EVALUATION OF RESILIENT MODULUS AND CORRELATION WITH ENGINEERING PROPERTIES FOR COMPACTED OKLAHOMA FINE-GRAINED SOILS

By

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Submitted to the Faculty of the Graduate College
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in partial fulfillment of the requirements
for the Degree of
DOCTOR OF PHILOSOPHY
July, 1984

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ACKNOWLEDGMENTS

The author takes this opportunity to express his sincere appreciation an gratitude to Dr. D. R. Snethen, Thesis Adviser and Chairman of the Advisory Committee, for his invaluable counsel, guidance, and encouragement prior to an during this research program.

The author is grateful to Drs. J. V. Parcher, V. A. Mast, and L. W. Reed, Committee Members, for their excellent instruction, advice, and interest prior to and during this research study.

The author is grateful to his fiancee, Mu Lan Zee, for her love and patience during his studies.

The author expresses his gratitude to the Department of Civil Engineering for the financial support and assistance through his Ph.D. program; and to Dr. P. G. Manke for support during the Summer, 1983.

Sincere appreciation is extended to Mr. Jim Nevels, Oklahoma Department of Transportation, for providing information which was used in this research study and to Mr. Cecil Sharp for his assistance in building the laboratory equipment.

IN MEMORY OF MY FATHER,

CHANG JUI,

for inspiring self motivation and confidence

and

TO MY MOTHER,

HSING SHOU-NAN,

for her devotion, understanding and encouragement during this study

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CHAPTER I

INTRODUCTION

Asphalt concrete pavement should be designed so that the thickness of pavement structure is sufficient to prevent not only excessive permanent deformation, but cracking of the asphalt concrete surface. A significant factor influencing the design thickness is subgrade support. Flexural fatigue cracking of the asphalt concrete is recognized as a significant mechanism contributing to the failure of otherwise well-designed pavements that exhibit minimal permanent deformations. Deformations that produce this fatigue cracking are essentially elastic and almost completely recoverable.

Resilient Modulus, M_R, is an elastic rebound modulus that is a measure of pavement materials recoverable elastic response to repeated loading. Design of a pavement structure on a poorly resilient soil (small M_R) may require resilient modulus testing and special pavement design procedure to minimize the possibility of failure of the pavement.

Although M_R testing provides the best way to understand the subgrade support condition, some other simple properties are widely used for pavement thickness design. One of these properties, Oklahoma Subgrade Index (OSI), will be evaluated in the thesis.

The objective of this research project is to evaluate the Resilient Modulus and correlate it with the Oklahoma Subgrade Index for typical Oklahoma soils. The factors involved in the M_R testing procedure and specimen preparation will be investigated for their influence on M_R. It is also the intent of this thesis to provide correlation between the results of the Asphalt Institute M_R-based thickness design procedure and the OSI method to determine a basis of comparison in order to develop a guide for design analysis of flexible pavement design.

CHAPTER II

LITERATURE REVIEW

Introduction

About thirty years ago, pavement fatigue failure was considered to be caused by permanent deformation of the subgrade soil. Present research (1) has shown that it is not sufficient to evaluate only the resistance to permanent or plastic deformation of the subgrade. Numerous investigations conducted by state transportation agencies have shown a close correlation between observations of cracking and fatigue-type failures in asphalt pavements and the measured deflections of these pavements due to passing wheel loads. In other words, resilient deformation is the primary factor causing pavement failure (2).

Most pavement designs are based on soil strength or resistance to deformation determined by some type of test in which the total load is slowly applied over a period of several minutes. Unfortunately, this does not simulate the real traffic loading conditions. Some researchers (2, 3, 4, 5, 6, 7, 8, 9) have discussed this misconception and have suggested that a 0.1 second to 0.3 second loading period would be more appropriate to evaluate the resilient properties of subgrade soil.

Resilient Testing

Several test devices have been developed for measuring soil resilient behavior. Typical equipment for measuring M_R includes trixial test equipment, repeated load application equipment, and repeated deformation measurement equipment.

The term, Resilient Modulus was introduced by Hveem (1) and defined as the ratio of the repeated axial deviator stress, σ_d, to the recoverable axial strain, ε_a.

$$M_R = \frac{\sigma_d}{\varepsilon_a}$$

The test may be conducted on all types of pavement materials ranging from cohesive soils to stabilized materials. However, test conditions (e.g., stress state, number of stress application) affect M_R responses for different materials in different ways.

Factors Affecting the M_R

Fine-Grained Soil

Extensive laboratory studies of the behavior of fine-grained soils under repeated-load testing have been conducted by Seed and others (2, 3, 4, 5, 6, 7, 8, 9). An investigation of Illinois soils under repeated load (unconfined) was conducted by Thompson and Robnett (10). Their investigations cover a wide range of soil types and conditions. The factors influencing the resilient behavior of fine-grained soils under repeated loading can be described as:

1. Method of Compaction. Different compaction methods produce different soil particle arrangements that result in variation of the M_R. The influence of compaction method is shown in Figure 1 (9). Kneading compaction on the wet side of optimum produces samples that exhibit large resilient axial deformation. However, samples compacted wet of optimum

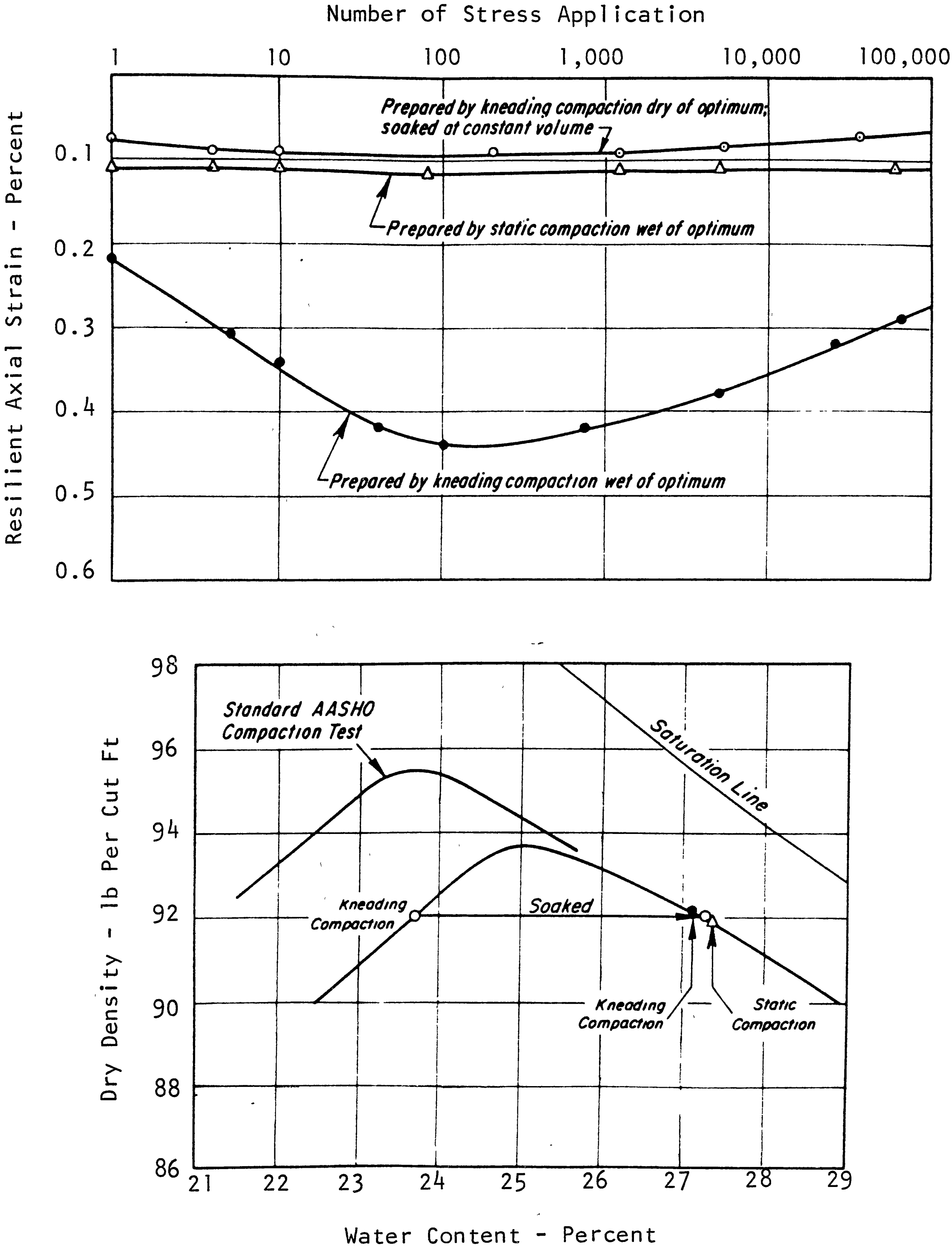


Figure 1. Comparison of Resilient Characteristics of Specimens Prepared by Static Compaction "Wet of Optimum" and by Kneading Compaction "Dry of Optimum" and Soaked to the Same Degree of Saturation (9)

using static compaction have resilient properties similar to those compacted on the dry side of optimum by kneading compaction and soaked to a similar degree of saturation.

2. Number of Stress Applications. Extensive studies (9) have shown that resilient deformation generally decreases as the number of load repetitions increases. This is because the soil mass is increasingly densified under repeated loadings.

3. Age at Initial loading. Compacted samples with high degrees of saturation increase in strength with time (2). The studies pointed out that thixotropic strength gain is not significant when good mixing procedures are used with statically compacted samples. The resilient strain determined for a small number of stress applications decreases as the time interval between compaction and testing increases.

4. Stress Intensity. Because traffic loading is not uniform, the influence of intensity of stress is important. Various researchers indicate that the M_R increases as the intensity of stress decreases. Typical results are shown in Figures 2 and 3 (9, 10).

5. Changes in Density and Water Content After Compaction. Seed et al. (9) show that as the water content of the soil increases (or moist density increases) the resilience increases and M_R decreases. However, there was a poor correlation between water content and M_R. M_R is more nearly a function of the degree of saturation, as shown in Figure 4.

Granular Materials

Only repeated-load triaxial tests can be used for this type of material. Granular materials need confining pressure to hold the molded sample in place during the test.

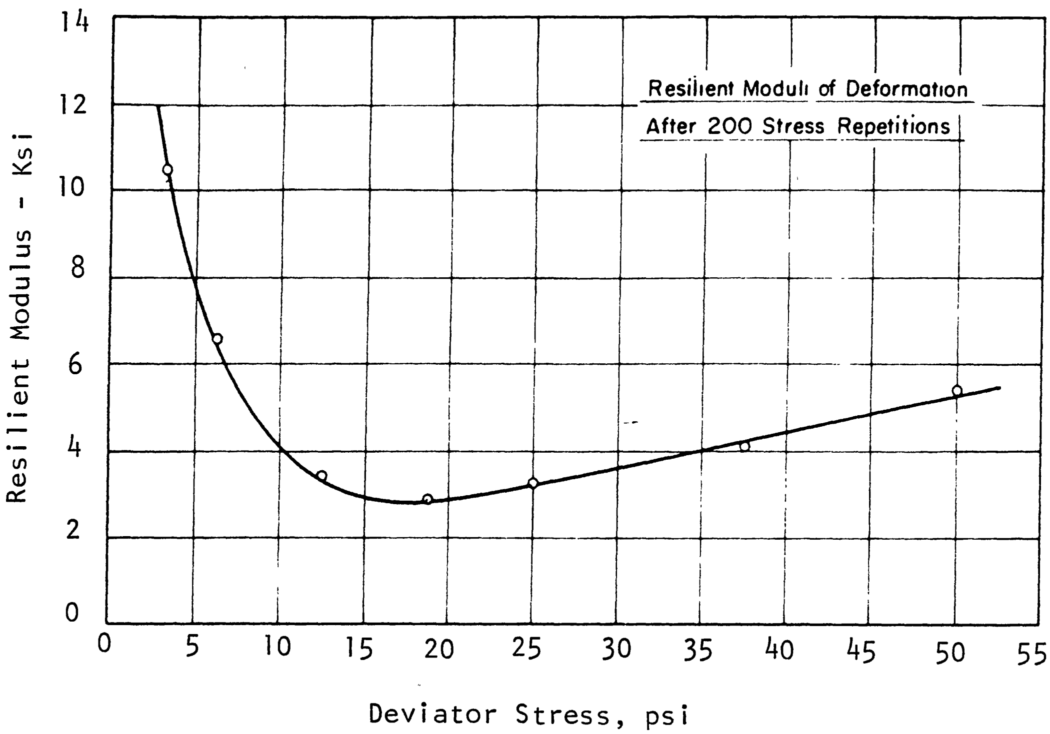


Figure 2. Effect of Stress Intensity on Resilience Characteristics; AASHO Road Test Subgrade Soil (9)

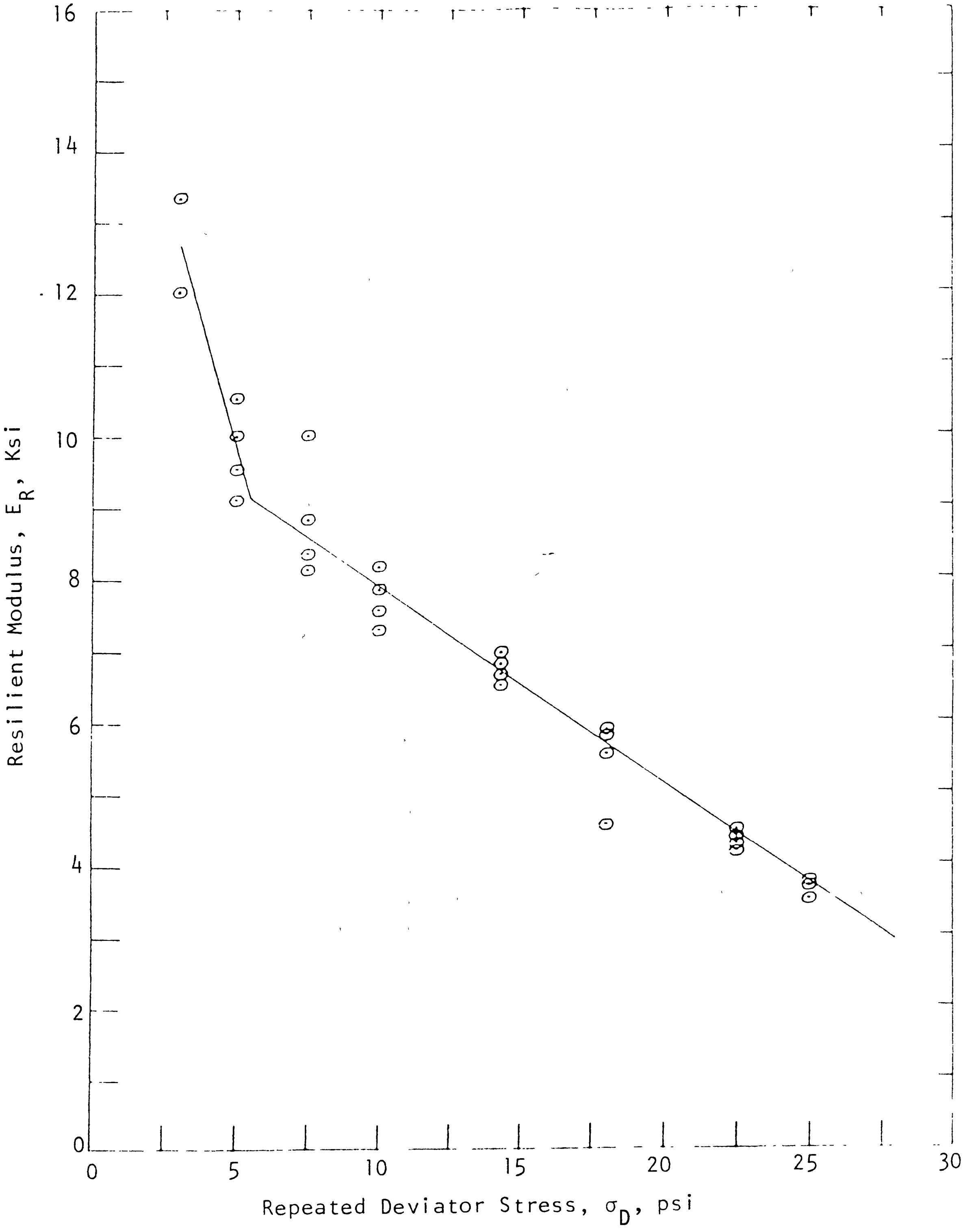


Figure 3. Effect of Stress Intensity on Resilience Characteristics (10)

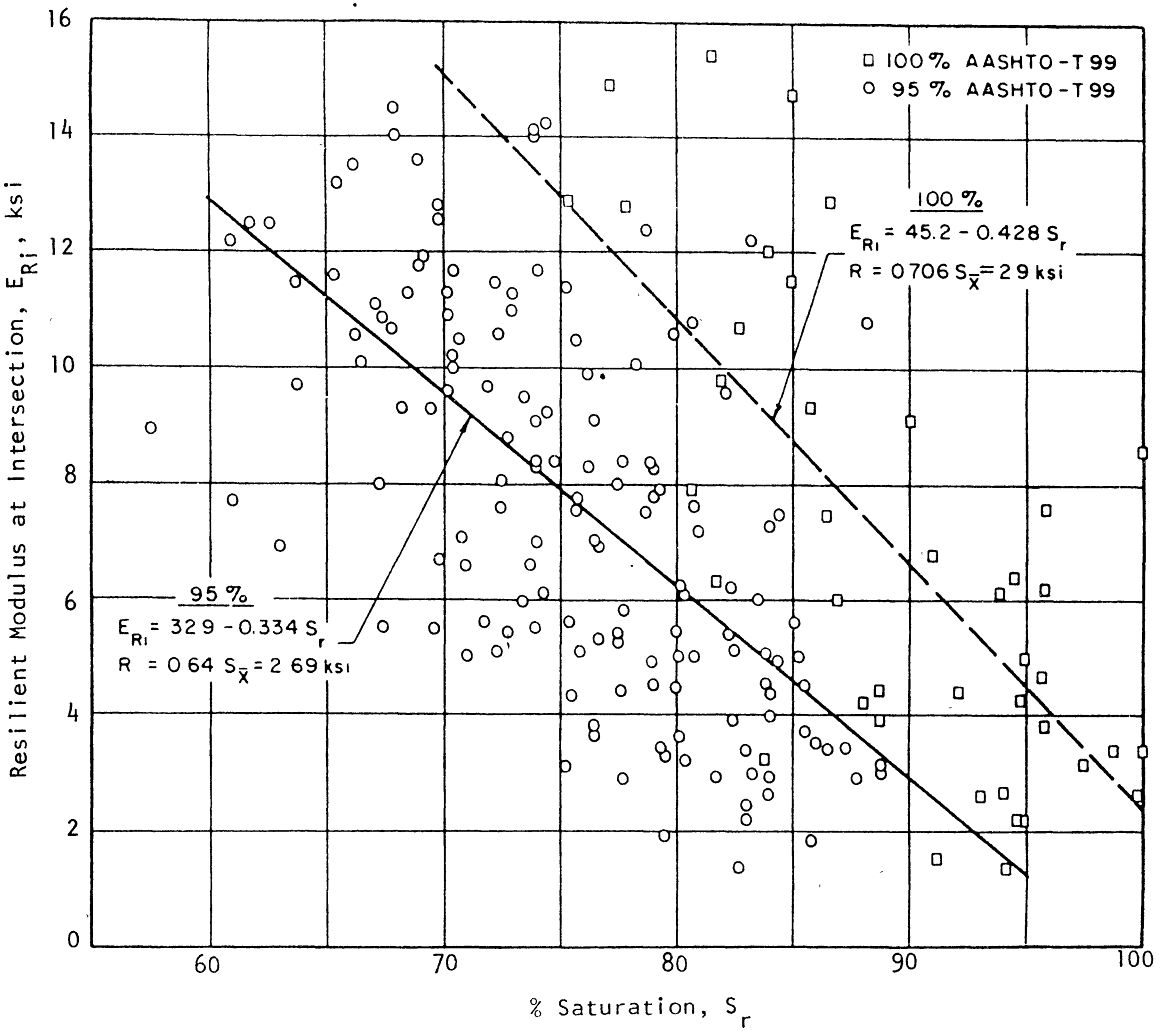


Figure 4. Resilient Modulus - % Saturation Relations for 95% and 100% Compaction (10)

Various investigations show a wide range of values for M_R. The factors which contribute to this variation are:

1. Duration of stress application and rate of deformation. Studies on silty sand and dry sand (11) indicated that the M_R increases with an increase in the duration of load application, but in spite of the large numbers of values investigated the change in the magnitude of M_R was relatively small.

2. Frequency of load application. The results from the studies (12) show that the higher the frequency the higher the modulus. This increases range from 50 to 100 percent, depending on water content and dry density.

3. Void Ratio. M_R can vary as much as 50% (13) for various void ratios (i.e., loose to dense state).

4. Type of aggregate and percentage of material passing the No. 200 sieve. The fines content (passing No. 200 sieve) influences the resilient property (14). Data shown in Table I indicate the changes of modulus caused by the various fines content. These data also present the effects of aggregate type on the modulus.

5. Degree of saturation. From Table I, regardless of aggregate type and fines content, M_R decreases with increasing degree of saturation.

6. Confining pressure. The significant influence of this factor on M_R has been investigated and noted in numerous studies. Typical testing results shown in Figure 5 (15) indicate a significant decrease in the M_R with decrease in the confining pressure.

The determination of an appropriate M_R value for cohesive (or cohesionless) soil is not a simple task, because it is necessary to

TABLE I

THE RESILIENT MODULUS WITH VARYING OF FINE CONTENT AND DEGREE OF SATURATION (14)

Material Tested	Percent Passing No. 200	Rebound Modulus (psi)		
		70% Sat.	80% Sat.	90% Sat.
Gravel	6.2	56,000	46,500	34,000
	9.1	---	40,000	31,000
	11.5	57,500	45,000	37,000
Crushed Rock	6.2	42,000	39,000	---
	9.1	39,000	29,000	---
	11.5	39,500	33,500	---

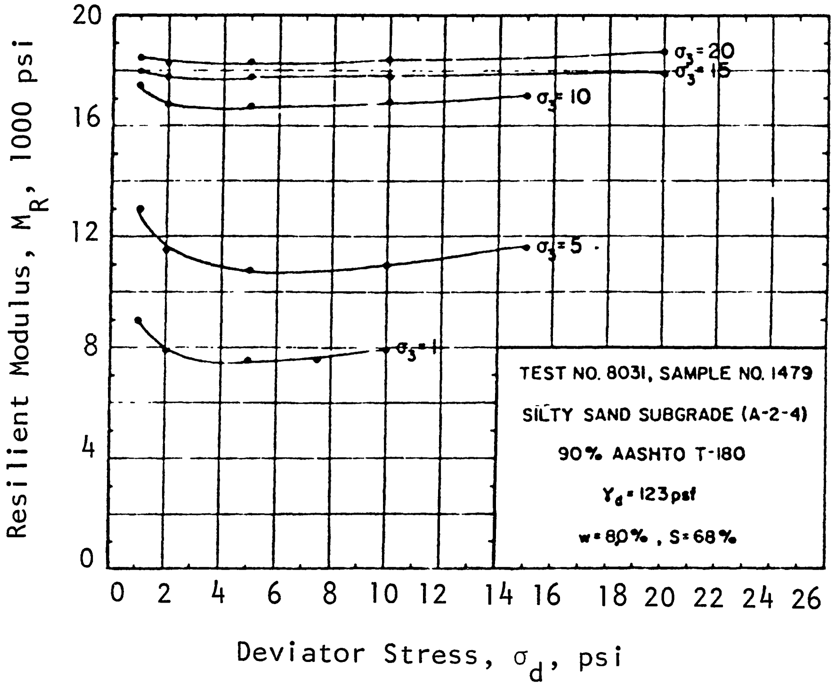


Figure 5. Arithmetic Plot of Resilient Modulus Test Results (15)

select the modulus data giving consideration to all the previously noted factors. But it is not as complex as it may appear. Using an appropriate method of sample preparation and specimen conditioning, it is possible to simulate closely in the laboratory any desired field condition for a soil.

M_R Used for Flexible Pavement Design Procedures

Several flexible pavement thickness design procedures have been developed in recent years. All of these procedures were based on subgrade soil support properties, such as CBR, R-value, or plate-bearing test results. Seed and others recently found that use of the M_R in thickness design is much to be preferred. Thus, correlations between the M_R and other properties such as the CBR, etc., have been developed for use in design.

Correlation Between M_R and Other Properties

The AASHTO Interim Guide is one of the most frequently used pavement design procedures. The assumed subgrade support value varies from 3.0, which represents a silty clay road bed, to 10.0 which represents crushed rock base material used on the AASHTO Road Test (16). The support value for other types of materials lies between these extremes. Soil support values have been proposed by several researchers. Van Til et al. (17) developed a relationship between soil support value and the resilient modulus of subgrade soil. Using 3,000 psi as the modulus of the subgrade soil at the AASHTO Road Test, a relationship between modulus and soil support was developed. The relationship is summarized in Figure 6. The correlation with other soil support properties is also shown in Figure 6.

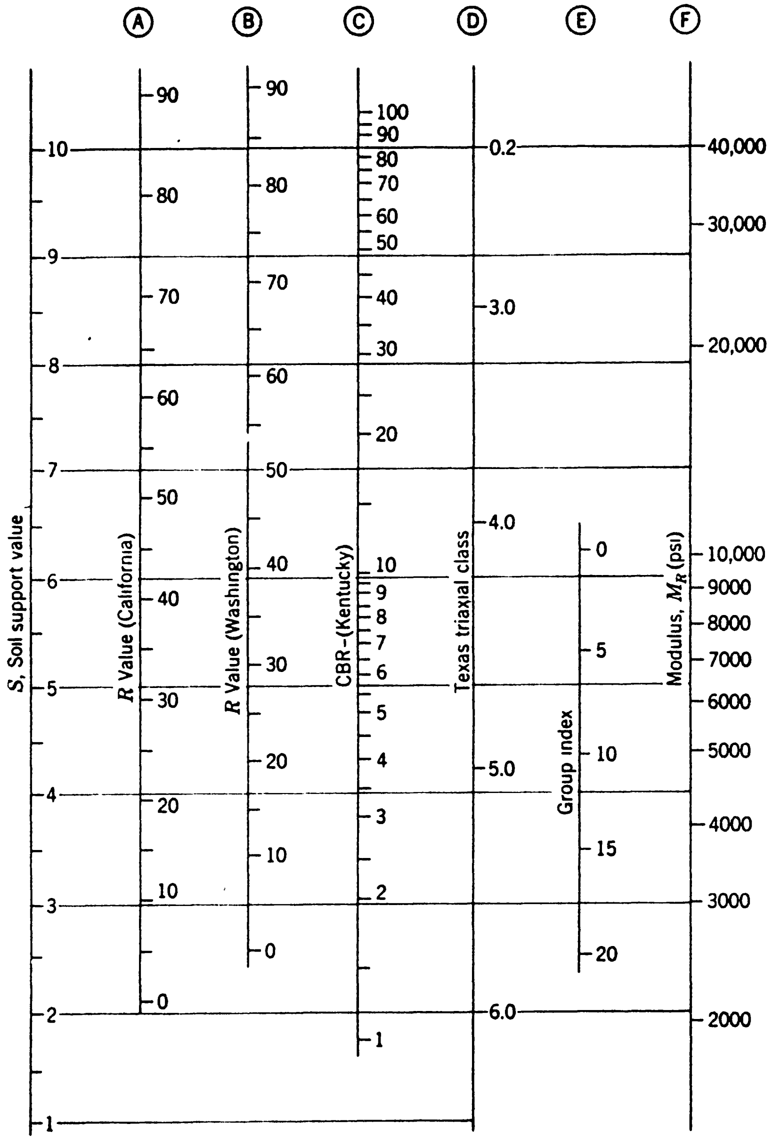


Figure 6. Correlation Chart for Estimating Soil Support(s), (17)

Following the development of the correlation chart, the M_R was first applied for pavement design. In the Van Til et al., study, no direct relationships were established between M_R and other soil support properties. Correlations of the other soil support properties were used for checking the validity of the soil support scale. As Seed and others pointed out, the M_R is the most significant factor affecting the performance of the pavement structure, and the other soil support properties may not represent the same value of M_R that Van Til et al., recommended for the soil support value.

Recent M_R-Based Design Procedure

Most pavement thickness design procedures are based on AASHTO Road Test results, a basis that poses significant limitations. Major limitations (17) include the following items.

1. There was a single subgrade soil and no variation in compaction or other conditions.

2. There was no variation in the base or subbase materials.

3. There was no direct provision made for different environmental conditions.

4. Annual rainfall for the area may not be representative of other locales.

These limitations restrict the use of many current thickness design procedures. For that reason, the mechanistic and empirical state-of-the-art information incorporated in the Asphalt Institute method provides a useful and reliable design procedure (18). In this most recent design procedure, the new principles involved may be described as follows (18):

1. The M_R of the subgrade soil is used directly in the procedure. The variation of M_R under the frozen, normal and saturated conditions was taken into consideration.

2. The M_R under various conditions (frozen, normal, and saturated) was taken into account for the materials used in the pavement structure (pavement, base and subbase).

3. More reliable environmental information was used including the local monthly averages of temperature and rainfall.

The Ninth Edition of MS-1 is believed to be as complete as circumstances will permit. This procedure will be used as a standard to check the validity of the local thickness design procedure (Oklahoma Subgrade Index, OSI) in the analysis chapter of this thesis.

Oklahoma Subgrade Index Pavement Thickness Design

Because of the variation of the soil strength properties found under various circumstances (i.e., climate, temperature, rainfall, etc.), the Oklahoma Department of Transportation developed a pavement thickness design procedure based on basic soil properties. The Oklahoma Subgrade Index is determined from selected soil properties (liquid limit, plastic index, and percent passing No. 200 sieve) (19).

Determination of OSI

The OSI is calculated from the liquid limit, plastic index, and percent passing No. 200 sieve. The procedures for calculation are shown in Figure 7 (19).

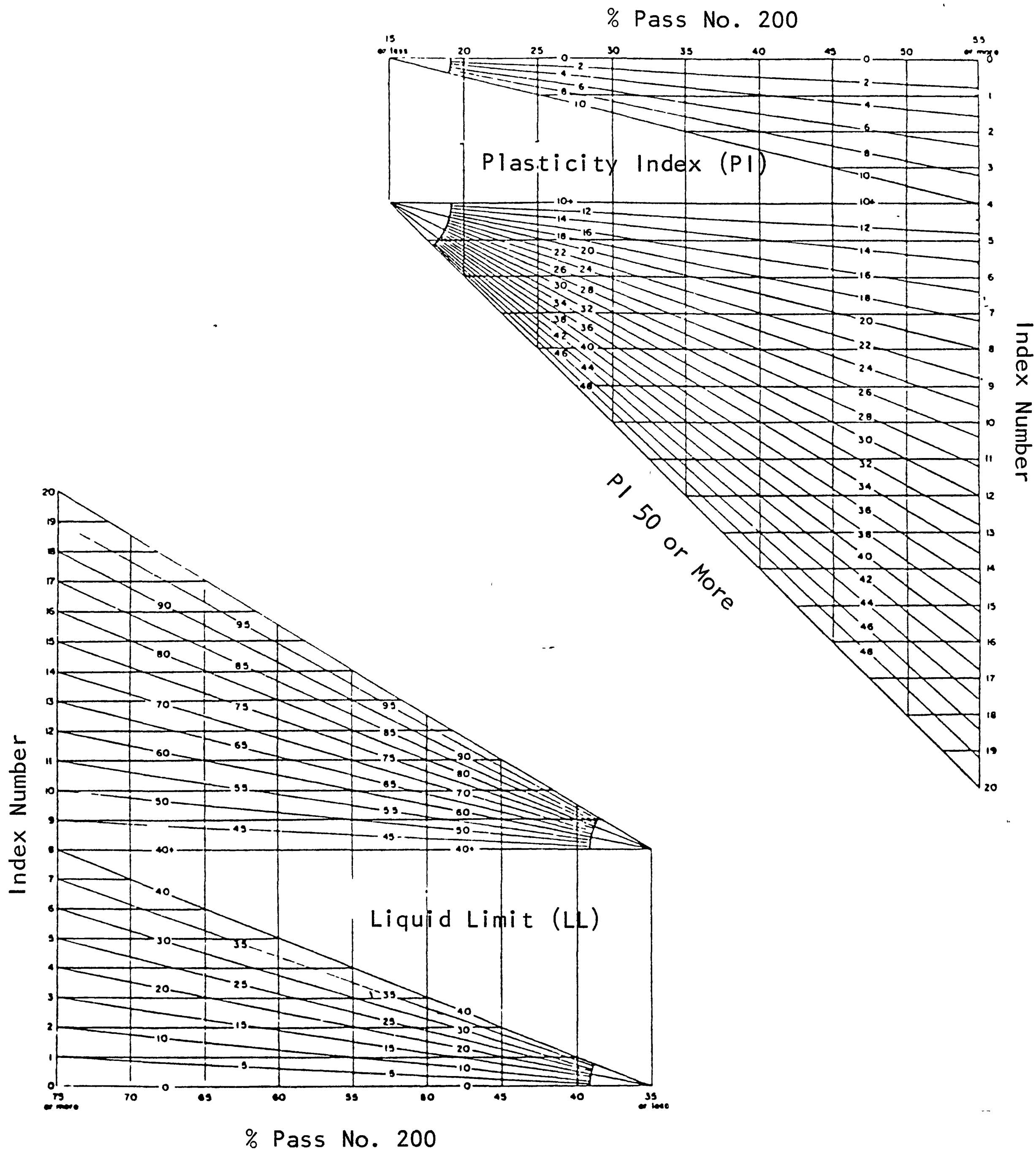


Figure 7. Oklahoma Subgrade Index Numbers Chart (19)

Determination of Equivalent Base Thickness (EBT)

The required equivalent base thickness is determined from the O.S.I. of the subgrade soil (see Figure 7). O.S.I. and wheel load are combined in Figure 8.

The Oklahoma design charts are based on equivalent base thickness (EBT). In using materials of different quality, the following conversions are used (19):

1" of Asphalt Concrete = 1½" of EBT

1" of Aggregate Base = 1" of EBT

1" of FABB (Fine Aggregate Bituminous Base) = 1" of EBT

1" of CABB (Coarse Aggregate Bituminous Base) = 1.25" of EBT

1" of UCAB (Untreated Coarse Aggregate Base) = 1" of EBT

1" of Soil Asphalt Base = 1" of EBT

1" of Cement Treated Base = 1" of EBT

1" of Subbase (Type I, II, or III) = ½" of EBT

1" of Subbase (Type IV) = 3/4" of EBT

1" of Lime Treated Subbase (6" Treatment) = ½" - 3/4" EBT

Other factors which affect the performance of the pavement are considered in the procedure. The equivalent base thickness may be adjusted for these factors (traffic, shoulder, and climate). The additional 2" EBT is used for this adjustment.

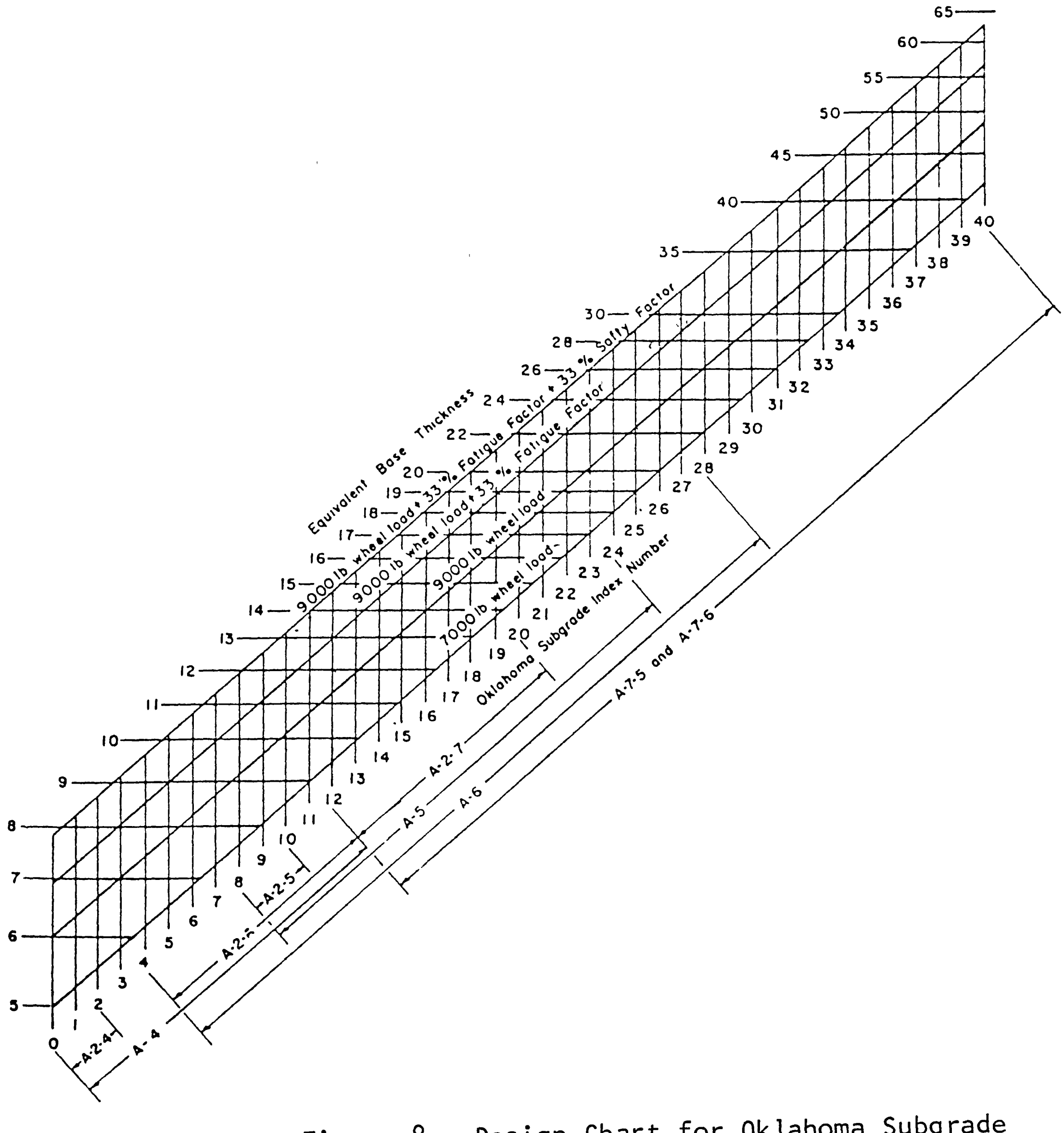


Figure 8. Design Chart for Oklahoma Subgrade Index Number (19)

CHAPTER III

THE RESEARCH PROGRAM

Introduction

In the Oklahoma Subgrade Index pavement thickness design procedure, there are some factors that have not been considered in the design criteria. They are:

1. The resilient subgrade property under repeated traffic load.
2. The climatic variation of subgrade properties.
3. The traffic volume (number of load repetitions) and its growth during the design period.

The resiliency of the subgrade is a significant factor affecting pavement performance. This research program was developed for identying problems associated with use of the M_R in pavement design.

The research involves determination of the M_R of compacted fine-grained soils and development of correlations between M_R and the OSI number of the soils. The first part of the research deals with the evaluation of selected engineering properties, including plasticity, specific gravity, and particle size distribution. The second part deals with the procedure used for resilient modulus testing and determination of the M_R. The third part involves correlation of the M_R with the OSI number and evaluation of the OSI design procedure based on selected conditions.

Sampling Program

The soils used for this study were selected on the basis of pedological classifications that represent typical Oklahoma fine-grained soils. The selected soil series were: Port, Renfrow, Lela, Kirkland, Norge, and Miller. Disturbed samples were obtained for each soil at the sampling sites described in Table II.

Testing Program

The testing program for this study consists of two parts. The first part deals with the determination of the basic engineering properties. The second part deals with the determination of the M_R.

Testing Program for Determining Engineering Properties

Atterberg Limits: Following air drying, grinding and sieving past the #40 sieve, the samples were mixed with appropriate quantities of distilled water and allowed to equilibrate for 24 hours. Testing procedure for the liquid limit followed the ASTM Designation D423-60. The plastic limit determination followed the ASTM Designation D424-59.

Specific Gravity: Appropriate quantities of the sample were soaked in distilled water for at least 24 hours. A calibrated 500 ml-flask was utilized during determination of the specific gravity of each sample. Testing procedures for specific gravity followed ASTM D854-58.

Sieve Analysis and Hydrometer Analysis: Particle size distribution was determined by means of a sieve analysis. Standard U.S. sieves (No. 10, 20, 40, 100, 200) were used for the sieve analysis. The appropriate amount of sample was soaked in distilled water at least 24 hours then

TABLE II

RESILIENT MODULUS PROJECT SAMPLING SITES

Name	Sampling Site
Port	Approximately 0.3 miles north of section line, adjacent to Black Bear Creek bridge on U.S. 77 in west right-of-way.
Renfrow	On U.S. 77W diagonally across intersection adjacent to Oklahoma Department of Transportation Division 4, Headquarters in Perry, Oklahoma.
Lela	Approximately one mile south of Red Rock Creek bridge on U.S. 77 in west right-of-way.
Kirkland	Approximately 0.2 miles north of entrance to Perry airport on U.S. 77 in east right-of-way, adjacent to small concrete culvert.
Norge	One-half mile east of SH 156 and U.S. 77 junction on SH 156, in south right-of-way, near the fence.
Miller	Approximately one mile south of Red Rock Creek bridge on U.S. 77 in west right-of-way.

washed over the sieves. The procedure for particle size analysis followed ASTM D422-63. That portion of the sample passing the No. 200 sieve was collected and oven dried for use in a hydrometer analysis. The sample was soaked in distilled water to insure that the particles were separated before the test. Sodium silicate (dispersing agent) was used for the sample preparation. The testing procedure for hydrometer analysis followed ASTM 422-63.

Testing Program for Determining the Resilient Modulus

The moisture content and dry density used for preparation of the M_R specimens were governed by the Standard Proctor Compaction Test data. Air-dried soil passing the No. 4 sieve was prewetted at five different moisture contents (from dry to wet conditions) and allowed to equilibrate for at least 24 hours. The testing procedure followed ASTM D498.

a. Selection of w% and γ_{dry} for Sample Preparation. Since moisture content is used to control the field compaction, the selection of the compaction parameters is based on this factor. Three moisture contents (Optimum Moisture Content, OMC; OMC-2.0; and OMC+2.0) were used for sample preparation. Dry densities for these moisture contents were determined from the Standard Proctor Compaction curves.

b. Preparation of the Soil for Laboratory Compacted Specimen. The procedure used to prepare soil samples for laboratory compaction followed AASHTO Designation: T274-82, Section 6.3. The prepared samples were sealed in plastic bags and cured at room temperature for 24 hours.

c. Compaction of Specimens. Static compaction was used for the resilient modulus testing program. The process involves compacting a known weight of wet soil to a volume that is fixed by the dimensions of the mold and compaction ram. The typical mold assembly for the preparation of a specimen involved a 2.8-in. diameter by 6-in. high cylinder which used 3 layers, see Figure 9a. The resulting specimen was 6 inches long. The compaction procedure is primarily based on the procedure of ASSHTO Designation: T 274-82, Section 6.4.4. Some changes were made for the OSU laboratory equipment. The modified procedure for the compaction of specimens is described in the following:

1. Three layers were used to compact the specimen. The weight of wet soil per layer (corresponding to the desired moisture content and dry density) was determined by

$$\bar{W}_L = \frac{\bar{W}_t}{3}$$

where:

$\bar{W}_L$ = Weight of wet soil per layer, gms

$\bar{W}_t$ = Total weight of specimen, gms

$$= \gamma_{dry} \times V_{specimen} \times (1 + w)$$

$$= \gamma_{dry} \times \frac{\pi (2.8)^2 \ (6)}{(4)(12^3)} \times (1 + w)$$

$$= (0.02138)\ (\gamma_{dry},\ \text{pcf})\ (1 + w)$$

An additional 1 to 2 grams of soil for each layer compensates for losses.

2. The lower loading ram was placed in the soil mold. The mass of soil, $\bar{W}_L$, determined in Step 1 was placed into the sample mold and the sample mold was gently vibrated to insure that the material was distributed properly in the mold.

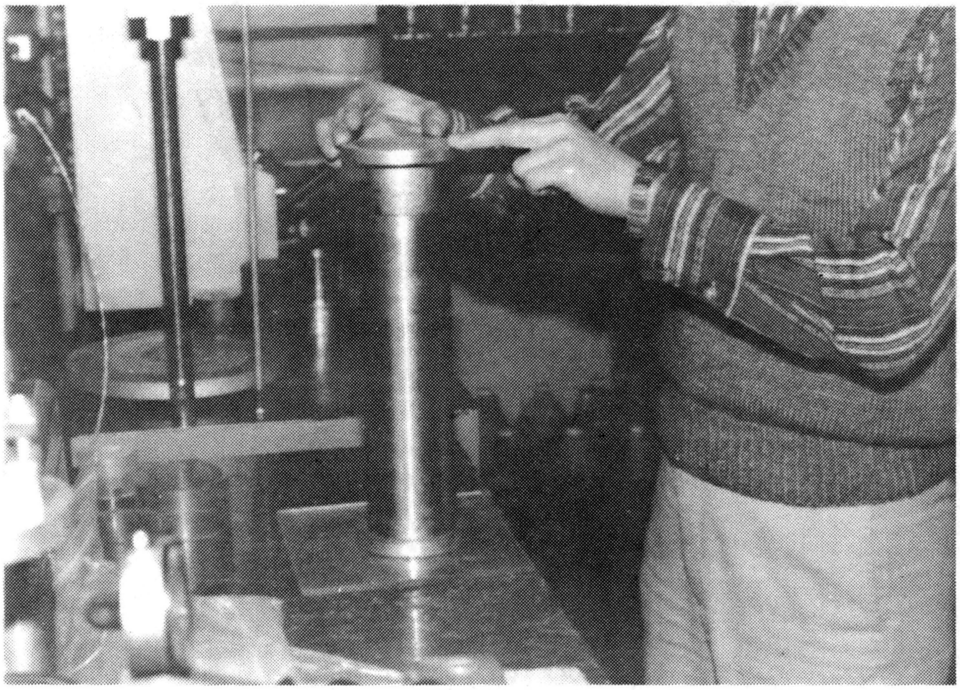


Figure 9a. Typical Assembly for the Specimen Compaction

3. The upper loading ram was inserted and the assembly placed in a Versa-Tester loading machine, and a small load was applied. The mold was centered in the machine to provide proper ram clearance, as shown in Figure 9b. The load was slowly increased until the load ram shoulder pressed against the mold. The load was held for approximately one minute. These procedures were found to reduce soil rebound.

4. After the load was removed, the assembly was taken from the machine. The ram was removed and the surface of compacted layer was scarified. The second layer of soil was then placed as described in Step 2. The mold was extended by inserting a 2-in. spacer ring and the second layer was compacted as in Step 3. See Figures 9c and 9d.

5. The procedures of Step 4 were repeated for the third layer.

6. The molded specimen was then extruded using the hydraulic jack system shown in Figure 9e.

7. The specimen was placed on a glass plate, and both ends were carefully trimmed and lightly brushed to remove loose soil. Careless trimming can result in poor contact which may affect the M_R test data.

8. The height of the specimen was measured to the nearest 0.002 in. and recorded.

9. The specimen was then placed in a triaxial test cell and enclosed in a membrane secured by O-Rings to the top and bottom platens (shown in Figure 9f). If back pressure saturation is not used, the specimen is now ready for resilient testing.

d. <u>Back-Pressure Saturation of Compacted Specimen</u>. For back-pressure saturation, the following procedure was used.

1. Filter paper was inserted between the porous stone and specimen before unrolling the rubber membrane.

Figure 9b. Centering the Mold

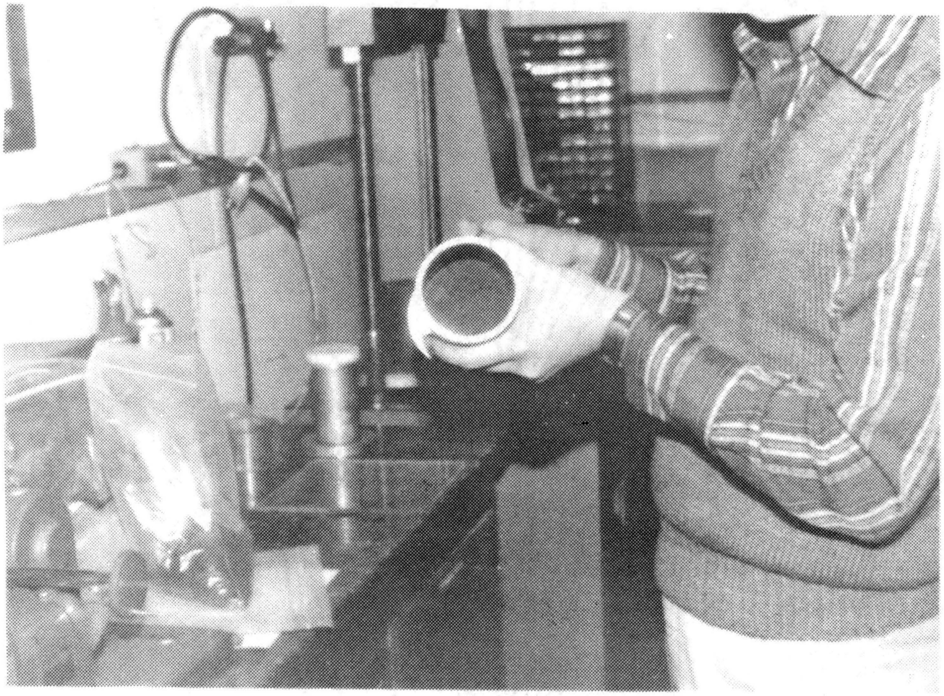


Figure 9c. Scarified Surface of Compacted Layer

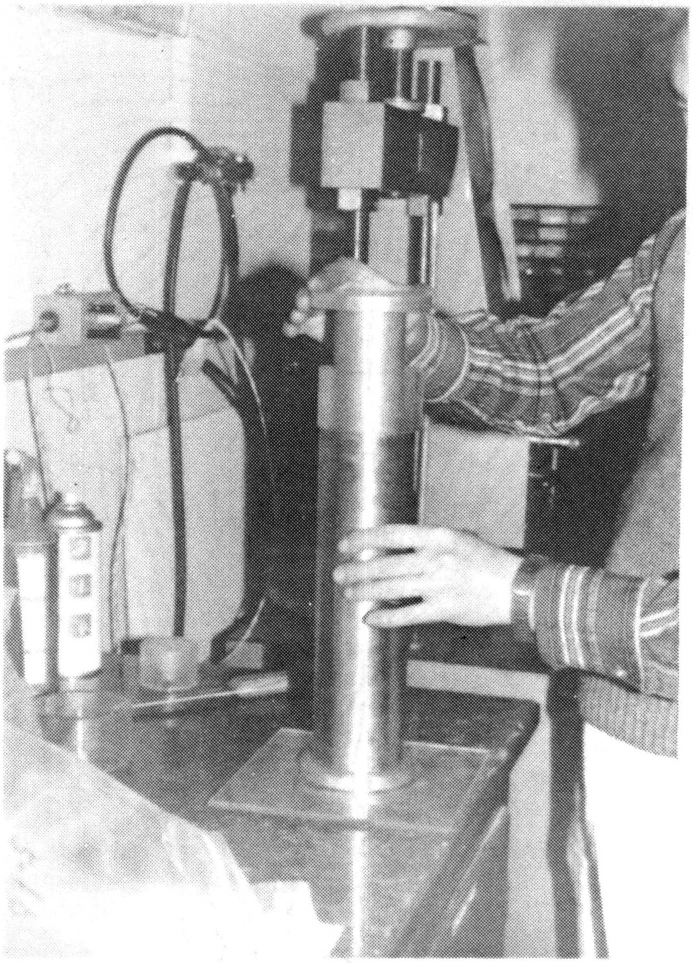


Figure 9d. Inserting 2-in Spacer Ring for the Second Layer Compaction

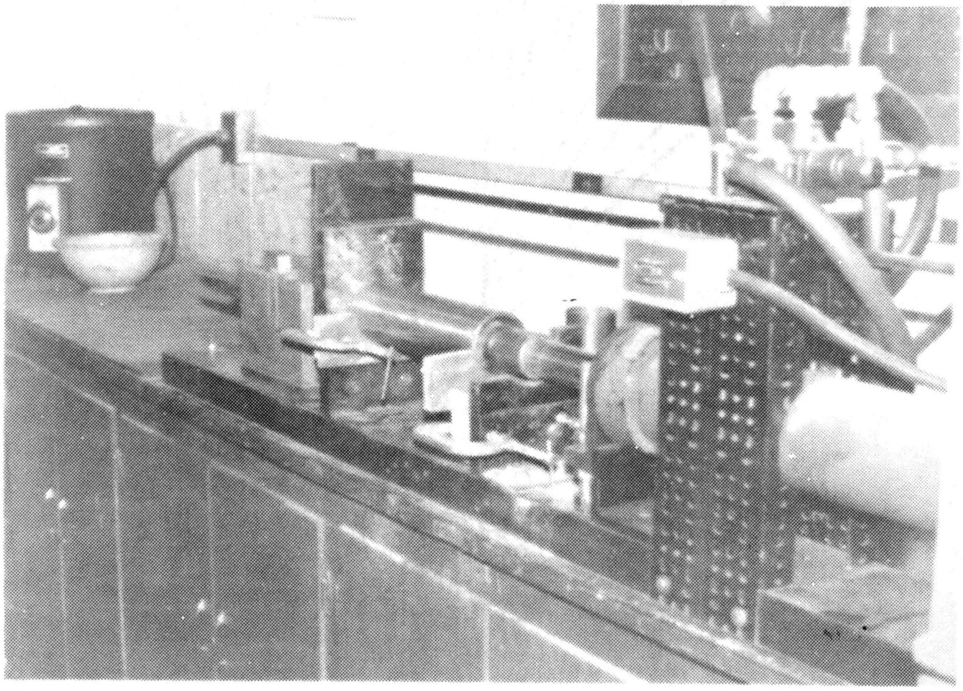


Figure 9e. Hydraulic Jack System for Extruding Specimen

Figure 9f. Unrolling the O-Ring and Membrane to Enclose the Specimen

2. A small pressure was applied to force the water through the line and saturate the porous stone.

3. Install the membrane and assemble the triaxial test cell.

4. The assembly was placed beneath the load cell and the ball bearing was inserted between the load cell and piston.

5. The bottom and top drainage lines were connected to the cell, keeping the valves closed.

6. The chamber pressure line was connected to the triaxial cell.

7. The chamber pressure and back pressure were applied in increments of 5 psi to 15 psi.

8. The volume of water movement in the burettes was recorded.

9. The pressure was decreased to 10 psi when the water stopped flowing and held for 30 minutes.

10. The pressure was reduced to 6 psi and held for 30 minutes.

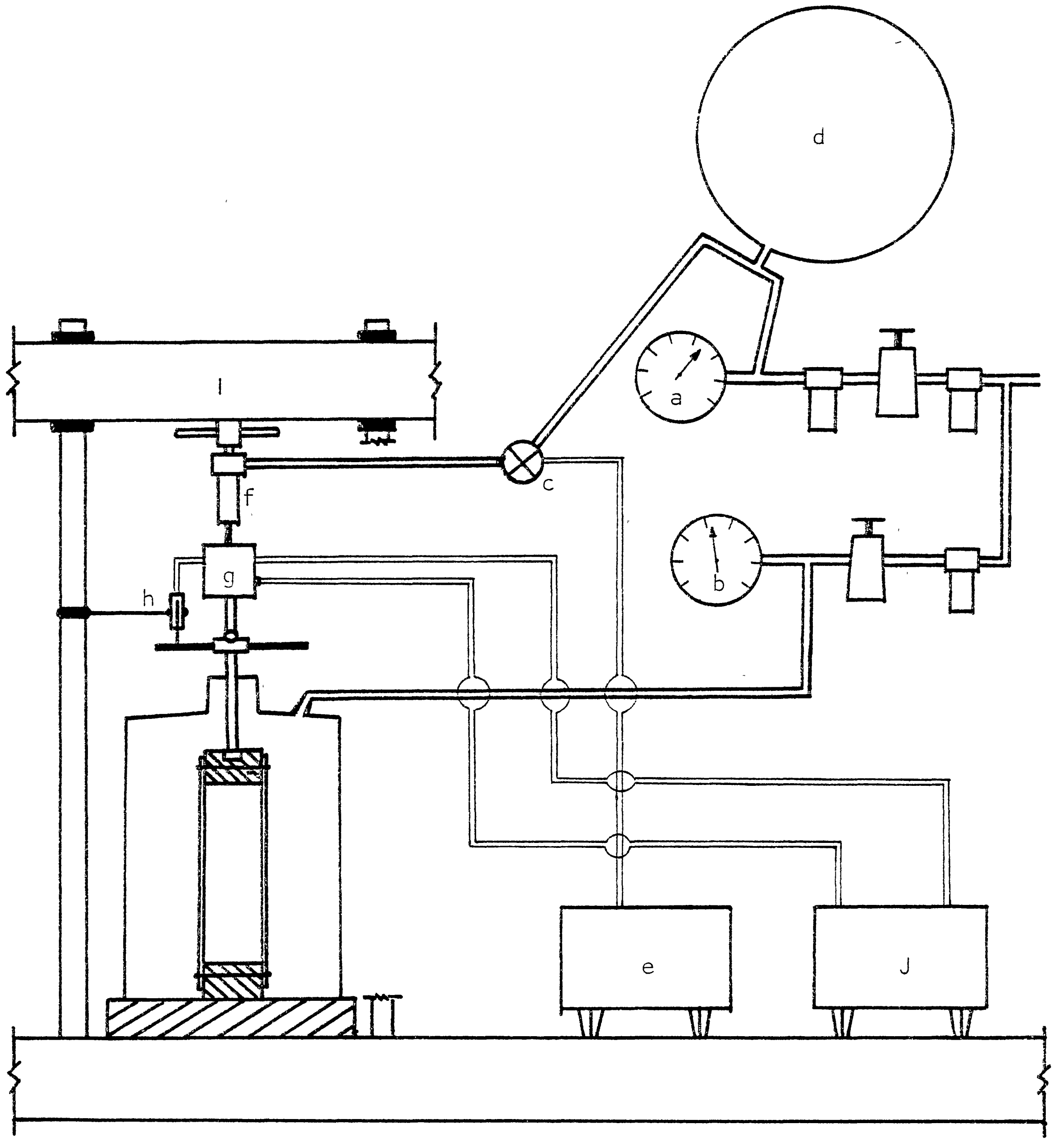
11. The back pressure was reduced to 0 psi and held for 30 minutes. Then, the chamber pressure line was disconnected.

12. Now the saturated specimen is ready for resilient testing.

Resilient Modulus Testing

a. Apparatus. Utilizing information from the technical literature as a beginning point, a resilience testing apparatus was designed and fabricated. The penumatic loading apparatus is capable of applying repeated dynamic loads of controlled magnitude and duration. Deformation is measured with a DCDT. A schematic diagram of the apparatus is presented in Figure 10.

Typical equipment used in the system included:



LEGEND:

a - Load Pressure Gage
b - Confining Pressure Gage
c - Solenoid Valve
d - Surge Tank
e - Micromaster Controller
f - Pneumatic Piston
g - Load Cell
h - DCDT
I - Load Frame
J - Strip Recorder

Figure 10. Resilient Modulus Testing Apparatus

1. Pressure gauges: 60 psi gauge for the loading system and 15 psi gauge for the confining system.

2. Solenoid valve: SMS Pneumatic Product Directional air solenoid valve.

3. Surge tank: Midwest Products Mobil air surge tank, 125 psi maximum capacity.

4. Micromaster controller: Minarik Electric, Model wp 6000 Microprocessor controller used to control the load frequency and duration.

5. Pneumatic piston: Bimba Model 172-D, pneumatic piston used to apply the repeated loading. Piston diameter = 1.5 in., rod diameter = 7/16 in., and stroke = 2.0 in.

6. Load cell: BLH Electronics Products, Model U3GL 300 pounds capacity.

7. DCDT: Hewlett Packard 7, DCDT-250 Model, maximum stroke ± 0.25 in.

8. Strip Recorder: Sargent Welch, Model DSRG-2, strip chart recorder used to record the deformation and loading.

9. DC power supply: two DC power supplies were used in the system, one for the DCDT and another for the load cell.

b. Testing Procedure: The testing procedure is primarily based on AASHTO Designation: T 274-82. Some adjustments were made to accommodate the testing equipment used. The following procedure is the one used throughout the testing program. For saturated specimens, Steps 1, 2, and 3 were not used.

1. The load cell was placed in contact with the ball bearing resting on the loading piston.

2. The top and bottom drainage valves leading into the specimen were opened.

3. For unsaturated specimens, the chamber pressure supply line was connected and a chamber pressure of 6 psi applied.

4. The load cell was adjusted and the DCDT unit placed on a flat steel plate fixed to the piston.

5. The recorders were zeroed for the DCDT and load cell.

6. The loading duration was set at 0.1 second and cycle duration at 3 seconds.

7. The test was begun by applying 200 repetitions of a deviator stress of 1.0 psi and followed by 200 repetitions each at 2, 4, 8 and 10 psi. This stress sequence constitutes specimen conditioning. That is, the elimination of the effects of the interval between compaction and loading and the elimination of initial loading versus unloading. This load conditioning also aids in minimizing the effects of initially imperfect contact between the end platens and the test specimen.

8. A suitable "mv" position was selected for the deformation channel for each deviator stress during the sample conditioning process. The selected "mv" positions were used during subsequent loadings.

9. The permanent deformation was recorded for adjusting the sample height.

10. The deviator stress was decreased to 1.0 psi and 200 repetitions of the deviator stress were applied and the average recoverable deformation during the last 10 strokes. A typical strip chart recording is shown in Figure 11.

11. 200 repetitions were applied using deviator stresses of 2, 4, 8 and 10 psi. The average recoverable deformation was recorded as in Step 10.

12. The chamber pressure was decreased to 3 psi and Steps 10 and 11 repeated.

13. The chamber pressure was decreased to zero and Steps 10 and 11 repeated.

14. After completion of the loading (with chamber pressure at zero) the triaxial cell was disassembled.

15. The entire specimen was used for determining the water content and dry density.

c. Resilient Modulus Calculation. The resilient modulus, M_R, was calculated by dividing the repeated axial stress (equal to the deviator stress, σ_D) by the resilient or recoverable strain, ε_R.

$$\varepsilon_R = \frac{\text{Recoverable Deformation}}{\text{Adjusted Sample Height}}$$

in which recoverable deformation can be determined from the recording on the strip chart (Figure 11).

Adjusted Sample Height = Initial Height minus the permanent deformation caused by sample conditioning $M_R = \frac{\sigma_D}{\varepsilon_R}$

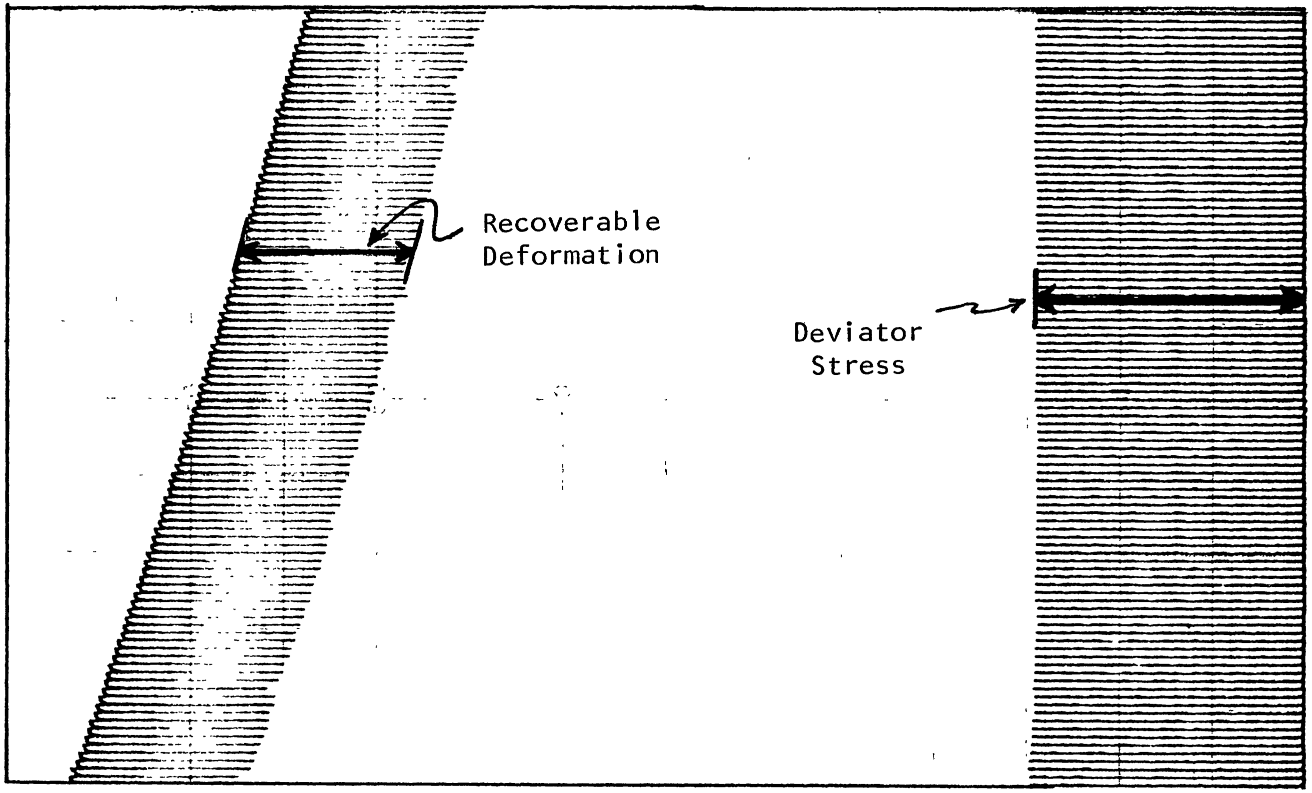


Figure 11. Typical Strip Chart Record

CHAPTER IV

PRESENTATION OF RESULTS

Introduction

The results of the testing program are presented in this chapter. The results for the resilient modulus tests are given in Appendix A. Since no significant difference in M_R occurred for the Renfrow, Lela and Kirkland soils with varying confining pressures (i.e. under normal conditions), average results for the duplicate specimens are presented in Appendix A. Appendix B contains the Standard Proctor Compaction curves for the soil samples.

Basic Engineering Properties

Atterberg Limits and Specific Gravity

The Atterberg Limits, Specific Gravity, and classification for the six selected samples are shown in Table III.

Grain Size Analysis

Grain size distribution curves are shown in Figures 12 and 13. The properties calculated from the grain size distribution curve are listed in Table IV.

TABLE III

ATTERBERG LIMITS AND SPECIFIC GRAVITIES FOR THE SELECTED SOIL SAMPLES

Soil Samples	Atterberg Limits			Specific Gravity	Unified Classification	AASHTO Classification
	LL	PL	PI			
Port	31.7	17.2	14.5	2.71	CL	A-6
Renfrow	42.1	17.0	25.1	2.81	CL	A-7
Lela	66.4	19.0	47.4	2.83	CH	A-7
Kirkland	47.8	16.0	31.8	2.79	CL-CH	A-7
Norge	34.7	18.8	15.9	2.76	CL	A-6
Miller	42.7	15.2	27.5	2.75	CL	A-7

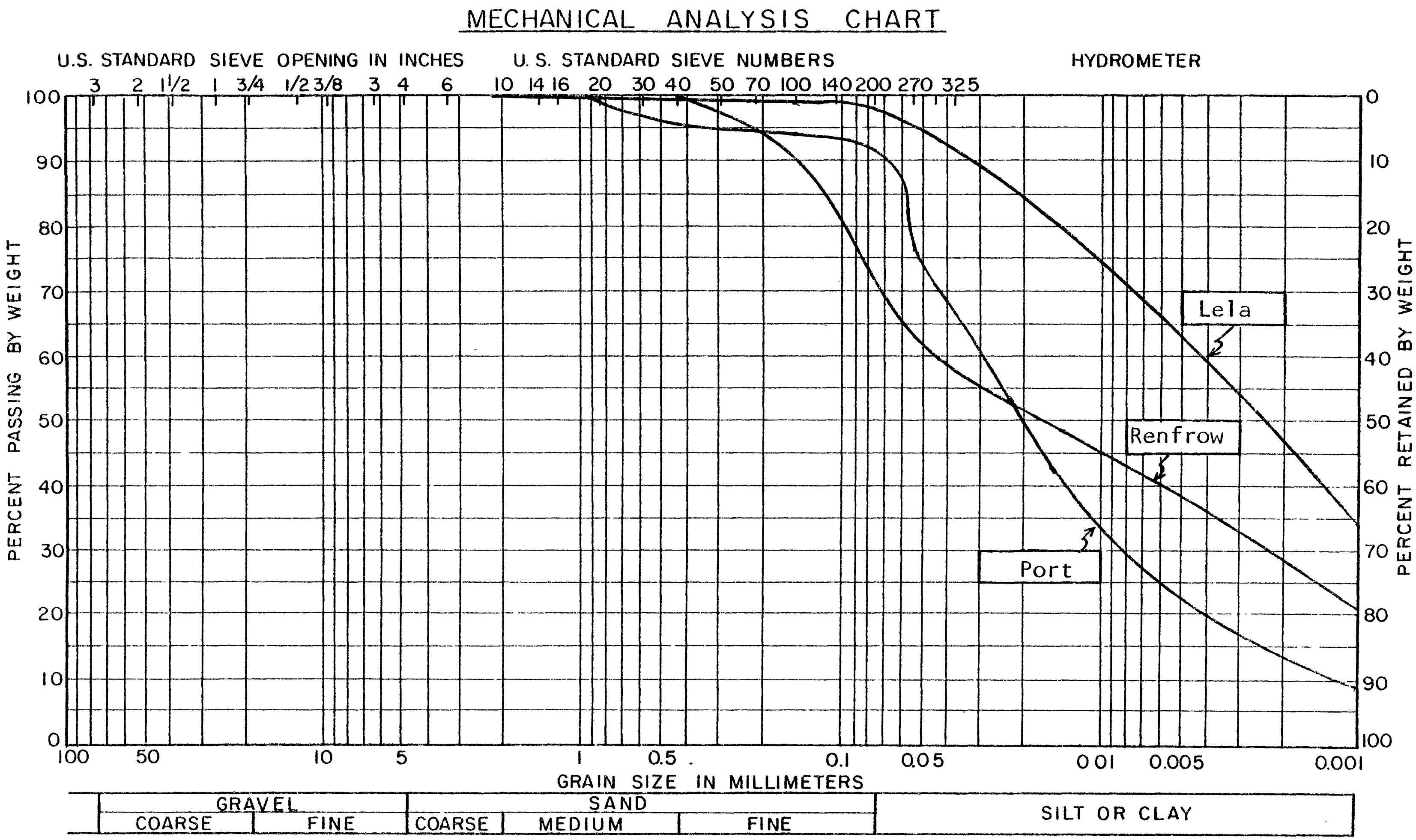


Figure 12. Grain Size Distribution Curve for Port, Renfrow, and Lela Soils

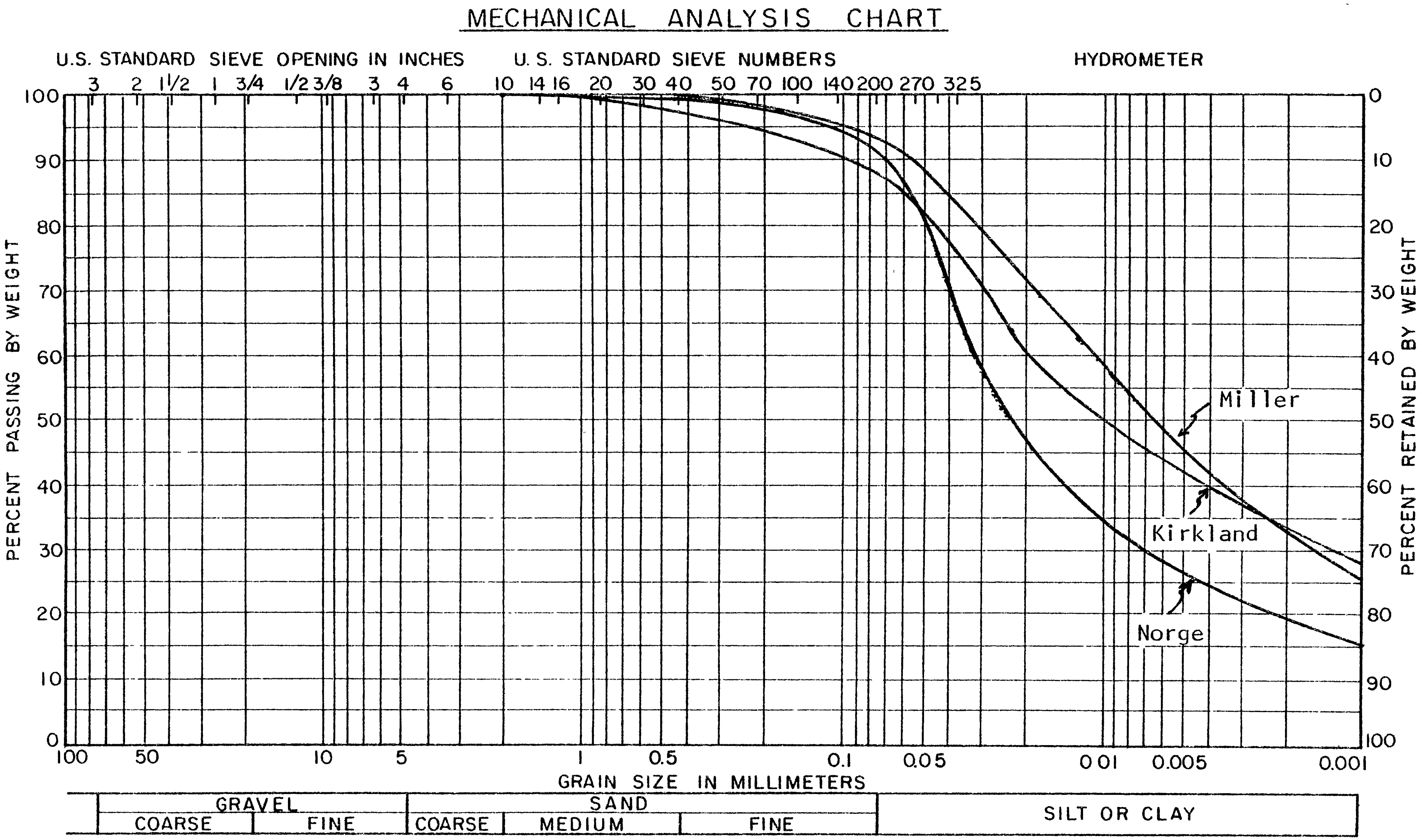


Figure 13. Grain Size Distribution Curves for Kirkland, Norge, and Miller Soils

TABLE IV

PROPERTIES OBTAINED FROM GRAIN SIZE DISTRIBUTION CURVES

Soil Samples	Soil Fraction, %			Uniformity of Coefficient, C_u $C_u = D_{60}/D_{10}$	Coefficient of Curvature, C_z $C_z = D_{30}^2/D_{10}D_{60}$
	Sand	Silt	Clay		
Port	9	78	13	25.0	0.18
Renfrow	29	43	28	88.0	2.20
Lela	2	51	47	8.2	0.33
Kirkland	12.5	54.5	33	200.0	0.72
Norge	9	72	19	63.3	3.20
Miller	7.5	59.5	33	36.7	0.68

Oklahoma Subgrade Index Determination

The Oklahoma Subgrade Index, based on the charts shown in Figure 7, are presented in Table V. The data used to determine the OSI, (% passing No. 200 sieve, plasticity index, and liquid limit) are also listed in the Table.

Standard Proctor Compaction Testing

Standard Proctor compaction curves for the selected soils are presented in Appendix B. The moisture contents and dry densities selected for the specimen preparation in the M_R testing program were obtained from these curves and a summary is presented in Table VI.

Results of Resilient Modulus Testing

The volume of test results is too large to present in its entirety, but the average results for the test series are shown in Tables VIIa - VIIf.

TABLE V

OKLAHOMA SUBGRADE INDEX (OSI) FOR THE SELECTED SOIL SAMPLES

Soil Samples	OSI(1) Determination			OSI(2) Determination			OSI = OSI(1) + OSI(2)
	% Pass #200	PI	OSI(1)	% Pass #200	LL	OSI(2)	
Port	92.4	16.4	6.5	92.4	33.0	6.8	13.3
Renfrow	71.6	25.1	10.0	71.6	42.1	8.4	18.4
Lela	98.3	47.4	18.9	98.3	66.4	13.3	32.2
Kirkland	88.4	31.8	12.6	88.4	47.8	9.7	22.3
Norge	92.9	17.5	7.0	92.9	35.7	7.2	14.2
Miller	94.9	27.5	11.0	94.9	42.7	8.6	19.6

TABLE VI

MOLDING CONDITIONS USED FOR THE PREPARATION OF THE M_R SPECIMENS

Soil Samples	Molding Conditions					
	OMC-2.0	γ_{dry}, pcf	OMC	γ_{dry}, pcf	OMC+2.0	γ_{dry}, pcf
Port	14.2	106.4	16.2	108.1	18.2	106.2
Renfrow	17.2	102.5	19.2	105.0	21.2	103.3
Lela	21.2	95.2	23.2	97.7	25.2	95.9
Kirkland	18.0	99.6	20.0	101.7	22.0	99.7
Norge	14.7	103.3	16.7	104.5	18.7	103.1
Miller	17.5	101.1	19.5	102.1	21.5	101.0

TABLE VIIa

RESILIENT MODULUS VALUES FOR PORT SOIL

σ_D, psi	OMC-2.0 σ_3, psi			OMC σ_3, psi			OMC+2.0 σ_3, psi			OMC/Saturated σ_3, psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	13.7	14.3	14.3	11.7	11.9	12.3	8.3	8.9	9.2	3.8	3.9	4.9
2	12.0	12.0	12.0	10.6	11.0	11.2	7.2	7.7	8.2	3.1	3.4	4.2
4	11.4	11.4	11.7	9.7	10.2	10.6	5.7	5.7	6.4	2.4	2.5	3.1
8	10.7	10.7	10.7	8.4	8.6	9.6	5.2	5.1	5.3	2.8	2.7	2.9
10	10.4	10.7	10.6	8.5	9.0	9.3	5.1	5.2	5.2	2.8	2.8	2.9

Units of M_R = Ksi.

TABLE VIIb

RESILIENT MODULUS VALUES FOR RENFROW SOIL

σ_D, psi	OMC-2.0 σ_3, psi			OMC σ_3, psi			OMC+2.0 σ_3, psi			OMC/Saturated σ_3, psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	15.1	15.1	15.6	15.0	15.0	15.1	14.4	14.4	14.4	4.5	5.2	5.9
2	13.6	13.5	13.9	13.4	13.4	13.5	12.6	12.6	12.6	3.6	4.5	5.1
4	12.0	12.0	12.4	10.9	10.9	10.8	9.7	9.7	9.7	2.6	2.9	3.9
8	11.8	11.8	11.9	10.3	10.3	10.4	6.9	6.9	6.9	2.2	2.4	2.6
10	11.7	11.7	11.7	9.9	9.9	9.8	6.4	6.4	6.4	2.1	2.4	2.6

Units of M_R = Ksi.

TABLE VIIc

RESILIENT MODULUS VALUES FOR LELA SOIL

σ_D, psi	OMC-2.0 σ_3,psi			OMC σ_3,psi			OMC+2.0 σ_3,psi			OMC/Saturated σ_3,psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	14.3	15.4	15.4	12.2	12.4	12.3	11.8	11.8	11.8	4.2	4.5	4.8
2	13.2	14.0	14.2	10.5	10.8	10.1	10.7	10.7	10.4	3.6	3.9	4.1
4	11.8	12.5	12.2	9.7	10.0	9.5	8.3	8.3	8.1	2.5	2.8	3.0
8	11.5	12.4	11.7	9.3	9.1	9.3	7.2	7.2	7.2	2.1	2.3	2.4
10	11.3	11.7	11.6	9.7	9.3	9.5	7.0	6.8	6.6	2.0	2.2	2.4

Units of M_R = Ksi.

TABLE VIId

RESILIENT MODULUS VALUES FOR KIRKLAND SOIL

σ_D, psi	OMC-2.0 σ_3, psi			OMC σ_3, psi			OMC+2.0 σ_3, psi			OMC/Saturated σ_3, psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	15.0	15.1	15.1	14.4	14.4	14.4	11.7	11.9	11.7	5.8	6.2	7.2
2	13.4	13.5	13.5	11.6	11.6	11.6	10.3	10.3	10.3	5.2	5.7	6.7
4	11.5	11.5	11.5	9.8	9.8	9.8	8.6	8.6	8.6	3.6	3.8	4.9
8	11.0	11.0	11.0	10.4	10.4	10.4	7.9	8.1	7.9	2.5	2.6	2.9
10	11.2	11.2	11.2	10.6	10.6	10.6	7.9	7.9	7.0	2.4	2.5	2.8

Units of M_R = Ksi.

TABLE VIIe

RESILIENT MODULUS VALUES FOR NORGE SOIL

σ_D, psi	OMC-2.0 σ_3, psi			OMC σ_3, psi			OMC+2.0 σ_3, psi			OMC/Saturated σ_3, psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	12.2	13.3	14.0	10.2	10.8	11.5	8.0	8.4	9.1	4.8	5.4	6.0
2	10.1	10.7	11.4	8.5	9.0	9.4	6.9	7.2	7.7	3.0	4.9	5.2
4	8.4	8.9	9.7	6.2	6.6	7.1	4.7	4.8	5.4	2.9	3.6	4.2
8	7.8	8.5	9.2	5.5	5.9	6.2	4.3	4.4	4.7	2.9	3.3	3.7
10	7.8	8.4	8.9	5.3	5.7	6.0	4.0	4.1	4.3	3.0	3.4	3.7

Units of M_R = Ksi.

TABLE VIIf

RESILIENT MODULUS VALUES FOR MILLER SOIL

σ_D, psi	OMC-2.0 σ_3, psi			OMC σ_3, psi			OMC+2.0 σ_3, psi			OMC/Saturated σ_3, psi		
	0	3	6	0	3	6	0	3	6	0	3	6
1	14.0	14.7	15.5	13.7	14.6	15.3	8.4	8.6	8.9	4.1	4.5	5.4
2	12.6	13.3	13.7	14.4	11.9	12.6	7.5	7.6	7.7	3.6	3.8	4.6
4	10.9	11.1	11.4	8.8	9.0	9.6	5.8	6.0	6.1	2.2	2.4	2.9
8	10.2	10.3	10.5	8.2	8.4	8.5	4.6	4.7	4.8	1.9	2.0	2.0
10	10.1	10.1	10.2	8.1	8.3	8.4	4.3	4.3	4.4	1.9	1.9	1.9

Units of M_R = Ksi.

CHAPTER V

ANALYSIS AND DISCUSSION OF RESULTS

Introduction

The purpose of this research was to correlate basic engineering properties and resilient modulus for typical Oklahoma cohesive soils. The properties investigated were chosen because of their practical significance to field applications and determination of the OSI number.

Major emphasis was placed on the evaluation of the resilient modulus testing procedure, including specimen preparation procedures, and on the possibility of a correlation between the M_R and OSI. Pavement design thicknesses obtained by the OSI and Asphalt Institute M_R-based methods were presented. The correlations and analyses included the following:

1. Resilient Modulus testing:
    a. Analyses of the influence of specimen preparation and conditioning procedures.
    b. Analysis of the effect of confining pressure on M_R.
    c. Analyses of the effect of specimen molding and saturation conditions on the M_R.
2. Analysis of the effect of soil type on M_R and the possibility of a correlation between M_R and OSI.
3. Comparison of OSI design thicknesses with those obtained from the Asphalt Institute M_R-based procedures for given traffic and loading conditions.

Results of Resilient Modulus Testing

Specimen Preparation and Conditioning

One of the purposes of sample conditioning is to minimize the effects of an initially imperfect contact between the end platens and the test specimen. Observations of the uneven movement of the DCDT support rod and recorder pen show that the effect of uneven trimming cannot be eliminated by conditioning, especially for stiffer specimens. The measurement of deformation outside the triaxial cell provides a better indication of this problem.

It was found that proper specimen conditioning can be obtained by:

1. Properly mixing the soil and water.
2. Following the standard sample preparation procedure including a consistent curing period.
3. Carefully trimming the ends of the specimen.
4. Completing the load conditioning cycles, prior to testing.

Effects of Confining Pressure on M_R

Although previous studies had indicated that confining pressure has little or no affect on the resilient modulus, the effects of varying the confining pressure (0, 3, and 6 psi) were evaluated in this study. In this respect, an analysis of the data presented in Table VI is given in Table VIII.

Considering the normal molding conditions, information in Table VIII demonstrates that the confining pressure has no effect for the Renfrow, Lela, and Kirkland soils and the effect increases for the Miller, Norge, and Port soils, respectively. For the saturated condition, M_R is

TABLE VIII

THE EFFECT OF CONFINING PRESSURE ON M_R FOR THE SELECTED SOIL SAMPLES

Soil Samples	Sample Condition OMC-2.0	OMC	OMC+2.0	OMC/Saturates
Port	No effect	Little effect. As σ_3 increases, the M_R increases about 5%.	Little effect at the low σ_D values, but no effect for the high σ_D values (i.e., 8 and 10 psi).	Some effect for σ_3 = 6 psi, at low σ_D values (i.e., 1, 2, and 4 psi).
Renfrow	No effect	No effect	No effect	The results show about 1.0% increase for all σ_D values as σ_3 increases.
Lela	No effect*	No effect*	No effect*	Significant effect for all values of σ_3 or σ_D**
Kirkland	No effect	No effect	No effect	Significant effect, especially for confining pressure of 6 psi**.
Norge	M_R increases as the confining pressure increases.	M_R increases as the confining pressure increases.	M_R increases as the confining pressure increases.	Significant effect**.
Miller	The effect is significant only at low σ_D values (i.e., 1 and 2 psi).	The effect is significant only at low σ_D values (i.e., 1 and 2 psi).	Effect is less significant.	Effect is increased at low σ_D (i.e., 1, 2, and 4 psi).

*The small fluctuation of results may be caused by the residual conditioning (sample conditioning still going on).

**The changes of M_R less than 2% are described as "no effect". The changes of M_R ranged from 2% to 5% are described as "little effect. The changes of M_R greater than 5% are described as "significant effect."

significantly influenced by confining pressure for all soils. Combining this with the engineering properties previously evaluated for the soils, the results may be summarized:

1. For high plasticity clay soils, the confining pressure has no effect on M_R.
2. The effect of confining pressure increase as the plasticity decreases.
3. For soils with similar clay contents, the degree of influence tends to increase with an increasing amount of silt.
4. Regardless of soil type, the confining pressure significantly affects M_R for the saturated condition.

Effect of Specimen Molding and Saturation Conditions

Specimen condition as determined from compaction curve is controlled by two factors: moisture content and dry density. The following analyses consider these two factors.

Dry Density. Observations from Table VI show that for all soils, the percent compaction for each condition selected is greater than 95%. Using an average M_R^* (see Figures 14a-14f), the 100% compacted specimens (i.e., at OMC) exhibited lower M_R values than for the OMC - 2.0 specimens for all soils. The M_R values obtained from OMC - 2.0 specimens exhibited the highest values and M_R values obtained from OMC + 2.0 specimens showed the lowest values. For 100% compacted specimens (at OMC), M_R values were

*In order to compare the influences of the sample conditions on M_R, the M_R values for each deviator stress, corresponding to confining pressure (0, 3, and 6 psi) were averaged.

between the highest and the lowest M_R values. Under the saturated condition, both the OMC-2.0 and OMC specimens were used to evaluated the M_R for the Port (mostly silt) and Lela (mostly clay) soils. The curves show no difference in M_R between the OMC-2.0 and OMC specimens. Following this study, only OMC specimens was used to determine the saturated M_R values for the other soils. The results represent the lowest M_R values for the selected soils.

Variation of dry density has no effect on M_R for not only normal conditions (i.e., OMC-2.0, OMC, and OMC+2.0), but also saturated condition. This may be the result of the high degree of compaction (above 95%) used for specimen preparation.

Moisture Content. Results shown that M_R decrease with the increasing moisture content (or degree of saturation).

Effect of Soil Type on M_R and Correlation Between M_R and OSI

Effect of Soil Type on M_R

For comparison purposes, the results from Figures 14a - 14f were used and separated by the four specimen placement conditions for the selected soil types, shown in Figures 15a - 15d. The numbers beside each curve indicate the soil samples. The analyses for the data represented by the different soils are based on each sample condition and the M_R under the loads corresponding the traffic load (6 psi or greater).

In Figures 15a - 15c, representing unsaturated soils, the magnitude of the M_R is approximately in direct relationship to the plastic index of soils. Minor departures from this may be the result of specific grada-

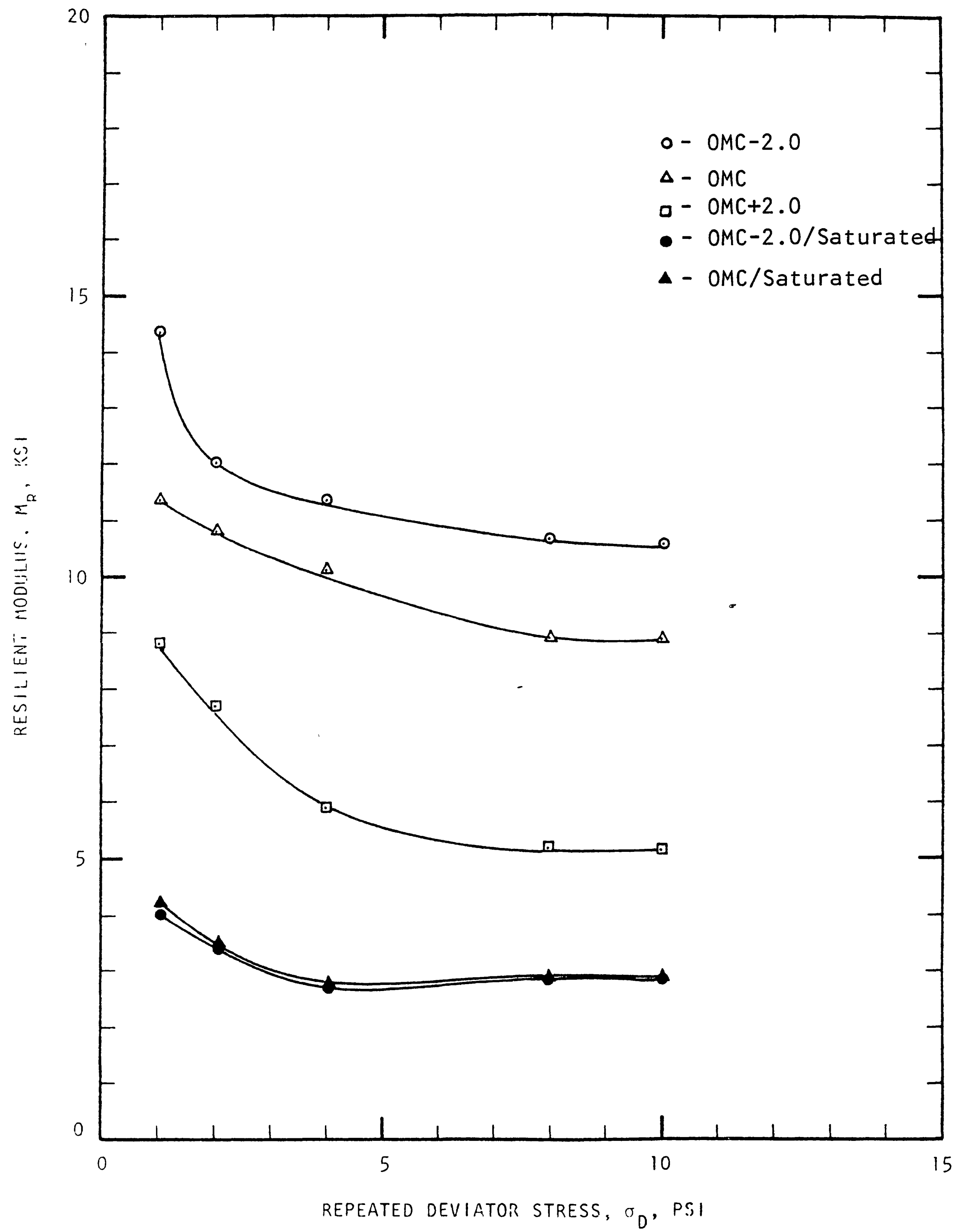


Figure 14a. Effect of Specimen Molding and Saturation Conditions on M_R for Port Soil

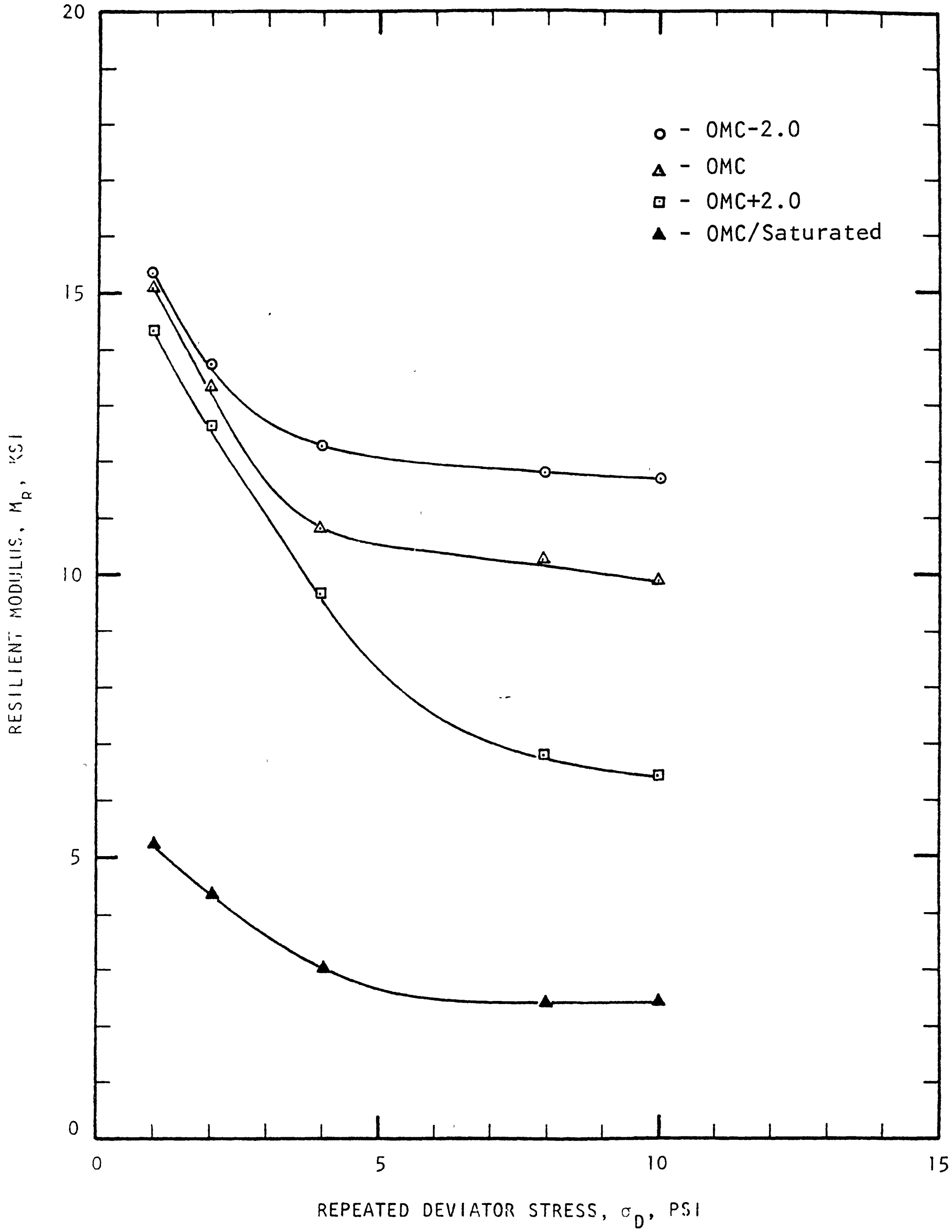


Figure 14b. Effect of Specimen Molding and Saturation Conditions on M_R for Renfrow Soil

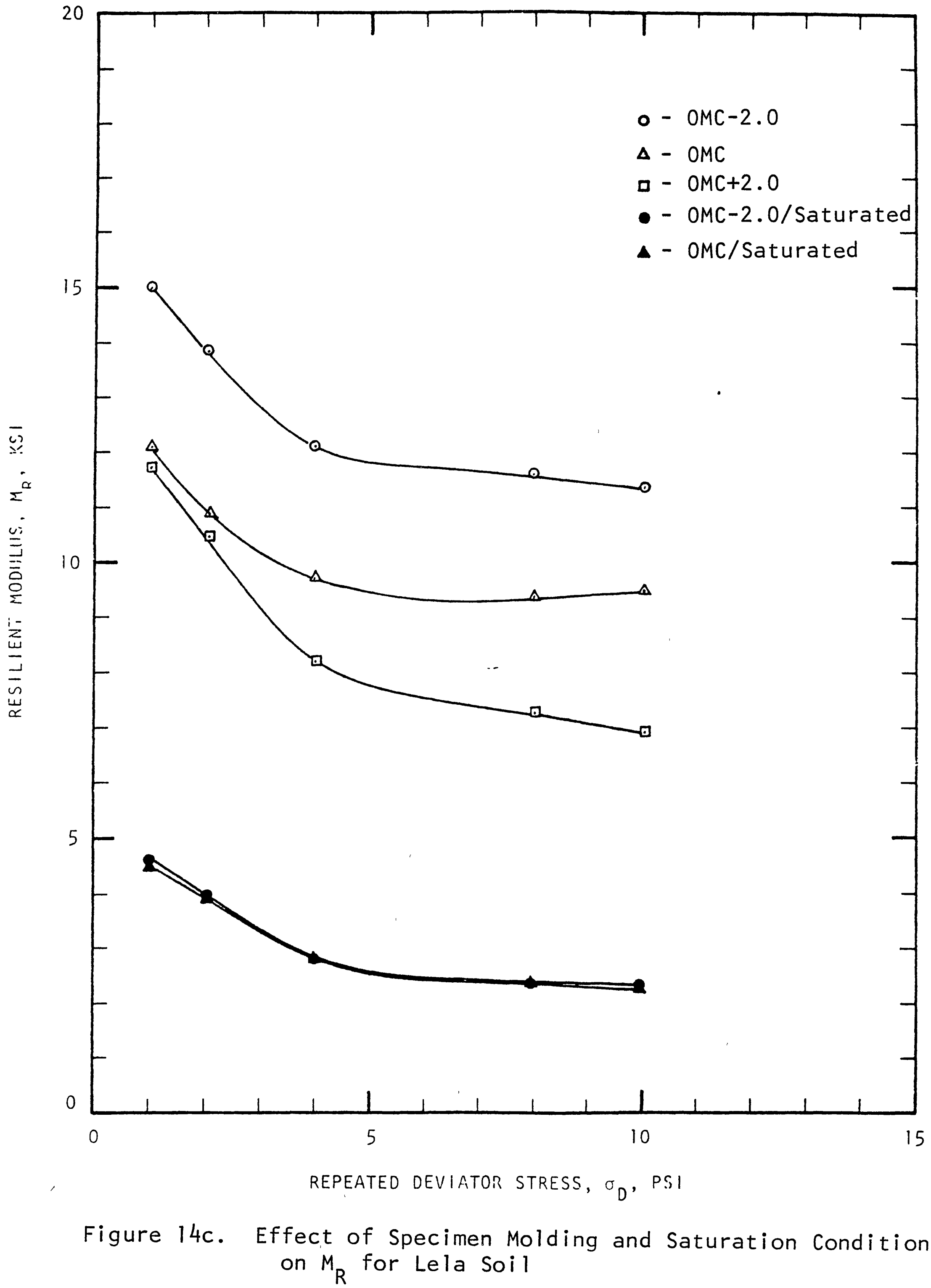


Figure 14c. Effect of Specimen Molding and Saturation Conditions on M_R for Lela Soil

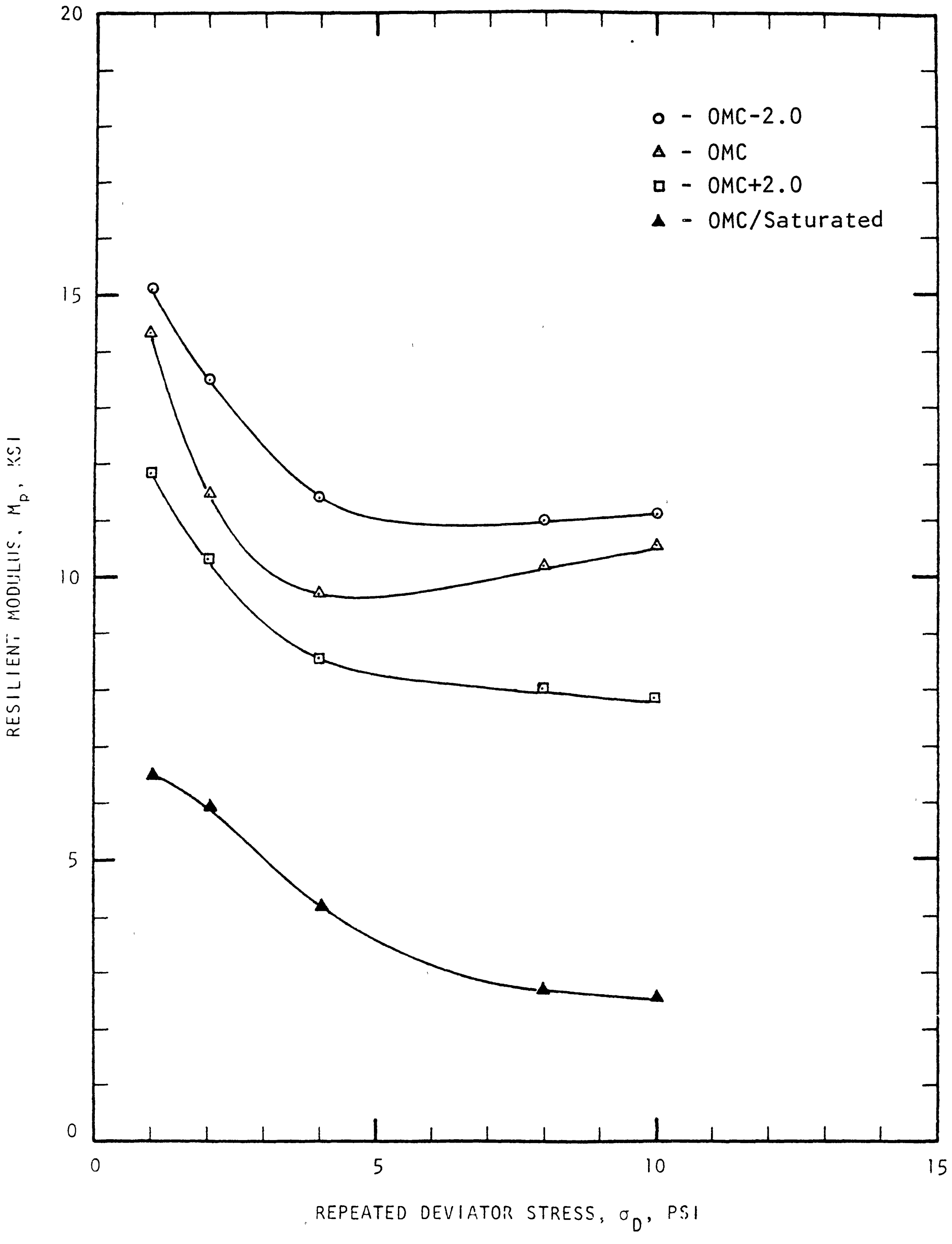


Figure 14d. Effect of Specimen Molding and Saturation Conditions on M_R for Kirkland Soil

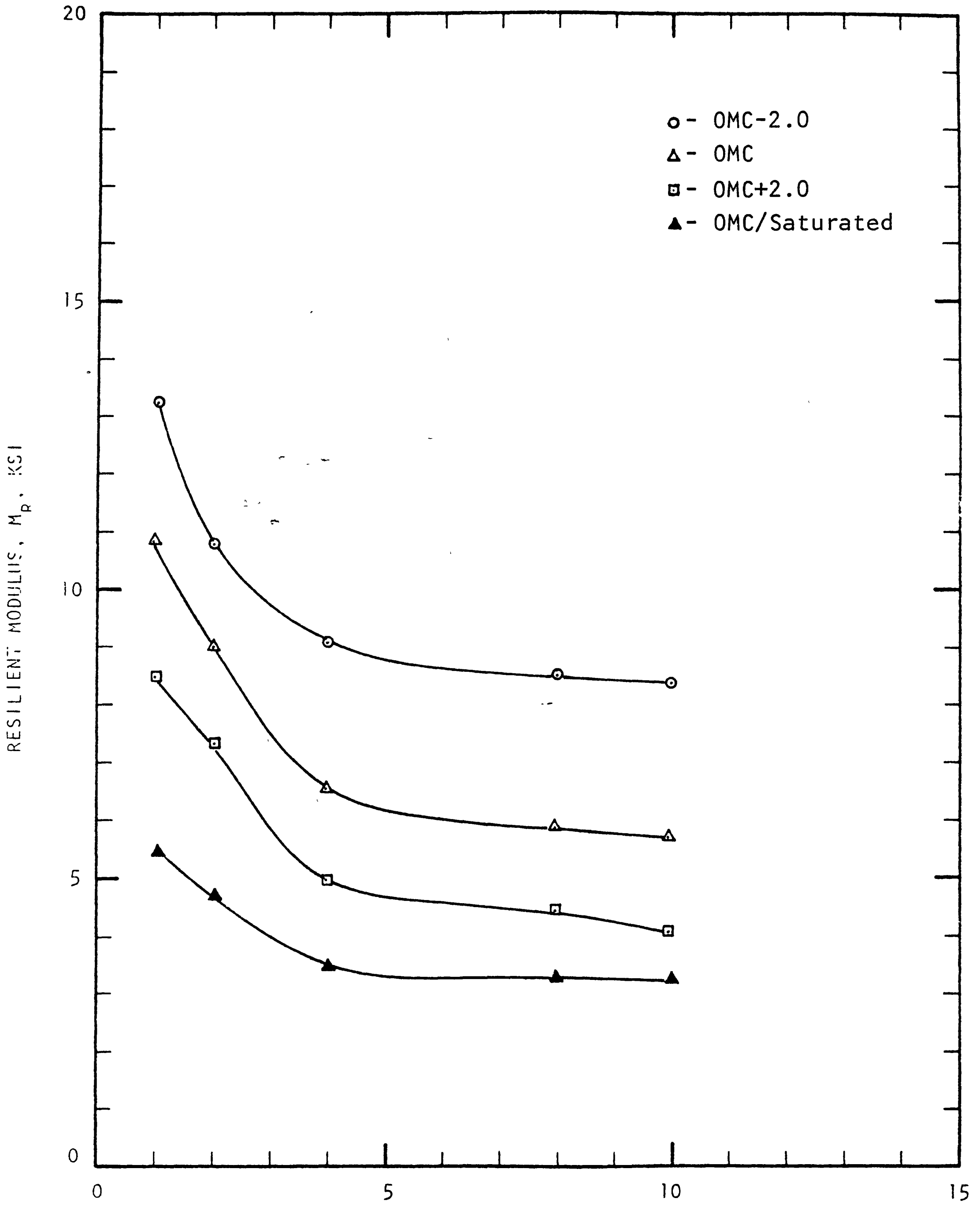


Figure 14e. Effect of Specimen Molding and Saturation Conditions on M_R for Norge Soil

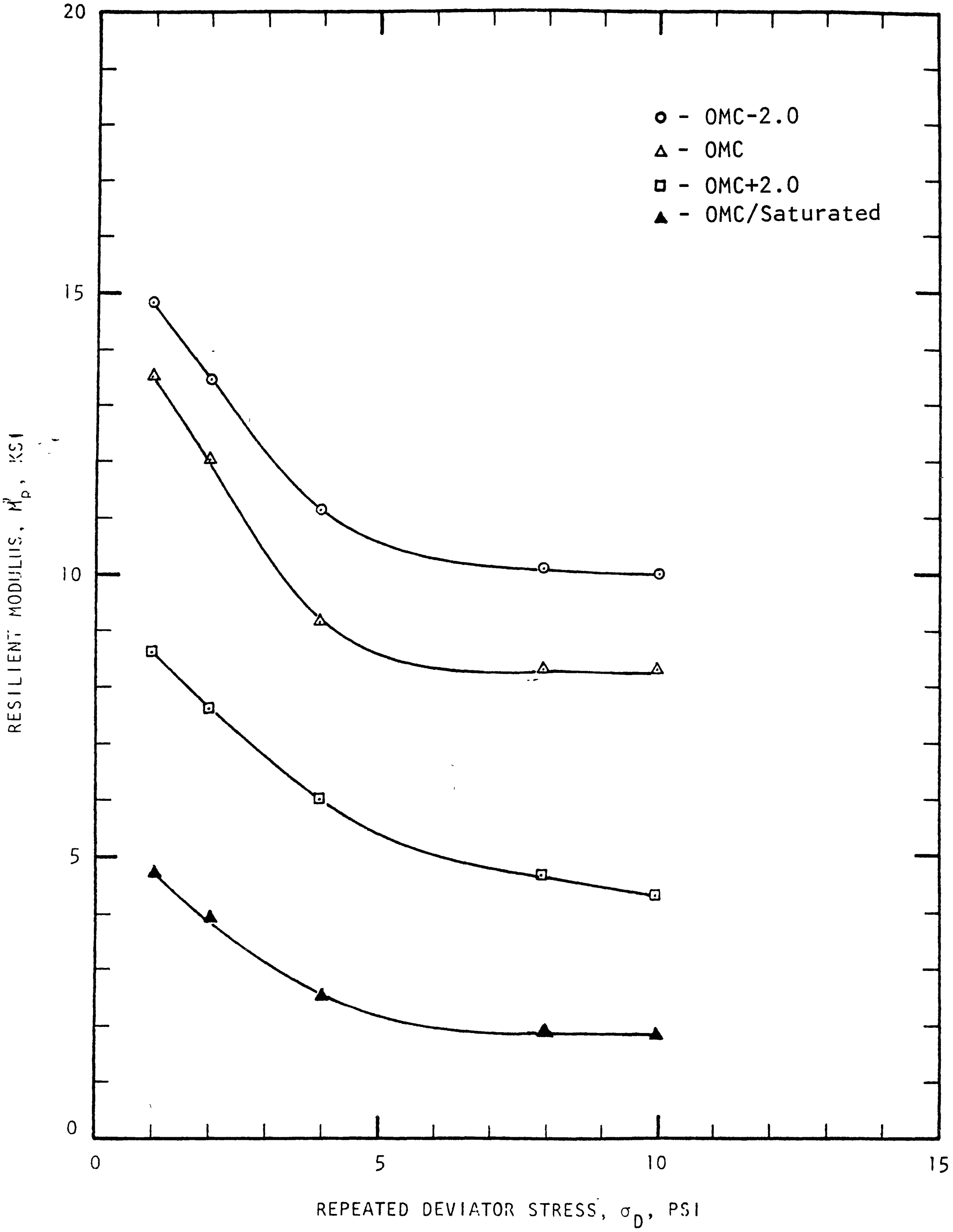


Figure 14f. Effect of Specimen Molding and Saturation Conditions on M_R for Miller Soil

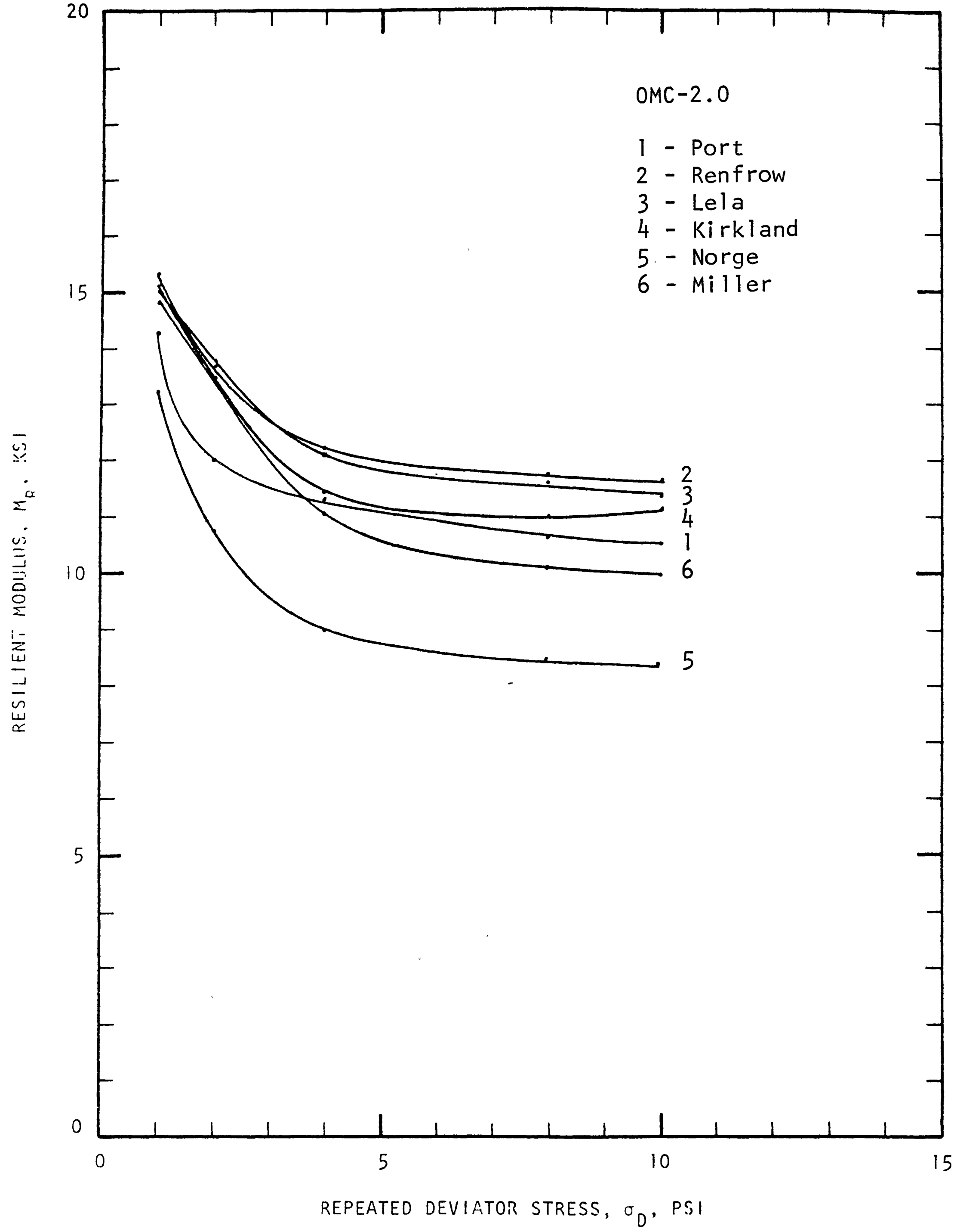


Figure 15a. Effect of Soil Type on M_R for OMC-2.0 Condition

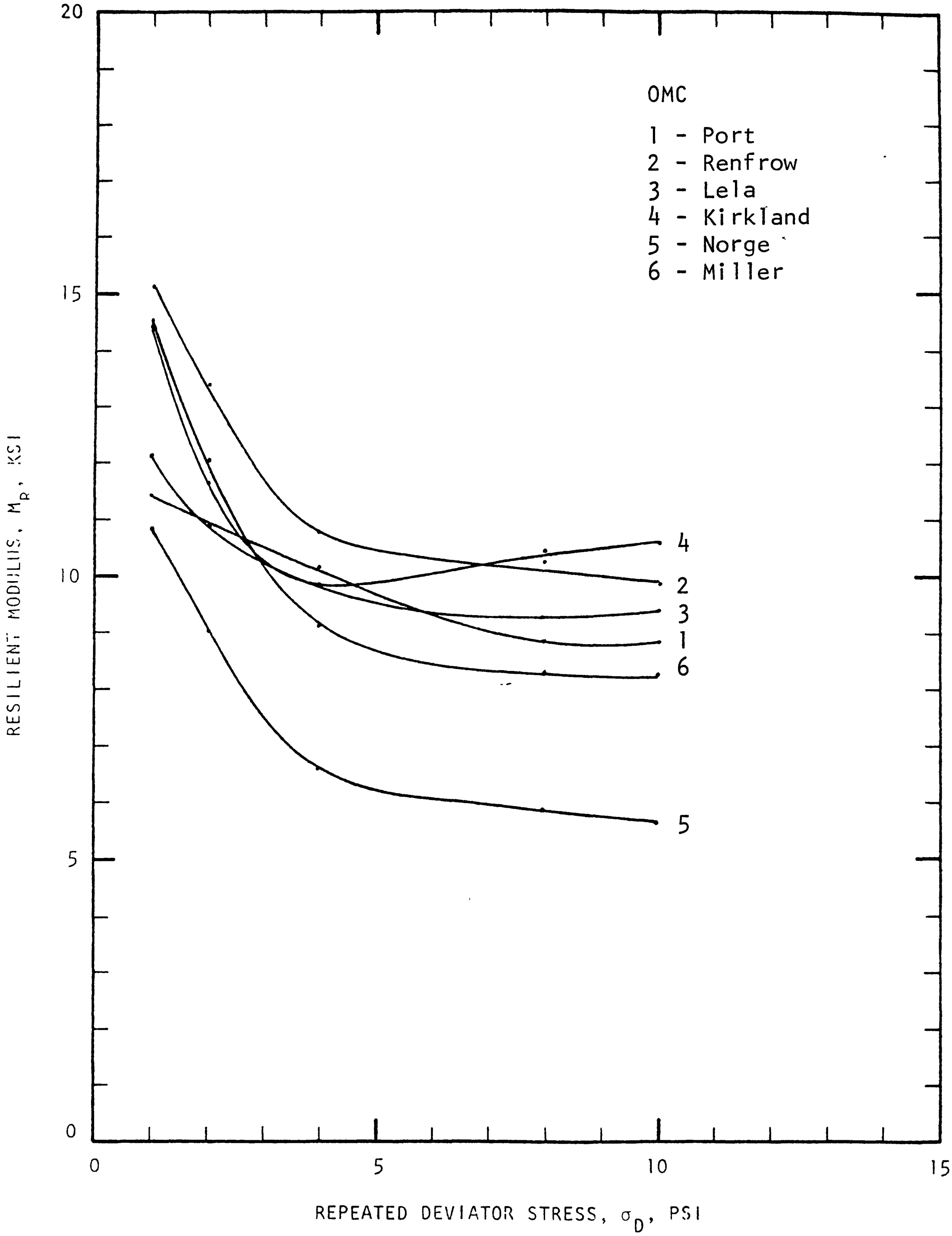


Figure 15b. Effect of Soil Type on M_R for OMC Condition

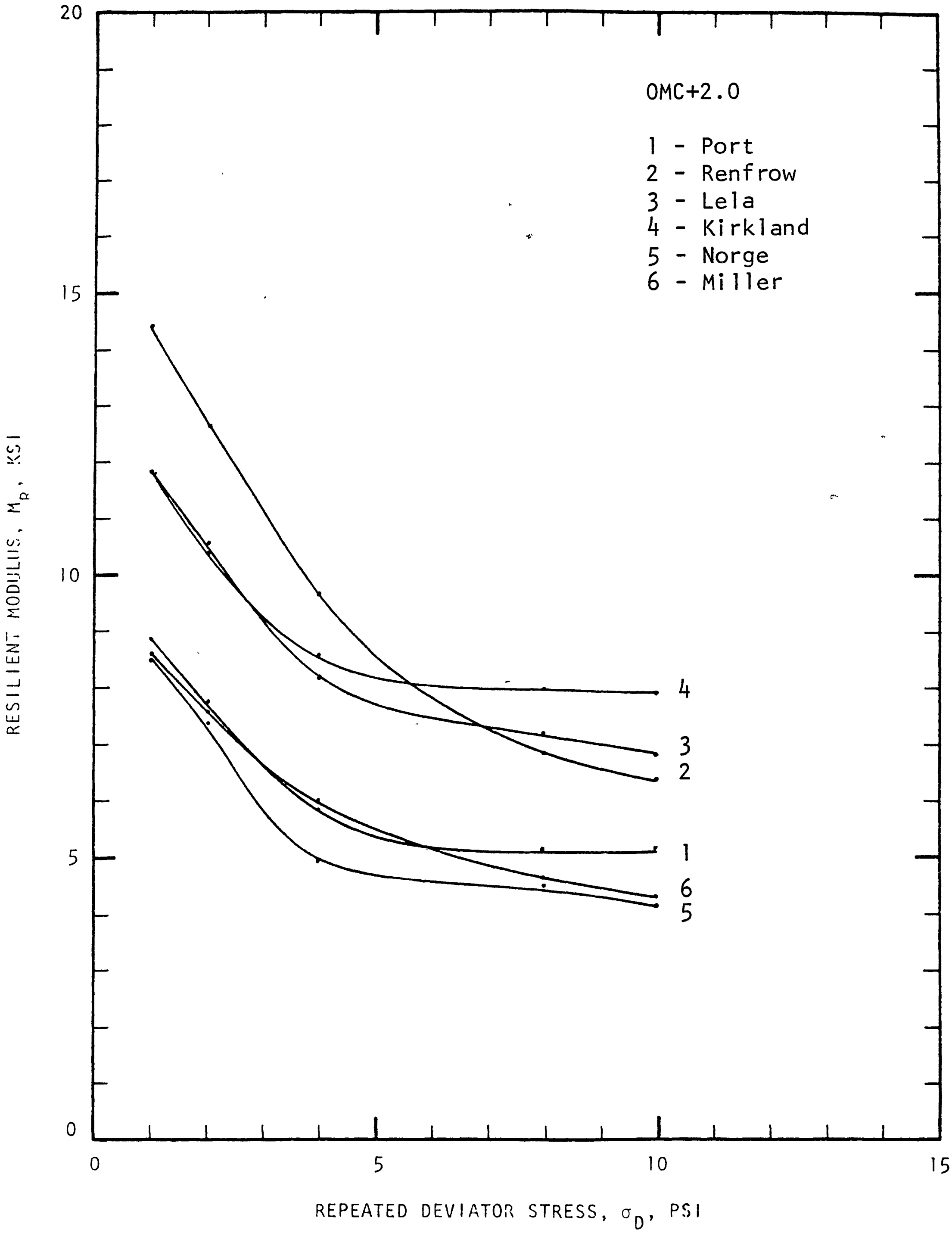


Figure 15c. Effect of Soil Type on M_R for OMC-2.0 Condition

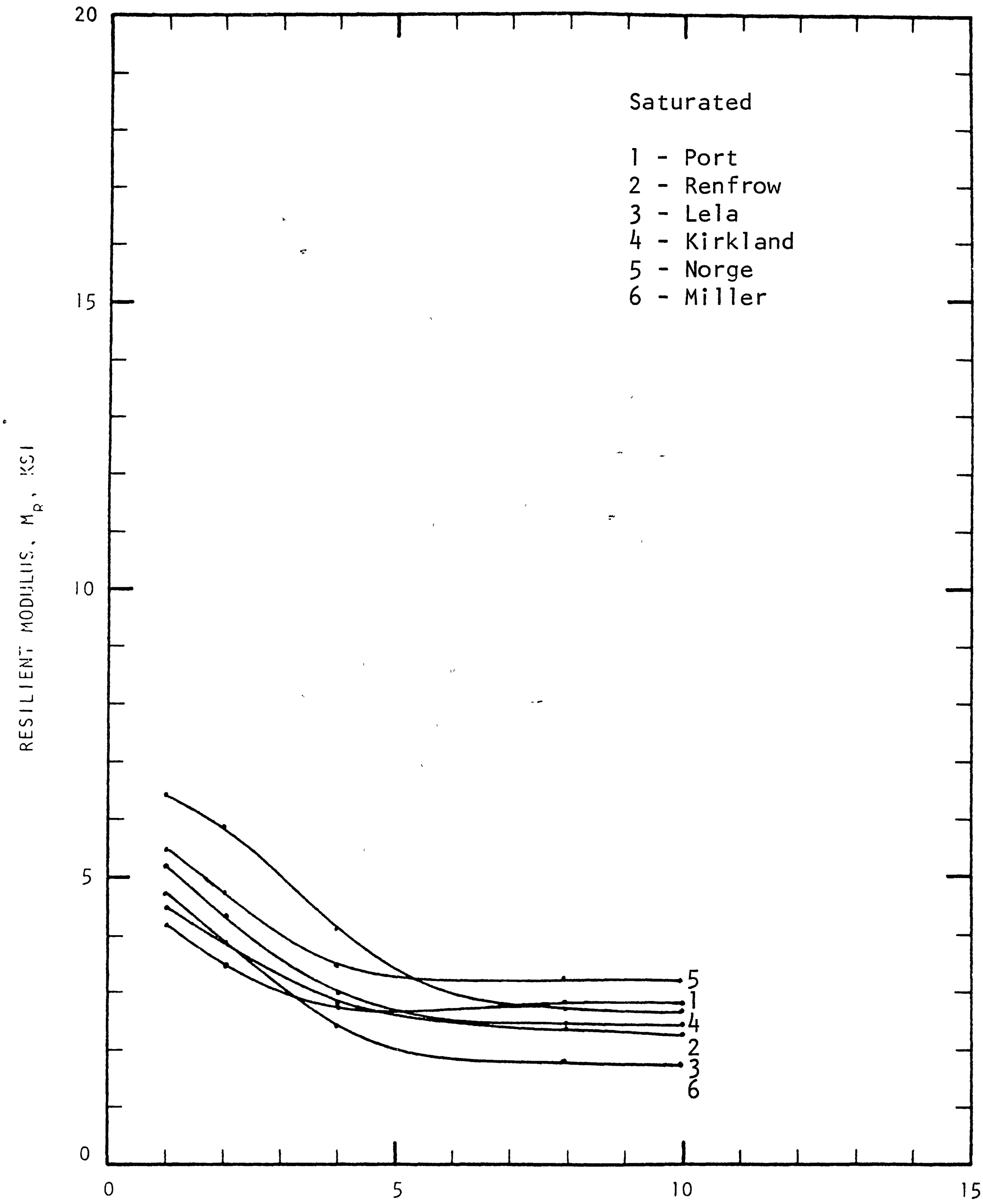


Figure 15d. Effect of Soil Type on M_R for Saturation Condition

tion.

Evaluating the order of M_R values (from highest to lowest) from Figure 15d, it appears to be independent of plasticity index (3, 4, 6, 2, 5, 1). The silty soils represent the high values of M_R and the clayey soils provided the low values of M_R under the saturated condition.

Based on the above analyses for the four sample conditions, the following may be concluded:

1. The higher the plasticity index of the soil, the higher the M_R for normal placement conditions.
2. For the saturated condition, the higher the plasticity index of the soil, the lower the M_R.
3. For the OMC-2.0 and OMC placement conditions, the greater the sand content of the soil, the higher the M_R. However, the M_R tended to decrease rapidly for the OMC+2.0 and saturated placement conditions.
4. Under all four conditions, the higher the silt content of the soil, the lower the M_R.
5. Higher value of Cu (well-graded) resulted in higher M_R values. This influence is more significant at the OMC sample placement condition.

Correlation Between M_R and OSI Number

In order to evaluate a correlation between M_R and OSI, the selected M_R values for each sample condition are based on the recommendation of Asphalt Institute (i.e., under condition of confining pressure equal to 2 psi and repeated deviator stress equal to 6 psi). Since a confining pressure of 2 psi was not used, the M_R curves for the nearest confining pressure (3 psi) and 6 psi deviator stress were used to establish the M_R value. The M_R values for all four sample placement conditions are listed in Table IX. For the normal conditions, M_R (log-scale) versus OSI is shown in Figure 16. For the saturated condition, because of the differences of M_R are small, an arithmetic scale (M_R versus OSI) was used in Figure 17.

The results in Figures 16 and 17, show that there is no significant correlation between M_R and OSI. This is not surprising, since most of the factors described in the previous analyses (i.e. sand content, silt content, and gradation) are not used in the determination of OSI.

Comparison of OSI and Asphalt Institute M_R-Based Design Thickness

Based on the previous review information, the latest flexible pavement design procedure (M_R-based) published by the Asphalt Institute was used to evaluate the suitability and validity of the OSI thickness design procedure. Design thicknesses determined using the OSI method for a typical Oklahoma State highway are listed in Table X. The aggregate base course thickness selected is the same as that assumed in the M_R-based design chart. The 2" adjustment of EBT used in Table X was recommended by the Oklahoma Department of Transportation, regardless of soil type and traffic volume. An example of the procedure for determininng the design thickness is described below.

Example: For Lela soil, 20 years design life.

OSI number = 32.2 (From Table V)

EBT = 37.0 in (From Figure 8)

TABLE IX

M_R VALUES FOR σ_3 = 3 PSI AND σ_D = 6 PSI

Soil Samples	Sample Conditions OMC-2.0	OMC	OMC+2.0	Saturated	OSI
Port	11.2	9.3	4.8	2.5	13.3
Renfrow	11.9	10.4	7.6	2.3	18.4
Lela	11.7	9.3	7.6	2.3	32.2
Kirkland	10.9	9.7	8.1	2.9	22.3
Norge	8.5	5.9	4.5	3.2	14.2
Miller	10.5	8.5	5.1	2.0	19.6

Unit of M_R = Ksi.

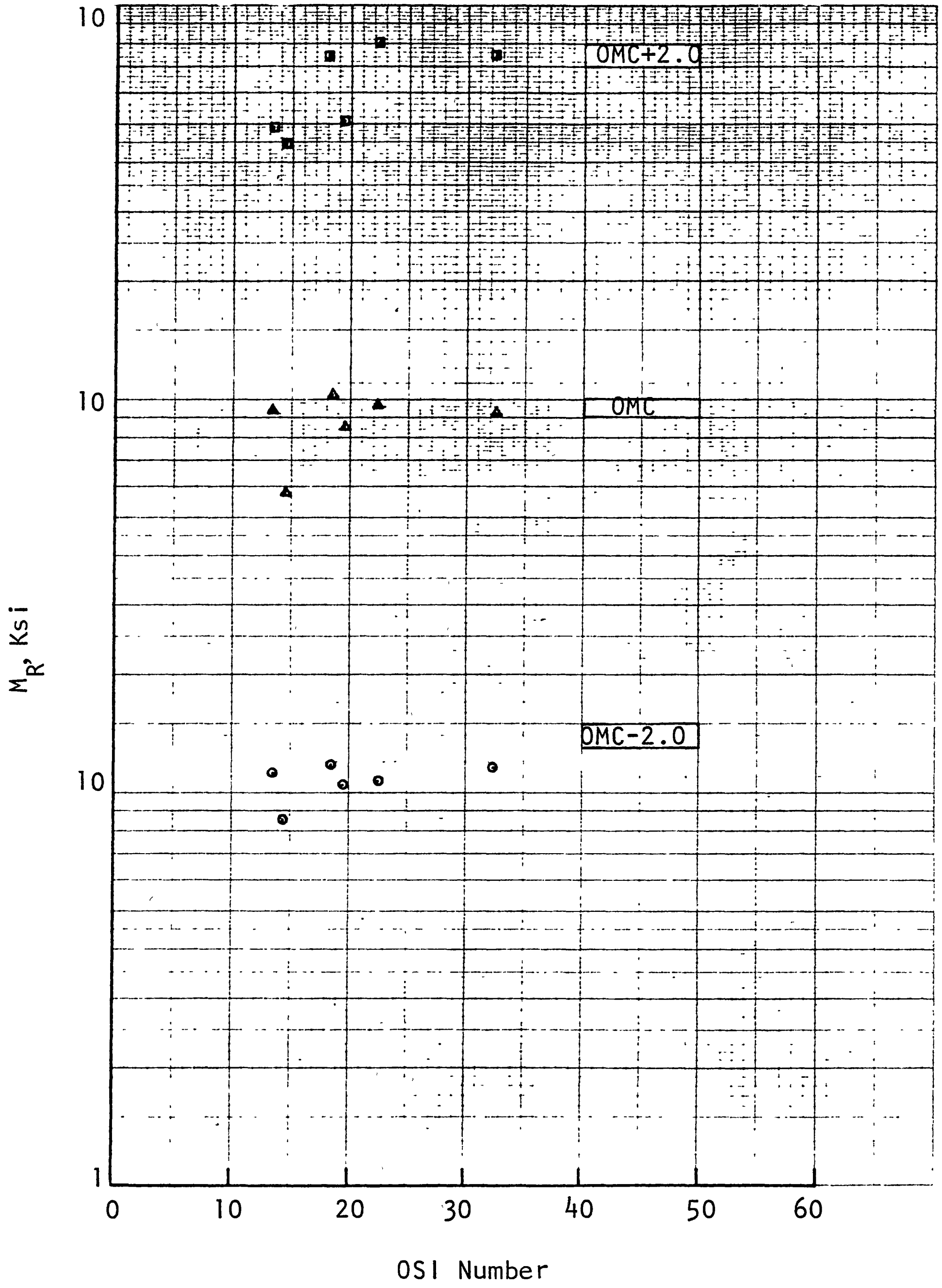


Figure 16. Relationship Between M_R and OSI

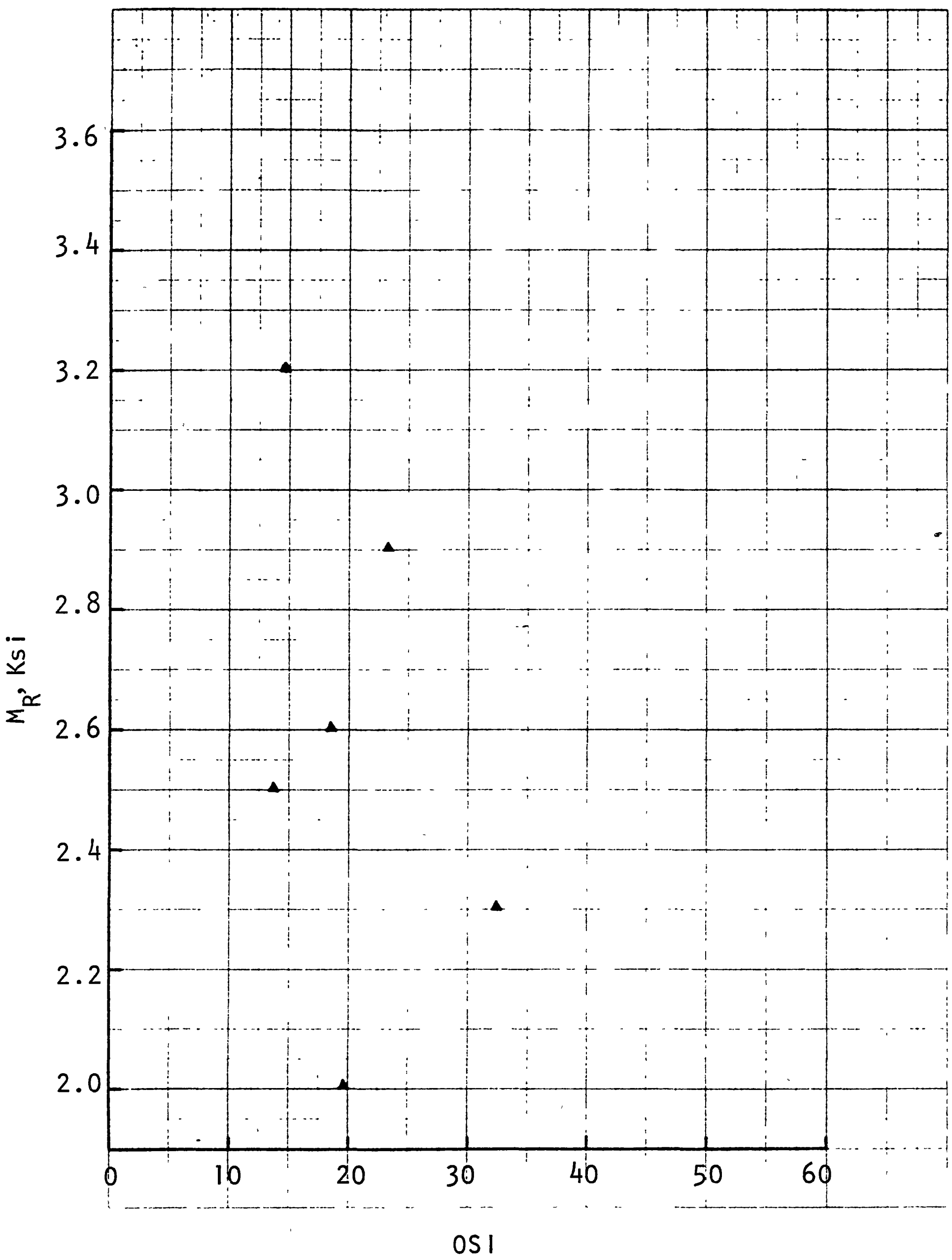


Figure 17. Relationship Between M_R and OSI for Saturated Specimens

TABLE X

DESIGN THICKNESSES BASED ON OSI METHOD

Soil Samples	OSI	Equivalent Base Thickness EBT (Inches)	Adjusted EBT (Inches)	Untreated Coarse Aggregate Base Thickness (Inches)	Asphalt Concrete Thickness (Inches)
Port	13.3	Minimum Requirement	Minimum Requirement	8.0	4.5
Renfrow	18.4	18.5	20.5	12.0	6.0
Lela	32.2	37.0	39.0	18.0	14.0
Kirkland	22.3	22.5	24.5	12.0	8.5
Norge	14.2	Minimum Requirement	Minimum Requirement	8.0	4.5
Miller	19.6	19.5	21.5	12.0	6.5

Adjusted EBT = 37.0 + 2.0

= 39.0 in

Thickness of Untreated Coarse Aggregate Base = 18.0 in

(Selected same thickness as the typical M_R-based design chart used.)

= 18.0 in of EBT

EBT for Asphalt Concrete = 39.0 - 18.0

= 21.0

Thickness of Asphalt Concrete = 21.0/1.5

= 14.0 in

The design thicknesses for the M_R-based procedure used the M_R value for a subgrade with moisture content close to OMC, and the estimated 20 year traffic volume provided by Oklahoma Department of Transportation. The design charts used for determining the thickness of asphalt concrete are shown in Reference 18. The required pavement thicknesses shown in Table XI correspond to the same depth of aggregate base course used in the OSI design data.

The data given in Tables X and XI indicate that:

1. Under normal placement conditions (OMC-2.0, OMC, OMC+2.0), the OSI design method requires a thinner pavement section for Port and Norge soils than is required by the M_R-based method.

2. For the Lela and Kirkland soils, a thicker pavement section is required by the OSI method than by the M_R-based method.

3. For the Renfrow and Miller soils, the two design methods produce about the same results.

These findings indicate that the OSI thickness design method may provide either too little or too much pavement structure for some types of Oklahoma soils. For the low M_R values associated with wet periods

TABLE XI

REQUIRED ASPHALT CONCRETE THICKNESSES USING M_R-BASED DESIGN CHARTS

Soil Samples	Sample Conditions	M_R (10^3 psi)	20 Years Traffic Volume*	Required Thickness of A.C. (in)
Port	OMC-2.0	11.2	9.5×10^5	6.0
	OMC	9.3	9.5×10^5	6.5
	OMC+2.0	4.8	9.5×10^5	9.0
Renfrow	OMC-2.0	11.9	9.5×10^5	5.5
	OMC	10.4	9.5×10^5	6.0
	OMC+2.0	7.6	9.5×10^5	7.0
Lela	OMC-2.0	11.7	9.5×10^5	5.5
	OMC	9.3	9.5×10^5	6.0
	OMC+2.0	7.6	9.5×10^5	6.5
Kirkland	OMC-2.0	10.9	9.5×10^5	5.5
	OMC	9.7	9.5×10^5	6.0
	OMC+2.0	8.1	9.5×10^5	6.5
Norge	OMC-2.0	8.5	9.5×10^5	7.0
	OMC	5.9	9.5×10^5	8.5
	OMC+2.0	4.5	9.5×10^5	9.0
Miller	OMC-2.0	10.5	9.5×10^5	6.0
	OMC	8.5	9.5×10^5	6.5
	OMC+2.0	5.1	9.5×10^5	7.5

*Number of 20 years EAL (Equivalent 18,000-lb Single Axle Load), provided by Oklahoma Department of Transportation.

(saturated condition) rapid progress toward fatigue failure may occur. It appears that the OSI method places undue emphasis on the potential for damage from unstable plastic soils as they become saturated. Greater design thicknesses are used for the more plastic soils (Lela and Kirkland) and the lesser design thicknesses for the less plastic soils (Port and Norge) than can be justified by the M_R-based design method.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

The objectives of this investigation were to determine the basic engineering and resilient modulus properties for typical Oklahoma cohesive soils. The objectives were obtained through a laboratory testing program and analyses.

Conclusions

The results of the testing program and the analyses described herein indicate the following conclusions:

1. For fine-grained soils, the confining pressure has little or no influence on the M_R under the normal conditions (OMC-2.0, OMC, OMC+2.0).

2. Under saturated condition, the confining pressure significantly influences the resilient modulus of fine-grained soil.

3. The plasticity of fine-grained soil is a major factor controlling the magnitude of M_R. Other factors affecting the resilient property include sand content, silt content, and gradation. The influences of these factors, except gradation, under normal conditions is generally opposite that for saturated condition. For fine-grained soils with similar plasticity, the higher the sand content (or the lower the silt content) the higher the M_R under normal conditions.

4. For the high degree of compaction used (greater than 95%), the magnitude of M_R decreases with increasing degree of saturation. The dry

density shows no significant influence on M_R.

5. There was no correlation between OSI and M_R.

6. The OSI method provides the minimum required design thickness of pavement for silty subgrades. For high plasticity subgrades, the OSI method provides too large of design thickness, even so, fatigue failure still may not be prevented for the extremely low M_R values during the wet season. The OSI pavement thickness design method does not appear to provide the optimum design for the typical Oklahoma cohesive soils.

Recommendations

1. A high speed strip chart recorder could be used for measuring the resilient deformation in future studies. This type of recorder would provide more details about the resilient properties.

2. Optimum design thickness should be determined by considering weather changes, moisture condition changes and the growth of traffic volume. In any future studies, the computer program provided by the Asphalt Institute should be used.

3. A study on the effect of stabilization of the subgrade for improving the resilient properties under the saturated condition should be undertaken.

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APPENDIX A

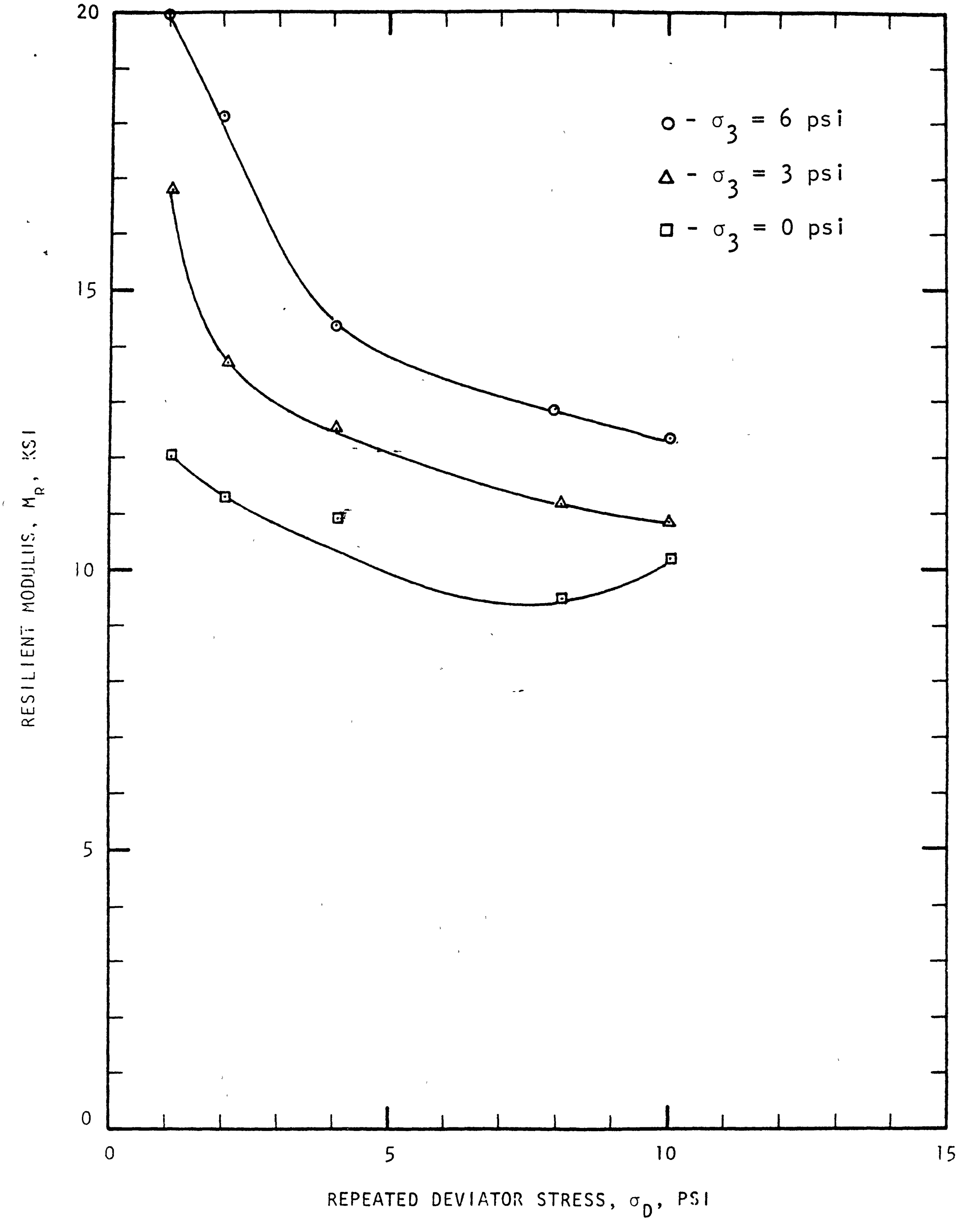

Figure 18. Resilient Modulus Testing Results for Port Soil, (OMC-2.0), Specimen 1

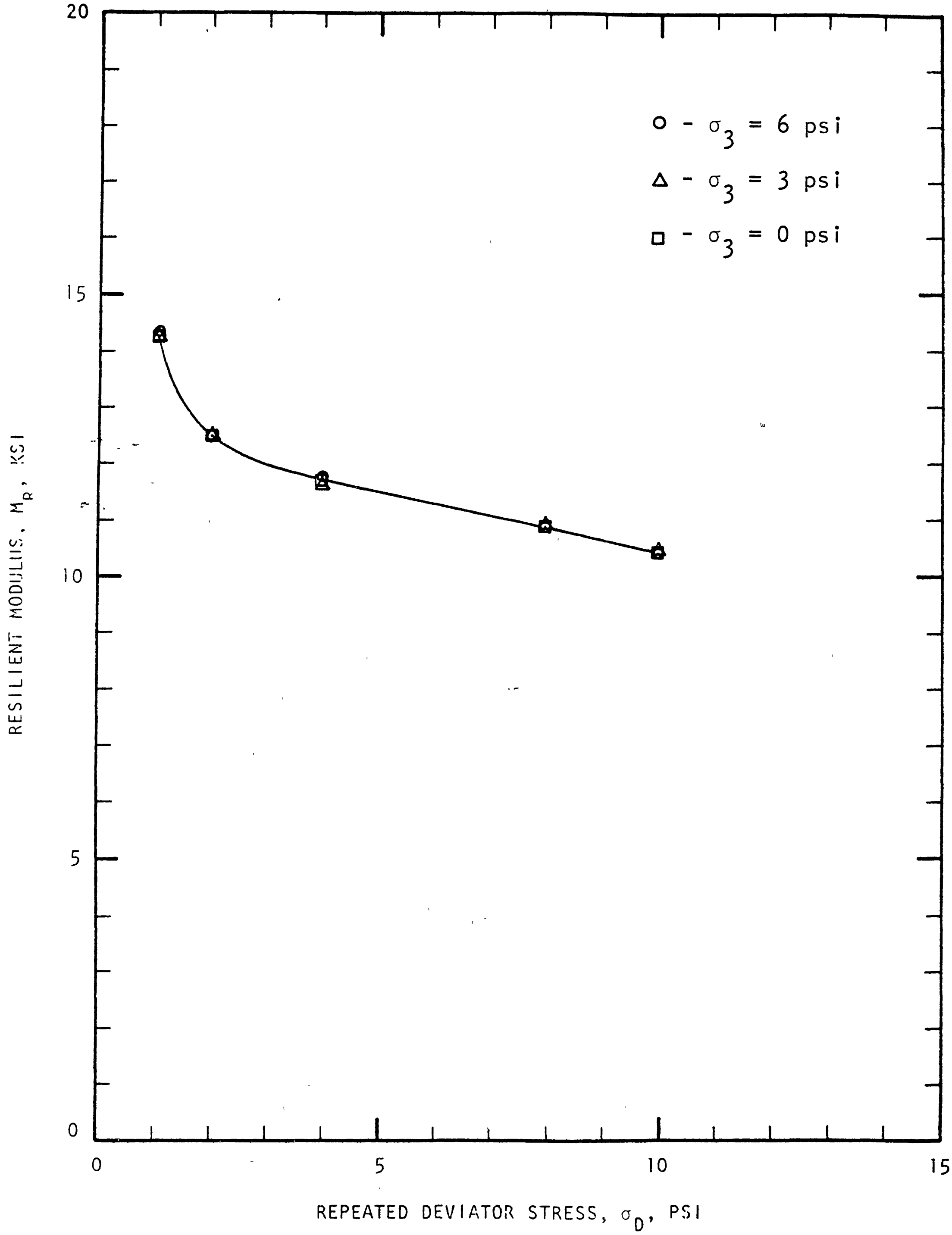

Figure 19. Resilient Modulus Testing Results for Port Soil, (OMC-2.0), Specimen 2

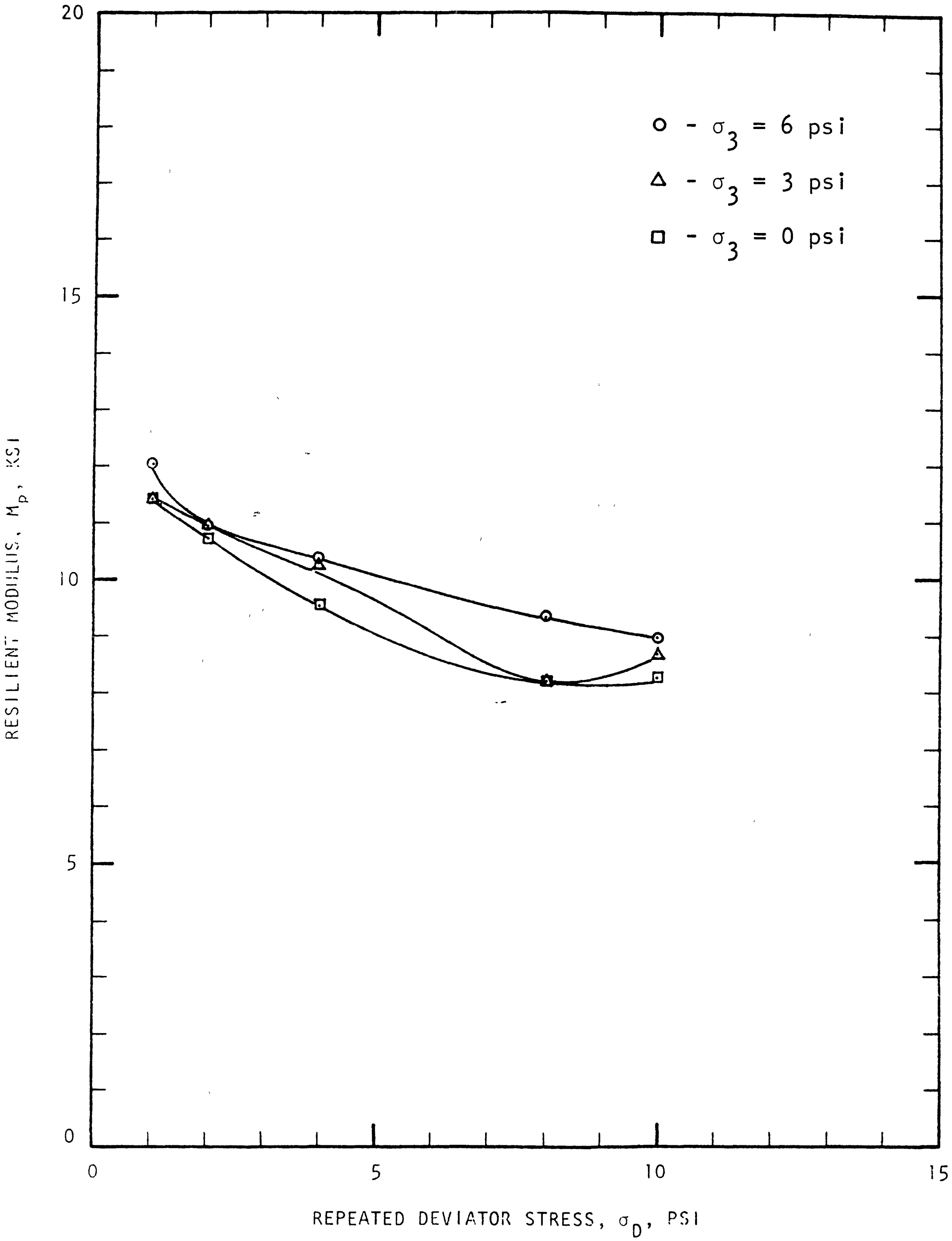

Figure 20. Resilient Modulus Testing Results for Port Soil, (OMC), Specimen 1

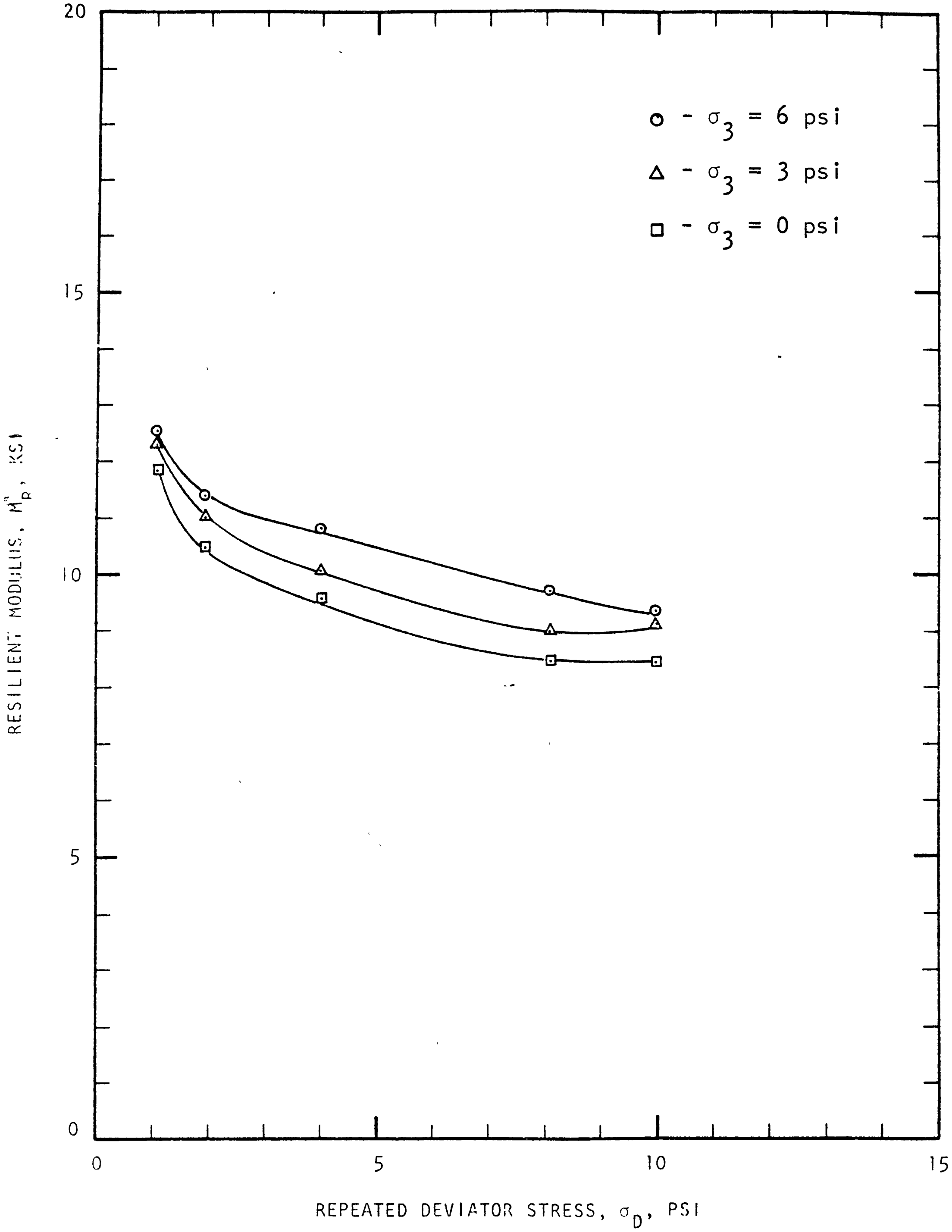

Figure 21. Resilient Modulus Testing Results for Port Soil, (OMC), Specimen 2

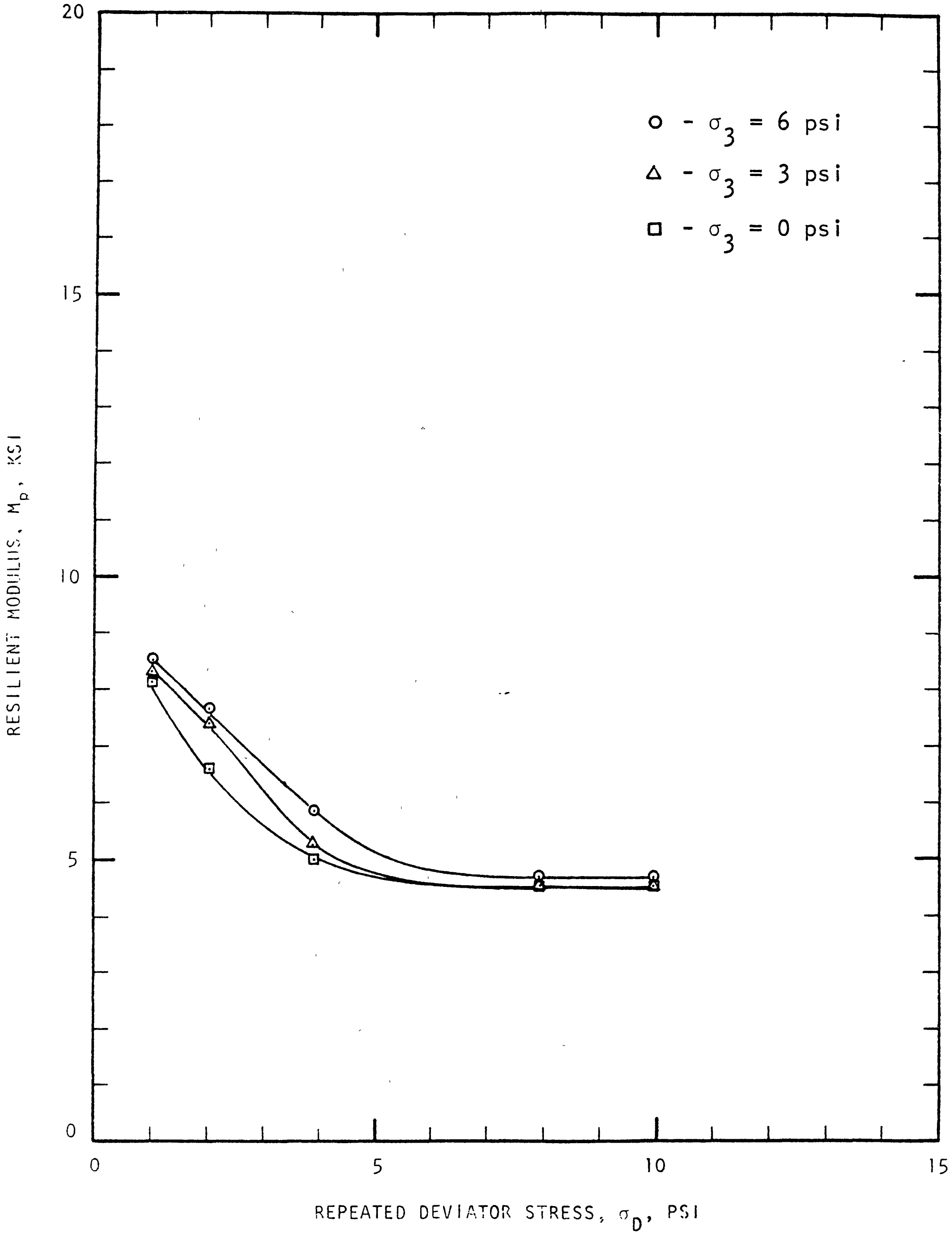

Figure 22. Resilient Modulus Testing Restuls for Port Soil, (OMC+2.0), Specimen 1

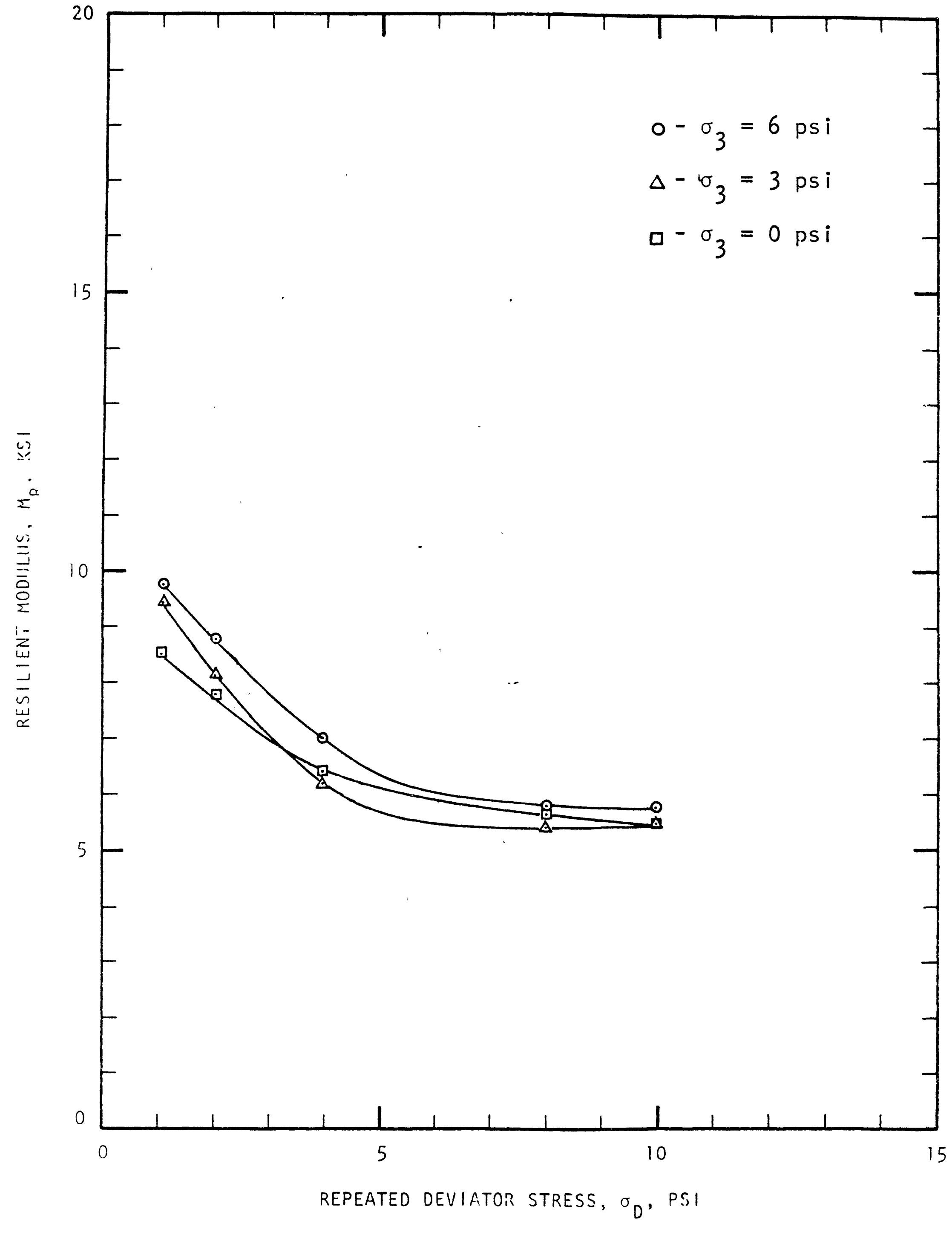

Figure 23. Resilient Modulus Testing Results fro Port Soil, (OMC+2.0), Specimen 2

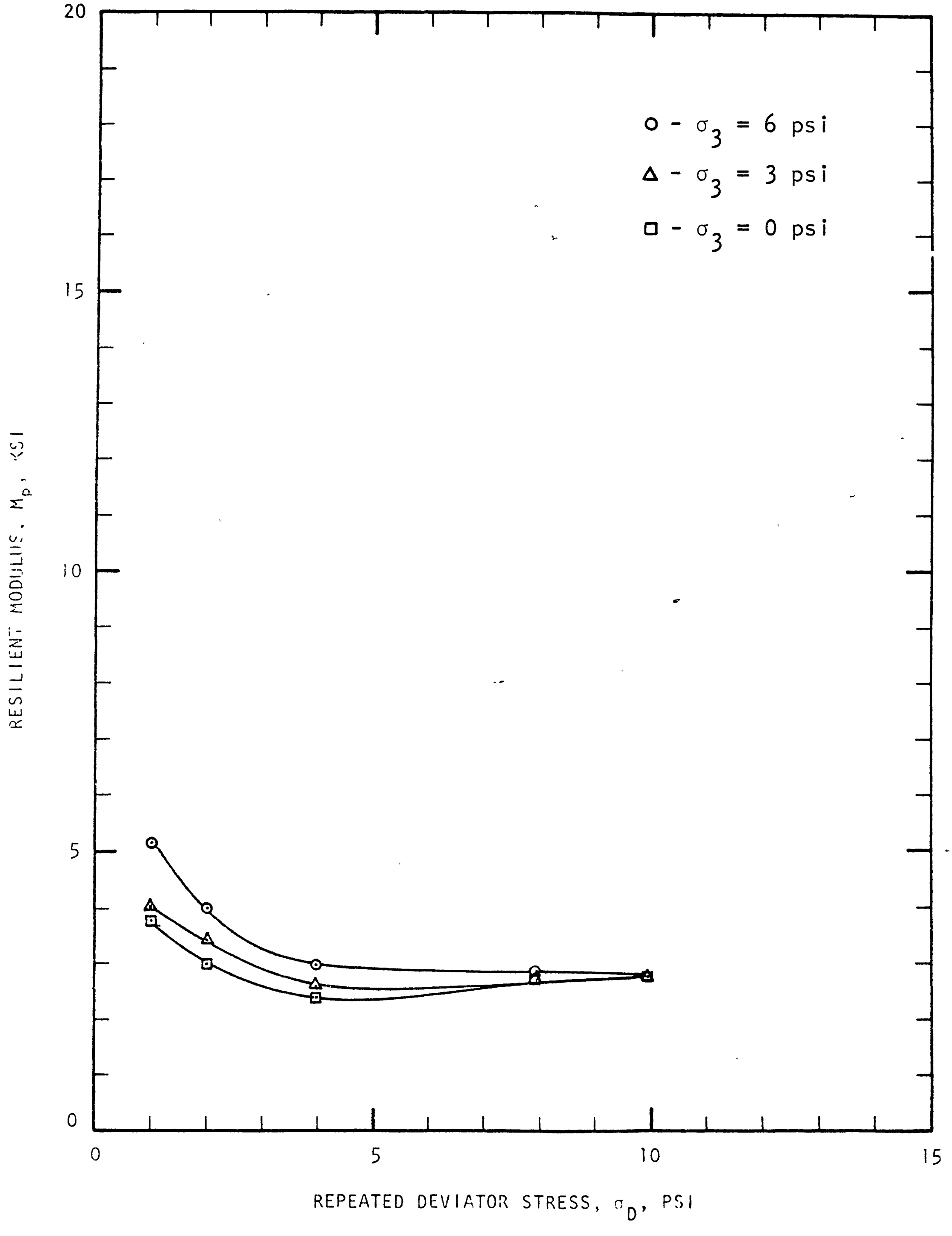

Figure 24. Resilient Modulus Testing Results for Port Soil, (Saturated), Specimen 1

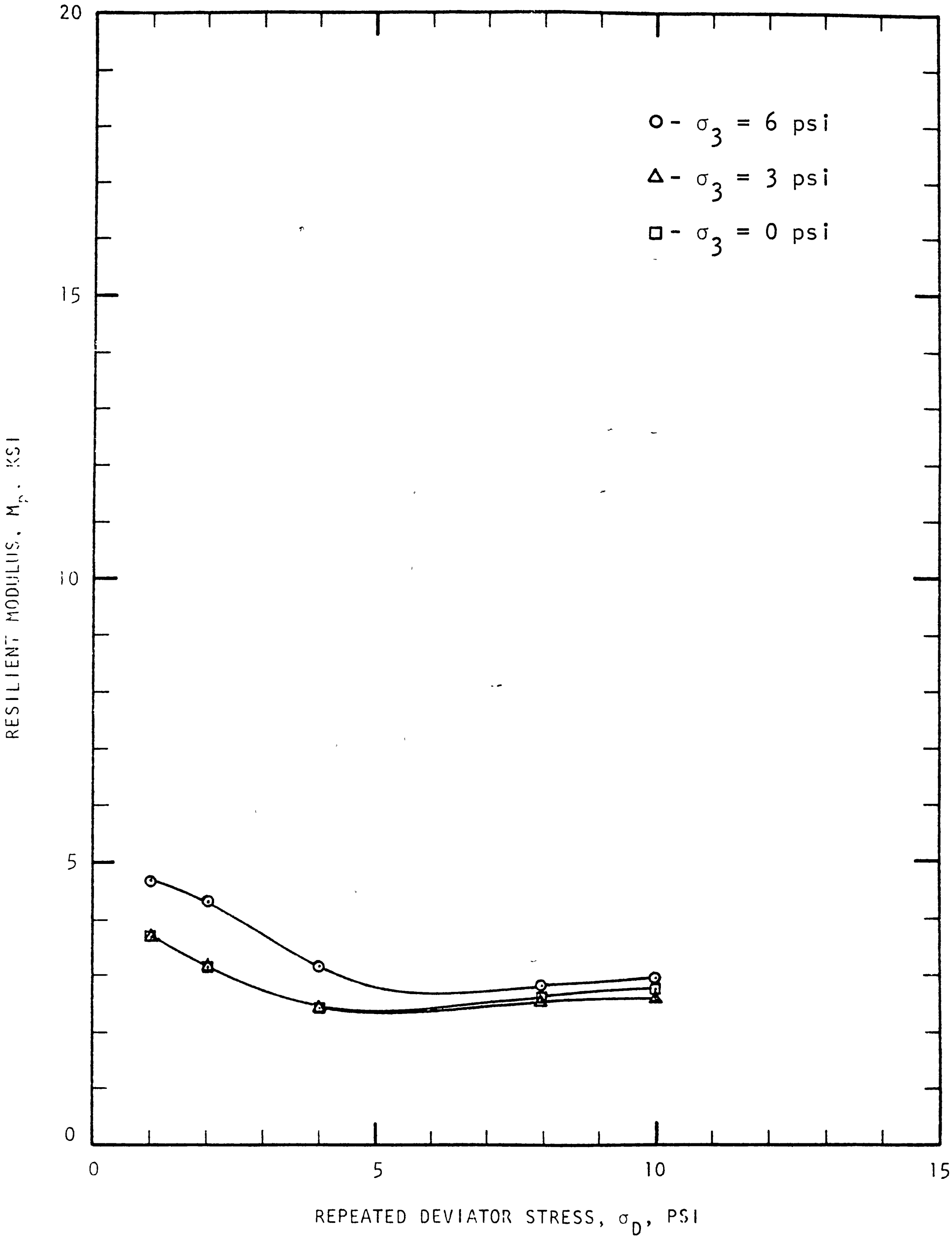

Figure 25. Resilient Modulus Testing Results for Port Soil, (Saturated), Specimen 2

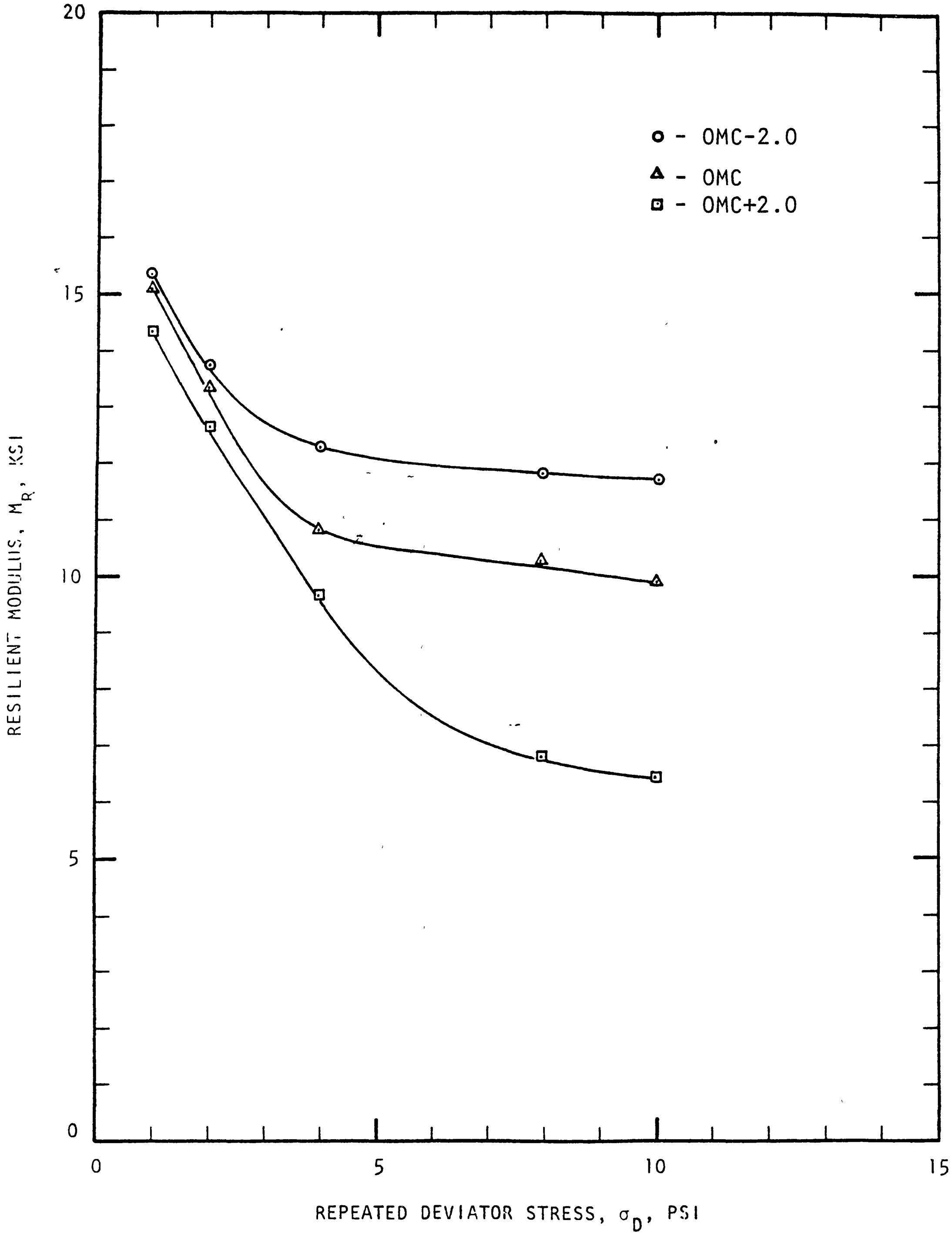

Figure 26. Resilient Modulus Testing Average Results for Each σ_3, Renfrow Soil

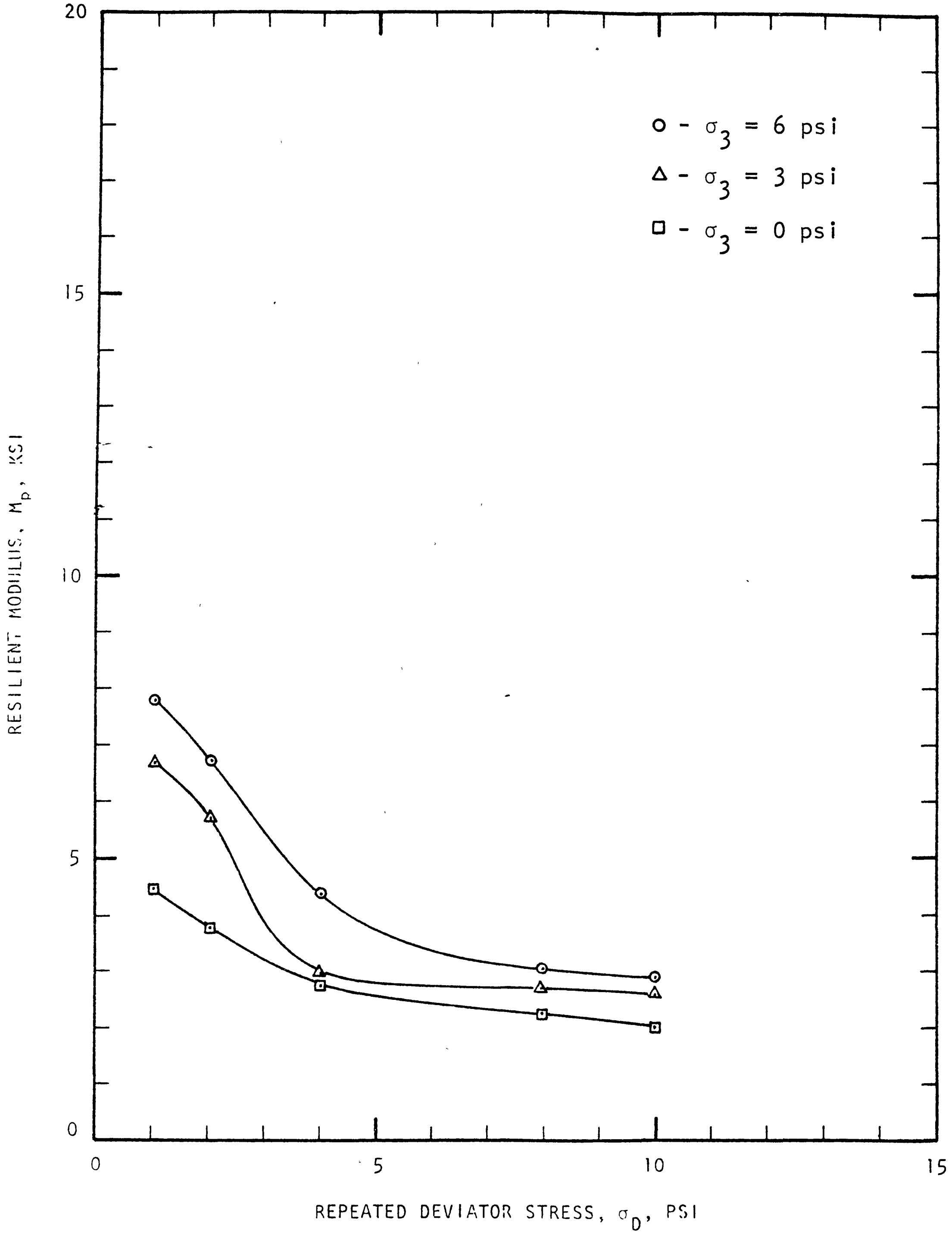

Figure 27. Resilient Modulus Testing Results for Renfrow Soil, (Saturated), Specimen 1

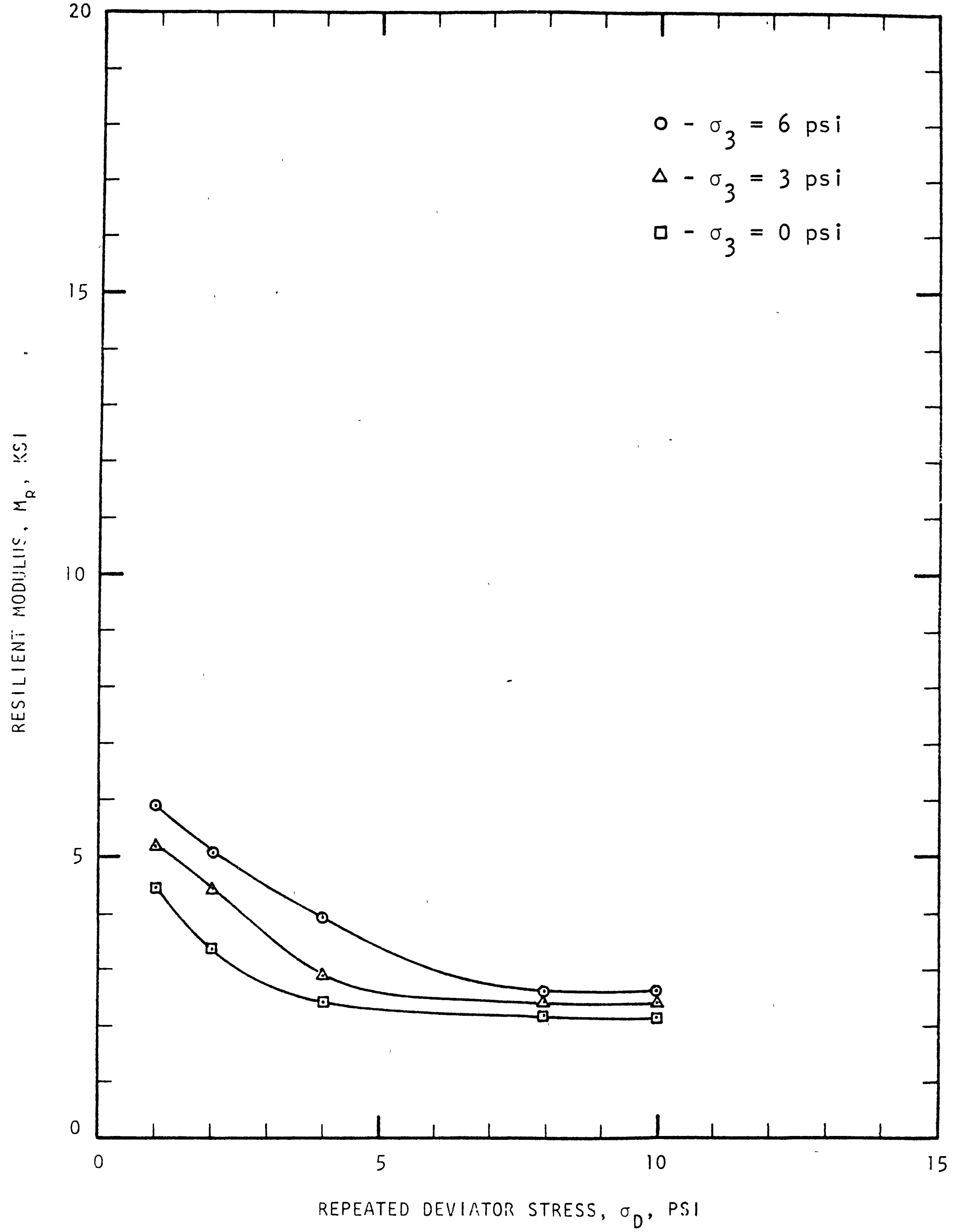

Figure 28. Resilient Modulus Testing Results for Renfrow Soil, (Saturated), Specimen 2

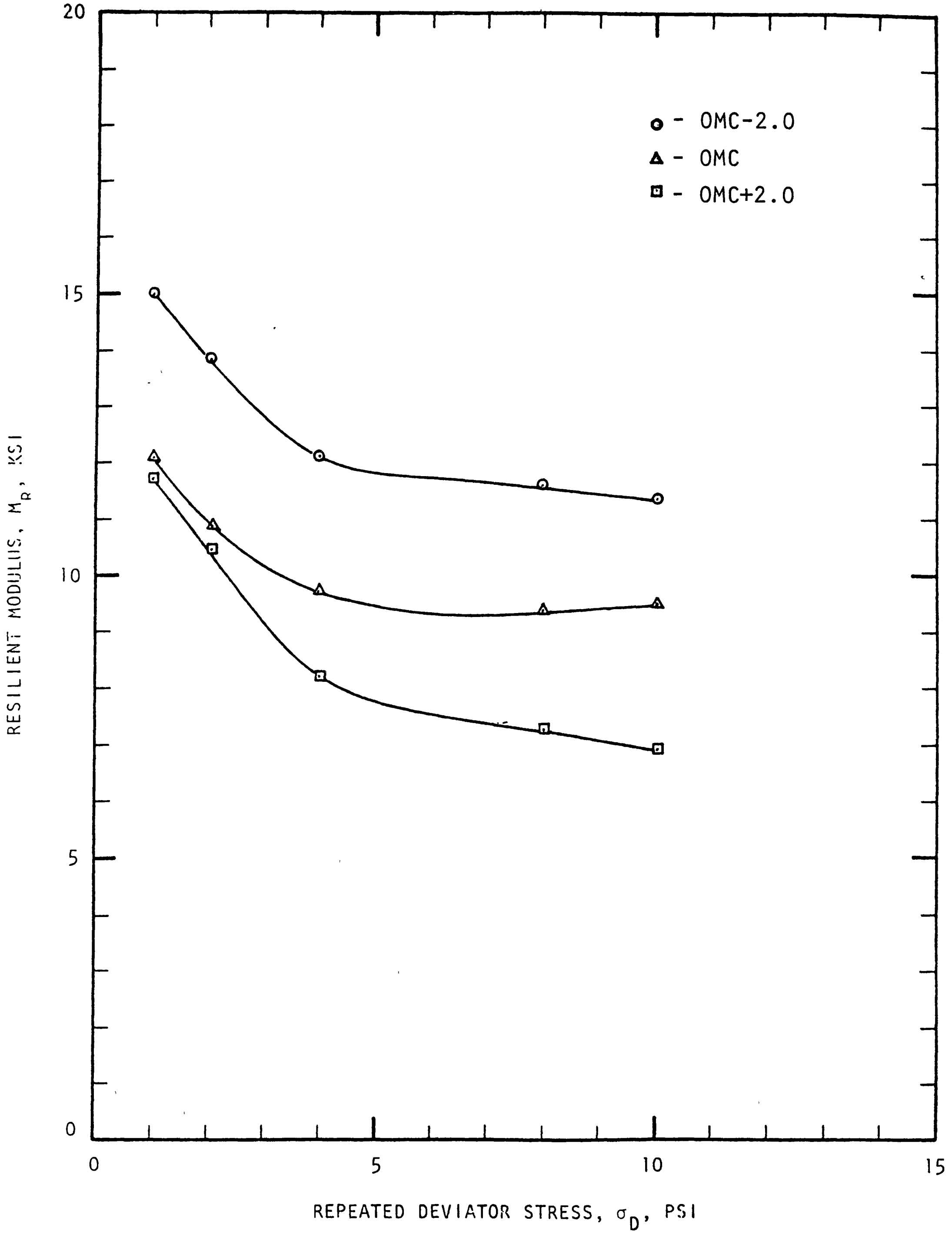

Figure 29. Resilient Modulus Testing Average Results for Each σ_3, Lela Soil

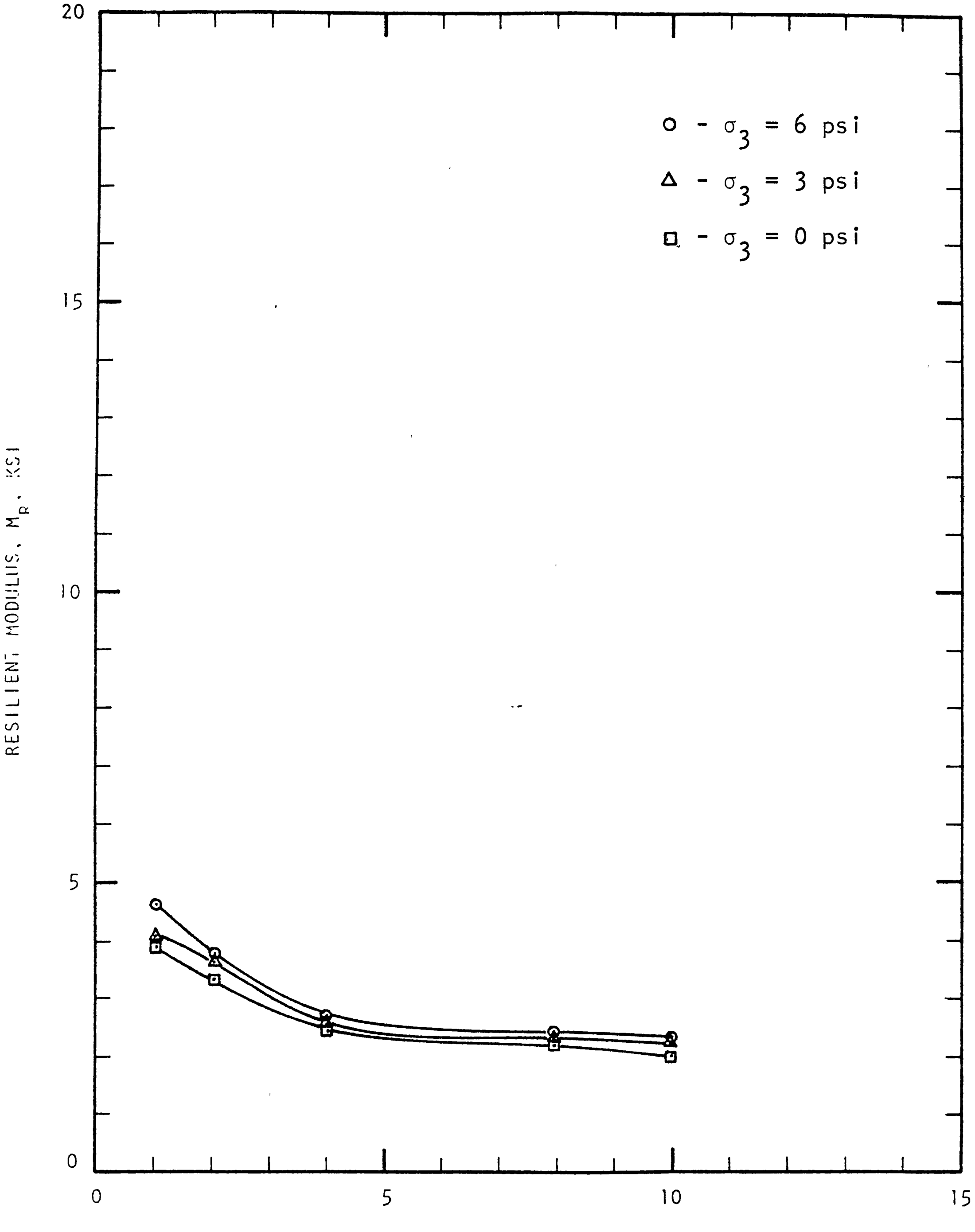

Figure 30. Resilient Modulus Testing Results for Lela Soil, Saturated, Specimen 1

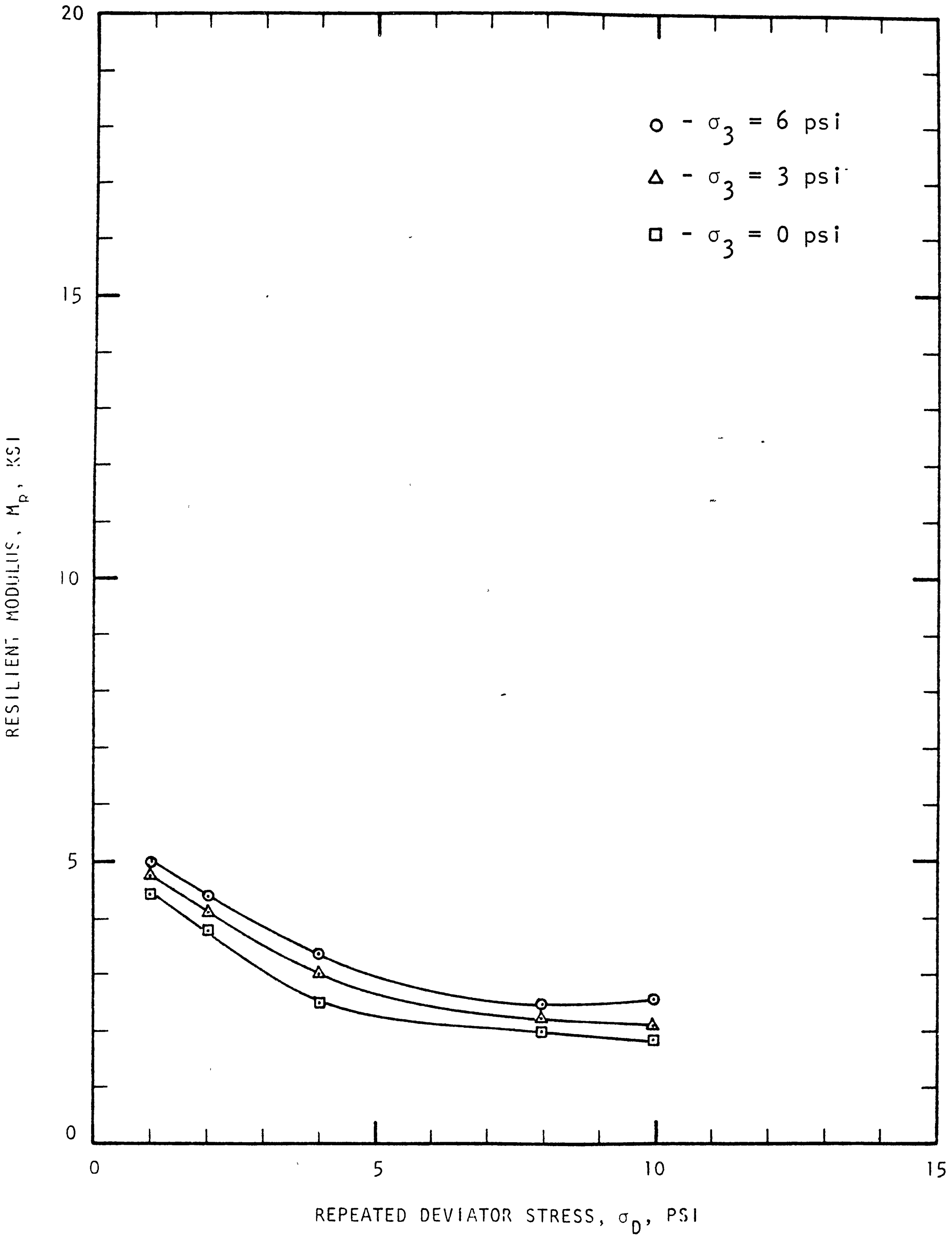

Figure 31. Resilient Modulus Testing Results for Lela Soil, Saturated, Specimen 2

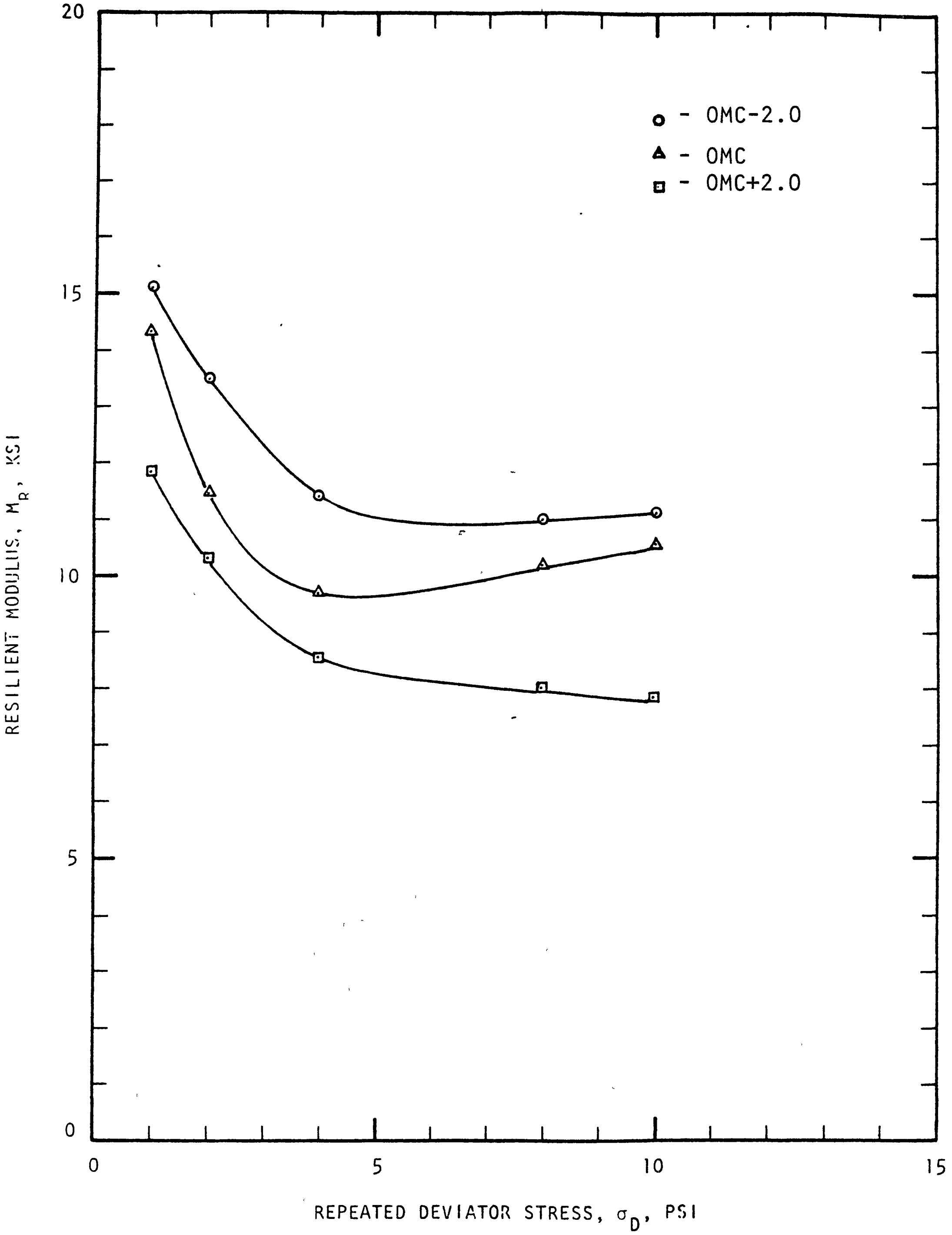

Figure 32. Resilient Modulus Testing Average Results for Each σ_3, Kirkland Soil

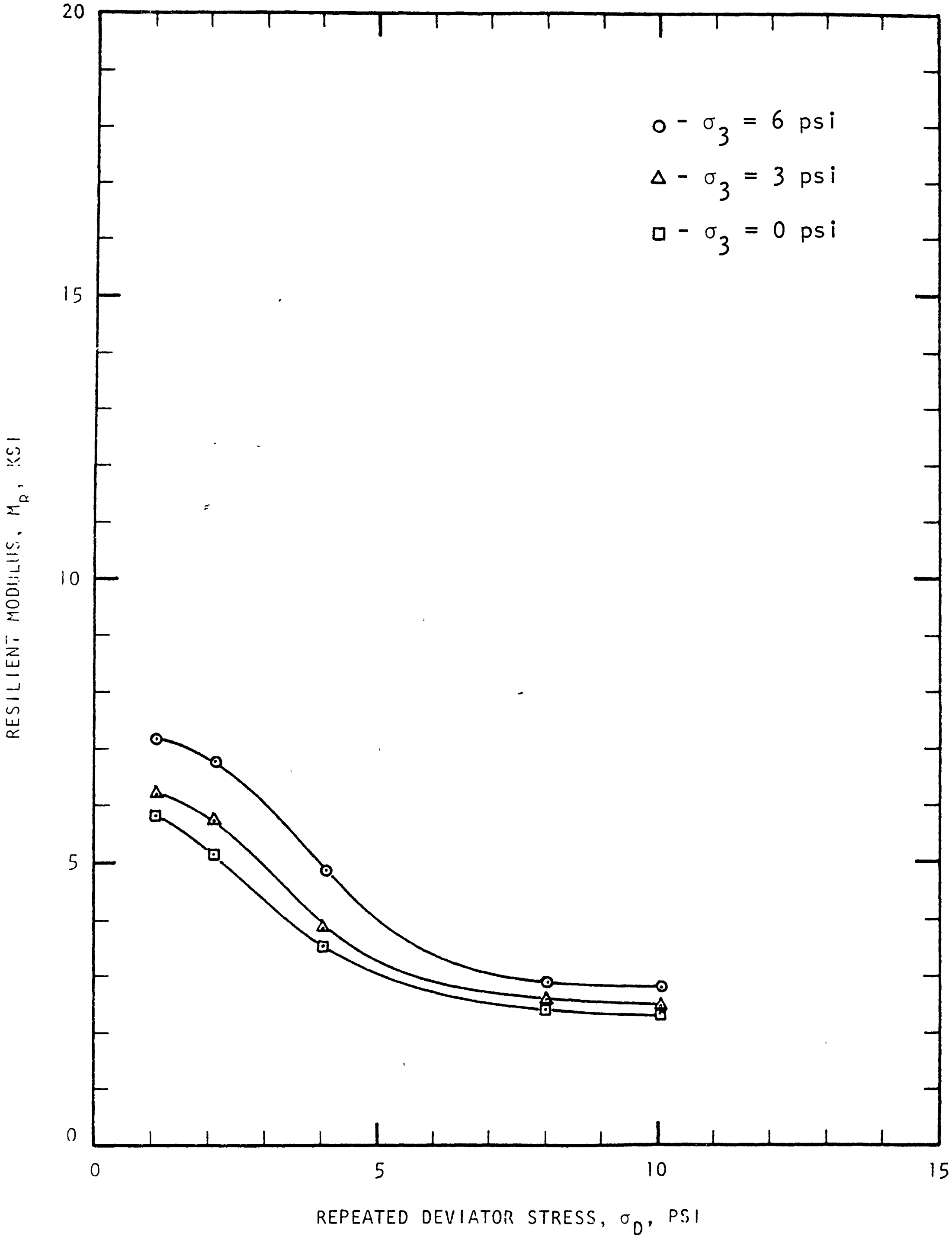

Figure 33. Resilient Modulus Testing Results for Kirkland Soil, Saturated, Specimen 1

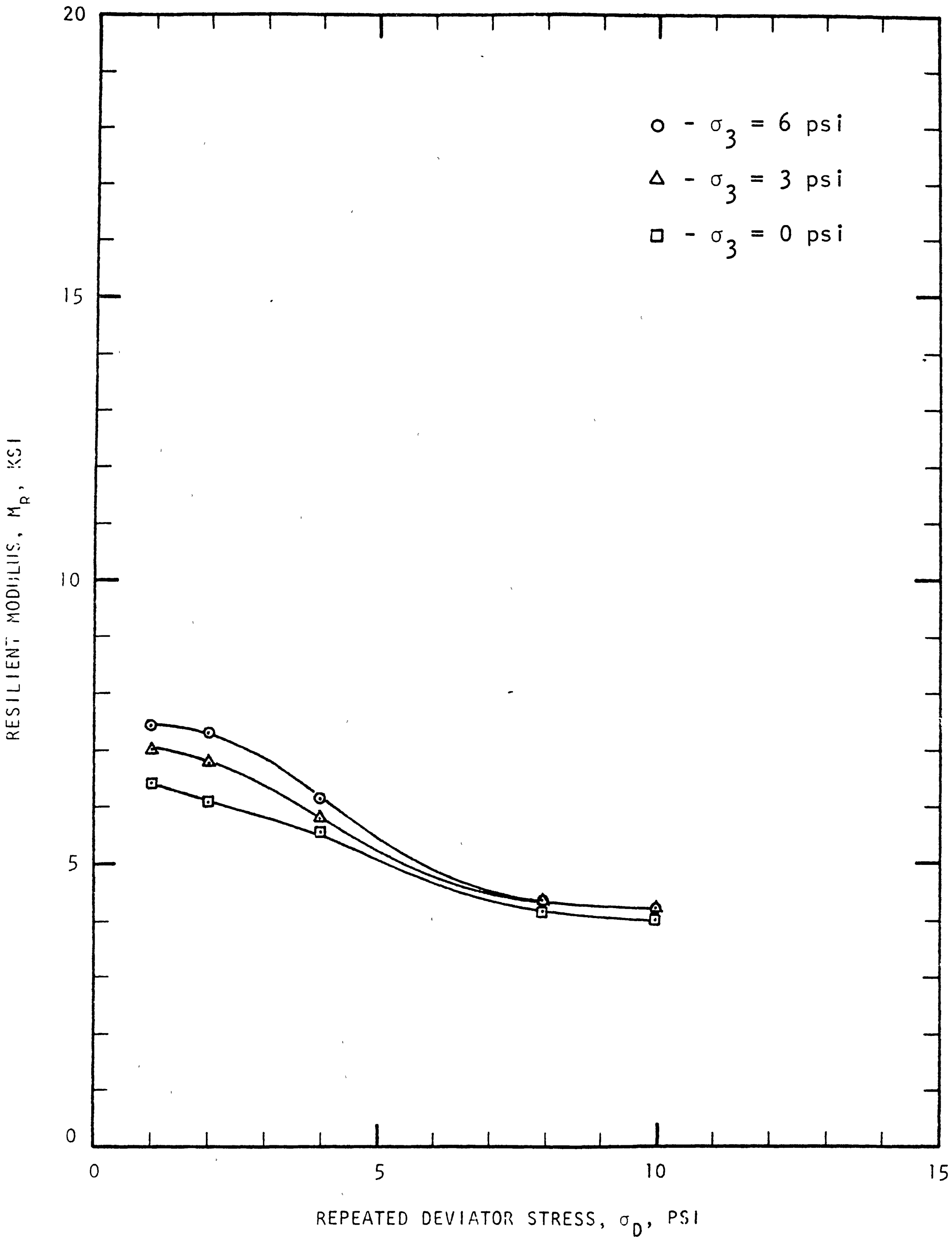

Figure 34. Resilient Modulus Testing Results for Kirkland Soil, Saturated, Specimen 2

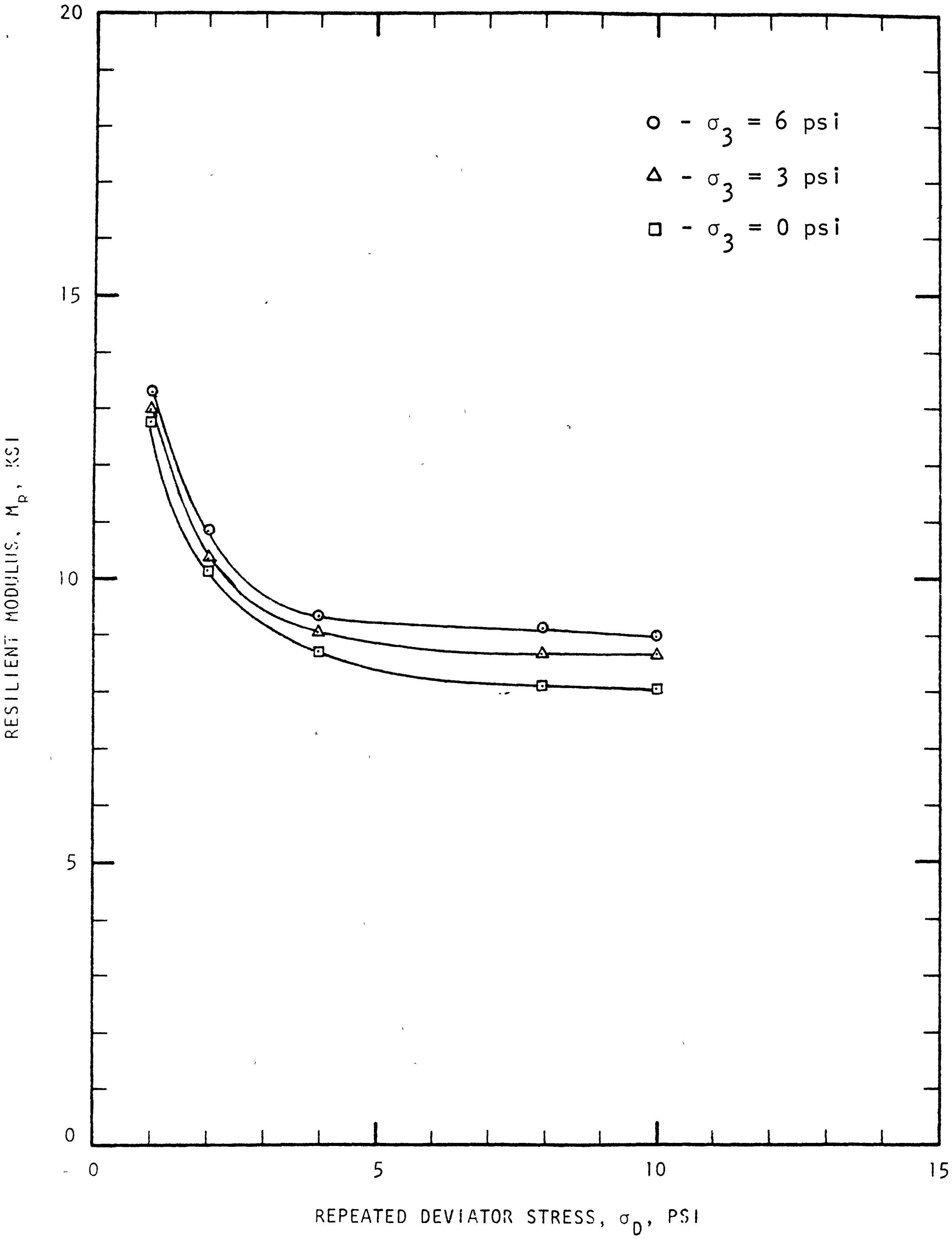

Figure 35. Resilient Modulus Testing Results for Norge Soil, OMC-2.0, Specimen 1

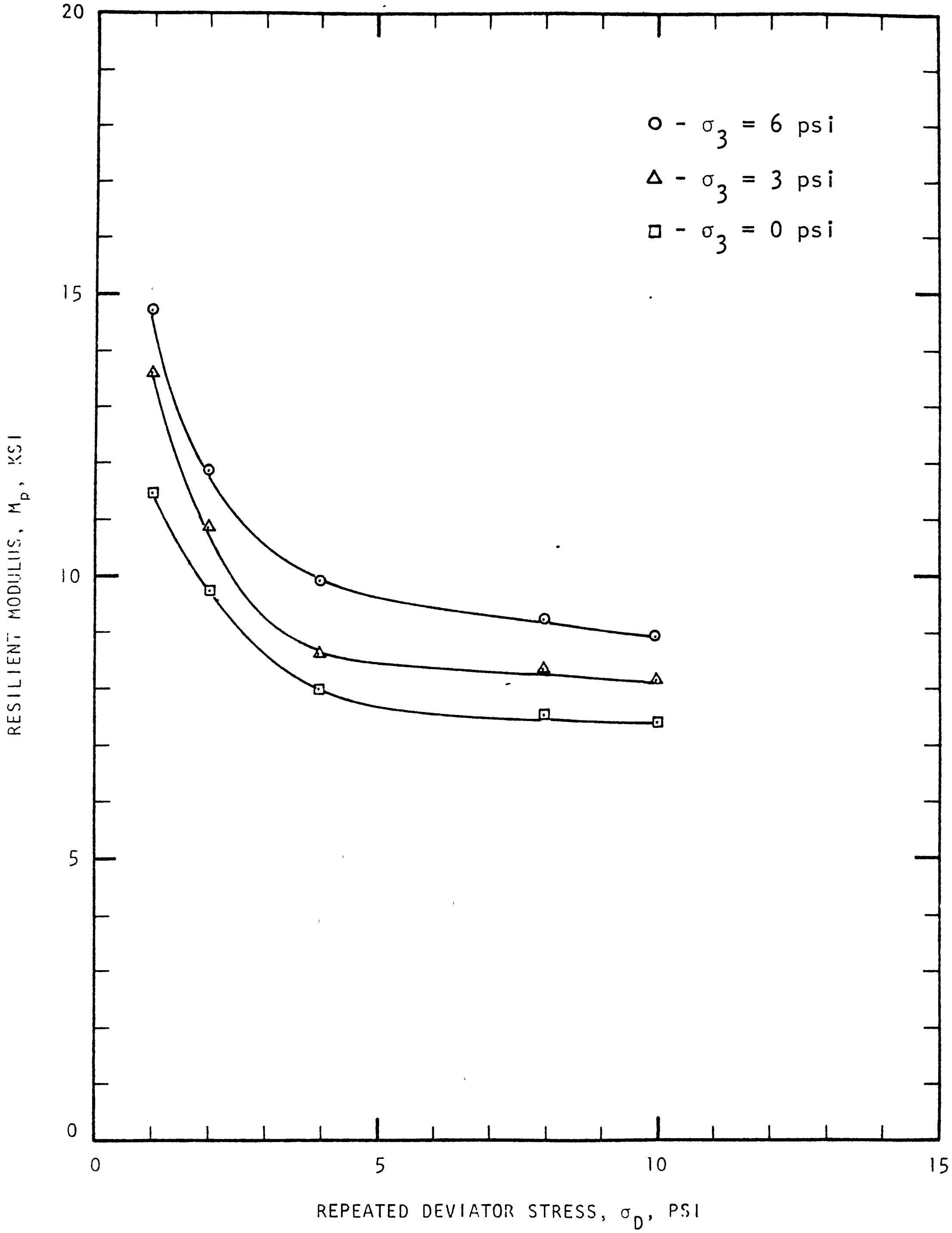

Figure 36. Resilient Modulus Testing Results for Norge Soil, OMC-2.0, Specimen 2

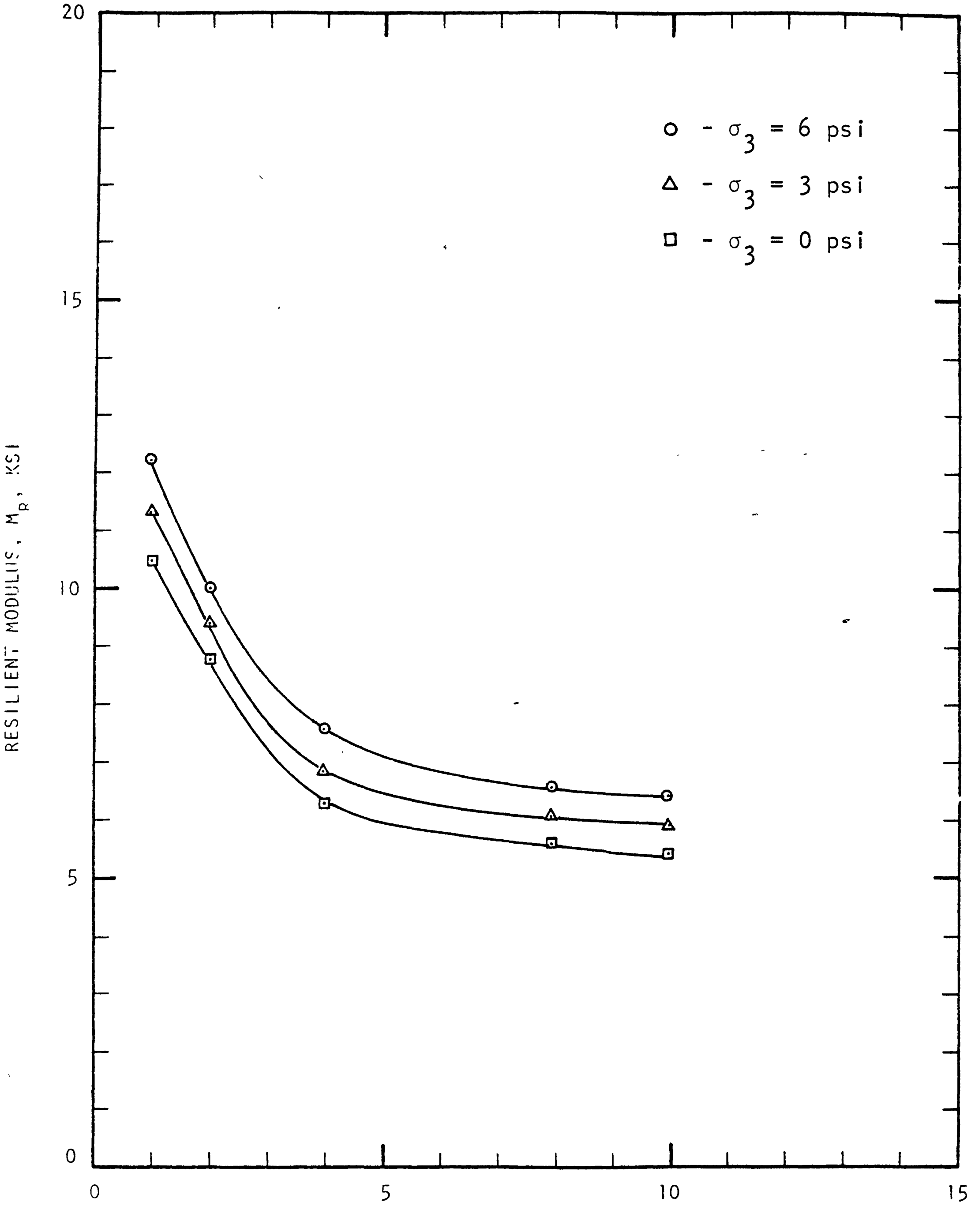

Figure 37. Resilient Modulus Testing Results for Norge Soil, OMC, Specimen 1

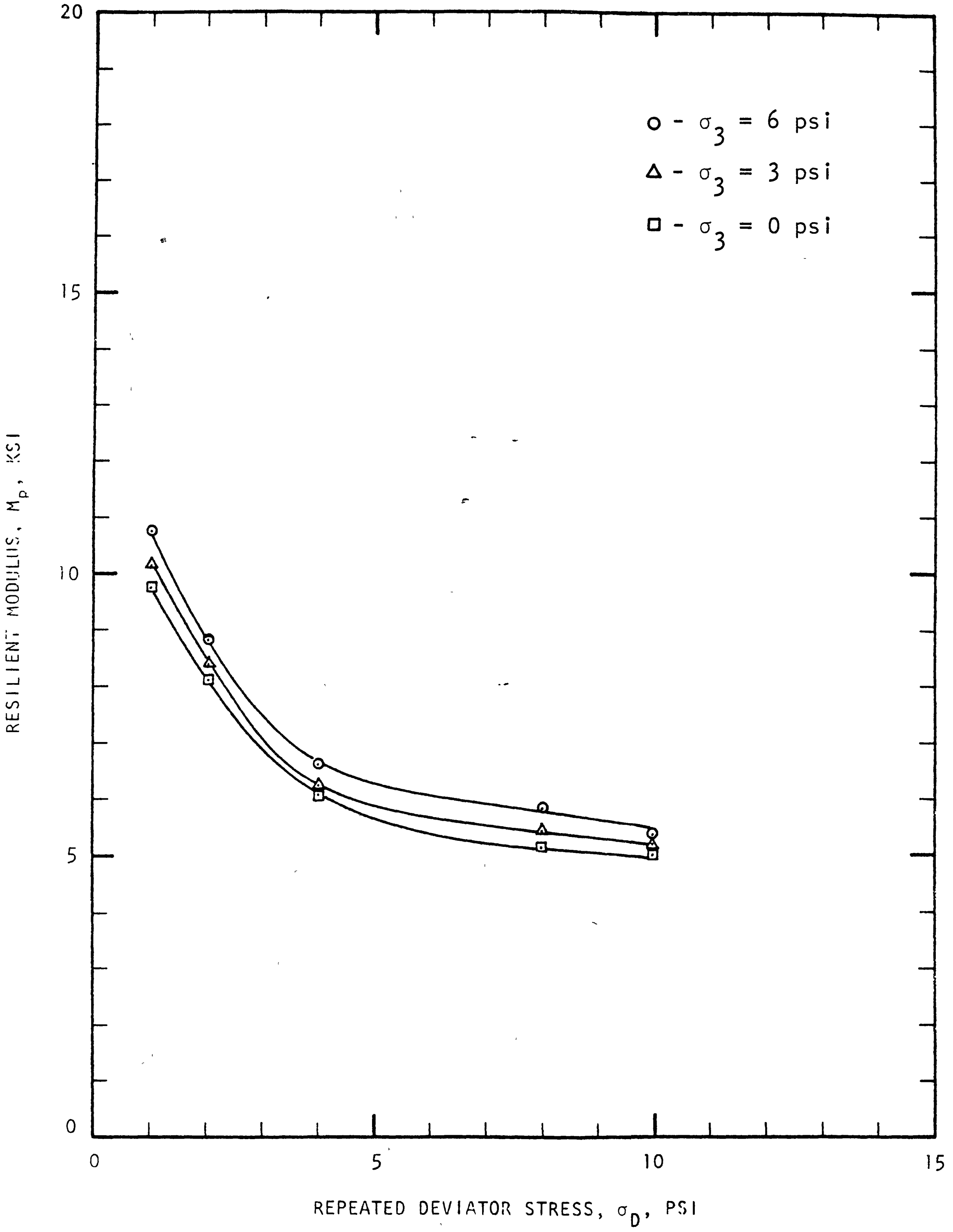

Figure 38. Resilient Modulus Testing Results for Norge Soil, OMC, Specimen 2

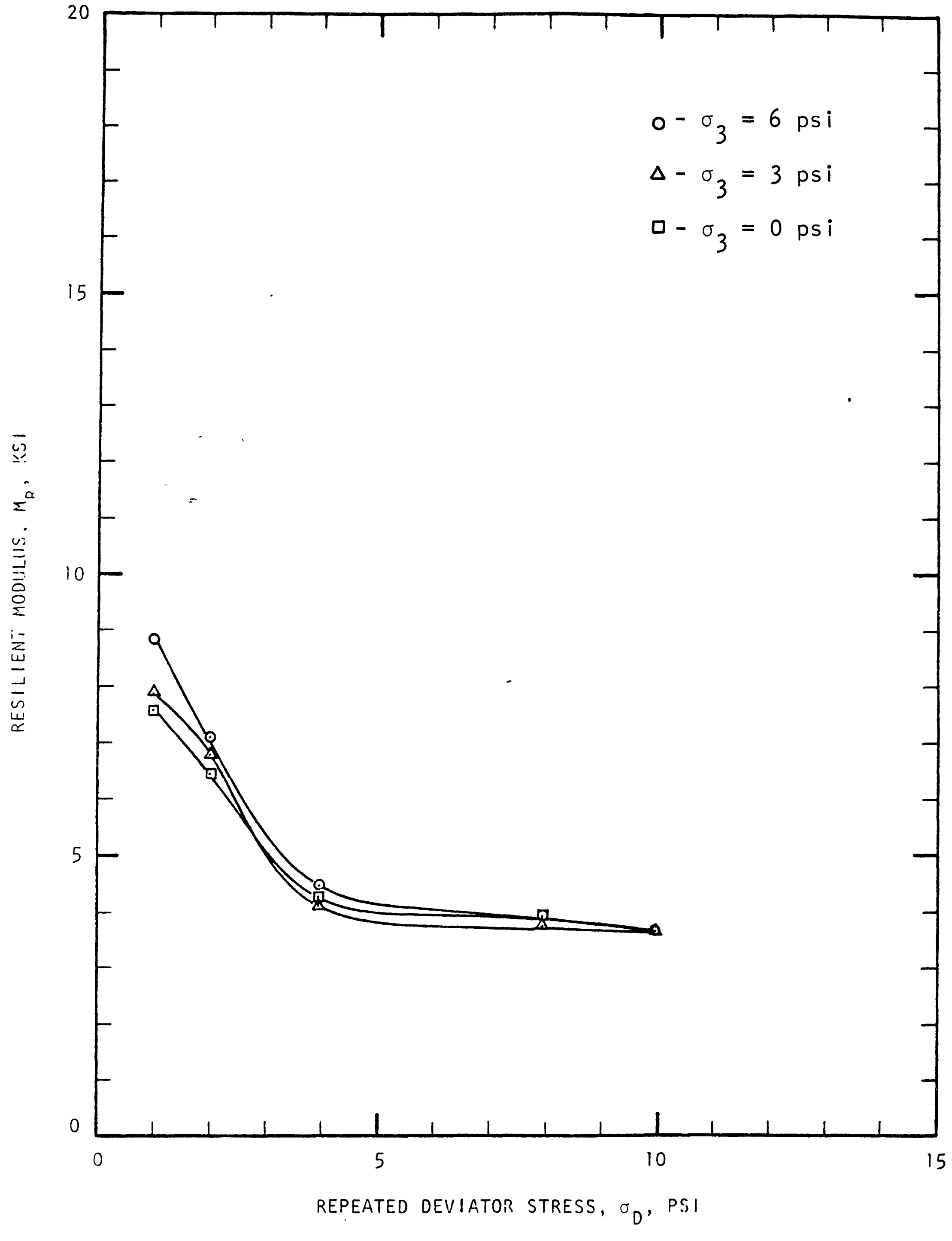

Figure 39. Resilient Modulus Testing Results for Norge Soil, OMC+2.0, Specimen 1

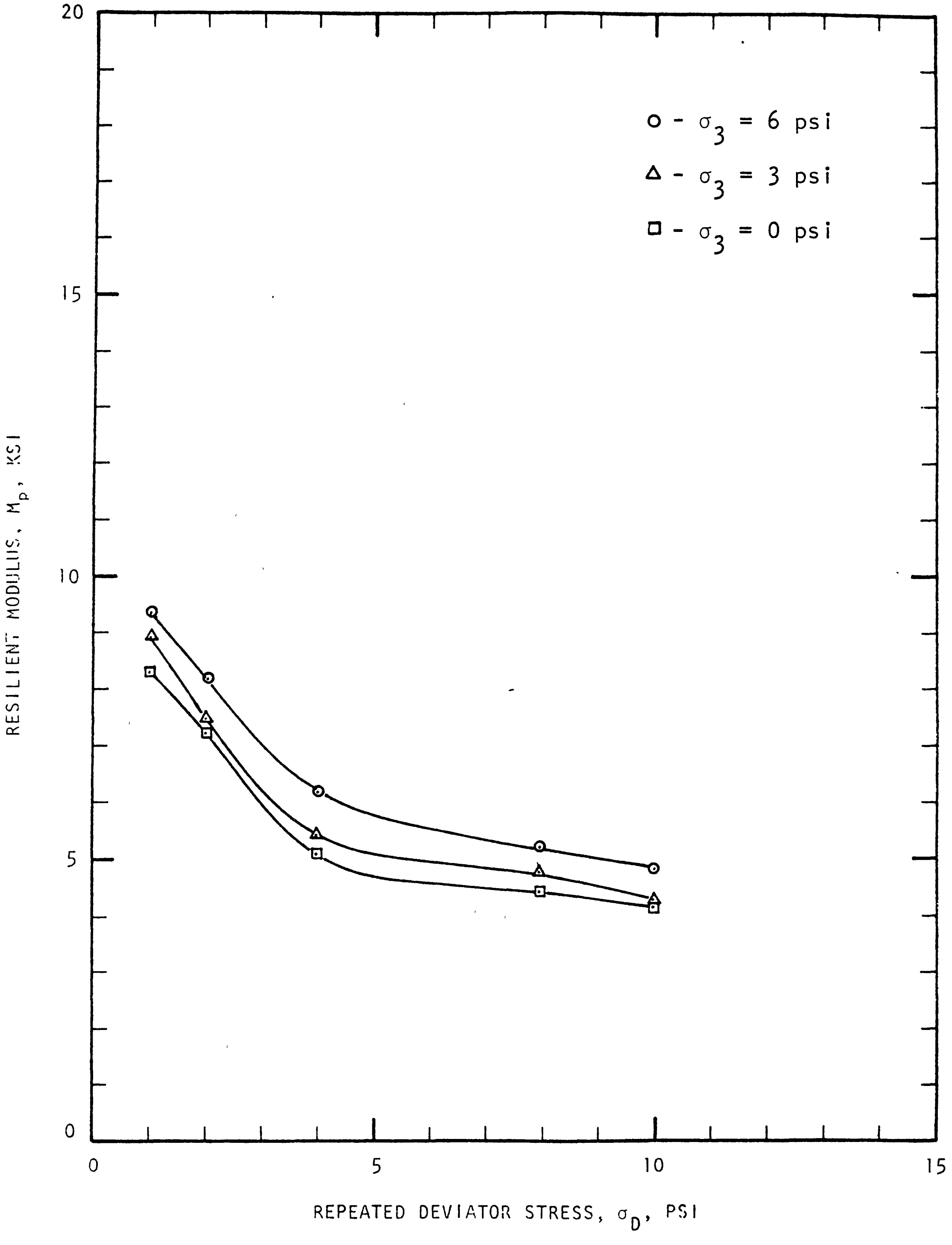

Figure 40. Resilient Modulus Testing Results for Norge Soil, OMC+2.0, Specimen 2

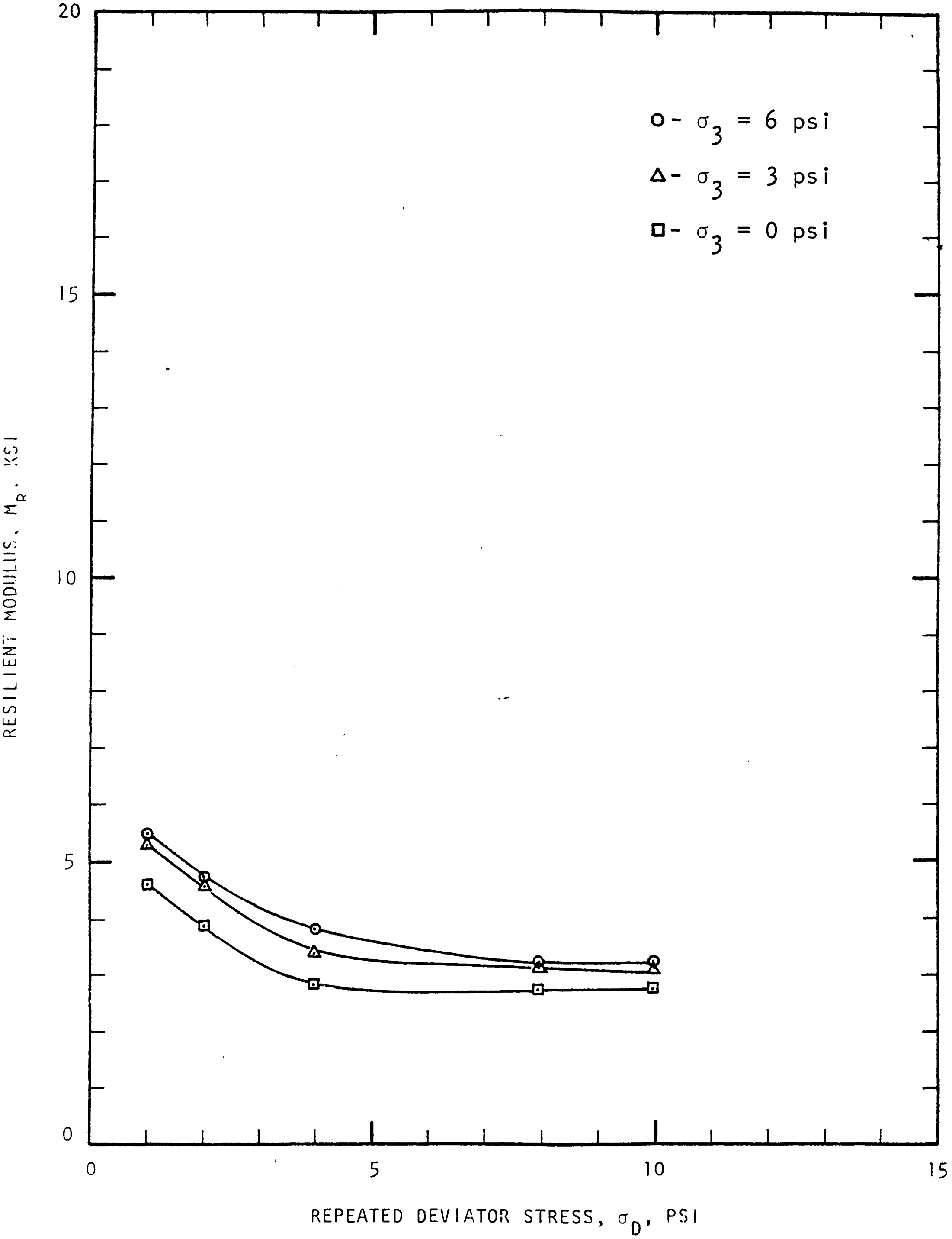

Figure 41. Resilient Modulus Testing Results for Norge Soil, Saturated, Specimen 1

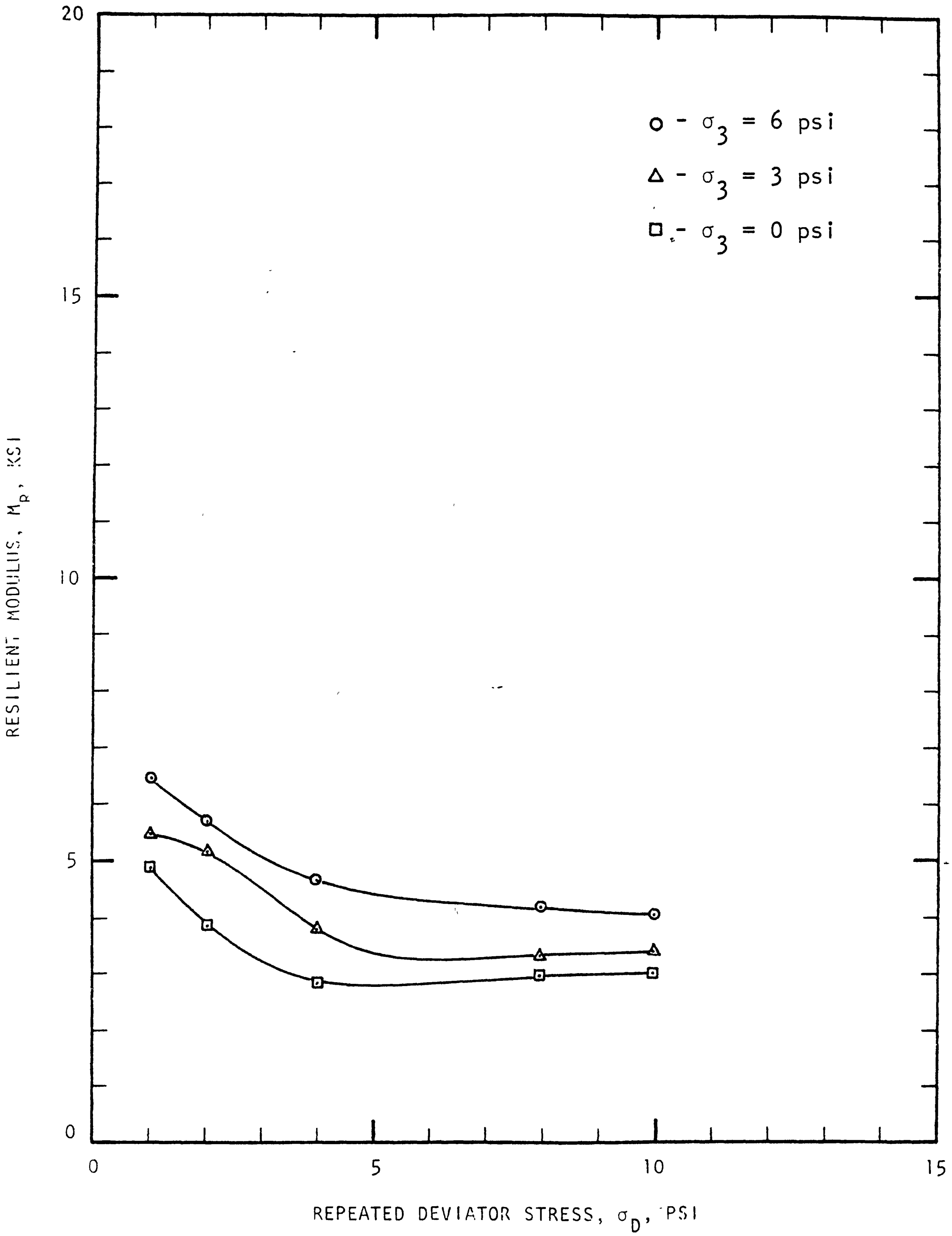

Figure 42. Resilient Modulus Testing Results for Norge Soil, Saturated, Specimen 2

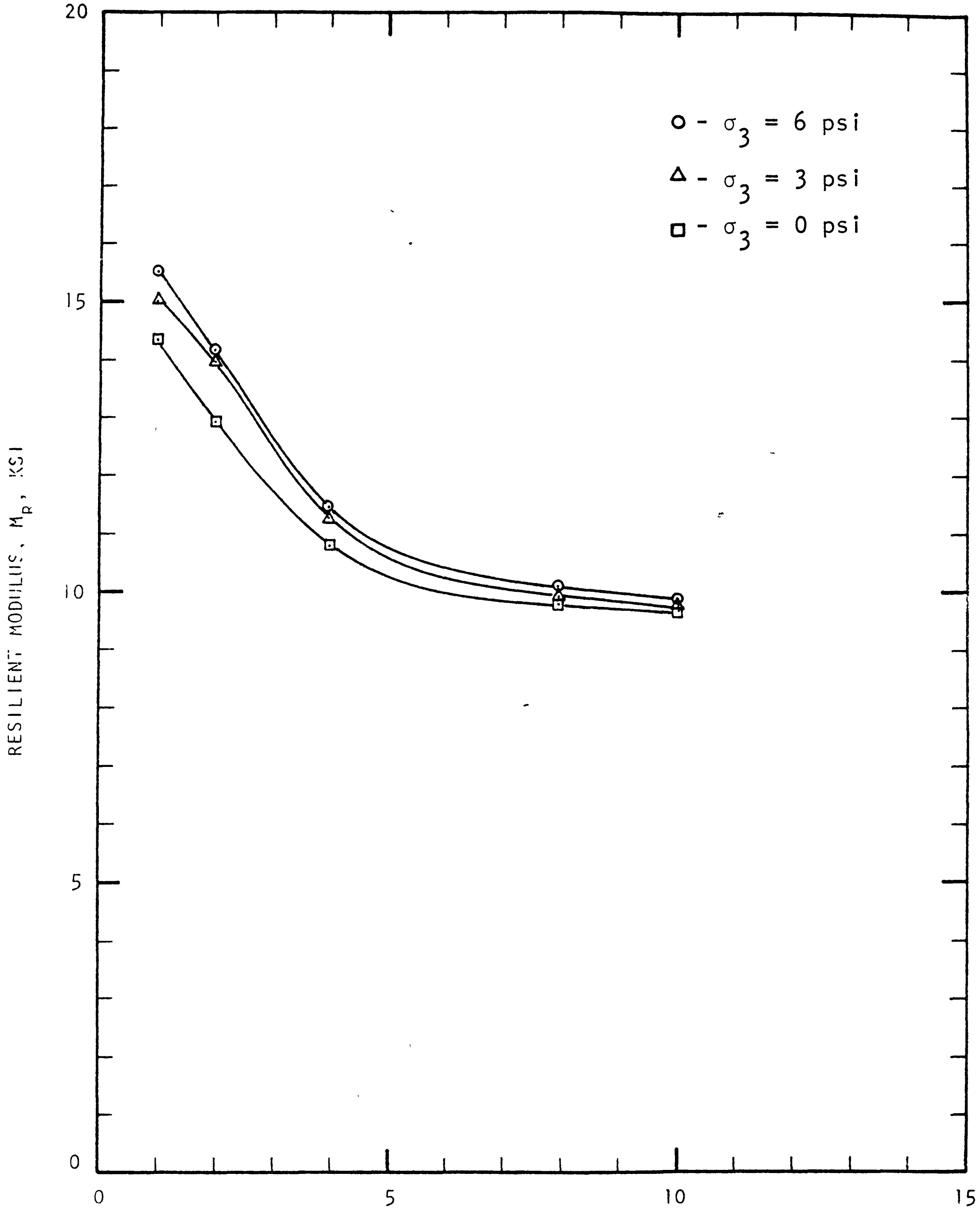

Figure 43. Resilient Modulus Testing Results fro Miller Soil, OMC-2.0, Specimen 1

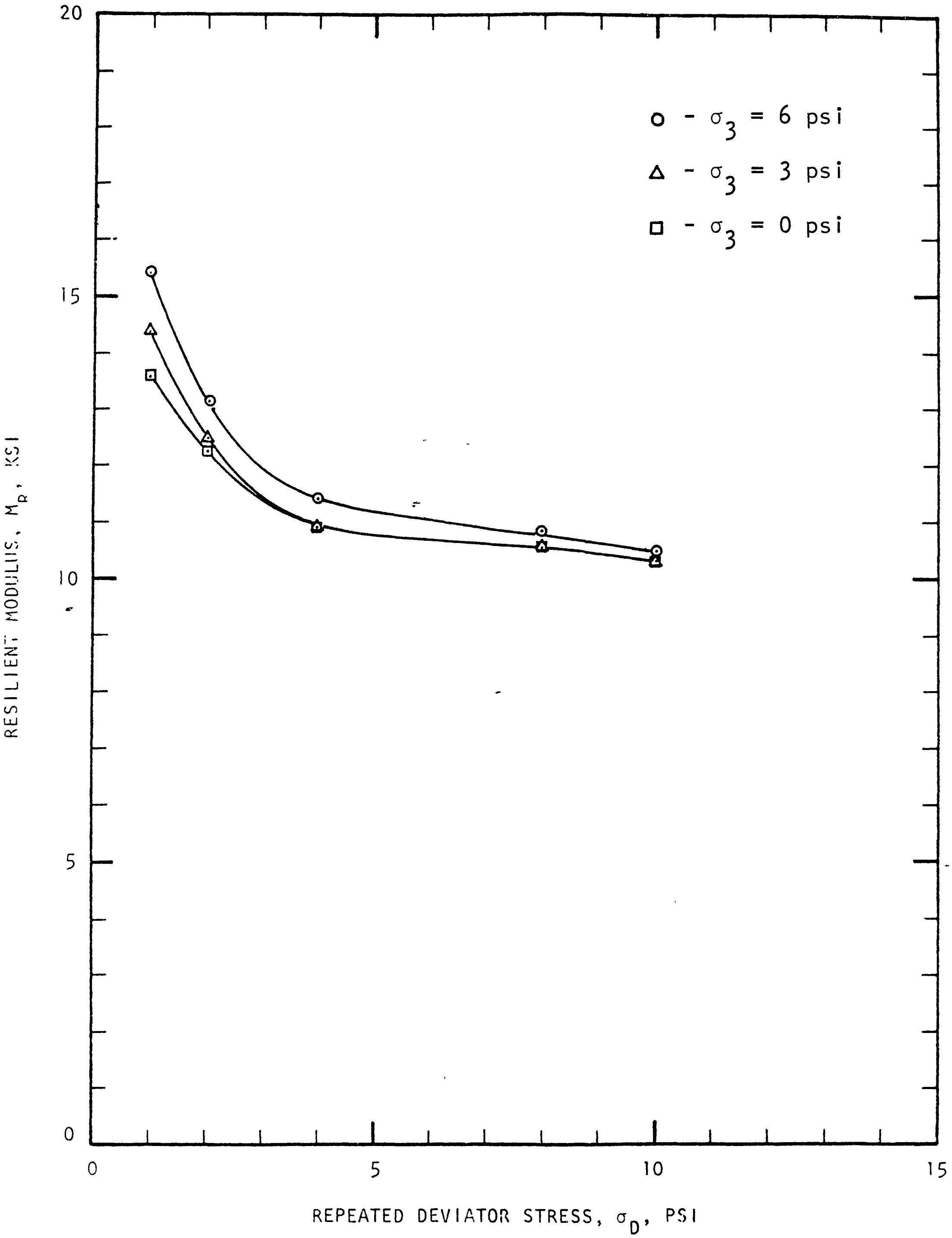

Figure 44. Resilient Modulus Testing Results for Miller Soil, OMC-2.0, Specimen 2

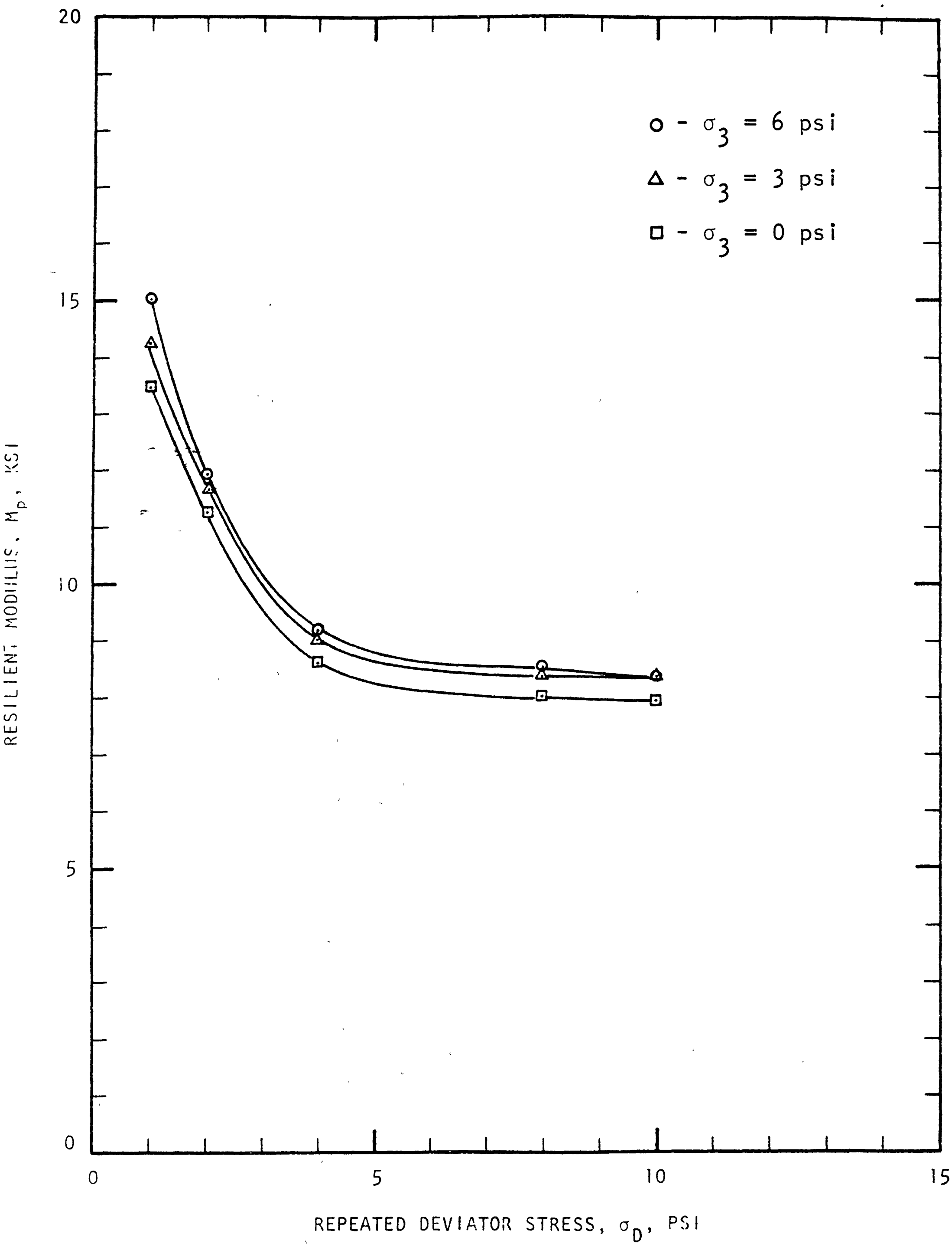

Figure 45. Resilient Modulus Testing Results for Miller Soils, OMC, Specimen 1

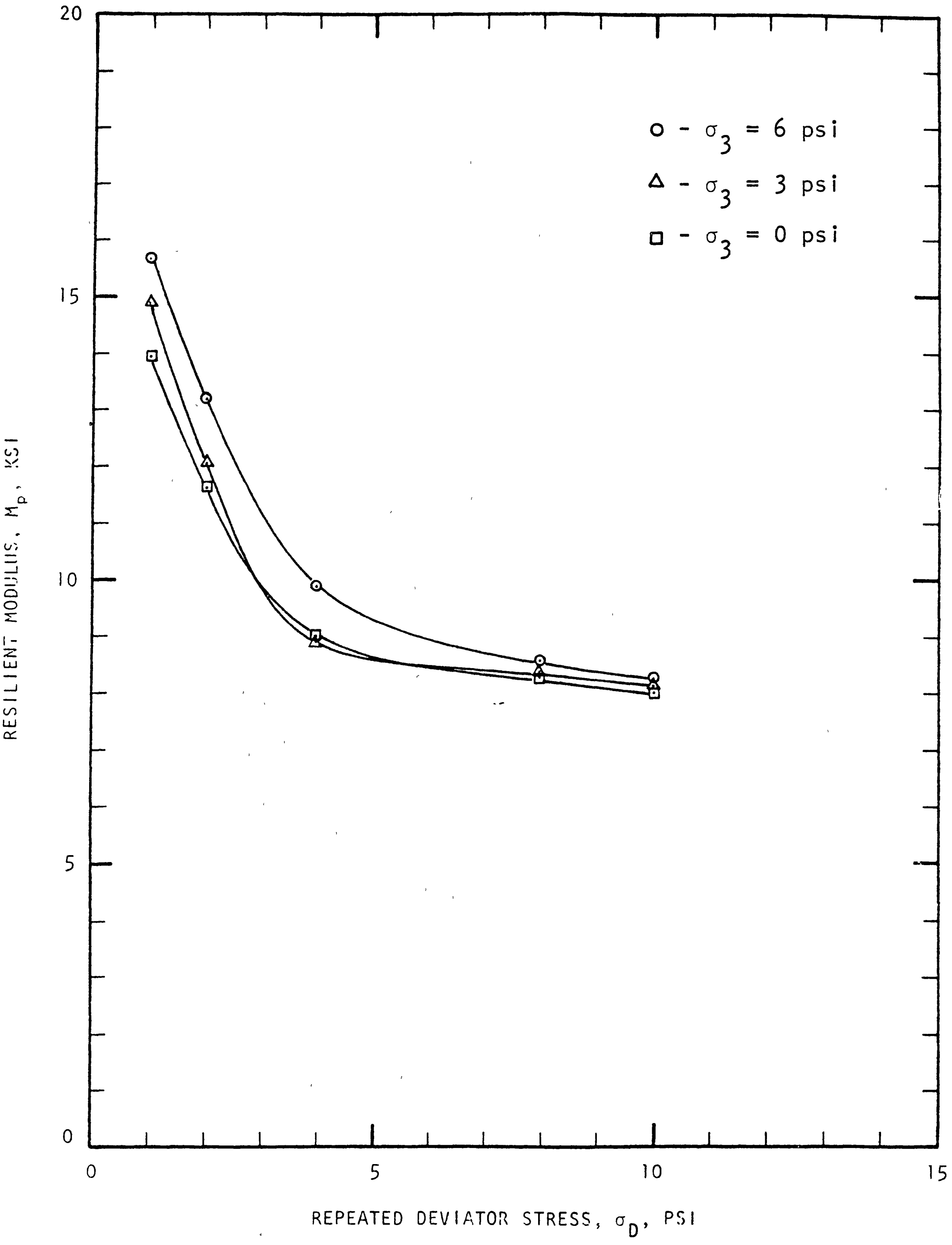

Figure 46. Resilient Modulus Testing Results for Miller Soil, OMC, Specimen 2

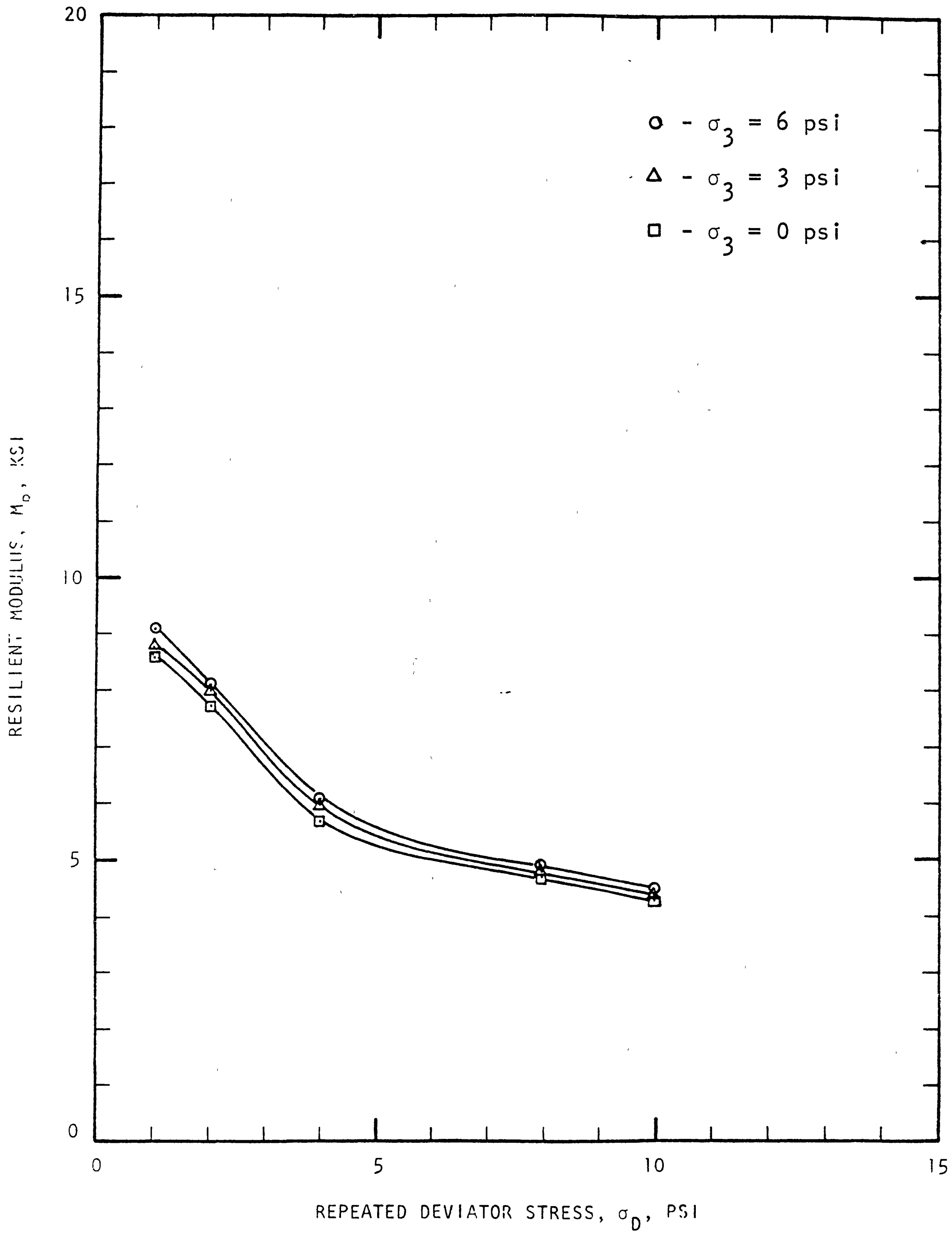

Figure 47. Resilient Modulus Testing Results for Miller Soil, OMC+2.0, Specimen 1

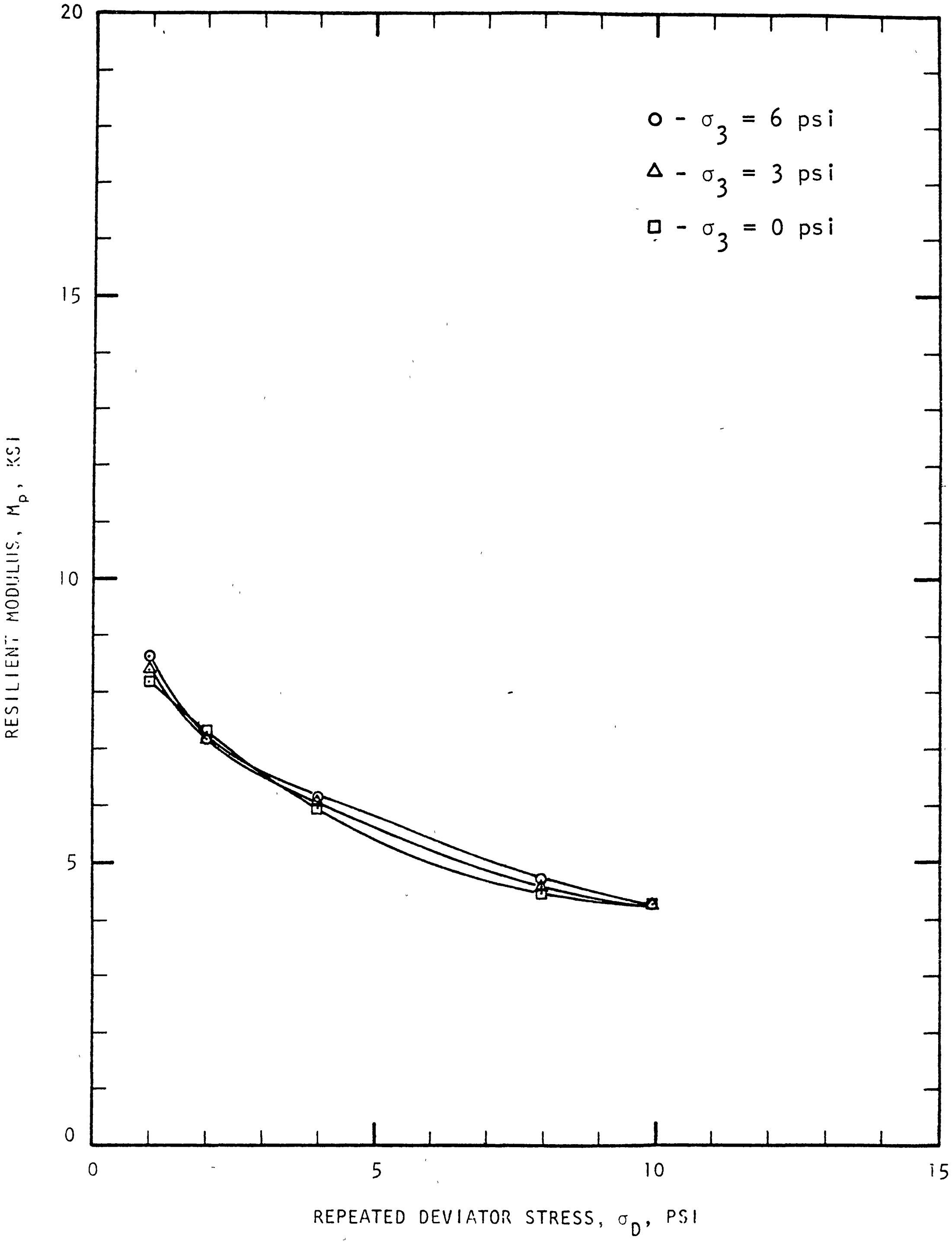

Figure 48. Resilient Modulus Testing Results for Miller Soil, OMC+2.0, Specimen 2

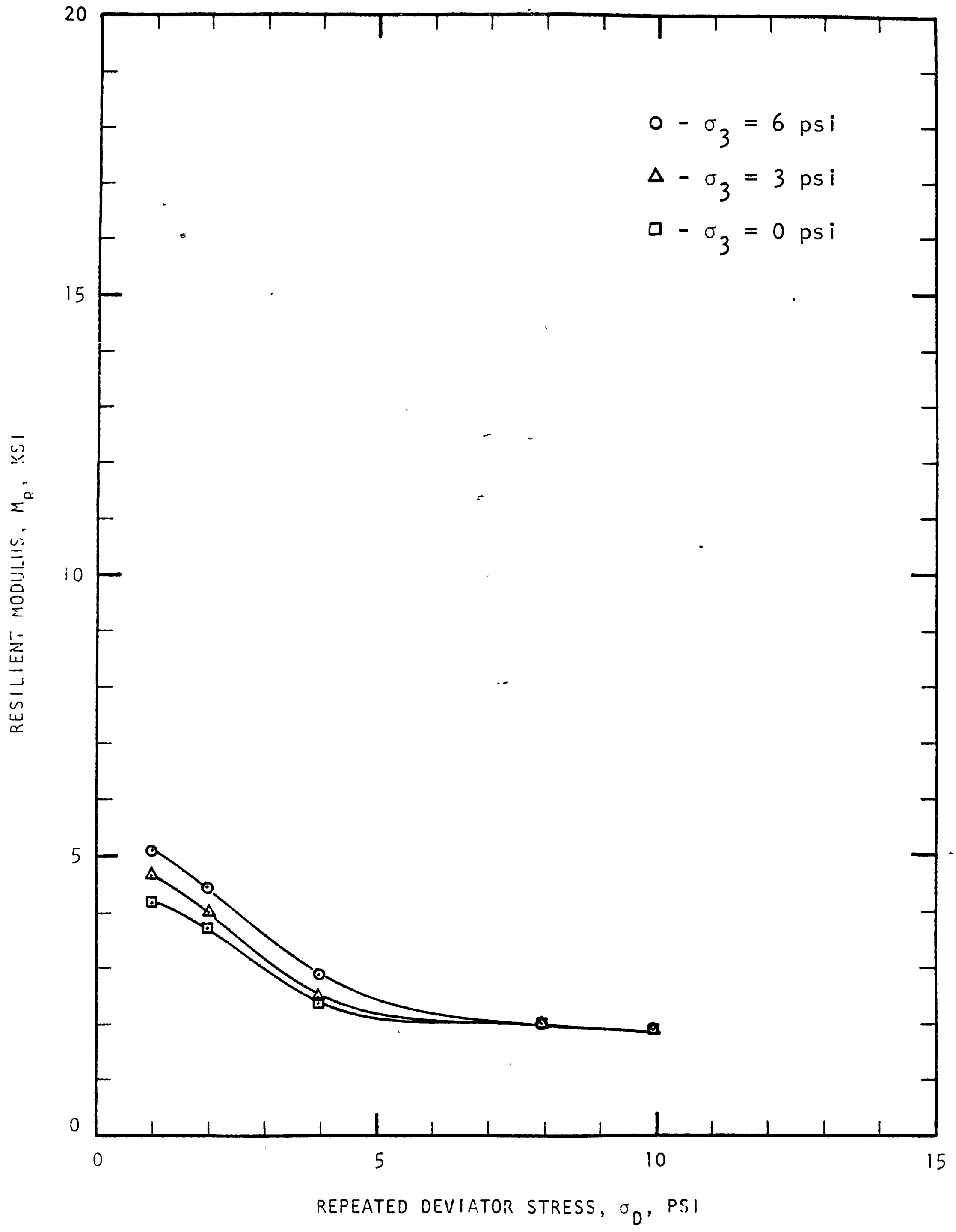

Figure 49. Resilient Modulus Testing Results for Miller Soil, Saturated, Specimen 1

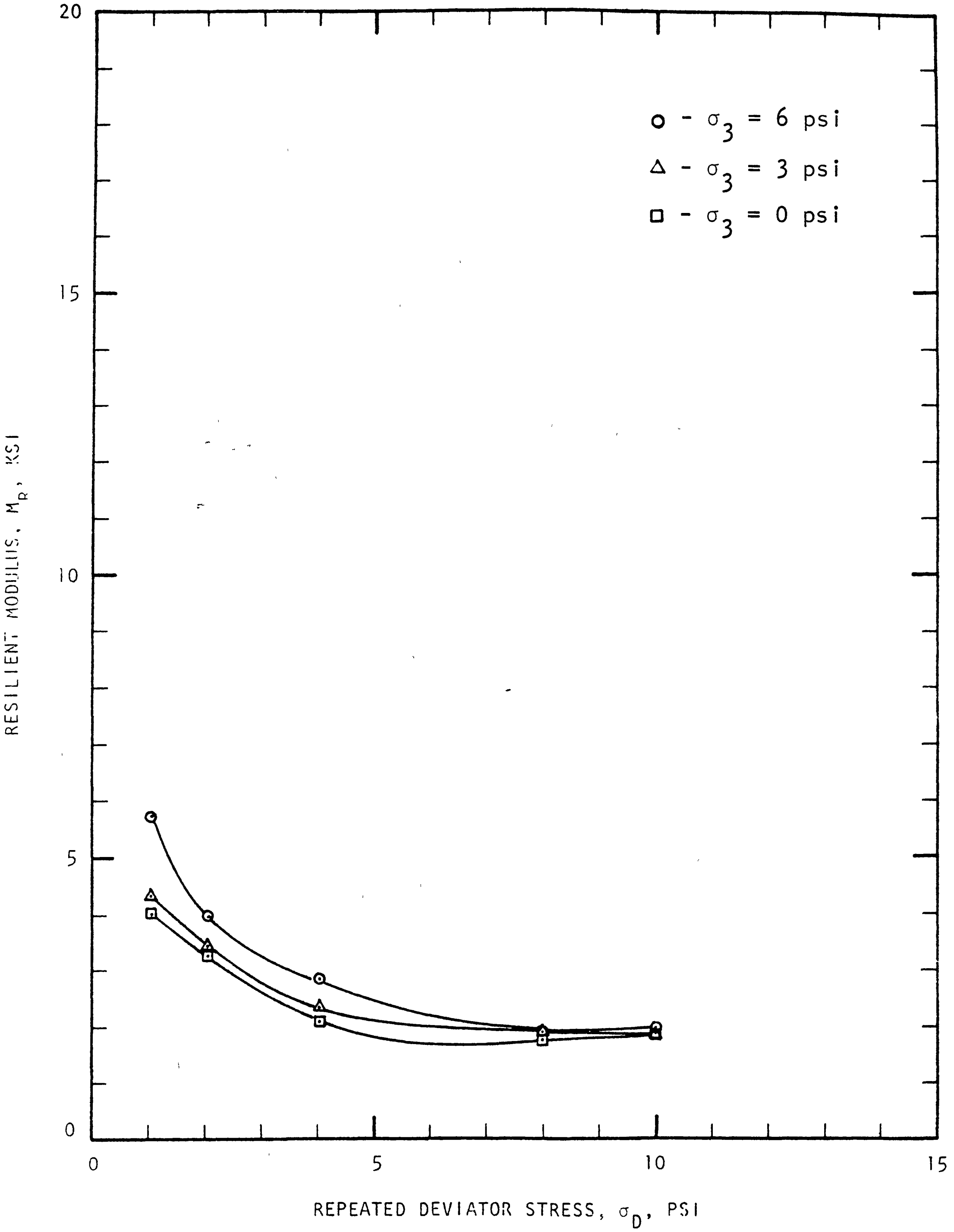

Figure 50. Resilient Modulus Testing Results for Miller Soil, Saturated, Specimen 2

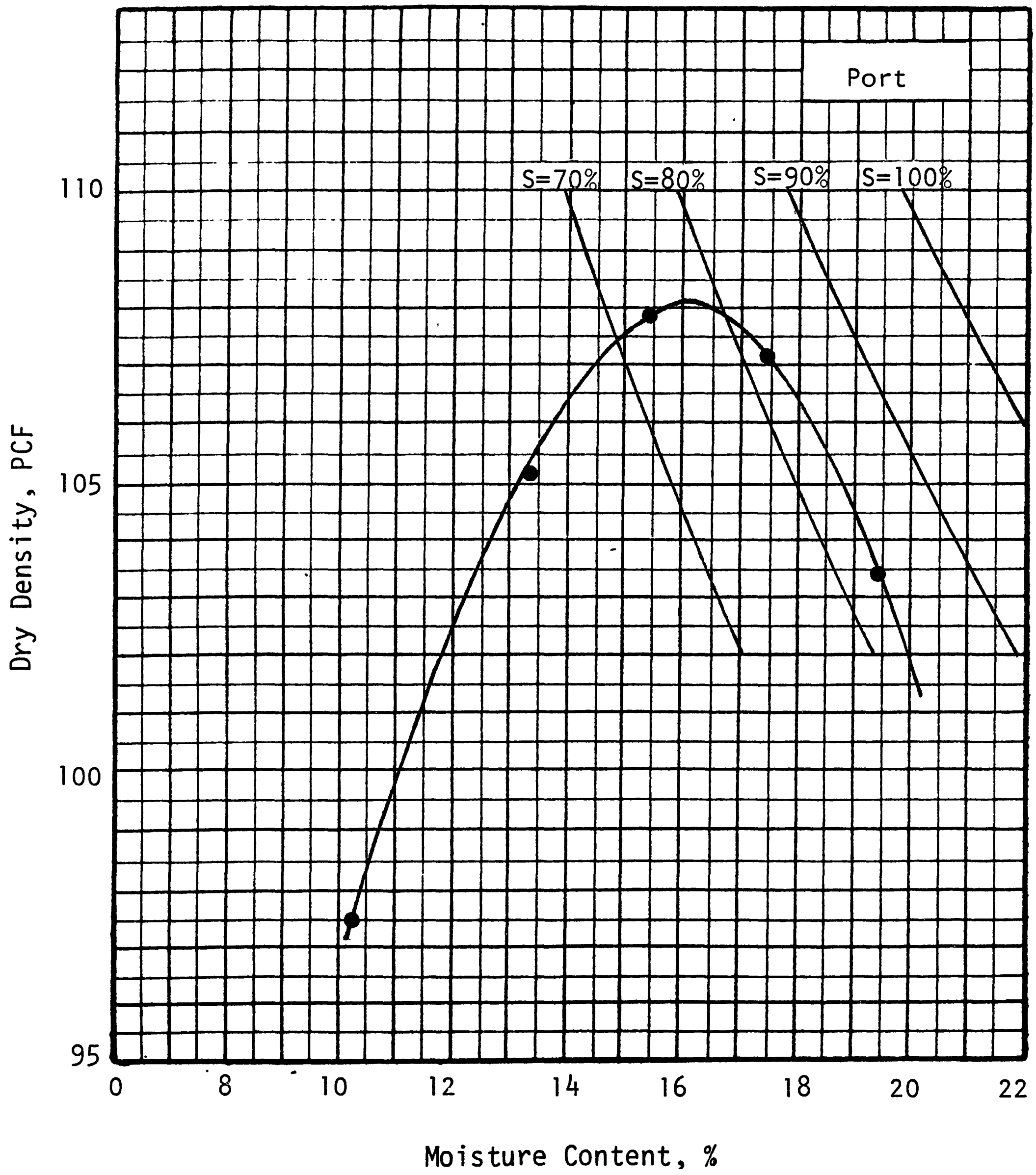

Figure 51. Standard Proctor Compaction Curve for Port Soil

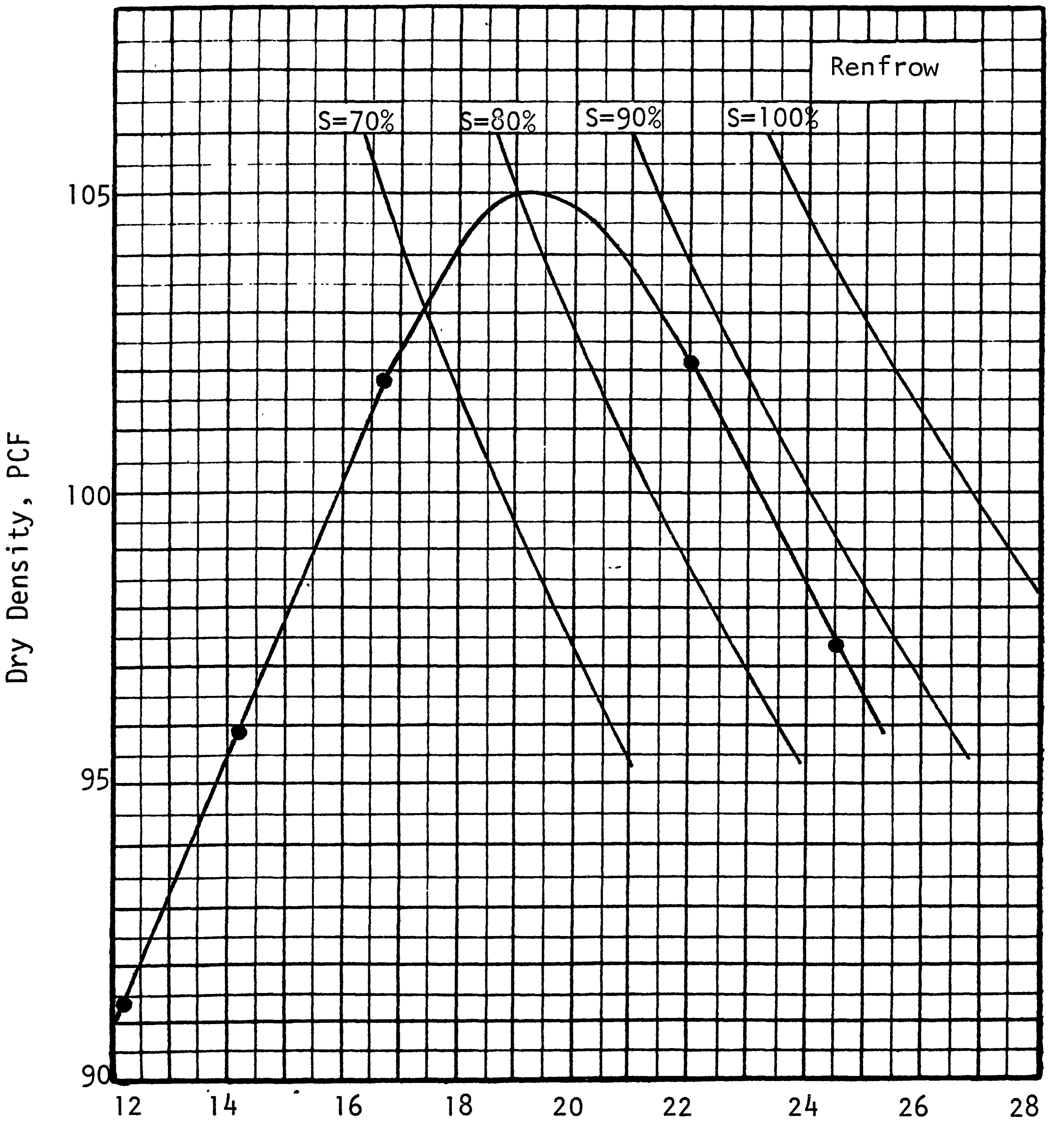

Figure 52. Standard Proctor Compaction Curve for Renfrow Soil

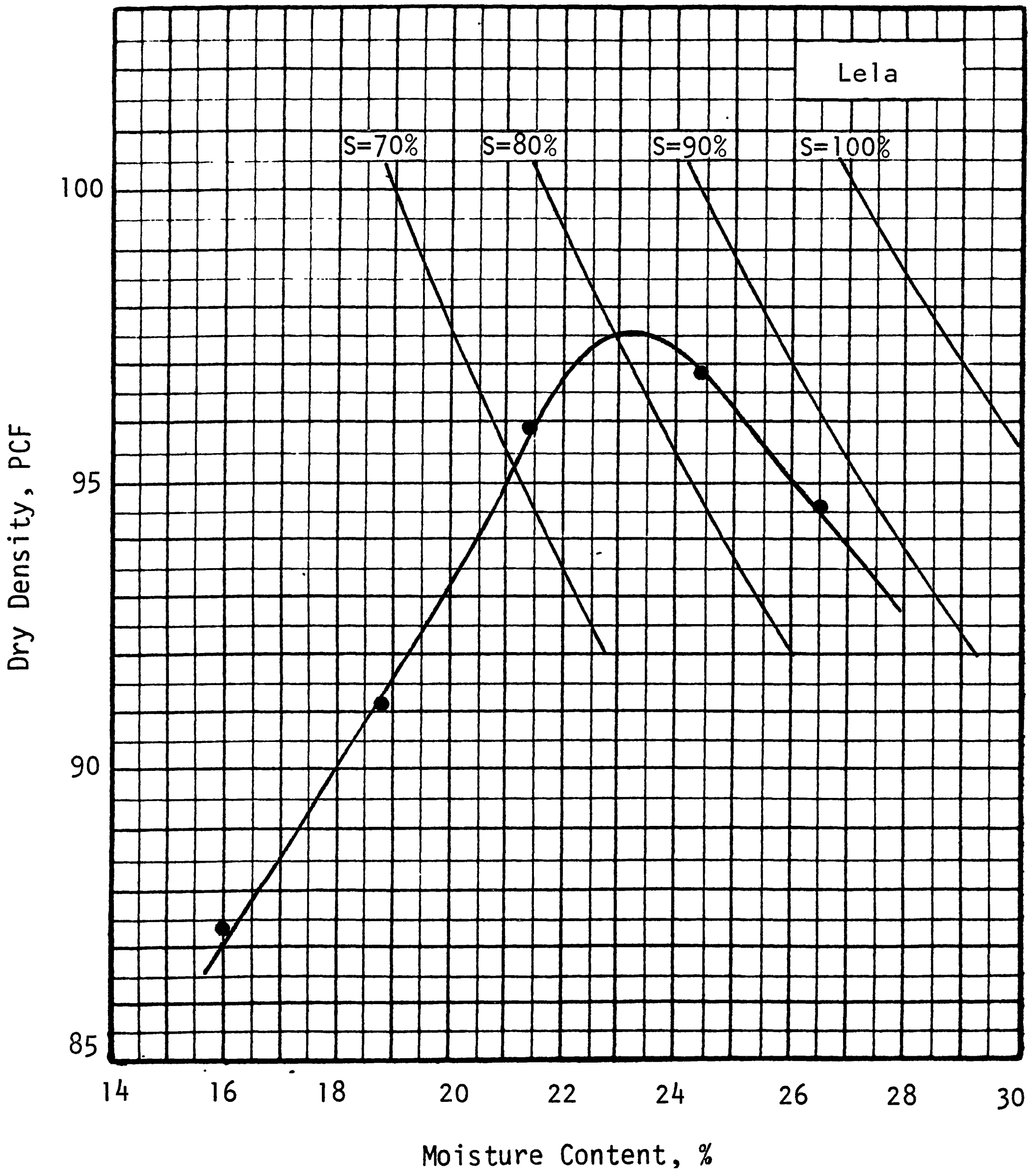

Figure 53. Standard Proctor Compaction Curve for Lela Soil

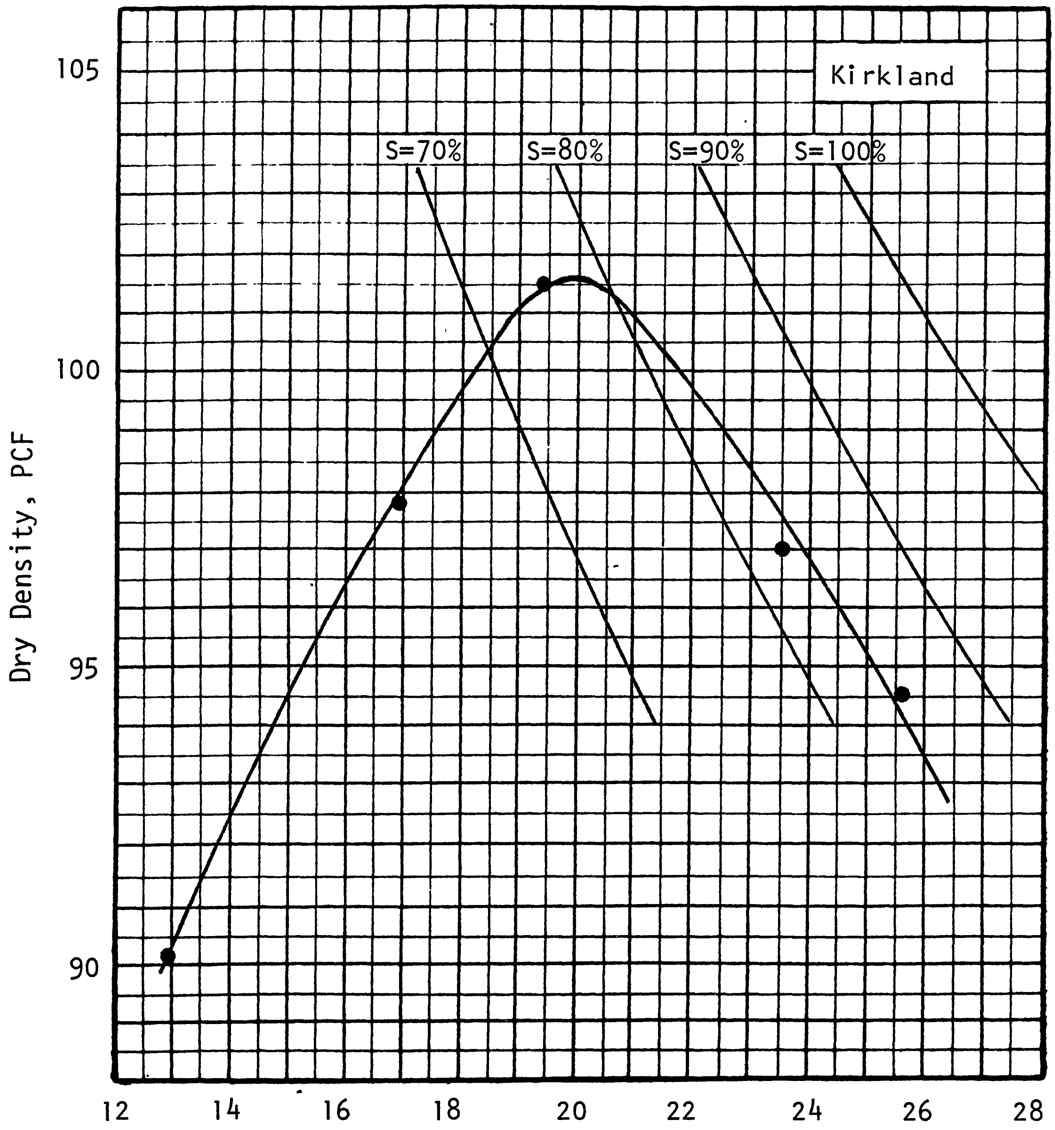

Figure 54. Standard Proctor Compaction Curve for Kirkland Soil

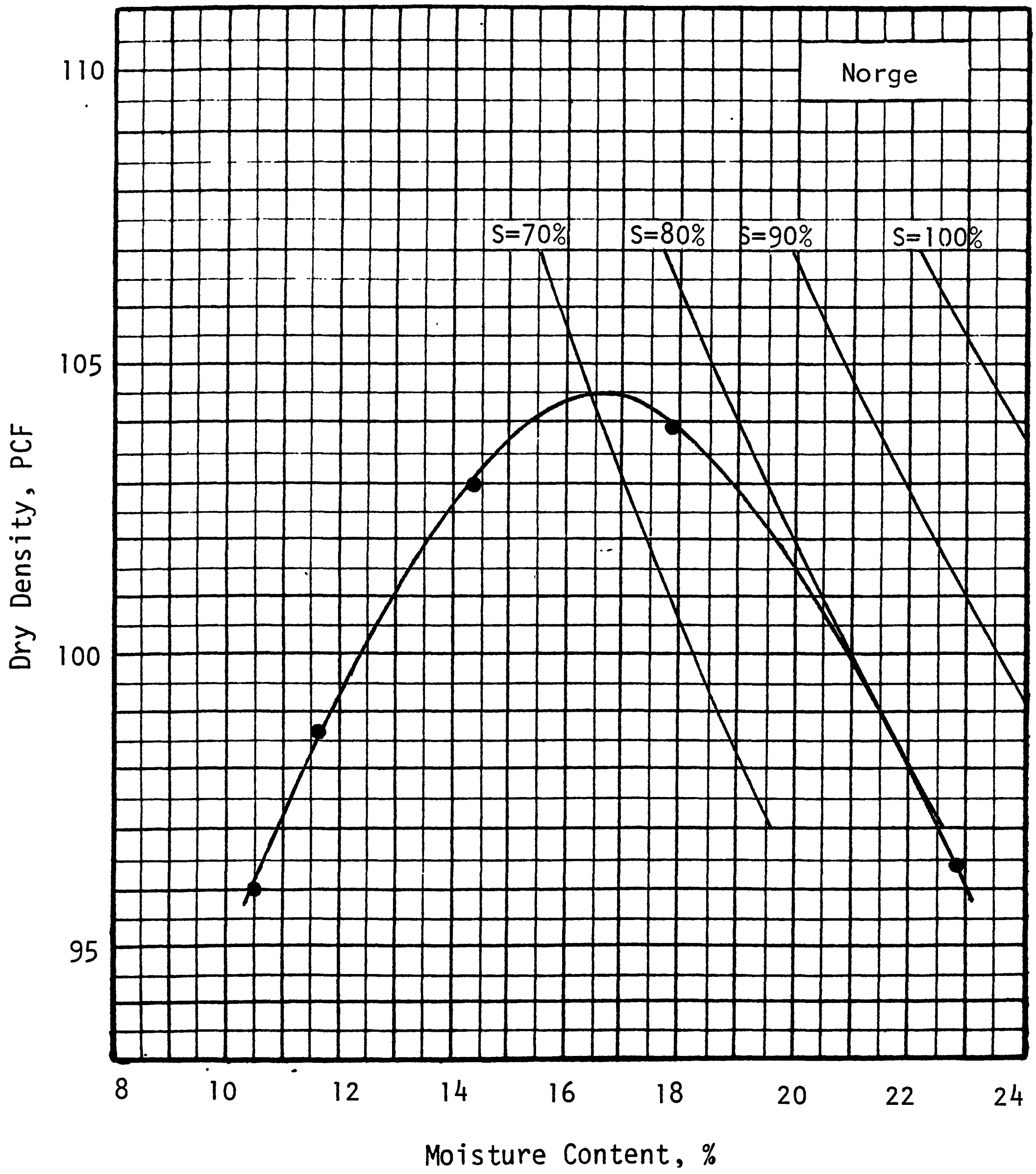

Figure 55. Standard Proctor Compaction Curve for Norge Soil

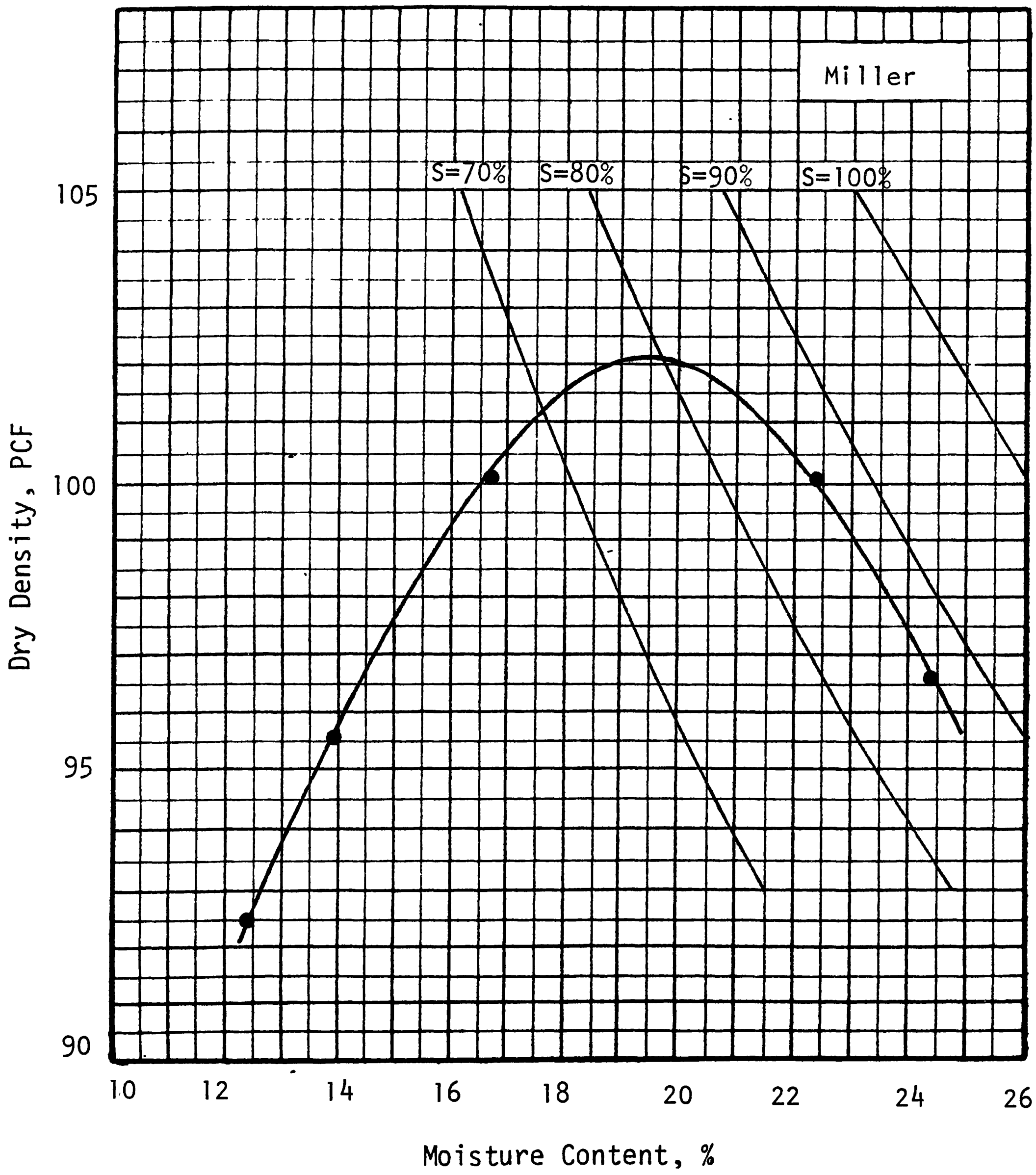

Figure 56. Standard Proctor Compaction Curve for Miller Soil

2

VITA

Ta Teh Dave Chang

Candidate for the Degree of

Doctor of Philosophy

Thesis: EVALUATION OF RESILIENT MODULUS AND CORRELATION WITH ENGINEERING PROPERTIES FOR COMPACTED OKLAHOMA FINE-GRAINED SOILS

Major Field: Civil Engineering

Biographical:

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