Resilient Properties of Laboratory Compacted Subgrade Soils

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The subgrade of road bed soils is generally constructed near the optimum moisture content-dry density combination standard Proctor tests. However, environmental and seasonal variations alter the design moisture contents and the resilient properties of subgrade soils. These changes influence structural performance of pavements. Recent pavement design guides, which use the resilient modulus for characterizing pavement materials, require additional information on seasonal moisture content variations and their influence on resilient properties. This report presents results from an experimental study which investigated the influence of moisture content variations on the resilient modulus of soils. Two soil types, sand and silty clay compacted at, above, and below optimum moisture content levels were tested using the AASHTO T-294 procedure. Two in-cell displacement measurement systems measured displacements with respect to the ends of the specimen and the middle one-third of the specimen. Results indicated that the influence of moisture content on moduli appears to be more evident in clays than in sands. The effect of moisture content on the resilient modulus computed from both measurement systems is discussed. The measurement coefficients used to convert end system moduli to middle system moduli are provided for various moisture content levels in both soils. The influence of conditioning confining stresses and moisture contents on accumulated plastic deformations is also evaluated. The regression model results for the tests conducted in the investigation are also included.

The 1986 AASHTO design guide recommends using a resilient modulus as the property for characterizing flexible pavement materials (*1*). The resilient modulus is defined as the ratio of repeated deviatoric stress to recoverable axial strain. The subgrade soils are usually tested in repeated load triaxial tests at the optimum moisture content since most embankment subgrade soils are compacted close to the optimum moisture content and the maximum dry density levels determined from standard Proctor tests. However, moisture content levels in the subgrade soils will vary as a result of seasonal and environmental fluctuations. The changes in moisture contents in the field will result in the variation of soil saturation levels, which in turn alter the strength and resilient properties of subgrade soils. Therefore, characterization based on resilient modulus (M_r) values at optimum moisture contents may not be accurate or reasonable enough to provide realistic designs for the design life of the pavement.

In addition, it is reported that most pavement failures occur as a result of wet and dry moisture content cycles of subgrade layers (*2*). Saturation due to flooding of underlying soil layers results in erosion of subgrades (*2*). On the other hand, extreme drying in summer periods causes shrinkage cracks in plastic clays. When wetted, the cracks soften the subgrade layers and decrease the strength of the pavement. Thus, it is important to identify the properties on the dry and wet side of the optimum moisture content levels which are more representative of field conditions. This is the rationale for the use of relative damage factors which represent moisture content fluctuations due to seasonal temperature variations in determining the effective resilient modulus. This modulus is used in the design of flexible pavements.

Two types of soils were tested at three different moisture contents. The soils were a blasting sand and a silty clay. The tests were conducted on the dry of optimum, at optimum and on the wet of optimum moisture content. This allowed to study of the influence of moisture content variation on the resilient modulus and plastic deformations developed during the conditioning and testing phases. Also, as a part of the laboratory investigations, the influence of the location of internally placed linear variable differential transformers (LVDTs) in providing accurate and realistic measurements is evaluated.

BACKGROUND AND OBJECTIVE

Several investigators recognize moisture content as an important soil parameter for determining the resilient modulus of soils for the design and characterization of flexible pavements (*2–5*). Elliott et al. (*5*) listed drainability, hydraulic conductivity, soil type, geometry of the road, topography, water table depth, precipitation, and temperature as important factors that affect moisture contents in the field. Some of these factors and their influence on pavement performance are described below.

Cohesive subgrade soils pose a problem in practice, since the pore pressure developed during traffic loading will not be dissipated immediately due to the low hydraulic conductivity of soils. As a result, the effective stresses and subsequently the strength of the subgrade soils will be decreased. The resilient properties also decrease and may cause rutting failures in pavement when subjected to higher traffic loads. The hydraulic conductivity of the compacted soils depends on several factors like soil fabric, mineralogy, and moisture content or saturation level. The moisture content is the controlled variable in this study. Therefore, any factors that influence the moisture contents of soils will also influence the performance of pavement sections.

Several research studies in the past have attempted to investigate the influence of moisture content on resilient modulus results. Thompson (*3*) provided regression equations relating the breakpoint resilient modulus values at a deviatoric stress of 6 lb/in.^{2} and the degree of saturation. These equations, which showed the combined effects of moisture contents and densities, become independent of the degree of saturation with an increase in clay content and plasticity. Pezo et al. (*6*) investigated influence of moisture content and plasticity indices by conducting M_r tests on several subgrade soils

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from Texas. This investigation indicated that an increase in moisture content resulted in a decrease of the resilient moduli. It is also reported that the influence of moisture content is more significant on soils with lower plasticity indices than those with high plasticity indices.

Conclusions drawn from previous studies regarding the role of moisture contents are similar. This research study is also aimed at understanding the resilient behavior of two soils at various moisture contents. This investigation is, however, different from other studies. Previous studies utilized external or internal end measurements and the soil samples were tested under AASHTO procedures T-274 and T-292. This study utilized two types of internal measurement systems and the samples were tested using the recent AASHTO T-294 procedure (*7*). The results are used to understand the influence of moisture contents on the resilient modulus and plastic deformations of soils, as well as to determine the influence of internal measurement systems in obtaining realistic measurements. It should be noted that the relative compaction level of dry, wet, and at optimum moisture contents varied between 98 and 100 percent, which indicates that the effect of dry density is not a controlling factor in this study.

EQUIPMENT AND SOIL DESCRIPTION

Loading System

An MTS model 810 closed loop servo-hydraulic material testing system was used for applying repeated loading. A detailed description of the equipment is presented elsewhere (*8–10*). An automated test software was developed and used for conducting tests, and performing data acquisition, reduction, and analysis tasks.

Measurement Systems

One of the aspects of interest in this study is the location of LVDTs on the specimens. It should be mentioned that the T-294-1992 procedure, which was used for conducting tests in the present program, suggests the use of an external LVDT system while the T-292-1991 procedure requires the use of both external and internal LVDT systems. The internal LVDT system located inside the chamber is a better system than those used outside the chamber since the measurements by internal systems are less influenced by system compliance errors.

This study used two diametrically placed internal LVDTs fixed on a Plexiglas clamp system. One system measured the deformations with respect to the ends of the specimen and the other measured the deformations in the middle one-third of the specimen. These systems are referred to as end system and middle systems, respectively.

Description of Soils and Specimen Preparation

Two locally available soils, a uniform blasting sand and a silty clay, were used in this study. The blasting sand exhibited dry densities of γ_{max} = 17.7 kN/m^3 (110.9 lb/ft^3) and γ_{min} = 15.8 kN/m^3 (99.0 lb/ft^3). The silty clay had an optimum water content of 20.6 percent, a maximum dry density of 16.3 kN/m^3 (101.6 lb/ft^3) and a plasticity index (PI) of 22. The silty clay and blasting sand were classified as A-7-6 and A-3 using AASHTO classification. Figure 1 presents standard Proctor density curves of these soils. Sand specimens were compacted in-place in the triaxial cell to reduce sample disturbance. Cohesive specimens were compacted in molds and carefully extruded for testing. Both specimens were 71.1 mm (2.8 in.) in diameter and 142.2 mm in height. Specimens were compacted at three different moisture contents and dry density combinations which cover the optimum, dry of optimum, and wet of optimum ranges. A list of these levels is given in Table 1.

Conditioning and Testing Procedure

The specimens were first subjected to a conditioning phase, followed by testing at various confining and deviatoric stress levels. The prescribed deviatoric load of haversine shape was applied in both conditioning and testing phases. The tests on both materials were performed at the confining and deviatoric stress levels recommended in the latest versions of AASHTO T-294-92. Detailed description of the AASHTO testing procedure can be found elsewhere (*8–10*). The data obtained from the acquisition were analyzed and reduced to determine resilient strains, plastic strains, and the moduli values for each of the confining and deviatoric stresses.

ANALYSIS OF RESULTS

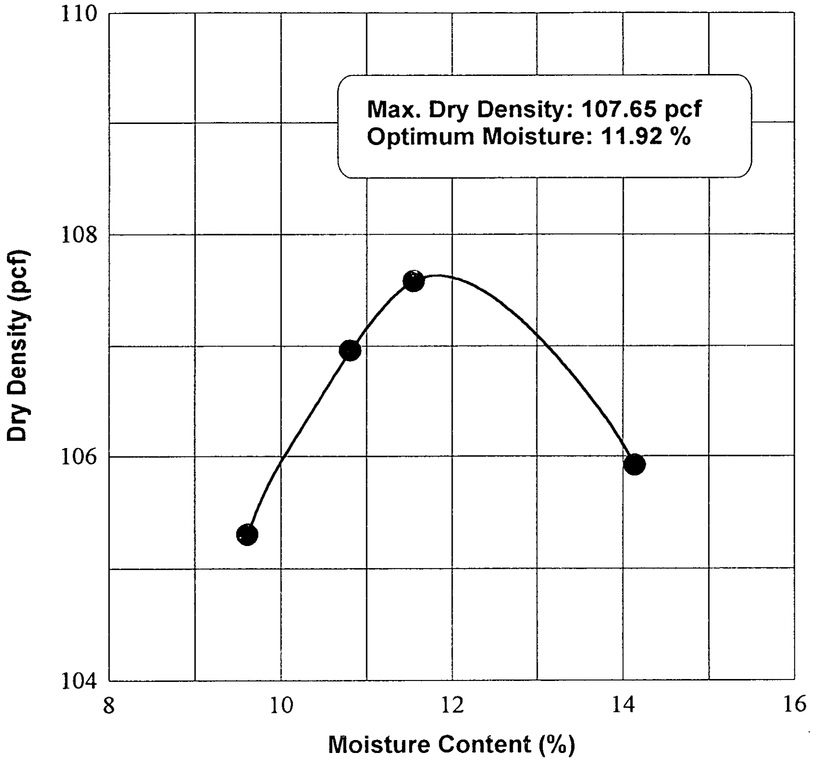
Sands

Table 2, a, b, and c, presents the resilient modulus test results for each moisture content level. The data include the end and middle resilient modulus, along with their means, standard deviations (*STD*), and coefficient of variations (*CV*) for each set of five tests. The coefficient of variation varied between 0.1 and 15.0 with most of the values being around 3.0. This implies that the test results are highly repeatable. The higher coefficients of variations (*CV* values around 10) were obtained for the end resilient moduli data from tests conducted above and below optimum moisture contents at low confining stresses.

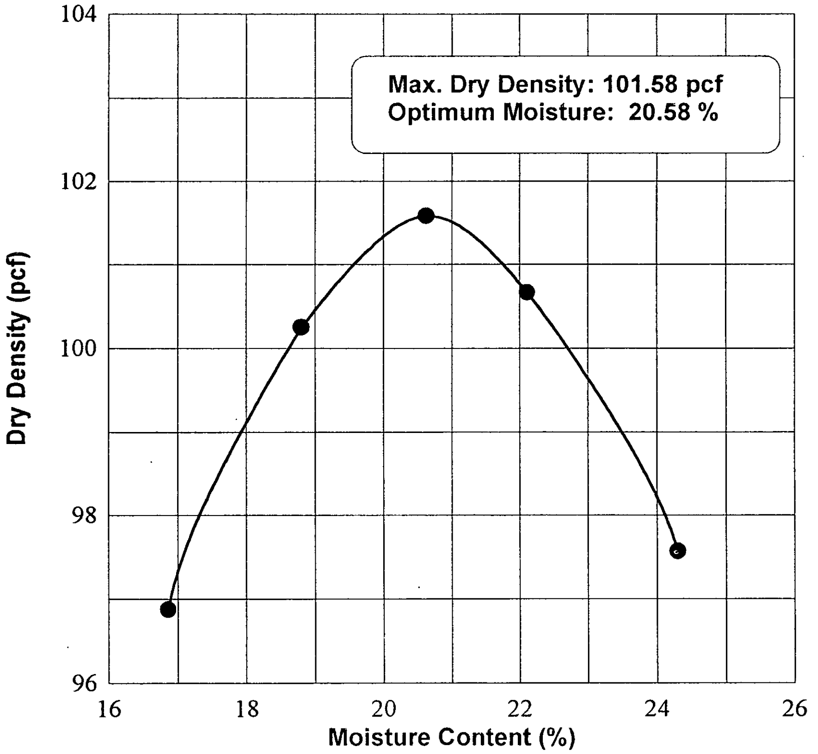
Figure 2 shows the variation of end resilient moduli at different moisture contents and dry densities for various confining stresses of 21 kPa (3 lb/in.^{2}), 70 kPa (10 lb/in^2), and 140 kPa (20 lb/in.^{2}), respectively. It should be noted that the resilient modulus increases with an increase in the confining pressure. This is attributed to the stiffness properties which increase with an increase in confining pressure (*9*). The decrease in dilational behavior at higher confining pressures also results in lesser axial strains and higher moduli values.

The influence of deviatoric stress can also be deduced from Figure 2. The deviatoric stress increase resulted in a slight increase in the end moduli values at each confining pressure. Overall, the deviatoric stress has a minor influence on resilient properties when compared with the confining pressure. This is because the deviatoric load levels applied are lesser than the peak deviatoric loads which can induce significant changes and deformations in the specimens. Therefore, the deformation responses in these tests are directly proportional to the applied deviatoric loading which results in slightly higher M_r values.

The compaction moisture content appears to have some influence on M_r results at low confining pressures. Figure 2 also indicates that higher moduli values are obtained at dry and wet of optimum moisture content level. An increase in strength at dry of optimum resulted in a slight increase in M_r values over the optimum M_r values. This is in agreement with those reported by other investiga-



(a) Blasting Sand



(b) Silty Clay

FIGURE 1 Standard proctor curves.

TABLE 1 Density-Moisture Content Levels of the Soil Specimens

Soil	Description (Optimum)	Dry Density (pcf)	Moisture Content (%)	Relative Compaction (%)
Blasting Sand	Dry	105.5	9.67	98
	Near	107.65	11.92	100
	Wet	106.65	13.50	99
Silty Clay	Dry	96.50	18.00	98
	Near	101.58	20.58	100
	Wet	96.50	23.00	98

tions (*2,3,5*). On the other hand, the moduli at wet of optimum are higher than those of optimum at low confining pressures. This trend cannot be explained except for the fact that leakage problems were experienced during the wet of optimum specimen tests. This may have decreased the moisture contents in the soil specimens which probably resulted in higher resilient properties.

Results at higher confining pressures appear to provide similar resilient properties at all moisture contents. This is attributed to the small variations in relative compaction levels as well as the smaller role of moisture contents in sands at higher confining pressures. As mentioned earlier, the relative compactions at the moisture content-density levels vary from 98 to 100 percent which indicate that the corresponding relative densities of the sand do not vary considerably. This variation in relative density is not significant enough to provide distinctly different M_r values.

Clays

Table 3 presents the resilient modulus test results at dry of optimum, optimum and wet of optimum, respectively. The coefficients of variation of clay test results are higher than those obtained for sands. However, this number is relatively small when compared with the variations in modulus values. This indicates that the clay test results are also repeatable.

Figure 3 presents clay specimen results at dry, near, and wet of optimum moisture content levels. The end measurement results are depicted in this figure. An increase in confining pressure resulted in an increase in moduli values. This is attributed to an increase in stiffness with an increase in confining pressure. Results at three moisture content levels show that the M_r values at dry and optimum levels are closer but significantly higher than those at wet of optimum.

TABLE 2 Sand Test Results

(a) Dry of Optimum

σ_3	σ_d	Mr, ends (MPa)			Mr,middle (MPa)		
kPa	kPa	Mean	STD	CV	Mean	STD	CV
139.3	104.7	375.2	22.0	5.9	429.8	17.7	4.1
20.9	20.9	151.9	21.2	14.0	190.7	11.7	6.1
21.1	35.1	160.5	22.5	14.0	196.7	9.4	4.8
21.1	52.1	164.5	23.5	14.3	202.6	9.5	4.7
21.1	69.4	165.1	24.6	14.9	201.9	10.0	5.0
34.8	35.2	204.4	27.5	13.4	258.6	9.1	3.5
34.9	69.7	209.9	27.1	12.9	256.7	10.4	4.1
34.9	104.4	205.8	25.0	12.1	244.0	11.2	4.6
34.9	137.9	194.6	23.4	12.0	221.1	14.5	6.6
69.7	35.1	277.8	26.6	9.6	338.9	13.5	4.0
69.7	69.7	287.0	23.9	8.3	338.9	14.6	4.3
69.7	138.8	291.1	21.5	7.4	332.8	17.8	5.4
69.6	207.7	280.3	19.1	6.8	311.9	15.7	5.0
104.6	69.6	334.2	19.4	5.8	387.2	15.2	3.9
104.6	104.6	341.6	19.4	5.7	390.9	16.6	4.2
104.5	138.7	345.5	18.8	5.4	391.9	16.9	4.3
104.5	207.8	347.0	18.0	5.2	385.8	17.2	4.5
139.4	69.5	379.6	18.2	4.8	435.8	15.8	3.6
139.4	104.5	387.8	18.3	4.7	438.5	17.0	3.9
139.4	138.7	393.2	17.7	4.5	443.8	18.5	4.2
139.4	276.9	396.7	17.5	4.4	434.1	19.1	4.4

TABLE 2 *(continued)*

(b) Optimum

σ_3 (kPa)	σ_d (kPa)	Mr, ends (MPa)			Mr, middle (MPa)		
		Mean	STD	CV	Mean	STD	CV
139.0	104.5	351.4	9.0	2.6	434.3	20.6	4.8
20.4	20.7	122.3	4.1	3.3	193.1	13.1	6.8
20.4	34.9	130.5	3.0	2.3	196.5	12.4	6.3
20.4	51.7	137.5	1.9	1.4	205.4	14.3	7.0
20.4	69.2	143.5	1.3	0.9	211.8	14.8	7.0
34.2	35.1	177.9	1.9	1.1	263.9	15.6	5.9
34.2	69.6	184.3	2.1	1.2	265.1	14.1	5.3
34.2	104.4	188.9	2.4	1.3	258.9	15.3	5.9
34.1	138.3	193.2	3.0	1.6	252.7	15.5	6.2
69.1	35.1	266.3	3.5	1.3	355.2	15.7	4.4
69.1	69.6	278.3	2.7	1.0	352.1	13.1	3.7
69.2	138.8	289.7	3.7	1.3	351.4	11.4	3.3
69.2	207.8	286.5	4.6	1.6	339.1	12.8	3.8
104.0	69.6	331.9	5.3	1.6	406.9	13.8	3.4
104.0	104.5	341.8	4.6	1.4	410.0	13.3	3.2
104.0	138.7	348.7	4.7	1.4	412.0	13.7	3.3
104.0	207.7	355.0	5.0	1.4	411.6	13.3	3.2
139.0	69.5	385.5	4.2	1.1	464.5	15.7	3.4
139.0	104.4	394.1	4.5	1.1	466.0	13.9	3.0
139.0	138.7	402.2	5.2	1.3	469.7	14.2	3.0
138.9	276.9	412.5	4.7	1.1	468.2	15.4	3.3

(c) Wet of Optimum

σ_3 (kPa)	σ_d (kPa)	Mr, ends (MPa)			Mr, middle (MPa)		
		Mean	STD	CV	Mean	STD	CV
139.5	104.7	373.4	11.2	3.0	439.1	17.1	3.9
21.0	20.7	149.4	17.7	11.9	200.7	13.6	6.8
21.0	35.0	158.2	19.1	12.1	201.6	8.9	4.4
20.9	51.9	166.2	20.8	12.5	206.7	6.4	3.1
21.0	69.2	171.4	21.4	12.5	210.4	5.5	2.6
34.6	35.1	208.4	25.2	12.1	267.1	10.8	4.1
34.8	69.6	213.8	24.2	11.3	263.0	8.1	3.1
34.8	104.3	212.3	21.8	10.3	253.4	6.9	2.7
34.8	138.0	212.2	19.9	9.4	247.3	9.1	3.7
69.8	35.1	289.2	22.7	7.9	380.0	41.9	11.0
69.8	69.6	296.0	20.1	6.8	361.7	21.3	5.9
69.7	138.8	301.9	17.5	5.8	348.9	12.8	3.7
69.8	207.8	295.3	13.3	4.5	331.6	9.4	2.8
104.7	69.6	343.8	13.5	3.9	416.0	25.1	6.0
104.7	104.6	351.8	13.0	3.7	412.0	19.0	4.6
104.7	138.7	357.5	11.9	3.3	411.1	16.1	3.9
104.7	207.7	361.1	10.6	2.9	404.8	11.5	2.8
139.3	69.5	390.3	10.8	2.8	470.6	34.2	7.3
139.2	104.4	399.8	9.7	2.4	466.6	24.7	5.3
139.4	138.7	407.4	10.2	2.5	467.0	19.6	4.2
139.4	276.9	413.5	10.8	2.6	456.5	12.9	2.8

Note:

σ_3 = Confining Pressure

σ_d = Deviatoric Stress

STD = Standard Deviation in MPa

CV = Coefficient of Variation in percent

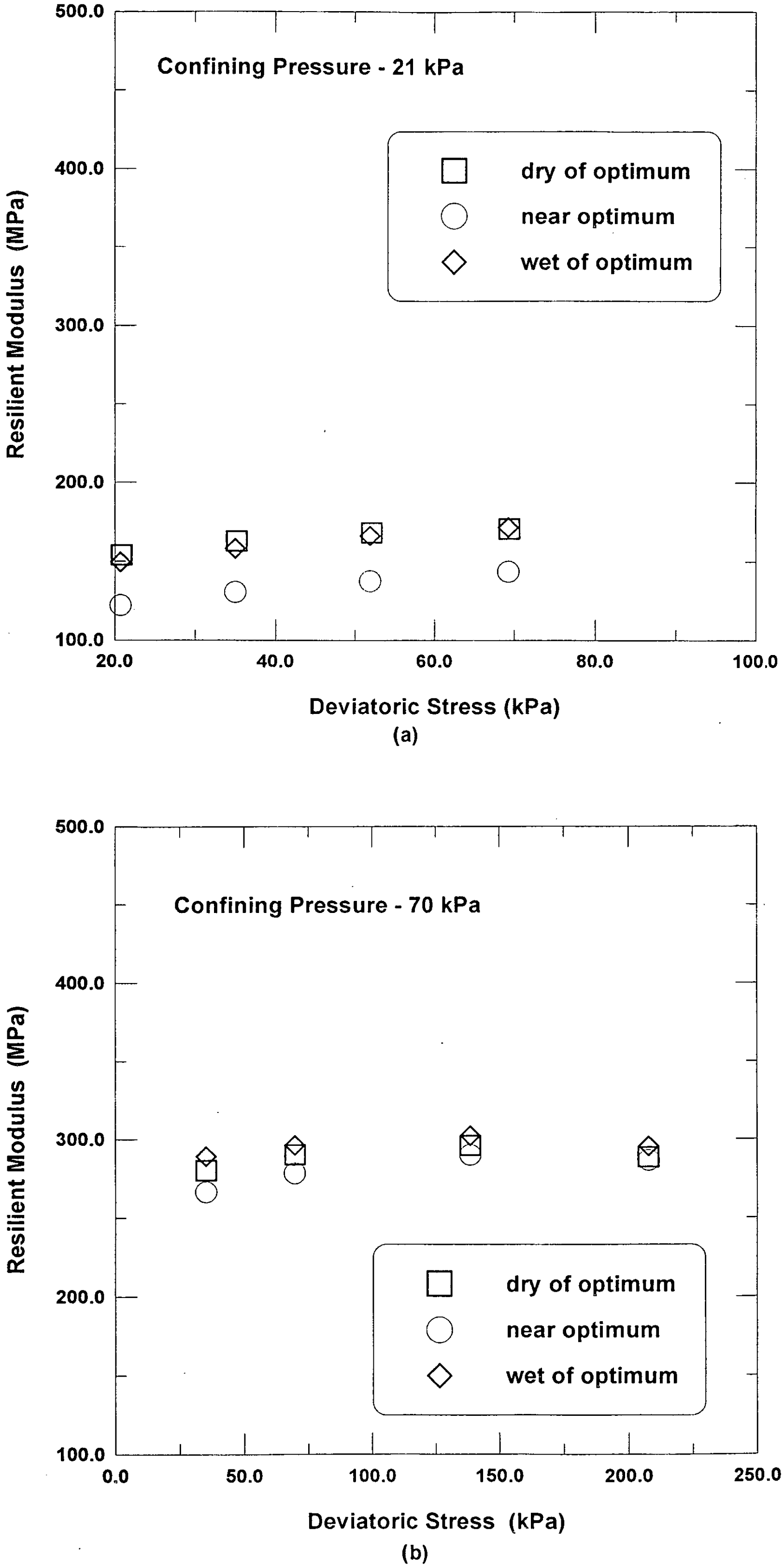


FIGURE 2 Influence of confining stress on resilient modulus of sands at various moisture contents.

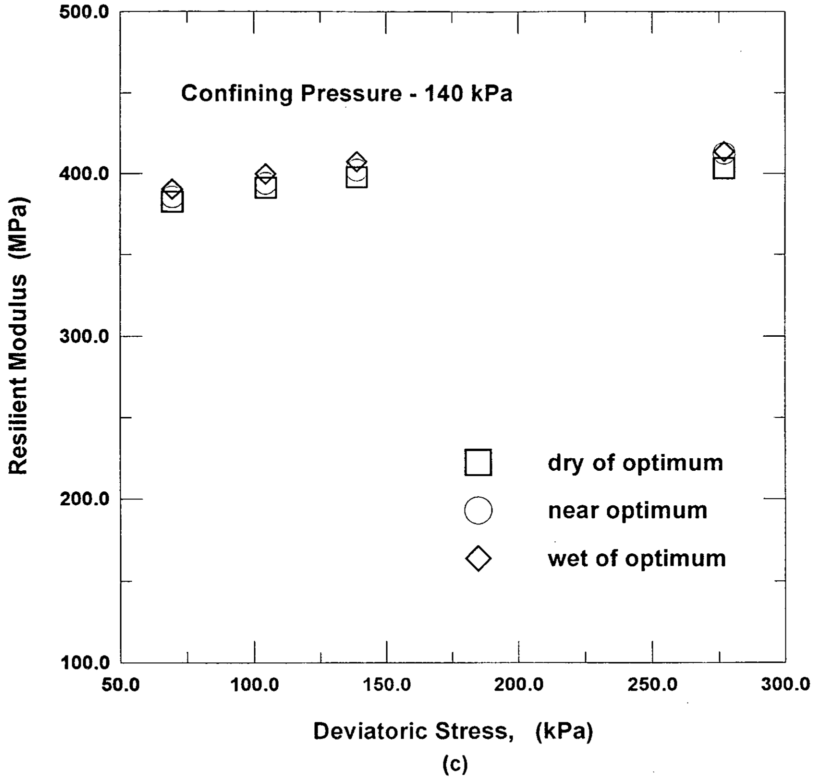


FIGURE 2 (*continued*)

The decrease in M_r at wet of optimum is attributed to the strength decrease at a higher saturation level as a result of smaller cohesion and friction angles and higher pore pressure developments.

An increase in the deviatoric stress resulted in a reduction of moduli values. This observation is consistent with those reported in other studies (*2–4*). This phenomenon is attributed to factors such as positive pore pressure development and fabric changes due to stress cycles. The pore pressures which increase with deviatoric load magnitudes, cycles and saturation levels of the specimens result in the reduction of overall strength. The lower strength specimens provide lower moduli values.

The fabric describes the arrangement of particles, particle groups, and pore spaces in a soil and its influence on mechanical properties of cohesive soils are well documented (*11*). It is assumed that the

TABLE 3 Silty Clay Test Results

(a) Dry of Optimum

σ_3 (kPa)	σ_d (kPa)	Mr, ends (MPa)			Mr, middle (MPa)		
		Mean	STD	CV	Mean	STD	CV
42.1	28.3	302.9	64.7	21.4	380.9	135.8	35.7
42.2	14.0	315.2	63.5	20.2	436.9	157.2	36.0
42.1	28.3	303.8	63.7	21.0	379.1	130.6	34.4
42.2	41.7	286.4	68.5	23.9	340.6	122.3	35.9
42.2	55.7	272.1	72.4	26.6	308.7	117.3	38.0
42.2	69.7	258.6	74.8	28.9	284.9	114.3	40.1
21.2	14.0	276.5	47.6	17.2	406.4	141.4	34.8
21.4	28.3	265.4	54.7	20.6	347.3	124.2	35.8
21.4	41.8	254.2	60.0	23.6	316.5	115.7	36.6
21.4	55.9	244.1	63.5	26.0	294.2	114.0	38.7
21.4	69.9	236.3	67.1	28.4	276.0	111.8	40.5
0.0	13.9	211.1	28.2	13.4	383.9	139.4	36.3
0.0	28.2	199.9	33.4	16.7	326.7	118.8	36.4
0.0	41.7	193.3	39.2	20.3	296.5	112.6	38.0
0.0	55.7	190.2	45.4	23.9	276.2	110.4	40.0
0.0	69.7	190.1	50.0	26.3	259.9	108.9	41.9

(*continued on next page*)

TABLE 3 (*continued*)

(b) Optimum

σ_3	σ_d	Mr, ends (MPa)			Mr, middle (MPa)		
(kPa)	(kPa)	Mean	STD	CV	Mean	STD	CV
42.1	27.4	224.5	25.8	11.5	261.0	48.9	18.8
42.1	13.5	240.0	22.8	9.5	288.5	65.7	22.8
42.1	27.6	226.8	24.9	11.0	261.3	49.1	18.8
42.1	40.7	203.5	26.4	13.0	232.9	43.5	18.7
42.2	54.3	185.6	26.4	14.3	208.1	40.0	19.2
42.2	67.8	171.1	27.0	15.8	187.6	39.3	20.9
21.5	13.5	218.3	19.4	8.9	271.4	56.0	20.6
21.4	27.4	199.9	22.2	11.1	238.8	44.2	18.5
21.3	40.6	182.5	23.4	12.8	215.9	41.5	19.2
21.4	54.3	167.2	24.3	14.5	196.2	39.8	20.3
21.4	67.8	155.6	25.4	16.3	180.0	40.0	22.2
0.0	13.3	178.1	15.9	8.9	254.7	52.3	20.5
0.0	27.1	159.9	18.1	11.3	222.4	44.0	19.8
0.0	40.1	145.1	19.9	13.7	200.5	42.2	21.0
0.0	53.8	134.6	21.1	15.7	182.6	42.0	23.0
0.0	67.3	127.9	22.3	17.4	167.1	41.6	24.9

(c) Wet of Optimum

σ_3	σ_d	Mr, ends (MPa)			Mr, middle (MPa)		
(kPa)	(kPa)	Mean	STD	CV	Mean	STD	CV
42.3	28.1	164.5	15.2	9.3	187.1	21.4	11.4
42.3	14.0	187.3	12.7	6.8	221.3	22.4	10.1
42.3	28.1	166.8	14.2	8.5	189.0	20.5	10.8
42.3	41.3	138.3	17.6	12.7	154.3	22.4	14.5
42.3	54.7	116.3	19.0	16.3	125.8	23.0	18.3
42.3	68.2	100.1	18.8	18.8	104.1	21.8	20.9
21.3	13.9	174.4	13.1	7.5	202.4	25.2	12.5
21.2	28.1	148.6	16.3	11.0	164.2	22.7	13.8
21.2	41.4	123.9	18.6	15.0	134.4	23.9	17.8
21.2	55.0	103.7	19.3	18.6	110.7	23.7	21.4
21.2	68.3	89.4	18.2	20.4	93.7	22.0	23.5
0.0	13.8	144.4	11.1	7.7	180.5	24.9	13.8
0.0	28.0	118.7	13.4	11.3	142.9	23.9	16.7
0.0	41.1	98.0	15.4	15.8	115.3	24.6	21.3
0.0	54.5	83.3	15.8	19.0	95.0	23.1	24.3
0.0	68.0	73.3	15.2	20.8	80.4	20.6	25.7

Note:
σ_3 = Confining Pressure
σ_d = Deviatoric Stress
STD = Standard Deviation in MPa
CV = Coefficient of Variation in percent

fabric at the end of conditioning is far more dispersed (a state at which no face-to-face association in particles exist) than at the beginning of conditioning. The tests which start at the end of conditioning are therefore conducted on specimens with dispersed fabric. The fabric becomes further dispersed with the deviatoric load and the number of cycles. The increasing dispersion results in the decline of resilient properties since this type of orientation generally exhibits lower shear strength components, cohesion and friction angle of soils. Experimental verification of this assumption is beyond the scope of this investigation, but still needs to be assessed.

Measurement Coefficients

Resilient deformations are considerably small and should be measured as accurately as possible by reducing sources of errors in the test. The influence of the location of the LVDT systems in providing accurate measurements was evaluated by using two types of internal measurement systems placed at different locations on the specimen.

The influence of the measurement system is presented in the form of measurement coefficients (*MC*) (*9,10*). The measurement coeffi-

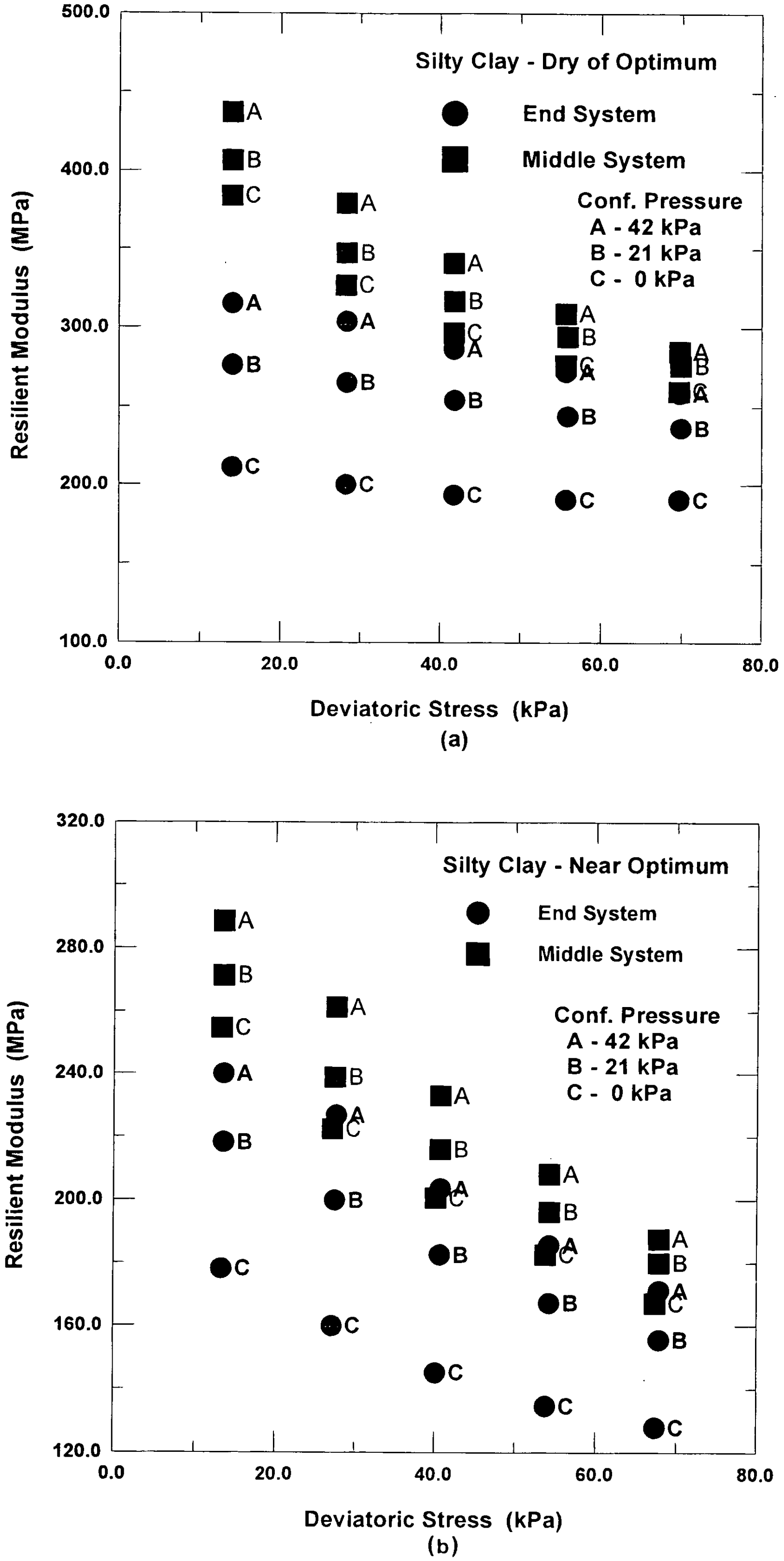


FIGURE 3 Influence of moisture content on resilient modulus test results of silty clay.

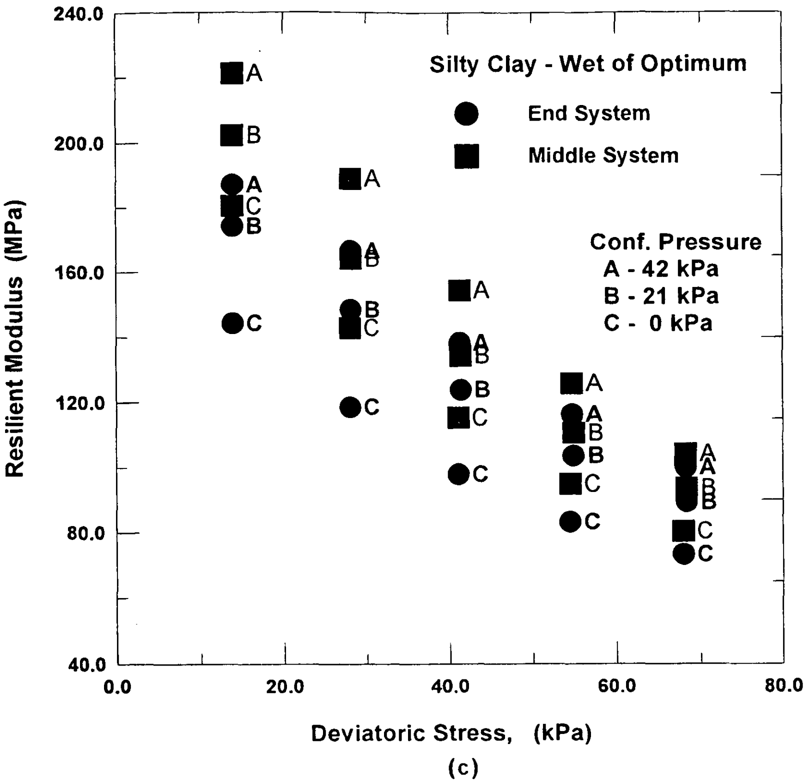


FIGURE 3 (*continued*)

cient is defined as the ratio of the resilient modulus determined by the middle system to that determined by the end system. The coefficient values are determined for various confining and deviatoric stresses. The coefficients can be used to convert the end measurement results to more realistic middle measurement system results. The middle system is assumed as a more appropriate and realistic method since its measurements are not influenced by end friction effects and system compliance errors. Measurement coefficients are calculated for the tests conducted in this study and are presented in Figures 4 (sand) and 5 (silty clay).

Figure 4 shows the relationships between M_r values from the middle measurement system and the end measurement system. The three moisture content levels and their results are plotted in the same figure. The slopes of these lines represent the measurement coefficients. The influence of the stresses on measurement coefficients was not considered in this case since the stresses did not alter the *MC* values considerably. The *MC* values are presumed to depend on the relative stiffness variation between samples and the Plexiglas clamps used for holding middle measurement LVDT clamp system. In other words, a softer sample allows the clamps to slip which induces significant errors into the measurements, while a stiff sample may not permit free movement of the LVDTs of middle system. Therefore, variations in soil stiffness during testing may affect the performance of middle measurement system and thus the measurement coefficients.

The measurement coefficients of sands obtained from Figure 4 are 1.15 (at dry of optimum), 1.22 (at optimum), and 1.18 (at wet of optimum). The differences in *MC* values obtained at various moisture contents are small. This indicates that the moisture content of the sand does not significantly affect the measurement system capabilities. This is attributed to the permeable nature of sands which immediately dissipates the developed pore pressures during testing, thereby keeping the stiffness properties intact during testing.

Figure 5 compares the measurement coefficients of clays at different deviatoric stresses for various confining pressures (0 to 42 kPa or 0 to 6 lb/in.^{2}). The influence of confining pressure appears to be more evident and, therefore, the stresses were included in the analysis to provide expressions for *MC* values. Higher *MC* values were obtained for an unconfined state and low deviatoric stress. The values decreased with an increase in the confining stress. This indicates that higher confining pressures provide better contact between LVDTs and the specimens which allows more precise and accurate measurements.

Linear regression analysis on the clay results provided the following measurement coefficient equations as a function of confining (e_3) and deviatoric stresses (e_d).

$$MC = (0.00335 * \sigma_3 - 0.051)\,\sigma_d + (1.83 - 0.0702 * \sigma_3) \quad (dry) \quad (1)$$

$$MC = (0.00032 * \sigma_3 - 0.013)\,\sigma_d + (1.43 - 0.0402 * \sigma_3) \quad (opt) \quad (2)$$

$$MC = (0.000298 * \sigma_3 - 0.017)\,\sigma_d + (1.26 - 0.0124 * \sigma_3) \quad (wet) \quad (3)$$

Increasing in moisture content levels decreased the *MC* values in the clay test results as opposed to the similar values obtained in the sand tests. Higher variation of *MC* values was obtained in the dry state than at the wet of optimum (Figure 5). This is attributed to pore

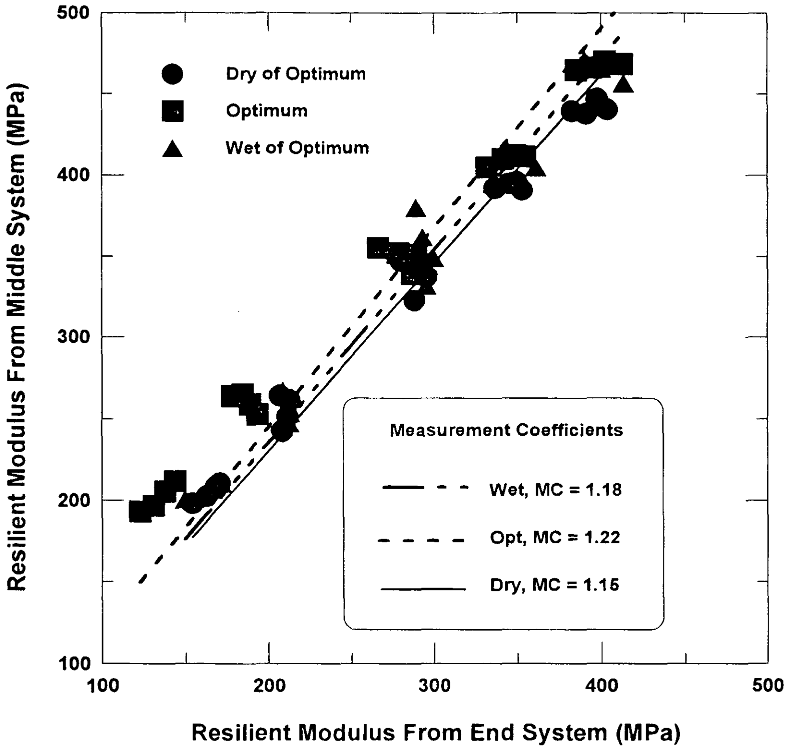


FIGURE 4 Measurement coefficients for sands.

pressure developments and fabric changes in the clay specimen. Although the tests were conducted in drained conditions, semi-drained conditions prevailed during testing due to low hydraulic conductivity of clays. This resulted in the development of pore pressures in the specimen. The wet of optimum state of a soil, which has a higher degree of saturation than at dry and optimum states, produces higher and more uniform pore pressures in the specimen. The uniform distribution of pore pressures may have resulted in uniform measurements at wet of optimum and smaller measurement coefficients.

Also, the fabric at the wet of optimum state is more dispersed structure, whereas the fabric at the dry of optimum state is more of flocculated (*11*). The flocculated structure can undergo greater particle dispersions or orientations than the dispersed structure during repeated loading. Therefore fabric changes in the specimen during testing are more significant at the dry of optimum than at the wet of optimum. Changes in particle orientations at dry level results in variations in measurements and the corresponding higher coefficients.

Plastic Deformation Development

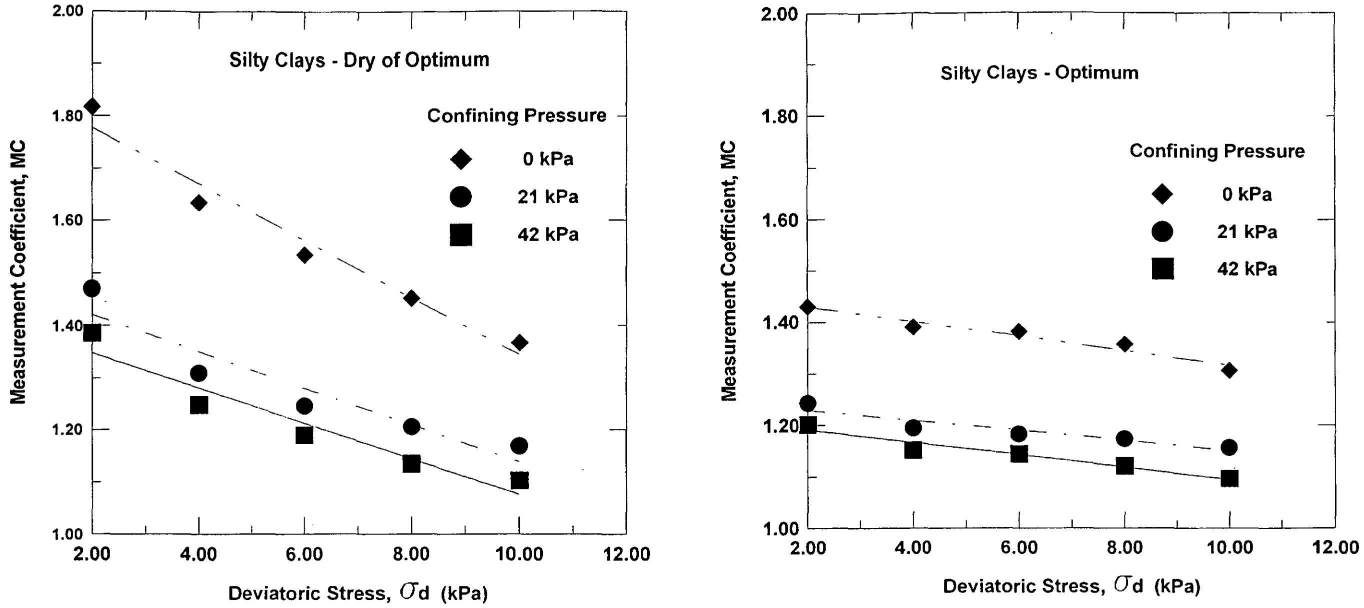
Figure 6 presents the plastic deformations of sands measured by the end system during the conditioning and the testing phases. All three moisture content levels are reported in this figure. The plastic deformations reported in the figure represent the accumulated deformations of one thousand cycles during conditioning and four hundred cycles for each confining stress during testing phase. The testing phase four hundred cycles were obtained by summing the deformations from individual sets of cycles (one hundred each) for four sets of deviatoric loads.

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

For sands, lower plastic deformations were obtained at all testing confining stresses, 21, 70, 105, and 140 kPa with the exception of 35 kPa. This indicates that conditioning not only reduced the plastic deformations in the first test confining pressure (which is 21 kPa), but also in the case of three upper confining stresses, 70, 105, and 140 kPa which are somewhat closer to the conditioning confining stress of 140 kPa. This is a significant finding since no specific guidelines are available in the literature for determining the magnitudes of conditioning confining stresses for granular field core samples based on the plastic deformation criterion. The conditioning confining stress for cores should be greater than the lateral confining pressure of the depth at which the soil samples are retrieved. In cases when the soil sample represents a significant depth of subgrade (either a sample from a deeper depth or samples retrieved from a range of depths), the lateral pressure of the bottom layer of the subgrade or the deepest depth should be used as the confining pressure for conditioning.

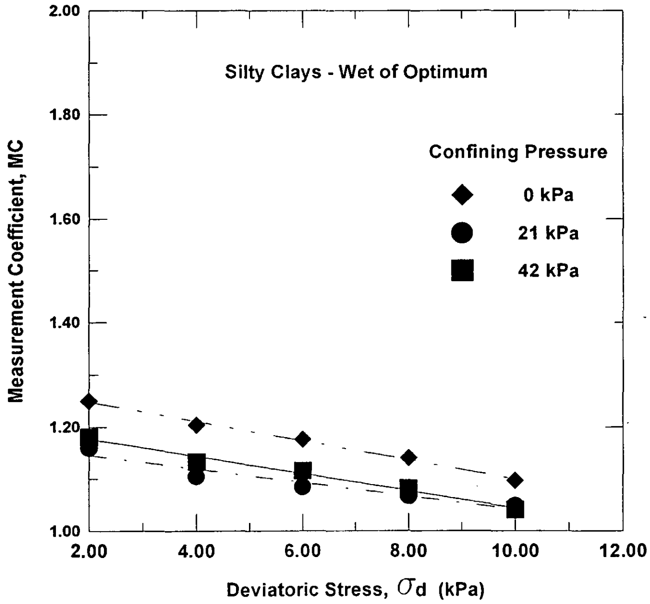
Higher plastic deformations were observed at confining stress of 35 kPa, possibly due to significant fluctuations in the confining pressures in the preceding two stages, 140 kPa (conditioning) and 21 kPa (first level of testing).

Figure 6 also presents the plastic deformations developed during testing of the clay specimens. Results from three moisture



(a) Dry of Optimum

(b) Optimum

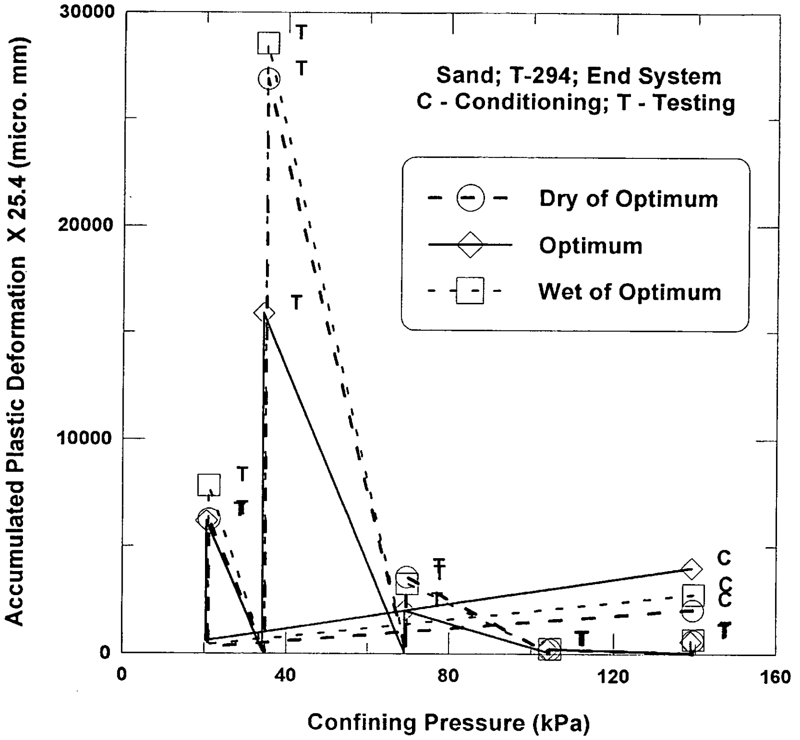


(c) Wet of Optimum

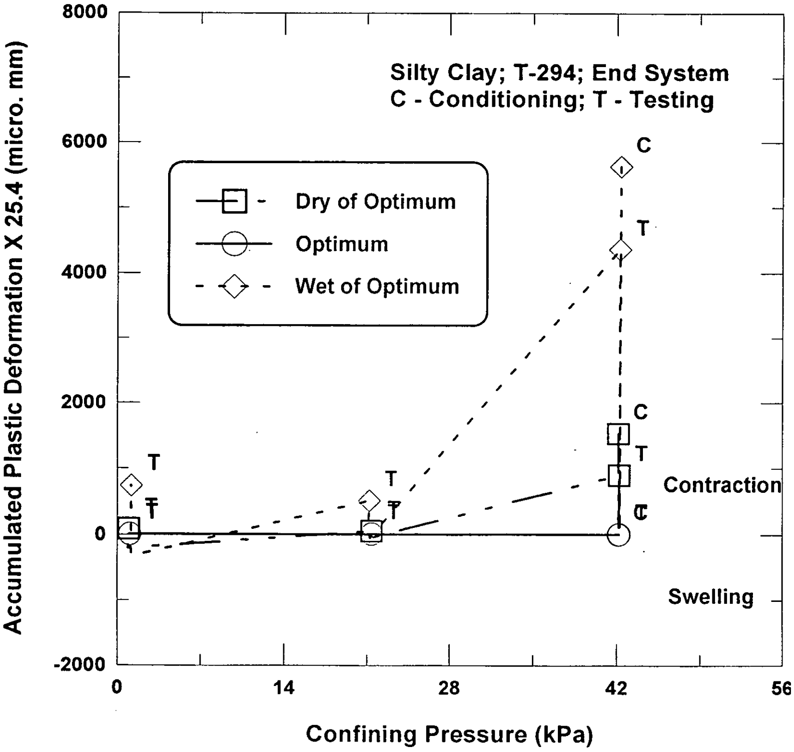
FIGURE 5 Measurement coefficients for silty clay.

contents and densities are depicted in this figure. The clay samples were conditioned at 42 kPa and the first test was conducted at the same confining pressure. This is followed by the testing at other confining pressures of 21 and 0 kPa. The figure suggests that the accumulated plastic deformations were larger at a confining pressure of 42 kPa (6 lb/in.^{2}) and then decreased with decreasing confining stresses. Even though conditioning did not result in the reduction of plastic deformations at first testing confining pressure (42 kPa), it significantly decreased the plastic deformations at other confining pressures (21 and 0 kPa). The purpose of conditioning in clays is probably achieved at confining stresses lower than the conditioning confining stress. Reason for this is attributed to the stiffening or over-consolidation of the specimen at a conditioning confining pressure of 42 kPa. The stiffened specimen, therefore, appears to induce smaller strains at lower confining pressures. This implies that the field cohesive core samples require a conditioning confining stress that is significantly higher than the lateral confining pressure corresponding to the retrieval depth location.

Smaller plastic deformations were measured by the middle system than the end system, possibly due to the differences in the lengths of the specimens that these systems were accounted for.



(a) Blasting Sand



(b) Silty Clay

FIGURE 6 Accumulated plastic deformations versus confining stresses for sands.

Regression Model Analysis

Regression models are generally used in the form of equations for predicting the moduli values. Theta (θ) or bulk stress and deviatoric stress are used in the models as predictors based on whether the soil is cohesionless or cohesive. These models are recommended in AASHTO procedures:

$$Mr = k_1 * \theta^{k2} \quad \text{for grandular soils} \tag{4}$$

$$Mr = k_3 * \sigma^{k4} \quad \text{for cohesive soils} \tag{5}$$

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

The regression coefficients are determined from the test results for both soils and are reported in Table 4a and b (sands and silty clays). Typical model results for sands and clays at wet of optimum are depicted in Figure 7. It is interesting to note that k_2 and k_4, which represent the slopes of the lines in the respective models, appear to be dependent on the type of soil and the moisture content level. The constants k_1 and k_3, which represent the intercepts in the figures, depend on the measurement systems, the moisture contents and density levels. As expected, higher k_1 and k_3 values are obtained for the middle system because of higher resilient moduli measurements. Although these constants varied with moisture content, no particular trend is observed in both soils.

SUMMARY AND CONCLUSIONS

The resilient modulus test results provided the following conclusions:

1. Sands exhibited higher resilient moduli at dry and wet of optimum than at optimum moisture content. Higher M_r values at dry of optimum over optimum is attributed to the higher strengths. The same at wet of optimum, on the other hand, did not follow a consistent trend possibly due to leakage problems associated during the wet of optimum tests. Overall, the statistical variation between the results at all three moisture content levels is insignificant, probably due to very small range of relative compactions used and lesser influence of moisture contents in sands.
2. The moduli values of clays decrease with an increase in moisture content. This is attributed to the increase in positive pore pressure development with an increase in moisture content or degree of saturation. Higher pore pressures decrease the effective stresses and the shear strength of the clay specimens, thereby resulting in smaller resilient moduli.
3. The measurement coefficients of the sand tested are 1.15 (dry of optimum), 1.22 (optimum), and 1.18 (wet of optimum). The small variations in these coefficients indicate that moisture contents in sands did not influence the measurement systems.
4. The measurement coefficients of clays for each moisture content level are expressed as a function of confining and deviatoric stresses. Higher values are produced for the dry of optimum moisture content level. Lesser fabric changes and uniform pore pressure developments at wet of optimum may have resulted in smaller variations between the end and middle measurements.
5. Conditioning resulted in smaller plastic deformations at most of the confining stress levels for sands and at confining stresses lower than the conditioning confining stress for clays.
6. The regression model constants appear to depend on the moisture content, density levels, and the measurement system. Though the model constants varied with respect to moisture contents, no particular or significant trends between them are noticed.

ACKNOWLEDGMENTS

This work was supported by the Louisiana Transportation Research Center in cooperation with the Louisiana Department of Transportation and Development (LADOTD) state project no. 92-2GT. The authors express their appreciation for this support.

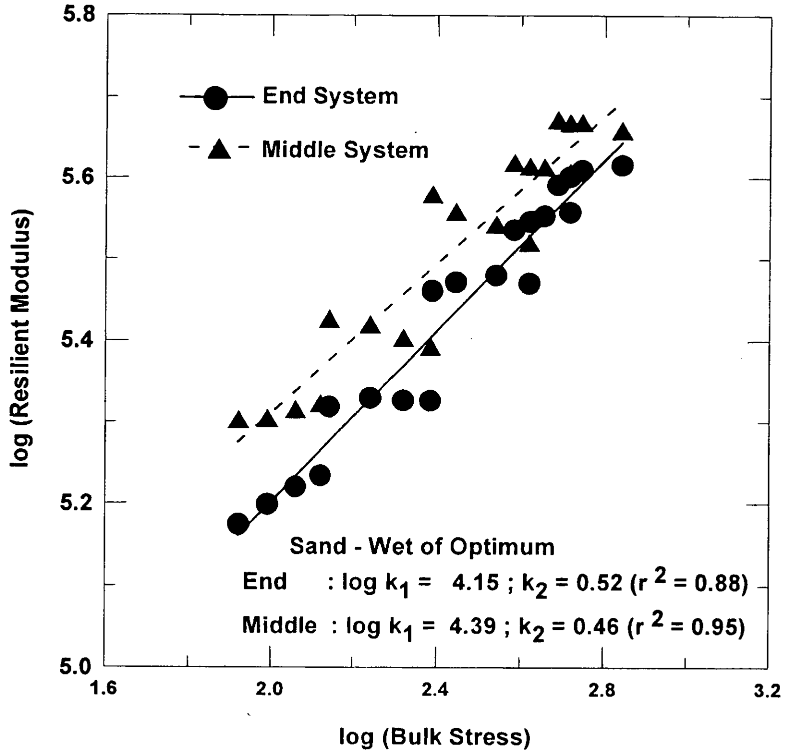
TABLE 4 Regression Constants for Sand and Silty Clay

(a) Sand

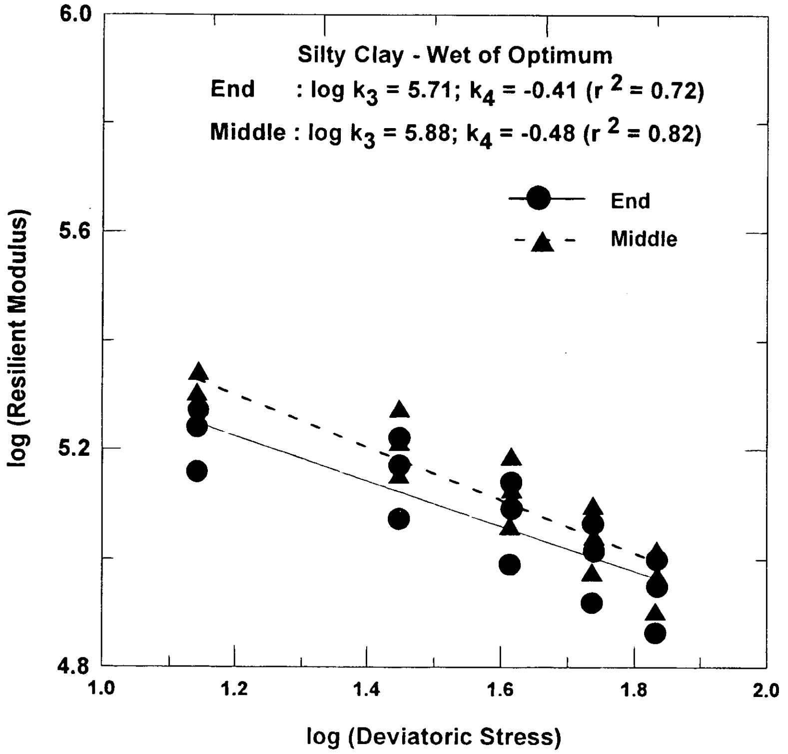
Moisture Content (%)	Dry Density (pcf)	End System			Middle System		
		$\log(k_1)$	k_2	R^2	$\log(k_1)$	k_2	R^2
9.67	105.50	4.44	0.43	0.90	4.20	0.50	0.94
11.92	107.65	4.38	0.48	0.92	4.91	0.63	0.96
13.50	106.65	4.39	0.46	0.88	4.15	0.52	0.95

(b) Silty Clay

Moisture Content (%)	Dry Density (pcf)	End System			Middle System		
		$\log(k_3)$	k_4	R^2	$\log(k_3)$	k_4	R^2
18.0	96.5	5.83	-0.09	0.88	6.67	-0.24	0.78
20.6	101.6	5.90	-0.21	0.84	6.29	-0.25	0.50
23.0	96.5	6.25	-0.41	0.82	6.63	-0.48	0.76



(a) Blasting Sand



(b) Silty Clay

FIGURE 7 Typical regression model results.

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Publication of this report sponsored by Committee on Soil and Rock Properties.