TECHNICAL REPORT STANDARD PAGE

1. Report No. FHWA/LA-94/283	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Investigation of the Use of Resilient Modulus for Louisiana Soils in the Design of Pavement	5. Report Date June 1994 6. Performing Organization Code	
7. Author(s) Louay N. Mohammad, Ph.D. Anand J. Puppala, Ph.D., P.E., Prasad Alavilli	8. Performing Organization Report No. 283	
9. Performing Organization Name and Address Louisiana Transportation Research Center 4101 Gourrier Avenue Baton Rouge, La. 70808	10. Work Unit No. 11. Contract or Grant No. 92-2GT	
12. Sponsoring Agency Name and Address DOTD P.O. Box 44245 Baton Rouge, LA 70804	13. Type of Report and Period Covered Final Report September 1992 - June 1994 14. Sponsoring Agency Code	

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

16. Abstract

The *1986 AASHTO Guide for the Design of Pavement Structures* has adopted the use of resilient modulus (M_r) as a fundamental property to characterize flexible pavement materials. The resilient modulus is defined as the ratio of the repeated axial deviatoric stress to the recoverable axial strain. The Louisiana Department of Transportation and Development, like many other state transportation agencies, has started implementing the AASHTO design procedure. At present, there are several types of dynamic testing devices which can be used to measure the resilient modulus. The repeated load triaxial test device is the most popular one because of the repeatability of the test results. In addition, it also allows the incorporation of field stresses.

A research study was initiated to develop a laboratory test procedure to characterize subgrade soils based on the structural properties obtained from repeated load triaxial testing. The objectives of this study were: (1) to develop a method for determination of the resilient modulus of Louisiana soil; (2) to evaluate the influence of using two separate internal measurement systems on the resilient modulus test results; (3) to provide a preliminary estimate of resilient modulus from physical properties of soil and (4) to compare laboratory determined resilient modulus with the one determined from field nondestructive deflection data. A statistically designed test factorial was used to examine the influence of the measurement system and AASHTO testing procedure on the resilient modulus test results. Two in-cell axial deformation measurement systems (one at the ends and the other one at the middle one-third of the specimen height), two AASHTO test procedures, T-292 and T-294, and two soil types (cohesive and granular) were used. In addition, this study examined the influence of moisture content and dry density variation on the test results. Three levels of moisture content and dry density were used in both cohesive and granular soils.

The results of the test program indicated that both measurement systems and testing procedures provided repeatable results. The resilient modulus of sands and silty clays determined from the middle measurement system is significantly different from that determined from the end measurement system. The resilient modulus of sands obtained from T-292 procedure is significantly different from those obtained from T-294 procedure. The influence of testing procedure and measurement systems on resilient modulus results is presented in the form of normalized factors. These factors are discussed with respect to confining stress, deviatoric stress, moisture content and dry densities. Resilient modulus prediction models using bulk stress and deviatoric stress are evaluated for the soil types tested. The influence of testing procedures, measurement systems and moisture content variations on the regression model constants are presented. Preliminary comparisons between laboratory determined resilient moduli and resilient moduli computed from field nondestructive methods showed that field methods predict lower values of resilient properties.

17. Key Words Dynamic Materials Characterization, Triaxial Testing, Resilient Modulus, California Bearing Ratio, Sand, Clay		18. Distribution Statement Unrestricted. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) N/A	20. Security Classif. (of this page) N/A	21. No. of Pages 123	22. Price

Form DOT F1700.7 (1-92)

INVESTIGATION OF THE USE OF RESILIENT MODULUS FOR LOUISIANA SOILS IN THE DESIGN OF PAVEMENTS

FINAL REPORT

By

LOUAY N. MOHAMMAD, Ph.D.
ASSISTANT PROFESSOR-RESEARCH OF CIVIL ENGINEERING
COORDINATOR, ENGINEERING MATERIALS CHARACTERIZATION RESEARCH AND FACILITY

ANAND J. PUPPALA, Ph.D., P.E.
MATERIALS RESEARCH ASSOCIATE
ENGINEERING MATERIALS CHARACTERIZATION RESEARCH FACILITY

and

PRASAD ALAVILLI
GRADUATE RESEARCH ASSISTANT

Research Report No. 283
Research Project No. 92-2GT

Conducted By

LOUISIANA TRANSPORTATION RESEARCH CENTER
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

June 1994

ABSTRACT

The *1986 AASHTO Guide for the Design of Pavement Structures* has adopted the use of resilient modulus (M_r) as a fundamental property to characterize flexible pavement materials. The resilient modulus is defined as the ratio of the repeated axial deviatoric stress to the recoverable axial strain. The Louisiana Department of Transportation and Development, like many other state transportation agencies, has started implementing the AASHTO design procedure. At present, there are several types of dynamic testing devices which can be used to measure the resilient modulus. The repeated load triaxial test device is the most popular one because of the repeatability of the test results. In addition, it also allows the incorporation of field stresses.

A research study was initiated to develop a laboratory test procedure to characterize subgrade soils based on the structural properties obtained from repeated load triaxial testing. The objectives of this study were: (1) to develop a method for determination of the resilient modulus of Louisiana soil; (2) to evaluate the influence of using two separate internal measurement systems on the resilient modulus test results; (3) to provide a preliminary estimate of resilient modulus from physical properties of soil and (4) to compare laboratory determined resilient modulus with the one determined from field nondestructive deflection data. A statistically designed test factorial was used to examine the influence of the measurement system and AASHTO testing procedure on the resilient modulus test results. Two in-cell axial deformation measurement systems (one at the ends and the other one at the middle one-third of the specimen height), two AASHTO test procedures, T-292 and T-294, and two soil types (cohesive and granular) were used. In addition, this study examined the influence of moisture content and dry density variation on the test results. Three levels of moisture content and dry density were used in both cohesive and granular soils.

The results of the test program indicated that both measurement systems and testing procedures provided repeatable results. The resilient modulus of sands and silty clays determined from the middle measurement system is significantly different from that determined from the end measurement system. The resilient modulus of sands obtained from T-292 procedure is significantly different from those obtained from T-294 procedure. The influence of testing procedure and measurement systems on resilient modulus results is presented in the form of normalized factors. These factors are discussed with respect to confining stress, deviatoric stress, moisture content and dry densities. Resilient modulus prediction models using bulk stress and deviatoric stress are evaluated for the soil types tested. The influence of testing procedures, measurement systems and moisture content variations on the regression model constants are presented. Preliminary comparisons between laboratory determined resilient moduli and resilient moduli computed from field nondestructive methods showed that field methods predict lower values of resilient properties.

IMPLEMENTATION STATEMENT

A state of the art testing capability for determining resilient modulus of soil has established. This positions LADOTD and LTRC in a leading role for the implementation c AASHTO pavement design procedure. This testing capability also provides the necessary su for the characterization of subgrade soils used in the LTRC Accelerated Loading Fa experiments. In addition, this capability provides LADOTD with an investigative tool tha be used in troubleshooting of construction projects. Based on the present experimental stu it is recommended that the AASHTO T-294 procedure be adapted for conducting res modulus test on soils.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The Louisiana Department of Transportation and Development (LADOTD), like many other state transportation agencies, uses empirical procedures in the design of highway pavements. These empirical design procedures require soil parameters like soil support value, California Bearing Ratio (CBR) and Texas triaxial value. Testing methods to obtain the soil parameters employ static compression loading of the specimen. The test methods do not adequately assess or represent the response of the pavement materials to the dynamic loading to which they are actually exposed under service conditions. Recognizing this deficiency, *the 1986 AASHTO Guide for Design of Pavement Structures* recommended using resilient modulus (M_r) as a definitive property to characterize flexible pavement materials. The resilient modulus is a dynamic response defined as the ratio of the repeated axial deviator stress to the recoverable axial strain. Sophisticated laboratory and field testing procedures are required for determining this property. The concept of using resilient modulus in mechanistic design methods provides a better understanding of pavement behavior, efficient use of pavement materials, and a reliable and realistic pavement design (Barksdale et al. 1990).

Currently, LADOTD uses a relationship developed for Louisiana soils to estimate soil support values. The relationship requires an R value which can be derived from soil classification and engineering properties. The resilient modulus can also be determined from an R value - M_r relationship developed by Temple and Shah (1987). This relationship was implemented in an automated procedure for estimating the road bed resilient modulus by simply inputting basic soil classification properties (Temple and Carpenter, 1990). This implementation has allowed LADOTD to use this software for designing flexible pavements. The above relationship was empirical and was not fully supported by the soil laboratory test results. It should also be noted that this procedure was developed as an interim design procedure and needs to be replaced with a procedure which implements AASHTO recommended methods.

After the 1986 AASHTO design guide recommendations, several transportation a started implementing laboratory testing procedures for determining the resilient moduli soils. Some of the agencies are in the process of modifying or reevaluating the existing procedures for testing their locally available soils (1988 Oregon Conference Publicatio The transportation agencies in Louisiana will be able to use AASHTO recommended laboratory procedures based on the success of the present investigation which is aimed implementing an AASHTO testing procedure to determine resilient modulus.

OBJECTIVES OF THE RESEARCH

The main objective of the research is to provide a laboratory methodology for re modulus testing which can be implemented to design flexible pavements mechanistical Louisiana. The results from laboratory methodologies will be used to validate the exis properties - M_r relationships used in Louisiana. The outcome of this research study wi development of a laboratory research tool that will provide LTRC with the ability to c diagnostic work for LADOTD and also to use it as a part of the materials characteriz process in the newly acquired Accelerated Loading Facility at LTRC. Other objectives study are to understand the influence of testing procedures (AASHTO T-294, T-292), LVDT's measurement locations and the physical soil characteristics on the resilient mo of soils. Field nondestructive testing procedures will be evaluated by comparing them laboratory results conducted on core samples.

SCOPE

Two soil types, a blasting sand and a silty clay, will be investigated under condi representing a simulation of the physical and stress states of soils beneath flexible pav subjected to moving wheel loads. The conclusions based on the test results are valid f soils tested in this investigation.

METHODOLOGY

BACKGROUND

Definition of Resilient Modulus and Significance

The resilient modulus is defined as the ratio of the repeated axial deviatoric stress to the recoverable or resilient axial strain,

$$M_r = \frac{\sigma_d}{\epsilon_d} \tag{1}$$

where σ_d is the deviatoric stress; and ϵ_d is the resilient strain. Figure 1 presents a graphical definition of the resilient modulus. This is a fundamental property of a material that is subjected to cyclic or repeated loading. Subgrade soils experience cyclic loading due to traffic loading and, therefore, this property will be more representative than static properties for characterizing such materials.

Repeated Loading Tests

The repeated loading tests are generally used to determine the resilient modulus. The shape and duration of the loading should simulate the actual traffic loading conditions. Previous investigations related to this subject showed that the traffic movement over a pavement surface can be experienced by materials beneath the pavement as an applied stress pulse. The magnitudes of stresses vary with the vehicle load, with a maximum value at a certain point in the pavement being obtained when the wheel load is directly over that point. In another case, the magnitude will be zero when the wheel load is at a sufficient distance from the point under consideration. This implies that the pavement materials are subjected to two phases of loadings. The first phase has pulse type loading with a peak load of a certain magnitude. This loading phase will be followed by a relaxation phase in which no lo... applied.

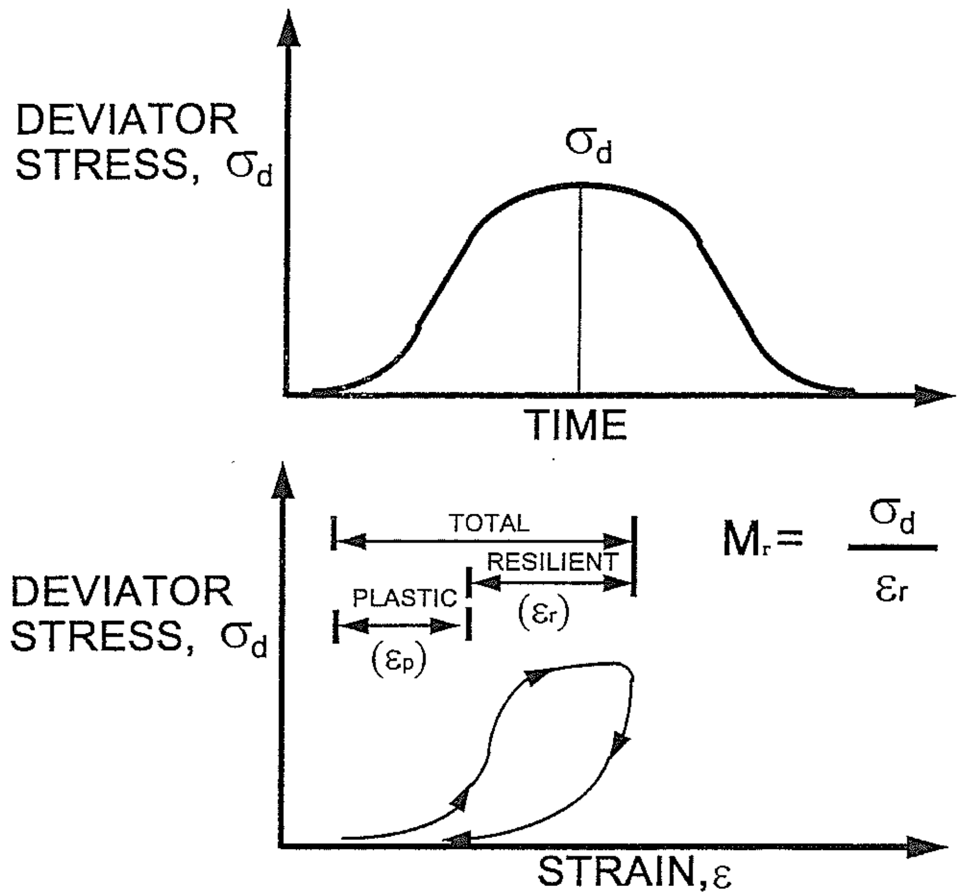


Figure 1. Definition of Resilient Modulus in a Repeated Loading Test

Several different pulse shapes are used by investigators to simulate the loading on the pavement. The pulse can be approximated by a haversine or a triangular or a square function. Vertical stress pulses measured by the AASHO road tests showed that the shape is similar to a haversine. Recognizing this, AASHTO recommended a haversine shape for pulse loading. The AASHTO testing procedures based on previous investigations recommended a loading period of 0.1 seconds and a relaxation period of 0.9 seconds. It should be noted that the repeated load tests differ from cyclic tests with respect to the loading type used. Repeated loading has only loading characteristics where as cyclic loading has both loading and unloading characteristics. Though both loading types were used in the resilient modulus testing procedures, the cyclic loading is the type of loading that is expected due to reciprocating machines and earthquakes. This cyclic loading is not representative of traffic loading and, thus, not recommended in the current test procedures.

Another interesting aspect of this testing is the use of AASHTO procedures (T-294, T-292, T-274) to conduct tests. Since their introduction, these procedures have been subjected to several criticisms and discussions (Houston et al. 1992). As a result of these discussions, old procedures were modified and released as new procedures. An example is the sequence of applying the confining and deviatoric stresses in the T-292 was modified in the T-294 procedure. Some transportation agencies and groups use their own procedures for the testing since AASHTO procedures resulted in the disintegration and breaking of the specimens (Ho 1988).

The stress states recommended by the AASHTO procedures involve testing the specimens at different confining and deviatoric stresses. The confining lateral stresses are caused by residual lateral stresses developed because of construction and compaction procedures. Traffic loading and the viscous damping nature of the materials also influence the residual lateral stresses. The largest residual stresses appear to be developed in granular materials due to large compaction stresses, traffic loading, and visco-elastic material behavior (Barksdale et al. 1990). The same for clays is also quite significant and this has been pointed out by Uzan (1985) and Duncan and Seed (1986). The general practice in geotechnical engineering is to estimate the residual lateral stress by taking the product of vertical stress and earth pressure coefficient at rest. The samples will be tested at this calculated lateral pressure. The deviatoric stresses are mainly induced by vehicle loads and are estimated using elastic analysis.

The AASHTO procedures (T-274-1974, T-292-1981, and T-294-1992) require several combinations of confining and deviatoric stresses to be performed on laboratory prepared specimens. The deviatoric and confining stresses are selected based on an elastic analysis of different traffic vehicle loads, soil types and their locations. The range of these stresses is significantly larger for granular sands than for cohesive soils. This is attributed to the stress state of soils which influence their soil strength. For example, sands derive their strength mainly through the frictional characteristics which in turn depend on the stress states at which they are tested. Cohesive materials, on the other hand, derive their strength mainly through the cohesive property rather than from the frictional characteristics and confining pressures. Therefore, the range of stresses for cohesive soils is significantly smaller than that of granular soils. For field core samples, the stresses expected in the field can be determined by performing a non-linear elastic analysis on the in situ pavement system. Tests will be then conducted at these analyzed stresses. In cases where the artificial laboratory specimens fail at a certain deviatoric stress, the number of stress states can be adjusted by considering the failure deviatoric stress as the limit stress. Detailed description of such case can be found in Houston et al. (1992).

The influence of overloading on the specimen by normal and shear stresses is explained in detail by Houston et al. (1992). They concluded that the damage caused by shear overstressing is more significant than that caused by normal stress. In the field, the traffic loads produce increments in both shear and normal stresses. In these cases, the specimens should be tested at stresses closer to these incremented stresses. Houston et al. (1992) concluded that the T-274 procedure recommended stress levels which are significantly higher than the in situ pavement stresses. Therefore, the T-274 procedure yields moduli values which will be significantly different from realistic moduli results. Houston et al.

(1992) recommended a new conditioning procedure for field core samples. This procedure uses an elastic multi-layer analysis on the field pavement site and then prepares a stress triangle. The conditioning stresses depend on the stress triangle values. Further detailed descriptions on this procedure can be found in Houston et al. (1992).

The number of cycles of conditioning and testing phases are different for each AASHTO procedure. The conditioning eliminates the imperfect contacts between end platens, minimizes disturbance effects caused by sampling or laboratory preparation, and, more importantly, reduces the plastic strain development. Houston et al. (1992) reported that the number of cycles prescribed by an earlier procedure, T-274 (200 cycles of conditioning) was not adequate enough to reduce plastic strains and it should be increased to 1000 to 2000 cycles. Both T-292 and T-294 procedures adapt conditioning with 1000 cycles of a certain deviatoric loading at a certain confining pressure. The number of cycles are the same for both granular and cohesive soils.

The testing phase requires 50 cycles (in the case of T-292) and 100 cycles (in the case of T-294) of deviatoric loading. The resilient strains are measured for the last five cycles and then averaged to calculate the resilient modulus. The number of cycles for testing can be increased or decreased based on the plastic strain criterion. If significant plastic strains are still measured (greater than 10 percent), the number of cycles in the testing phase needs to be increased.

Previous Investigations

Several laboratory methods such as triaxial, simple shear, torsional shear using the on any plane can be determined (Barksdale et al. 1990). In addition, the triaxial eq provides repeatable and reliable results and also allows the measurements to be tak and radial directions.

Other equipment such as resonant column and torsional shear test devices are in determining the M_r values. These methods appear to provide reasonable results strains (less than 0.1 %) need to be measured (Barksdale, 1990). The major proble associated with these tests, including the trixial test is their incapability of simulatio principal axes rotation. These axes rotation is graphically illustrated in Figure 2. W wheel load is exactly on top of a certain point in the pavement, the major principa acts in a vertical direction at that point. When the load is away from the point, the stress acts at a certain angle. Therefore, the movement of a vehicle wheel over a po rotates the principal axes at the point. The hollow cylinder test can be used to simu movement of axes rotation. However, this test is tedious, complicated and requires instrumentation (Barksdale et al. 1990). These are probably the reasons that this tes usage. Although the triaxial test has fixed principal axes, it still offers a procedure represent certain extreme stress conditions expected in the field. Further details on t equipment can be obtained from the NCHRP report documented by Barksdale et al.

Other aspects in the laboratory testing involve the investigation of the influenc types and their characteristics on resilient behavior. The clay content, Atterberg Indi density, water content, and degree of saturation have been investigated (Baladi, 198 Thompson, 1989; Vinson, 1989). Table 1 also presents a summary of major finding various investigations on these aspects. Most of the studies concluded that the granu shows an increase in the resilient modulus with an increase in the deviatoric and con stresses and cohesive soils display a decrease in M_r with an increase in the deviatoric (Ho, 1989; Dhamrait, 1989; Pezo et al., 1992).

The triaxial tests use an external measurement system for monitoring vertical displacements since the external system is easy to use and allows resetting the initial

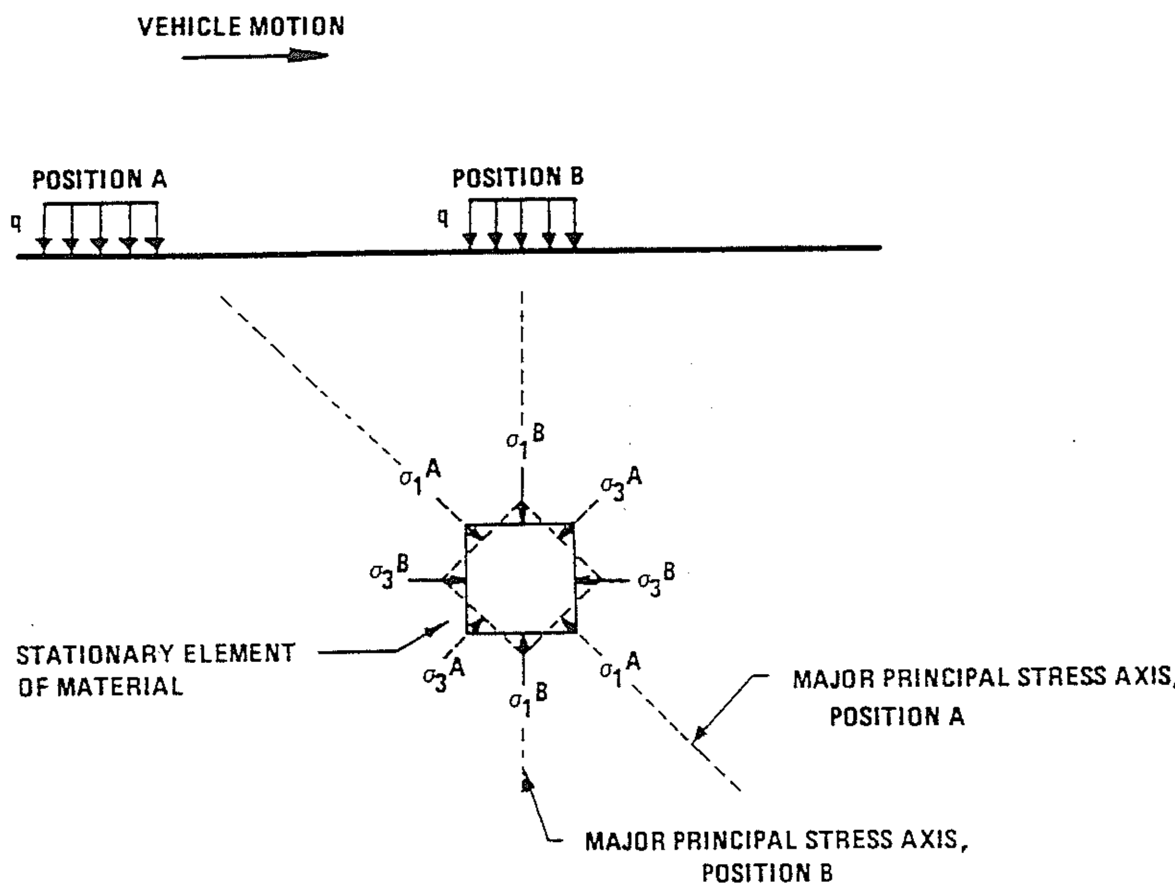


Figure 2: Principal Axes Rotation During Traffic Loading

to zero prior to the start of the test. However, the external measurement system is g influenced by system compliance errors and end friction effects which induce signifi errors into the measurements. Previous investigations reported that several different sensors and their locations on the specimen are evaluated to reduce the errors in the measurements (Barksdale et al. 1990). One of the suggestions culminating from thes investigations was to adapt an internal measurement system placed on the middle one height of the specimen in the triaxial cell. This will reduce the system compliance er end friction effects. Recognizing this, the T-292 procedure recommended the use of internal LVDT measurement systems, one placed at the ends of specimens and the ot the middle one third points of the specimen. These systems are hereafter known as en system and middle system, respectively. Although external measurement LVDT syste recommended in the new T-294 procedure, further research studies are still needed to understand the influence of measurement systems on the resilient modulus of soils. Th authors believe that the measurement system influence will be more significant on the resilient response of soft cohesive soils. This is because the cohesive soils are subject stresses of lower magnitudes which generate significantly smaller resilient strains. The it is necessary to have a measurement system which can capture small displacements strains. Hence, it is decided to adapt the internal measurement systems in the testing.

Nondestructive Testing (NDT)

Problems in simulating the in situ conditions, in particular, the moisture content, dry density, and the loading history similar to those in the subgrades have led the investigators to use in situ nondestructive testing methods. The nondestructive testing methods have been used by highway engineers to determine the structural capacity and integrity of pavements. These methods involve applying dynamic loading and measurin corresponding deflections. The moduli of the surface layer, base, and subgrades are the evaluated by conducting an elastic analysis on the pavement system. The Dynaflect and Falling Weight Deflectometer are two currently and frequently used nondestructive dev A decription of them is given below.

Table 1. A Summary of Resilient Modulus Tests Conducted on Soils

Reference	Types of Equipment	Soils Tested	Parameters Studied	Procedure Used	Major Conclusions
1) Seed et al., (1967)	Repeated load triaxial tests and plate load tests	Coarse grained and fine grained soils	σ_3, σ_d, water content, degree of saturation	--	1. Plate load tests on clay soils show that the modulus varies extensively with applied pressure and water content. 2. Resilient deformations from plate load tests of base course increases with confining pressure, thickness of base and the degree of saturation. 3. At equal levels of stress applications, the values from plate load and triaxial tests for subgrade cohesive soils are same. 4. Triaxial tests showed that both soil M_r values vary with stresses. 5. Theories using laboratory M_r values predicted measured deformations quite well.
2) Marshall Thompson and Robnett, Q. C. (1979)	Pneumatic repeated dynamic tests	Different Illinois soils	PI, liquid limit, unconfined compression strength, S_r, compaction effects	--	1. Several correlations between M_r and soil properties were developed. 2. The influence of increased compaction on resilient properties was demonstrated.
3) Pumphrey, Jr, N.D., and Lentz, R. W. (1986)	Repeated load triaxial tests	Florida subgrade sands	σ_3, σ_d, dry unit weight, water content, number of cycles	--	1. Increased confining pressure causes no change in permanent strains for low stress ratio tests and decreases the strains for high stress ratio tests. 2. The permanent strain increases with the number of cycles. 3. At optimum and below optimum, the increase in density causes a decrease in permanent strain. 4. Effects of moisture content on permanent strains cannot be stated due to limited data and conflicting results.
4) Elliott et al., (1988)	Repeated load triaxial tests	Arkansas subgrade soils	σ_3, σ_d, PI, dry density, freeze thawing, compaction method	T-274	1. Moisture content, freeze thaw and deviatoric stress have major influence on the M_r of soils. 2. Density within the range of 95 to 100 percent of maximum density does not have a significant effect on M_r. 3. The number of cycles in T-274 can be reduced.
5) Ho, R. K. H., (1989)	Triaxial system with MTS accessories	Florida soils	σ_3, σ_d, water content, LVDT system	T-274	1. The M_r values seem to be independent of number of repetitions. 2. M_r values computed from internal and external LVDTs are different. 3. Conditioning is too severe for these soils.
6) Dhamrait, J. S. (1989)	Triaxial testing equipment	Illinois subgrade soils	Amount of clay fraction, PI, water content	--	1. Several correlations between M_r and the amount of clay fraction, PI and water contents are prepared. 2. M_r equations are also developed as a function of FWD displacements.

Reference	Types of Equipment	Soils Tested	Parameters Studied	Procedure Used	Major Conclusions
7) Seim, D. K. (1989)	Triaxial testing equipment	Sand and miller clay	σ_3, σ_d	T-274	1. Skilled technicians and periodic calibration checks are essential for obtaining good results. 2. In sands, greater variability of M_r values are found at deviatoric stresses lower than 5 psi. 3. Statistical analysis showed that confining pressure, rather than deviatoric stress has more influence on M_r value.
8) Woolstrum, G (1990)	Triaxial tests	Nebraska soils and aggregates	σ_d, Group Index, water content	--	1. The M_r value can be reasonably determined by indirect methods. 2. The Nebraska Group Index is a reliable indicator of the M_r of soils with a Nebraska Group Index range of -3 to 28 3. Dynaflect results show no variation with M_r values.
9) Thompson, M. R. and Smith, K. L. (1990)	Repeated load triaxial tests	Granular materials	Gradation, number of load repetitions, σ_3 and σ_d	--	1. The resilient modulus parameters, k and n fall within the expected range. 2. The shear strength of a conditioned sample is significantly higher than an unconditioned sample. 3. Permanent deformations under repeated loading provides a more definitive evaluation of granular base materials than does shear strength and M_r values.
10) Andrews, R. C., Mundy, M.J. and Stacy, W.S. (1992)	Repeated triaxial testing	Granular materials	σ_3, σ_d, water content and cement content	--	1. Importance of asphalt life (fatigue) and permanent deformations (rutting) of subgrades are discussed. 2. Selection of cement content should be based on stiffness characteristics.
11) Pezo, R.F., Claros, G. and Hudson, W.R. (1992)	MTS testing with triaxial cell	Texas subgrade soils	σ_3, σ_d, compaction methods, effect of grouting	T-274	1. Grouting the specimens to the end platens is justified for accurate measurements of modulus. 2. Sample conditioning can be eliminated, if grouting is used. 3. Only a few stress repetitions are sufficient for accurate estimation of modulus values.
12) Kim, D. S, and Stokoe, II, K. H. (1992)	Resonant column tests (RC) and torsional shear tests (TS)	Compacted subgrade soils	Testing methods, σ_3, σ_d, strain amplitudes and loading frequency	--	1. The elastic threshold strain below which M_r values are independent of strain amplitudes increases with increase in PI. 2. M_r increases linearly as a function of logarithm of loading frequency. 3. Moduli obtained from RC and TS agreed well at strains above 0.01% with Triaxial values.
13) Houston, W. N., Houston, S. L. and Anderson, T. [illegible]	Triaxial system	Arizona subgrade soils	σ_3 and σ_d	T-274	1. Non-linear response of subgrade soils is reduced with the load repetitions. 2. Preconditioning from T-274 was inadequate to remove plastic strains.

Reference	Types of Equipment	Soils Tested	Parameters Studied	Procedure Used	Major Conclusions
14) Reed, L et al., (1992)	Triaxial system	Four sands with different gradations	Gradation, number of repetitions, σ_3 and σ_d	T-274	1. Dynamic loading induces pore pressure in the layers which reduces effective stresses, thereby reduces frictional resistance. 2. Aggregate gradation has major influence on the excess pore pressure generated. Open graded aggregates seem to be more resistant to pore pressure generation than dense graded aggregates. 3. The M_r value decreases with an increase in pore pressure ratios.
15) George, K. P., (1992)	Gyratory testing	Different soils	Soil gradation, stress state, moisture content, testing methods	T-274	1. Comparison between M_r values from triaxial testing and gyratory testing methods showed reasonable agreement. 2. M_r from gyratory tests are less influenced by the moisture content unlike the same from triaxial testing. 3. Gyratory testing simulates shear stress reversals very well, a phenomenon that occurs under traffic loads.
16) Mohammad et al., (1993, 1994)	MTS testing with triaxial accessories	Silty clay and blasting sand	σ_3, σ_d, LVDTs location, testing procedure, type of soil	T-292 and T-294	1. The influence of measurement system (location) is more evident on silty clays than on sands. 2. Influence of testing procedure have some effect on both soils. 3. Regression model constants can be used to understand the influence of testing procedures and LVDT locations.

Note: σ_3-confining stress; σ_d - deviatoric stress; w - water content; PI - plasticity index; S_r - degree of saturation

Dynaflect

This is the most commonly used nondestructive equipment in the United States. I a steady state vibratory device and consists of a force generator and five geophones housed in a small trailer which is towed by a light vehicle. The loading system consis two counter rotating eccentric masses. A load of 4.45 kN at a frequency of 8 cps is applied through two steel wheels that are 0.51 m apart. The resulting deflections caus by this loading are measured by the geophones mounted on a trailer bar at 30 cm intervals. The first geophone is located midway between the loading wheels. An elastic analysis is generally performed for the interpretation of the Dynaflect results to estima moduli of the surface layers, base, and subgrade. This analysis assumes that the soil in subgrade is an elastic, uniform half-space or an elastic stratum of finite thickness. The measured deflection basin parameters are used for checking the service integrity of the service pavements, evaluating the pavement response, and measuring the moduli pavem layers.

Falling Weight Deflectometer (FWD)

This is a deflection testing device which operates on the impulse loading principle. The test consists of dropping a known mass from a predetermined height. The falling weight strikes a plate placed on the pavement and transmits a force to the pavement. Th deflections will be measured by the geophones placed at several radial distances from th center of the plate. The FWD profile is then used for backcalculation of the moduli of pavement layers. Multi-layer theories are used in determining these moduli using linear-elastic iterative computer programs. The advantage of FWD over Dynaflect is the abili to apply variable and heavier dynamic loads and variable energies. The advantages and disadvantages of the NDT methods are presented below (Smith 1992; Irwin 1992).

Advantages:

1) Can be used in the design of pavements for determining the acceptable deflections, elastic layer characterization, and structural numbers.

2) Can be used as a diagnostic tool to locate areas of localized problems during and after construction.

3) Can be used in the analysis of performance of pavements in the areas such as seasonal load limits on paved and unpaved roads, joint load transfer across joints, and the loss of support for concrete pavement slabs.

4) Can be used in identifying the amount of stabilization to the subgrades and in determining the need for rehabilitation or overlay based on changes in deflection.

5) Can be used in mechanistic design, predicting the remaining life, and changes in the layer properties.

Limitations:

1) Method is not sensitive to thin layers.

2) Cannot identify adjacent layers of similar moduli.

3) Because of large modular ratios, this method provides erroneous results for the tests conducted on soft layers beneath rigid pavements.

4) Backcalculation software still needs to be corrected to obtain realistic results.

In order to overcome a few of these limitations, several ideas were suggested by Irwin (1992). They include modifying the equipment with rolling wheel loads and improving the analytical backcalculation subroutine software. The latter approach has been under investigation over the past few years (Maestas and Mamlouk 1992; Zhou et al. 1992; Hossain and Schofield 1992). The proposed experimental field study program will attempt to compare the back calculated results of the nondestructive tests using MODULUS software and laboratory tests.

EXPERIMENTAL SETUP AND ITS ACCESSORIES

Repeated loading triaxial tests are conducted using an MTS test system. Descrip of this equipment is given below.

MTS Equipment

An MTS model 810 closed loop servo-hydraulic material testing system is used applying repeated loading. The major components of this system are the loading syste digital controller, and load unit control panel.

Loading System

The MTS loading system consists of a load frame and hydraulic actuator. The dynamic force and displacement ratings of this system are 100 kN (22 kips) and ± 75 (± 3 in.), respectively. Figure 3 presents a photograph of the loading system. The loac frame is a free-standing, self-supporting, two column type unit with a moveable crosshead. The hydraulic actuator is supported on this crosshead. The load cell is attac to the end of the piston rod of the actuator and located inside the triaxial chamber.

Digital Controller

Figure 3 also shows the digital controller of the equipment. This acts as an inter between the computer and the rest of the system. The interface includes a machine con either in displacement or force mode, conditioning of sensors, and connections for external equipment. It also provides control of the hydraulic power supply and the hydraulic service manifold and provides 16 channels for analog inputs and outputs. Th computer using the machine software, *TESTSTAR*, downloads the program code to the

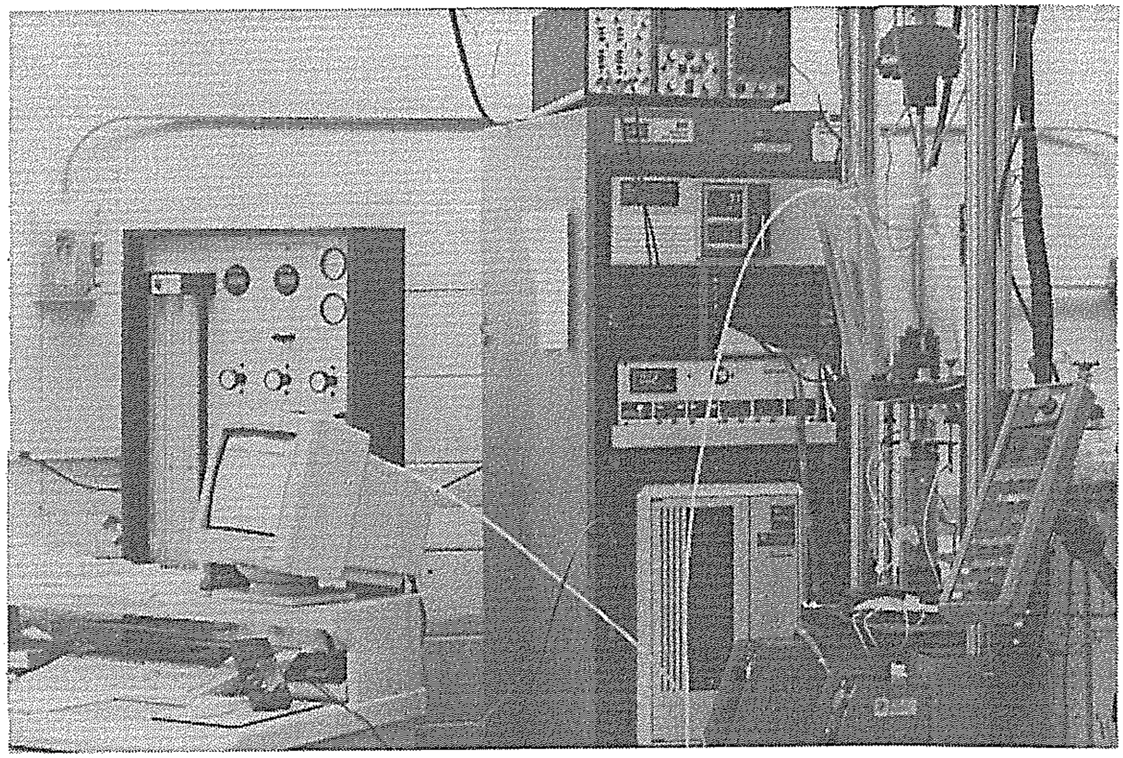


Figure 3: Photograph of MTS Equipment and Schematic of Testing Operation

digital controller. This provides the digital controller with the code that controls the whole system.

The test equipment uses a closed loop control system for applying forces to the specimen. The digital controller acts as a stabilizing unit in this operation. Figure 4 shows the schematic of the closed loop control action. A simplified closed loop control consists of a controlling element, which in this case is the computer and digital controller and a controlled element, which consists of a servovalve, hydraulic actuator, and test specimen.

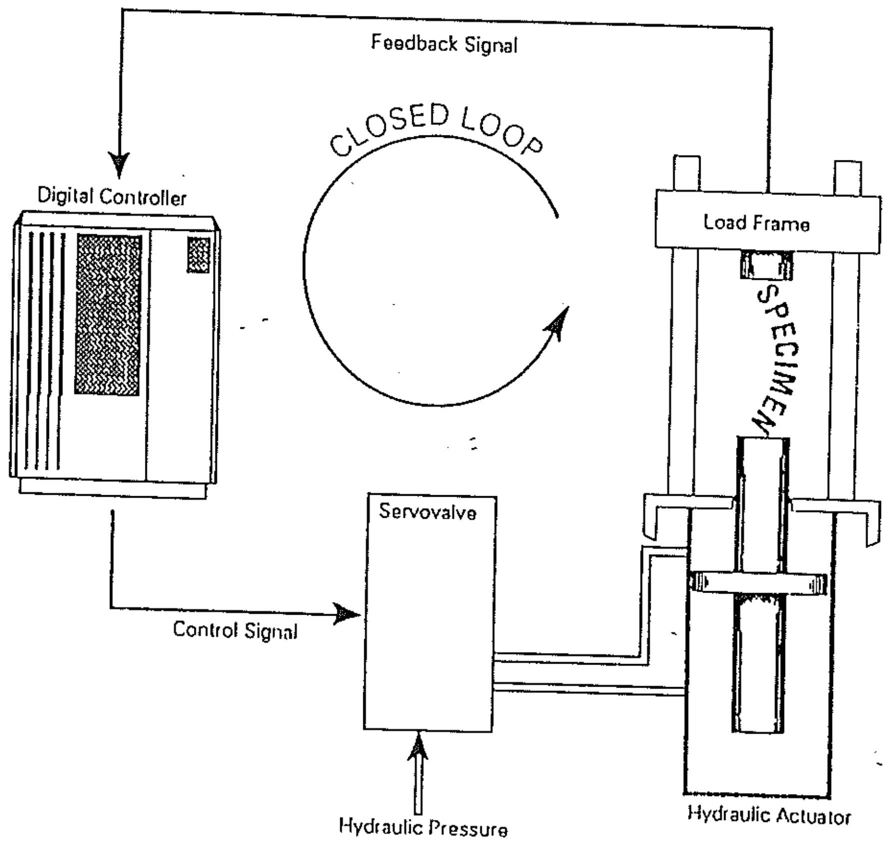


Figure 4: Schematic of the Closed Loop Control System

When the load is applied to the specimen, the feedback sensors send the signal back to the digital controller. The controller compares the feedback with the control signal and adjusts the control signal to correct the differences. This procedure provides accurate repeated loading magnitudes.

Load Unit Control Panel

This panel allows the users to control the load hydraulics while placing the specimen inside the triaxial cell. It has a 13 line by 40 characters long LCD display which shows machine status and custom messages. A switch and control knob on this panel can be used to manage the actuator position for specimen installation. It also has switches to control program start, stop, hold, and resume operations. Other accessories used in the experimental program are listed below.

Triaxial Cell

The plexiglas triaxial cell, manufactured by Research Engineering, is 203 mm (8 in.) in diameter and 330 mm (13 in.) in height. This cell has features that allows in-cell axial displacement measurements and can accommodate samples of 71.1 mm (2.8 in.) in diameter. Confining pressures of up to 700 kPa (98 psi) can be applied in this cell. Compressed air is used as the medium since it is easy to work with and requires no special insulation for in-cell measurement devices and other electrical connectors.

Pressure Control Panel

The control or pressure panel is used in applying the confining pressures to the specimen. Pressure regulators on the panel are used for this purpose. The minimum pressure that can be applied with this system is 0.35 kPa (0.05 psi). This control panel also has a venturi type pressure pump which will provide suction pressures. Figur shows a photograph of the triaxial cell and control panel.

LVDTs and Load Cell

In the recent AASHTO testing procedure, T-294, the LVDTs for measuring displacements are recommended to be placed outside the triaxial chamber. This ex measurement system is easy to install and also provides a simplified procedure to externally reset the initial LVDT readings to zero without having to remove the tri chamber. However, the external LVDT location results are significantly influenced friction effects and system compliance errors (Barksdale et al. 1991). One of the suggestions to minimize these errors is to use internal LVDTs in place of external LVDTs. Thus, in this study, the internal LVDT systems were selected and used.

The measurement systems have two diametrically placed internal LVDTs. One system is used to measure displacements at the ends of the specimen and the other is placed at the middle one-third point of the specimen. The LVDTs of the middle sys have a full scale stroke of ± 3.05 mm (± 0.12 in.) with a non-linearity of ± 0.007 mm (±0.0003 in.). The LVDTs of the end system have a full scale stroke of ± 6.35 (± 0.25 in.) with a non-linearity of ± 0.0158 mm (±0.000625 in.). The output from each LVDT was monitored independently and compared with the output of the other LVDT of the same system. If the difference between the axial deformations was not within the assigned tolerance, then the tests were discarded. This ensures good seating uniform loading on the specimen. An internal load cell with a capacity of 1.36 kN (300 lbs) is used. This allows the researchers to apply even small loads without much disturbance.

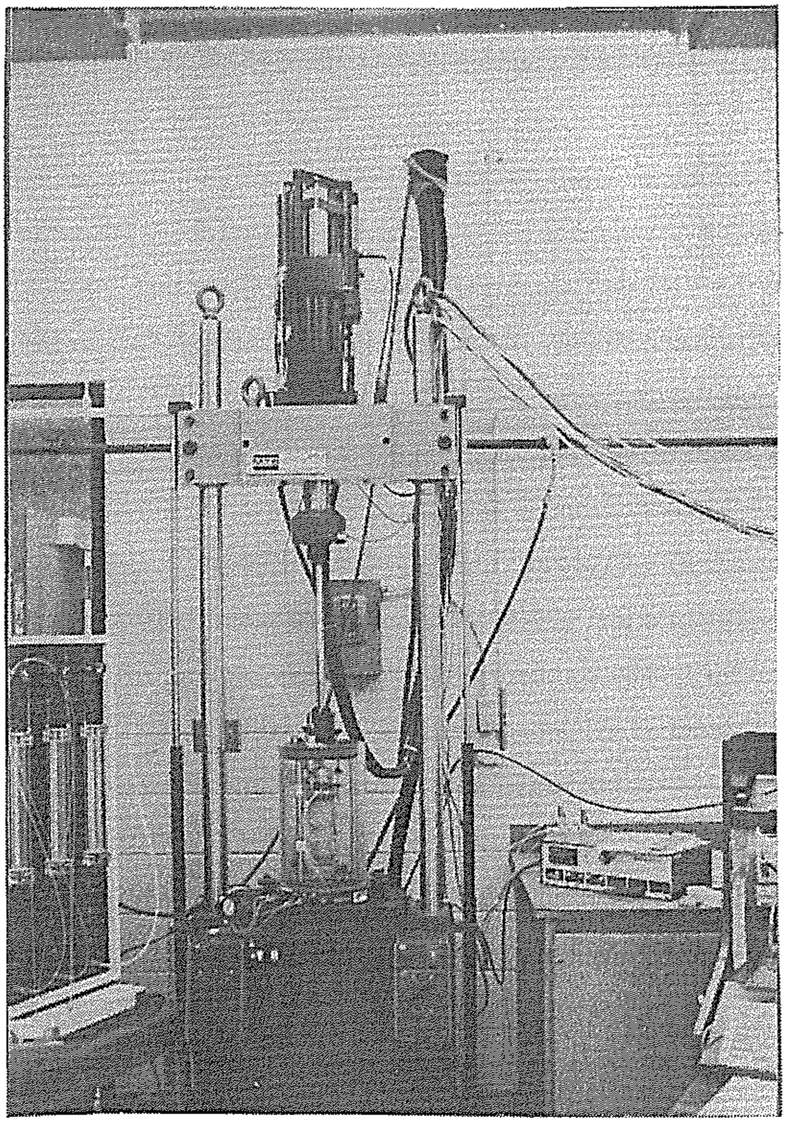


Figure 5: Photograph Showing the Triaxial Cell and Control Panel

DATA ACQUISITION AND EQUIPMENT CONTROL

The data acquisition system plays a major role in determining the M_r. Since th is of a repeated loading type, an accurate and faster sampling data acquisition system required to capture material response. The data acquisition system consists of a sign conditioner, data acquisition board, and software for equipment control, data reducti and analysis.

Signal Conditioner

A signal conditioner provides excitation signals for the LVDTs and the pressu transducers and amplifies the low level output signals from these measurement devic high level signals. High level signals can be carried long distances without causing m noise. Hence, a signal conditioner is kept close to the testing equipment. A ± 5 volt is used for all sensors. The loads are monitored from the MTS system and have a fix output level of ± 10 Volts for ± 1.36 kN (300 lbs).

Data Acquisition Board

A 486-based microcomputer is used together with a 12-bit interfacing board fro Metrabyte to collect, store, and analyze the data. Servovalve, strain gages, and LVDT signals are interfaced to the Metrabyte board through the signal conditioner. Custom application software was developed using the drivers and routines supplied by this bo This board has adjustable gain settings which can be used for achieving the required resolution. The minimum values which can be read from this system for both LVDTs pressure transducer, and load cell are 0.00309 mm (0.00012 in.), 0.00104 mm (0.00 in.), 3.3 N (0.73 lb) and 0.35 kPa (0.05 psi), respectively.

Software

The *TESTSTAR* software, provided with the equipment, is used for data acquisition and equipment control. The testing templates or procedures are written using the features in *TESTSTAR* software. Templates are made separately for conditioning and testing phases of M_r tests. For accurate measurements of high frequency dynamic loading and micro deformations, a sampling rate of 500 Hz per signal was used.

Data analysis for each confining pressure and deviatoric stress is performed by scanning the test data and analyzing it to determine the peak loads, resilient or elastic, permanent deformations, and resilient modulus properties. The software also provides a mechanism for verifying the sample alignment. This is done by comparing the deformations of both LVDTs from the end and middle measurement systems. Any significant differences in the deformations suggest that the sample is not properly aligned.

It should be mentioned that the first few tests in this study were conducted using a different MTS system, termed hereafter as old equipment. The old equipment is similar to the equipment described in this chapter. The replacement equipment is referred to as new equipment in this report. Results from similar tests conducted with both systems showed that the equipment variability is minimal and can be neglected.

AASHTO TESTING PROCEDURES

In the past, the T-274 procedure was generally used for M_r testing. Ever since the 1986 AASHTO recommendations, several research studies have been attempted to modify the old testing procedures. This, along with the SHRP related research resulted in the development of two AASHTO procedures, T-292 and T-294. The T-294 procedure is a modification of the T-292 procedure with respect to conditioning and testing stress sequence.

The tests on both soil types were performed at the confining and deviatoric stres levels recommended in the latest versions of AASHTO T-292-91 and T-294-92. The s samples were conditioned by applying one thousand repetitions of a specified deviator stress at a certain confining pressure. Conditioning eliminates the effects of specimen disturbances from sampling, compaction, and specimen preparation procedures and minimizes the imperfect contacts between end platens and the specimen. The specimen then subjected to different stress sequences. The stress sequence is selected to cover th expected in-service range that a pavement or subgrade material experiences because of traffic loading.

The conditioning on sands is performed at a higher confining and deviatoric stre The tests of T-292 are conducted from higher confining pressure level (140 kPa) to lo confining pressure level (21 kPa), whereas for T-294, the tests are conducted in the reverse order. In order to differentiate, these stresses are plotted in the form of bulk stresses per sequence/step order of testing (Figure 6). It should be noted that the T-29 sequence appears to have a higher magnitude of stress differences from step to step.

The tests for clays show lower magnitudes and ranges of stresses because cohesi soils are not as stress dependent as granular soils. Conditioning for both procedures is conducted at the same confining pressures. However, in the testing phase three confini stress levels are used in a T-294 test and only one confining pressure level is used in 292 test. The bulk stress variation of these procedures is presented in Figure 7.

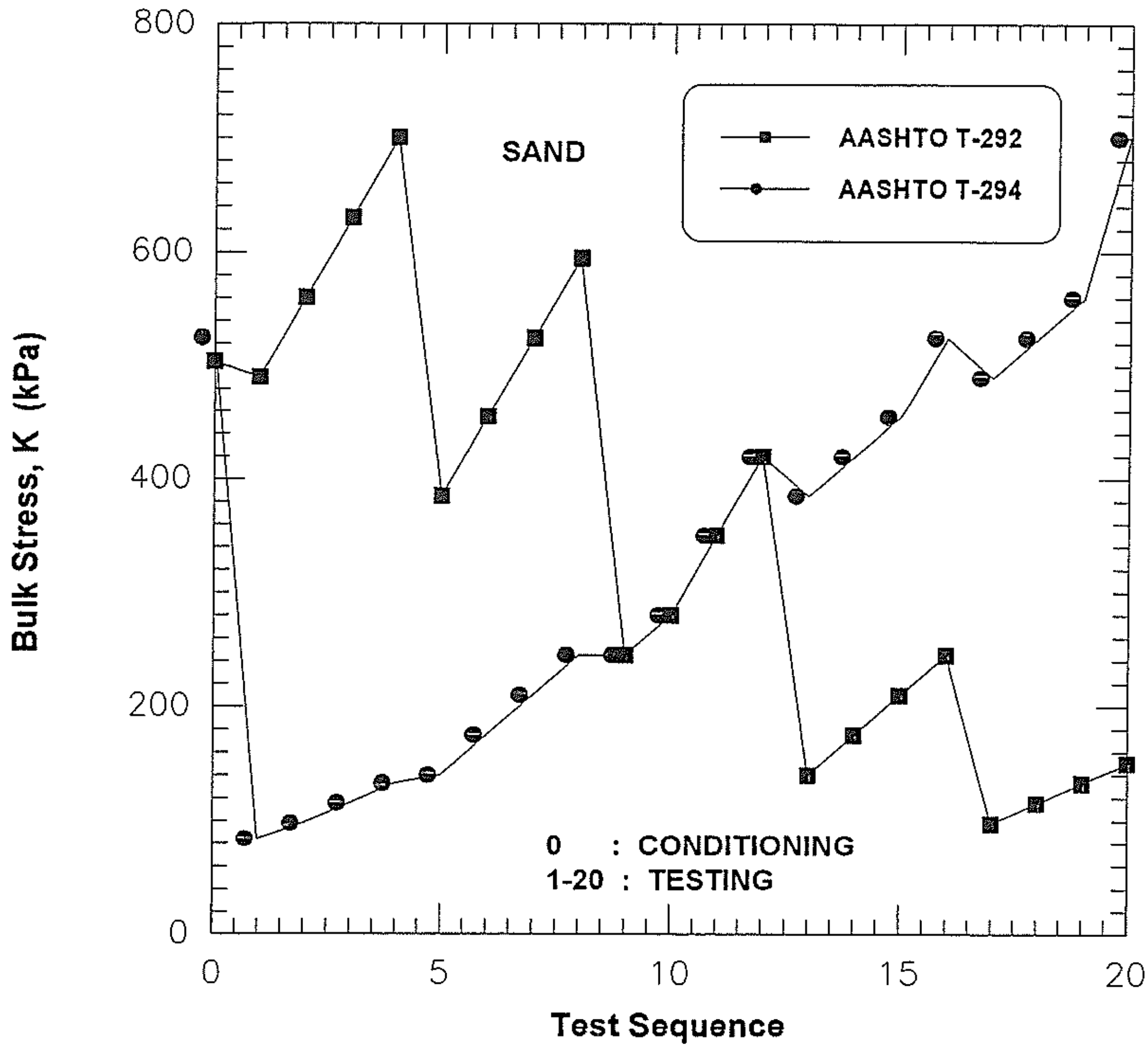


Figure 6: Bulk Stress Variation of AASHTO Procedures in Sands

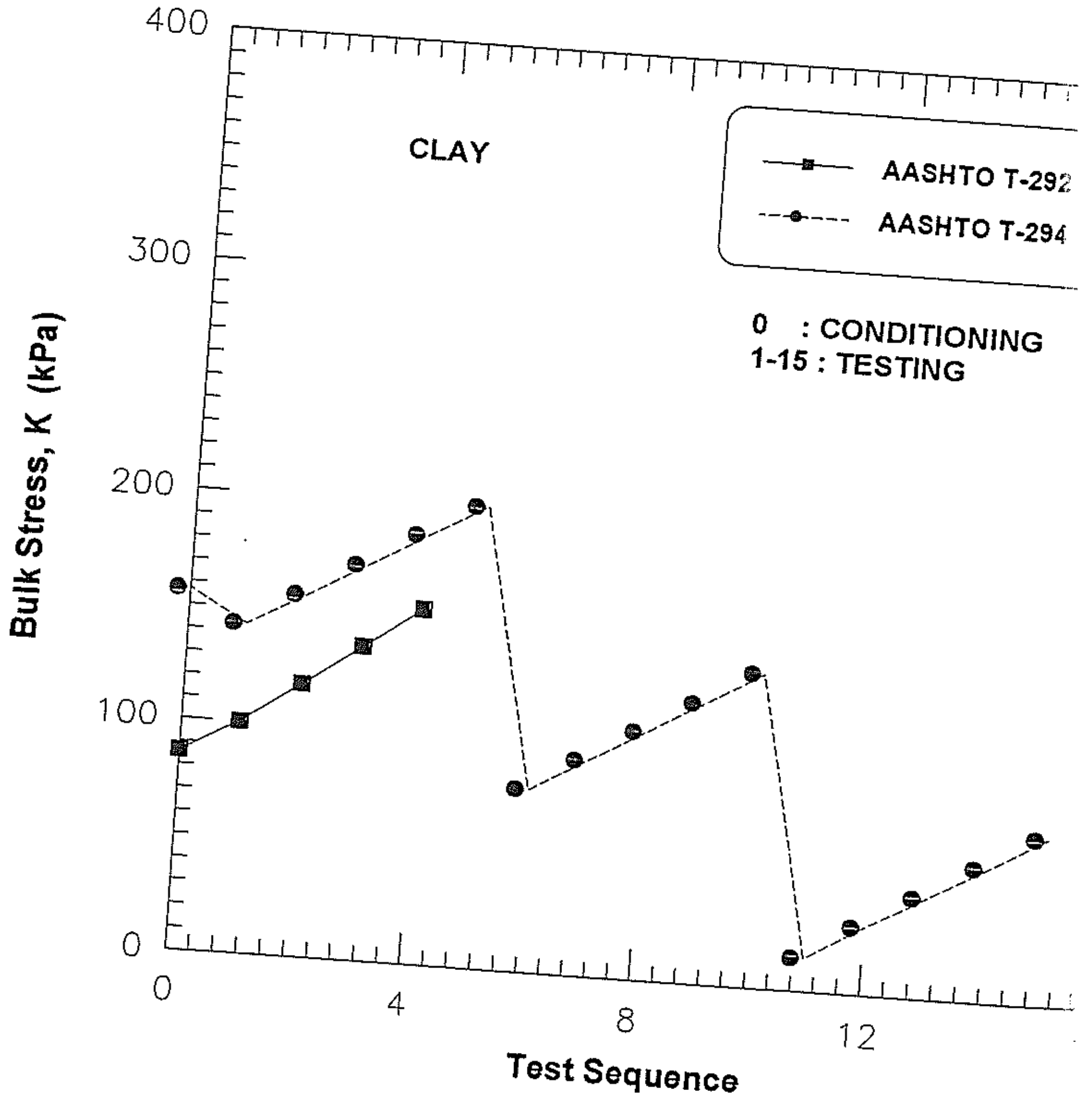


Figure 7: Bulk Stress Variation of AASHTO Procedures in Clays

LABORATORY TESTING PROGRAM

Variables Studied

The variables studied in these experiments are soil characteristics, AASHTO testing procedures and measurement systems.

Description of the Soils Used

Two locally available soil types, a uniform blasting sand and a silty clay, were used in this study. The blasting sand exhibited dry densities of γ_{max} = 17.7 kN/m^3 (110.9 pcf) and γ_{min} = 15.8 kN/m^3 (99.0 pcf) from maximum and minimum soil density tests, respectively. The silty clay had an optimum water content of 21.2 percent, a maximum dry density of 16.0 kN/m^3 (100.3 pcf) and a plasticity index of 22. The silty clay and blasting sand were classified as A-7 and A-3, respectively, in AASHTO classification. The grain size distribution and standard proctor density curves of these soils are given in Figures 8, 9, 10 and 11. It should be mentioned that the blasting sand tested with the old MTS equipment is slightly different from the sand used with the new MTS equipment. The variation is observed with respect to the optimum density and optimum moisture content levels assumed to be caused by the variations in sampling of these soils.

Specimen Preparation

Specimens were prepared using a compaction method. The sand specimens were compacted in-place in the triaxial cell to reduce sample disturbance. The cohesive specimens, on the other hand, were compacted in the steel molds and were carefully extruded for testing. Both specimens were 71.1 mm (2.8 in.) in diameter. A height to diameter ratio of two, as required by AASHTO procedures, was used to reduce the end friction effects.

Both specimens were compacted at various water contents and dry density combinations. A list of them are given in Table 2.

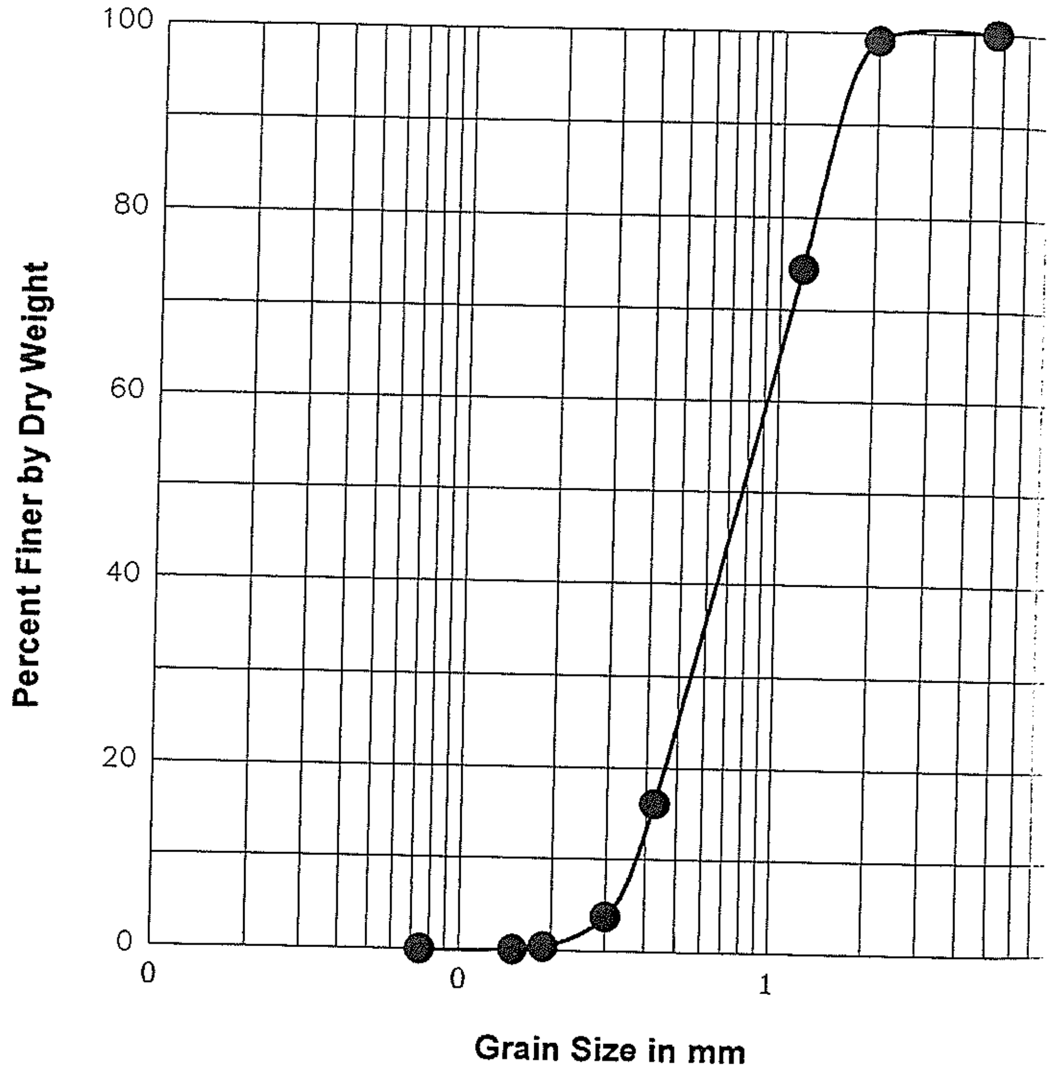


Figure 8: Grain Size Distribution of Blasting Sand

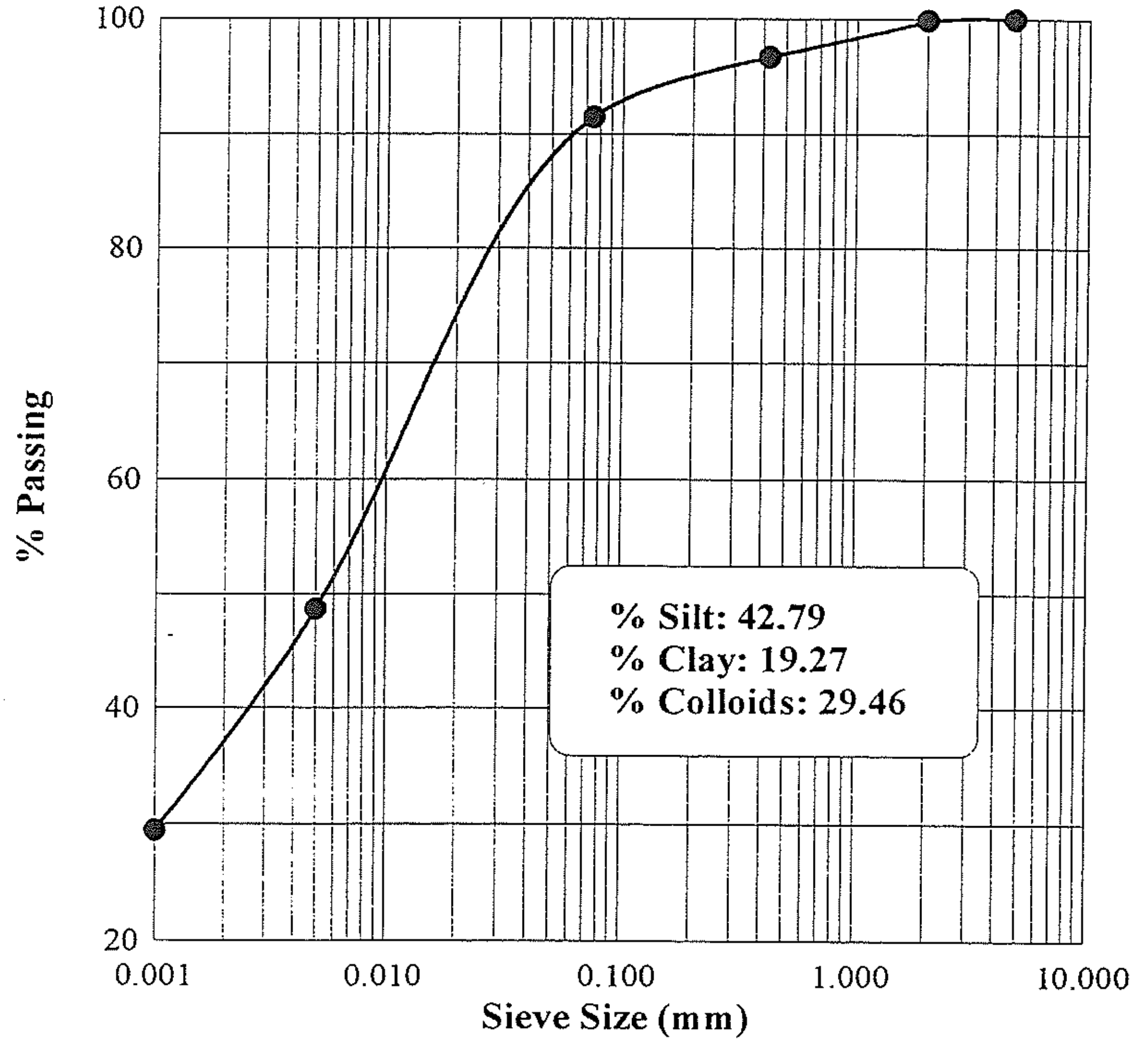


Figure 9: Grain Size Distribution of Silty Clay

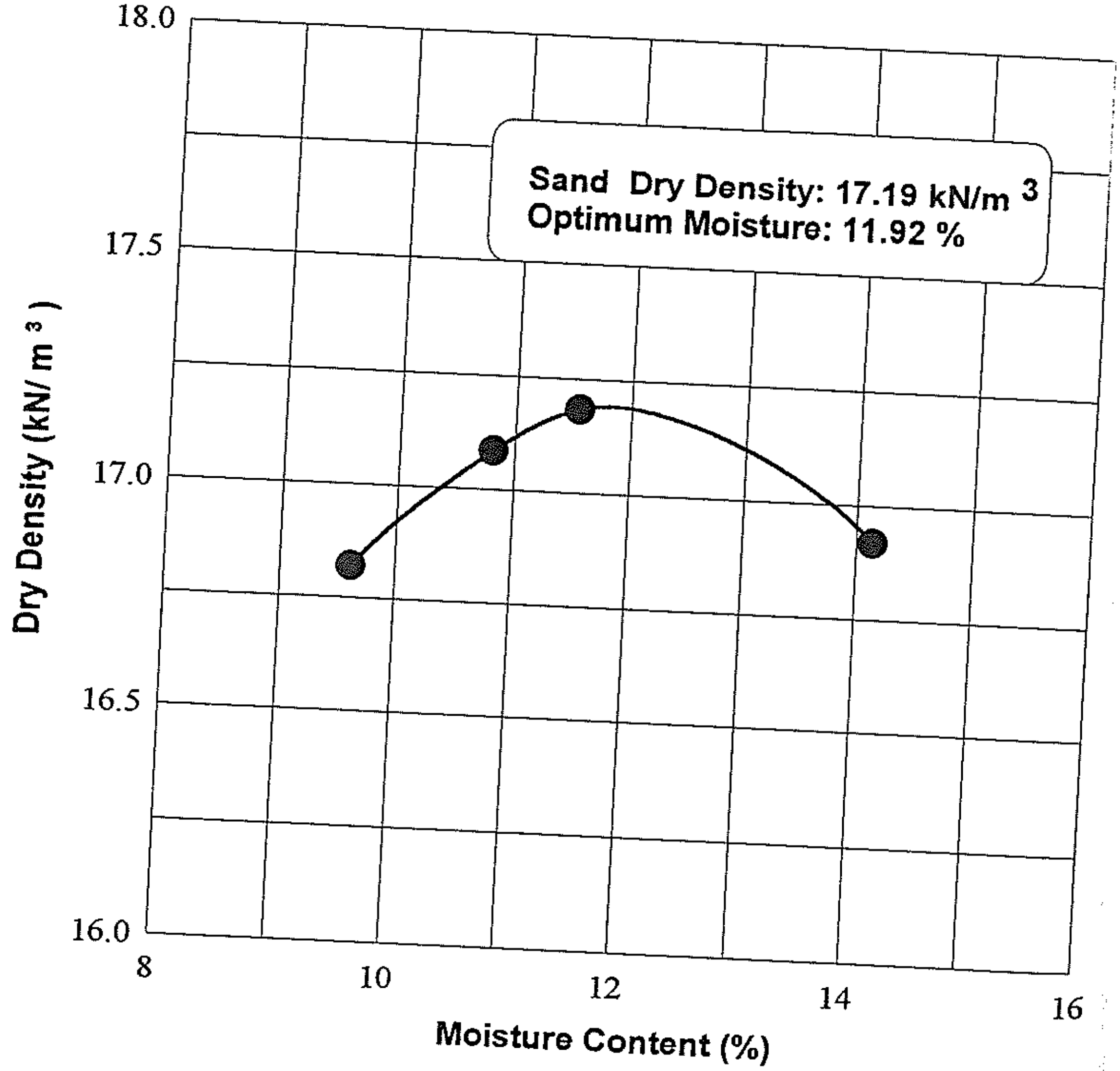


Figure 10: Proctor Density Results of Blasting Sand

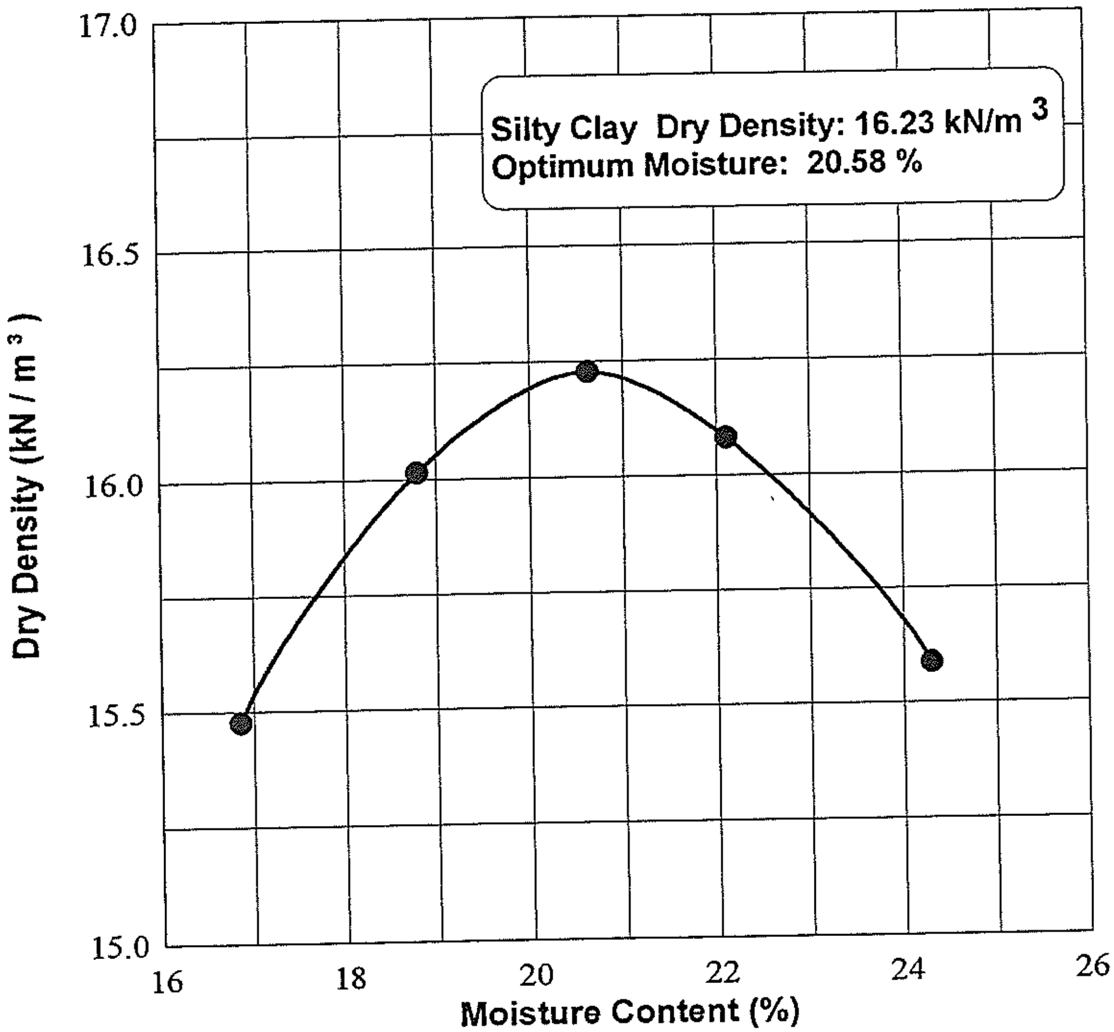


Figure 11: Proctor Density Results of Silty Clay

Table 2: Density - Water Content Levels of the Soil Specimens

Soil	Description (Optimum)	Dry Density (kN/m^3)	Moisture Content (%)	Relati Compac (%)
Blasting Sand	Dry	16.85	9.67	98
	Near	17.19	11.92	100
	Wet	17.03	13.50	99
Silty Clay	Dry	15.40	18.00	98
	Near	16.23	20.58	100
	Wet	15.40	23.00	98

Sand Specimens

The sand specimens were prepared by following the steps:

1. The split specimen mold is placed on the bottom platen of the triaxial cell over the porous stone. The membrane placed inside the mold is stretched over it such th is no slack in the mold. A vacuum is also applied outside the mold for this purpo o-rings are then placed around the membrane at the bottom platen and at the top mold.

2. The rammer is fixed to the piston rod (Figure 12). At this point, LVDT readings zeroed. The compaction program which invokes the rammer to apply a certain num blows for each layer is written with the features available in the MTS so *TESTSTAR*.

3. Four layers are used in this preparation. After compacting each layer, the top su scarified before placing the next layer. This provides good bonding between suc layers.

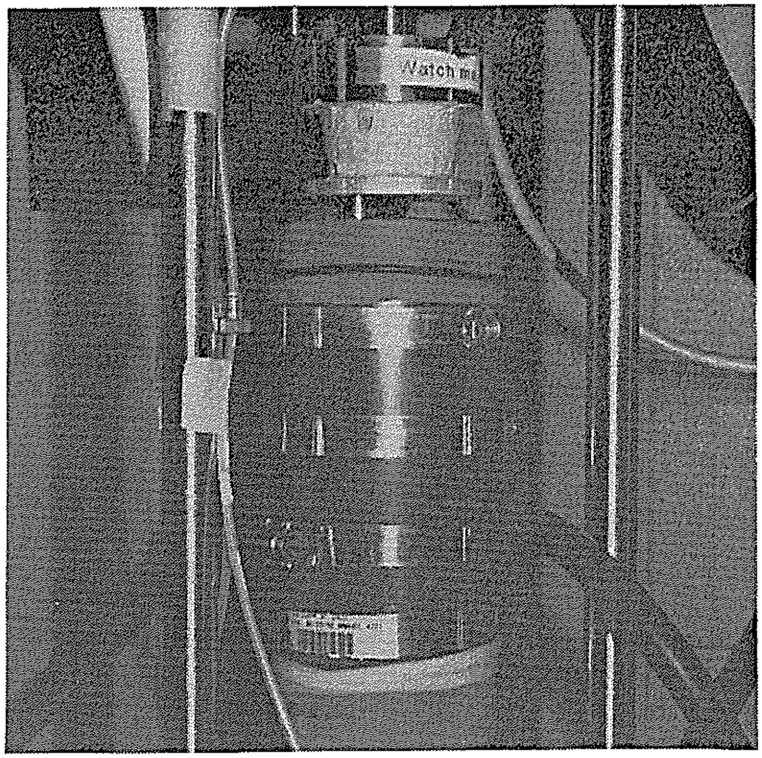


Figure 12: Rammer Assembly Used in Specimen Preparation

4. This procedure provides homogenous and repeatable specimens. The crushing of grains is also assessed by slicing the specimen into halves and passing them through a set of sieves. This grain size distribution along with the virgin distribution (Figure 13) shows that there is no crushing involved in this preparation.

5. After compaction of the top layer, the top porous stone and top platen are placed over the surface of the specimen. The membrane is stretched over the porous stone and platen and o-rings are then placed around the membrane and top platen.

6. The connections in the triaxial cell are attached to the control panel for applying pressures. The suction pressures of magnitudes 28 to 42 kPa are then applied inside the triaxial cell. The molds around specimens are removed and the average height and diameter of the specimens are measured. The LVDTs are carefully placed at the middle one third and at the ends of the specimen and zeroed.

7. The triaxial cell is positioned on top of the specimen assembly and all the triaxial chamber are tightened. The confining pressure is applied to the s releasing the suction pressures.

8. The load cell and the piston rod are placed into the groove of the top pla seating load is always applied in order to have perfect contact between the p mass.

9. The conditioning and testing protocols are prepared using features of software. The protocol, when invoked, applies a certain load to the sp prescribed number of times. The testing procedure steps are discussed in det section.

Clay Specimens

The clay specimen preparation is relatively easy when compared with that o procedure is a static compaction type as given by ASTM D1632. An extension sl placed on the top of the mold. The soil mass required for certain density is w placed inside the mold. The extension sleeve is then removed and a separating di on the surface of the specimen. The piston is then placed on top of the soil and a is applied by using a compression machine. Compaction will be continued until height is achieved. The specimens are placed inside the triaxial cell and necessary pressures are applied as required in the conditioning phase. Suction pressures are no since the cohesion property of the specimen will be able to prevent collaps specimen.

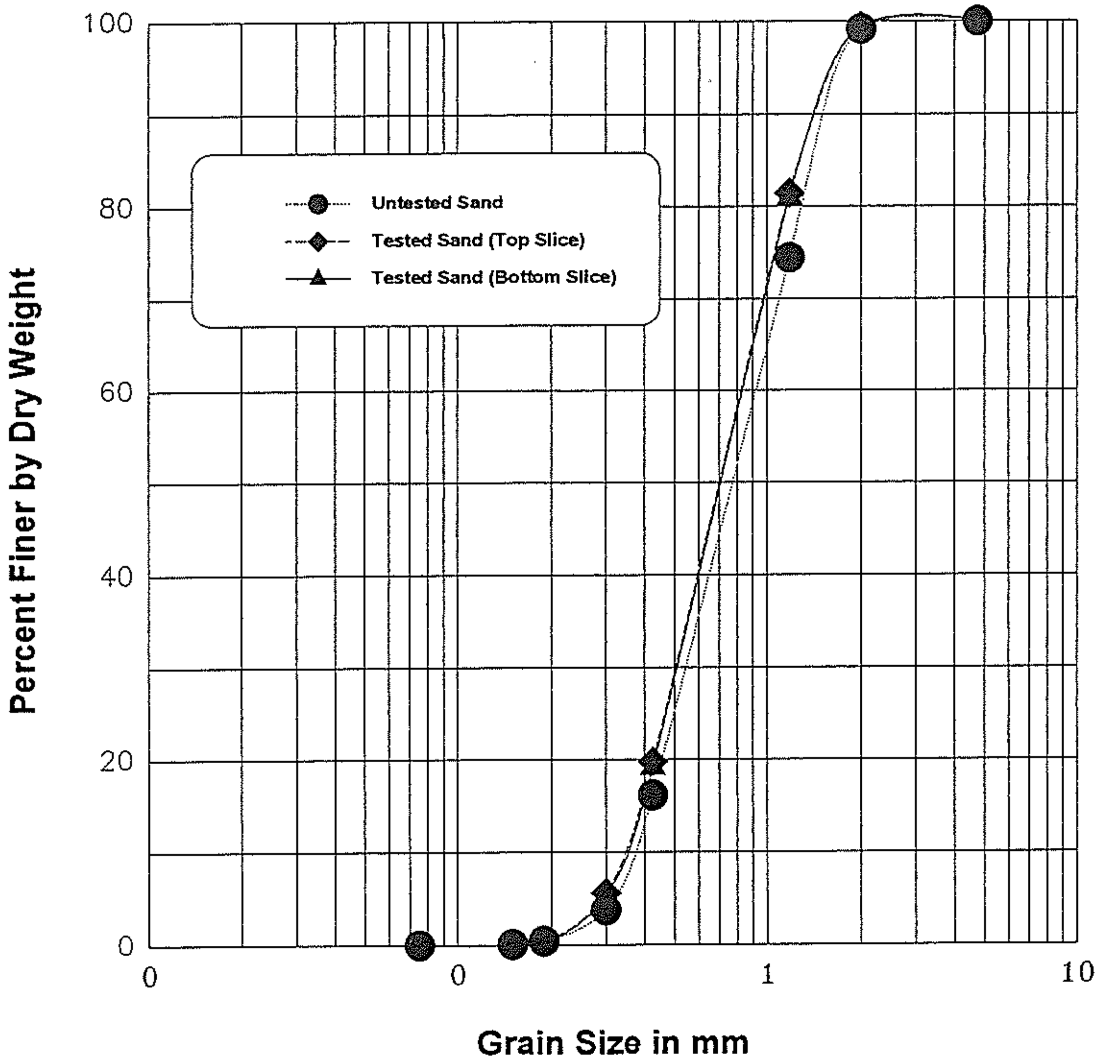


Figure 13: Grain Size Distribution Curves of Compacted Sand

Testing Procedure

The AASHTO testing procedures, T-292 and T-294 were previously discussed. Co programs for the conditioning and testing phases of the test procedures are writte TESTSTAR software features. After placing the sample inside the triaxial cell and pos the load cell rod on top of the specimen, a certain conditioning confining p recommended by the AASHTO procedure is applied. The respective computer pro then invoked. The prescribed deviatroic load of haversine shape is applied to the sp up to one thousand cycles. This is followed by a testing phase in which the program the user to provide certain pauses to allow the time to change the confining pressur program applies different deviatoric loads at each confining pressure. The data acq system collects the data from the load cells, and LVDTs of the end and middle system. data are then analyzed and reduced to resilient strains and plastic strains. A separate co program is written for this analysis. The resilient strains are used to determine the m values for each confining and deviatoric stress level.

Specimens were prepared at different moisture content levels using the above de procedure and were then tested. The test results are discussed in the next chapter.

FIELD TESTING PROGRAM

The field testing program consists of testing with nondestructive devices at tw followed by resilient modulus testing on core samples retrieved from the same locatio two sites selected are the airport paving strip at Opelousas, Louisiana (site 1) a Accelerated Loading Facility or ALF testing facility at Port Allen, Louisiana (site 2). soils from sites 1 and 2 are classified as silty clay and heavy clay. The properties of t are presented in Table 3.

Table 3: Soil Properties of Field Sites

Property	Opelousas (SITE 1)	Port Allen (SITE 2)
Soil Type	Silty Clay - Clay	Heavy Clay
% Passing No. 200 Sieve	93.2	98.0
Liquid Limit	28	93
Plasticity Index	6	66
Group Index	8.0	16.0
Organic Content	4.7	9.2
AASHTO Classification	A-4	A-7-6
Unified Soil Classification	ML-CL	CH

The LTRC coring rig was used in boring and sampling operations. Shelby tubes were used in sampling. Once the samples were retrieved, the ends of the shelby tubes were closed with polyethylene covers and were kept in a moisture controlled humidity room. This prevented moisture content loss during transportation and other handling processes.

Both Dynaflect and Falling Weight Deflectometer (FWD) were used in the non-destructive testing. Both devices were utilized at site 1 and only Dynaflect was used at site 2.

Modulus 4.1, a backcalculation software developed by Texas Transportation Institute, was used to analyze the FWD data and to determine the moduli of subgrade layers. Design charts were used to analyze the Dynaflect data.

ANALYSIS OF RESULTS

DESIGN OF THE LABORATORY EXPERIMENT

A statistically designed experiment, described below, is used to evaluate the variation in test results between the two methods and to ascertain the test repeatability for each method. For a simple random sample of size n drawn from a population having a mean and standard deviation σ, the 100 $(1-\alpha)\%$ confidence interval for the mean μ, is given b

$$\mu = x \pm z_{\alpha/2}(\sigma/\sqrt{n})$$

where,

$z_{\alpha/2}$ = upper $\alpha/2$ critical value for the standard normal distribution obtained from standard normal tables

x = sample mean

α = confidence level.

The error estimation e, defined as the maximum amount by which the estimate in perc differs from population mean μ, is given by:

$$e = | x-\mu | = (z_{\alpha/2}\sigma/\sqrt{n})$$

Rearranging equation 2,

$$n = (z_{\alpha/2}\sigma/e)^2$$

However, in the above equation, σ and e are initially unknown. In such a case, statisticians have established that for a normal distribution a sample size of thirty would well define the pattern of variation of the variable. Hence, in the initial phase of the experimental program, thirty specimens are selected for each soil type.

The sample mean x and sample variance s^2 are computed from the tests conducted on thirty specimens. Equation (4) is used to estimate the number of specimens for the remaining phase of the experimental study.

$$n = (t_{\alpha/2}s/e)^2 \tag{4}$$

where,

$t_{\alpha/2}$ = upper $\alpha/2$ critical value for the t-distribution

s = sample standard deviation

α = confidence level.

The number of specimens is approximately five based on the initial thirty test results. Thus, five good tests are conducted for each variable investigated in the experimental program and the average of these five test results are taken as the resilient properties of that variable. The criterion for a test to be accepted is described in the previous chapter.

ANALYSIS OF LABORATORY RESULTS

Table 4 provides the total number of tests conducted on artificial laboratory and natural field core specimens. Results of each test include confining and deviatoric stresses and their corresponding resilient modulus values. Test results are first used to assess repeatability, equipment variability, procedure variability and the number of cycles required in the testing phase of the experiment.

Table 4: Total Number of Resilient Modulus Tests Conducted on Laboratory and Field Specimens

Laboratory / Field	Soil Type	Number of Tests
Laboratory	Blasting Sand	30(O)
Laboratory	Blasting Sand	25(N)
Laboratory	Silty Clay	30(O),25(N)
Field	Silty Clay	3(N)
Field	Heavy Clay	3(N)

Note: O - old MTS equipment

N - new MTS equipment

Repeatability

In performance assessment of tests, repeatability of test results is to be considered. Test results of sands are presented in Table 5. These results represent an optimum den and water content combination level. The coefficient of variation C_v, used to measure of repeatability of the tests, was calculated and presented in the same table. The C_v values sands range from 0.84 to 10.3, with most of the values between 1.0 and 5.0. These numbers are considered small, indicating that the tests are repeatable.

The silty clay test results at optimum density and water content level are reported in Table 6. The coefficient of variation varies between 2 and 20 percent with most values around 15. These results can also be termed as repeatable since the C_v values are still considered small when compared to the magnitudes of resilient modulus values.

Table 5: Resilient Modulus Laboratory Results Conducted on Blasting Sand with Old MTS Equipment (Both AASHTO Procedures)

Cf. Pr.	D Str.	AASHTO Procedure								AASHTO Procedure							
		T−292				T−294				T−292				T−294			
		MRE	SD	CV	R	MRE	SD	CV	R	MRM	SD	CV	R	MRM	SD	CV	R
21	21					159.2	4.7	3.0	Y					189.0	8.6	4.6	Y
21	35	153.0	8.4	5.5	Y	164.2	6.7	4.1	Y	189.0	12.6	6.7	Y	193.2	10.7	5.5	Y
21	52.5	162.1	10.1	6.2	Y	173.6	5.6	3.3	Y	195.0	11.1	5.7	Y	206.5	6.1	2.9	Y
21	70	168.7	9.7	5.8	Y	180.1	4.3	2.4	Y	200.9	11.6	5.8	Y	212.1	8.7	4.1	Y
21	87.5	173.6	10.9	6.3	Y					205.5	11.9	5.8	Y				
35	42	168.0	9.6	5.7	Y	209.0	4.9	2.4	Y	194.6	12.1	6.2	Y	250.6	10.3	4.1	Y
35	70	183.8	9.6	5.3	Y	220.5	6.1	2.8	Y	217.0	9.5	4.4	Y	259.0	7.4	2.9	Y
35	105	199.5	10.1	5.0	Y	220.5	4.3	2.0	Y	231.7	9.9	4.3	Y	255.5	9.6	3.8	Y
35	140	206.5	12.3	6.0	Y	219.0	3.0	1.4	Y	237.3	13.9	5.8	Y	254.5	11.3	4.4	Y
70	35	226.1	11.9	5.3	Y	283.9	4.8	1.7	Y	269.9	16.6	6.1	Y	345.8	18.7	5.4	Y
70	70	238.0	10.1	4.3	Y	293.3	3.3	1.1	Y	275.8	12.9	4.7	Y	341.6	16.5	4.8	Y
70	140	257.6	10.0	3.9	Y	302.6	3.7	1.2	Y	295.4	11.5	3.9	Y	345.8	12.3	3.6	Y
70	210	274.1	9.6	3.5	Y	295.4	3.0	1.0	Y	309.8	11.8	3.8	Y	336.7	11.0	3.3	Y
105	70	287.0	12.0	4.2	Y	337.6	6.2	1.8	Y	336.7	21.4	6.4	Y	386.4	18.4	4.8	Y
105	105					346.5	4.6	1.3	Y					394.1	16.0	4.1	Y
105	140	304.5	12.4	4.1	Y	353.5	3.7	1.0	Y	347.2	19.3	5.6	Y	399.7	14.3	3.6	Y
105	210	317.8	11.6	3.7	Y	356.3	3.3	0.9	Y	358.4	17.6	4.9	Y	403.2	14.4	3.6	Y
105	280	327.3	10.7	3.3	Y					365.4	14.6	4.0	Y				
140	70	352.5	17.8	5.1	Y	380.8	4.8	1.3	Y	423.5	43.6	10.3	Y	445.9	26.1	5.9	Y
140	105					392.0	4.9	1.2	Y					448.7	23.2	5.2	Y
140	140	366.8	16.5	4.5	Y	396.2	3.3	0.8	Y	424.9	35.2	8.3	Y	456.1	22.4	4.9	Y
140	210	369.6	13.9	3.8	Y					420.0	28.1	6.7	Y				
140	280	372.4	11.8	3.2	Y	406.0	4.4	1.1	Y	417.9	21.9	5.3	Y	465.5	26.6	5.7	Y

Cf. Pr. : Confining Pressure (in kPa)

D Str. : Deviatoric Pressure (in kPa)

MRE : Resilient Modulus from End Measurement System (in MPa)

MRM : Resilient Modulus from Middle Measurement System (in MPa)

SD : Standard Deviation (MPa)

CV : Coefficient of Variation in percent

R : Repeatability

Y : Indicates Test is Repeatable as per ASTM C670.

Table 6: Resilient Modulus Laboratory Results Conducted on Silty Clay with Old Equipment (Both AASHTO Procedures)

Cf. Pr	D Str.	Measurement System								Pr
		End				Middle				
		MRE	SD	CV	R	MRM	SD	CV	R	
42	14	243.2	23.0	9.5	Y	293.4	42.2	14.4	Y	T-
42	28	216.9	14.2	6.5	Y	266.4	26.9	10.1	Y	
42	42	195.9	10.1	5.1	Y	234.9	24.8	10.6	Y	
42	55	179.8	10.1	5.6	Y	214.2	26.1	12.2	Y	
42	69	152.7	10.0	6.1	Y	194.4	27.3	14.1	Y	
21	14	204.3	17.9	8.8	Y	265.9	33.5	12.6	Y	T-
21	28	186.2	9.9	5.3	Y	244.7	31.6	12.9	Y	
21	42	171.7	11.6	6.8	Y	226.1	28.7	12.7	Y	
21	55	157.7	11.3	7.2	Y	204.5	27.9	13.7	Y	
21	69	145.9	12.5	8.6	Y	187.3	26.0	13.9	Y	
21	35	171.9	17.6	10.2	Y	240.6	57.5	23.9	Y	T-2
21	52	158.4	18.9	11.9	Y	213.5	44.1	20.7	Y	
21	69	141.7	19.6	13.8	Y	180.3	35.2	19.5	Y	
21	86	127.4	19.8	15.5	Y	154.8	31.9	20.6	Y	
0	14	161.5	15.5	9.6	Y	257.2	22.9	8.9	Y	T-2
0	28	141.4	14.6	10.3	Y	223.3	26.8	12.0	Y	
0	42	129.9	13.8	10.6	Y	206.8	23.1	11.2	Y	
0	55	122.4	13.0	10.6	Y	189.1	23.9	12.6	Y	
0	69	116.8	14.6	12.5	Y	172.9	23.9	13.8	Y	

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE/M : Resilient Modulus from End/Middle System (in MPa)
SD : Standard Deviation (in MPa)
CV : Coefficient of Variation in percent
R : Repeatability
Y : Indicates Test is Repeatable as per ASTM C670.

Equipment Variability

Two similar types of MTS equipments or devices are used in the entire study. The first device, termed old equipment, is used in the early phase of the experimental program, which included tests conducted for assessing repeatability. The second equipment or new device is used in the remainder of the experimental program. Though both MTS devices are similar, it is still necessary to evaluate the equipment variability and its influence on test results.

In order to assess this equipment variability, tests conducted on sands at the same water content and density combinations are compared. The water content and dry density of these tests were 9.86 % and 17.5 kN/m^3. Results presented in Figure 14 indicate that the new equipment under the T-294 procedure provided slightly higher values than those obtained with the old equipment. However, this slight increase is very small and it can be seen from the figure that the comparison results are very close to the equality line. The t-test conducted on these results is also indicated that the variation between these results is statistically not significant. This indicates that the equipment variability is insignificant and can be disregarded. The same operator used both devices and therefore operational errors are insignificant in this investigation.

Test Procedure Variability

It is interesting to note that the C_v values of sand test results from the T-294 procedure are lower than those from the T-292 procedure. This is attributed to the differences in the conditioning and testing phases in the respective procedures. The T-294 procedure, which is a slight modification of the T-292 procedure, is assumed to have resulted in less soil disturbance and stress sensitivity/dependency on the specimens. Also for sands, the T-294

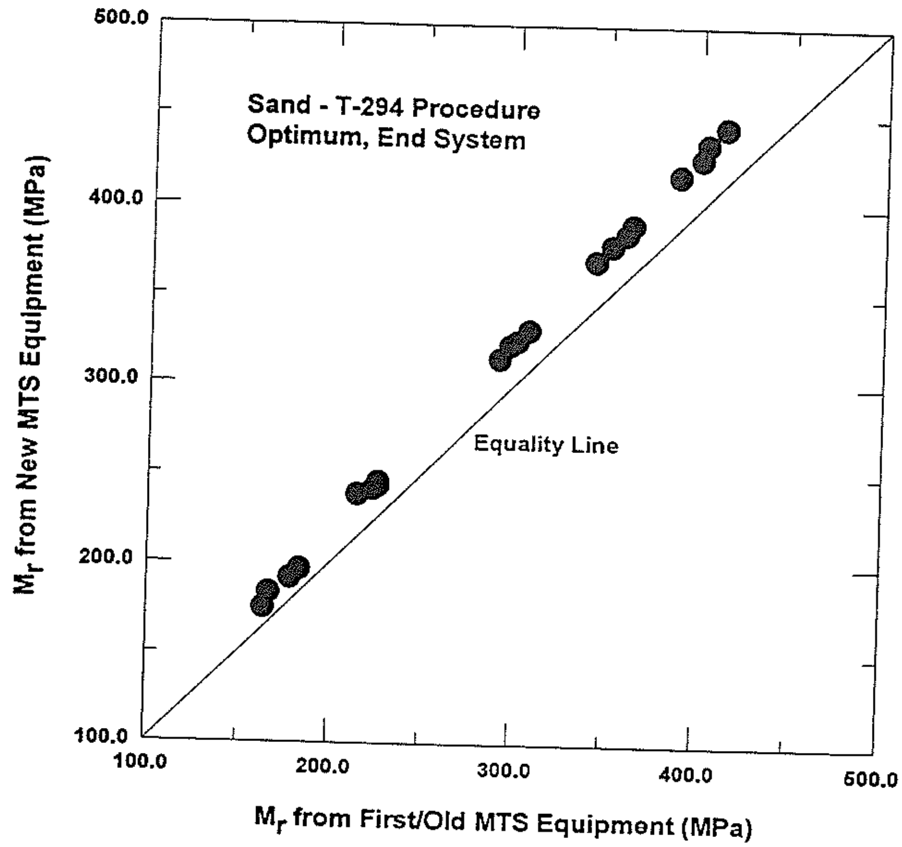


Figure 14: Equipment Variability

procedure produced higher resilient moduli results than the T-292 procedure. Th reasoning as above can be given for this observation.

The differences between results from both procedures for silty clays is not as as in the case for sands. This may be due to the fact that stress magnitudes of silt are significantly smaller than those of sands. These smaller magnitudes did not re significant differences in the resilient moduli values from both procedures.

The Number of Cycles

The number of cycles for the conditioning and testing phases are 1000 and 100 cycles, respectively. It should be mentioned that both AASHTO procedures use distinct testing cycles (fifty for T-292 and one hundred for T-294). However, it was decided to use one hundred cycles for testing in both procedures. This puts both procedures on the same platform as far as the number of cycles is concerned. This also allows investigators to understand the influence of testing procedures on the resilient properties of soils. It is still important, however, to understand the significance of the number of cycles and also assess whether the number of cycles are sufficient enough to reduce the plastic strain developments prior to resilient strain measurements.

The permanent strains and the differences between resilient strains at different cycles control the total number of cycles for each testing stress level. The number of cycles during the conditioning period will be selected such that the plastic strains are minimized at the end of conditioning. It should also be noted that AASHTO recommends total permanent strain at any time during the testing period should not exceed 5 and 10 percent for T-292 and T-294, respectively. These criterion should be considered when deciding the number of cycles.

The resilient, plastic strains, and resilient modulus values are determined for both soils. The accumulated plastic strains versus the number of cycles are shown in Figures 15 (sands) and 16 (silty clays). Both soils exhibited a continuous increase in permanent or plastic strains with the number of cycles. It should be noted that the plastic strains developed at the end of testing cycles are still increasing for both soils, more significantly for sands. Higher plastic strains for sands are reported, probably as a result of higher deviatoric stresses applied. The AASHTO procedures assume that conditioning and testing cycles on the specimens remove plastic strains prior to obtaining resilient measurements.

This phenomenon is not observed in the present testing since plastic strains are still increasing in the testing phase even in the last cycle of various stress levels.

The influence of continuous development of plastic strains on M_r is investigated by comparing the moduli values with each cycle. Figures 17 (sand) and 18 (silty clay) show the resilient moduli variation with the number of cycles. It can be seen from these figures that the resilient modulus of both soils does not show significant variation with the numb of cycles. The slopes of the best fit lines of sands and silty clays are 0.07 and 0.08. The slopes become flatter with the increase in the number of cycles. These small slopes sugg that the influence of the number of cycles on the resilient modulus is not significant after certain number of cycles which is about twenty-five for the soils of the present study. Th implies that the accumulated plastic deformation with number of cycles beyond twenty fi does not influence the resilient modulus results.

The question that arises now is the need for taking one hundred cycles to measure th resilient modulus when there is a minor variation in resilient modulus beyond certain cycles. This has been investigated by several researchers (Pezo et al., 1992; Ho, 1989). The investigations concluded that the number of cycles suggested in AASHTO procedu is high and needs to be reduced. The observation, however, may not be valid for all s The authors recommend that the number of cycles should be selected based on the typ soil and their characteristics. For example, based on the results of the soils tested in th study, the number of cycles can be decreased to twenty-five since the M_r does not sho significant variation beyond twenty-five cycles.

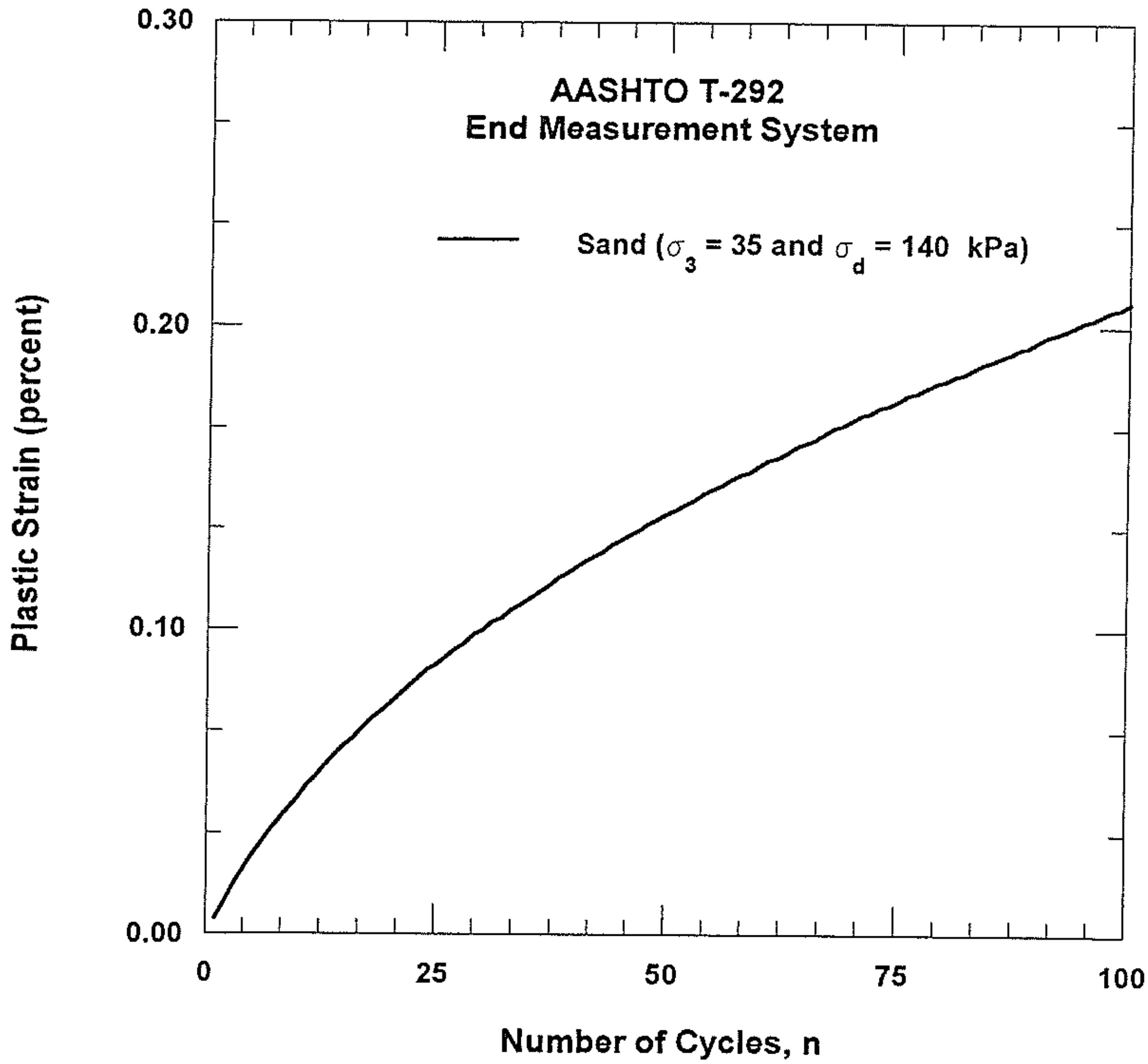


Figure 15: Accumulated Plastic Strains Versus Number of Cycles from Sand Test

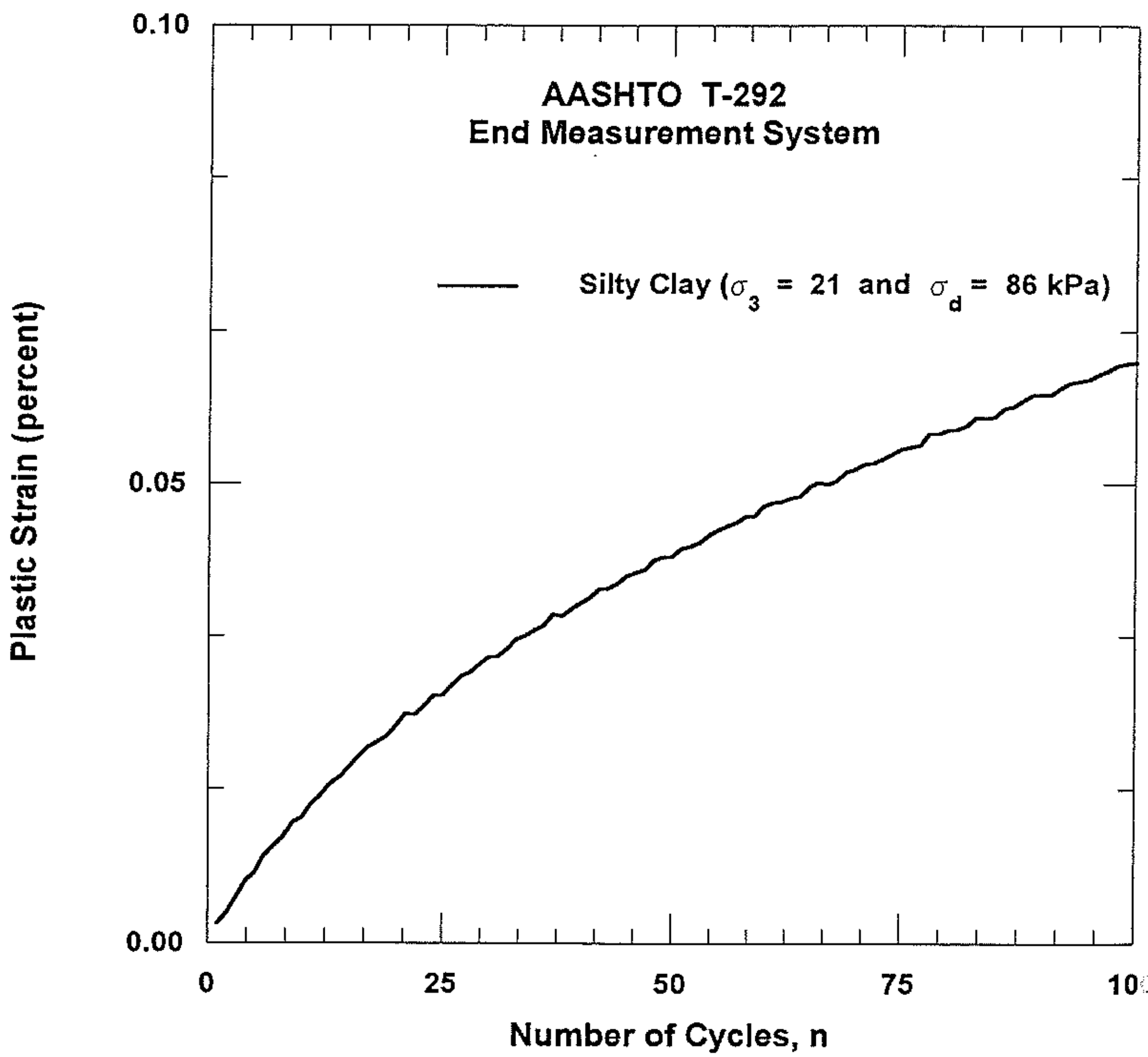


Figure 16: Accumulated Plastic Strains Versus Number of Cycles from Silty Clay

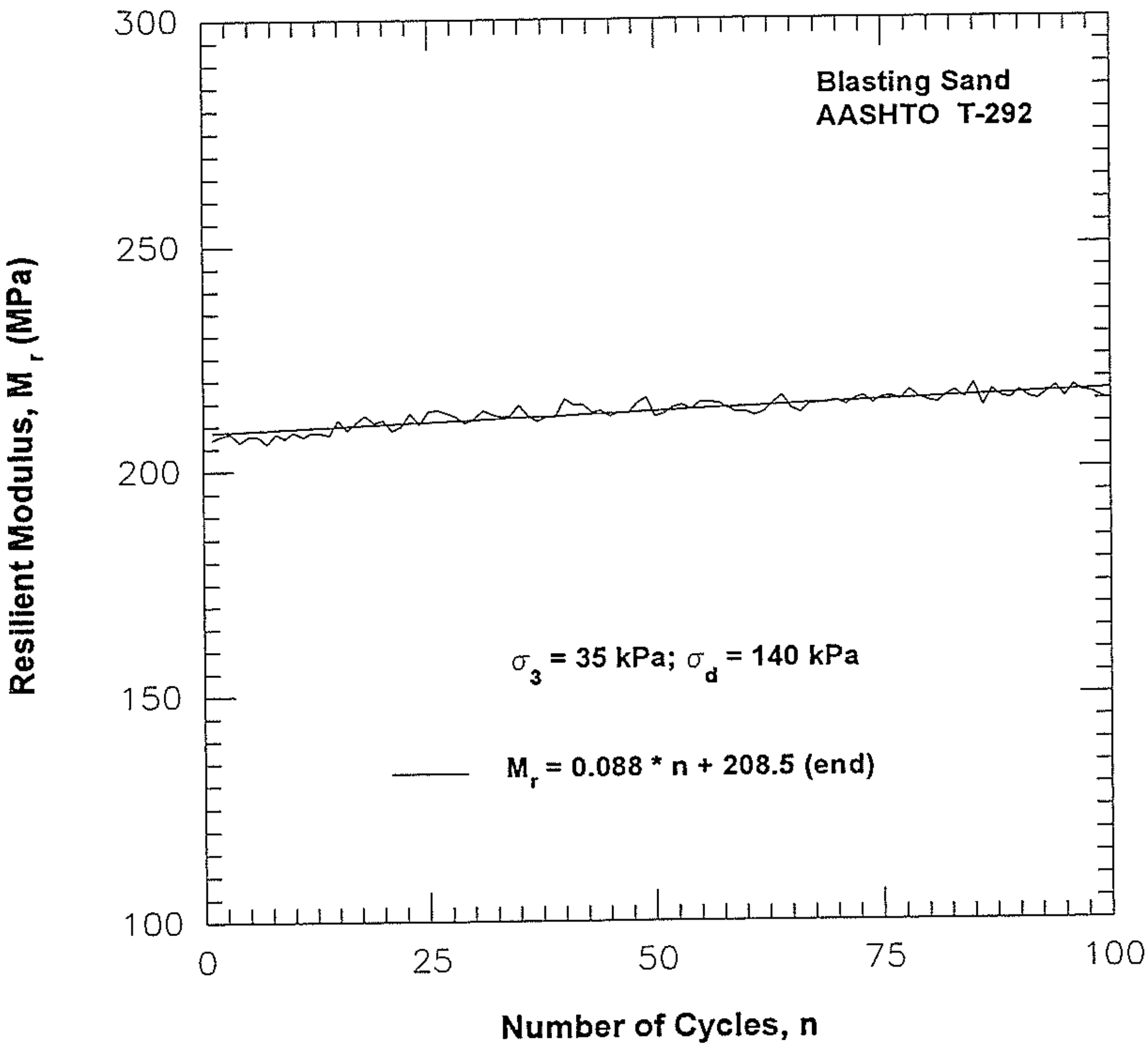


Figure 17: Resilient Modulus Versus Number of Cycles: Sands

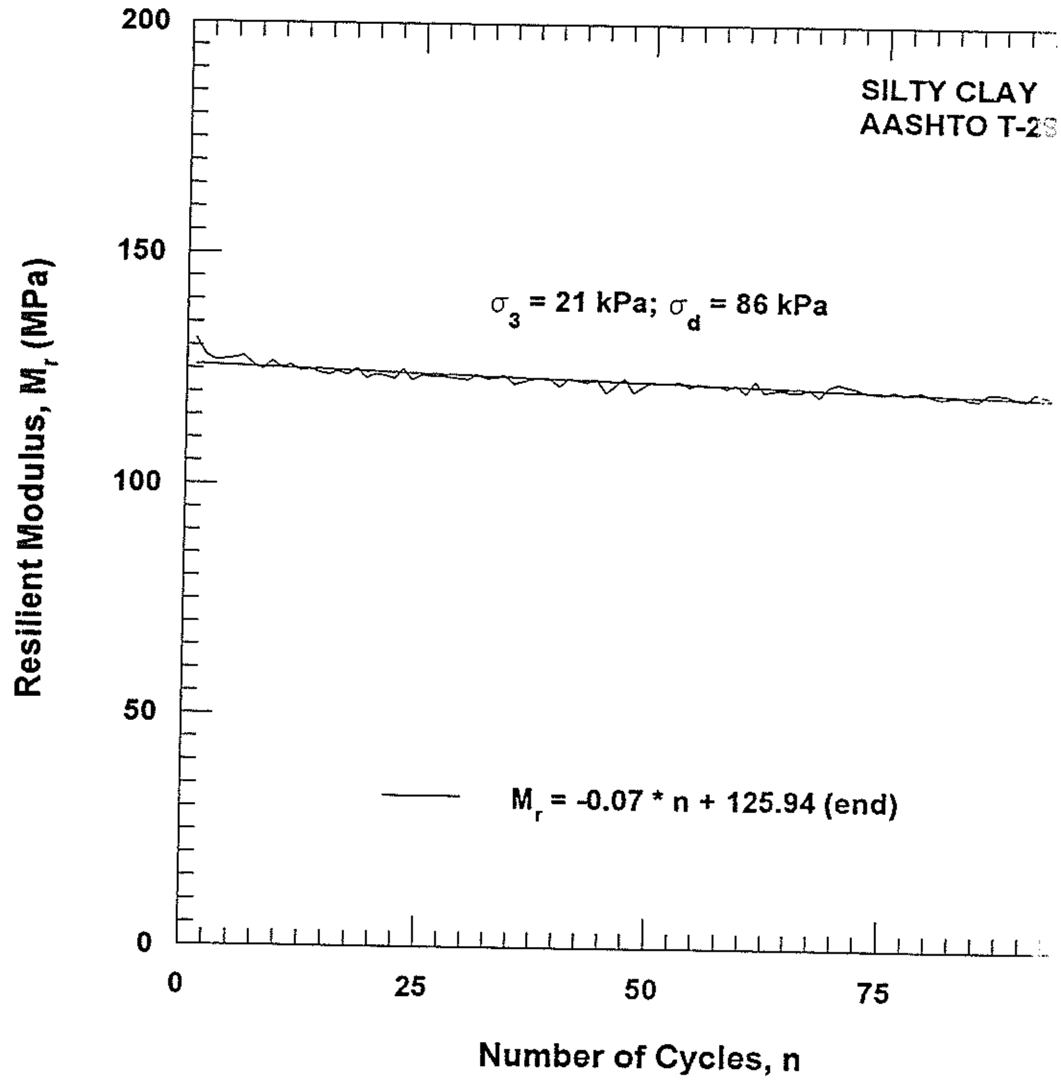


Figure 18: Resilient Modulus Versus Number of Cycles: Silty Clays

SOIL VARIABLES

Sands - M_r Results

The sand test results conducted using AASHTO procedure T-294 are presented in Tables 7 (optimum), 8 (dry of optimum), and 9 (wet of optimum). Values reported in these tables are obtained from averaging five test results. The coefficients of variation vary between 0.1 and 15.0 with most of them being around 3.0. This again suggests and confirms that the tests are highly repeatable. Slightly higher coefficients of variation (C_v values around 10) are observed for end resilient modulus values obtained from tests conducted at above and below the optimum moisture contents and at low confining stresses. Lower C_v values (less than 10) are reported for sands tested at optimum levels.

Figures 19 to 23 show the variation of resilient modulus on sands at different moisture contents and dry densities for various confining stresses of magnitudes 21 kPa, 35 kPa, 70 kPa, 105 kPa, and 140 kPa, respectively. Only end resilient moduli results are reported in these figures. It is noted that the resilient modulus increases with an increase in the confining pressure. This is attributed to two reasons. The first one corresponds to the stiffness characteristics of the sand which increases with an increase in confining pressure. The second reason is the dilational characteristics of sands. The dilational behavior of sands is more evident at higher densities and low confining pressures. This dilational behavior, when restrained at higher confining pressures, results in lesser axial strains and higher moduli values.

Table 7: Resilient Modulus Test Results on Blasting Sand at Optimum Moist
Dry Density Combination (T-294)

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
139.0	104.5	351.4	9.0	2.6	434.3	20.6	[illegible]
20.4	20.7	122.3	4.1	3.3	193.1	13.1	6[illegible]
20.4	34.9	130.5	3.0	2.3	196.5	12.4	6[illegible]
20.4	51.7	137.5	1.9	1.4	205.4	14.3	7.[illegible]
20.4	69.2	143.5	1.3	0.9	211.8	14.8	7.[illegible]
34.2	35.1	177.9	1.9	1.1	263.9	15.6	5.[illegible]
34.2	69.6	184.3	2.1	1.2	265.1	14.1	5.[illegible]
34.2	104.4	188.9	2.4	1.3	258.9	15.3	5.[illegible]
34.1	138.3	193.2	3.0	1.6	252.7	15.5	6.[illegible]
69.1	35.1	266.3	3.5	1.3	355.2	15.7	4.[illegible]
69.1	69.6	278.3	2.7	1.0	352.1	13.1	3.7
69.2	138.8	289.7	3.7	1.3	351.4	11.4	3.3
69.2	207.8	286.5	4.6	1.6	339.1	12.8	3.8
104.0	69.6	331.9	5.3	1.6	406.9	13.8	3.4
104.0	104.5	341.8	4.6	1.4	410.0	13.3	3.2
104.0	138.7	348.7	4.7	1.4	412.0	13.7	3.3
104.0	207.7	355.0	5.0	1.4	411.6	13.3	3.2
139.0	69.5	385.5	4.2	1.1	464.5	15.7	3.4
139.0	104.4	394.1	4.5	1.1	466.0	13.9	3.0
139.0	138.7	402.2	5.2	1.3	469.7	14.2	3.0
138.9	276.9	412.5	4.7	1.1	468.2	15.4	3.3

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 8: Resilient Modulus Test Results on Blasting Sand at Dry of Optimum (T-294)

Cf. Pr.	D Str.	MRE (MPa)			MRM (MPa)		
kPa	kPa	Mean	STD	CV	Mean	STD	CV
139.3	104.7	375.2	22.0	5.9	429.8	17.7	4.1
20.9	20.9	151.9	21.2	14.0	190.7	11.7	6.1
21.1	35.1	160.5	22.5	14.0	196.7	9.4	4.8
21.1	52.1	164.5	23.5	14.3	202.6	9.5	4.7
21.1	69.4	165.1	24.6	14.9	201.9	10.0	5.0
34.8	35.2	204.4	27.5	13.4	258.6	9.1	3.5
34.9	69.7	209.9	27.1	12.9	256.7	10.4	4.1
34.9	104.4	205.8	25.0	12.1	244.0	11.2	4.6
34.9	137.9	194.6	23.4	12.0	221.1	14.5	6.6
69.7	35.1	277.8	26.6	9.6	338.9	13.5	4.0
69.7	69.7	287.0	23.9	8.3	338.9	14.6	4.3
69.7	138.8	291.1	21.5	7.4	332.8	17.8	5.4
69.6	207.7	280.3	19.1	6.8	311.9	15.7	5.0
104.6	69.6	334.2	19.4	5.8	387.2	15.2	3.9
104.6	104.6	341.6	19.4	5.7	390.9	16.6	4.2
104.5	138.7	345.5	18.8	5.4	391.9	16.9	4.3
104.5	207.8	347.0	18.0	5.2	385.8	17.2	4.5
139.4	69.5	379.6	18.2	4.8	435.8	15.8	3.6
139.4	104.5	387.8	18.3	4.7	438.5	17.0	3.9
139.4	138.7	393.2	17.7	4.5	443.8	18.5	4.2
139.4	276.9	396.7	17.5	4.4	434.1	19.1	4.4

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 9: Resilient Modulus Test Results on Blasting Sand at Wet of Optimum (T-294)

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
139.5	104.7	373.4	11.2	3.0	439.1	17.1	3.9
21.0	20.7	149.4	17.7	11.9	200.7	13.6	6.8
21.0	35.0	158.2	19.1	12.1	201.6	8.9	4.4
20.9	51.9	166.2	20.8	12.5	206.7	6.4	3.1
21.0	69.2	171.4	21.4	12.5	210.4	5.5	2.6
34.6	35.1	208.4	25.2	12.1	267.1	10.8	4.1
34.8	69.6	213.8	24.2	11.3	263.0	8.1	3.1
34.8	104.3	212.3	21.8	10.3	253.4	6.9	2.7
34.8	138.0	212.2	19.9	9.4	247.3	9.1	3.7
69.8	35.1	289.2	22.7	7.9	380.0	41.9	11.0
69.8	69.6	296.0	20.1	6.8	361.7	21.3	5.9
69.7	138.8	301.9	17.5	5.8	348.9	12.8	3.7
69.8	207.8	295.3	13.3	4.5	331.6	9.4	2.8
104.7	69.6	343.8	13.5	3.9	416.0	25.1	6.0
104.7	104.6	351.8	13.0	3.7	412.0	19.0	4.6
104.7	138.7	357.5	11.9	3.3	411.1	16.1	3.9
104.7	207.7	361.1	10.6	2.9	404.8	11.5	2.8
139.3	69.5	390.3	10.8	2.8	470.6	34.2	7.3
139.2	104.4	399.8	9.7	2.4	466.6	24.7	5.3
139.4	138.7	407.4	10.2	2.5	467.0	19.6	4.2
139.4	276.9	413.5	10.8	2.6	456.5	12.9	2.8

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

The influence of deviatoric stress on the test results can also be seen from the same figures. They indicate that the deviatoric stress increase results in little or no significant change in the moduli values at low confining pressures. At higher confining pressures, the moduli remain constant. This indicates that the deviatoric stress has minor influence at higher confining pressures. The sands at higher confining stresses exhibit higher strength. The deviatoric loads applied at this higher strength are assumed to be lower than the peak deviatoric loads which can cause significant changes in the specimen. Thus, the deformation response in these tests is directly proportional to the deviatoric loading applied which results in the same M_r values at all deviatoric stresses.

The moisture content appears to have some influence on M_r results at low confining pressures (less than 70 kPa). Figures 19 to 23 indicate that higher moduli results are obtained for dry of optimum moisture content level than at other moisture content levels. The M_r values at the optimum level, on the other hand, are lower than those at the wet of optimum level. Leakage problems occurred in the wet of optimum tests which may have caused this discrepancy.

Ta

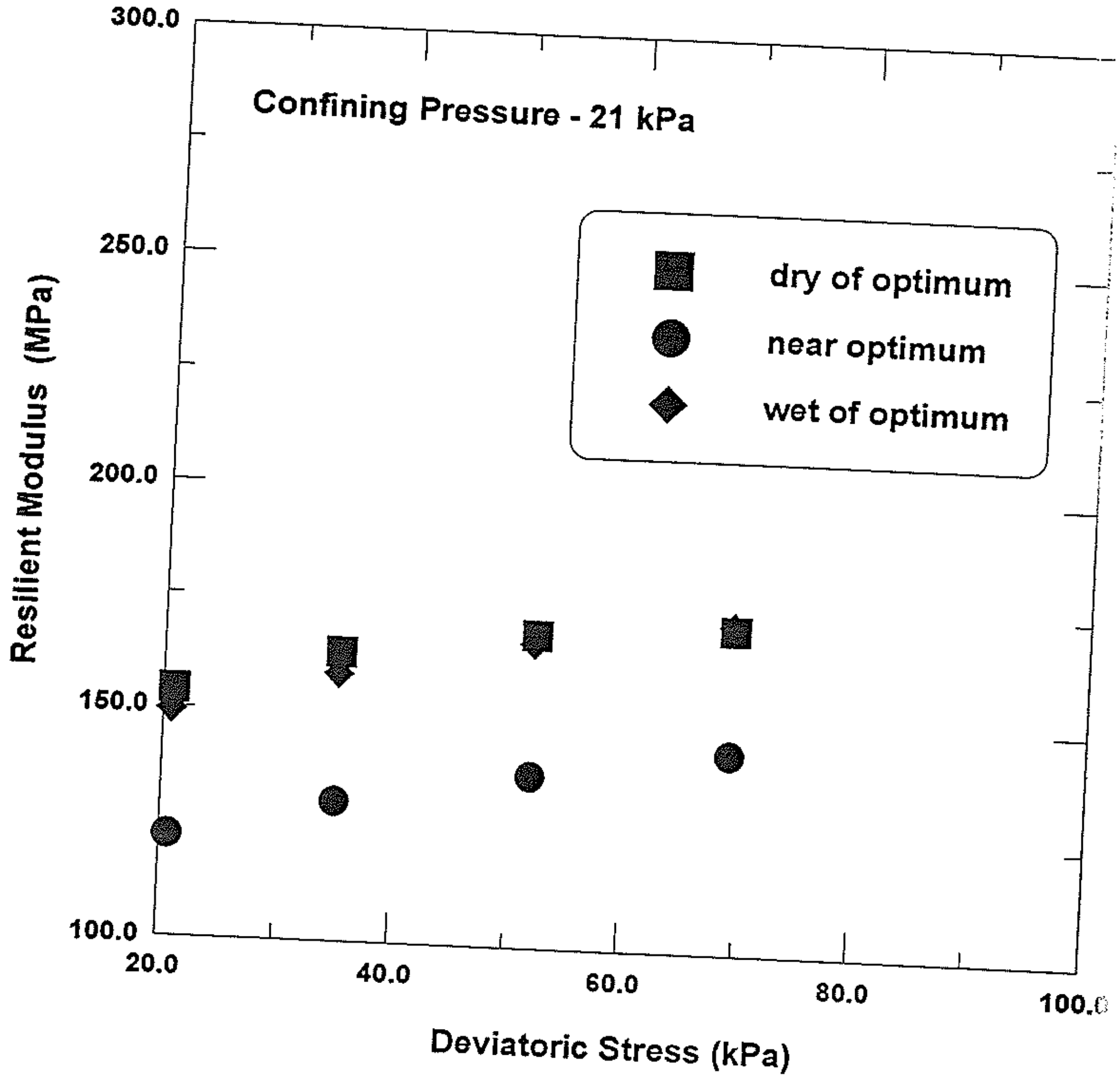


Figure 19: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus Sands at a Confining Pressure of 21 kPa

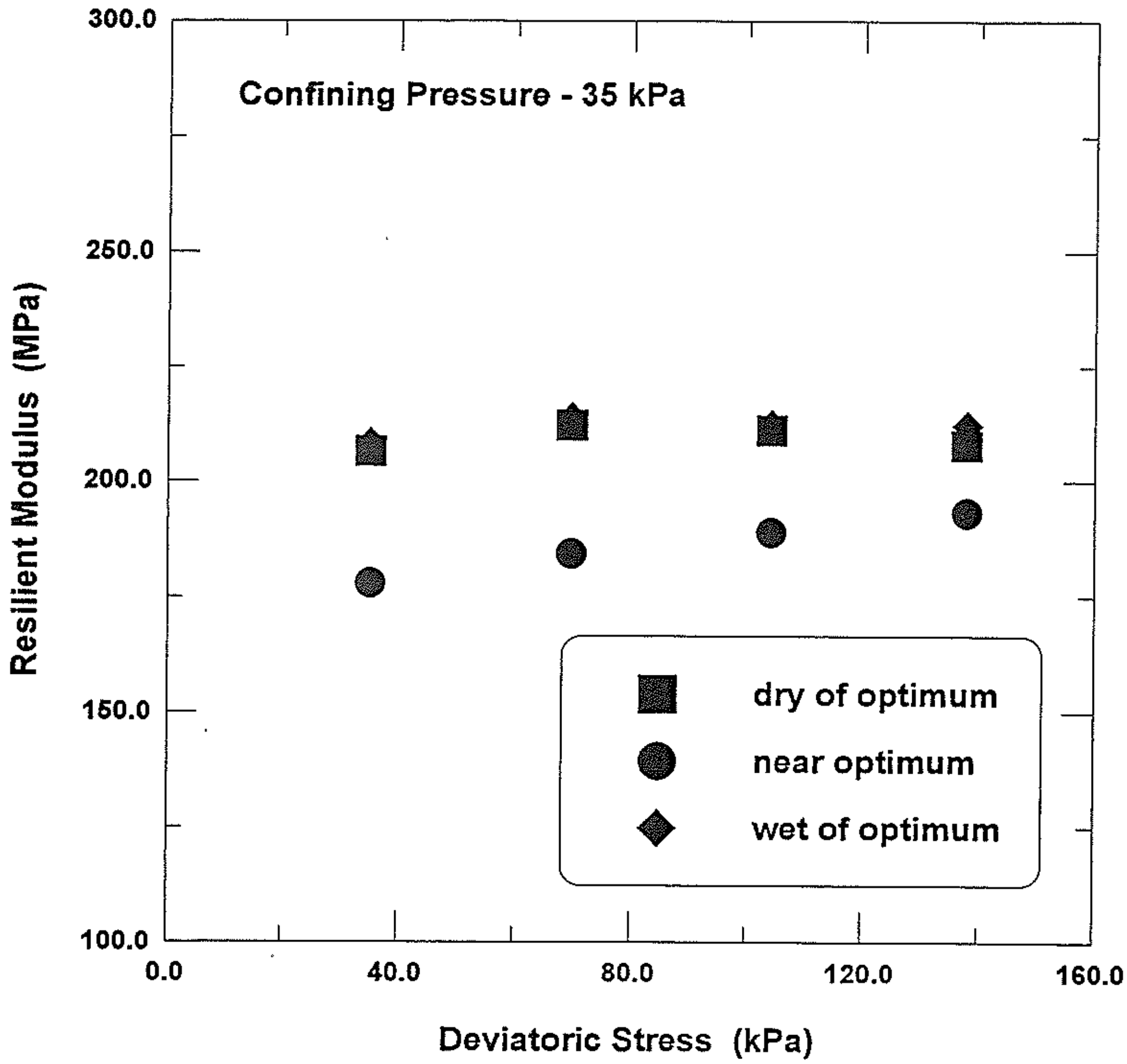


Figure 20: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus of Sands at a Confining Pressure of 35 kPa

Table

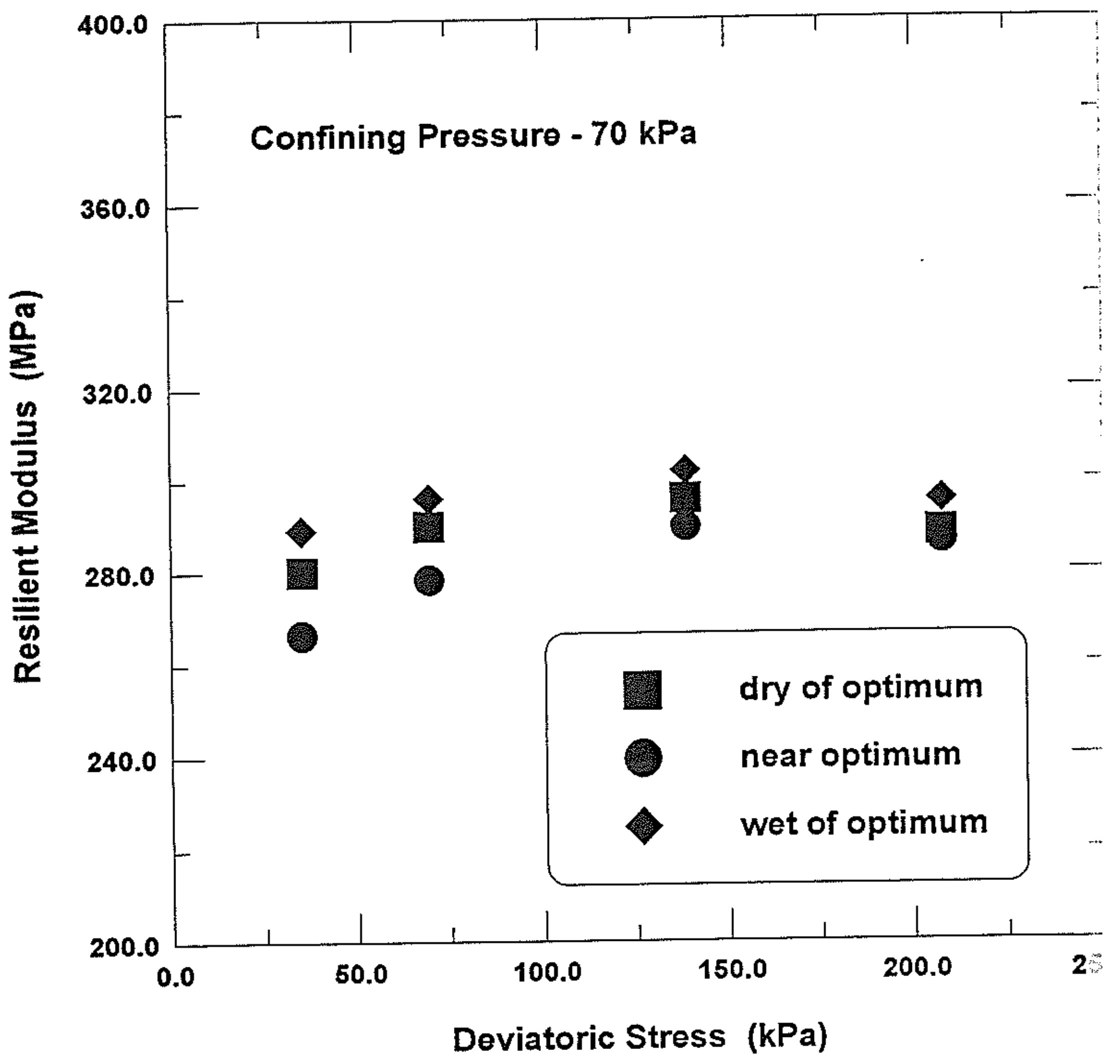


Figure 21: Influence of Moisture Content and Deviatoric Stress on Resilient Mod
Sands at a Confining Pressure of 70 kPa

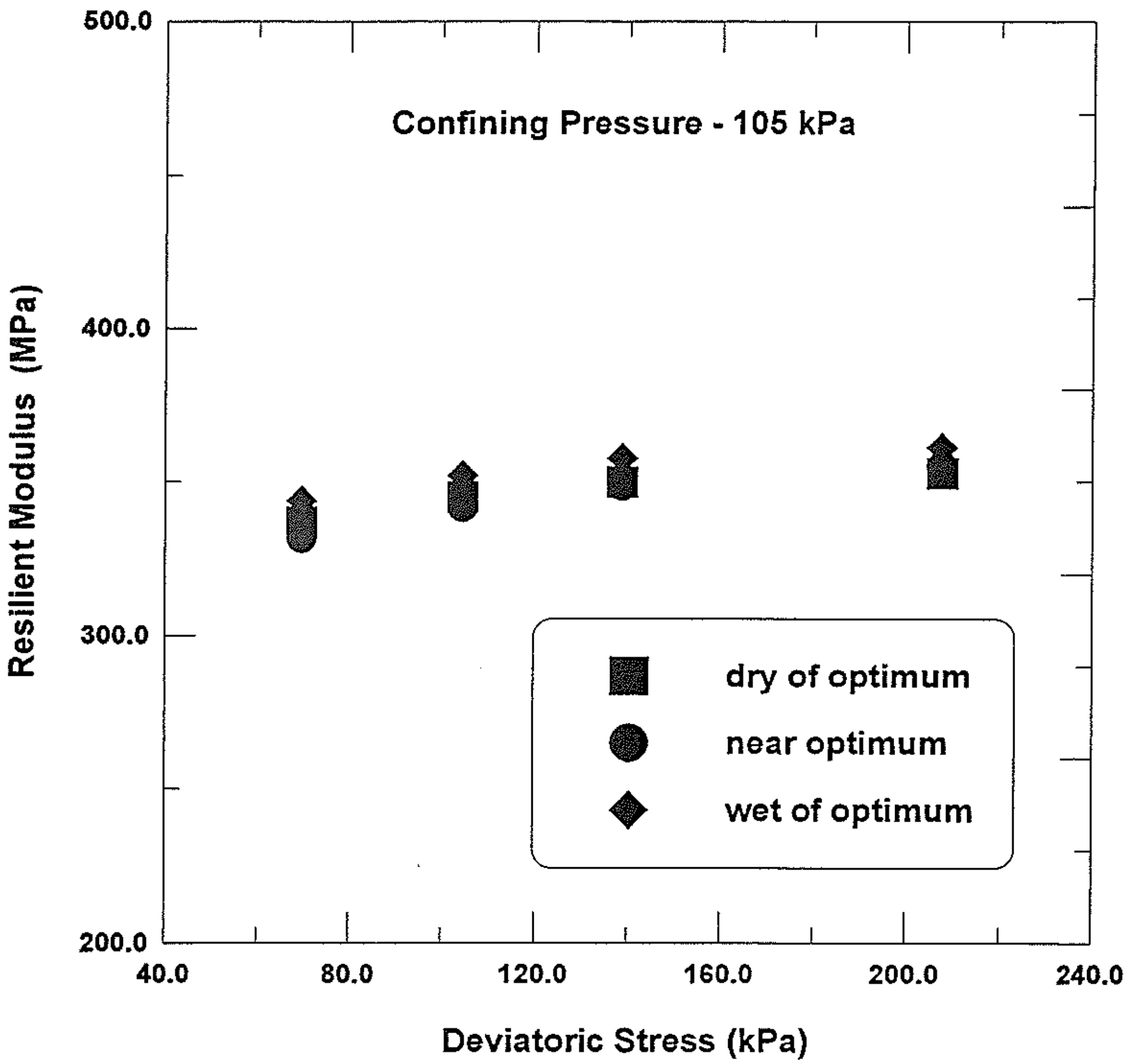


Figure 22: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus of Sands at a Confining Pressure of 105 kPa

Tabl

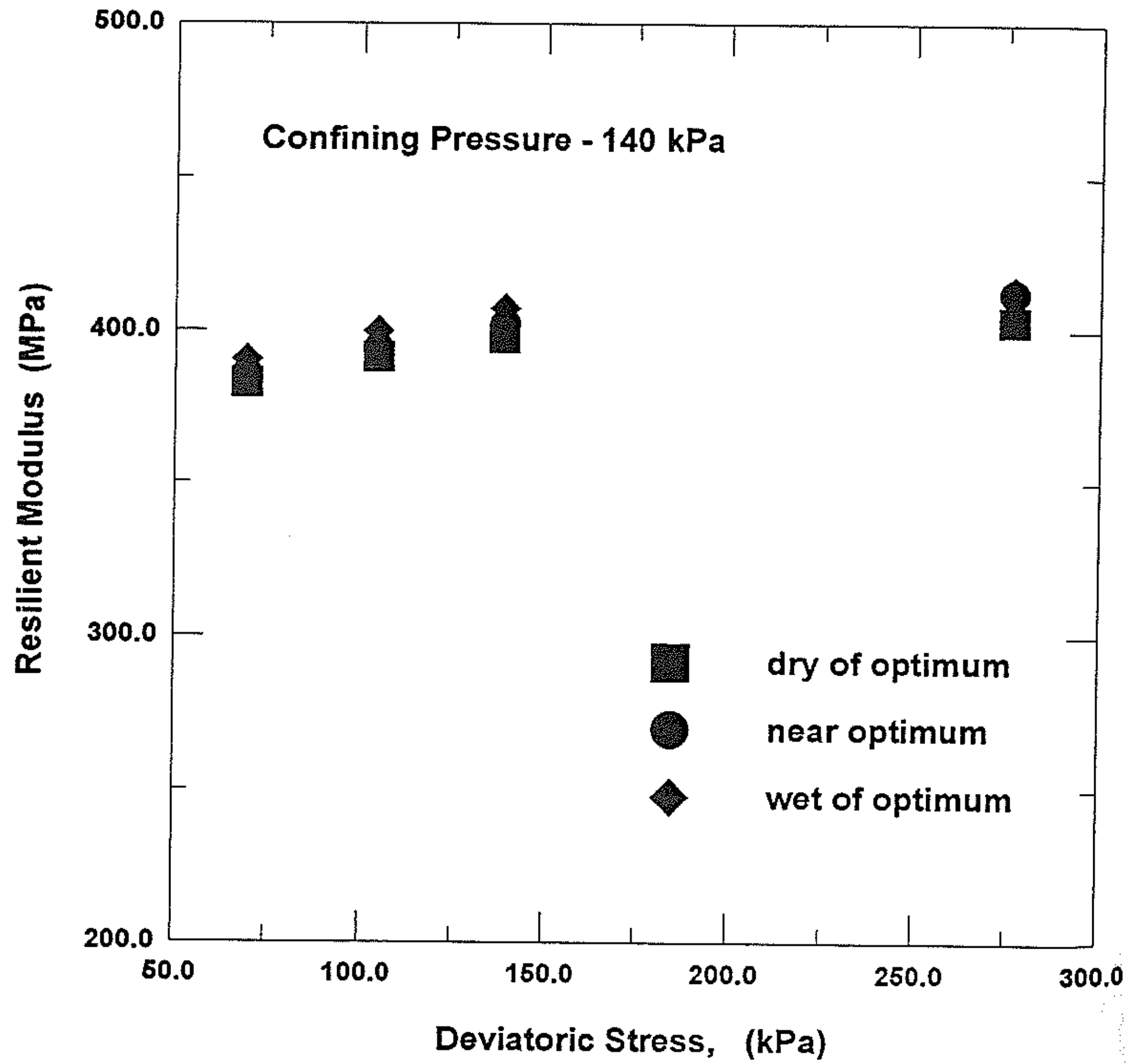


Figure 23: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus of Sands at a Confining Pressure of 140 kPa

The results at three moisture content levels appear to be close and similar. It should be noted that the relative compactions at these three moisture content-dry density levels vary from 98 to 100 percent which produce similar relative densities. This variation in relative compaction density is not significant enough to produce distinctly different M_r values.

Plastic Deformation Development

Figure 24 presents plastic deformations of sands measured by the end system during the conditioning and testing phases. All three moisture content levels are reported in this figure. The plastic deformations in the figure represent the accumulated deformations of one thousand cycles in the case of conditioning and four hundred cycles for each confining stress in the case of the testing phases. The testing phases have four hundred cycles obtained by summing the individual number of cycles (one hundred each) under four sets of deviatoric loads.

These results provided significant understanding of the conditioning role in this kind of testing. One of the main objectives of conditioning, as reported by the AASHTO T-292 procedure, is to reduce the plastic deformation development in the specimens. The deviatoric stress influence is apparent since higher deviatoric loads generally result in larger plastic deformations. The influence of confining pressure on the plastic deformations is more intricate and therefore requires further scrutiny and attention.

Small plastic deformations are obtained for sands at all testing confining stresses, 21, 70, 105 and 140 kPa, other than at 35 kPa. This indicates that the conditioning not only reduced the plastic deformations in the immediate testing confining pressure (which is 21 kPa), but also in the case of confining stresses (70, 105 and 140 kPa) which are closer to the conditioning confining stress of 140 kPa. This is a significant finding since no

Table 9:

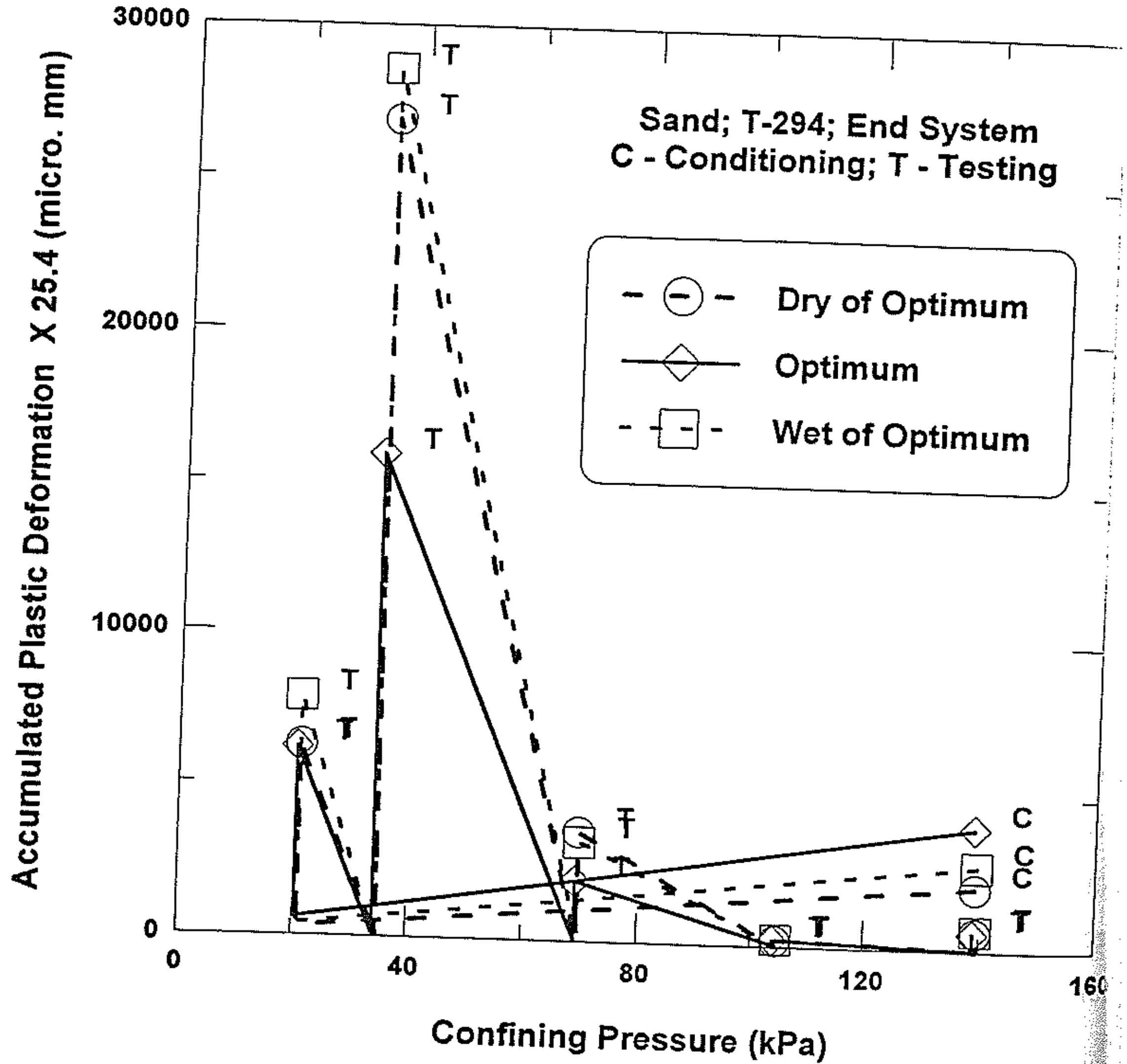


Figure 24: Plastic Deformations Developed at the End of Conditioning and Testing Phase Versus Confining Pressures (Sands, End System)

specific guidelines are available in the literature for determining the magnitudes of conditioning confining stresses of field core granular samples based on the plastic deformation criterion. The conditioning confining stress for cores should be greater than the lateral confining pressure of a depth at which the soil samples are retrieved. In certain cases, when the soil sample represents a significant depth of subgrade, the lateral pressure corresponding to the bottom layer of the subgrade should be used as the confining pressure for conditioning.

Higher plastic deformations are observed at 35 kPa confining stress, possibly due to significant fluctuations in the confining pressures in the preceding two stages, 140 kPa (conditioning) and 21 kPa (first level of testing).

Silty Clays - M_r Results

The silty clay results at optimum, dry of optimum and wet of optimum are shown in Tables 10, 11, and 12. The coefficient of variation of test results are slightly higher than those of sands. However, this number is still considered small when compared with the overall magnitudes of modulus values. This again indicates that the test results in silty clays are also repeatable. The slight increase in C_v values is probably due to the stress dependency and changes in fabric due to repeated loading. Detailed explanations of the stress dependency and fabric changes are explained in subsequent sections.

Figures 25, 26, and 27 present the silty clay specimen results at dry, optimum, and wet of optimum moisture content levels. End measurement results are presented in this figure. The increase in confining pressure resulted in an increase in moduli values. This, for silty clays, is attributed to a slight increase with the confining pressure in the overall strength. The results at three moisture content levels show that the M_r values at dry and optimum levels are close but significantly higher than those at wet of optimum. This is

Table 9

attributed to the strength decrease at wet of optimum as a result of presumed larger pressure development at higher saturation levels.

Table 10: Resilient Modulus Test Results on Silty Clay at Optimum Moisture Conte: Dry Density Combination (T-294)

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
42.1	27.4	224.5	25.8	11.5	261.0	48.9	18.8
42.1	13.5	240.0	22.8	9.5	288.5	65.7	22.8
42.1	27.6	226.8	24.9	11.0	261.3	49.1	18.8
42.1	40.7	203.5	26.4	13.0	232.9	43.5	18.7
42.2	54.3	185.6	26.4	14.3	208.1	40.0	19.2
42.2	67.8	171.1	27.0	15.8	187.6	39.3	20.9
21.5	13.5	218.3	19.4	8.9	271.4	56.0	20.6
21.4	27.4	199.9	22.2	11.1	238.8	44.2	18.5
21.3	40.6	182.5	23.4	12.8	215.9	41.5	19.2
21.4	54.3	167.2	24.3	14.5	196.2	39.8	20.3
21.4	67.8	155.6	25.4	16.3	180.0	40.0	22.2
1.1	13.3	178.1	15.9	8.9	254.7	52.3	20.5
1.0	27.1	159.9	18.1	11.3	222.4	44.0	19.8
1.1	40.1	145.1	19.9	13.7	200.5	42.2	21.0
1.0	53.8	134.6	21.1	15.7	182.6	42.0	23.0
1.1	67.3	127.9	22.3	17.4	167.1	41.6	24.9

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 11: Resilient Modulus Test Results on Silty Clay at Dry of Optimum (T-294)

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa) Mean	MRE (MPa) STD	MRE (MPa) CV	MRM (MPa) Mean	MRM (MPa) STD	MRM (MPa) CV
42.1	28.3	302.9	64.7	21.4	380.9	135.8	35.7
42.2	14.0	315.2	63.5	20.2	436.9	157.2	36.0
42.1	28.3	303.8	63.7	21.0	379.1	130.6	34.4
42.2	41.7	286.4	68.5	23.9	340.6	122.3	35.9
42.2	55.7	272.1	72.4	26.6	308.7	117.3	38.0
42.2	69.7	258.6	74.8	28.9	284.9	114.3	40.1
21.2	14.0	276.5	47.6	17.2	406.4	141.4	34.8
21.4	28.3	265.4	54.7	20.6	347.3	124.2	35.8
21.4	41.8	254.2	60.0	23.6	316.5	115.7	36.6
21.4	55.9	244.1	63.5	26.0	294.2	114.0	38.7
21.4	69.9	236.3	67.1	28.4	276.0	111.8	40.5
0.9	13.9	211.1	28.2	13.4	383.9	139.4	36.3
1.0	28.2	199.9	33.4	16.7	326.7	118.8	36.4
0.9	41.7	193.3	39.2	20.3	296.5	112.6	38.0
1.0	55.7	190.2	45.4	23.9	276.2	110.4	40.0
1.0	69.7	190.1	50.0	26.3	259.9	108.9	41.9

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 9:

Table 12: Resilient Modulus Test Results on Silty Clay at Wet of Optimum (T-29

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
42.3	28.1	164.5	15.2	9.3	187.1	21.4	11.4
42.3	14.0	187.3	12.7	6.8	221.3	22.4	10.1
42.3	28.1	166.8	14.2	8.5	189.0	20.5	10.8
42.3	41.3	138.3	17.6	12.7	154.3	22.4	14.5
42.3	54.7	116.3	19.0	16.3	125.8	23.0	18.3
42.3	68.2	100.1	18.8	18.8	104.1	21.8	20.9
21.3	13.9	174.4	13.1	7.5	202.4	25.2	12.5
21.2	28.1	148.6	16.3	11.0	164.2	22.7	13.8
21.2	41.4	123.9	18.6	15.0	134.4	23.9	17.8
21.2	55.0	103.7	19.3	18.6	110.7	23.7	21.4
21.2	68.3	89.4	18.2	20.4	93.7	22.0	23.5
1.2	13.8	144.4	11.1	7.7	180.5	24.9	13.8
1.1	28.0	118.7	13.4	11.3	142.9	23.9	16.7
1.2	41.1	98.0	15.4	15.8	115.3	24.6	21.3
1.2	54.5	83.3	15.8	19.0	95.0	23.1	24.3
1.2	68.0	73.3	15.2	20.8	80.4	20.6	25.7

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Increase in the deviatoric stress results in the reduction of moduli values of silty clays. This observation is consistent with those reported in other studies (Barksdale et al. 1990) and is attributed to several factors such as pore pressure development and fabric changes due to stressing cycles. The pore pressures which increase with deviatoric load magnitudes and cycles as well as saturation levels of the specimen result in the reduction of overall strength. The lower strength specimens yield lower moduli values. The fabric is defined as the particle orientation with respect to another particle. It is assumed that the fabric in the specimens at the beginning of testing (less dispersed) is significantly different from the one at the end of testing (more dispersed). Increased dispersion results in lower strength and resilient properties. The experimental verification of this assumption is beyond the scope of this investigation, but is still needs to be assessed.

Plastic Deformation Development

Figure 28 presents the plastic deformations for silty clay specimens developed during testing. Results from three moisture contents and densities are depicted in this figure. The figure suggests that the plastic deformations were larger at the confining pressure of 42 kPa and then started decreasing at lower confining stresses of 21 and 0 kPa. It should be mentioned that the tests for clays started at the same conditioning and first testing confining pressure of 42 kPa. This was followed by the testing performed at remaining confining pressures of 21 and 0 kPa. Even though conditioning did not result in the

Table 9:

reduction of plastic deformations at 42 kPa, it significantly decreased the plastic deformations at the other confining pressures of 21 and 0 kPa. The role of condition[illegible] silty clays is probably realized at lower confining stresses. The reason for not obtai[illegible] lower plastic deformations at 42 kPa is attributed to the stiffening or over consolidat[illegible] the specimen at 42 kPa confining pressure. This implies that in the case of field coh[illegible] core samples, a conditioning confining stress which needs to be significantly higher [illegible] the lateral confining pressure of the retrieval depth location is required to reduce pla[illegible] deformations. As expected, smaller plastic deformations are measured by the middle system than the end system, possibly due to the differences in the lengths that the measurement systems are accounted for.

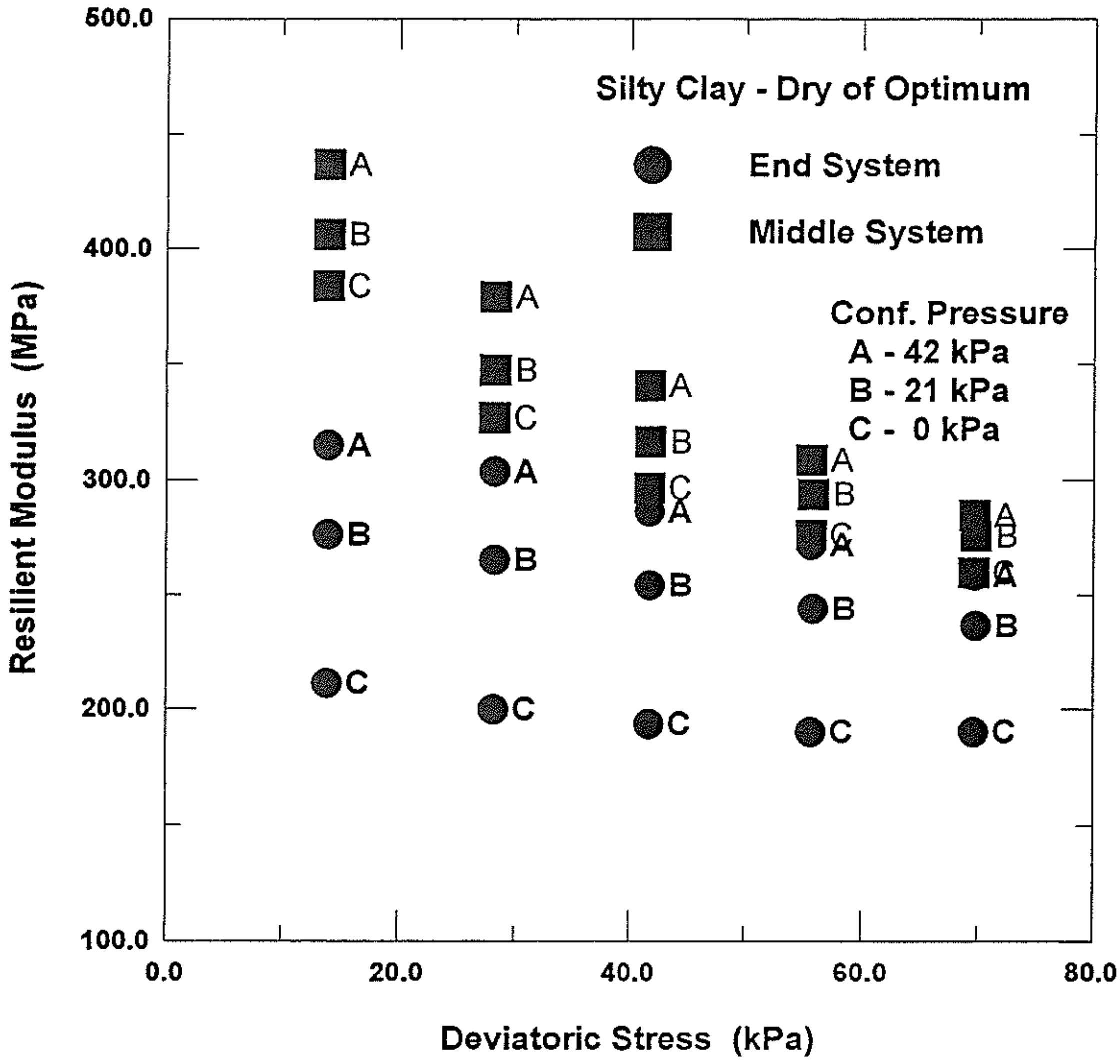


Figure 25: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus at Dry of Optimum (Silty Clays)

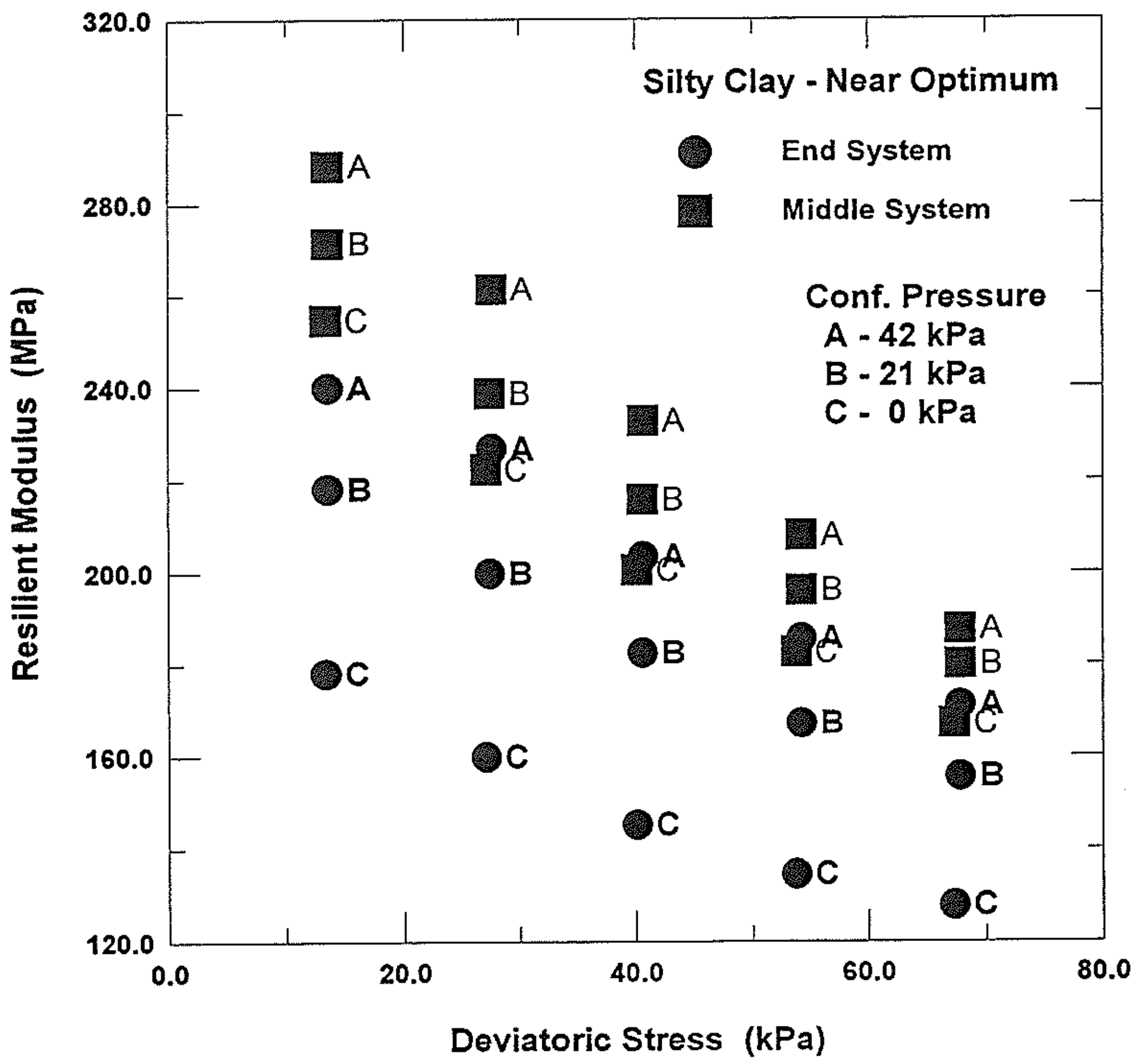


Figure 26: Influence of Moisture Content and Deviatroic Stress on Resilient Moudul

Optimum (Silty Clays)

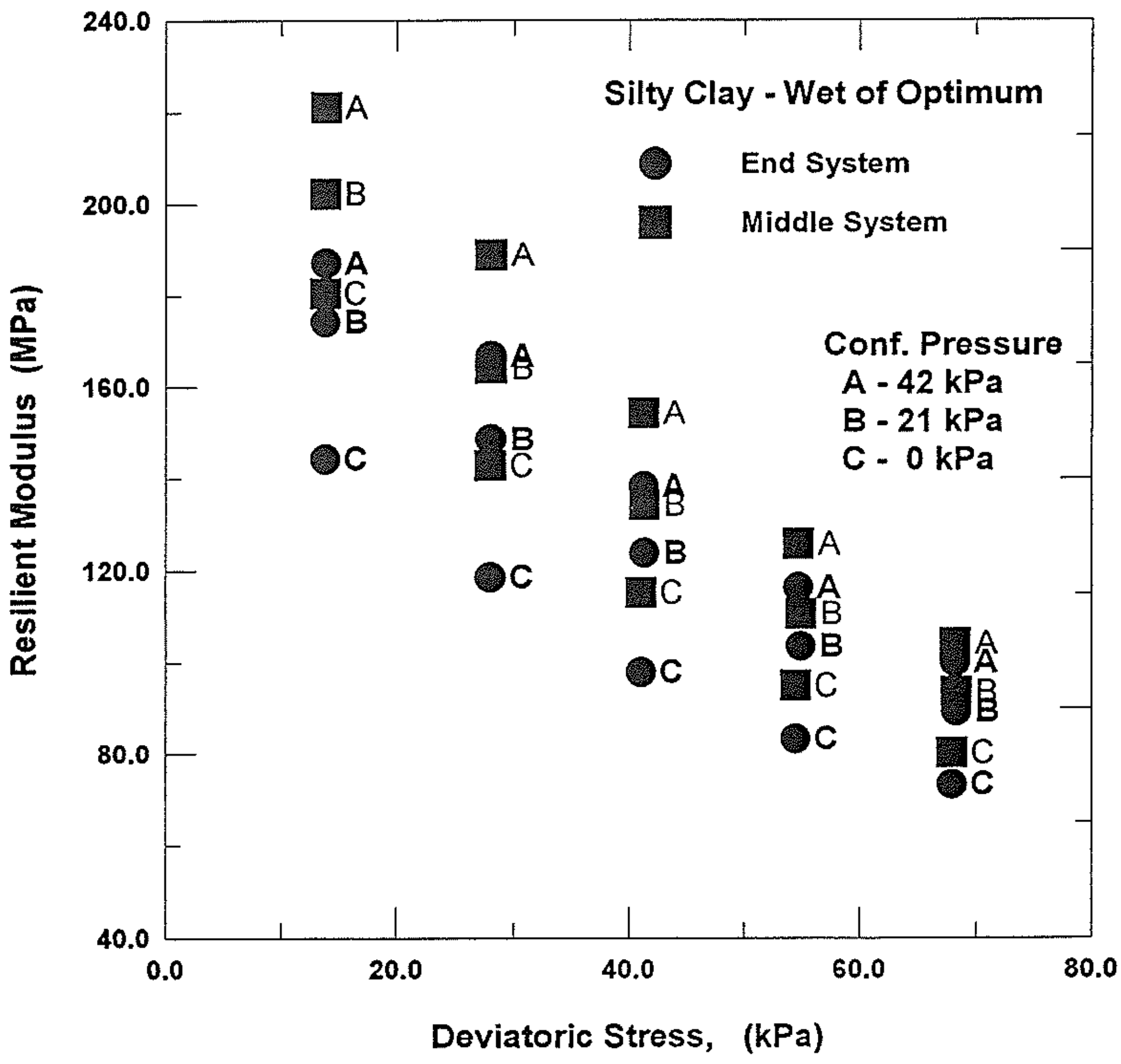


Figure 27: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus at Wet of Optimum (Silty Clays)

PROCEDURE VARIABLES

Two types of AASHTO procedures were used in the testing. The differences between these procedures were explained earlier. The results on both soils are presented in the form of a simplified normalized factor termed the procedure coefficient. The procedure coefficient (PC) is defined as the ratio of the M_r value obtained from the AASHTO T-294 procedure to that obtained from the AASHTO T-292 procedure. The T-292 procedure value is taken as the reference value to which the comparisons are made. In other words, the PC values represent the variations of M_r from the T-294 procedure with respect to the same from the T-292 procedure. The PC values of sands for confining and deviatoric stresses are determined for each measurement system. Results obtained with the old equipment were used for this purpose.

The PC values are always greater than one, which implies that the T-294 procedure provided higher modulus values than the T-292 procedure. This, as explained earlier, is attributed to the bulk stress variations in both procedures. The PC values of the sand for both measurement systems are shown in Figure 29. The PC values are as high as 1.28 at low confining (35 to 105 kPa) and deviatoric stresses ($<$ 70 kPa) and are reduced to around 1.15 with the increase in these stresses. Both measurement systems produced similar results. At low confining stresses (35 to 105 kPa) and deviatoric stresses (less than 70 kPa), the previous sequence of the testing had a certain influence on the moduli which resulted in higher PC values. This influence, however, is not observed at higher devi stresses (greater than 70 kPa) which implies that test procedures have a minor influen M_r values at these stresses. This is probably because the higher deviatoric stresses ap to the specimen will overcome the stress dependency effects due to previous testing st sequences. However, surprisingly for both measurement systems, lower PC values wi average value of 1.08 are observed for the tests conducted at the lowest (21 kPa) and highest (140 kPa) confining pressures. The lower values at higher confining pressure be reasoned from previous explanations. However, the same can not be explained in t case of lowest confining pressure (21 kPa) results. After additional examinations, the following observation can be assumed to be one of the reasons for the lower values. procedures tested the samples at this confining stress, 21 kPa, either at the end of the testing as in the case of T-292 or in the beginning of T-294 in which this test was prec by a conditioning at a high deviatoric stress (140 kPa). Therefore, in both procedures, previous conditioning (T-294) and testing (T-292) stabilized the sample and reduced th stress dependency behavior to a certain degree beyond which the testing procedures di result in any significant variation in the results.

The following equation is derived based on the results reported in Figure 29. This equation, which provides the procedure coefficients, is valid for both measurement sys and confining pressures of magnitudes 35, 70, and 105 kPa. For other confining press of 21 and 140 kPa, the coefficients remain constant for all deviatoric stresses and are around 1.08.

$$PC = 1.28 - 0.00115 * \sigma_d \tag{5}$$

where σ_d is the deviatoric stress in kPa.

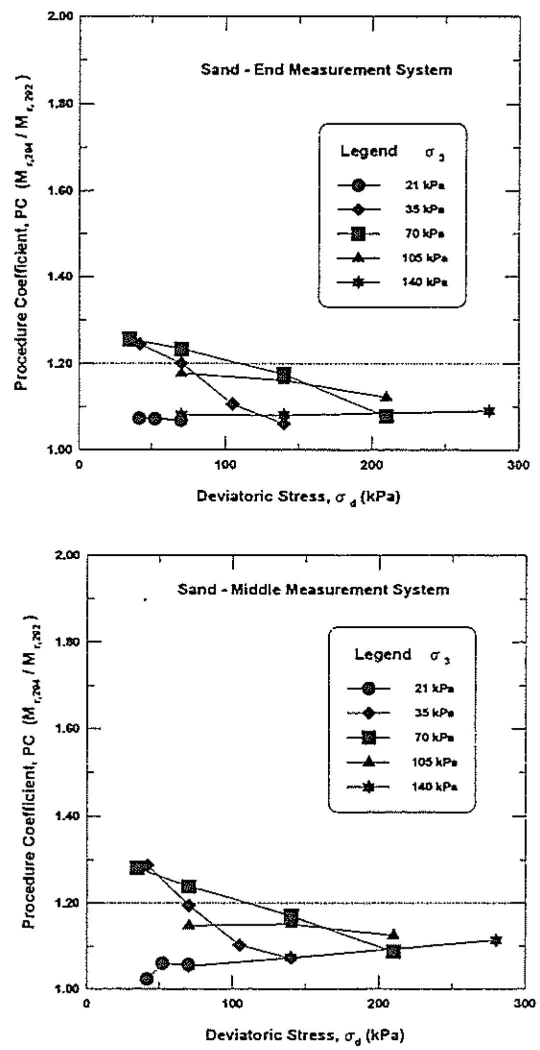


Figure 29: Procedure Coefficients of Sands: Influence of Testing Procedures

Figure 30, depicting the procedure variation on silty clays, has not provided enough information for discussion. This is because in testing on silty clays, the AASHTO procedures have different confining pressure and deviatoric stress sequences. The only common test stresses at which both procedures are conducted are the confining pressure 21 kPa and the deviatoric stress of 72 kPa. In order to determine another PC value, re from the deviatoric stress of 55 kPa in T-294 and 52 kPa in T-292 are assumed to be equivalent. The PC values of these two deviatoric stresses are calculated and are shown the same figure. The coefficients from both measurements are approximately 1.0 excep the middle system which has a value of 0.8 at 72 kPa deviatoric stress and 21 kPa confining pressure. Swelling phenomena and stress dependency may have occurred for specimens tested under the T-294 procedure at 21 kPa confining pressure due to a drop from the previous confining stress which is 42 kPa. This phenomenon appears to have more influence on middle measurement results. Therefore, lower M_r and PC values are calculated by the middle measurement system at 72 kPa deviatoric stress. Overall, the procedure variation on M_r values is not as significant as in the case of sands. Two reas for this are that the test procedure for silty clay specimens do not have a wide range of testing stresses, and the magnitudes of confining pressures for cohesive soils are low (0 42 kPa range).

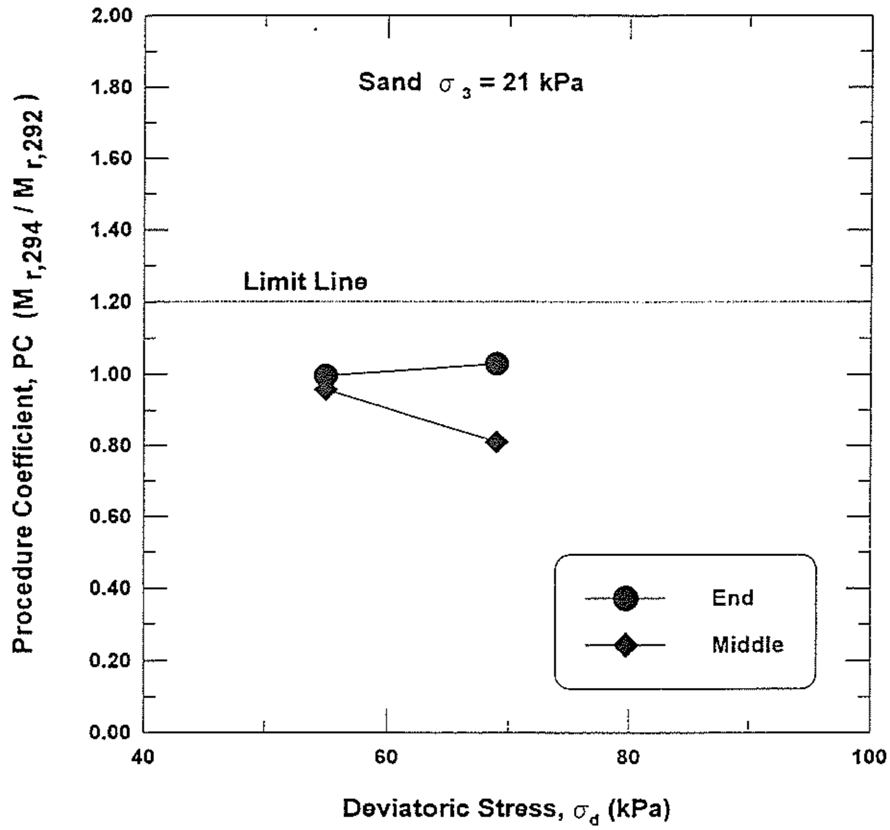


Figure 30: Procedure Coefficients of Silty Clays: Influence of Testing Procedures

MEASUREMENT VARIABLES

In this section, the discussion is devoted to the statistical variations between end and middle measurements. This is followed by another section in which the measurement coefficients which quantify the variations between end and middle measurement moduli are introduced.

Statistical Significance

To ascertain the significance of the difference in M_r values computed based on the deformations measured by the two measurement systems on the same specimen, the following three hypothesis tests need to be performed:

i. $H_0 : \mu_e = \mu_m \qquad H_a : \mu_e \neq \mu_m$

where μ_e is the mean M_r for the end measurement system and μ_m is the mean M_r for the middle measurement system. This test compares the means of the two groups, the end system, and the middle system at the overall level. At the overall level, the data set for each group contains M_r values at all stress levels for all thirty specimens.

ii. $H_0 : \mu_{ce} = \mu_{cm} \qquad H_a : \mu_{ce} \neq \mu_{cm}$

where μ_{ce} is the mean M_r for the end measurement system at a certain confining pressure, σ_3, and μ_{cm} is the mean M_r for the middle measurement system at the same confining pressure, σ_3. This test compares the means of the two groups, the end system, and the middle system at each confining pressure level. At each confining pressure level, the data set for each group contains M_r values at all deviatoric stress levels for all the thirty specimens at that confining pressure.

iii. $H_0 : \mu_{de} = \mu_{dm}$ $H_a : \mu_{dc} \neq \mu_{dm}$

where μ_{de} is the mean M_r for the end measurement system at a certain deviatoric stress, σ_d, and μ_{dm} is the mean M_r for the middle measurement system at the same deviatoric stress, σ_d. This test compares the means of the two groups, the end system, and the middle system at each deviatoric stress level. At each deviatoric stress level, the data set for each group contains M_r values at each deviatoric stress level for all the thirty specimens at each confining pressure.

Assuming that the two groups are independent and each of them follows a normal distribution, the two sample t-test procedure is used for the above hypothesis tests. A 95 percent confidence level is used.

The paired t-test was performed on sand and silty clay test results obtained at optimum water content - dry density combinations. These levels typically represent other level moisture contents also, therefore, conclusions based on t-tests of the above results are applicable to other levels. The results on sands indicate that at the 95 percent confidence level, the middle measurements provided higher moduli than the end measurements. This observation is valid in all the above three cases.

Measurement Coefficients

In general, external measurements (outside the triaxial cell) were used in the rep triaxial tests. However, due to the magnitudes of the strains measured (less than one percent in most cases), it is necessary to use high precision LVDTs on the specimen inside the triaxial cell. Otherwise, air gaps between specimens and accessories such porous stones and platens, system compliance, and errors such as sample alignment bedding problems would induce significant errors to the measurements. The internal measurements, though hard to install, provide results which are less influenced by th system compliance of triaxial cell accessories. This is the reason behind using the in measurements in the present tests.

The influence of the measurement system is presented in the form of measureme coefficients (MC). This coefficient is defined as the ratio of the resilient modulus or strain measured by the middle system to that measured by the end system. These coefficients are determined for both procedures and test stresses. The coefficient ca used to convert the end measurement results to more realistic middle measurement s results.

The next two sections discuss the influence of stresses and moisture contents on measurement coefficients (MC). Results from both soils at optimum dry density - w content level are used to understand the influence of stresses, whereas the complete test results are used to understand the influence of moisture contents.

Influence of Confining and Deviatoric Stresses

The sand (optimum moisture content) and the silty clay (optimum moisture content) results obtained from the old equipment are used in this section. Figure 31 presents the variation of MC values of sands for both AASHTO procedures. The MC values range from 1.20 at lower confining and deviatoric stresses to 1.08 at higher confining and deviatoric stresses. The lower value at the higher stresses is probably obtained because of the near perfect contacts between the end platens, porous stones and the specimen ends. This may be the reason that both measurement systems yielded similar values. Because of the small variation of MC values at various confining stresses, an average measurement coefficient value of 1.14 is recommended at all confining pressures for converting M_r values from the end system to the M_r values of the middle system.

Figure 32 presents the MC values obtained from results on silty clay specimens. The influence of the measurement system can be clearly seen from this figure. MC values ranging from 1.5 to 1.6 are observed for unconfined conditions. These significantly higher coefficients are due to the complex behavior of silty clay specimens which can result from the specimen preparation, the stress history due to the stress sequence of the testing the imperfect end contacts, and system compliance errors. It should be noted that T-292 sh only loading sequence and T-294 shows both loading and unloading sequences. Specim preparation using standard proctor tests may not produce the same soil fabric. This, coupled with the variations as a result of test stress sequences which induce stress dependency behavior and errors due to improper instrumentation, significantly influen the displacement measurements. The end system which measures the displacements ove the full length of the specimen will be influenced more by these problems than the mid system. The end system, therefore, measured significantly higher displacements resultin in lower M_r values and higher measurement coefficients. In addition, these problems w be more significant in unconfined conditions than in confined conditions. This is the re that higher MC values are obtained in the unconfined state. The MC values decrease w an increase in confining stress and to a certain extent with an increase in deviatoric str The MC values from both test procedures, which match at 21 kPa confining pressure, compared in the same figure. These values are similar and vary between 1.2 to 1.52.

The following measurement coefficient equation for silty clays is derived based on th results from Figure 32. The deviatoric stress is not taken into account in the equation s its influence on MC value is relatively insignificant when compared with the confining pressure.

$$MC = 1.52 * e^{-0.00594 * \sigma 3}$$

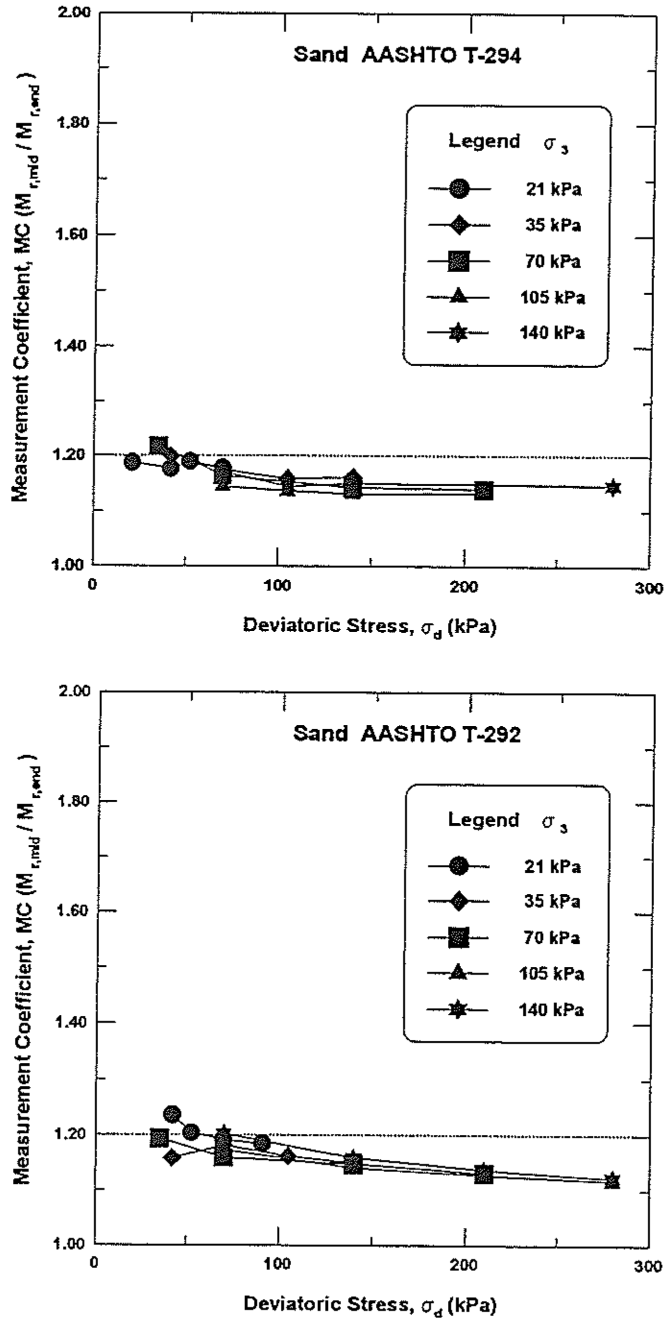


Figure 31: Measurement Coefficients of Sands: Influence of LVDT's Location

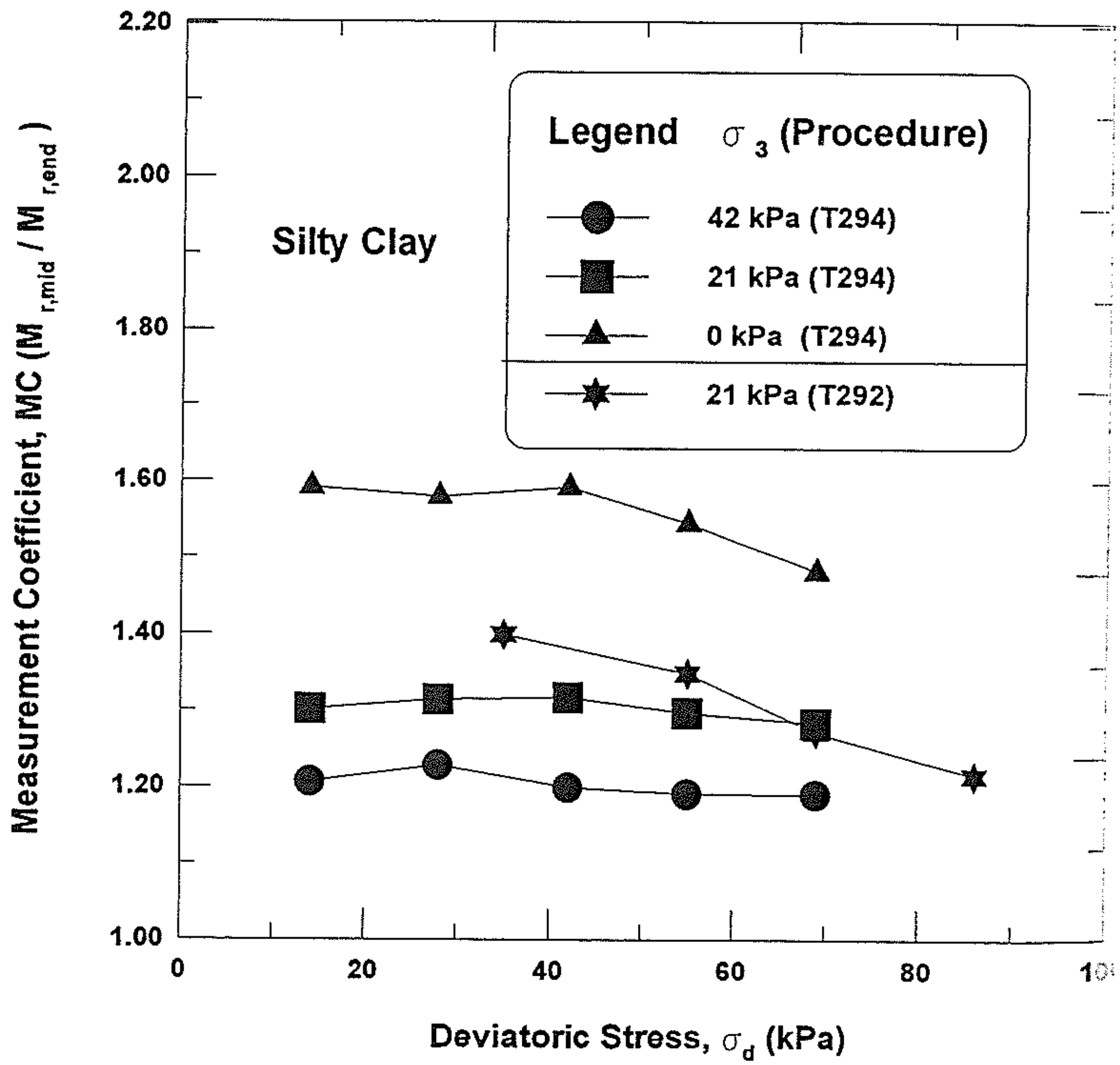


Figure 32: Measurement Coefficients of Silty Clays: Influence of LVDT's Locat

Influence of Moisture Contents on Measurement Coefficients

Sands

Results obtained from the T-294 tests conducted on sands at three moisture content levels are used to determine the measurement coefficients. These results were used to prepare Figure 33. The figure for sand test results is plotted by showing M_r values obtained from the middle system on the Y axis and M_r values from the end system on the X axis. All three moisture content results are plotted in the same figure. The best fit lines are plotted passing through the data and the origin. The slopes of these lines are the measurement coefficients. The influence of stresses on these results are ignored since the sand results reported in the previous section indicated that the stress influence does not significantly alter the MC values.

The measurement coefficients obtained from the figure are 1.15 (dry of optimum), 1.18 (optimum), and 1.22 (wet of optimum). Slightly higher values are obtained for wet of optimum tests since this level in the specimen indicates softness of the material due to increase in degree of saturation. The strength at higher moisture content levels is considerably less and this loss of strength appears to influence the flexibility of the specimen in holding on to the clamp system. This led to significant differences in the moduli results of the measurement systems, particularly at wet of optimum level.

Silty Clays

Figure 34 compares the measurement coefficients versus deviatoric stresses for va confining pressures (0 to 42 kPa). The influence of confining pressure appears to be evident in this case and, therefore, the stresses are included in the following analysis. Higher measurement coefficients were obtained for an unconfined state. Reasons for t explained in the earlier sections. These values decrease with an increase in the confini stress. This probably indicates that higher confining stresses provide better contact bet LVDTs and specimens and allow more accurate measurements. Even though the autho not notice any visual slipping problems in this testing, slipping at unconfined states ma have occurred which probably influenced the middle measurement results. The results figures are used to provide the following equations for measurement coefficients. Line regression analysis is used to obtain the following equations.

$$MC = (0.00335*\sigma_3-0.051)\ \sigma_d + (1.83-0.0702*\sigma_3) \qquad (7$$

$$MC = (0.00032*\sigma_3-0.013)\ \sigma_d + (1.43-0.0402*\sigma_3) \qquad (8$$

$$MC = (0.000298*\sigma_3 - 0.017)\ \sigma_d + (1.26 - 0.0124*\sigma_3) \qquad \textbf{(9)}$$

Unlike in sands, the increase in moisture content levels showed a significant variation in the silty clay test results. Higher variation of MC values were obtained at the dry state than at the wet of optimum state (Figure 34). This is attributed to the pore pressure development as well as the fabric changes. The wet of optimum state has higher degree of saturation level than the dry and optimum state. The pore pressure developments at this state are assumed to be uniform throughout the specimen and, consequently, the variations in measurements are also uniform.

The fabric at wet of optimum is a more dispersed structure, whereas the fabric at dry of optimum is a more flocculated structure. The fabric can change more significantly at the dry of optimum than at the wet of optimum due to repeated loading. Therefore the fabric results in variations in measurements and MC values in silty clays.

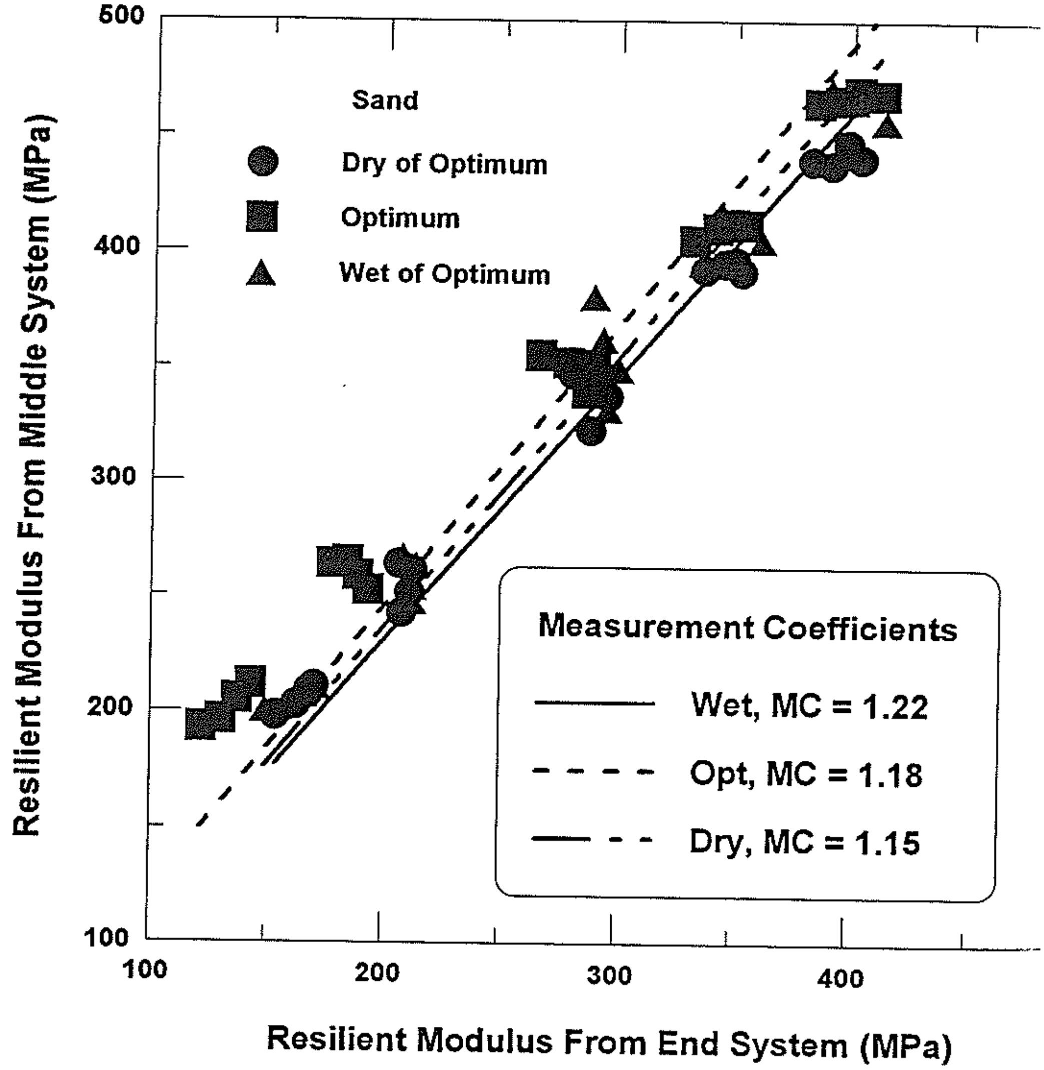


Figure 33: Influence of Moisture Contents, Confining Pressure and Devitoric St

Measurement Coefficients of Sands

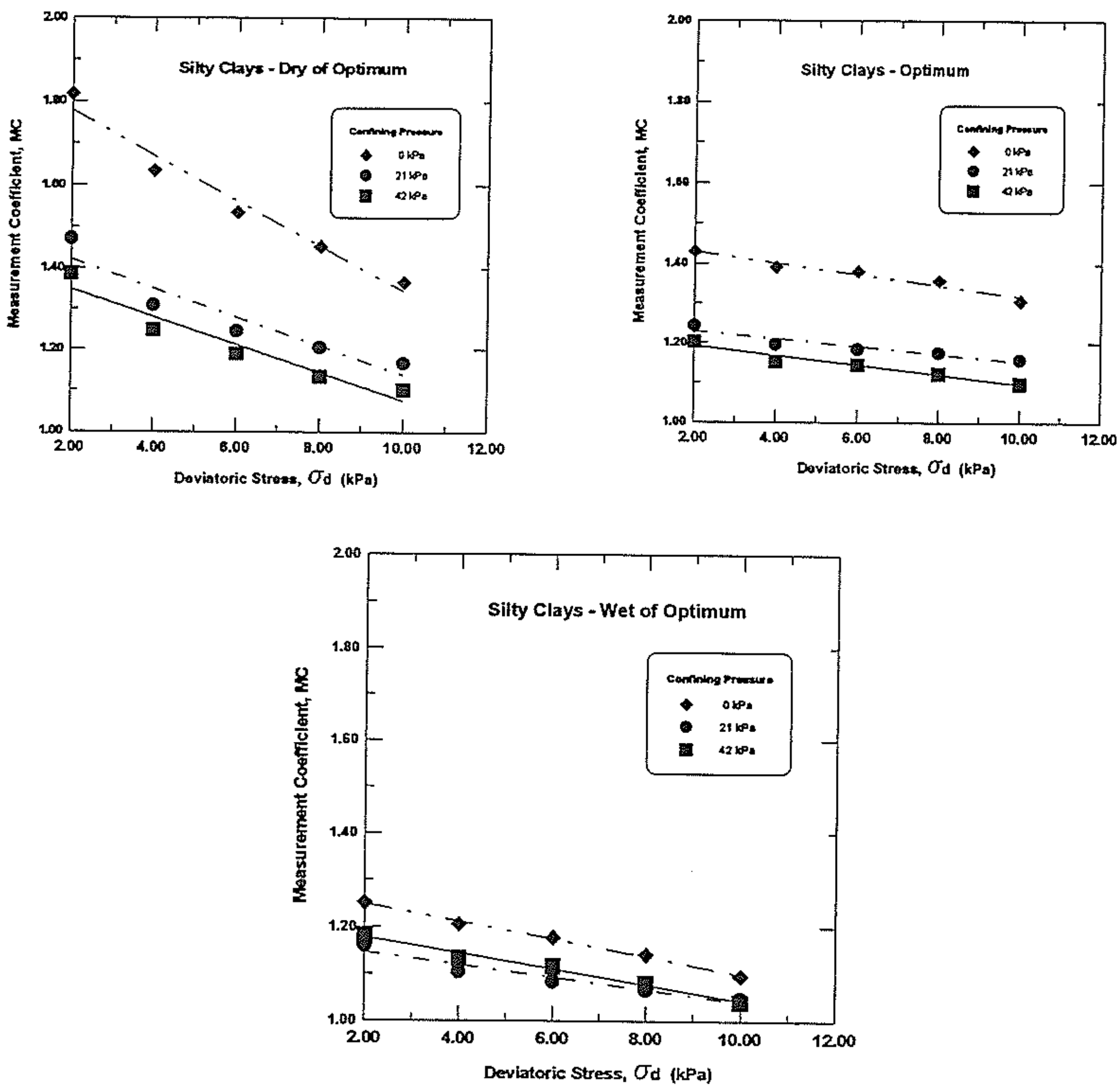


Figure 34: Influence of Moisture Contents, Confining Pressure and Deviatoric Stresses on Measurement Coefficients of Silty Clays

REGRESSION MODELS AND CONSTANTS

Regression models are used in the form of equations for predicting the moduli. Th (θ) or the bulk stress and the deviatoric stress are used as predictors in these models on whether the soil is cohesionless or cohesive. These models were recommended in AASHTO T-292, and T-294. The models can be expressed as:

$$M_r = k_1 * \theta^{k2} \quad \textbf{Granular Soils}$$

$$M_r = k_3 * \sigma_d^{k4} \quad \textbf{Cohesive Soils}$$

where k_1 and k_2 (granular soils); k_3 and k_4 (cohesive soils) are regression coefficients.

The regression coefficients are determined from the test results of both soils (Figur 36, 37 (sands), 38, 39, 40 (silty clays), and 41 (sand results from old MTS equipment regression model constants are presented in Tables 13 (sands) and 14 (silty clays). The regression constants are analyzed in the following sections with respect to AASHT procedures and measurement locations as well as soil characteristics.

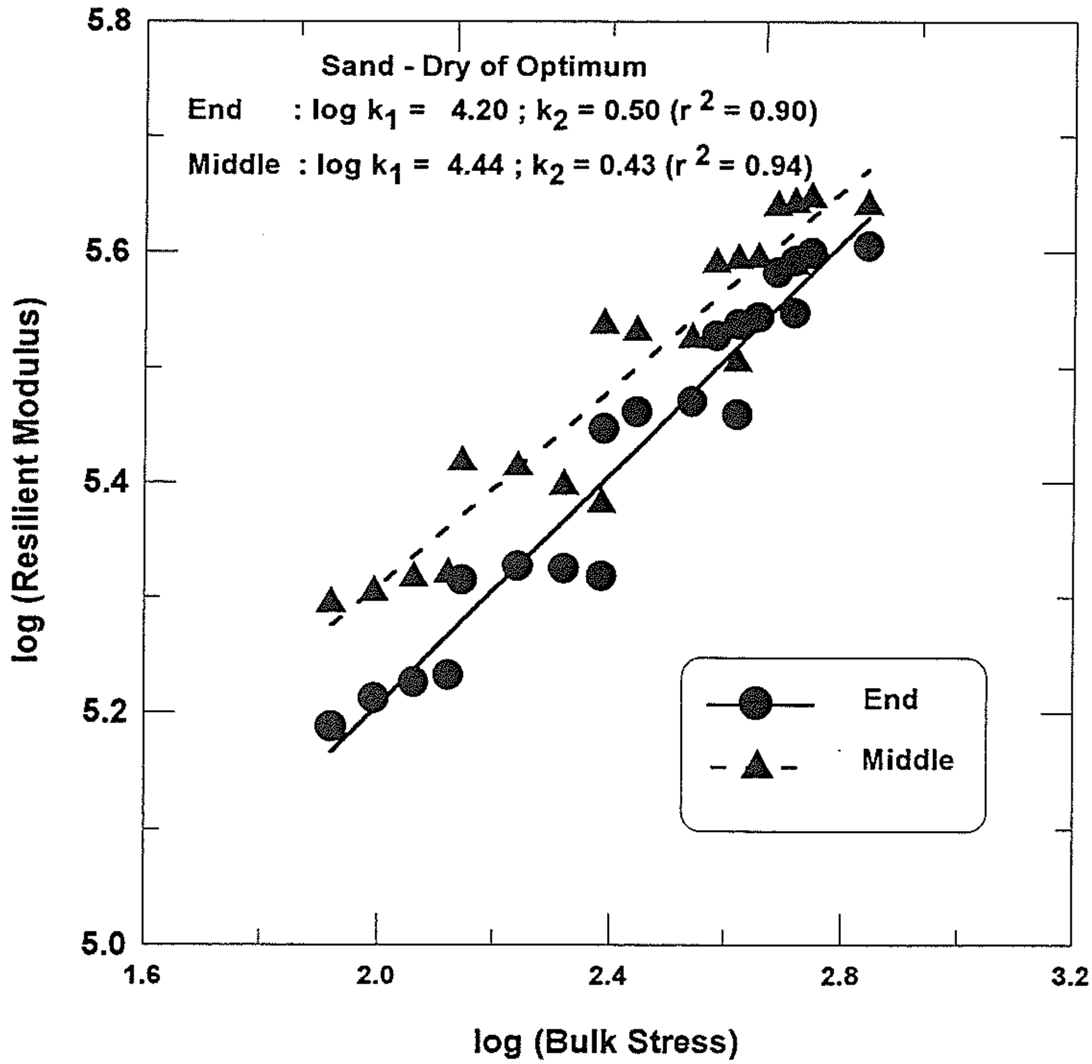


Figure 35: Regression Model for Sands at Dry of Optimum

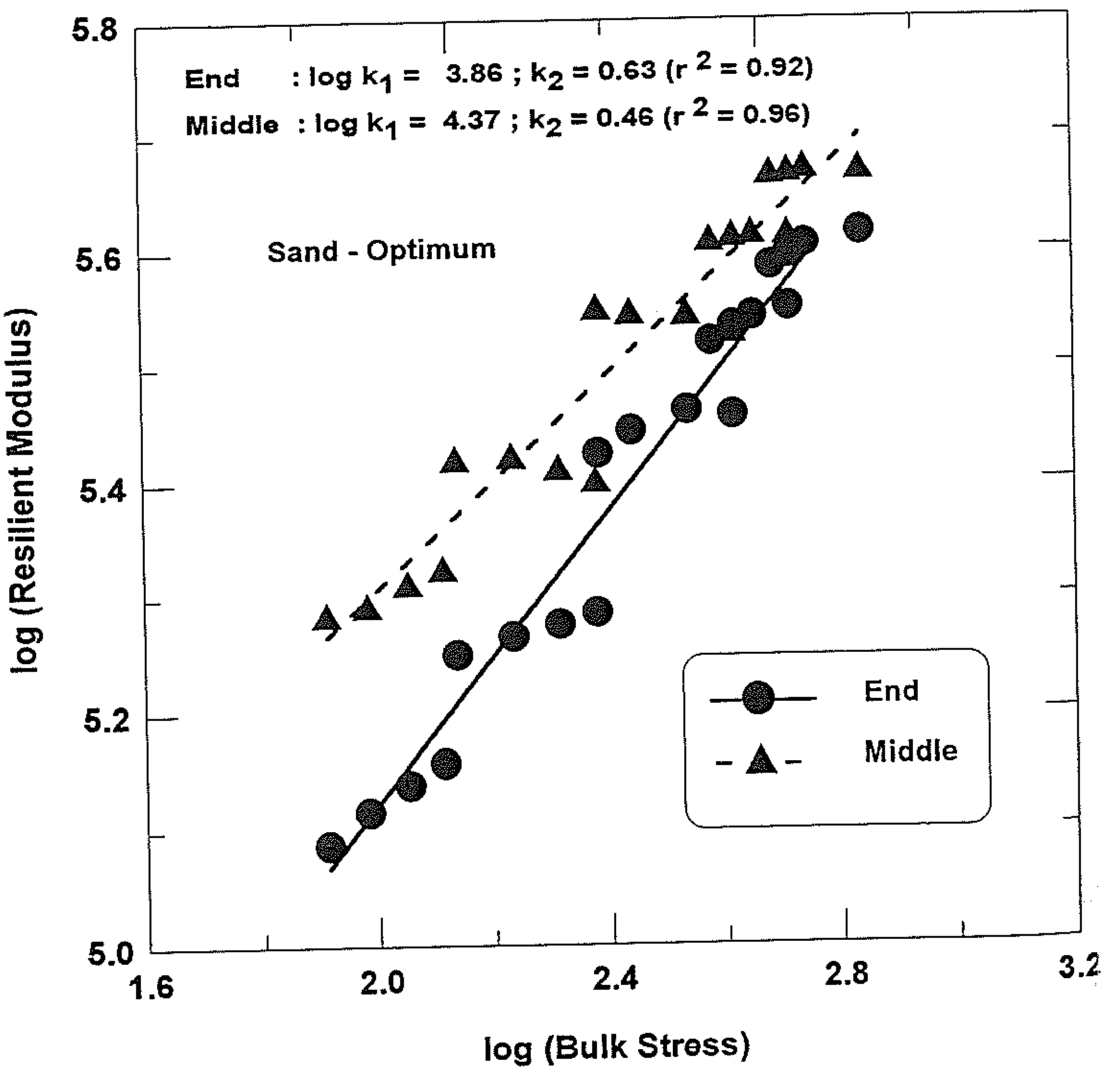


Figure 36: Regression Model for Sands at Optimum

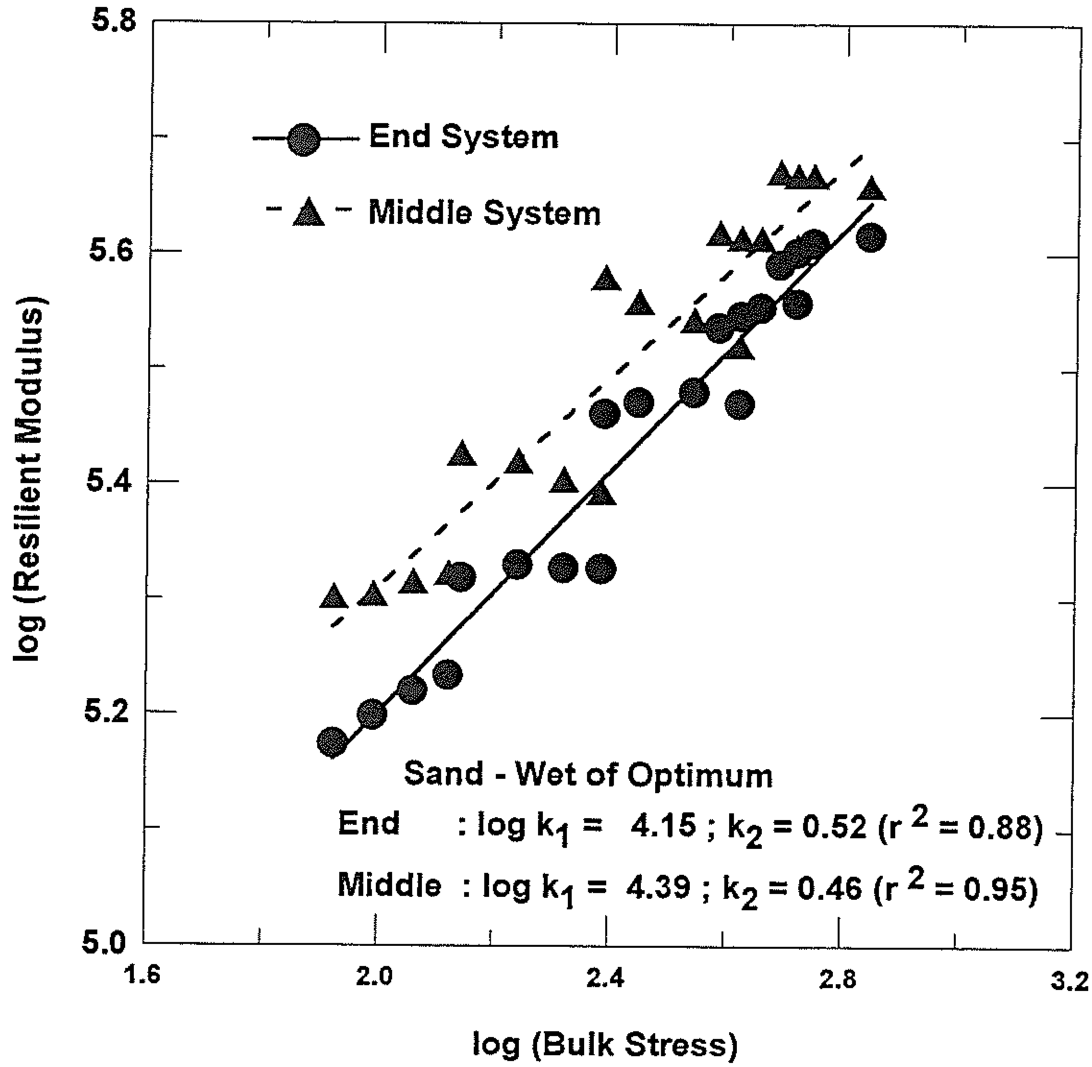


Figure 37: Regression Model for Sands at Wet of Optimum

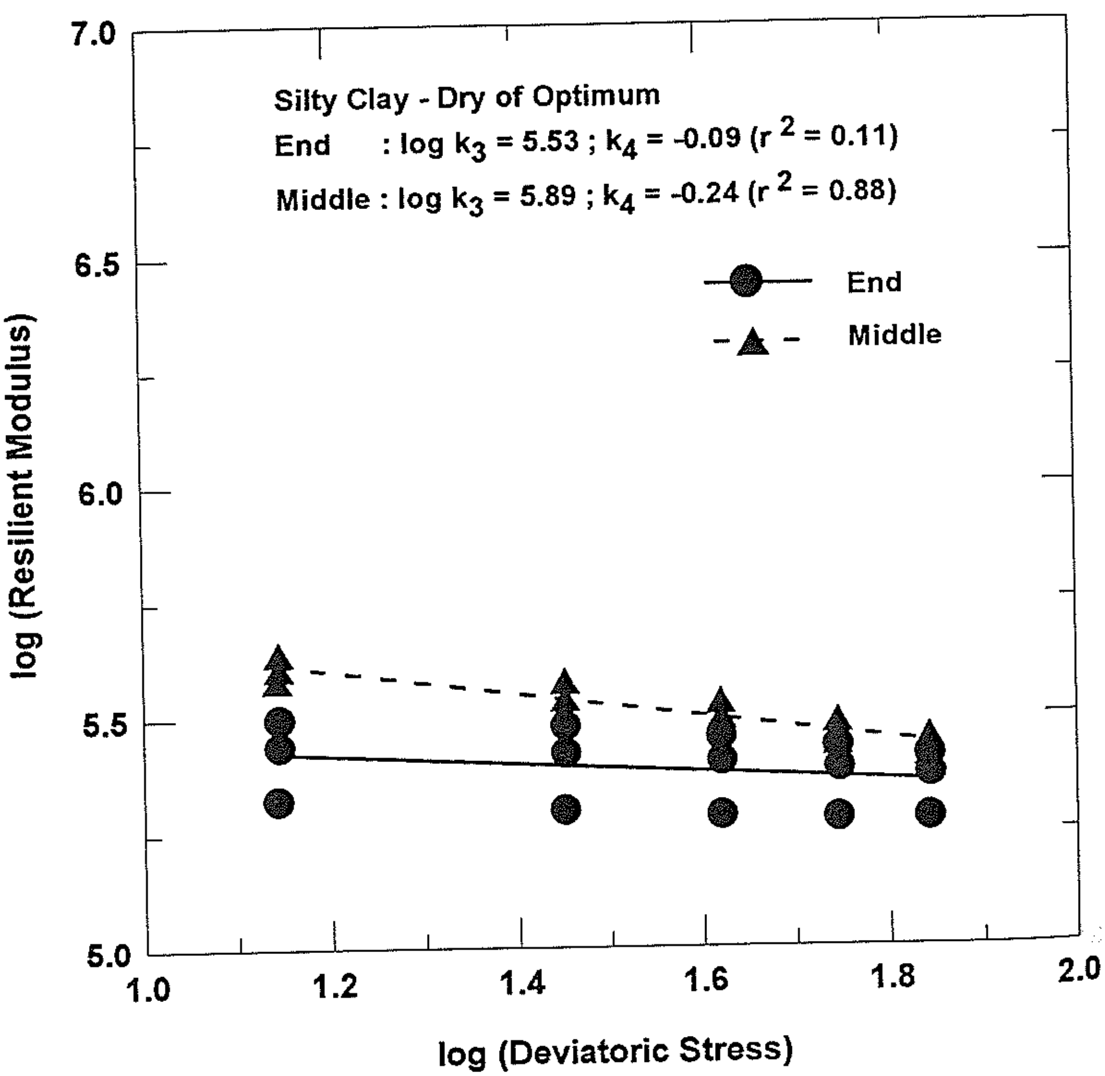


Figure 38: Regression Model for Silty Clays at Dry of Optimum

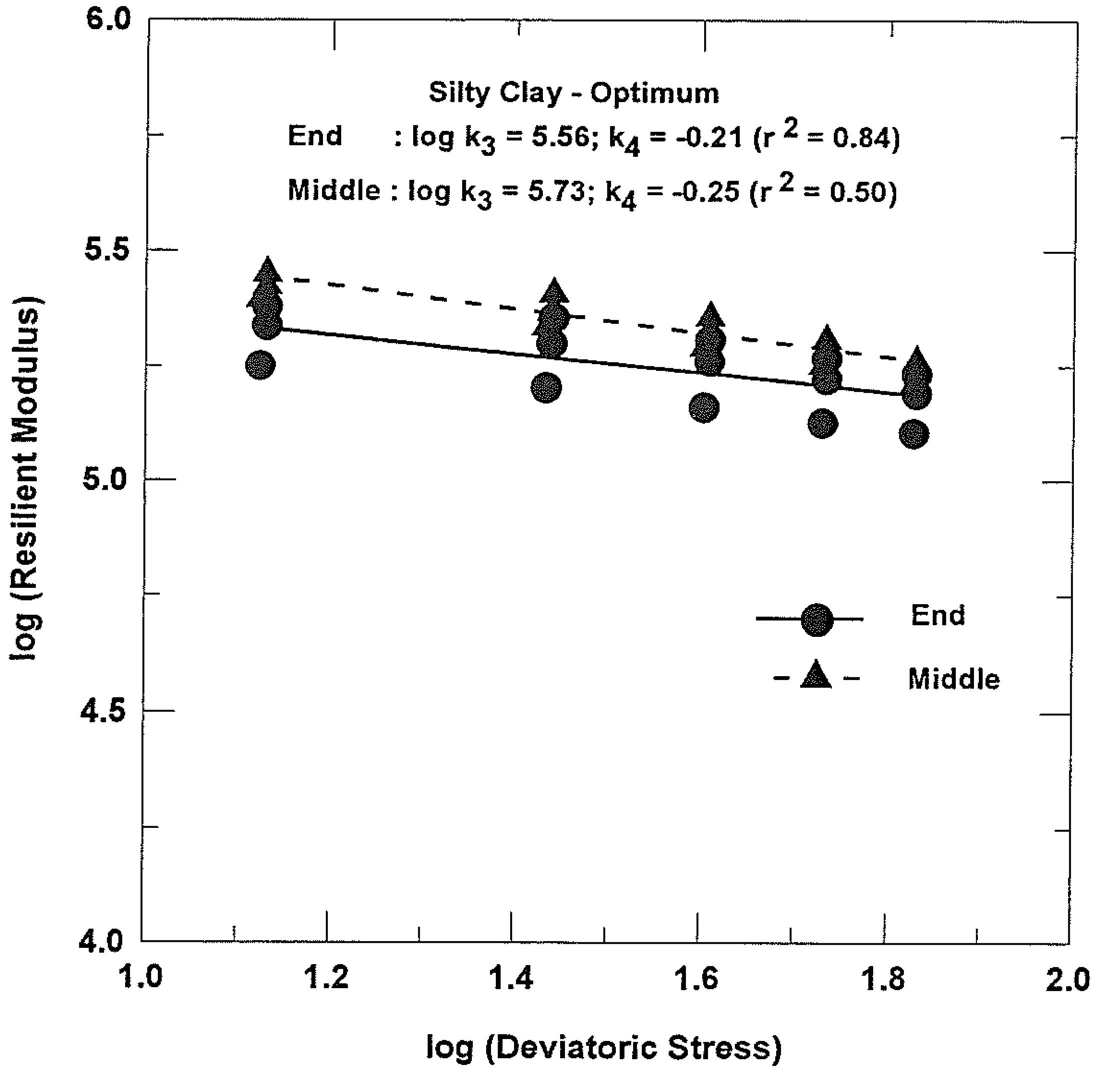


Figure 39: Regression Model for Silty Clays at Optimum

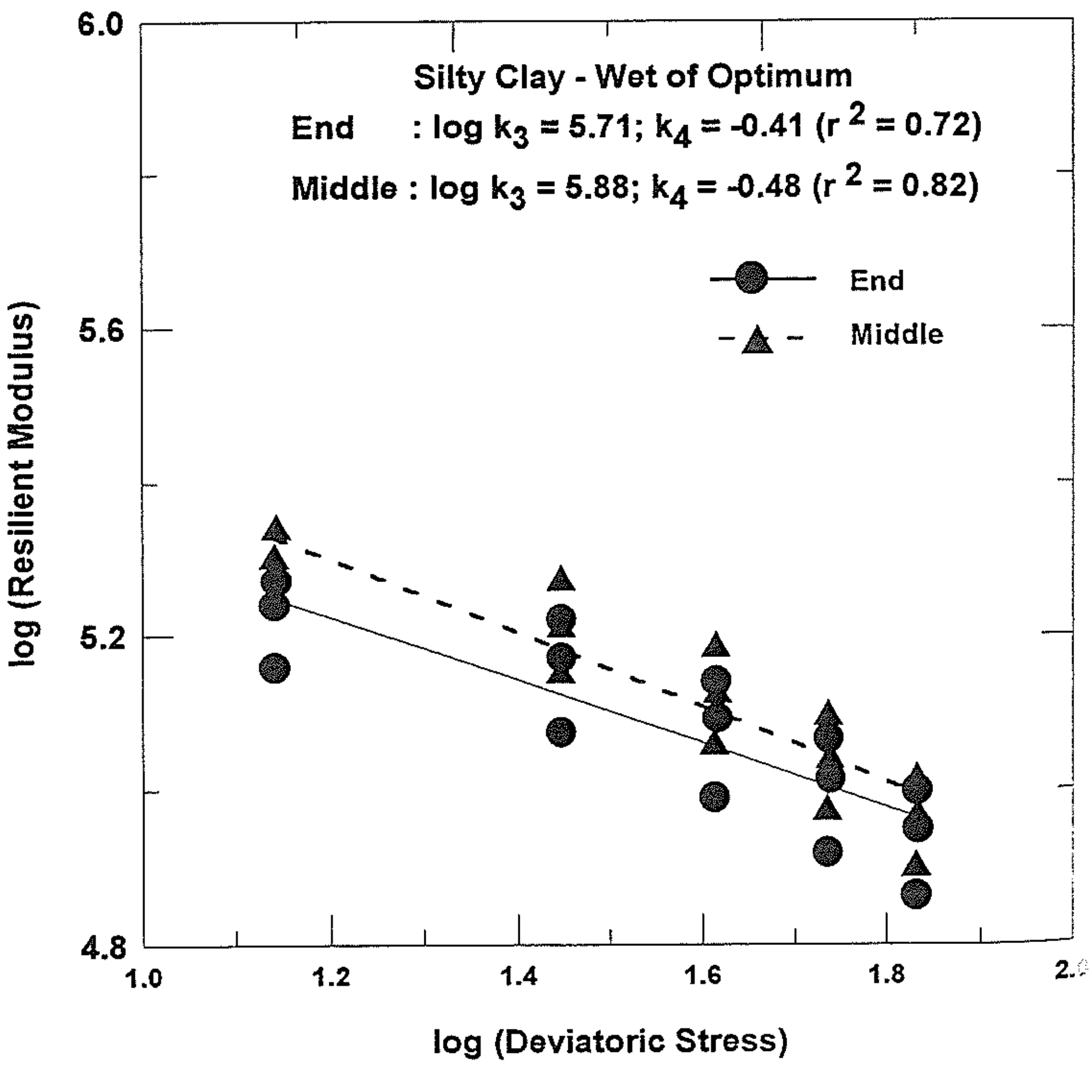


Figure 40: Regression Model for Silty Clays at Wet of Optimum

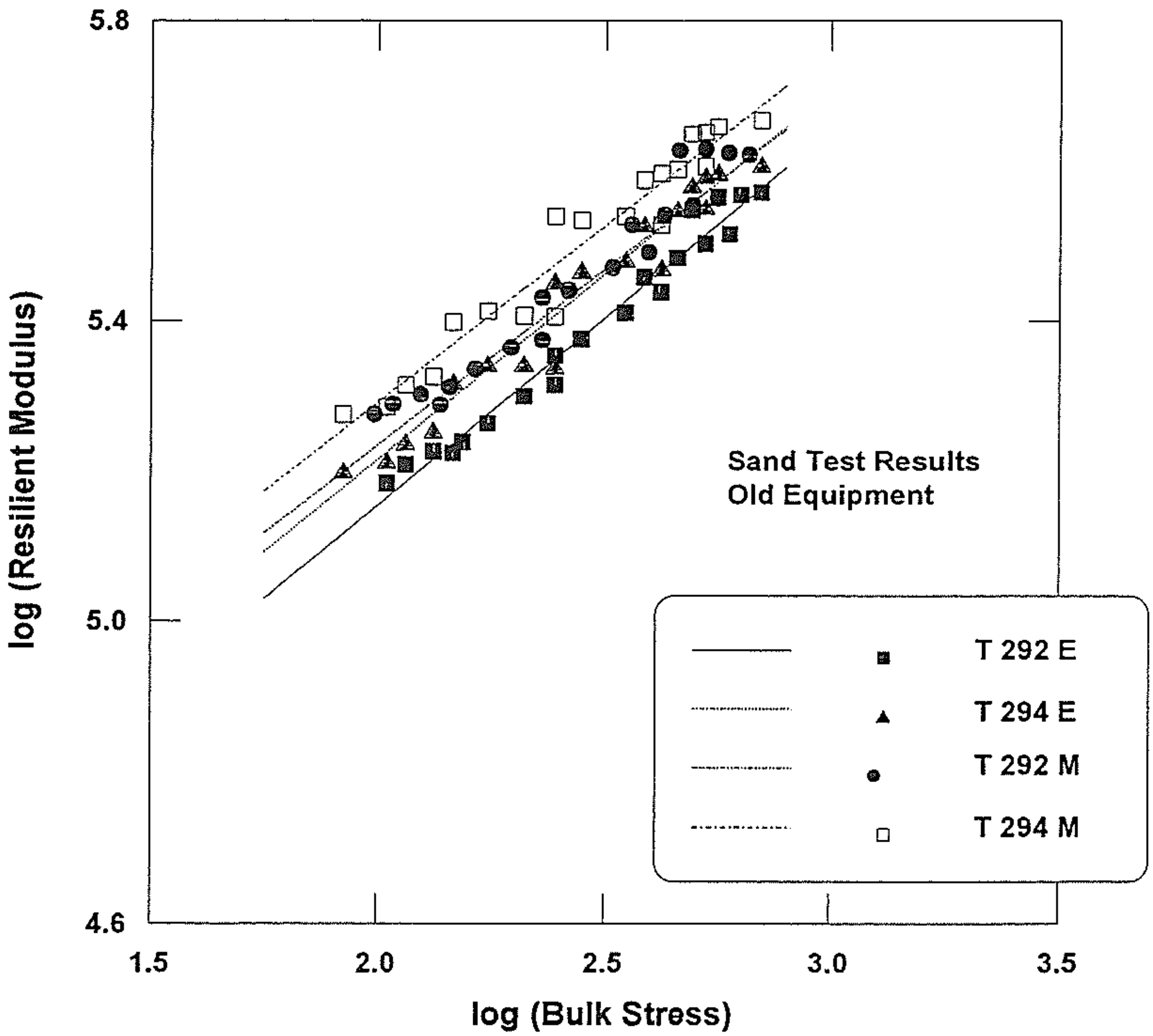


Figure 41: Regression Model for Sands at Optimum (Results Obtained From Old Equipment)

Table 13: Regression Constants for Sand Test Results

Moisture Content (%)	Dry Density (kN/m^3)	End System		Middle System	
		$\log(k_1)$	k_2	$\log(k_1)$	k_2
9.67	16.85	4.44	0.43	4.20	0.50
11.92	17.19	4.38	0.48	4.91	0.63
12.0 (O)	17.57	4.23	0.49	4.35	0.47
12.0 (O)*	17.57	4.15	0.49	4.30	0.47
13.50	17.03	4.39	0.49	4.15	0.52

Note: * - AASHTO T-292 Procedure Used for these Tests;
O - First MTS Equipment is Used in these Tests.

Table 14: Regression Constants for Silty Clay Test Results

Moisture Content (%)	Dry Density (kN/m^3)	End System		Middle System	
		$\log(k_3)$	k_4	$\log(k_3)$	k_4
18.0	15.4	5.83	-0.09	6.67	-0.2[illegible]
20.6	16.23	5.90	-0.21	6.29	-0.2[illegible]
21.2 (T-292)	16.02	5.75	-0.14	6.15	-0.2[illegible]
23.0	15.4	6.25	-0.41	6.63	-0.4[illegible]

Influence of Procedure and Measurement System on Model Constants

It is interesting to note that k_2 and k_4, which are slopes of the lines in the theta and deviatoric stress models, appear to be dependent on the soil type and to a certain extent on the conditioning and testing procedures. In the case of sands, the variation in k_2 obtained from AASHTO procedures (most of them have a range of 0.40 to 0.50 with one of them having a value of 0.63) is not significant. The lower k_2 values are reported for the end measurement system. This is because the end system provided lower moduli values than the middle system. The k_4 of silty clays is significantly different from both AASHTO procedures and this is attributed to the variations in the conditioning and testing in the procedures. Other model constants, k_1 and k_3, which are intercepts in the figures, depend on the testing procedures and the measurement systems. As expected, higher k_1 and k_3 values are obtained for the middle system than the end system due to higher measurements of resilient moduli by the middle system. Figures 42 and 43 graphically present the influence of soil type, the testing procedure and the measurement system on the regression coefficients. The influence of soil characteristics on model constants is presented in another section.

The ranges of regression constants in sands vary from 14,000 to 17,000 kPa for k_1 and 0.4 to 0.7 for k_2. The constants magnitudes of this study are in agreement with these ranges.

Influence of Physical Soil Characteristics on Model Constants

One of the objectives of this investigation is to prepare preliminary correlations between M_r values and physical soil characteristics. The important soil characteristics under consideration are CBR values, density-moisture contents, and grain size properties. For each soil, the M_r values depend on confining and deviatoric stresses. The two ways of correlating these properties with soil characteristics are either including these stresses in the correlations or using the regression constants obtained from bulk stress or deviatoric stress mod latter procedure is used since it simplifies the analysis. It should be mentioned tha two soil types are used in this investigation. This implies that the data is not suffici enough to provide the significant variation expected in all soils or to provide mean complete conclusions. The grain size properties are not used in the analysis since o sets of soils are tested which will not provide wide variation in grain size properti Therefore, the CBR, density and moisture contents are the only soil characteristics considered in this analysis.

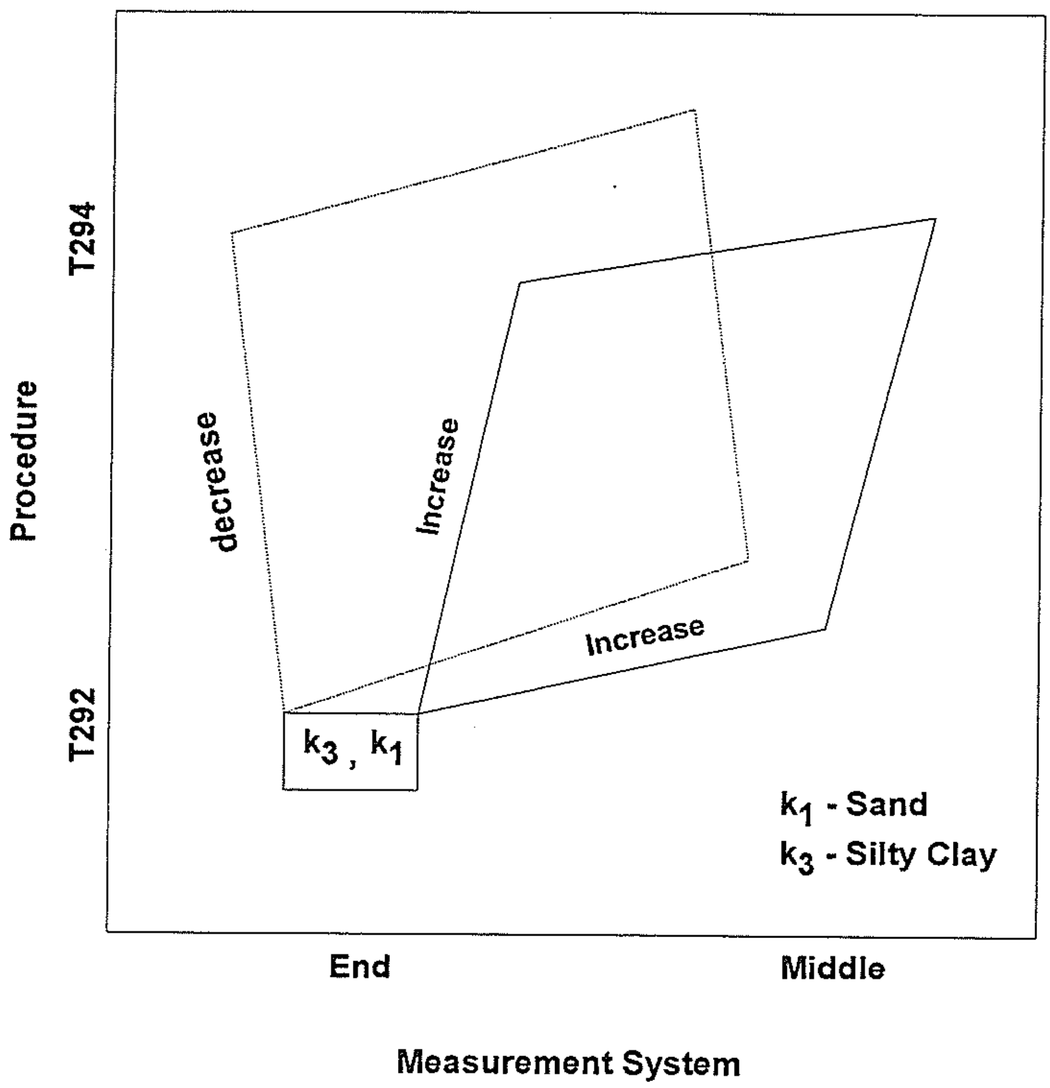


Figure 42: Graphical Representation of Testing Procedure and Measurement Locations Influence on Regression Model Constants (Optimum Results)

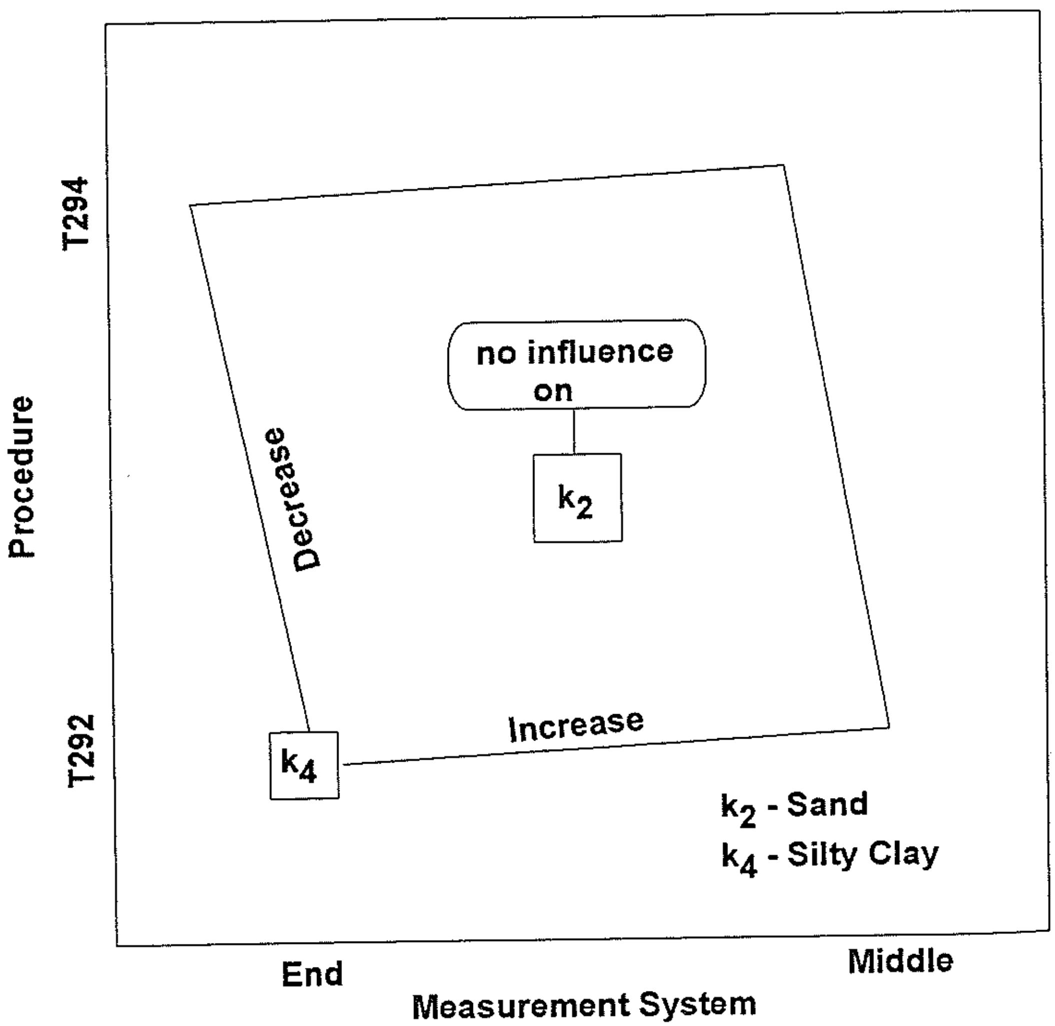


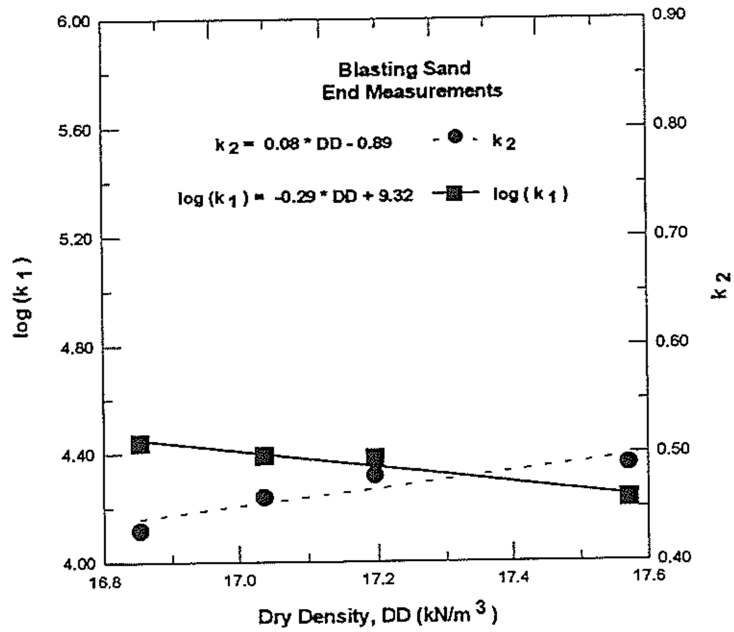
Figure 43: Graphical Representation of Testing Procedure and Measurement Locatio
Influence on Regression Model Constants (Optimum Results)

Blasting Sand

The regression constants, k_1 and k_2 from the bulk stress model are plotted against density and water contents in Figures 44 and 45 and against CBR in Figure 46. The regression equations are provided in the same figures. The coefficient of determination (R-square) values of the best fit lines are very low suggesting that the correlations are poor. This is expected since best fit lines are drawn based on only three sets (dry, optimum and wet) of results. Further testing data from similar type of test results on various types of sands may improve these correlations. Based on the figures, it can also be concluded that the moisture contents and CBR values are more appropriate than the density for correlating with regression constants. The reason for this is that the density mainly depends on moisture content and therefore cannot be considered as an independent variable. Thus, considering density alone as a single variable will not provide an accurate estimation of regression constants.

Silty Clay

The regression constants, k_3 and k_4, from the deviatoric stress model are plotted against density and moisture contents in Figures 47 and 48 and against CBR in Figure 49. The best fit equations are also provided in these figures. Poor correlations are obtained mainly due to limited test data. A larger test database is needed to improve these correlations. In spite of this, the figures can still be used to determine the model constants in the case of silty clays. Once these constants are determined, the resilient properties can be estimated by either assuming stresses expected in the subgrades or determining stresses in the subgrades from an elastic analysis.



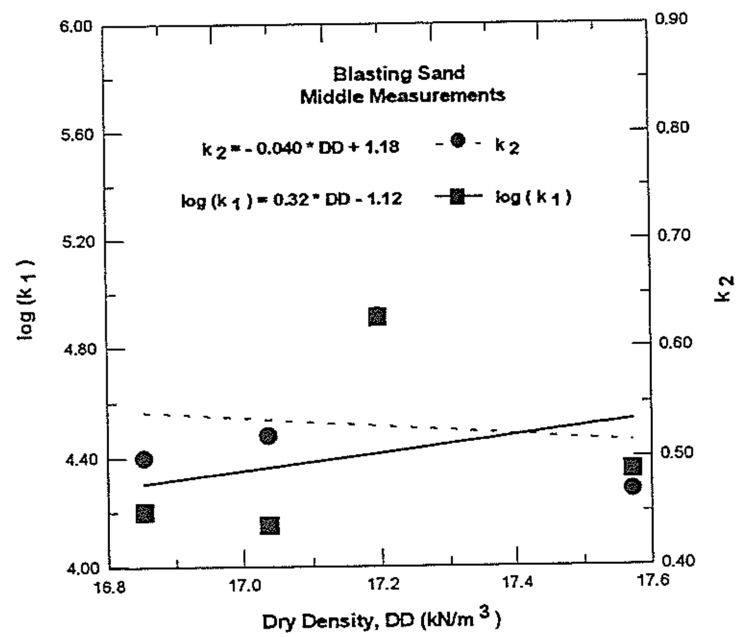
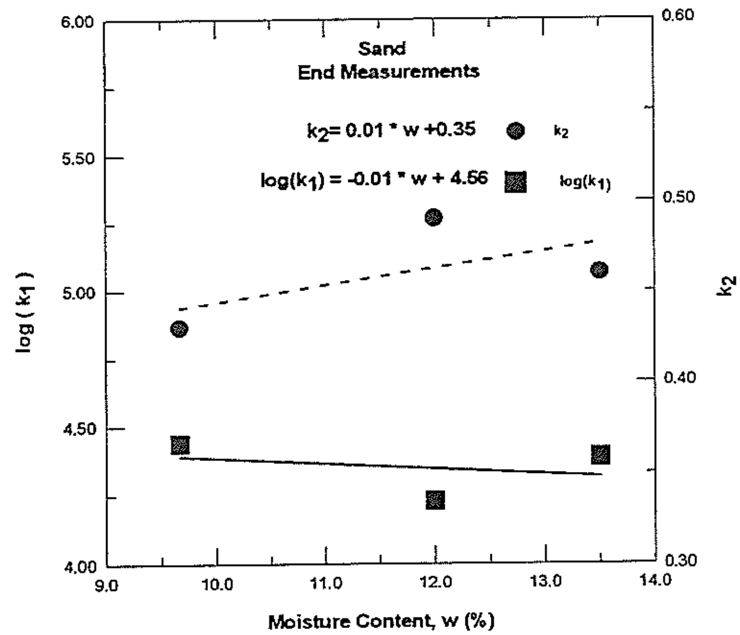


Figure 44: Influence of Dry Density on Sand Regression Constants



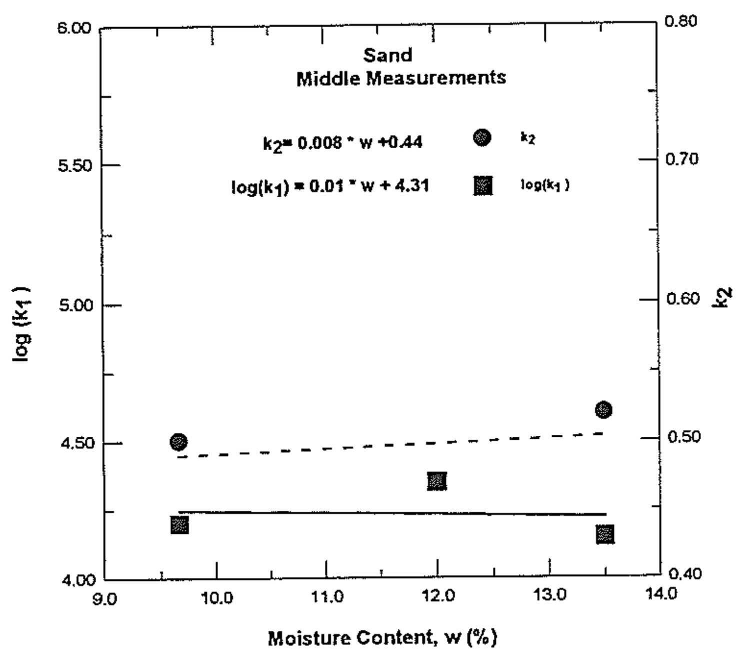


Figure 45: Influence of Moisture Contents on Sand Regression Constants

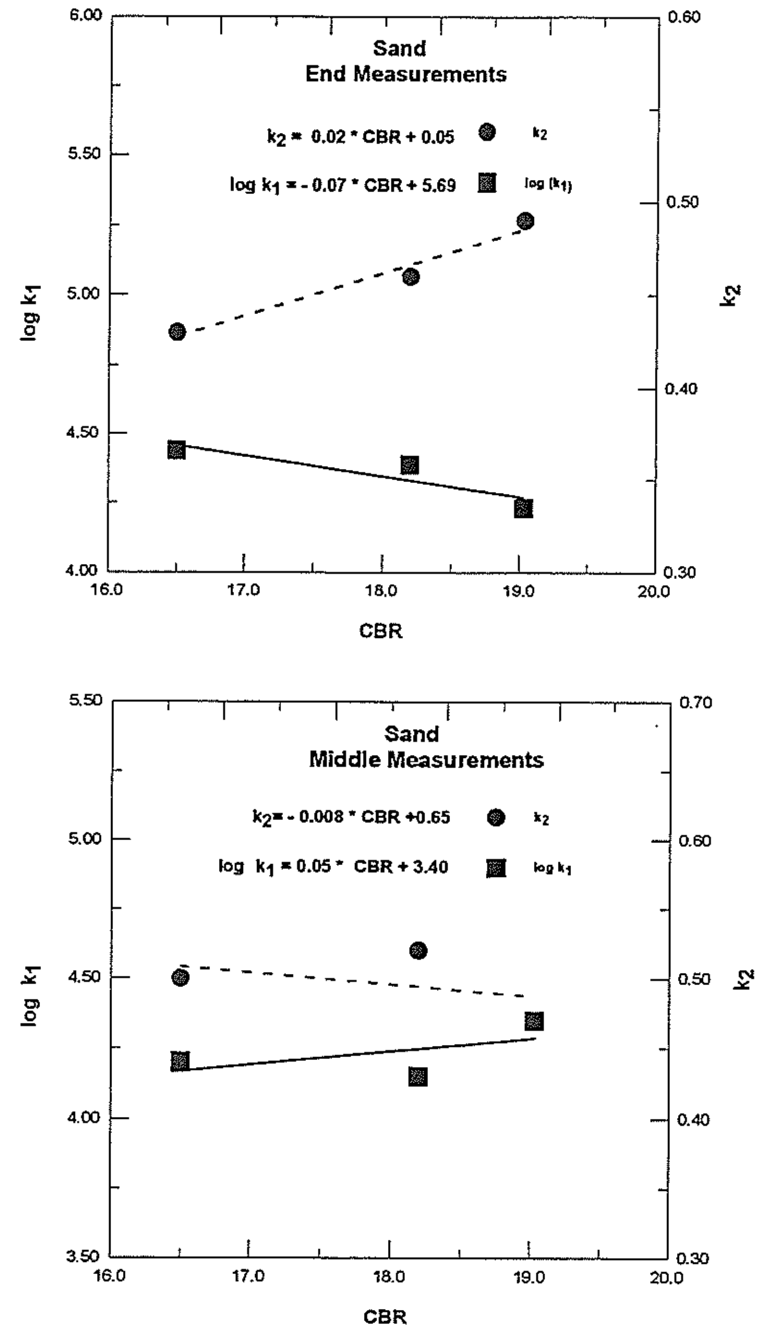
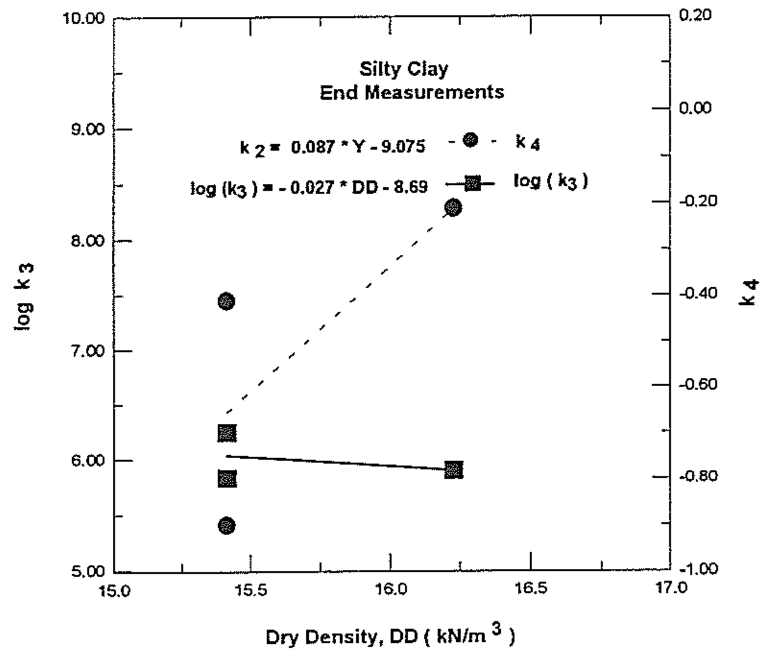


Figure 46: Influence of CBR on Sand Regression Constants



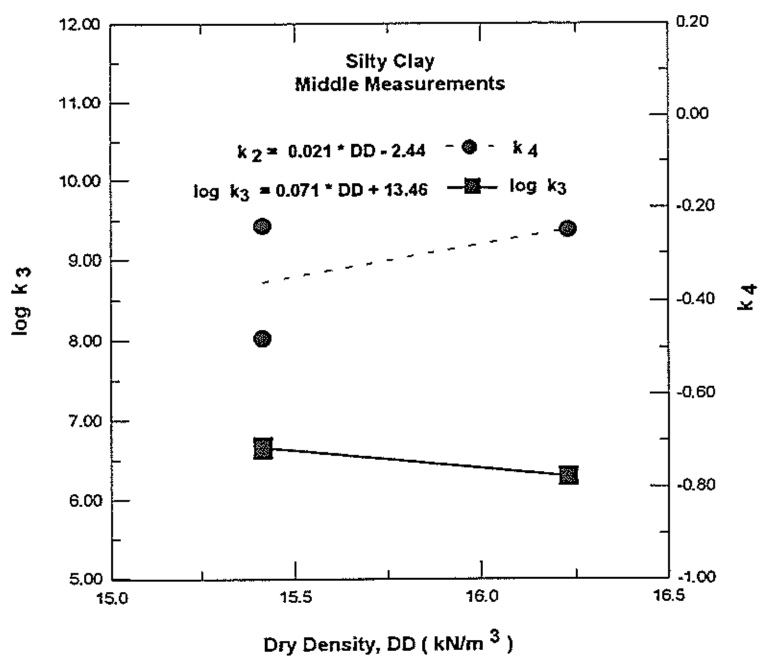
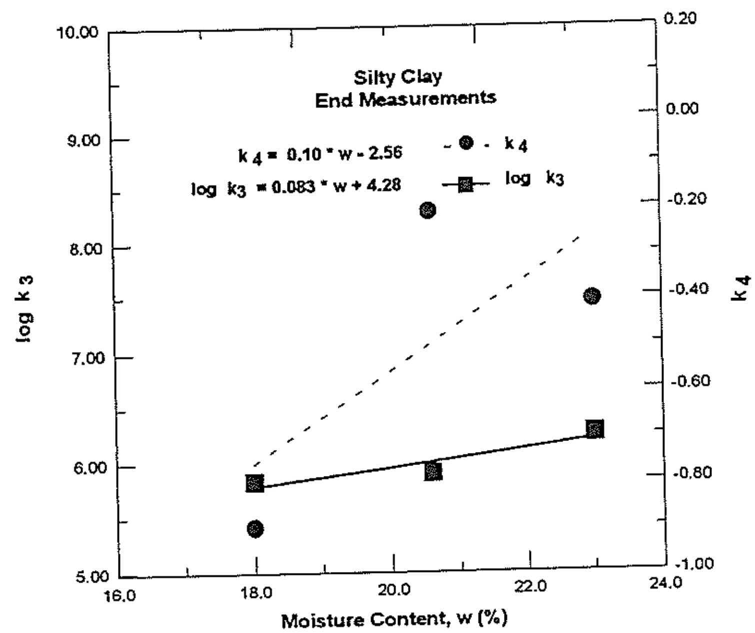


Figure 47: Influence of Dry Density on Silty Clay Regression Constants



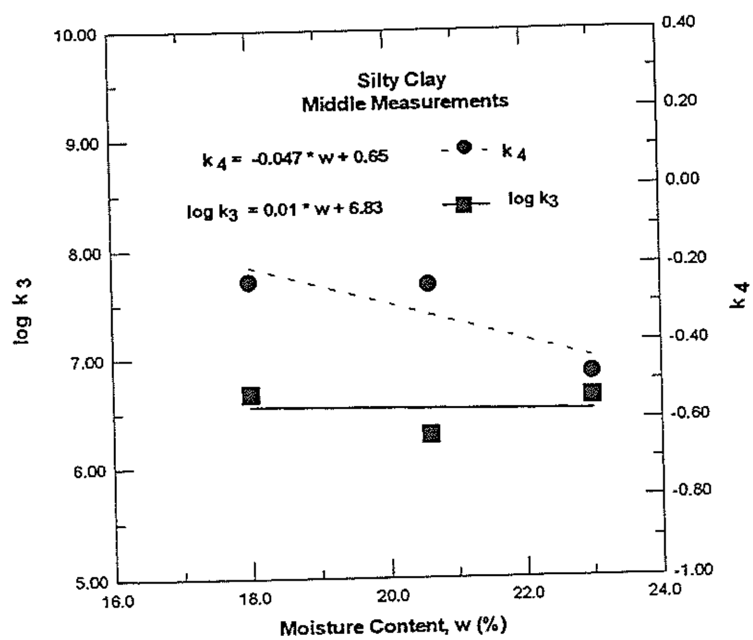


Figure 48: Influence of Moisture Contents on Silty Clay Regression Constants

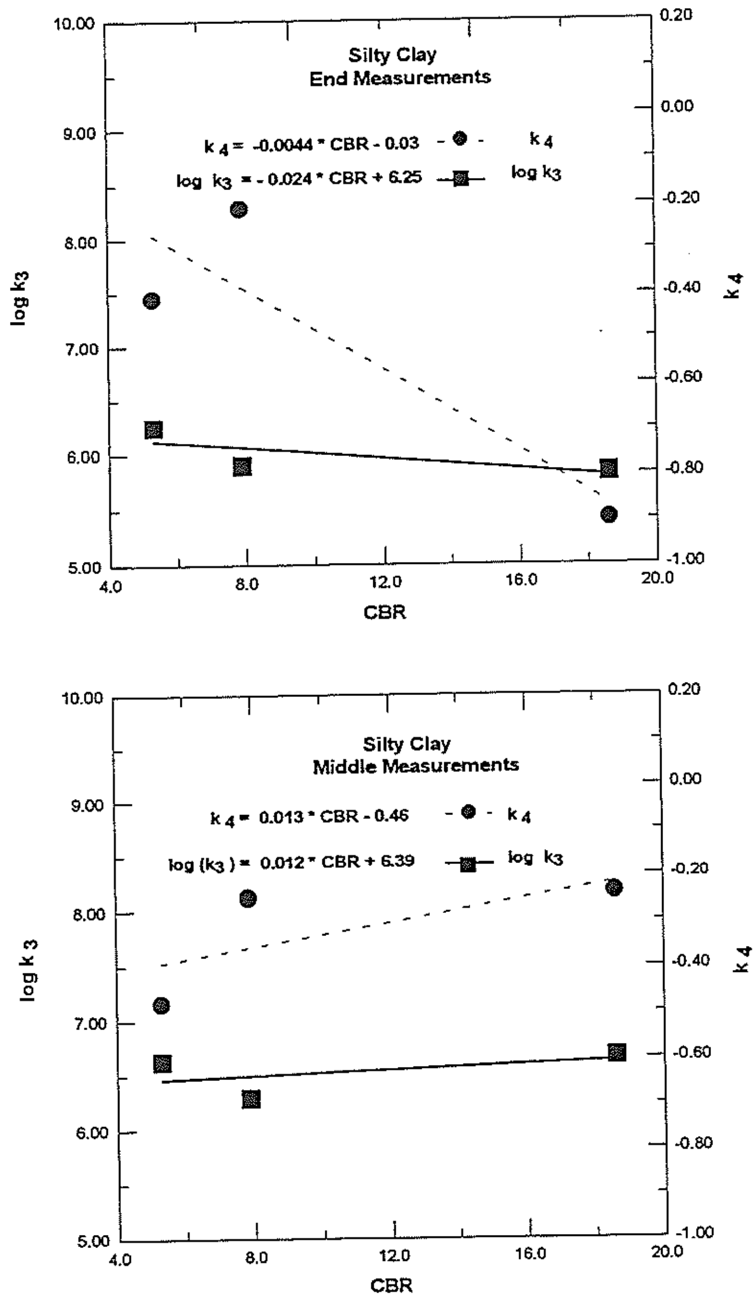


Figure 49: Influence of CBR on Silty Clay Regression Constants

ANALYSIS OF FIELD TEST RESULTS

Opelousas Site

The soil from the site at St. Landry Parish, Opelousas, is classified as silty clay (A- AASHTO classification) with liquid limit and plasticity index values of 22 and 6, respectively. The resilient modulus results from laboratory tests on field core specime presented in Table 15. The AASHTO T-294 procedure for silty clays was used in the testing. The results from middle measurements varied between 140 to 280 MPa based c magnitudes of confining and deviatoric stresses.

The falling weight deflectometer and dynaflect data were analyzed using backcalcu and analytical charts. The modulus 4.1 backcalculation software described earlier was u in the backcalculation analysis of FWD data. This program reads the measured geopho deflection data and then computes the deflections by assuming different moduli for pav layers. The measured and computed data will be compared and the one which provide best comparisons is the final modulus value. Table 16 provides a comparison of these with respect to laboratory determined values. Results indicated that the moduli compu from nondestructive devices are significantly lower than the laboratory moduli determi three different confining pressures. This observation is in agreement with the AASHTO findings (1993 AASHTO Design Guide).

Comparisons of silty clay results obtained from the laboratory (Table 10) and field testing (Table 15) indicate that the laboratory artificially prepared sample results are q close to the natural core sample test results.

Table 15: Resilient Modulus Test Results on Field Core Samples Obtained from St. Landry Parish, Opelousas Site

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
42.0	27.9	206.1	11.0	5.3	246.4	20.3	8.2
42.0	13.8	220.8	7.6	3.4	285.5	24.6	8.6
42.0	28.0	209.0	10.2	4.9	249.2	21.1	8.5
41.9	41.3	192.3	10.2	5.3	220.2	15.0	6.8
41.9	55.2	178.6	8.9	5.0	198.0	13.5	6.8
42.0	69.2	168.4	8.5	5.0	181.9	11.3	6.2
20.9	13.8	191.9	9.5	5.0	259.9	21.5	8.3
21.2	27.9	176.5	10.1	5.7	214.3	15.9	7.4
21.1	41.4	165.0	9.3	5.6	192.4	13.2	6.9
21.1	55.4	156.0	9.0	5.8	175.7	11.2	6.4
21.1	69.4	149.7	8.6	5.7	164.3	10.7	6.5
0.7	13.7	144.4	10.9	7.6	223.4	17.6	7.9
0.7	27.8	133.7	10.5	7.9	182.7	12.3	6.7
0.7	41.2	126.9	9.5	7.4	163.2	11.1	6.8
0.7	55.2	122.7	8.6	7.0	149.7	10.4	7.0
0.7	69.3	120.4	8.2	6.8	140.3	9.8	7.0

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 16: Comparison of NDT (Dynaflect and FWD) and Laboratory Modulus of Soils :
Opelousas Site

Dynaflect Moduli (MPa)	FWD Moduli (MPa)	Laboratory Results (Middle System)		
		σ_3 (kPa)	σ_d (kPa)	M_r (MPa)
86.8	**96.6**	**42**	**42**	**220.2**
86.8	**96.6**	**21**	**42**	**192.4**
86.8	**96.6**	**0**	**42**	**163.2**

Note: 14 psi = 100 kPa; 1 ksi = 7.1 MPa

Port Allen Site

The soil from Port Allen is classified as heavy clay (AASHTO classification A-6-7) liquid limit and plasticity index values are 93 and 66, respectively. The AASHTO T-2 procedure was used for conducting resilient modulus tests. The results are given in Ta 17. Results indicate that the M_r values varied from 3.2 to 4.1 ksi (40 to 60 MPa) for confining stresses of 0 to 6 psi (0 to 42 kPa). The moduli values were very low due to strength of the heavy clay soil. The backcalculated moduli data from the Dynaflect ar compared in Table 18. Results appear to be in agreement with one another. FWD test not conducted on this site and therefore not included in this assessment. Significant re with field studies is still needed to understand the applicability of backcalculation pro of NDT methods in providing realistic resilient properties of soils.

COMPARISONS WITH EXISTING CORRELATIONS

Existing correlations developed by Temple and Carpenter (1990) use soil properties including grain size and Atterberg properties to empirically estimate the resilient modulus. The group index is first determined based on the soil characteristics, including grain size data and Atterberg properties. The group index will be then used to determine the R values. The R values of 240 psi of exudation pressure are used for this purpose. Figure 4.7 of the FHWA/LA-90/218 is then used to estimate the M_r values.

Table 19 shows the comparisons of the results obtained from the correlations and the laboratory investigations. The correlation values are closer to the lower range modulus values of experimental investigations except for heavy clay test results. This type of variation is expected when empirical correlations are used. The empirical correlations need to be updated by including confining and deviatoric stresses as well as moisture content - dry density variations.

Table 17: Resilient Modulus Test Results on Field Core Samples Obtained from Port All

Site

Cf. Pr. (kPa)	D Str. (kPa)	MRE (MPa)			MRM (MPa)		
		Mean	STD	CV	Mean	STD	CV
41.7	14.0	33.9	6.0	17.8	33.0	7.1	21.7
41.7	6.8	39.0	6.5	16.7	38.8	8.3	21.5
41.7	14.1	34.2	6.1	17.8	33.2	7.2	21.8
41.7	21.1	29.7	5.5	18.4	28.6	6.2	21.8
20.5	6.8	35.2	5.7	16.3	35.6	7.5	21.1
20.6	14.1	30.3	5.3	17.6	29.9	6.5	21.7
20.6	21.1	26.5	4.9	18.6	26.1	5.7	22.0
1.2	6.7	28.4	4.6	16.2	32.2	7.2	22.2
1.3	14.0	23.8	4.4	18.5	26.7	6.3	23.7
1.3	21.0	20.5	4.2	20.5	22.9	5.7	24.8

Cf. Pr. : Confining Pressure (in kPa)
D Str. : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation (MPa)
CV : Coefficient of Variation in percent

Table 18: Comparison of NDT (Dynaflect and FWD) and Laboratory Modulus of Soils from Port Allen Site

Dynaflect Moduli (MPa)	FWD Moduli (MPa)	Laboratory Results (Middle)		
		σ_3(kPa)	σ_d(kPa)	M_r(MPa)
16.8	NA	42	21	28.6
16.8	NA	21	21	26.1
16.8	NA	0	21	22.9

Table 19: Comparison of M_r values from Correlations and Laboratory Tests

Type of Soil	Correlations -I Modulus (from R Value) (MPa)			Laboratory Results (ranges)		
	GI	R	M_r	σ_3 (kPa)	σ_d (kPa)	M_r (MPa)
Sand (E)	0	70	>105	0-42	0-140	140-420
Silty Clay (E)	7	36	105	0-42	0-70	126-280
Silty Clay (F)	8	32	91	0-42	0-70	120-280
Heavy Clay (F)	16	11	49	0-42	0-70	20-38

Note: E - Experimental; F - Field
14 psi = 100 kPa; 1 ksi = 7.1 MPa

CONCLUSIONS AND RECOMMENDATIONS

OBJECTIVES ACCOMPLISHED

The resilient modulus triaxial testing program was successfully conducted on laborat prepared and field core specimens. Two types of soils, a granular sand and a silty clay, were used in the laboratory investigations and the field cores consisted of silty clay and heavy clay soils. The successful accomplishment of the testing program provided a laboratory methodology for determining the resilient properties of local subgrade soils. research study also provided some insight into the influence of AASHTO testing proce and LVDTs measurement locations on the resilient modulus of soils. Preliminary correlations are also developed between resilient modulus properties and CBR, dry den and moisture contents.

CONCLUSIONS

The testing program was developed and conducted based on statistical concepts, w reduced the operational and equipment related errors in this study. Quality control procedures indicated that the AASHTO specimen preparation methods provided reaso homogenous specimens. The coefficient of variation of clays, an indicator for the vari in test results, ranged from 1 to a maximum of 30 with most of them around 10 perc The coefficients of variation of sands was smaller (less than 10) when compared with of clays. These smaller coefficients in both soils indicate that good to excellent repeat was achieved in the testing program.

The following are the major conclusions drawn from the test results of this study.

1. The sand results show a significant range in the magnitude of resilient modulus values based on the confining stresses and moisture content levels. The range at optimum moisture content varies from 100 to 500 MPa (20 to 50 ksi) for confining pressures of 21 to 140 kPa. The range of resilient modulus for silty clays is about 100 to 400 MPa (20 to 40 ksi).

2. The conditioning performed to reduce the plastic strain developments has not resulted in the complete elimination of plastic strains in both soils. Significant plastic strains are observed even at the end of the conditioning cycles. However, these plastic strain developments have not influenced the resilient strain measurements in the testing phase since M_r values in the testing phase are not significantly changed with the number of cycles. This observation implies that the number of cycles can be reduced from the AASHTO prescribed one hundred cycles.

3. The M_r of both soils increases with an increase in the confining pressure. This increase in sands is attributed to an increase in stiffness as well as the reduction in dilatancy properties. The increase of resilient properties in silty clays is attributed to a slight increase in strength as well as the assumed reduction in the pore pressure development at higher confining pressures.

4. The deviatoric stress influence on sands is significant only at low confining pressures. At higher confining pressures, the influence of deviatoric stress is not noticed. For silty clays, an increase in the deviatoric stress results in the development of higher pore pressures which reduces the overall strength of the specimen. Therefore, the resilient modulus decreases with an increase in deviatoric stress in silty clays.

5. The moisture content, as expected, appears to have more influence on silty clay M_r results than on the sand results. This is mainly attributed to the pore pressure development in silty clays.

6. The T-294 procedure resulted in higher moduli than the T-292 procedure. This is du smaller bulk stress fluctuations in conditioning and testing stress levels in the T-294 procedure which is assumed to provide less disturbance to the soil.

7. The measurement coefficients (MC) which are defined as the ratios of end resilient measurements/moduli to middle resilient measurements/moduli were introduced. The coefficients which are greater than one, indicate that the middle system provides hig moduli than the end system, possibly due to fewer system compliance errors and end friction effects.

8. The MC values of sands varied from 1.1 to 1.2 and they appear to be less influenced confining and deviatoric stresses. The MC values of silty clay vary from 1.1 to 1.7. noted in the silty clay test results is that the confining stress appears to have a major influence on the MC values. Larger MC values are determined for silty clays at unconfined conditions.

9. The Louisiana developed correlations, though not accounting for confining and devia stresses, have provided M_r values on field core samples which are quite close to laboratory investigations at low confining and deviatoric stresses. However, there is a need to incorporate confining and deviatoric stresses and moisture content and dry density properties into the existing empirical correlations to make these correlations m practical.

RECOMMENDATIONS

Future research in this area should attempt to cover a wider range of the different soil types in Louisiana subgrades and provide ranges of resilient modulus values for those soils at various moisture contents and dry density levels. The AASHTO recommended T-294 procedure needs to be used in the testing phase. Any such study should address the influences of soil grain size and shapes, moisture-density relations, specimen preparation procedures, and testing stresses on the resilient modulus values.

REFERENCES

__________, "Guide for Design of Pavement Structures," *American Association of St Highway and Transportation Officials*, 1986.

American Association of State Highway and Transportation Officials, Interim Specifications Transportation materials and Methods of Sampling and Testing, Part II Interim Test Metho 1992.

Andrews, R. C., Mundy, M. J. and Stacy, W. S. "Properties of Granular Materials Mechanistic Pavement Design," Asphalt Conference, Brisbane, March.

Badu-Tweneboah, K., Manzione, C.W., Ruth, B.E., and Miley, W.G. "Prediction of Flex Pavement Layer Moduli from Dynaflect and FWD Deflections," *Nondestructive Testin Pavements and Backcalculation of Moduli, ASTM STP 1026*, A.J. Bush III and G. Y. Ba Eds., American Society for Testing and Materials, Philadelphia, 1989, pp. 245-277

Baladi, G. "Resilient Modulus," Proceedings, *Workshop on Resilient Modulus Tes State-of-the-Practices*, Oregon State University, Corvallis, Oregon, March 1989.

Barksdale, R.D., et al. "Laboratory Determination of Resilient Modulus For Flexible Paver Design," *NCHRP Report 1-28*, Georgia Institute of Technology, December 1990.

Baus, R. L. and Ray, R. P. "Investigation of Subgrade Resilient Modulus in Flexible Pave Design - Phase I," *Research Report No. F92-112*, Geotechnical Research Studies, Depar of Civil Engineering, University of South Carolina, Columbia, South Carolina, Dece 1992.

Bishop, A.W. and Henkel, D.J. "The Triaxial Test," *Edward Arnold*, London, 1962.

Elliot, R. P., Thornton, S. I., Foo, K. Y., Siew, K. W. and Woodbridge, R. "Res Properties of Arkansas Subgrades," *Report No. FHWA/AR-89/004*, Arkansas Highwa Transportation Research Center, University of Arkansas, Fayetteville, November 1988.

Ho, R. "Repeated Load Tests on Untreated Soils: A Florida Experience" Procee *Workshop on Resilient Modulus Testing State-of-the-Practices*, Oregon State Unive Corvallis, Oregon, March 1989.

Hossain, M. and Scofield, L. A. "Correlation Between Backcalculated and Laboratory-Determined Asphalt Concrete Moduli," *Transportation Research Record 1377*, National Research Counsil, Washington, D.C., 1992.

Houston, W. N., Houston, S. L. and Anderson, T. W. "Stress State Considerations for Resilient Modulus Testing of Pavement Subgrade," *Transportation Research Board*, 71st Annual Meeting, Jan 12-16, Washington, D.C., 1992.

Irwin, L. H. "Report of the Discussion Group on Practical Limitations and What Can be Done to Overcome Them," *Transportation Research Record 1377*, Transportation Research Board, National Research Counsil, Washington, D.C., 1992.

Jackson, N. "Thoughts on AASHTO T-274-82 Resilient Modulus of Subgrade Soils," Proceedings, *Workshop on Resilient Modulus Testing State-of-the-Practices*, Oregon State University, Corvallis, Oregon, March 1989.

Jaky, J. "The Coefficient of Earth Pressure at Rest," *Journal of the Society of Hungarian Architects and Engineers*, pp. 355-358, 1944.

Kim, Dong-Soo and Stokoe, K. H. "Characterization of Resilient Modulus of Compacted Subgrade Soils Using Resonant Column and Torsional Shear Tests," *Transportation Research Board*, 71st Annual Meeting, January 12-16, Washington, D.C., 1992.

Maestas, J. M. and Mamlouk, M. S. "Comparison of Pavement Deflection Analysis Methods Using Overlay Design," *Transportation Research Record 1377*, Transportation Research Board, National Research Counsil, Washington, D.C., 1992.

Mohammad, L. N., Puppala, A. J. and Alavilli, P. "Influence of Moisture Content on Resilient Modulus of Compacted Soils" National Academy of Science, Transportation Research Board, 1994 (in press)

Mohammad, L. N., Puppala, A. J. and Alavilli, P. "Resilient Modulus of Soils: Influence of Testing Procedure and LVDT's Location" National Academy of Science, Transportation Research Board, 1994 (in press)

Mohammad, L.N., Puppala, A.J., and Alavilli, P. "Effect of Strain Measurements on Resilient Modulus of Sands," Dynamic Geotechnical Testing: Second Volume, ASTM STP 1213, Ronald J. Ebelhar, Vincent P. Drnevich, and Bruce L. Kutter, Eds., American Society for Testing and Materials, Philadelphia, 1994.

Mohammad, L. N., Alavilli, P. and Puppala, A. J. "Data Acquisition System for Determini the Resilient Modulus of Soils," Advances in Site Characterization: Data Acquisitio Management and Interpretation, Geotechnical Special Publication No. 37., ASCE Conventio Dallas, 1993, pp. 27-41.

Parker, Jr. F. "Estimation of Paving Materials Design Moduli from Falling Weig Deflectometer Measurements," Transportation Research Record 1293, National Resear Counsil, Washington, D.C., 1991.

Pezo, R. F., Claros, G., Hudson, W. R., and Stokoe, II, K. H. "Development of a Relia Resilient Modulus Test for Subgrade and Non-Granular Subbase Materials for Use in Rout Pavement Design," Research Report 1177-4f, Center for Transportation Research, Univers of Texas, Austin, Texas, January, 1992.

Pezo, R. F., Kim, D. S., Stokoe II, K. H., and Hudson, W. R. "A Reliable Resilient Modu Testing System," Transportation Research Record No. 1307, Transportation Research Boa National Research Council, Washington, D.C., 1991.

Pezo, R. F., Claros, G., and Hudson, W. R. "An Efficient Resilient Modulus Test Procedure for Subgrade and Non-Granular Subbase Materials," Transportation Research Boa 71st Annual Meeting, January 12-16, Washington, D.C., 1992.

Pumphrey, Jr., N. D., and Lentz, R. W. "Deformation Analyses of Florida Highway Subgra Sand Subjected to Repeated Load Triaxial Tests," Transportation Research Record 10 Geotechnical Engineering, Transportation Research Board, National Research Coun Washington, D.C., 1986.

Roberts, F. L., Wedgeworth, J. B. and Pumphrey, Jr., N. D. "Analysis and Evaluation Methods for Backcalculation of M_r Values," Three Volumes, Department of Civil Engineeri Louisiana Tech University, Ruston, 1992.

Seed, H. B., Mitry, F. G., Monismith, C. L., and Chan, C. K. "Prediction of Flex Pavement Deflections from Laboratory Repeated Load Tests," NCHRP 35, HRB, 1967.

Seim, D. "A Comprehensive Study on the Resilient Modulus of Subgrade Soils," Proceedin Workshop on Resilient Modulus Testing State-of-the-Practices, Oregon State Univers Corvallis, Oregon, March 1989.

Shook, J. F., Finn, F. N., Witczak, M. W., and Monismith, C. L. "Thickness Design Asphalt Pavements - The Asphalt Institute Method," Proceedings, 5th International Confere on the Structural Design of Asphalt Pavements,Vol. I., pp. 17-44, 1982.

Smith, R. E. "Report of the Discussion Group on Current and Future Applications of Nondestructive Deflection Measurements," Transportation Research Record 1377, Transportation Research Board, National Research Counsil, Washington, D.C., 1992.

Sneddon, R. V. "Resilient Modulus Testing of 14 Nebraska Soils," Project No. RES1(0099)P404, University of Nebraska-Lincoln, November, 1988.

Strategic Highway Research Program "Resilient Modulus of Unbound Granular Base, Subbase Materials and Subgrade Soils, SHRP Protocol P-46, UG07, SS07, 1989.

Temple, W. H., and Shah, S. C. "Louisiana Experimental Base Project," Research Project No. 74-1G, FHWA/LA-87/192, Louisiana Transportation Research Center, Baton Rouge, 1987, 125 pages.

Thompson, M. "Factors Affecting the Resilient Moduli of Soils and Granular Materials," Proceedings, Workshop on Resilient Modulus Testing State-of-the-Practices, Oregon State University, Corvallis, Oregon, March 1989.

Thompson, M. "Resilient Modulus of Subgrade Soils," Proceedings, Workshop on Resilient Modulus Testing State-of-the-Practices, Oregon State University, Corvallis, Oregon, March 1989.

Vinson, T. "Fundamentals of Resilient Modulus Testing," Proceedings, Workshop on Resilient Modulus Testing State-of-the-Practices, Oregon State University, Corvallis, Oregon, March 1989.

Wilson, B. E., et al. "Multiaxial Testing of Subgrade," Proceedings, Transportation Research Board 69th Annual Meeting, January 1990, Washington, D.C.

Woolstrum, G. "Dynamic Testing of Nebraska Soils and Aggregates,", Proceedings, Transportation Research Board 69th Annual Meeting, January 1990, Washington, D.C.

Zhou, H., Huddleston, J. and Lundy, J. "Implementation of Backcalculation in Pavement Evaluation and Overlay Design in Oregon," Transportation Research Record 1377, National Research Council, Washington, D.C., 1992