### Title and Subtitle
Investigation of the Use of Resilient Modulus for Louisiana Soils in the Design of Pavement

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### 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.

### Key Words
Dynamic Materials Characterization, Triaxial Testing, Resilient Modulus, California Bearing Ratio, Sand, Clay

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

FINAL REPORT

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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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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 of AASHTO pavement design procedure. This testing capability also provides the necessary support for the characterization of subgrade soils used in the LTRC Accelerated Loading Plate experiments. In addition, this capability provides LADOTD with an investigative tool that can be used in troubleshooting of construction projects. Based on the present experimental status, it is recommended that the AASHTO T-294 procedure be adapted for conducting resilient modulus test on soils.
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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 agencies started implementing laboratory testing procedures for determining the resilient modulus of 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 Publication). 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 at 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 resilient modulus testing which can be implemented to design flexible pavements mechanistically. The results from laboratory methodologies will be used to validate the existing properties - M* relationships used in Louisiana. The outcome of this research study will include the development of a laboratory research tool that will provide LTRC with the ability to do diagnostic work for LADOTD and also to use it as a part of the materials characterization process in the newly acquired Accelerated Loading Facility at LTRC. Other objectives of the 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 modulus of soils. Field nondestructive testing procedures will be evaluated by comparing them to laboratory results conducted on core samples.

SCOPE

Two soil types, a blasting sand and a silty clay, will be investigated under conditions representing a simulation of the physical and stress states of soils beneath flexible pavements subjected to moving wheel loads. The conclusions based on the test results are valid for these 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,

\[ M_r = \frac{\sigma_d}{\varepsilon_d} \]  \hspace{1cm} (1)

where \( \sigma_d \) is the deviatoric stress; and \( \varepsilon_d \) 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 load applied.
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 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.
Previous Investigations

Several laboratory methods such as triaxial, simple shear, torsional shear, using the

increased.

still measured (greater than 10 percent), the number of cycles in the testing phase needs to be

increased or decreased based on the plastic strain criterion. If significant plastic strains are

then averaged to calculate the resilient modulus. The number of cycles for testing can be

of T-294) or deviatoric loading. The resiliene strains are measured for the last five cycles and

The resilient phase requires 50 cycles (in the case of T-292) and 100 cycles (in the case

both granular and cohesive soils.

deviatoric loading at a certain confining pressure. The number of cycles are the same for
cycles. Both T-292 and T-294 procedures are identical with 1000 cycles or a certain

not excessive enough to reduce plastic strains and it should be increased to 1000 to 2000

number of cycles prescribed by an earlier procedure, T-294 (200 cycles of conditioning) was

importantly, reduces the plastic strain development. Houston et al. (1997) reported that the

minimizes disturbance effects caused by sampling or laboratory preparation, and, more

AASHTO procedure. The conditioning elminates the imptectic connecte between end plates,

The number of cycles of conditioning and testing phases are different for each

descriptions on this procedure can be found in Houston et al. (1997).
on any plane can be determined (Barksdale et al. 1990). In addition, the triaxial equipment provides repeatable and reliable results and also allows the measurements to be taken in any radial directions.

Other equipment such as resonant column and torsional shear test devices are also used in determining the $M_r$ values. These methods appear to provide reasonable results at strains (less than 0.1 %) need to be measured (Barksdale, 1990). The major problem associated with these tests, including the triaxial test is their incapability of simulating principal axes rotation. These axes rotation is graphically illustrated in Figure 2. When the wheel load is exactly on top of a certain point in the pavement, the major principal stress acts in a vertical direction at that point. When the load is away from the point, the principal stress acts at a certain angle. Therefore, the movement of a vehicle wheel over a point rotates the principal axes at the point. The hollow cylinder test can be used to simulate movement of axes rotation. However, this test is tedious, complicated and requires instrumentation (Barksdale et al. 1990). These are probably the reasons that this test is not in use. Although the triaxial test has fixed principal axes, it still offers a procedure to represent certain extreme stress conditions expected in the field. Further details on the equipment can be obtained from the NCHRP report documented by Barksdale et al.

Other aspects in the laboratory testing involve the investigation of the influence of types and their characteristics on resilient behavior. The clay content, Atterberg limits, density, water content, and degree of saturation have been investigated (Baladi, 1989; Thompson, 1989; Vinson, 1989). Table 1 also presents a summary of major findings of various investigations on these aspects. Most of the studies concluded that the granular materials show an increase in the resilient modulus with an increase in the deviatoric and confining stresses and cohesive soils display a decrease in $M_r$ with an increase in the deviatoric stress (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 value.
Figure 2: Principal Axes Rotation During Traffic Loading
to zero prior to the start of the test. However, the external measurement system is greatly influenced by system compliance errors and end friction effects which induce significant errors into the measurements. Previous investigations reported that several different types of 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 these investigations was to adapt an internal measurement system placed on the middle one third height of the specimen in the triaxial cell. This will reduce the system compliance error and end friction effects. Recognizing this, the T-292 procedure recommended the use of two internal LVDT measurement systems, one placed at the ends of specimens and the other at the middle one third points of the specimen. These systems are hereafter known as end system and middle system, respectively. Although external measurement LVDT system 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. The 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 subjected to stresses of lower magnitudes which generate significantly smaller resilient strains. Therefore, it is necessary to have a measurement system which can capture small displacements or 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 measuring corresponding deflections. The moduli of the surface layer, base, and subgrades are then evaluated by conducting an elastic analysis on the pavement system. The Dynaflect and Falling Weight Deflectometer are two currently and frequently used nondestructive devices. A description of them is given below.
Table 1. A Summary of Resilient Modulus Tests Conducted on Soils

<table>
<thead>
<tr>
<th>Reference</th>
<th>Types of Equipment</th>
<th>Soils Tested</th>
<th>Parameters Studied</th>
<th>Procedure Used</th>
<th>Major Conclusions</th>
</tr>
</thead>
</table>
| 1) Seed et al., (1967) | Repeated load triaxial tests and plate load tests | Coarse grained and fine grained soils | $\sigma_0$, $\sigma_o$, 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 | $\sigma_p$, $\sigma_0$, 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 | $\sigma_p$, $\sigma_0$, 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., (1999) | Triaxial system with MTS accessories | Florida soils | $\sigma_p$, $\sigma_0$, 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. (1999) | 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. |
<table>
<thead>
<tr>
<th>Reference</th>
<th>Types of Equipment</th>
<th>Soils Tested</th>
<th>Parameters Studied</th>
<th>Procedure Used</th>
<th>Major Conclusions</th>
</tr>
</thead>
<tbody>
<tr>
<td>7) Selin, D. K. (1989)</td>
<td>Triaxial testing equipment</td>
<td>Sand and miller clay</td>
<td>$\sigma_3$, $\sigma_4$</td>
<td>T-274</td>
<td>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.</td>
</tr>
<tr>
<td>8) Woolstrum, G (1990)</td>
<td>Triaxial tests</td>
<td>Nebraska soils and aggregates</td>
<td>$\sigma_3$, Group Index, water content</td>
<td>--</td>
<td>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. Dynaflact results show no variation with $M_r$ values.</td>
</tr>
<tr>
<td>9) Thompson, M. R. and Smith, K. L. (1990)</td>
<td>Repeated load triaxial tests</td>
<td>Granular materials</td>
<td>Gradation, number of load repetitions, $\sigma_3$ and $\sigma_4$</td>
<td>--</td>
<td>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.</td>
</tr>
<tr>
<td>10) Andrews, R. C., Mundy, M.J. and Stacy, W.S. (1982)</td>
<td>Repeated triaxial testing</td>
<td>Granular materials</td>
<td>$\sigma_3$, $\sigma_4$, water content and cement content</td>
<td>--</td>
<td>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.</td>
</tr>
<tr>
<td>11) Pezo, R.F., Claros, G. and Hudson, W.R. (1992)</td>
<td>MTS testing with triaxial cell</td>
<td>Texas subgrade soils</td>
<td>$\sigma_3$, $\sigma_4$, compaction methods, effect of grouting</td>
<td>T-274</td>
<td>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.</td>
</tr>
<tr>
<td>12) Kim, D. S. and Stokoe, II, K. H. (1992)</td>
<td>Resonant column tests (RC) and torsional shear tests (TS)</td>
<td>Compacted subgrade soils</td>
<td>Testing methods, $\sigma_3$, $\sigma_4$, strain amplitudes and loading frequency</td>
<td>--</td>
<td>1. The elastic threshold strain below which $M_r$ values are independent of strain amplitudes increases with increase in $P_t$. 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.</td>
</tr>
<tr>
<td>13) Houston, W. N., Houston, S. L. and Anderson, T. (1992)</td>
<td>Triaxial system</td>
<td>Arizona subgrade soils</td>
<td>$\sigma_3$ and $\sigma_4$</td>
<td>T-274</td>
<td>1. Non-linear response of subgrade soils is reduced with the load repetitions. 2. Fewer repetitions from T-274 were found to be adequate to represent the soil.</td>
</tr>
<tr>
<td>Reference</td>
<td>Types of Equipment</td>
<td>Soils Tested</td>
<td>Parameters Studied</td>
<td>Procedure Used</td>
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<tr>
<td>14) Reed, L et al., (1992)</td>
<td>Triaxial system</td>
<td>Four sands with different gradations</td>
<td>Gradation, number of repetitions, ( \sigma_s ) and ( \sigma_d )</td>
<td>T-274</td>
<td>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 ) value decreases with an increase in pore pressure ratios.</td>
</tr>
<tr>
<td>15) George, K. P., (1992)</td>
<td>Gyratory testing</td>
<td>Different soils</td>
<td>Soil gradation, stress state, moisture content, testing methods</td>
<td>T-274</td>
<td>1. Comparison between ( M ) values from triaxial testing and gyratory testing methods showed reasonable agreement. 2. ( M ) 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.</td>
</tr>
<tr>
<td>16) Mohammad et al., (1993, 1994)</td>
<td>MTS testing with triaxial accessories</td>
<td>Silty clay and blasting sand</td>
<td>( \sigma_s ), ( \sigma_f ), LVDTs location, testing procedure, type of soil</td>
<td>T-292 and T-294</td>
<td>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.</td>
</tr>
</tbody>
</table>

Note: \( \sigma_s \) - confining stress; \( \sigma_d \) - deviatoric stress; \( w \) - water content; PI - plasticity index; S, - degree of saturation.
Dynaflect

This is the most commonly used nondestructive equipment in the United States. It is 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 consists of 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 caused 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 estimate the moduli of the surface layers, base, and subgrade. This analysis assumes that the soil is 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 of pavement 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. The deflections will be measured by the geophones placed at several radial distances from the 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 ability 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. Description of this equipment is given below.

MTS Equipment

An MTS model 810 closed loop servo-hydraulic material testing system is used for applying repeated loading. The major components of this system are the loading system, 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 µm (±3 in.), respectively. Figure 3 presents a photograph of the loading system. The load 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 attached 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 interface between the computer and the rest of the system. The interface includes a machine control 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. The computer uses the machine software, TESTSTAR, downloads the program code to the
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. Figure 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 external 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 triaxial chamber. However, the external LVDT location results are significantly influenced by 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 system have a full scale stroke of ± 3.05 mm (± 0.12 in.) with a non-linearity of ± 0.0075 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 this is of a repeated loading type, an accurate and faster sampling data acquisition system is required to capture material response. The data acquisition system consists of a signal conditioner, data acquisition board, and software for equipment control, data reduction and analysis.

Signal Conditioner

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

Data Acquisition Board

A 486-based microcomputer is used together with a 12-bit interfacing board from Metrabyte to collect, store, and analyze the data. Servo valve, 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 board. 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 (0.0004 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 $M_i$ 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 $M_i$ 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 stress levels recommended in the latest versions of AASHTO T-292-91 and T-294-92. The samples were conditioned by applying one thousand repetitions of a specified deviatoric 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 the 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 stress. The tests of T-292 are conducted from higher confining pressure level (140 kPa) to low 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-292 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 cohesion 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 confining 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 $\gamma_{\text{max}} = 17.7$ kN/m$^3$ (110.9 pcf) and $\gamma_{\text{min}} = 15.8$ 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

% Silt: 42.79
% Clay: 19.27
% Colloids: 29.46
Figure 10: Proctor Density Results of Blasting Sand
Figure 11: Proctor Density Results of Silty Clay

Silty Clay Dry Density: 16.23 kN/m$^3$
Optimum Moisture: 20.58 %
Table 2: Density - Water Content Levels of the Soil Specimens

<table>
<thead>
<tr>
<th>Soil</th>
<th>Description (Optimum)</th>
<th>Dry Density (kN/m³)</th>
<th>Moisture Content (%)</th>
<th>Relative Compact (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blasting Sand</td>
<td>Dry</td>
<td>16.85</td>
<td>9.67</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Near</td>
<td>17.19</td>
<td>11.92</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Wet</td>
<td>17.03</td>
<td>13.50</td>
<td>99</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>Dry</td>
<td>15.40</td>
<td>18.00</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>Near</td>
<td>16.23</td>
<td>20.58</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Wet</td>
<td>15.40</td>
<td>23.00</td>
<td>98</td>
</tr>
</tbody>
</table>

_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 that there is no slack in the mold. A vacuum is also applied outside the mold for this purpose. 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 are zeroed. The compaction program which invokes the rammer to apply a certain number of blows for each layer is written with the features available in the MTS soil TESTSTAR.

3. Four layers are used in this preparation. After compacting each layer, the top surface is scarified before placing the next layer. This provides good bonding between soil layers.
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 releasing the suction pressures.

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

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

*Clay Specimens*

The clay specimen preparation is relatively easy when compared with that of procedure is a static compaction type as given by ASTM D1632. An extension of placed on the top of the mold. The soil mass required for certain density is placed inside the mold. The extension sleeve is then removed and a separating disk 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 not since the cohesion property of the specimen will be able to prevent collapse specimen.
Figure 13: Grain Size Distribution Curves of Compacted Sand
Testing Procedure

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

Specimens were prepared at different moisture content levels using the above described 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 two locations followed by resilient modulus testing on core samples retrieved from the same locations. The two sites selected are the airport paving strip at Opelousas, Louisiana (site 1) and the Accelerated Loading Facility or ALF testing facility at Port Allen, Louisiana (site 2). The soils from sites 1 and 2 are classified as silty clay and heavy clay. The properties of these soils are presented in Table 3.
Table 3: Soil Properties of Field Sites

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

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 \( \sigma \), the 100 \((1-\alpha)\)% confidence interval for the mean \( \mu \), is given by:

\[
\mu = x \pm z_{\alpha/2} \left( \frac{\sigma}{\sqrt{n}} \right)
\]

where,

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

\( x = \) sample mean

\( \alpha = \) confidence level.

The error estimation \( e \), defined as the maximum amount by which the estimate in percent differs from population mean \( \mu \), is given by:

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

Rearranging equation 2,

\[
n = \left( \frac{z_{\alpha/2} \sigma}{e} \right)^{2}
\]
However, in the above equation, $\sigma$ 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.

$$n = (t_{\alpha/2}s/e)^2$$  \hspace{1cm} (4)

where,

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

$s = \text{sample standard deviation}$

$\alpha = \text{confidence level}$.

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

<table>
<thead>
<tr>
<th>Laboratory / Field</th>
<th>Soil Type</th>
<th>Number of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratory</td>
<td>Blasting Sand</td>
<td>30(O)</td>
</tr>
<tr>
<td>Laboratory</td>
<td>Blasting Sand</td>
<td>25(N)</td>
</tr>
<tr>
<td>Laboratory</td>
<td>Silty Clay</td>
<td>30(O),25(N)</td>
</tr>
<tr>
<td>Field</td>
<td>Silty Clay</td>
<td>3(N)</td>
</tr>
<tr>
<td>Field</td>
<td>Heavy Clay</td>
<td>3(N)</td>
</tr>
</tbody>
</table>

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 density and water content combination level. The coefficient of variation $C_v$, used to measure repeatability of the tests, was calculated and presented in the same table. The $C_v$ values for 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)

<table>
<thead>
<tr>
<th>Cf. Pr.</th>
<th>D Str.</th>
<th>AASHTO Procedure</th>
<th>AASHTO Procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>T-292</td>
<td>T-294</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MRE</td>
<td>SD</td>
</tr>
<tr>
<td>21</td>
<td>21</td>
<td>159.2</td>
<td>4.7</td>
</tr>
<tr>
<td>21</td>
<td>35</td>
<td>153.0</td>
<td>8.4</td>
</tr>
<tr>
<td>21</td>
<td>52.5</td>
<td>162.1</td>
<td>10.1</td>
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<td>21</td>
<td>70</td>
<td>188.7</td>
<td>9.7</td>
</tr>
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<td>21</td>
<td>87.5</td>
<td>173.6</td>
<td>10.9</td>
</tr>
<tr>
<td>35</td>
<td>42</td>
<td>168.0</td>
<td>9.5</td>
</tr>
<tr>
<td>35</td>
<td>70</td>
<td>183.8</td>
<td>9.6</td>
</tr>
<tr>
<td>35</td>
<td>105</td>
<td>195.5</td>
<td>10.1</td>
</tr>
<tr>
<td>35</td>
<td>140</td>
<td>206.5</td>
<td>12.3</td>
</tr>
<tr>
<td>70</td>
<td>35</td>
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<tr>
<td>70</td>
<td>140</td>
<td>257.6</td>
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<td>70</td>
<td>210</td>
<td>274.1</td>
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<tr>
<td>105</td>
<td>70</td>
<td>287.0</td>
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<td>105</td>
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<td>304.5</td>
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<td>317.8</td>
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<tr>
<td>105</td>
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<td>140</td>
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<tr>
<td>140</td>
<td>280</td>
<td>372.4</td>
<td>11.8</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Cf. Pr</th>
<th>D Str.</th>
<th>Measurement System</th>
<th>End</th>
<th>Middle</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>MRE</td>
<td>SD</td>
</tr>
<tr>
<td>42</td>
<td>14</td>
<td>243.2</td>
<td>23.0</td>
<td>9.5</td>
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<td>42</td>
<td>28</td>
<td>216.9</td>
<td>14.2</td>
<td>6.5</td>
</tr>
<tr>
<td>42</td>
<td>42</td>
<td>195.9</td>
<td>10.1</td>
<td>5.1</td>
</tr>
<tr>
<td>42</td>
<td>55</td>
<td>178.8</td>
<td>10.1</td>
<td>5.6</td>
</tr>
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<td>42</td>
<td>69</td>
<td>152.7</td>
<td>10.0</td>
<td>6.1</td>
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<td>14</td>
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<td>17.9</td>
<td>8.8</td>
</tr>
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<td>21</td>
<td>28</td>
<td>186.2</td>
<td>9.9</td>
<td>5.3</td>
</tr>
<tr>
<td>21</td>
<td>42</td>
<td>171.7</td>
<td>11.6</td>
<td>6.8</td>
</tr>
<tr>
<td>21</td>
<td>55</td>
<td>157.7</td>
<td>11.3</td>
<td>7.2</td>
</tr>
<tr>
<td>21</td>
<td>69</td>
<td>145.9</td>
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<td>8.6</td>
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<td>35</td>
<td>171.9</td>
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<td>10.2</td>
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<td>52</td>
<td>158.4</td>
<td>18.9</td>
<td>11.9</td>
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<td>69</td>
<td>141.7</td>
<td>19.6</td>
<td>13.8</td>
</tr>
<tr>
<td>21</td>
<td>86</td>
<td>127.4</td>
<td>19.8</td>
<td>15.5</td>
</tr>
<tr>
<td>0</td>
<td>14</td>
<td>161.5</td>
<td>15.5</td>
<td>9.6</td>
</tr>
<tr>
<td>0</td>
<td>28</td>
<td>141.4</td>
<td>14.6</td>
<td>10.3</td>
</tr>
<tr>
<td>0</td>
<td>42</td>
<td>129.9</td>
<td>13.8</td>
<td>10.6</td>
</tr>
<tr>
<td>0</td>
<td>55</td>
<td>122.4</td>
<td>13.0</td>
<td>10.6</td>
</tr>
<tr>
<td>0</td>
<td>69</td>
<td>116.8</td>
<td>14.6</td>
<td>12.5</td>
</tr>
</tbody>
</table>

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/m³. 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 Cᵥ 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
procedure produced higher resilient moduli results than the T-292 procedure. The reasoning as above can be given for this observation.

The differences between results from both procedures for silty clays is not as significant as in the case for sands. This may be due to the fact that stress magnitudes of silty clays are significantly smaller than those of sands. These smaller magnitudes did not result in 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 number 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 suggest that the influence of the number of cycles on the resilient modulus is not significant after a certain number of cycles which is about twenty-five for the soils of the present study. This implies that the accumulated plastic deformation with number of cycles beyond twenty five does not influence the resilient modulus results.

The question that arises now is the need for taking one hundred cycles to measure the 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 procedure is high and needs to be reduced. The observation, however, may not be valid for all soils. The authors recommend that the number of cycles should be selected based on the type of soil and their characteristics. For example, based on the results of the soils tested in this study, the number of cycles can be decreased to twenty-five since the $M_r$ does not show 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

\[ \sigma_3 = 35 \text{ kPa}; \sigma_d = 140 \text{ kPa} \]

\[ M_r = 0.088 \times n + 208.5 \text{ (end)} \]
Figure 18: Resilient Modulus Versus Number of Cycles: Silty Clay.

$\sigma_3 = 21 \text{ kPa}; \sigma_d = 86 \text{ kPa}$

$M_r = -0.07 \times n + 125.94 \text{ (end)}$
SOIL VARIABLES

Sands - M, 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 Moisture Dry Density Combination (T-294)

<table>
<thead>
<tr>
<th>Cf. Pr. (kPa)</th>
<th>D Str. (kPa)</th>
<th>MRE (MPa)</th>
<th>MRM (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>STD</td>
</tr>
<tr>
<td>139.0</td>
<td>104.5</td>
<td>351.4</td>
<td>9.0</td>
</tr>
<tr>
<td>20.4</td>
<td>20.7</td>
<td>122.3</td>
<td>4.1</td>
</tr>
<tr>
<td>20.4</td>
<td>34.9</td>
<td>130.5</td>
<td>3.0</td>
</tr>
<tr>
<td>20.4</td>
<td>51.7</td>
<td>137.5</td>
<td>1.9</td>
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<td>34.2</td>
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<td>104.4</td>
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<td>138.7</td>
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<tr>
<td>138.9</td>
<td>276.9</td>
<td>412.5</td>
<td>4.7</td>
</tr>
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</table>

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)

<table>
<thead>
<tr>
<th>Cf. Pr. kPa</th>
<th>D Str. kPa</th>
<th>MRE (MPa) Mean</th>
<th>MRE (MPa) STD</th>
<th>MRE (MPa) CV</th>
<th>MRM (MPa) Mean</th>
<th>MRM (MPa) STD</th>
<th>MRM (MPa) CV</th>
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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)
STD: Standard Deviation (MPa)
CV: Coefficient of Variation in percent
Table 9: Resilient Modulus Test Results on Blasting Sand at Wet of Optimum (T-294)

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Cf. Pr.: Confining Pressure (in kPa)
D Str.: Deviotoric 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.
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
Figure 21: Influence of Moisture Content and Deviatoric Stress on Resilient Modulus of 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
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, 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 Phases 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, 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
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 Content Dry Density Combination (T-294)

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<th>MRM (MPa)</th>
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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)

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

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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
reduction of plastic deformations at 42 kPa, it significantly decreased the plastic
dehformations at the other confining pressures of 21 and 0 kPa. The role of conditioning
silty clays is probably realized at lower confining stresses. The reason for not obtaining
lower plastic deformations at 42 kPa is attributed to the stiffening or over consolidation
the specimen at 42 kPa confining pressure. This implies that in the case of field core
core samples, a conditioning confining stress which needs to be significantly higher to
the lateral confining pressure of the retrieval depth location is required to reduce plastic
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 Deviatoric Stress on Resilient Modulus 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 deviatoric stresses (greater than 70 kPa) which implies that test procedures have a minor influence on M, values at these stresses. This is probably because the higher deviatoric stresses applied to the specimen will overcome the stress dependency effects due to previous testing stress sequences. However, surprisingly for both measurement systems, lower PC values with an 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 pressures can be reasoned from previous explanations. However, the same cannot be explained in the 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. In 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 preceded 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 the stress dependency behavior to a certain degree beyond which the testing procedures did not 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 systems and confining pressures of magnitudes 35, 70, and 105 kPa. For other confining pressures,
of 21 and 140 kPa, the coefficients remain constant for all deviatoric stresses and are around 1.08.

\[
PC = 1.28 - 0.00115 \times \sigma_d
\]  

(5)

where \(\sigma_d\) 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, refer 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 in the same figure. The coefficients from both measurements are approximately 1.0 except 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, and PC values are calculated by the middle measurement system at 72 kPa deviatoric stress. Overall, the procedure variation on M, values is not as significant as in the case of sands. Two reasons 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. \( H_0 : \mu_e = \mu_m \quad H_a : \mu_e \neq \mu_m \)

where \( \mu_e \) is the mean \( M_r \) for the end measurement system and \( \mu_m \) 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. \( H_0 : \mu_{ee} = \mu_{em} \quad H_a : \mu_{ee} \neq \mu_{em} \)

where \( \mu_{ee} \) is the mean \( M_r \) for the end measurement system at a certain confining pressure, \( \sigma_3 \), and \( \mu_{em} \) is the mean \( M_r \) for the middle measurement system at the same confining pressure, \( \sigma_3 \). 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. $H_0: \mu_{de} = \mu_{dm}$  $H_a: \mu_{de} \neq \mu_{dm}$

where $\mu_{de}$ is the mean $M_r$ for the end measurement system at a certain deviatoric stress, $\sigma_d$, and $\mu_{dm}$ is the mean $M_r$ for the middle measurement system at the same deviatoric stress, $\sigma_d$. 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 repeated 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 specimens inside the triaxial cell. Otherwise, air gaps between specimens and accessories such as porous stones and platens, system compliance, and errors such as sample alignment and bedding problems would induce significant errors to the measurements. The internal measurements, though hard to install, provide results which are less influenced by the system compliance of triaxial cell accessories. This is the reason behind using the internal measurements in the present tests.

The influence of the measurement system is presented in the form of measurement 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 can be used to convert the end measurement results to more realistic middle measurement 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 - wet...
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 six only loading sequence and T-294 shows both loading and unloading sequences. Specimen 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 influence the displacement measurements. The end system which measures the displacements over the full length of the specimen will be influenced more by these problems than the middle system. The end system, therefore, measured significantly higher displacements resulting in lower M, values and higher measurement coefficients. In addition, these problems will be more significant in unconfined conditions than in confined conditions. This is the reason that higher MC values are obtained in the unconfined state. The MC values decrease with an increase in confining stress and to a certain extent with an increase in deviatoric stress. The MC values from both test procedures, which match at 21 kPa confining pressure, are 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 the results from Figure 32. The deviatoric stress is not taken into account in the equation since its influence on MC value is relatively insignificant when compared with the confining pressure.

\[ MC = 1.52 \times e^{-0.00594 \times \sigma_3} \]
Figure 31: Measurement Coefficients of Sands: Influence of LVDT's Location
Figure 32: Measurement Coefficients of Silty Clays: Influence of LVDT's Location
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 $M_i$ 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 various 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 this are explained in the earlier sections. These values decrease with an increase in the confining stress. This probably indicates that higher confining stresses provide better contact between LVDTs and specimens and allow more accurate measurements. Even though the author did not notice any visual slipping problems in this testing, slipping at unconfined states may have occurred which probably influenced the middle measurement results. The results in the figures are used to provide the following equations for measurement coefficients. Linear regression analysis is used to obtain the following equations.

\[
MC = (0.00335 \sigma_3 - 0.051) \sigma_d + (1.83 - 0.0702 \sigma_3)
\]  

\[
MC = (0.00032 \sigma_3 - 0.013) \sigma_d + (1.43 - 0.0402 \sigma_3)
\]
\[ MC = (0.000298 \sigma_3 - 0.017) \sigma_d + (1.26 - 0.0124 \sigma_3) \]  \hspace{1cm} (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 Deviatoric Stress 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, $T(\theta)$ or the bulk stress and the deviatoric stress are used as predictors in these models depending 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:

$$M_r = k_1 * \theta^{k_2} \quad \text{Granular Soils}$$

$$M_r = k_3 * \sigma_d^{k_4} \quad \text{Cohesive Soils}$$

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 (Figure 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 AASHTO 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

End: $\log k_1 = 3.86; k_2 = 0.63 \ (r^2 = 0.92)$

Middle: $\log k_1 = 4.37; k_2 = 0.46 \ (r^2 = 0.96)$
Figure 37: Regression Model for Sands at Wet of Optimum
Silty Clay - Dry of Optimum

End : \log k_3 = 5.53 \; ; \; k_4 = -0.09 \; (r^2 = 0.11)

Middle : \log k_3 = 5.89 \; ; \; k_4 = -0.24 \; (r^2 = 0.88)

Figure 38: Regression Model for Silty Clays at Dry of Optimum
Figure 39: Regression Model for Silty Clays at Optimum
Silty Clay - Wet of Optimum

End: \( \log k_3 = 5.71; k_4 = -0.41 \) (\( r^2 = 0.72 \))

Middle: \( \log k_3 = 5.88; k_4 = -0.48 \) (\( r^2 = 0.82 \))

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

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (kN/m³)</th>
<th>End System</th>
<th>Middle System</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>log(k₁)</td>
<td>k₂</td>
<td>log(k₁)</td>
</tr>
<tr>
<td>9.67</td>
<td>4.44</td>
<td>0.43</td>
<td>4.20</td>
</tr>
<tr>
<td>11.92</td>
<td>4.38</td>
<td>0.48</td>
<td>4.91</td>
</tr>
<tr>
<td>12.0 (O)</td>
<td>4.23</td>
<td>0.49</td>
<td>4.35</td>
</tr>
<tr>
<td>12.0 (O)*</td>
<td>4.15</td>
<td>0.49</td>
<td>4.30</td>
</tr>
<tr>
<td>13.50</td>
<td>4.39</td>
<td>0.49</td>
<td>4.15</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Dry Density (kN/m³)</th>
<th>End System</th>
<th>Middle System</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>log(k₃)</td>
<td>k₄</td>
<td>log(k₃)</td>
</tr>
<tr>
<td>18.0</td>
<td>5.83</td>
<td>-0.09</td>
<td>6.67</td>
</tr>
<tr>
<td>20.6</td>
<td>5.90</td>
<td>-0.21</td>
<td>6.29</td>
</tr>
<tr>
<td>21.2 (T-292)</td>
<td>5.75</td>
<td>-0.14</td>
<td>6.15</td>
</tr>
<tr>
<td>23.0</td>
<td>6.25</td>
<td>-0.41</td>
<td>6.63</td>
</tr>
</tbody>
</table>
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 that
two soil types are used in this investigation. This implies that the data is not sufficient
enough to provide the significant variation expected in all soils or to provide meaningful
complete conclusions. The grain size properties are not used in the analysis since only
sets of soils are tested which will not provide wide variation in grain size properties.
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 Location 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 (AASHTO classification) with liquid limit and plasticity index values of 22 and 6, respectively. The resilient modulus results from laboratory tests on field core specimens 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 on magnitudes of confining and deviatoric stresses.

The falling weight deflectometer and dynaflect data were analyzed using backcalculation and analytical charts. The modulus 4.1 backcalculation software described earlier was used in the backcalculation analysis of FWD data. This program reads the measured geophone deflection data and then computes the deflections by assuming different moduli for pavements layers. The measured and computed data will be compared and the one which provides the best comparisons is the final modulus value. Table 16 provides a comparison of these results with respect to laboratory determined values. Results indicated that the moduli computed from nondestructive devices are significantly lower than the laboratory moduli determined at 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 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

<table>
<thead>
<tr>
<th>Cf. Pr. (kPa)</th>
<th>D Str. (kPa)</th>
<th>MRE (MPa)</th>
<th>MRM (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>STD</td>
</tr>
<tr>
<td>42.0</td>
<td>27.9</td>
<td>206.1</td>
<td>11.0</td>
</tr>
<tr>
<td>42.0</td>
<td>13.8</td>
<td>220.8</td>
<td>7.6</td>
</tr>
<tr>
<td>42.0</td>
<td>28.0</td>
<td>209.0</td>
<td>10.2</td>
</tr>
<tr>
<td>41.9</td>
<td>41.3</td>
<td>192.3</td>
<td>10.2</td>
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<tr>
<td>41.9</td>
<td>55.2</td>
<td>178.6</td>
<td>8.9</td>
</tr>
<tr>
<td>42.0</td>
<td>69.2</td>
<td>168.4</td>
<td>8.5</td>
</tr>
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<td>20.9</td>
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<td>9.5</td>
</tr>
<tr>
<td>21.2</td>
<td>27.9</td>
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<td>10.1</td>
</tr>
<tr>
<td>21.1</td>
<td>41.4</td>
<td>165.0</td>
<td>9.3</td>
</tr>
<tr>
<td>21.1</td>
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<tr>
<td>21.1</td>
<td>69.4</td>
<td>149.7</td>
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<td>0.7</td>
<td>13.7</td>
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<td>10.9</td>
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<td>27.8</td>
<td>133.7</td>
<td>10.5</td>
</tr>
<tr>
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<td>41.2</td>
<td>126.9</td>
<td>9.5</td>
</tr>
<tr>
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<td>55.2</td>
<td>122.7</td>
<td>8.6</td>
</tr>
<tr>
<td>0.7</td>
<td>69.3</td>
<td>120.4</td>
<td>8.2</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Dynaflect Moduli (MPa)</th>
<th>FWD Moduli (MPa)</th>
<th>Laboratory Results (Middle System)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\sigma_3$ (kPa)</td>
</tr>
<tr>
<td>86.8</td>
<td>96.6</td>
<td>42</td>
</tr>
<tr>
<td>86.8</td>
<td>96.6</td>
<td>21</td>
</tr>
<tr>
<td>86.8</td>
<td>96.6</td>
<td>0</td>
</tr>
</tbody>
</table>

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) with liquid limit and plasticity index values of 93 and 66, respectively. The AASHTO T-28 procedure was used for conducting resilient modulus tests. The results are given in Table 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 the low strength of the heavy clay soil. The backcalculated moduli data from the Dynaflect are compared in Table 18. Results appear to be in agreement with one another. FWD test results are not conducted on this site and therefore not included in this assessment. Significant research work with field studies is still needed to understand the applicability of backcalculation procedures 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 Ali Site

<table>
<thead>
<tr>
<th>Cf. Pr. (kPa)</th>
<th>D Str. (kPa)</th>
<th>MRE (MPa)</th>
<th>MRM (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>STD</td>
</tr>
<tr>
<td>41.7</td>
<td>14.0</td>
<td>33.9</td>
<td>6.0</td>
</tr>
<tr>
<td>41.7</td>
<td>6.8</td>
<td>39.0</td>
<td>6.5</td>
</tr>
<tr>
<td>41.7</td>
<td>14.1</td>
<td>34.2</td>
<td>6.1</td>
</tr>
<tr>
<td>41.7</td>
<td>21.1</td>
<td>29.7</td>
<td>5.5</td>
</tr>
<tr>
<td>20.6</td>
<td>6.8</td>
<td>35.2</td>
<td>5.7</td>
</tr>
<tr>
<td>20.6</td>
<td>14.1</td>
<td>30.3</td>
<td>5.3</td>
</tr>
<tr>
<td>20.6</td>
<td>21.1</td>
<td>26.5</td>
<td>4.9</td>
</tr>
<tr>
<td>1.3</td>
<td>6.7</td>
<td>28.4</td>
<td>4.6</td>
</tr>
<tr>
<td>1.3</td>
<td>14.0</td>
<td>23.8</td>
<td>4.4</td>
</tr>
<tr>
<td>1.3</td>
<td>21.0</td>
<td>20.5</td>
<td>4.2</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Dynaflect Moduli (MPa)</th>
<th>FWD Moduli (MPa)</th>
<th>Laboratory Results (Middle)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\sigma_3$ (kPa)</td>
</tr>
<tr>
<td>16.8</td>
<td>NA</td>
<td>42</td>
</tr>
<tr>
<td>16.8</td>
<td>NA</td>
<td>21</td>
</tr>
<tr>
<td>16.8</td>
<td>NA</td>
<td>0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Correlations -I Modulus (from R Value) (MPa)</th>
<th>Laboratory Results (ranges)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GI</td>
<td>R</td>
</tr>
<tr>
<td>Sand (E)</td>
<td>0</td>
<td>70</td>
</tr>
<tr>
<td>Silty Clay (E)</td>
<td>7</td>
<td>36</td>
</tr>
<tr>
<td>Silty Clay (F)</td>
<td>8</td>
<td>32</td>
</tr>
<tr>
<td>Heavy Clay (F)</td>
<td>16</td>
<td>11</td>
</tr>
</tbody>
</table>

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 laboratory 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. A research study also provided some insight into the influence of AASHTO testing procedure and LVDTs measurement locations on the resilient modulus of soils. Preliminary correlations are also developed between resilient modulus properties and CBR, dry density, and moisture contents.

CONCLUSIONS

The testing program was developed and conducted based on statistical concepts, which reduced the operational and equipment related errors in this study. Quality control procedures indicated that the AASHTO specimen preparation methods provided reasonably homogenous specimens. The coefficient of variation of clays, an indicator for the variance in test results, ranged from 1 to a maximum of 30 with most of them around 10 percent. The coefficients of variation of sands was smaller (less than 10) when compared with that of clays. These smaller coefficients in both soils indicate that good to excellent repeatability 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 due to 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. These coefficients which are greater than one, indicate that the middle system provides higher 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 by confining and deviatoric stresses. The MC values of silty clay vary from 1.1 to 1.7. It is 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 deviatoric 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 more 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


