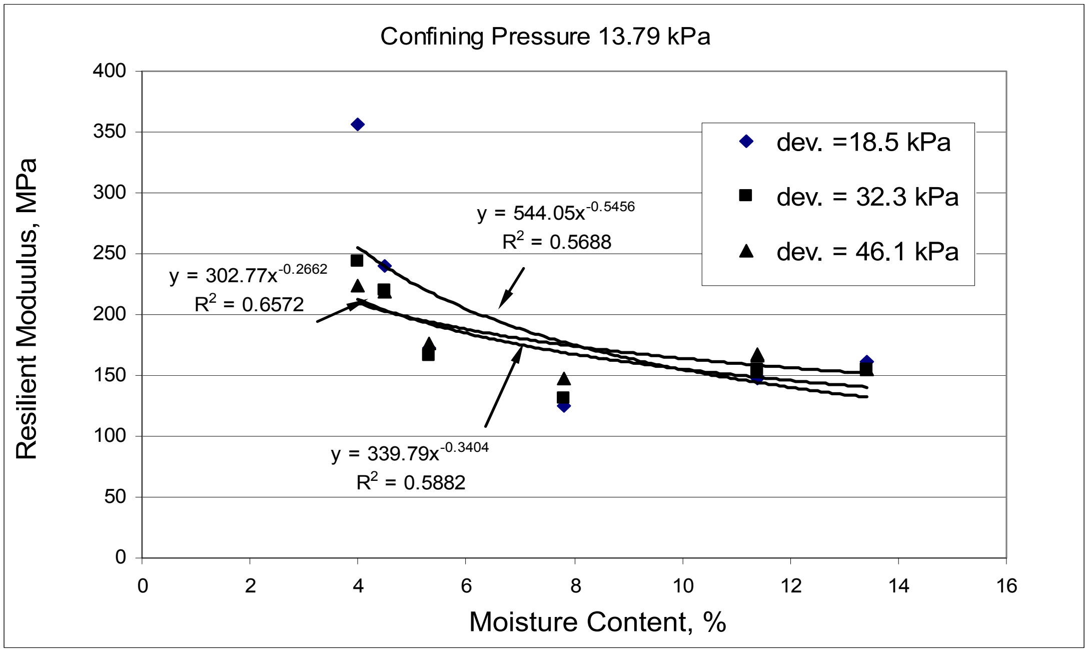


Figure 6.10 Mr vs. Moisture Content at Different Deviator Stresses for SR70 A-3 Soil

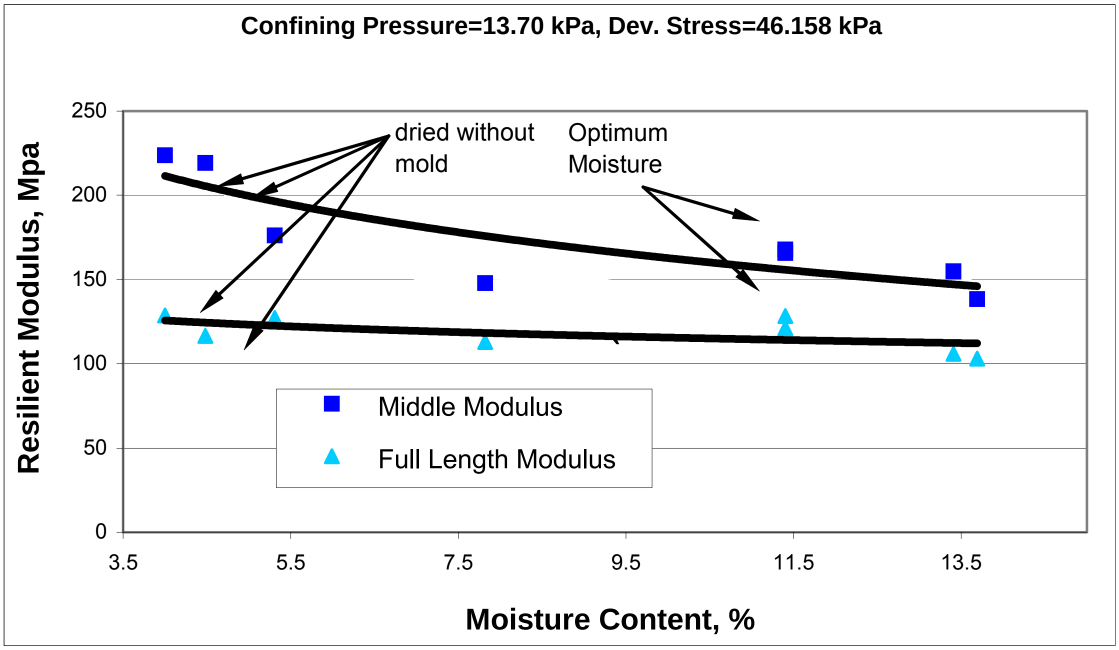


Figure 6.11 Mr vs. Moisture Content for SR70 A-3 Soil

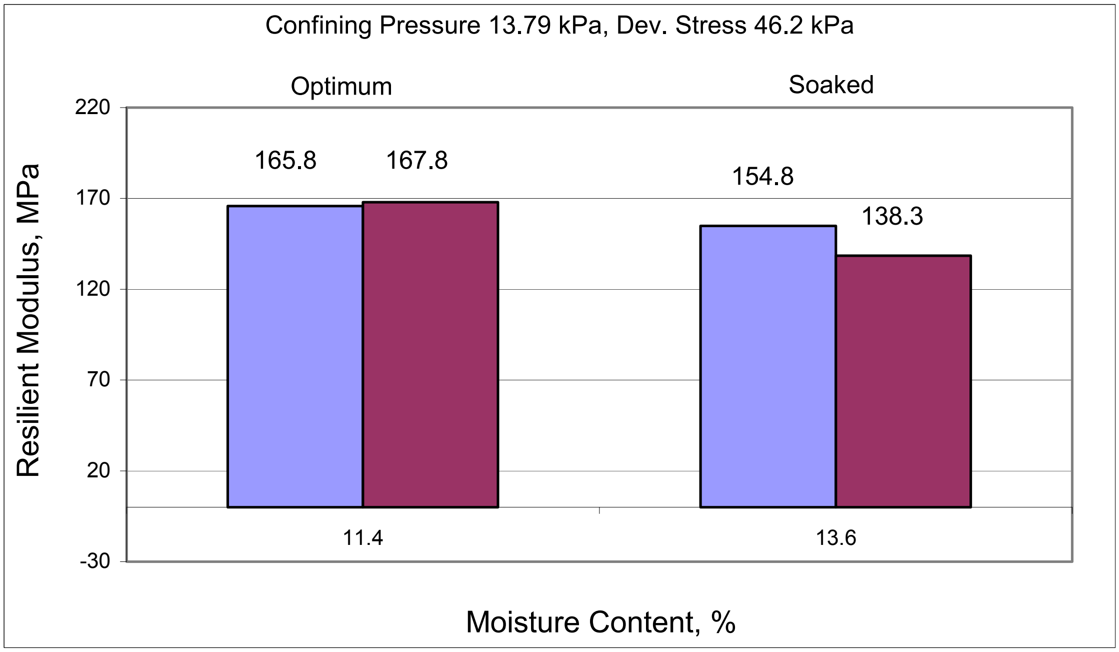


Figure 6.12 Mr vs. Moisture Content at Optimum and Soaked Conditions for SR70 A-3 Soil

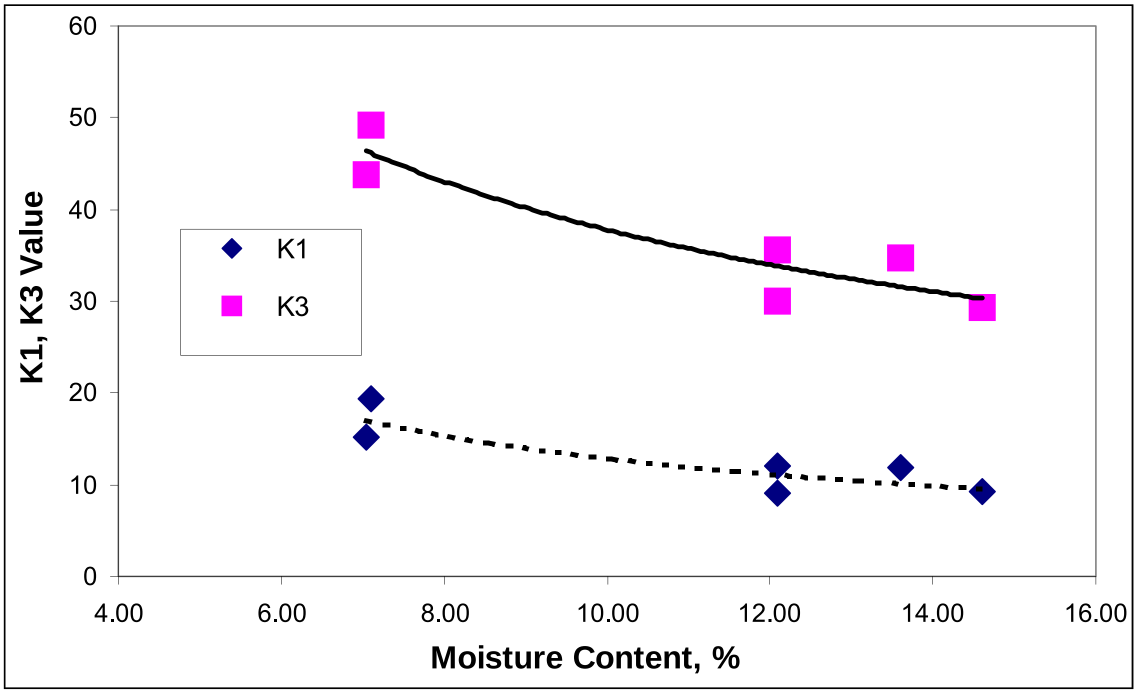


Figure 6.13 k1, k3 vs. Moisture Content for A-2-4 12% Soil

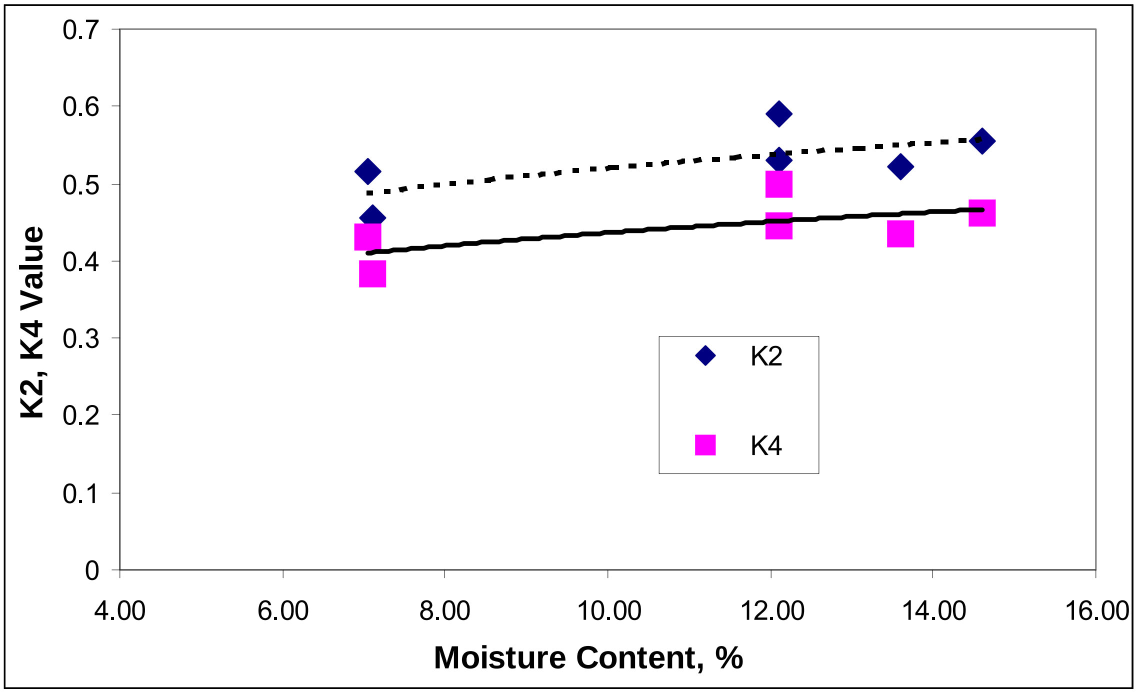


Figure 6.14 k2, k4 vs. Moisture Content for A-2-4 12% Soil

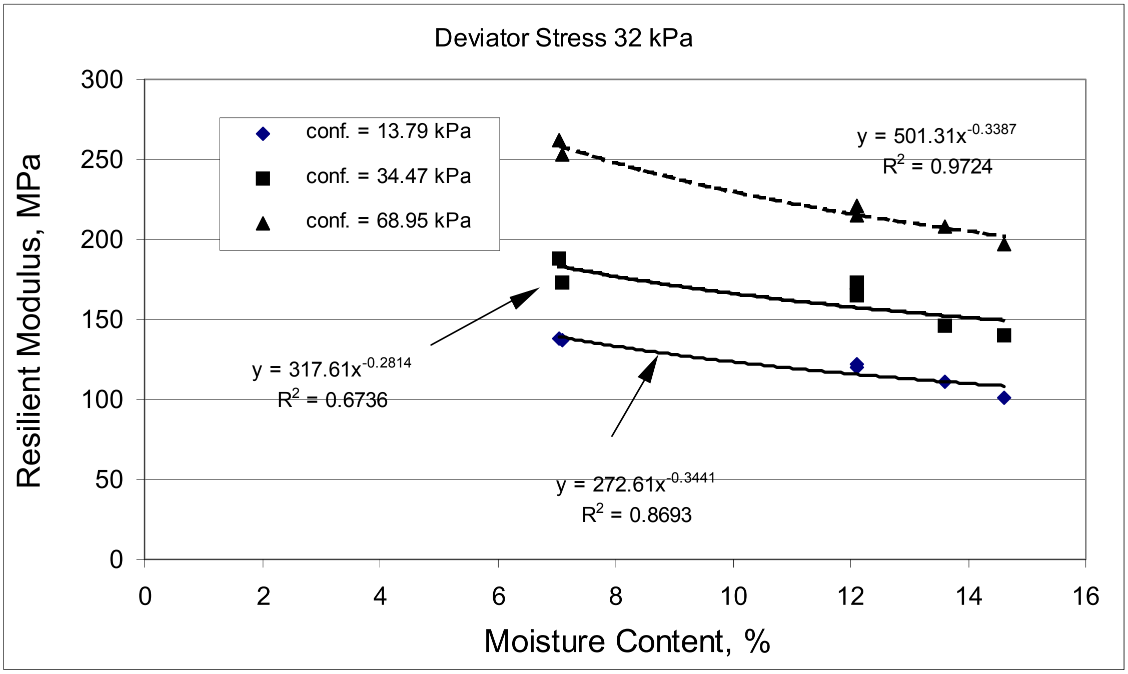


Figure 6.15 Mr vs. Moisture Content at Different Confining Pressures for A-2-4 12% Soil

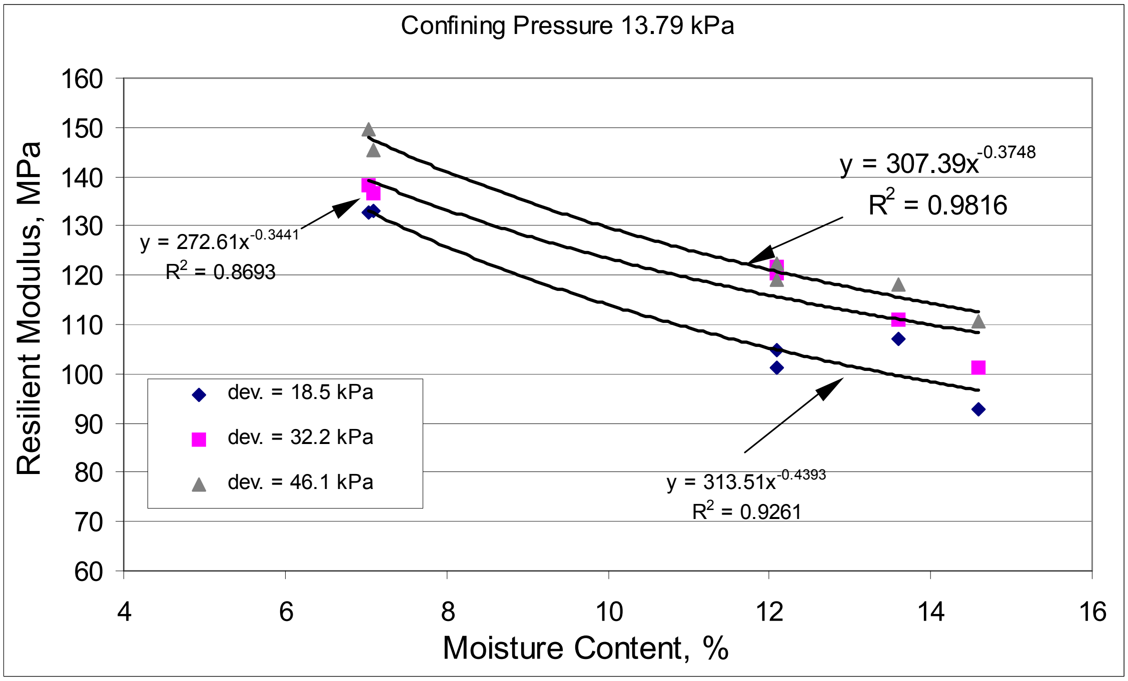


Figure 6.16 Mr vs. Moisture Content at Different Deviator Stresses for A-2-4 12% Soil

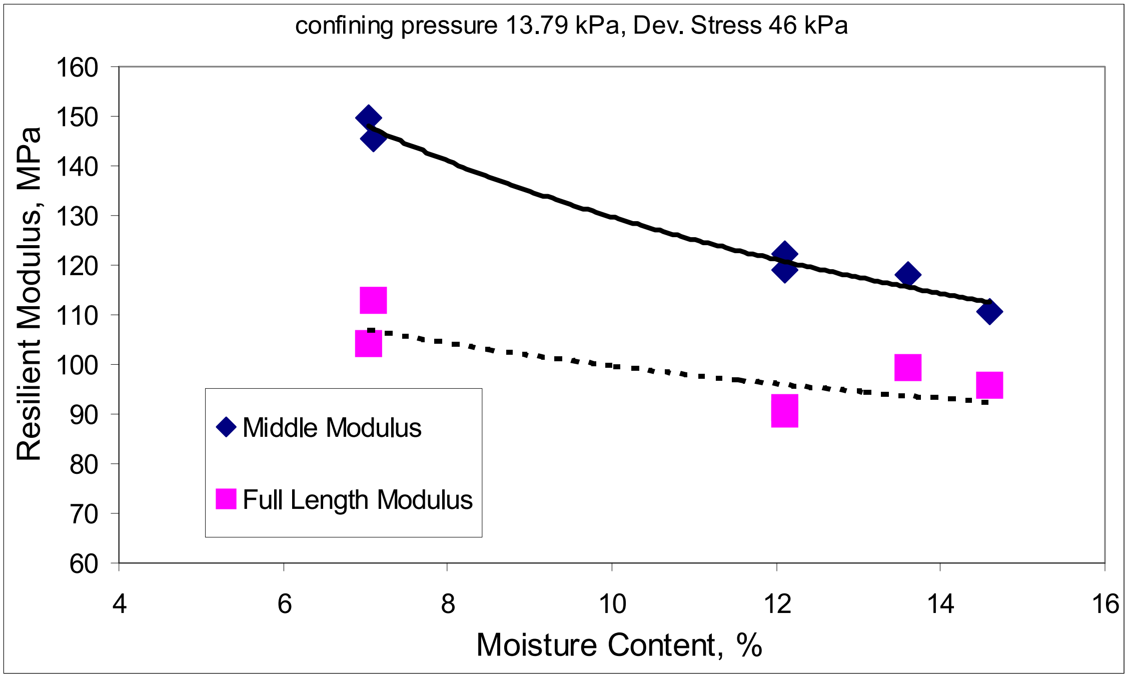


Figure 6.17 Mr vs. Moisture Content for A-2-4 12% Soil

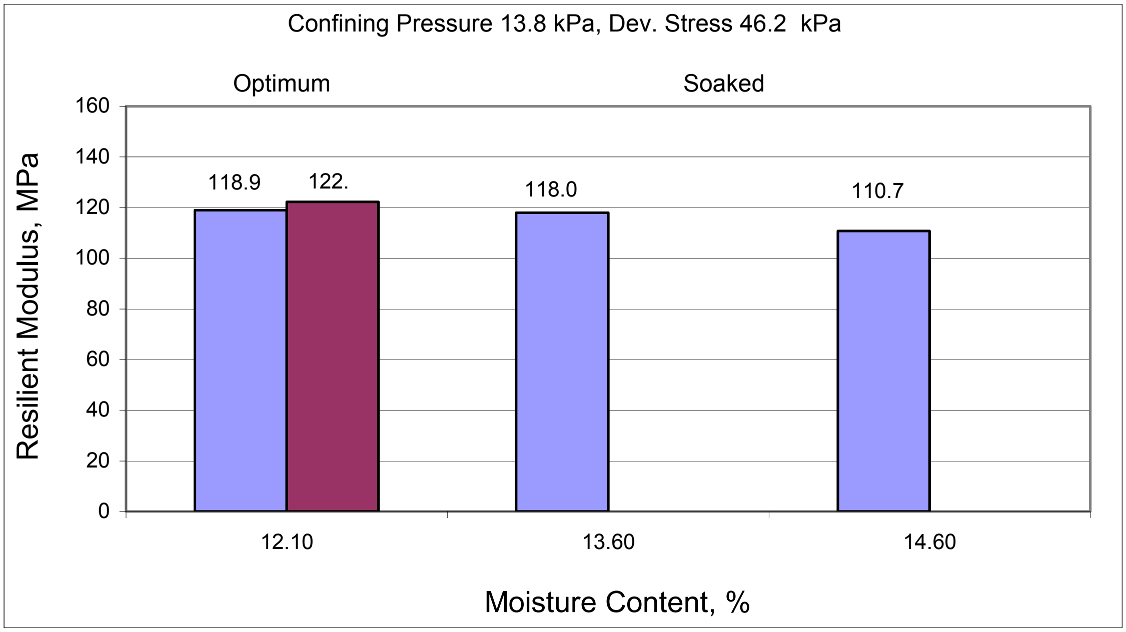


Figure 6.18 Mr vs. Moisture Content at Optimum and Soaked Conditions for A-2-4 12% Soil

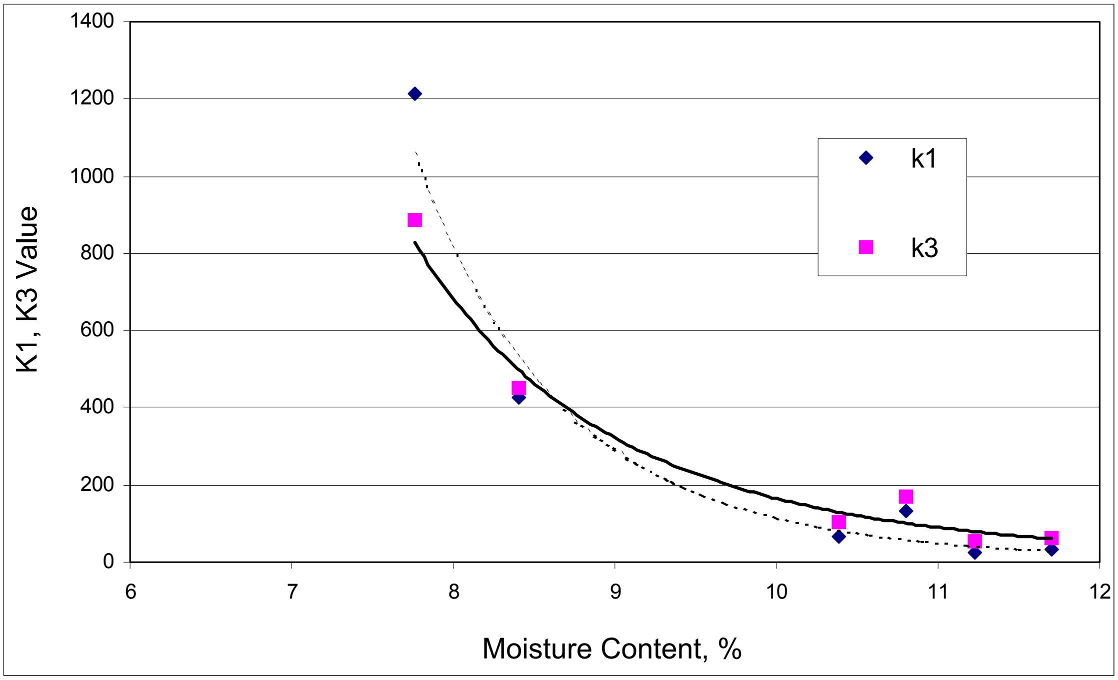


Figure 6.19 k1, k3 vs. Moisture Content for SR70 A-2-4 Soil

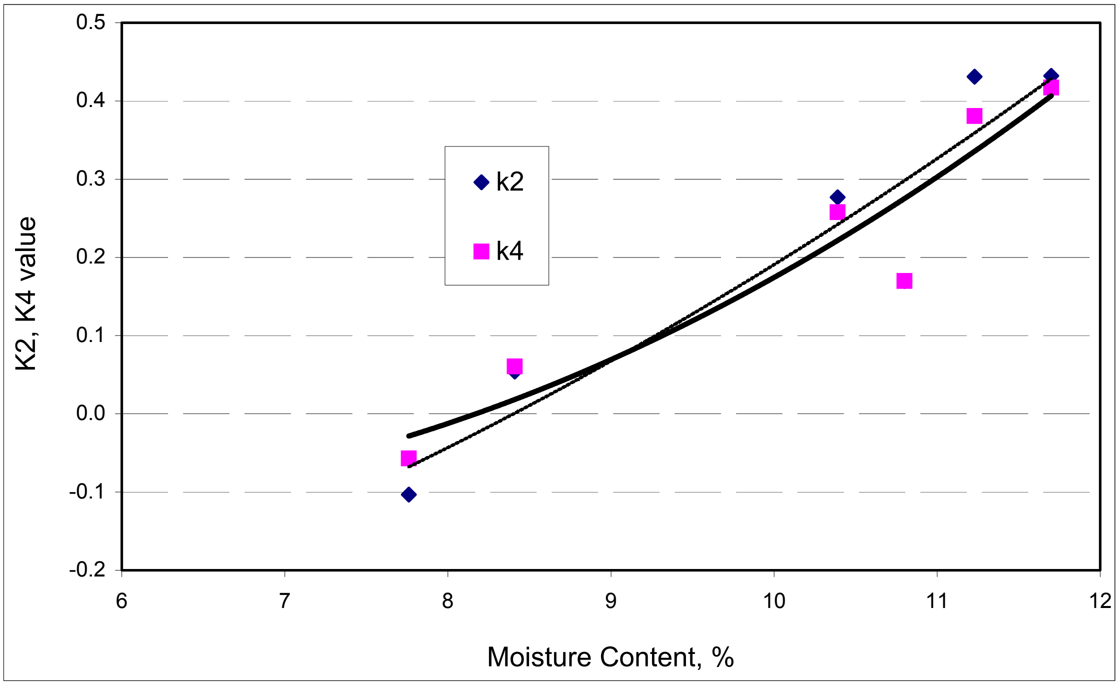


Figure 6.20 k2, k4 vs. Moisture Content for SR70 A-2-4 Soil

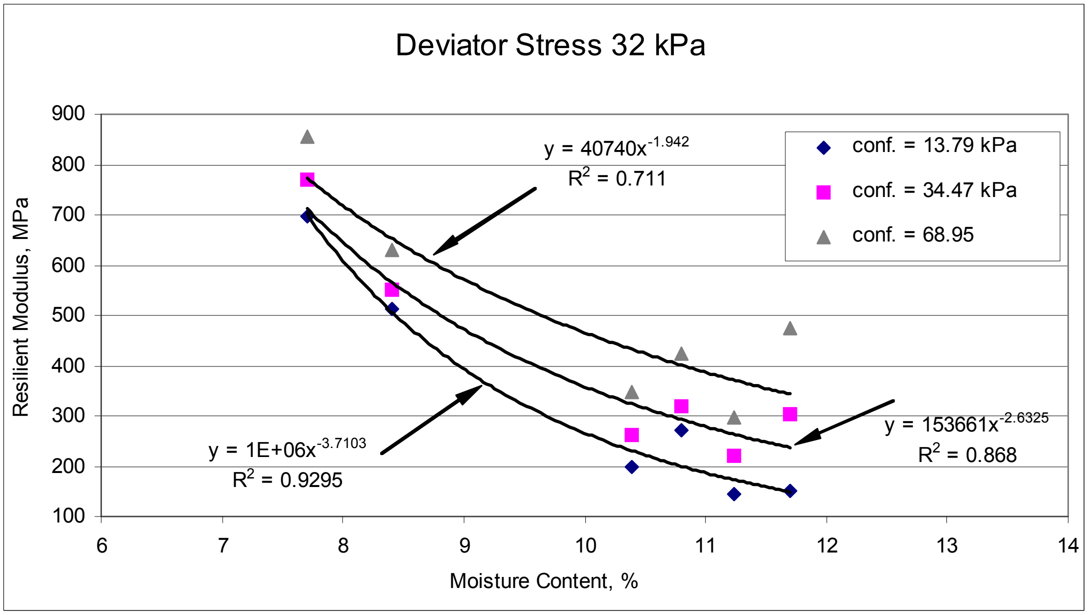


Figure 6.21 Mr vs. Moisture Content at Different Confining Pressures for SR70 A-2-4 Soil

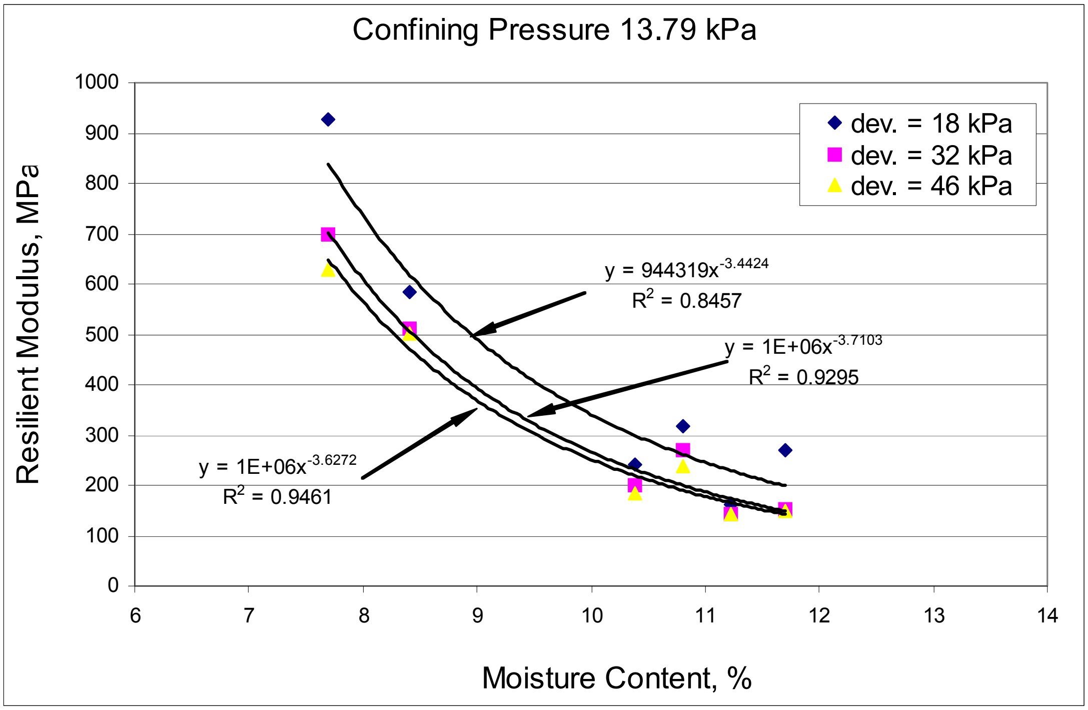


Figure 6.22 Mr vs. Moisture Content at Different Deviator Stresses for SR70 A-2-4 Soil

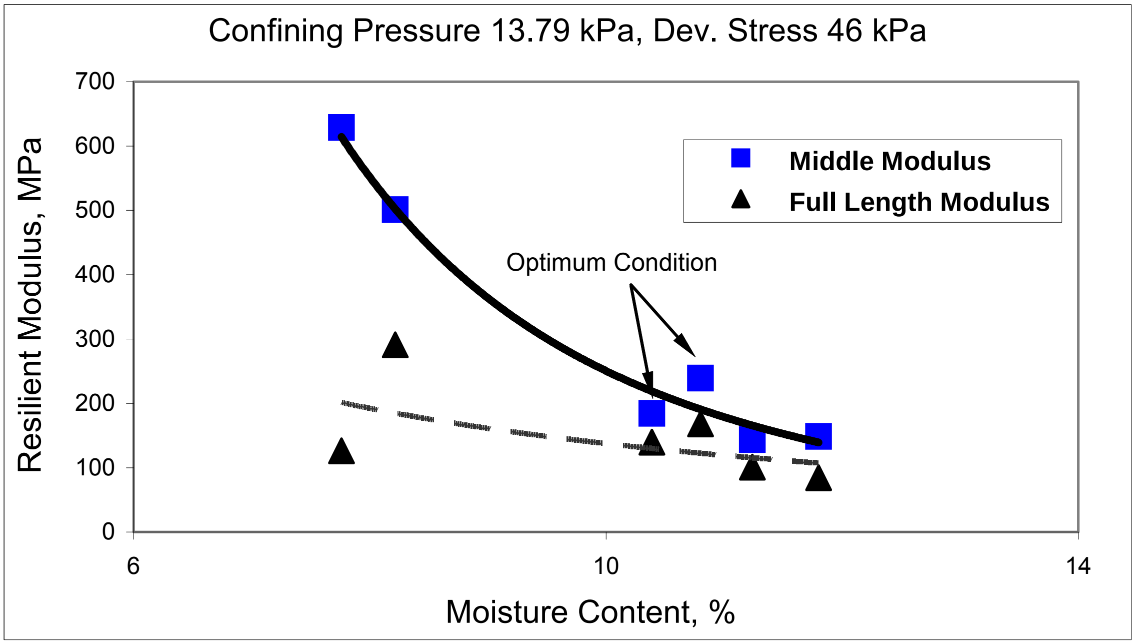


Figure 6.23 Mr vs. Moisture Content for SR70 A-2-4 Soil

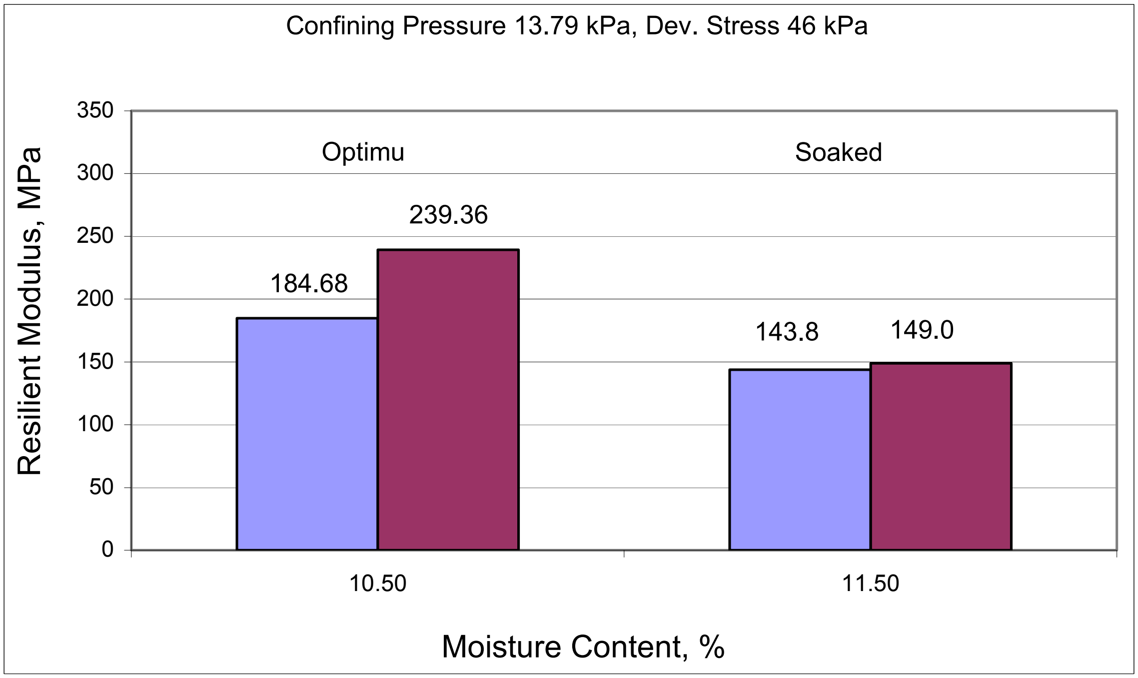


Figure 6.24 Mr vs. Moisture Content at Optimum and Soaked Conditions for SR70 A-2-4 Soil

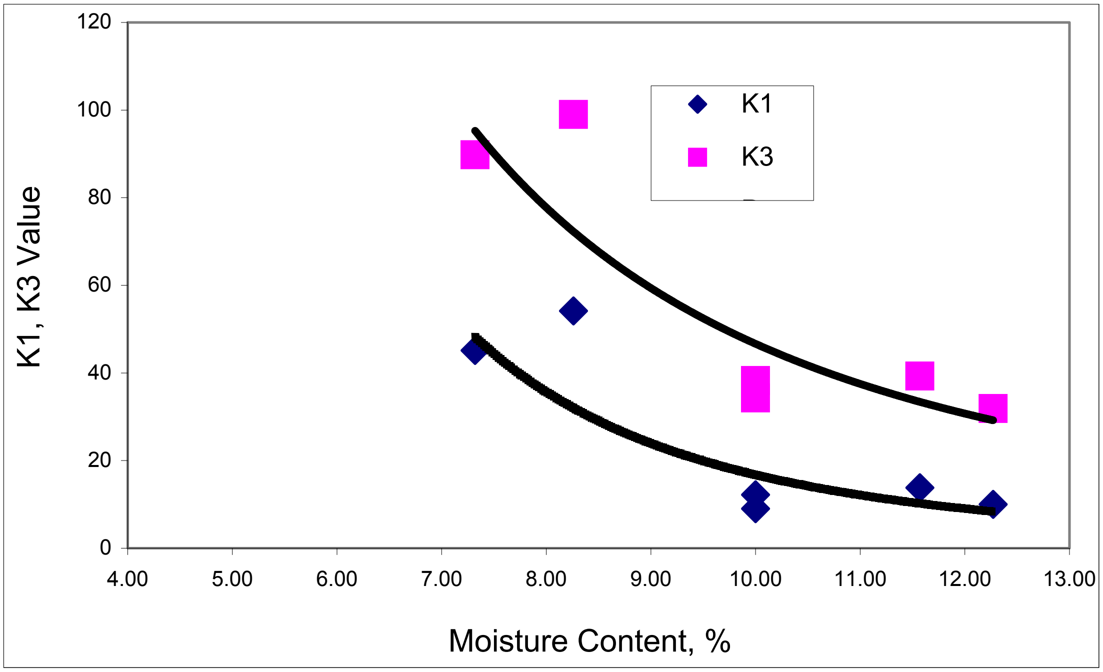


Figure 6.25 k1, k3 vs. Moisture Content for A-2-4 20% Soil

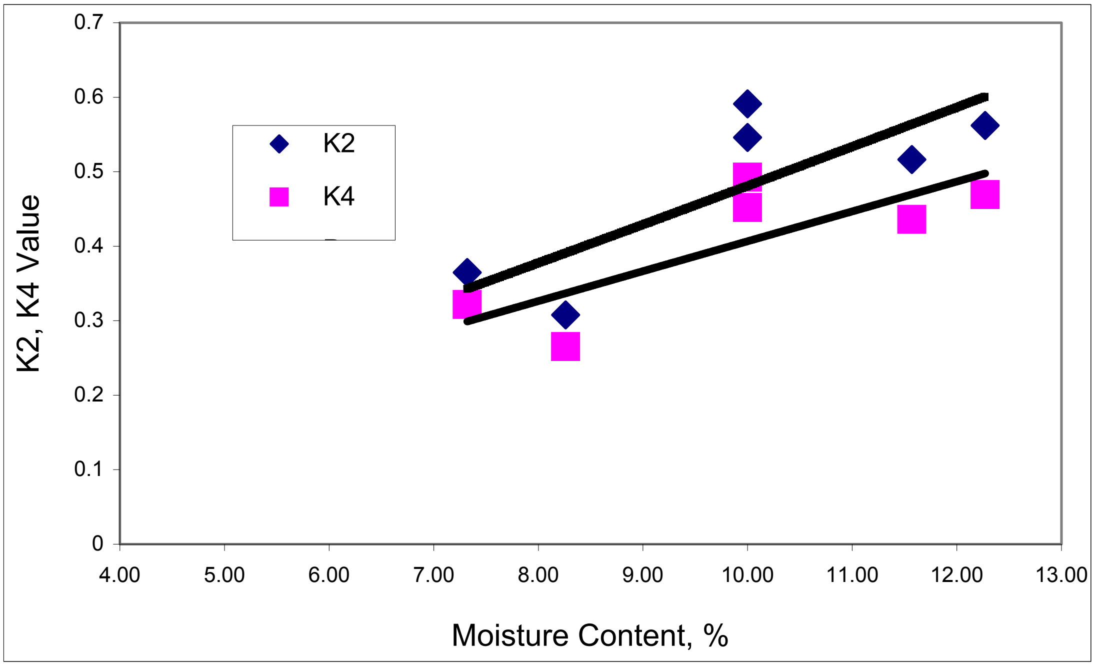


Figure 6.26 k2, k4 vs. Moisture Content for A-2-4 20% Soil

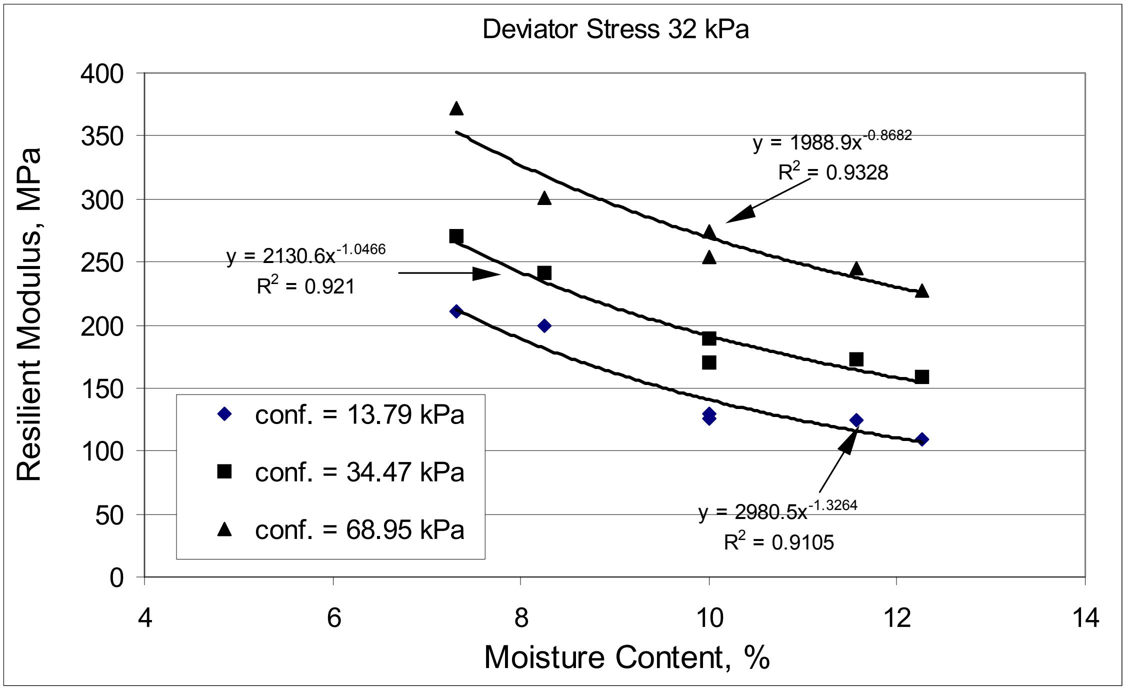


Figure 6.27 Mr vs. Moisture Content at Different Confining Pressures for A-2-4 20% Soil

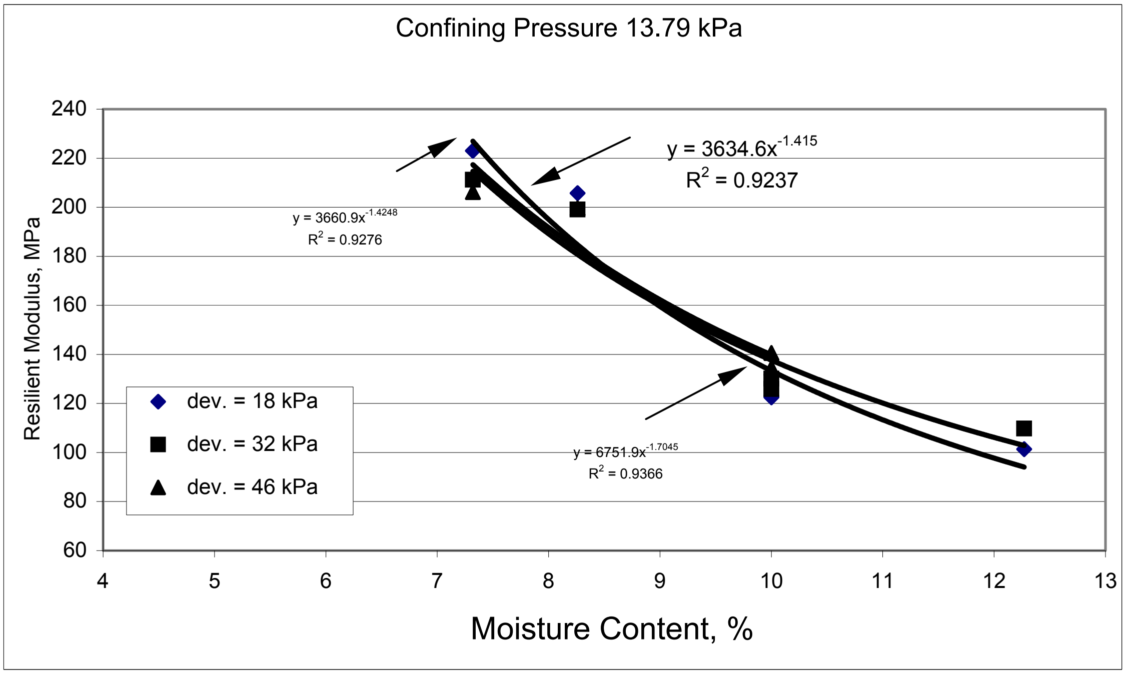


Figure 6.28 Mr vs. Moisture Content at Different Deviator Stresses for A-2-4 20% Soil

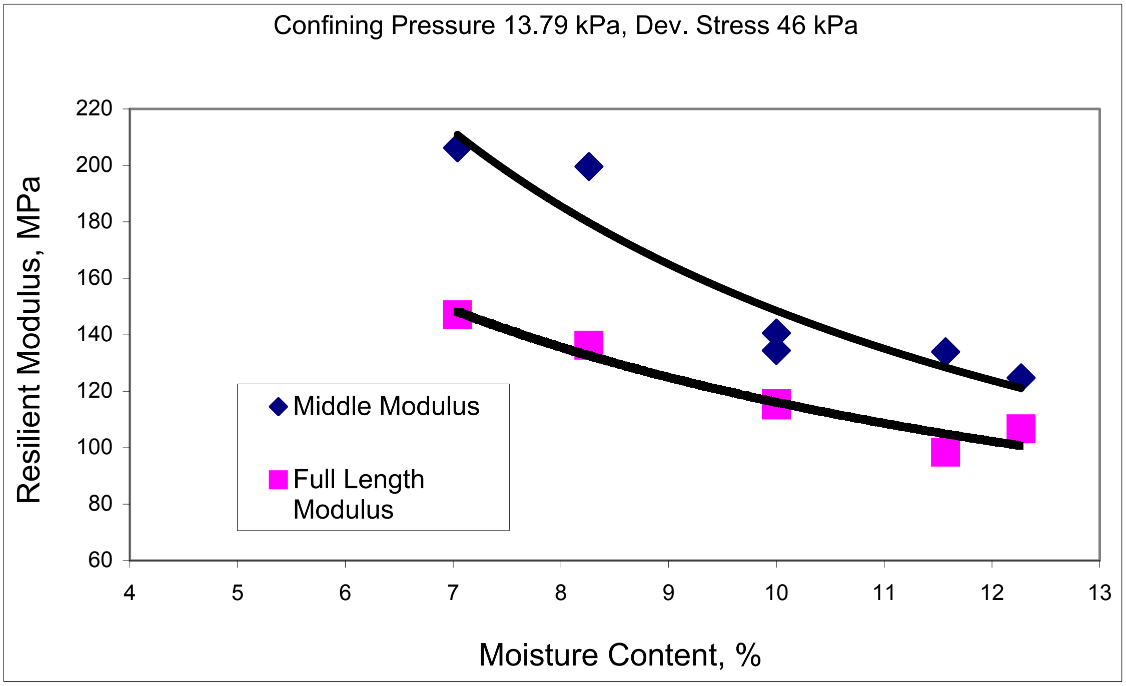


Figure 6.29 Mr vs. Moisture Content for A-2-4 20% Soil

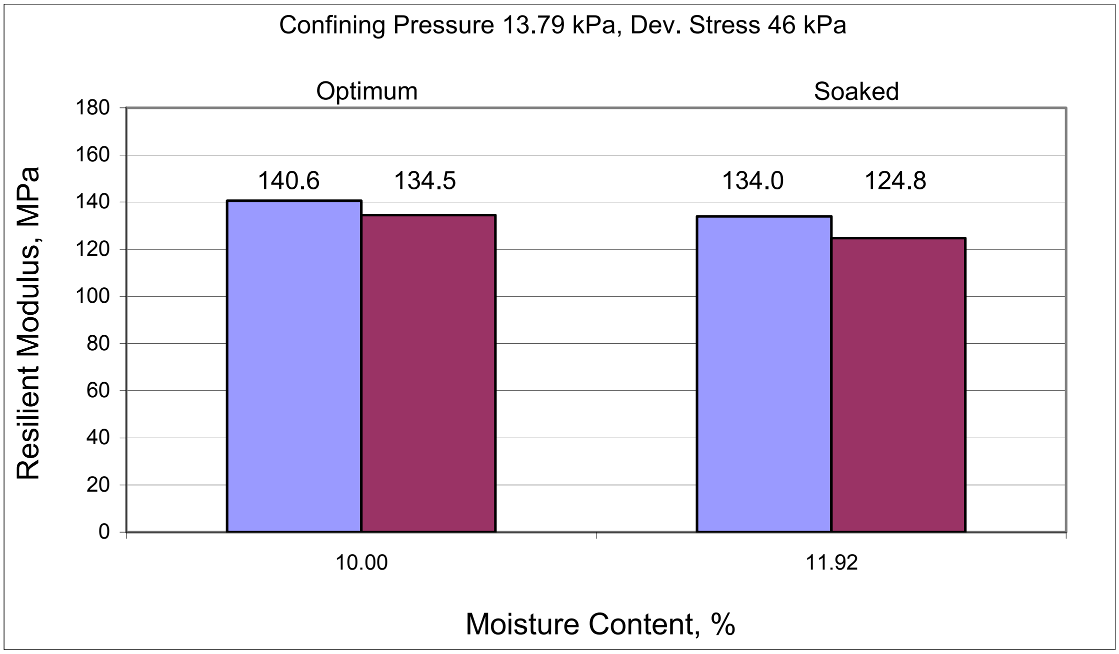


Figure 6.30 Mr vs. Moisture Content at Optimum and Soaked conditions for A-2-4 20% Soil

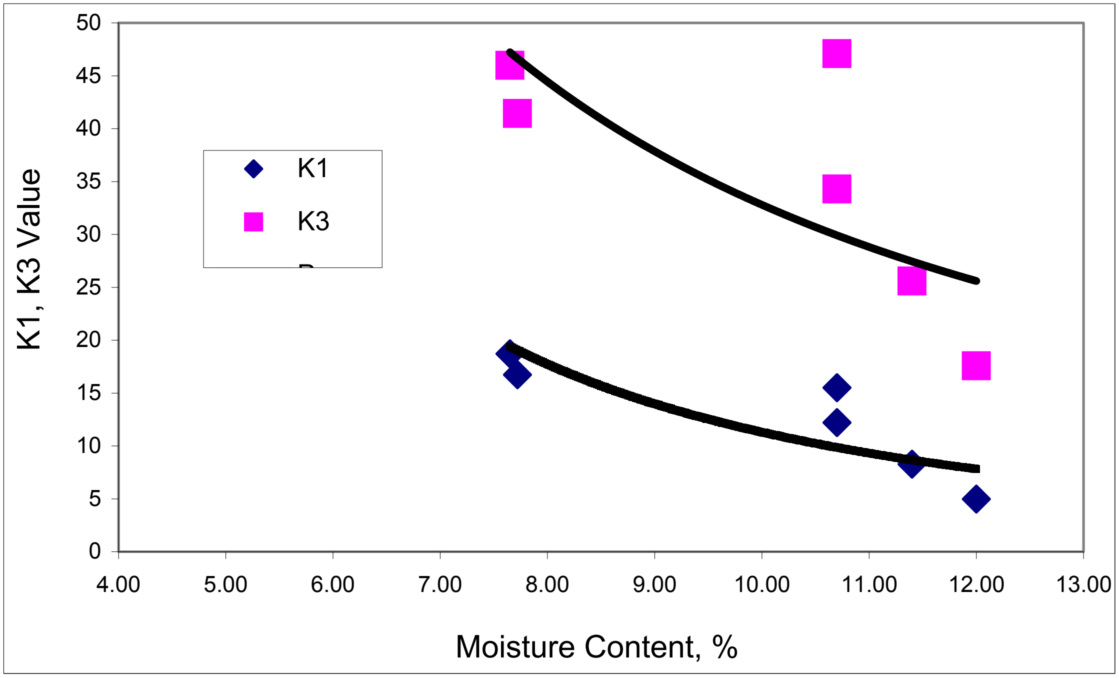


Figure 6.31 k1, k3 vs. Moisture Content for A-2-4 24% Soil

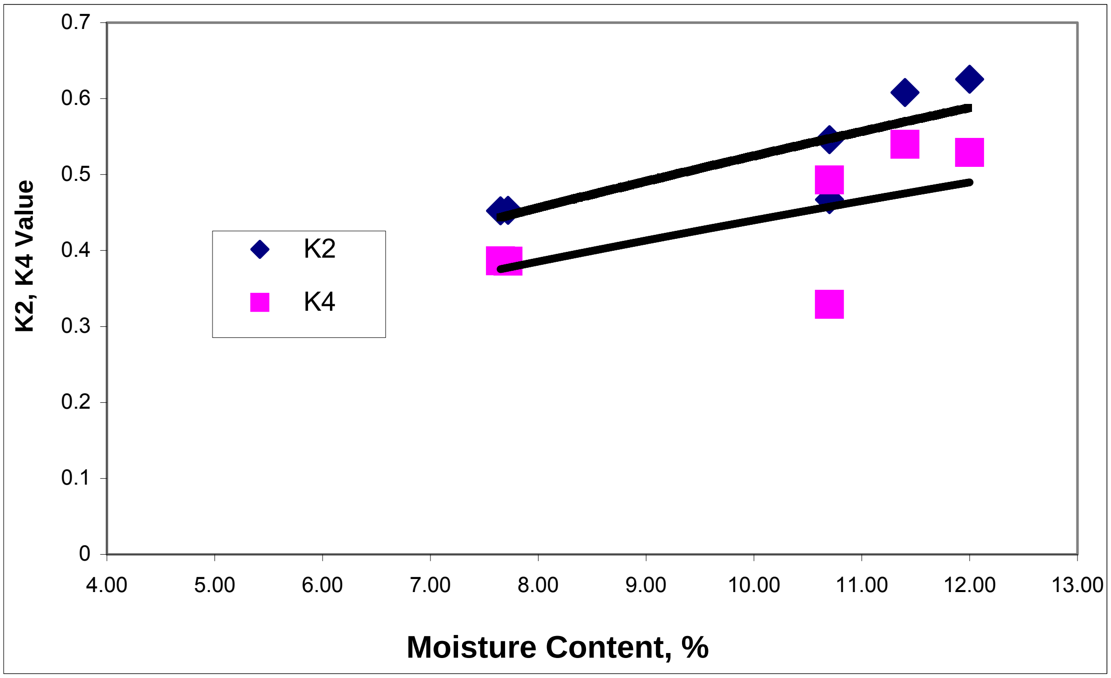


Figure 6.32 k2, k4 vs. Moisture Content for A-2-4 24% Soil

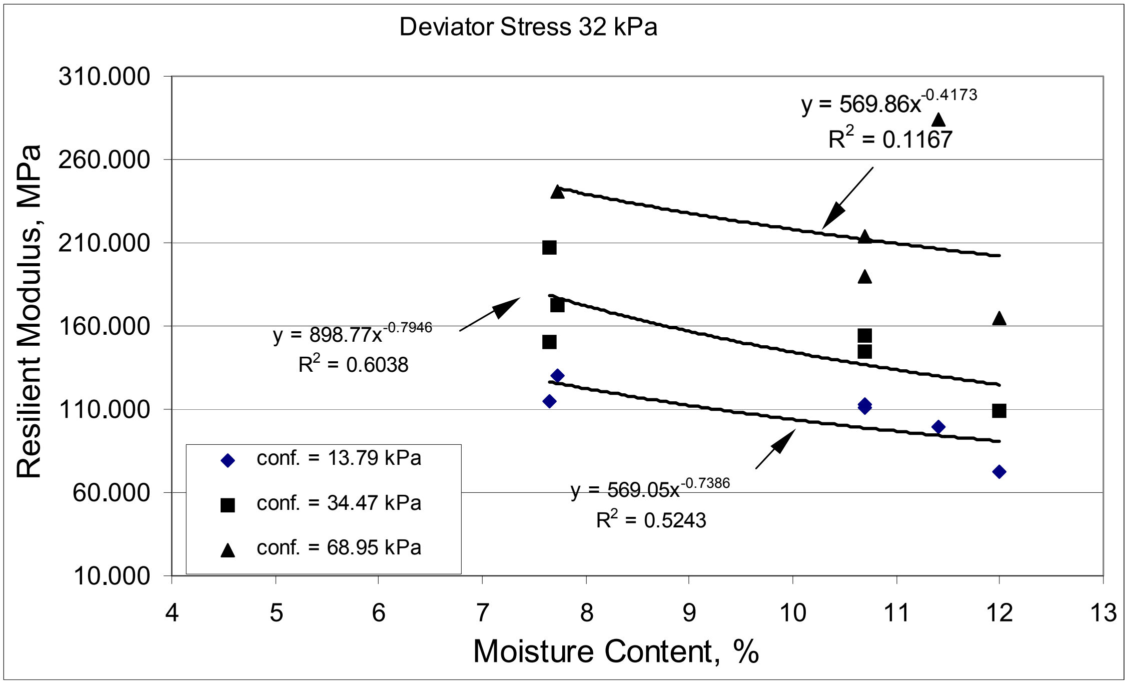


Figure 6.33 Mr vs. Moisture Content at Different Confining Pressures for A-2-4 24% Soil

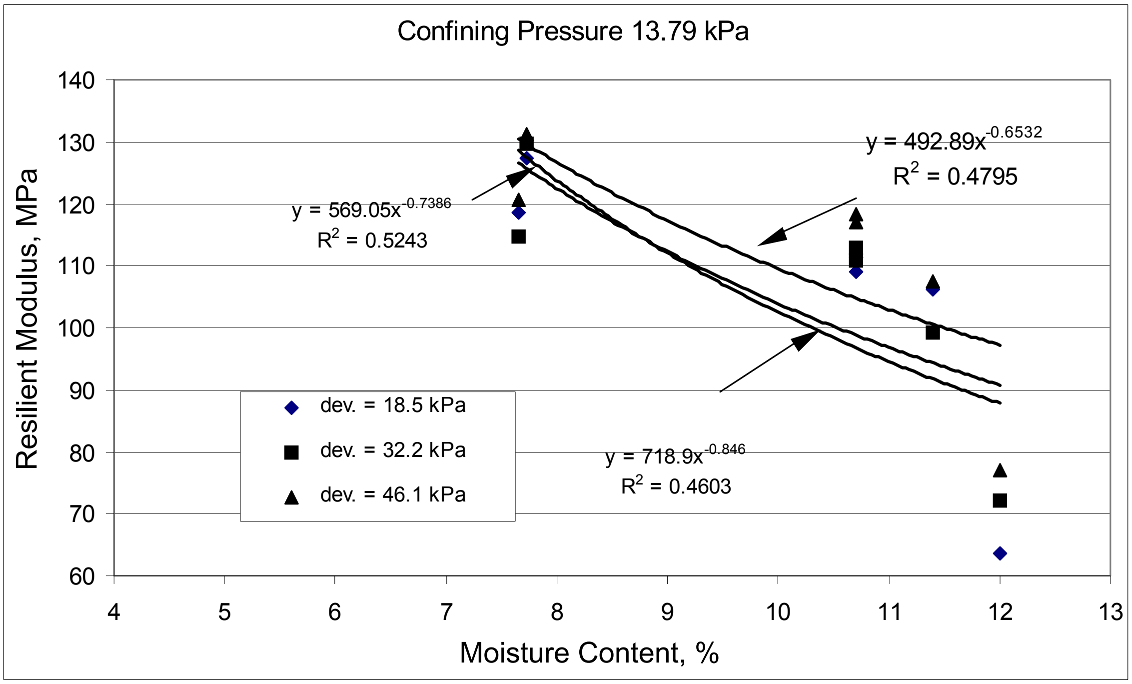


Figure 6.34 Mr vs. Moisture Content at Different Deviator Stresses for A-2-4 24% Soil

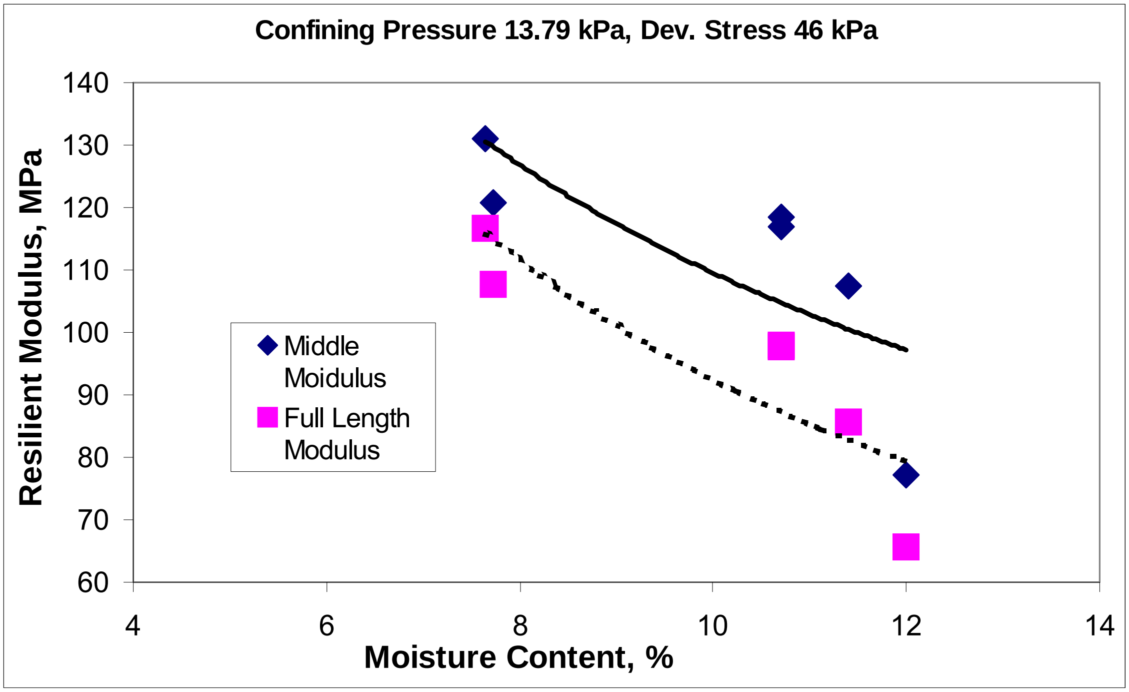


Figure 6.35 Mr vs. Moisture Content for A-2-4 24% Soil

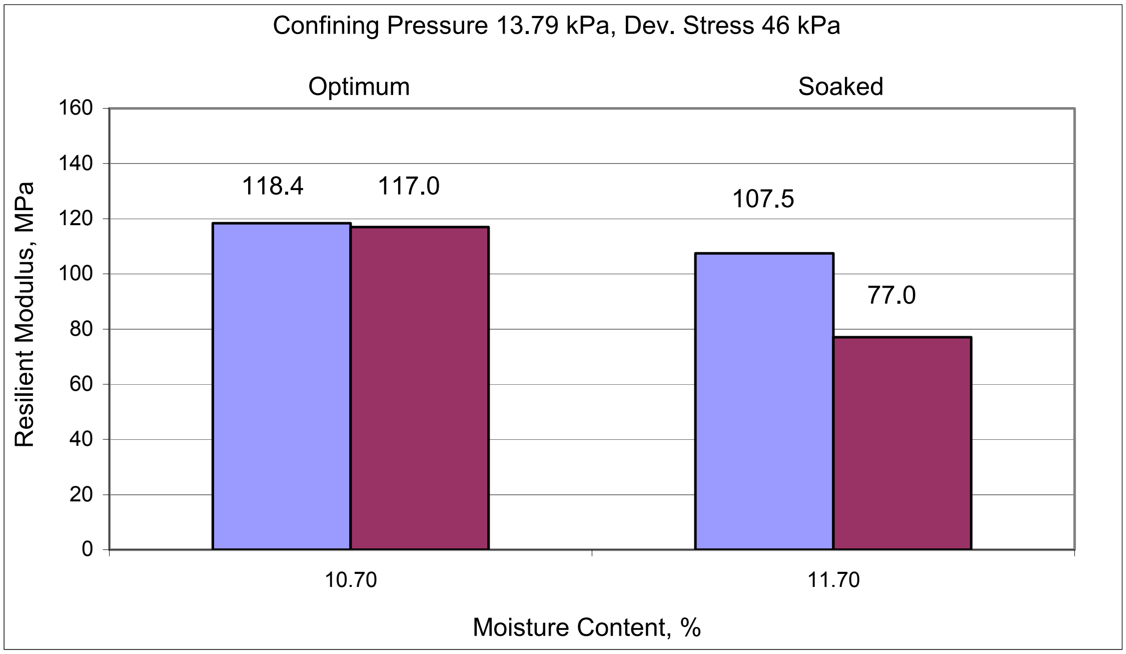


Figure 6.36 Mr vs. Moisture Content at Optimum and Soaked Conditions for A-2-4 24% Soil

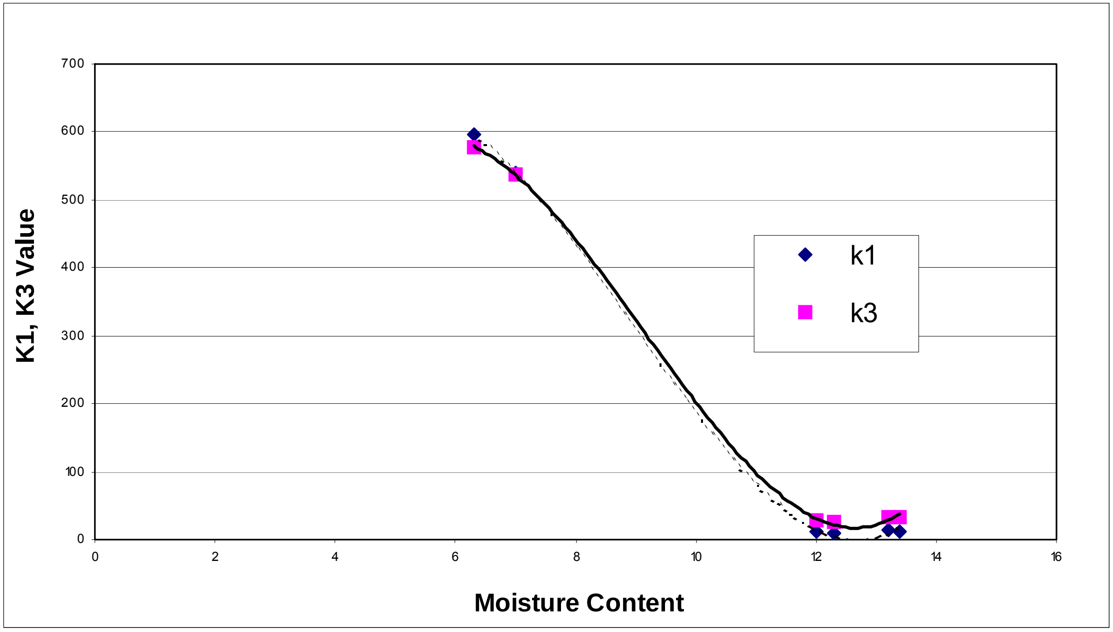


Figure 6.37 k1, k3 vs. Moisture Content for A-2-4 30% Soil

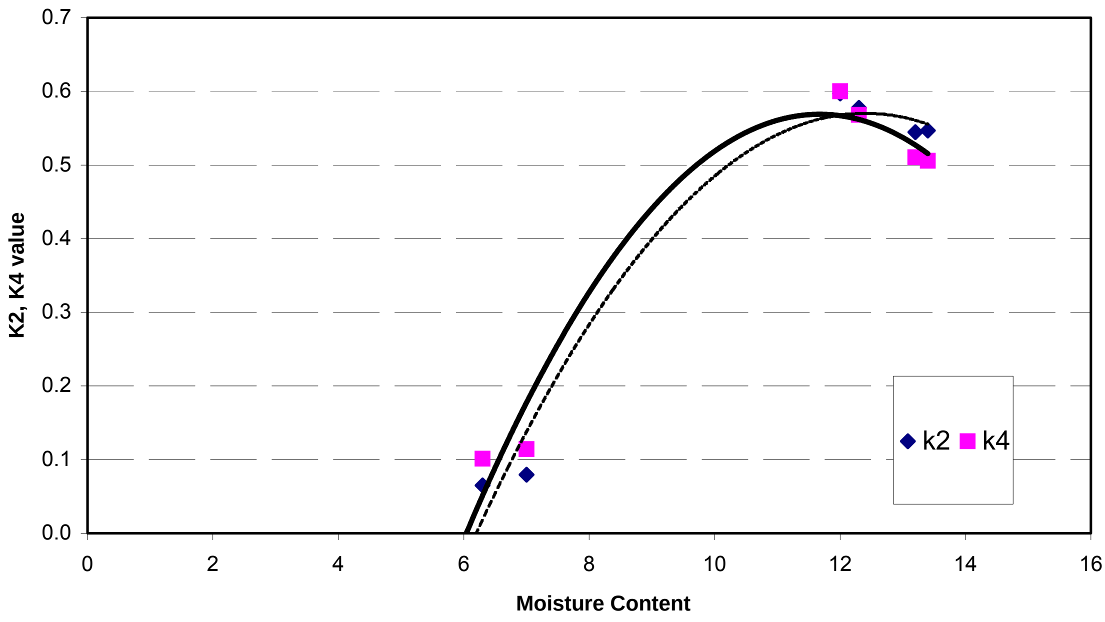


Figure 6.38 k2, k4 vs. Moisture Content for A-2-4 30% Soil

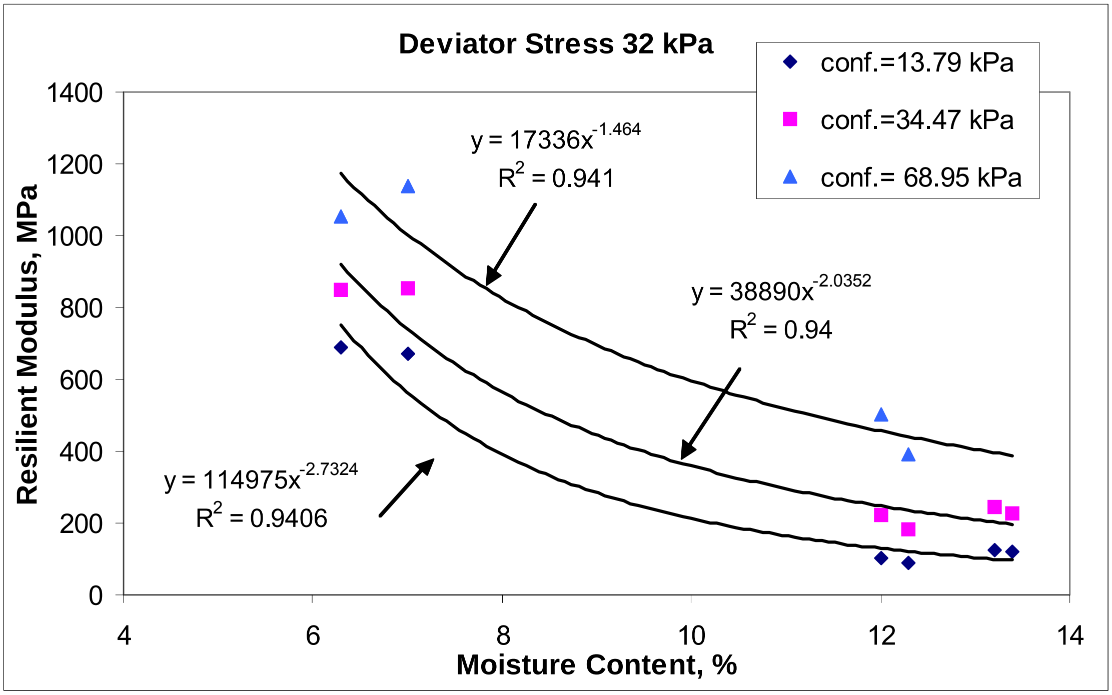


Figure 6.39 Mr vs. Moisture Content at Different Confining Pressures for A-2-4 30% Soil

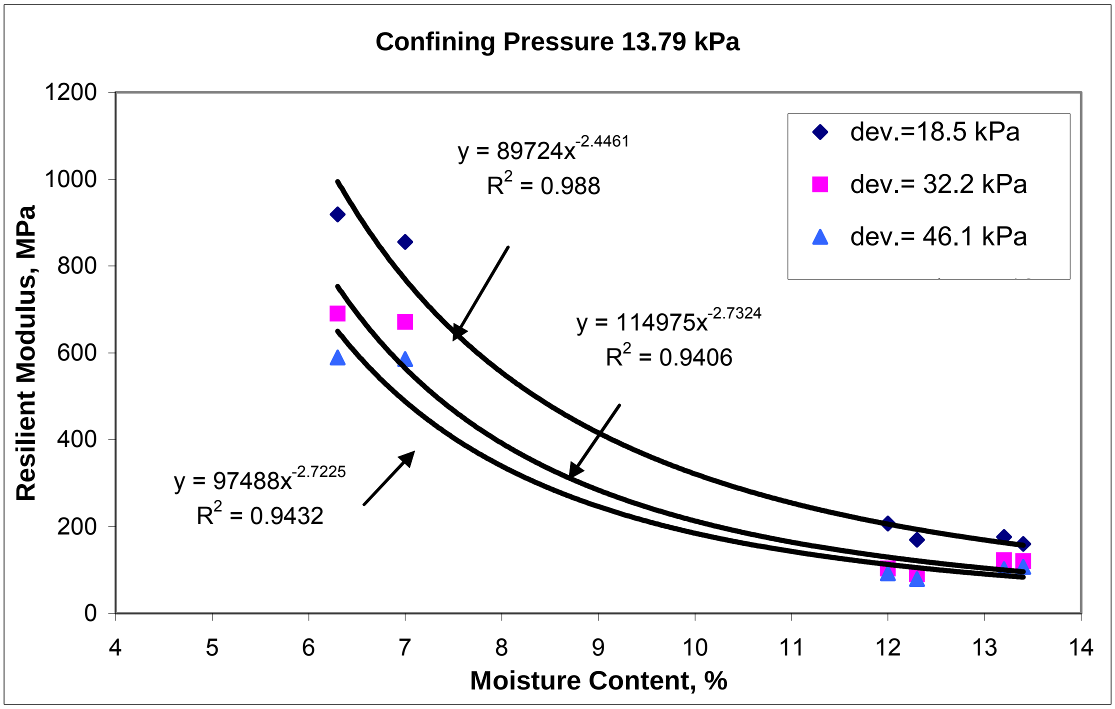


Figure 6.40 Mr vs. Moisture Content at Different Deviator Stresses for A-2-4 30% Soil

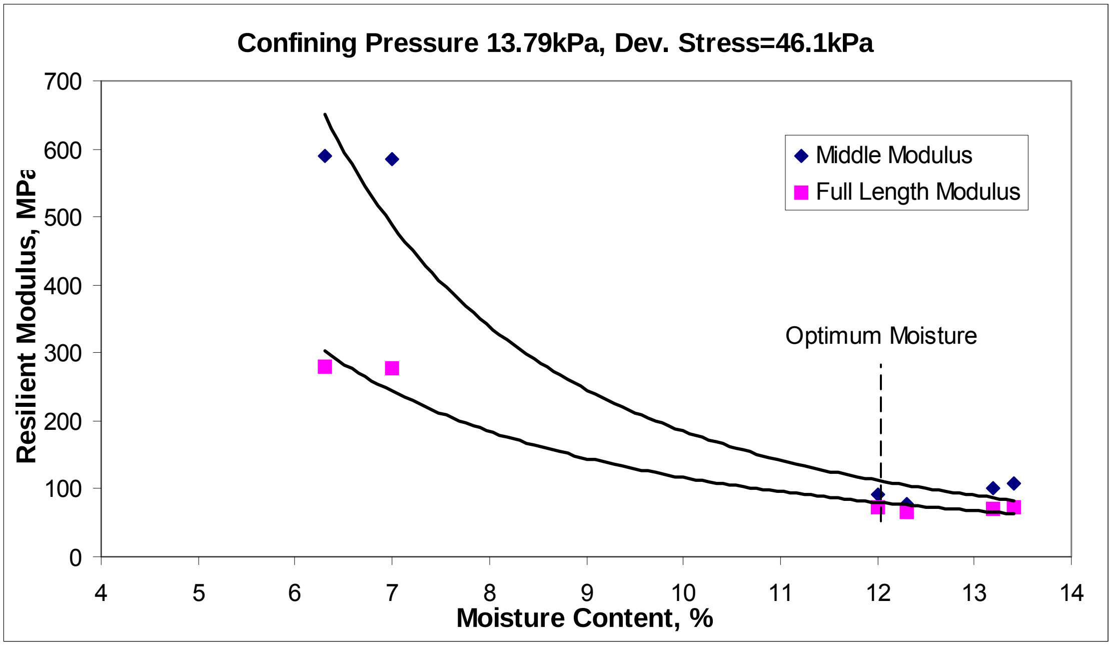


Figure 6.41 Mr vs. Moisture Content for A-2-4 30% Soil

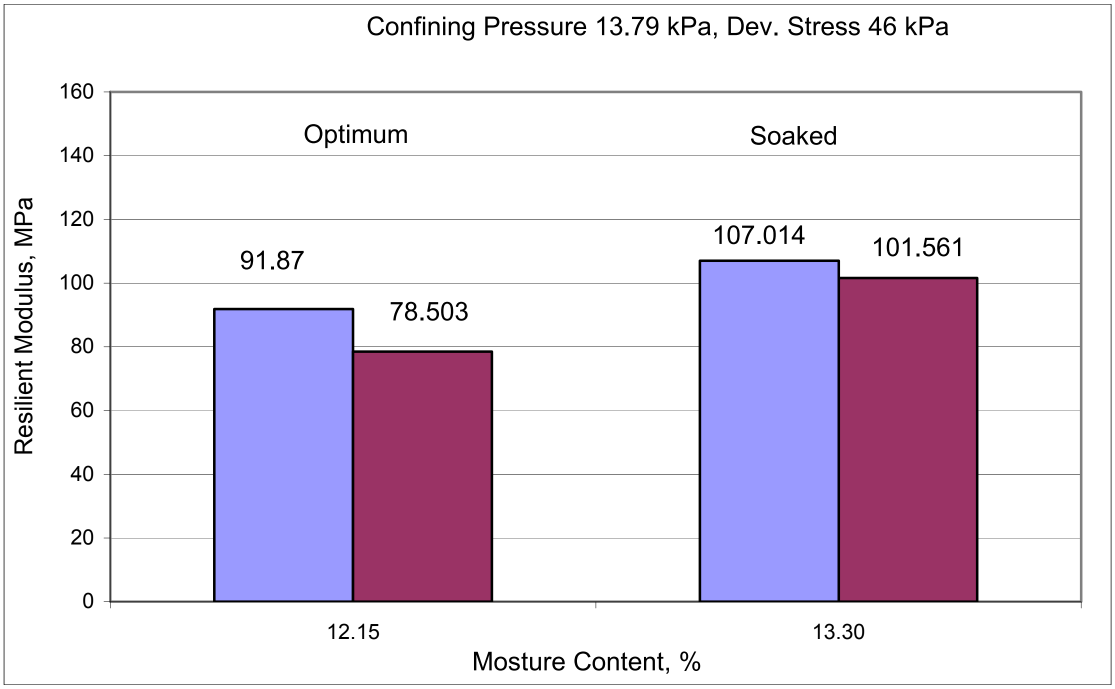


Figure 6.42 Mr vs. Moisture Content at Optimum and Soaked conditions for A-2-4 30% Soil

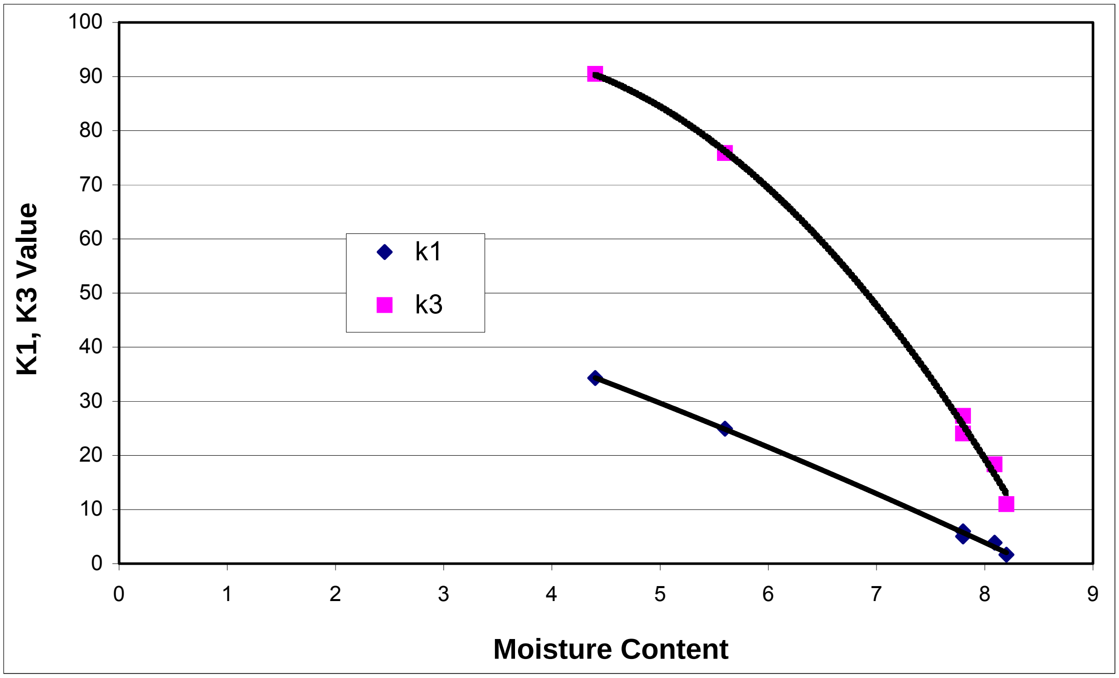


Figure 6.43 k1, k3 vs. Moisture Content for Crushed Miami Oolite A-1 Soil

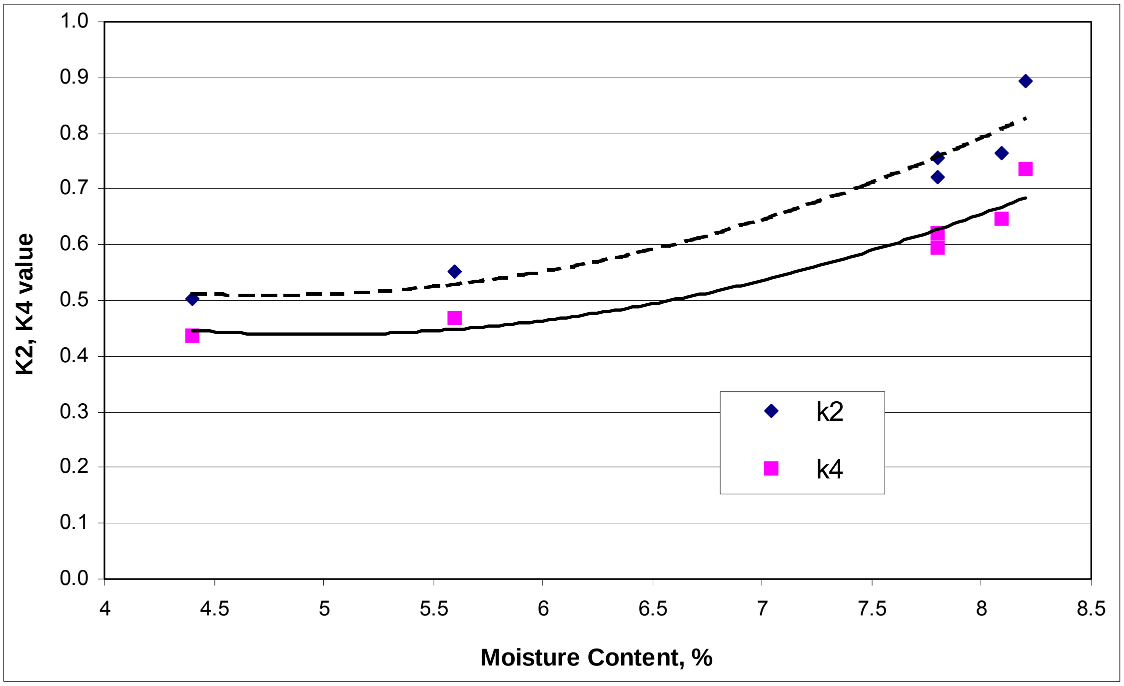


Figure 6.44 k2, k4 vs. Moisture Content for Crushed Miami Oolite A-1 Soil

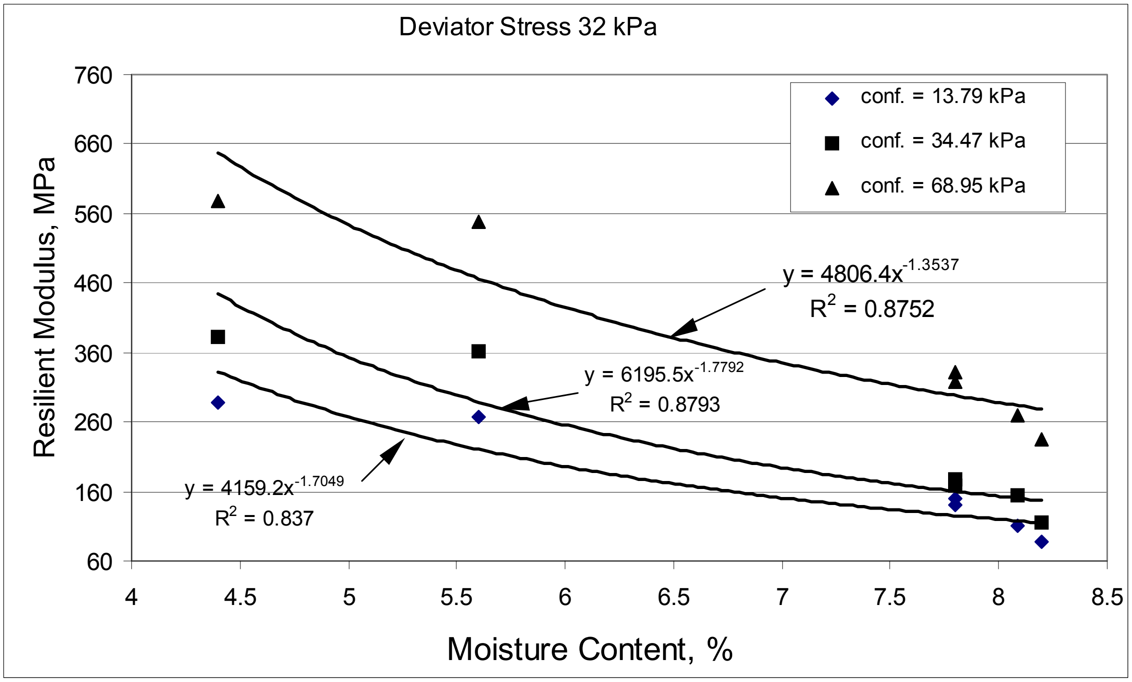


Figure 6.45 Mr vs. Moisture Content at Different Confining Pressures for Crushed Miami Oolite A-1 Soil

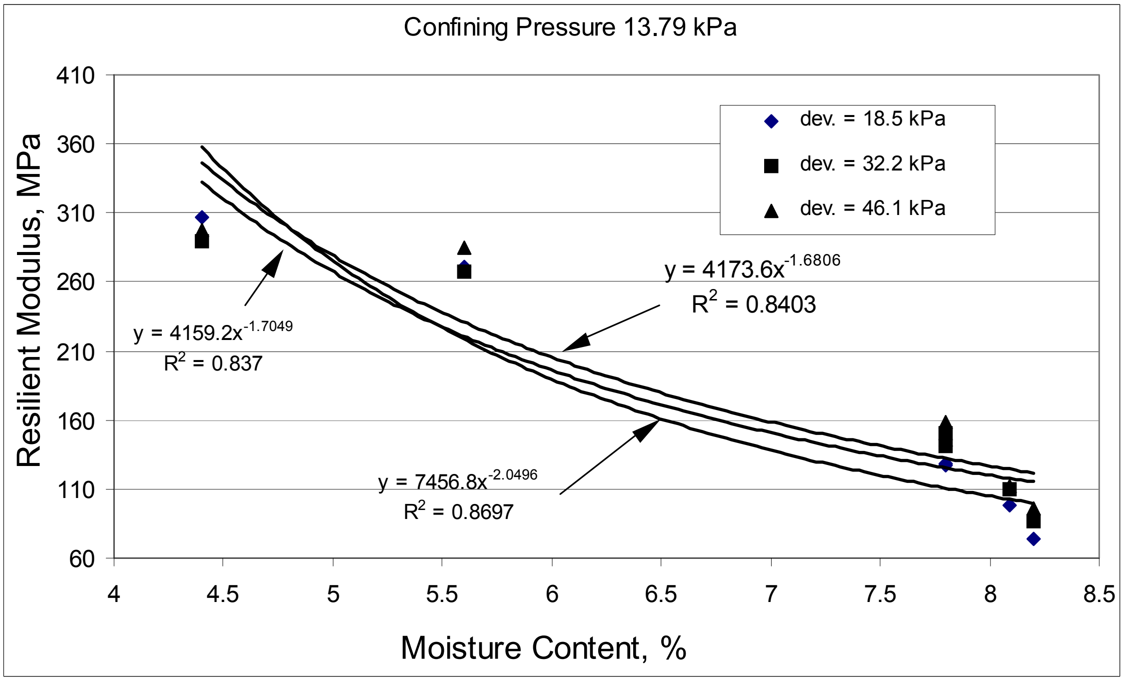


Figure 6.46 Mr vs. Moisture Content at Different Deviator Stresses for Crushed Miami Oolite A-1 Soil

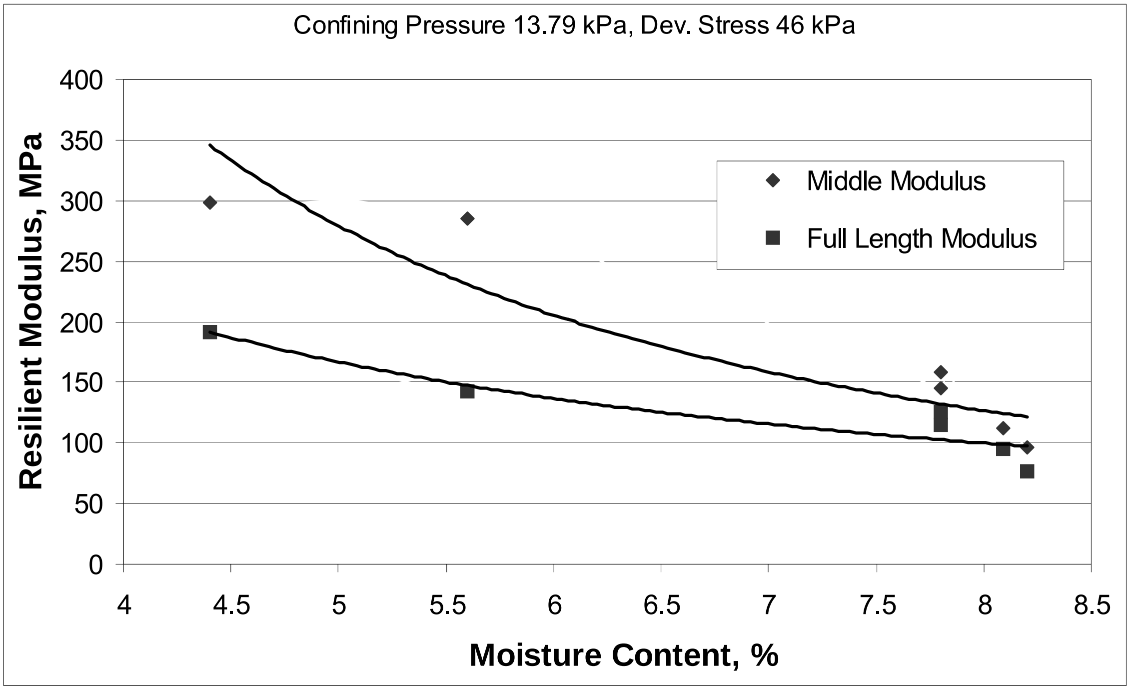


Figure 6.47 Mr vs. Moisture Content for Crushed Miami Oolite A-1 Soil

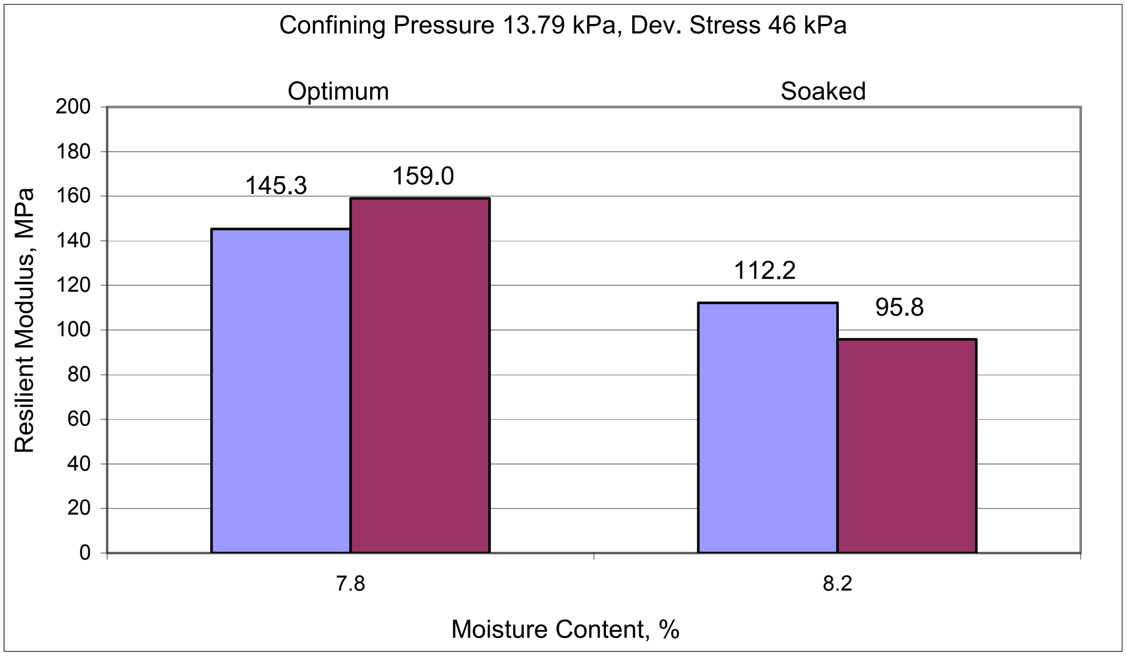


Figure 6.48 Mr vs. Moisture Content at Optimum and Soaked Conditions for Crushed Miami Oolite A-1 Soil

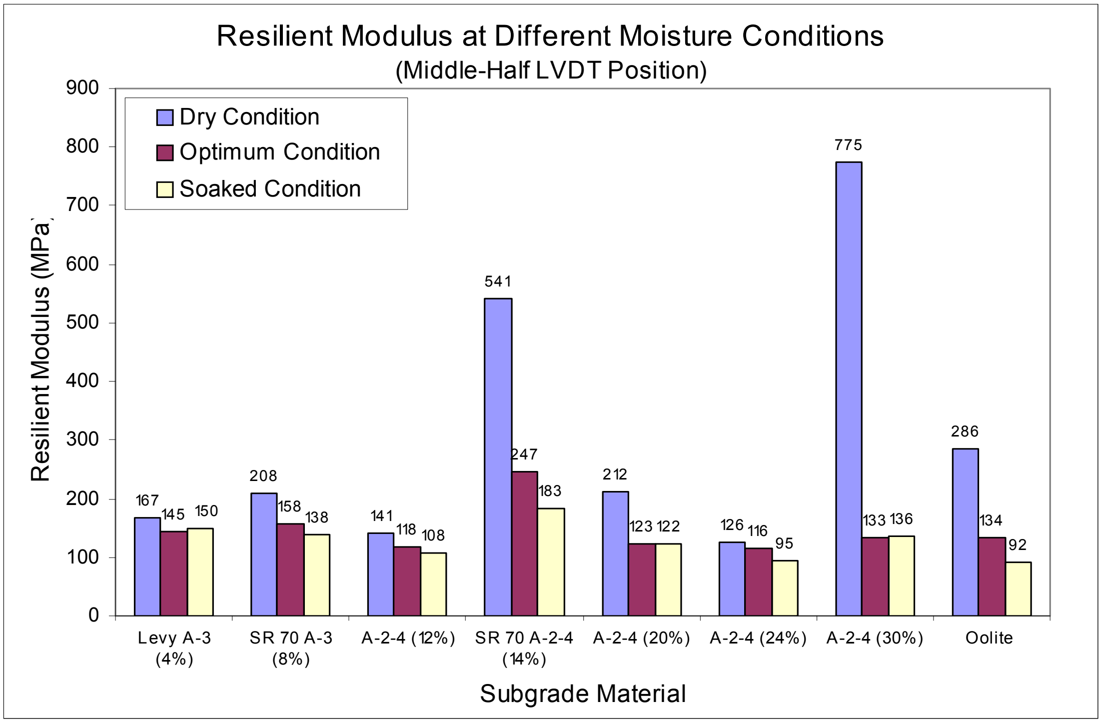


Figure 6.49 Summary of Average Lab Resilient Moduli (Middle-Half LVDT Position)

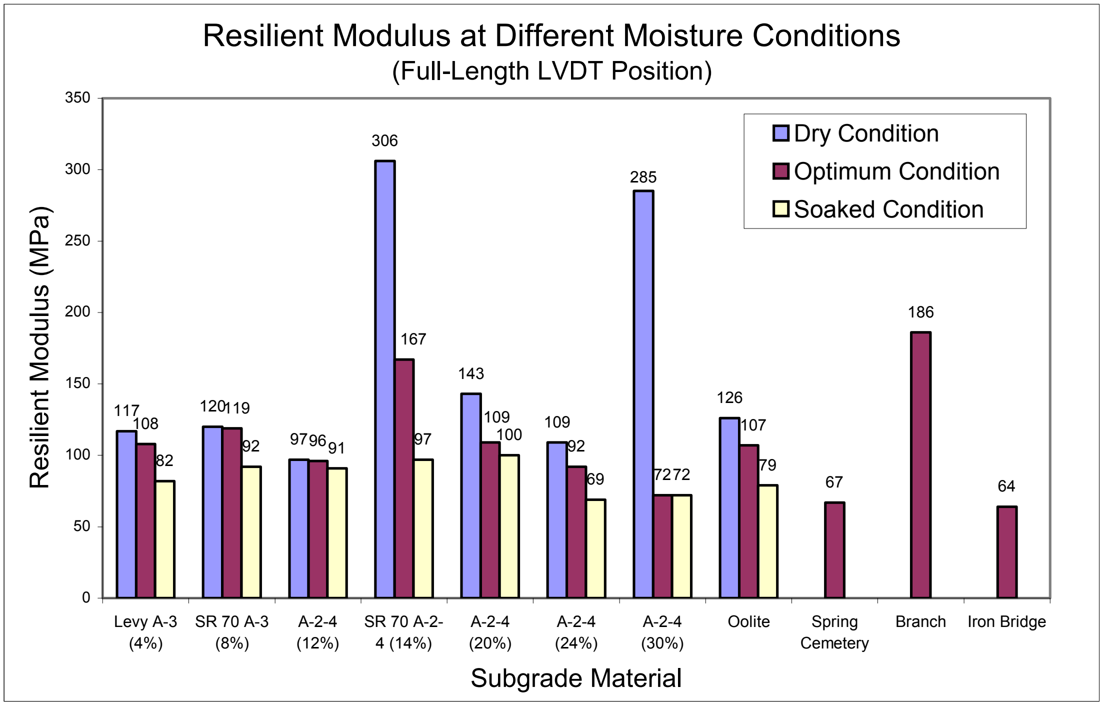


Figure 6.50 Summary of Average Lab Resilient Moduli (Full-Length LVDT Position)

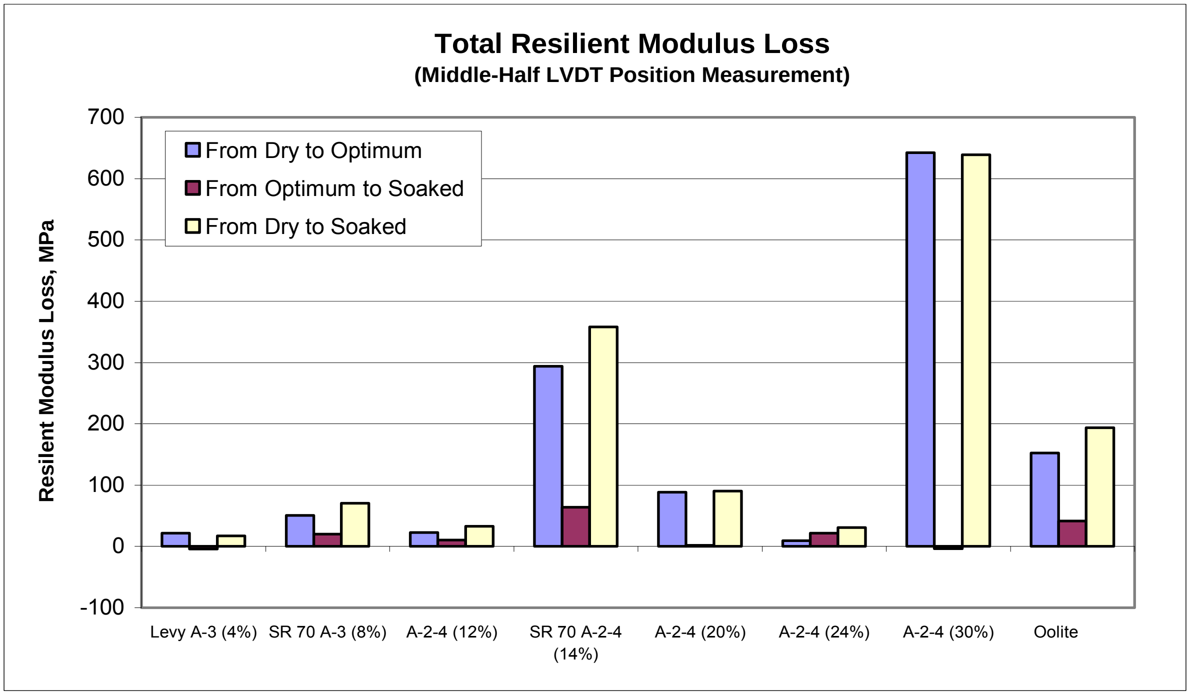


Figure 6.51 Total Resilient Modulus Loss (Middle-Half LVDT Position)

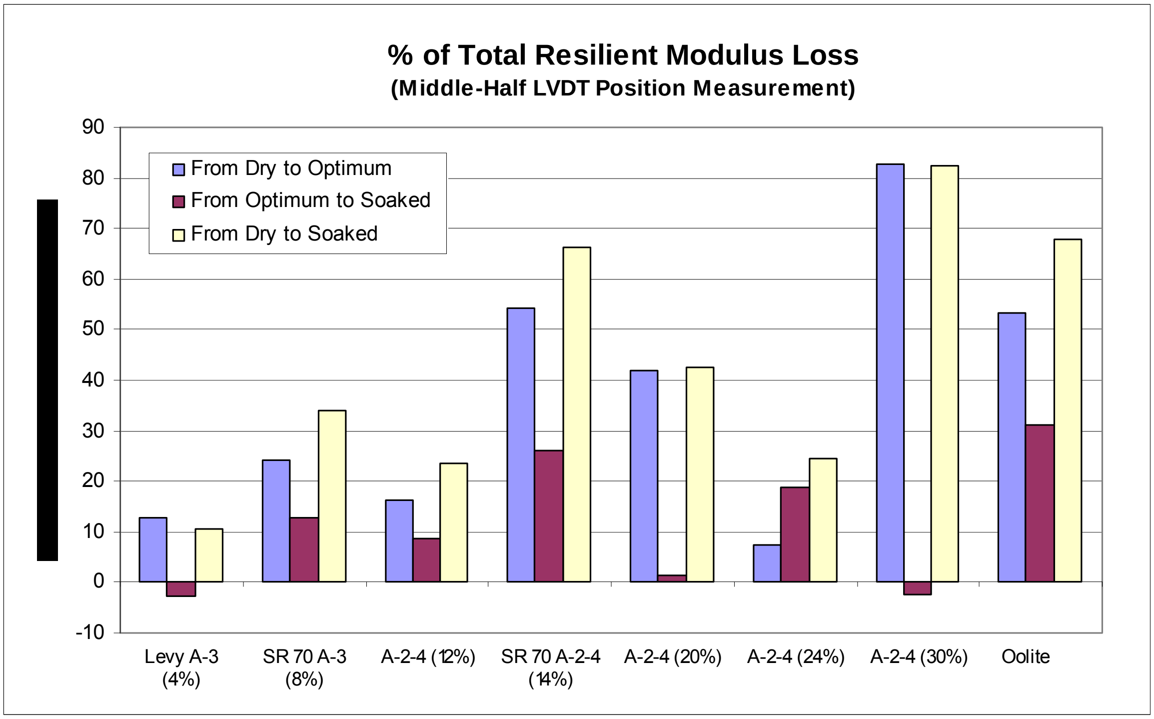


Figure 6.52 Percent of Resilient Modulus Loss (Middle-Half LVDT Position)

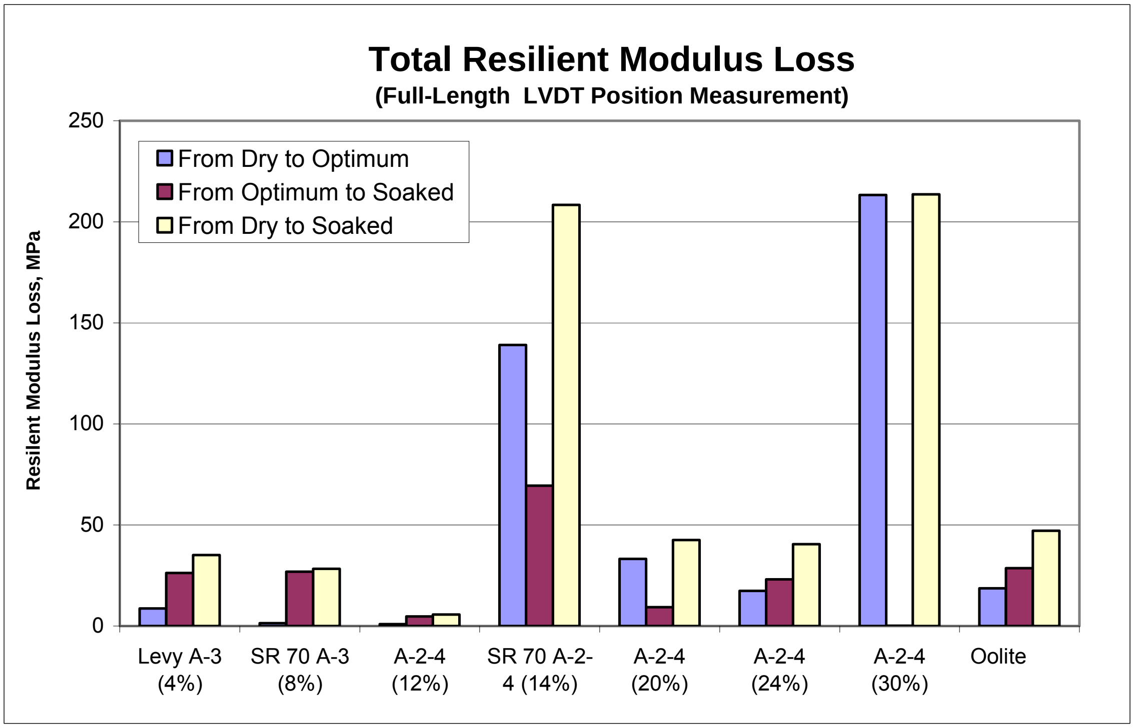


Figure 6.53 Total Resilient Modulus Loss (Full-Length LVDT Position)

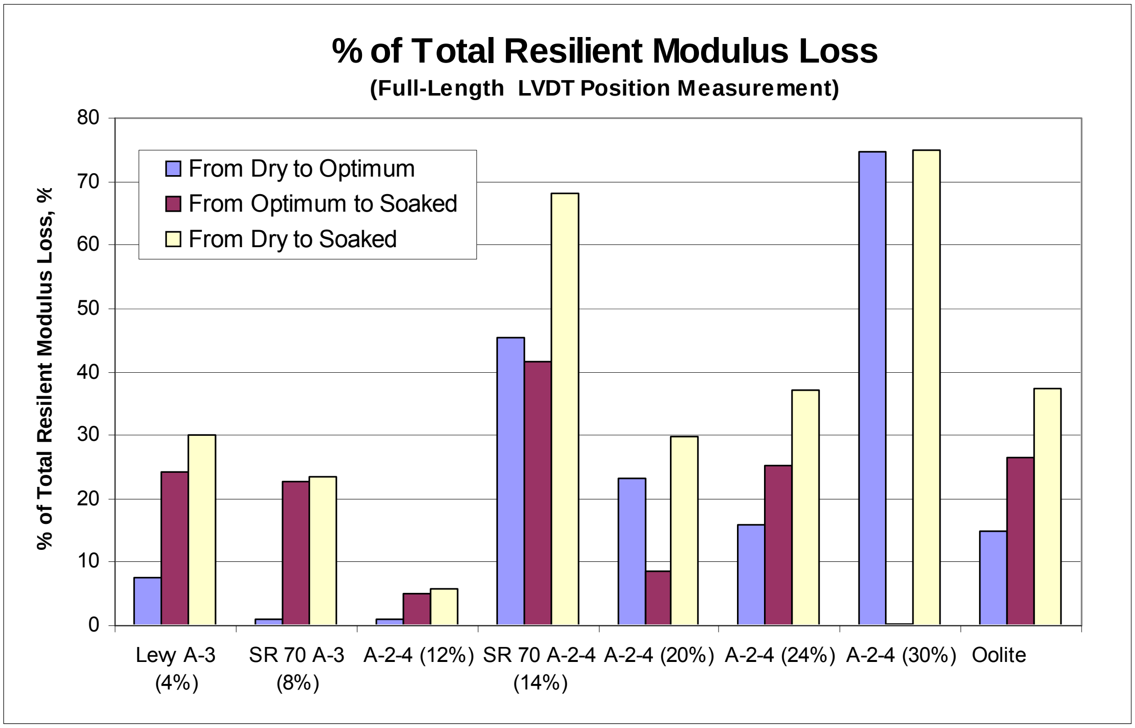


Figure 6.54 Percent Resilient Modulus Loss (Full-Length LVDT Position)

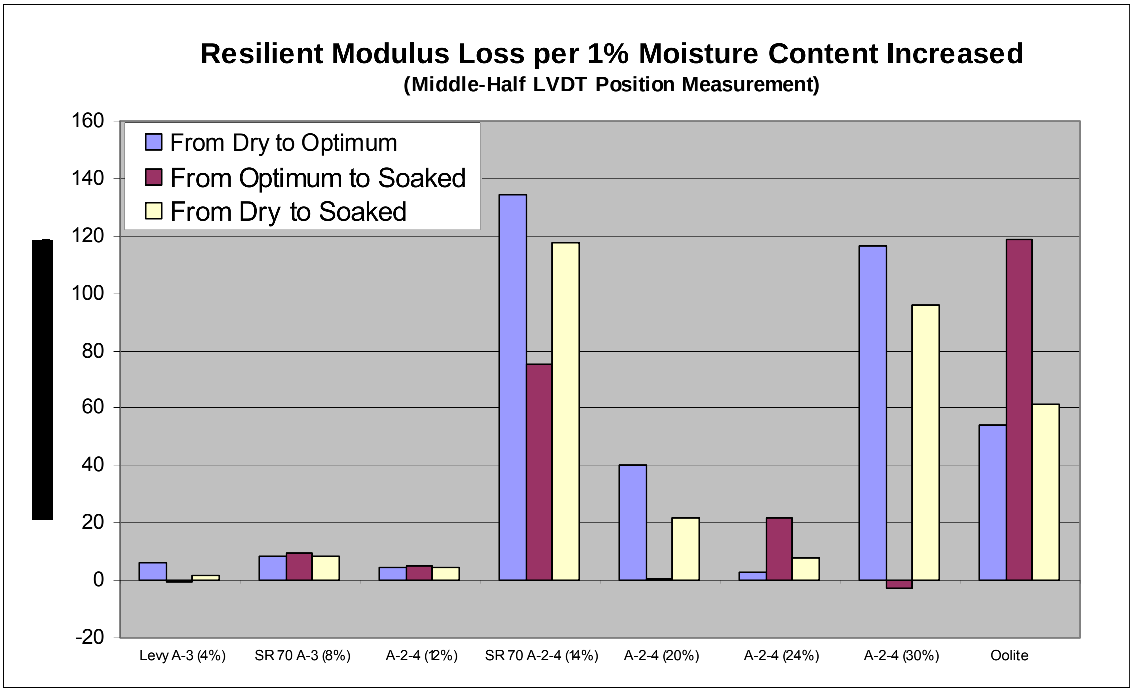


Figure 6.55 Resilient Modulus Loss Per 1% Increase in Moisture Content (Middle-Half LVDT Position)

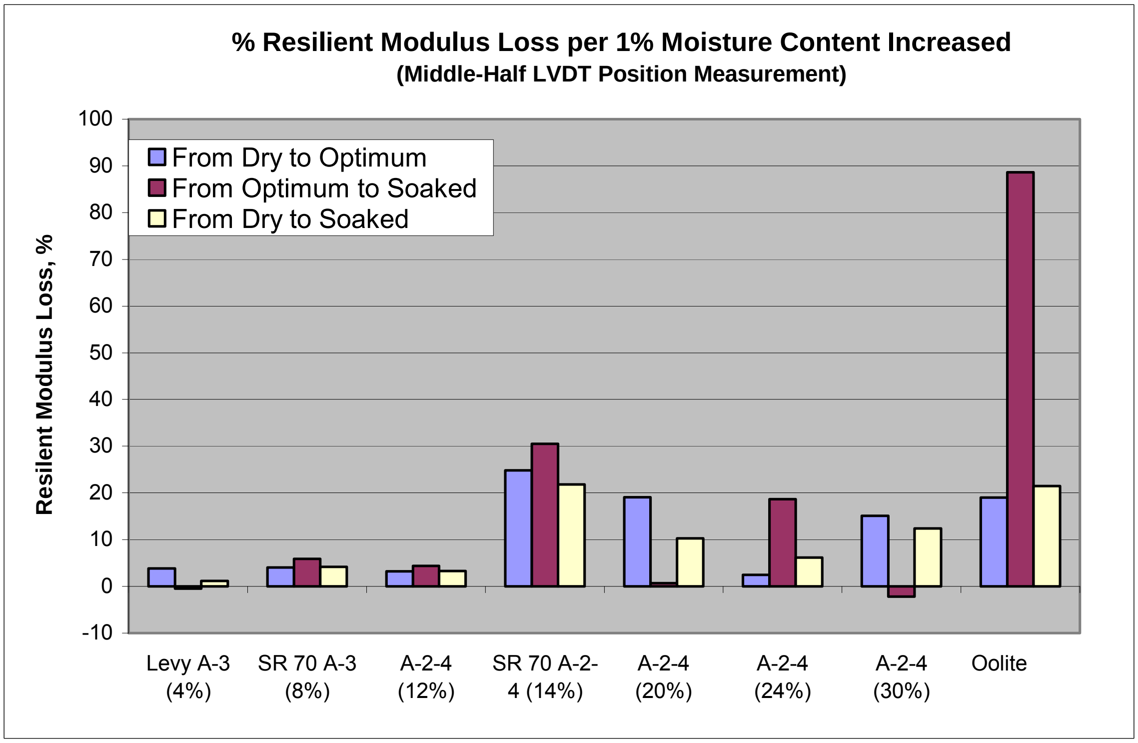


Figure 6.56 Percent Resilient Modulus Loss Per 1% Increase in Moisture Content (Middle-Half LVDT Position)

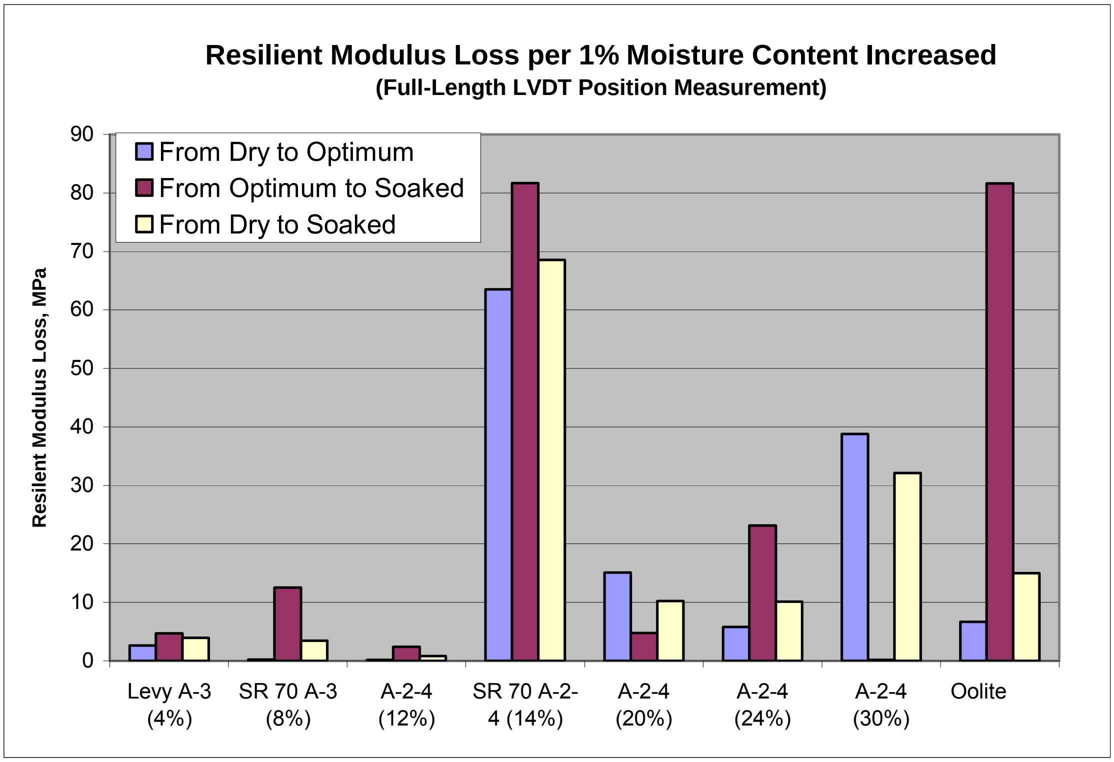


Figure 6.57 Resilient Modulus Loss Per 1% Increase in Moisture Content (Full-Length LVDT Position)

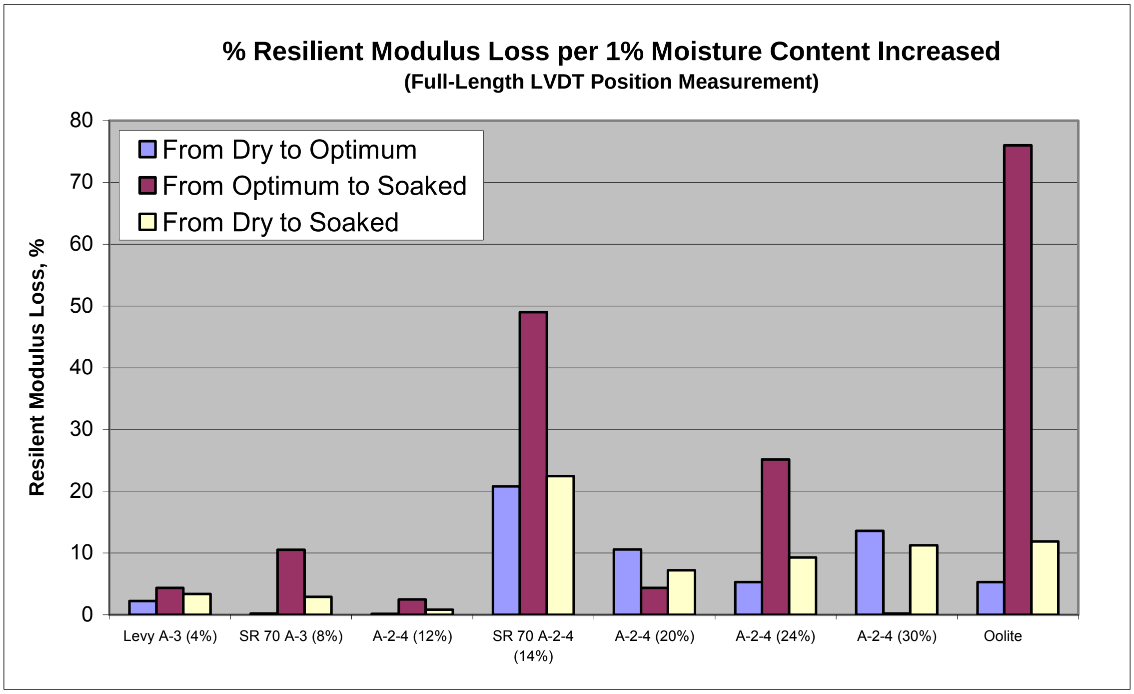


Figure 6.58 Percent Resilient Modulus Loss Per 1% Increase in Moisture Content (Full-Length LVDT Position)

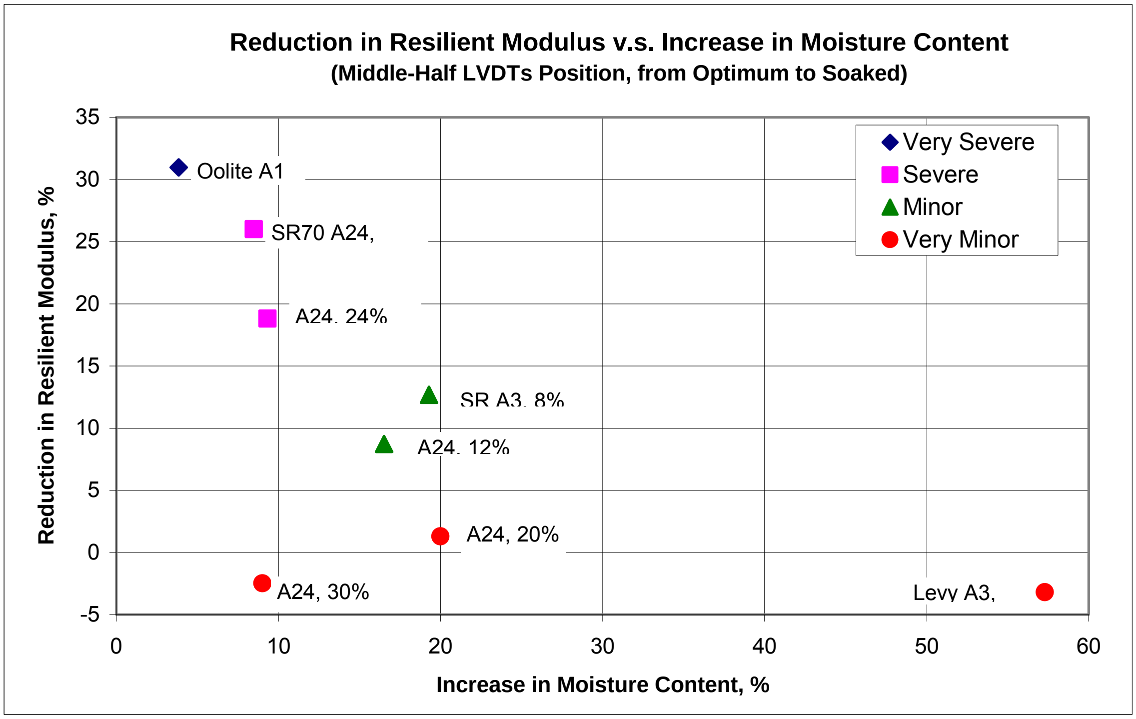


Figure 6.59 Reduction Rate of Resilient Modulus vs. Increase Rate of Moisture Content for Eight Soils (Middle-Half)

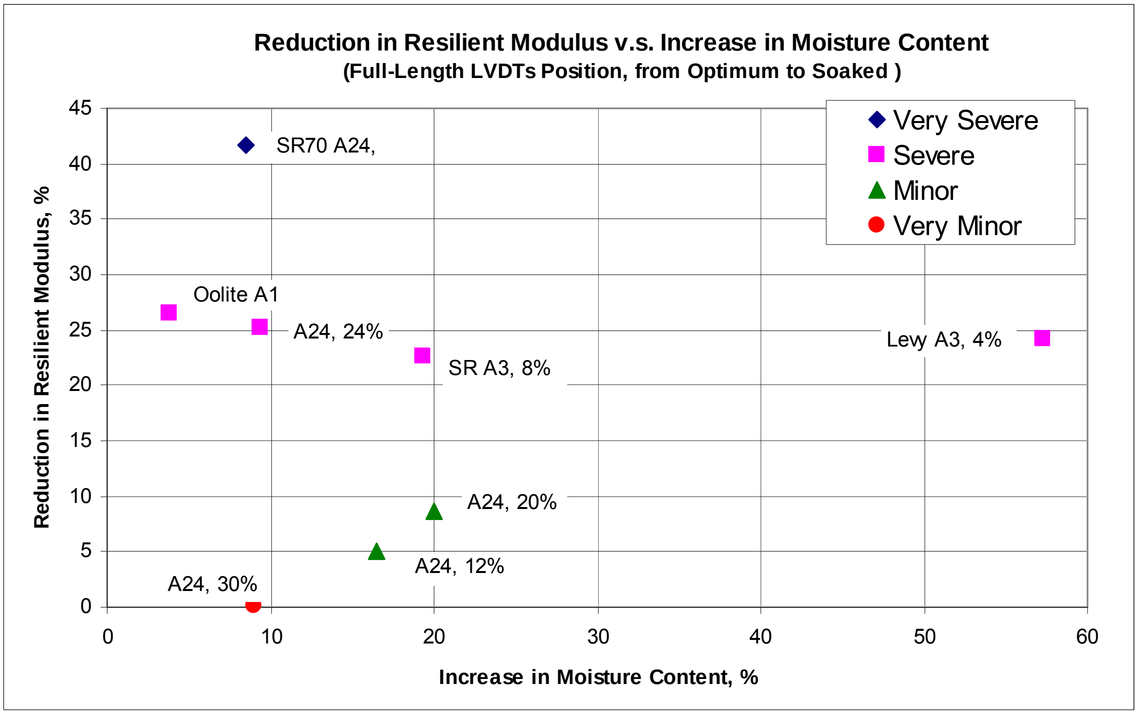


Figure 6.60 Reduction Rate of Resilient Modulus vs. Increase Rate of Moisture Content for Eight Soils (Full-Length)

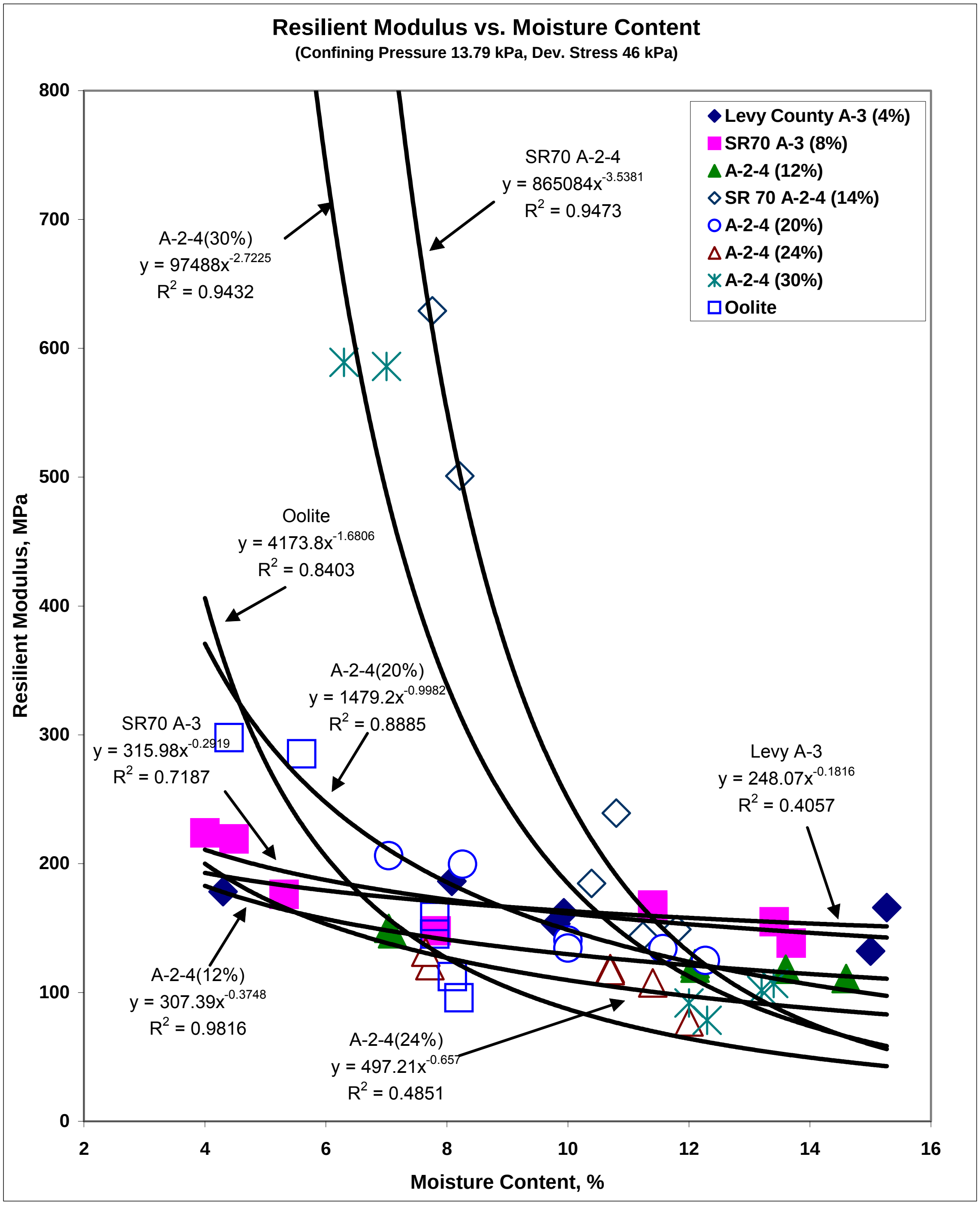


Figure 6.61 Mr vs. Moisture Content for Phase I and II Soils

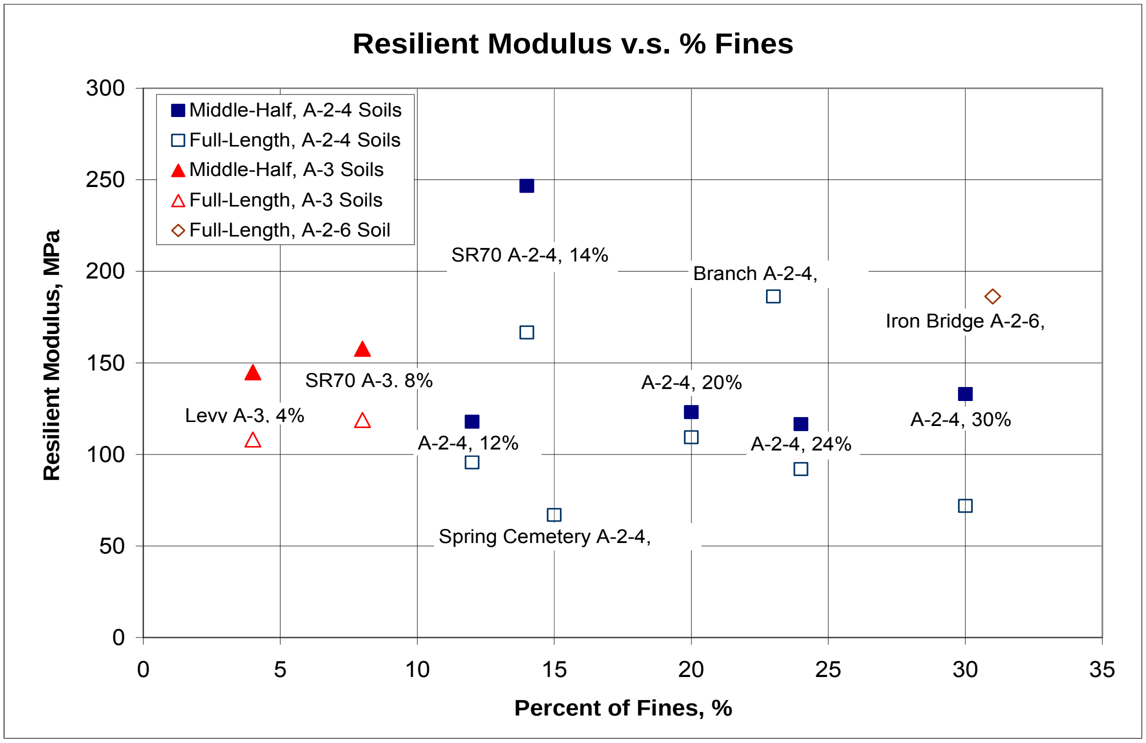


Figure 6.62 Resilient Modulus vs. Percent of Fines

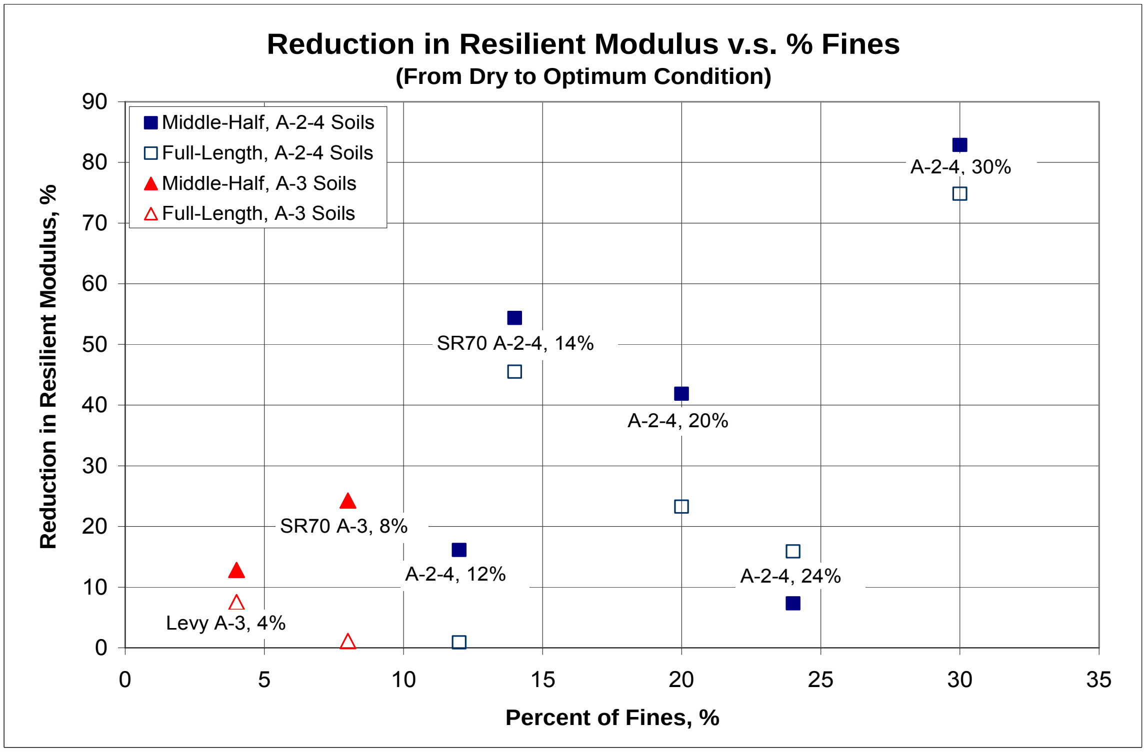


Figure 6.63 Reduction in Resilient Modulus vs. Percent of Fines (from Dry to Optimum Condition)

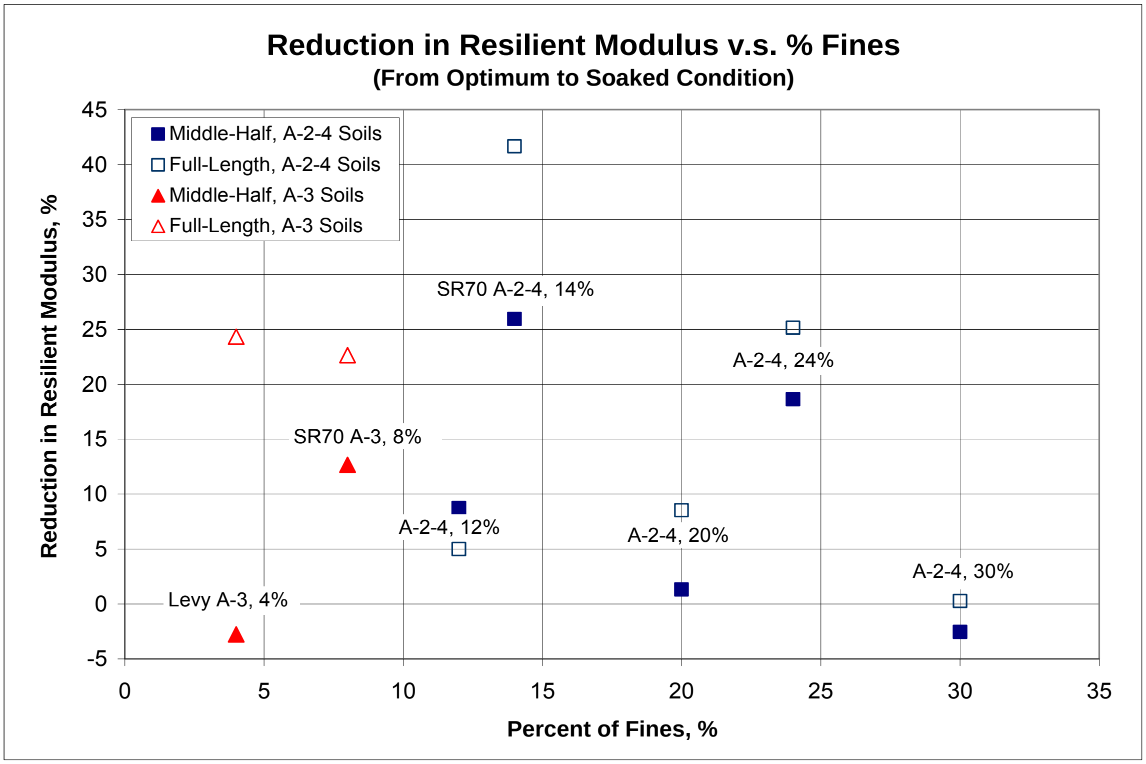


Figure 6.64 Reduction in Resilient Modulus vs. Percent of Fines (from Optimum to Soaked Condition)

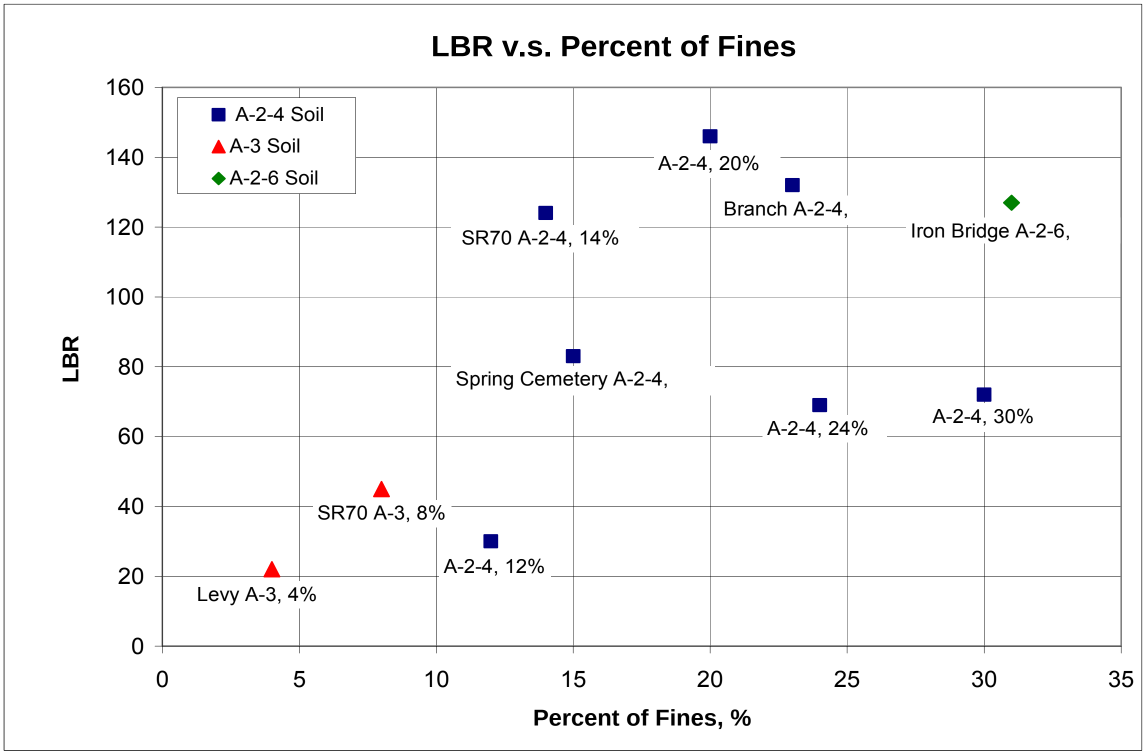


Figure 6.65 LBR vs. Percent of Fines

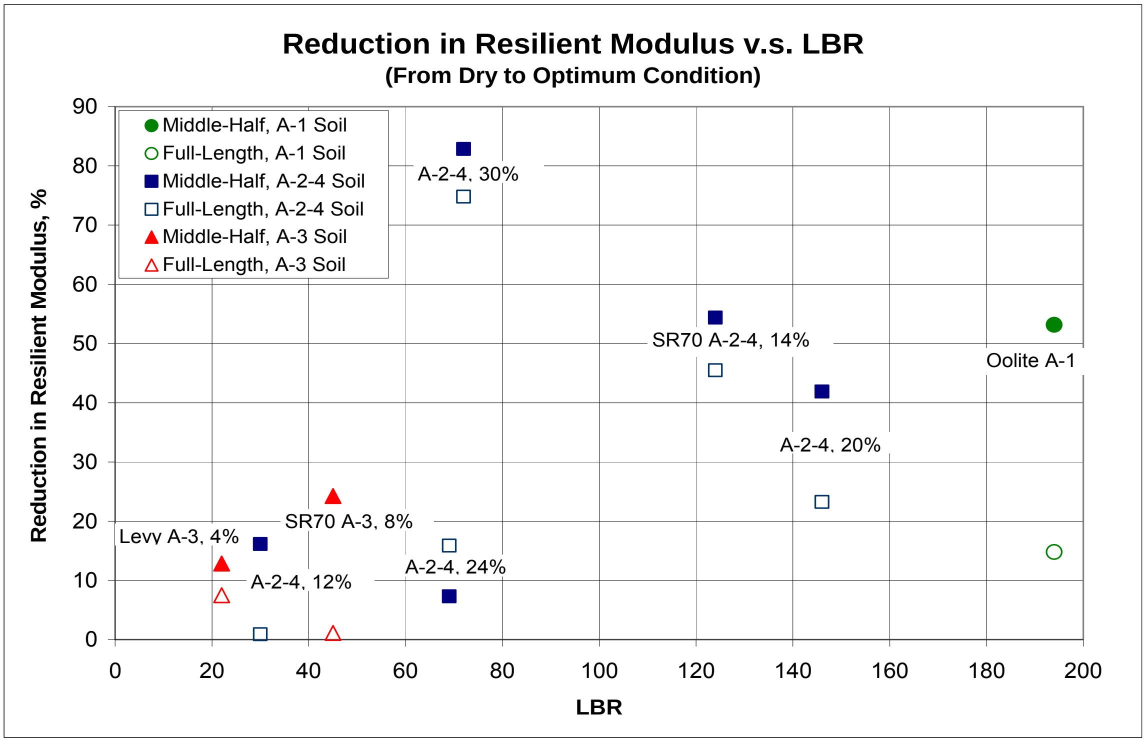


Figure 6.66 Reduction in Resilient Modulus vs. LBR (from Dry to Optimum Condition)

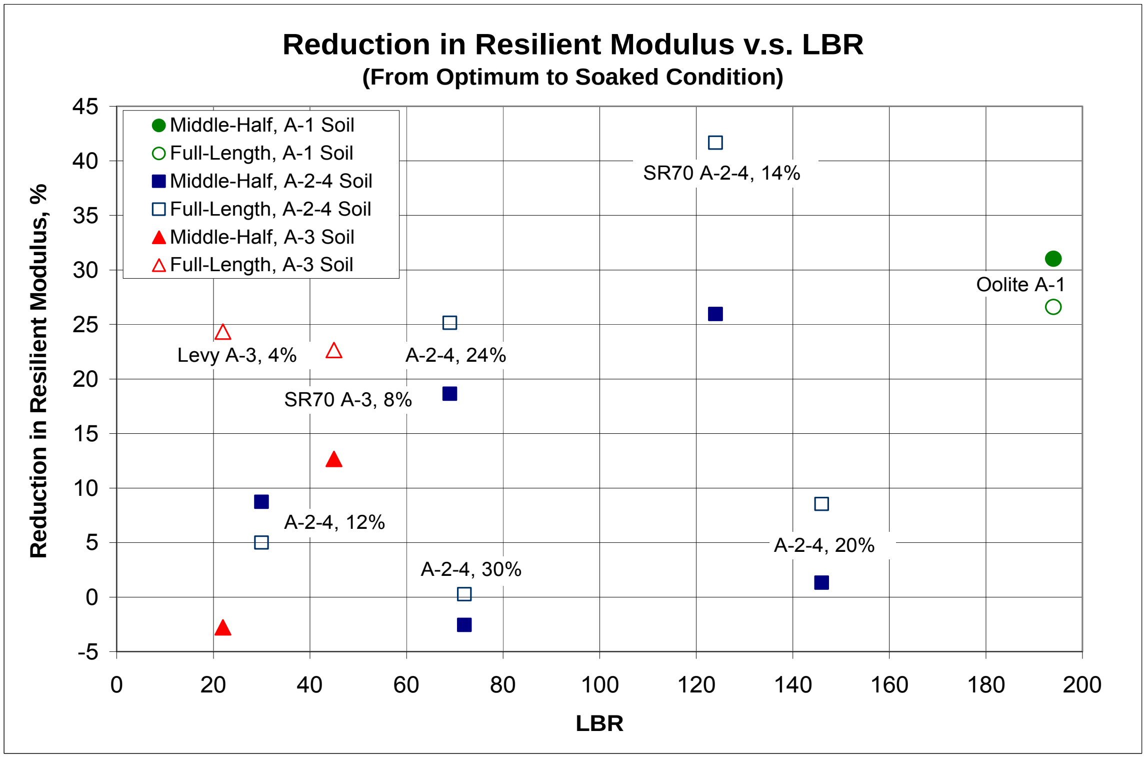


Figure 6.67 Reduction in Resilient Modulus vs. LBR (from Optimum to Soaked Condition)

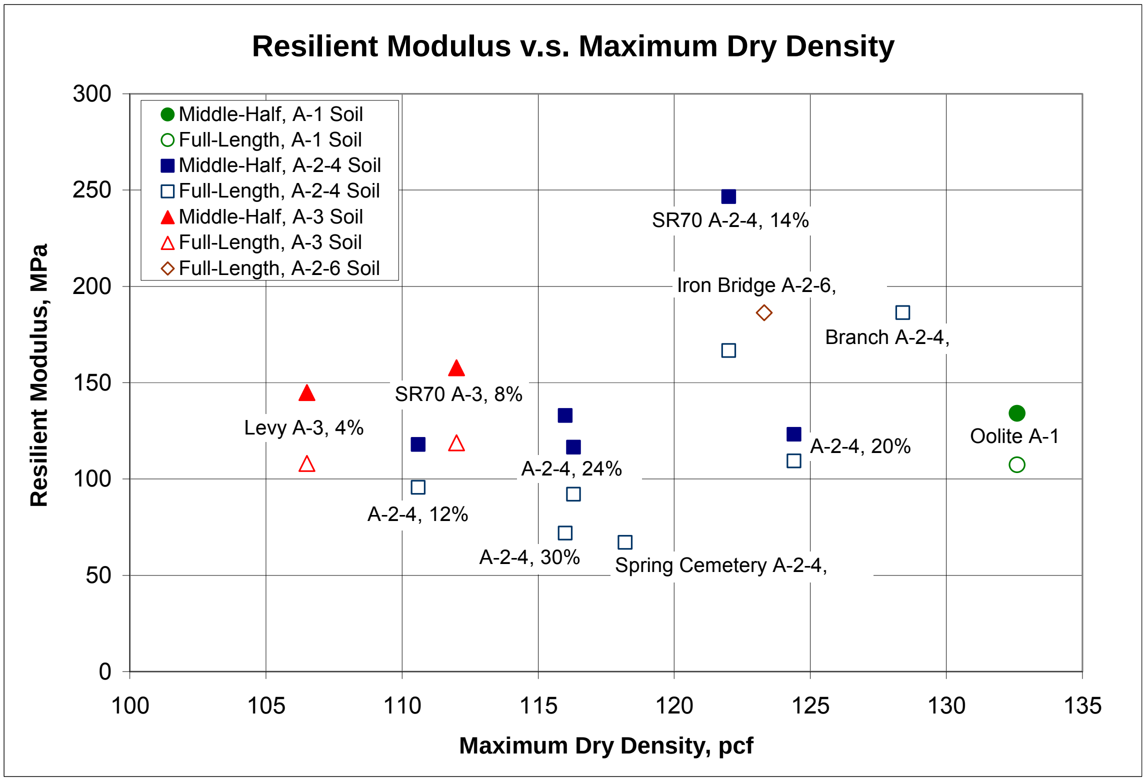


Figure 6.68 Resilient Modulus vs. Maximum Dry Unit Weight

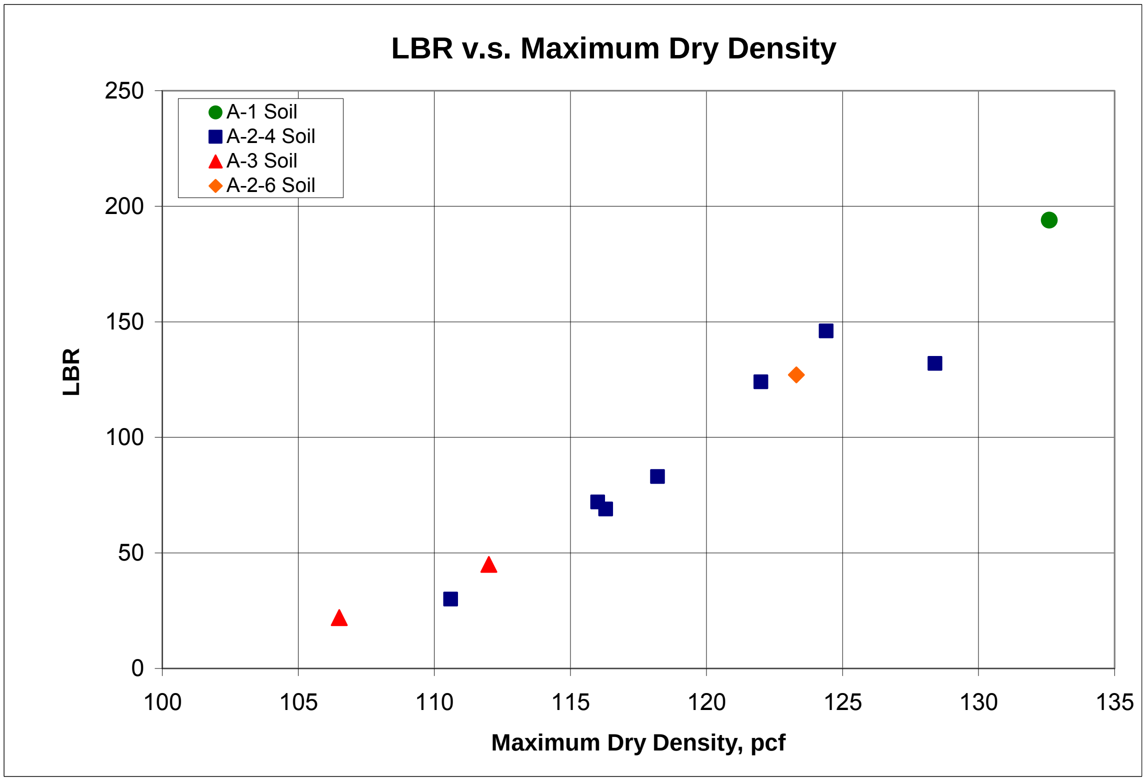


Figure 6.69 LBR vs. Maximum Dry Unit Weight

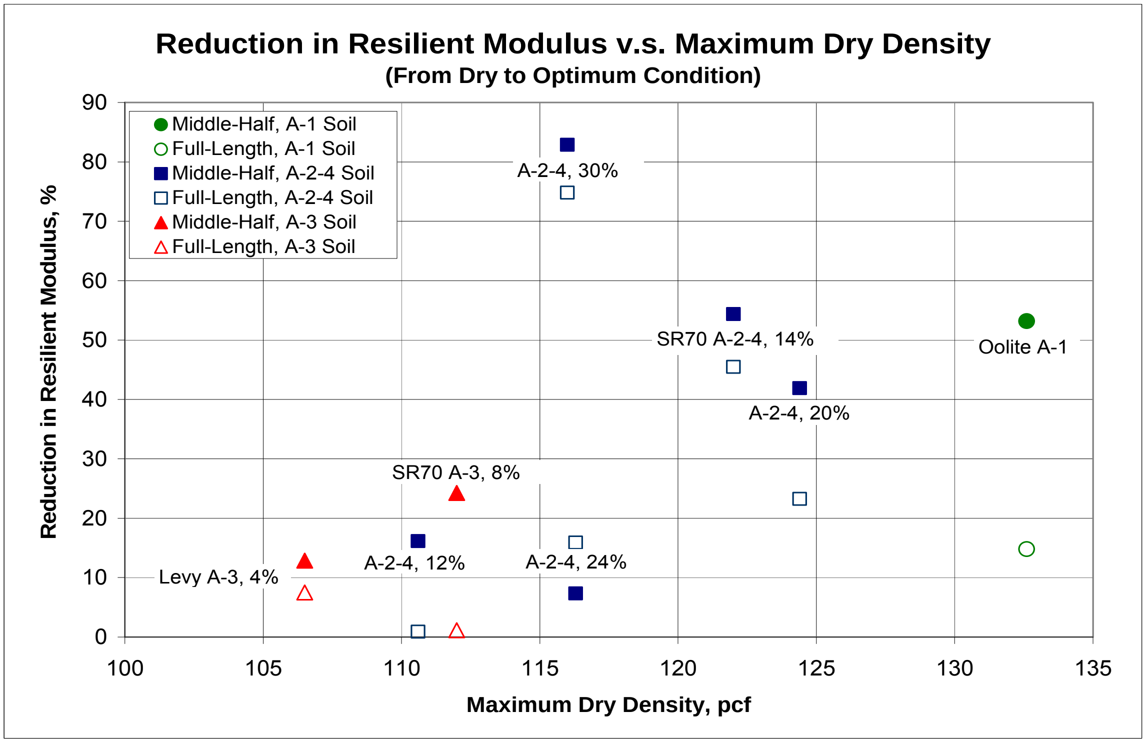


Figure 6.70 Reduction in Resilient Modulus vs. Maximum Dry Unit Weight (from Dry to Optimum Condition)

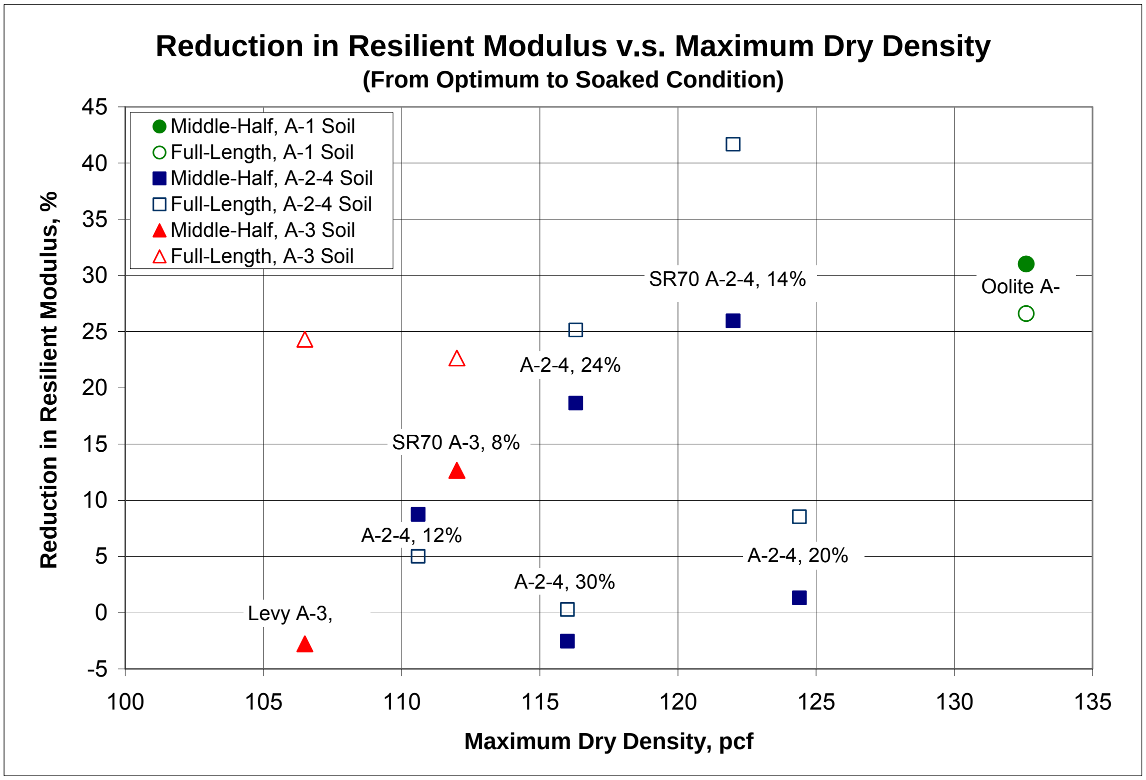


Figure 6.71 Reduction in Resilient Modulus vs. Maximum Dry Unit Weight (from Optimum to Soaked Condition)

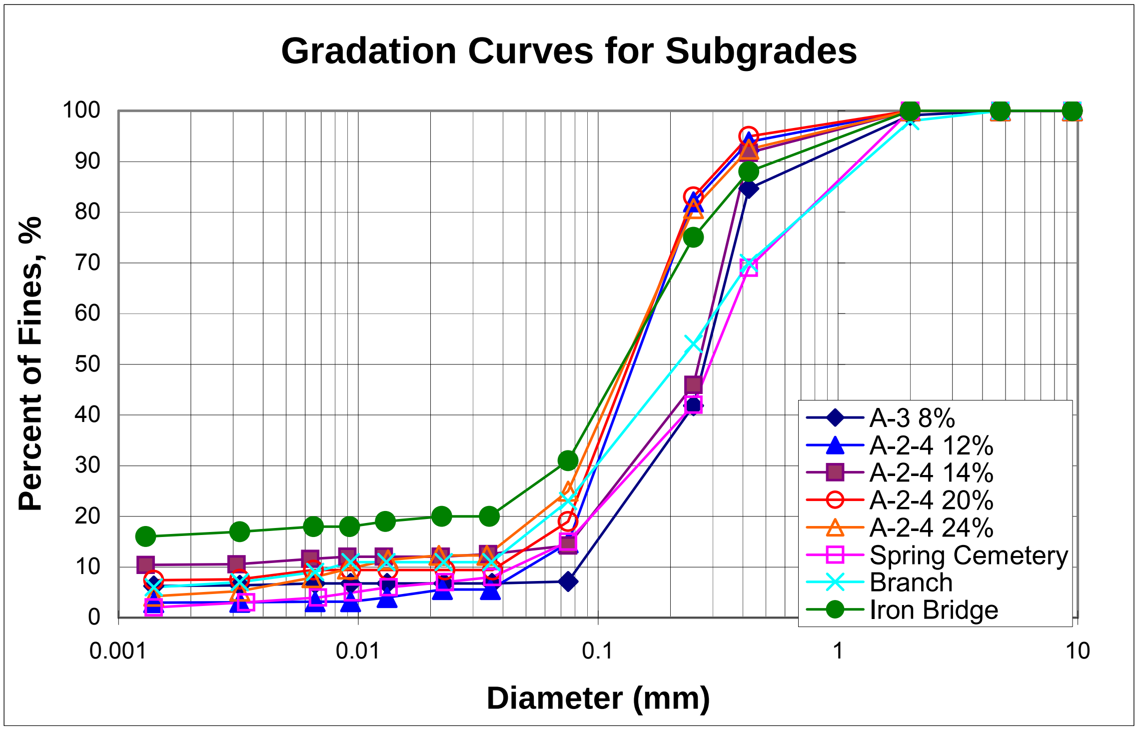


Figure 6.72 Gradation Curves for Eight Subgrade Soils

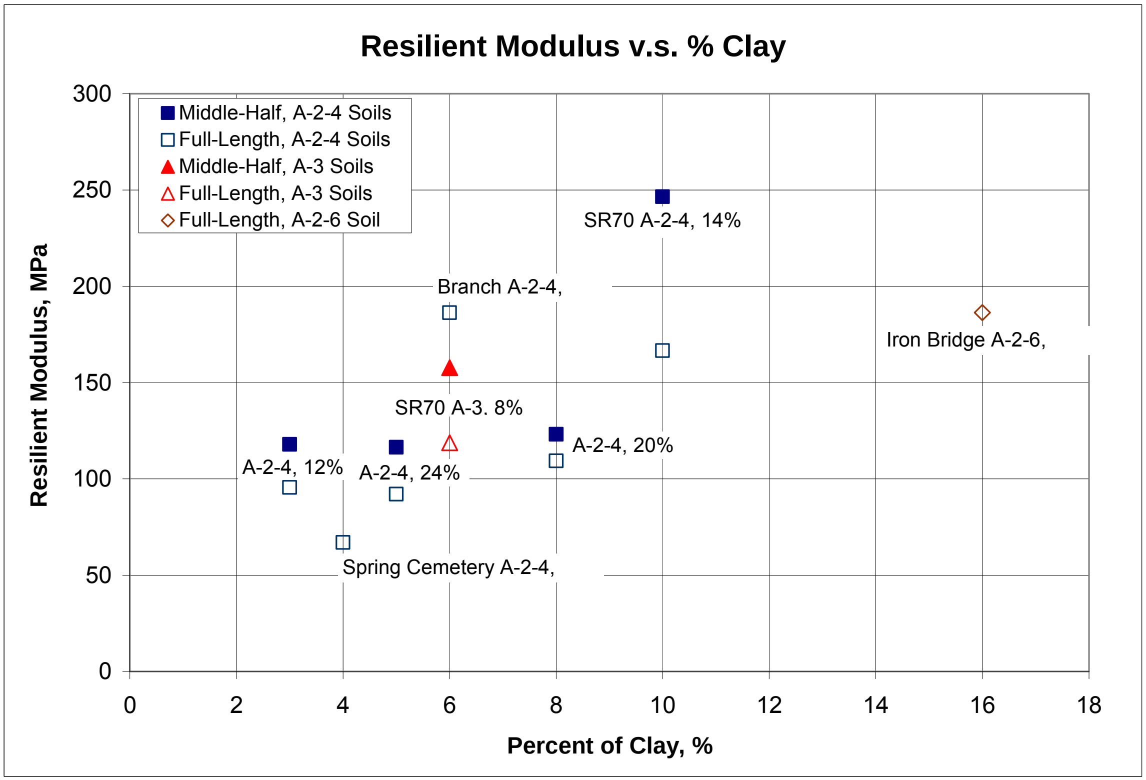


Figure 6.73 Resilient Modulus vs. Percent of Clay

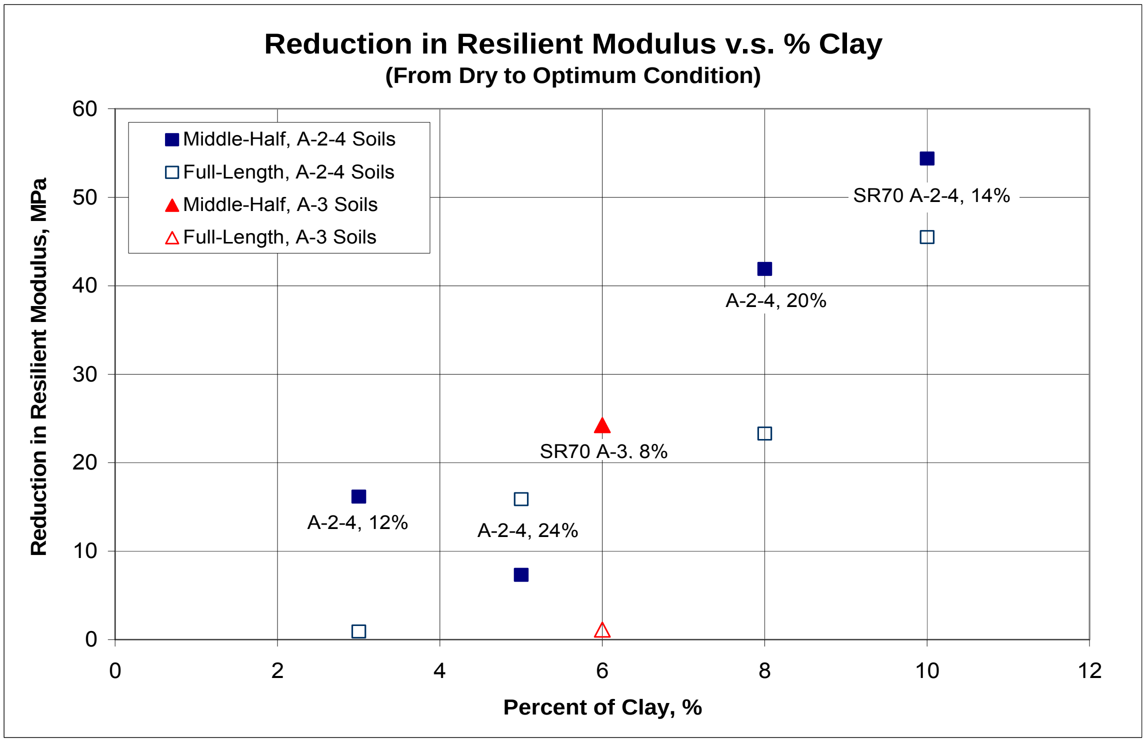


Figure 6.74 Reduction Rate in MR vs. Percent of Clay (from Dry to Optimum Condition)

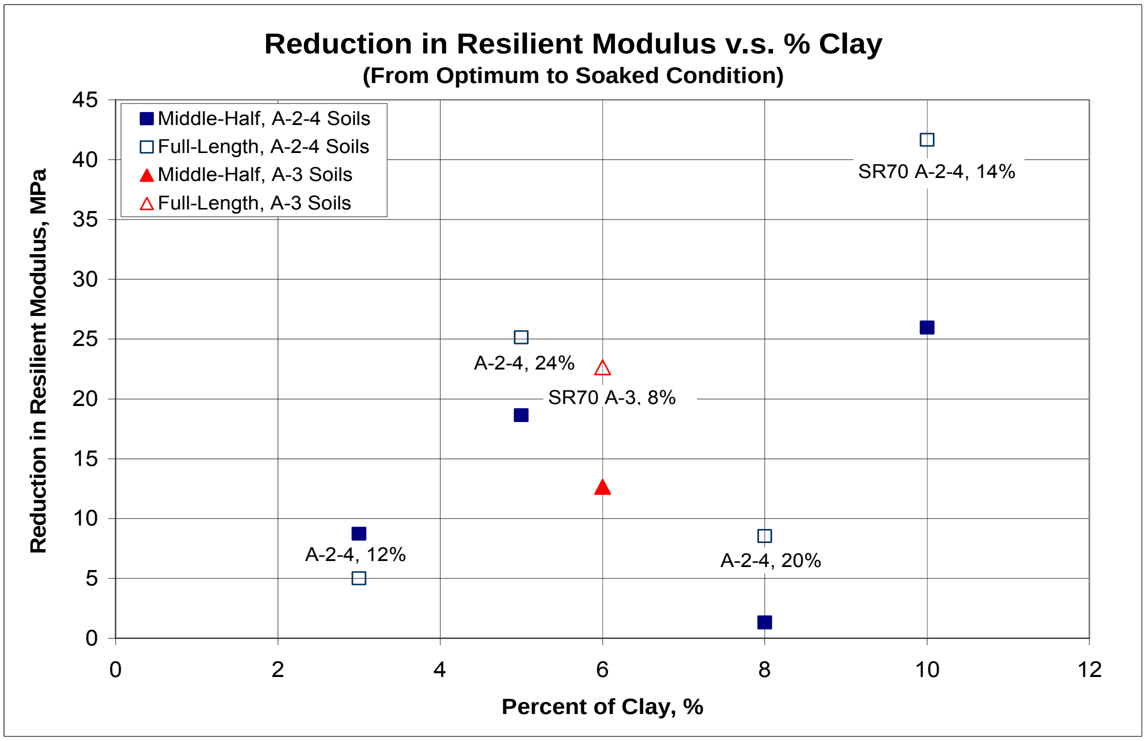


Figure 6.75 Reduction Rate in MR vs. Percent of Clay (from Optimum to Soaked Condition)

CHAPTER 7
ANALYSIS OF TEST-PIT TEST RESULTS

7.1 GENERAL

The test-pit test results are separated into two parts: a) the equivalent modulus of subgrade materials under designated plate loads as a result of water table adjustments and moisture changes; b) the moisture profile of subgrade materials as a result of groundwater table variation. The analysis of the moisture effect on the equivalent modulus resulting from the groundwater table changes for different subgrade materials is presented in this chapter. The analysis of moisture profiles includes the drainage effect and capillary rise study.

7.2 DRAINAGE ANALYSIS

Moisture could evaporate into the air from the top layer of the material in test pit, and could also drain to layers below; it depends on the ambient conditions and soil properties. However, the moisture content of each layer measured from TDR probes after the materials are placed should not vary too much at the beginning of the test. The data showed that the moisture content varied across the layers, and that there was a significant difference in moisture content between the layers for different test pits. The variation in moisture content due to the deviation of TDR probes will be discussed according to different test pits. Therefore, use of TDR data should be limited to comparison purposes only in this study.

In this section, short-term and long-term moisture variations after drainage were evaluated for subgrade materials bearing different permeability values. The moisture variations after drainage are summarized in Table 7.1 for Phase I and Table 7.2 for Phase II subgrade materials. The short-term rate of moisture dissipation is shown in Figure 7.1 for Levy County A-3 soil and in Figure 7.2 for SR70 A-3 Soil. The long-term rate of moisture dissipation for SR70 A-3, SR70 A-2-4, A-2-4(12%), A-2-4(20%), A-2-4(24%), A-2-4(30%) and Miami Oolite A-1 soils are shown in Figures 7.3, 7.4, 7.5, 7.6, 7.7, 7.8 and 7.9. Drainage analysis is not available for Phase III soils.

7.2.1 Observation of Drainage Data

7.2.1.1 Test Pit 1 – Levy A-3 Soil

Levy A-3 soil is the first test-pit test in this study. The degree of saturation was about 70% to 80% for the top layer to the bottom when the groundwater level is at the bottom of the Limerock. Most water was drained within one day. The moisture content dropped rapidly, from 15.1% to 8.7%, within two hours for the top layer. 8.8% of water was drained for the top layer and only 1% for the bottom layer. The test pit was evacuated after two days. There should be more water to be drained afterwards. The data should only be applied to short-term drainage.

7.2.1.2 Test Pit 2 – SR70 A-3 and SR70 A-2-4 Soils

Both short-term and long-term drainage analyses are available for SR70 A-3 soil. The four middle layers were fully saturated and the moisture content of the bottom layer for the SR70 A-3 soil was found to be much lower than the moisture content of the layers above when the water table was at the bottom of the limerock. 9.2% of water for the top layer to 6.5% of water for the bottom layer was drained within two weeks. The water was reduced about 11% to 8.2%, within 86 days, for the layers from top to bottom.

For SR70 A-2-4 soil, the water content of the top layer shown in Figure 7.4 is obviously wrong. It is extremely high (over 30%) for the top layer and too low (11%) for the layer of 12-18 in. above the embankment. The error should be most likely attributed to damage to the TDR probes in the test pit.

Unlike for the SR70 A-3 soil, the water did not drain much as time elapsed for the layers below the layer of 18-24 in. above the embankment. The material held more water when the water table went down to 24 in. below the top of the embankment. The water in the top layer did not start to drain until two weeks later. The water was reduced about 16.6%, 5.8%, and 4.8% for the top three layers within 86 days.

7.2.1.3 Test Pit 3 – A-2-4 12%, 20%, 24% Soils

Unlike in Test Pit 2, the water only drained to 12 in. above the embankment in Test Pit 3. The effect should be weighted when compared to the results from other test pits. From Figure 7.5, the water content of the 24-30 in. layer for the A-2-4 12% soil was extremely low (only 7.5%) compared to other layers. Again, the water content of the bottom layer was lower than normal, while the layer of 12-18 in. had the highest water content. The measuring from the TDR probe should be the key factor to the errors. There was almost no water content change for the bottom five layers except for the top layer, which had its water content reduced about 3.5% within 59 days. This rate is much lower compared to other soils. The effect is mostly attributed to the high suction value (440 kPa) of the A-2-4 12% soil.

The moisture profile for the A-2-4 20% soil was quite different from the one for A-2-4 12% soil. The layer of 18-24 in. had the highest moisture content when the water table was at bottom of the limerock. The water in the top layer dropped immediately as the water table went down to 12 in. above the embankment and the layer of 24-30 in. did not change until 24 days later. There was no water loss for the rest of layers. The total water loss was about 2.6% for the top layer and 2.9% for the layer of 24-30 in. above the embankment within 59 days. The suction value was 373 kPa for the A-2-4 20% soil.

There was no significant water change for the A-2-4 24% soil as the groundwater table went down to 12 in. above the embankment, except for the top layer, which had a quick drop after 28 days. The layer of 24-30 in. above the embankment had a relatively low moisture content compared to the layers next to it.

7.2.1.4 Test Pit 4 – A-2-4 30% and Miami Oolite A-1 Soils

A-2-4 30% soil had a normal moisture profile except at the bottom layer. This might be due to the material change at the interface between the bottom layer and the existing A-2-4 embankment underneath. No water was shown to have drained after the water table went down to 12 in. above the embankment. Water was retained in between the soil particles due to the higher percentage of fines and suction value.

The water content of the Miami Oolite A-1 soil was low when the water table was at the bottom of limerock. The lab optimum water content is about 7.6%, but the water content in the test pit was about 6.6% to 3.1%. The water content did not change as the water table changed except for the top layer, which had a slight reduction of about 0.7% within 59 days.

7.2.2 Discussion of Drainage Behavior

The variation in the absolute rate of drainage for the subgrade materials was attributed to the difference in coefficient of permeability and suction value, which were related to their void ratio (gradation and grain size). With the percent of fines passing No. 200 sieve increasing from 4% for Levy County A-3 soil to 30% for A-2-4 (30%) soil, the coefficient of permeability decreased from the order of magnitude 10^{-3} to 10^{-5} cm/sec (refer to Table 4.14 and Figure 4.14) and the suction value increased (refer to Figure 4.13). The result was a more time-consuming moisture dissipation process before the final equilibrium was established.

The rate of drainage was directly related to the permeability of the soil. In a saturated (or nearly saturated) state, the permeability for a specific soil was a function of the void ratio. As the draining process continued, the soil became partially saturated. In this case, the permeability was significantly affected by the combined change in void ratio and degree of saturation. Since water flowed through the pore space also occupied by water, the percentage of the voids that were filled with water was an important factor. After the water dissipated first from the large pores from flooded condition, air took its place. Water had to flow through smaller pores filled with water, which provided a more narrow passage for downward seepage. On the other hand, with the increase of soil suction (because of a decrease in moisture) as drainage continued, the air-soil interface (capillary meniscus) was drawn closer to the soil particles, which led to a further decrease in the volume of void filled with water. As a result, the permeability of soil (or the rate of drainage) rapidly decreased after a short-term drainage.

Generally, the closer to the top of subgrade, the more moisture reduction occurred due to drainage. The drainage rate decreased with an increase in percent of fines. For A-2-4(30%) and Miami Oolite A-1 soils, there was no significant moisture change for all the sensors.

7.3 CAPILLARY RISE ANALYSIS

In the capillary rise study, the height of the capillary rise was the vertical distance between the water table and the highest elevation where the increase in moisture existed. When the water table changed from a level below 0 in. (the interface between subgrade and embankment) to 0 in., the moisture profile and the time were recorded. For Levy A-3 soil, the water table was raised from −20 in. to 0 in. above the embankment. For SR70 A-3 and

SR70 A-2-4 soils, the groundwater level was raised from -24 in. to -12 in. above the embankment, and then from –12 in. to 0 in. above the embankment. For the Phase II and III soil types (A-2-4 12%, A-2-4 20%, A-2-4 24%, A-2-4 30%, Miami Oolite A-1, Spring Cemetery A-2-4, Branch A-2-4, and Iron Bridge A-2-6), the groundwater level was raised from –24 in. to 0 in. above the embankment. The moisture data with the groundwater level raised from 0.0 in. to +12 in. above the embankment was also utilized for capillary rise analysis.

The moisture data after the adjustment of the groundwater level from drained conditions to 0.0 in. above the embankment and from 0.0 in. to +12.in. above the embankment, which are useful for the capillary rise study, are summarized in Table 7.3 for Phase I soils, Table 7.4 for Phase II soils, and Table 7.5 for Phase III soils.

The moisture profile at each elevation for the adjustment of the groundwater level from drained conditions to 0.0 in. above the embankment and from 0.0 in. to +12.0 in. above the embankment can be found in Figures 7.10(A) to 7.33(A), while Figures 7.10(B) to 7.33(B) illustrate the moisture content versus time lapse for each layer. The water content of the top layer (30 in. to 36 in. above the embankment) was not available due to the limitation of the TDR probes for the Phase III soils until the water table was raised up to groundwater level C (+24.0 in. above the embankment). The capillary rise could have been higher than what was observed if the water content of the top layer had been recorded.

7.3.1 Observation of Capillary Moisture Data

7.3.1.1 Levy County A-3 Soil

Levy A-3 soil had a wide range of water content from 5% to 12.5% after reaching equilibrium at the initial placement. The capillary rise effect in Levy A-3 soil with the groundwater level from 20 in. to 0 in. below the embankment is illustrated in Figure 7.10(A) and 7.10(B). The water content increased about 2.5% for the bottom layer and 4.7% for the layer above within 28 days. Figure 7.10(B) showed no moisture increase for the top two layers caused by capillary rise, and the capillary rise was about 24 in., as seen in Figure 7.10(A). Beyond that point, the behavior was more than offset by the evaporation rate (high drying rate for A-3 soil). The water content change due to capillary rise was not significant for the Levy A-3 soil when compared to other soils.

As shown in Figure 7.11(A) and Table 7.11(B), the capillary rise effect in the Levy A-3 soil with the groundwater level from 0 in. to +12 in. was more than 24 in. The water content kept increasing after a 47-day equilibrium. The degree of saturation was about 78% for the bottom layer on day 47.

7.3.1.2 SR70 A-3 Soil

The capillary rise of the SR70 A-3 soil with the groundwater level from –24 in. to -12 in. is illustrated in Figure 7.12(A) and 7.12(B). The bottom layer had a low water content compared to other layers. The total capillary rise was about 6 in. in addition to the 12 in. of that passing through the standard A-3 sand within embankment, which was 18 in. high. The change in water content was less than 1%. The capillary rise was insignificant in this condition.

Figure 7.13(A) and 7.13(B) show the capillary rise effect when the water table was raised from -12 in. to 0 in. Although there was a significant moisture increase of 7.2% close to the embankment within a seven-day period, the capillary water could only ascend to a height of 18 in. Beyond that point, the moisture increase caused by the capillary rise was more than offset by the evaporation rate (high drying rate for A-3 soil). The degree of saturation for the bottom layer was about 88% and the equilibrium was reached within 18 days.

The capillary rise effect is illustrated in Figure 7.14(A) and 7.14(b) when the water table was raised from 0 in. to +12 in. above the embankment. The water content increased rapidly and had a degree of saturation of 100% for the layer of 6-12 in., while there was no increase in the water content for the bottom layer within one day. The capillary rise was about 18 in. with an increase in water content of about 7% to 1.3% for the top five layers. The capillary rise was insignificant for the top two layers.

7.3.1.3 SR70 A-2-4 Soil

For the SR70 A-2-4 soil shown in Figures 7.15(A) and 7.15(B), the water content increased at a slow rate and the total amount of increase in moisture was limited due to the capillary rise for the groundwater level raised from -24 in. to -12 in.

After the water table was raised from -12 in. to 0 in., the short-term increase of moisture content which resulted from the capillary rise was not obvious, as shown in Figures 7.16(A) and 7.16(B). Even with a long-term high groundwater standing duration of 7 to 42 days, there was only a 2% to 0.7% increase in moisture content from the bottom layers up within 42 days. The capillary rise was about 12 in. The degree of saturation was about 80% for the bottom layer.

Similar to the SR70 A-3 soil, Figures 7.17(A) and 7.17(B) showed that there was an instant increase in moisture in the layer of 6-12 in. above the embankment within four days, while there was no change in the water content for the bottom layer when the groundwater level was raised to +12 in. above the embankment. The capillary rise was about 12 in. The degree of saturation was about 80% for the bottom layer and 100% for the layer of 6-12 in. above the embankment.

7.3.1.4 A-2-4, 12% Soil

The capillary rise of A-2-4 12% soil with a groundwater level change from –24 in. to 0 in. above the embankment is illustrated in Figures 7.18(A) and 7.18(B). After the water content was stabilized, the layer of 12-18 in. above the embankment had a greater increase in water than the bottom layer. The capillary rise was about 36 in. and it took 33 days to reach equilibrium. The soil had a low degree of saturation from 20% for the layer of 24-30 in. above the embankment to 60% for the layer of 6-18 in. above the embankment. The water content increased about 7% for the layer of 12-18 in. above the embankment. Figures 7.19(A) and 7.19(B) illustrate the capillary rise effect for the A-2-4 12% soil when the water table was raised from 0 in. to +12 in. Unlike the previous soils, the A-2-4 12% soil did not have an immediate response to the water level change until the second day. The water content kept increasing at a slow speed and still increased after 86 days. The same phenomenon applied to the A-2-4 (12%) soil; the bottom layer had the lower water content relative to the layer above when the groundwater level was up to +12 in. above the embankment. The capillary rise was more than 24 in. after 86 days. The layer of 12-18 in. had the highest water content, and the degree of saturation was about 69%. Compared to other soils, the degree of saturation was low. The delay of response to the water change was noticeable for the A-2-4 (12%) soil.

7.3.1.5 A-2-4, 20% Soil

The capillary rise effect was insignificant when the groundwater level was raised from -24 in. to 0 in. above the embankment. No increase in moisture content was shown for any of the six layers above the embankment. The capillary rise can be considered to be 0 in. for a 53-day period, as shown in Figures 7.20(A) and 7.20(B).

A similar phenomenon applied when the water table was raised from 0 in. to +12 in. above the embankment. The water content did not change for the bottom layer and only slightly increased for the layers above when the groundwater level was at 12 in. above the embankment. This is abnormal. Even the water content did not change much; the capillary rise was more than 24 in. at a slow increase rate after 86 days.

7.3.1.6 A-2-4, 24% Soil

Figures 7.22(A) and 7.22(B) illustrate the capillary rise effect when the groundwater level was raised from -24 in. to 0 in. above the embankment. It was abnormal that the water content of the layer of 0-6 in. did not change much, while the layers above had more water content change than the bottom layer. The capillary rise can be more than 36 in. The water content increased about 3.5% for the layer of 12-18 in with a stabilized degree of saturation of 61%, while the bottom layer had the smallest increase, about 0.2%, with a 43% degree of saturation.

The capillary rise effect for the water table raised from 0in. to +12 in. is shown in Figures 7.23(A) and 7.23(B). The bottom two layers did not experience any increase in water content. The layer of 18-24 in. had the highest increase in water content, 3.7%, with a stabilized degree of saturation of 73%. The capillary rise was more than 24 in. after an 86-day period. It was noted that the A-2-4 24% soil had the same one-day delay of water increase behavior as found also in the A-2-4 12% soil.

7.3.1.7 A-2-4, 30% Soil

The A-2-4 30% soil had a quick increase in moisture of about 4% after 4 days for the bottom layer when the water table was raised up to 0 in. above the embankment, as shown in Figures 7.24(A) and 7.24(B). After that, the increase rate became slow, decreasing even to no increase at all for the bottom layer. The capillary rise was about 36 in. The degree of saturation was 95% for the bottom layer.

Figures 7.25(A) and 7.25(B) show that the water increased rapidly for the layer of 6-18 in. above the embankment after the groundwater level was raised to +12 in. above the embankment. There was no increase in moisture for the bottom layer. The layer of 6-18 in. above embankment was saturated. The capillary rise was more than 24 in.

7.3.1.8 Miami Oolite A-1 Soil

Miami Oolite A-1 soil had a very low water content, for an average of about 3%, after it reached equilibrium when initially placed, except for the layer of 18-24 in. above the embankment. There was not much increase in moisture after the groundwater level was raised to 0 in. above the embankment. The capillary rise was only about 18 in. with a 0.6% increase in moisture for the layer of 6-12 in. above the embankment. The water content decreased after four days for the top three layers. This can be mainly attributed to the large void ratio and the fact that the water can be easily drained downwards. The degree of saturation was low for all six layers. Figures 7.26(A) and 7.26(B) illustrate the phenomenon discussed above.

When the groundwater level was raised to +12 in. above the embankment, the water did not go up until two days later. The water content even decreased after 23 days. The capillary rise was about 12 in. with a slight increase of 1.2% in water content for the layer of 12-18 in. above the embankment. Just as when the groundwater level was at the top of the embankment, the layer of 18-24 in. above the embankment had a relatively high water content compared to the other layers.

7.3.1.9 Spring Cemetery A-2-4 Soil

The Spring A-2-4 soil had a rapid increase in water content from 5.5% to 12.2% for the bottom layer and from 5.8% to 9.2% for the layer of 6-12 in. above the embankment within the first day when the groundwater level was raised from 24 in. below the embankment to the top of the embankment. The water content of the top three layers even decreased. After that, the water content had a slow increase in moisture, with an increase of 0.8% for the bottom layer and 1.5% for the layer of 12-18 in. above the embankment. The water content reached equilibrium after 17 days. The capillary rise was about 18 in., as shown in Figures 7.28(A) and 7.28(B).

After raising water table up to +12 in. above embankment, the capillary rise effect reached to the top layer with a 0.7% increase in water content for the top layer and about 2.5% for the two layers underneath within 56 days, as shown in Figures 7.29(A) and 7.29(B). The capillary rise was about 18 in.

7.3.1.10 Branch A-2-4 Soil

As shown in Figures 7.30(A) and 7.30(B), the water content did not change much for the first day after the groundwater level was raised to the top of the embankment. The two layers above the bottom layer had the greatest increase in moisture, with an increase of about 1.8%, which is low compared to the other soils. The capillary rise was about 18 in. and the degree of saturation was about 92%.

Just as when the groundwater level was raised to +12 in. above the embankment, the capillary rise reached to the top layer. The increase in the water content was significant (5%) for the top layer. The bottom layer had only little increase in moisture (0.3%) and the layer in between increased at an average of 1.5% increase. Figures 7.31(A) and 7.31(B) show that the capillary rise for the Branch A-2-4 soil was more than 18 in. The degree of saturation was about 94% for the bottom layer, and the soil was saturated for the layer of 6-12 in, above the embankment after a 56-day period.

7.3.1.11 Iron Bridge A-2-6 Soil

The Iron Bridge A-2-6 soil had a delay of increase in moisture on the first day, and after that the water content increased 3% within three days and about 5.4% after 17 days for the bottom layer. The layer of 12-18 in. above the embankment had a relatively low water content compared to the layers next to it and the water content decreased during the first week. Figures 7.32(A) and 7.32(B) show that the capillary rise was about 18 in. with a 0.6% increase in water content for the top layer. The degree of saturation was about 81%.

When the groundwater level was raised to +12 in. above the embankment, the water rose to the top layer after one week. The layer of 6-12 in. had less water content increase than the other layers. Figures 7.33(A) and 7.33(B) show that the capillary rise was more than 18 in.

7.3.2 Discussion on Capillary Rise Behavior

The height of the capillary rise for all eleven soils is summarized in Table 7.6 with the time spent to reach equilibrium. The final height of the capillary rise is shown in Figure 7.34 for a raise in the groundwater level to both 0 in. and +12 in. above the embankment. The A-2-4 (12%), A-2-4 (24%), and A-2-4 (30%) soils had the highest capillary rise height (reached to the top layer of the subgrade), while the A-2-4 (20%) soil had no capillary rise (0 in.).

The rate of the capillary rise, which is the height of capillary rise versus time to reach that height, is illustrated in Figure 7.35 for all eleven soils with groundwater level at the top of the embankment. The rate of capillary rise is affected by many factors such as permeability, porosity, capillary rise height, etc. Levy County A-3 (4%) had the highest permeability value; its capillary rise speed should be the highest one. But there is no such a relationship between the capillary rise rate and the permeability value due to the interference of several affecting factors. The capillary rise could not be determined accurately, because the moisture profile throughout the entire test pit could not be made uniformly; there was only one TDR probe available to measure the moisture content for each level (the elevation difference was 6 in.). From the recorded moisture profiles for all eleven soils, there was likely some malfunction with the TDR probes function. The accurate prediction of the capillary rate demanded successful permeability modeling which simulates the variation of unsaturated permeability as moisture develops within capillary fringe and the accurate moisture content profile with time.

Figures 7.36 and 7.37 address the capacity of the soil to take on the water due to the capillary rise effect. From the figures, it can be seen that the A-2-4 (12%) soil exhibited a high level of capillary rise ability, and had a water content increase of 1.8% for the top layer and about 27% in total for all six layers when the water table was raised from drained conditions to the top of the embankment. The capillary rise was much faster in the beginning and gradually slowed down afterwards. This can be observed in Figure 7.35.

Both the capillary rise from the groundwater level from drained condition to 0 in. above the embankment and from 0 in. to +12 in above the embankment were studied. Since the capillary rise can be limited by the height of the placement, the data from the case in which the water table was raised to 0 in. above the embankment was more representative of the capillary rise behavior. The data showed that the A-2-4 (12%) soil had both the highest capillary rise and the highest increase in moisture. This can be attributed the higher suction value. But with the high suction value, the A-2-4 (20%) had almost no capillary rise. This behavior is abnormal.

The accuracy of the moisture data should be reexamined due to the abnormality of the TDR measurements. The moisture data obtained from the analysis of the moisture effect in this study should only be considered as a reference, even though it gives us a general understanding of capillary rate and capacity for each soil, which would be helpful in design and construction for some sudden increases in groundwater table.

7.4 TEST-PIT EQUIVALENT MODULUS STUDY

The equivalent modulus values for the eleven subgrade materials for various groundwater levels are described and analyzed in this section using the experimental results presented in Chapter 5. The average equivalent modulus from 10,000 to 30,000 cycles was used for analysis. The test results will be further discussed in Chapter 8 as a case study.

7.4.1 Observation of Experimental Results

7.4.1.1 Levy County A-3 Soil

Since the moisture differences within the top layer of soil for the groundwater levels of -20 in., 0.0 in. and +12 in. above the embankment were quite limited (Figure 5.21), they led to no considerable changes in the equivalent modulus for the A-3 sandy soil. It showed that the increase in moisture in the middle and lower layers of the soil had only a limited influence on the decrease of the soil modulus.

In general, the equivalent modulus for this soil was less sensitive to the variation of moisture content (Figures 5.21, 5.22 and 5.23), especially in a situation when the moisture content of the subgrade near the loading point was below optimum moisture (non-flooded situation).

The degree of saturation under test conditions can be found in Table 7.7. The average equivalent modulus for each water condition with the water content range for Levy County A-3 soil is presented in Table 7.18.

7.4.1.2 SR70 A-3 Soil

No significant difference of the equivalent modulus was showed when the moisture differences within the top layer of the subgrade soil were quite limited for low groundwater level (Figure 5.24). Although a significant difference existed in the equivalent modulus between the drained condition and groundwater level being at 12 in. above the embankment (Figures 5.25), generally the equivalent modulus decreased slightly with the increase of moisture content in A-3 soil.

In the flooded conditions (Figure 5.27), when the moisture content reached a certain level the differences in soil modulus could be insignificant. In the 9/29/99 plate load test (Test No. 2-4), no significant equivalent modulus change was detected when the water level was adjusted from +12 in. to +36 in. above the embankment, even though there was quite a difference for moisture content levels and degrees of saturation (Figure 5.25).

Since the loading location remained the same for this soil in all tests previous to Test No. 2-6, it might be suspected that the test results would not be satisfactory due to preloading of the site. However, the relocated test (Test No.2-5, conducted on 10/5/99 under the same +36 in. water table) revealed a temporary decrease for the value of the equivalent modulus between 50 and 10,000 load cycles, but eventually achieved the same result for higher repetitions of load.

The degree of saturation under test conditions can be found in Table 7.8. The average equivalent modulus at each water condition with the water content range for SR70 A-3 soil is presented in Table 7.19.

7.4.1.3 SR70 A-2-4 Soil

No obvious moisture difference existed within the top layer of the SR70 A-2-4 soil when the groundwater level was raised from 0.0 in. to 12.0 in. above the embankment (Figure 5.28). The decrease in modulus was caused mainly by an increase in moisture content within the middle and bottom layers of soil.

For the A-2-4 soil, the equivalent modulus was more sensitive to changes in moisture when the groundwater level was changed to the top of the subgrade under a 50-psi plate load with limerock base layer, as illustrated in Figures 5.29.

The degree of saturation under test conditions can be found in Table 7.9. The average equivalent modulus for each water condition with the water content range for SR70 A-2-4 soil is presented in Table 7.20.

7.4.1.4 A-2-4(12%) Soil

As can be seen in Figures 5.32(A) and 5.32(B), when the water table was raised from 0 in. to +12 in., there was a moisture difference of about 2% in each layer of the subgrade. The modulus did not change much due to the increase in moisture. However, when the water table was raised again from +12 in. to +36 in., the modulus decreased by about 28%, as shown in Figures 5.33(A) and 5.33 (B). The A-2-4 (12%) soil is sensitive to changes in high groundwater levels.

The degree of saturation under test conditions can be found in Table 7.10. The average equivalent modulus for each water condition with the water content range for A-2-4 (12%) soil is presented in Table 7.21.

7.4.1.5 A-2-4(20%) Soil

As can be seen in Figures 5.34(A) and 5.34(B), when the water table was raised from 0 in. to +12 in., the decrease in the equivalent modulus was insignificant. When the groundwater level was raised from +12 in. to +36 in. above the embankment, there was a decrease in modulus of about 17%, even though the water content doubled for the top layer of soil, as shown in Figures 5.35(A) and 5.35(B). The A-2-4(20%) soil is not considered sensitive to the change in moisture in terms of equivalent modulus.

The degree of saturation under test conditions can be found in Table 7.11. The average equivalent modulus for each water condition with the water content range for A-2-4 (20%) soil is presented in Table 7.22.

7.4.1.6 A-2-4 (24%) Soil

From Figures 5.36(A) and 5.36(B), there was a drop in modulus of about 28% when the groundwater level was raised to +12 in. above the embankment. When the groundwater level was raised from +12 in. to +36 in. above the embankment, the modulus decreased to about 21% with a significant increase in moisture of 7%, as shown in Figures 5.37(A) and 5.37(B). The A-2-4(24%) soil is not sensitive to the change in moisture.

The degree of saturation under test conditions can be found in Table 7.12. The average equivalent modulus for each water condition with the water content range for A-2-4 (24%) soil is presented in Table 7.23.

7.4.1.7 A-2-4 (30%) Soil

As shown in Figures 5.38(A) and 5.38(B), when the groundwater level was raised from 0 in. to +12 in., the decrease in modulus was insignificant. But when the groundwater level was raised from +12 in. to +36 in., the modulus had an obvious decrease from 260 MPa to 99 MPa with a limited change in water content for all layers, as shown in Figures 5.39(A) and 5.39(B). This soil type, A-2-4 (30%), is very sensitive to the change in moisture in response to the high groundwater levels. It was abnormal that the moisture profiles did not change due to the change in groundwater levels. The explanation for this was that the TDR probes were damaged during installation and compaction.

The degree of saturation under test conditions can be found in Table 7.13. The average equivalent modulus for each water condition with the water content range for A-2-4 (30%) soil is presented in Table 7.24.

7.4.1.8 Miami Oolite A-1 Soil

Because the Miami Oolite A-1 is very stiff, only modulus data under 50-psi plate load test were measured with a limerock base layer. The modulus was reduced 61% when the water table was raised from +12 in. to +36 in. above the embankment, as shown in Figure 5.40(A). However, a decrease in moisture content of only about 1% for the top layer is shown in Figure 5.40(B). This soil is sensitive to the water change.

The degree of saturation under test conditions can be found in Table 7.14. The average equivalent modulus for each water condition with the water content range for the Miami Oolite A-1 soil is presented in Table 7.25.

7.4.1.9 Spring Cemetery A-2-4 Soil

As shown in Figure 5.41(A), there was no obvious difference in equivalent modulus when the water table was raised from +0 in. to +12 in., while there was a reduction of about 28% in equivalent modulus when the water table was raised to +24 in. above the embankment for the 20 psi plate load without a limerock base layer. The difference was due to the increase in moisture in the top layer (Figure 5.41(B)).

After the limerock base was placed, the plate load tests were conducted at groundwater levels of 0 in., +12 in., +24 in. and +36 in. with a wetting process under a 50-psi plate load. When the water table went down from +36 in. to +24 in. and then to +12 in. above the embankment (drying process), the soil retained more water than was retained with the wetting process at the same water level. Therefore, the equivalent moduli from drying process were lower than those from wetting process. As shown in Figures 5.42(A) and 5.42(B), the equivalent moduli were low when the groundwater levels were at the +24 in. and +36 in. above the embankment.

The degree of saturation under test conditions can be found in Table 7.15. The average equivalent modulus for each water condition with the water content range for Spring Cemetery A-2-4 soil is presented in Table 7.26.

7.4.1.10 Branch A-2-4 Soil

As shown in Figure 5.43(A), there was no obvious difference in equivalent modulus when the groundwater level was raised from +0 in. to +12 in., while there was a reduction of about 34% in equivalent modulus when the water table was raised to +24 in. above the embankment under a 20-psi plate load without the limerock base layer.

After the limerock base was placed, the plate load tests were conducted at groundwater level of 0 in., +12 in., +24 in. and +36 in. above the embankment with a wetting process under a 50-psi plate load. There was only a 10% reduction in equivalent modulus when the groundwater level was raised from 0 in. to +12 in. above the embankment while the reduction rate was about 42% when the groundwater level was raised from +12 in. to +24 in. above the embankment. When the groundwater level was at +36 in. above the embankment, the equivalent modulus decreased 77%, from 671 MPa to 157 MPa. When the groundwater level went down from +36 in. to +24 in. and then to +12 in. above the embankment (drying process), the soil had more water retained than was retained with the wetting process at the same water level. Therefore, the equivalent moduli from the drying process were lower than those from the wetting process. As shown in Figures 5.44(A) and 5.44(B), the equivalent modulus was low when the groundwater level was at +24 in. and +36 in. above the embankment. Figure 5.44(B) shows that the water content was higher for the top layers when the groundwater level was at +36 in. and +24 in., which was drawn down from +36 in. above embankment.

The degree of saturation under test conditions can be found in Table 7.16. The average equivalent modulus for each water condition with the water content range for Branch A-2-4 soil is presented in Table 7.27.

7.4.1.11 Iron Bridge A-2-6 Soil

As shown in Figure 5.45(A), the equivalent modulus slightly increased as the groundwater level rose from 0 in. to +12 in.

above the embankment, but had a significant decrease of 33% when the groundwater level was raised from +12 in. to +24 in. above the embankment. As can be seen in Figure 5.45(B), the increase in water content with the groundwater level from 0 in. to +12 in. above the embankment was much higher than when the groundwater level was raised from +12 in. to +24 in. above the embankment.

After the limerock base was placed, the plate load tests were conducted at gorundwater level 0 in., +12 in., +24 in. and +36 in. above the embankment with a wetting process under a 50-psi plate load. The equivalent modulus was about three times that which was obtained without the limerock base. As shown in Figure 5.46(A), the equivalent modulus had a decrease of 43% when the groundwater level was raised from 0 in. to +24 in. above the embankment and a decrease of 81% from 0 in. to +36 in. above embankment. When the water table went down from +36 in. to +24 in. and then to +12 in. above the embankment (drying process), the soil had more water retained than that which was obtained with the wetting process at the same water level. Therefore, the equivalent moduli from drying process were lower than those from the wetting process. As shown in Figures 5.46(A) and 5.46(B), the equivalent modulus was low when the water table was at the +36 in. and +24 in. water table level. Figure 5.46(B) shows that the water content was higher for the top layers when the water table was at +36 in. above the embankment.

The degree of saturation under test conditions can be found in Table 7.17. The average equivalent modulus for each water condition with water content range for Iron Bridge A-2-6 soil is presented in Table 7.28.

7.4.2 Analysis of Experimental Results

The plate load test results are summarized in Table 7.29 and presented in Figures 7.38, 7.39, and 7.40. The equivalent modulus (EQ modulus) values for different types of soil were affected to a different extent under various levels of groundwater table. The reduction rates in EQ modulus for different water level changes are presented in Table 7.30 and shown in Figures 7.41 and 7.42.

For the change of water table level from 0 in. to +12 in. under a 20-psi plate load without limerock, there was not much change of equivalent modulus values for most soils except for A-2-4 (24%) soil, which had a reduction of 28% in equivalent modulus, as shown in Table 7.30 and Figure 7.41. For the Phase III soils (Spring Cemetery, Branch, and Iron Bridge), the reduction rate in equivalent modulus became much higher when the groundwater level was raised up to +24 in. above the embankment.

For the change of water table level from +12 in. to +36 in. under a 50-psi plate load with a limerock base built on top, there was significant change of modulus values for the eleven soils except for the A-2-4 (20%) soil. The most sensitive soils were SR70 A-2-4 (14%), A-2-4 (30%), Miami Oolite A-1, Branch A-2-4 (23%), and Iron Bridge A-2-6 (31%). The reduction rates were 53%, 62%, 61%, 74% and 79%, respectively, as shown in Table 7.30 and Figure 7.42.

Analysis was also done with the water drained from +36 in. to +24 in. and from +24 in. to +12 in. above the embankment for the Phase III soils. From Table 7.30, the Iron Bridge A-2-6 soil showed the highest increase rate, 96%, with the groundwater level lowered from +36 in. to +24 in. above the embankment, and the rate of 237% with the water table lowered from +36 in. to +12 in. above the embankment.

When considering the effect of the plate load and limerock base layer, the increase in equivalent modulus ranged from a low of 42% for A-2-4 (30%) soil with a 2-ft base clearance to a high of 233% for the Iron Bridge A-2-6 soil with a 3-ft base clearance. The Iron Bridge A-2-6 soils had the highest increase rates in equivalent modulus when plate load increased from 20 psi to 50 psi with limerock built on the top of soil, as shown in Table 7.31 for the three water levels of 0 in., 12 in., and 24 in. above the embankment. In general, the benefit from adding a base layer was significant for the high-fine soils. The analysis for the water table at 0 in. and 24 in. above the embankment under a 50-psi plate load were only available for Phase III soils.

There was no simple relationship between the reduction in equivalent modulus and the percent of fines in soils. However, with the condition of a 50-psi plate load with limerock base, the two A-3 soils (Levy A-3 and SR70 A-3) were less sensitive to the groundwater level adjustments, while the A-2-6 soil (Iron Bridge) had the highest reduction rate in equivalent modulus when subjected to the groundwater level change. As for the A-2-4 soils, A-2-4 (20%) did not change much as the groundwater level changed, while the SR70 A-2-4 and A-2-4 (30%) had a relatively high reduction rate (over 50%) than the other A-2-4 soils.

The fluctuation of equivalent modulus values as a result of the change in levels of the groundwater table illustrated that the mere soil structure itself was not the controlling factor for elastic deformation. But the presence of water did not necessarily mean a decrease in equivalent modulus of the soils. For example, no significant difference occurred for the equivalent modulus of extremely coarse gravel whether it was flooded or completely drained.

7.4.3 Discussion

Many properties such as clay content, permeability, suction value, gradation, etc. of soils can affect the resilient modulus value directly or indirectly. This research study was focused more on the effect of moisture on the resilient modulus of pavement subgrades. In the literature, the suggestion has been raised that correlating the resilient behavior of soil with the suction value it assumes, may be more appropriate than using moisture content or degree of saturation as indicators for the analysis of subgrade resilient behavior. For a specific subgrade soil, the resilient modulus is more or less dependent on the capillary moisture developed from the groundwater table. However, for different subgrade materials, the resilient modulus is more dependent on the capillary potential of each individual soil (suction value) rather than the capillary moisture accumulated within a capillary zone (Liu, 2001).

One of the concerns for an experimental program of test-pit tests was to find out whether cyclic loading has any effect on the moisture content of the subgrade materials in the test-pit. For all of the plate load tests conducted in the test-pit, moisture readings from the TDR probes from 3 in. to 33 in. below the plate loading area showed that there were no changes of moisture content before and after the implementation of cyclic loading.

The TDR probes deployed within the test-pit did not serve their function as moisture-detecting sensors very well. The fluctuations of the moisture content readings showed that either there were damages on the TDR probes or the soils around the TDR probes were disturbed. For a precise measurement of moisture content, more TDRs should be used and distributed evenly in the test pit for each layer to avoid the deviations. An adequate calibration of the TDR probes with each individual soil before that soil is compacted into the test-pit for investigation should increase the reliability of the data.

In test-pit tests, the equivalent modulus values were dependent on the effect of the bottom embankment layer as well as the top limerock layer. From Figure 7.40, the beneficial effect of adding a base layer with limerock is clearly demonstrated. When a 5-in.-thick layer of limerock was added and the load was increased from 20 psi to 50 psi, the equivalent modulus values were almost doubled under the same level of groundwater table at +12 in. above the embankment.

The modulus from the plate load tests was the equivalent modulus for the combined subgrade and embankment underneath, and even the base limerock layer. The modulus value for limerock layer may be estimated from this simple comparison. The subgrade layer modulus for various levels of high groundwater table can be computed from simulation computer programs when the moduli of the base limerock and embankment are available. A layered system established to estimate the resilient modulus of subgrade layer using the KENLAYER program for each soil type is discussed in next section.

7.5 LAYERED SYSTEM SIMULATION FOR TEST-PIT STUDY

7.5.1 Purpose

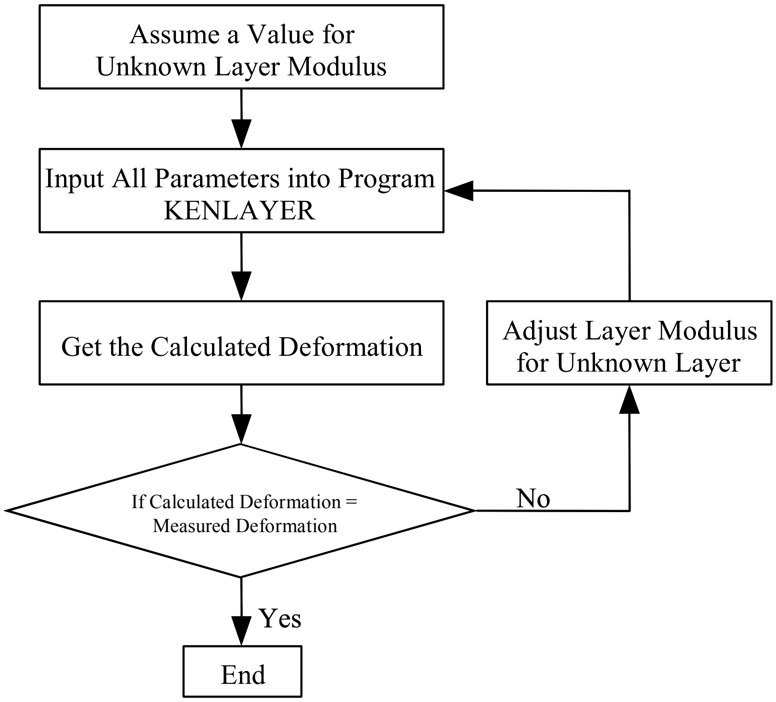
In reality, the pavement has several layers. For a general pavement profile, there are the asphalt concrete layer, base layer, subgrade layer and embankment layer, from top to bottom. For a more complicated layer system, there may be other layers such as an asphalt crack relief layer, drainage layer, and so on. In test-pit tests, there were at least two layers; embankment and subgrade layers. For some tests, the third layer, a 5-in. limerock layer, was added on the top.

Because the water table had different levels in different periods, the subgrade layer should be divided into several layers. To simplify the problem, all six subgrade layers (lifts) were considered as one single layer.

The purpose for setting up a layer system for a test-pit test is to get the modulus for each layer (actually for the 36-in.-thick subgrade layer for each soil type) instead of the equivalent modulus for all of the layers. Then the layer modulus for each subgrade can be compared with the results coming from the laboratory resilient modulus tests.

7.5.2 Layered System Calculations and Analysis

To calculate the layer modulus of the subgrade soils, a pavement analysis and design software, "KENPAVE," developed by Huang (1993), was utilized. The following flow chart shows the procedures of calculation for layer moduli in the KENPAVE program:



Based on the two-layer system, the layer modulus of a subgrade without a limerock layer on top could be calculated by providing the saturated embankment moduli (assume 11,207 psi for a soaked embankment). The summary of the layer moduli at different groundwater levels (0 in., +12 in. and +24 in. above embankment) under a 20-psi plate load without a limerock base layer are presented in Tables 7.32, 7.33, and 7.34.

To get the subgrade layer moduli for those subgrades with limerock built on top, the limerock layer moduli should be obtained first. This can be done by treating the layers below the limerock layer as one layer based on the two-layer system. Tables 7.35, 7.36, and 7.37 show the calculations of the limerock layer moduli for the eleven subgrades when the groundwater levels were stabilized at 0 in., +12 in., and +24 in. above the embankment under a 50-psi plate load. The subgrade layer moduli can be obtained by providing the limerock moduli on top and the saturated embankment moduli below based on a three-layer system. The subgrade layer moduli for the eleven soil types are summarized in Tables 7.38 through Table 7.43.

The layer moduli for the eleven soils under all the different conditions are summarized in Table 7.44. The reduction rates are shown in Table 7.45. The results show that when the groundwater level increased from 0 in. to +12 in. above the embankment, the layer modulus for the subgrade decreased by a degree. The A-2-4 (24%) soil had the highest reduction rate (33%) compared to other soils, while the A-2-4 (12%) soil had almost no change in reduction rate. In contrast, the Iron Bridge Soil even had an increase (14%) in layer modulus when the groundwater level increased.

When the water table continued to increase from +12 in. to +36 in. above the embankment, the subgrade layer would be totally emerged in water. The layer modulus for each subgrade had a big drop. These results also proved the theory that water can have a great effect on the pavement modulus.

It is well known that granular materials and subgrade soils are nonlinear with elastic modulus varying with the level of stresses. The nonlinear material properties have been incorporated in KENLAYER. According to the theoretical development of KENLAYER, use of KENLAYER was adequate in order to estimate the layer modulus in the study. The comparison between the layer modulus and the laboratory resilient modulus will be further discussed in Chapter 9.

Table 7.1 Moisture Profile after Drainage for Subgrade Materials in Test-pit Test (Phase I)

Levy County A-3 soil (#200 Passing 4%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	16.6	16.4	16.4	14.8	14.7	15.1	0 hour
		15.8	15.7	15.1	13.8	13.1	8.7	2 hours
		15.7	13.6	11.1	9.8	8.8	6.9	26 hours
		15.6	12.3	10.3	9.1	8.3	6.3	50 hours
SR-70 A-3 Soil (#200 Passing 8%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	16.8	19.9	20.4	21.4	19.6	17.1	0 hour
		15.4	19.3	20	21.1	19.3	16.6	3 hours
		15.1	18.8	20	20.7	16.1	12.9	27 hours
		14.6	18.5	18.9	19.5	15.8	10.4	51 hours
		10.3	13.1	12.8	12.4	11.2	7.9	2 weeks
		9.1	11.9	11.9	11.3	9.7	6.9	30 days
		8.7	10.8	10.9	10.3	8.6	6.3	66 days
		8.6	10.5	10.6	10	8.29	6.13	86 days
SR-70 A-2-4 Soil (#200 Passing 14%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	14.7	18	11.2	16	19.2	30.5	0 hour
		14.4	17.7	11.1	15.8	19.1	30.4	3 hours
		14.4	17.7	11.1	15.8	19	30.5	27 hours
		14.4	17.7	11.1	15.7	18.6	30.5	51 hours
		14.3	17.5	10.9	12.6	14.7	28	2 weeks
		14.2	17.2	10.8	12.1	14.5	21.1	30 days
		14.2	15.3	10.2	11.6	14	16.2	66 days
		14	14.6	9.9	11.2	13.4	13.9	86 days

Table 7.2 Moisture Profile after Drainage for Subgrade Materials in Test-pit Test (Phase II)

A-2-4 (12%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	11.3	13.2	13.5	11.9	7.5	11.4	0 days
		11.1	13	13.3	11.8	7.5	10.9	10days
		11.1	13	13.3	11.7	7.5	10	23days
		11.1	12.9	13.3	11.7	7.9	8.8	39days
		11	12.9	13.2	11.6	6.7	7.9	59days
A-2-4 (20%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	9.64	9.67	9.89	11.17	9.47	8.08	0 days
		9.57	9.56	9.81	11.07	9.3	7.68	10days
		9.58	9.56	9.8	11.07	9.32	7	23days
		9.58	9.54	9.78	11.07	8.04	6.31	39days
		9.56	9.52	9.76	11.03	6.6	5.45	59days
A-2-4 (24%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	7.59	8.76	11.42	12.08	9.7	13.01	0 days
		7.42	8.6	11.24	11.88	9.59	12.93	10days
		7.44	8.6	11.23	11.84	9.54	12.81	23days
		7.42	8.57	11.21	11.82	9.53	10.27	39days
		7.37	8.5	11.12	11.73	9.04	7.83	59days
A-2-4 (30%)	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	15.58	16.82	16.31	14.83	11.42	8.62	0 days
		15.42	16.57	16.04	14.64	11.3	8.52	10days
		15.4	16.58	16.05	14.64	11.3	8.47	23days
		15.41	16.59	16.06	14.66	11.31	8.36	39days
		15.34	16.52	16	14.63	11.3	8.25	59days
Oolite	Elevation (in.)	3	9	15	21	27	33	Elapse Time
	Moisture Profile after Drainage (%)	3.17	3.24	3.83	6.61	4.56	4.44	0 days
		3.15	3.21	3.78	6.46	4.52	4.25	10days
		3.15	3.21	3.78	6.4	4.54	4.13	23days
		3.15	3.2	3.77	6.34	4.48	3.96	39days
		3.14	3.2	3.71	6.25	4.44	3.67	59days

Table 7.3 Moisture Profile for Subgrades after the Adjustment of Groundwater Level in Test-pit Test (Phase I)

Subgrade Material	Water Table (in.)	Elapsed Time (days)	Moisture Profile in Each Elevation above Embankment, %					
			3 in.	9 in.	15 in.	21 in.	27 in.	33 in.
Levy County A-3 (4%)	-20 to 0	0	12.50	9.64	8.35	7.20	6.97	4.97
		1	12.76	9.73	8.34	7.17	6.95	4.95
		3	13.54	9.96	8.31	7.09	6.86	4.86
		7	14.12	10.49	8.47	7.06	6.77	4.76
		14	14.18	10.48	8.47	7.00	6.65	4.62
		21	14.94	13.32	9.12	7.14	6.63	4.55
		28	15.00	14.35	10.81	7.96	6.74	4.55
	0 to +12	0	15.09	14.50	10.89	8.01	6.81	4.58
		4	15.34	14.71	14.13	10.38	7.12	4.31
		7	15.42	14.80	14.45	12.19	8.26	4.60
		14	15.57	14.92	14.60	12.94	8.96	5.43
		21	15.83	15.18	14.99	13.33	9.17	5.54
		28	16.11	15.46	15.42	13.73	9.58	6.23
		47	16.46	15.74	15.75	14.03	9.56	6.25
SR70 A-3 (8%)	-24 to -12	0	8.63	10.62	10.80	10.78	9.73	7.21
		1	9.20	10.63	10.80	10.78	9.71	7.19
		3	9.36	10.64	10.78	10.77	9.69	7.17
		7	9.45	10.66	10.77	10.74	9.62	7.11
	-12 to 0	0	9.52	10.66	10.69	10.64	9.32	6.93
		1	16.48	11.54	10.69	10.62	9.30	6.92
		4	16.58	12.34	10.84	10.61	9.25	6.89
		7	16.72	12.77	10.97	10.63	9.21	6.87
		14	17.15	13.00	11.07	10.63	9.13	6.83
		18	17.34	13.06	11.09	10.62	9.08	6.80
	0 to +12	0	17.17	13.44	11.17	10.59	8.86	6.68
		1	17.15	19.92	12.84	10.62	8.85	6.68
		4	17.05	20.64	15.93	11.82	8.86	6.66
		7	17.01	21.03	16.82	12.32	8.97	6.66
		14	16.89	20.97	17.60	12.75	9.28	6.66
		28	16.78	20.70	18.64	13.27	9.64	6.72
		43	16.69	20.38	19.30	13.80	10.68	7.97
SR70 A-2-4 (14%)	-24 to -12	0	12.09	11.13	8.90	8.78	9.03	7.41
		1	12.12	11.16	8.96	8.79	9.08	7.50
		3	12.15	11.19	8.93	8.81	9.07	7.45
		7	12.20	11.23	8.95	8.84	9.11	7.48
		15	12.31	11.33	9.01	8.89	9.17	7.53
		24	12.41	11.44	9.08	8.95	9.24	7.56
	-12 to 0	0	12.41	11.44	9.08	8.95	9.24	7.56
		1	12.57	11.47	9.09	8.97	9.27	7.59
		4	13.17	11.50	9.09	8.96	9.23	7.55
		7	13.60	11.57	9.13	9.00	9.28	7.59
		14	13.92	11.66	9.13	8.99	9.27	7.58
		21	14.08	11.75	9.12	8.97	9.24	7.54
		28	14.23	11.90	9.18	9.04	9.31	7.59
		35	14.37	12.05	9.22	9.09	9.37	7.62
		42	14.46	12.16	9.22	9.08	9.34	8.28
	0 to +12	0	14.46	12.16	9.22	9.08	9.34	8.28
		1	14.58	15.37	9.26	9.12	9.35	8.21
		4	14.58	18.50	9.42	9.11	9.33	8.16
		7	14.61	18.51	9.57	9.20	9.44	8.25
		14	14.65	18.52	9.81	9.23	9.42	8.17
		28	14.63	18.39	10.42	9.28	9.39	8.12
		43	14.58	18.25	10.80	9.73	9.61	11.46

Table 7.4 Moisture Profile for Subgrades after the Adjustment of Groundwater Level in Test-pit Test (Phase II)

Subgrade Material	Water Table (in.)	Elapsed Time (days)	Moisture Profile in Each Elevation above Embankment, %					
			3 in.	9 in.	15 in.	21 in.	27 in.	33 in.
A-2-4 (12%)	-24 to 0	0	5.08	5.06	4.60	4.12	2.50	2.99
		4	6.55	5.89	4.82	4.11	2.46	2.74
		7	9.90	8.65	6.27	4.47	2.44	2.66
		14	10.03	10.10	7.21	5.31	2.72	2.85
		27	10.28	11.80	12.18	9.58	4.01	3.43
		33	10.25	11.79	11.69	7.87	3.96	4.64
		53	10.39	12.06	11.60	7.83	3.92	4.64
	0 to +12	0	10.39	12.06	11.60	7.83	3.92	4.64
		1	10.40	12.08	11.62	7.82	3.93	4.67
		3	10.51	12.29	12.48	10.84	5.20	5.86
		7	10.53	12.33	12.53	10.93	5.24	6.12
		14	10.61	12.43	12.65	11.05	5.22	6.14
		28	10.73	12.63	12.86	11.39	5.54	6.38
		50	10.87	12.80	13.09	11.62	5.66	6.43
		77	11.11	12.95	13.20	11.04	6.42	7.67
		86	11.15	12.95	13.27	11.19	6.44	7.68
A-2-4 (20%)	-24 to 0	0	9.50	9.18	9.31	10.42	4.28	3.57
		4	9.51	9.20	9.31	10.40	4.21	3.39
		7	9.52	9.21	9.30	10.37	4.07	3.28
		14	9.54	9.23	9.31	10.33	3.93	3.32
		27	9.55	9.26	9.31	10.31	3.85	3.33
		53	9.55	9.32	9.39	10.36	3.92	3.26
	0 to +12	0	9.55	9.32	9.39	10.36	3.92	3.26
		1	9.55	9.32	9.39	10.36	3.92	3.27
		3	9.57	9.35	9.40	10.36	3.92	3.29
		7	9.57	9.37	9.43	10.37	3.93	3.32
		14	9.57	9.39	9.49	10.43	3.99	3.39
		28	9.58	9.45	9.62	10.57	4.19	3.54
		50	9.58	9.45	9.71	10.74	4.79	3.51
		86	9.60	9.51	9.77	10.88	5.61	3.83
A-2-4 (24%)	-24 to 0	0	7.04	6.84	6.67	7.12	5.49	5.23
		4	7.19	7.11	6.80	7.16	5.47	5.25
		7	7.25	7.68	7.08	7.26	5.48	5.26
		14	7.27	8.00	7.52	7.50	5.59	5.35
		27	7.27	8.39	8.96	7.92	5.78	5.47
		45	7.20	8.50	10.20	8.48	6.10	5.65
		53	7.22	8.55	10.20	8.43	6.00	5.25
	0 to +12	0	7.22	8.55	10.20	8.43	6.00	5.25
		1	7.23	8.55	10.19	8.40	5.97	5.30
		3	7.29	8.62	11.09	9.02	6.13	5.46
		7	7.28	8.60	11.12	10.62	6.50	5.66
		14	7.28	8.62	11.20	11.23	6.72	5.88
		28	7.31	8.61	11.32	11.88	6.93	6.00
		57	7.41	8.69	11.47	12.02	6.88	5.46
		86	7.39	8.63	11.37	12.08	7.02	6.05

Table 7.4 - Continued

Subgrade Material	Water Table (in.)	Elapsed Time (days)	Moisture Profile in Each Elevation above Embankment, %					
			3 in.	9 in.	15 in.	21 in.	27 in.	33 in.
A-2-4 (30%)	-24 to 0	0	11.93	12.17	11.73	12.48	10.82	7.87
		4	16.23	13.22	12.54	13.22	11.08	8.39
		7	16.21	13.60	12.56	13.21	11.06	8.16
		14	16.13	13.80	12.58	13.08	10.98	8.18
		21	16.10	13.90	12.72	13.24	11.10	8.32
		28	16.09	13.88	12.61	13.21	11.05	8.47
		40	15.99	14.22	12.52	12.94	10.89	7.98
	0 to +12	0	15.99	14.22	12.52	12.94	10.89	7.98
		1	15.96	16.37	12.59	12.86	10.85	7.89
		7	15.90	16.86	16.58	13.25	10.97	7.87
		14	15.84	16.82	16.52	13.48	11.18	8.18
		22	15.95	16.97	16.67	13.85	11.34	8.54
		28	15.84	16.84	16.43	13.81	11.25	8.26
		127	15.56	16.82	16.16	14.04	11.24	8.05
Oolite	-24 to 0	0	3.00	2.54	2.49	6.04	4.75	3.99
		4	3.20	3.09	3.22	6.12	4.73	3.91
		7	3.20	3.08	2.84	5.98	4.70	3.84
		40	3.19	3.09	2.49	5.85	4.48	3.66
	0 to +12	0	3.19	3.09	2.49	5.85	4.48	3.66
		1	3.20	3.19	2.51	5.82	4.46	3.64
		2	3.20	3.20	2.51	5.82	4.43	3.61
		7	3.20	3.23	2.97	5.87	4.42	3.58
		14	3.19	3.25	3.37	5.97	4.42	3.69
		23	3.18	3.24	3.70	6.26	4.55	4.24
		28	3.19	3.24	3.65	6.19	4.59	3.95
		127	3.17	3.24	3.53	6.05	4.37	3.49

Table 7.5 Moisture Profile for Subgrades after the Adjustment of Groundwater Level in Test-pit Test (Phase III)

Subgrade Material	Water Table (in.)	Elapsed Time (days)	Moisture Profile in Each Elevation above Embankment, %					
			3 in.	9 in.	15 in.	21 in.	27 in.	33 in.
Spring Cemetery A-2-4 (15%)	-24 to 0	0	5.50	5.80	7.30	5.70	4.10	
		1	12.20	9.20	7.00	5.10	4.00	
		3	12.50	9.20	7.80	4.80	4.30	
		7	13.00	11.20	9.10	5.10	4.30	
		17	13.00	11.40	8.50	5.00	4.00	
	0 to +12	0	13.00	11.40	8.50	5.00	4.00	
		1	13.30	12.30	11.40	6.60	4.20	
		4	13.10	12.60	11.00	7.90	4.40	
		7	13.10	12.30	11.70	7.60	4.60	
		14	13.40	12.20	11.50	8.30	4.70	
		28	13.40	12.50	11.80	8.30	4.70	
		56	13.50	12.80	12.00	8.70	4.70	
Branch A-2-4 (23%)	-24 to 0	0	9.40	7.57	6.86	6.76	5.22	
		1	9.55	7.61	7.31	6.76	4.98	
		3	9.65	8.60	6.86	6.61	4.98	
		7	10.19	8.85	8.69	6.47	5.22	
		17	10.20	9.39	8.69	6.76	5.22	
	0 to +12	0	10.20	9.39	8.69	6.76	5.22	
		1	10.64	10.88	9.13	7.15	5.22	
		4	10.64	11.32	10.17	8.03	4.98	
		7	10.64	11.42	10.02	8.13	4.84	
		14	10.49	11.42	10.32	9.16	6.61	
		28	10.34	11.17	10.61	8.87	8.67	
		56	10.49	10.88	11.06	9.26	10.06	
Iron Bridge A-2-4 (31%)	-24 to 0	0	5.10	7.30	6.20	7.10	5.70	
		1	5.10	7.50	5.90	7.40	5.60	
		3	8.10	7.60	5.90	7.40	5.40	
		7	8.80	9.20	5.40	7.40	5.90	
		17	10.50	10.40	9.20	7.20	6.00	
	0 to +12	0	10.50	10.40	9.20	7.20	6.00	
		1	12.00	10.60	9.90	7.20	6.00	
		4	13.50	10.70	11.70	8.50	5.90	
		7	13.00	10.70	12.00	9.10	5.70	
		14	13.00	10.80	12.50	10.50	8.40	
		28	13.30	11.40	12.30	10.30	10.40	
		56	12.40	10.80	12.30	10.70	10.70	

Table 7.6 Summary of Capillary Rise for Subgrade Materials in Test-pit Test

Subgrade Soils	Water Table Level Change	Capillary rise (inch)	Moisture Gain (%)						Modulus Change (MPa)	Time to reach equilibrium (days)	Moistured stablized
			0-6	6-12	12-18	18-24	24-30	30-36			
Levy County A-3	-20 to 0	24	2.6	4.9	2.5	0.8	-0.2	-0.4	-33	28	yes
	0 to +12	24	1.4	1.2	4.9	6.0	2.7	1.7	-13	>47	no
SR70 A-3	-24 to -12	18	0.9	0.0	-0.1	-0.1	-0.4	-0.3		7	yes
	-12 to 0	18	7.6	2.8	0.5	0.0	-0.5	-0.2		18	yes
	0 to 12	18	-0.5	6.9	8.1	3.2	1.8	1.3	-30	42	yes
SR70 A-2-4	-24 to -12	24	0.3	0.3	0.2	0.2	0.2	0.2		24	yes
	-12 to 0	12	2.1	0.7	0.1	0.1	0.1	0.7		>42	no
	0 to +12	12	0.1	6.1	1.6	0.6	0.3	3.2	-29	>43	no
A-2-4, 12%	-24 to 0	36	5.3	7.0	7.0	3.7	1.4	1.6	-49	33	yes
	0 to +12	24	0.8	0.9	1.7	3.4	2.5	3.0	0	86	no
A-2-4, 20%	-24 to 0	0	0.1	0.1	0.1	-0.1	-0.4	-0.3	-13	0	yes
	0 to +12	24	0.1	0.2	0.4	0.5	1.7	0.6	-8	86	yes
A-2-4, 24%	-24 to 0	36	0.2	1.7	3.5	1.3	0.5	0.0	16	45	yes
	0 to +12	24	0.2	0.1	1.2	3.6	1.0	0.8	-52	51	yes
A-2-4, 30%	-24 to 0	36	4.1	2.1	0.8	0.5	0.1	0.1	-23	21	yes
	0 to +12	24	-0.4	2.6	3.6	1.1	0.4	0.1	-3	22	yes
Oolite	-24 to 0	18	0.2	0.6	0.0	-0.2	-0.3	-0.3	-37	4	yes
	0 to +12	12	0.0	0.1	1.0	0.2	-0.1	-0.2	190	23	yes
Spring Cemetery	-24 to 0	18	7.5	5.6	1.2	-0.7	-0.1			7	yes
	0 to +12	18	0.5	1.4	3.5	3.7	0.7		-5	21	yes
Branch	-24 to 0	18	0.8	1.8	1.8	0.0	0.0			16	yes
	0 to +12	18*	0.3	1.5	2.4	2.5	4.8		-8	>56	no
Iron Bridge	-24 to 0	18	5.4	3.1	3.0	0.1	0.3			17	no
	0 to +12	18*	1.9	0.4	3.1	3.5	4.7		17	40	yes

*There was no water content recorded for the top layer of Phase III soils (30 in. to 36 in. above the embankment). The capillary rise could be higher than that which was observed.

Table 7.7 Levy County A-3 Soil, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=W \times G_s \times \gamma_d / (\gamma_w \times G_s - \gamma_d)$)													
	US lb/ft^3	SI kN/m^3	-20 in.		0.0 in.		+12 in.		+12 in.		+12 in.		+36 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S	W	S
33	107.6	16.91	5.2	24.8	4.5	21.4	5.5	26.2	6.2	29.5	6.2	29.5	15.0	71.5	15.0	71.5
27	106.9	16.80	7.2	33.7	6.8	31.8	9.1	42.6	9.5	44.5	9.5	44.5	14.7	68.8	14.7	68.8
21	107.5	16.89	7.4	35.2	8.0	38.0	13.2	62.7	14.0	66.6	14.0	66.6	14.8	70.4	14.8	70.4
15	107.1	16.83	8.6	40.5	10.9	51.3	14.9	70.1	15.7	73.9	15.7	73.9	16.3	76.7	16.4	77.2
9	107.2	16.85	9.9	46.7	14.5	68.4	15.2	71.7	15.7	74.1	15.7	74.1	16.3	76.9	16.4	77.4
3	107.7	16.92	12.9	61.6	15.1	72.2	15.8	75.5	16.3	77.9	16.3	77.9	16.7	79.8	16.7	79.8
Test Number			1-1		1-2		1-3		1-4		1-5		1-6		1-7	

Table 7.8 SR70 A-3 Soil, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=W \times G_s \times \gamma_d / (\gamma_w \times G_s - \gamma_d)$)													
	US lb/ft^3	SI kN/m^3	-24 in.		-24 in.		0.0 in.		+12 in.		+12 in.		+36 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S	W	S
33	112.4	17.66	6.2	33.5	6.1	33.0	6.7	36.2	6.8	36.7	8.0	43.2	17.1	92.4	14.5	78.4
27	110.2	17.32	8.4	42.8	8.3	42.3	8.9	45.4	9.7	49.5	10.7	54.6	19.8	100.0	15.4	78.5
21	110.6	17.38	10.1	52.0	10.0	51.5	10.6	54.6	13.4	69.0	13.8	71.1	21.6	100.0	16.1	83.0
15	109.8	17.25	10.7	54.0	10.6	53.5	11.2	56.5	18.8	94.9	19.3	97.4	20.6	100.0	15.6	78.7
9	110.4	17.35	10.6	54.3	10.5	53.8	13.4	68.7	20.6	100.0	20.4	100.0	20.2	100.0	17.9	91.7
3	109.9	17.27	8.6	43.5	8.6	43.5	17.2	87.0	16.7	84.5	16.7	84.5	16.9	85.5	15.4	77.9
Test Number			2-6		2-7		2-1		2-2		2-3		2-4		2-8	

Table 7.9 SR70 A-2-4 Soil, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=W x G_s x \gamma_d / (\gamma_w x G_s - \gamma_d)$)													
	US lb/ft^3	SI kN/m^3	-24 in.		-24 in.		0.0 in.		+12 in.		+12 in.		+36 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S	W	S
33	115.6	18.17	14.6	86.1	13.9	81.9	8.1	47.7	8.1	47.7	11.4	67.2	33.2	100.0	22.7	100.0
27	117.8	18.51	13.4	84.0	13.4	84.0	9.4	58.9	9.3	58.3	9.6	60.2	19.5	100.0	16.4	100.0
21	117.8	18.51	11.1	69.6	11.2	70.2	9.1	57.0	9.3	58.3	9.7	60.8	16.0	100.0	12.8	80.2
15	117.9	18.53	9.8	61.6	9.9	62.2	9.2	57.8	10.5	66.0	10.8	67.9	11.1	69.8	10.0	62.8
9	116.8	18.35	14.6	89.0	14.6	89.0	12.1	73.7	18.3	100.0	18.3	100.0	18.0	100.0	15.8	96.3
3	113.1	17.77	13.9	76.5	14.0	77.1	14.4	79.3	14.6	80.4	14.6	80.4	14.6	80.4	14.0	77.1
Test Number			3-5		3-6		3-1		3-2		3-3		3-4		3-7	

Table 7.10 A-2-4 (12%), Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=W x G_s x \gamma_d / (\gamma_w x G_s - \gamma_d)$)									
	US lb/ft^3	SI kN/m^3	0.0 in.		+12 in.		+12 in.		+12 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S
33	110.6	17.38	4.8	24.7	6.4	33.0	7.7	39.7	7.7	39.7	11.1	57.2
27	110.6	17.38	4.0	20.6	5.6	28.9	6.4	33.0	6.4	33.0	7.5	38.6
21	110.6	17.38	7.9	40.7	11.5	59.3	11.1	57.2	12.1	62.3	11.8	60.8
15	110.6	17.38	11.7	60.3	12.9	66.5	13.2	68.0	13.3	68.5	13.5	69.6
9	110.6	17.38	12.0	61.8	12.7	65.4	12.9	66.5	13.0	67.0	13.2	68.0
3	110.6	17.38	10.4	53.6	10.8	55.6	11.1	57.2	11.2	57.7	11.3	58.2
Test Number			4-1		4-2		4-3		4-4		4-5	

Table 7.11 A-2-4 (20%), Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=WxG_sx\gamma_d / (\gamma_wxG_s - \gamma_d)$)											
	US lb/ft^3	SI kN/m^3	0.0 in.		+12 in.		+12 in.		+12 in.		+12 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S
33	124.4	19.55	3.3	25.3	3.5	26.5	3.3	24.9	3.7	28.4	3.8	28.6	8.0	61.0
27	124.4	19.55	4.0	30.1	4.6	35.3	4.7	35.7	5.7	43.0	5.7	43.2	9.4	71.3
21	124.4	19.55	10.4	78.8	10.7	81.3	10.7	81.4	10.9	82.5	10.9	82.5	11.1	84.7
15	124.4	19.55	9.4	71.3	9.7	73.7	9.7	73.8	9.8	74.4	9.8	74.3	9.9	74.9
9	124.4	19.55	9.3	70.9	9.5	71.9	9.5	71.9	9.5	72.3	9.5	72.4	9.6	73.3
3	124.4	19.55	9.6	72.6	9.6	72.9	9.6	72.9	9.6	73.0	9.6	73.0	9.6	73.1
Test Number			5-1		5-2		5-3		5-4		5-5		5-6	

Table 7.12 A-2-4 (24%), Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=WxG_sx\gamma_d / (\gamma_wxG_s - \gamma_d)$)											
	US lb/ft^3	SI kN/m^3	0.0 in.		+12 in.		+12 in.		+12 in.		+12 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S
33	116.3	18.28	5.3	31.5	6.0	35.9	5.7	34.0	6.1	36.4	6.1	36.4	13.1	78.4
27	116.3	18.28	6.0	36.1	7.0	42.1	7.0	41.8	7.0	42.3	7.0	42.2	9.7	58.5
21	116.3	18.28	8.4	50.7	12.1	72.5	12.2	73.1	12.1	72.4	12.1	72.5	12.1	72.7
15	116.3	18.28	10.2	61.3	11.3	68.1	11.4	68.4	11.3	68.1	11.4	68.4	11.4	68.7
9	116.3	18.28	8.6	51.4	8.6	51.4	8.6	51.7	8.6	51.7	8.6	51.8	8.8	52.6
3	116.3	18.28	7.2	43.4	7.3	43.8	7.3	44.0	7.4	44.2	7.4	44.4	7.6	45.5
Test Number			6-1		6-2		6-3		6-4		6-5		6-6	

Table 7.13 A-2-4 (30%), Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\% = W \times G_s \times \gamma_d / (\gamma_w \times G_s - \gamma_d)$)											
	US lb/ft^3	SI kN/m^3	+12.0 in.		+12 in.		+12 in.		+12 in.		+12 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S
33	116	18.23	8.0	47.9	8.0	47.8	7.9	47.2	8.1	48.3	8.1	48.2	8.6	51.5
27	116	18.23	11.2	66.9	11.2	66.9	11.2	66.7	11.3	67.2	11.3	67.3	11.5	68.4
21	116	18.23	13.8	82.4	13.8	82.4	13.9	82.6	14.0	83.5	14.0	83.4	14.9	88.9
15	116	18.23	16.3	97.1	16.3	97.1	16.2	96.8	16.2	96.5	16.2	96.5	16.4	97.9
9	116	18.23	16.8	99.9	16.8	99.9	16.8	100.0	16.8	99.8	16.8	99.9	17.0	100.0
3	116	18.23	15.8	93.9	15.8	93.9	15.7	93.4	15.5	92.5	15.5	92.6	15.7	93.3
Test Number			7-1		7-2		7-3		7-4		7-5		7-6	

Table 7.14 Miami Oolite A-1, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density (γ_d)		Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\% = W \times G_s \times \gamma_d / (\gamma_w \times G_s - \gamma_d)$)											
	US lb/ft^3	SI kN/m^3	+12 in.		+12 in.		+12 in.		+12 in.		+12 in.		+36 in.	
			W	S	W	S	W	S	W	S	W	S	W	S
33	132.6	20.84	3.5	35.1	3.5	34.7	3.5	34.6	3.5	34.9	3.5	35.0	4.4	43.6
27	132.6	20.84	4.5	44.4	4.5	44.4	4.4	44.1	4.3	43.2	4.3	43.2	4.6	45.5
21	132.6	20.84	6.0	59.8	6.0	59.9	6.0	59.6	5.9	58.9	6.0	59.9	6.6	65.2
15	132.6	20.84	3.7	37.1	3.7	37.1	3.8	37.3	2.8	28.1	3.5	35.0	3.8	37.7
9	132.6	20.84	3.2	32.2	3.2	32.2	3.2	32.2	3.2	32.3	3.2	32.1	3.3	32.4
3	132.6	20.84	3.2	31.7	3.2	31.7	3.2	31.7	3.2	31.7	3.2	31.5	3.2	31.7
Test Number			8-1		8-2		8-3		8-4		8-5		8-6	

Table 7.15 Spring Cemetery, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density	Average Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=WxG_sx\gamma_d$ / $(\gamma_wxG_s - \gamma_d)$)																	
		0 in.		12 in.		24 in.		0 in.		12 in.		24 in.		36 in.		24 in.		12 in.	
	kN/m^3	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S
Base								7.9		8.2		11.8		17.7		13.6		10.4	
33	18.22					7.2	44.9	4.5	28.1	4.8	29.7	10.3	64.3	13.0	81.1	12.9	80.5	5.9	37.1
27	18.22	4.2	26.2	4.9	30.4	10.4	64.9	6.5	40.6	8.8	54.7	12.5	78.2	13.3	83.0	13.5	84.2	9.9	61.7
21	18.56	5.0	33.2	8.1	54.1	13.9	92.4	6.8	45.0	11.7	77.6	11.6	77.3	12.7	84.2	12.6	84.0	12.0	79.6
15	18.56	7.4	49.4	11.6	76.9	12.9	85.5	9.0	59.8	11.5	76.7	11.9	78.9	13.1	86.9	12.5	83.1	13.2	87.7
9	18.56	9.5	63.4	12.4	82.7	13.5	90.0	10.7	71.1	12.7	84.2	12.7	84.2	13.3	88.2	13.2	87.5	13.9	92.3
3	18.56	12.3	81.8	13.2	88.0	14.2	94.6	13.0	86.6	13.4	89.3	13.1	87.1	13.6	90.6	13.6	90.2	13.4	89.2
Test Number		9-1,2,3		9-4,5,6		9-7,8,9		9-10,11,12		9-12,14,15		9-16,17,18		9-19,20,21		9-22,23,24		9-25,26,27	

Table 7.16 Branch, Degree of Saturation under Plate Load Test

Elevation above embankment (in.)	Dry Density	Average Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=WxG_sx\gamma_d$ / $(\gamma_wxG_s - \gamma_d)$)																	
		0 in.		12 in.		24 in.		0 in.		12 in.		24 in.		36 in.		24 in.		12 in.	
	kN/m^3	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S
Base								4.5		4.7		6.2		9.9		8.8		5.1	
33	20.73					3.6	37.2	5.8	59.4	6.2	63.8	8.3	85.3	9.6	98.7	9.3	95.7	8.0	82.0
27	20.73	5.1	52.5	9.8	100.1	12.8	130.8	9.1	93.7	12.2	124.6	12.9	132.0	13.3	135.9	13.1	133.9	12.9	131.7
21	20.17	6.8	61.6	9.1	82.5	9.7	87.5	9.3	83.6	9.7	87.5	9.5	85.8	9.9	89.2	9.8	88.8	9.7	87.3
15	20.17	8.8	79.0	10.8	97.7	10.4	93.5	10.3	93.0	10.9	98.4	10.8	97.4	10.8	97.5	11.3	101.5	11.4	102.8
9	20.17	9.3	83.7	10.9	98.1	10.2	91.9	10.0	90.5	10.7	96.6	10.8	97.0	10.8	97.5	11.2	100.6	11.3	101.7
3	20.17	10.2	92.2	10.3	92.6	10.3	93.2	10.0	90.6	10.3	92.8	10.0	90.6	10.3	93.2	10.4	94.1	9.8	88.8
Test Number		10-1,2,3,4		10-5,6,7,8		10-9,10,11		10-12,13,14		10-15,16,17		10-18,19,20,21		10-22,23,24		10-25,26,27		10-28,29,30	

Table 7.17 Iron Bridge, Degree of Saturation under Plate Load Test

Elevation above Embankment (in.)	Dry Density	Average Water Content (W %) & Degree of Saturation (S %) under Different Water Table of Plate Load Test ($S\%=W x G_s x \gamma_d / (\gamma_w x G_s - \gamma_d)$)																	
		0 in.		12 in.		24 in.		0 in.		12 in.		24 in.		36 in.		24 in.		12 in.	
	kN/m^3	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S	W	S
Base								6.2		6.9		7.9		12.6		9.0		6.6	
33	20.38					8.4	80.0	8.6	81.9	9.3	88.6	10.3	98.5	13.5	129.1	11.3	108.1	10.2	98.0
27	20.38	6.0	56.9	10.7	102.0	12.2	117.0	9.4	89.6	11.0	105.2	11.7	111.4	13.0	124.7	11.9	114.1	11.5	109.8
21	19.36	7.4	56.7	10.7	82.5	10.8	83.3	10.9	84.3	11.1	85.3	11.3	87.1	11.2	86.6	11.5	88.9	11.3	87.2
15	19.36	6.5	50.5	12.5	96.4	12.8	98.4	11.5	88.4	12.4	95.8	12.6	97.1	12.8	98.4	12.7	97.9	12.6	97.4
9	19.36	9.8	75.4	11.1	85.6	11.2	86.6	11.1	85.8	10.9	84.3	11.2	86.3	11.4	88.1	11.4	88.1	11.2	86.2
3	19.36	9.6	74.0	12.6	96.9	11.9	91.7	11.4	87.6	12.1	93.0	12.0	92.9	12.1	93.0	12.2	93.8	11.7	90.2
Test Number		11-1,2,3,4		11-5,6,7		11-8,9,10		11-11,12,13		11-14,15,16		11-17,18,19		11-20,21,22		11-23,24,25		11-26,27,28	

Table 7.18 Summary of Plate Load Test for Levy County A-3 Soil

Test No.	Water Table*	Test Load	5-in. Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10,000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
1-1	-20	20	NO	12.9---5.2	3.8	108	178	25819	0.16307	0.00642	178	25819	0.16307	0.00642
1-2	0	20	NO	15.1---4.5			145	20986	0.20091	0.00791	145	20986	0.20091	0.00791
1-3	12	20	NO	15.8---5.5			132	19082	0.22047	0.00868	132	19082	0.22047	0.00868
1-4	12	20	YES	16.3---6.2			226	32831	0.12827	0.00505	226	32831	0.12827	0.00505
1-5	12	50	YES	16.3---6.2			264	38313	0.27508	0.01083	264	38313	0.27508	0.01083
1-6	36	50	YES	16.7---14.7			196	28479	0.36982	0.01456	196	28479	0.36982	0.01456
1-7	36	20	YES	16.7---14.7			170	24674	0.17069	0.00672	170	24674	0.17069	0.00672

Table 7.19 Summary of Plate Load Test for SR70 A-3 Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
2-1	0	20	NO	17.2---6.7	6.2	108	204	29545	0.14275	0.00562	204	29545	0.14275	0.00562
2-2	12	20	NO	20.6---6.8	5.7	110.5	174	25307	0.16637	0.00655	174	25307	0.16637	0.00655
2-3	12	50	YES	20.4---8.0	10.8	120.6	300	43469	0.24232	0.00954	300	43469	0.24232	0.00954
2-4	36	50	YES	21.6---17.1	10.8	120.6	230	33301	0.31598	0.01244	225	32571	0.32334	0.01273
2-5	36	50	YES	21.5---17.2			220	31842	0.33071	0.01302				
2-6	-24	50	YES	10.7---6.2	3.1	122.5	499	72379	0.14554	0.00573	454	65791	0.16167	0.00637
2-7	-24	50	YES	10.6---6.1	2.7	123.5	408	59204	0.17780	0.00700				
2-8	36	50	YES	17.9---14.5	10.3	122.5	208	30160	0.34900	0.01374	208	30160	0.34900	0.01374

Table 7.20 Summary of Plate Load Test for SR70 A-2-4 Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
3-1	0	20	NO	14.4---8.1	9.6	110.4	183	26534	0.15875	0.00625	183	26534	0.15875	0.00625
3-2	12	20	NO	18.3---8.1	8.2	110.4	154	22276	0.18898	0.00744	154	22276	0.18898	0.00744
3-3	12	50	YES	18.3---9.7---11.4	11	120.6	227	32860	0.32055	0.01262	227	32860	0.32055	0.01262
3-4	36	50	YES	14.6---11.1---33.2			106	15334	0.68707	0.02705	106	15334	0.68707	0.02705
3-5	-24	50	YES	14.6---9.8---14.6	5.1	121.2	233	33798	0.31140	0.01226	308	44703	0.25044	0.00986
3-6	-24	50	YES	14.6---9.9---13.9	3.2	120.9	383	55607	0.18948	0.00746				
3-7	36	50	YES	14.0---10.0---22.7	13	115.1	61	8803	1.20879	0.04759	61	8803	1.20879	0.04759

Table 7.21 Summary of Plate Load Test for A-2-4 (12%) Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
4-1	-24	20	NO	5.09---2.99	3.1	111.3	178	25816	0.16307	0.00642	174	25236	0.16675	0.00657
4-2	-24	20	NO	5.08---2.99	3	112	170	24656	0.17043	0.00671				
4-3	0	20	NO	10.38---4.75	4	112	123	17839	0.23597	0.00929	125	18201	0.23139	0.00911
4-4	0	20	NO	10.38---4.75	4.1	112.1	128	18563	0.22682	0.00893				
4-5	12	20	NO		3.2	111	127	18419	0.22835	0.00899	126	18285	0.23012	0.00906
4-6	12	20	NO	10.77---6.39	4.5	111.9	125	18151	0.23190	0.00913				
4-7	12	50	YES	11.11---7.69	8.5	116.3	254	36825	0.28626	0.01127	241	34996	0.30175	0.01188
4-8	12	50	YES	11.15---7.68	9.1	116.3	229	33168	0.31725	0.01249				
4-9	36	50	YES	11.3---11.05			173	25134	0.41859	0.01648	174	25253	0.41669	0.01641
4-10	36	50	YES	11.26---11.22			175	25372	0.41478	0.01633				

Table 7.22 Summary of Plate Load Test for A-2-4 (20%) Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
5-1	-24	20	NO	9.48---4.21	4.5	115	215	31182	0.13513	0.00532	196	28354	0.15011	0.00591
5-2	-24	20	NO	9.49---4.03	4.2	115.3	176	25526	0.16510	0.00650				
5-3	0	20	NO	9.55---3.42	5.6	115.6	178	25816	0.16307	0.00642	183	26468	0.15926	0.00627
5-4	0	20	NO	9.55---3.33	4.2	115.5	187	27121	0.15545	0.00612				
5-5	12	20	NO	9.58---4.64	3.2	115.2	181	26194	0.16053	0.00632	175	25322	0.16637	0.00655
5-6	12	20	NO	9.58---3.28	4.2	115.3	169	24450	0.17221	0.00678				
5-7	12	50	YES	9.6---3.73	8.1	116.4	222	32181	0.32690	0.01287	250	36204	0.29426	0.01159
5-8	12	50	YES	9.6---3.76	9.6	116.6	277	40226	0.26162	0.01030				
5-9	36	50	YES	9.62---7.98			170	24701	0.42570	0.01676	208	30214	0.36017	0.01418
5-10	36	50	YES	9.61---8.02			246	35727	0.29464	0.01160				

Table 7.23 Summary of Plate Load Test for A-2-4 (24%) Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
6-1	-24	20	NO	7.05---5.37	6.3	112.5	174	25236	0.16713	0.00658	171	24728	0.17069	0.00672
6-2	-24	20	NO	6.99---5.16	6.3	112.5	167	24220	0.17424	0.00686				
6-3	0	20	NO	7.21---5.36	4.2	112.6	176	25526	0.16510	0.00650	187	27106	0.15596	0.00614
6-4	0	20	NO	7.22---5.25	3.8	111.9	198	28686	0.14681	0.00578				
6-5	12	20	NO	7.29---5.98	5.6	111.9	141	20384	0.20650	0.00813	135	19595	0.21514	0.00847
6-6	12	20	NO	7.33---5.66	5	112	130	18805	0.22377	0.00881				
6-7	12	50	YES	7.36---6.05	8.5	116.5	218	31572	0.33325	0.01312	214	31001	0.33947	0.01337
6-8	12	50	YES	7.39---6.06	8.3	116.6	210	30429	0.34569	0.01361				
6-9	36	50	YES	7.55---13.07			176	25526	0.41275	0.01625	169	24492	0.43142	0.01699
6-10	36	50	YES	7.58---13.05			162	23459	0.45009	0.01772				

Table 7.24 Summary of Plate Load Test for A-2-4 (30%) Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
7-1	-24	20	NO	11.96---8.05	8	118.9	209	30312	0.13919	0.00548	209	30312	0.13919	0.00548
7-2	0	20	NO	16.01---8.04	7.4	118.4	196	28426	0.14783	0.00582	186	26904	0.15710	0.00619
7-3	0	20	NO		7.7	118.6	175	25381	0.16637	0.00655				
7-4	12	20	NO	15.75---8.04	7.4	118.4	186	26912	0.15621	0.00615	183	26534	0.15862	0.00625
7-5	12	20	NO	15.76---8.02	7.2	118.1	180	26155	0.16104	0.00634				
~~7-6~~	~~12~~	~~20~~	~~NO~~	~~15.68---7.92~~	~~6.9~~	~~118.6~~	~~133~~	~~19217~~	~~0.21895~~	~~0.00862~~				
7-7	12	50	YES	15.53---8.1	10.8	116.4	238	34547	0.30582	0.01204	260	37654	0.28232	0.01112
7-8	12	50	YES	15.54---8.09	8.1	116.9	281	40761	0.25883	0.01019				
7-9	36	50	YES	15.66---8.64			93	13559	0.77699	0.03059	99	14356	0.73558	0.02896
7-10	36	50	YES	15.64---8.64			104	15153	0.69418	0.02733				

Table 7.25 Summary of Plate Load Test for Miami Oolite A-1 Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range (from bottom to top)	Nuclear Gage M.C.	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
	in.	psi		%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
8-1	-24	50	NO	2.98---4.02	4.1	134	424	61494	0.17170	0.00676	424	61494	0.17170	0.00676
8-2	0	50	NO	3.18---3.94	4.2	134.5	407	59028	0.17831	0.00702	387	56055	0.18821	0.00741
8-3	0	50	NO	3.18---3.82	4	134.3	366	53082	0.19812	0.00780				
~~8-4~~	~~12~~	~~50~~	~~NO~~	~~3.17---3.56~~	~~3.2~~	~~135.5~~	~~29224~~	~~4238434~~	~~0.00279~~	~~0.00011~~				
8-5	12	50	NO	3.18---3.53	3.2	135.5	633	91768	0.11481	0.00452	577	83736	0.12632	0.00497
8-6	12	50	NO	3.18---3.49	3.2	134.9	557	80819	0.13030	0.00513				
8-7	12	50	NO	3.18---3.47	3.5	135	542	78619	0.13386	0.00527				
~~8-8~~	~~12~~	~~50~~	~~YES~~	~~3.18---3.51~~	~~8.3~~	~~116.6~~	~~395~~	~~57285~~	~~0.18364~~	~~0.00723~~				
8-9	12	50	YES	3.16---3.52	7.9	116.7	630	91326	0.11532	0.00454	630	91326	0.11532	0.00454
8-10	36	50	YES	3.18---4.38			295	42818	0.24562	0.00967	246	35673	0.30721	0.01210
8-11	36	50	YES	3.18---4.38			197	28528	0.36881	0.01452				

Table 7.26 Summary of Plate Load Test for Spring Cemetery Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range		Nuclear Gage Misture Content	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
				Subgrage Soil (from bottom to top)	Limerock Base										
	in.	psi		%	%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
9-1	0	20	No	12.2 - 4		5	119.1	115	16743	0.25146	0.00990	114	16558	0.25434	0.01001
9-2	0	20	No	12.2 - 4.3		5.6	120.2	110	16012	0.26289	0.01035				
9-3	0	20	No	12.5 - 4.3		5.4	119.3	117	16920	0.24867	0.00979				
9-4	12	20	No	13.1 - 4.6				104	15046	0.27965	0.01101	109	15796	0.26678	0.01050
9-5	12	20	No	13.3 - 4.9				110	15884	0.26492	0.01043				
9-6	12	20	No	13.3 - 5.1				113	16457	0.25578	0.01007				
9-7	24	20	No	14.1 - 7.4		6.4	123.2	79	11405	0.36906	0.01453	82	11915	0.35382	0.01393
9-8	24	20	No	14.3 - 7.4		6.7	122	86	12544	0.33553	0.01321				
9-9	24	20	No	14.3 - 6.8		7.5	118.3	81	11796	0.35687	0.01405				
9-10	0	50	Yes	13 - 4.6	8.2	6	122.7	244	35423	0.29708	0.01170	247	35795	0.29409	0.01158
9-11	0	50	Yes	13.1 - 4.3	7.7	7.1	120.1	243	35287	0.29823	0.01174				
9-12	0	50	Yes	13 - 4.6	7.7	5.4	120.4	253	36675	0.28695	0.01130				
9-13	12	50	Yes	13.4 - 4.9	8.2	6.9	122	197	28582	0.36819	0.01450	196	28387	0.37088	0.01460
9-14	12	50	Yes	13.5 - 4.7	8.2	6.4	126.3	191	27636	0.38089	0.01500				
9-15	12	50	Yes	13.4 - 4.7	8.3	7.5	121.4	200	28944	0.36357	0.01431				
9-16	24	50	Yes	13.4 - 10.7	12.1	10.6	122.9	149	21666	0.48573	0.01912	144	20896	0.50436	0.01986
9-17	24	50	Yes	13.1 - 10	11.6	11	122.3	146	21218	0.49596	0.01953				
9-18	24	50	Yes	12.8 - 10.2	11.6	11.4	111.2	137	19804	0.53138	0.02092				
9-19	36	50	Yes	13.5 - 12.7	12.8			109	15760	0.66780	0.02629	108	15734	0.66984	0.02637
9-20	36	50	Yes	13.4 - 13.1	17.5			113	16342	0.64396	0.02535				
9-21	36	50	Yes	14 - 13.2	17.9			104	15099	0.69774	0.02747				
9-22	24	50	Yes	13.5 - 12.9	13.6			131	18967	0.55483	0.02184	135	19527	0.53930	0.02123
9-23	24	50	Yes	13.7 - 12.9	13.6			134	19404	0.54234	0.02135				
9-24	24	50	Yes	13.5 - 12.9	13.6			139	20209	0.52073	0.02050				
9-25	12	50	Yes	13.4 - 6	10.3			177	25679	0.40981	0.01613	182	26377	0.39954	0.01573
9-26	12	50	Yes					192	27798	0.37859	0.01491				
9-27	12	50	Yes	13.4 - 5.9	10.5			177	25653	0.41022	0.01615				

Table 7.27 Summary of Plate Load Test for Branch Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range		Nuclear Gage Misture Content	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
				Subgrage Soil (from bottom to top)	Limerock Base										
	in.	psi		%	%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
10-1	0	20	No	10.2 - 5		3.8	119.8	314	45567	0.09240	0.00364	312	45192	0.09336	0.00368
~~10-2~~	~~0~~	~~20~~	~~No~~	~~10.2 - 5.2~~		~~3.5~~	~~124~~	~~439~~	~~63708~~	~~0.06608~~	~~0.00260~~				
10-3	0	20	No	10.3 - 5.2		3.6	123.3	328	47540	0.08854	0.00349				
10-4	0	20	No	10.2 - 5.1				293	42468	0.09913	0.00390				
10-5	12	20	No	10.2 - 9.6				321	46614	0.09031	0.00356	295	42778	0.10014	0.00394
10-6	12	20	No	10.2 - 9.6				321	46502	0.09057	0.00357				
10-7	12	20	No	10.2 - 9.8				243	35217	0.11953	0.00471				
~~10-8~~	~~12~~	~~20~~	~~No~~	~~10.5 - 10.1~~				~~458~~	~~66420~~	~~0.06338~~	~~0.00250~~				
10-9	24	20	No	10.3 - 3.9*		3.5	121.8	250	36243	0.11615	0.00457	205	29777	0.14533	0.00572
10-10	24	20	No	10.5 - 3.3*		4.6	127.2	200	28980	0.14526	0.00572				
10-11	24	20	No	10.2 - 3.7*		5.2	124.6	166	24109	0.17460	0.00687				
10-12	0	50	Yes	10.1 - 5.7	4.7	2.7	121	593	86048	0.12236	0.00482	671	97295	0.10986	0.00433
10-13	0	50	Yes	10.1 - 5.6	4.5	2.2	123	790	114553	0.09187	0.00362				
10-14	0	50	Yes	9.9 - 6.1	4.5	1.8	122.1	629	91285	0.11534	0.00454				
10-15	12	50	Yes	10.3 - 6	4.7	1.8	116.3	568	82400	0.12772	0.00503	607	88097	0.12089	0.00476
10-16	12	50	Yes	10.2 - 6.4	4.8	2	125.7	704	102048	0.10315	0.00406				
10-17	12	50	Yes	10.3 - 6.3	4.7	2.4	125.3	551	79844	0.13181	0.00519				
10-18	24	50	Yes	10.3 - 8	5.9	2.2	120.6	396	57421	0.18330	0.00722	413	59913	0.17590	0.00693
10-19	24	50	Yes	10.5 - 8.6	6.2	2	116.2	433	62812	0.16754	0.00660				
~~10-20~~	~~24~~	~~50~~	~~Yes~~	~~10.1 - 8.3~~	~~6.2~~	~~3.5~~	~~119.9~~	~~223~~	~~32394~~	~~0.32491~~	~~0.01279~~				
10-21	24	50	Yes	9.3 - 8.4	6.7	2.8	120.5	410	59506	0.17687	0.00696				
10-22	36	50	Yes	10.3 - 9.6	9.8			169	24558	0.42866	0.01688	157	22792	0.46934	0.01848
10-23	36	50	Yes	10.2 - 9.7	9.8			131	18976	0.55571	0.02188				
10-24	36	50	Yes	10.5 - 9.6	10.1			171	24843	0.42366	0.01668				
10-25	24	50	Yes	10.5 - 9	8.7			198	28740	0.36618	0.01442	192	27814	0.37968	0.01495
10-26	24	50	Yes	10.5 - 9.3	8.7			176	25562	0.41171	0.01621				
10-27	24	50	Yes	10.3 - 9.7	8.9			201	29141	0.36115	0.01422				
10-28	12	50	Yes	9.6 - 8	5.1			390	56527	0.18618	0.00733	414	60034	0.17975	0.00708
10-29	12	50	Yes	10.3 - 7.9	5.3			346	50200	0.20964	0.00825				
10-30	12	50	Yes	9.7 - 8.2	5.1			506	73375	0.14342	0.00565				

Table 7.28 Summary of Plate Load Test for Iron Bridge Soil

Test No.	Water Table*	Test Load	5-in Limerock Layer	Moisture Content Range		Nuclear Gage Misture Content	Nuclear Gage Density	Average EQ.Modulus (after 10000 Cycles)		Average Deformation (after 10000 Cycles)		Average EQ.Modulus (at Each Condition)		Average Deformation (at Each Condition)	
				Subgrage Soil (from bottom to top)	Limerock Base										
	in.	psi		%	%	%	pcf	MPa	psi	mm.	in.	MPa	psi	mm.	in.
~~11-1~~	~~0~~	~~20~~	~~No~~	~~8.8 - 5.9~~		~~5.2~~	~~122.1~~	~~80~~	~~11637~~	~~0.36195~~	~~0.01425~~	178	25750	0.16843	0.00663
11-2	0	20	No	9.4 - 5.9		5.2	125	155	22538	0.18677	0.00735				
11-3	0	20	No	9.7 - 6		5.4	125.3	223	32371	0.13010	0.00512				
11-4	0	20	No	10.5 - 6				154	22342	0.18842	0.00742				
11-5	12	20	No	12.8 - 10.6				187	27143	0.15523	0.00611	195	28259	0.15154	0.00597
11-6	12	20	No	12.3 - 10.6				168	24394	0.17262	0.00680				
11-7	12	20	No	12.6 - 10.8				229	33239	0.12678	0.00499				
11-8	24	20	No	11.9 - 8.7		8.6	135.4	142	20553	0.20482	0.00806	119	17226	0.24866	0.00979
11-9	24	20	No	11.9 - 7.9		9.4	137.6	105	15246	0.27611	0.01087				
11-10	24	20	No	11.9 - 8.8		10	136.4	109	15881	0.26506	0.01044				
11-11	0	50	Yes	11.9 - 8.7	6.2	4.6	109.5	553	80254	0.13121	0.00517	591	85743	0.12349	0.00486
11-12	0	50	Yes	11.3 - 8.5	6.2	3.3	123.9	655	95002	0.11078	0.00436				
11-13	0	50	Yes	10.9 - 8.5	6.2	3.1	122.4	565	81972	0.12847	0.00506				
11-14	12	50	Yes	12.2 - 9.3	6.9	3.2	123.1	583	84595	0.12442	0.00490	542	78621	0.13455	0.00530
11-15	12	50	Yes	12 - 9.4	6.9	3	118.6	552	80067	0.13144	0.00517				
11-16	12	50	Yes	12 - 9.1	6.9	3.4	121.9	491	71202	0.14780	0.00582				
11-17	24	50	Yes	-		5.3	118.2	370	53716	0.19591	0.00771	335	48601	0.21940	0.00864
11-18	24	50	Yes	12.2 - 10.5	7.8	3.8	121.2	351	50908	0.20675	0.00814				
11-19	24	50	Yes	11.9 - 10.1	8	5.5	116	284	41179	0.25555	0.01006				
11-20	36	50	Yes	11.9 - 13.7	12.5			103	14966	0.70413	0.02772	112	16262	0.66502	0.02618
11-21	36	50	Yes	12.3 - 13.5	13.2			95	13744	0.76653	0.03018				
11-22	36	50	Yes	12 - 13.3	12.1	5.5	116	138	20077	0.52441	0.02065				
11-23	24	50	Yes	12 - 11.4	8.9			238	34487	0.30514	0.01201	220	31851	0.33198	0.01307
11-24	24	50	Yes	12.2 - 11.4	8.8			201	29137	0.36118	0.01422				
11-25	24	50	Yes	12.3 - 11.1	9.3			220	31927	0.32961	0.01298				
11-26	12	50	Yes	11.8 - 10.3	6.6			392	56876	0.18503	0.00728	378	54841	0.19237	0.00757
11-27	12	50	Yes	-				352	51060	0.20610	0.00811				
11-28	12	50	Yes	11.7 - 10.3	6.6			390	56586	0.18598	0.00732				

Table 7.29 Average Plate Load EQ Modulus for Eleven Soils

Plate Load EQ Modulus : 1.38 pa / Resilient Deformation																		
Test Condition	A		B		C		D		E		F		G		H		I	
Water Table (Above Embankment)	+0 in.		Raised up to +12 in.		Raised up to +24 in.		Drawn down to +0 in.		Raised up to +12 in.		Raised up to +24 in.		Raised up to +36 in.		Drawn down to +24 in.		Drawn down to +12 in.	
Base Clearance	3 ft		2 ft		1 ft		3 ft		2 ft		1 ft		0 ft		1 ft		2 ft	
Limerock Base Layer	No		No		No		Yes		Yes		Yes		Yes		Yes		Yes	
Plate Load	20 psi		20 psi		20 psi		50 psi		50 psi		50 psi		50 psi		50 psi		50 psi	
Unit	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa
Levy A-3	20986	145	19082	132	-	-	-	-	38313	264	-	-	28479	196	-	-	-	-
SR 70 A-3	29545	204	25307	174	-	-	-	-	43469	300	-	-	32571	225	-	-	-	-
SR 70 A-2-4	26534	183	22276	154	-	-	-	-	32860	227	-	-	15334	106	-	-	-	-
A-2-4 (12%)	18201	125	18201	125	-	-	-	-	34996	241	-	-	25253	174	-	-	-	-
A-2-4 (20%)	26468	183	25322	175	-	-	-	-	36204	250	-	-	30214	208	-	-	-	-
A-2-4 (24%)	27106	187	19595	135	-	-	-	-	31001	214	-	-	24492	169	-	-	-	-
A-2-4 (30%)	26904	186	26534	183	-	-	-	-	37654	260	-	-	14356	99	-	-	-	-
Oolite A-1	-	-	-	-	-	-	-	-	91326	630	-	-	35673	246	-	-	-	-
Spring Cemetery A-2-4	16558	114	15796	109	11915	82	35795	247	28387	196	20896	144	15734	108	19527	135	26377	182
Branch A-2-4	45192	312	42778	295	29777	205	97295	671	88097	607	59913	413	22792	157	27814	192	60034	414
Iron Bridge A-2-6	25750	178	28259	195	17226	119	85743	591	78621	542	48601	335	16262	112	31851	220	54841	378

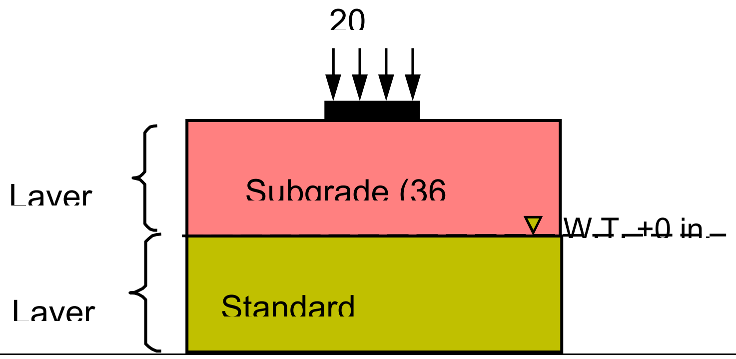
Table 7.30 Plate Load EQ Modulus Reduction Rate for Eleven Soils

Plate Load EQ Modulus Reduction Rate (%)										
Plate Load	20 psi	20 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Limerock Base layer	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Test Condition	from A to B	from A to C	from D to E	from D to F	from D to G	from E to G	from G to H	from G to I	from F to H	from E to I
Change of Base Clearance	from 3 ft to 2 ft	from 3 ft to 1 ft	from 3 ft to 2 ft	from 3 ft to 1 ft	from 3 ft to 0 ft	from 2 ft to 0 ft	from 0 to 1 ft (Draw down)	from 0 to 2 ft (Draw down)	from 1 ft (raise up) to 1 ft (draw down)	from 2 ft (raise up) to 2 ft (draw down)
Levy A-3	9.1	-	-	-	-	25.7	-	-	-	-
SR 70 A-3	14.3	-	-	-	-	25.1	-	-	-	-
SR 70 A-2-4	16.0	-	-	-	-	53.3	-	-	-	-
A-2-4 (12%)	0.0	-	-	-	-	27.8	-	-	-	-
A-2-4 (20%)	4.3	-	-	-	-	16.5	-	-	-	-
A-2-4 (24%)	27.7	-	-	-	-	21.0	-	-	-	-
A-2-4 (30%)	1.4	-	-	-	-	61.9	-	-	-	-
Oolite A-1	-	-	-	-	-	60.9	-	-	-	-
Spring Cemetery A-2-4	4.6	28.0	20.7	41.6	56.0	44.6	-24.1	-67.6	6.6	7.1
Branch A-2-4	5.4	34.0	9.5	38.4	76.6	74.1	-22.0	-163.4	53.6	31.9
Iron Bridge A-2-6	-9.7	33.1	8.3	43.3	81.0	79.3	-95.9	-237.2	34.5	30.2

Table 7.31 Plate Load EQ Modulus Increase Rate for Eleven Soils Due to Limerock Base Layer Effect

Plate Load EQ Modulus Reduction Rate due to Limerock Effect (%)			
Test Condition	from A to D	from B to E	from C to F
Base Clearance	3 ft	2 ft	1 ft
Levy A-3	-	101	-
SR 70 A-3	-	72	-
SR 70 A-2-4	-	48	-
A-2-4 (12%)	-	92	-
A-2-4 (20%)	-	43	-
A-2-4 (24%)	-	58	-
A-2-4 (30%)	-	42	-
Oolite A-1	-	-	-
Spring Cemetery A-2-4	116	80	75
Branch A-2-4	115	106	101
Iron Bridge A-2-6	233	178	182

Table 7.32 Subgrade Layer Modulus under 20-psi Cyclic Plate Stress w/o Limerock, W.T. at 0 in.

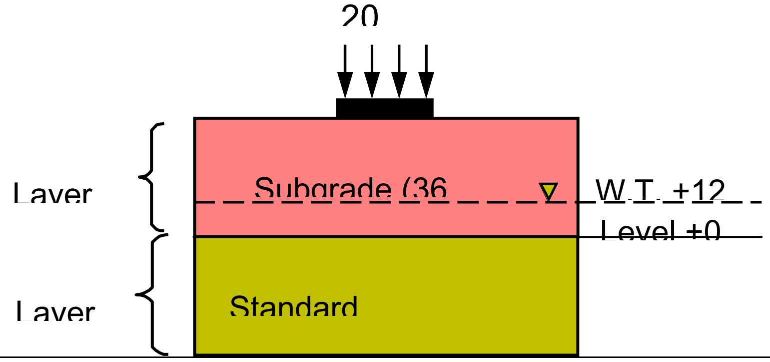
<table>
<tr><td>Schematic View</td><td colspan="5"> </td></tr>
<tr><td rowspan="2">Layer Layout (top to bottom)</td><td colspan="5">Layer 1 in program (modulus $E_{2@+0}$):
Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="5">Layer 2 in program (modulus: $E_{3@+0}$):
Embankment Layer* (36 in.)</td></tr>
<tr><td rowspan="2">Subgrade Soil</td><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{3@+0}$</td><td colspan="2">$E_{2@+0}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Levy A-3</td><td>0.00791</td><td>11207</td><td>77</td><td>34000</td><td>234</td></tr>
<tr><td>SR70 A-3</td><td>0.00562</td><td>11207</td><td>77</td><td>52400</td><td>361</td></tr>
<tr><td>SR70 A-2-4</td><td>0.00625</td><td>11207</td><td>77</td><td>45700</td><td>315</td></tr>
<tr><td>A-2-4 (12%)</td><td>0.00911</td><td>11207</td><td>77</td><td>28600</td><td>197</td></tr>
<tr><td>A-2-4 (20%)</td><td>0.00627</td><td>11207</td><td>77</td><td>45500</td><td>314</td></tr>
<tr><td>A-2-4 (24%)</td><td>0.00614</td><td>11207</td><td>77</td><td>46800</td><td>323</td></tr>
<tr><td>A-2-4 (30%)</td><td>0.00619</td><td>11207</td><td>77</td><td>46300</td><td>319</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01001</td><td>11207</td><td>77</td><td>25520</td><td>176</td></tr>
<tr><td>Branch A-2-4</td><td>0.00368</td><td>11207</td><td>77</td><td>93500</td><td>644</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00663</td><td>11207</td><td>77</td><td>42400</td><td>292</td></tr>
</table>

1. $E_{x@y}$ is designated as the x layer(x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

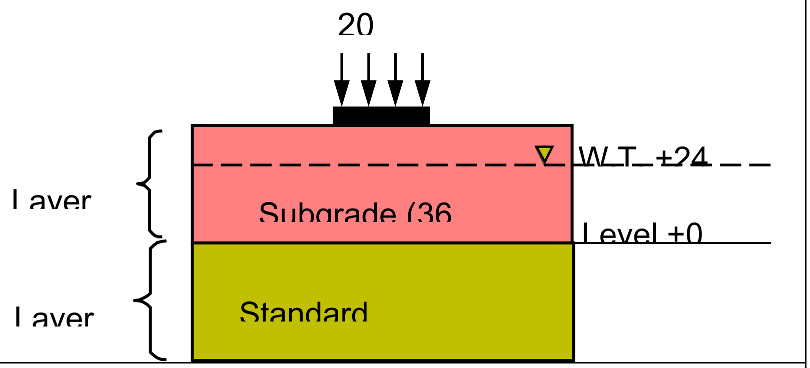
3. The E values in bold face were back calculated by KENLAYER.

Table 7.33 Subgrade Layer Modulus under 20-psi Cyclic Plate Stress w/o Limerock, W.T. at 12 in.

Schematic View					
Layer Layout (top to bottom)	Layer 1 in program (modulus: $E_{2@+12}$): Subgrade Layer (36 in.)				
	Layer 2 in program (modulus: $E_{3@+12}$): Embankment Layer (36 in.)				
Subgrade Soil	Resilient deformation Δ_R (in.)	$E_{3@+12}$		$E_{2@+12}$ (KENLAYER)	
		psi	MPa	psi	MPa
Levy A-3	0.00868	11207	77	**30330**	**209**
SR70 A-3	0.00655	11207	77	**43050**	**297**
SR70 A-2-4	0.00744	11207	77	**36650**	**253**
A-2-4(12%)	0.00906	11207	77	**28800**	**199**
A-2-4 (20%)	0.00655	11207	77	**43050**	**297**
A-2-4 (24%)	0.00847	11207	77	**31250**	**216**
A-2-4 (30%)	0.00625	11207	77	**45500**	**312**
Spring Cemetery A-2-4	0.01050	11207	77	**24120**	**166**
Branch A-2-4	0.00394	11207	77	**84700**	**584**
Iron Bridge A-2-6	0.00597	11207	77	**48500**	**334**

1. $E_{x@y}$ is designated as the x layer(x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.
2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.
3. The E values in bold face were back calculated by KENLAYER.

Table 7.34 Subgrade Layer Modulus under 20-psi Cyclic Plate Stress w/o Limerock, W.T. at 24 in.

<table>
<tr><td>Schematic View</td><td colspan="5">
</td></tr>
<tr><td rowspan="2">Layer Layout (top to bottom)</td><td colspan="5">Layer 1 in program (modulus: $E_{2@+24}$):
Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="5">Layer 2 in program (modulus: $E_{3@+24}$):
Embankment Layer (36 in.)</td></tr>
<tr><td rowspan="2">Subgrade Soil</td><td rowspan="2">Resilient deformation Δ_R (in.)</td><td colspan="2">$E_{3@+24}$</td><td colspan="2">$E_{2@+24}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01393</td><td>11207</td><td>77</td><td>**17340**</td><td>**120**</td></tr>
<tr><td>Branch A-2-4</td><td>0.00572</td><td>11207</td><td>77</td><td>**51300**</td><td>**354**</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00979</td><td>11207</td><td>77</td><td>**26230**</td><td>**181**</td></tr>
</table>

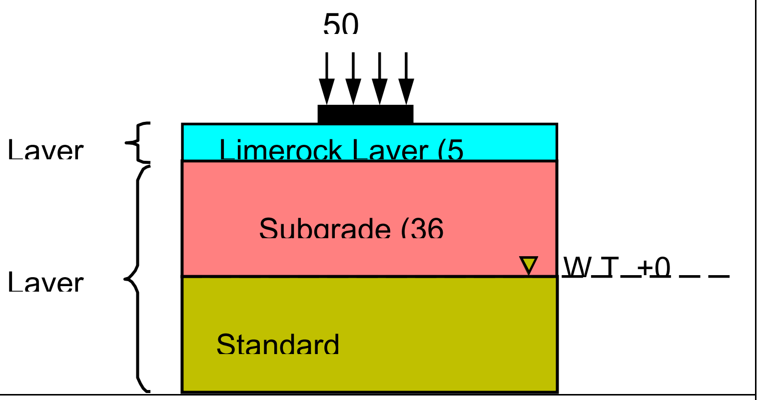
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. The E values in bold face were back calculated by KENLAYER.

4. No test data were available for the other soils.

Table 7.35 Limerock Layer Modulus under 50-psi Cyclic Plate Stress w/ Limerock, W.T. at 0.0 in.

<table>
<tr><td>Schematic View</td><td colspan="5">
</td></tr>
<tr><td rowspan="2">Layer Layout (top to bottom)</td><td colspan="5">Layer 1 in program (modulus: $E_{1@+0}$):
Limerock Layer (5 in.)</td></tr>
<tr><td colspan="5">Layer 2 in program (modulus: $E_{2-3@+0}$):
Subgrade + Embankment Layers (72 in.)</td></tr>
<tr><td rowspan="2">Subgrade Soil</td><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{2-3@+0}$</td><td colspan="2">$E_{1@+0}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01158</td><td>16558</td><td>114</td><td>548000</td><td>3778</td></tr>
<tr><td>Branch A-2-4</td><td>0.00433</td><td>45192</td><td>344</td><td>1400000</td><td>9653</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00486</td><td>25750</td><td>178</td><td>3440000</td><td>23719</td></tr>
</table>

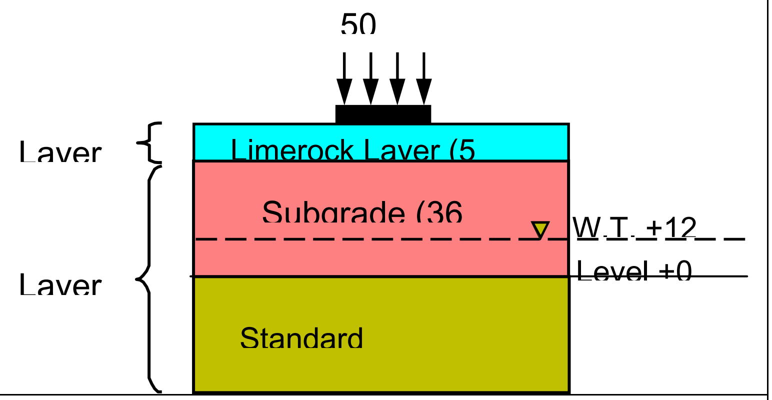
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. $E_{2-3@+0}$ is the EQ modulus for the layer 2-3 under 20 psi plate stress without limerock with the W.T. at the bottom of test material.

3. The E values in bold face were back calculated by KENLAYER.

4. No test data were available for the other soils.

Table 7.36 Limerock Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +12 in.

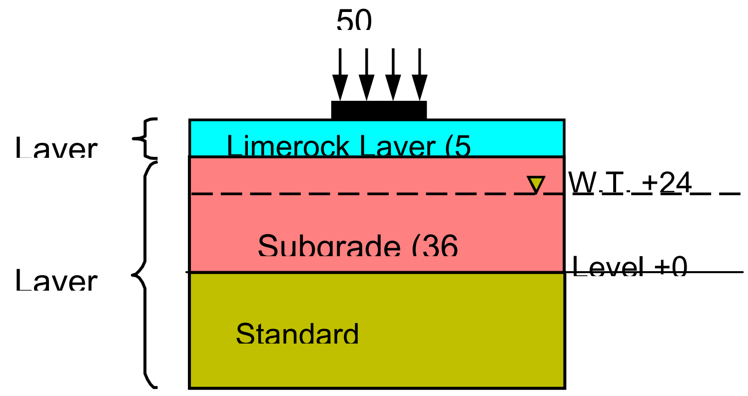
<table>
<tr><td>Schematic View</td><td colspan="5">
</td></tr>
<tr><td rowspan="2">Layer Layout (top to bottom)</td><td colspan="5">Layer 1 in program (modulus: $E_{1@+12}$):
Limerock Layer (5 in.)</td></tr>
<tr><td colspan="5">Layer 2 in program (modulus: $E_{2\text{-}3@+12}$):
Subgrade + Embankment Layers (72 in.)</td></tr>
<tr><td rowspan="2">Subgrade Soil</td><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{2\text{-}3@12}$</td><td colspan="2">$E_{1@+12}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Levy A-3</td><td>0.01083</td><td>19082</td><td>132</td><td>490000</td><td>3379</td></tr>
<tr><td>SR70 A-3</td><td>0.00954</td><td>25307</td><td>174</td><td>365000</td><td>2517</td></tr>
<tr><td>SR70 A-2-4</td><td>0.01262</td><td>22276</td><td>154</td><td>178000</td><td>1227</td></tr>
<tr><td>A-2-4(12%)</td><td>0.01188</td><td>18151</td><td>125</td><td>402500</td><td>2775</td></tr>
<tr><td>A-2-4(20%)</td><td>0.01159</td><td>25322</td><td>175</td><td>171500</td><td>1183</td></tr>
<tr><td>A-2-4(24%)</td><td>0.01337</td><td>19595</td><td>135</td><td>206500</td><td>1424</td></tr>
<tr><td>A-2-4(30%)</td><td>0.01112</td><td>26534</td><td>183</td><td>234000</td><td>1613</td></tr>
<tr><td>Oolite A-1</td><td>0.00454</td><td>70332</td><td>485</td><td>354000</td><td>2440</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01460</td><td>15796</td><td>109</td><td>277500</td><td>1913</td></tr>
<tr><td>Branch A-2-4</td><td>0.00476</td><td>42778</td><td>295</td><td>1155000</td><td>7963</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00530</td><td>28259</td><td>195</td><td>2125000</td><td>14652</td></tr>
</table>

1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. $E_{2\text{-}3@+0}$ is the EQ modulus for the layer 2-3 under 20 psi plate stress without limerock with the W.T. at the bottom of test material.

3. The E values in bold face were back calculated by KENLAYER.

Table 7.37 Limerock Layer Modulus under 50-psi Cyclic Plate Stress w/ Limerock, W.T. at +24 in.

<table>
<tr><td>Schematic View</td><td colspan="5">
</td></tr>
<tr><td rowspan="2">Layer Layout (top to bottom)</td><td colspan="5">Layer 1 in program (modulus: $E_{1@+24}$):
Limerock Layer (5 in.)</td></tr>
<tr><td colspan="5">Layer 2 in program (modulus: $E_{2\text{-}3@+24}$):
Subgrade + Embankment Layers (72 in.)</td></tr>
<tr><td rowspan="2">Subgrade Soil</td><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{2\text{-}3@+24}$</td><td colspan="2">$E_{1@+24}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01986</td><td>11915</td><td>82</td><td>184800</td><td>1274</td></tr>
<tr><td>Branch A-2-4</td><td>0.00693</td><td>29777</td><td>205</td><td>770000</td><td>5309</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00864</td><td>17226</td><td>119</td><td>1322000</td><td>9115</td></tr>
</table>

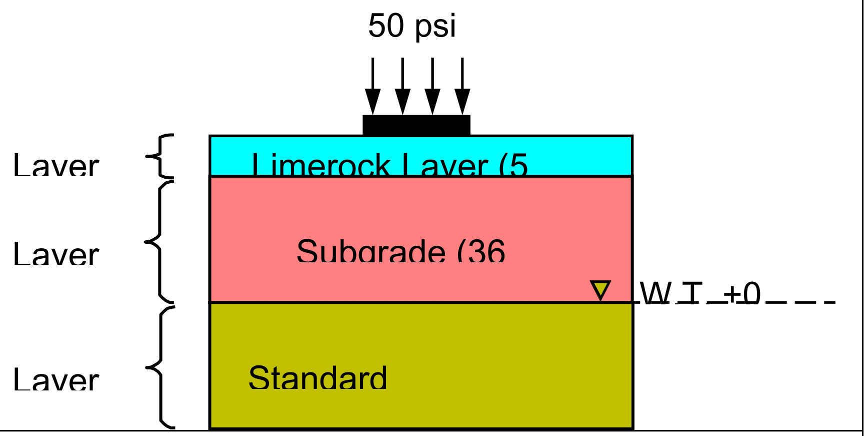
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. $E_{2\text{-}3@+0}$ is the EQ modulus for the layer 2-3 under 20 psi plate stress without limerock with the W.T. at the bottom of test material.

3. The E values in bold face were back calculated by KENLAYER.

4. No test data were available for the other soils.

Table 7.38 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at 0.0 in.

<table>
<tr><td>Schematic View</td><td colspan="6">
</td></tr>
<tr><td rowspan="5">Subgrade Soil</td><td colspan="6">Layer 1 in program (modulus: $E_{1@+0}$):
Limerock Layer (5 in.)</td></tr>
<tr><td colspan="6">Layer 2 in program (modulus: $E_{2@+0}$):
Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="6">Layer 3 in program (modulus: $E_{3@+0}$):
Embankment Layer (36 in.)</td></tr>
<tr><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{3@+0}$</td><td colspan="2">$E_{1@+0}$</td><td colspan="2">$E_{2@+0}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01158</td><td>11207</td><td>77</td><td>548000</td><td>3778</td><td>18800</td><td>130</td></tr>
<tr><td>Branch A-2-4</td><td>0.00433</td><td>11207</td><td>77</td><td>1400000</td><td>9653</td><td>108500</td><td>748</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00486</td><td>11207</td><td>77</td><td>3440000</td><td>23719</td><td>54300</td><td>374</td></tr>
</table>

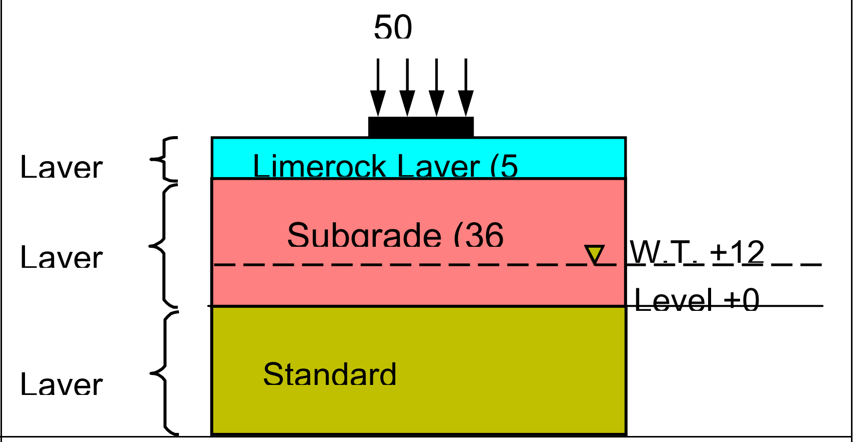
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. $E_{1@+0}$ is the calculated limerock layer modulus from Table 7.35.

4. The E values in bold face were back calculated by KENLAYER.

Table 7.39 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +12 in.

Schematic View							
Subgrade Soil	Layer 1 in program (modulus: $E_{1@+12}$): Limerock Layer (5 in.)						
	Layer 2 in program (modulus: $E_{2@+12}$): Subgrade Layer (36 in.)						
	Layer 3 in program (modulus: $E_{3@+12}$): Embankment Layer (36 in.)						
	Resilient Deformation Δ_R (in.)	$E_{3@+12}$		$E_{1@+12}$		$E_{2@+12}$ (KENLAYER)	
		psi	MPa	psi	MPa	psi	MPa
Levy A-3	0.01083	11207	77	490000	3379	**23820**	**164**
SR70 A-3	0.00954	11207	77	365000	2517	**35300**	**243**
SR70 A-2-4	0.01262	11207	77	178000	1227	**29500**	**203**
A-2-4(12%)	0.01188	11207	77	402500	2775	**21700**	**150**
A-2-4(20%)	0.01159	11207	77	171500	1183	**35450**	**244**
A-2-4(24%)	0.01337	11207	77	206500	1424	**23380**	**161**
A-2-4(30%)	0.01112	11207	77	234000	1613	**31450**	**217**
Oolite A-1	0.00454	11207	77	354000	2440	**194000**	**1337**
Spring Cemetery A-2-4	0.01460	11207	77	277500	1913	**17300**	**119**
Branch A-2-4	0.00476	11207	77	1155000	7963	**93500**	**645**
Iron Bridge A-2-6	0.00530	11207	77	2125000	14652	**57200**	**394**

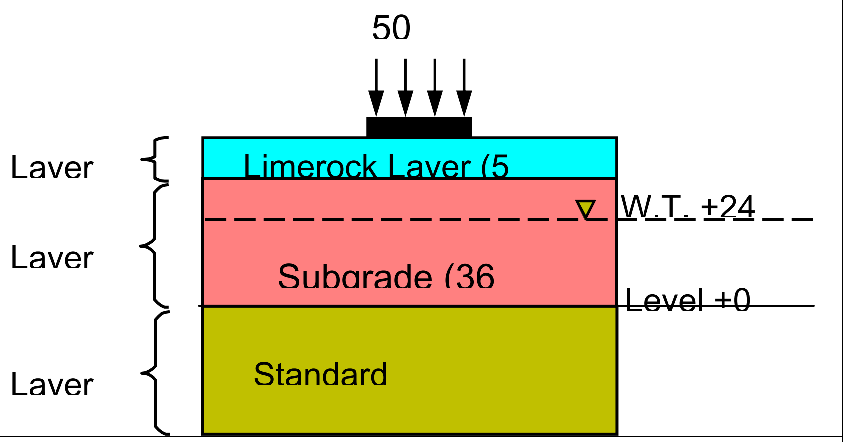
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. $E_{1@+0}$ is the calculated limerock layer modulus from Table 7.36.

4. The E values in bold face were back calculated by KENLAYER.

Table 7.40 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +24 in.

<table>
<tr><td>Schematic View</td><td colspan="7">
</td></tr>
<tr><td rowspan="5">Subgrade Soil</td><td colspan="7">Layer 1 in program (modulus: $E_{1@+24}$):
Limerock Layer (5 in.)</td></tr>
<tr><td colspan="7">Layer 2 in program (modulus: $E_{2@+24}$):
Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="7">Layer 3 in program (modulus: $E_{3@+24}$):
Embankment Layer (36 in.)</td></tr>
<tr><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{3@+24}$</td><td colspan="2">$E_{1@+24}$</td><td colspan="2">$E_{2@+24}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01986</td><td>11207</td><td>77</td><td>184800</td><td>1274</td><td>11800</td><td>81</td></tr>
<tr><td>Branch A-2-4</td><td>0.00693</td><td>11207</td><td>77</td><td>770000</td><td>5309</td><td>44960</td><td>310</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00864</td><td>11207</td><td>77</td><td>1322000</td><td>9115</td><td>22350</td><td>154</td></tr>
</table>

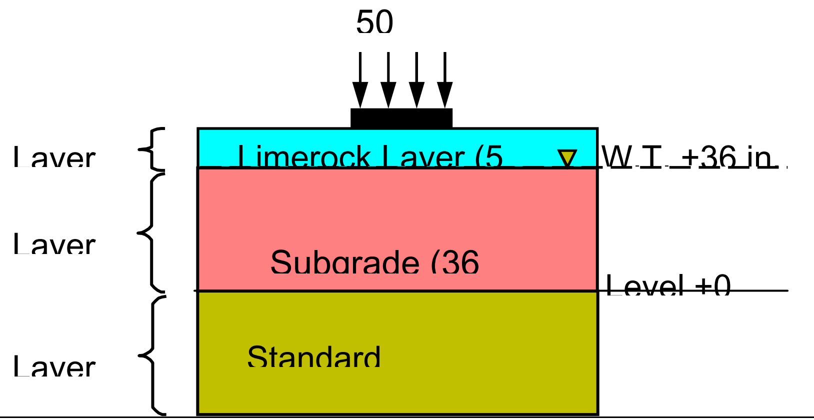
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. $E_{1@+0}$ is the calculated limerock layer modulus from Table 7.37.

4. The E values in bold face were back calculated by KENLAYER.

Table 7.41 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +36 in.

<table>
<tr><td>Schematic View</td><td colspan="7">
</td></tr>
<tr><td rowspan="5">Subgrade Soil</td><td colspan="7">Layer 1 in program (modulus: $E_{1@+36}$): Limerock Layer (5 in.)</td></tr>
<tr><td colspan="7">Layer 2 in program (modulus: $E_{2@+36}$): Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="7">Layer 3 in program (modulus: $E_{3@+36}$): Embankment Layer (36 in.)</td></tr>
<tr><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{3@+36}$</td><td colspan="2">$E_{1@+36}$</td><td colspan="2">$E_{2@+36}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Levy A-3</td><td>0.01456</td><td>11207</td><td>77</td><td>364070</td><td>2510</td><td>15160</td><td>105</td></tr>
<tr><td>SR70 A-3</td><td>0.01273</td><td>11207</td><td>77</td><td>273385</td><td>1885</td><td>22700</td><td>157</td></tr>
<tr><td>SR70 A-2-4</td><td>0.02705</td><td>11207</td><td>77</td><td>83126</td><td>573</td><td>10490</td><td>72</td></tr>
<tr><td>A-2-4(12%)</td><td>0.01641</td><td>11207</td><td>77</td><td>290202</td><td>2000</td><td>13480</td><td>93</td></tr>
<tr><td>A-2-4(20%)</td><td>0.01418</td><td>11207</td><td>77</td><td>142860</td><td>985</td><td>26160</td><td>180</td></tr>
<tr><td>A-2-4(24%)</td><td>0.01699</td><td>11207</td><td>77</td><td>159418</td><td>1100</td><td>16800</td><td>116</td></tr>
<tr><td>A-2-4(30%)</td><td>0.02896</td><td>11207</td><td>77</td><td>89154</td><td>615</td><td>8995</td><td>62</td></tr>
<tr><td>Oolite A-1</td><td>0.01210</td><td>11207</td><td>77</td><td>138000</td><td>952</td><td>36800</td><td>254</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.02637</td><td>11207</td><td>77</td><td>153735</td><td>1081</td><td>7725</td><td>53</td></tr>
<tr><td>Branch A-2-4</td><td>0.01848</td><td>11207</td><td>77</td><td>299145</td><td>2063</td><td>10520</td><td>73</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.02618</td><td>11207</td><td>77</td><td>439875</td><td>3033</td><td>3932</td><td>27</td></tr>
</table>

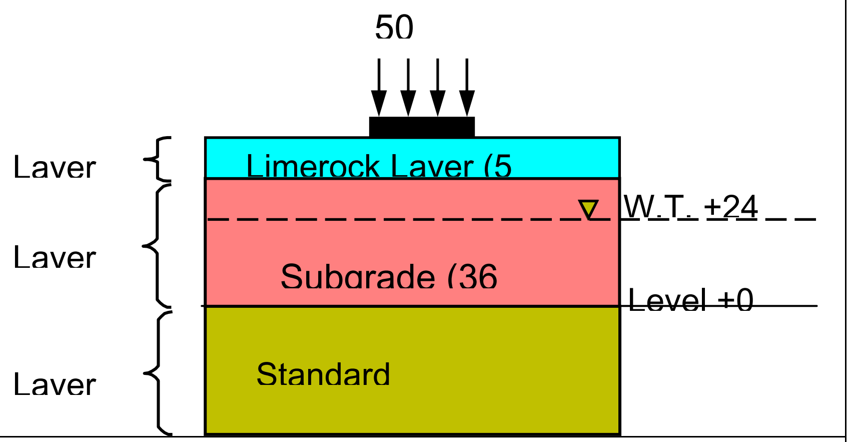
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. $E_{1@+36}$ = ($E_{1@+12}$ in Table 7.36) * (1 - reduction rate from test condition E to G in Table 7.30)

4. The E values in bold face were back calculated by KENLAYER.

Table 7.42 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +24 in. (Draw down)

Schematic View							
Subgrade Soil	Layer 1 in program (modulus: $E_{1@+24}$): Limerock Layer (5 in.)						
	Layer 2 in program (modulus: $E_{2@+24}$): Subgrade Layer (36 in.)						
	Layer 3 in program (modulus: $E_{3@+24}$): Embankment Layer (36 in.)						
	Resilient Deformation Δ_R (in.)	$E_{3@+24}$		$E_{1@+24}$		$E_{2@+24}$ (KENLAYER)	
		psi	MPa	psi	MPa	psi	MPa
Spring Cemetery A-2-4	0.02123	11207	77	181100	1249	**10520**	**73**
Branch A-2-4	0.01495	11207	77	313400	2161	**15550**	**107**
Iron Bridge A-2-6	0.01307	11207	77	1091000	7522	**9360**	**65**

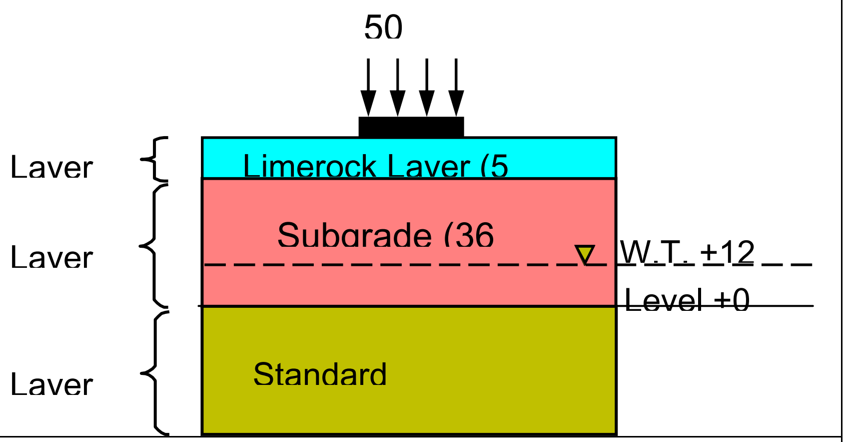
1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. $E_{1@+24}$ was obtained using linear interpolation of the water content – resilient modulus

4. The E values in bold face were back calculated by KENLAYER.

Table 7.43 Subgrade Layer Modulus under 50 psi Cyclic Plate Stress w/ Limerock, W.T. at +12 in. (Draw down)

<table>
<tr><td>Schematic View</td><td colspan="7"> </td></tr>
<tr><td rowspan="5">Subgrade Soil</td><td colspan="7">Layer 1 in program (modulus: $E_{1@+12}$): Limerock Layer (5 in.)</td></tr>
<tr><td colspan="7">Layer 2 in program (modulus: $E_{2@+12}$): Subgrade Layer (36 in.)</td></tr>
<tr><td colspan="7">Layer 3 in program (modulus: $E_{3@+12}$): Embankment Layer (36 in.)</td></tr>
<tr><td rowspan="2">Resilient Deformation Δ_R (in.)</td><td colspan="2">$E_{3@+12}$</td><td colspan="2">$E_{1@+12}$</td><td colspan="2">$E_{2@+12}$ (KENLAYER)</td></tr>
<tr><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td><td>psi</td><td>MPa</td></tr>
<tr><td>Spring Cemetery A-2-4</td><td>0.01573</td><td>11207</td><td>77</td><td>254700</td><td>1756</td><td>**15600**</td><td>**108**</td></tr>
<tr><td>Branch A-2-4</td><td>0.00708</td><td>11207</td><td>77</td><td>856500</td><td>5906</td><td>**43500**</td><td>**300**</td></tr>
<tr><td>Iron Bridge A-2-6</td><td>0.00757</td><td>11207</td><td>77</td><td>2591000</td><td>17865</td><td>**19450**</td><td>**134**</td></tr>
</table>

1. $E_{x@y}$ is designated as the x layer (x=1 limerock; x=2 subgrade; x=3 embankment; x=2-3 subgrade + embankment) modulus when the water table level is at y in.

2. The soaked embankment layer modulus was assumed as 11207 psi based on other research tests.

3. The E values in bold face were back calculated by KENLAYER.

Table 7.44 Subgrade Layer Modulus Computed from KENLAYER Program for Eleven Soils

Subgrade Layer Modulus from KENLAYER Program																		
Test Condition	A		B		C		D		E		F		G		H		I	
Water Table (Above Embankment)	+0 in.		Raised up to +12 in.		Raised up to +24 in.		Drawn down to +0 in.		Raised up to +12 in.		Raised up to +24 in.		Raised up to +36 in.		Drawn down to +24 in.		Drawn down to +12 in.	
Base Clearance	3 ft		2 ft		1 ft		3 ft		2 ft		1 ft		0 ft		1 ft		2 ft	
Limerock Base Layer	No		No		No		Yes		Yes		Yes		Yes		Yes		Yes	
Plate Load	20 psi		20 psi		20 psi		50 psi		50 psi		50 psi		50 psi		50 psi		50 psi	
Unit	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa
Levy A-3	33938	234	30312	209	-	-	-	-	23785	164	-	-	15228	105	-	-	-	-
SR 70 A-3	52357	361	43075	297	-	-	-	-	35243	243	-	-	22770	157	-	-	-	-
SR 70 A-2-4	45685	315	36693	253	-	-	-	-	29442	203	-	-	10442	72	-	-	-	-
A-2-4 (12%)	28571	197	28861	199	-	-	-	-	21755	150	-	-	13488	93	-	-	-	-
A-2-4 (20%)	45540	314	43075	297	-	-	-	-	35388	244	-	-	26106	180	-	-	-	-
A-2-4 (24%)	46846	323	31327	216	-	-	-	-	23350	161	-	-	16824	116	-	-	-	-
A-2-4 (30%)	46265	319	45250	312	-	-	-	-	31472	217	-	-	8992	62	-	-	-	-
Oolite A-1	-	-	-	-	-	-	-	-	193909	1337	-	-	36838	254	-	-	-	-
Spring Cemetery A-2-4	25526	176	24075	166	17404	120	18854	130	17259	119	11748	81	7687	53	10587	73	15664	108
Branch A-2-4	93500	644	84700	584	51342	354	108500	748	93500	645	44960	310	10520	73	15500	107	43500	300
Iron Bridge A-2-6	42350	292	48441	334	26251	181	54242	374	57143	394	22335	154	3916	27	9427	65	19434	134

Table 7.45 Subgrade Layer Modulus Reduction Rate for Eleven Soils

Subgrade Layer Modulus Reduction Rate (%)										
Plate Load	20 psi	20 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi	50 psi
Limerock Base layer	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Test Condition	from A to B	from A to C	from D to E	from D to F	from D to G	from E to G	from G to H	from G to I	from F to H	from E to I
Change of Base Clearance	from 3 ft to 2 ft	from 3 ft to 1 ft	from 3 ft to 2 ft	from 3 ft to 1 ft	from 3 ft to 0 ft	from 2 ft to 0 ft	from 0 to 1 ft (Draw down)	from 0 to 2 ft (Draw down)	from 1 ft (raise up) to 1 ft (draw down)	from 2 ft (raise up) to 2 ft (draw down)
Levy A-3	10.7	-	-	-	-	36.0	-	-	-	-
SR 70 A-3	17.7	-	-	-	-	35.4	-	-	-	-
SR 70 A-2-4	19.7	-	-	-	-	64.5	-	-	-	-
A-2-4 (12%)	-1.0	-	-	-	-	38.0	-	-	-	-
A-2-4 (20%)	5.4	-	-	-	-	26.2	-	-	-	-
A-2-4 (24%)	33.1	-	-	-	-	28.0	-	-	-	-
A-2-4 (30%)	2.2	-	-	-	-	71.4	-	-	-	-
Oolite A-1	-	-	-	-	-	81.0	-	-	-	-
Spring Cemetery A-2-4	5.7	31.8	8.5	37.7	59.2	55.5	-37.7	-103.8	9.9	9.2
Branch A-2-4	9.4	45.1	13.8	58.6	90.3	88.7	-47.3	-313.5	65.5	53.5
Iron Bridge A-2-6	-14.4	38.0	-5.3	58.8	92.8	93.1	-140.7	-396.3	57.8	66.0

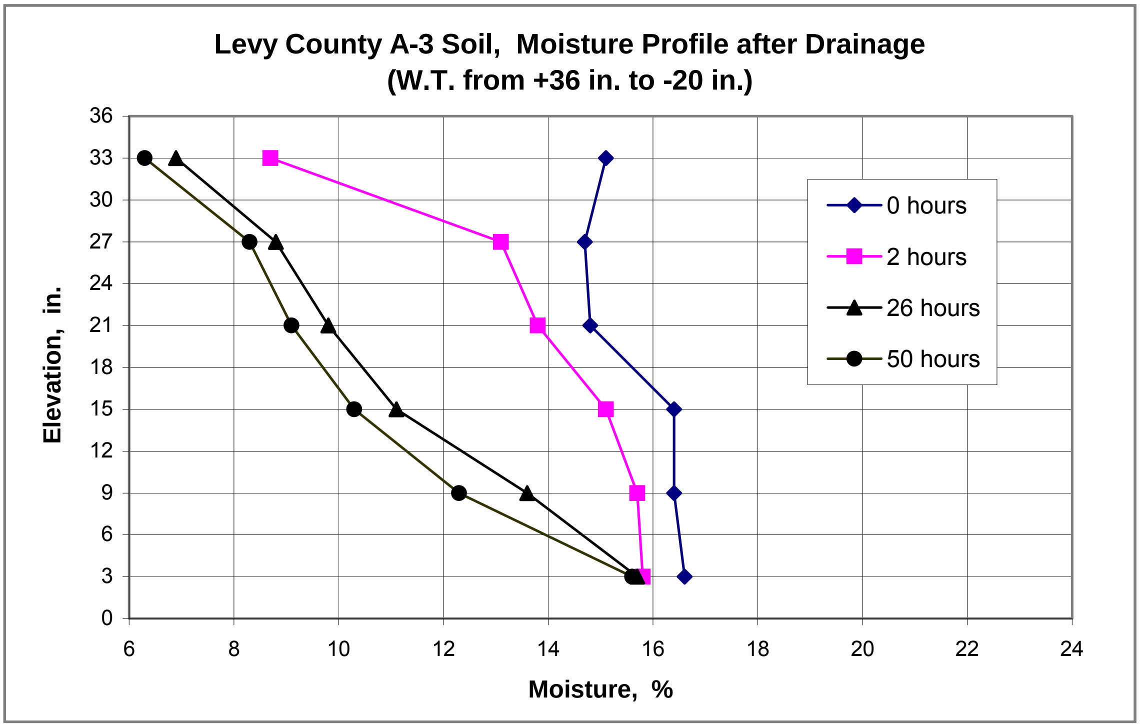


Figure 7.1(A) Levy County A-3 Soil Moisture Profile after Drainage (short-term) (Water Table from +36 in. to -20 in.)

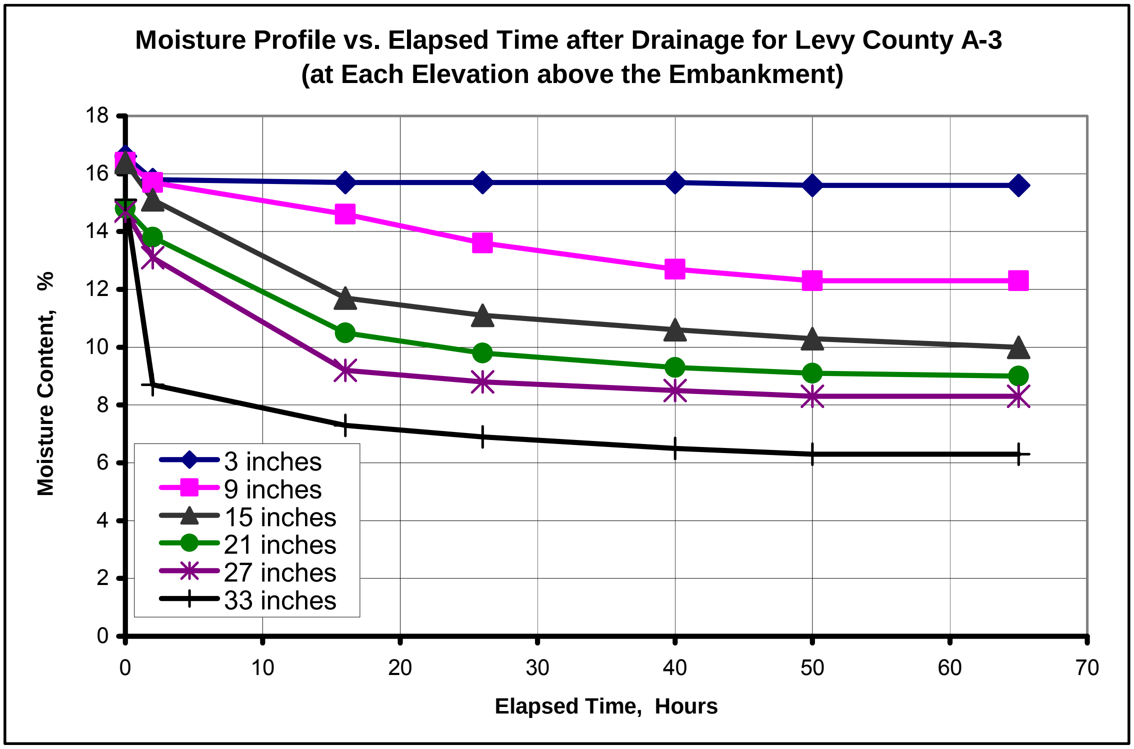


Figure 7.1(B) Moisture Profile vs. Elapsed Time after Drainage for Levy County A-3 Subgrade

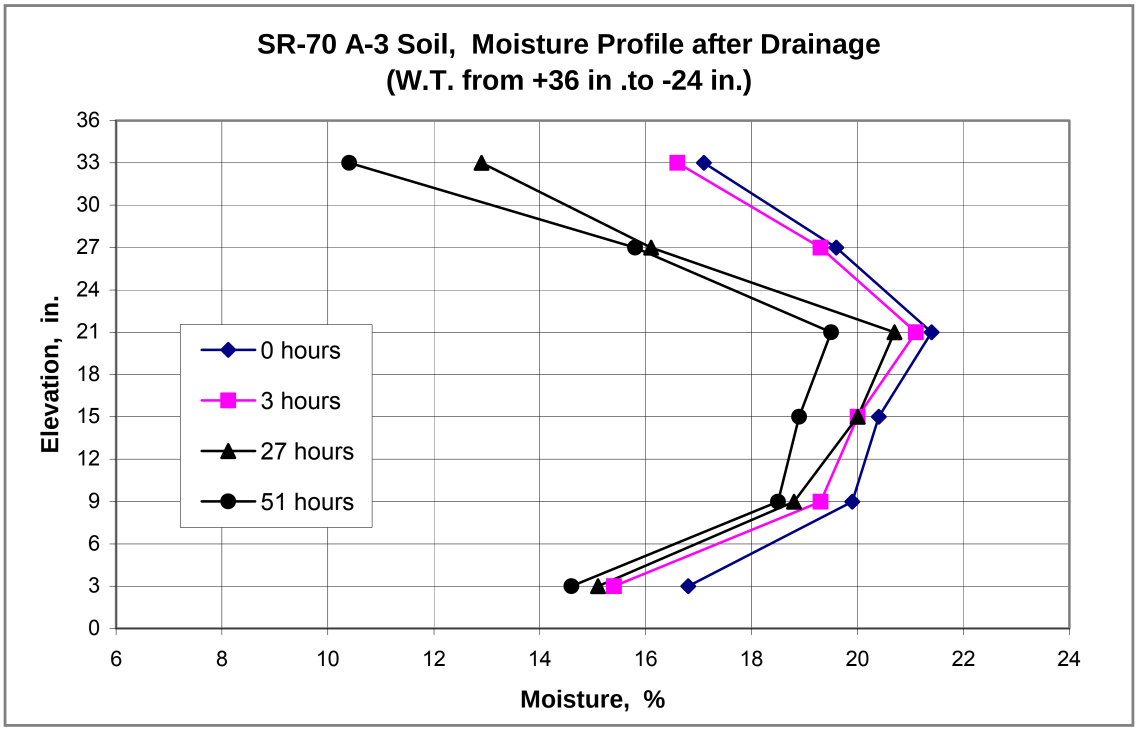


Figure 7.2(A) SR70 A-3 Soil Moisture Profile after Drainage (short-term) (Water Table from +36 in. to -24 in.)

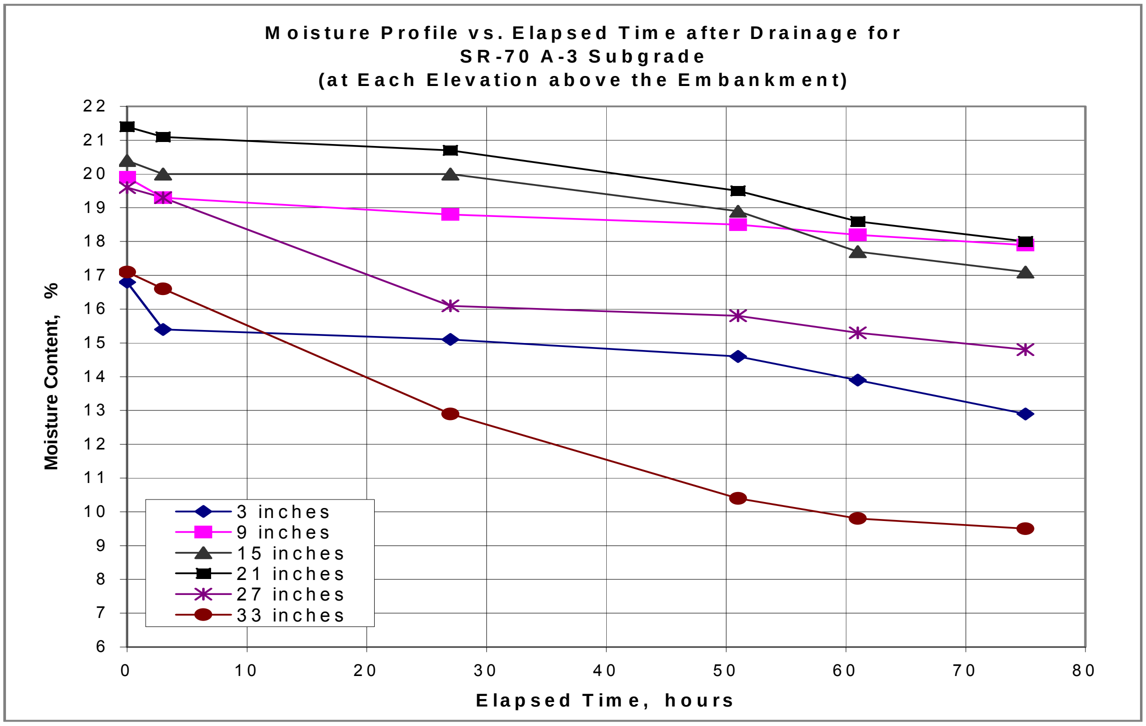


Figure 7.2(B) Moisture Profile vs. Elapsed Time after Drainage for SR70 A-3 Subgrade

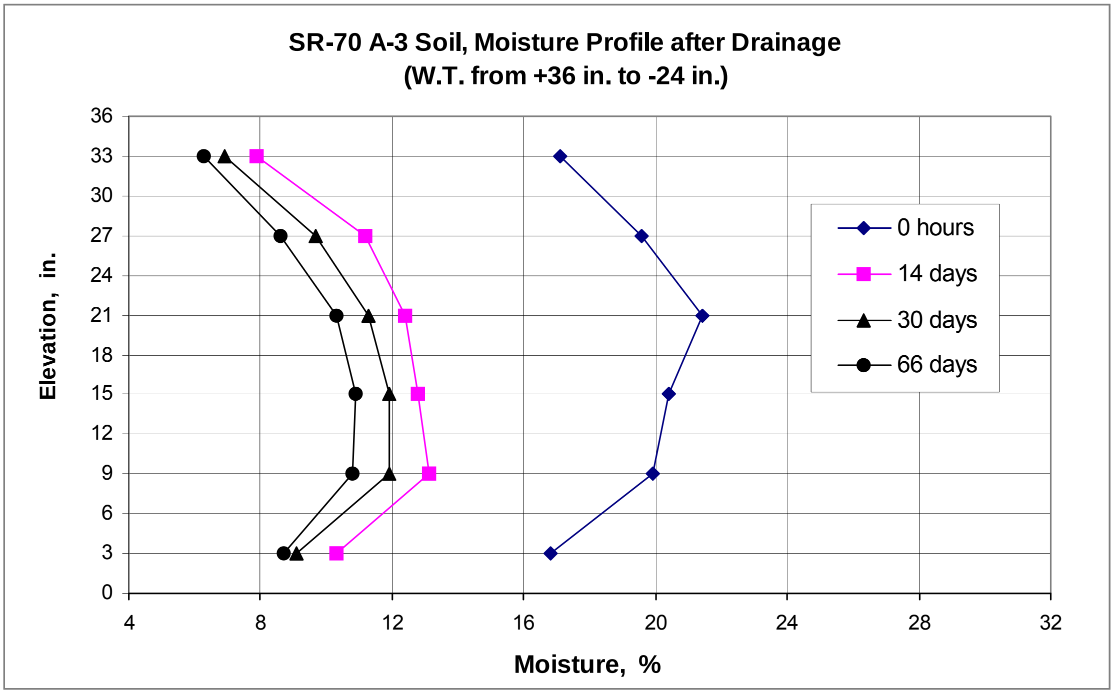


Figure 7.3(A) SR70 A-3 Soil Moisture Profile after Drainage (long-term) Water Table from +36 in. to -24 in.)

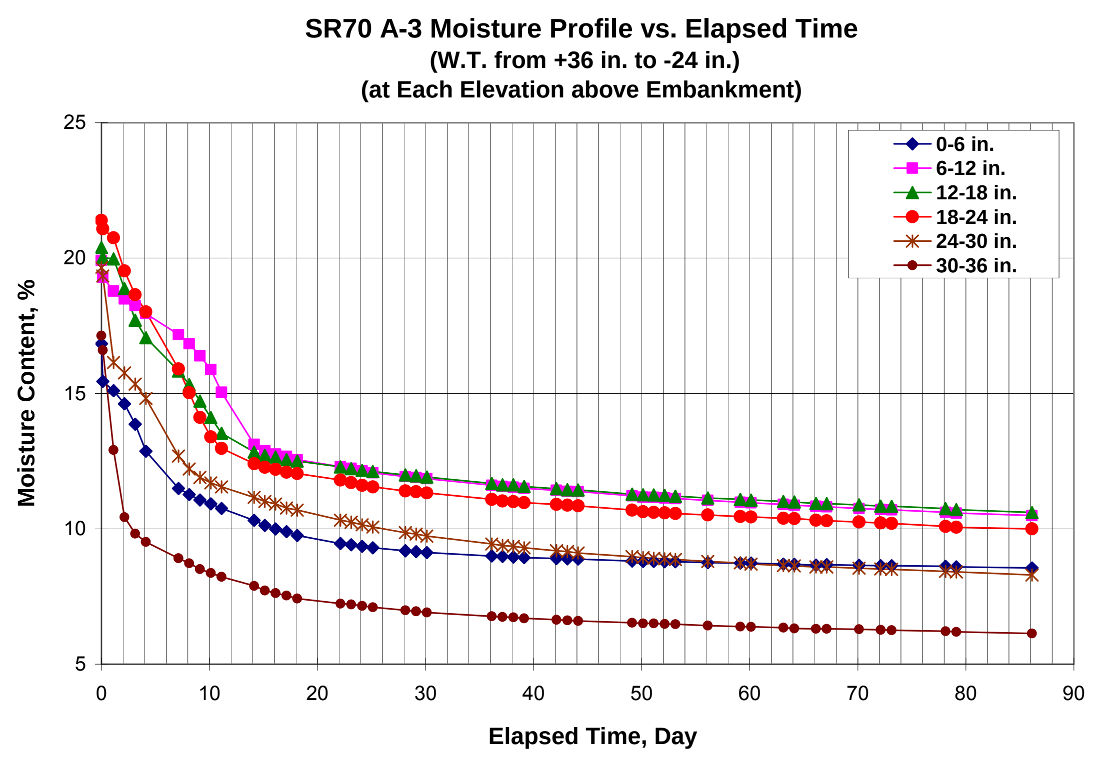


Figure 7.3(B) Moisture Profile vs. Elapsed Time after Drainage for SR70 A-3 Subgrade

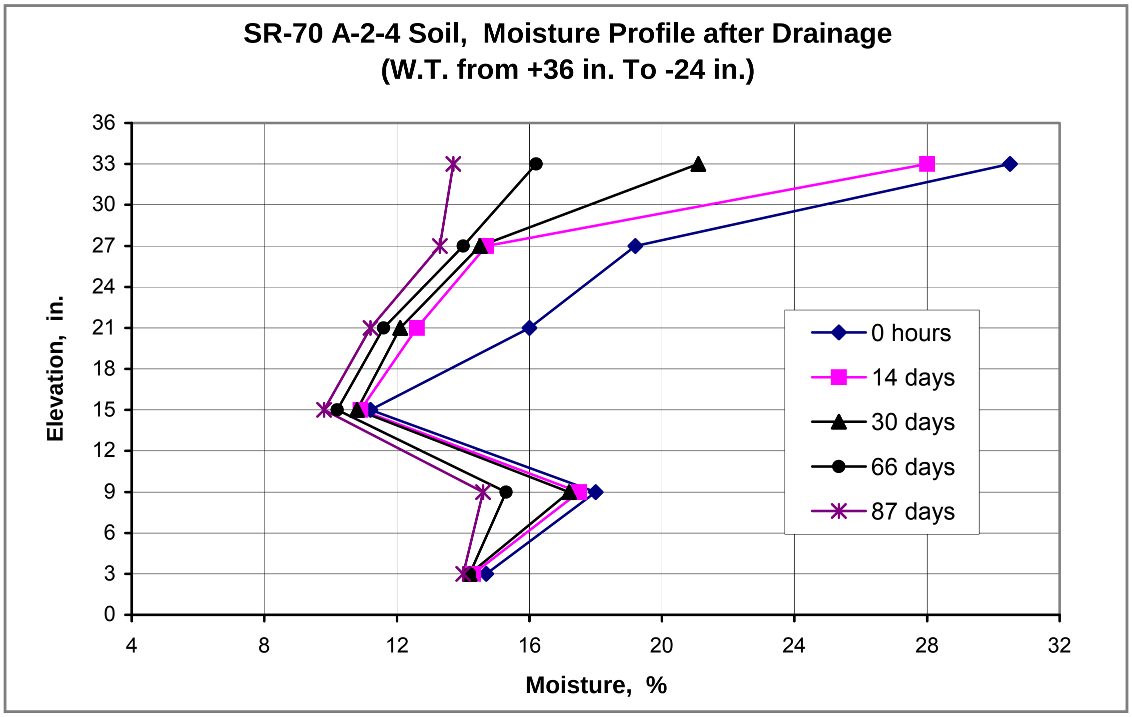


Figure 7.4(A) SR70 A-2-4 Soil Moisture Profile after Drainage (Water Table from +36 in. to -24 in.)

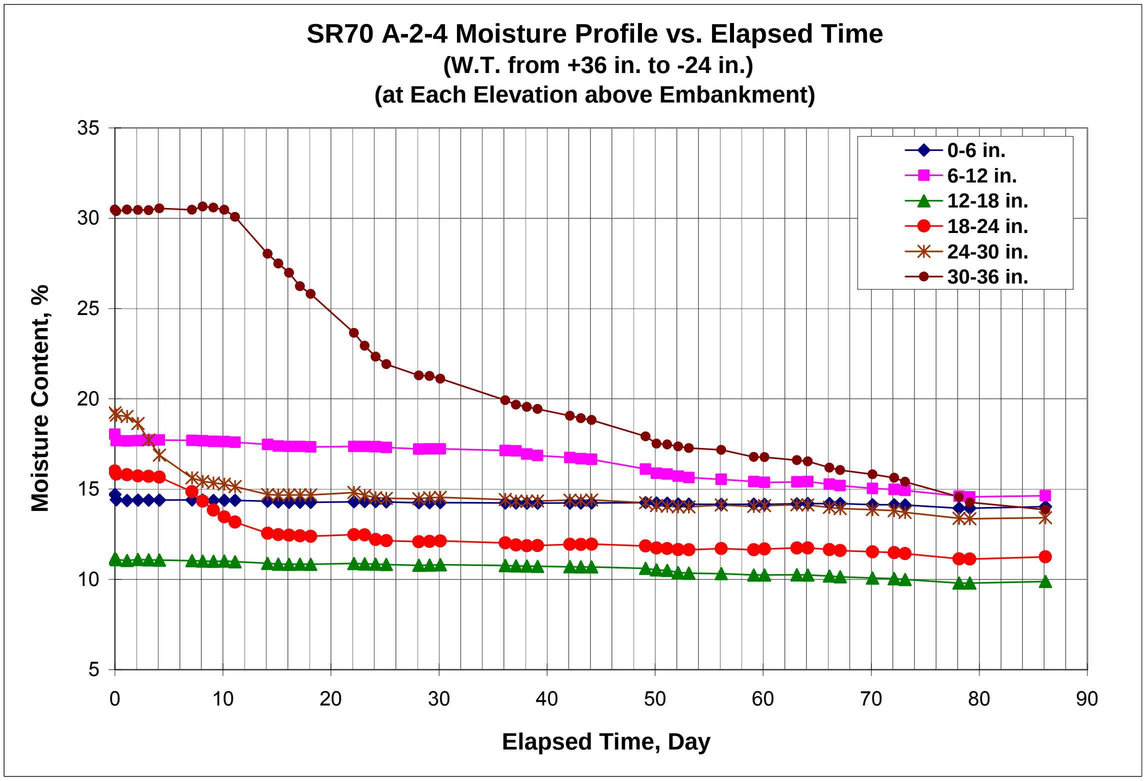


Figure 7.4(B) Moisture Profile vs. Elapsed Time after Drainage for SR70 A-2-4 Subgrade

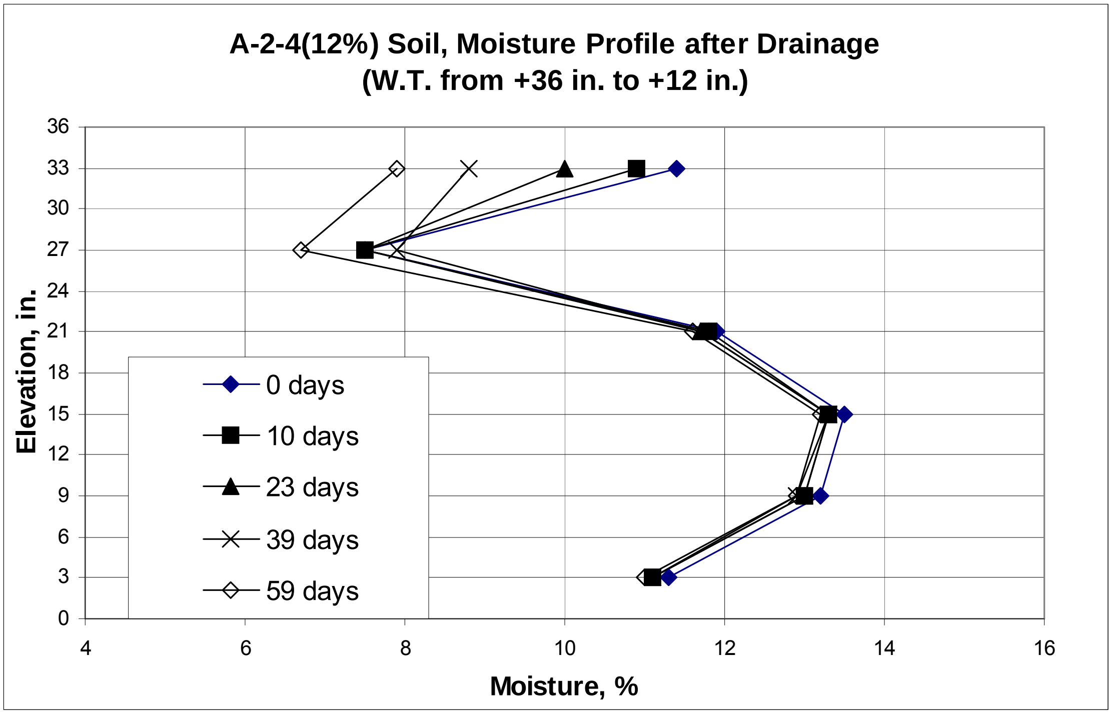


Figure 7.5(A) A-2-4(12%) Soil Moisture Profile after Drainage (Water Table from +36 in. to +12 in.)

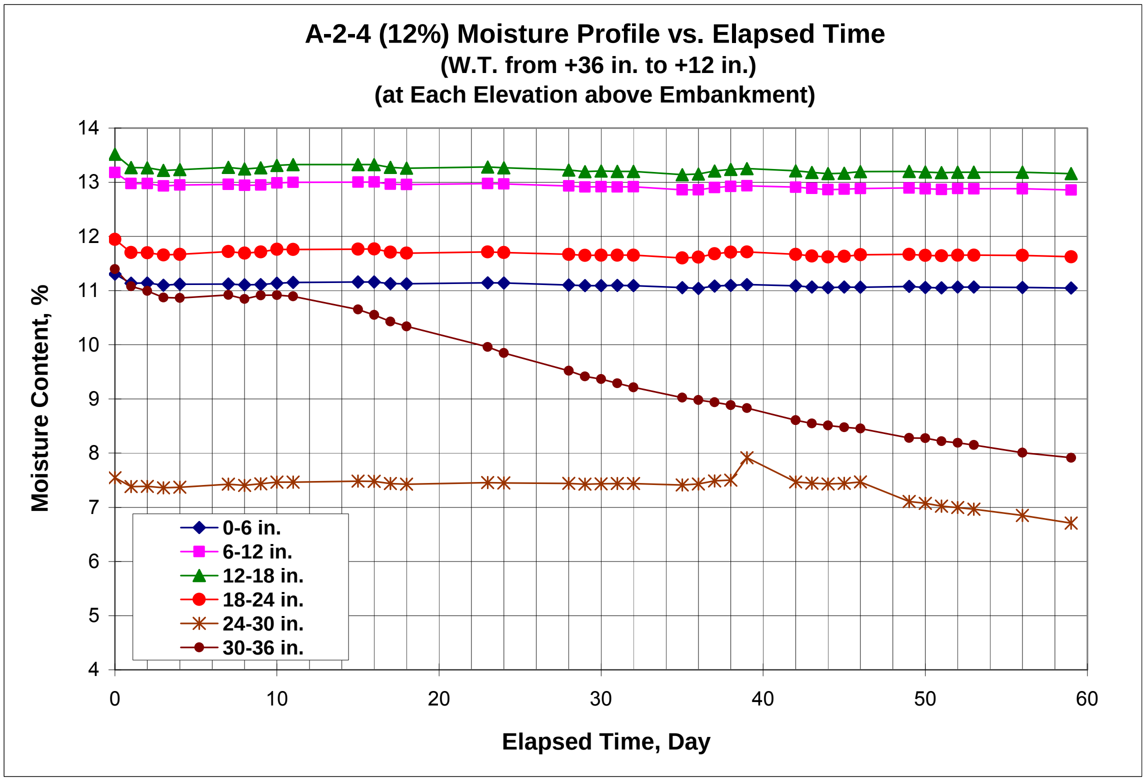


Figure 7.5(B) Moisture Profile vs. Elapsed Time after Drainage for A-2-4(12%) Subgrade

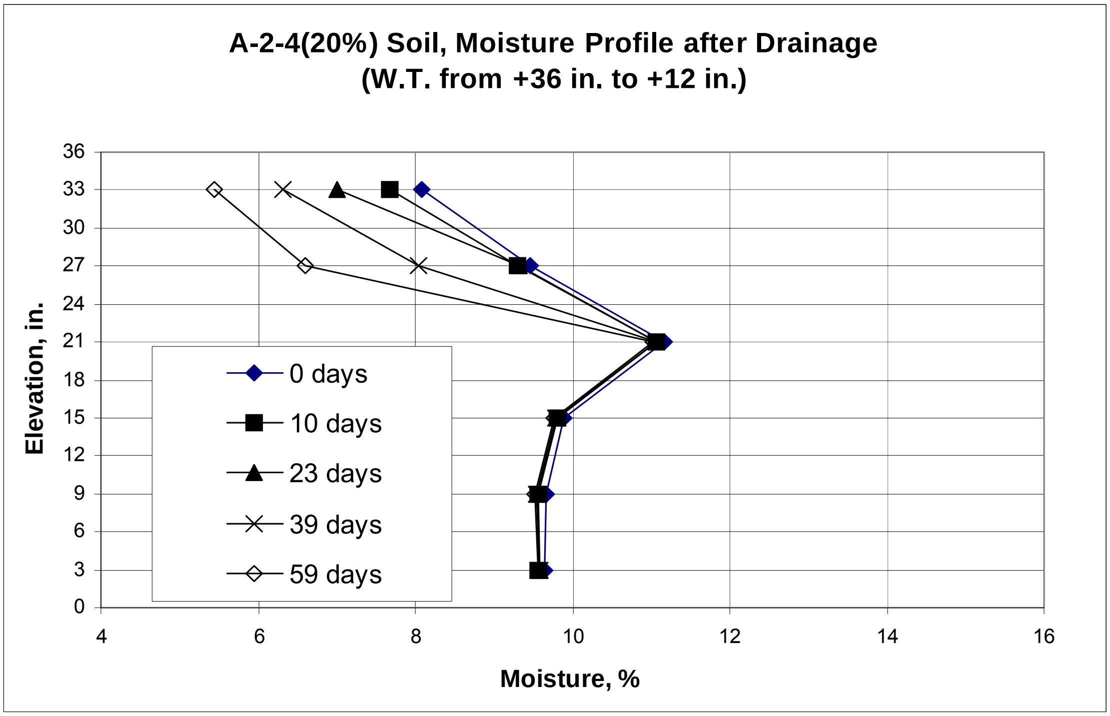


Figure 7.6(A) A-2-4(20%) Soil Moisture Profile after Drainage (Water Table from +36 in. to +12 in.)

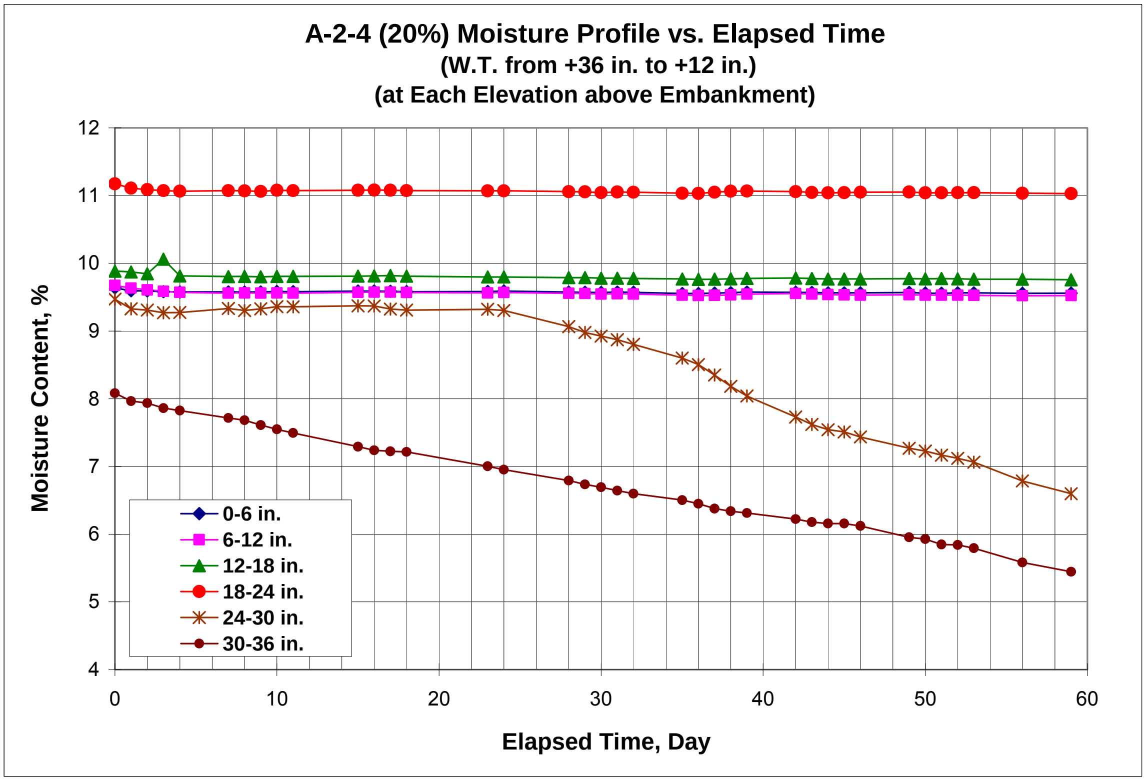


Figure 7.6(B) Moisture Profile vs. Elapsed Time after Drainage for A-2-4(20%) Subgrade

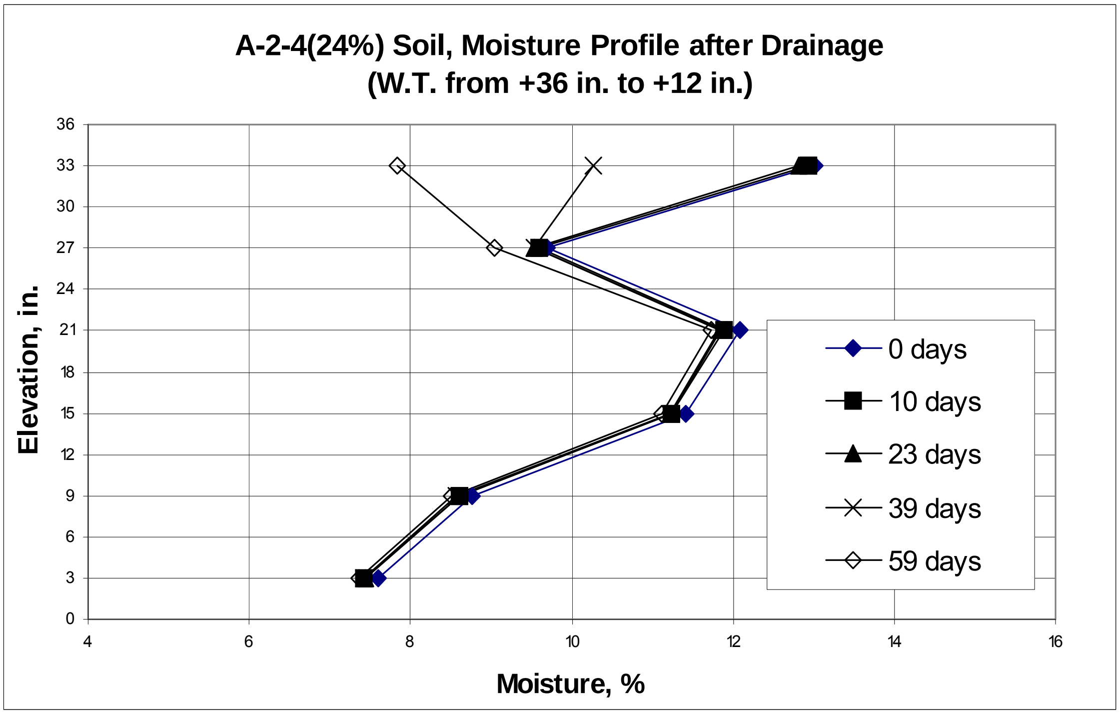


Figure 7.7(A) A-2-4(24%) Soil Moisture Profile after Drainage (Water Table from +36 in. to +12 in.)

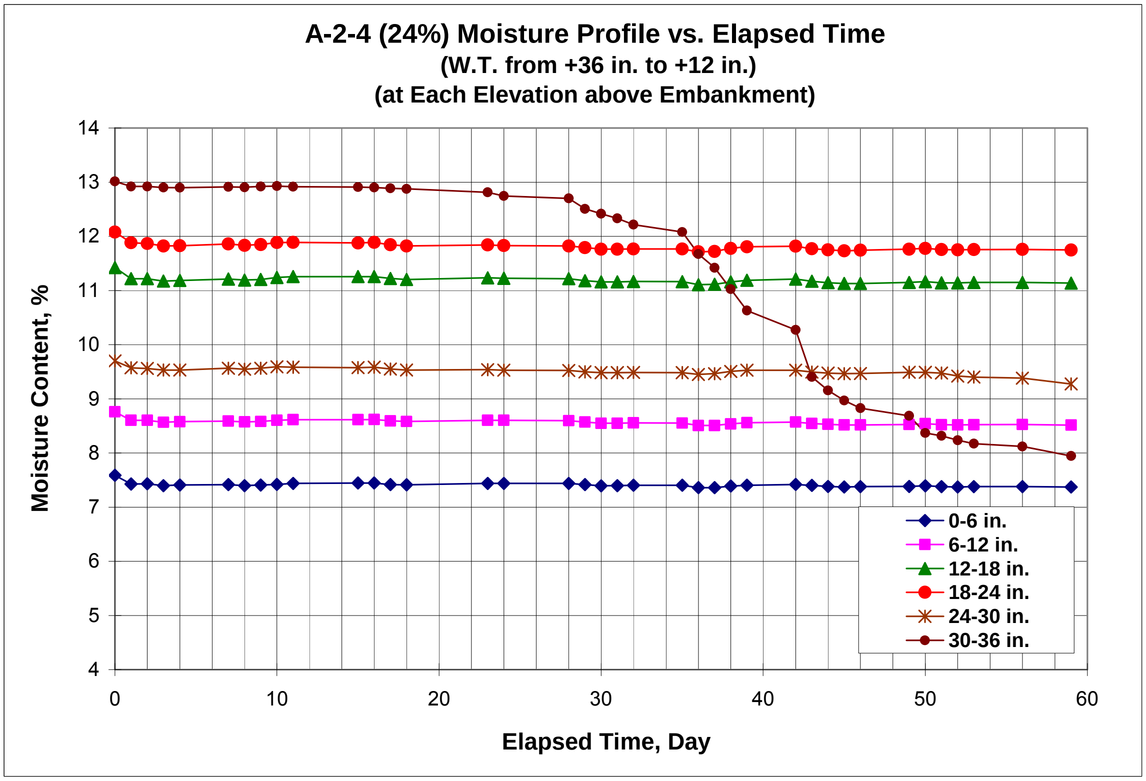


Figure 7.7(B) Moisture Profile vs. Elapsed Time after Drainage for A-2-4(24%) Subgrade

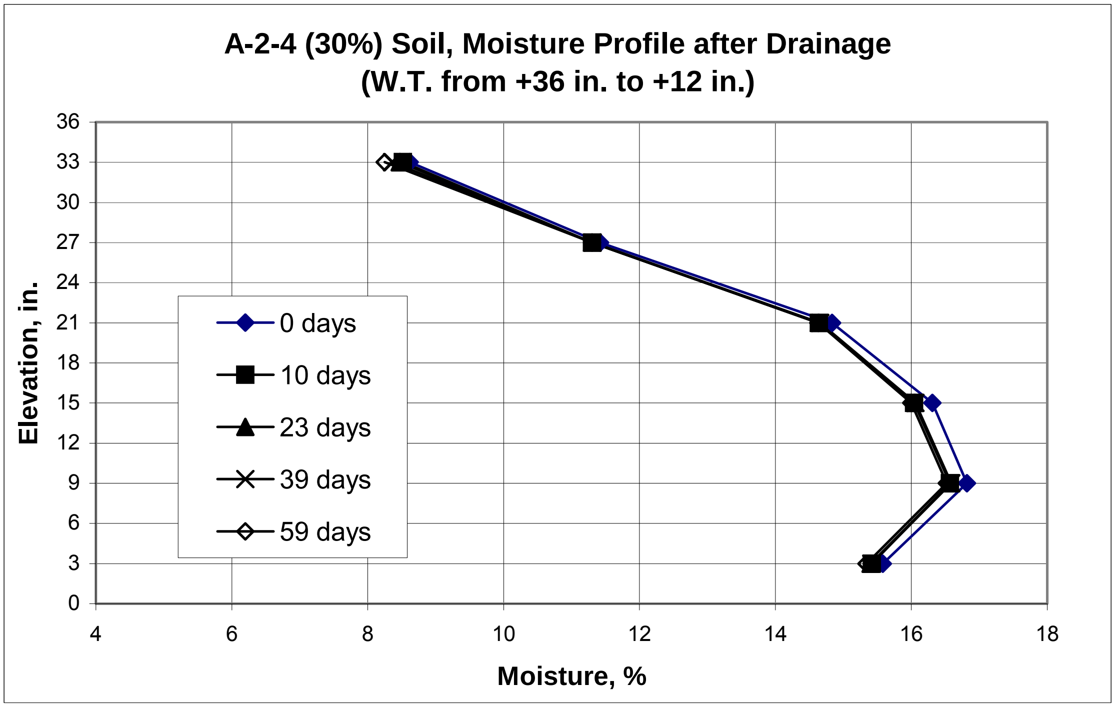


Figure 7.8(A) A-2-4(30%) Soil Moisture Profile after Drainage (Water Table from +36 in. to +12 in.)

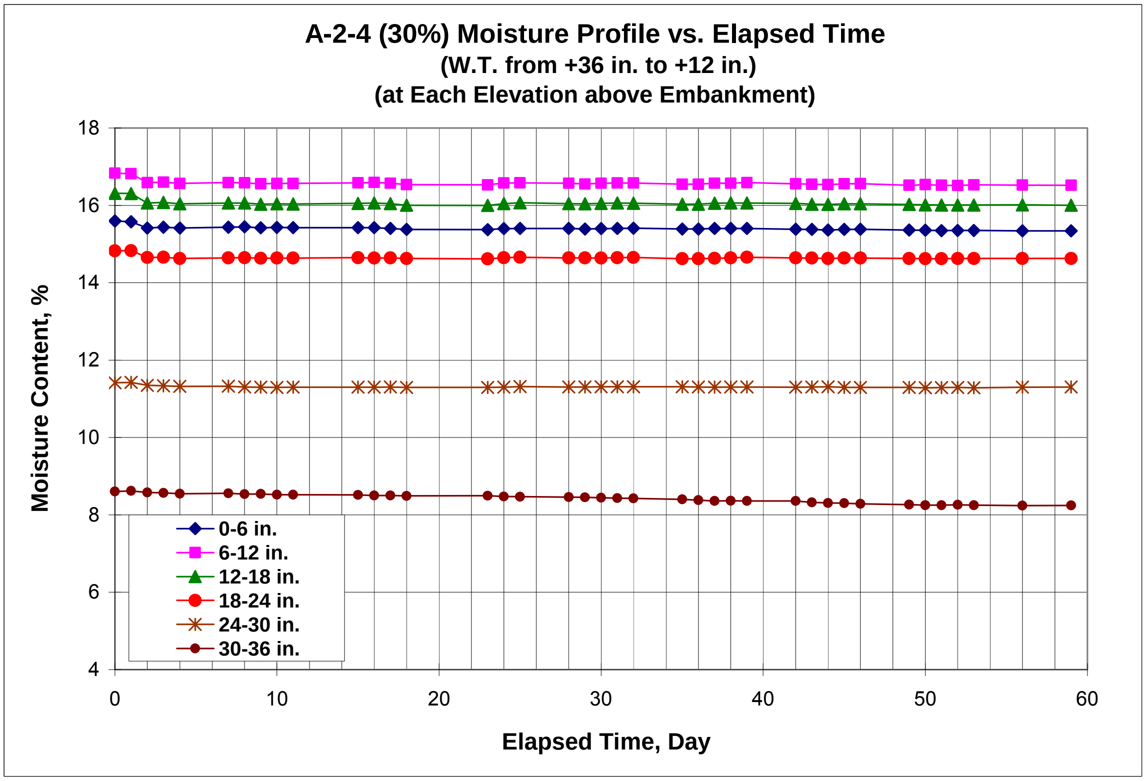


Figure 7.8(B) Moisture Profile vs. Elapsed Time after Drainage for A-2-4(30%) Subgrade

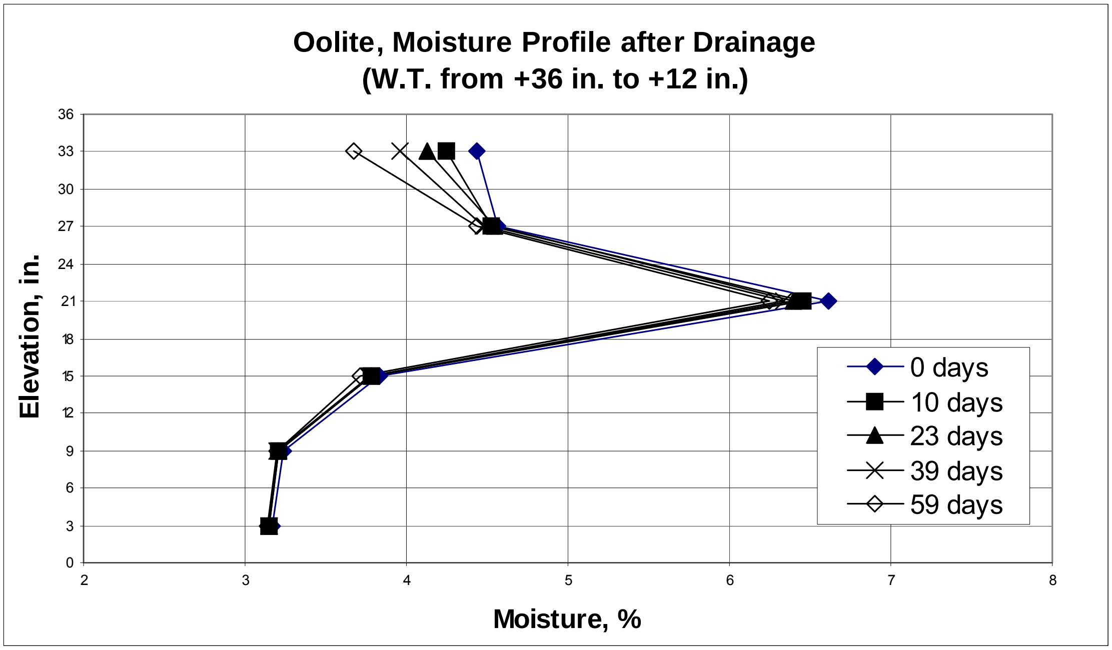


Figure 7.9(A) Miami Oolite A-1 Moisture Profile after Drainage (Water Table from +36 in. to +12 in.)

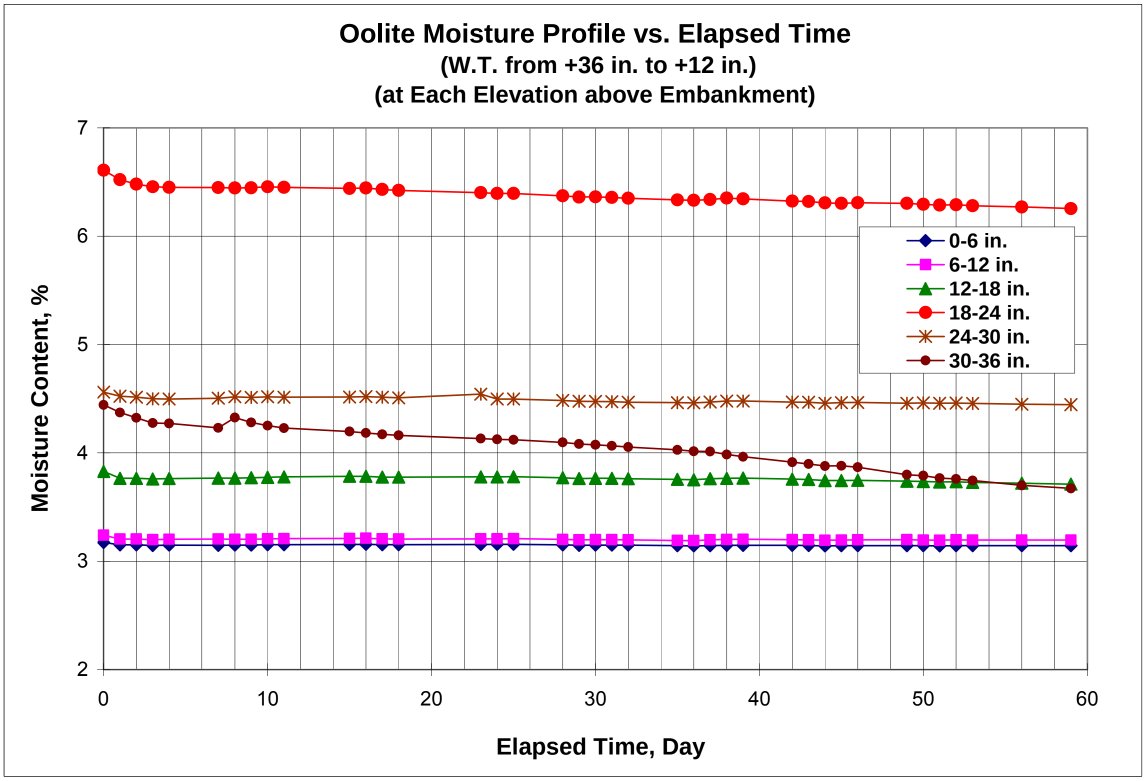


Figure 7.9(B) Moisture Profile vs. Elapsed Time after Drainage for Oolite Subgrade

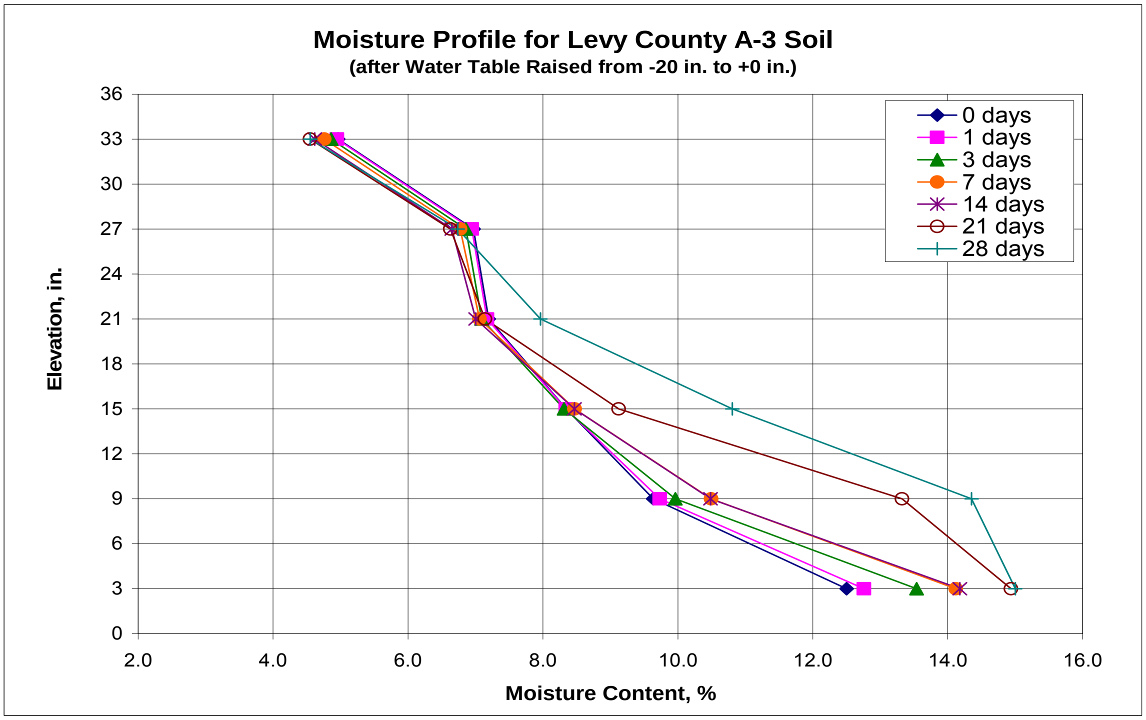


Figure 7.10(A) Moisture Profile after Groundwater Level Raised from -20 in. to 0 in. for Levy County A-3 Soil

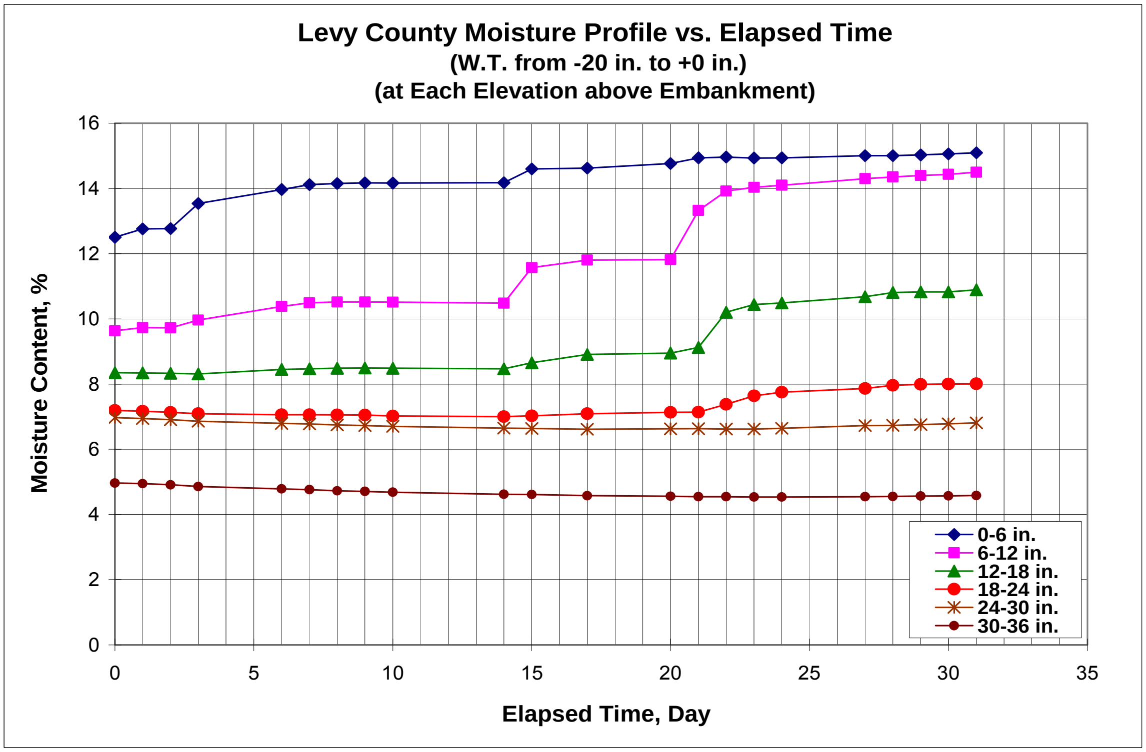


Figure 7.10(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -20 in. to 0 in. for Levy County A-3 Soil

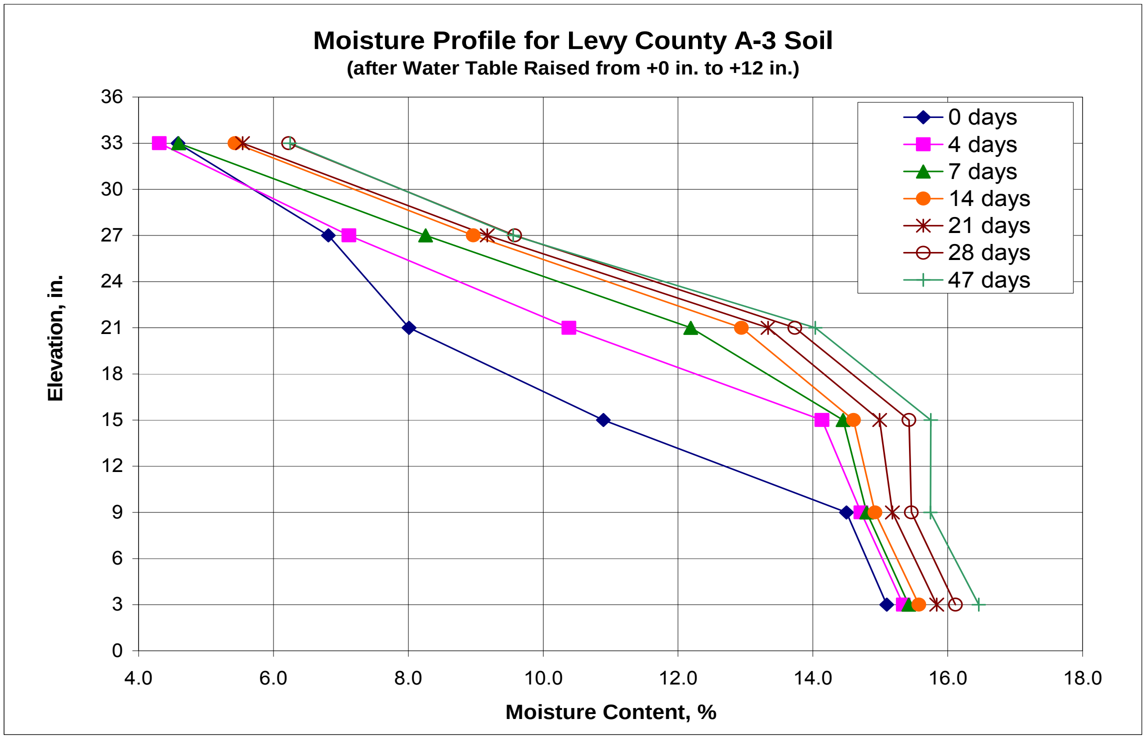


Figure 7.11(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for Levy County A-3 Soil

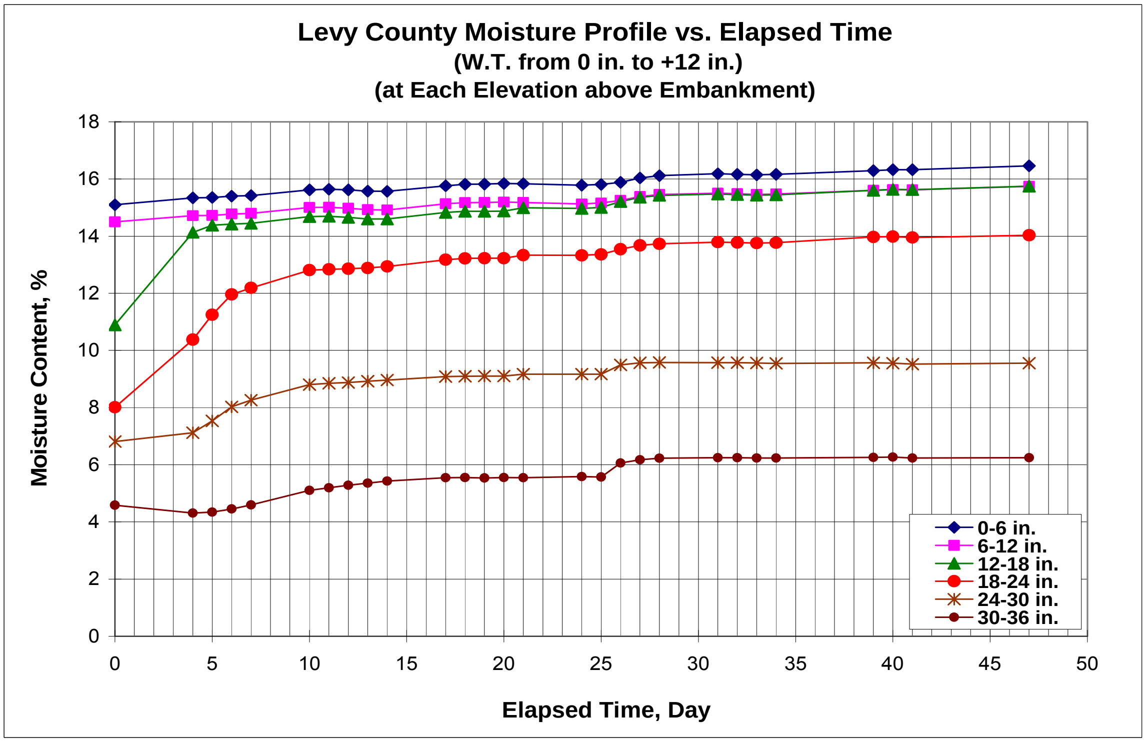


Figure 7.11(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Levy County A-3 Soil

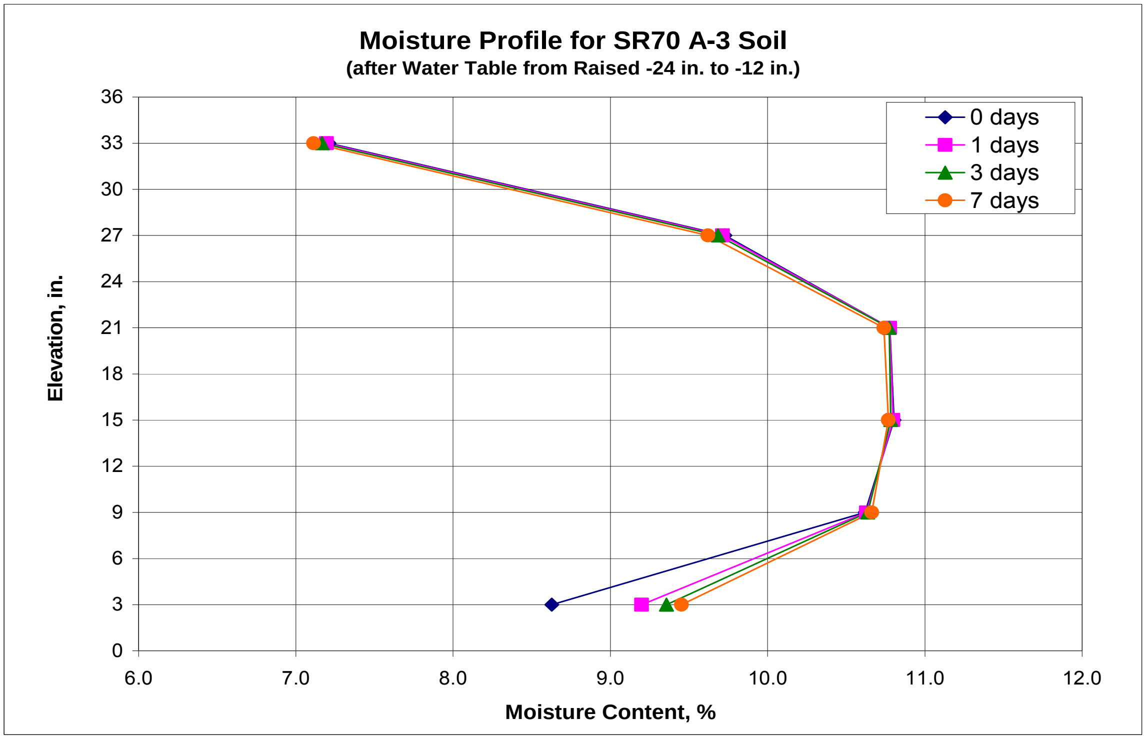


Figure 7.12(A) Moisture Profile after Groundwater Level Raised from -24 in. to -12 in. for SR70 A-3 Soil

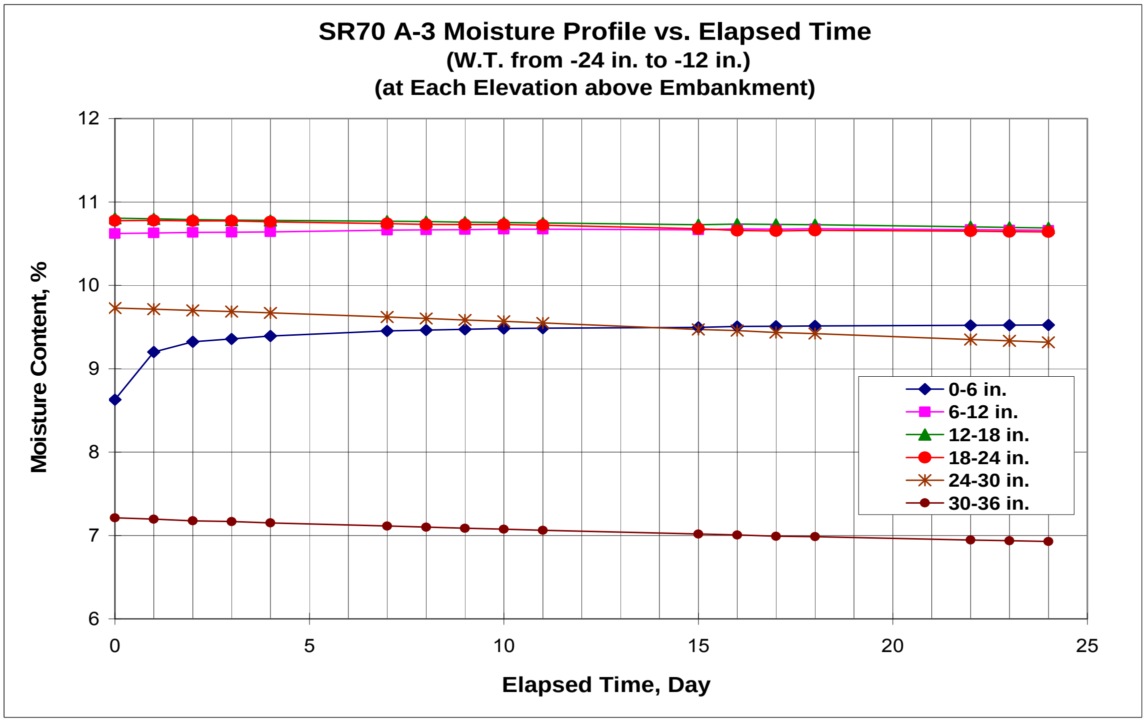


Figure 7.12(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to -12 in. for SR70 A-3 Soil

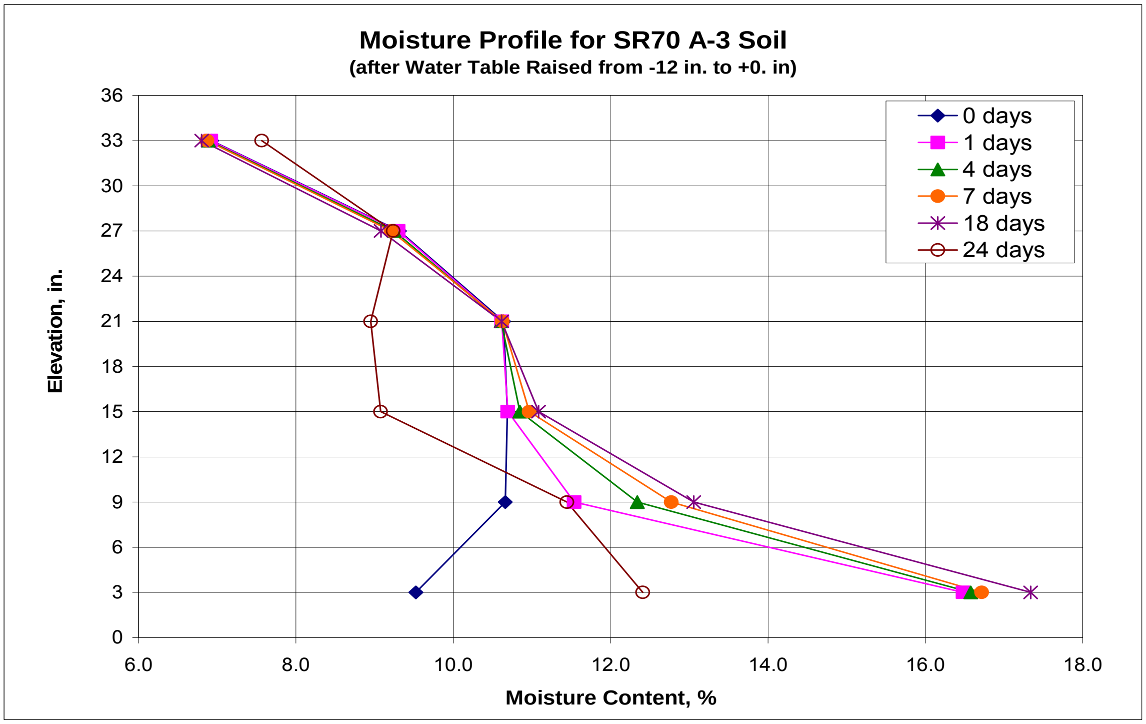


Figure 7.13(A) Moisture Profile after Groundwater Level Raised from -12 in. to 0 in. for SR70 A-3 Soil

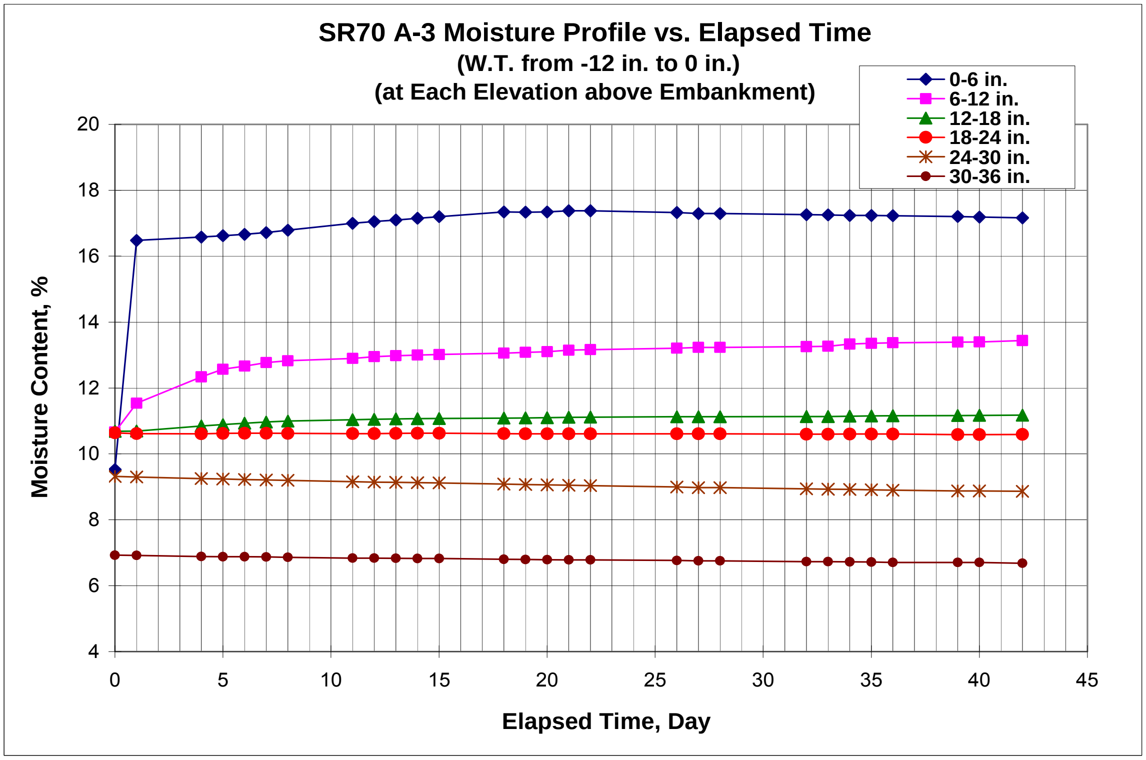


Figure 7.13(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -20 in. to 0 in. for Sr-70 A-3 Soil

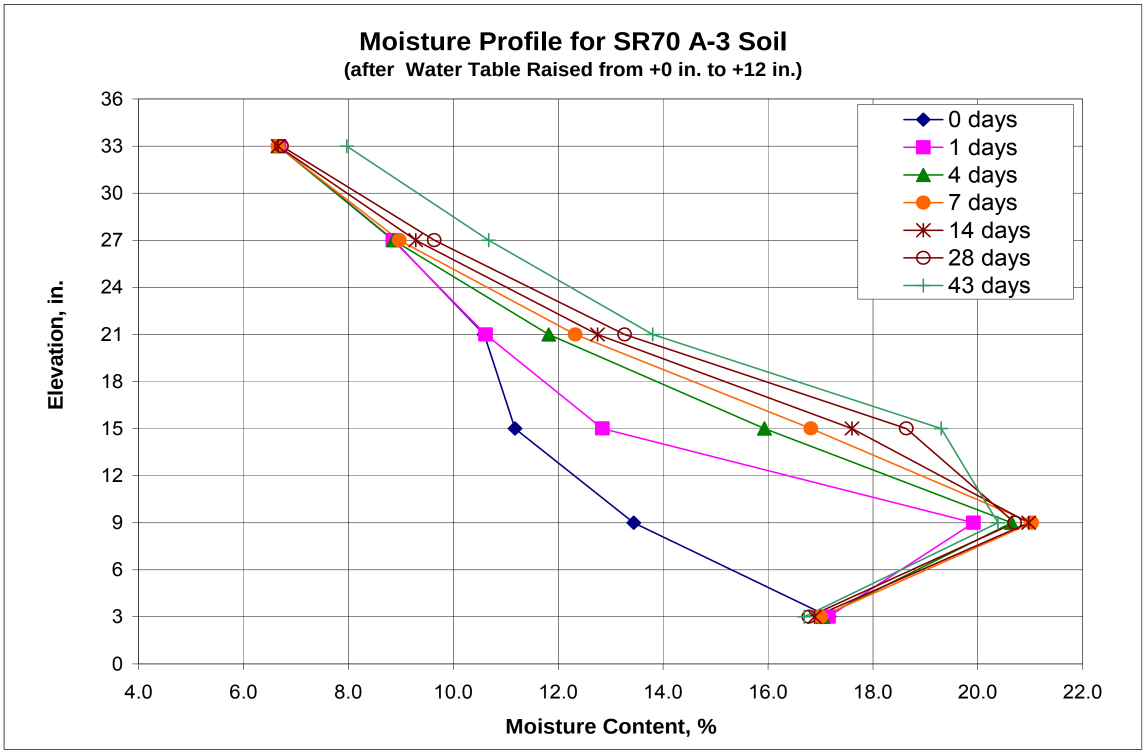


Figure 7.14(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for SR70 A-3 Soil

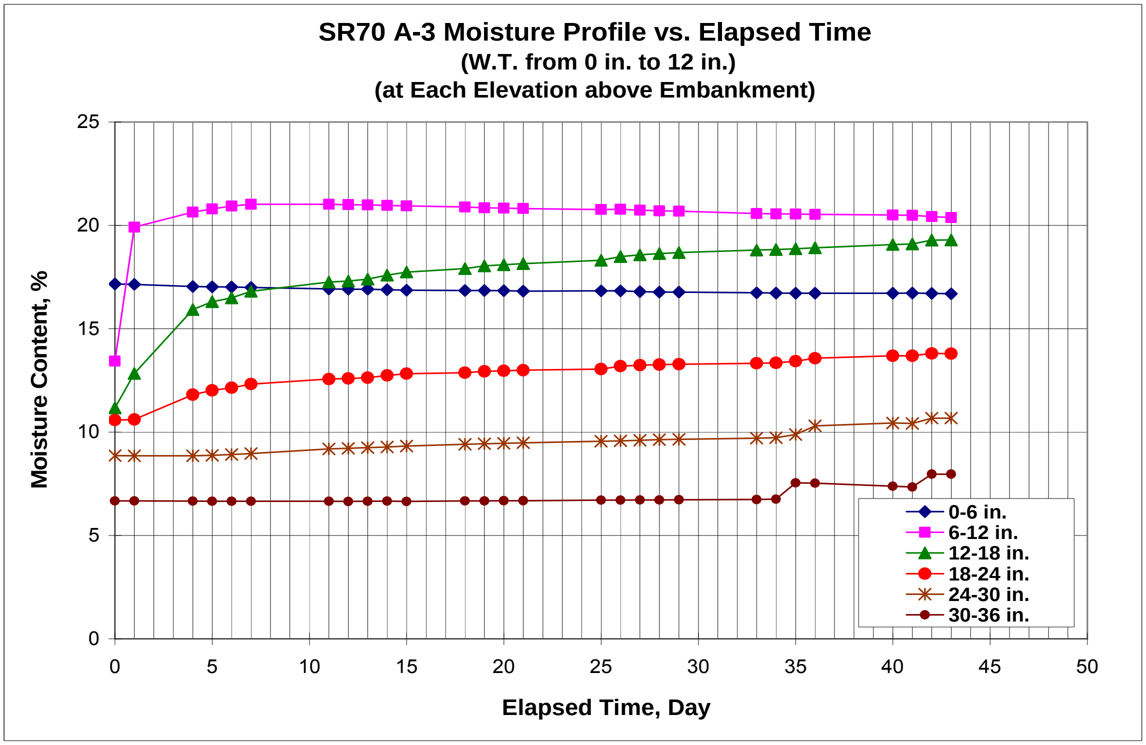


Figure 7.14(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for SR70 A-3 Soil

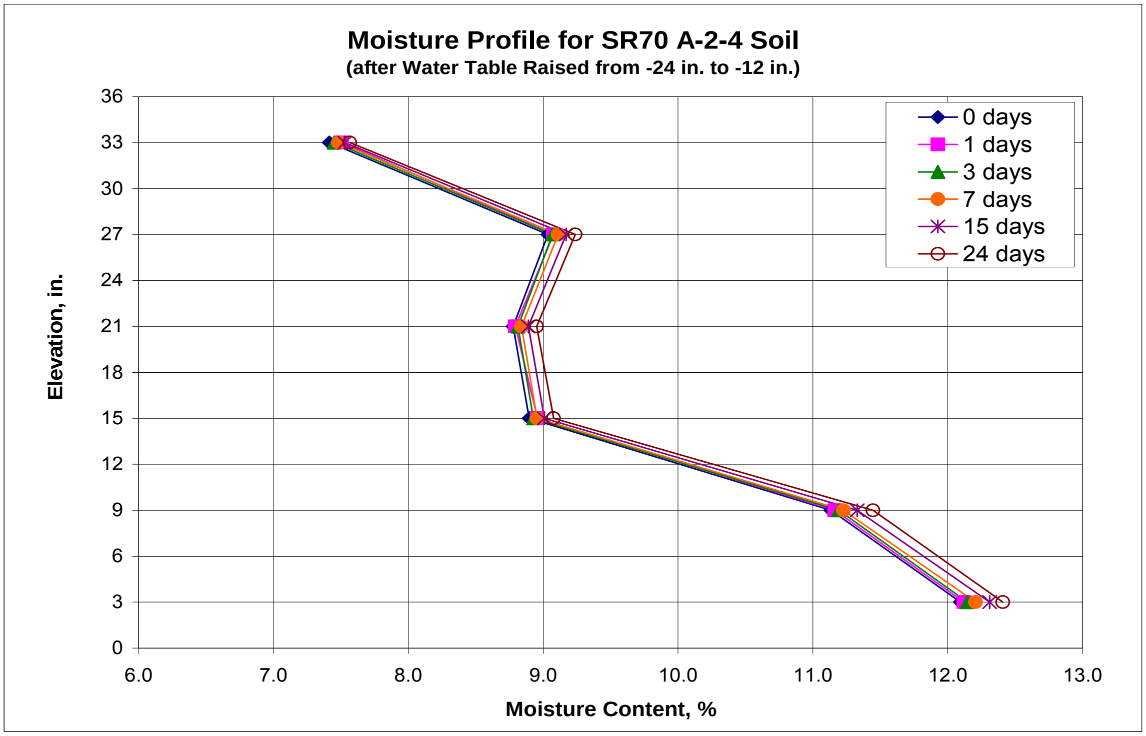


Figure 7.15(A) Moisture Profile after Groundwater Level Raised from -24 in. to -12 in. for SR70 A-2-4 Soil

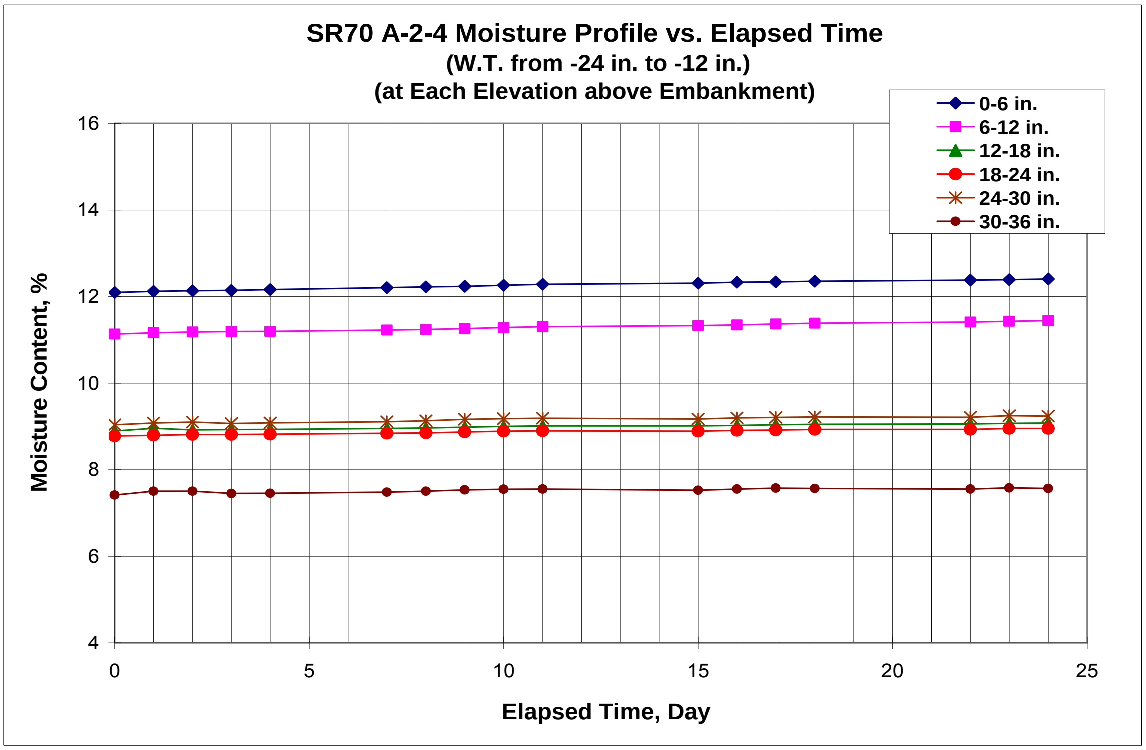


Figure 7.15(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to -12 in. for SR70 A-2-4 Soil

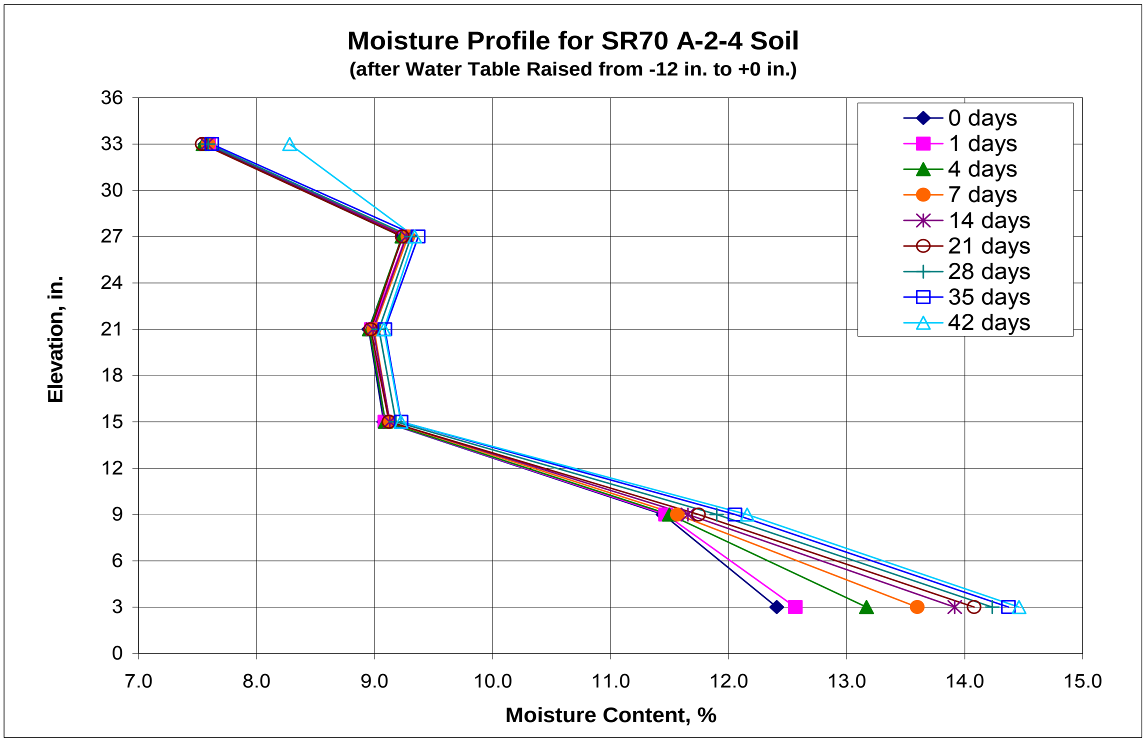


Figure 7.16(A) Moisture Profile after Groundwater Level Raised from -12 in. to +0 in. for SR70 A-2-4 Soil

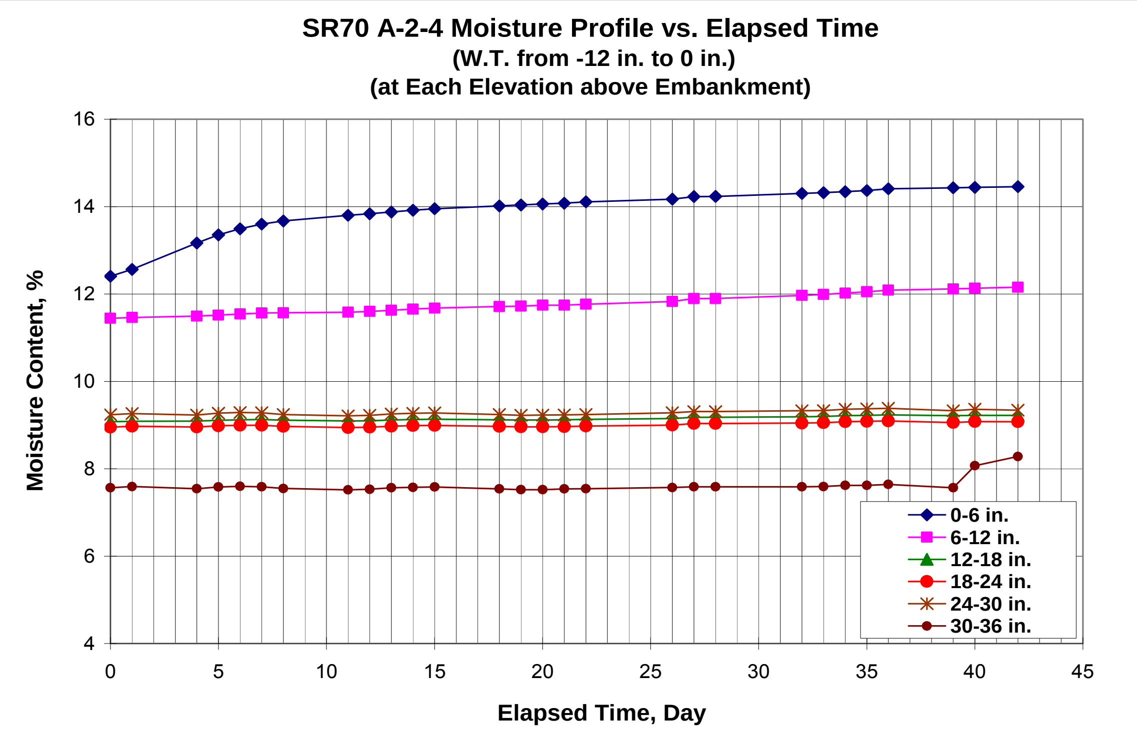


Figure 7.16(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -12 in. to 0 in. for SR70 A-2-4 Soil

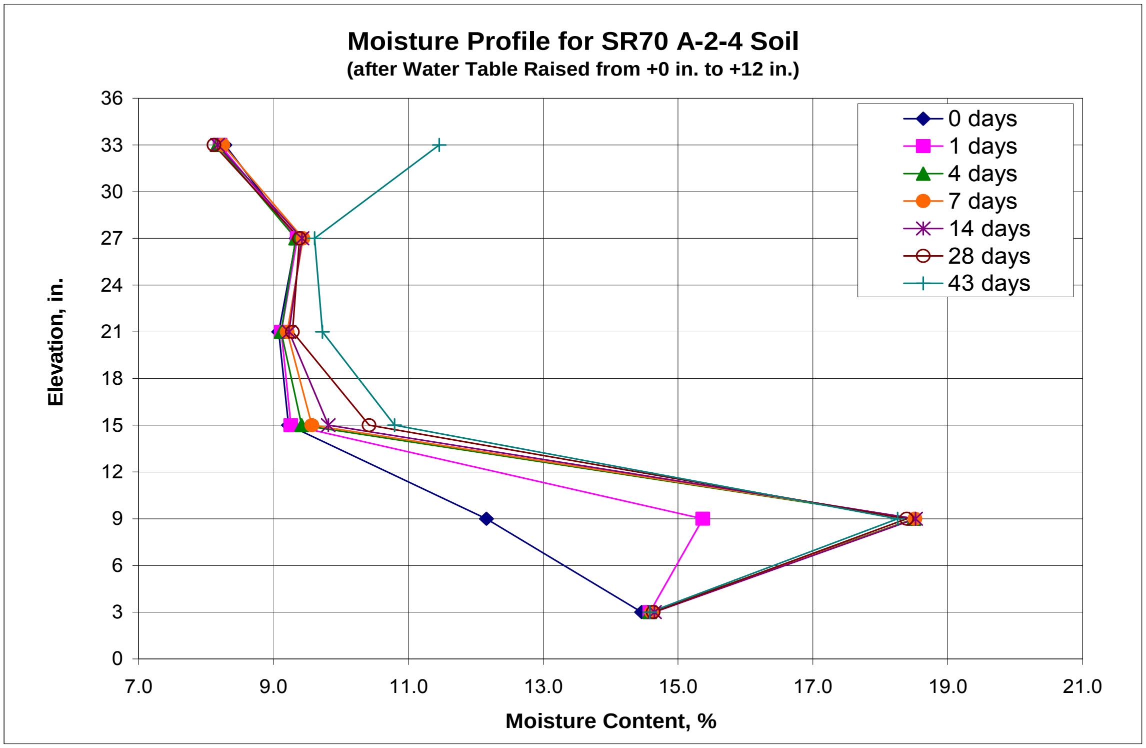


Figure 7.17(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for SR70 A-2-4 Soil

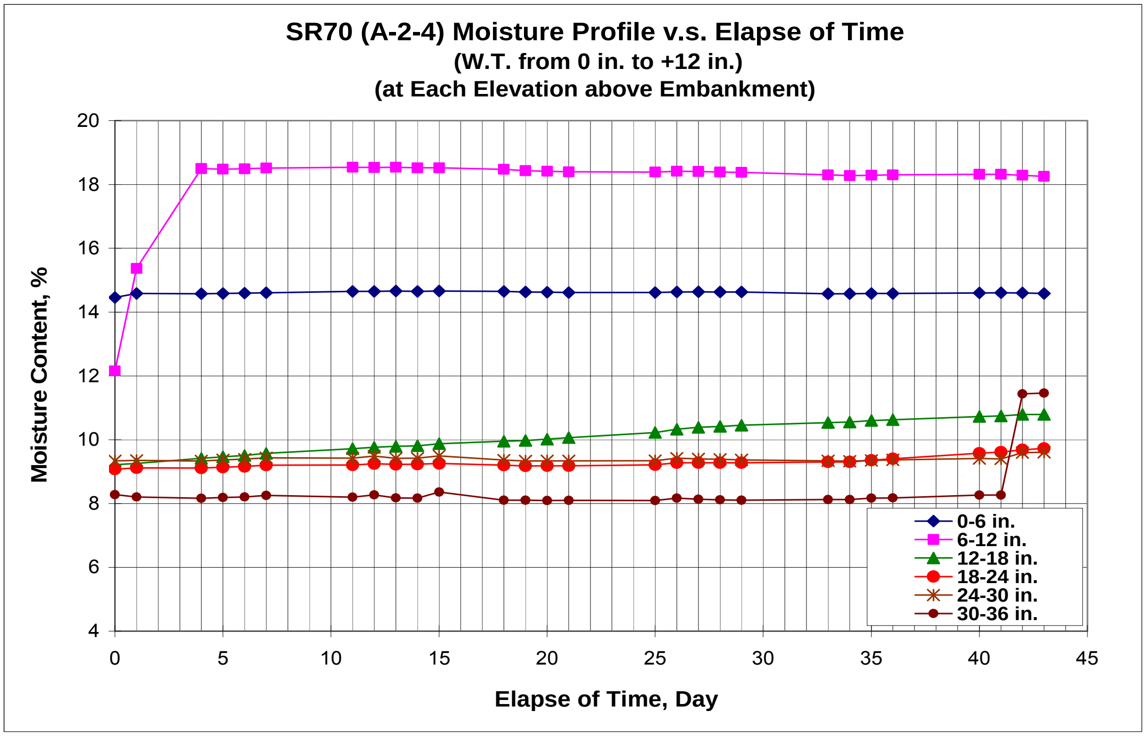


Figure 7.17(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Sr-70 A-2-4 Soil

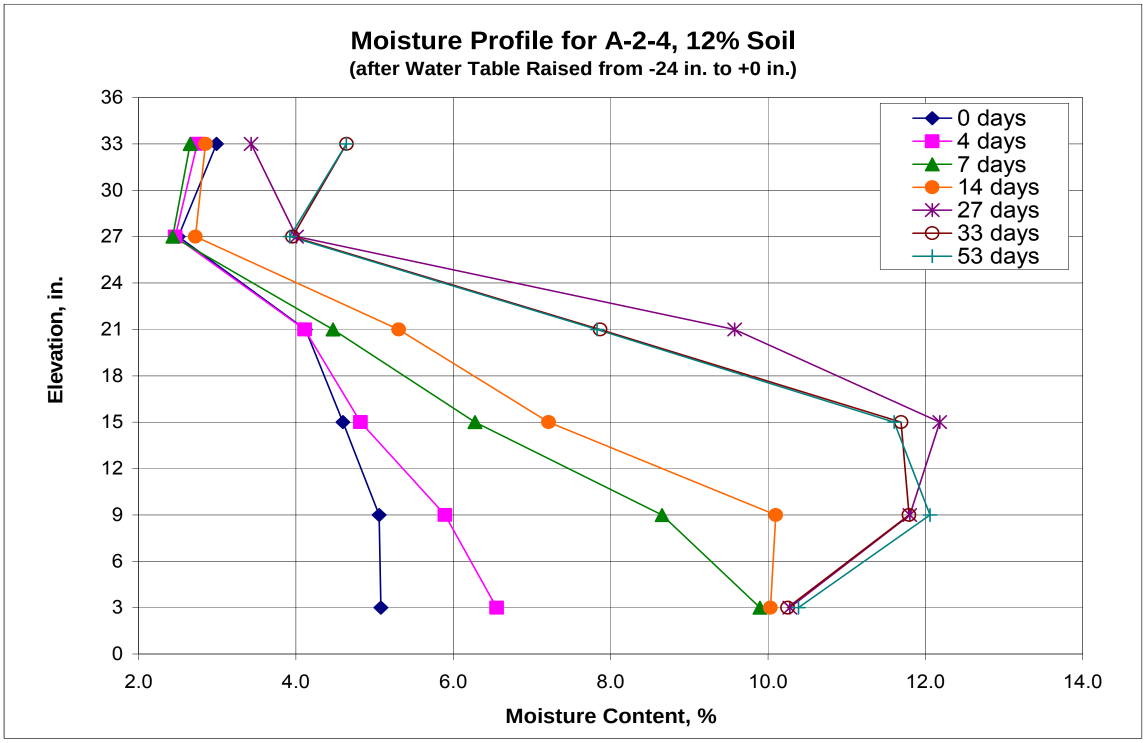


Figure 7.18(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 12% Soil

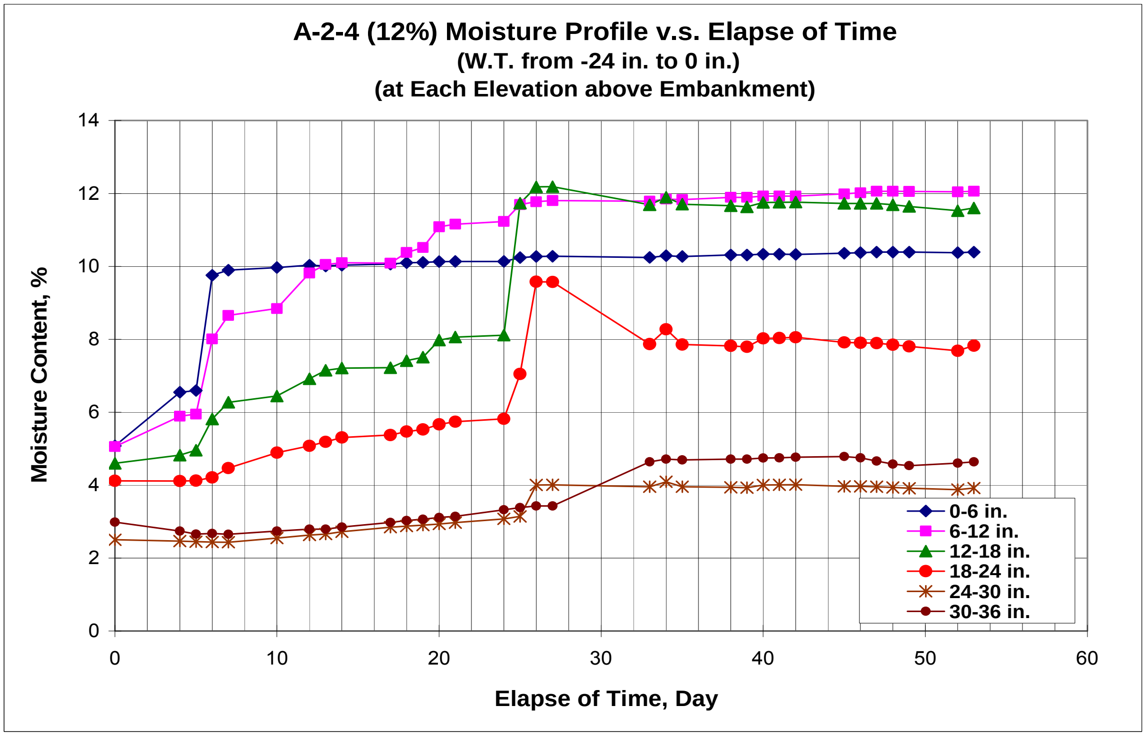


Figure 7.18(B) Moisture Profile vs. Elapsed Time after Raised W.T. from -24 in. to 0 in. for A-2-4, 12% Soil

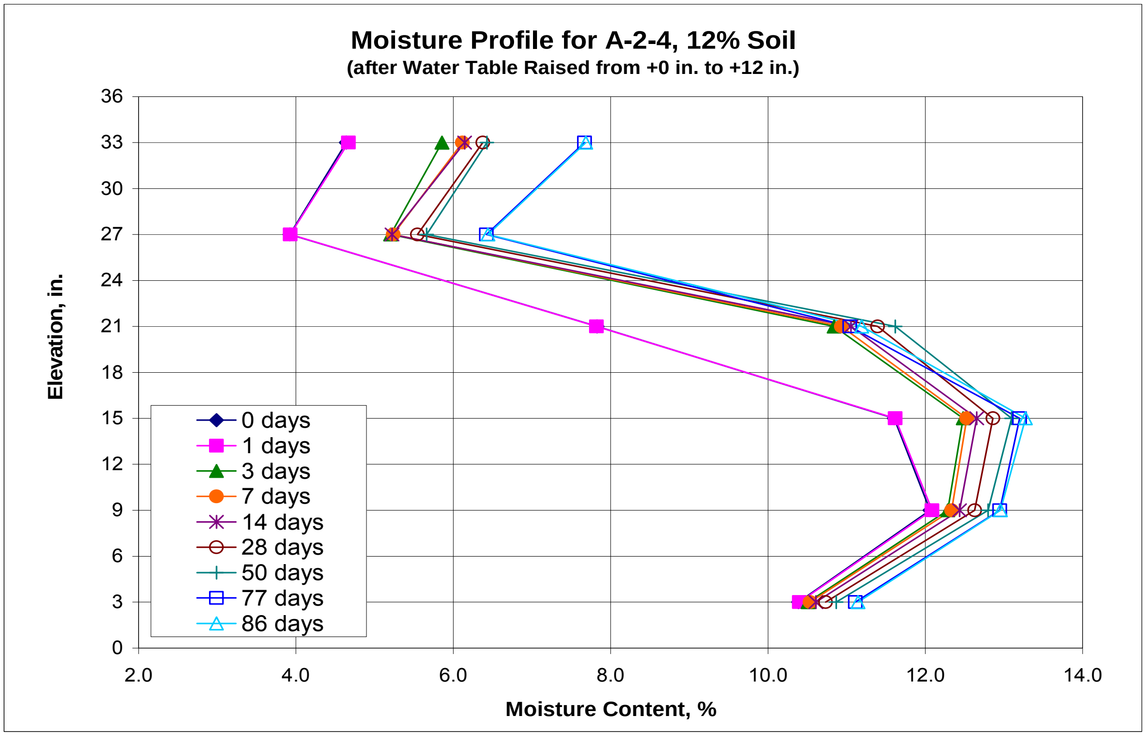


Figure 7.19(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 12% Soil

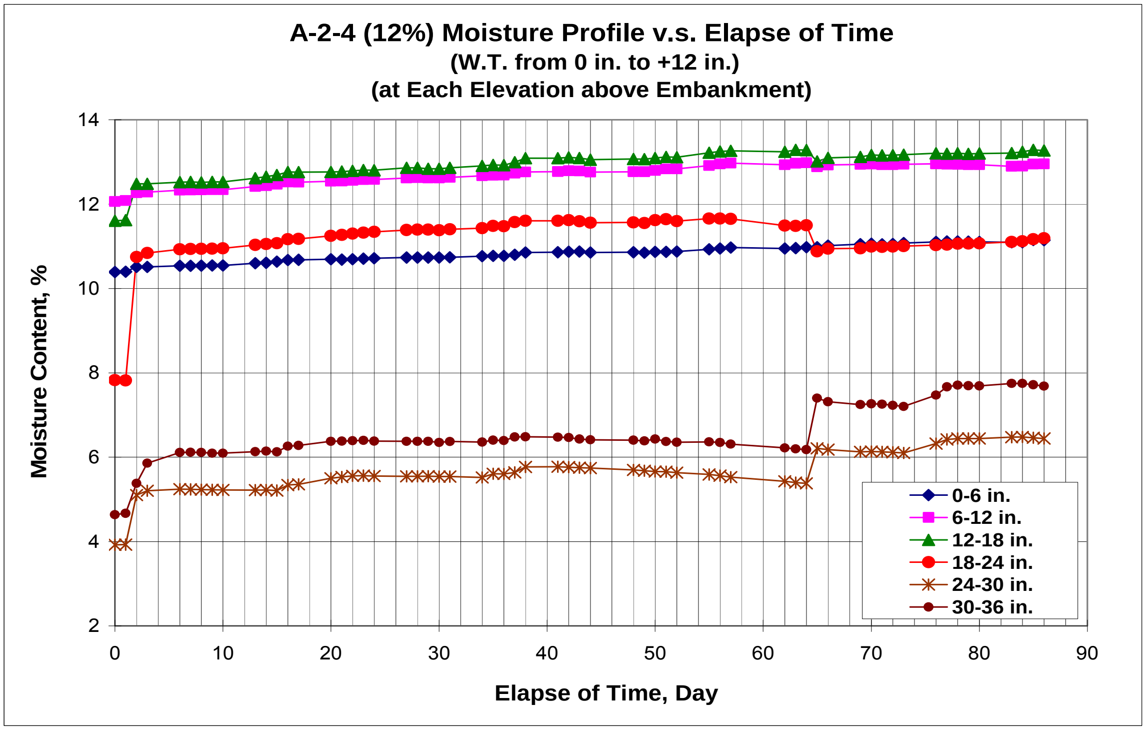


Figure 7.19(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 12% Soil

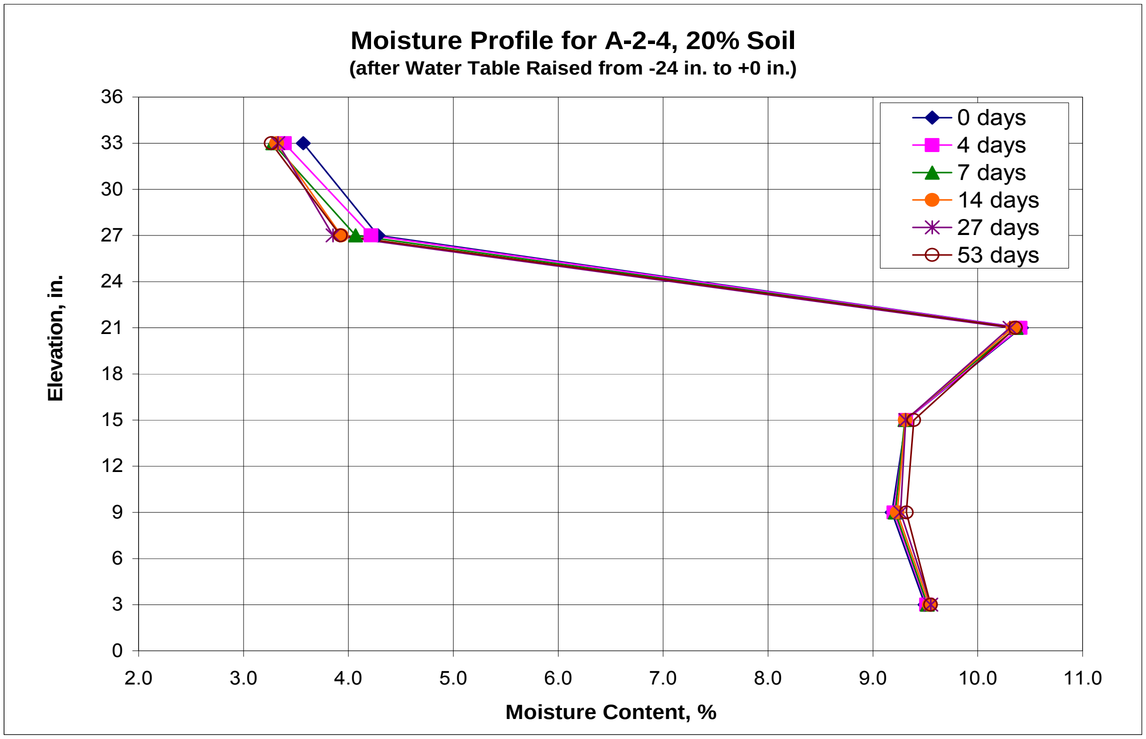


Figure 7.20(A) Moisture Profile after Groundwater Level Raised from -24 in. to +0 in. for A-2-4, 20% Soil

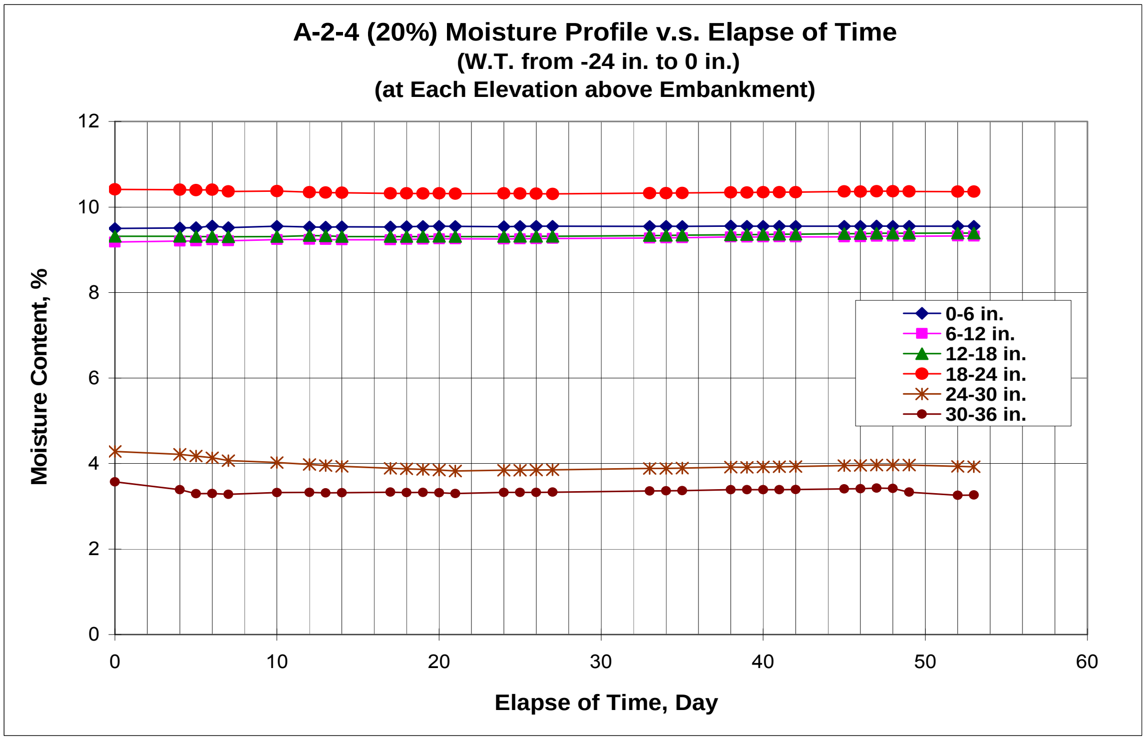


Figure 7.20(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 20% Soil

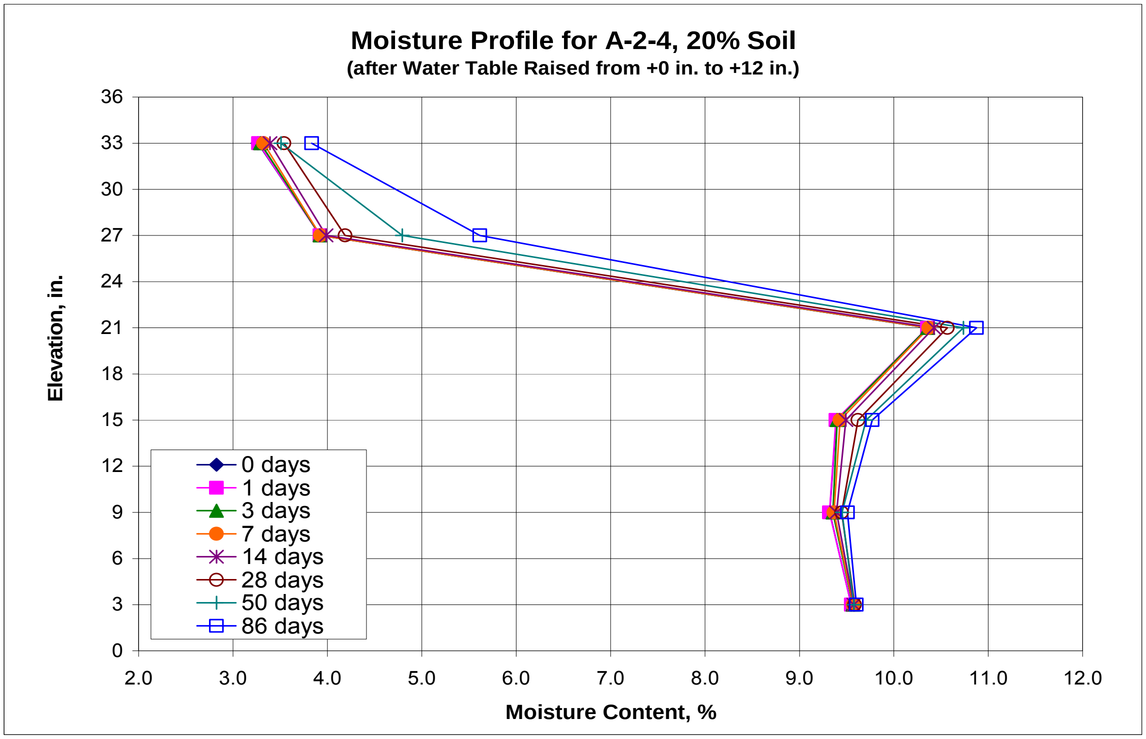


Figure 7.21(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 20% Soil

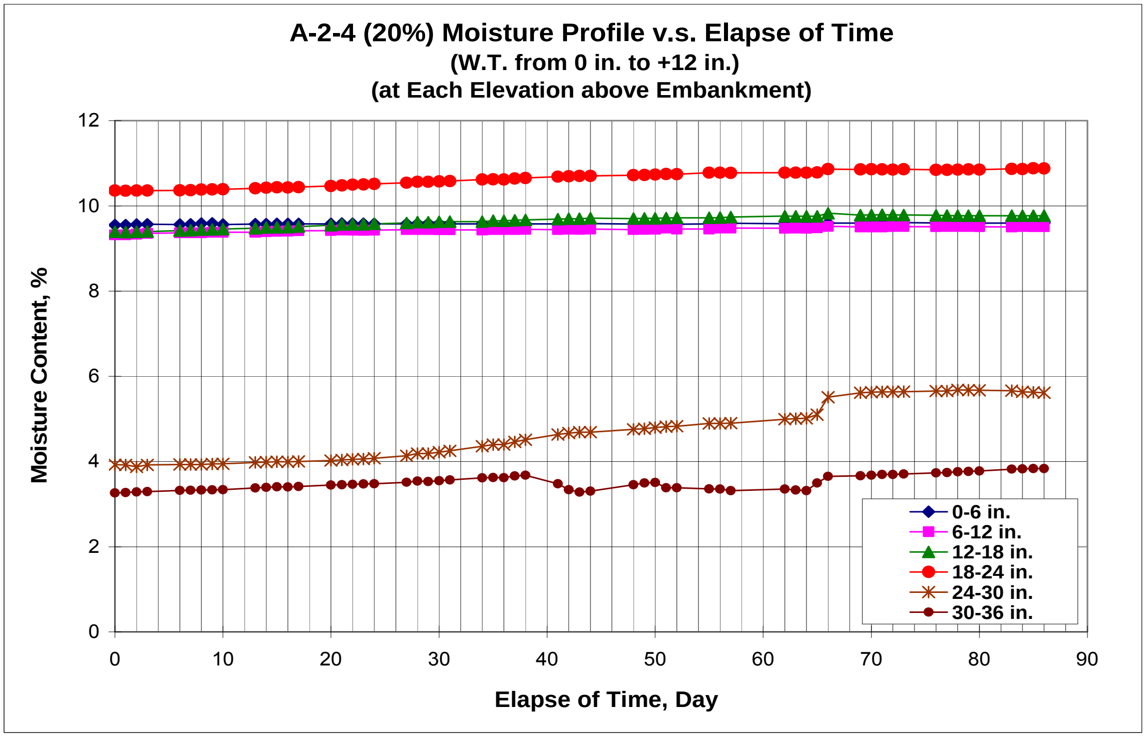


Figure 7.21(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 12% Soil

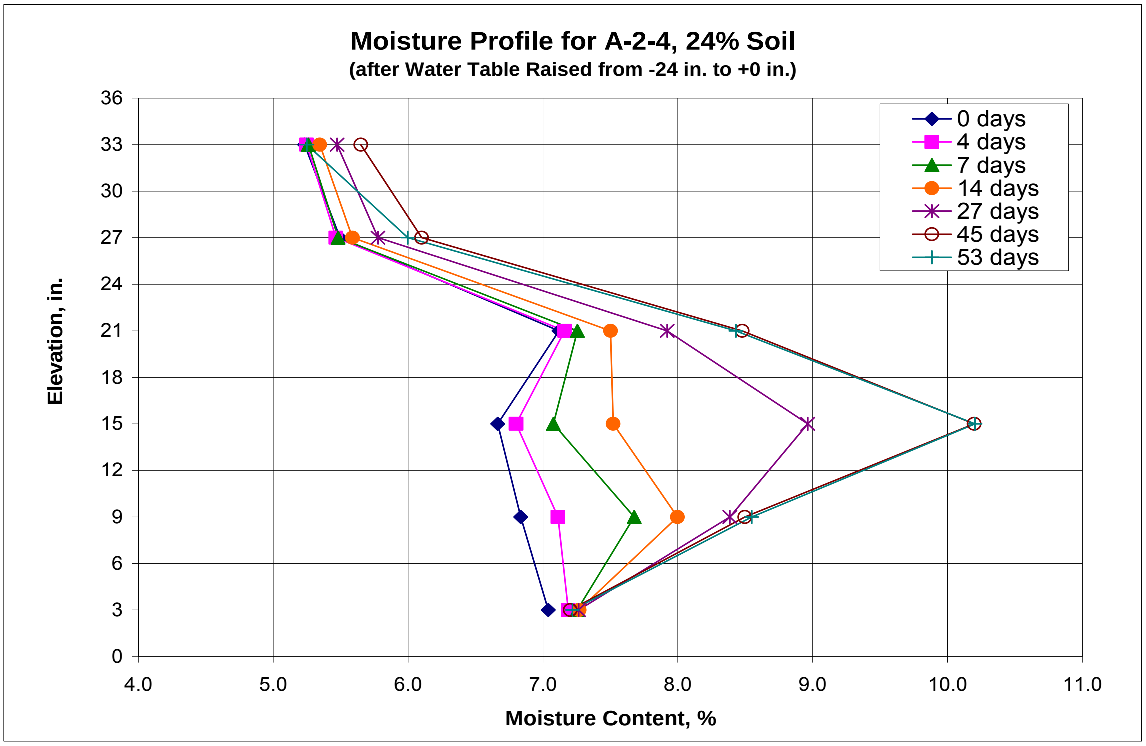


Figure 7.22(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 24% Soil

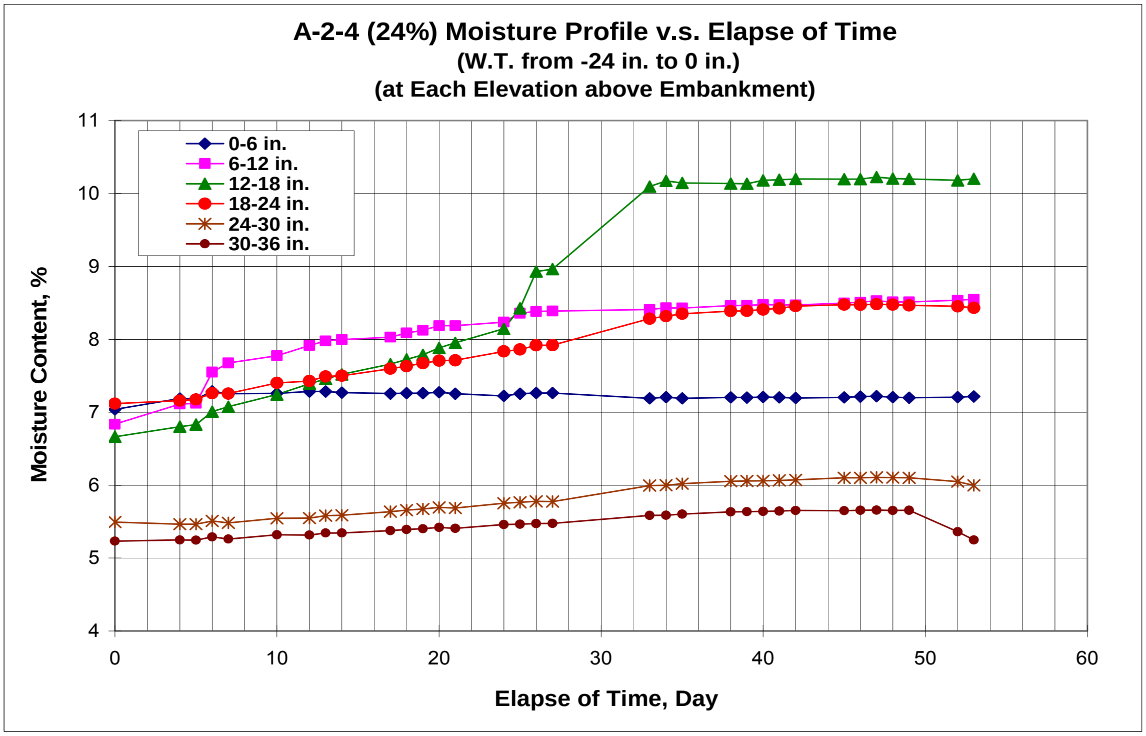


Figure 7.22(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 24% Soil

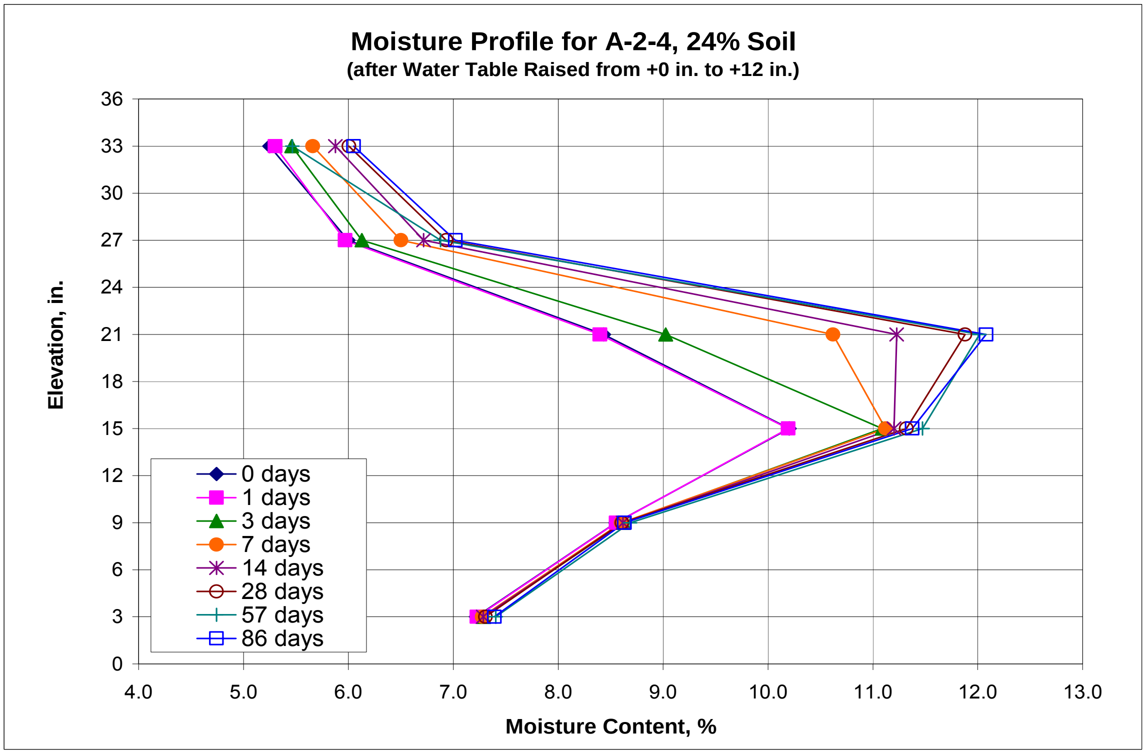


Figure 7.23(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 24% Soil

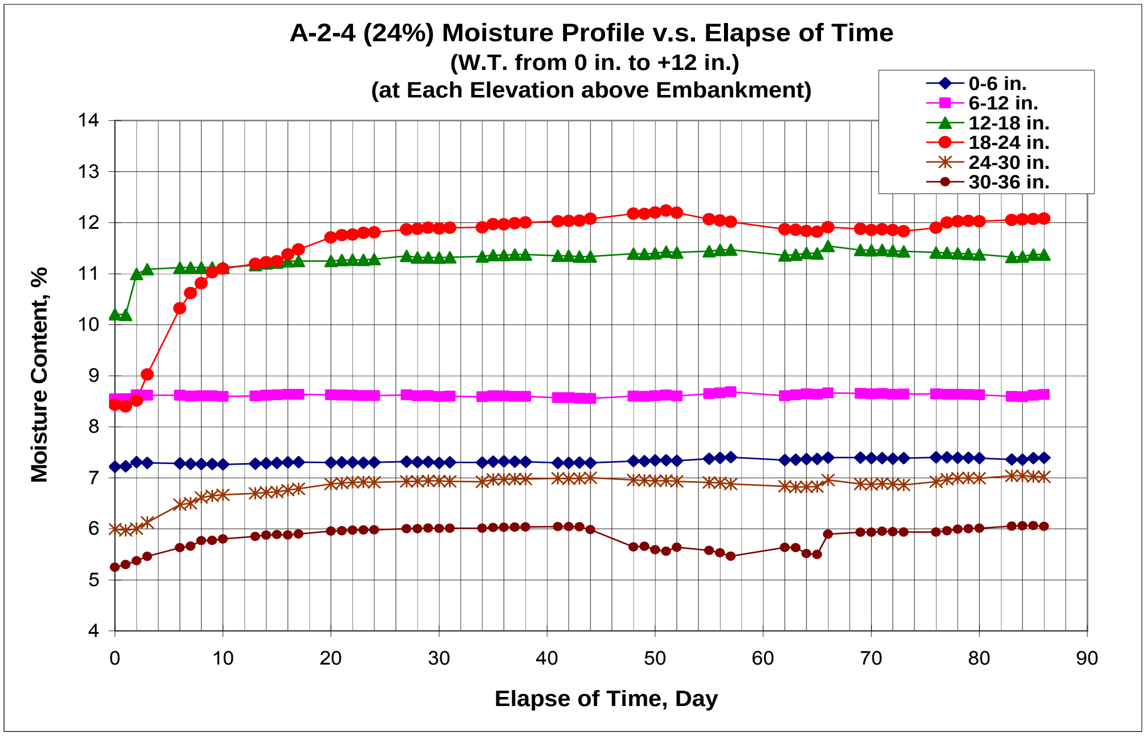


Figure 7.23(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 24% Soil

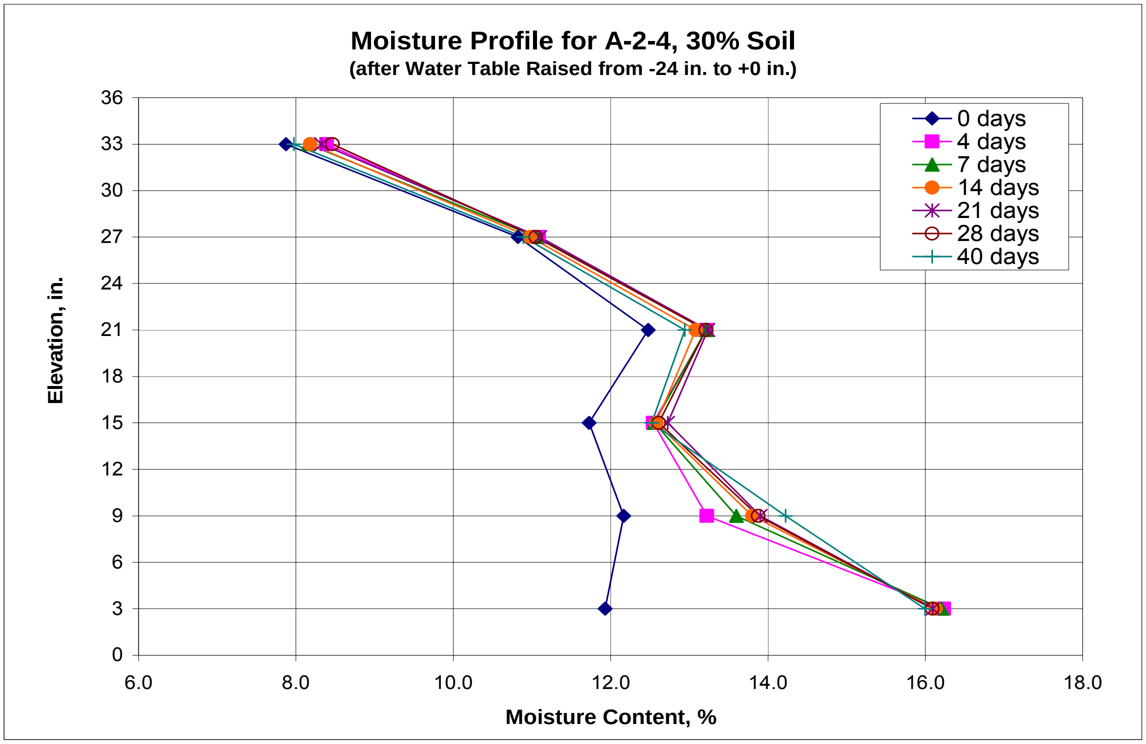


Figure 7.24(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 30% Soil

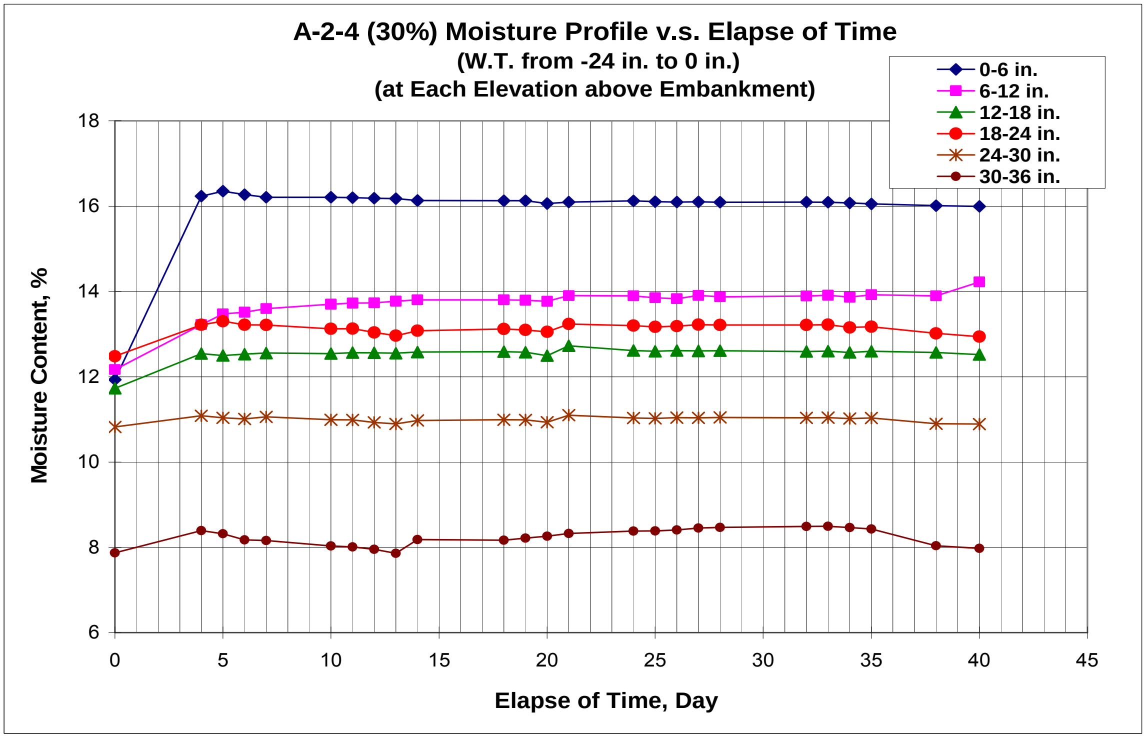


Figure 7.24(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for A-2-4, 30% Soil

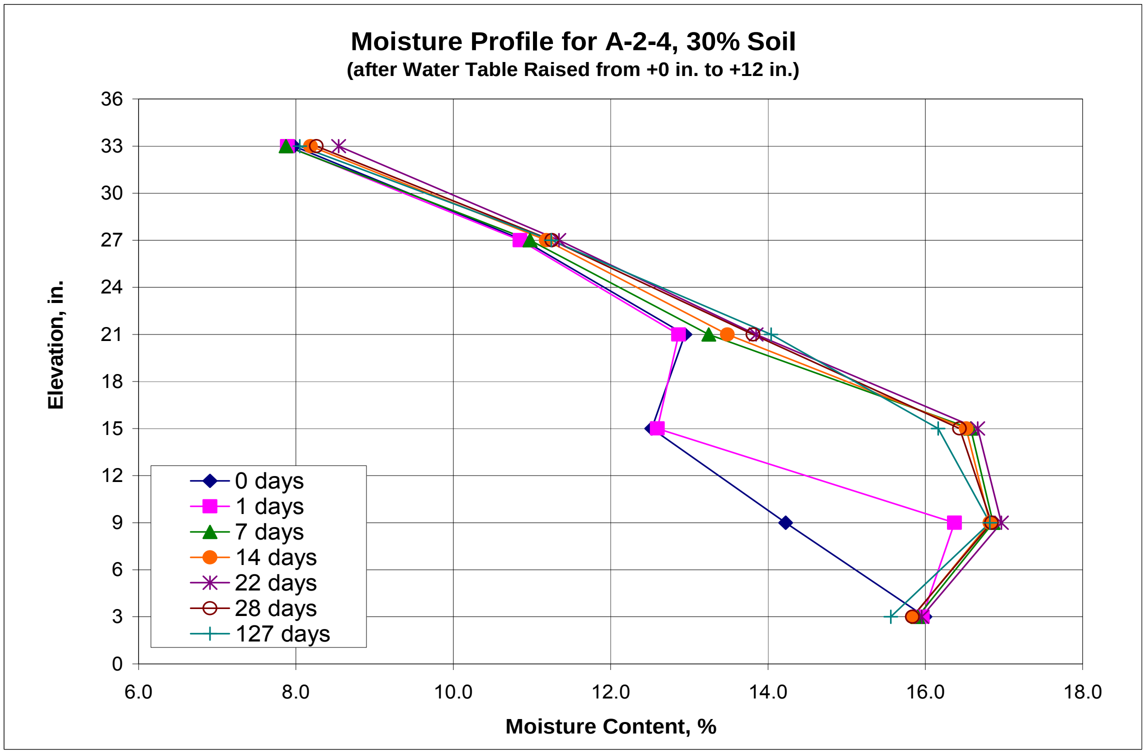


Figure 7.25(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 30% Soil

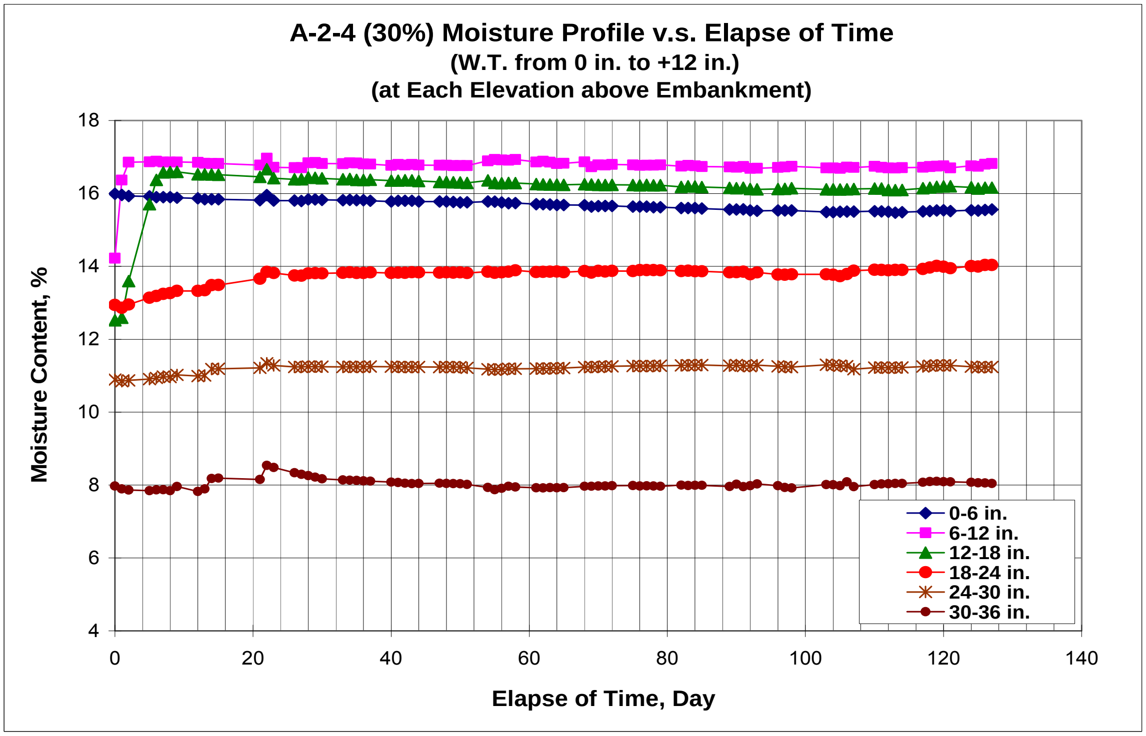


Figure 7.25(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for A-2-4, 30% Soil

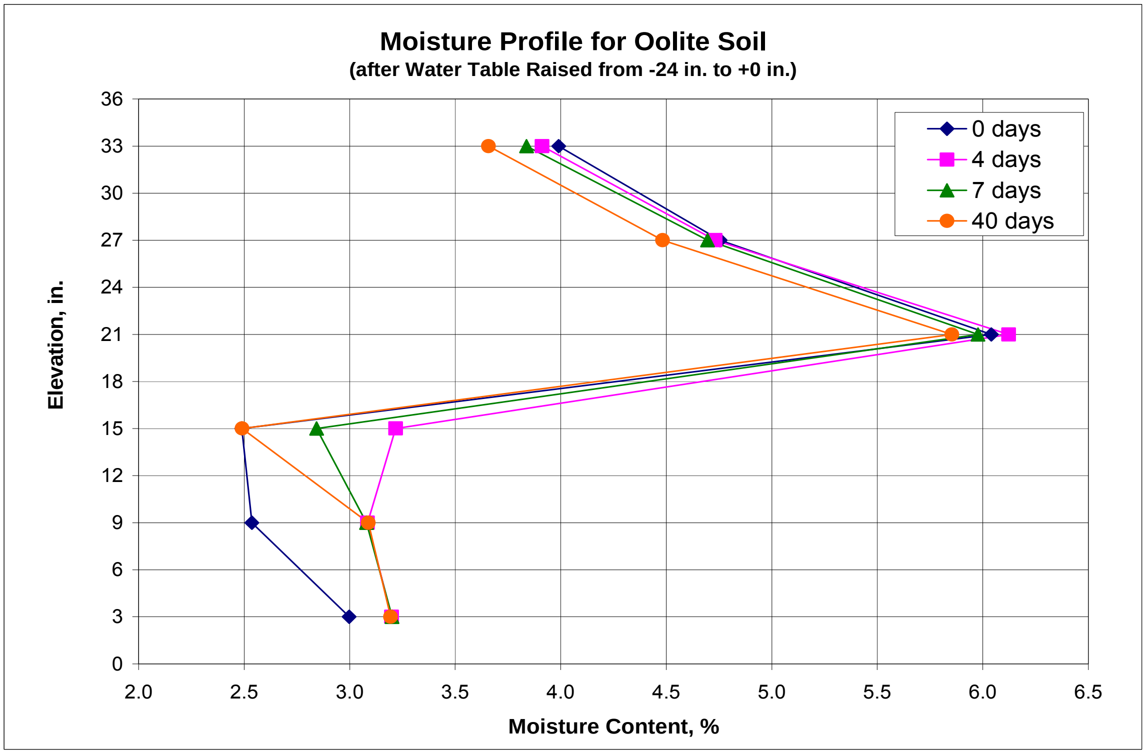


Figure 7.26(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for Miami Oolite A-1 Soil

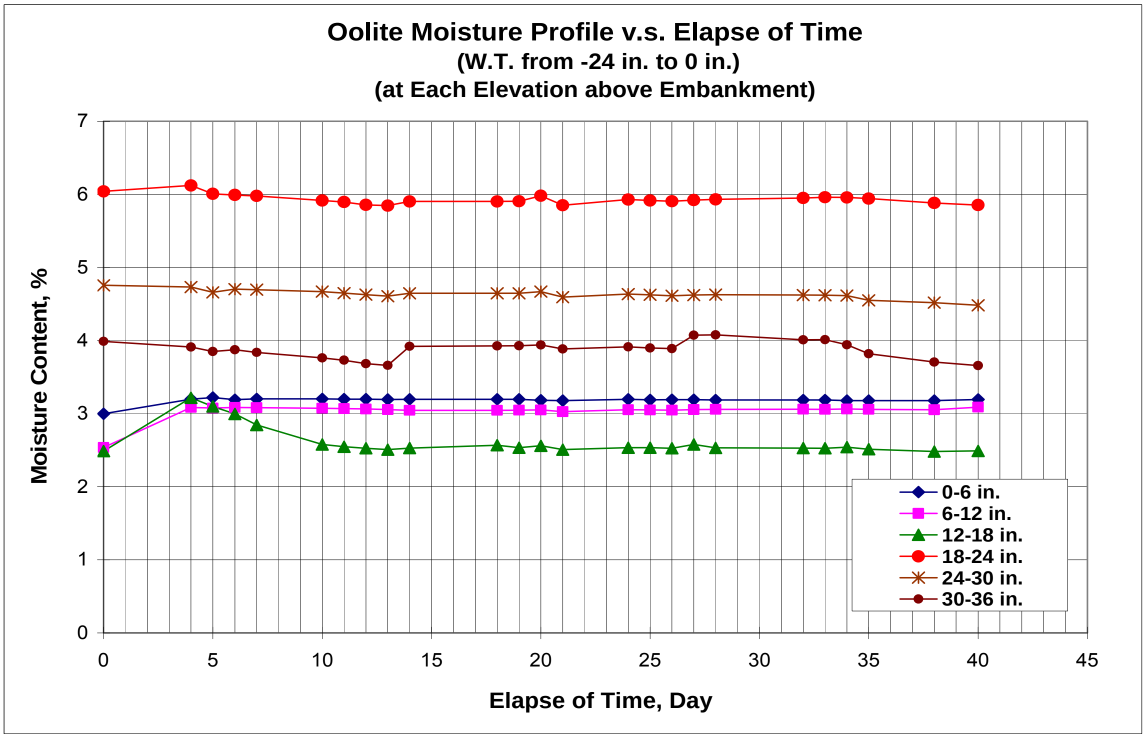


Figure 7.26(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for Miami Oolite A-1 Soil

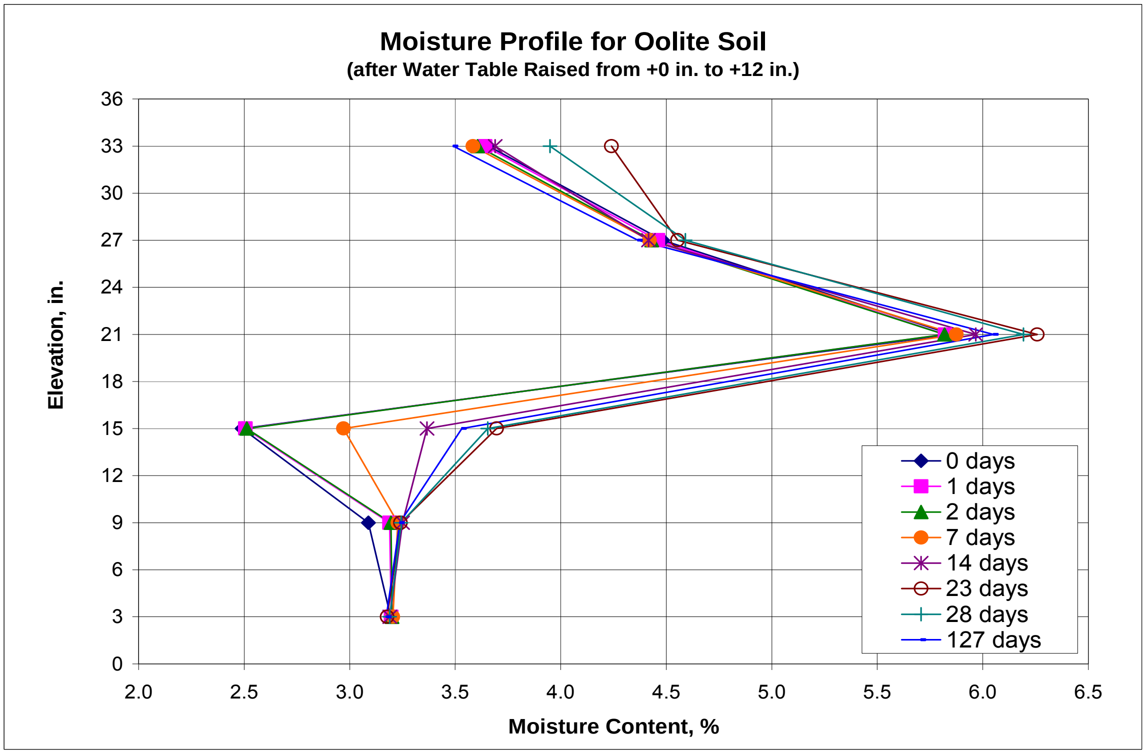


Figure 7.27(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for Miami Oolite A-1 Soil

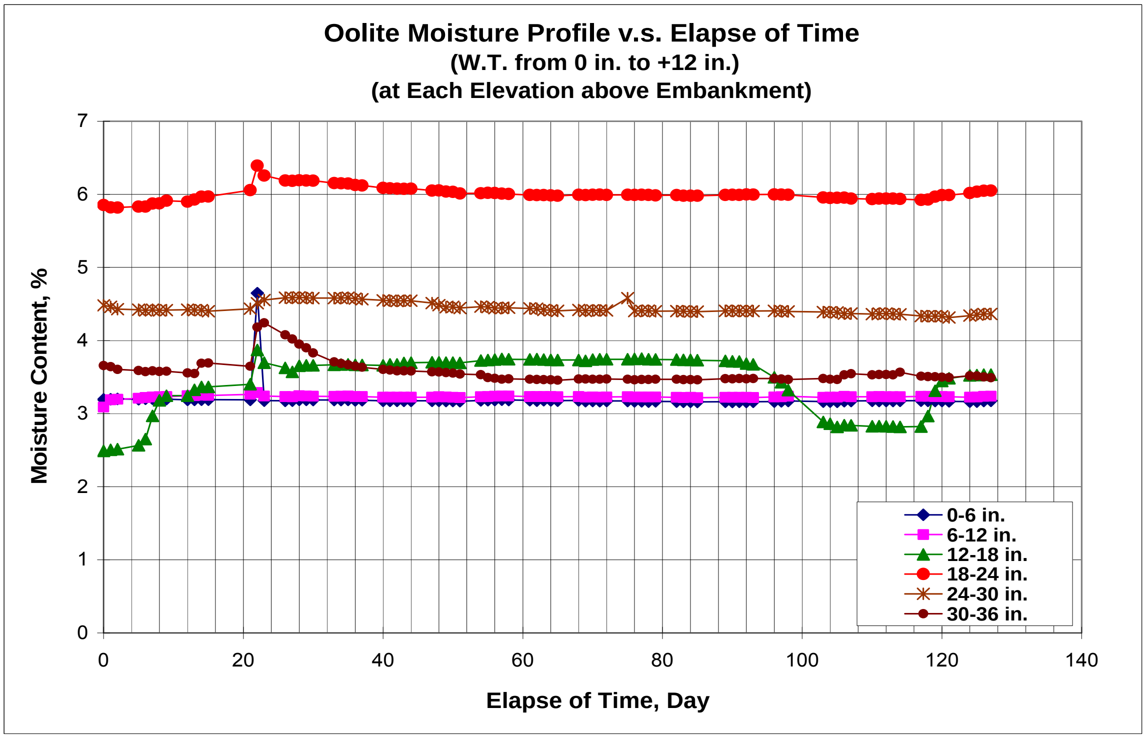


Figure 7.27(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Miami Oolite A-1 Soil

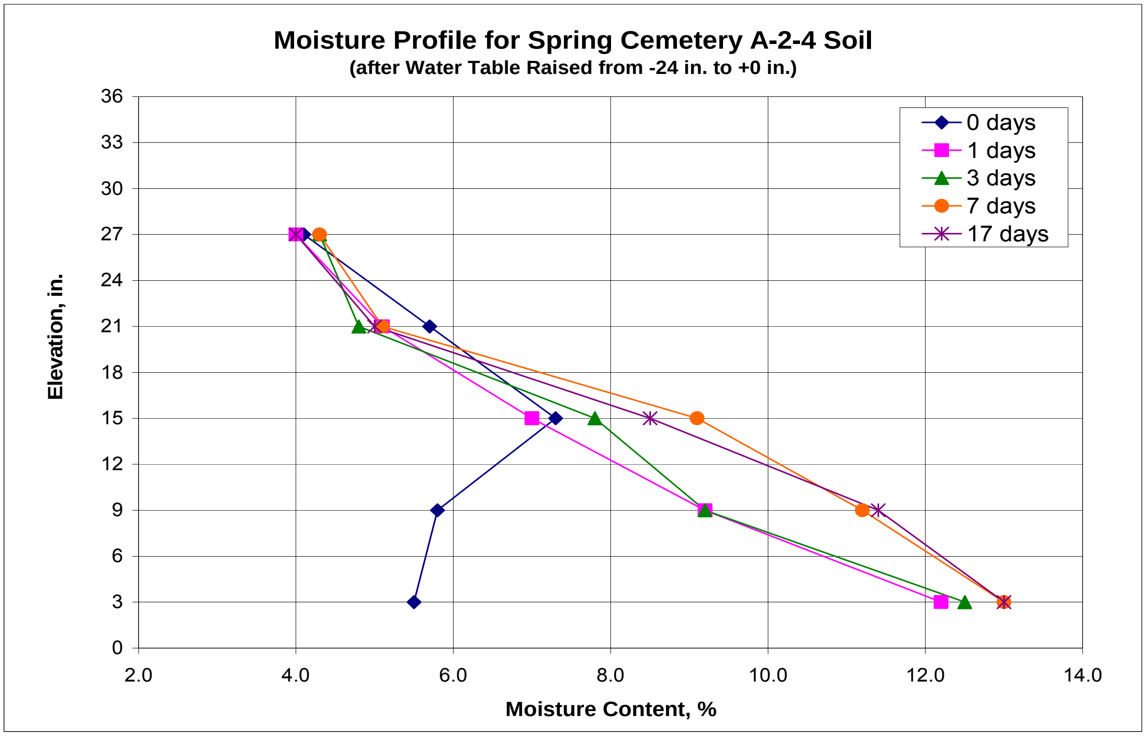


Figure 7.28(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for Spring Cemetery A-2-4 Soil

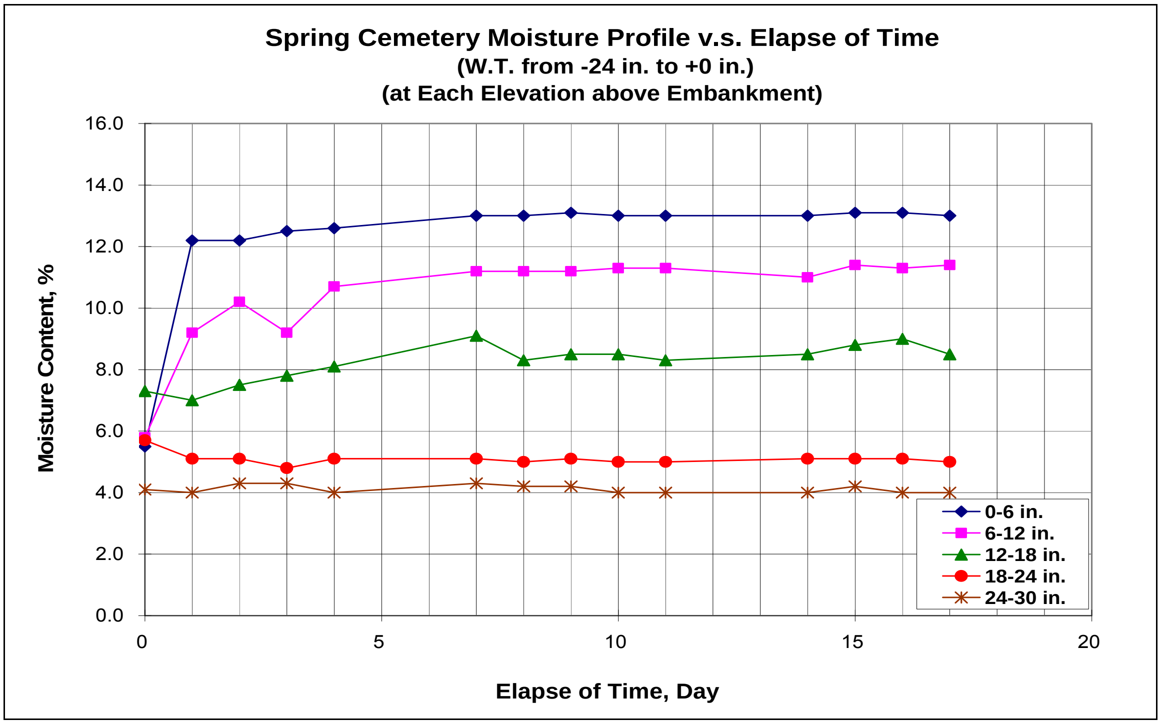


Figure 7.28(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for Spring Cemetery A-2-4 Soil

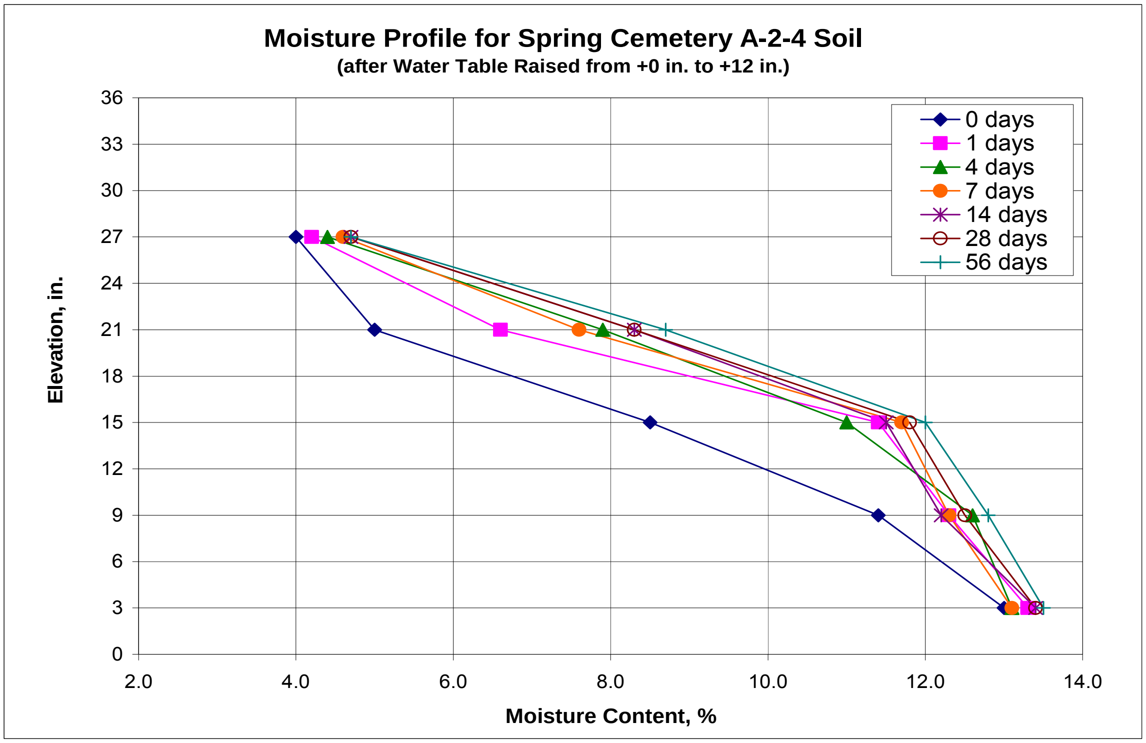


Figure 7.29(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for Spring Cemetery A-2-4 Soil

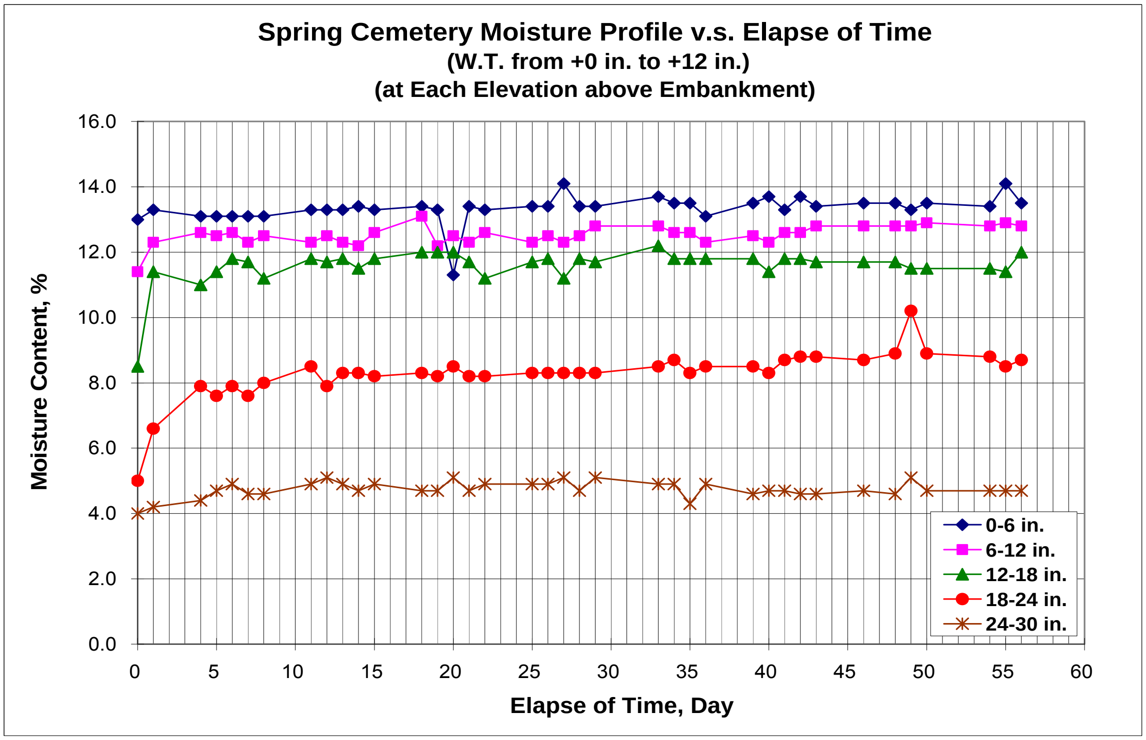


Figure 7.29(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Spring Cemetery A-2-4 Soil

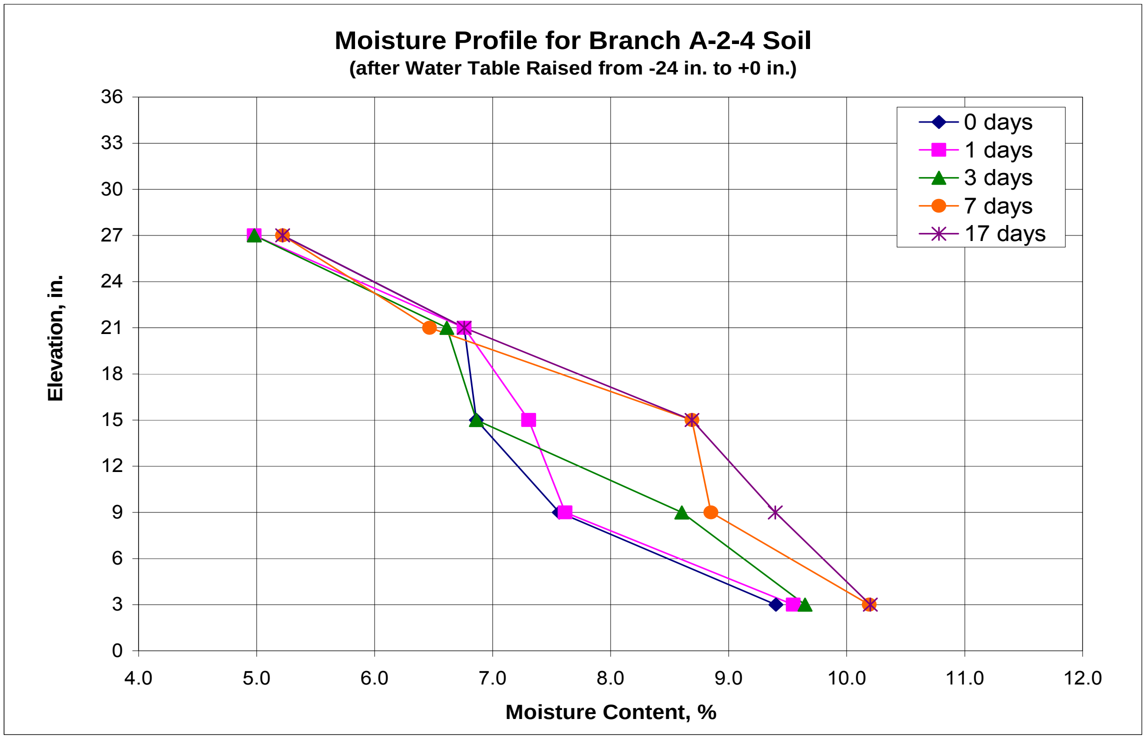


Figure 7.30(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for Branch A-2-4 Soil

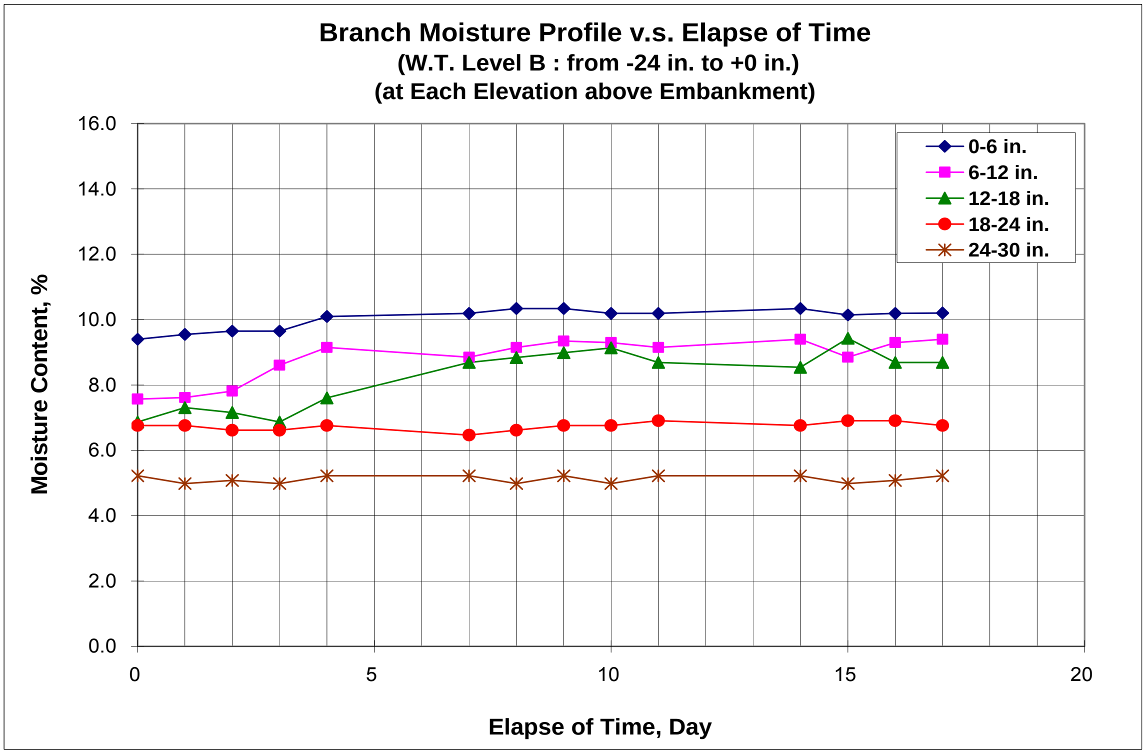


Figure 7.30(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for Branch A-2-4 Soil

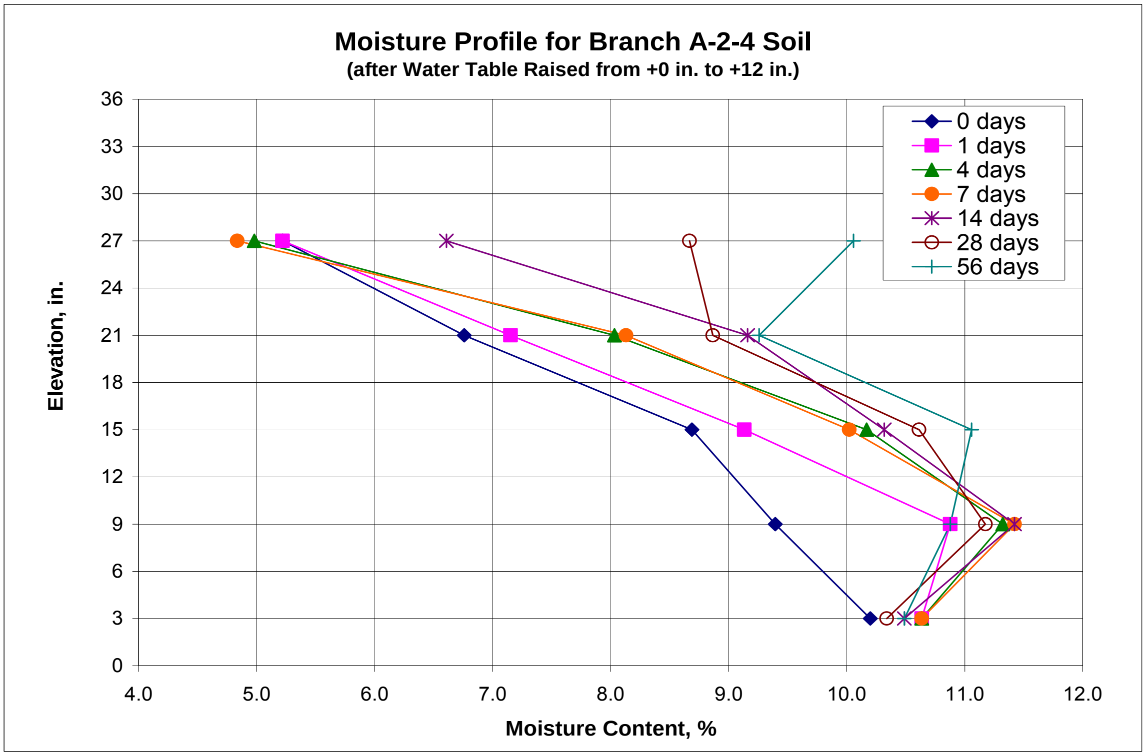


Figure 7.31(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for Branch A-2-4 Soil

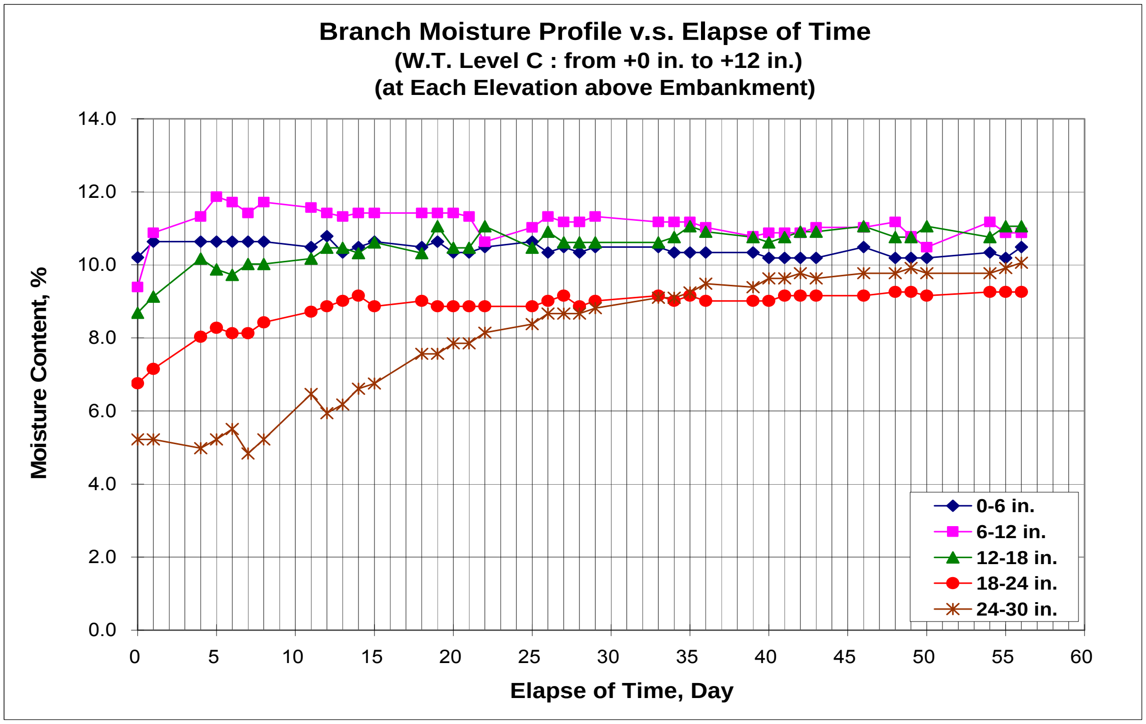


Figure 7.31(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Branch A-2-4 Soil

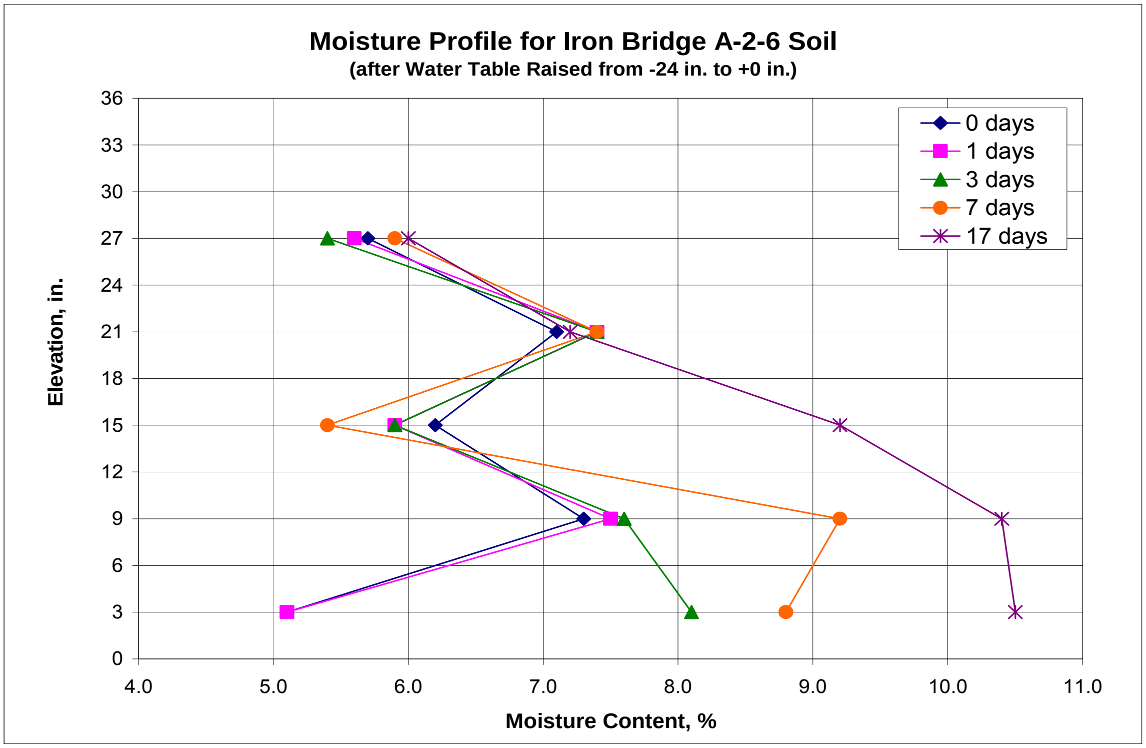


Figure 7.32(A) Moisture Profile after Groundwater Level Raised from -24 in. to 0 in. for Iron Bridge A-2-6 Soil

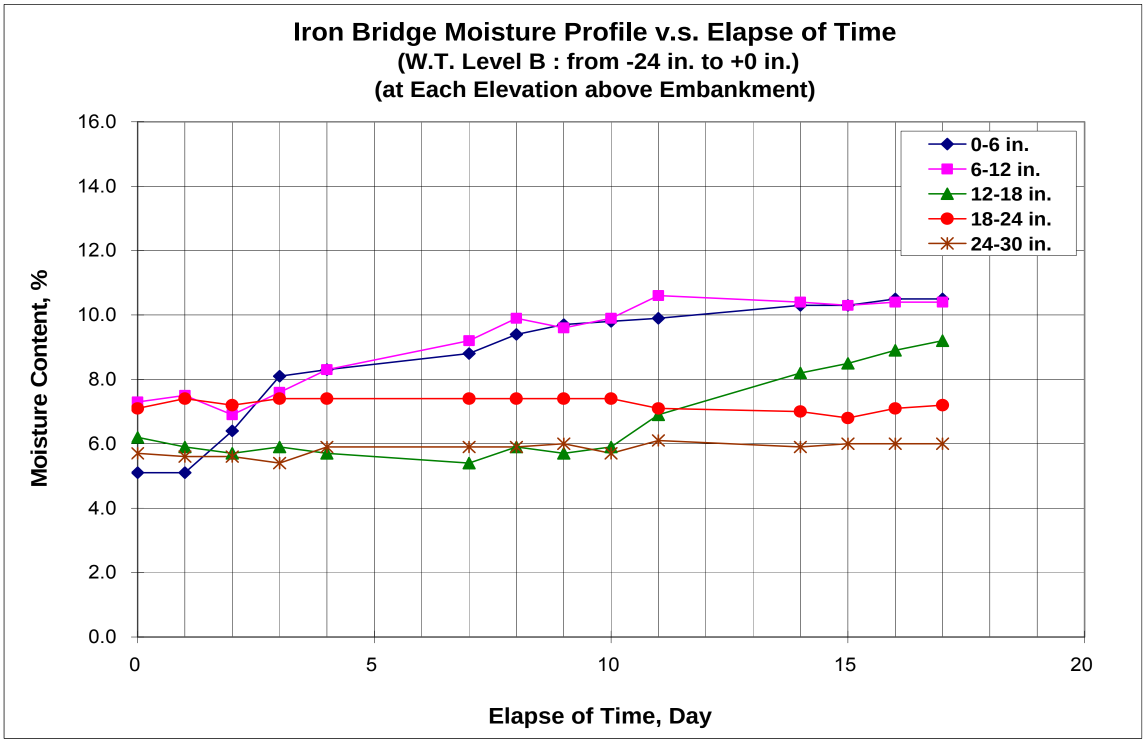


Figure 7.32(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from -24 in. to 0 in. for Iron Bridge A-2-6 Soil

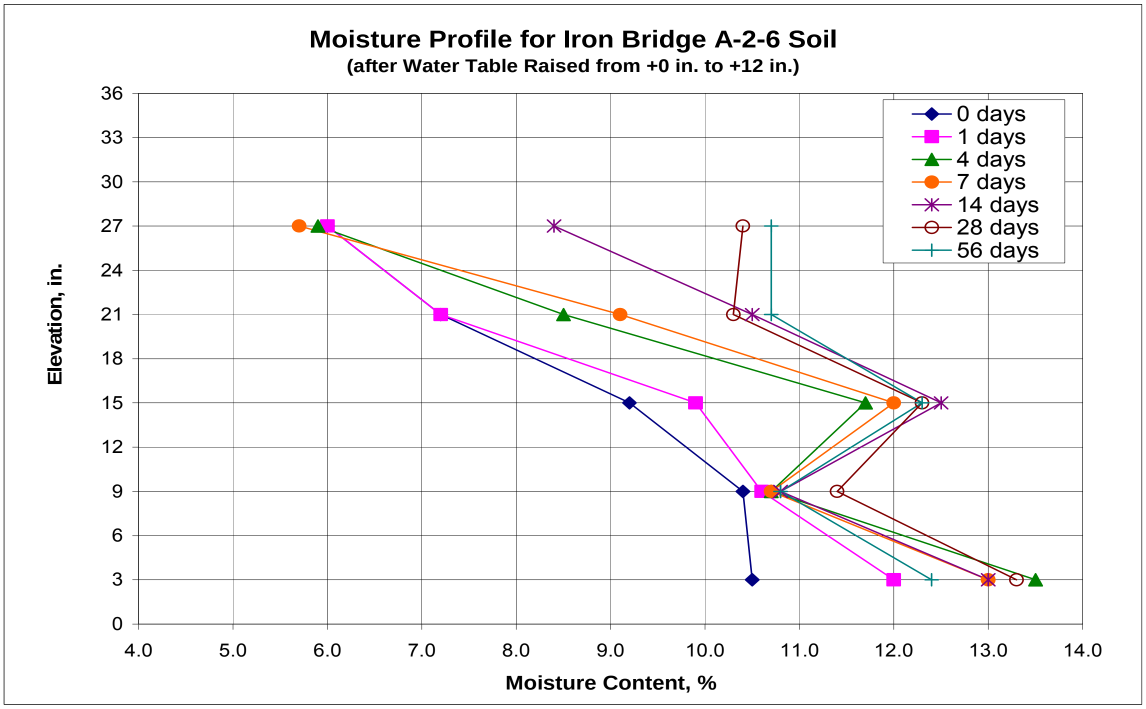


Figure 7.33(A) Moisture Profile after Groundwater Level Raised from +0 in. to +12 in. for Iron Bridge A-2-6 Soil

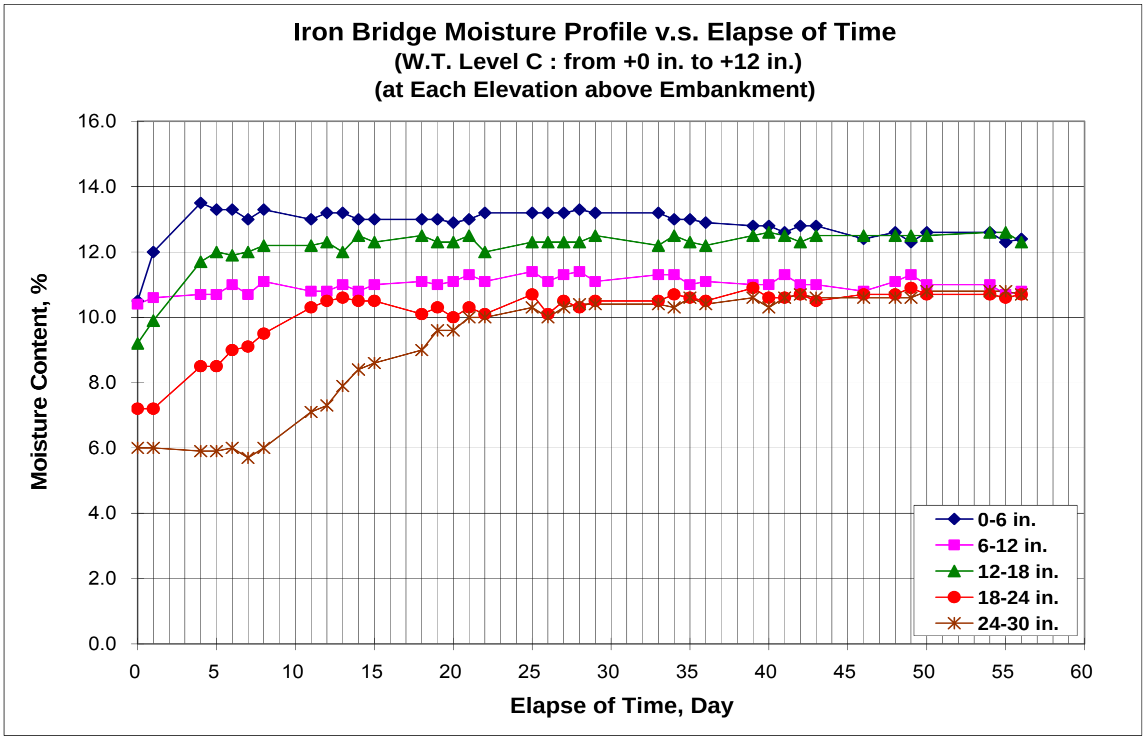


Figure 7.33(B) Moisture Profile vs. Elapsed Time after Groundwater Level Raised from +0 in. to +12 in. for Iron Bridge A-2-6 Soil

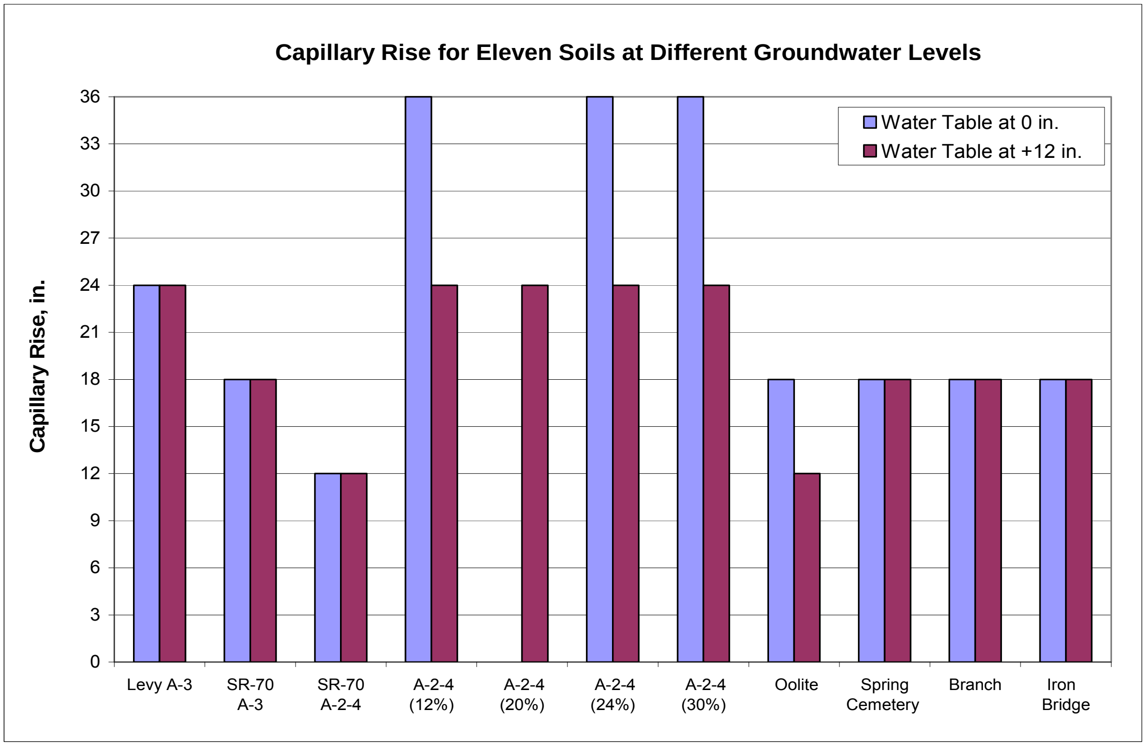


Figure 7.34 Capillary Rise for Eleven Soils at Different Groundwater Levels

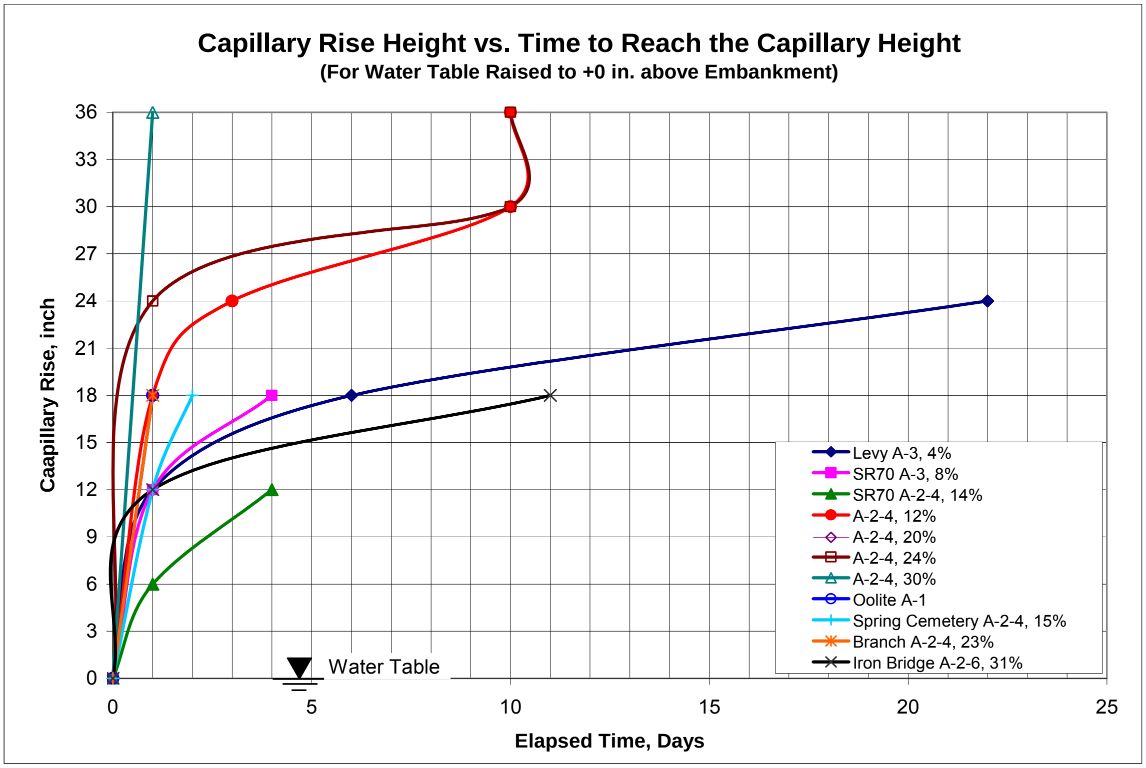


Figure 7.35 Rate of Capillary Rise for Eleven Soils with Groundwater Level at 0 in.

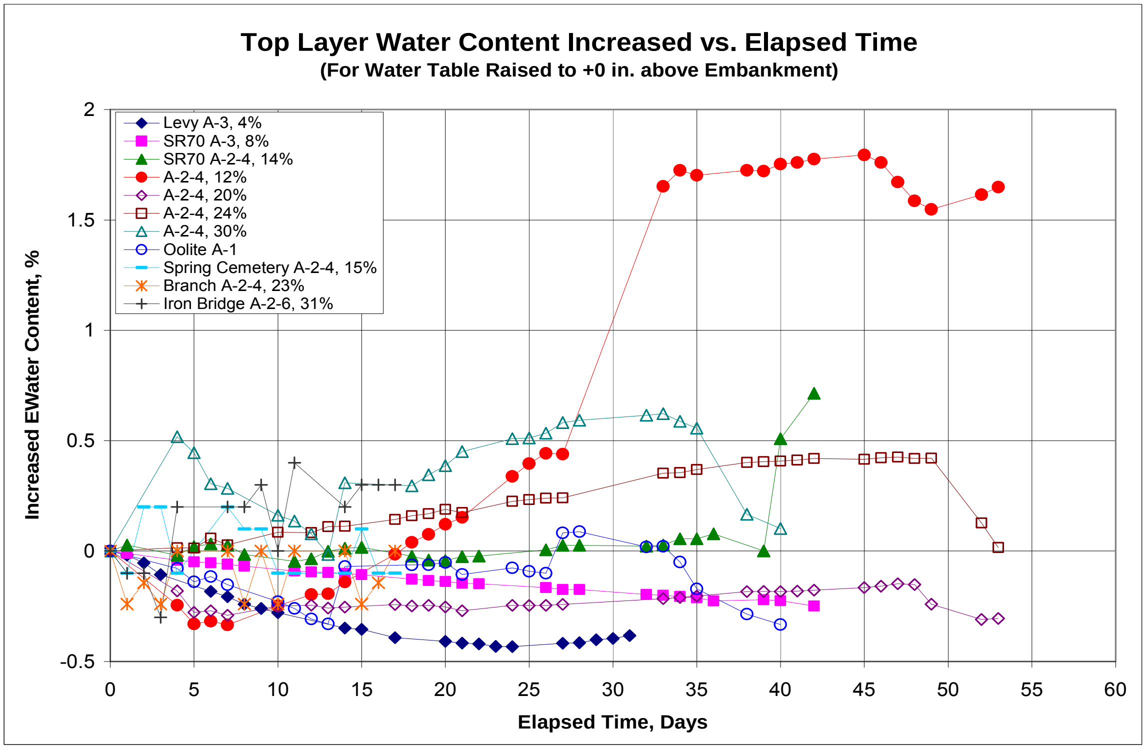


Figure 7.36 Increased Water Content of the Top Layer for Eleven Soils with Water Table at +0 in. above the Embankment

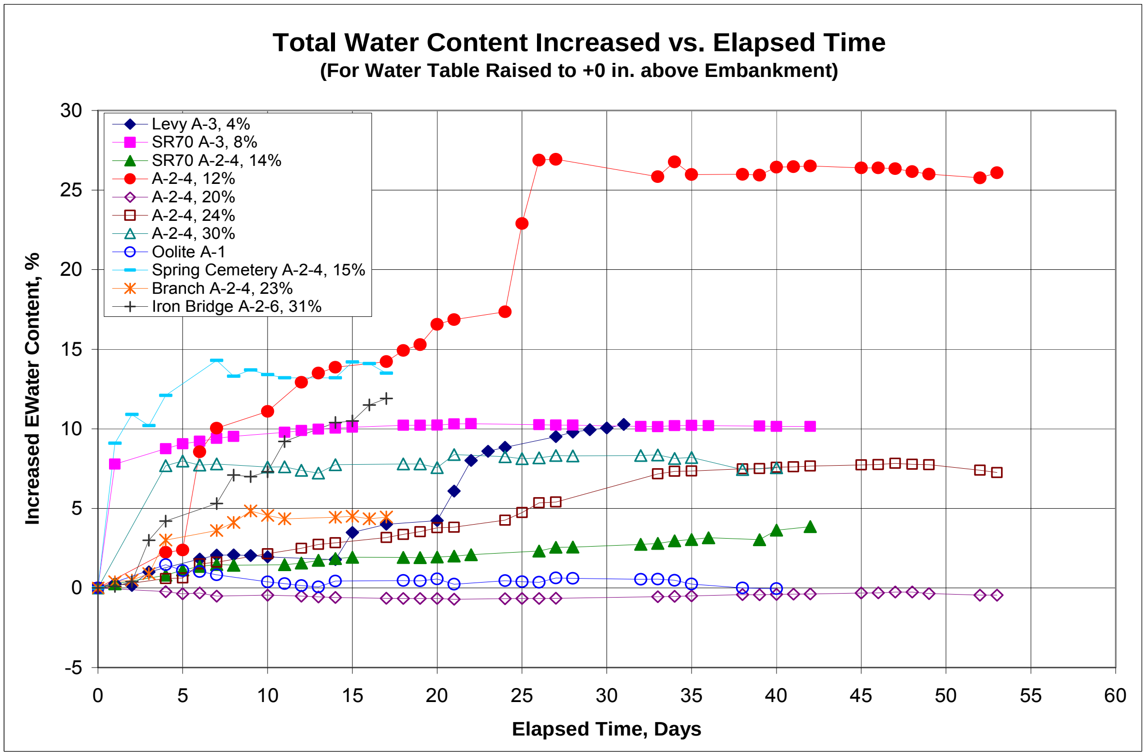


Figure 7.37 Total Increased Water Content for Eleven Soils with Water Table at +0 in. above the Embankment

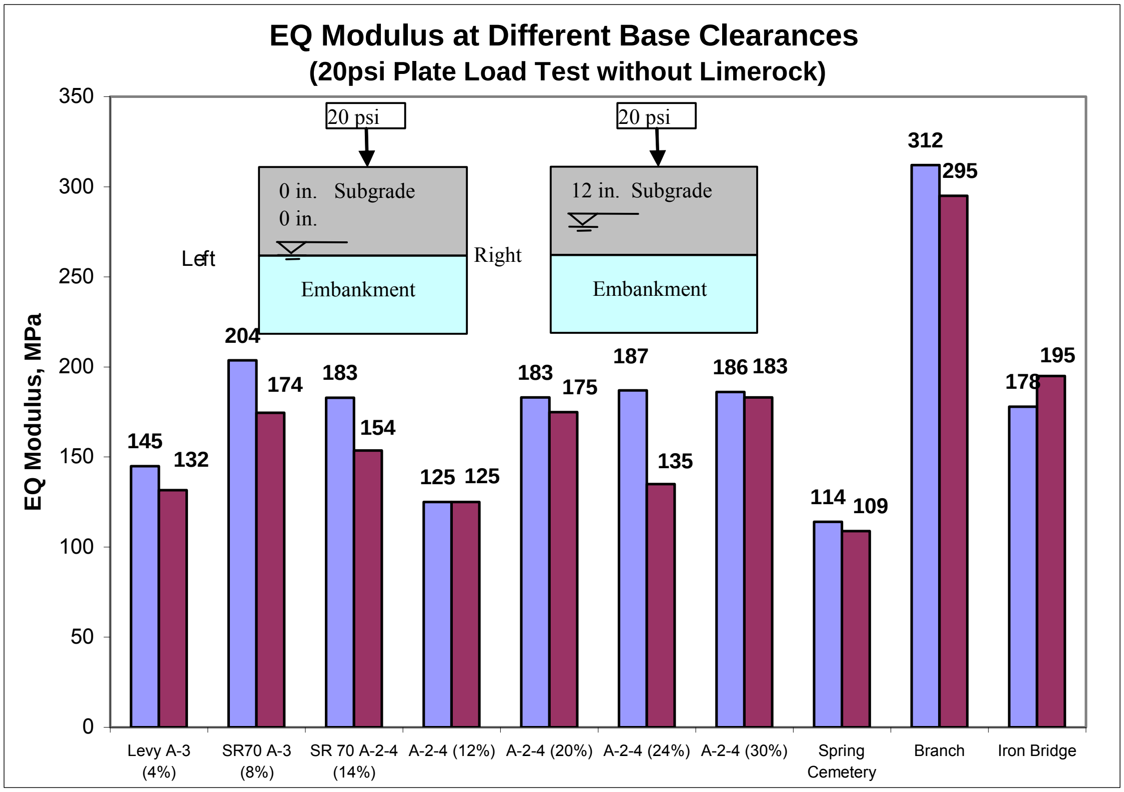


Figure 7.38 EQ Modulus at Different Base Clearances (20 psi without Limerock)

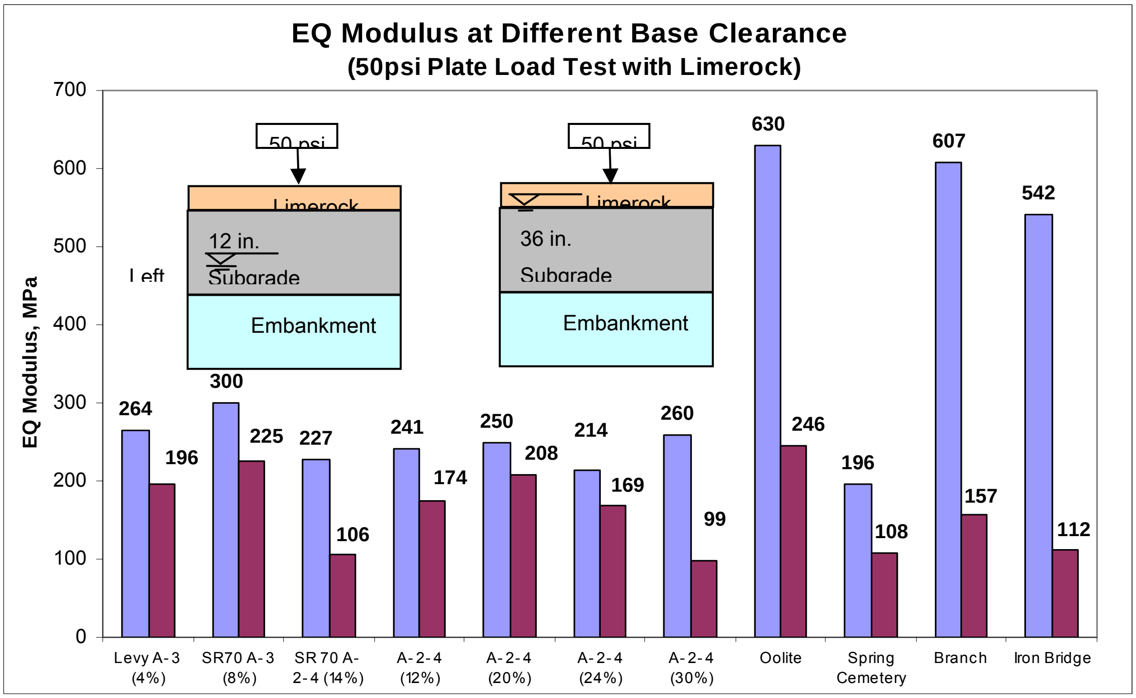


Figure 7.39 EQ Modulus at Different Base Clearances (50 psi with Limerock)

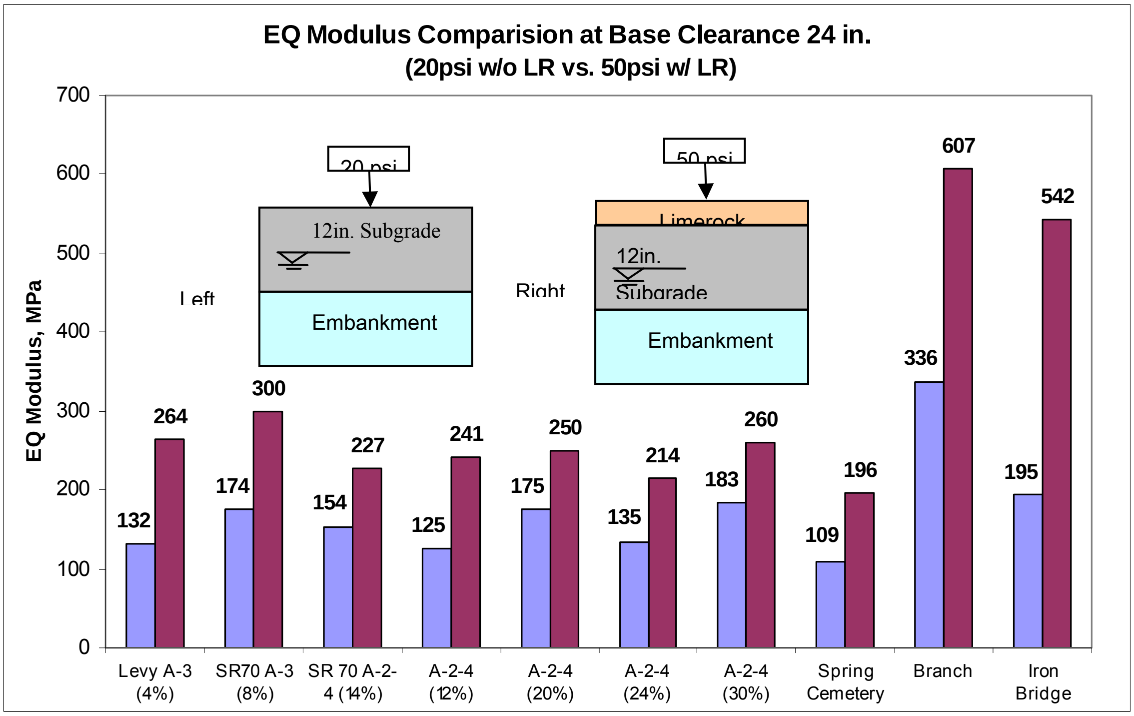


Figure 7.40 EQ Modulus Comparisons at Base Clearance 2 ft (20 psi w/o Limerock vs. 50 psi w/ Limerock)

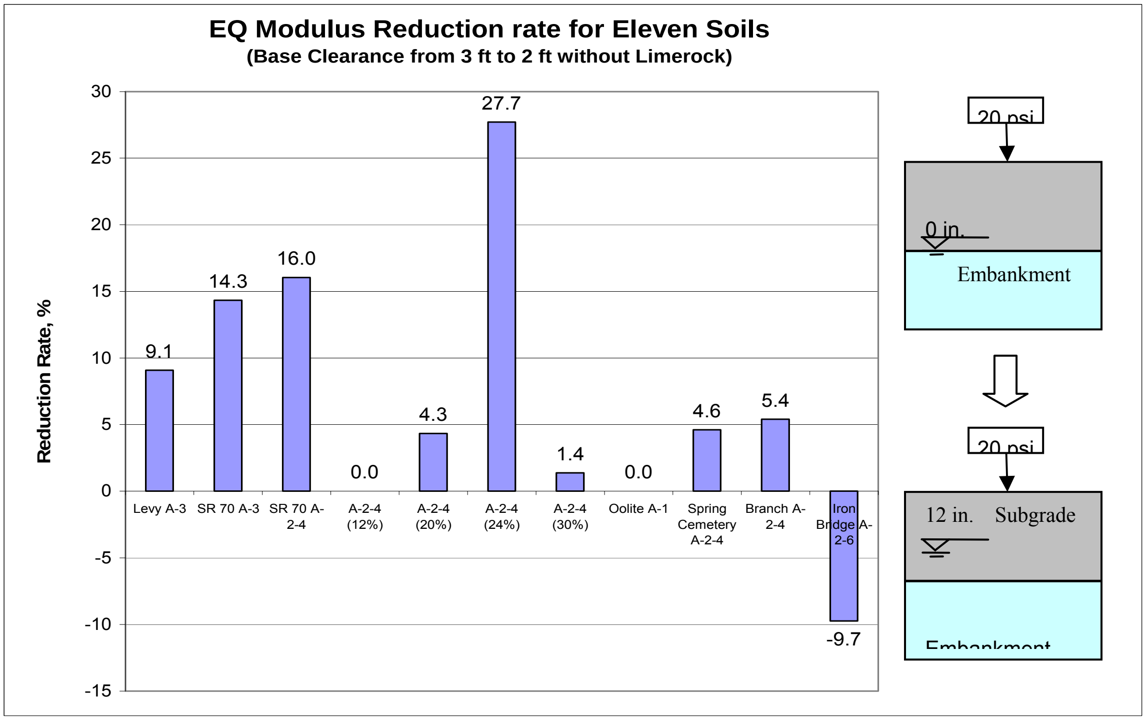


Figure 7.41 EQ Modulus Reduction Rate for Base Clearance from 3 ft to 2 ft (20 psi without Limerock)

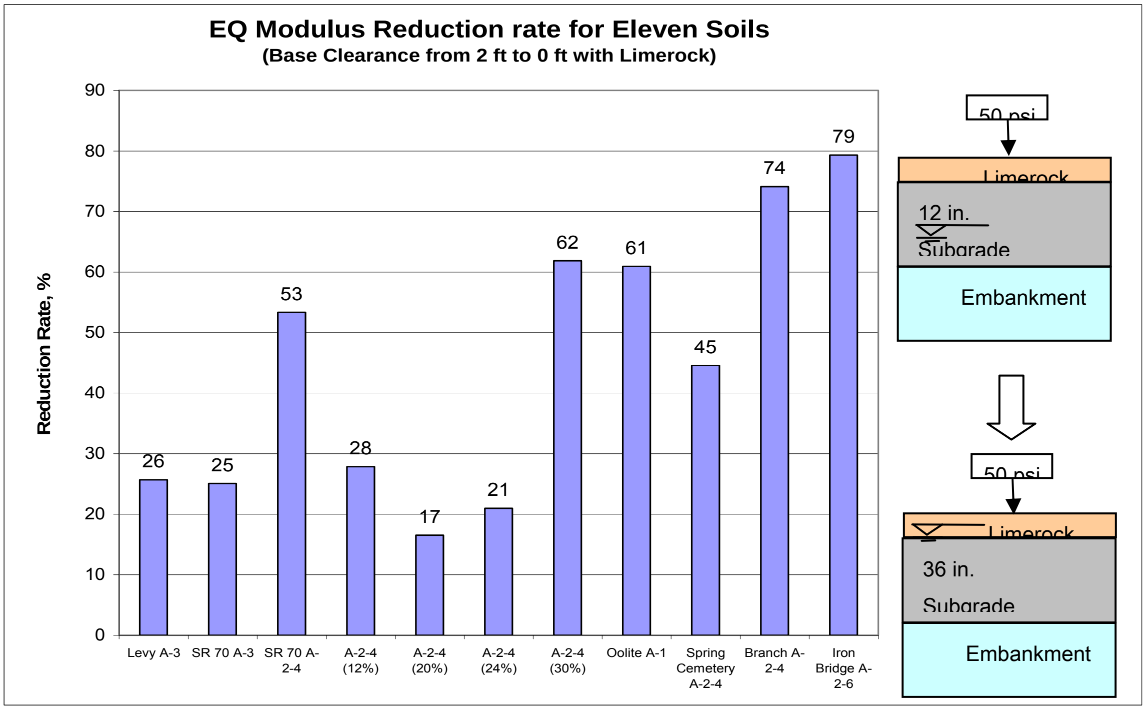


Figure 7.42 EQ Modulus Reduction Rate for Base Clearance from 2 ft to 0 ft (50 psi with Limerock)

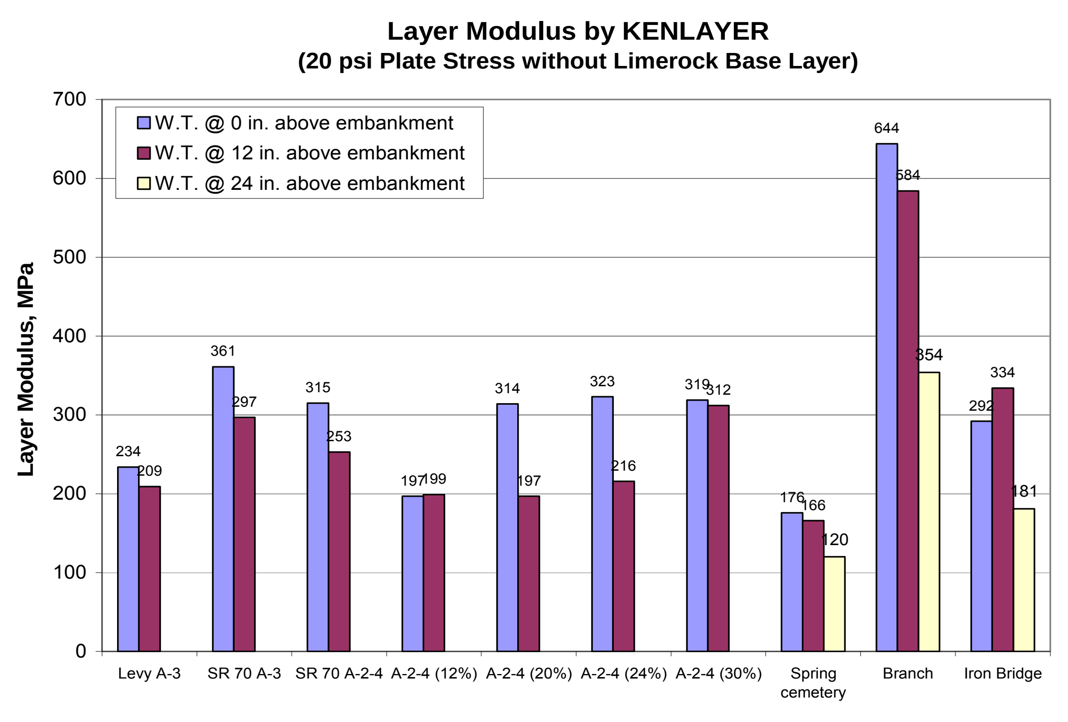


Figure 7.43 Summary of Test-pit Layer Moduli at Different Groundwater Levels (20 psi without Limerock Layer)

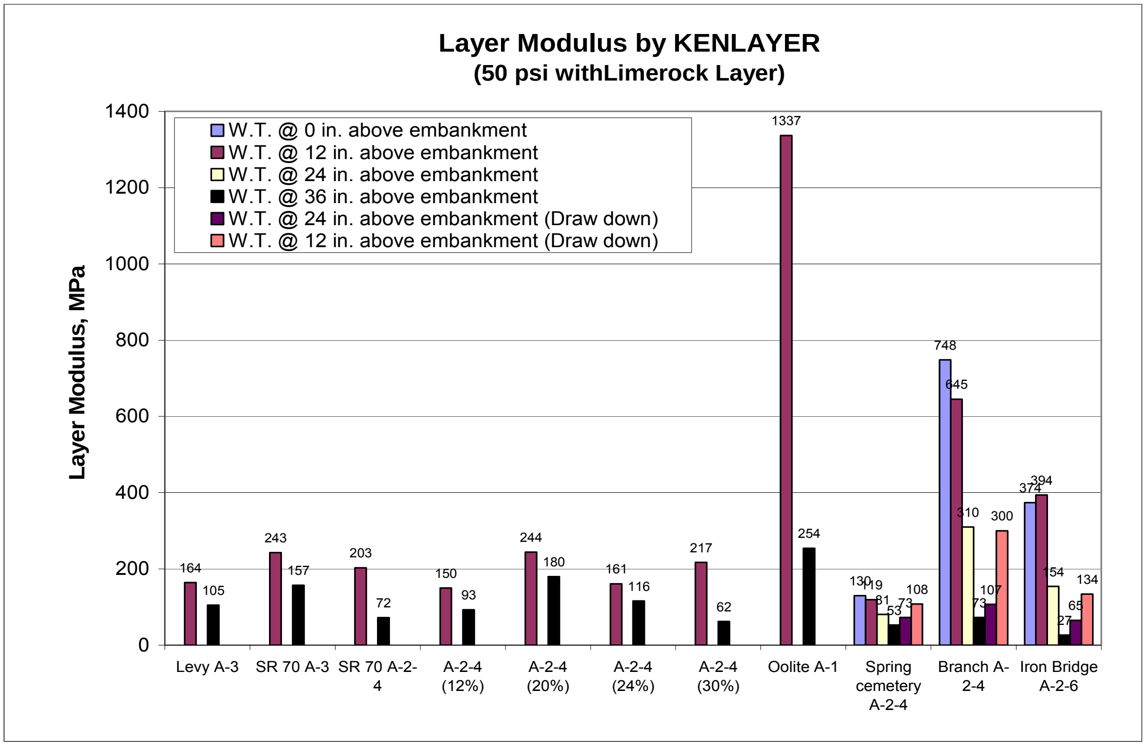


Figure 7.44 Summary of Test-pit Layer Moduli at Different Groundwater Levels (50 psi with Limerock layer)

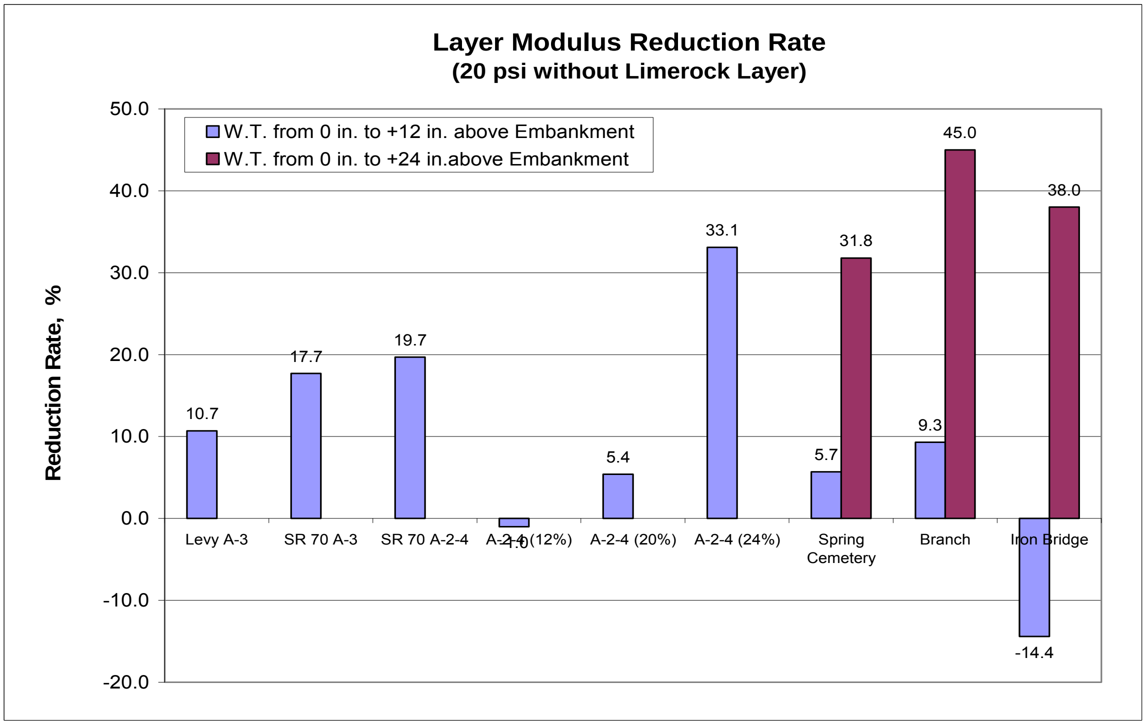


Figure 7.45 Reduction Rates of Test-pit Layer Moduli at Different Base Clearances (20 psi without Limerock Layer)

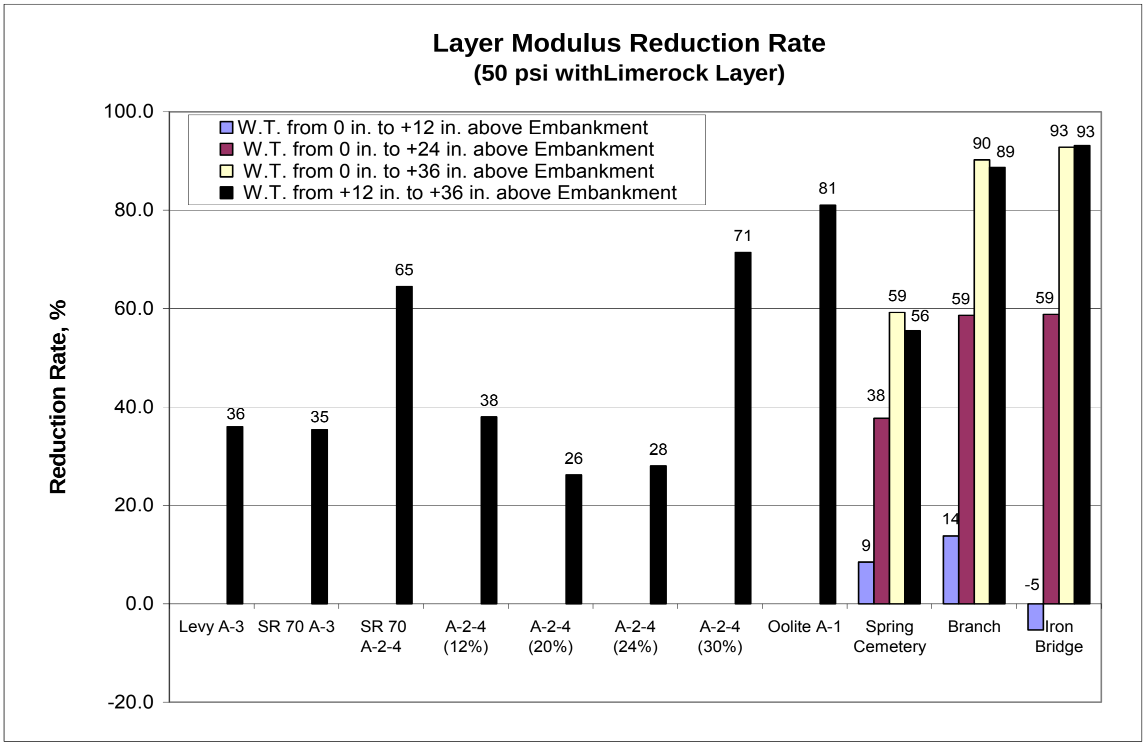


Figure 7.46 Reduction Rates of Test-pit Layer Moduli at Different Base Clearances (50 psi with Limerock Layer)

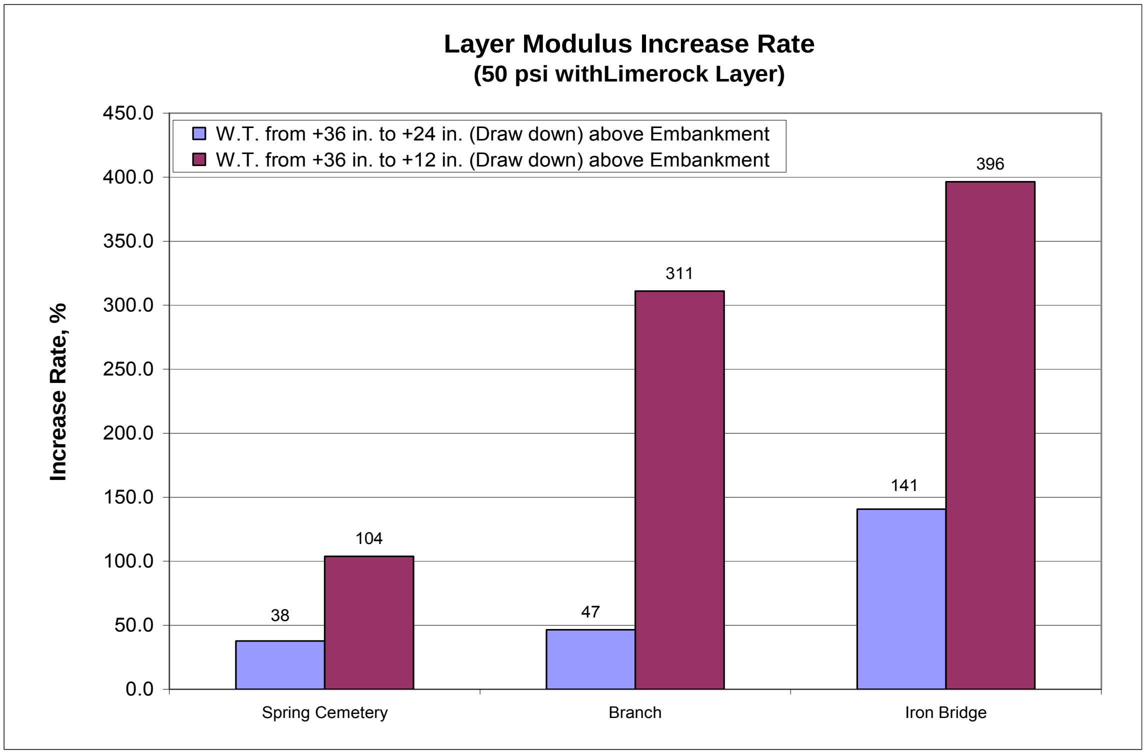


Figure 7.47 Increase Rates of Test-pit Layer Moduli at Draw-down Conditions (50 psi with Limerock Layer)

CHAPTER 8
CASE STUDY AND FIELD MONITOR PROGRAM

8.1 CASE STUDY FOR HIGH GROUNDWATER EFFECT

The practical significance of designing pavements with base clearances is to optimize the thickness of the pavement layers above the high groundwater level including a structural asphalt concrete layer satisfying both the economical and safety designs. A case study utilizing the measured equivalent modulus data to design the required thickness of flexible pavement layer with respect to different high groundwater levels would help to develop insight into the economic aspect of importance for such base clearances. The schemes for this case study using the measured equivalent modulus in test-pit tests are illustrated in Figures 8.1 and 8.2.

The AASHTO Guide for Design of Pavement Structures (1986 and 1993) was adopted for this case study relative to the change of groundwater table. In this design approach, the effective roadbed soil resilient modulus (M_r) to be used in the AASHTO design equation was taken from the equivalent modulus of composite pavement profile in test-pit tests, which is summarized in Tables 7.29, 7.30, 7.31, 7.32, 7.33, 7.34, and 7.35. Two schemes were studied (Figures 8.1 and 8.2) in the following ways:

1) 20-psi plate loading with the equivalent modulus of the composite section for assumed 5-in. or 10-in. limerock base, 36-in. stabilized subgrade plus embankment

2) 50-psi plate loading with the equivalent modulus of the composite section for 5-in. limerock base, 36-in. stabilized subgrade layer, and embankment

The detailed design source data, assumptions, and procedures are discussed in the following sections.

In this case study, the pavement design includes the eleven soil types as the subgrade to calculate required asphalt concrete thickness.

8.1.1 Traffic Data

Traffic is one of the most important parameters in pavement design. The Florida Department of Transportation (FDOT) collects and stores a broad range of traffic data to assist highway engineers in designing and maintaining safe, state-of-the-art, and cost effective facilities. Traffic data are collected by the Central Office, districts, local governments, and consultants, and include volume and vehicle classification counts, speed surveys, and truck weight measurements. The traffic data are based upon cumulative expected 18-kip (80 kN) equivalent single-axle load (ESAL). In order to calculate accumulated ESAL, Average Annual Daily Traffic (AADT), truck factor and some other traffic factors were needed for Equation 8-1.

The AADT is the estimate of typical daily traffic on a road segment for all the days of the week, over the period of one year. The most critical factor for pavement design is the percentage of trucks using a roadway. The structural design is primarily dependent upon the heavy axle loads generated by commercial truck traffic. The estimated future truck volume is needed for calculating the 18 kip (80 KN) Equivalent Single Axle Loads (ESAL) for pavement design. Design traffic calculations use the factor T, the percentage of trucks for 24 hours (one day).

The 18K ESAL required for pavement design purposes can be computed using the following equation:

$$W_{t18} = AADT \times T_{24} \times D \times LF \times E_{18} \times 365 \qquad (8\text{-}1)$$

Where,

W_{t18} = number of 18-kip (80KN) ESAL in the design lane during a given year

AADT = average annual daily traffic

T_{24} = percentage of heavy trucks, 24 hours

D = directional distribution factor

LF = lane factor, covert directional truck to design lane trucks

E18 = 18K ESAL equivalency factor, the damage caused by one average heavy truck

Since no data were available for the prediction of traffic growth, an annual growth rate of 2% was assumed for calculation of ESALs, based on the experience of the traffic growth rate for the last ten years.

To evaluate the groundwater level (different moisture content condition) effect on the required thickness of the asphalt concrete layer, two traffic levels were used. According to Table 8.1 from Asphalt Institute, an ESAL values of 1.3×10^7 was used to present the traffic condition.

8.1.2 Resilient Modulus Based Design Procedure

The AASHTO Guide for Design of Pavement Structures (AASHTO 1986, 1993) is considered the standard for pavement design using the resilient modulus. In this case study, the AASHTO design equation (Equation 8-2) is introduced to determine the required thickness of asphalt concrete utilizing the composite soil modulus obtained from the plate load test under different groundwater table levels.

$$\log(W_{18}) = Z_R \times S_0 + 9.36\times\log(SN+1) - 0.20 + \frac{\log[\Delta PSI/(4.2-1.5)]}{0.4+1094/(SN+1)^{5.19}} + 2.32\times\log M_R - 8.07 \quad (8\text{-}2)$$

Where,

SN = Structural number required

W_{18} = Number of 18-kip (80KN) ESAL in the design lane during a given year (smaller than W_{t18} to achieve a higher level of reliability)

Z_R = Standard normal deviate

S_0 = Standard deviation

ΔPSI = Change in serviceability

M_R = Effective roadbed soil resilient modulus (psi)

For a design using resilient modulus, 95% of reliability and 0.45 of standard deviation (Standard Normal Deviation –1.645) were selected according to the AASHTO suggested value. The serviceability of a pavement (PSI) is defined as its ability to serve the type of traffic that uses the facility. PSI is the primary measure of serviceability in current use. In this case, a total PSI loss of 1.7 was assumed, and a terminal serviceability level of 2.5 was selected.

For the asphalt concrete layer, the resilient modulus was assumed to be 350 ksi. For the flexible pavement design in the case of 20-psi plate load without limerock, a 5-in. or 10-in. limerock base was assumed above the stabilized subgrade layer, and the resilient modulus was taken as 31 ksi for the limerock layer. The layer coefficients for the asphalt concrete and limerock base were valued as 0.44 and 0.18, respectively, from their resilient modulus.

In this case study, a good drainage was assumed. The percent of time the pavement structure was exposed to moisture levels approaching saturation was 5-25% and the drainage coefficient was set to 1.0. Based upon the required structural number obtained from Equation 8-2, the required thickness of the asphalt concrete layer was determined.

8.1.3 Design Results and Analysis

The results of the required structural number are summarized in Tables 8.2 and 8.3 for the design layers above the tested condition under different groundwater level variations. The required thicknesses of asphalt concrete layer are summarized in Tables 8.4, 8.5, and 8.6 with different loading conditions and water table levels.

Figures 8.3(A), 8.3(B), 8.4(A), and 8.4(B) show that, under 20-psi plate loading condition, the Levy A-3, SR70 A-3, A-2-4(12%), A-2-4(20%), Spring Cemetery A-2-4, Branch A-2-4, and Iron Bridge A-2-6 soils were required very little increase of the asphalt concrete layer (less than 0.5 in.) when the groundwater table was raised from base clearance 3 ft to 2 ft, while the SR70 A-2-4 and A-2-4 (24%) had a higher increase in AC thickness. As seen in Figure 8.3(B), the required layer thickness of asphalt concrete was significantly increased when the base clearance was reduced to 1 ft. The increased AC thickness was about the same when the thickness of the limerock base layer was increased from 5 in. to 10 in.

As shown in Figures 8.5(A) and 8.5(B), the Levy A-3, SR70 A-3, A-2-4(12%), A-2-4(20%), A-2-4(24%) soils did not increase much of the AC thickness (less than 1 in.) when the groundwater level was raised from +12 in. to +36 in. above embankment (i.e., base clearance 2 ft to 0 ft), while the SR70 A-2-4, A-2-4 (30%), Branch, and Iron Bridge had higher increased AC thickness (more than 2.5 in.).

The results of this case study indicated that for some sensitive soil types as the subgrade, an increase of the groundwater table (12 in. or higher above the embankment) would demand a significant increase of the thickness of the asphalt concrete layer in order to have the same quality pavement performance. Thus, the most safe and economical way for the design of pavement is to maintain an adequate base clearance between the groundwater table and the bottom of the base layer, which is essential for fine-grained subgrade materials.

8.2 FIELD MONITORING PROGRAM

The development of moisture within a subgrade material may exert a detrimental effect on the pavement while under the surcharge provided by moving vehicles. The main purpose for the research of design highwater clearances is to evaluate the influence of moisture within the subgrade material upon the soil modulus, so as to recommend an adequate distance of base clearance between the high groundwater table and the bottom of base layer. To achieve this objective, a field-monitoring test evaluating the moisture variations caused by the capillary rise behavior within actual field geologic strata was desirable. Being exposed to the open environment, the climatic factors such as precipitation and atmospheric temperature were introduced into the moisture measurement for SR70 field monitoring program. The critical moisture conditions acquired through the field test can be correlated with the resilient behavior of the same subgrade material sharing the similar moisture profile in a test-pit test, in order to predict the pavement performance. In the two-year monitoring period, due to the road construction and an equipment problem, there is no data record for almost half a year. In the summer season, due to occasional heavy precipitation, the groundwater table will rise in the following days, and then come back to original height.

8.2.1 Field Installation

The field test was conducted at State Road 70 near Fort Pierce, Florida. Two test sites, 300 ft. apart, were selected for the installation of TDR probes. Twelve TDR probes were installed in each excavated test pit, from 0.5 ft. below the asphalt concrete layer down to 6 ft. below the asphalt concrete layer. All TDR probes were connected with a datalogger powered by a solar panel for data acquisition and data storage. The moisture data recorded within the datalogger can be transferred to an indoor terminal through a public telephone by activating PC208W software in the computer. To correlate the moisture condition of the pavement with the climatic factor such as precipitation, a rain gauge was also installed near the test site. The acquisition interval for the precipitation data was fifteen minutes and activated in synchrony with the data logger.

The installation and instrumentation for the field-monitoring program is described in detail in Appendix B. The results of this monitoring program are presented and discussed in Appendix C.

8.2.2 Discussion on Field Monitoring Program

One major question for the moisture measurement in SR70 was to find out to what extent the test-pit study conducted in the laboratory could simulate the practical moisture variation along the pavement profile in the field. In the test-pit test, the groundwater was taken as the only source of moisture within the pavement. In the field monitoring program conducted at State Road 70, both the downward moisture percolation as a result of precipitation and the upward moisture migration (capillary rise) as a result of the groundwater table change were observed. But the moisture increase within the top layer of subgrade (A-3 soil, for both test site No. 1 and No. 2) incurred by precipitation was transient, and the degree of saturation was low according to the field test results. When compared with what was achieved in the test-pit test for the SR70 A-3 soil, with roughly the same moisture content and the degree of saturation resulted from the water table adjustment, the effect of moisture damage on the subgrade stiffness (resilient modulus) was minimal. In addition, the asphalt concrete layer provided protection against the seepage from precipitation.

The A-2-4 soil was not encountered within three feet below the base layer at the test sites. The moisture resulting from climatic change in the A-2-4 soil layer fluctuated in a way quite similar to what was observed in the test-pit test subjected to groundwater table adjustments. The moisture variation of the A-2-4 soil at SR70 was relatively small compared with the A-3 soil. However the A-2-4 soil layer (with some organic content) existed between 3.5 ft. and 4.5 ft. below the asphalt concrete layer at test site No. 2, and functioned as a barrier for both the downward and upward migration of moisture. The effect of hysterisis was quite obvious for the SR70 A-2-4 soil due to high percentage of fines and higher soil suction. Discussions on the field monitoring results are presented in Appendix C.

Table 8.1 Traffic Classification

Traffic class	Type of street or highway	Range of heavy trucks expected in design period	ESAL
I	Parking lots, driveways Light traffic residential streets Light traffic farm roads	Less than 7000	5×10^3
II	Residential streets Rural farm and residential roads	7000 to 15,000	10^4
III	Urban minor collector streets Rural minor collector roads	70,000 to 150,000	10^5
IV	Urban minor arterial and light industrial streets Rural major collector and minor arterial highways	700,000 to 1,500,000	10^6
V	Urban freeways, expressways, and other principal arterial highways Rural interstate and other principal arterial highways	2,000,000 to 4,500,000	3×10^6
VI	Urban interstate highways Some industrial roads	7,000,000 to 15,000,000	10^7

Note: Whenever possible, more rigorous traffic analysis should be used for roads and streets in traffic category IV or higher. (Source: Asphalt Institute, 1981b)

Table 8.2 Required Structural Number for the Layer above Tested Subgrade Layers (Plate Load 20 psi)

Required Structural Number under Plate Load Test (20psi) without Limerock			
Traffic Data (ESAL)	1.30E+07		
W.T. above Embankment	0 in.	12 in.	24 in.
Levy County A-3 (4%)	3.80	3.94	-
SR70 A-3 (8%)	3.34	3.54	-
SR70 A-2-4 (14%)	3.48	3.72	-
A-2-4 (12%)	4.01	4.00	-
A-2-4 (20%)	3.48	3.54	-
A-2-4 (24%)	3.45	3.90	-
A-2-4 (30%)	3.46	3.61	-
Oolite A-1	-	-	-
Spring Cemetery A-2-4	4.15	4.22	4.65
Branch A-2-4	2.84	2.90	3.33
Iron Bridge A-2-6	3.52	3.40	4.09

Table 8.3 Required Structural Number for the Layer above Tested Layers (Plate Load 50 psi)

Required Structural Number under Plate Load Test (50psi) with 5 in. Limerock						
Traffic Data (ESAL)	1.30E+07					
W.T. above Embankment	0 in.	12 in.	24 in.	36 in.	24 in.	12 in.
Levy County A-3 (4%)	-	3.02	-	3.39	-	-
SR70 A-3 (8%)	-	2.88	-	3.22	-	-
SR70 A-2-4 (14%)	-	3.21	-	4.26	-	-
A-2-4 (12%)	-	3.13	-	3.54	-	-
A-2-4 (20%)	-	3.09	-	3.37	-	-
A-2-4 (24%)	-	3.28	-	3.59	-	-
A-2-4 (30%)	-	3.04	-	4.36	-	-
Oolite A-1	-	2.16	-	3.11	-	-
Spring Cemetery A-2-4	3.10	3.39	3.81	4.22	3.90	3.49
Branch A-2-4	2.11	2.19	2.54	3.69	3.42	2.54
Iron Bridge A-2-6	2.22	2.29	2.76	4.17	3.24	2.63

Table 8.4 Required Thickness of AC Layer with 5 in. Limerock under 20-psi Plate Load

Required Thickness of AC Layer (in.) (20psi with Assumed Limerock Base 5 in.)				Difference in AC Thickness (in.)	
W.T. above Embankment	0 in.	12 in.	24 in.	0 to 12 in.	0 to 24 in.
Levy County A-3 (4%)	6.59	6.90	-	0.31	-
SR70 A-3 (8%)	5.54	6.00	-	0.46	-
SR70 A-2-4 (14%)	5.86	6.40	-	0.54	-
A-2-4 (12%)	7.06	7.04	-	-0.02	-
A-2-4 (20%)	5.87	6.00	-	0.13	-
A-2-4 (24%)	5.80	6.81	-	1.01	-
A-2-4 (30%)	5.82	6.16	-	0.34	-
Oolite A-1	-	-	-	-	-
Spring Cemetery A-2-4	7.38	7.54	8.53	0.16	1.15
Branch A-2-4	4.40	4.54	5.52	0.14	1.12
Iron Bridge A-2-6	5.95	5.67	7.24	-0.28	1.29

Table 8.5 Required Thickness of AC Layer with 10 in. Limerock under 20-psi Plate Load

Required Thickness of AC Layer (in.) (20psi with Assumed Limerock Base 10 in.)				Difference in AC Thickness (in.)	
W.T. above Embankment	0 in.	12 in.	24 in.	0 to 12 in.	0 to 24 in.
Levy County A-3 (4%)	4.55	4.86	-	0.31	-
SR70 A-3 (8%)	3.50	3.96	-	0.46	-
SR70 A-2-4 (14%)	3.81	4.36	-	0.55	-
A-2-4 (12%)	5.01	5.00	-	-0.01	-
A-2-4 (20%)	3.82	3.96	-	0.14	-
A-2-4 (24%)	3.75	4.77	-	1.02	-
A-2-4 (30%)	3.77	4.11	-	0.34	-
Oolite A-1	-	-	-	-	-
Spring Cemetery A-2-4	5.33	5.49	6.49	0.16	1.16
Branch A-2-4	2.32	2.49	3.48	0.17	1.16
Iron Bridge A-2-6	3.91	3.64	5.20	-0.28	1.29

Table 8.6 Required Thickness of AC Layer with 5 in. Limerock under 50-psi Plate Load

Required Thickness of AC Layer (in.) (50psi with Assumed Limerock Base 5 in.)							Difference in AC Thickness (in.)					
Water Table above Embankment	0 in. (E)	12 in. (F)	24 in. (G)	36 in. (H)	24 in. (I)	12 in. (J)	E to F	E to G	E to H	F to H	Between G and I	Between F and J
Levy A-3 (4%)	-	6.87	-	7.70	-	-	-	-	-	0.83	-	-
SR70 A-3 (8%)	-	6.54	-	7.31	-	-	-	-	-	0.77	-	-
SR70 A-2-4 (14%)	-	7.29	-	9.68	-	-	-	-	-	2.39	-	-
A-2-4 (12%)	-	7.11	-	8.06	-	-	-	-	-	0.95	-	-
A-2-4 (20%)	-	7.02	-	7.52	-	-	-	-	-	0.5	-	-
A-2-4 (24%)	-	7.45	-	8.15	-	-	-	-	-	0.7	-	-
A-2-4 (30%)	-	6.92	-	9.91	-	-	-	-	-	2.99	-	-
Oolite A-1	-	4.91	-	7.06	-	-	-	-	-	2.15	-	-
Spring Cemetery	7.05	7.71	8.65	9.60	8.87	7.92	0.66	1.6	2.55	1.89	0.22	0.21
Branch	4.79	4.98	5.78	8.38	7.77	5.78	0.19	0.99	3.59	3.4	1.99	0.8
Iron Bridge	5.03	5.21	6.27	9.48	7.37	5.98	0.18	1.24	4.45	4.27	1.1	0.77

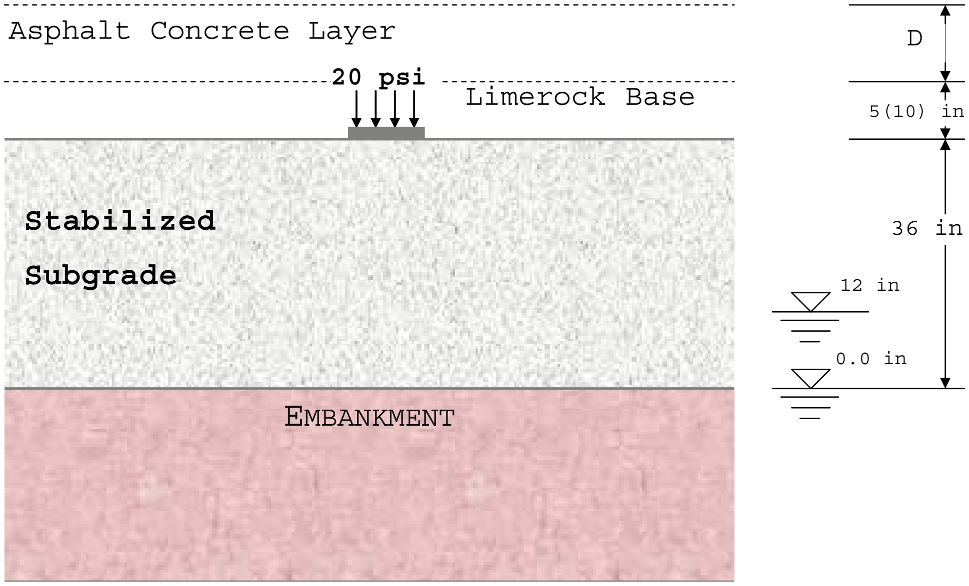


Figure 8.1 Case Study for SR70 (20 psi without Limerock Base)

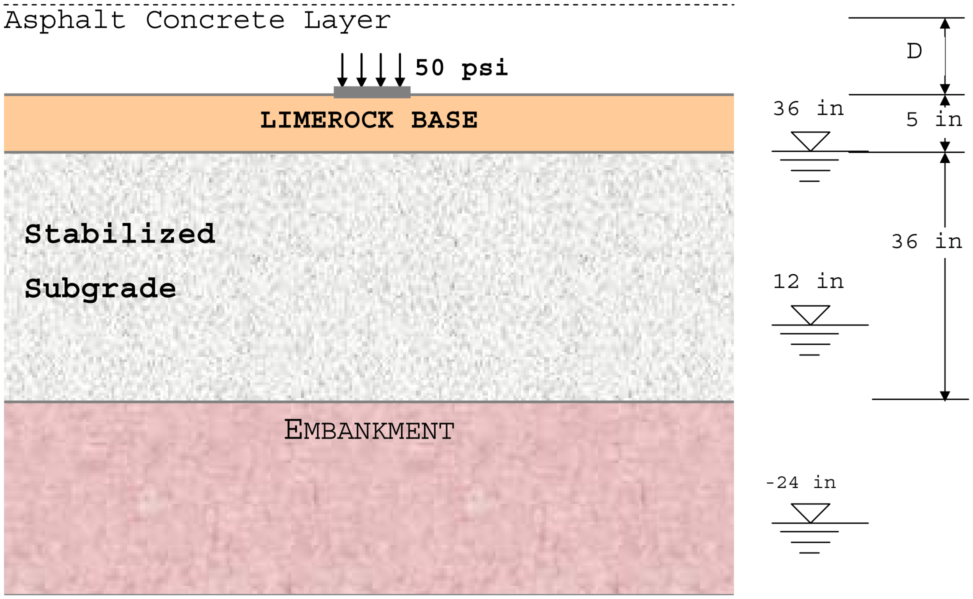


Figure 8.2 Case Study for SR70 (50 psi with Limerock Base)

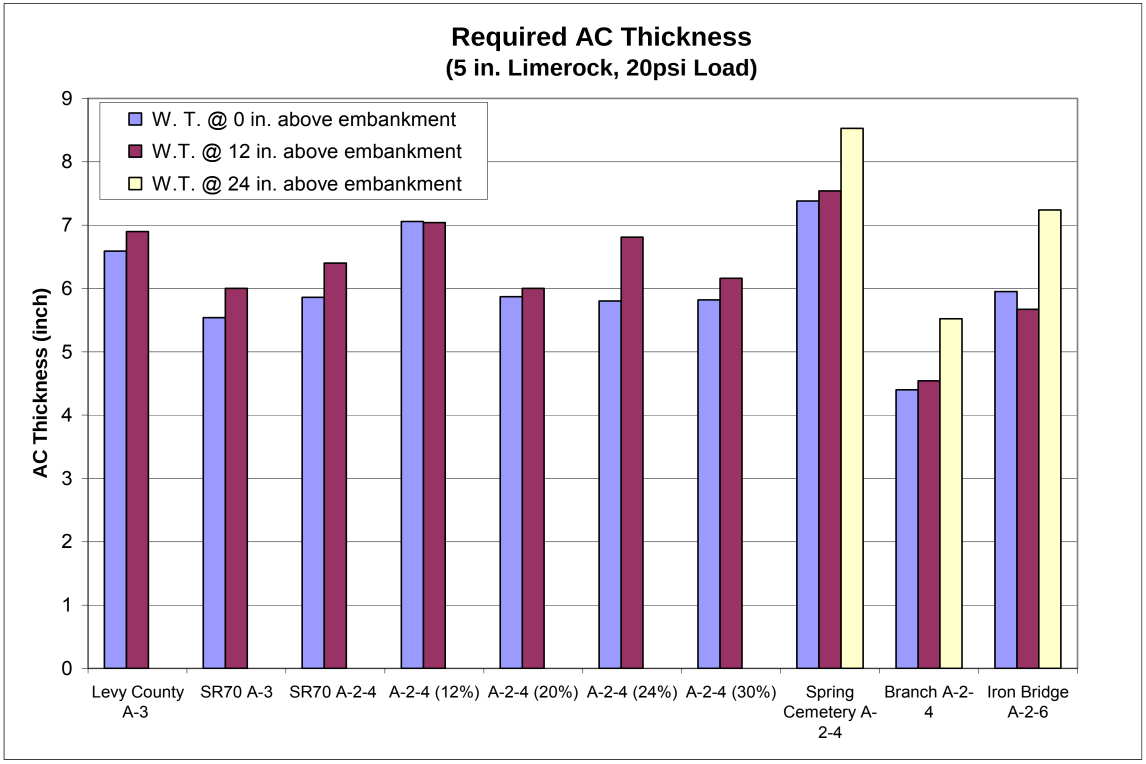


Figure 8.3(A) Required AC Thickness at Different Water Tables (20-psi Plate Load with 5-in. Limerock Base Below)

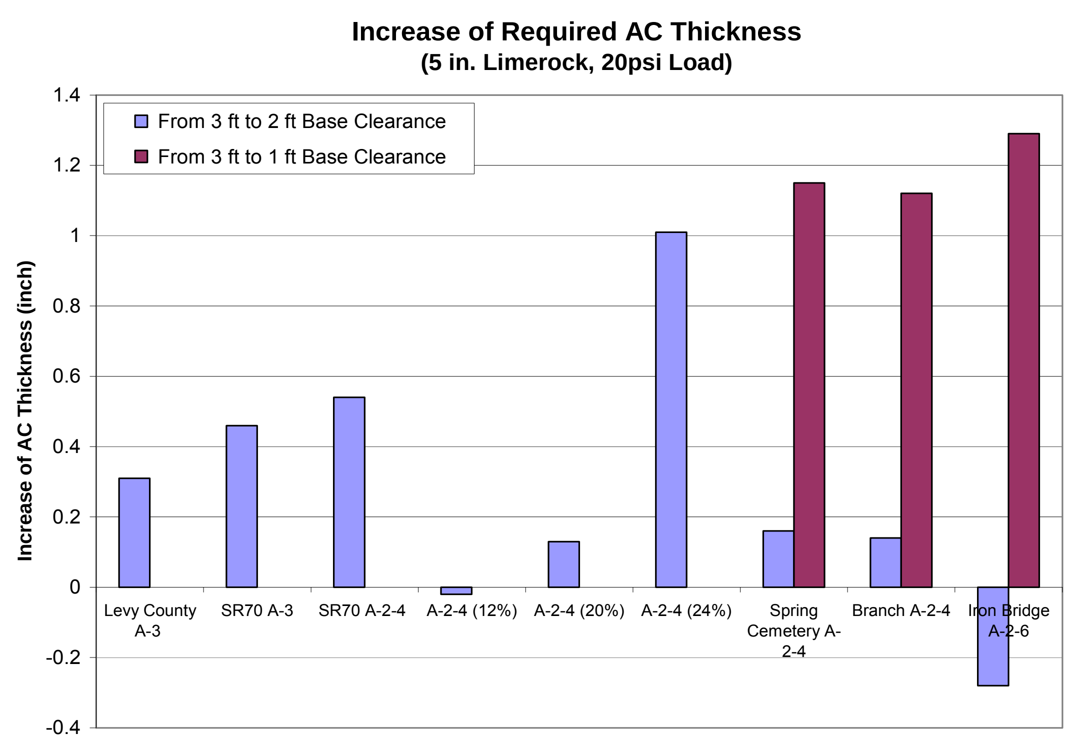


Figure 8.3(B) Increase of Required AC Thickness at Different Base Clearance (20-psi Plate Load with 5-in. Limerock Base Below)

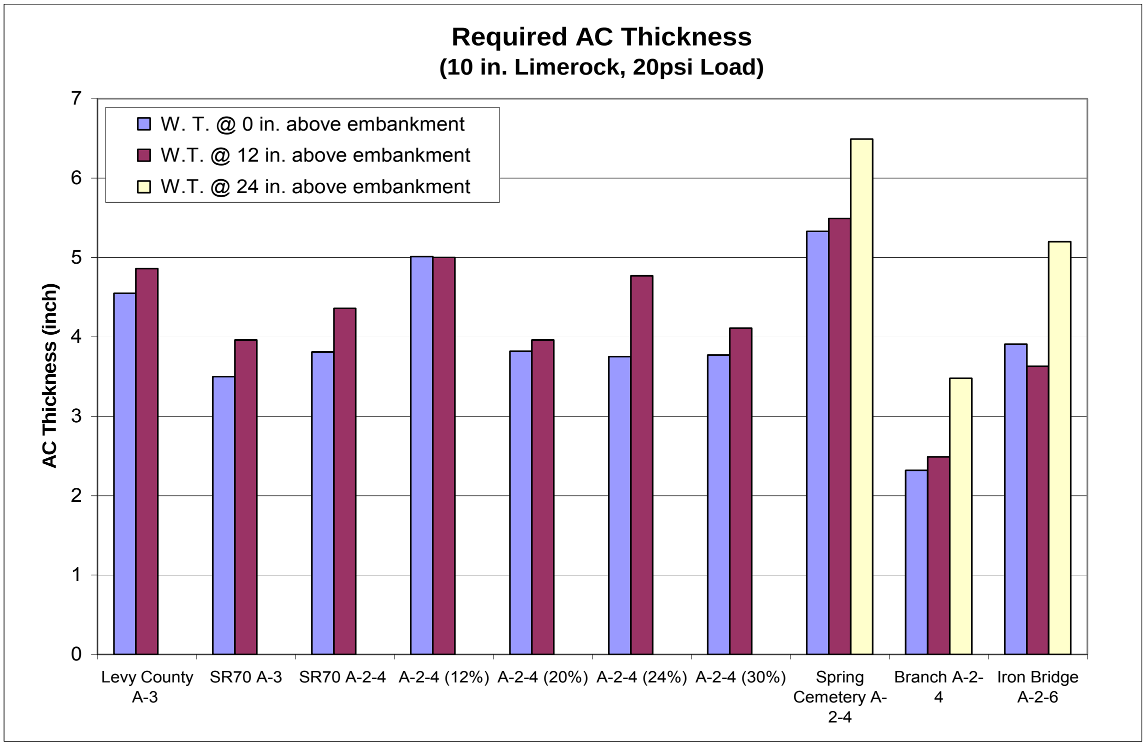


Figure 8.4(A) Required AC Thickness at Different Water Tables (20-psi Plate Load with 10-in. Limerock Base Below)

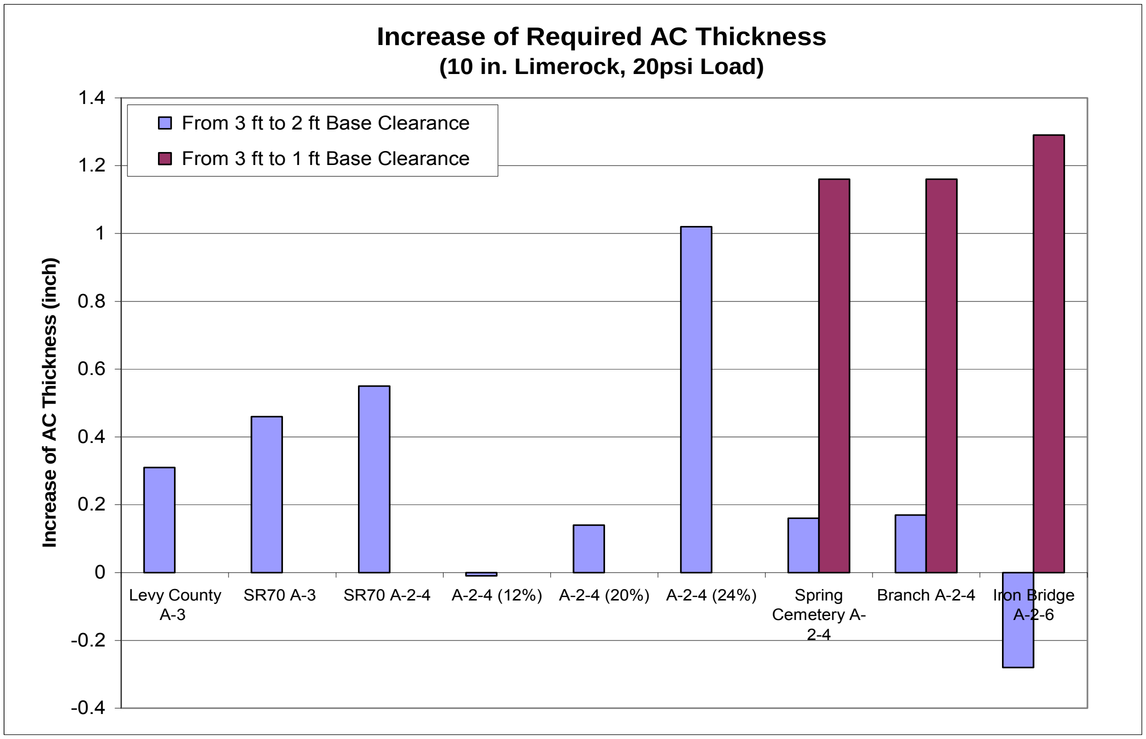


Figure 8.4(B) Increase of Required AC Thickness at Different Base Clearance (20-psi Plate Load with 10-in. Limerock Base Below)

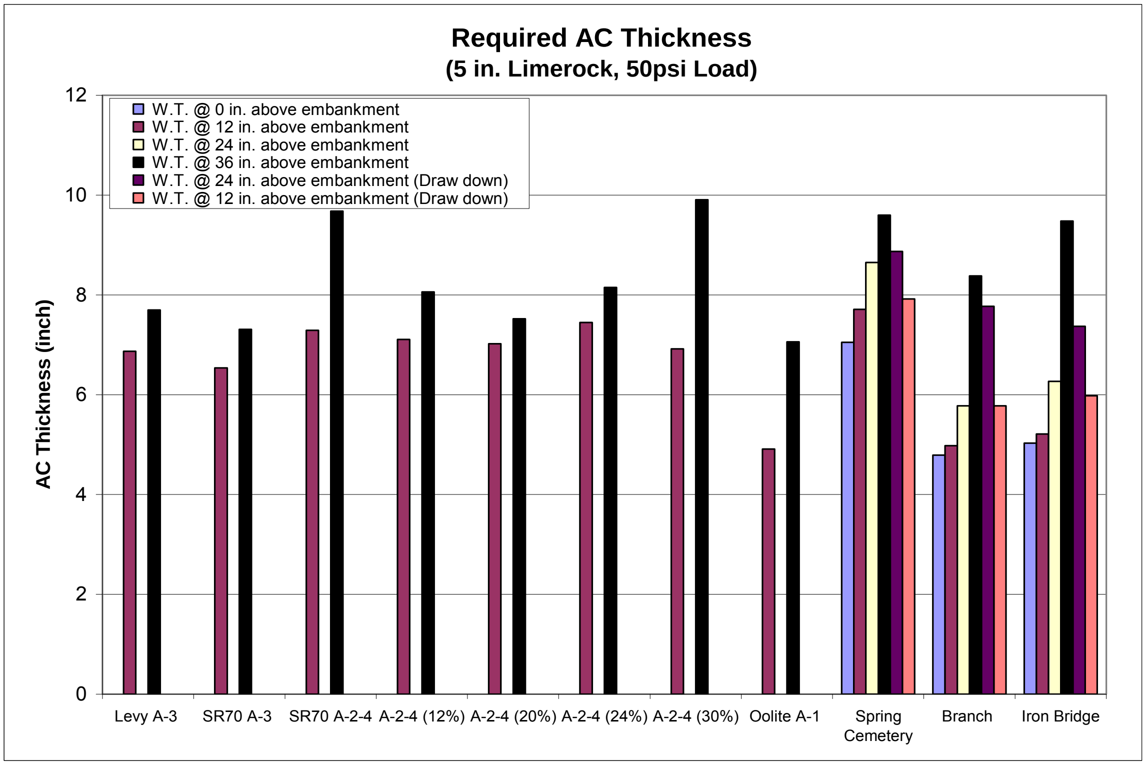


Figure 8.5(A) Required AC Thickness at Different Water Tables (50-psi Plate Load with 5-in. Limerock Base Below)

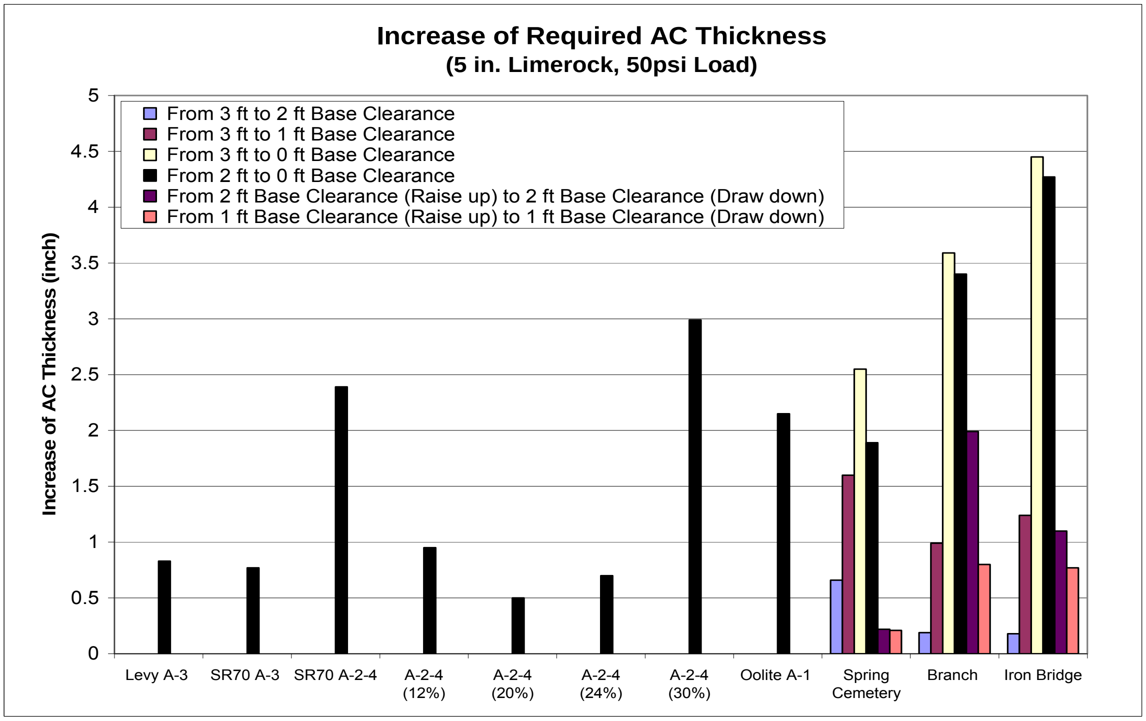


Figure 8.5(B) Increase of Required AC Thickness at Different Base Clearance (50-psi Plate Load with 5-in. Limerock Base Below)

CHAPTER 9
SUMMARIES AND DISCUSSIONS

9.1 GENERAL

The summaries of the design highwater clearance study are presented in this chapter. Laboratory tests conducted at the College of Engineering in Tallahassee and test-pit tests performed at the FDOT Gainesville Materials Office, along with the test results, are summarized and compared. The effect of high groundwater levels on the performance of pavement subgrade is evaluated in terms of the detrimental effect of the moisture on the resilient modulus of subgrade materials.

9.2 TEST SUBGRADE MATERIALS

The soils under evaluation in this research were the typical A-3 and A-2-4 subgrade materials in use in the Sate of Florida. A total of eleven types of soil were evaluated. The materials were further divided into three groups according to the test schedule as follows:

(I) Phase I: (From Dec. 1999 to Feb. 2000)

1. Levy A-3 soil - 4% fines
2. SR70 A-3 soil - 8% fines

3. SR70 A-2-4 soil – 14% fines

(II) Phase II: (From Jun. 2000 to Mar. 2001)

4. A-2-4 soil - 12% fines
5. A-2-4 soil - 20% fines
6. A-2-4 soil - 24% fines
7. A-2-4 soil - 30% fines
8. Miami Oolite A-1 soil

(III) Phase III: (From Jul. 2005 to Apr. 2007)

9. Spring Cemetery A-2-4 soil - 15% fines
10. Branch A-2-4 soil - 23% fines
11. Iron Bridge A-2-6 soil - 31% fines

The basic properties of the test materials were provided by the FDOT Gainesville State Material Office. It should be noted that the A-2-4 (30%) soil was obtained from blending two different source materials.

9.3 LABORATORY RESILIENT MODULUS TESTS

Three major laboratory tests were performed to study the factors influencing the resilient modulus of each subgrade soil. The test conditions are summarized in Table 9.1. The summary of the test results is presented in the following sections.

9.3.1 Resilient Modulus Test

The tests were performed using the AASHTO T292-91I test standard for the Phase I and II soils, with both middle-half and full-length LVDT position measurements, while the tests for the Phase III soils were conducted using the AASHTO T307-99 test standard with only full-length LVDT position measurement. Since the resilient modulus tests were only performed at the optimum water content for the Phase III soils, the analysis of the moisture effect on the resilient modulus was only available for the Phase I and II soils. As for the compaction effort, the 100% Modified Proctor was used for the Phases I and Phase II soils, while the 100% Standard Proctor was used for the Phase III soils. The resilient modulus data obtained from the bulk stress of 75.8 kPa (11 psi), which was three times the confining pressure of 13.8 kPa (2 psi) plus one deviator stress of 34.5 kPa (5 psi), were used for analysis.

Test results showed that SR70 A-2-4 (14%) and A-2-4 (30%) had very high resilient moduli under dry conditions, while SR70 A-2-4 (14%) and Branch A-2-4 (23%) had high resilient moduli under the optimum condition. The average resilient modulus from middle-length LVDT position measurement was about 1.36 times that from the full-length LVDT position measurement. The test results from the middle-length LVDT position measurements were considered more representative of the actual resilient modulus due to less end effect.

Analysis of the results indicated that the moisture had a detrimental effect on the resilient modulus of subgrade soils. An increase in moisture caused a reduction in the resilient modulus. The degree of reduction was different among various types of soil. The Levy County A-3, SR70 A-3, and A-2-4 (12%) soils had lower reduction rates, while the SR70 A-2-4 and Oolite A-1 soils had higher reduction rates. The A-2-4 (20%), A-2-4 (24%), and A-2-4 (30%) behaved differently between the wetting and drying processes. The A-2-4 (30%) soil had a higher reduction rate under dry conditions than that under wet conditions. The degree of reduction for A-2-4 soils was more apparent than that of A-3 soils; the exception of this criterion was the A-2-4 (12%) soil.

In general, the resilient modulus increases with an increase in confining pressure and deviator stress for coarse-grain soils, but decreases with an increase in deviator stress for fine-grain soils. The data showed that the resilient moduli of the SR70 A-2-4 (14%), A-2-4 (30%), and Branch A-2-4 (23%) soils decreased with an increase in deviator stress, while the deviator stress had no effect on the resilient moduli of the A-2-4 (20%) or Miami Oolite A-1 soils.

There was no apparent relationship between both resilient modulus value and reduction rate and percent of fines passing No. 200 sieve. However, for the A-3 soils, the reduction rate increased with an increase in percent of fines. This trend was not significant for the A-2-4 soils.

The effect of the maximum dry unit weight, LBR, percent of clay and the graduation characteristics were also considered in the analysis. The rate of reduction in resilient modulus increased with an increase in the maximum dry unit weight, LBR and percent of clay. The poorly graded soils had higher reduction rates, as observed in the SR70 A-2-4(14%), Branch A-2-4 (23%), and Iron Bridge A-2-6 (31%) soils.

9.3.2 Suction Test

The suction tests adopted the AASHTO Designation T273-86 to determine the soil suction force utilizing the thermocouple psychrometers of the Spanner. Tests were performed at different moisture content levels for all the first eight soil types. Test results showed that, in general, suction increased with a decrease in water content. No trend between the suction value and the percent of fines was found; neither was there a relationship between suction and optimum water content.

9.3.3 Permeability Test

Permeability values for each subgrade soil under saturated conditions were obtained using the laboratory permeability test method. For A-2-4 and A-2-6 soils, the ASTM Designation D5084-90 Flexible Wall Permeameter (FWP) method was used, while the ASTM Designation D2434-68 Constant Head method was adopted for SR70 and Levy County A-3 soils. Test results showed that the permeability decreased with an increase in percentage of fines.

The percentage of fines is a good indicator to use in order to predict the permeability property for soils.

9.4 TEST-PIT EQUIVALENT MODULUS TESTS

A full-scale simulation was conducted to evaluate the effect of a high groundwater level on the modulus of the subgrade soil in the test-pit experimental program. With adjustment of the groundwater level in the subgrade, the dynamic plate load tests were performed to measure the flexible deformations; from this, the equivalent moduli of the materials in the test pit were derived.

The equivalent moduli were, however, measured for the composite layers of subgrade and embankment under the plate loading, with an additional limerock base layer. A layer system using KENLAYER was setup to estimate the resilient modulus value for the individual subgrade layer under the high groundwater level.

It should be noted that five test pits were constructed in a slightly different way, and the water table conditions were not the same, either. Furthermore, the compaction effort for these five test pits were not quite the same either. The major difference among these five test pits is illustrated in Table 9.2.

9.4.1 Moisture Study Summary

The drainage was analyzed by lowering the groundwater level from +36 in. above the embankment to a lower level. The lower groundwater level for Test Pits 3 and 4 were only set at +12 in. above the embankment, which was much higher than the groundwater level set for Test Pits 1 and 2 (20 in. and 24 in. below the embankment). The drainage rates for Test Pits 3 and 4 could potentially be higher if the lower groundwater levels were set at the same level as that which was set for Test Pits 1 and 2.

A general trend was found in Figure 9.1, though; the drainage rate was proportional to both the percent of fines and the permeability. As shown in Table 9.3, the Levy A-3 soil had the highest drainage rate compared to other soils, while the A-2-4 (30%) and Miami Oolite A-1 soils did not drain much at all.

Capillary rise is discussed in Section 7.3 for both the groundwater level from drained condition to both 0 in. and +12 in. above the embankment. Since the capillary rise can be limited by the height of the placement, the data from the case at which the water table was raised to 0 in. above the embankment was more representative of the capillary rise behavior. Table 9.4 and Figure 9.2 showed that the A-2-4 (12%) had the highest height and percent of water increased due to capillary rise, both of which can be attributed the higher suction value. But with the high suction value, the A-2-4 (20%) soil had almost no capillary rise. This behavior is abnormal.

The accuracy of the moisture data should be reexamined due to the abnormality of the TDR measurements. The moisture data obtained from the analysis of the moisture effect can only considered as a reference in this study.

9.4.2 Plate Load Test Summary

When the groundwater level was raised from the interface of the subgrade and embankment layers to +12 in. above the interface (i.e., from base clearance 3 ft to 2 ft), the equivalent modulus values for the subgrade soils were only decreased slightly except that the A-2-4 (24%) soil had a 28% reduction in resilient modulus (Table 7.29). In contrast, when the groundwater level was changed from +12 in. to +36 in. above the embankment, the reduction rate increased significantly, especially for the SR70 A-2-4 (14%), A-2-4 (30%), Miami Oolite A-1, Branch A-2-4, and Iron Bridge A-2-6 Soils.

From Phase III plate load tests, the reduction in resilient modulus was evaluated when the groundwater level was raised from +12 in. to +24 in. above the embankment. The data showed that the reduction rate increased significantly compared to that which was obtained when the groundwater level was raised from the top of the embankment to +12 in. above the embankment.

Other than the plate load test, for which, the modulus represented the composite layers below the load, the layer modulus results represent the modulus for the individual layers of subgrade materials. The layer moduli of the eleven subgrade soils under various groundwater levels were computed with KENLAYER to be compared with the results from the plate load tests. The results showed that the SR70 A-2-4 (14%), A-2-4 (30%), Miami Oolite A-1, Branch A-2-4, and Iron Bridge A-2-6 Soils are extremely sensitive to the change in groundwater level from +12 in. to +36 in. above the embankment. The A-2-4 (20%) soil is the least sensitive soil in response to the groundwater level changes.

The plate load tests under a 20-psi load pressure without a limerock base were compared with those under a 50-psi load pressure with a limerock base at the groundwater level at +12.0 in. above the embankment. The results showed that with a 5-inch thick limerock layer as the base layer, the equivalent modulus values were almost doubled. All the subgrade soils had significant increase for their equivalent modulus values due to the addition of a limerock base layer. The limerock base layer certainly improved the dynamic performance of the pavement.

9.5 COMPARISON OF LABORATORY AND TEST-PIT TEST RESULTS

The laboratory and test-pit tests were performed under various water conditions. In the laboratory tests, moisture was added into the dry soils in the laboratory, whereas in the test-pit, the groundwater level was raised or lowered to a stabilized condition within the test pit. The laboratory resilient modulus and equivalent layer modulus generally represented the same kind of engineering property of the subgrade performance. However, the physical conditions (water content) were very much different for deriving the resilient modulus and equivalent layer modulus.

The laboratory resilient modulus versus the test-pit equivalent modulus and subgrade layer modulus are presented in Tables 9.5 and 9.6, respectively. The laboratory resilient moduli at the optimum moisture content were lower than the equivalent layer modulus values for groundwater level at +12 in. above the embankment. Comparing the resilient modulus results from the laboratory with the subgrade layer moduli from the test-pit, the modulus values were generally within the same range for the subgrade soils.

As shown in Tables 9.7 and 9.8, the reductions in the layer modulus when the groundwater level was raised from 12 in. to 36 in. above the embankment are much more severe than those in the laboratory resilient modulus from optimum to soaked water conditions. However, the relative severity of moisture damage is the same for each subgrade soil.

From the test pit equivalent modulus data, the SR70 A-2-4 (14%), A-2-4 (30%), Miami Oolite A-1, Branch A-2-4, and Iron Bridge A-2-6 soils are the soils most sensitive to moisture damage. The A-2-4 (20%) soil is the soil least sensitive soil to moisture damage. The reductions due to moisture damage can be used in pavement design to estimate the resilient modulus values under various moisture conditions.

9.6 DISCUSSIONS OF GRADATION EFFECT ON THE RESILIENT MODULUS

Based on the laboratory and test-pit test results, the SR70 A-2-4 (14%) and A-2-4 (30%), Miami Oolite A-1, Branch A-2-4 (23%), and Iron Bridge A-2-6 (31%) soils are the most sensitive to the moisture damage. Other soil types are not as sensitive to changes in moisture content. According to the studies from both Zhang (2004) and Ling (2007), the coefficient of uniformity (C_u) and coefficient of curvature (C_c) are two factors that can be used to predict the moisture sensitivity of granular soils. For a well-graded granular soil, C_u should be over 5 and C_c should be in the range of 1 to 3. From Table 6.16 the gradation properties for the subgrade soils, the SR70 A-2-4 (14%) and A-2-4 (24%) soils have C_c values higher than 3. Based on this research study, a simple criterion is proposed to classify soil moisture sensitivity as follows:

Moisture Sensitivity	Criteria
High	$C_c > 5$
Intermediate	$3 <= C_c <= 5$
Low	$C_c < 3$

The proposed criteria will need additional research to further evaluate the moisture sensitivity of granular soils.

Table 9.1 Laboratory Tests Comparison

Phase	Test Materials	Compaction Effort	Test Method	LVDT Position	Moisture Condition	Suction Test	Permeability Test
I	Levy A-3	100% Modified Proctor	AASHTO T292-91I	Middle-Half Full-Length	Dry Optimum Soaked	Yes	Yes
	SR70 A-3					Yes	Yes
	SR70 A-2-4					Yes	Yes
II	A-2-4 (12%)					Yes	Yes
	A-2-4 (12%)					Yes	Yes
	A-2-4 (12%)					Yes	Yes
	A-2-4 (12%)					Yes	Yes
	Oolite A-1					Yes	Not available
III	Spring Cemetery A-2-4	100% Standard Proctor	AASHTO T307-99	Full-Length	Optimum	Not available	Yes
	Branch A-2-4					Not available	Yes
	Iron Bridge A-2-6					Not available	Yes

Table 9.2 Test Pit Comparison

Test Pit	Size (in.)	Embankment Material	Test materials	Compaction Effort	Plate Load Location	Groundwater Level (in. above the embankment)	Remarks
1	72x96	Standard Embankment : 12 in. A-3 Soil Builder's Sand : 12 in. River Gravel : 12 in.	Levy A-3	Lift 1~6 : 100% Modified	Fixed	-20, 0, +12, +36, -24	Groundwater level fluctuated at +12 in. Level
2	72x96		SR70 A-3, SR70 A-2-4		Fixed with some scattered	-24, -12, 0, +12, +36, -24, +36, +41	Groundwater level fluctuated at +36 in. Level
3	72x96		A-2-4 (12%), A-2-4 (20%), A-2-4 (24%)		Scattered	-24, -12 , 0, +12, +36, +41, +30 , +12, +36, +12	Groundwater level fluctuated at 0 in. and +36 in. Level
4	72x96		A-2-4 (30%), Miami Oolite A-1		Scattered	-24, -12, 0, +12, +36, +12	
5	96x108	Standard Embankment : 24in. A-2-4 Soil Builder's Sand : 9 in. River Gravel : 9 in.	Spring Cemetery A-2-4, Branch A-2-4, Iron Bridge A-2-6	Lift 1~4 : 100% Standard Lift 5~6 : 98% Modified	Scattered	-24, 0, +12, +24, 0, +12, +24, +36, +24, +12	

Table 9.3 Drainage Rates for Phase I and Phase II Soils

Test Pit	Groundwater level Adjustment	Material	Time to Complete Drainage	Top Layer*** Water Content Drained	Drainage Rate	Top Layer Degree of Stauration
	in.		days	%	% Water content / day	%
1	+36 to -20	Levy A-3	3	15.1 -> 6.3	2.933	31.2
2	+36 to -24	SR70 A-3	59	17.1 -> 6.4	0.181	38.9
		SR70 A-2-4	59	30.5** -> 16.8	0.232	84.6
3	+36 to +12	A-2-4, 12%	59	11.4 -> 7.9	0.059	42.4
		A-2-4, 20%	59	8.1 -> 5.5	0.044	44.0
		A-2-4, 24%	59	13.0 -> 7.9	0.086	49.3
4	+36 to +12	A-2-4, 30%	59	8.6 -> 8.3	0.005	51.5
		Oolite A-1	59	4.4 -> 3.7	0.012	39.5
5	Not Available					

* Above embankment

** The top layer was over flooded, and the water content was too high.

*** Analysis is for the top layer only (+30 to +36 in. above the embankment)

Table 9.4 Capillary Rise Rate for the Eleven Soils with the Groundwater Level from Drained Condition to +0 in. above the Embankment

Test Pit	Groundwater Level Adjustment	Material	Time to Reach Capillary Height	Height of Capillary Rise	Time to Reach Equlibrium	Water Increase by Capillary Rise (Total)	Water Increase by Capillary Rise (Top Layer)	Rise Delayed
	in.		days	in.	days	%	%	days
1	-20 to 0	Levy A-3	22	24	28	10.3	-0.4	0
2	-12 to 0	SR70 A-3	4	18	18	10.2	-0.2	0
		SR70 A-2-4	34	30	>42	3.9	0.7	0
3	-24 to 0	A-2-4, 12%	20	36	33	26.1	1.6	0
		A-2-4, 20%	0	0	0	-0.5	-0.3	0
		A-2-4, 24%	13	36	45	7.3	0.0	0
4	-24 to 0	A-2-4, 30%	4	36	21	7.5	0.1	0
		Oolite A-1	4	18	4	0.0	-0.3	0
5	-24 to 0	Spring Cemetery A-2-4	2	30*	7	13.5	-0.1	0
		Branch A-2-4	11	24	16	4.5	0.0	0
		Iron Bridge A-2-6	4	30*	17	11.9	0.3	1

* Capillary rise is limited due to unavailability of the moisture content of the top layer.

Table 9.5 Comparison of Lab MR Test Results to Test-Pit EQ Modulus Results

Soil		Lab Reislient Modulus			Test-Pit Average Equivalent Modulus after 10000 Cycles								
		Dry	Optimum	Soaked	20 psi Plate Load w/o Limerock			50 psi Plate Load w/ Limerock					
		Full Length	Full Length	Full Length	W.T.:0 in. above embankment	W.T.:12 in. above embankment	W.T.:24 in. above embankment	W.T.:0 in. above embankment	W.T.:12 in. above embankment	W.T.:24 in. above embankment	W.T.:36in. above embankment	Draw down to W.T. 24 in. above embankment	Draw down to W.T. 12 in. above embankment
Levy A-3 (%4)	MC, %	6.2	9.55	15.15	15.1---4.5	15.8---5.5			16.3---6.2		16.7---14.7		
	MR, psi	16957	15684	11868	20986	19082			38313		28479		
SR70 A-3 (8%)	MC, %	5.4	11.4	13.55	17.2---6.7	20.6---6.8			20.4---8.0		21.6---17.1		
	MR, psi	17427	17231	13328	29545	25307			43469		32571		
SR 70 A-2-4 (14%)	MC, %	8.41	10.6	11.45	14.4---8.1	18.3---8.1			18.3---11.4		14.6---33.2		
	MR, psi	44332	24165	14099	26534	22276			32860		15334		
A-2-4 (12%)	MC, %	7.05	12.1	14.1	10.38---4.75	10.77---6.39			11.15---7.68		11.3---11.05		
	MR, psi	13996	13868	13175	18201	18201			34996		25253		
A-2-4 (20%)	MC, %	7.8	10	11.95	9.55---3.33	9.58---3.28			9.6---3.73		9.61---8.02		
	MR, psi	20673	15862	14509	26468	25323			36204		30214		
A-2-4 (24%)	MC, %	7.7	10.7	11.7	7.22---5.25	7.33---5.66			7.39---6.05		7.58---13.05		
	MR, psi	15871	13350	9994	27106	19595			31001		24492		
A-2-4 (30%)	MC, %	6.65	12.15	13.3	16.01---8.04	15.76---7.92			15.54---8.09		15.66---8.64		
	MR, psi	41360	10419	10392	26904	26534			37655		14356		
Oolite A-1	MC, %	5	7.8	8.15					3.18---3.52		3.18---4.38		
	MR, psi	18280	15575	11431					91326		35673		
Spring Cemetery A-2-4 (15%)	MC, %		9.25		12.5---4	13.3---4.6	14.3---6.8	13.1---4.3	13.5---4.7	13.4---10	14---12.7	13.7---12.9	13.4---5.9
	MR, psi		9719		16558	15796	11914	35796	28387	20896	15733	19527	26377
Branch A-2-4 (23%)	MC, %		8.8		10.3---5	10.2---9.6	10.5---3.3	10.1---5.6	10.3---6	10.5---8	10.5---9.6	10.5---8	10.3---7.9
	MR, psi		27024		45192	42778	29777	97295	88097	59913	22792	27814	60034
Iron Bridge A-2-6 (31%)	MC, %		10.35		10.5---5.9	12.8---10.6	11.9---7.9	11.9---8.5	12.2---9.1	12.2---10.1	12.3---13.5	12.3---11.1	11.8---10.3
	MR, psi		9252		25751	28258	17227	85743	78621	48600	16263	31851	54841

Table 9.6 Comparison of Lab MR Test Results to Layer Modulus Results

Soil		Lab Reislient Modulus			Subgrade Layer Modulus Back Calculated from KENLAYER								
		Dry	Optimum	Soaked	20 psi Plate Load w/o Limerock			50 psi Plate Load w/ Limerock					
		Full Length	Full Length	Full Length	W.T.:0 in. above embankment	W.T.:12 in. above embankment	W.T.:24 in. above embankment	W.T.:0 in. above embankment	W.T.:12 in. above embankment	W.T.:24 in. above embankment	W.T.:36in. above embankment	Draw down to W.T. 24 in. above embankment	Draw down to W.T. 12 in. above embankment
Levy A-3 (%4)	MC, %	6.2	9.55	15.15	15.1---4.5	15.8---5.5			16.3---6.2		16.7---14.7		
	MR, psi	16957	15684	11868	33938	30312			23785		15228		
SR70 A-3 (8%)	MC, %	5.4	11.4	13.55	17.2---6.7	20.6---6.8			20.4---8.0		21.6---17.1		
	MR, psi	17427	17231	13328	52357	43075			35243		22770		
SR 70 A-2-4 (14%)	MC, %	8.41	10.6	11.45	14.4---8.1	18.3---8.1			18.3---11.4		14.6---33.2		
	MR, psi	44332	24165	14099	45685	36693			29442		10442		
A-2-4 (12%)	MC, %	7.05	12.1	14.1	10.38---4.75	10.77---6.39			11.15---7.68		11.3---11.05		
	MR, psi	13996	13868	13175	28571	28861			21755		13488		
A-2-4 (20%)	MC, %	7.8	10	11.95	9.55---3.33	9.58---3.28			9.6---3.73		9.61---8.02		
	MR, psi	20673	15862	14509	45540	43075			35388		26106		
A-2-4 (24%)	MC, %	7.7	10.7	11.7	7.22---5.25	7.33---5.66			7.39---6.05		7.58---13.05		
	MR, psi	15871	13350	9994	46846	31327			23350		16824		
A-2-4 (30%)	MC, %	6.65	12.15	13.3	16.01---8.04	15.76---7.92			15.54---8.09		15.66---8.64		
	MR, psi	41360	10419	10392	46265	45250			31472		8992		
Oolite A-1	MC, %	5	7.8	8.15					3.18---3.52		3.18---4.38		
	MR, psi	18280	15575	11431					193909		36838		
Spring Cemetery A-2-4 (15%)	MC, %		9.25		12.5---4	13.3---4.6	14.3---6.8	13.1---4.3	13.5---4.7	13.4---10	14---12.7	13.7---12.9	13.4---5.9
	MR, psi		9719		25526	24075	17404	18854	17259	11748	7687	10587	15664
Branch A-2-4 (23%)	MC, %		8.8		10.3---5	10.2---9.6	10.5---3.3	10.1---5.6	10.3---6	10.5---8	10.5---9.6	10.5---8	10.3---7.9
	MR, psi		27024		93500	84700	51342	108500	93500	44960	10520	15500	43500
Iron Bridge A-2-6 (31%)	MC, %		10.35		10.5---5.9	12.8---10.6	11.9---7.9	11.9---8.5	12.2---9.1	12.2---10.1	12.3---13.5	12.3---11.1	11.8---10.3
	MR, psi		9252		42350	48441	26251	54242	57143	22335	3916	9427	19434

Table 9.7 Comparison of Lab MR Reduction Rate to Test-Pit EQ Modulus Reduction Rate

Soil		Lab Reislient Modulus Reduction Rate , R (%)		Test Pit Equivalent Modulus Reduction Rate, R (%)					
		From Dry to Optimum	From Optimum to Soaked	20 psi Plate Load w/o Limerock		50 psi Plate Load w/ Limerock			
		Full Length	Full Length	From 3 ft to 2 ft Base Clearance	From 3 ft to 1 ft base Clearance	From 3 ft to 2 ft Base Clearance	From 3 ft to 1 ft Base Clearance	From 3 ft to 0 ft Base Clearance	From 2 ft to 0 ft Base Clearance
Levy A-3	MC, %	6.2 -> 9.56	9.55 -> 15.16	(15.1-4.5)-(15.8-5.5)					(16.3--6.2)-(16.7-14.7)
	R , %	8	24	9.1					25.7
SR70 A-3 (8%)	MC, %	5.4 -> 11.5	11.4 -> 13.56	(17.2-6.7)-(20.6-6.8)					(20.4-8.0)-(21.6--7.1)
	R , %	1	23	14.3					25.1
SR 70 A-2-4 (14%)	MC, %	8.41 -> 10.7	10.6 -> 11.46	(14.4-8.1)-(18.3-8.1)					(18.3-11.4 -(4.6-33.2)
	R , %	45	42	16.0					53.3
A-2-4 (12%)	MC, %	7.05 -> 12.2	12.1 -> 14.2	(10.4-4.8)-(10.8-6.4)					(11.2-7.7)-(11.3-11.1)
	R , %	1	5	0.0					27.8
A-2-4 (20%)	MC, %	7.8 ->11	11 -> 11.95	(9.6-3.3)-(9.6-3.3)					(9.6-3.7)-(9.6-8.0)
	R , %	23	9	4.3					16.5
A-2-4 (24%)	MC, %	7.7 -> 10.8	10.7 -> 11.8	(7.2-5.4)-(7.3-5.7)					(7.4-6.1)-(7.6-13.1)
	R , %	16	25	27.7					21.0
A-2-4 (30%)	MC, %	6.65 -> 12.16	12.15 -> 13.4	(16.0-8.0)-(15.8-7.9)					(15.5-8.1)-(15.7-8.6)
	R , %	75	0	1.4					61.9
Oolite A-1	MC, %	6 -> 7.8	7.8 -> 8.16						(3.2-3.5)-(3.2-4.4)
	R , %	15	27						60.9
Spring Cemetery A-2-4 (15%)	MC, %			(12.5-4)-(13.3-4.6)	(13.3-4.6)-(14.3-6.8)	(13.1-4.3)-(13.5-4.7)	(13.5-4.7)-(13.4-10)	(13.4-10)-(14-12.7)	(13.5-4.7)-(14-12.7)
	R , %			4.6	28.0	20.7	41.6	56.0	44.6
Branch A-2-4 (23%)	MC, %			(10.3-5)-(10.2-9.6)	(10.2-9.6)-(10.5-3.3)	(10.1-5.6)-(10.3-6)	(10.3-6)-(10.5-8)	(10.5-8)-(10.5-9.6)	(10.3-6)-(10.5-9.6)
	R , %			5.4	34.0	9.5	38.4	76.6	74.1
Iron ridge A-2-6 (31%)	MC, %			(10.5-5.9)-(12.8-10.6)	(12.8-10.6)-(11.9-7.9)	(11.9-8.5)-(12.2-9.1)	(12.2-9.1)-(12.2-10.1)	(12.2-10.1)-(12.3-13.5)	(12.2-9.1)-(12.3-13.5)
	R , %			-9.7	33.1	8.3	43.3	81.0	79.3

Table 9.8 Comparison of Lab MR Reduction Rate to Layer Modulus Reduction Rate

Soil		Lab Reislient Modulus Reduction Rate , R (%)		Subgrade Layer Modulus Reduction Rate, R (%)					
		From Dry to Optimum	From Optimum to Soaked	20 psi Plate Load w/o Limerock		50 psi Plate Load w/ Limerock			
		Full Length	Full Length	From 3 ft to 2 ft Base Clearance	From 3 ft to 1 ft base Clearance	From 3 ft to 2 ft Base Clearance	From 3 ft to 1 ft Base Clearance	From 3 ft to 0 ft Base Clearance	From 2 ft to 0 ft Base Clearance
Levy A-3	MC, %	6.2 -> 9.56	9.55 -> 15.16	(15.1-4.5)-(15.8-5.5)					(16.3--6.2)-(16.7-14.7)
	R , %	7.5	24.3	10.7					36.0
SR70 A-3 (8%)	MC, %	5.4 -> 11.5	11.4 -> 13.56	(17.2-6.7)-(20.6-6.8)					(20.4-8.0)-(21.6--7.1)
	R , %	1.1	22.7	17.7					35.4
SR 70 A-2-4 (14%)	MC, %	8.41 -> 10.7	10.6 -> 11.46	(14.4-8.1)-(18.3-8.1)					(18.3-11.4)-(4.6-33.2)
	R , %	45.5	41.7	19.7					64.5
A-2-4 (12%)	MC, %	7.05 -> 12.2	12.1 -> 14.2	(10.4-4.8)-(10.8-6.4)					(11.2-7.7)-(11.3-11.1)
	R , %	0.9	5.0	-1.0					38.0
A-2-4 (20%)	MC, %	7.8 ->11	11 -> 11.95	(9.6-3.3)-(9.6-3.3)					(9.6-3.7)-(9.6-8.0)
	R , %	23.3	8.5	5.4					26.2
A-2-4 (24%)	MC, %	7.7 -> 10.8	10.7 -> 11.8	(7.2-5.6)-(7.3-5.7)					(7.4-6.1)-(7.6-13.1)
	R , %	15.9	25.1	33.1					28.0
A-2-4 (30%)	MC, %	6.65 -> 12.16	12.15 -> 13.4	(16.0-8.0)-(15.8-7.9)					(15.5-8.1)-(15.7-8.6)
	R , %	74.8	0.3	2.2					71.4
Oolite A-1	MC, %	6 -> 7.8	7.8 -> 8.16						(3.2-3.5)-(3.2-4.4)
	R , %	14.8	26.6						81.0
Spring Cemetery A-2-4 (15%)	MC, %			(12.5-4)-(13.3-4.6)	(13.3-4.6)-(14.3-6.8)	(13.1-4.3)-(13.5-4.7)	(13.5-4.7)-(13.4-10)	(13.4-10)-(14-12.7)	(13.5-4.7)-(14-12.7)
	R , %			5.7	31.8	8.5	37.7	59.2	55.5
Branch A-2-4 (23%)	MC, %			(10.3-5)-(10.2-9.6)	(10.2-9.6)-(10.5-3.3)	(10.1-5.6)-(10.3-6)	(10.3-6)-(10.5-8)	(10.5-8)-(10.5-9.6)	(10.3-6)-(10.5-9.6)
	R , %			9.3	45.0	13.8	58.6	90.2	88.7
Iron Bridge A-2-6 (31%)	MC, %			(10.5-5.9)-(12.8-10.6)	(12.8-10.6)-(11.9-7.9)	(11.9-8.5)-(12.2-9.1)	(12.2-9.1)-(12.2-10.1)	(12.2-10.1)-(12.3-13.5)	(12.2-9.1)-(12.3-13.5)
	R , %			-14.4	38.0	-5.3	58.8	92.8	93.1

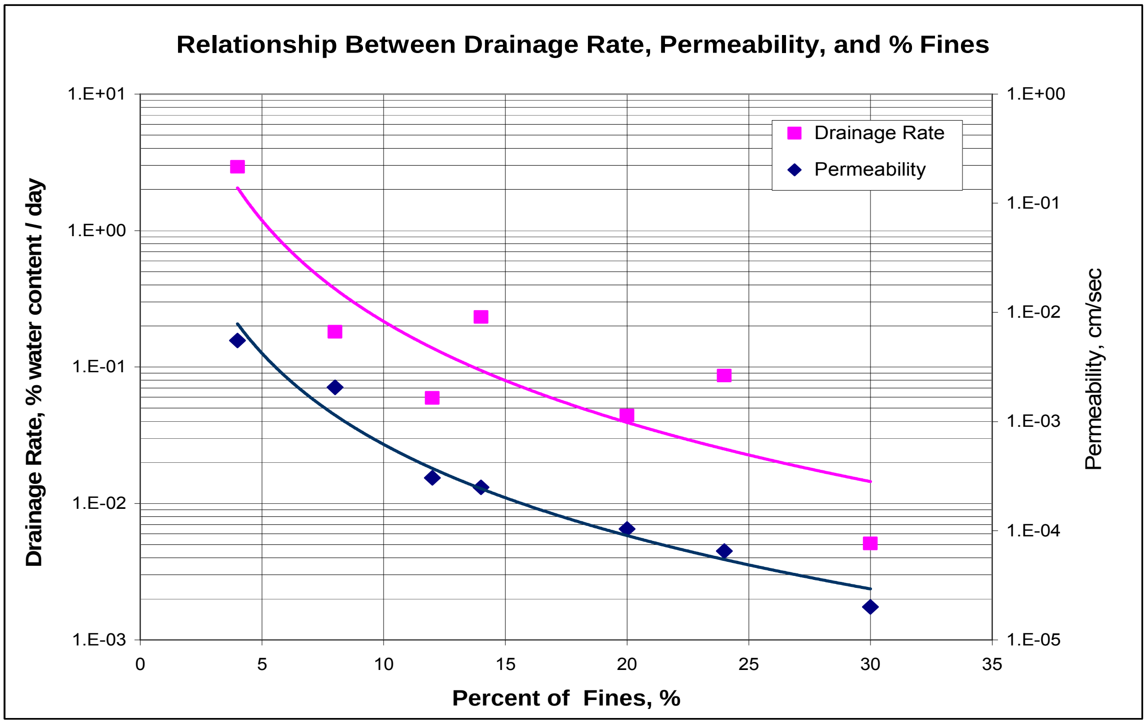


Figure 9.1 Relationship between Drainage Rate, Permeability, and Percent of Fines for Phase I and II soils

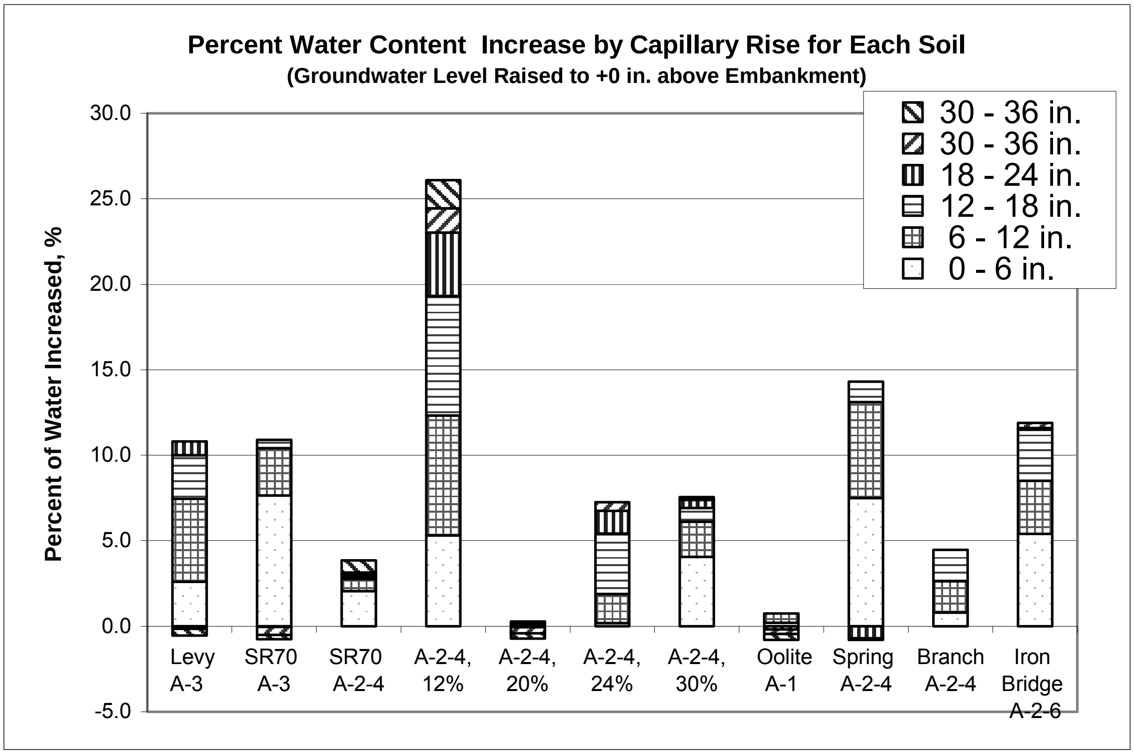


Figure 9.2 Percent Water Content Increased by Capillary Rise for Eleven Soils

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

10.1 CONCLUSIONS

Based upon the analyses and findings of this experimental study, the conclusions are summarized as follows:

Laboratory Experimental Program

1. Based on laboratory resilient modulus test, the resilient modulus value of each subgrade soil decreased with an increase in moisture content. However, the rates of reduction for these soils were not at the same level. The SR70 A-2-4 (14%) and Oolite A-1 soils were very sensitive to the change of moisture content from the optimum to soaked conditions. These two soils had the reduction rates of 26% and 31%. The other soil types were not as sensitive to the moisture content change (with reduction rates lower than 20%) as those two soils.
2. The moisture content in subgrade soil was a major factor affecting the resilient modulus. In addition, the test results showed that other factors including dry unit weight, LBR, percent of clay, coefficient of uniformity (C_u) and coefficient of curvature (C_c) also significantly affected the resilient modulus. The C_u and C_c were considered as two good

indicators for correlating the moisture sensitivity of granular soils.

3. No relationship existed between the reduction rate and the percentage of fines in soil. The percentage of fines was not a good indicator for categorizing the soils in terms of the sensitivity of resilient modulus to moisture effect. However, the percentage of fines was a good indicator to predict the permeability properties of soil. The permeability value under saturated condition decreased with an increase in percentage of fines.

Test-Pit Experimental Program

4. The A-2-4 (24%) soil was very sensitive to the change of high groundwater level from +0.0 in. to +12.0 in. above the embankment (i.e., lowering base clearance from 3 ft. to 2 ft.), the plate load equivalent modulus values were reduced 28%.
5. The SR70 A-2-4 (14%), A-2-4 (30%), Oolite A-1, Branch A-2-4 (23%), and Iron Bridge A-2-6 (31%) soils were extremely sensitive to the change of high groundwater level from +12.0 in. to +36.0 in. above the embankment (i.e., lowering base clearance from 2 ft. to 0 ft.). The plate load equivalent modulus reduction rates were more than 50%. For the Levy A-3 (4%), SR70 A-3 (8%), A-2-4 (12%,) A-2-4 (24%), and Spring Cemetery A-2-4 (15%) soils, the reduction rates were also

significant for the base clearance from 2 ft. to 0 ft. with the plate load equivalent modulus reduction rates in the range of 21% to 45%. The A-2-4 (20%) soil was the least sensitive soil in response to the change of high groundwater level with the plate load equivalent modulus reduction rate about 17%.

6. Adding a 5-in. limerock base layer was very beneficial to the pavement resistance, and the equivalent modulus values were almost doubled. The added limerock base layer certainly improved the dynamic performance of the pavement.
7. Comparing the laboratory resilient modulus results with the subgrade layer modulus values from test-pit, the modulus values were generally within the same range from the same type of soil. The laboratory resilient modulus value at optimum condition was lower than the layer modulus (about 50% to 70%) for the same type of soil tested in the test-pit with a base clearance of two feet (24 in.), except that the SR70 A-2-4 (14%) soil had about the same modulus for both tests.
8. When a pavement design is prepared, pavement designers and geotechnical engineers typically do not know the exact soil that will be used for the embankment. Due to the lack of a direct relationship between percent fines and modulus reduction and the high variability of the moduli reductions, cautions should be exercised when reducing base clearance below three feet. It was evident in this research that when

base clearances were reduced to two feet, the plate load equivalent modulus reductions were up to 28%. When base clearances were further reduced to one foot, the plate load equivalent modulus reductions were up to 43%. Furthermore, with base clearances at zero foot, the modulus reductions were up to 81%.

Case Study for High Groundwater Effect

9. The results of the case study indicated that for some sensitive soils, such as SR70 A-2-4 (14%) and A-2-4 (30%) soils, an increase of high groundwater table would demand a significant increase of the required thickness of asphalt concrete layer in order to have the same quality of pavement performance. The most severe condition was for base clearance reduced from two feet to zero foot. The other subgrade soils also required some increase of asphalt concrete layer thickness.
10. In areas with high groundwater levels, adequate base clearance should be maintained to minimize the moisture damage and to achieve quality performance of the pavement.

10.2 RECOMMENDATIONS

1. In future test-pit tests, TDR sensors should be calibrated against each soil type to have more accurate measurements of moisture content along pavement profile. More TDRs should be placed at different locations to avoid the errors from

malfunctions of the TDRs, such as the damage from installation and compaction or during plate load tests, disturbed soil around the TDR, etc.

2. Tests should be performed at the identical water and stress condition for each soil. The wetting and drying process should be the same too. To have more precise comparison between laboratory and test-pit modulus tests, the water content of the subgrade soils at each water table level in the test pit plate load tests should be well quantified to the same water conditions as those in the laboratory resilient modulus tests.
3. Limerock base should be compacted on the top of the stabilized subgrade prior the plate load test so as to have a direct comparison of the effect of the groundwater level on the subgrade modulus under an identical pavement profile.
4. Upon excavation of the test-pit, the density of the embankment layer should be re-evaluated to identify any possible consolidation or densification of the embankment layer due to the plate loading and compaction.

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