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Proceedings of the Workshop on Resilient Modulus Testing

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

These proceedings represent the results of a 3 day workshop on the state of the practice of resilient modulus testing. The resilient modulus is one of the most important pavement material characterization properties. It is used in the mechanistic approach to pavement design and recognized by the 1986 American Association of State Highway and Transportation Officials, "Guide for Design of Pavement Structures." The test procedures for material analysis are relatively new to the State highway agencies and consequently needs further development. These proceedings provided information on current techniques, equipment and results.

These proceedings will be of interest to engineers and technicians involved in pavement design and analysis of pavement materials.

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Stanley Byington

Stanley R. Byington, Director
Office of Implementation

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16. Abstract

These proceedings are the results of a 3 day resilient modulus testing workshop. The objective of the workshop was to address the significance of utilizing and evaluating the resilient moduli of highway materials. At the workshop, 25 presentations were made on the various aspects of test techniques and systems used to evaluate resilient modulus. Workshop discussions also included the importance of this property to characterize highway materials as used in the mechanistic approach to pavement design and the 1986 American Association of State Highway and Transportation Officials (AASHTO) "Guide for Design of Pavement Structures."

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SI* (MODERN METRIC) CONVERSION FACTORS

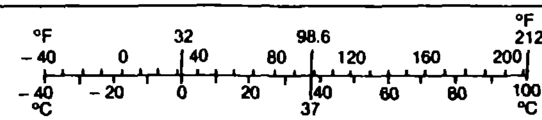
APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

NOTE: Volumes greater than 1000 L shall be shown in m^3.

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

PROCEEDINGS OF THE WORKSHOP ON RESILIENT MODULUS TESTING

Report No. FHWA-TS-90-031

Sponsored by

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McLean, Virginia 22101

Oregon State Highway Division
Oregon Department of Transportation
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PREFACE

The 1986 AASHTO Guide for the Design of Pavement Structures recommends that resilient modulus be used as the definitive property to characterize highway materials in the mechanistic approach to pavement design. In response to the AASHTO recommendations, the Oregon Department of Transportation sponsored this workshop on resilient modulus to address the needs of the pavement-engineering community. The workshop presented the first opportunity for pavement engineers to evaluate the resilient moduli of highway materials and to select equipment to measure resilient moduli appropriate to their laboratory needs.

Workshop instructors provided participants with background information on the mechanistic approach to pavement design and test techniques and on systems used to evaluate resilient moduli. Equipment manufacturing representatives displayed and demonstrated their test systems and participated in a "round robin" test program that evaluated the resilient moduli of samples of pavement materials. The results of the evaluations were discussed and compared during the workshop.

Through workshop presentations, demonstrations, and discussions, significant progress has been made toward understanding the complexities of resilient-modulus testing. Continued use of mechanistic design approaches to pavement engineering will ensure the future enhancement of the repeated-load test systems demonstrated during this conference.

ODOT WORKSHOP ON RESILIENT MODULUS

STATE OF THE PRACTICE

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EXECUTIVE SUMMARY OF THE ODOT WORKSHOP ON RESILIENT MODULUS TESTING: STATE OF THE PRACTICE

The Workshop on Resilient Modulus Testing, held over a three day period, was structured to ensure that all participants would have a background in the mechanistic approach to pavement design that would further allow them to appreciate resilient modulus testing, the applications of resilient modulus test results, the interpretation of test results, and the future of resilient modulus testing. Professor Dave Newcomb, University of Minnesota, gave an introduction to the mechanistic approach to pavement design and design examples. Professor Ted Vinson, Oregon State University, gave a background on the history of resilient modulus testing, justification for resilient modulus testing, and the conduct of the tests. Professors Marshall Thompson, University of Illinois, and John Epps, University of Nevada, made presentations on the factors affecting resilient moduli of subgrade soils, granular materials, and asphalt concrete. Ample time was allowed for questions from the audience following these presentations.

On the afternoon of the first day, five states presented applications of resilient modulus test results in their pavement engineering practices. Dave Seim, assistant soils engineer, New York State Department of Transportation, gave the results of a comprehensive study on the resilient modulus of subgrade soils. George Cochran presented the Minnesota Department of Transportation's experience with laboratory resilient modulus testing. Tom Moses reported on the Alaska Department of Transportation and Public Facilities applications of resilient modulus test results in their pavement engineering practice. The applications include research as well as design based on laboratory determined resilient moduli (specifically the Seward Highway, Sterling Highway, and Cold Bay Airport). The use of a two-axis, computerized resilient modulus test system was presented by Glenn Fager of the Kansas Department of Transportation. Finally, Dick Hines, flexible pavement engineer, presented the Colorado Department of Highway's experience with resilient modulus testing of asphalt concrete over the past 15 years. These presentations illustrated the ways in which the states have used commercially available equipment to evaluate resilient moduli of bound and unbound pavement materials. The states were chosen because they represented five different equipment manufacturers who have provided resilient modulus test equipment over the past 15 years.

During the second day of the conference, laboratory demonstrations were given by ten equipment manufacturers. The equipment manufacturers included AMI, Cox and Sons, Digit^R al Control Systems, H & V Materials R & D, Interlocken Technology, MTS, Research Engineering, Retsina, SBEL, and VTI. The equipment manufacturers displayed a range of sophistication as well as a range of cost. Not all equipment manufacturers were able to evaluate resilient moduli of both bound and unbound materials.

Andrew Brickman, Oregon State University, gave an overview of resilient modulus test systems before the laboratory demonstration. The overview included a consideration of methods to apply loads, methods to control loads, system instrumentation, signal conditioning, and data acquisition and control. Future developments to test systems, including the role of the personal computer in resilient modulus test systems, the trend toward closed-loop control, better fixtures to hold diametral specimens and displacement transducers, and the use of noncontact displacement transducers, also were discussed by Mr. Brickman. Throughout the presentation, emphasis was given to the goal common to the design of any quality measuring instrument, namely:

* accuracy and repeatability
* ease of use
* ruggedness and dependability
* reasonable cost

In the afternoon of the second day, five presentations were given. Professor Chris Bell, Oregon State University, talked about the influence of the method of sample preparation on resilient modulus of asphalt concrete; Professor Gilbert Baladi, Michigan State University, discussed the influence of test apparatus boundary conditions on resilient moduli determined in diametral tests; Bud Furber, of Pavement Services, Inc., gave a presentation on the effects of strain and temperature on resilient moduli of asphalt concrete determined under diametral test conditions. The influence of soil sampling operations on resilient moduli was presented by Newt Jackson of the Washington Department of Transportation. Finally, Professor Marshall Thompson talked about the influence of soil sample preparation on resilient moduli.

As part of the workshop, the equipment manufacturers were asked to participate in a round-robin test program. Jim Huddleston and Haiping Zhou of the Oregon Department of Transportation summarized the results of the round-robin test program. Unfortunately, the asphalt concrete cores provided to the equipment manufacturers before the workshop displayed a great deal of variability in compaction density and this variability was reflected in the test results.

During the third day of the conference, Carl Monismith, professor of civil engineering at the University of California, Berkeley, gave a presentation on the use of laboratory test results for design purposes. Professor Monismith noted that while it may appear desirable at times for the engineer to adopt a single value for material parameters as a means to control specific forms of distress, caution should be exercised.

Professor Bob Lytton, Texas A & M, presented the relationship of resilient moduli to backcalculated moduli using nondestructive testing equipment. It is recognized, and this was brought out by Professor Lytton's presentation, that substantial differences exist between backcalculated moduli and laboratory determined moduli. Professor Lytton encouraged the workshop participants to consider these differences in the design process, but pointed out the advantages of both moduli to the pavement engineering profession. The future of resilient modulus testing was expressed by representatives from four state departments of transportation.

Robert Ho presented the Florida Department of Transportation's experience on the use of repeated-load tests on two granular subgrades and one base material. David Allen discussed the present and future of resilient modulus testing at the Kentucky Transportation Center. He noted that an expanded role for the use of the resilient modulus test is anticipated in Kentucky. Further, he noted there is a real possibility that the test may become a part of the normal series of specifications for state acceptance of a particular mix design. Jagat Dhamrait presented the Illinois Department of Transportation's experience with resilient modulus testing. This information reflected the many research studies conducted by the University of Illinois in cooperation with the Illinois DOT and FHWA.

Finally, Ok-Kee Kim presented the CALTRANS' resilient modulus test experience of yesterday, today, and tomorrow. He noted that since the early 1970s, CALTRANS has incorporated resilient modulus testing in several research studies. He further noted that the resilient modulus testing by

CALTRANS may not extend beyond research purposes in the immediate future. If improved test equipment provides acceptable repeatability, accuracy, and correlation with other design parameters, then the resilient modulus of pavement materials may be adopted as an additional design property. In conclusion, he noted that it is expected that the use of resilient modulus values for pavement structure evaluation and analysis will increase in the future at CALTRANS.

The third day also included two panel discussions, the first moderated by Jim Sorenson of the FHWA. The panelists included Professor Carl Monismith, Professor Marshall Thompson, Professor Ron Terrel, Oregon State University, Jim Huddleston, Newt Jackson, and Jim Wilson, a representative of AMI Consultants. The panel and audience discussion were recorded and transcribed. Among the questions asked were: Why resilient modulus? What is the appropriate stress level that one should use in a resilient modulus test? Have there been successful attempts to correlate resilient modulus to an asphalt concrete mix's rutting potential? What is the relationship between resilient modulus of an asphalt concrete material and the creep characteristic? Should compaction prestresses be accounted for in selecting the appropriate stress level for resilient modulus of unbound materials? Is it necessary to lubricate the end platens in a triaxial, repeated-load test? How can a resilient modulus test be performed on a moist sample of fine-grain noncohesive material without the water in the sample flowing to the bottom of the specimen during the test? Are the same amount of conditioning cycles needed for cohesionless and cohesive soils? What are the relative merits of internal versus external LVDT clamps?

Other questions were specific to the differences in the types of test equipment and included: What is the difference between electrohydraulic and electropneumatic equipment? What are the load wave forms available? and How much do you have to pay for such versatility? The answers to these questions, as well as others, were transcribed and are included as part of the Workshop Proceedings.

The views of ASTM, AASHTO, NCHRP, and SHRP were presented by representatives of these organizations during the second panel discussion. It was noted that many of the professional societies are in the process of revising their test procedures and that the proceedings and discussions of the workshop would provide valuable input to their processes.

Professor R. Gary Hicks, Oregon State University, gave the closing remarks to the workshop on resilient modulus testing. He concluded that the workshop provided answers to the following questions:

* What types of equipment are available and where can they be obtained?
* What factors affect resilient moduli of bound and unbound materials?
* What are the limitations of the resilient modulus tests as they presently exist?
* Are tests between the various types of equipment repeatable?
* How can modulus results be used in the pavement design process?
* What does the future of resilient modulus testing appear to be?

He noted that throughout the workshop there was excellent response from both speakers and participants in an effort to address these questions.

Perhaps the most significant fact brought out by the workshop is that there is a strong need to educate the pavement engineering community on the advantages and disadvantages of resilient modulus testing. Also, while specific answers were given to many questions, there still remain questions that could not be answered because the necessary research work has not been conducted.

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ELASTIC PAVEMENT ANALYSIS

by

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for

Workshop on Resilient Modulus Testing
at
Oregon State University
Corvallis, OR 97331-2302

March 28-30, 1989

SECTION OUTLINE
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1. INTRODUCTION

   1.1 Purpose

   1.2 Inputs

   1.3 Outputs

   1.4 Assumptions

   1.5 Implementation of elastic analysis

2. TYPICAL EXAMPLES AND RESULTS

   2.1 Typical pavement structures

   2.2 Examples of changed inputs

   2.3 Results of sensitivity analysis

3. DESIGN CONSIDERATIONS

4. RELATIONSHIP BETWEEN PAVEMENT RESPONSE AND PAVEMENT PERFORMANCE

   4.1 Introduction

   4.2 Typical relationships

   4.3 More examples

APPENDIX 1

SECTION OUTLINE
ELASTIC PAVEMENT ANALYSIS

1. INTRODUCTION

Elastic pavement analysis is a computational procedure whereby stresses, strains or deflections can be calculated for any point of interest in a pavement structure for any given tire load(s). Knowledge of such parameters coupled with appropriate limited values (particularly for strains) can be used to design new pavement structures as well as evaluate existing structures. This section introduced this topic and illustrates the important material property of "elastic modulus."

2. TYPICAL EXAMPLES AND RESULTS

Typical elastic analysis results are shown for three "common" structural thicknesses of pavements. Different wheel load and material property combinations are used to illustrate how pavement surface deflection, horizontal strain at the bottom of the asphalt concrete layer, and vertical strain at the top of the subgrade varies. Finally, a "deflection basin" is calculated and illustrated.

3. DESIGN CONSIDERATIONS

A brief discussion is provided to show how elastic analysis can be used in the design of pavement structures.

4. RELATIONSHIP BETWEEN PAVEMENT RESPONSE AND PAVEMENT PERFORMANCE

The important linkage between pavement response to load (strain) to limiting strain values (failure criteria) is illustrated for estimating fatigue cracking and rutting. Further, examples are included in this section which illustrate how load equivalencies can be calculated from the use of elastic analysis and failure criteria.

SECTION
ELASTIC PAVEMENT ANALYSIS

1. INTRODUCTION

1.1 Purpose for this type of pavement analysis

1.1.1 For new pavement design

(a) Accommodate changing load requirements
(b) Better utilize available materials
(c) Accommodate new materials
(d) Improve reliability for performance prediction
(e) Better define the role of construction
(f) Material properties (elastic) relate better to actual pavement performance

1.1.2 For pavement rehabilitation

(a) Better define properties of existing layers (the measured properties tie closer to expected pavement performance and more data can be obtained by nondestructive field testing)
(b) Accommodate new materials and/or pavement layers
(c) Accommodate analysis of various load levels (past, present, future)
(d) Accommodate material property changes due to environment and aging

1.2 Inputs

1.2.1 Traffic

(a) Loads
(b) Geometry

1.2.2 Materials

(a) Resilient modulus (elastic modulus or "stiffness")
(b) Poisson's ratio

1.2.3 Pavement geometry

(a) Layer thickness

1.2.4 Pavement response locations for

(a) Stress
(b) Strain
(c) Deflection

1.3 Outputs (refer to Figure 1)

Pavement response to applied load(s) in terms of

(a) Stress
(b) Strain
(c) Deflection

1.4 Elastic pavement analysis assumptions

1.4.1 Material properties in each layer are homogeneous (elastic properties same at all points in a given material).

1.4.2 Material properties in each layer are isotropic (elastic properties are the same in all directions at any point).

1.4.3 Each layer has a <u>finite</u> thickness except the lowest layer (presumably the subgrade) and all are <u>infinite</u> in the lateral dimensions.

1.4.4 Elastic analysis solutions require <u>two</u> material properties for each layer:

E (or E_T or M_r) = elastic modulus, psi
μ = Poisson's ratio

(a) Elastic modulus

(i) Elastic modulus is sometimes called Young's modulus since Thomas Young published the concept of elastic modulus in 1807 (not exactly a new idea). Essentially, elastic modulus can be determined for any solid material and represents a <u>constant ratio</u> of <u>stress</u> and <u>strain</u>.

E = stress/strain

Thus, the "flexibility" of any object (be it pavement or airplane) depends on its elastic modulus and geometrical shape; however, it is important to note that <u>strength</u> (stress needed to break something) is not the same thing as <u>stiffness</u> (as measured by elastic modulus).

(ii) Typical values of elastic modulus for various materials include

	<u>Material</u>	<u>E (psi)</u>
1.	Rubber	1,000
2.	Wood	1-2,000,000
3.	Aluminum	10,000,000
4.	Steel	30,000,000
5.	Diamond	170,000,000
6.	Portland cement concrete	3-6,000,000

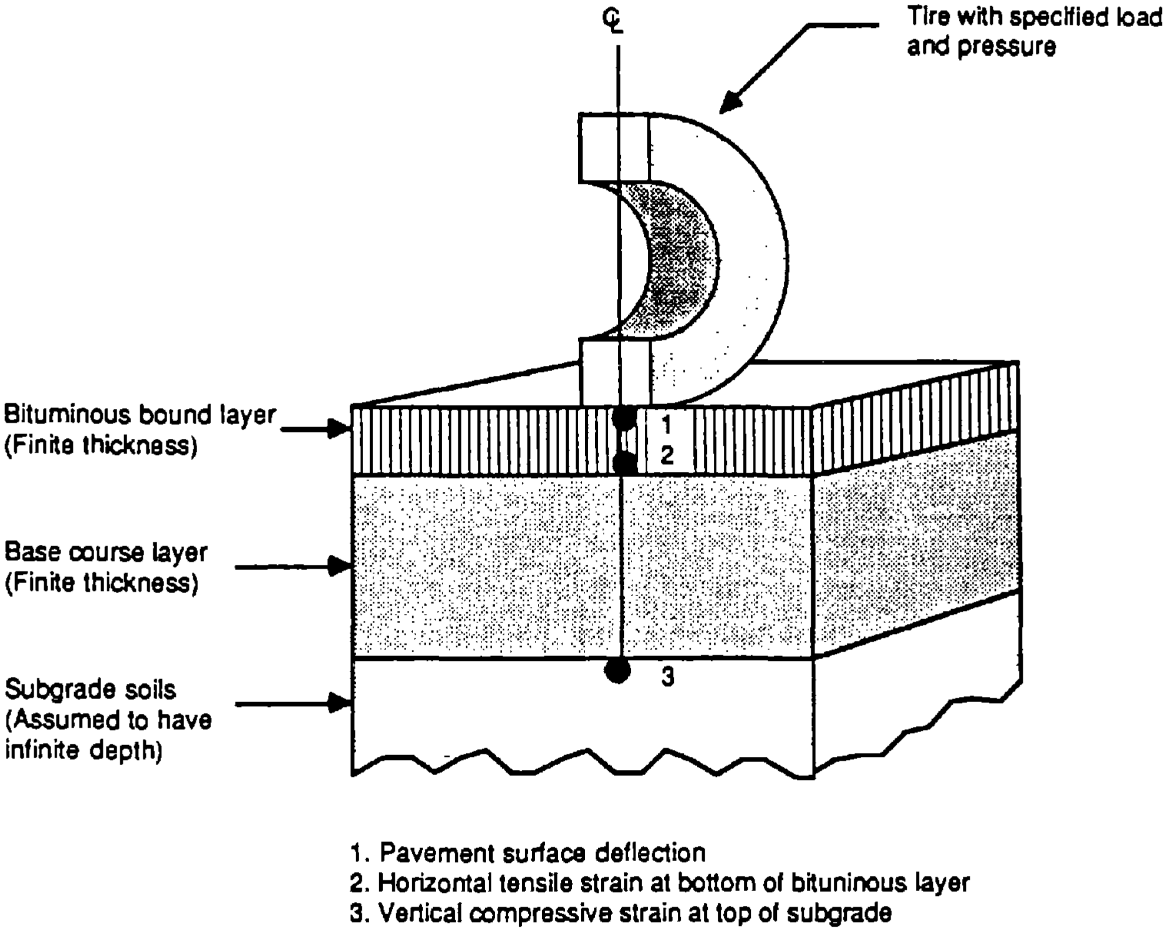


Figure 1. Pavement Response Locations Used in Evaluating Load Effects

7. Flexible pavement materials

• Asphalt concrete (at 70°F)	500,000 (±)
• Asphalt concrete (at 32°F)	2,000,000 (±)
• Asphalt concrete (at 120°F)	20,000 (±)
• Crushed stone	20-40,000 (±)
• Silty soils	5-20,000 (±)
• Clayey soils	5-10,000 (±)

(b) Poisson's ratio

Ratio of lateral strain to the longitudinal strain within the elastic range of a material. Typical values include

	Material	Poisson's ratio μ
1.	Steel	0.25 - 0.30
2.	Aluminum	0.33
3.	Portland cement concrete	0.15 - 0.20
4.	Flexible pavement materials	
	(a) Asphalt concrete	0.35 (±)
	(b) Crushed stone	0.40 (±)
	(c) Soils (fine-grained)	0.45 (±)

1.5 Implementation of elastic analysis

Pavement elastic analysis has existed since the 1800's; however, multilayered elastic analysis which can be used to realistically characterize actual pavement structures (with several, separate layers) has existed since only the 1960's. During the early 1960's, the first computer programs were prepared to do the complex and time consuming calculations required of multilayered elastic analysis. During the 1980's, as these "mainframe" computer programs were converted to run on "microcomputers," greater potential exists to use elastic analysis tools in pavement design and rehabilitation decision making.

2. TYPICAL EXAMPLES AND RESULTS

2.1 Typical pavement structures

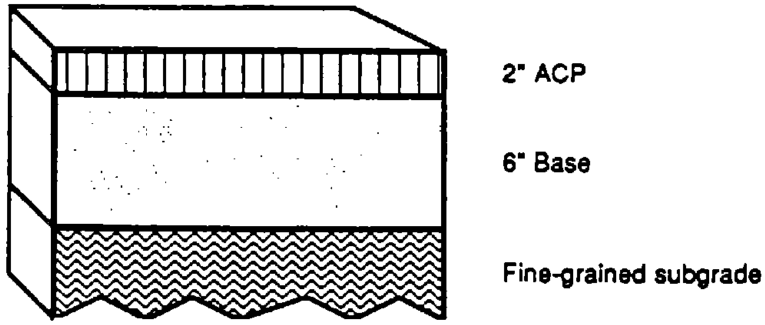
Three "typical" pavement structures are shown in Figure 2 (these pavements were selected for illustration purposes only). The initial "inputs" are

2.1.1 Tire load = 9,000 pounds

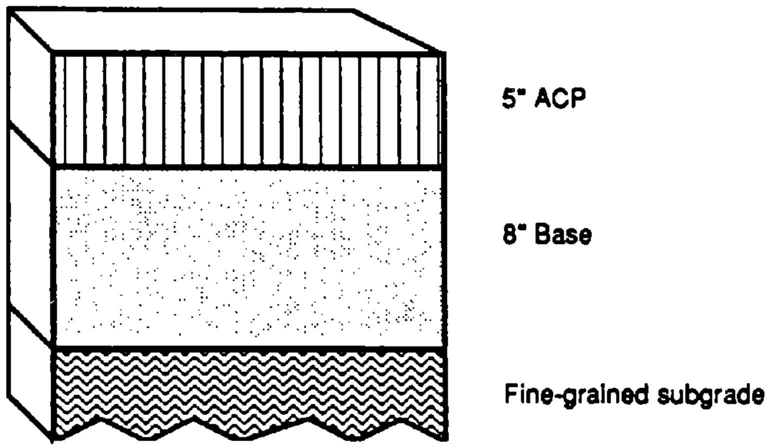
2.1.2 Tire pressure = 80 psi

2.1.3 Elastic moduli

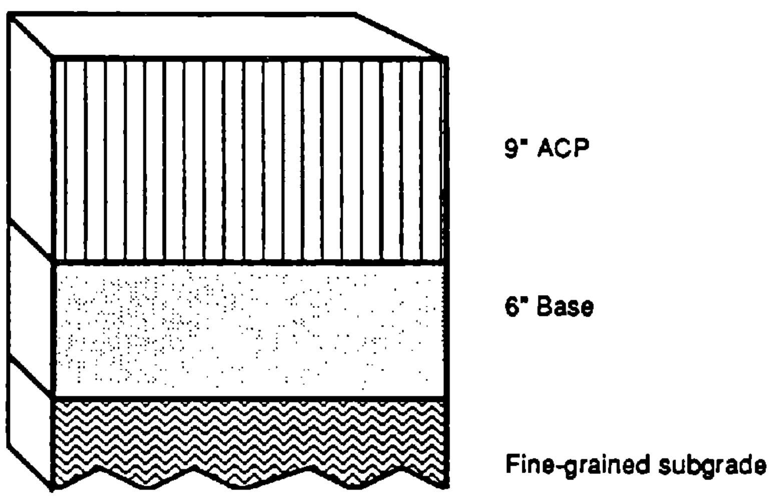
(a) Asphalt concrete = 500,000 psi
(b) Crushed stone base = 25,000 psi
(c) Fine-grained subgrade = 7,500 psi



Section A (Thin Thickness Section)



Section B (Medium Thickness Section)



Section C (Thick Section)

Figure 2. "Typical" Pavement Sections

2.1.4 Layer thickness

(a) Section A (low thickness): ACP = 2 inches
Base = 6 inches

(b) Section B (medium thickness): ACP = 5 inches
Base = 8 inches

(c) Section C (high thickness): ACP = 9 inches
Base = 6 inches

2.2 Example of changed inputs

Several of the inputs were changed to show how the pavement responds to these changes. These include

2.2.1 Tire load (from 9,000 pounds at 80 psi to 900 pounds at 30 psi)

2.2.2 Tire pressure (from 80 psi to 140 psi)

2.2.3 Stabilize subgrade modulus (from no stabilization (7,500 psi) to lime stabilization (50,000 psi) of top 6 inches of subgrade)

2.2.4 Base course modulus (from crushed stone base (25,000 psi) to asphalt stabilized base (500,000 psi))

2.2.5 Surface course modulus (from 500,000 psi to 200,000 psi possibly due to moisture sensitivity of the asphalt concrete)

2.3 Results of sensitivity analysis (also refer to Table 1)

2.3.1 Change in tire load

(a) Surface deflection

(i) Section A (thin): decreases 88 percent
(ii) Section B (medium): decreases 89 percent
(iii) Section C (thick): decreases 89 percent

(b) Horizontal tensile strain bottom of AC

(i) Section A (thin): decreases 74 percent
(ii) Section B (medium): decreases 84 percent
(iii) Section C (thick): decreases 88 percent

Table 1. Sensitivity Analysis of Various Input Parameters

	Pavement Response Parameter	Standard Pavement	Low Tire Load	High Tire Pressure	Stabilized Subgrade	Asphalt Treated Base	Moisture Sensitive
1.	Surface deflection top of AC (in.)						
1.1	Section A (thin)	0.048	0.006	0.052	0.036	0.021	0.053
1.2	Section B (med)	0.027	0.003	0.028	0.023	0.014	0.033
1.3	Section C (thick)	0.018	0.002	0.019	0.016	0.012	0.024
2.	Horizontal tensile strain bottom of AC or ATB (in/in x 10^{-6})						
2.1	Section A (thin)	469	121	735	369	193	482
2.2	Section B (med)	279	44	352	246	88	433
2.3	Section C (thick)	145	17	161	128	67	258
3.	Vertical compressive strain top of subgrade (in/in x 10^{-6})						
3.1	Section A (thin)	-2,239	-284	-2,554	-956	-508	-2,604
3.2	Section B (med)	-755	-79	-790	-431	-222	-1,037
3.3	Section C (thick)	-371	-38	-375	-245	-169	-608

Note: Tension (+)
Compression (-)

(c) Vertical compressive strain top of subgrade

(i) Section A (thin): decreases 87 percent
(ii) Section B (medium): decreases 90 percent
(iii) Section C (thick): decreases 90 percent

2.3.2 Change in tire pressure

(a) Surface deflection

(i) Section A (thin): increases 8 percent
(ii) Section B (medium): increases 4 percent
(iii) Section C (thick): increases 6 percent

(b) Horizontal tensile strain bottom of AC

(i) Section A (thin): increases 57 percent
(ii) Section B (medium): increases 26 percent
(iii) Section C (thick): increases 11 percent

(c) Vertical compressive strain top of subgrade

(i) Section A (thin): increases 14 percent
(ii) Section B (medium): increases 5 percent
(iii) Section C (thick): increases 1 percent

2.3.3 Lime stabilize subgrade

(a) Surface deflection

(i) Section A (thin): decreases 25 percent
(ii) Section B (medium): decreases 15 percent
(iii) Section C (thick): decreases 11 percent

(b) Horizontal tensile strain bottom of AC

(i) Section A (thin): decreases 21 percent
(ii) Section B (medium): decreases 12 percent
(iii) Section C (thick): decreases 12 percent

(c) Vertical compressive strain top of unstabilized subgrade

(i) Section A (thin): decreases 57 percent
(ii) Section B (medium): decreases 43 percent
(iii) Section C (thick): decreases 34 percent

2.3.4 Base course changed to ATB

(a) Surface deflection

(i) Section A (thin): decreases 56 percent
(ii) Section B (medium): decreases 48 percent
(iii) Section C (thick): decreases 33 percent

(b) Horizontal tensile strain bottom of ATB

(i) Section A (thin): decreases 59 percent
(ii) Section B (medium): decreases 68 percent
(iii) Section C (thick): decreases 54 percent

(c) Vertical compressive strain top of subgrade

(i) Section A (thin): decreases 77 percent
(ii) Section B (medium): decreases 71 percent
(iii) Section C (thick): decreases 54 percent

2.3.5 Moisture sensitive asphalt concrete layer

(a) Surface deflection

(i) Section A (thin): increases 10 percent
(ii) Section B (medium): increases 22 percent
(iii) Section C (thick): increases 33 percent

(b) Horizontal tensile strain bottom of AC
(i) Section A (thin): increases 3 percent
(ii) Section B (medium): increases 55 percent
(iii) Section C (thick): increases 78 percent

(c) Vertical compressive Strain top of subgrade

(i) Section A (thin): increases 16 percent
(ii) Section B (medium): increases 37 percent
(iii) Section C (thick): increases 64 percent

(d) Figure 3 is used to further illustrate the way the thin, medium and thick pavement structures response to a constant load (recall P=9,000 pounds at 80 psi) if the surface course elastic modulus is equal to 500,000 psi. In this case, the estimated surface deflections are shown for various locations which form "deflection basins." Clearly, the "thin" pavement has a much larger deflection basin than the medium or thick pavements.

Figure 4 also shows these deflection basins for the thin, medium and thick pavements with surface course moduli = 500,000 psi but also the basins for surface course moduli = 200,000 psi. From Figure 4 one can see that the change in the deflection basin is affected for a greater horizontal distance for the thicker surface course pavements.

3. DESIGN CONSIDERATIONS

The primary goal in using elastic layered analysis in design is to minimize critical stresses, strains and deflections.

Some of the ways this can be done include

3.1 Reduce the modulus ratio between the upper layers (i.e., decrease E1 ÷ E2 ratio).

3.2 Increase the thickness ratio of the upper layers (i.e., increase h1 ÷ h2 ratio).

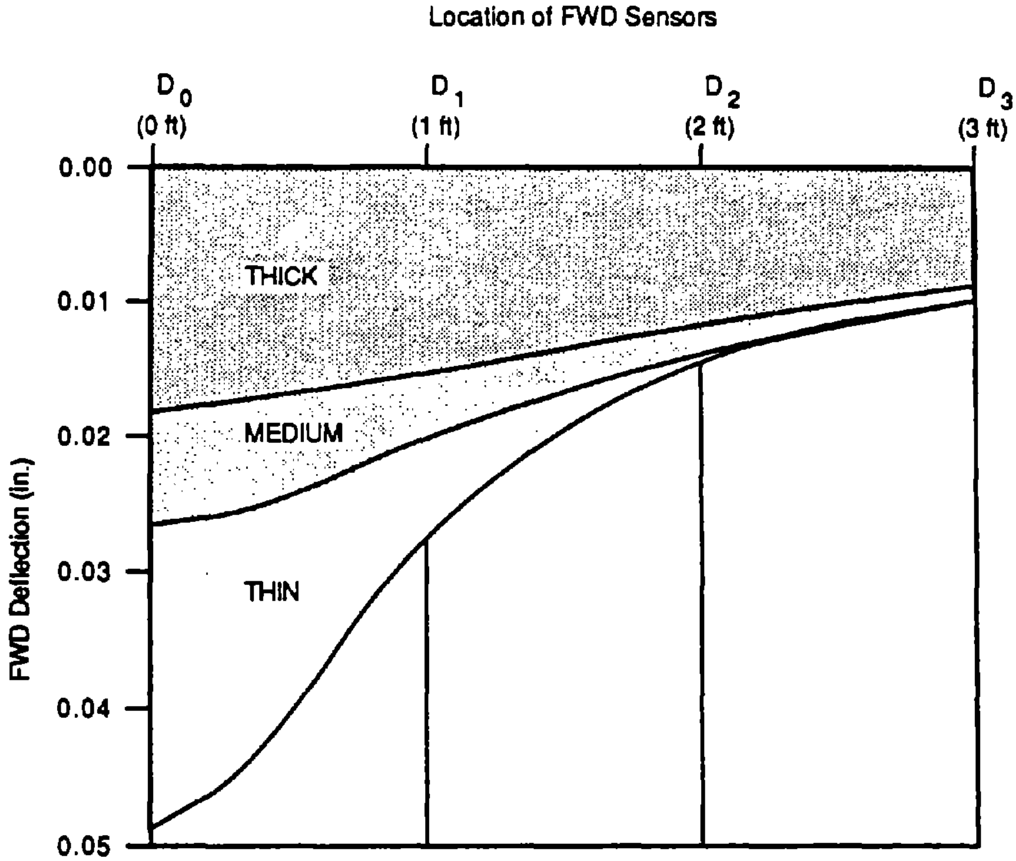


Figure 3. Comparison of Deflection Basins for the "Standard" Typical Pavements

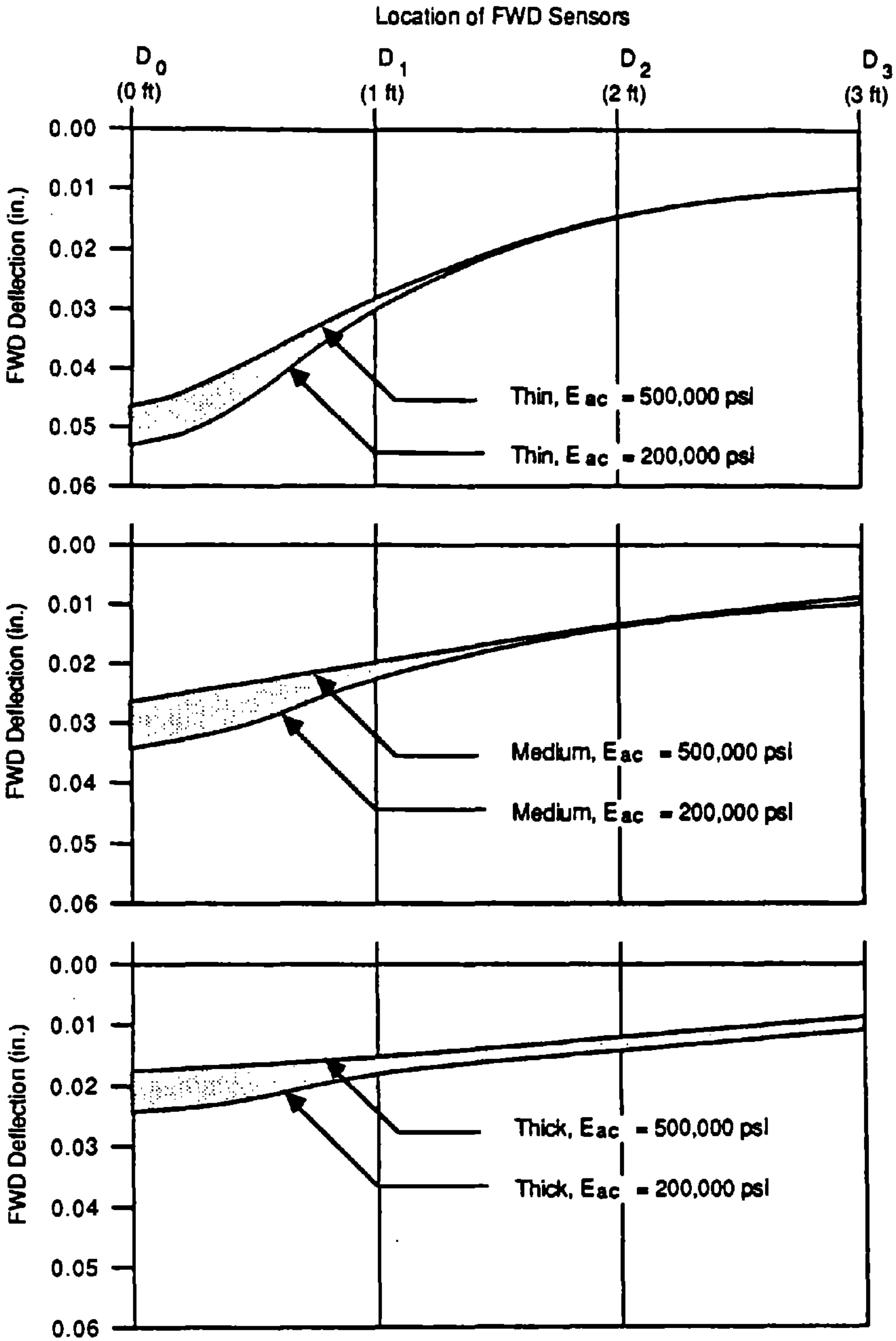


Figure 4. Differences in Deflection Basins for the "Standard" vs. "Moisture Sensitive" Pavement Sections

4. RELATIONSHIP BETWEEN PAVEMENT RESPONSE AND PAVEMENT PERFORMANCE

4.1 Introduction

The relationship between pavement response (such as stress, strain or deflection) and pavement performance (such as fatigue cracking or rutting) is complicated and difficult to quantify.

4.2 Typical relationships

"Typical" relationships used to estimate asphalt concrete fatigue cracking and rutting are shown as Figures 5 and 6. For example, the use of these relationships can be illustrated as follows:

4.2.1 Fatigue cracking

For the "standard" pavement and load (Figure 2 and Table 1), the estimated tensile strain at the bottom of the asphalt concrete layer for Section B is 279×10^{-6}in/in. This results in an estimated "life" of about 392,000 repetitions of a 9,000 pound load on a tire with 80 psi inflation pressure.

4.2.2 Rutting

If the calculated vertical compressive strain at the top of the fine-grained subgrade for Section B is 755×10^{-6}in./in., then the estimated "life" to a rutting failure is about 134,000 repetitions of a 9,000 pound load on a tire with 80 psi inflation pressure.

4.2.3 The fatigue relationship for asphalt concrete can be changed significantly due to mixture and construction variables. For example, excessive air voids can significantly reduce the fatigue life (refer to Appendix 1 — "Effect of Compaction on Asphalt Concrete Life" — for additional information).

4.3 More examples — calculation of relative damage to pavements due to different axle loads or structural conditions

4.3.1 Fourth power law

The relative damage of one axle load when compared to another is referred to as the "fourth power law" (confirmed by the AASHO Road Test). For example, if we want to determine (roughly) how damaging a 40,000 pound single axle is compared to a 20,000 pound single axle, then

$$\left(\frac{40{,}000}{20{,}000}\right)^4 \cong 16$$

Thus, the 40,000 pound axle is 16 times more damaging than the 20,000 pound axle. Similarly, if we compare an auto axle of say 1,800 pounds to a 18,000 pound single truck axle, then

$$\left(\frac{18{,}000}{1{,}800}\right)^4 \cong 10{,}000$$

Thus, based on the fourth power law, a typical auto axle is 10,000 times less damaging than an 18,000 pound truck axle.

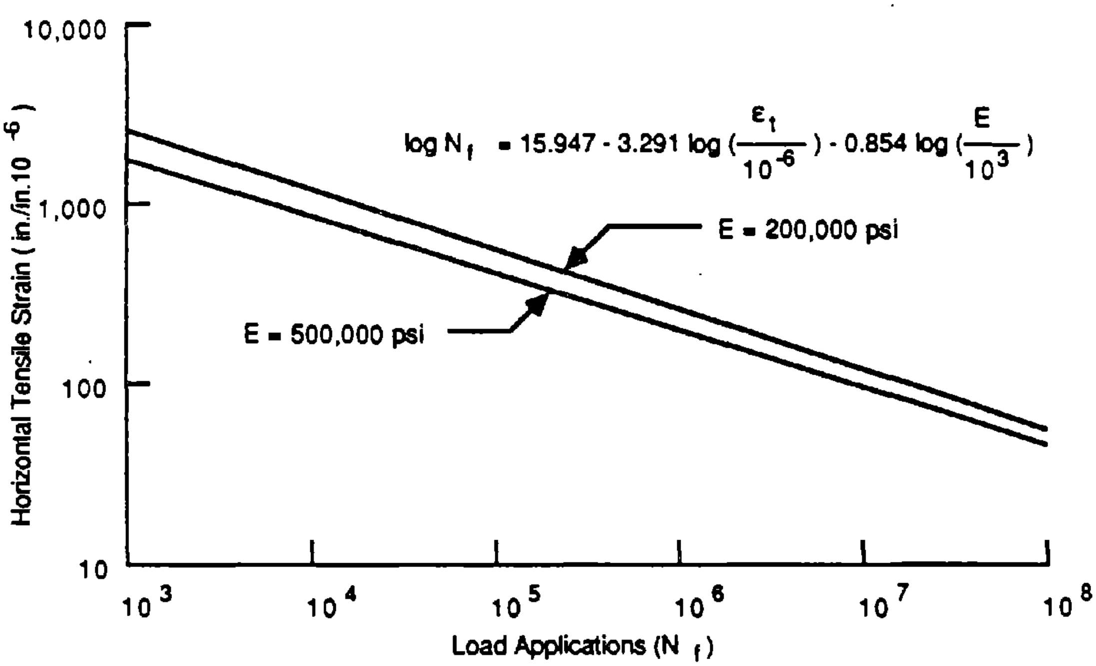


Figure 5. Limiting Horizontal Strain Criterion for Asphalt Concrete Fatigue Cracking.

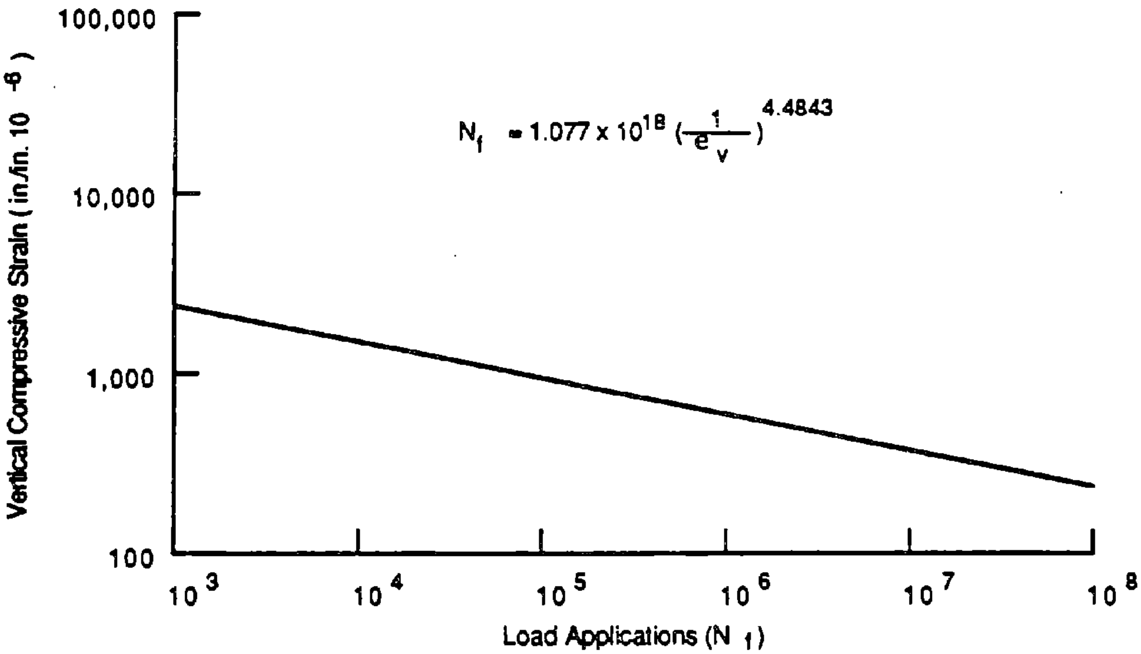


Figure 6. Limiting Subgrade Strain Criterion for Rutting

4.3.2 Relative damage based on "typical" pavement sections

4.3.2.1 The equivalency of one axle load to another can be represented by the following:

$$F_i = \frac{N_{18,000}}{N_i}$$

where

F_i	=	equivalency factor
$N_{18,000}$	=	repetitions to failure for an 18,000 lb. single-axle load
N_i	=	repetitions to failure for the axle and tire configuration in question

4.3.2.2 For Section B (N values obtained from Figure 5), if we change the AC modulus from 500,000 psi ("standard pavement") to 200,000 psi (an AC layer with high air voids and possibly moisture sensitive), then the following loads to failure result:

N_{500ksi}	$\cong$	392,000 (18,000 lb. single axle, fatigue failure, AC = 500 ksi),
N_{200ksi}	$\cong$	202,000 (18,000 lb. single axle, fatigue failure, AC = 200 ksi), and
F	=	$\frac{392,000}{202,000} = 1.94$

Thus, for the same truck tire load, a pavement with moisture damage (as characterized by lower AC elastic modulus) is affected almost <u>twice as much</u> (actually, 1.94 times by the <u>same load</u>); or stated another way, it may reduce the service life by almost 50 percent.

4.3.2.3 An extension of changing the AC modulus from 500,000 psi to 200,000 psi for the thin, medium and thick pavement sections illustrates the changes in fatigue life for each. If we do this for the standard load, the following results (based on data in Table 1 and Figure 5):

(a) Thin pavement
$N_{500ksi} \cong 71,000$ (for $\epsilon_t = 469 \times 10^{-6}$ in./in.)
$N_{200ksi} \cong 142,000$ (for $\epsilon_t = 482 \times 10^{-6}$ in./in.)

(b) Medium pavement
$N_{500ksi} \cong 392,000$ (for $\epsilon_t = 279 \times 10^{-6}$ in./in.)
$N_{200ksi} \cong 202,000$ (for $\epsilon_t = 433 \times 10^{-6}$ in./in.)

(c) Thick pavement
$N_{500ksi} \cong 3,381,000$ (for $\epsilon_t = 145 \times 10^{-6}$ in./in.)
$N_{200ksi} \cong 1,110,000$ (for $\epsilon_t = 258 \times 10^{-6}$ in./in.)

(d) Thus, a loss in AC "stiffness" has a serious impact on the medium and thick surface courses. Further, this analysis suggests that thin surface courses may perform better at lower stiffness levels; however, if a surface course is truly moisture sensitive, it never performs better. Recall that the allowable fatigue criterion is a function of stiffness as well as strain.

APPENDIX 1
EFFECT OF COMPACTION ON ASPHALT CONCRETE LIFE

1. ASPHALT CONCRETE PERFORMANCE

1.1 Fatigue

(a) What is fatigue of asphalt concrete? Fatigue of asphalt concrete refers to cracking caused by repeated bending due to traffic. Fatigue related pavement cracking is often referred to as "alligator cracking."

(b) How do air voids affect asphalt concrete fatigue life? Various authors [1, 2, 3] have shown that the fatigue life of asphalt concrete can be reduced from 10 to 30 percent for each 1 percent increase in air voids. For example, if 7 percent air voids were achievable for a mix and 11 percent air voids resulted following construction, then one can conservatively expect about a 40 percent reduction in pavement surfacing life.

(c) How do air voids affect the thickness of asphalt concrete? At first glance, this question makes no sense. However, due to fatigue considerations, Finn and Epps [1] showed that the effective thickness of asphalt concrete layers are reduced due to increasing air voids. They considered two thicknesses of asphalt concrete, 4 and 6 inches both at a starting point of 7 percent air voids (generally considered achievable in normal paving construction). The following summarizes their example [after Ref. 1]:

Asphalt Concrete Air Voids (%)	Effectiveness Thickness Asphalt Concrete (inches)	
7	4.0	6.0
8	3.5	5.0
9	3.0	4.5
10	2.5	4.0
12	2.0	4.0

Thus, if the air voids are increased from a desirable level of 7 percent to a poor compaction level of 12 percent, then a 4-inch-thick asphalt concrete layer will effectively last as long as only a 2-inch layer (and a 6-inch layer would effectively be reduced to a 4-inch-thick layer).

1.2 Aging

(a) What is "aging" of asphalt concrete? Aging can be defined in many ways. One way is to base a judgment of aging by determining asphalt cement properties such as asphalt penetration.

(b) What do typical test results show relative to air voids and asphalt hardening (lower asphalt penetration)? A study by Goode and Owings [4] showed for asphalt concrete mixtures 4 years after construction that the asphalt penetration is reduced about 6 percent for each 1 percent increase in air voids. For example, they showed that mixes retained about 75 percent of their original (immediately after construction) asphalt penetration for an air void level of 6 percent. If the air voids immediately after construction were 12 percent, then the asphalt penetration was only 30 percent of is original after construction level.

Again, the lower the asphalt penetration, the more susceptible the mixture is to various effects such as freeze-thaw cycles, moisture damage and cracking. This was well illustrated by Hubbard and Gollumb [5] which showed that asphalt penetrations of 30 or less (at 77 degrees F) generally leads to distressed asphalt concrete. High air void pavements as shown by Goode and Owings [4] can be expected to result in these low penetration levels for original (before mixing) 85–100 penetration asphalt cement (similar penetration levels for the WSDOT AR–4000 specification).

2. RECOMMENDATIONS

The recommendations by Finn, et al. [6], appropriate with respect to the maximum air voids for asphalt concrete construction:

Asphalt Concrete Layer		Maximum Air Voids (percent) Light Traffic	Moderate to Heavy Traffic
(a)	Upper 1.5-2 inches of asphalt concrete pavement	8	7
(b)	Asphalt concrete pavement deeper than 2 inches	7	6

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FUNDAMENTALS OF RESILIENT MODULUS TESTING

by

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Paper prepared for the

Workshop on Resilient Modulus Testing
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Corvallis, OR

March 28-30, 1989

Fundamentals of Resilient Modulus Testing[1]

Ted S. Vinson[2]

Introduction

A prediction of stresses, strains, and deflections induced in a layered pavement structure under traffic wheel loadings requires an understanding of the stress-strain behavior of the materials comprising the structure. Combined stress-strain behavior is typically expressed in terms of modulus. The major component of deformation or strain that is induced by a single wheel load is not permanent or associated with rupture; it is recoverable. Therefore, it is appropriate to identify the resilient modulus as the required input to determine the stresses, strains, and deflections in a pavement structure under wheel loadings.

Over the past three decades, a number of test systems and procedures have been used to determine the resilient modulus of pavement materials. These include the follow basic repeated-load tests:

(1) direct tension
(2) beam flexure (bending or rotating cantilever)
(3) indirect diametral tension
(4) triaxial compression

Of these four tests, diametral indirect tension and triaxial compression are presently considered to be the simplest, most practical, and economical methods to obtain resilient moduli of pavement materials. In general, the diametral indirect tension test is used for bound materials, such as asphalt concrete or cement-treated base, and the triaxial compression test is used for unbound materials, such as cohesionless base course soils or cohesive subgrade soils.

Purpose

The purpose of this paper is to present the fundamental aspects of resilient modulus (i.e., repeated load) testing of pavement materials using diametral indirect tension or triaxial compression test systems. A historical background of repeated load test methods and

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load application conditions is presented so that the currently used diametral and triaxial test systems and procedures may be better appreciated.

Historical Background - Test Methods

Pavement thickness design prior to World War II was basically empirical, based on experience, soil classification, and the response of a pavement structure to static load, e.g., a plate load or CBR test. A minimum thickness for a surface course was often selected based on plastic deformation as the only failure criterion. The possibility of extensive cracking of asphalt concrete pavements resulting from excessive elastic deflections and without any significant plastic deformation was not even considered. Concerns about this approach were expressed by many, however, including Professor A. Casagrande (Burmister, 1943), who wrote:

> "Irrespective of the theoretical method of evaluation of load tests, there remains the important question as to what extent individual static load tests reflect the results of thousands of dynamic load repetitions under actual traffic. Experience and large-scale traffic tests have already indicated that various types of soils react differently, and that the results of static load tests by no means bear a simple relation to pavement behavior."

Shortly after World War II, several investigators (e.g., Campen and Smith (1947), McLeod (1947), Phillippe (1947), Hittle and Goetz (1947)) used repeated plate load tests on model pavement sections with the number of load repetitions of the order of 10. The primary objective of their investigations was to determine the effect of repetition of load on the deformation and not to determine the resilient modulus.

Repeated plate load tests were costly to perform and involved considerable time. Consequently, within a few years after the transition from static to repeated plate load tests in the field a transition was made from static triaxial to repeated load triaxial tests in the laboratory. A substantial contribution in this pioneering effort was made by Seed, Chan, and Monismith (1955), Seed and McNeill (1958), and Seed, Chan, and Lee (1963). The collective work of these investigators focused on the determination of the deformation characteristics and resilient modulus of compacted subgrades. In the early stages of their work, a comparison was made between values of initial tangent modulus determined from the stress versus strain relationships in unconfined compression tests and the values of resilient modulus determined by repeated application of a 25 psi axial stress on unconfined specimens. The results, shown on Figure 1, indicate the differences between these moduli are considerable, suggesting the behavior of soils under traffic loading can only be obtained from repeated loading tests. This conclusion is further substantiated with data obtained by the California Division of Highways (Hveem, 1955)

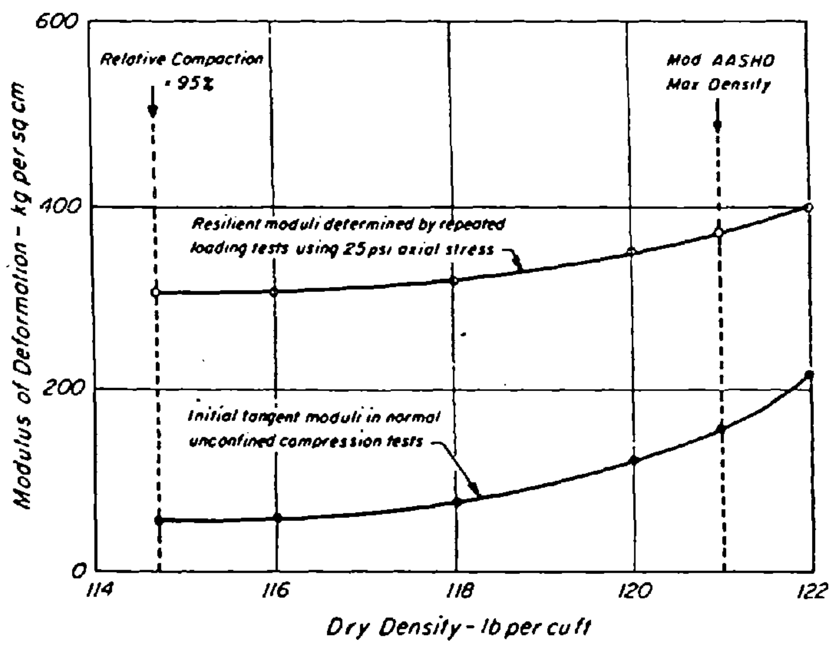


Figure 1. Comparison of Moduli of Deformation Determined by Normal Unconfined Compression and Repeated Loading Tests (after Seed and McNeill, 1958)

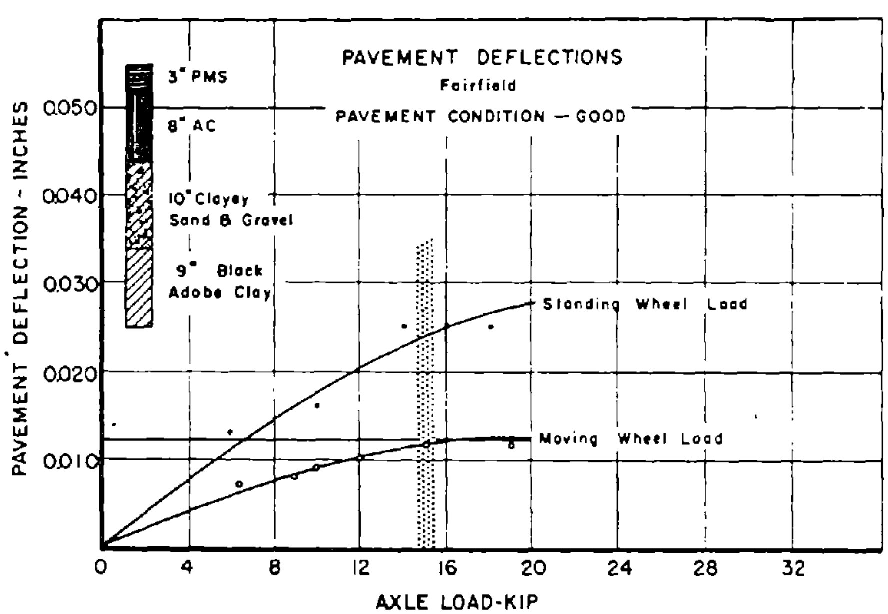


Figure 2. Pavement Deflections Under Standing and Moving Wheel Loads (after Hveem, 1955)

presented in Figure 2 which shows the marked difference in pavement deflections occurring under standing and slowly moving wheel loads.

Seed et al. recognized that their work was only a useful first step towards the ultimate solution of the problem. A determination of pavement deflections and the prediction of the fatigue life of the pavement also requires a determination of the resilient modulus of the base course and asphalt concrete surface, as well as characterization of the fatigue life of the asphalt concrete.

Early work to evaluate the characteristics of asphalt concrete under repeated loadings were conducted in the United States by Papazion and Baker (1959), Monismith, Secor, and Blackmer (1961), Jimenez (1962), and Jimenez and Galloway (1963), Monismith (1963, 1964, 1965), Deacon (1965), Deacon and Monismith (1966), Monismith, Kasianchuk, and Epps (1967), Epps (1968), and Kasianchuk (1968). In Europe, early work included contributions by Nijboer (1959), Pell, McCarthy, and Gardner (1961), Pell and McCarthy (1962), and Pell (1963, 1966, and 1967). While the focus of much of this work was on the influence of mix characteristics on asphalt concrete fatigue life, a byproduct of the work was a determination of resilient modulus. The majority of the U.S. work referenced is associated with repeated loads applied to beam specimens in a test system configuration as shown in Figure 3. A point load rotating cantilever system was used by Pell et al., as shown in Figure 4. The resilient modulus, MR, for beam specimens may be calculated from an equation of the form

$$MR = K \frac{P}{I \cdot \Delta} \tag{1}$$

in which

K = constant depending on load, end constraint, and specimen geometry

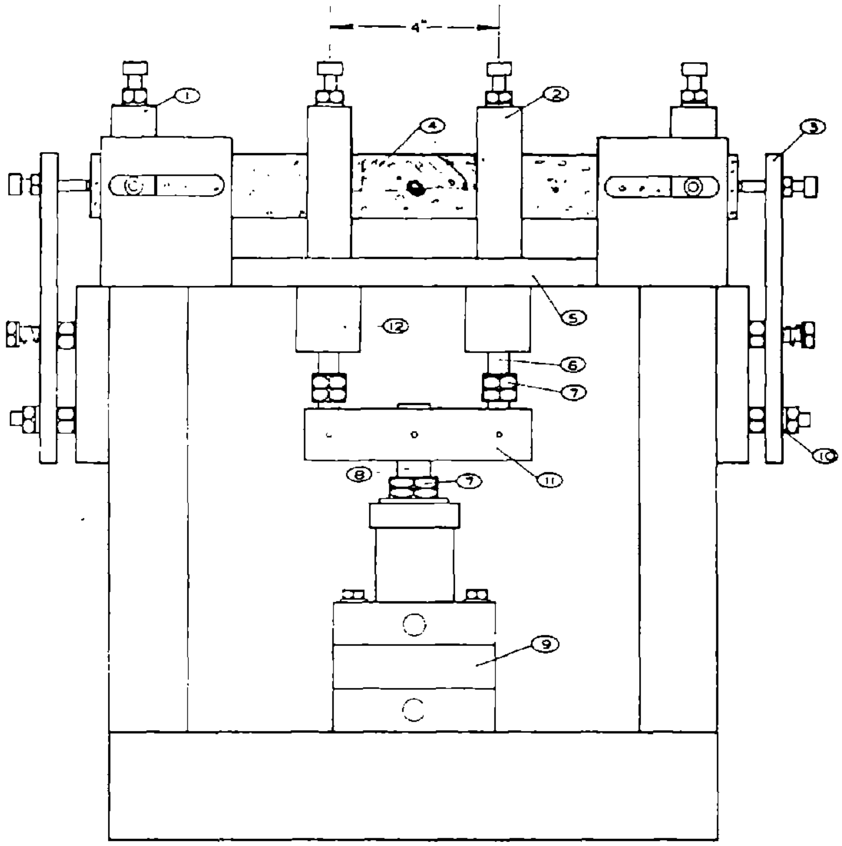
P = repeated load

Δ = measured deflection at critical location for load geometry (e.g., center or loaded end)

I = moment of inertia of beam cross section.

Repeated load triaxial compression tests were conducted on asphalt concrete specimens to obtain resilient moduli by Secor and Monismith (1961, 1964), Terrel (1967), and Dehlen (1969). Fatigue life characteristics could not be determined with the test.

In the early 1970s, Schmidt (1972) proposed the use of the repeated load indirect diametral tension test to obtain resilient moduli of asphalt concrete specimens. The test was immediately recognized to be simple and less expensive than either the beam flexure tests or triaxial compression test. Also, the fatigue life characteristics of asphalt concrete could be determined with the indirect tensile test.



Key:

1. Reaction clamp	5. Base plate	9. Double-acting, Bellofram cylinder
2 Load clamp	6. Loading rod	10. Rubber washer
3. Restrainer	7. Stop nut	11. Load bar
4 Specimen	8. Piston rod	12. Thomson ball bushing

Figure 3. Repeated Load Beam Flexure Apparatus (after Epps, 1968)

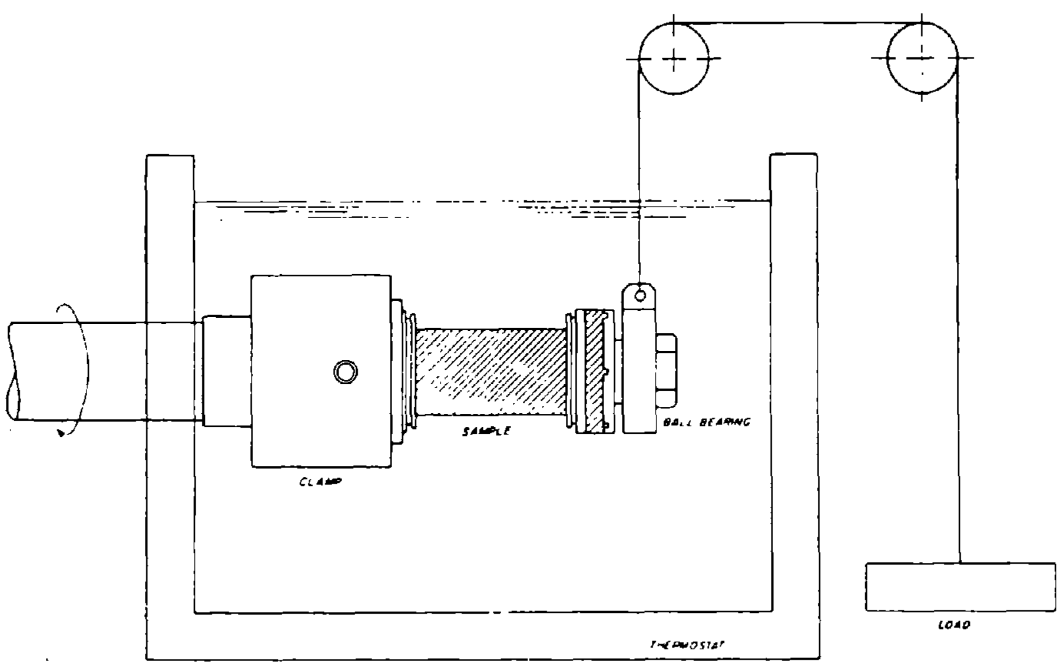


Figure 4. Point Load Rotating Cantilever Apparatus (after Saal and Pell, 1960)

Schmidt (1972) compared the resilient moduli obtained in the repeated load indirect tension test to resilient moduli determined using direct tension, triaxial compression, and beam flexure tests. The comparative results are shown in Figure 5. A favorable comparison was obtained.

Seed and Monismith (1963) in their Moderators' Report to the International Conference on the Structural Design of Asphalt Pavements noted

> "On the basis of the results reported at this Conference there would seem to be great need for increased emphasis on the study of base-course materials. The base-course characteristics may play a large part in determining both transient pavement deflections and curvature, yet apart from the new device, the resiliometer, there are no laboratory testing techniques available to evaluate base-course characteristics in this regard. The large scatter of values for base-course moduli reported by different authors is somewhat disturbing and the development of new procedures for evaluating the transient deformation characteristics of base-course materials together with a systematic study of different materials and conditions, would be highly desirable."

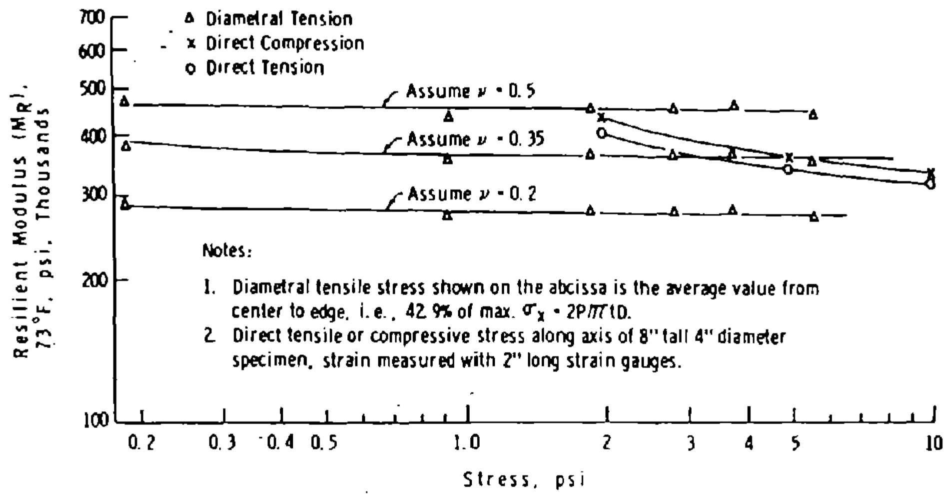
Of course, it was recognized by Seed and Monismith that much of the scatter of values for base course moduli was associated with the stress-state dependency of modulus of granular materials and the lack of incorporation of this dependency in an interpretation of resilient modulus results for these materials.

The repeated load triaxial compression test is the logical choice of test system to evaluate resilient moduli of base course materials. Mitry (1964), Seed et al. (1967), Shifley (1967), Kallas and Riley (1967), Kasianchuk (1968), Dehlen (1969), and Hicks (1970) conducted tests on granular materials which, taken collectively, provided a comprehensive picture of the influence of type and gradation of the aggregate, void ratio (dry density), and degree of saturation on resilient moduli.

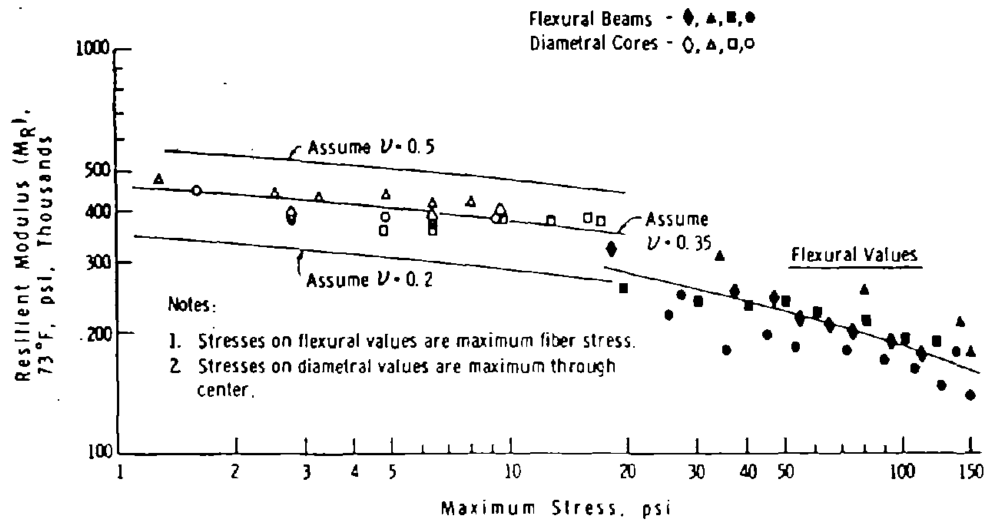
A method to address the stress-state dependency of base course materials was suggested by Biarez (1962) who established that the resilient modulus of a uniform sand measured after a few cycles of compressive stress in a triaxial apparatus could be represented by the following equation:

$$MR = K\,\sigma_m^n \tag{2}$$

in which



(a) Direct tension, compression, and diametral methods



(b) Flexural and diametral methods

Figure 5. Comparison of Resilient Moduli of AC Specimens Using Direct Tension, Compression, Flexural, and Diametral Methods (after Schmidt, 1972)

K and n = constants which depend on the material characteristics

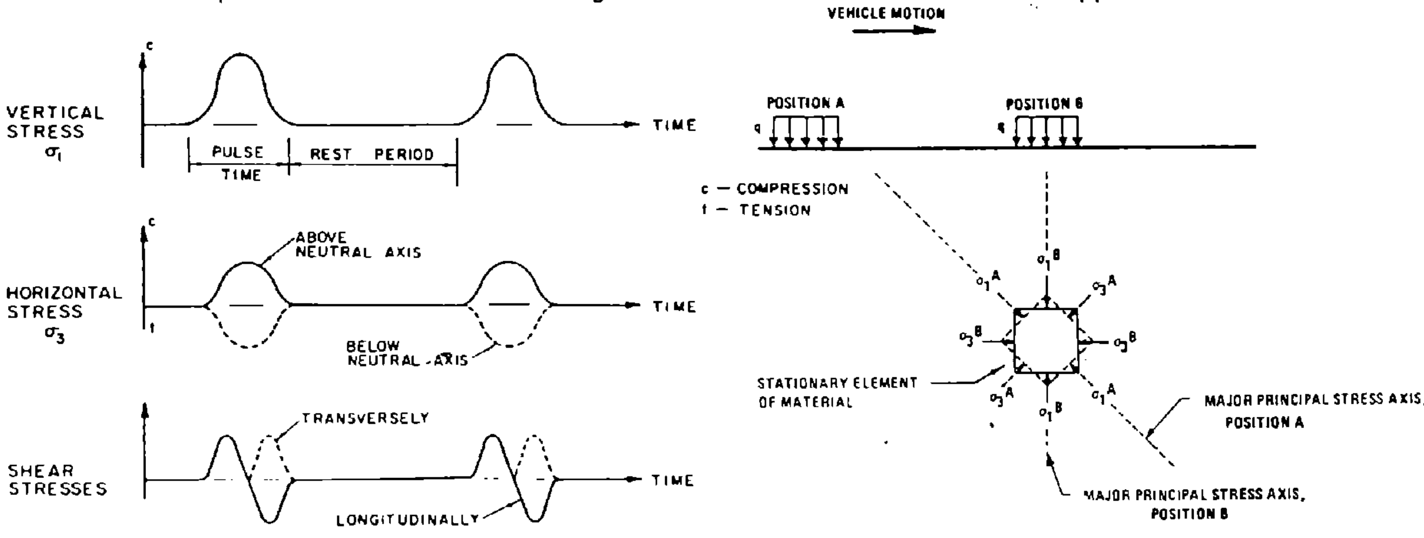
σ_m = mean normal stress

$$= \frac{\text{sum of principal stresses, } \sigma_1 + \sigma_2 + \sigma_3}{3}$$

Historical Background - Applied Load

Ideally, to estimate resilient moduli of the materials comprising a pavement structure in the laboratory one would apply stress state histories to a specimen associated with a moving wheel load passing over a representative element at some depth in the structure. Specifically, the elements in a pavement structure are subjected to a series of rapidly applied and rapidly released stresses on vertical and horizontal planes. The stress variations are shown in Figure 6a. While the magnitudes of the stress variation will differ between points in the same layer, the basic pattern is similar throughout the pavement structure. Another way of representing this situation is that the orientation of the principal stress axes of an element of material is gradually rotated as a wheel load moves along the surface. This is shown in Figure 6b. At an instant when the load is directly above the element, the principal stresses are oriented horizontally and vertically. Due to the fact that the principal stress rotates as the wheel load approaches and passes over an element, a reversal of shear stresses occurs on vertical and horizontal planes of an element.

Seed and McNeill (1958) made one of the earliest attempts to duplicate the stress state history by considering the actual variation in vertical stress on a soil element at a depth of 27 in. below the pavement surface at the Stockton test track, shown in Figure 7. Owing to the limitations of their test equipment, they did not use the actual form of the vertical stress that was observed but chose to use a square wave as shown in Figure 7. Seed and McNeill also applied a



(a) Stress variations in typical asphalt concrete element due to moving loads (after Morris and Haas, 1974)

(b) Rotation of principal stress axis of an element as a vehicle moves over the surface (after Barksdale, 1971)

Figure 6. Stress Variations and Rotation of Principal Stress Axis Due to Moving Wheel Loads

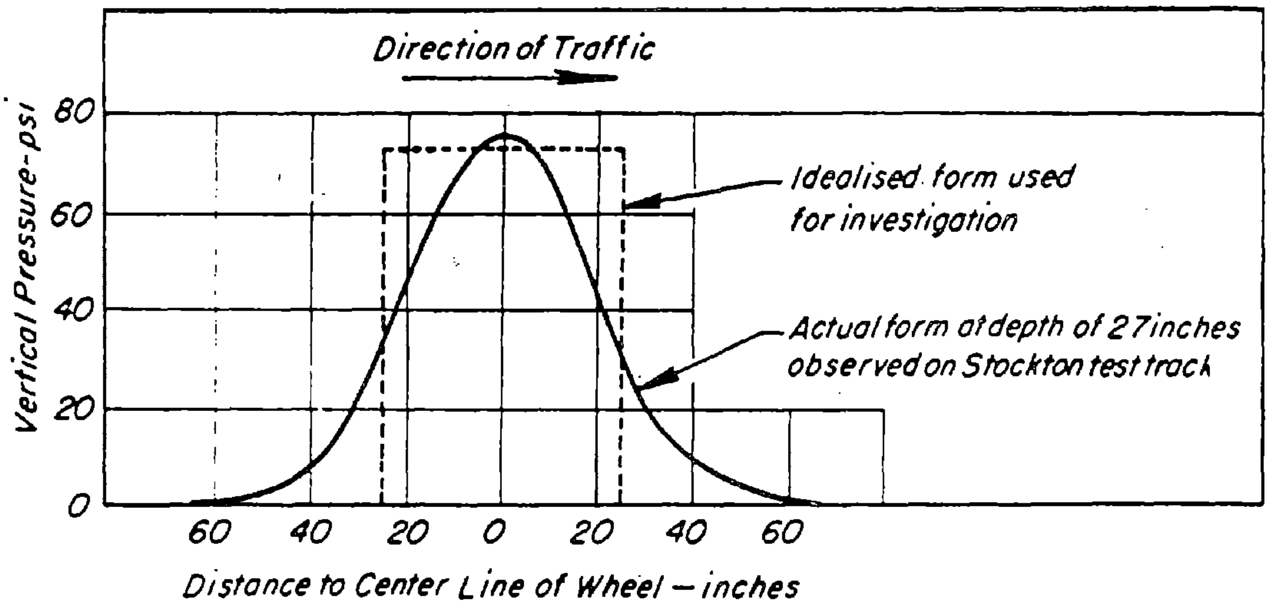


Figure 7. Changes in Stress on Soil Element Due to Moving Load (after Seed and McNeill, 1958)

repeated confining pressure to the specimens in their program to assess the consequences of changes in both the vertical and horizontal stress states on the permanent deformation characteristics of subgrade soils. Beyond this initial effort, however, only a few researchers have cycled both the vertical and horizontal stresses in repeated load triaxial tests.

Barksdale (1971) noted that vehicle speed and depth beneath the pavement surface are of great importance in selecting the appropriate axial compressive stress pulse time to use in repeated loading testing. Based on the results of a linear elastic finite element representation of a typical pavement structure, he established the relationship shown in Figure 8. For full-depth construction with 5 to 12 in. of asphalt concrete and vehicle speeds of 50 to 60 mph, pulse times of 0.03 to 0.05 sec are appropriate.

Terrel, Awad, and Foss (1974) note that since asphalt mixes exhibit viscoelastic behavior, a computed value of modulus will be dependent upon the rest period between individual stress pulses. The dependency is not significant at low temperatures and short stress durations, but may be considerable at warm temperatures and long stress durations. In the latter instance the viscoelastic response must be included as a parameter in the material characterization.

Terrel, Awad, and Foss (1974) also investigated the influence of the shape of the wave pulse on the total and resilient strains induced in an asphalt treated base material. The tests were conducted on a cylindrical specimen in a triaxial cell using a full size, triangular, and square wave pulse. Based on their test results, they concluded

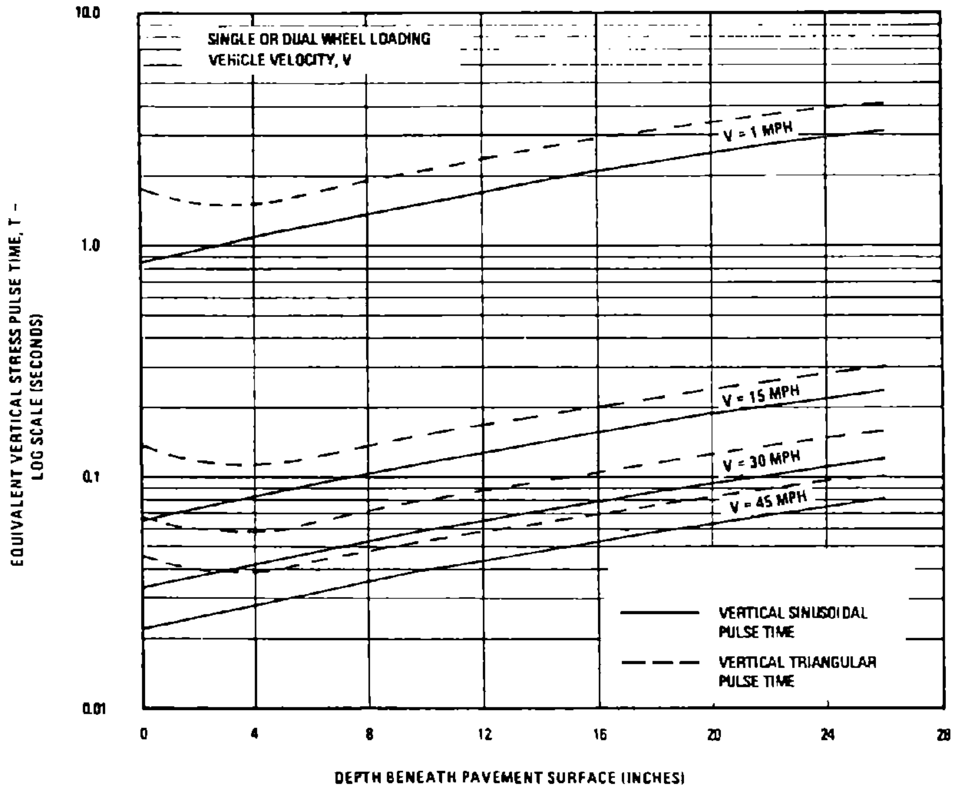


Figure 8. Variation of Equivalent Vertical Stress Pulse Time with Vehicle Velocity and Depth (after Barksdale, 1971)

(1) there is no significant difference in the magnitude of the total or the resilient stress between the triangular or the sinusoidal stress pulse,

(2) an equivalent square pulse can be obtained by applying (i) the same stress for a duration of 33 percent of the equivalent sinusoidal, or (ii) 66 percent of the stress with the same duration as the equivalent sinusoidal,

(3) a square vertical stress pulse is a reasonable approximation of the actual conditions within a pavement layer.

Fundamentals of Repeated Load Triaxial Testing

The resilient modulus of cohesionless base course materials or cohesive subgrade materials is determined in a repeated load triaxial compression test. A triaxial cell suitable for use in repeated load testing of soils is shown in Figure 9. This equipment is similar to

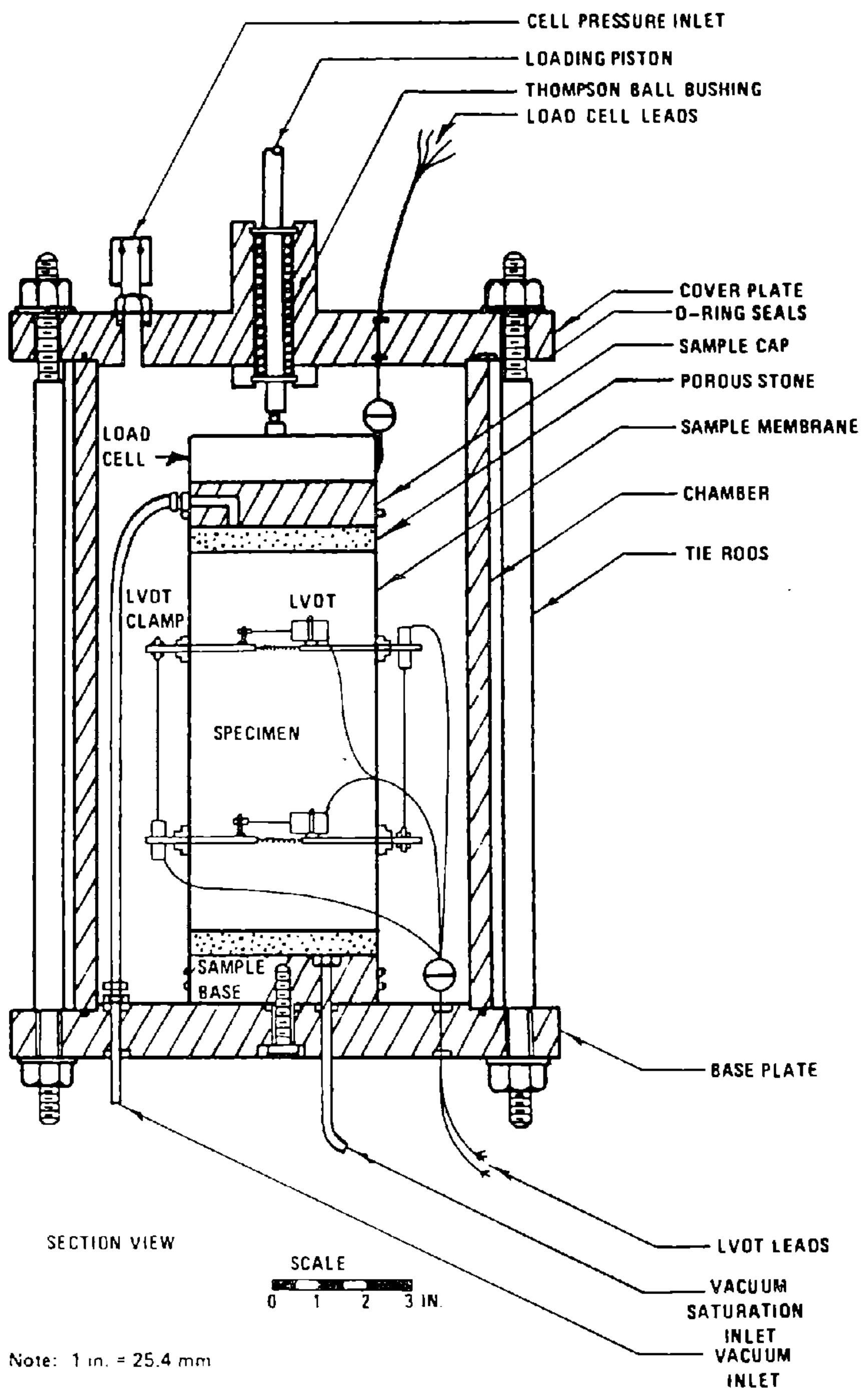


Figure 9. Triaxial Cell Suitable for Repeated Load Testing of Soils

most standard cells except that it is somewhat larger to facilitate the internally mounted load and deformation measuring equipment and has additional outlets for the electrical leads for the measuring devices. For the type of equipment shown in Figure 9, air would be used as the cell fluid. The external loading source may be any device capable of providing a variable load of fixed cycle and load duration. The device can range from simple cam and switch control of static weights or air pistons to closed loop electrohydraulic systems.

The test is conducted by placing a specimen in the triaxial cell. The specimen is subjected to an all around confining pressure, σ_c (= σ_3), and a repeated axial stress, σ_d (= $\sigma_1 - \sigma_3$), is applied to the sample. During the test the recoverable axial strain, ϵ_a, is determined by measuring the recoverable deformations across a known gage length. The resilient modulus is calculated from the following equation:

$$MR = \frac{\sigma_d}{\epsilon_a} \tag{3}$$

The pattern of soil deformation under repeated loading and a sustained confining pressure is illustrated in Figure 10. There is a small volumetric compression of the specimen when the confining pressure is first applied. Under the first application of the deviator stress there is an immediate deformation followed by a plastic deformation while the load is sustained, and then partial recovery or rebound when the load is removed. Similar patterns of deformation occur with successive load applications, although the magnitude of the deformation occurring decreases with each successive stress application. The rebound or resilient deformation remains approximately the same but the total deformation and cumulative permanent deformation both increase progressively as the number of stress applications is increased. Thus, if the total deformation is plotted against the number of applications, a curve such as that shown in Figure 10c is obtained.

The repeated axial or deviator stress is calculated from the following equation:

$$\sigma_d = \frac{P}{A} \tag{4}$$

in which

P = repeated load

A = cross-sectional area

The cross-sectional area is assumed to remain constant during a test. This is a reasonable assumption since damaging stresses are usually avoided during a test. If a load cell is internal to the cell the effect of any load piston friction is eliminated.

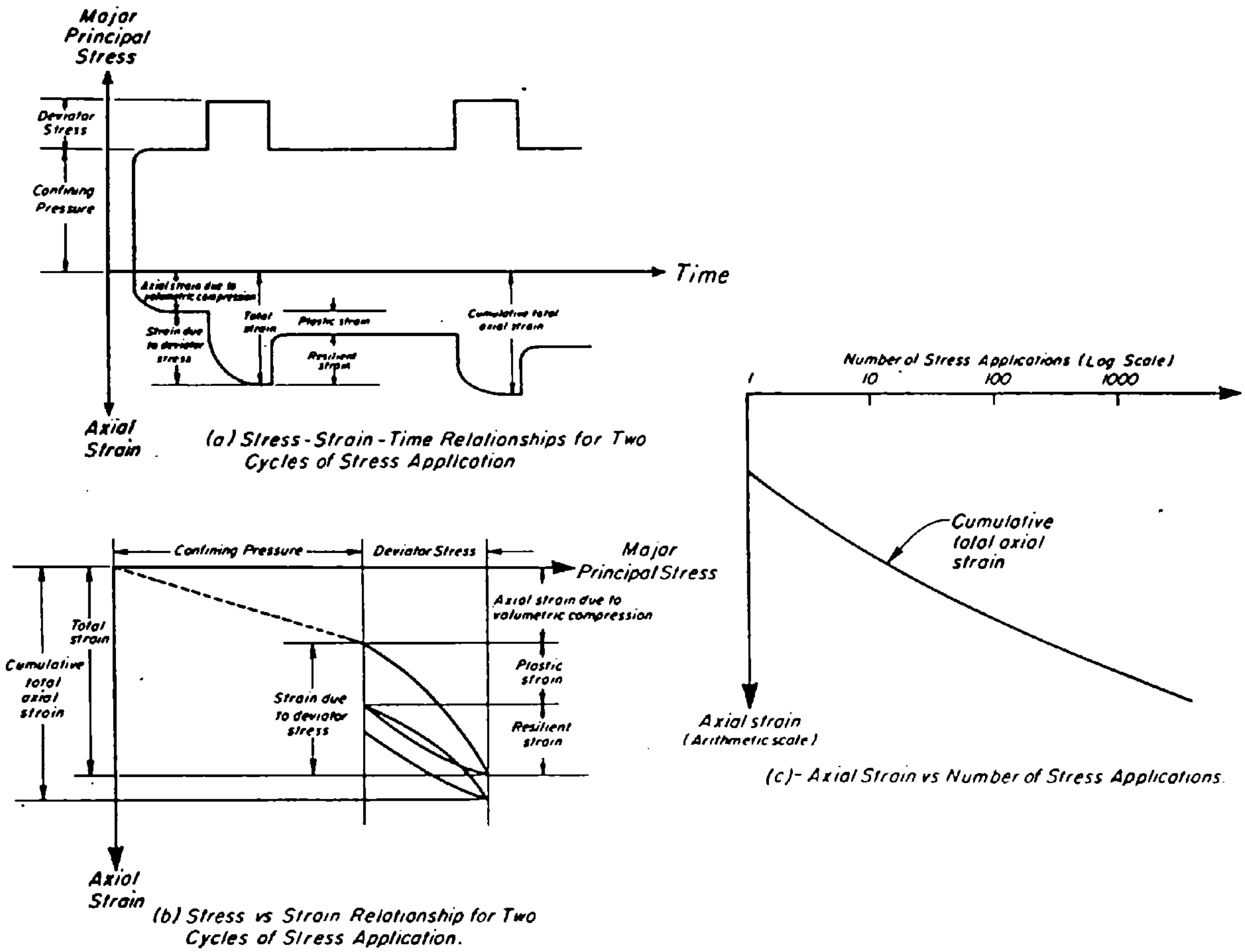


Figure 10. Pattern of Soil Deformation under Repeated Loading and a Sustained Confining Pressure (after Seed and McNeill, 1958)

The deformation measurements which support the calculation of axial strain may be made with displacement transducers which are external to the cell or displacement transducers which are clamped to the specimen. The axial strain is calculated from the following equation

$$\epsilon_a = \frac{\Delta}{L_g} \tag{5}$$

in which

Δ = axial deformation

L_g = gage length

With external displacement transducers the gage length is the specimen height. It is possible that deformations in the load train can occur during load application. For stiff materials (generally considered to be materials with MR $\geq$ 15,000 psi), the error may be unacceptable and the internal displacement transducers should be used.

Poisson's ratio may be calculated from a measurement of radial and axial deformations of the specimen which are then used to calculate radial and axial strain. The following equation is employed

$$\mu = \frac{\epsilon_r}{\epsilon_a} \tag{6}$$

in which

ϵ_r = radial strain

The standard method of test for "Resilient Modulus of Subgrade Soils" was specified in 1982 by the American Association for State Highway and Transportation Officials (AASHTO T274-82). It is generally accepted that the method is fundamentally sound. However, the test procedure requires fairly rigorous conditioning of soil specimens (i.e, the elimination of the effects of (1) the interval between compaction and loading, (2) initial loading versus reloading, and (3) initially imperfect contact between the end platens and the test specimen). The method also requires an evaluation of resilient moduli under a substantial number of stress states for both cohesive and granular (cohesionless) soils. In the opinion of many researchers and practitioners, the conditioning history and number of stress states required may be excessive and unnecessary.

For base course and other granular (cohesionless) soils, the resilient moduli obtained in a repeated load triaxial test program are expressed by an equation of the form

$$MR = K\,\theta^{C} \tag{7}$$

$$(= K_1\,\theta^{K_2})$$

in which

K (or K_1) and C (or K_2) = experimental constants which depend on the material characteristics

θ = bulk stress

$= \sigma_1 + \sigma_2 + \sigma_3$

$= \sigma_1 + 2\sigma_3$ (for a triaxial test)

$= \sigma_d + 3\sigma_c$ (for a triaxial test)

Equation (7), which is similar to Equation (2), was used by Hicks and Monismith (1971) and Kalcheff and Hicks (1973) to interpret test results for base course material. The constants K and C are derived from a set of test results by rewriting equation (7) as follows:

$$\log MR = \log K + C \log \theta \tag{8}$$

In this form it may be noted that the relationship between MR and θ should be a straight line on a log-log plot. K is the antilog of the y-intercept and C is the slope of the line. Representative test results which illustrate this relationship are shown in Figure 11. The data shown were obtained following the stress sequence and stress ratios presented in Table 1, which were recommended by Kalcheff and Hicks (1973). In some cases the stress sequence would be repeated to provide a more complete data set for a given material condition. Measurements of load and deformation for the determination of resilient properties were made after approximately 100-150 stress repetitions at a given stress condition.

The last column in Table 1 is the stress ratio, σ_1/σ_3. At failure, the stress ratio for a cohesionless material is given by the following equation:

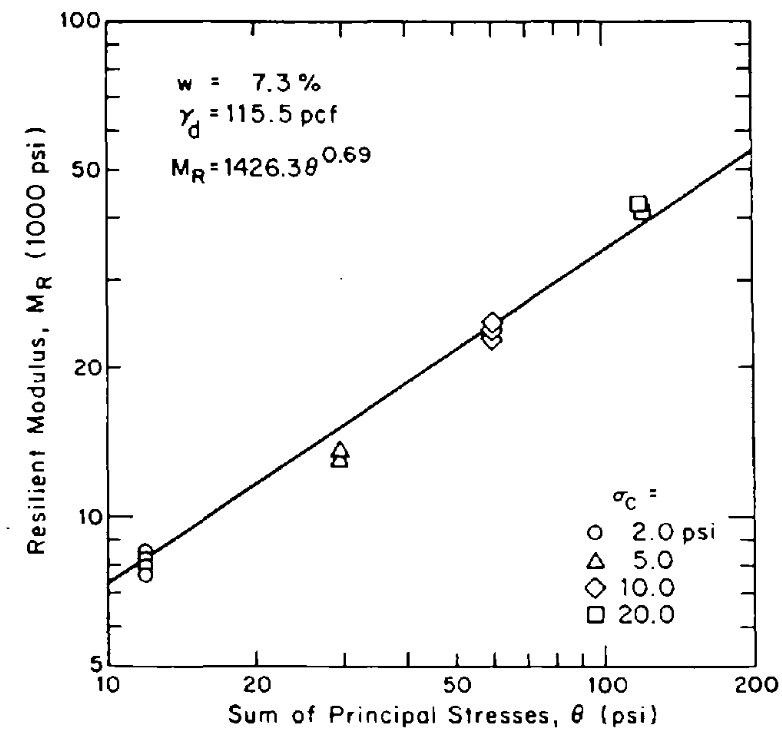


Figure 11. Resilient Modulus versus Sum of Principal Stresses for Fremont Highway Base Material (after Hull et al., 1980)

Table 1. Stress Sequence and Ratios Used for Repeated Load Tests

Material (1)	Confining Pressure, σ_3, in psi (2)	Deviator Stress, σ_d (= $\sigma_1-\sigma_3$) in psi (3)	Stress Ratio, σ_1/σ_3 (4)
Base Course (Cohesionless)	2	6	4.0
	5	15	4.0
	10	30	4.0
	20	60	4.0
	2	6	4.0
Subgrade (Cohesive)	6	8,6,4,8	2.3,2.0,1.7,2.3
	4	8,6,4,8	3.0,2.5,2.0,3.0
	2	6,4,2,6	4.0,3.0,2.0,4.0
	6	8,6,4,8	2.3,2.0,1.7,2.3

Note: Testing sequence is vertical for base materials; testing sequence is horizontal then vertical for subgrade materials.

$$\sigma_1/\sigma_3 = \tan^2 (45 + \phi/2) \tag{9}$$

in which

ϕ = angle of internal friction of the material

If this ratio is exceeded for the test material the specimen will fail along a plane or bulge during the conduct of the test. For cohesionless soils a generally accepted range of ϕ values is 30° to 45°. This suggests a corresponding range of stress ratios at failure of 3 to 5.8.

For cohesive subgrade soils the resilient modulus obtained in a repeated load triaxial test program is not expressed by an equation but is generally represented as shown in Figure 12. The stress sequence and stress ratios presented in Table 1 were used to obtain the data.

Fundamentals of Repeated Load Diametral Indirect Tension Tests

The resilient modulus of an asphalt concrete or bound (i.e., stabilized) material specimen may be determined either with the repeated load triaxial or diametral test equipment. The diametral equipment is generally used because of its simplicity. It consists of four components as follows:

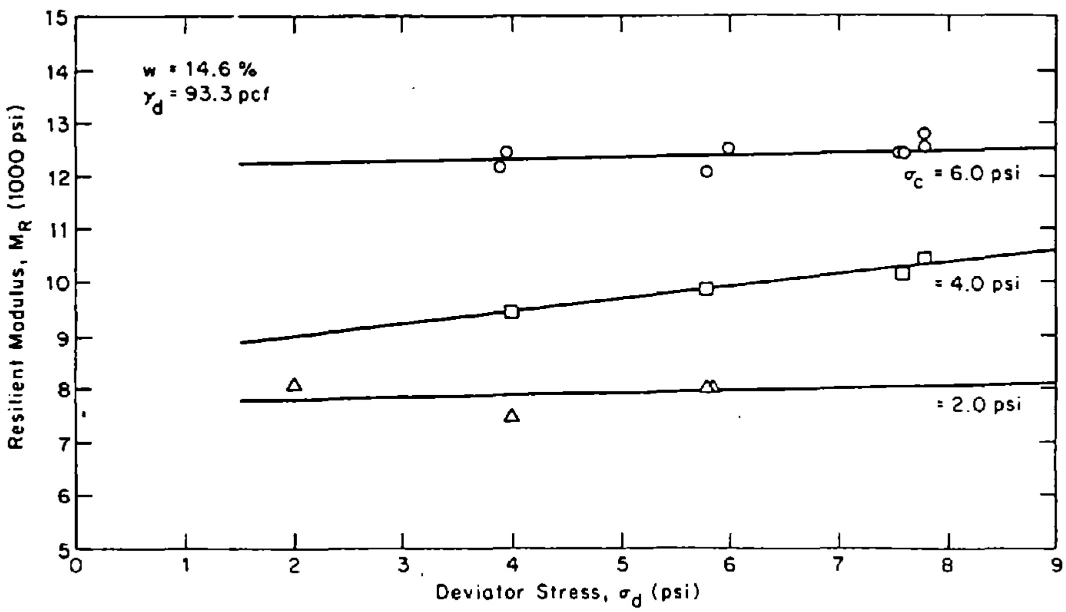


Figure 12. Resilient Modulus versus Deviator Stress for McKenzie Highway Subgrade Material (after Hull et al., 1980)

(1) control cabinet
(2) load frame
(3) diametral yoke
(4) recording system with transducer signal conditioning (e.g., strip recorder or personal computer)

An environmental cabinet may be placed around the load frame and diametral yoke to achieve temperature control of the specimen during testing. Alternatively, the specimen may be heated (or cooled) in an oven (or low temperature cabinet) to a desired test temperature, then rapidly removed for testing. (Obviously, this is not as desirable as the use of an environmental cabinet.)

To determine the resilient modulus of an asphalt concrete specimen, the specimen is placed in the diametral yoke as shown in Figure 13. The yoke with specimen is next placed in the load frame on a load cell platen with bottom loading strip attached to the cell as shown in Figure 14. The piston loading ram is used to apply an initial seating load to a top loading strip. Linear variable differential transformer (LVDT) gauge heads which are mounted on the diametral yoke are used to measure lateral deformation during repeated load. The lateral deformations are very small, of the order of 50-100 micro-inches. Consequently, the LVDTs employed with the diametral yoke must be very sensitive. Further, they must be adjusted prior to repeated load application to ensure they are in contact with the diametral

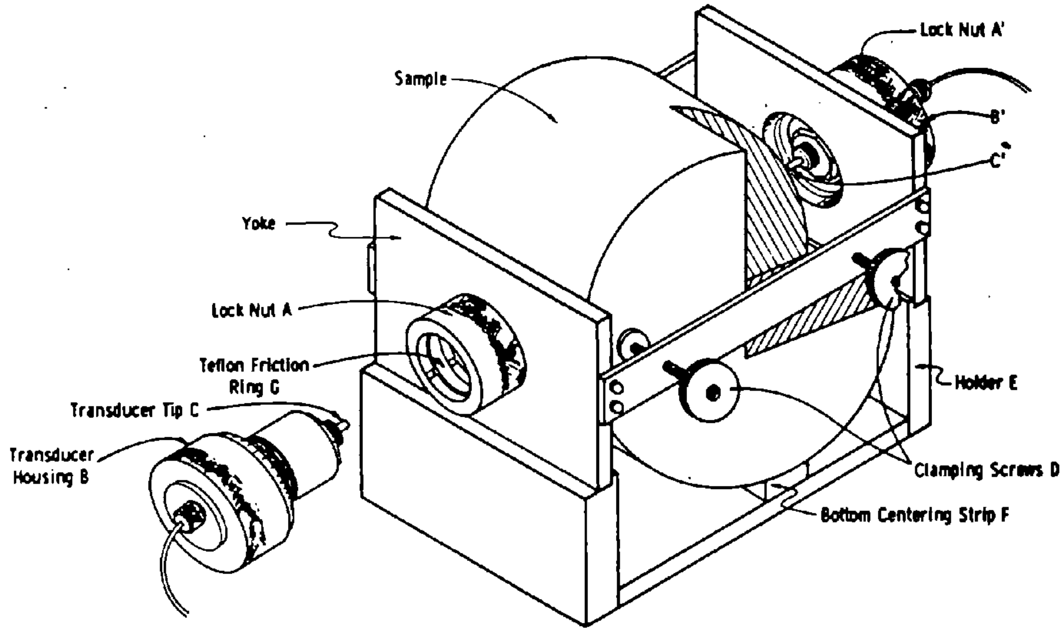


Figure 13. Diametral Resilient Modulus Device Yoke and Alignment Stand (after Schmidt, 1972)

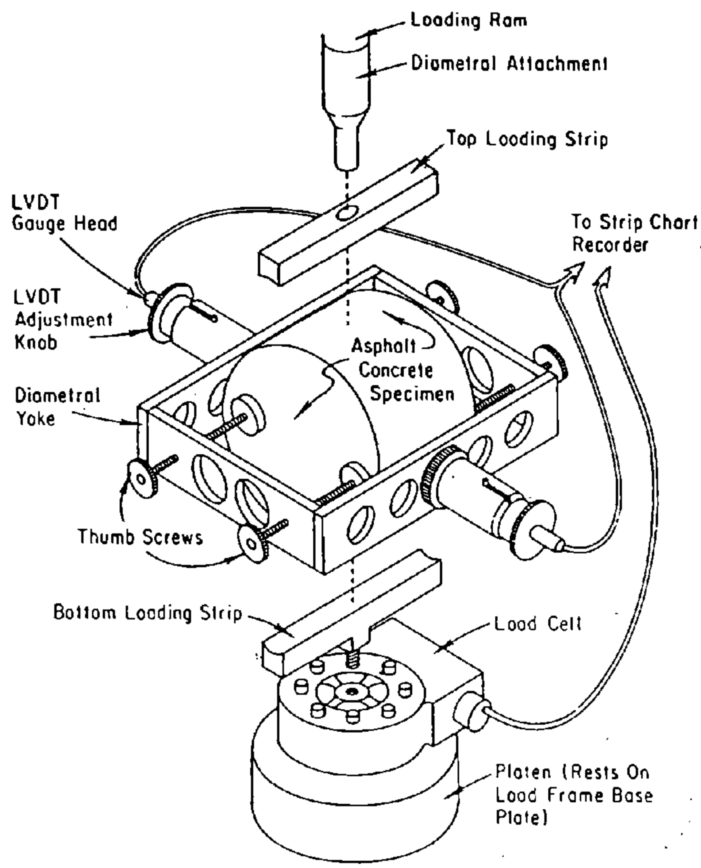


Figure 14. Test Specimen with Diametral Yoke and Loading Ram

specimen and are in their linear range of response. This adjustment is very often time consuming.

With the specimen in the diametral yoke, the seating load applied, and the LVDTs in contact and within their linear range of response, a test may be conducted. The load is applied by a suitable loading system connected to the loading ram. The duration and frequency of the repeated load are controlled with a suitable device. The magnitude of the repeated load and resulting deformation are recorded and/or processed.

The material properties of the test specimen (i.e., resilient modulus and Poisson's ratio) may be calculated with the following equation:

$$MR = (P/Ht)\ (\upsilon + .27) \tag{10}$$

in which

MR = resilient modulus

P = repeated load

H = total recoverable horizontal deflection

t = specimen thickness

υ = Poisson's ratio

The tensile strain at the center of the specimen is given by

$$\epsilon_t = \left[\frac{0.16 + 0.48\ \upsilon}{0.27 + \upsilon}\right] H \tag{11}$$

in which

ϵ_t = tensile strain at the center of the specimen (microstrain, μ)

If vertical deformations are also recorded during the test then Poisson's ratio may be calculated from the following equation:

$$\upsilon = \frac{-3.50\ -0.27\ (V/H)}{0.063 + (V/H)} \tag{12}$$

where

V = total recoverable vertical deflection

The American Society of Testing and Materials has published a standard method of "Indirect Tension Test for Resilient Modulus of Bituminous Mixtures" (ASTM D 4123-82). The procedure described in the standard method identifies a range of temperatures, loads, and loading frequencies. Specifically, the recommended series consists of tests conducted at 41, 77, and 104°F (5, 25, and 40°C) at one or more loading frequencies, typically, 0.33, 0.5, and 1.0 Hz for each temperature. This recommended series will result in nine test values for one specimen which can be used to evaluate the overall resilient behavior of a bituminous mixture. Both laboratory molded specimens (prepared in accordance with accepted procedures such as ASTM D1561, D1559, D3496, D3187) and core specimens (with relatively smooth, parallel surfaces) may be used. Specimens will normally be either a nominal 4 or 6 in. in diameter. A metal loading strip with concave surfaces and a radius of curvature equal o the nominal radius of the test specimen is required. The load strips must be either 0.5 or 0.75 in. wide for the diameters identified, respectively. The specimens should have a thickness of at least 2 in. and a diameter of 4 in. for aggregate up to 1 in. maximum size, and a thickness of at least 3 in. and a minimum diameter of 6 in. for aggregate up to 1.5 in. size.

The test proceeds with a specimen placed in the temperature control cabinet set at a specified test temperature. If the temperature of the specimen is monitored (for example, with a thermistor attached to the specimen), then the test may proceed when the specimen in noted to be within ±2°F (±1°C) of the specified temperature. If the specimen is not monitored, then the specimen should remain in the cabinet at the specified temperature for at least 24 hours prior to testing. When the specimen is at the prescribed temperature it is next preconditioned by applying a repeated haversine (or other simple wave form) load for a period of time sufficient to obtain a uniform horizontal deformation response. Depending upon the load frequency and temperature, a minimum of 50-200 load repetitions may be required. The recommended load duration is from 0.1 to 0.4 sec., with 0.1 sec. considered to be more representative of transient pavement loading. The recommended load range is that required to induce 10-50% of the tensile strength. If tensile strength data are not available, then load ranges from 4-200 lbs/in. of specimen thickness may be used (loads as low as 10 lbs have been used by some researchers). It is noted that if the cumulative vertical deformation exceeds .001 in. during the test, the applied load or the test temperature, or both, should be reduced.

Following the conduct of a test at a specific load (magnitude, frequency, duration) and temperature, the specimen is rotated approximately 90° and the test is repeated. The average recoverable horizontal and vertical deformations of the specimen are obtained over at least three load cycles after the repeated resilient deformation has stabilized. To reduce permanent damage to the specimens, it is suggested that testing begin at the lowest temperatures, shortest load duration, and smallest load. Subsequent tests on the specimen should be for conditions that produce progressively lower moduli.

It may be noted from the description of the "standard" test procedure that resilient moduli may be substantially different for different test conditions. For example, a modulus obtained at a low temperature, and under high frequency and low load duration, may be substantially greater than a modulus obtained at high temperatures under low load frequency and extended load duration. Obviously, this suggests the need of identifying the specific test conditions (e.g., specimen temperature, load magnitude, load frequency, load duration, load wave form, and tensile strain) to insure that equivalent moduli are being compared.

Equations (10), (11), and (12) are supported by the work of Hadley et al. (1970). He developed equations to evaluate the material properties (i.e., elastic modulus, E, and Poisson's ratio, υ) of diametrically loaded cylindrical specimens. These equations have subsequently been applied to the elastic response of specimens subjected to repeated loads. The following assumptions are made in the development of the equations:

(1) The material is elastic, thus Hooke's Law is valid.

(2) The material is homogeneous and isotropic, allowing the use of a single value for the modulus and Poisson's ratio.

(3) Plane-stress conditions exist and, therefore, the problem can be modeled as two-dimensional.

(4) The x- and y-axes are principal planes. This assumption follows from the stress analysis, in which $\tau_{r\theta} = 0$ along these axes.

(5) A uniform strip load is applied directly to the specimen.

The stress analysis of a perfectly elastic, homogeneous, isotropic, and weightless, circular element with a diametrically applied compressive strip was performed by Hondros (1959) using a Fourier series. The definition sketch for the theoretical development is shown in Figure 15. Figure 16 illustrates the unit stress distributions that result from Hondros' analysis for a 4-in. diameter disk with a 0.5-in. loading strip width.

Any of the four stress distributions ($\sigma_{\theta x}$, σ_{rx}, $\sigma_{\theta y}$, σ_{ry}) can be written

$$\sigma = \pm(2P/\pi at)[f(r,R,\alpha)] \qquad (13)$$

If the width of the loading strip is fixed (a = 0.5-in. for this case), Equation (13) can be rewritten

$$\sigma = (P/t)\{2/\pi a[f(r,R,\alpha)]\} = (P/t)\sigma^* \qquad (14)$$

in which

σ^* = unit stress

For any differential element along the x-axis, Hooke's Law is expressed as

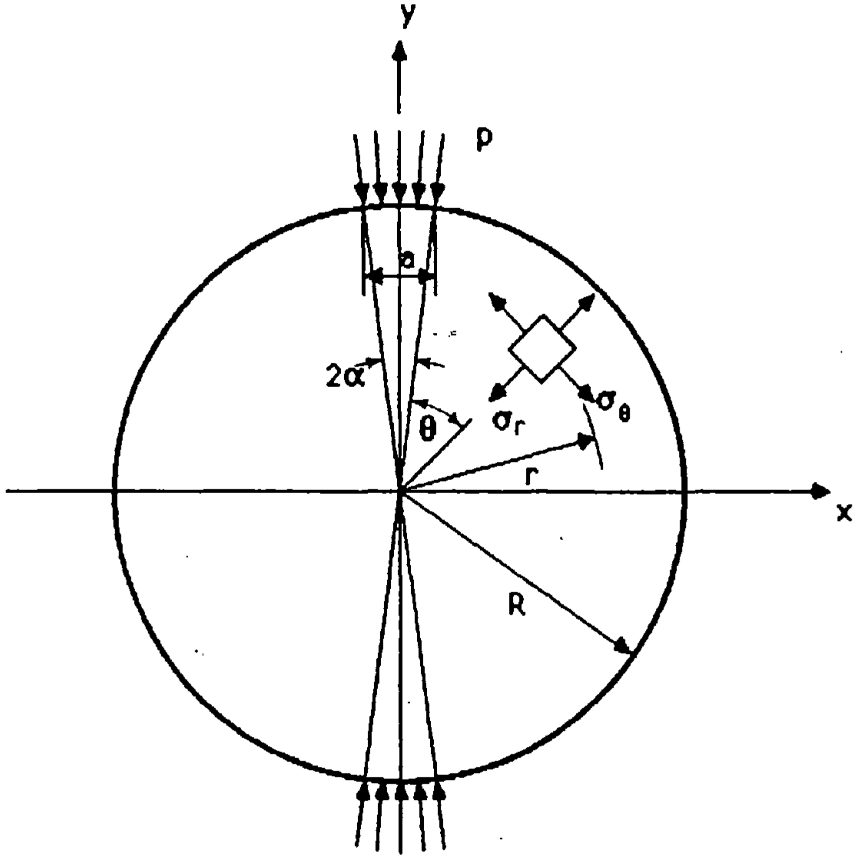


Figure 15. Definition Sketch of a Diametrally Loaded Circular Element

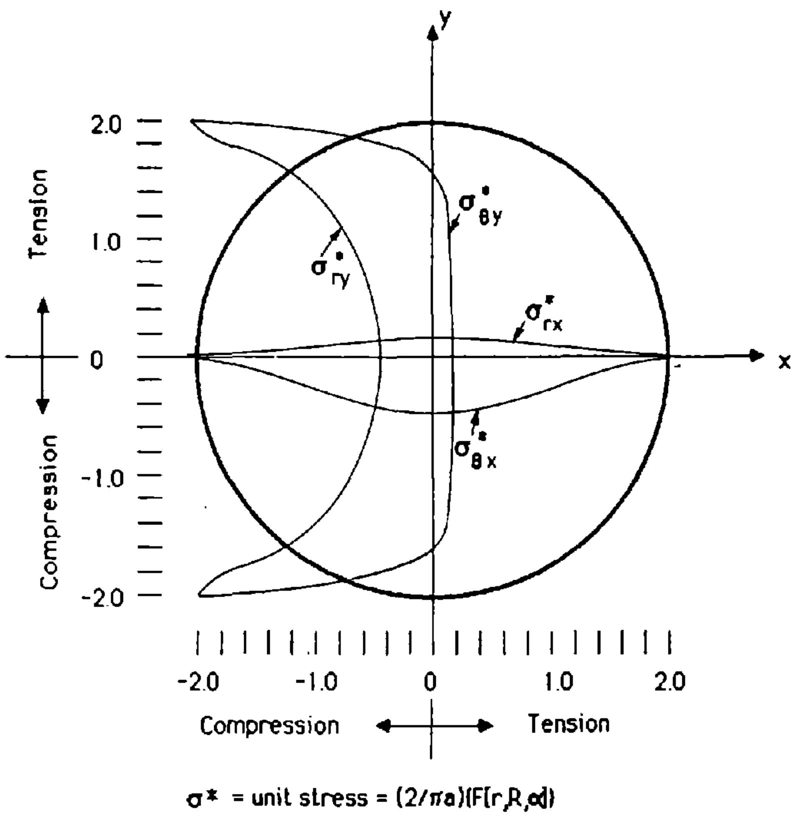


Figure 16. Unit Stress Distribution Along the Vertical and Horizontal Axes for a Diametrically Loaded 4-Inch Diameter Circular Element with a 0.5-Inch Strip Load (after Hondros, 1959)

$$\epsilon_x = 1/E(\sigma_{rx} - \upsilon\sigma_{\theta x}) \tag{15}$$

In terms of the total horizontal deflection,

$$H = \int_D \epsilon_x = \int_D 1/E(\sigma_{rx} - \upsilon\sigma_{\theta x})$$

from which

$$H = 1/E \left[\int_D \sigma_{rx} - \upsilon \int_D \sigma_{\theta x}\right] \tag{16}$$

in which

$\int_D$ = integral over the diameter

Next, the previously defined expression for unit stress is substituted into Equation (16). Solving for E,

$$E = P/Ht \left[\int_D \sigma^*_{rx} - \upsilon \int_D \sigma^*_{\theta x}\right] \tag{17}$$

Performing the same operations along the y-axis results in

$$E = P/Vt \left[\int_D \sigma^*_{ry} - \upsilon \int_D \sigma^*_{\theta y}\right] \tag{18}$$

Equations (17) and (18) can now be equated to solve for υ

$$\upsilon = \frac{\int_D \sigma^*_{ry} - \frac{V}{H}\int_D \sigma^*_{rx}}{\int_D \sigma^*_{\theta y} - \frac{V}{H}\int_D \sigma^*_{\theta x}} \tag{19}$$

An equation for tensile strain can now be obtained by first expressing Hooke's Law for the deflection over a finite length at the center of the specimen,

$$H_1 = \int_1 \epsilon_x = \int_1 1/E(\sigma_{rx} - \upsilon\sigma_{\theta x}) \tag{20}$$

in which

$\int_1$ = integral over a finite length at the center

By the definition of strain,

$$\epsilon_{x1} = H_1/l = 1/El \left[\int_l \sigma_{rx} - \upsilon \int_l \sigma_{\theta x} \right] \tag{21}$$

Expressing Equation (21) in terms of unit stress and solving for E,

$$E = (P/tl\epsilon_{x1}) \left[\int_l \sigma^*_{rx} - \upsilon \int_l \sigma^*_{\theta x} \right] \tag{22}$$

The modulus, E, has now been expressed in terms of the total horizontal deflection and in terms of tensile strain. Equating these two expressions results in

$$\epsilon_{x1} = \frac{H}{l} \left[\frac{\int_l \sigma^*_{rx} - \upsilon \int_l \sigma^*_{\theta x}}{\int_D \sigma^*_{rx} - \upsilon \int_D \sigma^*_{\theta x}} \right] \tag{23}$$

The Mean Value Theorem can now be applied to the expressions $\int_l \sigma^*_{rx}/l$ and $\int_l \sigma^*_{\theta x}/l$ to arrive at

$$\epsilon_{x1} = H \left[\frac{\sigma^*_{rx}\big|_{r=0} - \upsilon\, \sigma^*_{\theta x}\big|_{r=0}}{\int_D \sigma^*_{rx} - \upsilon \int_D \sigma^*_{\theta x}} \right] \tag{24}$$

where

$\big|_{r=0}$ indicates "evaluated at r = 0"

The expressions for Poisson's ratio, resilient modulus (MR = E), and tensile strain at the center ($\epsilon_t = \epsilon_{x1}$) can be solved by numerically integrating the unit stress over the diameter of the specimen and solving the unit stress at the origin:

$$MR = (P/Ht)(0.27 + \upsilon) \tag{10}$$

$$\epsilon_t = \left[\frac{0.16 + 0.48\,\upsilon}{0.27 + \upsilon} \right] H \tag{11}$$

$$\upsilon = \frac{-3.59 - 0.27\,(V/H)}{0.063 + (V/H)} \tag{12}$$

It should be noted that the calculation of MR and ϵ_t requires the determination of Poisson's ratio. The determination of Poisson's ratio requires measurements of both horizontal and vertical deflections. From a practical standpoint, it is difficult to measure the vertical deflection of a test specimen during the performance of a resilient modulus test. However, if a value of Poisson's ratio is selected as input to Equations (10) and (11), the vertical deflection is not needed.

A typical value of Poisson's ratio for asphaltic concrete is 0.35 (Yoder and Witczak, 1975). Based on this assumption, Equations (10) and (11) may be rewritten as:

$$MR = 0.62 \; (P/Ht) \tag{25}$$

$$\epsilon_t = 0.52 \; H \times 10^6 \tag{26}$$

The values of MR and ϵ_t are obtained using the following procedure:

(1) the values of repeated load and horizontal deflection are recorded from tests on two mutually perpendicular diametral axes; and

(2) the average value of horizontal deflection (the load usually remains constant) and the value of the repeated load are input to Equations (25) and (26).

A typical load-time pulse and deformation-time relationship for an asphalt concrete specimen is shown in Figures 17 and 18. The results shown identify two possible measurements for vertical and horizontal measurements. In one case, the instantaneous resilient horizontal and vertical measurements, H_I and V_I, are measured. Substitution of these values into Equations (10), (11), and (12) result in a determination of instantaneous resilient modulus, tensile strain, and Poisson's ratio. In the second case, total resilient horizontal and vertical deformations are measured which result in a determination of total resilient modulus, tensile strain, and Poisson's ratio. Obviously, care must be exercised when reporting values of resilient properties to distinguish between instantaneous versus total response.

The results of a theoretical investigation of the influence of test specimen boundary conditions on indirect tension test resilient moduli are presented in Appendix A. A summary of this work is as follows:

(1) The diametral resilient modulus test is adequately represented by elastic theory based on an assumption of plane stress response of the specimen.

(2) For a resilient modulus test performed under typical loading conditions (i.e., a steel/asphalt concrete interface), the estimate of resilient modulus is less accurate when vertical deflections are employed to first estimate Poisson's ratio. However, the estimate using vertical deflections is more accurate if a low

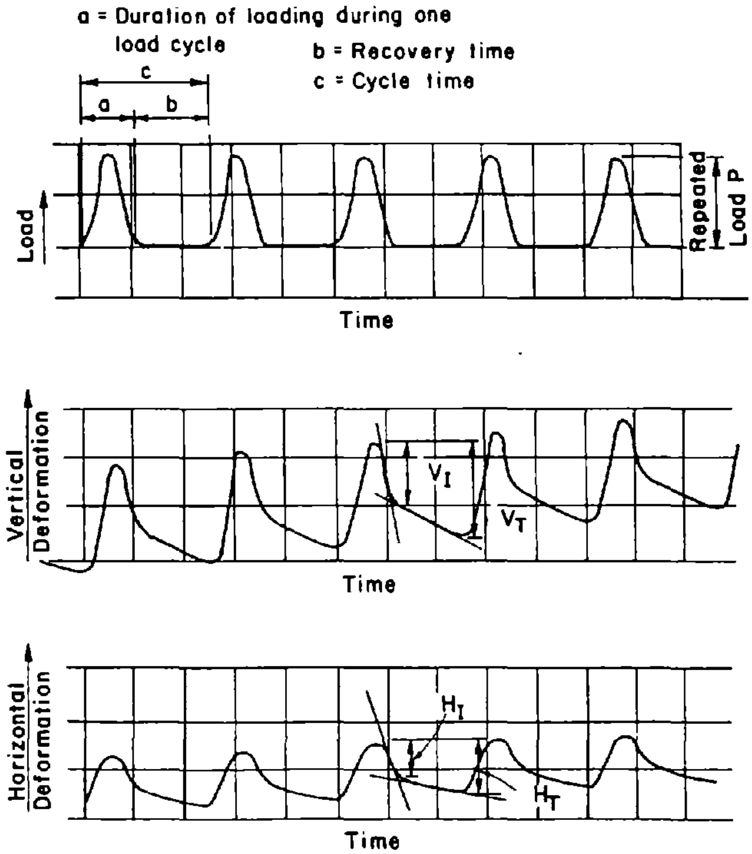


Figure 17. Load Pulse and Associated Deformation Relationships for the Repeated Load Indirect Tension Test (after Kennedy and Anagnos, 1983)

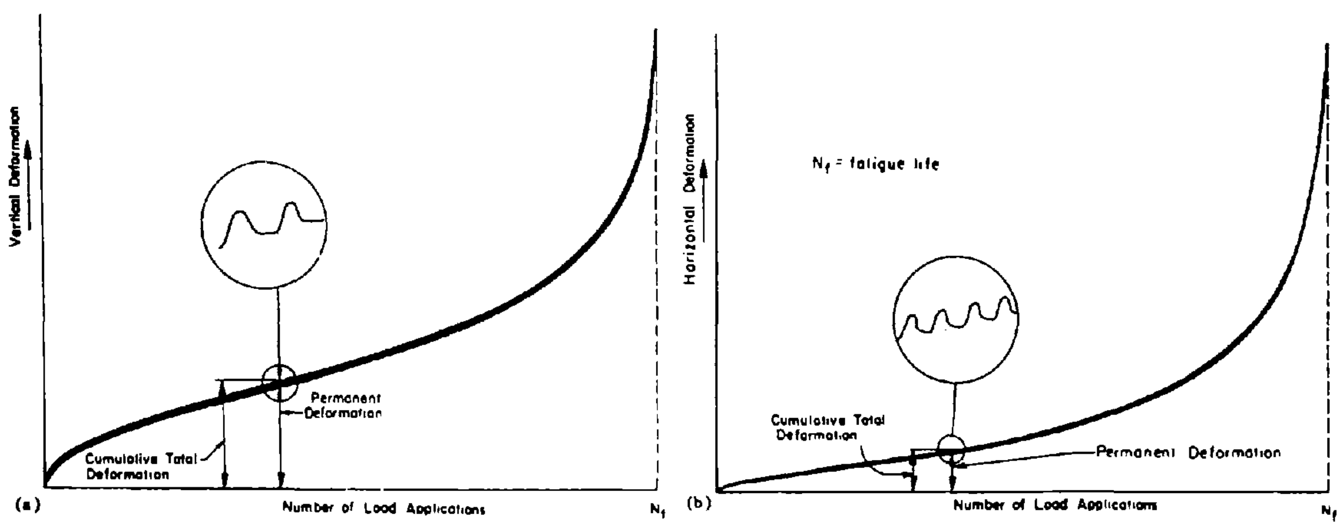


Figure 18. Relationships Between Number of Load Applications and Vertical and Horizontal Deformation for Repeated Load Indirect Tension Test (after Kennedy and Anagnos, 1983)

modulus material is placed between the steel platen and asphalt concrete.

(3) Placing a low modulus material between the steel load platen and the test specimen significantly reduces the induced shear stresses, and the normal stress distribution is more closely modeled by the theoretical stress distribution. Therefore, only horizontal deflections should be used in the determination of the resilient modulus if the diametral test is performed with a steel loading platen/asphalt concrete interface. Vertical deflections will improve the estimate of the resilient modulus if a low modulus material is placed between the steel and the asphalt concrete at this interface.

(4) Although the assumed value of Poisson's ratio directly influences the magnitude of the modulus, it has little effect on the accuracy of the estimation of the resilient modulus.

Summary and Conclusions

The work of a number of researchers and practitioners has been presented to provide an appreciation of the fundamentals of resilient modulus testing. Particular emphasis has been given to the repeated load triaxial compression and diametral indirect tension tests. These tests are based on sound theoretical principles, and if they are properly conducted on specimens that represent in situ conditions, they will provide values of resilient moduli that support the mechanistic approach to pavement design.

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APPENDIX A

Influence of Test Specimen Boundary Conditions on Indirect Tension Test Resilient Moduli

by

J. Heinicke and T.S. Vinson

The finite element structural analysis program ANSYS PC/LINEAR (Gorman, 1986) was used to investigate the influence of an assumed plane stress condition and an assumed uniform strip load applied directly to the specimen on resilient moduli determined in an indirect tension test. The effect of Poisson's ratio on the accuracy of the determination of the resilient modulus was also studied.

Under the assumption of a plane stress condition, the out-of-plane stresses are set equal to zero. However, out-of-plane stresses are present under actual test conditions for the following reasons: (1) the test specimen has a finite thickness which necessarily creates out-of-plane resistance to deflection, and (2) the presence of the thumbscrews used to attach the diametral yoke (see Figure 14) provide out-of-plane resistance at their respective locations.

The assumption that a uniform strip load is applied directly to the specimen ignores the surface traction forces and related shear stresses. These forces result from the material incompatibility at the boundary of the steel platen (high modulus) and the test specimen (low modulus). Further, the load distribution is slightly nonuniform owing to the method of application, i.e., a point load is applied to the steel platen at the center of the test specimen.

The finite element models considered in this study include:

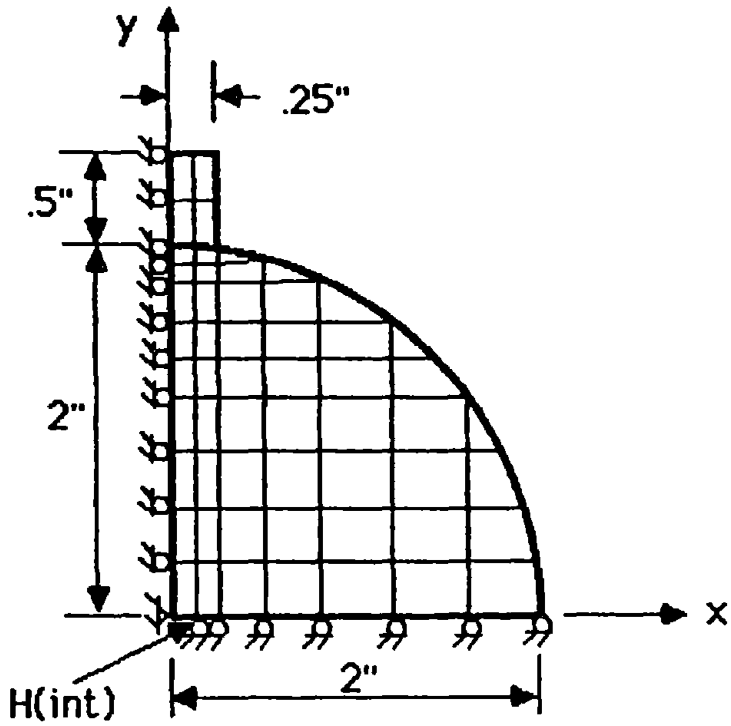
Case 1: plane stress, with a uniformly distributed load applied directly to the circular model;

Case 2: plane stress, with a uniformly distributed load applied to the circular model through a steel loading platen;

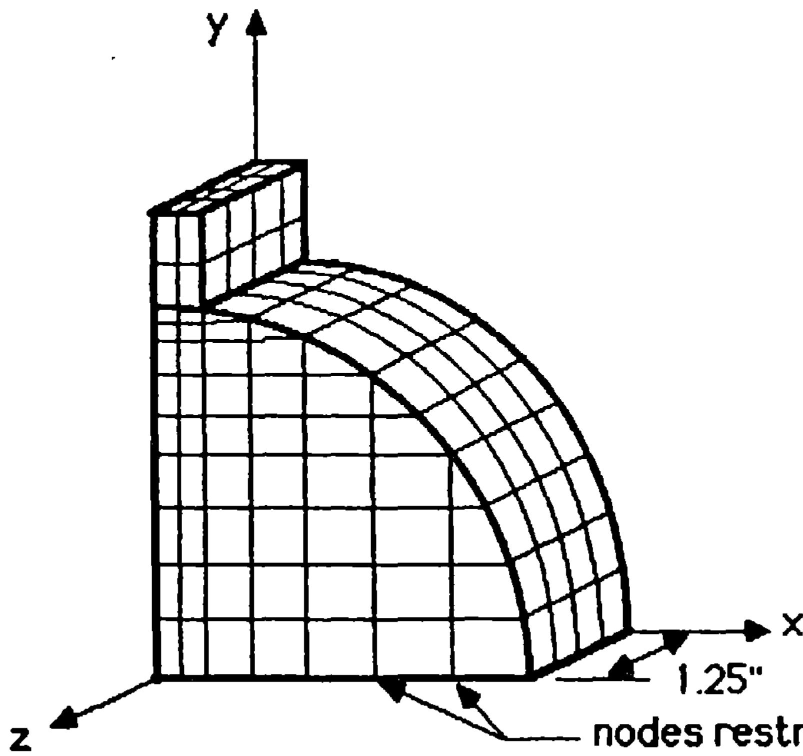
Case 3: three-dimensional, with a point load applied to the cylindrical model through a steel loading platen; and

Case 4: three-dimensional, with a point load applied to the cylindrical model through a teflon coated steel loading platen.

The two-dimensional mesh utilized for Case 2 is shown in Figure A1a. The same mesh is used for Case 1; however, the steel loading platen is absent from the model. Only one-quarter of the circular



(a) Two-dimensional Model.



(b) Three-dimensional Model.

Figure A1. Meshes Utilized in Finite Element Analyses model is needed owing to symmetry. The three-dimensional models can be visualized by expanding the two-dimensional mesh in the z-direction, as shown in Figure A1b. Again, symmetry may be employed, resulting in a model that represents one-eighth of the test specimen. The actual boundary conditions are simulated in the three-dimensional model by: (1) applying a point load through the steel loading platen to model the loading ram, and (2) restricting the outward (z-direction) deflection of two nodes lying on the x-axis at the position where the thumb screws confine the specimen (see Figure A1b). No slippage is permitted at the loading interface for Cases, 2, 3, and 4.

The resilient modulus test procedure involves first applying a small seating load (5% of the repeated load) to maintain the alignment of the steel platen and the test specimen. The specimen is then subjected to the repeated load. This loading condition (i.e., seating load plus repeated load) could not be modeled using the ANSYS PC/LINEAR program. Therefore, initial seating load has been ignored, and a static load was used to represent the repeated load in the analysis. The use of such loading is reasonable since the model is assumed to be linear elastic.

The following test and material conditions (control conditions) were input to the finite element models:

(1) Asphalt concrete properties,

$MR = 300$ ksi

$\upsilon = 0.35$ for Cases 2 and 4

$\upsilon = 0.15, 0.35,$ and 0.45 for Cases 1 and 3

(2) Steel loading strip properties,

E = elastic modulus = 29×10^3 ksi

$\upsilon = 0.30$

(3) Load (backcalculated using Equations (6) and (7) with $\epsilon_t = 100\ \mu$),

$P = 232$ lbs

(4) Specimen thickness,

$t = 2.5$ in.

The output from the finite element analyses includes values of horizontal and vertical deflections at each node. These can be used as input to the theoretically developed equations to compare differences between the finite element solution and the theoretical solution. Further, the stresses output at each node can be compared to Hondros' theoretical stress distribution.

The deflection results of the finite element analyses and the values backcalculated from these results are given in Table A1. The theoretical solutions, which represent the initial (and compatible) conditions, are identified as control. The resilient modulus is calculated by the following methods:

Method 1. The output values of H and V (total horizontal and vertical deflection obtained from the exterior nodes on the x- and y-axes) are input to Equation (12) to solve for υ. This computed value of υ and H and P are used in Equation (10) to solve for MR.

Method 2. The control value of υ and P and the output value of H are input to Equation (10) to solve for MR.

Note that Method 2 represents the typical procedure for the determination of the resilient modulus from laboratory data (i.e., horizontal deflection and repeated load are measured, and υ is assumed). Thus, Method 1 may be viewed as an attempt to improve the estimation of the resilient modulus by obtaining the vertical deflection to calculate the actual value of Poisson's ratio.

The % error and tensile strain given in Table 2 are determined as follows:

(1) % error equals the difference between MR (control) and MR (backcalculated), divided by MR (control); and

(2) tensile strain equals the horizontal deflection of the central element, H(int), divided by the width of the element (0.125 in.).

The Case 1 model best represents the conditions assumed in the theoretical solution. Thus, the accuracy of the mesh employed in this study can be verified by comparing Case 1 to the control condition. As noted in Table A1, the output moduli are within ±3% for either method of calculation and Poisson's ratio is within 5%. There is similar close agreement between the tensile strain levels and the stress distributions. Based on this comparison, the accuracy of the finite element representation is acceptable.

The inclusion of the steel platen (Case 2) does not significantly affect the horizontal deflection. Based on Method 2, the output modulus is 1% less than the control modulus and the tensile strain is within 6%. However, owing to the reduction in vertical deflection, Poisson's ratio exceed 0.50 when Method 1 is employed. This value is theoretically impossible and represents a 50% increase from the control value of 0.35. The resulting value of the resilient modulus is 27% greater than the control value.

The moduli of Cases 3 and 4 that are backcalculated using Method 2 also match the control moduli reasonably well. However, there is a significant difference between the two models when the vertical

Table A1. Results of Finite Element Analyses

(1) Model	Input υ	Deflections (× .001 inch) (2) H(int)	(3) H	(4) V	Method 1 (calculated using vertical and horizontal deflections) (5) υ	(6) MR (ksi)	(7) % Error	Method 2 (calculated using horizontal deflections) (8) MR (ksi)	(9) % Error	(10) Strain (μ)
Control	0.15	——	0.130	-1.142	0.15	300	0	300	0	72
	0.35	——	0.192	-1.182	0.35	300	0	300	0	102
	0.45	——	0.223	-1.201	0.45	300	0	300	0	117
Case 1	0.15	0.0088	0.134	-1.102	0.17	304	1	291	-3	70
	0.35	0.0125	0.196	-1.098	0.38	306	2	294	-2	100
	0.45	0.0143	0.227	-1.096	0.48	307	2	295	-2	114
Case 2	0.35	0.0120	0.195	-0.887	0.53	380	27	296	-1	96
Case 3	0.15	0.0079	0.126	-0.986	0.19	340	13	310	3	63
	0.35	0.0110	0.183	-0.985	0.40	341	14	315	5	88
	0.45	0.0125	0.212	-0.956	0.53	352	17	316	5	100
Case 4	0.35	0.0111	0.182	-1.099	0.33	305	2	316	5	89

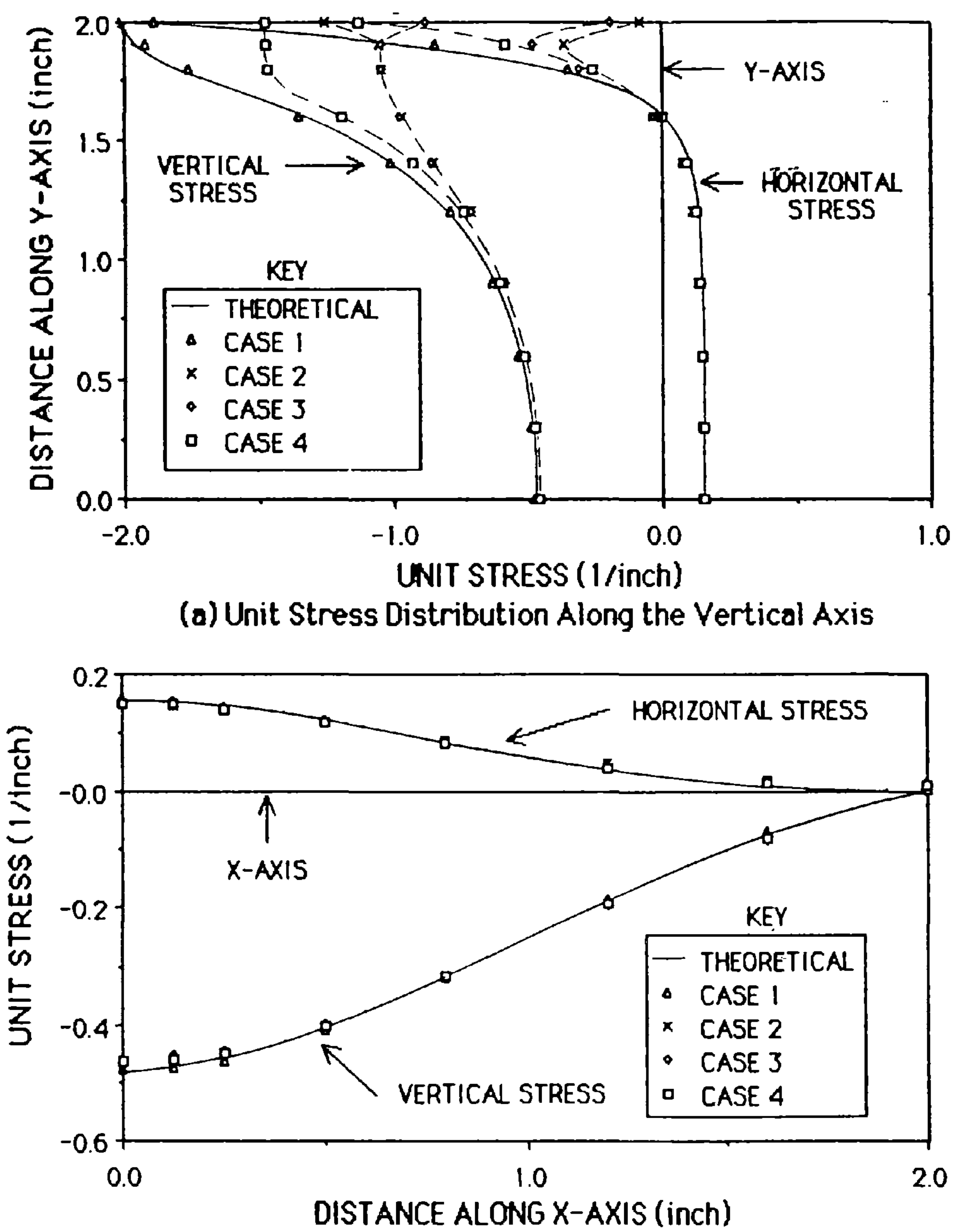
deflections are considered. Using Method 1, the moduli of Case 3 are from 13 to 17% greater than the control moduli. This represents an error that is approximately 10% greater than the error resulting from Method 2. The Case 4 model behaves in a opposite manner; i.e., the modulus is improved by 3% when the vertical deflections are taken into account. This implies that if measurements of vertical deflection are obtained in the laboratory, the estimate of modulus is not improved unless a low modulus material is positioned between the steel loading platen and the test specimen.

Figure A2 illustrates Hondros' theoretical unit stress distribution with the nodal unit stress values for Cases 1 through 4 (with $\upsilon = 0.35$) superimposed. The stress values for Cases 3 and 4 represent the weighted average of the nodal stress values along the width (z-direction) of the specimen. The stress distributions along the horizontal axis are practically identical to the theoretical solution for each model and similar agreement may be noted along the inner two-thirds of the vertical axis. However, the stress distributions diverge at the exterior of the specimen (near the load) as follows: (1) for Case 1, the stresses are almost identical to the theoretical stresses, (2) for Case 4, the vertical stresses decrease by 25% and the horizontal stresses decrease by 50%, and (3) for Cases 2 and 3, the vertical stresses decrease by 50% and the horizonal stresses decrease by 90%.

The differences in stress may be attributed to the effect of surface traction forces that result from the material incompatibility at the loading interface. In the theoretical solution, the horizontal and vertical axes are principal planes (i.e, there are no shear stresses along these axes). However, the finite element models confirm that shear stresses exist along these axes. Thus, the stresses output in the finite element analyses are normal stresses rather than principal stresses. The greater reductions of normal stress for Cases 2 and 3 reflect the high shear stresses that are induced at the load interface. The stresses for the Case 4 model are closer to the theoretical stresses owing to the inclusion of a low modulus material (e.g., Teflon, E = 100 ksi) between the high modulus steel and the relatively low modulus asphalt concrete. Comparing Case 4 to Case 3, the shear stresses at the interface are reduce approximately 300%.

As previously noted, the stress distributions for Cases 3 and 4 represent the weighted average of stresses across the width of the model. These stresses change from the middle of the model to the free face as follows: (1) the compressive stresses are higher in the middle of the specimen than at the free face, (2) the tensile stresses are lower in the middle of the specimen than at the free face, (3) the shear stresses are higher in the middle of the specimen than at the free edge, and (4) the out-of-plane compressive stress increases slightly near the location of the thumb screws.

Varying Poisson's ratio from 0.15 to 0.45 has little effect on the accuracy of the estimation of resilient modulus. The backcalculated moduli increase slightly (4% for Case 3) as Poisson's ratio



(a) Unit Stress Distribution Along the Vertical Axis

(b) Unit Stress Distribution Along the Horizontal Axis

Figure A2. Comparison of Unit Stress Distributions increases from 0.15 to 0.45. Also, the stress distributions are nearly identical for each assumed value of Poisson's ratio.

The results of the finite element models indicate that the resilient modulus diametral test is adequately represented by elastic theory based on the assumption of plane stress response of the test specimen. Although the actual boundary conditions create traction forces that result in the propagation of shear stresses through the specimen, the effect is relatively insignificant with respect to the horizontal deflection actually used in the determination of the resilient modulus. However, if vertical measurements are obtained in an effort to estimate Poisson's ratio, a low modulus material must be placed between the steel load platen and the test specimen.

Designation: D 4123 – 82 (1987)

Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures[1]

Standard Method of Test for

Resilient Modulus of Subgrade Soils

AASHTO DESIGNATION: T 274-82

1. SCOPE

1.1 These methods cover procedures for preparing and testing untreated soils for determination of dynamic elastic modulus under conditions that represent a reasonable simulation of the physical conditions and stress states of subgrade materials beneath flexible pavements subjected to moving wheel loads.

1.2 The methods described are applicable to undisturbed samples of natural and compacted subgrade materials and to distributed samples prepared for testing by compaction in the laboratory.

1.3 The values of resilient (dynamic elastic) modulus determined with these procedures can be used in the available linear-elastic and non-linear elastic layered system theories to calculate the physical response of pavement structures.

2. SUMMARY OF THE METHOD

2.1 A repeated axial deviator stress of fixed magnitude, duration, and frequency is applied to an appropriately prepared and conditioned cylindrical test specimen. During and between the dynamic deviator stress applications, the specimen is subjected to a static all-around stress provided by means of a triaxial pressure chamber. The resilient (recoverable) axial strain response of the specimen is measured and used to calculate the dynamic stress-dependent resilient moduli.

3. SIGNIFICANCE AND USE

3.1 The resilient modulus test provides the basic constitutive relationship between stress and deformation of flexible pavement construction materials for use in structural analysis of layered pavement systems.

3.2 The resilient modulus test provides a means of evaluating pavement construction materials, including subgrade soils, under a variety of environmental conditions and stress states that realistically simulate the conditions that exist in pavements subjected to moving wheel loads.

4. BASIC DEFINITIONS

4.1 σ_1 is the total axial stress (major principal stress)

4.2 σ_3 is the total radial stress; that is, the applied confining pressure in the triaxial chamber (minor and intermediate principal stresses)

4.3 $\sigma_d = \sigma_1 - \sigma_3$ is the deviator stress, that is, the repeated axial stress for this procedure

4.4 E_t is the total axial strain due to σ_d

4.5 E_r is the resilient (recovered) axial strain

4.6 $M_r = \sigma_d / E_r$ is the resilient modulus, i.e., the dynamic stress-strain relationship that can be substituted in analytical procedures involving dynamic traffic loading requiring a modulus of elasticity

4.7 Load duration is the time interval the specimen is subjected to a deviator stress

4.8 Cycle duration is the time interval between successive applications of a deviator stress

4.9 $\Upsilon_d = \dfrac{G\Upsilon_w}{1 + (WG/S)}$

where Y_d = weight of dry soil, pounds per cubic foot (kilo-newtons per cubic meter).

G = specific gravity of soil solids, dimensionless
W = water content of soil, (%)
S = degree of saturation, (%)
Υ_w = unit weight of water, pounds per cubic foot (kilo-newtons per cubic meter).

NOTE: both W and S must be expressed as either a decimal or a number, e.g., 20% is either .20 or 20, but is is imperative that there is consistency between the two

5. APPARATUS

5.1 *Triaxial Pressure Chamber*—The pressure chamber is used to contain the test specimen and the confining fluid during the test. A triaxial chamber suitable for use in resilience testing of soils is shown in Figure 1. The chamber is similar to most standard triaxial cells except that it is somewhat larger to facilitate the internally mounted load and deformation measuring equipment, and has additional outlets for the electrical leads from the measuring devices.

5.1.1 Standard triaxial cells with externally mounted load and deformation measuring equipment (Figure 2) may be used for materials whose maximum resilient modulus is less than 15,000 psi (104,000 kPa).

5.1.2 Air is used as the chamber fluid in both configurations. Water or water/alcohol mixture can also be used.

5.2 *Loading Device*—The external loading source may be any device capable of providing varying repeated loads in fixed cycles of load and release. These devices range from simple cam and switch control of static weights or air pistons to closed-loop electro-hydraulic systems. A load duration of 0.1 second and cycle duration of from 1 to 3 seconds is required. A sine, haversine, rectangular, or triangular shaped stress pulse form may be used.

5.3 *Load and Specimen Response Measuring Equipment*

5.3.1 The axial load measuring device is an electronic load cell. Preferably, the load is measured by placing the load cell between the specimen cap and the loading piston as shown in Figure 1. Load cells may also be mounted outside the test chamber provided corrections are made for any dynamic piston friction in the chamber gland.

5.3.2 Test chamber pressures are monitored with conventional pressure gauges, manometers or pressure transducers of suitable sensitivity ranges.

5.3.3 Axial deformation-measuring equipment for use with materials with maximum resilient modulus in excess of 15,000 psi (104,000 kPa) consists of 2 linear variable differential transformers (LVDT's) attached directly to the specimen by a pair of clamps. The clamps and LVDT's are shown in position on a test specimen in Figure 1. Details of the clamps are shown in Figure 3.

5.3.3.1 Axial deformation measurements on materials with maximum resilient modulus less than 15,000 psi (104,000 kPa) may be made with LVDT's clamped to the piston rod outside the test chamber as shown in Figure 2.

5.3.4 It is necessary to maintain suitable signal excitation, conditioning, and recording equipment in addition to the measuring devices for simultaneous recording of axial load and deformations. The LVDT's should be wired so that the average signal from the pair is recorded.

5.3.5 In order to minimize errors in test specimen response measurement and recording, the system is calibrated immediately before and after each test. A device found to be satisfactory for this purpose consists of a high quality load ring supported in an incompressible (steel) jig whose overall dimensions are similar to the test specimen's (Figure 4). To calibrate the system, the device is placed on the base of the triaxial chamber and the load cell and LVDT's are attached. The device is subjected to repeated axial loads of the magnitude and duration used for measuring the resilient response of the test specimen. By holding a card against the face of the load ring dial, the resulting dynamic deflections of the ring can be observed without difficulty. The load ring displacements are compared to the recorded LVDT trace to obtain the deformation calibration. The load ring's own force-displacement relationship is used to establish the magnitude of load represented by the recorded load cell traces.

TRIAXIAL CHAMBER WITH INTERNAL LVDT'S AND LOAD CELL

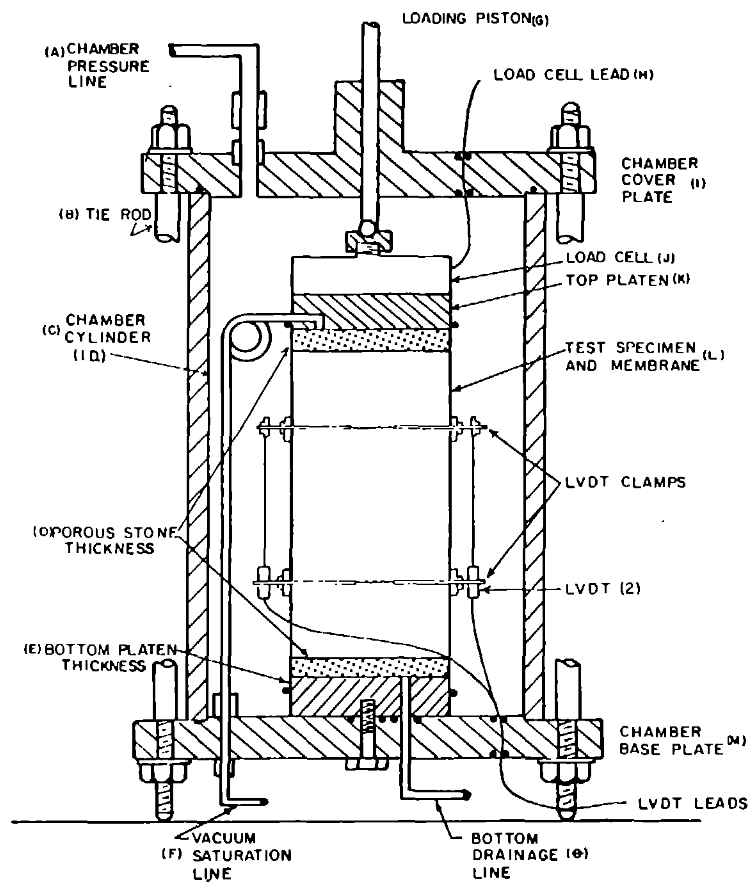


TABLE OF MEASUREMENTS (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H	I	J	K	L	M	N	
METRIC, mm	6.4	12.7	152.4	6.4	38.1	6.4	12.7	Note 1	19.1	Note 1	38.1	Note 2	25.4	6.4	
ENGLISH in.	0.25	0.50	6.00	0.25	1.50	0.25	0.50		0.75		1.50		1.0	0.25	

NOTE
1 Dimensions varies with manufacturer
2 Dimensions varies with specimen size

Figure 1

TRIAXIAL CHAMBER WITH EXTERNAL LVDT'S AND LOAD CELL

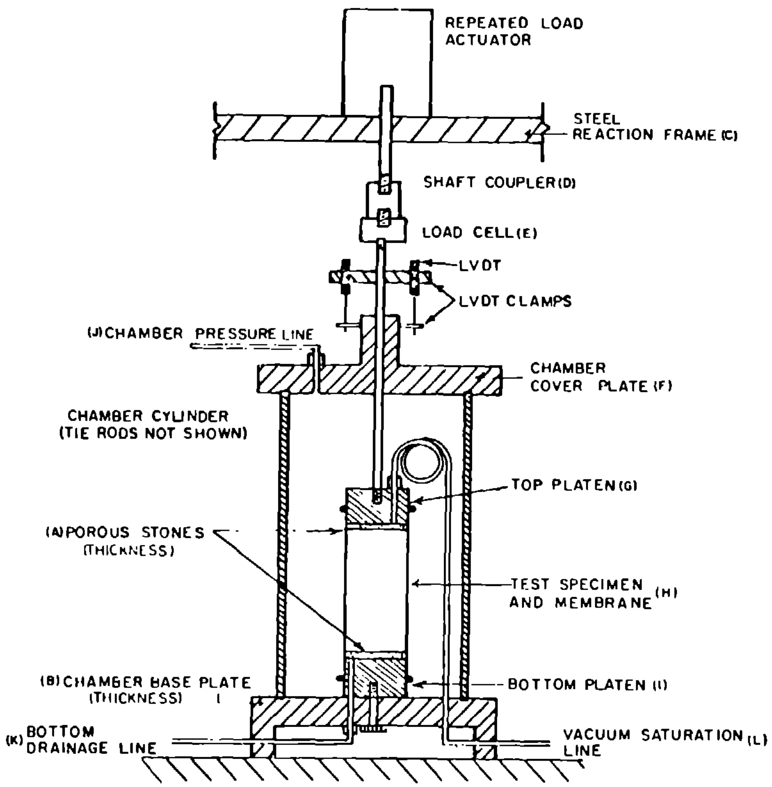


TABLE OF MEASUREMENTS (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H	I	J	K	L
METRIC, mm.	6.4	254	12.7	25 4	Note 1	19 1	38 1	Note 2	38.1	6 4	6.4	6 4
ENGLISH in	0 25	100	0 5	1 00		0.75	1.50		1.50	0 25	025	0 25

1 Dimension varies with manufacturer
2 Dimension varies with specimen size

Figure 2

5.4 *Specimen Preparation Equipment*—A variety of test specimen preparation equipment is required to prepare undisturbed samples for testing and to obtain compacted specimens that are representative of field conditions. Use of different materials and different methods of compacting in the field requires the use of varying compaction techniques in the laboratory. Typical equipment required is listed as follows:

5.4.1 Equipment for trimming test specimens from undisturbed samples as described in AASHTO T 234, Strength Parameters of Soils by Triaxial Compression.

5.4.2 Equipment for impact compaction as described in AASHTO T 99, Moisture-density Relations of Soils Using a 5.5-lb. (2.5 kg) Rammer and a 12-in. (305 mm) Drop or AASHTO T 180, Moisture-Density Relations of Soils Using a 10-lb. (4.54 kg) Rammer and an 18-in. (457 mm) Drop.

5.4.3 Apparatus for kneading compaction as described in AASHTO T 190, Resistance R-Value and Expansion Pressure of Compacted Soils or other apparatus which utilize kneading methods of compaction.

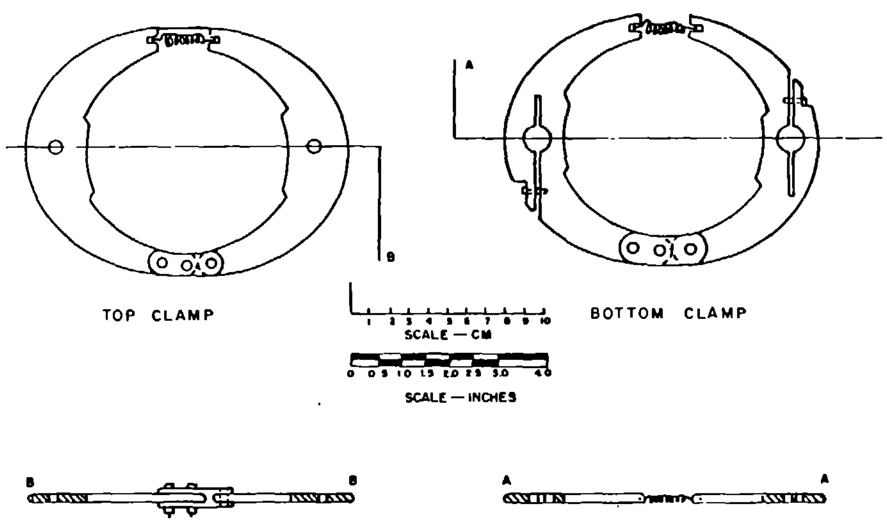
5.4.4 Apparatus for statically compacting a known weight of moist soil to a predetermined length and diameter fixed by the dimensions of a mold. A typical mold assembly for the preparation of 2.8-in. (71 mm) diameter by 6-in (152 mm) high specimen for 3-layer static compaction is shown in Figure 5.

5.4.5 Split mold and hand-held air-operated vibratory compactor as shown in Figure 6.

5.4.6 Static loading machine with an adequate capacity for compacting different materials.

5.5 *Miscellaneous Apparatus*—This includes calipers, micrometer gauge, steel rule (calibrated to 0.02 in. (0.5 mm)) rubber membranes from 0.01 to 0.025 in. (0.254 to 0.635 mm) in thickness, rubber O-rings, vacuum source with bubble chamber and regulator, membrane expander, porous stones, scales, moisture content cans, and data sheets as required.

LVDT CLAMP DETAIL



NOTE. Dimensions vary with manufacturer and specimen size

Figure 3

PROVING-RING CALIBRATION DEVICE

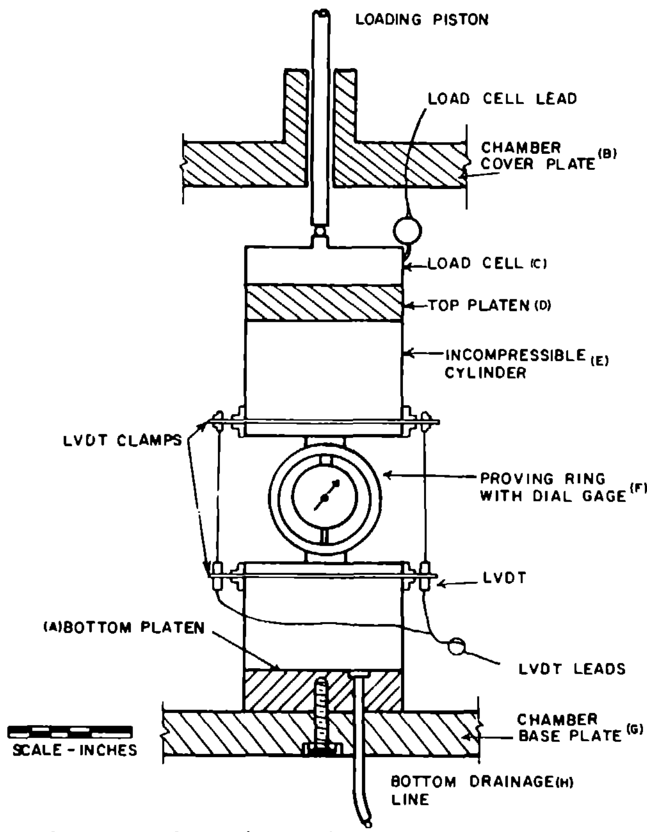
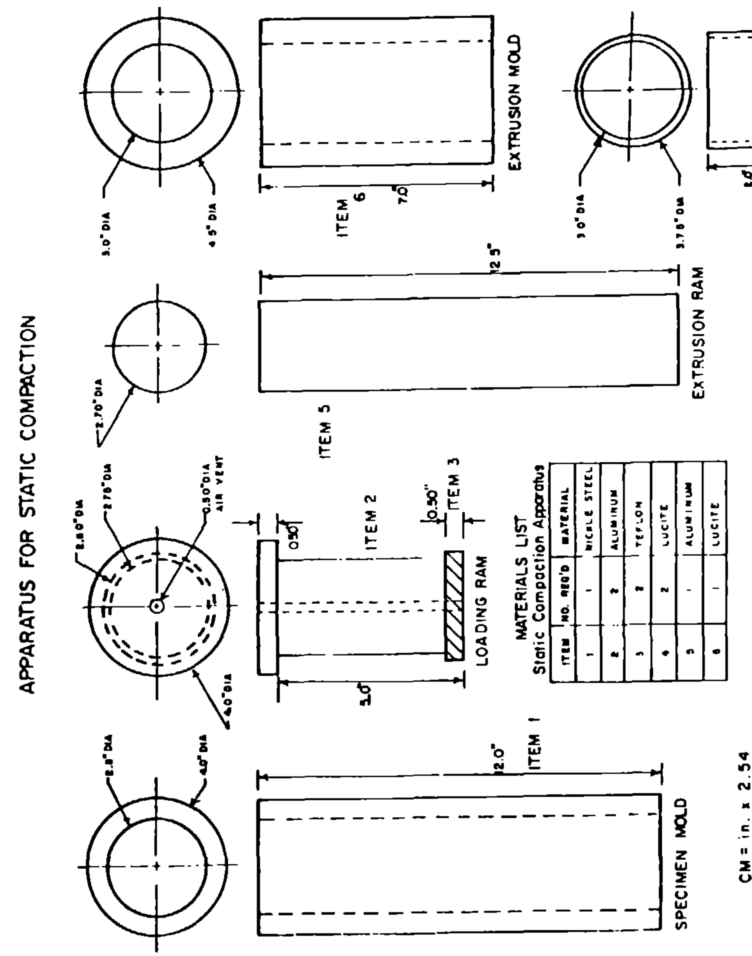


TABLE OF MEASUREMENTS (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H
METRIC mm	38.1	19.1	Note 1	38.1	Note 1	Note 1	25.4	6.4
ENGLISH in	1.50	0.75		1.50			1.0	0.25

1 Dimension varies with specimen size & equipment manufacturer

Figure 4



MATERIALS LIST
Static Compaction Apparatus

ITEM	NO. REQ'D	MATERIAL
1	1	NICKLE STEEL
2	2	ALUMINUM
3	2	TEFLON
4	2	LUCITE
5	1	ALUMINUM
6	1	LUCITE

Figure 5

6. PREPARATION OF TEST SPECIMENS

6.1 *Specimen Size*—Specimen length should be not less than two times the diameter. Minimum specimen diameter is the larger of 2.8-in. (71 mm) or six times the largest particle size. Four-inch (102 mm) diameter, 8-in. (203 mm) high specimens can be accomodated in the triaxial cell shown in Figure 1, and this is the minimum size specimen required when the ring clamp LVDT holders shown in Figure 3 are used.

6.2 *Undisturbed Specimens*—Undisturbed specimens are trimmed and prepared as described in AASHTO T 234, Strength Parameters of Soils by Triaxial Compression.

6.3 *Preparation of Soil for Laboratory Compacted Specimens*—The following procedure is used to prepare soil samples for laboratory compaction.

6.3.1 If the soil sample is damp when received from the field, dry it until it becomes friable under a trowel. Drying may be in air or by use of drying apparatus such that the temperature does not exceed 60C (140F). Then thoroughly break up the aggregations in such a manner as to avoid reducing the natural size of individual particles.

6.3.2 Sieve an adequate quantity of the representative pulverized soil over the ¾-in. (19.0 mm) sieve. Discard the coarse material, if any, retained on the ¾-in. (19.0 mm) sieve.

6.3.3 Determine the air-dry moisture content W_1 of the soil. The moisture sample shall weigh not less than 200 g for soils with a maximum particle size smaller than 0.187-in. (4.75 mm) and not less than 500 g for soils with maximum particle size greater than 0.187-in. (4 75 mm)

6.3.4 Determine the volume V of the compacted specimen to be prepared. For other than static compaction methods, the height of the compacted specimen must be slightly greater than that required for resilience testing to allow for trimming of the specimen ends. An excess of 0.5-in (13 mm) is generally adequate for this purpose.

6.3.5 Determine the weight of oven-dry soil solids W_s and water W_c required to obtain the desired dry density γ_d and water content w_c as follows:

$$W_s \text{ (pounds)} = \gamma_d \text{ (pounds per cubic foot)} \times V \text{ (cubic feet)}$$

$$W_s \text{ (grams)} = W_s \text{ (pounds)} \times 454$$

$$W_c \text{ (pounds)} = W_s \text{ (pounds)} \times w_c \left[\frac{\text{Percent}}{100}\right]$$

$$W_c \text{ (grams)} = W_c \text{ (pounds)} \times 454$$

6.3.6 Determine the weight of air-dried soil W_{ad} required to obtain W_s. An additional amount W_{as} of at least 500 grams should be allowed to provide material for the determination of water content at the time of compaction.

$$W_{ad} \text{ (grams} = (W_s + W_{as}) \times (1 + \frac{w_1}{100})$$

6.3.7 Determine the weight of water W_{aw} required to increase the weight from the existing W_1 to the weight of water W_c corresponding to the desired compaction water content w_c.

$$W_1 \text{ (grams)} = (W_s + W_{as}) \times (\frac{w_1}{100})$$

$$W_2 \text{ (grams} = (W_s + W_{as}) \times (\frac{w_c}{100})$$

$$W_{aw} \text{ (grams)} = W^2 - W^1$$

6.3.8 Determine the wet weight of the soil W_t to be compacted.

$$W_t \text{ (grams)} = W_s \times (1 + \frac{w_c}{100})$$

6.3.9 Place the mass of the soil W_{ad} determined in 6.3.6 into a mixing pan.

6.3.10 Add the water W_{aw} to the soil in small amounts and mix thoroughly after each addition.

6.3.11 Place the mixture in a plastic bag. Seal the bag and store it in an atmosphere of at least 75 percent relative humidity for 24 hours. Ensure a complete seal by using 2 or more bags.

6.3.12 After mixing and storage, weigh the wet soil and container to the nearest gram and record this value on the appropriate forms as shown in Figure 7 and 8.

6.4 *Compacting Specimens of Cohesive Soils*—The resilient behavior of compacted cohesive soils containing substantial amounts of clay is dependent on the structure imparted to the soil particles by the compaction process. Cohesive soils containing substantial amounts of clay are defined for this procedure as soils classified A-2-6, A-2-7, A-6 and A-7 using the criteria of AASHTO M 145, The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes.

APPARATUS FOR VIBRATORY COMPACTION OF COHESIONLESS SOILS

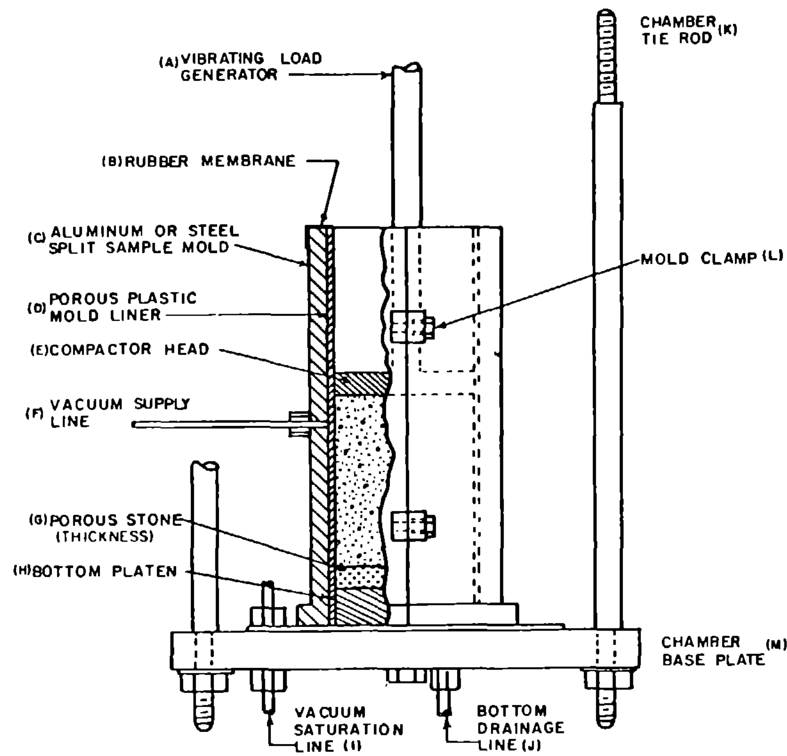


TABLE OF MEASUREMENTS (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H	I	J	K	L	M
METRIC, mm	Note 1	Note 2	Note 2	Note 2	Note 3	6.4	6.4	38.1	6.4	6.4	12.7	Note 1	254
ENGLISH, in.						0.25	0.25	1.50	0.25	0.25	0.50		100

NOTE:

1. Dimension varies with manufacturer
2. Dimension varies with specimen size
3. Diameter should be 0.25 ± 0.02 inch (6.35 ± 0.5 mm) smaller than specimen diameter

Figure 6

6.4.1 *Selection of Compaction Method*—The method of compaction and the compaction (molding) water content w_c of cohesive soils depends on the field condition to be simulated by the laboratory specimen.

6.4.1.1 Specimens representing cohesive subgrades compacted at water contents corresponding to less than 80 percent saturation which remain in the as-constructed condition can be compacted to the field dry density and water content by any standard gyratory, kneading, or static procedures.

6.4.1.2 Test specimens representing a subgrade that was originally compacted at a water content less than that corresponding to the 80 percent saturation value, but which has subsequently experienced an increase in in-service water content are compacted at the in-service water content using the static method described in 6.4.4.

6.4.1.3 Kneading is used for test specimens representing the field compaction and in-service conditions of 6.4.1.2 only if the specimens are compacted at the intital field (as-constructed) water content and then subjected to post-compaction changes in water content. Controlled post-compaction changes in water content are limited in the laboratory to the back pressure saturation techniques described in 6.4.5.

6.4.1.4 Test specimens representing cohesive subgrades compacted in the field at water contents greater than the 80 percent saturation value are compacted in the laboratory using kneading compaction. These test specimens may also be subjected to post-compaction water content increases if the field material to be represented has experienced post-compaction water content increases.

6.4.1.5 Table I summarizes the above discussion of compaction method selection.

6.4.2 *Moisture Density Relationships*—When the range of compaction conditions and the range of in-service conditions are known, select the required laboratory compaction method from the alternatives listed. If the in-service conditions are not well defined, prepare and test specimens over a range of dry densities and water contents. Four steps are followed to select the densities, water contents, and compaction methods used to prepare specimens representative of the range of resilient behavior:

6.4.2.1 Establish the moisture-density relationship for the soil according to the procedure of AASHTO T 99, Moisture Density Relations of Soils Using a 5.5 lb. (2.5 kg) Rammer and a 12-in. (30.5 cm) Drop.

6.4.2.2 Determine the specific gravity of the soil according to the procedure of AASHTO T 100, Specific Gravity of Soils.

6.4.2.2 Use the data obtained in 6.4.2.1 and 6.4.2.2 to determine 100 and 80 percent of saturation at various densities. Place this information on the graph of the moisture-density relationship determined in 6.4.2.1; that is, draw a 100 percent and an 80 percent saturation line.

6.4.2.4 Select the densities, water contents, and compaction methods to be used to prepare test specimens.

6.4.3 *Compaction by Kneading Methods*—Standard molds associated with kneading compaction methods such as AASHTO T 190 or the Harvard miniature method may not be of the correct dimensions for direct use in resilience testing. However, molds of the correct dimensions can be obtained, and the methods referred to above can be adapted to the new mold sizes This generally will require trial-and-error adjustments in the number of compacted layers or the number of tamps per layer (or both) to produce specimens of the required densities. Large size compacted specimens also can be prepared and the correct size test specimen trimmed from the larger compacted specimen. Eight steps are required for the kneading compaction procedure.

6.4.3.1 Establish the number of layers N to be used to compact the soil. Determine the wet weight of soil required per layer, W_L. Layer thickness should not exceed 2 inches.

$$W_L \text{ (grams)} = \frac{W_t}{N}$$

6.4.3.2 Place the mass of soil determined in Step 1 in the mold. Compact according to the procedure established for the mold dimensions and compactor used. Scarify the surface for the remaining layers.

6.4.3.3 Repeat Step 2 for the remaining layers.

6.4.3.4 After the specimen has been completed, determine (verify) the compaction water content, w_c of the remaining soil. The moisture sample shall weigh not less than 200 g for soils with a maximum particle size smaller than 0.187-in. (4.75 mm) and not less than 500 g for soils with maximum particle size greater than 0.187-in. (4.75 mm). Record this value on a form for cohesive soils as shown in Figure 7.

Soil Sample ______
Location ______
Sample No. ______
Specific Gravity ______

Soil Specimen Measurements
Diameter: Top ______ Middle ______ Bottom ______ Average ______
Membrane Thickness ______
Net Diameter ______
Ht Specimen + Cap + Base ______
Ht Cap + Base ______
Initial Length, L_o ______

Soil Specimen Weight
Initial Wt. of Container
+Wet Soil-gms. ______
Final Wt. of Container
+Wet Soil-gms. ______
Wt. Wet Soil Used ______

Soil Specimen Volume
Initial Area A_o $in^2(cm^2)$ ______
Volume A_oL_o $in^3(cm^3)$ ______
Wet Density-pcf(kN/m^3) ______
Compaction Water Content, w_c, % ______
% Saturation ______
Dry Density-pcf(kg/m^3) ______

Date ______
Compaction Method ______
*Vertical Spacing Between LVDT Clamps-in.(cm) ______

Constants
Vertical LVDT ______
Load Cell ______

Water Content After Resilience Testing % ______
Comments ______

Nominal Deviator Stress σ_d psi (kPa)	Chamber Pressure σ_3 psi (kPa)	Load Cell Chart Reading	Deviator Load lb (kN)	Applied σ_d psi (kPa)	Recoverable Deformation LVDT Chart Reading	Recoverable Deformation Inches (mm)	ϵ_r in/in (mm/mm)	$M_r = \sigma_d/\epsilon_r$ psi (kPa)

*Not applicable for externally mounted LVDT's.

Figure 7. Form for Resilient Modulus Tests on Cohesive Soils

Soil Sample ______	Soil Specimen Weight	Date ______
Location ______	Initial Wt. of Container ______	Compaction Method ______
Sample No. ______	+Wet Soil-gms. ______	*Vertical Spacing Between
Specific Gravity ______	Final Wt. of Container ______	LVDT Clamps-in.(cm) ______
	+Wet Soil-gms. ______	
Soil Specimen Measurements	Wt. Wet Soil Used ______	Constants
Top ______		Vertical LVDT ______
Diameter Middle ______	Soil Specimen Volume	Load Cell ______
Bottom ______	Initial Area A_o $in^2(cm^2)$ ______	
Average ______		Water Content After
Membrane Thickness ______	Volume A_oL_o $in^3(cm^3)$ ______	Resilience Testing % ______
Net Diameter ______	Wet Density-pcf(kN/m^3) ______	Comments ______
Ht Specimen + Cap + Base ______	Compaction Water Content, w_c % ______	______
Ht Cap + Base ______	% Saturation ______	______
Initial Length, L_o ______	Dry Density-pcf(kg/m^3) ______	______
Inside Diameter of Mold ______		

Chamber Pressure σ_3 psi (kPa)	Nominal σ_d psi (kPa)	Load Cell Chart Reading	Deviator Load lb (kN)	σ_d psi (kPa)	σ_1 psi (kPa)	Recoverable Deformation LVDT Chart Reading	Recoverable Deformation inches (mm)	ϵ_r in/in mm/mm	$M_r = \sigma_d/\epsilon_r$ psi (kPa)	$\theta = \sigma_1 + 2\sigma_3$ psi (kPa)

*Not applicable for externally mounted LVDT's.

Figure 8. Form for Resilient Modulus Tests on Granular Soils

6.4.3.5 Carefully remove the specimen from the mold. If the compacted specimen is not of the desired dimensions, trim the test specimen in accordance with the procedures described in AASHTO T 234, Strength Parameters of Soils by Triaxial Compression. If the compaction mold has the same dimensions as the desired test specimen, plane end surfaces can be obtained by applying a small static load to the specimen before it is carefully removed from the mold.

6.4.3.6 Weigh the specimen to the nearest gram. Determine the average height and diameter to the nearest 0.02-in. (0.5 mm). Record these values on a form for cohesive soils as shown in Figure 7.

6.4.3.7 Using a vacuum membrane expander, place the thin leak-proof membrane over the specimen. Place O-rings or other pressure seals around the membrane to provide a positive seal to top and bottom solid end platens like those used with the triaxial chamber.

6.4.3.8 Wrap the membrane-enclosed sample in a plastic bag, seal, and place it in an atmosphere of at least 75 percent relative humidity for a period of not less than 24 hours to insure a uniform moisture distribution. If no post-compaction conditioning such as freeze-thaw cycling or back pressure saturation is to be used, the specimen is now ready for transfer to the triaxial chamber for resilience testing.

6.4.4 *Compaction by Static Loading*—In the absence of standard methods for static compaction, the method described in this procedure is used. The process is one of compacting a known weight of wet soil to a volume that is fixed by the dimensions of the mold assembly. A typical mold assembly for the preparation of a specimen with a 2.8-in. (71 mm) diameter and a 6-in. (152 mm) height using 3 layers is shown in Figure 5. Other suitable equipment and number of layers necessary to produce specimens of larger dimensions can be developed. Sixteen steps are required for static compaction.

6.4.4.1 Establish the number of layers N to be used to compact the soil. The thickness of individual layers should be limited to 2 inches. Determine the weight of wet soil per layer.

$$W_L \text{ (grams)} = \frac{W_t}{N}$$

6.4.4.2 Place one of the loading rams into the sample mold.

6.4.4.3 Place the mass of soil W_L determined in Step 1 into the sample mold. Use a spatula to draw the soil away from the edge of the mold and form a slight mound in the center.

6.4.4.4 Insert the second loading ram and place the assembly in the static loading machine. Apply a small load. Adjust the mold so that it rests equidistant from the caps of the load rams. Soil pressure developed by the initial loading will serve to hold the mold in place. By having both loading rams each the zero volume change positions simultaneously, more uniform layer densities are obtained.

6.4.4.5 Slowly increase the load until the loading ram caps rest firmly against the mold. Hold the load at or near the maximum load for not less than one minute. The rate of loading and load duration depend on the amount of soil rebound. The slower the rate of loading and the longer the load is held, the less the rebound.

6.4.4.6 Decrease the load to zero and remove the assembly from the loading machine.

6.4.4.7 Remove the loading ram. Scarify the surface of the compacted layer, put the correct weight of soil W_L for a second layer in place, and adjust the soil as in Step 3. Add a spacer ring and insert the loading ram.

6.4.4.8 Invert the assembly and repeat Step 7.

6.4.4.9 Place the assembly in the loading machine. Load slowly while holding the load at or near maximum when the spacer disk firmly contacts the mold.

6.4.4.10 Repeat Steps 6, 7, 8 and 9 as required.

6.4.4.11 After the specimen has been completed determine (verify) the compaction water content w_c of the remianing soil. The moisture sample shall weigh not less than 200 g for soils with maximum particle size smaller than 0.187-in. (4.75 mm) and not less than 500 g for soils with maximum particle size greater than 0.187-in. (4.75 mm). Record this value on a form for cohesive soils as shown in Figure 7.

6.4.4.12 Place the extruder ram into the sample mold and force the specimen out of the sample mold into the extrusion mold.

6.4.4.13 Use the extrusion mold to carefully slide the compacted specimen onto a glass plate.

6.4.4.14 Determine the weight of the compacted specimen to the nearest gram. Measure the height and diameter to the nearest 0.02-in. (0.5 mm). Record these values on a form for cohesive soils as shown in Figure 7.

TABLE I SELECTION OF COMPACTION METHOD

GYRATORY	KNEADING	STATIC
Subgrades compacted at a water content less than 80% saturation and remain in that condition		
		Subgrades compacted at a water content less than 80% saturation and water content subsequently increases
	Sample compacted at initial field water content & subjected to post construction change in water content	
	Subgrades compacted at a water content greater than 80% saturation	

6.4.4.15 Using a vacuum membrane expander, place the thin leak-proof membrane over the specimen. Place O-rings or other pressure seals around the membrane to provide a positive seal to solid top and bottom end platens similar to those to be used with triaxial chamber.

6.4.4.16 Age the specimen as described in 6.4.3.8. If the no post-compaction conditioning such as post-compaction back pressure saturation or freeze-thaw cycling is to be used, the specimen is now ready for transfer to the triaxial chamber for resilience testing.

6.4.5 *Post-Compaction Back-Pressure Saturation of Undisturbed or Compacted Cohesive Soil Specimens*—If a specimen of undisturbed soil or cohesive soil compacted by the methods of 6.4.3 or 6.4.4 is to be saturated before testing, the following 22 steps are required.

6.4.5.1 Remove the test specimen from solid end platens by first removing the rubber O-rings and then carefully folding or rolling the membrane back from the ends of the specimen a distance of approximately one-quarter inch (6.4 mm).

6.4.5.2 Place a saturated porous stone on top of the pedestal or bottom end platen of the triaxial chamber. Saturate the bottom drainage line of the triaxial chamber and the pore pressure measuring device prior to beginning this process by forcing de-aired water through it. If a removable type bottom platen is used, tighten it firmly to the triaxial chamber to obtain an airtight seal.

6.4.5.3 With the bottom drainage valve closed, place the test specimen on the saturated porous stone, carefully fold down the membrane, and seal the membrane to the pedestal or bottom end platen with an O-ring or other pressure seal.

6.4.5.4 Place the top porous stone and top end platen (with vacuum saturation inlet) on top of specimen, fold up the membrane, and seal it to the top end platen.

6.4.5.5 With the drainage line to the bottom of the specimen closed, connect the vacuum inlet at the top of the specimen to a vacuum source through the medium of a bubble chamber and apply a vacuum of 5 psi (35 kPa). If bubbles are absent, an airtight seal has been obtained for the system. If bubbles are present, check for leakage caused by poor connections, holes in the membrane, or imperfect seals at the end platens.

6.4.5.6 When leakage has been eliminated, disconnect the vacuum supply. If specimen response is to be measured using internal clamp-mounted LVDT's, Steps 7, 8, and 9 are required. If externally mounted LVDT's are to be used, the method continues with Step 10.

6.4.5.7 Open the lower LVDT clamp and carefully clamp it at approximately the lower quarter point of the specimen.

6.4.5.8 Repeat Step 7 for the upper clamp, placing it at the upper quarter point. Ensure that both clamps lie in horizontal planes.

6.4.5.9 Connect the LVDT's to the recording unit and balance the recording bridges. This will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine to the nearest 0.02-in. (0.5 mm) the vertical spacing between the LVDT clamps and record this value on a form for cohesive soils as shown in Figure 7.

6.4.5.10 Set the load cell in place on the sample cap if the internal load cell configuration of Figure 1 is used.

6.4.5.11 Place the chamber cylinder and cover plate. Insert the loading piston and obtain a firm connection with the load cell.

6.4.5.12 Tighten the chamber tie rods firmly.

6.4.5.13 Slide the assembly apparatus into position under the axial loading device. Bring the loading device down and couple it to the triaxial chamber piston.

6.4.5.14 Connect the chamber pressure supply line and apply confining pressure of 5 psi (35 kPa).

6.4.5.15 Connect the bottom specimen drainage line to a reservoir of de-aired distilled water for which a back pressure can be controlled and monitored.

6.4.5.16 Reconnect the specimen top drainage line to the vacuum source through the bubble chamber. Apply a vacuum of 3 psi (21 kPa) to the top of the specimen.

6.4.5.17 Open the bottom drainage valve and allow water to be drawn up slowly through the specimen. When water appears to flow out of the specimen in the top drainage line, disconnect the vacuum source from the specimen.

6.4.5.18 Connect the top drainage line to a second reservoir of de-aired distilled water. Maintain the back pressure in this reservoir 5 psi (35 kPa) lower than the pressure in the reservoir connected to the bottom of the specimen.

6.4.5.19 Raise the chamber pressure and back pressure slowly in increments of 5 psi (35 kPa) to 75 psi (518 kPa) and 70 psi (483 kPa) respectively, being careful to maintain the chamber pressure approximately 5 psi (35 kPa) greater than the back pressure in the bottom drainage reservoir in order to prevent flow between the specimen and the membrane.

6.4.5.20 Continue to flush water through the system by maintaining the 5 psi (35 kPa) difference in back pressure applied to the top and bottom drainage line reservoirs until all air has been eliminated.

6.4.5.21 When all air has been eliminated from the test specimen, an increase in chamber pressure (with valves to the top and bottom back pressure reservoirs closed) will result in an approximately equal increase in pore pressure. When this condition is achieved (it may take several days) reduce the back pressure to zero and the chamber pressure to 5 psi (35 kPa), again being careful to maintain the chamber pressure 5 psi greater than the back pressure.

6.4.5.22 After both back pressures have been reduced to zero, disconnect both the top and bottom specimen drainage lines and open them to atmospheric pressure (outside the triaxial chamber). The specimen is now ready for resilience testing.

6.5 *Compacting Specimens of Granular Soils*—Granular soils that exhibit sufficient cohesion (apparent) to permit handling (removal from the mold, transporting, and sealing in the rubber membrane) can be compacted by the methods described in 6.4.3 and 6.4.4. However, it is generally not necessary to consider soil structure effects. The exceptions are some plastic silts that may also exhibit resilient properties that are dependent on compaction conditions. Granular materials that cannot be handled are compacted as described in 6.5.2.

6.5.1 *Moisture Density Relationships*—When the range of field densities and moisture conditions to be represented by the test specimen is known, laboratory test specimens can be compacted directly to the in-service water content using the methods of 6.4.3, 6.4.4, or 6.5.2. If the service conditions are not well defined, prepare and test specimens over a range of dry densities and water contents. Establish the moisture-density relationship of the soil according to the procedure of AASHTO T 99, Moisture-Density Relations of Soils Using a 5.5-lb. (2.5 kg) Rammer and a 12-in. (30.5 cm) Drop.

6.5.2 *Compacting Granular Soil Using a Split Mold and Vibrator*—Cohesionless granular materials are compacted readily by use of a split mold mounted on the base of the triaxial cell as shown in Figure 6. Compaction forces are generated by a vibrator, such as a small hand-operated air hammer. Twenty-six steps are required to compact the specimen.

6.5.2.1 Tighten the sample base into place on the triaxial cell base. It is essential that an air tight seal be developed.

6.5.2.2 Place the two porous stones plus the sample cap on the sample base. (Two stones are required for saturated specimens, but generally only the lower stone would be used for tests of unsaturated specimens). Determine the height of base, cap, and stones to the nearest 0.02-in. (0.5 mm), and record this value on a form for granular soils as shown in Figure 8.

6.5.2.3 Remove the sample cap and upper porous stone if used. Measure the thickness of the rubber membrane with a micrometer gage. Record this value on a form for granular soils as shown in Figure 8.

6.5.2.4 Place the rubber membrane over the sample base and lower porous stone. Fix the membrane in place with an O-ring seal.

6.5.2.5 Place the split-mold sample former around the sample base and draw the rubber membrane up through the mold. Tighten the split mold firmly into place. Exercise care to avoid pinching the membrane.

6.5.2.6 Stretch the membrane tightly over the rim of the mold. Apply a vacuum to the mold to remove all membrane wrinkles. The use of the porous plastic forming jacket liner as shown in Figure 6 helps to insure that the membrane fits smoothly around the inside perimeter of the mold. The vacuum is maintained throughout the compaction procedure.

6.5.2.7 Use calipers to determine to the nearest 0.02-in. (0.5 mm) the inside diameter of the membrane-lined mold. Determine to the nearest 0.02-in (0.5 mm) the distance from the top of the porous stone to the rim of the mold.

6.5.2.8 Determine the volume V of specimen to be prepared. The diameter of the specimen is the diameter in Step 7 and height is a value less than that determined in Step 7 but at least 2 times the diameter.

6.5.2.9 Determine the weight of material that must be compacted into the volume V determined in Step 8 to obtain the desired density and water content as described in 6.3.4 through 6.3.8.

6.5.2.10 Determine the number of layers N to be used for compaction. Normally, layer depths will be 1 to 1.5-in. (25.4 to 38.1 mm). Determine the weight of wet soil required for each layer W_L as in 6.4.3.1.

6.5.2.11 Place the total required mass of soil W_{ad} into a mixing pan. Add the required amount of water W_{aw} and mix thoroughly.

6.5.2.12 Determine the weight of wet soil plus mixing pan and record on a form for granular soils as shown in Figure 8.

6.5.2.13 Place the amount of wet soil W_L required for 1 layer into the mold. Exercise care to avoid spillage. Use a spatula to draw the material away from the edge of the mold and form a small mound at the center of the mold.

6.5.2.14 Insert the vibrator head and vibrate the soil until the distance from the surface of the compacted layer to the rim of the mold is equal to the distance measured in Step 7 minus the thickness of the lift selected in Step 10. This may require removal and reinsertion of the vibrator head several times until experience is obtained in gauging the required vibration time.

6.5.2.15 Repeat Steps 13 and 14 for each new lift. The measured distance from the surface of the compacted layer to the rim of the mold is successively reduced by the thickness of each new lift from Step 10. The final surface should be a smooth, horizontal plane.

6.5.2.16 When compaction is completed, observe the weight of the mixing plan plus excess soil and record it on a form for granular soils as shown in Figure 8. The weight determined in Step 12 less the weight observed is the weight of wet soil incorporated in the specimens. Determine (verify) the compaction water content w_c of the soil remaining in the pan. The moisture sample shall weigh not less than 200 g for soils with a maximum particle size smaller than 0.187-in. (4.75 mm). Record this value on a form for granular soil as shown in Figure 8.

6.5.2.17 Place the porous stone and top sample cap on the surface of the specimen. Roll the rubber membrane off the rim of the mold and over the sample cap. If the sample cap projects above the rim of the mold, the membrane should be sealed tightly against the cap with the O-ring seal. If it does not, the seal can be applied later.

6.5.2.18 Connect the vacuum-saturation inlet to a vacuum source and apply 5 psi (35 kPa) of vacuum through the medium of a bubble chamber. The vacuum serves to detect leakage and to impact a stress induced rigidity to the material to prevent collapse when the mold is removed.

6.5.2.19 Carefully remove the sample mold. Seal the membrane to the sample cap if this has not been done. Determine to the nearest 0.02-in. (0.5 mm) the height of specimen plus cap and base and the diameter of the specimen plus membrane. Record these values on a form for granular soils as shown in Figure 8.

6.5.2.20 Observe the presence or absence of air bubbles in the bubble chamber. If bubbles are absent, an airtight seal has been obtained. If bubbles are present, check for leakage caused by poor connections, holes in the membrane, or imperfect seals at the cap and base. The existence of an airtight seal ensures that the membrane will remain firmly in contact with the specimen. This is essential for use of the clamp mounted internal LVDT's. Leakage through holes in the membrane can frequently be eliminated by coating the surface of the membrane with liquid rubber latex or by using a second membrane.

6.5.2.21 When leakage has been eliminated, open the lower LVDT clamp and place it carefully over the specimen at approximately the lower quarter point of the specimen.

6.5.2.22 Repeat Step 21 for the upper clamp and place it at the upper quarter point. Ensure that both clamps lie in horizontal planes.

6.5.2.23 Connect the LVDT's to the recording unit and balance the recording bridges. This will require recorder adjustments and adjustment of the LVDT stems. When a recording bridge balance has been obtained, determine to the nearest 0.02-in. (0.5 mm) the vertical spacing between the LVDT clamps and record this value on a form for granular soils as shown in Figure 8.

6.5.2.24 Place the load cell on the specimen end platen, assemble the remainder of the cell, and tighten the tie rods firmly. Slide the assembled apparatus into position under the axial loading device, and couple the actuator and triaxial cell pistons.

6.5.2.25 Connect the chamber pressure supply line and apply a pressure of 5 psi (35 kPa).

6.5.2.26 Remove the vacuum supply from the vacuum saturation inlet and close this line. If the specimen is to be tested at the as-compacted water content, it is now ready for resilience testing. If the specimen is to be subjected to post-compaction back-pressure saturation, the steps listed in 6.5.3 are completed.

6.5.3 *Post-Compaction Back-Pressure Saturation of Granular Soils*—Test specimens of granular soil to be saturated by back pressure flushing are prepared by the method described in 6.5.2. After completing the steps of 6.5.2, the following additional steps are necessary to saturate the soil.

6.5.3.1 Connect the vacuum supply to the vacuum inlet (at the top of the specimen) and connect the bottom drainage line to a source of de-aired distilled water.

6.5.3.2 Apply a vacuum of 2 to 3 psi (14 to 21 kPa), open the bottom water drainage valve, and allow water to be drawn slowly upward through the specimen.

6.5.3.3 Continue to flush water through the system to remove all entrapped air. To evaluate the presence or absence of air the pore water pressure response to a chamber pressure increase is observed as described for cohesive soils in 6.4.5.21.

6.5.3.4 When all air has been eliminated, set the chamber pressure at 10 psi (69 kPa), apply a 5 psi (35 kPa) back pressure to the water supply while closing the vacuum inlet valve. The effective confining pressure (5 psi (35 kPa)) on the specimen is now equal to the chamber pressure (10 psi (69kPa)) minus the back pressure (5 psi (35 kPa)). The saturated specimen is now ready for resilience testing.

7. PROCEDURE

7.1 *Resilience Tests on Cohesive Soils*—The procedures described in this section are used for undisturbed and laboratory compacted specimens of cohesive subgrade soils as defined in 6.4.

7.1.1 *Assembly of Triaxial Chamber*—Resilience testing of specimens previously subjected to the back-pressure saturation procedures of section 6.4.5 begins with Step 7.1.2. Specimens trimmed from undisturbed samples and laboratory compacted specimens which have not been subjected to the post-compaction back-pressure saturation techniques are placed in the triaxial chamber and loading apparatus in the following steps.

7.1.1.1 Place the triaxial chamber base assembly on the platform of the loading machine. If the chamber has a removable bottom platen (sample base) tighten it firmly to obtain an airtight seal.

7.1.1.2 Remove the solid end platens from the previously membrane-enclosed test specimen by first removing the rubber O-rings and then carefully folding or rolling the membrane back from the ends of the specimen a distance of approximately one-quarter inch (6.4 mm).

7.1.1.3 Place a porous stone on the top of the pedestal or bottom end platen of the triaxial chamber.

7.1.1.4 Carefully place the specimen on the porous stone, fold down the membrane, and seal the membrane to the pedestal or end platen with an O-ring or other pressure seal.

7.1.1.5 Place the top platen (sample cap) and load-cell on the specimen, fold up the membrane, and seal it to the top platen.

7.1.1.6 Close the valve on the vacuum saturation line to the top platen (this line is not required for resilience testing of specimens not subjected to post-compaction saturation; closing the valve will prevent loss of air from the chamber during testing).

7.1.1.7 Connect the specimen's bottom drainage line to a vacuum source through the medium of a bubble chamber. Apply a vacuum of 3 psi (21 kPa). If bubbles are present, check for leakage as described in 6.4.5.5.

7.1.1.8 When leakage has been eliminated disconnect the vacuum supply. Install the LVDT's assemble the triaxial cell, and position it under the axial loading device as described in 6.4.5.7 through 6.4.5.14.

7.1.2 *Conduct of Resilience Test*—Twelve steps are necessary to conduct the resilient modulus test on cohesive soils which have been installed in the triaxial chamber and placed in the loading apparatus as described in either 6.4.5 or 7.7.1.

7.1.2.1 Open all drainage valves leading into the specimen.

7.1.2.2 If it is not already connected, connect the chamber pressure supply line and apply a confining pressure (chamber pressure) of 6 psi (41 kPa) to the test specimen.

7.1.2.3 Rebalance the recording bridges for the LVDT's and load-cell.

7.1.2.4 Begin the test by applying 200 repetitions of a deviator stress of 1 psi (6.9 kPa) and then 200 repetitions each of 2, 4, 8, and 10 psi (14, 28, 55, and 69 kPa). The foregoing stress sequence constitutes sample conditioning, that is, the elimination of the effects of the interval between compaction and loading and the elimination of initial loading versus reloading. This load conditioning also aids in minimizing the effects of initially imperfect contact between the end platens and the test specimen.

7.1.2.5 Decrease the deviator stress to 1 psi (6.9 kPa). Apply 200 repetitions of deviator stress and record the recovered deformations at the 200th repetition on a form for cohesive soils as shown in Figure 7.

7.1.2.6 Decrease the confining stress (chamber pressure) to 3 psi (21 kPa). Repeat Step 5.

7.1.2.7 Decrease the confining stress (chamber pressure) to zero. Repeat Step 5.

7.1.2.8 Increase the confining stress (chamber pressure) to 6 psi (41 kPa) and the deviator stress to 2 psi (14 kPa), apply 200 repetitions of load and record the vertical recovered deformations at the 200th repetition.

7.1.2.9 With the deviator stress at 2 psi (14 kPa), apply 200 deviator stress repetitions and record vertical recovered deformations at successive confining stresses (chamber pressures) of 3 psi (21 kPa) and zero.

7.1.2.10 Continue recording the vertical recovered deformations after 200 repetitions of the constant deviator stress-decreasing confining stress (chamber pressure) sequence for deviator stress values of 4, 8, and 10 psi (28, 55, and 69 kPa).

7.1.2.11 At the completion of the loading (with chamber pressure at zero) disassemble the triaxial cell and remove the LVDT clamps.

7.1.2.12 Use the entire specimen for determining the water content. Record this value on the form for cohesive soils as shown in Figure 7.

7.2 *Resilience Testing of Granular Soils*—The procedures listed in this section are used for both saturated and unsaturated specimens of cohesionless soils. For soils saturated after compaction using the steps of 6.5.3, the confining stresses called for in the conditioning phase are effective confining stresses; that is, the confining stress is equal to the chamber pressure less the back pressure.

7.2.1 After the test specimen has been prepared and placed in the loading device as described in 6.5.2 or 6.5.3, the following steps are necessary to conduct the resilient modulus testing:

7.2.1.1 If not already done, adjust the position of the axial loading device or triaxial chamber base support as necessary to couple the load-generation device piston and the triaxial chamber piston. The triaxial chamber piston should bear firmly on the load cell.

7.2.1.2 Rebalancing the recording bridges for the LVDT's and loadcell.

7.2.1.3 Set the confining stress to 5 psi (35 kPa) and apply 200 repetitions of an axial deviator stress of 5 psi (35 kPa). For saturated specimens the drainage valve from the base of the specimen to the back-pressure reservoir is open throughout the resilience testing.

7.2.1.4 Set the axial load generator to apply a deviator stress of 10 psi (69 kPa). Activate the load generator and apply 200 repetitions of this load.

7.2.1.5 Set the confining stress to 10 psi (69 kPa) and apply 200 repetitions of an axial deviator stress of 10 psi (69 kPa).

7.2.1.6 Apply 200 repetitions of an axial deviator stress of 15 psi (104 kPa).

7.2.1.7 Set the confining stress to 15 psi (104 kPa) and apply 200 repetitions of an axial deviator stress of 15 psi (104 kPa).

7.2.1.8 Apply 200 repetitions of an axial deviator stress of 20 psi (138 kPa).

7.2.1.9 If the specimen is one which has been saturated by the back-pressure saturation procedures of 6.5.3 reduce the back-pressure to zero.

7.2.1.10 Begin the recorded resilient modulus test by using a confining pressure of 20 psi (138 kPa) and a deviator stress of 1 psi (6.9 kPa). Record the vertical recovered deformations on a form for granular soils like that shown in Figure 8 after 200 repetitions have been applied.

7.2.1.11 Increase the deviator stress to 2 psi (14 kPa) and record the vertical recovered deformations after 200 repetitions. Continue to record vertical recovered deformations after 200 repetitions for deviator stress levels of 5, 10, 15, and 20 psi (35, 69, 104, and 138 kPa).

7.2.1.12 Reduce the confining pressure to 15 psi (104 kPa) and record vertical recovered deformations after application of 200 repetitions of each of the following deviator stress levels: 1, 2, 5, 10, 15, and 20 psi (6.9, 14, 35, 69, 104, and 138 kPa).

7.2.1.13 Reduce the confining pressure to 10 psi (69 kPa) and record vertical recovered deformations after application of 200 repetitions of each of the following deviator stress levels: 1, 2, 5, 10, and 15 psi (6.9, 14, 35, 69, and 104 kPa).

7.2.1.14 Reduce the confining pressure to 5 psi (35 kPa) and record vertical recovered deformations after application of 200 repetitions of each of the following deviator stress levels: 1, 2, 5, 10, and 15 psi (6.9, 14, 35, 69, and 104 kPa).

7.2.1.15 Reduce the confining pressure to 1 psi (6.9 kPa) and record vertical recovered deformations after application of 200 repetitions of each of the following deviator stress levels: 1, 2, 5, 7.5, and 10 psi (6.9, 14, 35, 52, and 69 kPa). Stop the loading after 200 repetitions of the last deviator stress level or when the specimen fails.

7.2.1.16 Reduce the chamber pressure to 0, dismantle the cell, and remove the LVDT clamps.

7.2.1.17 Use the entire test specimen to determine the water content. Record this value on the form for granular soils as shown in Figure 8.

8 CALCULATIONS

8.1 Calculations are performed by using the tabular arrangement from a form as shown in Figure 7 and 8.

9. REPORT

9.1 *Cohesive Soils*—The report for resilient modulus tests on cohesive material shall include the following.

9.1.1 Data sheets with calculations in tabular form as shown in Figure 7 for each specimen tested.

9.1.2 Plots showing variation in resilient modulus with deviator stress and confining stress of the form shown in Figure 9 for each specimen tested.

9.1.3 Plot of moisture-density relation for the soil tested showing the 100 and 80 percent saturation lines and the points (moisture-density coordinate) of the specimens tested

9.1.4 Remarks—note any unusual conditions or other data that would be considered necessary to properly interpret the results obtained.

9.2 *Granular Soils*—The report for resilient modulus tests on granular materials shall include the following:

9.2.1 Data sheets with calculations in tabular form as shown in Figure 8 for each specimen tested.

9.2.2 Plots showing variations in resilient modulus with deviator stress and confining stress of the form shown in Figure 9 for each specimen tested.

9.2.3 Log-log plot of resilient modulus versus the sum of the principal stresses of the form shown in Figure 10 for each specimen tested. Values of the regression constants K_1 and K_2 shall be stated on each plot.

9.2.4 Plot of moisture-density relation for the soil tested showing the 100 and 80 percent saturation lines and the points (moisture-density coordinates) of the specimens tested.

9.2.5 Remarks—note unusual conditions or other data that would be considered necessary to properly interpret the results obtained.

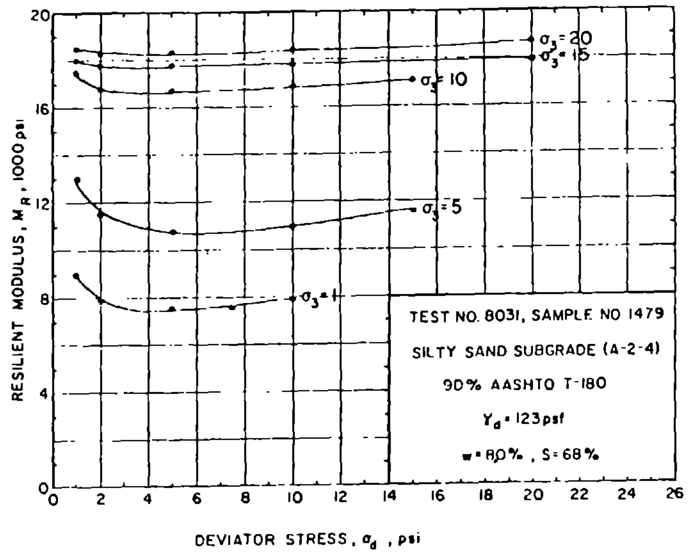


Figure 9. ARITHMETIC PLOT OF RESILIENT MODULUS TEST RESULTS

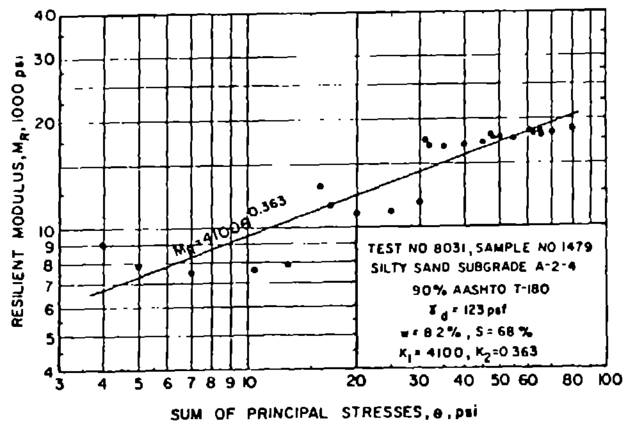


Figure 10. LOGARITHMIC PLOT OF RESILIENT MODULUS TEST RESULTS

FACTORS AFFECTING THE RESILIENT MODULI OF SOILS AND GRANULAR MATERIALS

by

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INTRODUCTION

Subgrade soils have a major impact on the design, construction, structural response, and performance of a pavement. All pavement structural design procedures require a "subgrade soil" input (i.e. CBR, soil support number, resilient modulus, k value, R value, etc).

The subgrade has a large effect on pavement construction operations and efficiency. Unstable subgrades present problems relative to placing and compacting subbase and base materials and providing adequate support for subsequent paving operations. Such construction operations as placing reinforcing steel, dowels, and jointing devices are sensitive to placement details. Without an adequate "working platform", critical pavement construction details may not be accomplished within acceptable tolerances. Frequently, such construction deficiencies are undetected because they are "hidden" in the finished pavement!

Pavement structural responses (stresses, strains, displacements) are significantly influenced by the subgrade. A large percentage of the surface deflection of a pavement is accumulated in the subgrade. If surface deflection is a design criteria, the need for good subgrade characterization is obvious. Adequate subgrade characterization requires consideration of the fluctuation of subgrade soil properties as a function of space (various locations with depth in the subgrade and longitudinal location along the project) and time (seasons of the year and yearly climate variation).

Granular base and subbase layers are essential components of a flexible pavement. Their function is to reduce the repeated wheel loading related stress state on underlying layers and minimize rutting within the granular layers. Granular layers are of particular significance in low ESAL (equivalent single axle load) applications where the AC (asphalt concrete) surface course is thin (less than 4-6 inches) or only a surface treatment is utilized.

The major recent emphasis in granular material and subgrade soil evaluation has been repeated load testing. Resilient moduli and permanent deformation behavior can be quantified based on appropriate repeated load testing data. In a well designed pavement system the permanent strain accumulated per load cycle is very small compared to the total strain.

In the 1986 AASHTO Guide (1), resilient moduli are used to characterize subgrade soils and assign "layer coefficients" to granular base and subbase layers. State Highway Agencies are experiencing considerable difficulty in establishing the appropriate "resilient modulus" inputs for these purposes.

Repeated Load Testing

Suggested procedures for repeated load testing have been proposed by several agencies and groups (2,3,4). AASHTO (5) has adopted a procedure (T274-82) for, "Resilient Modulus of Subgrade Soils." ASTM is currently developing a suggested procedure.

Triaxial test conditions (generally constant confining pressure) are used for granular materials. Cohesive soils can be tested in unconfined compression or under triaxial conditions.

Pneumatic and electrohydraulic repeated loading equipment have been successfully utilized. The equipment must be capable of producing a load pulse duration of approximately 25 to 150 msec. The load pulse is generally repeated 15 to 30 times a minute.

Specimen deformation over a portion, or in some cases the entire length, of the specimen is typically measured used LVDTs. Total, resilient (rebound), and permanent deformations are typically recorded.

Figure 1 illustrates the response of a soil to a repeated load pulse.

RESILIENT BEHAVIOR

A commonly used measure of resilient response is the "resilient modulus" as defined below:

$$E_R = \sigma_D/\epsilon_R$$

E_R = resilient modulus

σ_D = repeated deviator stress

ϵ_R = recoverable axial strain

The concept is apparent from an examination of Figure 1. The resilient response of granular materials and fine-grained soils is stress dependent.

Fine-Grained Soils

Cohesive soils display stress-softening resilient behavior under repeated loading. Robnett and Thomspson (2) have demonstrated that for practical pavement design and analysis purposes repeated unconfined (σ_3=0) compression testing is satisfactory for resilient testing of cohesive soils.

Two basic stress dependent behavior models have been utilized. The arithmetic model is demonstrated in Figures 2 and 3 and the semi-log model is shown in Figure 4.

Recent extensive resilient testing studies at the University of Illinois (6, copy attached) have been primarily analyzed based on the arithmetic model. E_{Ri}, see Figure 2, is a good indicator of a soil's resilient behavior. E_{Ri} is typically associated with a repeated deviator stress of about 6 psi. The slope values, K_1 and K_2, display less variability and influence pavement structural response to a smaller degree than does E_{Ri}.

There are many factors that influence the resilient response of fine-grained soils. A general summary of the major factors, based on the Illinois study (6) of 50 typical fine-grained soils, is presented below.

Soil Properties - For a given compaction condition (for example 95% AASHTO T99 density and optimum or optimum + water content), E_{Ri} is significantly correlated with liquid limit, plasticity index, group index, silt content, clay content, specific gravity, and organic carbon content. Those properties that tend to contribute to low resilient moduli (low E_{Ri}) are low plasticity (LL, PI), low group index, high silt content, low clay content, low specific gravity, and high organic carbon contents. Regression equations for predicting E_{Ri} based on soil properties were developed by Thompson and Robnett (6).

For Illinois fine-grained soils, Thompson & LaGrow (7) have proposed using the following relation for conventional flexible pavement design purposes.

$$E_{Ri}(OPT) = 4.46 + 0.098\ C + 0.119\ (PI)$$

$$R^2 = 0.63 \quad SEE = 2.7\ \text{ksi}$$

where:

$E_{Ri}(OPT)$ - "Breakpoint Resilient Modulus", ksi, (see Figure 3) at AASHTO T-99 optimum moisture content and 95 % compaction

C - less than 2 micron clay content (%)

PI - Plasticity index (%)

Soil Classification Effects - Analysis of variance results (6) indicated that the resilient behavior (E_{Ri}, K_1 response parameters) of the various groups in the classification systems (Unified, AASHTO, USDA) frequently are not significantly different. Thus, classifying the soil in the AASHTO, Unified, or USDA system does not place fine-grained soils into distinctive resilient behavior groups.

Moisture-Density Effects - Degree of saturation is a factor that reflects the combined effect of density and moisture content. E_{Ri} is strongly correlated with degree of saturation. The regression equations shown in Figure 5 indicate that E_{Ri} can be estimated based on degree of saturation. The E_{Ri}-degree of saturation regression equations differ for 95% AASHTO T-99 and 100% AASHTO T-99 compaction. One hundred percent compaction provides higher E_{Ri} for a given degree of saturation. The difference in E_{Ri} values for 100% and 95% AASHTO T-99 compaction is reduced at increased degrees of saturation.

The combined effects of compaction moisture content and density are easily discerned using the E_{Ri}-degree of saturation relations. For soils substantially wet of optimum, high degrees of saturation and low E_{Ri} values are characteristic, regardless of level of compaction.

Additional E_{Ri}-degree of saturation regression equations of the form $E_{Ri} = a + b\ S_R$ were developed (E_{Ri} = "breakpoint" resilient modulus, a = intercept, S_R = degree of saturation, %) for various soil classification groups. The regression coefficient, b, is indicative of moisture sensitivity. It was noted that for the B and C horizons those classification groups with higher clay contents and increased plasticity tend to be less sensitive to changes in degree of saturation. For example, the CH group in the Unified system and the silty clay loam, clay and silty clay groups in the USDA system have regression coefficients substantially less than the other groups.

Based on subsequent analyses of the U of I data, Thompson and LaGrow (7) proposed using the following "moisture adjustment" factors to adjust E_{Ri} (OPT) values for moisture contents in excess of optimum.

USDA Textural Class	Moisture Sensitivity *
clay, silty clay, and silty clay loam	0.7
silt loam	1.5
loam	2.1

* - E_{Ri} decrease (ksi) for a 1 % moisture increase

Compressive Strength Effects - University of Illinois data (6) indicate that E_{Ri} can be predicted using unconfined compressive strength. The regression equation is :

$$E_{Ri} = 0.86 + 0.307\ Q_u$$

$$R^2 = 0.47 \quad SEE = 2.61 \text{ ksi}$$

where:

E_{Ri} = "breakpoint" resilient modulus, ksi

Q_u = Unconfined compressive strength, psi

The unconfined compressive strength should be representative of the insitu conditions. It is important to note that insitu strength typically displays considerable seasonal variability. In the U of I study (6) the effect of increased (+ optimum) compaction moisture content on compressive strength was considered. For 32 B and C horizon soils, the unconfined strength at optimum + 2% moisture content was (on the average) 78 % of the strength at optimum. Increased silt content and lower PIs indicated increased moisture sensitivity (greater decreases) relative to both strength and modulus.

Freeze-Thaw Effects - Studies by Bergan and Fredlund (8), Culley (9), Chamberlain (10), Bergan and Monismith (11), Bergan and Culley (12), and Robnett and Thompson (13), have shown that the resilient behavior of fine-grained cohesive soils is also greatly affected by cyclic freeze-thaw action. The studies revealed that substantial increases in resilient deformation (reduced resilient moduli) were caused by the imposition of a small number of freeze-thaw cycles, even though no gross moisture changes were allowed (closed system freeze-thaw).

Typical data illustrating the freeze-thaw effect for Tama B [LL = 46, PI = 25, 32% < 2 clay, AASHTO Class A-7-6(27)] are shown in Figure 6. It is significant to note that one freeze-thaw cycle is sufficient to drastically reduce the resilient modulus of the soil.

Summary - It is apparent that many factors influence the resilient behavior of fine-grained soils. Careful engineering consideration must be given to determining the resilient properties (subgrade support values) utilized in pavement analysis and design.

Granular Materials

Granular materials "stiffen" (increased resilient modulus) as stress state increases. Repeated load triaxial testing is required to characterize the resilient behavior of granular materials. Resilient modulus is a function of the applied stress state. The "theta model" is frequently used to characterize this behavior. In fact the "theta model" is recommended in the 1986 AASHTO Guide (1). The model is :

$$E_R = k\theta^n$$

where:

E_R = resilient modulus

k & n = experimentally derived factors

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$

(Note $\theta = \sigma_1 + 2\sigma_3$ in the triaxial test)

Figure 7 is an E_R - θ relation for a sandy-gravel [AASHTO A-1-a(0)].

Several investigators (14-21) have conducted comprehensive laboratory studies on the repeated loading behavior of granular materials. The studies support the following generalized statements concerning resilient modulus trends for granular materials:

1. The resilient response after a limited number of load repetitions (100 or so) is representative of the response determined after several thousand repetitions.
2. The same specimen can be used to measure the resilient response over a wide range of stress levels, and the stresses can be applied in any order (with the caveat that the repeated stress states are not greater than approximately 60 % of the ultimate shear strength of the material).

3. Resilient modulus is only minimially affected by variations in stress pulse duration. In fact, Kalcheff and Hicks (22) demonstrated that if the stress pulse is rapidly applied, and then sustained; the resilient response is the same as that obtained from a rapidly applied and released short duration stress pulse of the same magnitude. In a recent TRB presentation, Barksdale and Itani (23) indicated that a "slow cyclic test" (they used a 0.25 % / minute strain rate and 5 cycles of loading)

> "can be used to evaluate the resilient modulus of unbound aggregate bases for design purposes. The modulus obtained from a slow cyclic test could, if desired, be increased by 10 percent to give better results, which is in agreement with other studies"

4. For practical purposes, CCP (constant confining pressure) and VCP (variable confining pressure) triaxial resilient moduli are similar.

6. For a given gradation, crushed materials provide increased resilient moduli.

7. For a given gradation and nature of material (crushed, uncrushed, etc), material source (limestone, granite, trap rock, etc) is not a highly significant factor.

8. For a given material, density has a limited impact on resilient modulus.

9. The effects of minor gradation changes are of limited significance. However, the resilient moduli of "open-graded" aggregates (such as AREA #4 or AREA #5 ballast) tend to be somewhat lower than for conventional dense-graded aggregates.

10. Increased moisture contents (above optimum) tend to decrease resilient modulus. Moisture sensitivity will vary depending on specific gradations and the amount and nature (primarily PI) of the minus # 200 material. Lary and Mahoney's study (24) developed moisture sensitivity data for several granular base materials sampled from typical U. S. Forest Service roads in the Northwest. Their regression equations indicate that for an initial modulus of 20 ksi, a 1 % increase in moisture content would induce resilient modulus decreases from about 0.6 to 1.6 ksi. Open-graded aggregates that do not contain fines are practically "moisture insensitive."

Rada and Witczak (18) summarized typical resilient property data (derived from available technical literature) for granular materials. Their findings are shown in Table 1 . Note that as k increases, n decreases. Rada and Witczak's proposed "k - n" relation is shown in Figure 8. Higher quality granular materials display larger k's and smaller n's.

Suggested typical k and n values presented in the AASHTO Guide (1) are shown in Table 2. Note that the moisture efeect is considered.

Elliott and Thompson (25) demonstrated that for a range of granular materials (k = 9000 psi, n= 0.33; k= 4000 psi, n= 0.5), the maximum effect on ILLI-PAVE calculated structural responses (deflections, subgrade stresses, and AC strain) is about 10 %. Their analyses of the AASHO Road Test deflection results for Loop 4 also indicated that the nature of the granular materials (crushed stone base / sandy gravel subbase) was not a significant factor. The combined thickness of the base and subbase layers was a significant factor.

Hicks et al (21) considered the effects of percent fracture and gradation on aggregate bases. The study demonstrated the relative insensitivity of predicted pavement life and required pavement thickness for the range of aggregate variables (percent fracrure, percent fines). It is important to note that "rutting" was not included in the performance assesment.

It is apparent that for practical apriori structural analysis design purposes typical resilient moduli data for generic type granular materials are probably satisfactory. This does not imply that the shear strength and "rutting potentials" of the granular materials are similar! In fact, many of the factors that have little or practically insignificant effects on resilient behavior very strongly and significantly influence the shear strength and rutting potential of a granular material. Thus it would appear that the granular material - resilient modulus relations presented in the 1986 AASHTO Guide (1) may well be inappropriate and misleading, particularly for those cases where granular layer lutting is a major factor influencing pavement performance.

SUMMARY

Repeated load testing data are important inputs for flexible pavement analysis and design. Static testing procedures are not adequate for characterizing the behavior of soils and granular materials subjected to the impulse type repeated loading representative of moving wheel loads.

Repeated load testing procedures can be used to quantify resilient moduli and permanent deformation behavior. General material resilient modulus and permanent deformation behavior models have been developed. Only resilient behavior is considered in this paper.

The repeated loading responses of fine-grained soils and granular materials are quite variable. Many factors influence resilient moduli.

The resilient moduli of cohesive soils cover a very wide range. Soil texture, PI, and degree of saturation (moisture content, density), are particularly important factors for fine-grained soils.

The resilient moduli of granular materials display more "generic" type behavior and are less variable than for fine-grained soils. Gradation, shape/angularity/surface texture (crushed - uncrushed), and moisture content (especially for "high fines content" materials) influence granular material resilient moduli. The magnitude of the repeated stress state (as expressed by the bulk stress - theta) is the most dominating and significant factor.

Data and study findings are presented indicating that the "resilient modulus" of a granular material may not be an appropriate property for assigning "layer coefficients" in the utilization of the 1986 AASHTO Guide. The rutting potentials of granular materials are not adequately defined by "resilient modulus" relations.

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TABLE 1

TYPICAL RESILIENT PROPERTY DATA

(Reference 18)

Granular Material Type	Number of Data Points	K* (psi) Mean	K* (psi) Standard Deviation	n* Mean	n* Standard Deviation
Silty Sands	8	1620	780	0.62	0.13
Sand-Gravel	37	4480	4300	0.53	0.17
Sand-Aggregate Blends	78	4350	2630	0.59	0.13
Crushed Stone	115	7210	7490	0.45	0.23

* $E_R = K\theta^n$ where

E_R = resilient modulus, psi

K,n = experimentally derived factors from repeated triaxial testing data

TABLE 2

Typical values for k_1 and k_2 for unbound base and subbase materials ($M_R = k_1 \theta^{k_2}$).

(Reference 1)

Moisture Condition	k_1*	k_2*
(a) Base		
Dry	6,000 - 10,000	0.5 - 0.7
Damp	4,000 - 6,000	0.5 - 0.7
Wet	2,000 - 4,000	0.5 - 0.7
(b) Subbase		
Dry	6,000 - 8,000	0.4 - 0.6
Damp	4,000 - 6,000	0.4 - 0.6
Wet	1,500 - 4,000	0.4 - 0.6

* Range in k_1 and k_2 is a function of the material quality.

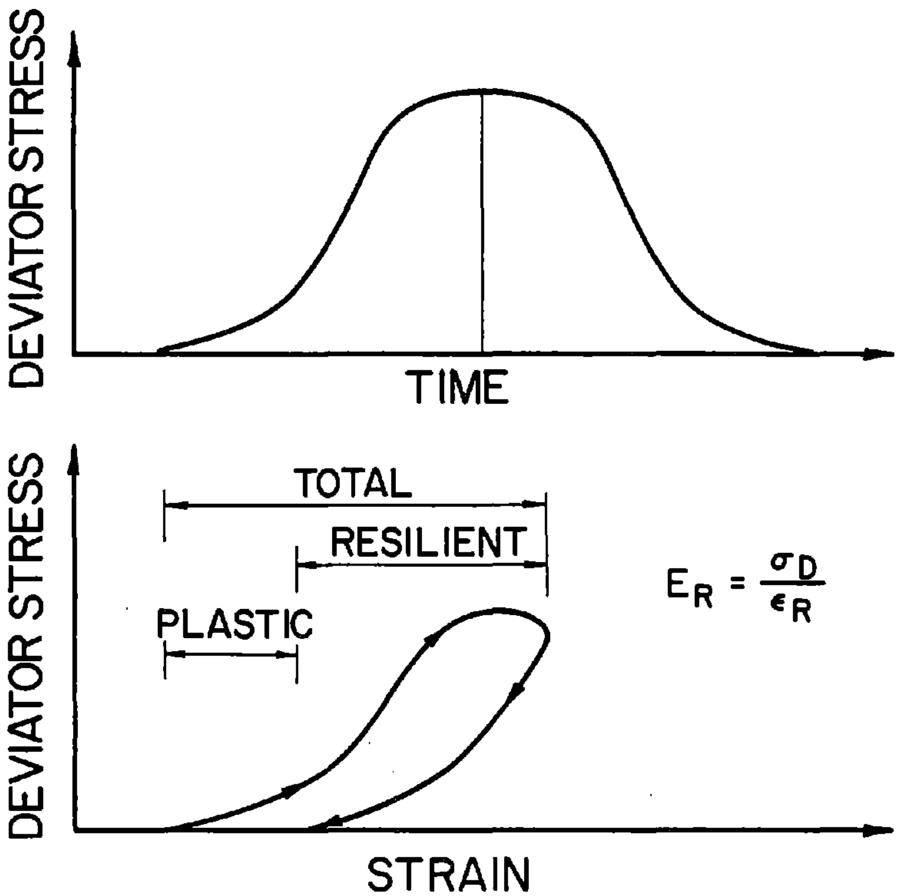

Figure 1. Repeated Load Testing Concepts.

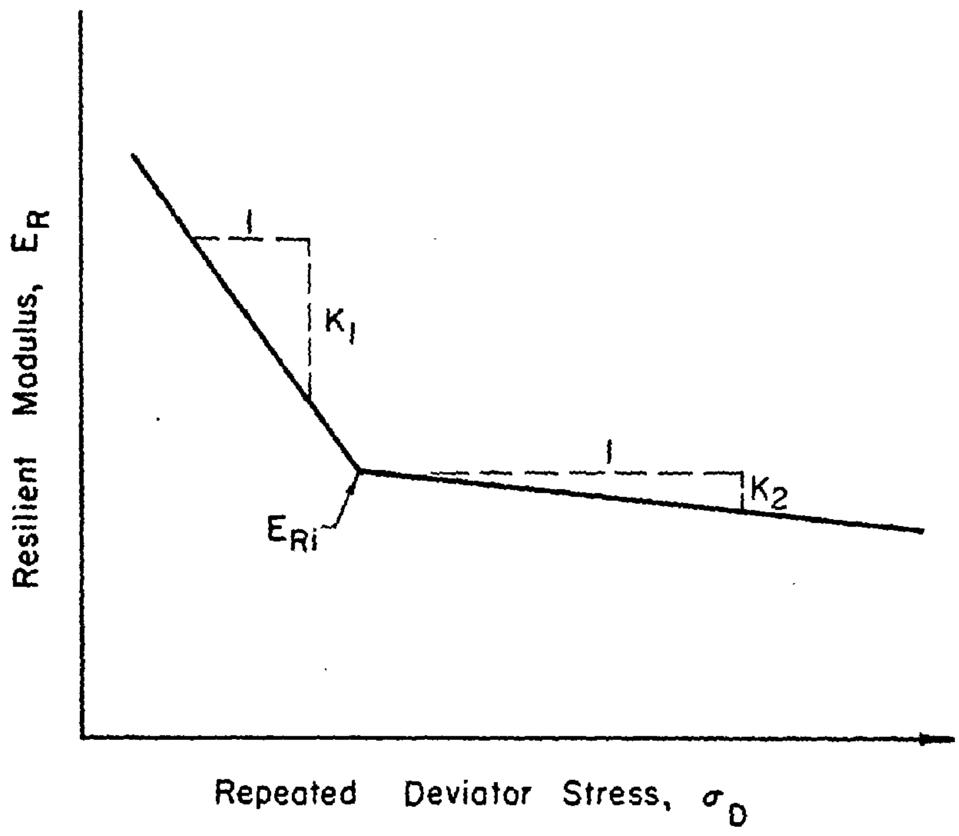

Figure 2. Arithmetic Model for Stress Dependent Resilient Behavior of Fine-Grained Soils.

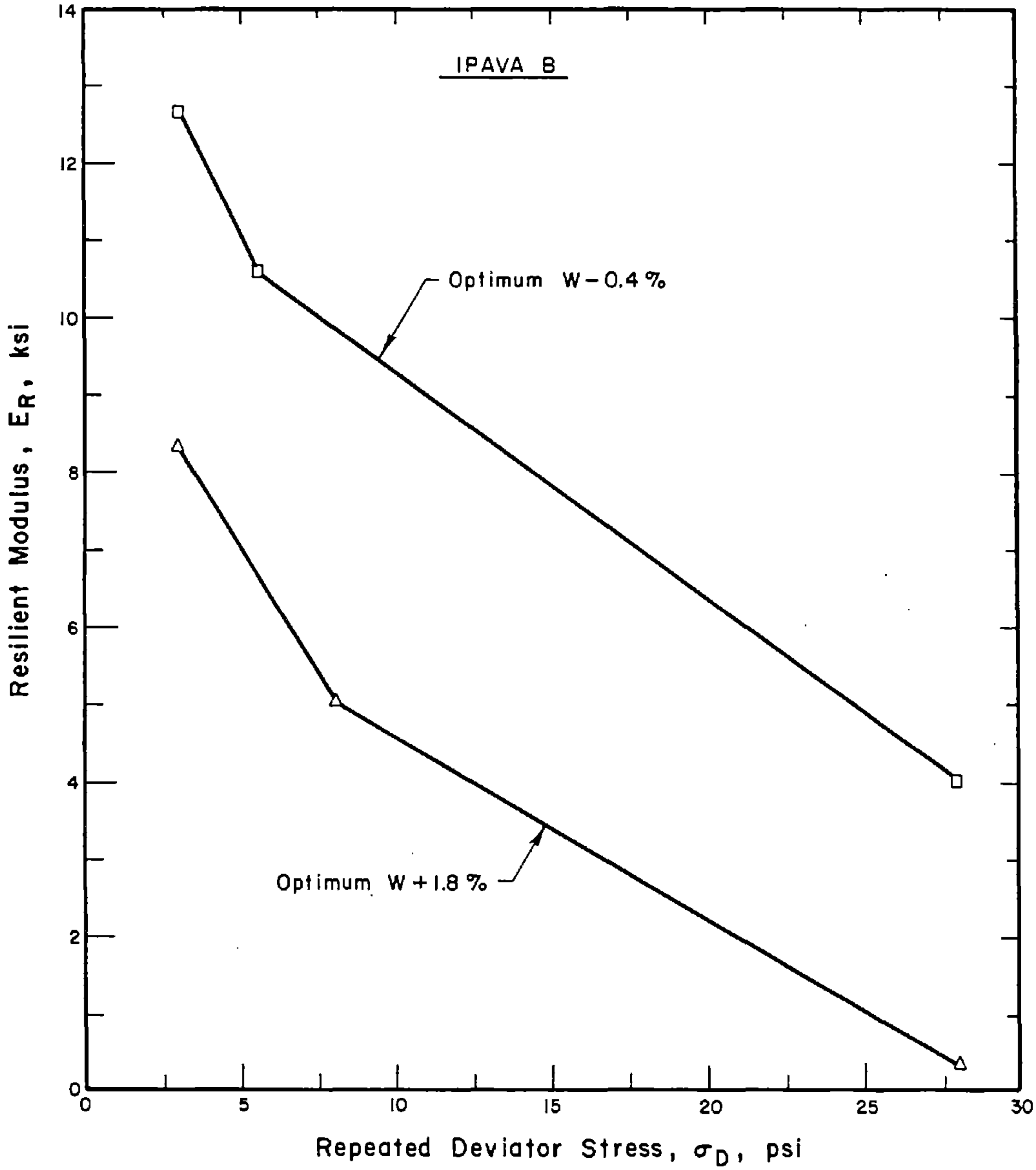

Figure 3. Typical Stress Dependent Resilient Behavior of a Fine-Grained Soil [AASHTO A-7-6(36)].

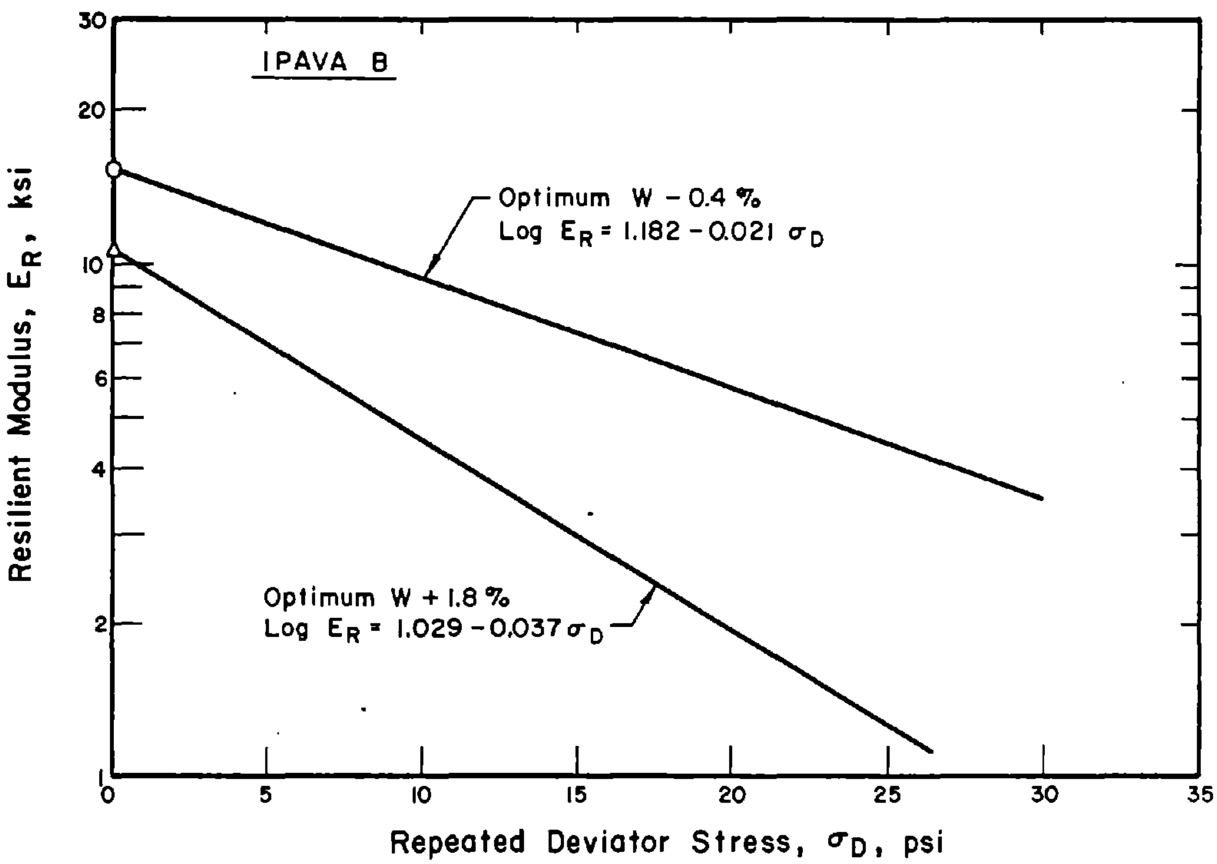

Figure 4. Semi-Log Model for Stress Dependent Resilient Behavior of a Fine-Grained Soil [AASHTO A-7-6(36)].

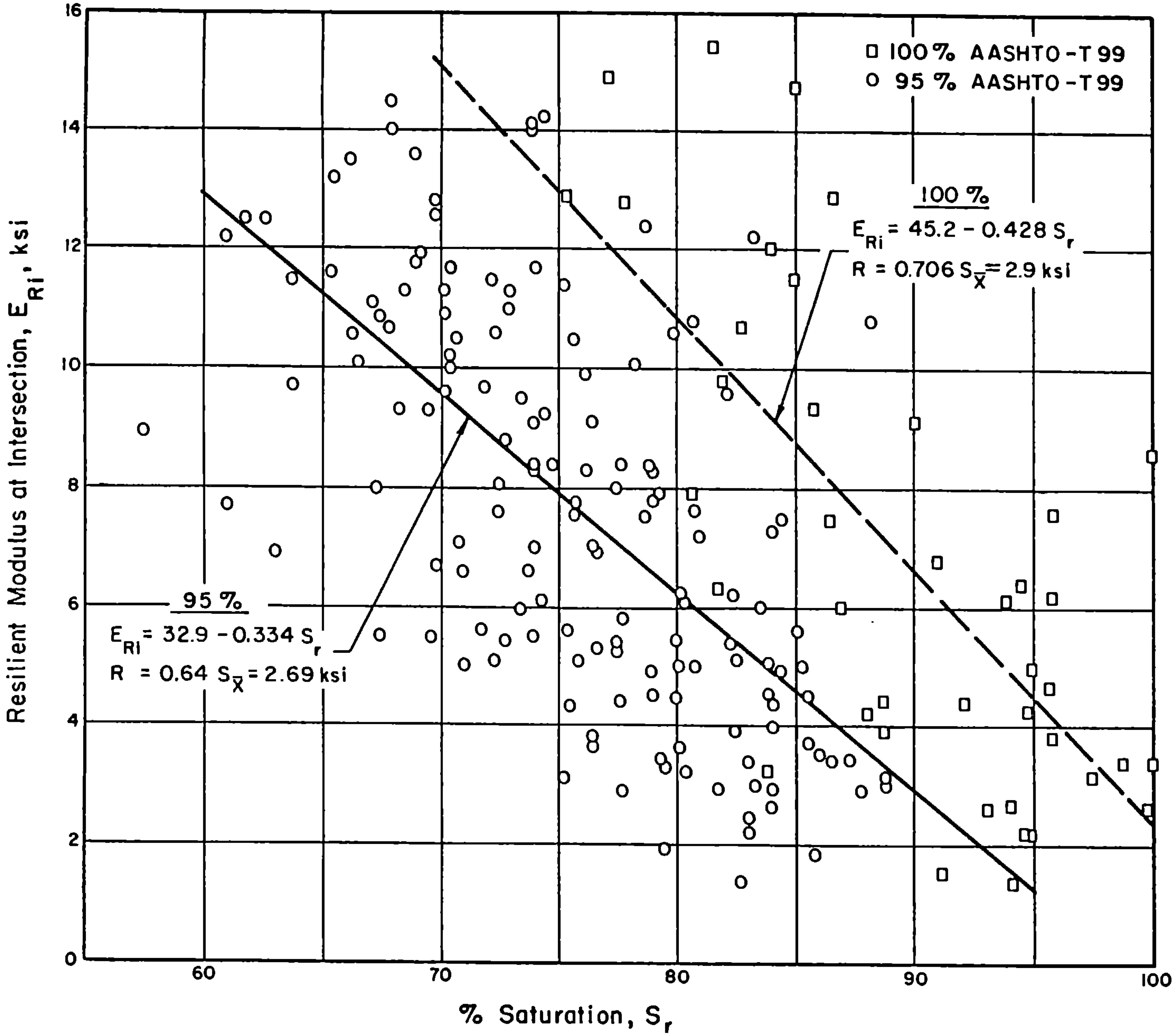

Figure 5. Resilient Modulus - % Saturation Relations for Fine-Grained Soils.

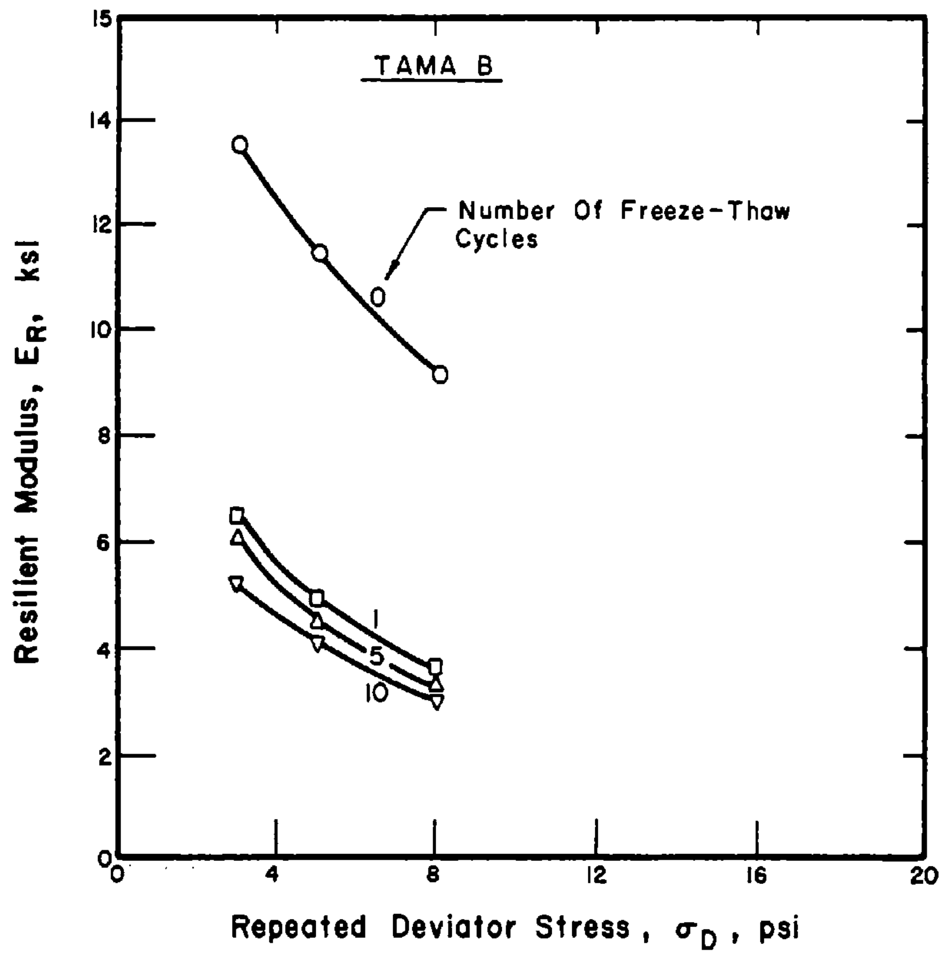

Figure 6. Influence of Cyclic Freeze-Thaw on the Resilient Behavior of a Fine-Grained Soil [AASHTO A-7-6(27)].

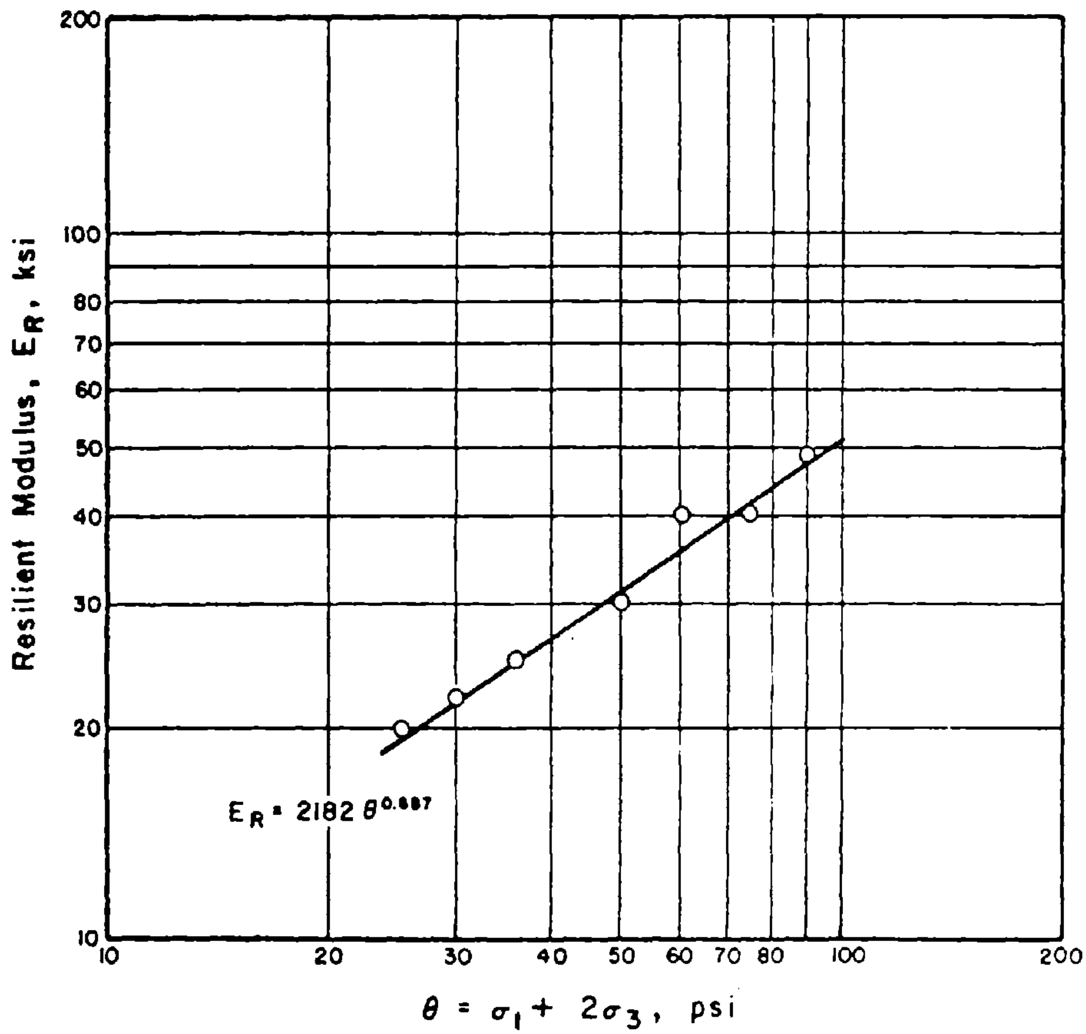

Figure 7. Resilient Modulus – θ Relation for a Sandy Gravel.

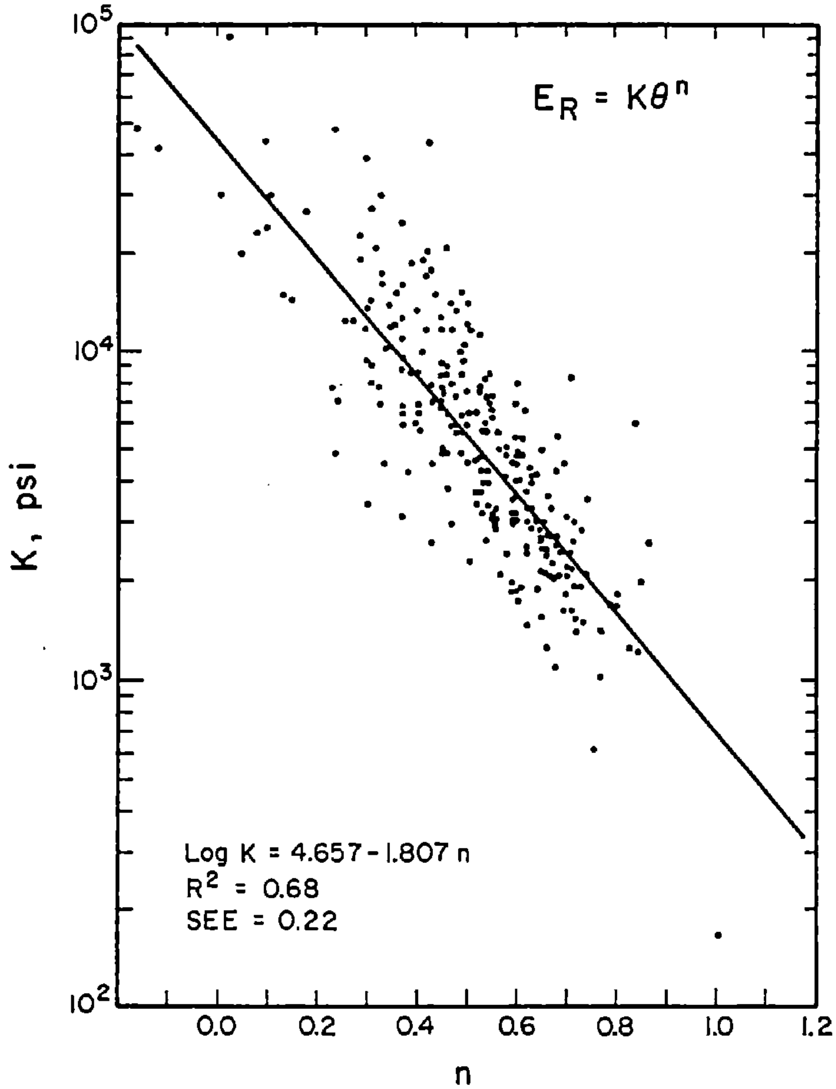

Figure 8. Relationship between K and n Values for Granular Materials Identified by Rada and Witczak.

Factors that Influence Resilient Modulus Values for Asphalt Concrete

by

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presented at

Workshop on Resilient Modulus Testing
Oregon State University
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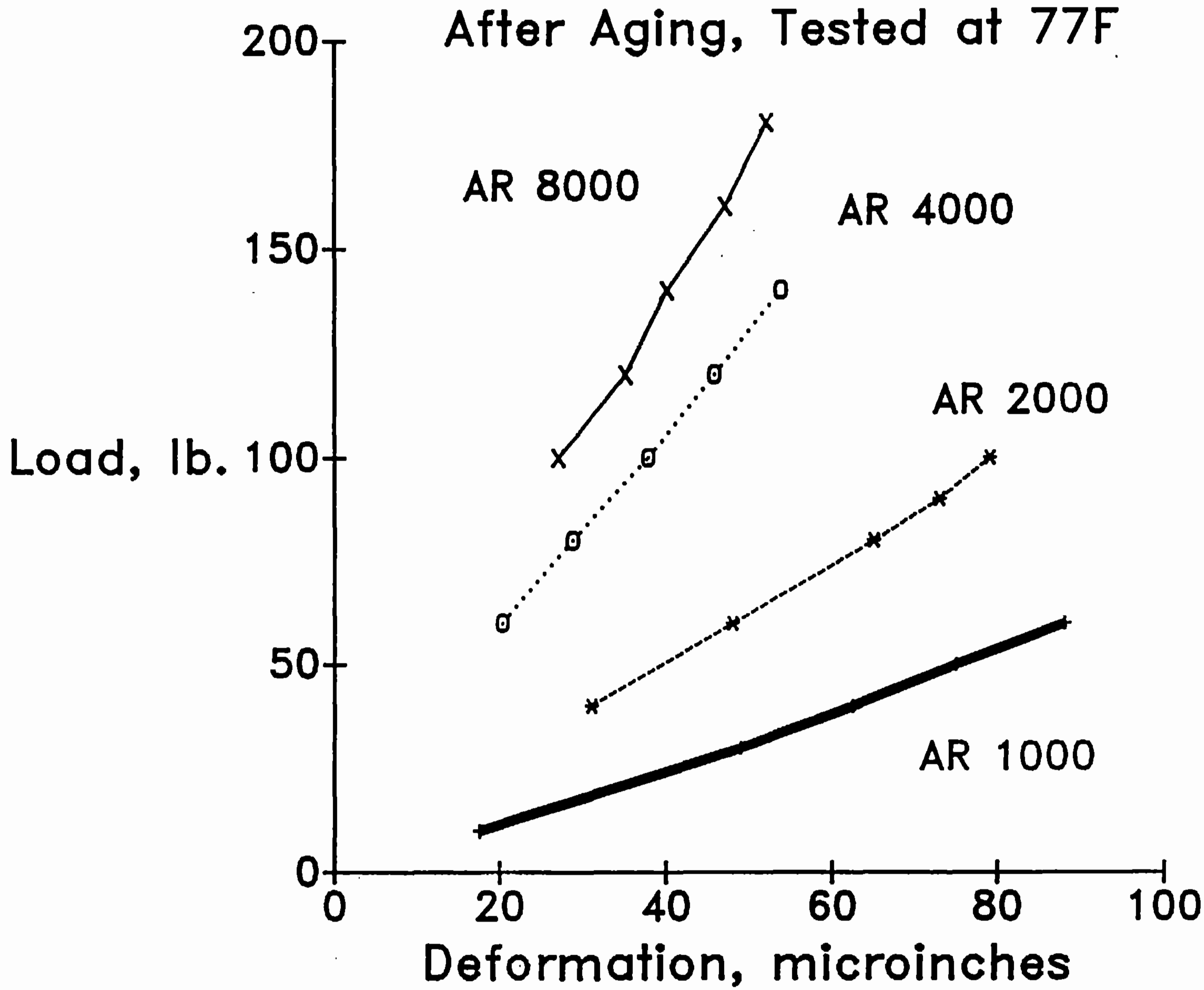
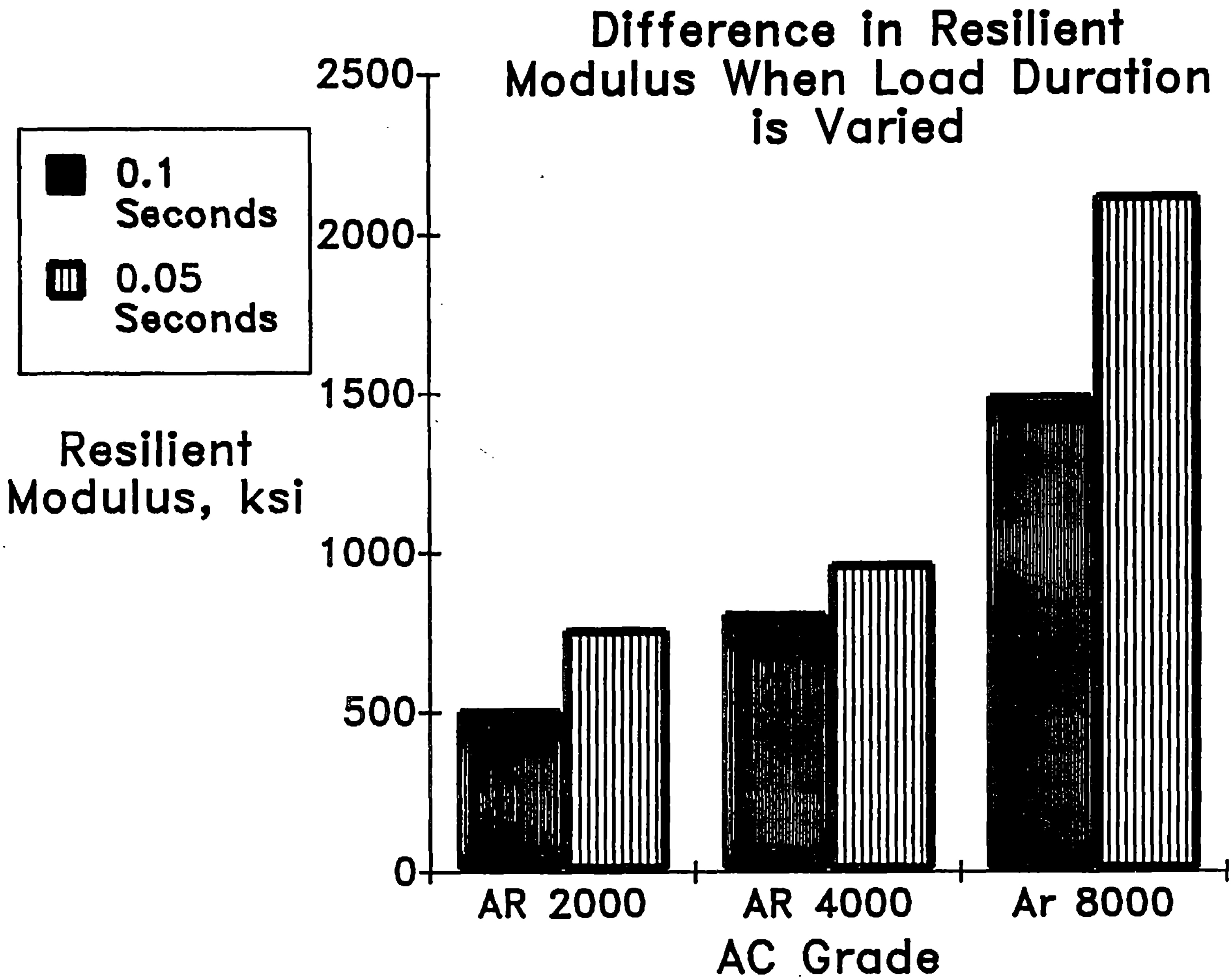
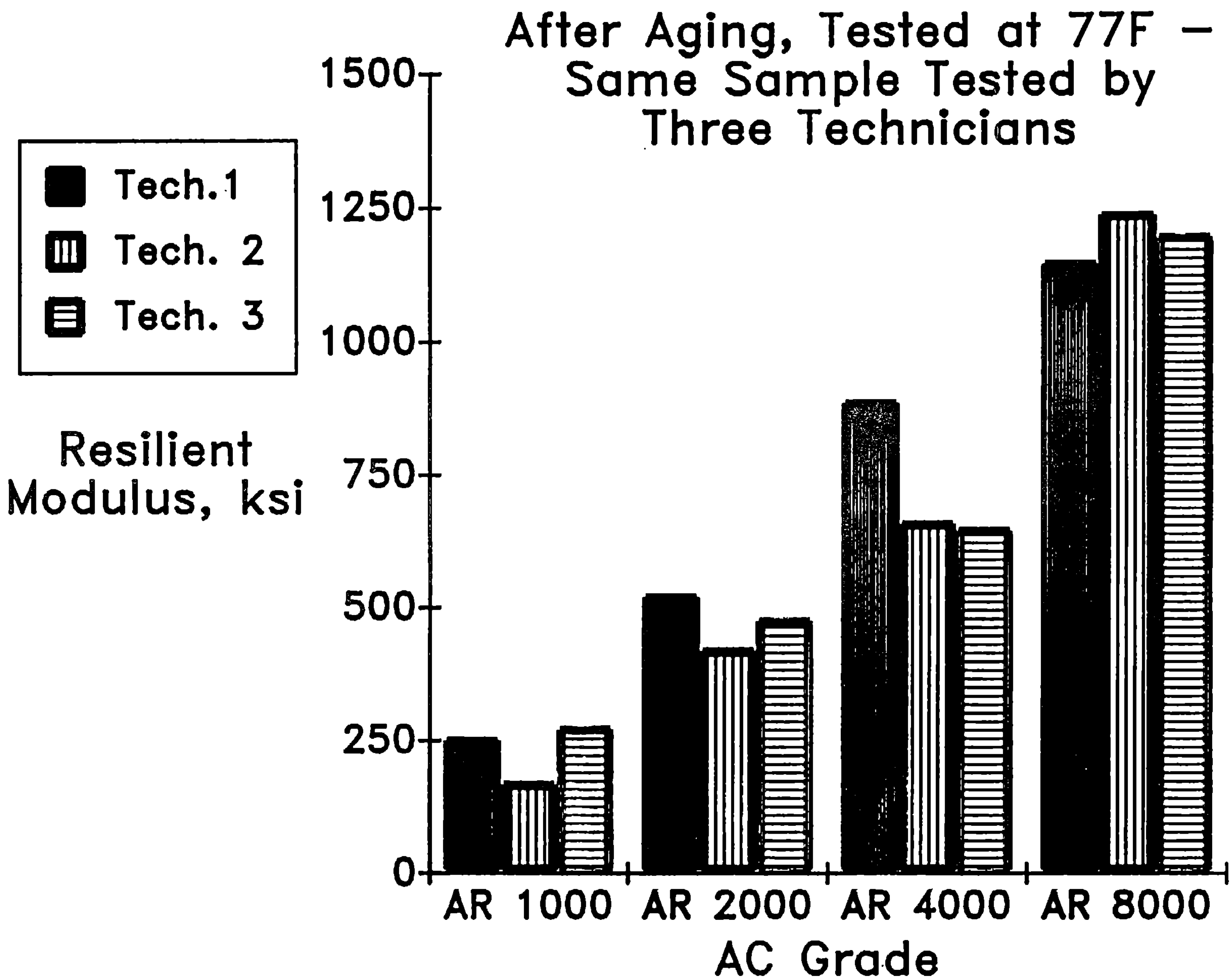
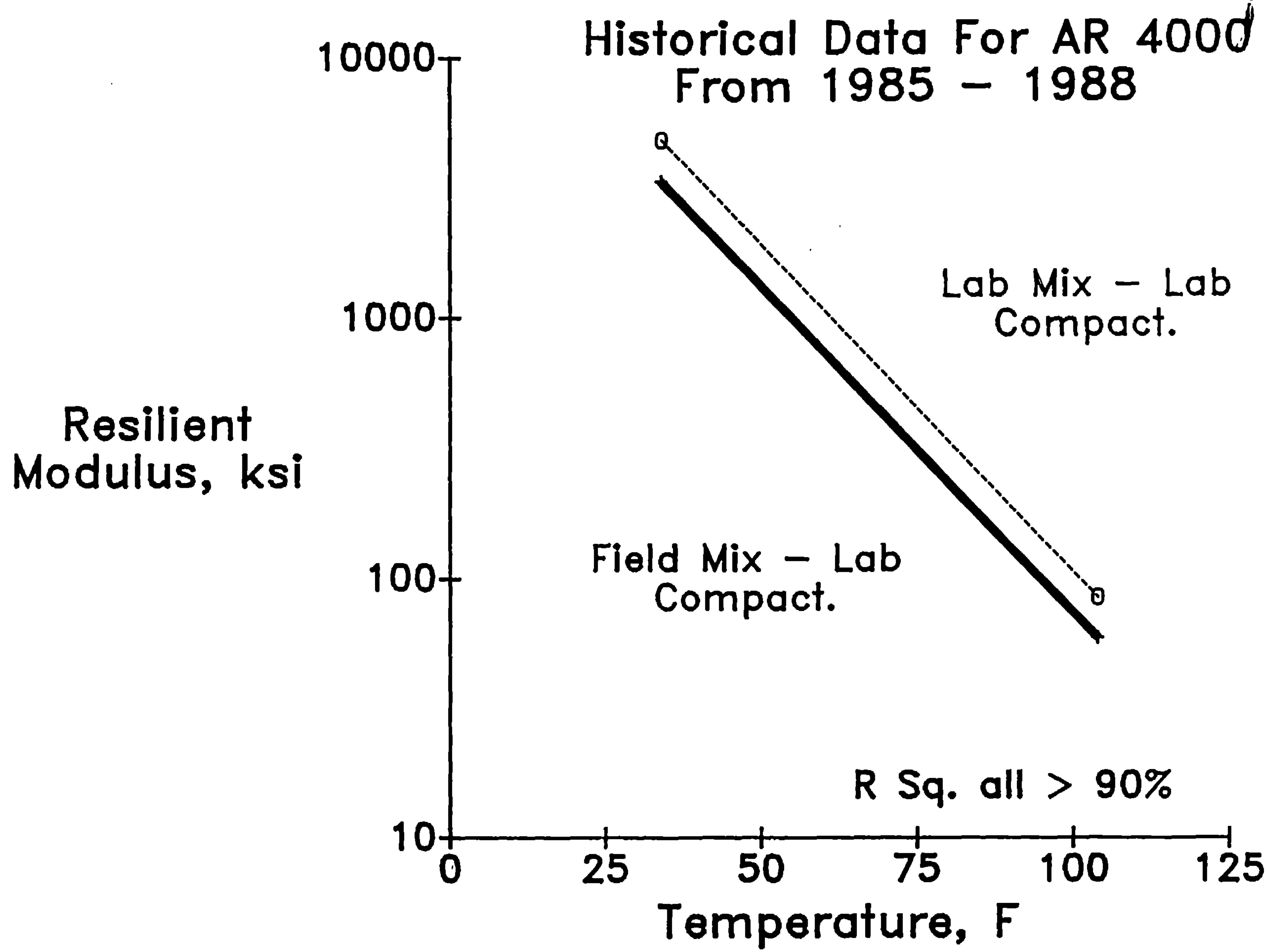
March 28-30, 1989

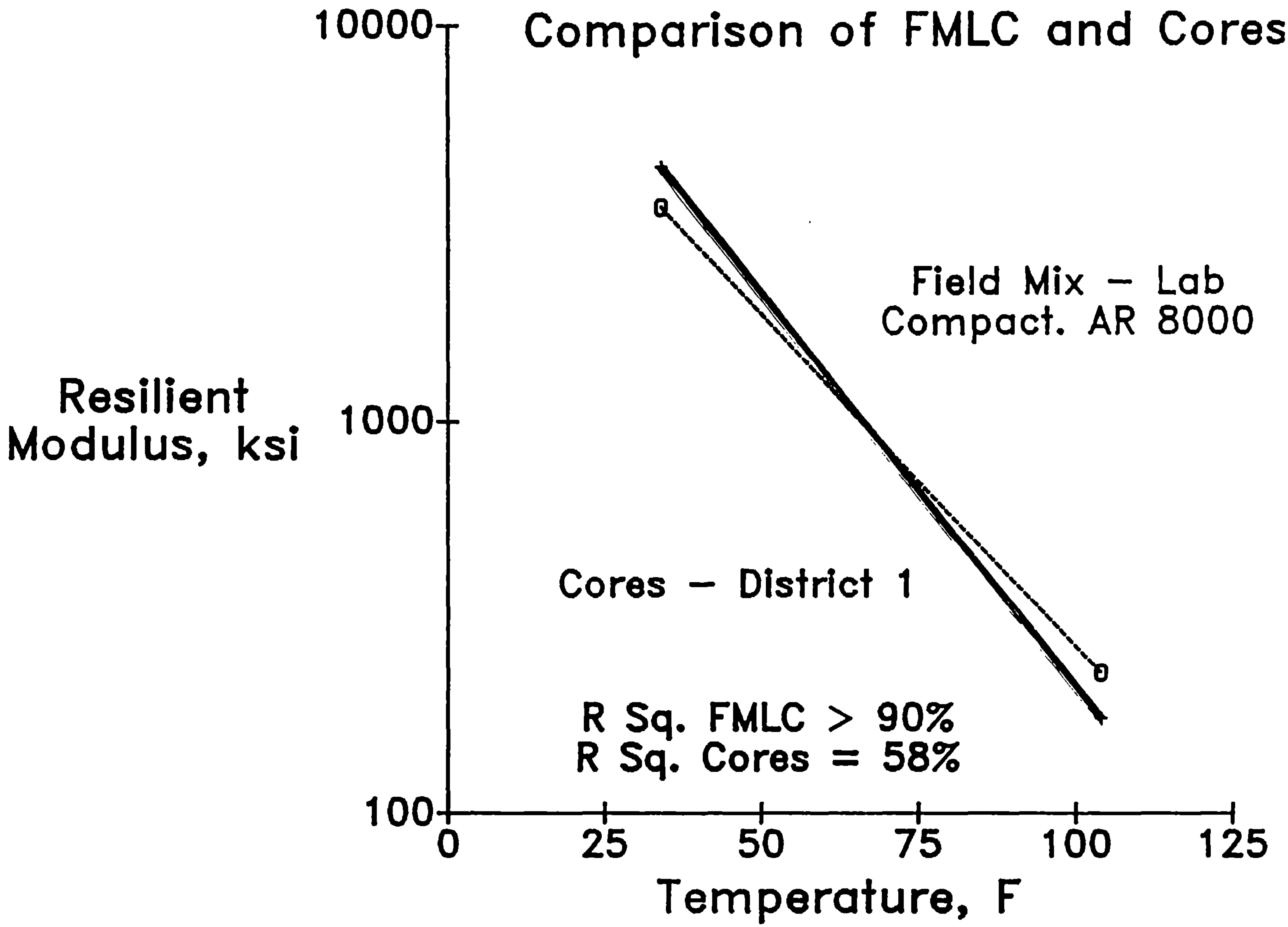
Factors That Influence Resilient Modulus Values for Asphalt Concrete

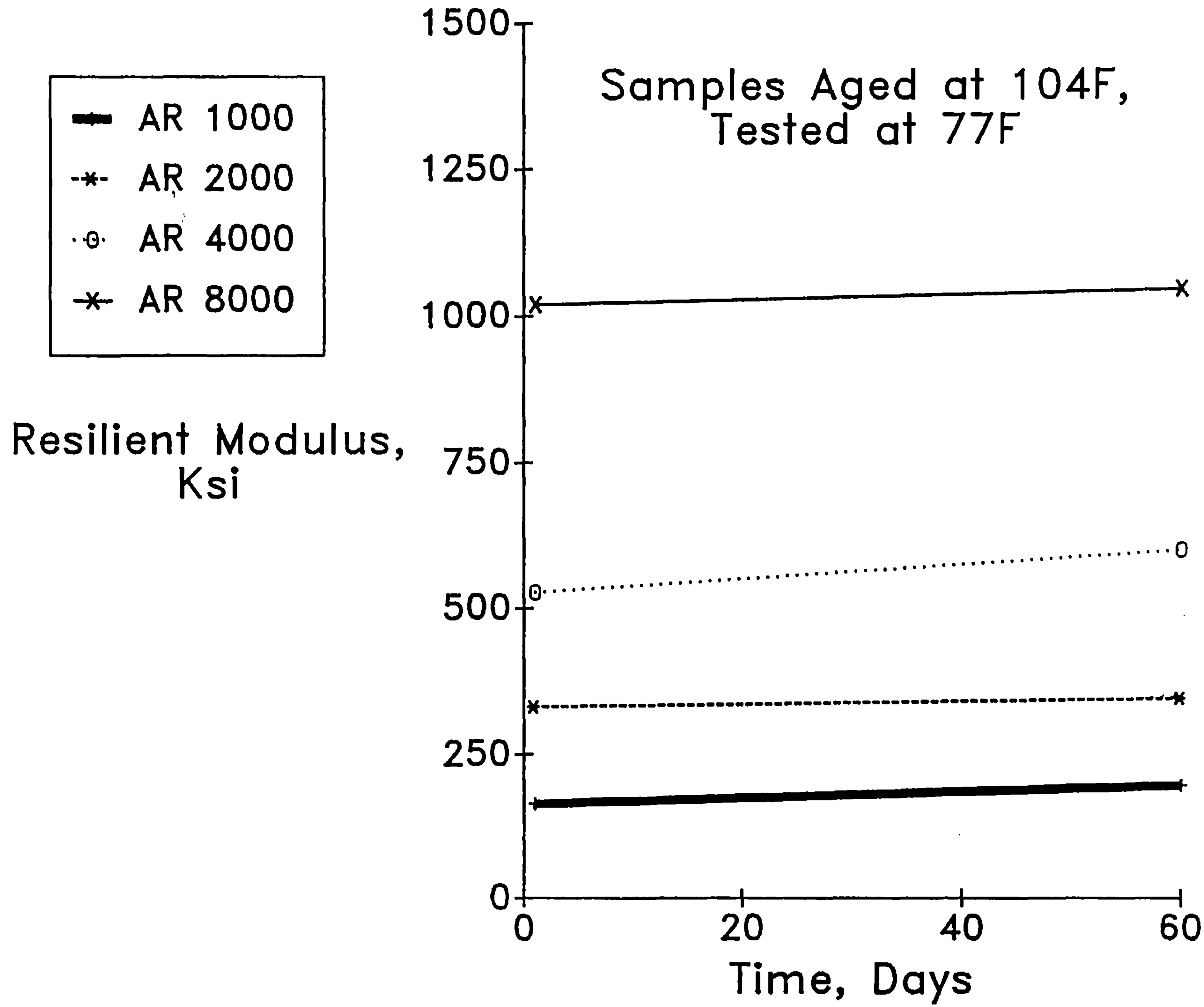

After Aging, Tested at 77F
Load, lb.
200
150
100
50
0
AR 8000
AR 4000
AR 2000
AR 1000
0
20
40
60
80
100
Deformation, microinches


Difference in Resilient Modulus When Load Duration is Varied
0.1 Seconds
0.05 Seconds
Resilient Modulus, ksi
2500
2000
1500
1000
500
0
AR 2000
AR 4000
Ar 8000
AC Grade


After Aging, Tested at 77F —
Same Sample Tested by
Three Technicians
Tech.1
Tech. 2
Tech. 3
Resilient
Modulus, ksi
1500
1250
1000
750
500
250
0
AR 1000
AR 2000
AR 4000
AR 8000
AC Grade


Historical Data For AR 4000
From 1985 - 1988
Resilient
Modulus, ksi
10000
1000
100
10
Lab Mix - Lab
Compact.
Field Mix - Lab
Compact.
R Sq. all > 90%
0
25
50
75
100
125
Temperature, F


Comparison of FMLC and Cores
10000
1000
100
Resilient Modulus, ksi
Field Mix - Lab Compact. AR 8000
Cores - District 1
R Sq. FMLC > 90%
R Sq. Cores = 58%
0
25
50
75
100
125
Temperature, F


AR 1000
AR 2000
AR 4000
AR 8000
Resilient Modulus,
Ksi
Samples Aged at 104F,
Tested at 77F
1500
1250
1000
750
500
250
0
0
20
40
60
Time, Days

Variations Due to Air Voids

Variations Due to the Process of Moisture Conditioning

Variations Due to Asphalt Content:

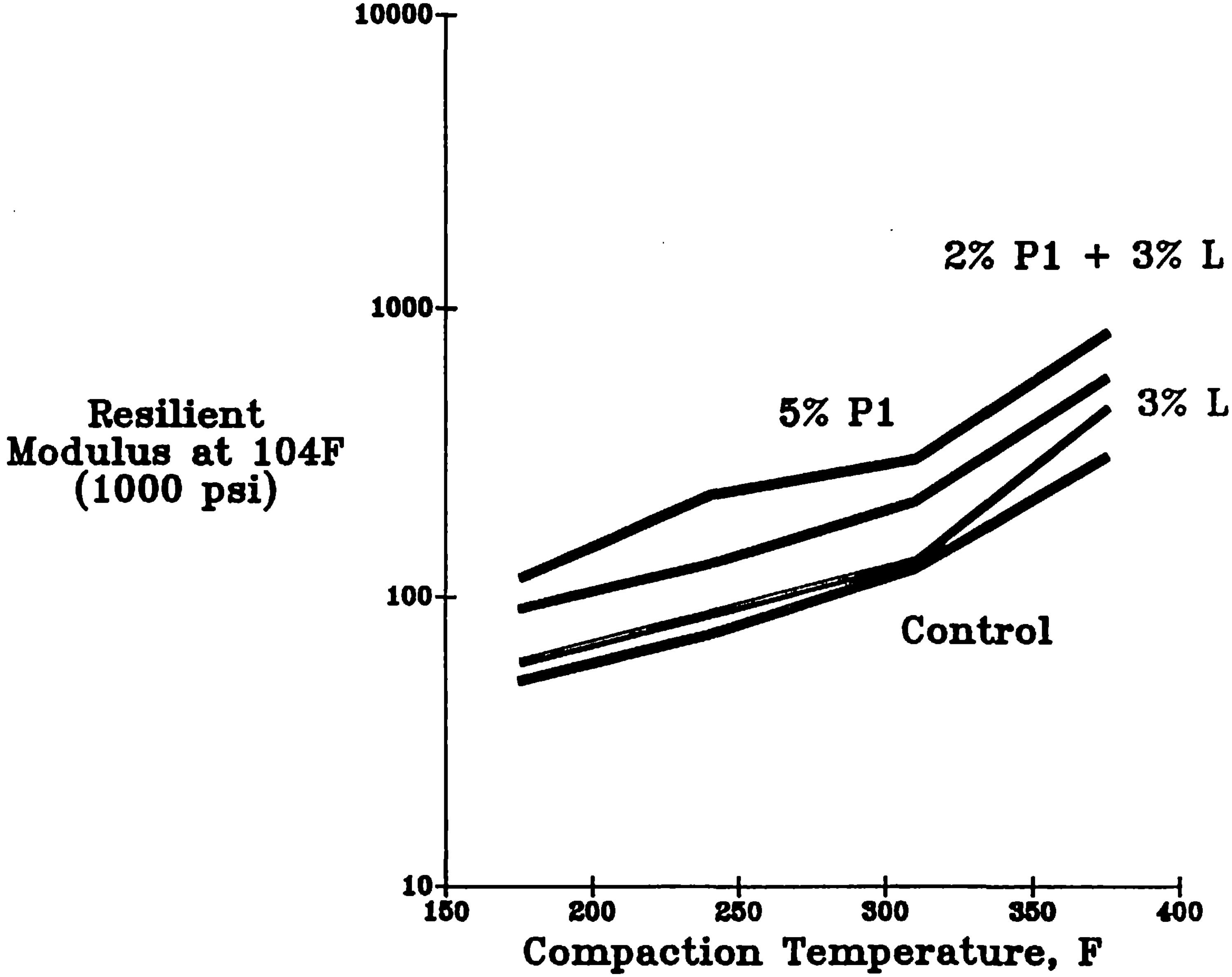
No Lime

Various Percentages of Lime

Variations Due to Modifiers:

Polyolefins

SBR Latex



10000
1000
100
10
Resilient
Modulus at 104F
(1000 psi)
2% P1 + 3% L
5% P1
3% L
Control
150
200
250
300
350
400
Compaction Temperature, F

Conclusions:

Mr at Diff. Temp. Varies with Method of Sampl Prep

Cores. vs. Fresh Mix Tend to Have Mr Softer at Cold Temp. and Stiffer at Warm Temp.

Mr Can Change with Aging

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A COMPREHENSIVE STUDY ON THE RESILIENT MODULUS OF SUBGRADE SOILS

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March, 1989

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2. INTRODUCTION

2.1 Background

Cyclic testing of subgrade soils has been used to investigate various roadway support problems. Some of these include liquefaction, seasonal strength variations, rutting and resiliency. This report deals with a test method for evaluating the latter.

The New York State Department of Transportation Soil Mechanics Bureau acquired its cyclic apparatus in the early 1980's for the purpose of testing Westway Project soils in New York City for possible liquefaction. Although Westway has been abandoned, the advent of the AASHTO pavement design guide provided an impetus for the SMB to develop its cyclic test system and a procedure to determine the resilient modulus (M_r) of subgrade soils. This task may become more pertinent should pavements be designed in the future in accordance with the AASHTO guide to design for the dynamic loading effects from heavy traffic.

The SMB's procedure is based on the AASHTO Test T 274-82, ("Standard Method of Test for Resilient Modulus of Subgrade Soils") modified to enable testing on the Bureau's equipment. This report describes the test development and the organization and results of a variability study.

2.2 Concept of Resilient Modulus

The resilient modulus parameter is a stress-strain relationship defined as:

$$M_r = \sigma_d / \epsilon_r \tag{1}$$

where σ_d = repeated deviator stress ($\sigma_d = \sigma_1 - \sigma_3$=repeated axial load/specimen area), and

ϵ_r = recoverable axial strain (ϵ_r = recoverable deflection/specimen height).

Simply stated, it is the repeated stress imposed on a soil specimen versus the strain recovered when the stress is released. It is dependent on soil type, soil stress history, test stress state, and soil moisture conditions.

The M_r is obtained by imposing 200 load-unload cycles on a specimen of known cross-sectional area, under a specific confining pressure. The load and the deflection recovered after the load is removed are measured at the 200th cycle and are used to compute the repeated stress and recoverable strain in equation (1). The deviator stress and/or the confining pressure are changed at every 200th cycle and the process is repeated. A M_r is computed for each stress state. Computation is illustrated in the diagram appearing in Figure 1.

Values could typically range from several thousand psi for a very soft or loose soil to 20,000 to 30,000-psi for a very stiff or dense soil.

2.3 Test Apparatus

The Soil Mechanics Laboratory (SML) cyclic loading test system includes the following pieces of equipment shown in Figure 2 through Figure 4.

1. The axial loading machine is Structural Behavior Engineering Laboratories' (SBEL) Model SDT-1000. The mechanics of operation are as follows: a solenoid valve, controlled by a timer, alternates the supply pressure to an air cylinder between on-load and off-load pressures. The air cylinder cyclically loads the specimen by pressing it against the machine's reaction frame. Separate valves control the rate of air flow entering and exhausting from the cylinder, allowing

shaping of the load vs. time wave. The timer controls the durations of load application and load release. Additional valves control the load applied during and between pulses. Loading capacity is from 0 to 1250 lb.

2. The triaxial cell is SBEL Model HX-100, which includes specimen platens of 1.4, 2.8 and 3.375 inches in diameter. During testing, the cell rests on the loading machine's platen. A ram extends from the top specimen cap through the cell top to the load cell mounted on the reaction frame. Confining pressure may be varied between 0 and 150 psi.

3. The load cell measures the load being applied to the specimen.

4. The SBEL Model RM 808-1 signal conditioning unit receives the load cell's electronic output, interfacing it with the recording system.

5. A linear motion potentiometer (LMP) strain transducer measures specimen deflections.

6. A Houston Instrument Microscribe 4521 strip chart recorder plots the load and deflection data from the load cell and LMP. This was purchased early in the trial testing to replace an XYY' plotter, the initial means of the cyclic system for recording load and deflection data. The plotter used paper having only a 15 in. long grid which could only record data from a small portion of the test. Constantly changing paper and reorienting pens proved very inconvenient and time consuming. The Microscribe recorder was chosen after investigating various brands and receiving several demonstrations. It is much better suited to the M_r test and saves about 1 hr of testing time per test.

7. Various other pieces of equipment are required for specimen preparation and conduct of the test, most of which were constructed in-house.

The SMB apparatus would lie relatively low on a scale for sophistication of cyclic equipment. Some systems have the capabilities of cyclic torsion, cyclically variable confining pressure, generation of multiple load-wave forms, tailoring of load vs. time waves to 0.01 sec and other capabilities that the SMB equipment cannot perform. With the exception of cyclic torsion, these capabilities can be pertinent to M_r testing. Costs range from about $20,000 to well over $300,000 with the SMB system at the low end. It should be noted, however, that some groups doing pavement related testing have obtained usable test results with medium to low cost or even homemade equipment. The more sophisticated high cost equipment generally was found to be used by organizations involved in very complex work such as earthquake studies.

2.4 Initial Experimentation And Testing

One of the first project objectives was to determine control settings on the loading machine that would achieve the deviator stresses required by AASHTO Test T 274-82. These stresses are listed in Table 1. This was accomplished in the following manner. A proving ring was mounted between the platen and the load cell in place of the triaxial cell. Pulsing was started and the load wave produced on the recorder was monitored. Control settings were varied until the load required to produce a specific stress for a 3.375 in. diam specimen was developed. Settings were found that would produce triangular and rectangular shaped waves at each stress level. The proving ring had a twofold purpose. First, it eliminated the need for setting up the numerous soil test specimens that would likely fail in such a trial and error process. Second, the proving ring, in the early experimentation, was used as a check for accuracy of the recorded load wave.

During the process of determining control settings, trials were made at different cyclic frequencies. The loading machine's timer has the capability of varying both the on-load and off-load times between 0.3 and 10.0 sec. However, when the on-load time was reduced to about 0.7 sec, the load wave recorded on the XYY' plotter became very erratic. Further reduction in time resulted in a straight-line load wave. These occurrences were significant in indicating the machine's limited ability to produce certain cyclic frequencies. Similar findings were made when the strip chart recorder was put into use; therefore all testing was done at a ½ Hz frequency with 1 sec on-load and off-load times.

Trial testing using natural soil specimens was next attempted, using the sequence of stress states specified by AASHTO. As can be seen in Table 1, the sequence is different for cohesive and cohesionless soils. However, in both cases a specimen conditioning phase precedes the data collection phase. The latter is the portion of the test where load and deflection data are recorded for use in computations. Specimen conditioning consists of subjecting the specimen to the range of deviator stress that will be imposed on it during the data phase. Its purpose is to eliminate strains from the M_r phase that would result from disturbances of sampling and specimen preparation and seating of the specimen caps.

The trial tests were run on both cohesive and cohesionless soils. The cohesionless specimens were prepared by mixing a fine sand to 5 percent moisture content, then compacting it with Standard Proctor effort in a mold fabricated from a 3.375 in. diameter thin wall undisturbed sampling tube. Cohesive specimens were taken from undisturbed sample tubes left over from a design project. Their visual descriptions were gray clay and, layered gray clay and silty clay. The soil was moist, had a firm consistency and was plastic.

Results were different for the two soil types. Cohesive specimens failed during specimen conditioning before any data could be obtained from the M_r phase. Testing of the sand progressed through conditioning without any visible signs of failure and with data always obtainable in the M_r phase.

3. DIFFERENCES BETWEEN AASHTO AND SMB TEST PROCEDURES AND APPARENT EFFECTS ON TEST RESULTS

Mechanical limitations of the equipment and results of the trial tests indicated modifications to the AASHTO procedure would be required to enable M_r testing on the SMB apparatus. The major changes are as follows:

1. Cyclic Frequency

A 0.1 sec load duration and a load release time ranging from 0.9 to 2.9 seconds are specified by the AASHTO procedure. Since the SMB equipment generated erratic load wave data for on-load times shorter than 0.7 sec, a load duration of 1.0 second was chosen for testing. The off-load time was also set at 1.0 sec and is within the specified range. The times are about as close to AASHTO specifications as the Bureau's system is capable. Indications from several sources are that this change will have no significant effect on test results. A paper by Allen and Thompson (4) dealing with resilient properties of granular materials, indicates that changes in M_r due to varying the pulse duration between 0.1 and 1.0 sec are small. Thompson emphasized this during a March 6, 1986 telephone conversation and mentioned that a similar trend exists for cohesive soils.

David Allen (1) indicates in a report concerning rutting in flexible pavements that cyclic frequency had no significant effects in his M_r testing on a dense graded aggregate. These authors refer to numerous other researchers whose testing has drawn similar conclusions for a variety of soil types.

2. Sequence of Stress States

AASHTO specifies that each test begin with a specimen conditioning phase preceding the data collection phase where load and deflection data are acquired. However, should too high a conditioning stress be applied, the specimen will fail prior to the data phase and no data can be obtained. Two methods can be used to avoid this. The first consists of obtaining strength data for the specimen. This information would be used to eliminate conditioning stress states that would likely cause failure. This method is not totally reliable however, since there has been little work done in estimating a soil's dynamic strength from routine triaxial tests. Professor Thompson uses UU (unconsolidated, undrained) triaxial tests, which yield conservative strength values to approximate a soil's dynamic strength. He concedes, however, that this provides only an estimate of the cyclic stress under which a specimen will fail, and is not foolproof. A specimen may fail at a lesser cyclic stress than the failure stress determined from a UU test.

Another drawback of this method is the increased cost and time required to perform the triaxial tests.

The second method involves rearranging the sequence of stress states to be used in testing. The sequence can be organized with a conditioning phase preceding the data phase at each deviator stress level. Using this method, a failure occurring during a conditioning phase does not affect data acquired from all previous data phases. Since this method requires no previous knowledge of the specimen's strength, it was used in design of the SMB procedure.

The sequence was further modified to include additional deviator stress levels. The AASHTO procedure specifies small deviator stress increases early in the test and larger ones as it progresses. The trial test specimens withstood the 1-, 2- and 4-psi levels but failed immediately at the 8-psi stress. Inclusion of additional intermediate deviator stress levels would allow the failure point to be approached in smaller increments, thereby obtaining more data from each test.

Table 2 shows the sequence used in the SMB procedure. A study by Allen and Thompson(3) indicates that reorganization of the specimen conditioning and M_r test phases and the additional stress levels incorporated into the SMB procedure would have no significant effect on the resilient modulus values.

3. Load Cell Location

The axial load imposed on the specimen is measured by an electronic load cell which is mounted on the crosshead of the loading machine's reaction frame. The AASHTO test method prefers mounting the load cell within the triaxial cell between the top cap and loading ram. Such positioning would require modifications to the triaxial cell to bring the electric leads outside to the recording equipment. An internal load cell would also have to be submersible if water is used for the confining fluid.

An internal load cell is available for the SMB's cyclic system from the manufacturer, but it would be mounted beneath the specimen. This location might allow the load data to be affected by inertial effects and is not addressed by AASHTO.

The AASHTO procedure does allow external load cells to be used if corrections are made for dynamic piston friction. To determine these corrections, the triaxial cell was cycled without a specimen at all test confining pressures with the piston in contact with the load cell. In all cases the friction amounted to a one pound load. However, this may not be an accurate indication of friction during actual testing. With the piston in contact with a specimen, it would slide a much shorter distance through the triaxial cell cover port.

The best way to determine corrections for friction would be to compare the outputs from load cells within and outside the triaxial cell. This is not possible with the Bureau's present apparatus. If, in fact, the friction during testing is approximately one pound, its effect on M_r values would be small.

4. Deformation Measuring Equipment

Axial deformation measuring equipment, specified by the AASHTO test consists of two linear variable differential transformers (LVDT's) wired so the average signal is recorded. Averaged signals result in more accurate deflection data. Soils with a maximum resilient modulus in excess of 15,000 psi are to have the LVDT's clamped directly to the specimen. This location would yield the most accurate data when extremely small deflections are measured. If the maximum modulus is less than 15,000 psi, the LVDT's may be clamped to the loading ram with the shafts connected to the triaxial cell.

The SMB's apparatus measures cyclic deflections by means of a single linear motion potentiometer (LMP) clamped to a post of the loading machine's reaction frame. An LVDT has a longer life, can be made with a finer resolution and better linearity and is considerably more expensive than an LMP. Mounting the measuring devices on the loading ram would automatically zero out deflections occurring within parts of the apparatus. Clamps for such mounting of the LMP are not included in the Bureau's apparatus but are available. However, this would only be a partial improvement at best. The problem with mounting one device to the loading ram is that measurements can be affected by any lateral movement of the ram which may be allowed by the small clearance between it and the port in the triaxial cell cover. Two LVDT's mounted 180° opposed, and wired for the average signal to be recorded, would be the most accurate method of measuring resilient deflections occurring only within the specimen.

5. Specimen Saturation

The AASHTO test contains a section for the specimen saturation procedure which is similar to this Bureau's for the consolidated undrained with pore pressure triaxial test. AASHTO does not specify that all specimens be saturated, and the SMB cyclic test system does not have this capability. However, if needed a saturation procedure incorporating back pressure and deaired water could be devised. Research by Thompson and Robnett (10) indicates that M_r values decrease with increasing percent saturation.

6. Specimen Compaction

The AASHTO test describes procedures for laboratory compaction of specimens by vibratory, kneading and static methods and contains a section explaining the selection of the appropriate method to simulate particular field conditions. The SMB does not use these compaction methods. All specimens are either undisturbed specimens, 3.375 in. in diameter or are compacted in a mold fabricated from a 3.375 in. diameter thin wall sampling tube in a manner similar to the Standard or Modified Proctor compaction test.

Professor Robert Elliot of the University of Arkansas at Fayetteville was contacted concerning M_r values obtained from specimens compacted by different methods. He stated in a October 28, 1986 telephone conversation that University research has shown the following: a) static compaction generally gives the highest but most variable resilient moduli. b) kneading yields the lowest and most consistent results, and c) M_r values and variability from specimens compacted by the Proctor methods fall in between those obtained from static and kneading compacted specimens but are closer to the kneading compaction results.

7. Length:Diameter Ratio

AASHTO specifies that the specimen length should not be less than two times the diameter. This condition is met by the SMB procedure with respect to cohesive specimens. The ratio of length to diameter for cohesionless specimens, however, is 1.93:1. The effect of this reduction is considered to be small since the ratio is very nearly 2:1.

4. DETERMINATION OF PROJECT GOALS

After a procedure was written, modifying AASHTO's test T 274-82 for SMB use, it was then deemed desirable to investigate the variability of test results which could be expected from the Bureau's system. In a November 27, 1985 memo, Lyndon H. Moore, then the Director of Technical Services, suggested that the SMB consider a parallel testing program involving the SMB and an outside organization such as Rensselaer Polytechnic Institute (RPI). RPI was mentioned because of its 10 year involvement with a cyclic program, its sophisticated testing equipment, and its proximity. As Mr. Moore also suggested, a Research Needs Statement was submitted to the Engineering Research and Development Bureau concerning the proposed program.

Meetings with personnel from the SMB, Research Bureau and the Pavement Management Section were held to discuss the feasibility of such a program. Some of the points expressed were as follows:

1. The SMB's test procedure is an adaptation of the AASHTO test which is a complex procedure. Obtaining good data can be difficult since many sensitive factors can influence test results. For example, a one percent variation in a silty specimen's moisture content could change the M_r by 1,500 psi.

2. Putting laboratory M_r values to use in design work using AASHTO's pavement design guide is a very complex, sometimes confusing matter.

3. Due to a lack of experience with this type of design and an unfamiliarity with AASHTO's methods, no one knew if employing M_r related design procedures would result in significant savings. Analyses were needed in this area.

4. The cost of the proposed parallel testing program could be as high as $50,000.

The following decisions were made to give some short term direction to the SMB's M_r work:

1. A sensitivity analysis would be done to investigate the likelihood of realizing significant savings by employing the M_r related AASHTO pavement design procedures.

2. Due to the uncertainty of the Bureau's future involvement in M_r testing, the potentially high cost SMB-RPI research project would be deferred.

3. An in-house study would be undertaken to investigate the variability of M_r test results obtained using the SMB's procedure and apparatus.

5. ORGANIZATION OF THE VARIABILITY STUDY

5.1 Objectives

The objectives of the SMB variability study and how they were met include the following:

1. The study was to examine the effects of deviator stress and confining pressure on M_r values. This was done with a two way analysis of variance (ANOVA) using the STATGRAPHICS computer program.

   Three cases were examined. First, an analysis was done using all M_r data. A second analysis was performed ignoring M_r values for deviator stress levels of 1-psi. The third case ignored M_r values for 1- and 2-psi deviator stress levels.

2. The project was to determine if the resilient modulus can be correlated with deviator stress and confining pressure and what the relationships are if correlations do exist. This objective was met by performing simple regressions using the STATGRAPHICS program. The same three cases mentioned above were examined.

3. A third objective was to determine if certain stress states of the test procedure yield more variable M_r results than others. This was accomplished by computing the certainty (i.e., confidence level) with which the mean M_r values of future SMB testing will lie within a pre-selected

range (i.e., confidence interval). The interval was set at the mean M_r value obtained for each stress state of the study testing, plus and minus 15 percent. This was the range used by the Engineering Research Bureau in their cyclic testing on asphalt pavements. A confidence level was calculated for each stress state since the resilient modulus is a stress dependent parameter. The t distribution was used in these calculations because the concern was the mean of a small sample.

5.2 Constituent Study Details

Based on consultation with Professor Dimitri Grivas of RPI, 10 tests were run on each soil type included in the study. Since procedures allowed convenient preparation of eleven cohesive specimens, twenty one tests were run; ten on a silty fine sand and eleven on a commercially available clay. The sand is mapped by the Soil Conservation Service as Colonie Sand and was obtained locally from the Pine Bush area of Albany. The clay, marketed as Millers Clay, was purchased from Northeast Ceramic Supply of Troy, N.Y. Specific gravity, Atterberg limits and hydrometer tests on soil from each batch showed the clay to be a homogeneous material. Results of index tests appear in Table 3.

The majority of test results were hand calculated while software was being developed. Calculations for the last few tests were performed by the RESMOD program, written in-house by Kevin H. Gary of the Soil Mechanics Laboratory. Required input data include soil type, specimen dimensions, weights, heights of load and deflection waves and strip chart recorder scale voltages for each wave.

The program presents test results in the typical form of M_r vs. deviator stress for each confining pressure for cohesive specimens, or log of M_r vs. log of the sum of the principle stresses for cohesionless soils. RESMOD also calculates coefficients of determination for correlation purposes and regression constants for cohesionless testing. Samples of the RESMOND plots are shown in Figures 5 and 6. A hard copy of the program is available from the SML.

Other details concerning the variability study are as follows:

Sine, haversine, rectangular and triangular shaped load waves are allowed by the AASHTO test. The SMB apparatus is capable of triangular or rectangular shapes. The former was used in the study due to much lower air demands. The SML compressor was unable to sustain rectangular load waves throughout the test.

As was previously mentioned, no corrections were made to account for ram friction since they could not be determined with confidence. This, however, should have only a small effect on M_r values and should not affect the study since any correction would be identical from test to test.

One technician performed all testing to further reduce causes of variability.

5.3 Specimen Preparation

All specimens were prepared using the following procedures to reduce specimen variability.

5.3.1. Cohesionless Specimens

The sand was thoroughly mixed in a large tare to ensure a homogeneous source, and allowed to air dry for several weeks. For each specimen 2,500 grams of sand were taken from the tare, thoroughly mixed with 125 cc of water and allowed to cure for several hours. This brought the soil to a 5 percent moisture content at which specimens remained intact after extrusion from the compaction mold. Excess soil remaining after specimen preparation was tested to check the cured moisture content. In every case, results were very nearly 5 percent.

Test specimens were compacted in a manner similar to the Standard Proctor compaction test in a mold fabricated from a Shelby tube. The Shelby mold makes specimens 3.375 in. in diameter, the largest the test apparatus will accept. Compactive effort was 12,421 ft-lb/ft^3. After compaction, each specimen was extruded, set up in the triaxial cell and tested immediately using the SMB procedure.

5.3.2 Cohesive Specimens

The Millers Clay was cut into small chunks and placed in a large container. By adding water and mixing with an electric drill, the clay was whipped into a slurry at a 75 percent moisture content. This content allowed pouring the slurry into consolidation equipment. A curing period of at least two weeks was allowed to soften any lumps to the slurry consistency.

Consolidation was necessary to densify the slurry to a firm consistency suitable for manufacturing specimens and was accomplished in the following manner. The slurry was poured into a split open-bottom container lined with a heavy fabric to expedite water drainage. Layers of fabric were also placed under the container and between the slurry and loading plate. The assembly was placed in a heavy frame and loads were applied incrementally by an air cylinder. Consolidation was monitored by a Hewlett-Packard computer which received data from an electronic strain transducer. The CONREAD program, required for acquisition and plotting of consolidation data, was written in-house. For each load, beginning at 62.5 psf, the computer plotted consolidation vs. the square root of elapsed time. This was used to determine when 90 percent of primary consolidation for each load increment was complete. At this point, the load was doubled and the process repeated until the normal stress reached 8,000 psf, the highest preload pressure that could safely be obtained in the laboratory. After 90 percent consolidation was achieved at the last increment, the load was maintained for an additional 21 days.

Shelby tubes were then pressed into the clay using either the SML's tensile test machine or a pneumatic cylinder. The tubes were sealed at both ends using molten wax and plastic caps, and then stored in SML's fog room until testing to prevent drying.

Consolidation testing was done on specimens taken from the top and bottom of each tube. The middle portion was used for cyclic testing. Preconsolidation pressures (Pp) determined from testing for the top specimens ranged mostly between 6,000 and 7,000 psf. Bottom-of-tube specimens generally showed Pp's between 4,500 and 6,000 psf. These Pp's are considered less accurate since the bottom of the tube is susceptible to increased disturbance.

6. RESULTS

6.1. Experimental

6.1.1. M_r Values

M_r values ranged from about 6400 to 27,200-psi for the sand testing and 1200 to 21,700 for clay testing. Average values were about 13,600 for sand and 5,500 for clay.

Certain observations were made when reviewing the data arranged in the form of Tables 6 and 7. Table 6 lists M_r data from sand testing at each stress state. It seems that when deviator stress is kept constant, M_r values vary substantially over the range of confining pressures. However, when confining pressure is kept constant and deviator stress is varied, the range of M_r values encountered is not nearly as large.

Table 7 lists M_r data from clay testing in the same form. The above trends of sand testing appear reversed with regards to clay. Varying deviator stress at constant confining pressure appears to have a significant effect on M_r results for clay testing. Varying confining pressure at constant deviator stress does not produce as great a range of M_r values as with the sand testing.

6.1.2. Apparatus

One occurrence which went unexplained is the final moisture content being greater than the initial generally by 0.5 percent to 2.5 percent for all 10 sand specimens. This continued to be the case during the sand testing in spite of the care taken to check for and prevent leaks. To find how leakage occurred, a dummy specimen was set up with a vacuum introduced inside the membrane and a confining pressure of 50-psi applied to the outside. Although these conditions were more severe than those during testing, no bubbles were noticed in the bubble chamber during an hour of monitoring.

Another observation made a number of times throughout the clay testing phase of the study, involved specimen deflections at various confining pressures. For the majority of cases, specimen deflections would increase as confining pressure decreased at a particular deviator stress level. However, there were a number of instances where deflections actually decreased under these conditions. This occurrence was attributed to the test system's deflection measuring equipment and will be discussed in a latter section of this report.

As the cyclic project progressed it became evident that certain aspects of the apparatus could cause erroneous data to be obtained unless dealt with carefully. Several points are discussed below.

1. The LMP has a "dead zone" at both ends of the shaft travel. While there is still room for travel in these zones, no output signal is generated. If sufficient permanent strain occurs to the specimen during the course of the test some portion of the shaft's cyclic movement may occur within these zones. The effect is to chop off the top or bottom of the deflection wave. Occurrence of this condition is obvious by the shape of the deflection wave, but will cause M_r data to be lost.

2. When testing cohesionless specimens, decreasing the deviator stress from a high level to the 1-psi level of the next confining pressure (see Table 2), could easily result in reducing the between-pulse load to zero. If this happens, the triaxial cell with the specimen will drop down from its floating between-pulse position. Thus, contact between the specimen and reaction frame is lost. The next pulse, however, will accelerate the specimen upwards impacting it with the frame. A greatly exaggerated deflection will be recorded. This condition is also obvious by reviewing the strip chart output, but M_r data again, may be lost.

3. The axial loading machine has several controls which together adjust the magnitude and shape of the load wave. However, a particular combination of control settings will not indefinitely produce the same wave. Continuous attention must be given to the strip chart output to adjust the load as needed.

6.2 Statistical Analysis

6.2.1 Confidence Levels

The analyses which calculated the confidence with which mean M_r values, at each stress state, will fall within a specific range, yielded different results for the two materials tested. Results of these analyses are summarized in Tables 4 and 5. As seen in Table 4, the confidence levels of sand testing are relatively low for the 1- and 2-psi deviator stresses with an average of 79%. Generally they are higher at deviator stresses of 5 psi and higher with an average of 96%. This is the same basic trend found by Allen of the KTRP in his cyclic testing as described in a telephone conversation. The KTRP's axial loading machine is the same as that owned by the SMB. Numerous other groups contacted also reported a larger scatter of data at lower stresses.

The analysis was repeated at the lower deviator stresses for both sand and clay testing after close scrutiny of the load and deflection waves from the strip chart recorder. Results obtained, after any possible erroneous data were eliminated, still indicate lower confidence levels at lower stresses.

Analysis of clay testing resulted in different trends than those of the sand testing. Table 5 shows that confidence levels remained relatively low at the 1- and 2-psi deviator stresses, and greatly increased at 3-psi. However, instead of slowly increasing or remaining fairly high, they dropped sharply at the 4- and 5-psi stresses. A small increase was seen at the 6-psi level. However, this is insufficient evidence to assume that the trend is reversing and is beginning to parallel that of the sand testing.

Results of the analysis were also reviewed by Peter Bellair of the Research Bureau's Geotechnics Section, who has a greater familiarity and involvement with statistical analysis methods. Bellair agreed with the comments on variability.

Based on results of these analyses, further analyses were performed on data from sand testing only.

Discussion:

A search for causes of the large scatter of clay test results began with a recheck of the analysis calculations. No errors were found. Load and deflection data and M_r calculations had been previously reviewed.

To examine the effect of the variation in preload pressures, average M_r values at each deviator stress were plotted vs. Pp values obtained from the consolidation test on the specimen taken from the top of each tube. This plot appears in Figure 7. As is seen, the M_r values do not seem to be affected by the fairly small range of Pp.

Mechanical factors were investigated next with the most likely cause being the LMP deflection transducer. Prior to the study, its accuracy was checked by comparing its output against a dial indicator when the plungers of both instruments were moved an equal amount. At that time, the LMP was consistent and closely agreed with the dial indicator. This comparison was repeated after the study using a depth micrometer with a resolution equal to the indicator (0.0001 in.). The LMP output was then inconsistent and erroneous. Also, equal deflections with the LMP plunger in different positions (i.e., extended or depressed) gave different output. In some cases, the difference was as much as 25 percent of the larger recorded deflection.

These facts point to the LMP as the cause of lower confidence levels for the clay testing. The relatively good results of the sand testing may be explained by how the LMP works. Randy Eller of Bourns Instruments Inc., manufacturer of the device, explained that the steel plunger shaft slides along a conductive plastic element, thus creating an output signal. Steel in moving contact with plastic means that the device will eventually wear out. After describing the SMB testing and accuracy checks on the LMP, Eller said he would strongly suspect that its useful life ended between the sand and clay phases of the study.

The cyclic apparatus load cell was checked against another and was in close agreement and was consistent. Thus, the load waves from the strip chart recorder were assumed to be accurate.

6.2.2. ANOVA

Analysis of variance results are summarized for sand testing in Table 8. The F-ratios listed are the analysis F-statistics which are calculated for the factors (confining pressure and deviator stress) whose effects on M_r were examined.

If the factor's calculated f-ratio is greater than the statistic taken from the F-distribution tables, the factor is considered to have an effect on the dependent variable (M_r).

The analysis was performed in the following three ways:

1. using all M_r data available from the sand testing phase,
2. ignoring M_r values from the 1-psi deviator stress level, and
3. ignoring M_r values from the 1- and 2-psi deviator stress levels.

These three approaches were taken since the analysis described in the previous section indicated the possibility that M_r test results of the sand testing are more variable at the lower deviator stress levels.

In all cases, the calculated F-ratios for the effects of confining pressure on M_r were much greater than the tabulated F-statistics for a 99 percent confidence level. However, F-ratios for the effects of deviator stress on M_r and the effects of the interaction of deviator stress and confining pressure on M_r were lower than the tabulated values, even at a 75 percent confidence level.

Testing by Seed, Mitry, Monismith and Chan (7) on a uniform sand (Monterey Sand) indicates that confining pressure had a greater effect on the soil's M_r values than did deviator stress.

6.2.3 Correlation and Simple Regression

Correlation and simple regression analyses were done on sand testing data using linear ($Y = aX+b$) and multiplicative ($Y=aX^b$) models for the three cases described in the previous section of this report. Results are summarized in Table 9. M_r appears uncorrelated with deviator stress with an average correlation coefficient of 0.18. M_r has somewhat better correlation with confining pressure having an average coefficient of 0.62.

Regression constants of the SMB sand testing were roughly 2/3 those of the testing done on Monterey Sand.

7. SUMMARY AND CONCLUSIONS

A test procedure based on AASHTO Test T 274-82 was developed for use with the SMB's cyclic apparatus to obtain the resilient modulus of subgrade soils. Deviations from the AASHTO test concerning cyclic frequency, stress state sequence, load and deflection measuring equipment, specimen saturation and compaction procedures and the specimen aspect ratio were required to enable testing on the Bureau's system. Possible effects of the modifications were explored and were expected to be minimal.

A study was conducted to examine the variability of test results. Procedures were developed to manufacture homogeneous sand and clay test specimens for the study. Analyses were performed to:

1. determine variability of test results at each stress state,

2. examine the effects of confining pressure and deviator stress on M_r values and

3. determine correlation coefficients and regression constants for relationships of M_r and confining pressure and M_r and deviator stress.

The latter two analyses were performed on data from the sand testing phase only because of mechanical problems occurring during the clay testing phase.

The following conclusions are made, based on data generated during the SMB M_r variability testing and analyses.

1. The complexity of the SMB resilient modulus test procedure and equipment will require skill and experience from the technician to obtain good test results.

2. Periodic calibration checks should be made to ensure the accuracy of the load and deflection, measurement and recording equipment.

3. Based on the sand testing phase of the study, greater variability of M_r values can be expected at deviator stress levels lower than 5-psi than at levels of 5-psi or higher. This is the same basic trend found by others using the same axial loading machine as the SMB.

4. Analysis of variance and correlation analyses indicate that confining pressure has greater effects on M_r than does deviator stress. Testing by others shows similar findings.

5. Regression constants of SMB sand testing using the multiplicative model are somewhat lower than constants generated by other groups testing.

8. RECOMMENDATIONS

The following recommendations are expected to reduce M_r variability and increase efficiency of the SMB cyclic test system:

1. The present LMP should be replaced before any further testing is done. At the very least, another LMP should be purchased. Further upgrading would require two such devices, the necessary electronic hardware for signal averaging and a clamp for mounting on the loading piston. The approximate total cost is $1000. To avoid periodic replacement of LMP's at $200 each and to achieve greater accuracy, LVDT's may be considered as an alternative. Although the approximate cost is $5000, LVDT's may be the best choice if cyclic testing programs are to be continued.*

2. After the problem concerning strain transducers is resolved, further testing may be advisable to confirm the system's repeatability. Specimens should be prepared to last further into the test before failing to obtain a wider range of data. Sand specimens should be compacted to the Modified Proctor effort. The clay can be preloaded to higher pressures if equipment modifications are made. The latter would absolutely require certain added safety precautions.

3. The 1- and 2-psi deviator stresses should be eliminated from production testing unless future studies show an increase in their repeatability and the necessity is demonstrated for data at these stress levels.

4. Consideration should be given to the purchase of an electronic data acquisition system if cyclic testing is to become a routine SMB procedure. With such a system, signals from the load cell and strain transducers are

* Shortly before publication of this report, two LVDT's were purchased to implement recommendation no. 1.

stored on a computer disk. Such an upgrade to the apparatus should increase both accuracy and efficiency. The University of Maine at Orono went this route at a cost of about $5000 which included an Apple McIntosh computer, signal interface, digital output and software. Systems for Hewlett-Packard computers may be less expensive since the SML already has an H.P. computer. However, frequent routine M_r testing would require a second computer since the first already has extensive use.

The Kentucky Transportation Research Program has automated their cyclic apparatus with an IBM PC XT Computer. Their costs were much less since the computer was already on hand, and all software was written in-house. David Allen of the KTRP feels that automating the system was a definite improvement. The loading machines of the KTRP and the University of Maine are the same as the SMB's.

5. Once the repeatability is established, a study to determine the accuracy of test results may be desirable. This would probably be similar to the SMB-RPI parallel program which had been proposed. An alternate and less expensive approach would involve performing tests on several materials having known elastic moduli in the range of 5000 to 25,000-psi.

6. A study to determine if the number of loading cycles per stress state may be reduced would be very worthwhile if cyclic testing programs are continued. Currently, specimens are tested with 200 cycles per stress state. The University of Arkansas at Fayetteville is doing work to investigate the possibility of cutting the cycling from 200 to 50. If this were possible for New York soils, testing time would be greatly reduced and productivity increased.

7. The minimum specimen diameter should be 3.375 in. Reduction of this diameter would likely decrease repeatability of the mid-range deviator

stresses. This effect is apparently characteristic of M_r test systems in general.

8. Specimen caps of 4 in. diam may be desirable for future production on Proctor mold specimens. These can be fabricated by the Materials Bureau machine shop.

9. ACKNOWLEDGMENTS

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Gorril, W., Professor of Civil Engineering, University of Maine, Orono.

Peters, A.J., Materials Engineer, Washington State Department of Transportation.

Sandford, T., Professor of Civil Engineering, University of Maine, Orono.

Schulze, D., Laboratory Director, Goldberg-Zoino Associates, Newton, Mass.

Thompson, M.R., Professor of Civil Engineering, University of Illinois, Urbana.

Zimmie, T.F., Associate Professor Of Civil Engineering, Rensselaer Polytechnic Institute, Troy, NY.

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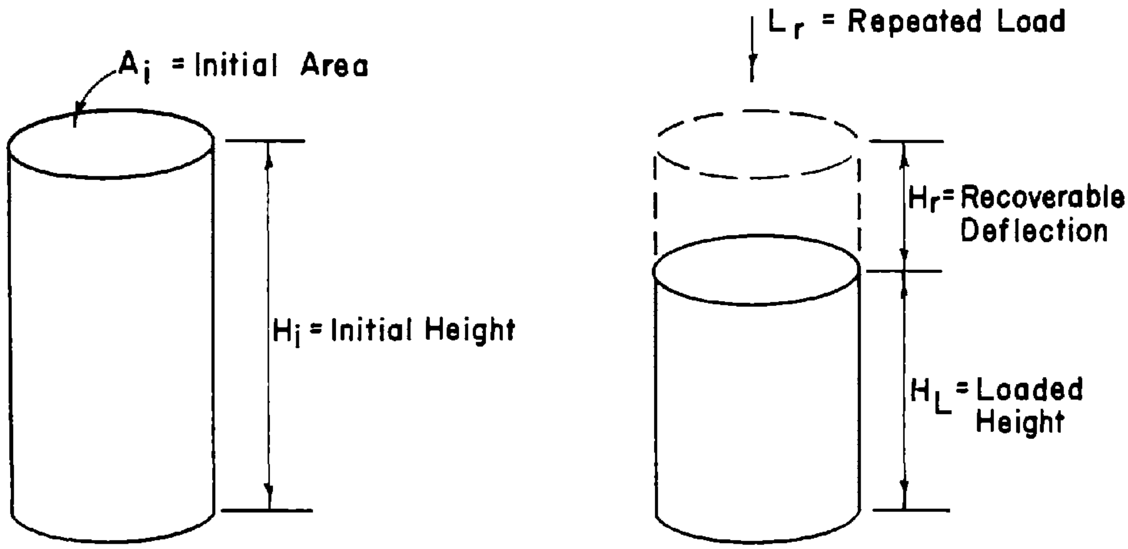
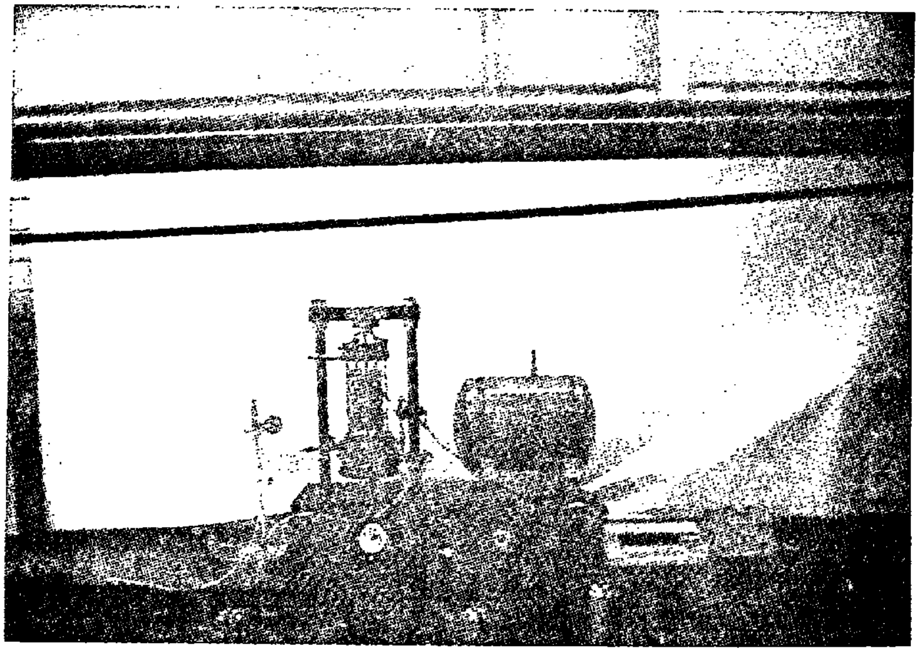
DEFINITION OF RESILIENT MODULUS



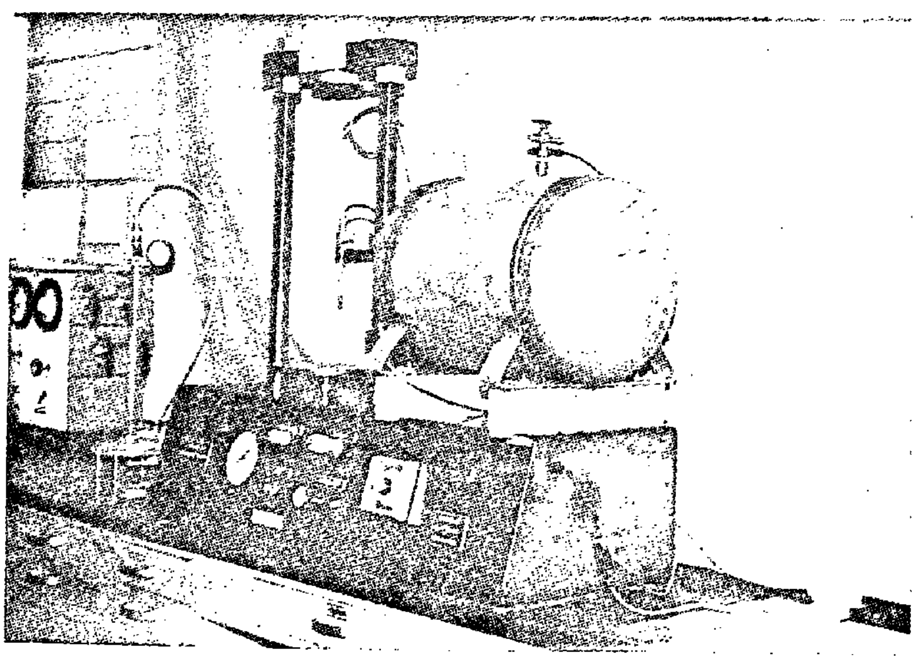
$$\frac{L_r}{A_i} = \sigma_r = \text{Repeated Deviator Stress}$$

$$\frac{H_r}{H_i} = \epsilon_r = \text{Recoverable Strain}$$

$$\text{Resilient Modulus} = M_r = \frac{\sigma_r}{\epsilon_r}$$

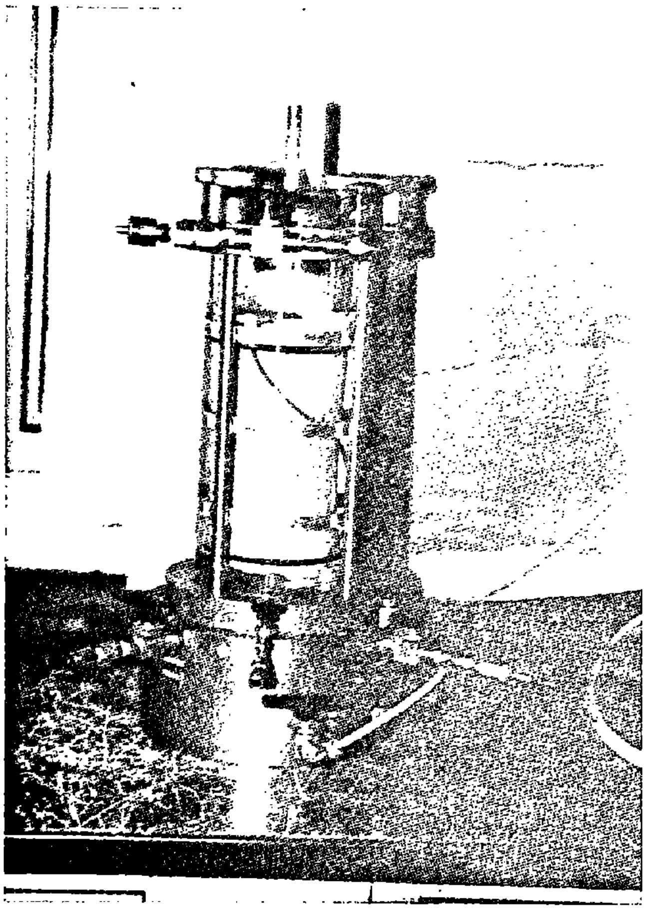


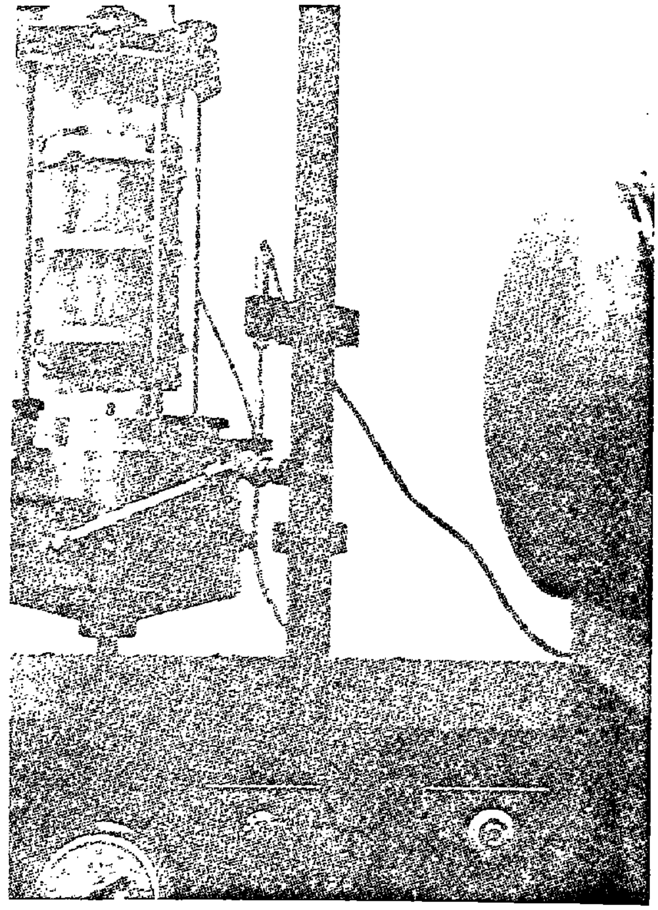
Entire test system.



Axial loading machine with air supply tank.

Figure 2





Above left: Triaxial cell with specimen and loading ram.
Above right: Linear motion potentiometer mounted on reaction frame.
Below: Load cell mounted on reaction frame crosshead.

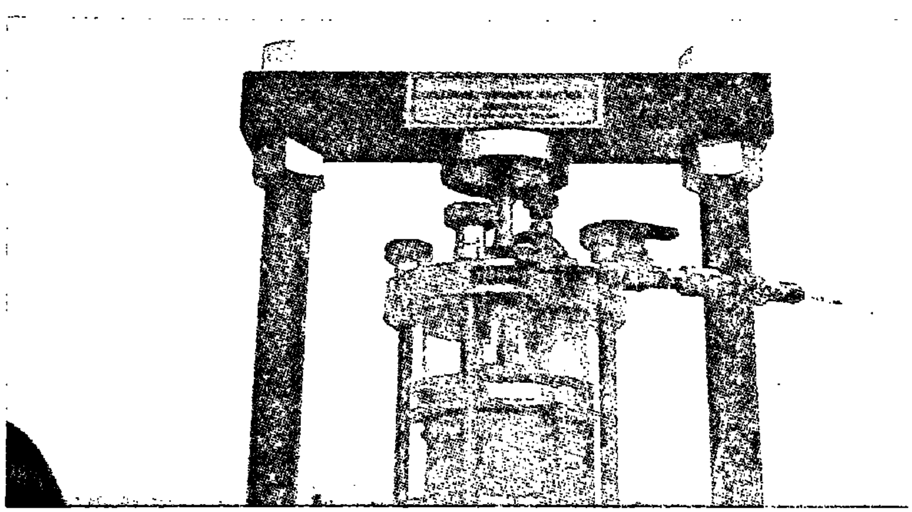
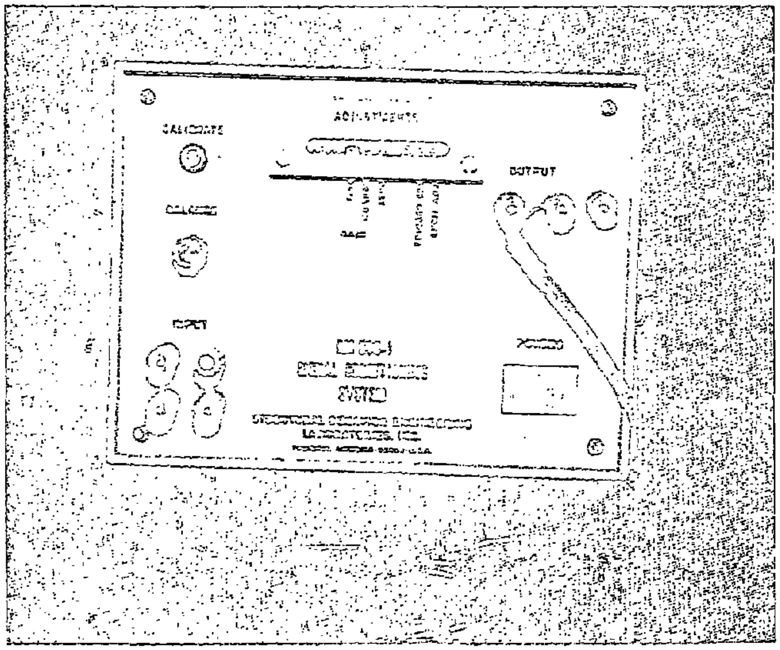
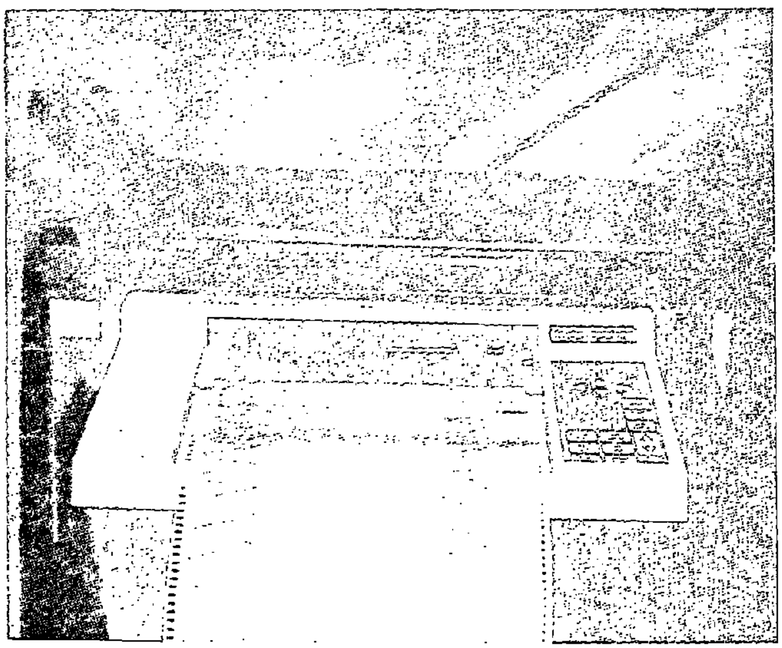
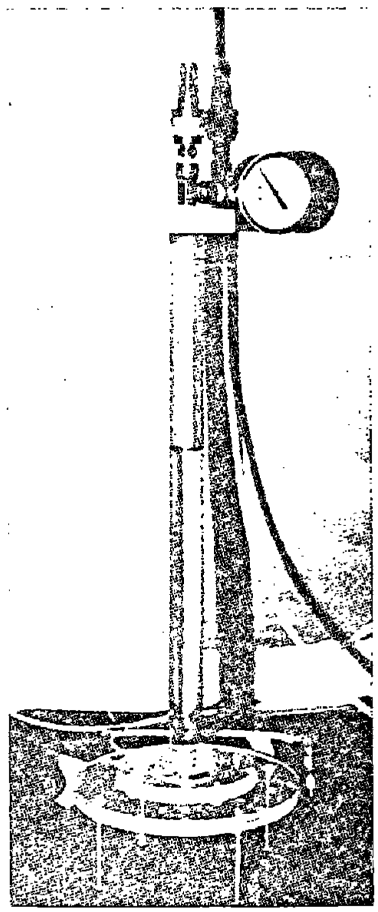


Figure 3



Left: Signal conditioner





Above: Strip chart recorder.

Right: Bubble chamber.

Figure 4

PIN : E104.04.701.54

HOLE # : COL. SAND

TUBE # : S-2

DEPTH (ft) : 0

TESTCODE : 0001

DATE : 1/6/87

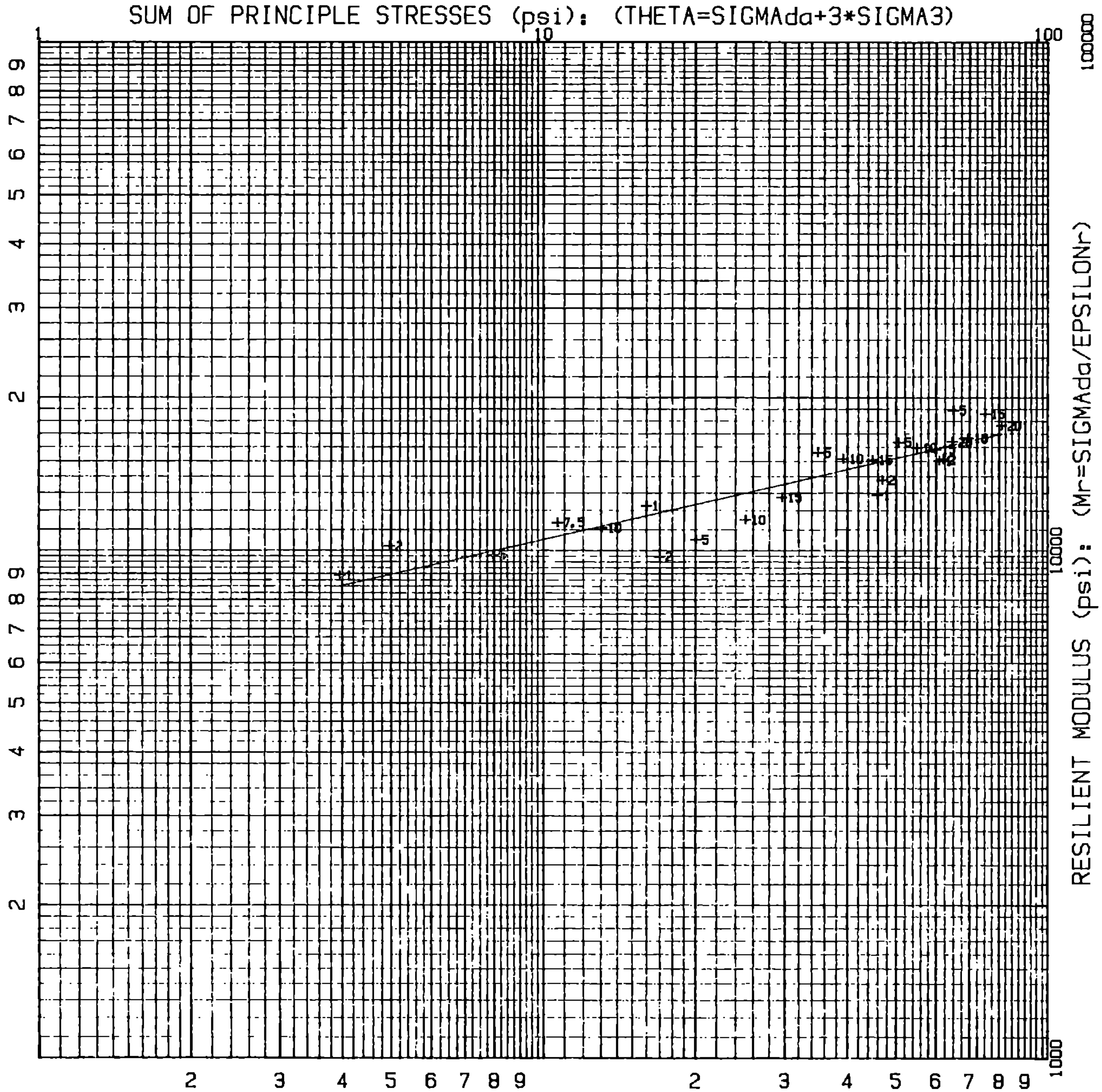
NUMBERS TO THE RIGHT OF +
ARE NOMINAL DEVIATOR
STRESSES (SIGMAdn) (psi).

Mr=a*THETA^b

POWER CURVE REGRESSION FIT
LINES DRAWN FOR :
OVERALL

REMARKS : ____________________

APPROVAL ____________________



Typical RESMOD plot for cohesionless specimens.
Figure 5

PIN : E104.04.701.54

HOLE # : BATCH-7

TUBE # : T-1

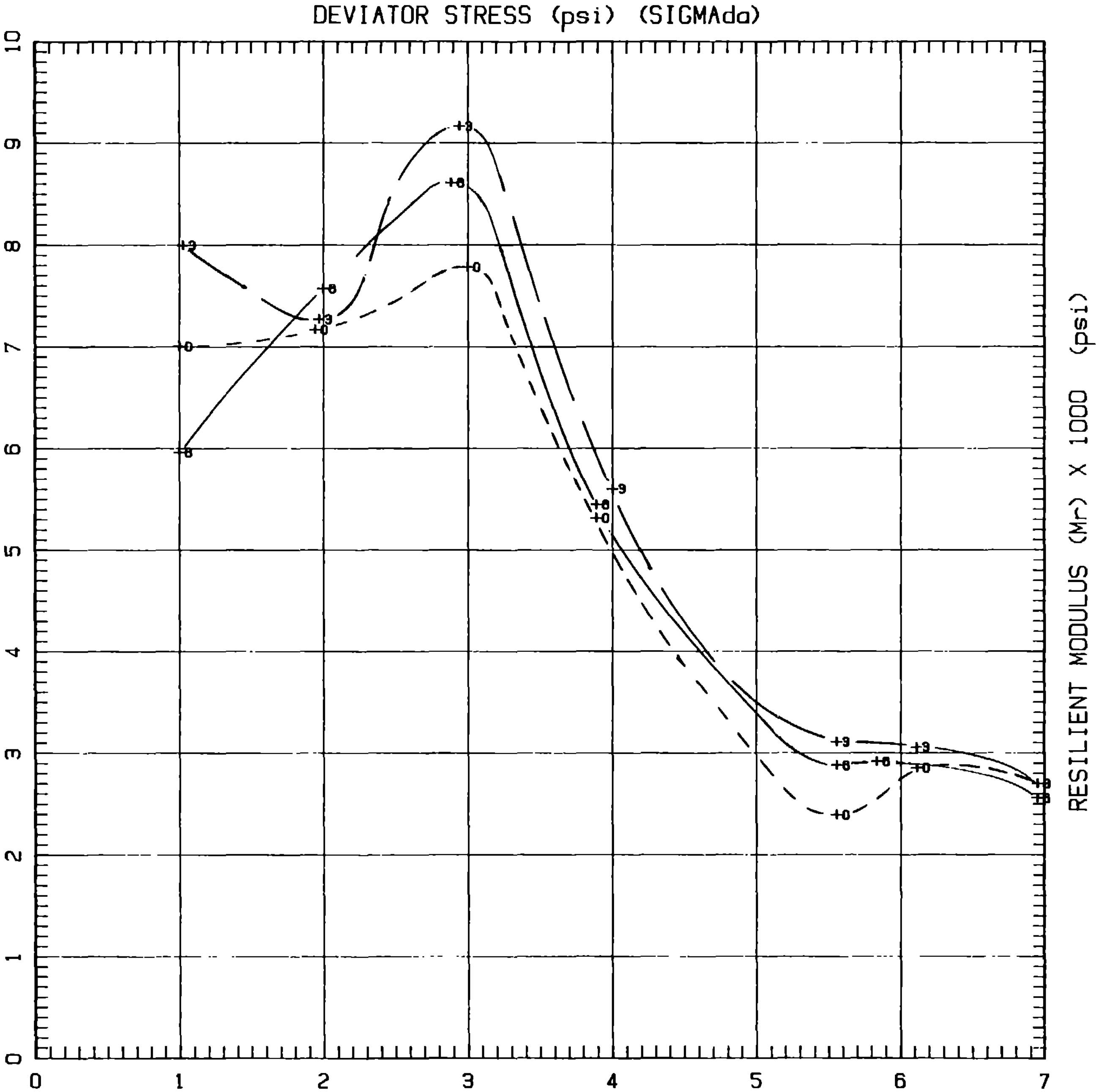
DEPTH (ft) : .4

TESTCODE : 0002

DATE : 12/05/86

NUMBERS TO THE RIGHT OF +
ARE CONFINING PRESSURES (psi).

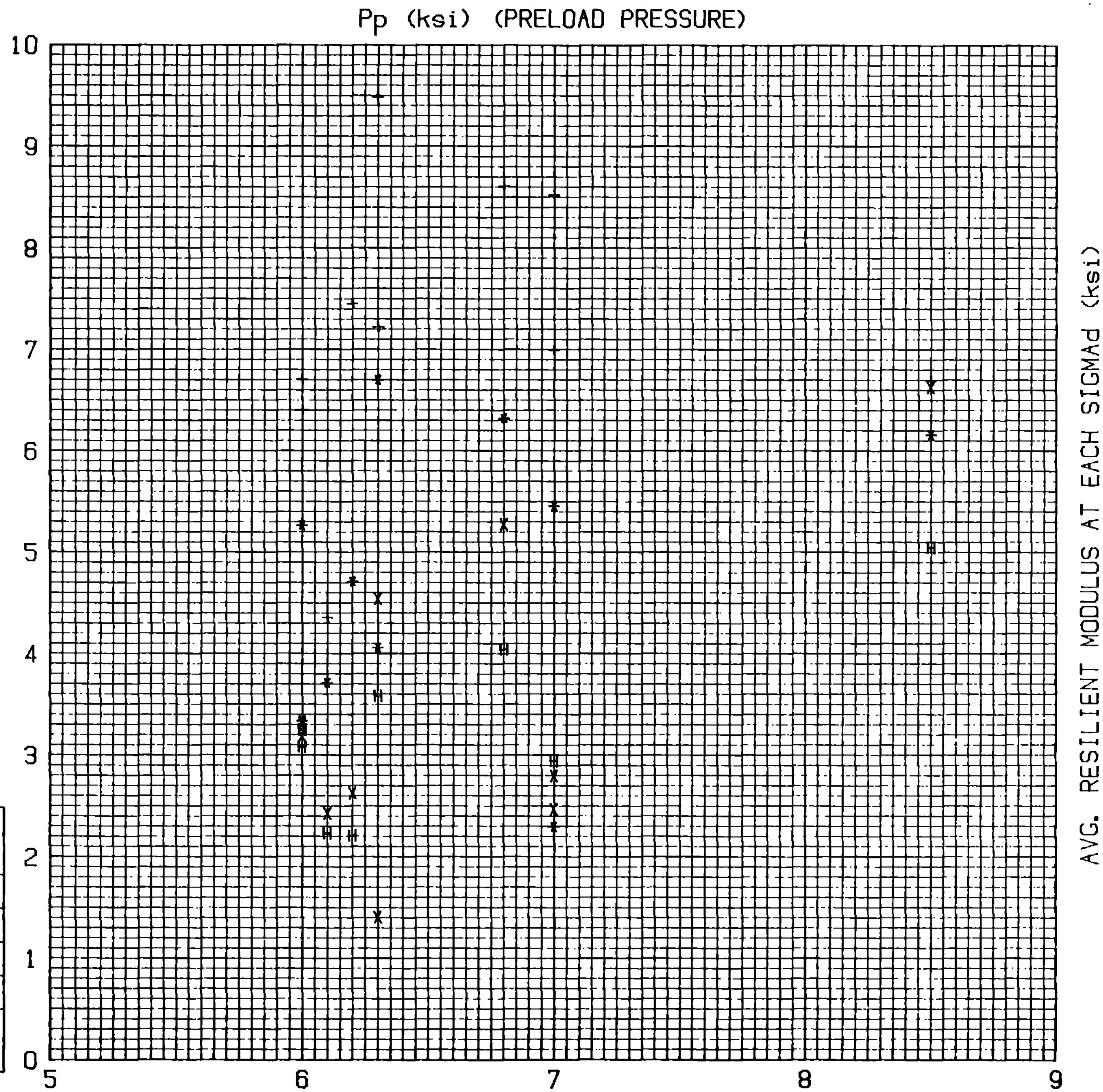
REMARKS : ______________________



Typical RESMOD plot for cohesive specimens.
Figure 6

FIGURE 7

Preload Pressure vs.
Resilient Modulus for
testing on Millers Clay



LEGEND

SYMBOL	SIGMAd
+	3
*	4
X	5
H	6

TABLE 1

STRESS STATE SEQUENCES FOR AASHTO TEST T 274

Cohesive Specimens		
Phase	Deviator Stress (psi)	Confining Pressure (psi)
*SC	1 2 4 8 10	6
**DC	1	6 3 0
	2	6 3 0
	4	6 3 0
	8	6 3 0
	10	6 3 0

* SC - Specimen Conditioning
* * DC - Data Collection

Cohesionless Specimens		
Phase	Deviator Stress (psi)	Confining Pressure (psi)
SC	5 10 10 15 15 20	5 5 10 10 15 15
DC	1 2 5 10 15 20	20
	1 2 5 10 15 20	15
	1 2 5 10 15	10
	1 2 5 10 15	5
	1 2 5 7.5 10	1

TABLE 2
STRESS STATE SEQUENCES FOR SMB TEST PROCEDURE

Cohesive Specimens		
Phase	Deviator Stress (psi)	Confining Pressure (psi)
* SC	1	6
** DC		6
		3
		0
SC	2	6
DC		6
		3
		0
SC	3	6
DC		6
		3
		0
SC	4	6
DC		6
		3
		0
SC	5	6
DC		6
		3
		0
SC	6	6
DC		6
		3
		0
SC	7	6
DC		6
		3
		0
SC	8	6
DC		6
		3
		0
SC	9	6
DC		6
		3
		0
SC	10	6
DC		6
		3
		0
SC	11	6
DC		6
		3
		0
SC	12	6
DC		6
		3
		0

Cohesionless Specimens		
Phase	Deviator Stress (psi)	Confining Pressure (psi)
SC	5	5
	10	5
	10	10
	15	10
	15	15
	20	15
DC	1	20
	2	
	5	
	10	
	15	
	20	
	1	15
	2	
	5	
	10	
	15	
	20	
	1	10
	2	
	5	
	10	
	15	
	1	5
	2	
	5	
	10	
	15	
	1	1
	2	
	5	
	7.5	
	10	

* SC - Specimen Conditioning
** DC - Data Collection

TABLE 3

COHESIVE SPECIMEN INDEX TESTING SUMMARY

Batch	Tube	M_T Test No.	(1) Specific Gravity Depth (ft.)	(1) Specific Gravity G_s	(2) Atterberg Limits PL	(2) Atterberg Limits LL	(2) Atterberg Limits PI	Hydrometer % Finer .002 mm
3	1	1	.05	2.68	18.4	37.8	19.4	50.5
			.8	2.68				
	2	2	.05	2.68				
			.8	2.68				
	3	3	.05	2.68				
			.8	2.67				
4	4	4	.05	2.68	18.1	37.2	19.1	47.1
			.8	2.68				
	5	5	.05	2.67				
			.8	2.67				
	6	6	.05	2.68				
			.85	2.68				
6	1	7	.05	2.68	18.1	38.2	20.1	——
	2	8	.05	2.69				
	3	9	.05	2.68				
7	1	10	.1	2.68				
			.9	2.68				
	2	11	.1	2.67				
			.9	2.68				
	3	——	.1	2.67				
			.3	2.67	18.5	39.3	20.8	50.5
			.6	2.68	18.3	40.4	22.1	50.0
			.9	2.67				

Notes

1. Specific gravity specimens were taken at depths shown from within the sampling tubes.

2. Limits and hydrometer specimens for batches 3, 4 and 6 were taken from excess soil trimmed off the outside of tubes after pressing the sampling tubes. Limits and hydrometer specimens were taken from within tube 3 of batch 7.

TABLE 4
SUMMARY OF SAND TESTING ANALYSIS

Deviator Stress (psi)	Confining Pressure (psi)	All Data Used In Calculations					Erroneous Data Eliminated				
		Avg. M_r (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)	Avg. M_r (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)
1		15,670	8,488	10	13,320 18,020	57.2	18,635	6,511	8	15,840 21,430	69.2
2		15,688	5,896	10	13,335 18,041	72.6					
5	20	17,515	2,676	10	14,888 20,142	98.3					
10		17,418	3,586	10	14,805 20,030	94.1					
15		16,078	2,337	10	13,666 18,489	98.6					
20		15,926	1,648	10	13,537 18,315	99.0					
1		13,087	2,871	9	11,124 15,050	91.5	12,578	2,599	8	10,691 14,464	90.2
2		16,609	12,411	10	14,118 19,100	43.6	12,736	2,135	9	10,826 14,646	96.1
5	15	14,357	3,304	10	12,203 16,510	91.5					
10		14,831	2,661	10	12,606 17,055	96.3					
15		15,230	2,809	10	12,946 17,514	96.0					
20		15,676	2,401	10	13,325 18,027	98.5					
1		15,566	5,676	8	13,231 17,901	67.3	14,217	4,537	7	12,084 16,349	69.2
2		13,762	3,216	9	11,698 15,826	91.0					
5	10	13,067	2,087	10	11,107 15,027	98.0					
10		13,048	2,266	10	11,091 15,005	96.8					
15		13,818	2,298	10	11,745 15,890	97.4					
1		11,833	5,749	9	10,058 13,608	59.1					
2		11,114	4,134	9	9,447 12,781	69.9					
5	5	10,978	2,843	10	9,331 12,625	88.0					
10		10,712	1,699	10	9,105 12,319	98.0					
15		10,342	2,099	10	8,791 11,893	94.5					

TABLE 4
SUMMARY OF SAND TESTING ANALYSIS (Cont'd)

Deviator Stress (psi)	Confining Pressure (psi)	All Data Used In Calculations					Erroneous Data Eliminated				
		Avg. M_T (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)	Avg. M_T (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)
1	1	12,107	6,363	8	10,291 13,923	52.2	10,159	3,440	7	8,635 11,683	66.5
2		12,064	8,755	9	10,254 13,873	42.4	9,281	2,825	8	7,889 10,673	75.6
5		10,028	2,128	4	8,524 11,532	67.4					
7.5		11,442	1,270	4	9,726 13,158	89.9					
10		10,320	1,324	3	8,772 11,868	74.3					

1) S - Standard Deviation
2) N - Number of data points
3) C Lim - Confidence limits
4) C Lev - Confidence level

TABLE 5
SUMMARY OF CLAY TESTING ANALYSIS

Deviator Stress (psi)	Confining Pressure (psi)	All Data Used In Calculations					Erroneous Data Eliminated				
		Avg. M_r (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)	Avg. M_r (psi)	1) S	2) N	3) C Lim (psi)	4) C Lev (%)
1	6	146,046	277,106	11	124,139 167,953	19.2	6,086	1,755	7	5,173 6,999	76.0
	3	156,291	276,131	11	132,847 179,735	20.6	6,619	3,036	6	5,626 7,612	50.2
	0	155,385	276,499	11	132,077 178,693	20.5	7,178	2,649	8	6,101 8,255	66.3
2	6	57,534	156,943	11	48,904 66,164	13.4	10,228	4,048	10	8,694 11,762	70.2
	3	11,715	5,781	11	9,958 13,472	63.6	10,386	3,943	10	8,828 11,944	72.2
	0	11,039	6,292	11	9,383 12,695	57.4	9,933	5,389	10	8,443 11,423	57.0
3	6	7,261	1,677	11	6,172 8,350	92.9					
	3	7,011	1,841	11	5,959 8,063	89.9					
	0	6,559	1,637	11	5,575 7,543	91.1					
4	6	4,775	1,640	11	4,059 5,491	80.3					
	3	4,470	1,358	11	3,800 5,141	84.4					
	0	4,530	1,745	11	3,851 5,210	74.3					
5	6	3,441	1,779	11	2,925 3,957	61.5					
	3	3,406	1,671	11	2,895 3,917	63.6					
	0	3,035	1,358	11	2,580 3,490	67.4					
6	6	3,255	1,211	9	2,767 3,743	69.8					
	3	3,073	1,090	9	2,612 3,534	72.0					
	0	2,907	1,057	9	2,471 3,343	70.9					
7	6	3,201	1,561	5	2,721 3,681	45.3					
	3	3,539	1,070	4	3,008 4,070	54.4					
	0	3,068	613	3	2,608 3,528	60.0					
8	6	3,451	____	1	____	____					
	3	2,847	____	1	____	____					

1) S - Standard Deviation
2) N - Number of data points
3) C Lim - Confidence limits
4) C Lev - Confidence level

TABLE 6

M_r DATA FROM SAND TESTING

DEVIATOR STRESS psi	CONFINING PRESSURE (psi) 1	5	10	15	20
1	13,263 12,132 9,213 6,437 11,148 4,945 13,976 Avg = 10,159	11,564 8,084 10,180 10,005 26,469 8,579 9,412 13,599 8,608 Avg = 11,833	21,861 14,340 8,206 10,296 14,301 12,871 17,642 Avg = 14,217	12,939 16,173 13,639 8,336 15,202 12,507 13,234 9,591 Avg = 12,578	25,315 12,874 15,377 27,197 12,435 13,238 26,469 16,173 Avg = 18,635
2	10,702 14,667 10,201 6,668 9,786 7,355 5,649 9,225 Avg = 9,281	9,632 7,566 10,915 8,821 21,768 9,626 10,652 10,442 10,604 Avg = 11,114	17,304 11,924 12,710 8,578 17,886 11,600 13,237 11,762 14,874 Avg = 13,762	12,662 13,420 13,601 10,588 15,541 8,700 15,126 13,056 11,930 Avg = 12,736	21,452 13,402 15,718 26,105 15,000 17,157 17,158 6,708 17,315 6,865 Avg = 15,688
5	9,352 9,040 13,179 8,539 Avg = 10,028	12,033 7,546 10,029 9,260 9,587 7,785 14,707 16,436 11,364 11,029 Avg = 10,978	15,479 12,033 16,180 10,870 12,602 10,002 13,888 14,702 13,887 11,029 Avg = 13,067	13,221 15,565 16,017 8,731 16,159 10,002 14,707 15,570 20,360 13,235 Avg = 14,357	18,699 15,528 17,860 18,908 19,346 13,234 16,545 22,054 18,908 14,067 Avg = 17,515
10	11,211 8,798 10,951 Avg = 10,320	13,739 10,090 11,844 9,218 9,696 8,877 13,197 10,638 10,634 9,190 Avg = 10,712	17,244 11,249 16,002 11,265 11,680 10,952 13,197 13,236 14,705 10,951 Avg = 13,048	15,759 13,414 16,150 11,970 15,038 10,723 14,706 17,748 19,609 13,197 Avg = 14,831	21,566 15,548 18,073 15,846 15,740 13,911 15,945 25,735 16,666 15,151 Avg = 17,418
15		11,849 10,266 12,982 8,959 10,666 7,190 13,206 7,966 11,675 8,659 Avg = 10,342	19,214 11,873 15,393 11,637 12,024 11,991 13,933 13,676 14,701 13,733 Avg = 13,818	16,316 13,294 16,142 12,575 14,684 12,744 14,710 15,647 22,269 13,917 Avg = 15,230	20,547 15,249 18,851 15,150 15,498 15,589 14,927 18,047 13,932 12,989 Avg = 16,078
20				13,463 13,582 16,545 13,763 15,027 12,553 18,008 16,133 20,589 14,100 Avg = 15,676	19,276 15,327 17,442 15,338 15,872 15,134 15,873 17,154 13,748 14,100 Avg = 15,926

Notes:

1. Some M_r values were not included in this table due to erroneous deflection data.

2. Not all stress states have the same number of M_r entries due to data elimination (see note 1) and specimen failures.

TABLE 7

M_r DATA FROM CLAY TESTING

DEVIATOR STRESS psi	CONFINING PRESSURE (psi) 0	3	6
1	4,256 7,006 9,809 10,040 5,776 7,622 3,121 9,790 Avg = 7,178	5,816 11,714 3,811 3,515 6,854 8,001 Avg = 6,619	3,257 6,562 5,405 5,726 9,160 6,527 5,963 Avg = 6,086
2	4,890 7,142 16,524 12,462 21,734 7,170 8,647 6,486 6,085 8,187 Avg = 9,933	9,608 9,160 15,823 13,329 17,988 7,273 8,225 6,916 6,845 8,692 Avg = 10,386	8,391 9,063 14,945 12,115 19,176 7,574 8,105 7,387 6,167 9,357 Avg = 10,258
3	7,271 8,243 8,850 5,974 3,542 7,337 6,648 7,785 6,348 4,017 6,126 Avg = 7,012	7,682 8,243 9,695 6,757 3,553 6,853 6,351 9,169 6,768 4,353 7,692 Avg = 7,011	7,408 9,341 9,931 7,366 4,651 7,475 7,114 8,613 6,104 4,697 7,175 Avg = 7,261
4	4,136 5,876 4,019 7,632 2,100 6,450 3,355 5,317 5,265 3,444 2,231 Avg = 4,530	4,482 6,108 4,077 4,972 2,662 6,439 3,355 5,605 5,265 3,916 2,294 Avg = 4,470	5,516 6,977 4,077 5,872 2,720 7,208 3,307 5,449 5,265 3,771 2,363 Avg = 4,775
5	2,514 5,143 1,329 5,418 1,537 4,154 3,137 2,395 3,193 2,442 2,121 Avg = 3,035	2,753 5,440 1,383 6,830 1,537 4,894 3,137 3,114 3,371 2,392 2,610 Avg = 3,406	2,609 5,239 1,503 7,632 1,647 4,568 3,287 2,883 3,371 2,442 2,671 Avg = 3,441
6	2,156 3,350 1,262 2,854 2,598 2,259 2,967 3,760 4,960 Avg = 2,907	2,209 3,198 1,342 3,058 3,502 2,089 3,204 4,118 4,941 Avg = 3,073	1,266 4,206 1,321 2,919 3,660 2,346 3,081 4,248 5,248 Avg = 3,255
7	2,725 3,775 2,703 Avg = 3,068	2,725 3,768 4,960 2,703 Avg = 3,539	1,230 2,725 4,267 5,223 2,561 Avg = 3,219

See TABLE 6 notes

TABLE 8

ANOVA SUMMARY FOR SAND TESTING

			Source of Variation: Deviator Stress	Source of Variation: Confining Pressure	Source of Variation: 2-Factor Interaction
F-Ratios	Case 1		0.512	26.487	0.719
F-Ratios	Case 2		0.568	26.630	0.628
F-Ratios	Case 3		0.074	28.439	0.640
Confidence Levels (%)	Case 1	99	3.11	3.41	2.10
		95	2.26	2.42	1.70
		90	1.88	1.97	1.51
		75	1.34	1.36	1.23
	Case 2	99	3.42	3.42	2.28
		95	2.42	2.42	1.80
		90	1.97	1.97	1.57
		75	1.36	1.36	1.25
	Case 3	99	2.93	3.46	2.64
		95	2.67	2.44	2.01
		90	2.13	1.99	1.71
		75	1.39	1.37	1.30

Notes:

Case 1 - uses all M_r data.

Case 2 - ignores M_r data at 1-psi deviator stress level.

Case 3 - ignores M_r data at 1- and 2-psi deviator stress levels.

TABLE 9

REGRESSION AND CORRELATION SUMMARY

FOR SAND TESTING

	REGRESSION MODEL	Regression of M_r on 3			Regression of M_r on d		
		a	b	Corr. Coeff.	a	b	Corr. Coeff.
Case 1	LINEAR	362.43	9,344	0.581	98.97	12,876	0.154
	MULT.	8,526	0.193	0.572	12,009	0.053	0.187
Case 2	LINEAR	365.07	9,225	0.628	123.59	12,545	0.202
	MULT.	8,336	0.202	0.607	11,282	0.081	0.239
Case 3	LINEAR	382.02	9,174	0.685	99.34	12,880	0.153
	MULT.	8,081	0.221	0.655	11,463	0.074	0.153

Notes:

1) Regression models
   Linear - $Y = aX + b$,
   Mult. - Multiplicative = $Y = aX^b$

2) See TABLE 7 notes for cases 1, 2 and 3.

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MINNESOTA DEPARTMENT OF TRANSPORTATION EXPERIENCE WITH LABORATORY MR TESTING

by

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March 1989

The purpose of this talk is to give an overview of resilient modulus testing in the Minnesota DOT. To begin, I would like to give a little background on our history in this area.

In 1961, I was chasing P waves and S waves around California with a geophone, oscilloscope, and sledge hammer. Fig. 1. Those wave velocity values, when coupled with measured inplace densities, gave values of Young's modulus, bulk modulus, shear modulus, and Poisson's ratio for several inplace soils. Figures 2, 3.

Later, Mn/DOT purchased a Road Rater and eventually an FWD. We have tested all our bituminous roadways and have accumulated a considerable data bank. Figure 4. Our people have considerable confidence in the back calculated subgrade resilient moduli. However, there is apprehension as to back calculated surfacing moduli. This is because we have found the back calculated values to be very sensitive to layer thickness and we have also found that in real life there is a very considerable variation in layer thickness.

We also managed somehow to purchase a laboratory resilient modulus device - an MTS. This was to be a more or less universal machine with a large triaxial chamber and temperature controlled, so that samples as large as 6" X 12" could be tested. We also constructed a diametrical loading device for testing bituminous lab samples and field cores.

Due to problems with waterproofing LVDT's, we later abandoned the liquid filling in favor of nitrogen.

After we gained some experience with both the FWD and the MTS, it was proposed that we conduct a comparison of laboratory and field test results from the same site. The results were not encouraging. At the time, the feeling in the Department was that the field values were correct and the laboratory values incorrect. In retrospect, it may have been that we really did not know which was really correct. This disappointment, however, lead us to rework the laboratory system. To this end, we sought assistance from Dr. Vardoulakis of the University of Minnesota CME Department. He provided guidance in reworking the LVDT mounting system. In addition, our lab technician visited CRREL and, as a result, gained experience in testing techniques. Finally, we happened upon a report from Professor Baladi regarding an improved indirect tensile loading device. We constructed two of these devices and have been very satisfied with them. They seem to give very consistent results. Our earlier, essentially homemade indirect tensile device, had produced inconsistent results; possibly as a result of the sample "rocking" in the test frame.

Due to a loss of valued personnel, we have not yet repeated the field versus laboratory tests. However, we expect to in the near future, especially in conjunction with the new Cold Regions Test Track. We feel that we have received a considerable education in field and laboratory resilient modulus testing. We also feel that in the meantime, we can perform meaningful laboratory and field resilient modulus tests.

At this point, I would like to give an overview of some recent Mn/DOT laboratory resilient modulus testing results. I would also like to note that all our laboratory testing has been accomplished by Neil McGee, the Foundations Lab Chief. He has been very enthusiastic and resourceful in making the system work. His reports are attached.

An early study using the the Baladi device simply compared the calculated resilient modulus of several bituminous Marshall specimens compacted at different AC contents and at different blows. Note that the center hammer consistently gave smaller values. Figure 5. This phenomena was disturbing, but no apparent reason could be found. Densities were within 1/2 pound. Subsequent studies only produced minor variations between hammers. Of more interest is the effect upon calculated resilient modulus value of using an assumed value of 0.35 for Poisson's ratio, as suggested in ASTM D4123, versus using actual measured horizontal and vertical displacements to calculate Poisson's ratio. Figure 5a. The differences from the two approaches are startling.

In view of the fact that the samples were in all probability not homogeneous, isotropic and truly elastic, perhaps instead of using the term "Poisson's ratio", we should instead use the term "horizontal deformation versus vertical deformation". Nevertheless, the use of an assumed Poisson's ratio of 0.35 is suspect - at least in our minds.

A later study compared resilient modulus values from asphalt concrete Marshall briquets. Again, the Baladi device was used. The control (virgin aggregate and asphalt) had a resilient modulus of approximately 500,000 psi ± at room temperature. These values reflect the use of measured Poisson's ratio. Again, if we had assumed a value of 0.35 much higher, Mr values would have resulted. Figure 6. Of interest to us was also the high Mr values generated by the 60% recycle 40% virgin mixture. Figures 7 & 8. There has been and continues to be considerable debate within the Department as to value of recycled mixtures. These tests seemed to allay some concerns - at least as far as Mr values are concerned. Figure 9. However, there is still the concern that a higher Mr value mix may be more brittle and thereby have an ultimately shorter fatigue life. Another question that came up was whether Poisson's ratio could be used as indicator of rutting potential. Conceivably, low Poisson's ratio mixes would have lower rutting potential than higher values. This is because the shear modulus which may be an indicator of rutting potential is directly proportioned to Young's modulus, but inversely proportional to Poisson's ratio.

$$\text{Shear Modulus} = \frac{\text{Young's Modulus}}{2\ (1 + \text{Poisson's ratio})}$$

Thus, two samples with similiar Young's moduli but different Poisson's ratio may have different rutting potential. Likewise, two samples with the same Poisson's ratio but different Young's moduli should have different rutting potential.

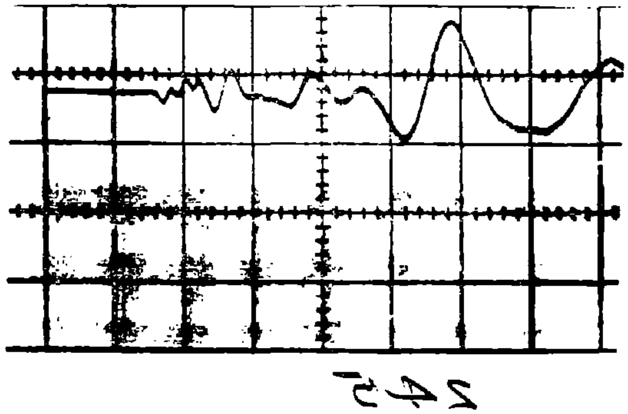
Another study of control and recycled mixtures conducted on cores from a 12 year old project on T.H. 12 west of the Twin Cities. These samples were run at three different temperatures. Figures 10 & 11. As might be expected, the Mr and Poisson's ratio varied with temperature. Figure 12, 13, 14. Both virgin and recycled mixes showed less change in Mr with temperature than fresh samples would - perhaps due to oxidation aging of the asphalt cement. The recycled cores were generally stronger than virgin cores, although both gave acceptable values. Again, the use of an assumed Poisson's ratio would give quite different test values. Finally, a plot of Mr values across the road show a pattern of variation which needs to be taken into account in any future field coring operations. Figure 15, 16. Whether that pattern is random or due to construction or traffic effects is not known. However, in any future coring it is our intention to be sure the cores be specifically taken in the wheel paths or between the wheel paths, and noted as such.

Testing of the proposed Cold Regions test track base aggregate was another study. The data does not provide any startling results, except perhaps that one of the crushed rock samples seemed to give unexpectedly low values. Figure 17. The values will be used in the design of test track sections.

Testing of test track subgrade soils was also performed for use in the design of test track sections. One question which comes up during this testing was the selection of a test moisture content. One value - optimum moisture content (T-99) was selected as a benchmark and possibly indicative of construction conditions. Another value - optimum plus 2% was also selected as perhaps more representative of in service or even springtime conditions. These assumptions were based on investigation performing during 1943 and 1945 by John Swanberg and C. C. Hansen of MHD (HRB proc. Vol 26). Figure 18. They found, based on extensive sampling, that the field soil moisture content was somewhat more than T99 opt. m.c. (about 104%), but that in the spring, the field m.c. generally increased by 2 percentage points.

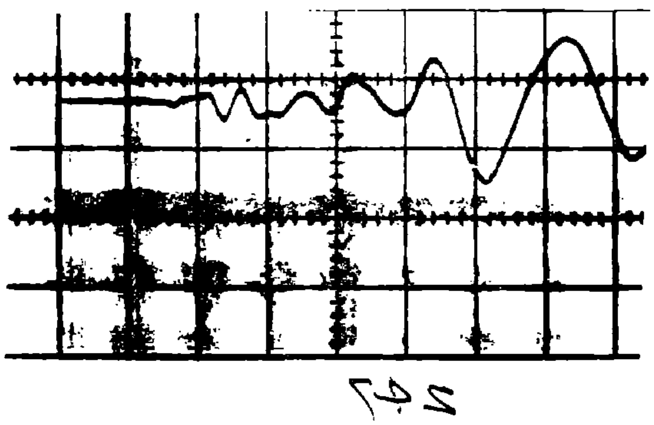
As can be seen from the data, the higher moisture content values have the effect of reducing Mr to about 1/2 of the Mr at opt. m.c. Figure 19, 20, 21. These values will be verified after construction by FWD testing.

In summary, we feel that laboratory and field resilient modulus testing are very useful tools for pavement design. The examples at hand show that quantitative comparisons can be made between recycled and virgin bituminous mixes. A similar comparison can be made regarding the effects of seasonal moisture content changes on subgrade soils. Perhaps most important, the utilization of actual Poisson ratio values may lead to a measure of rutting potential in bituminous pavements.



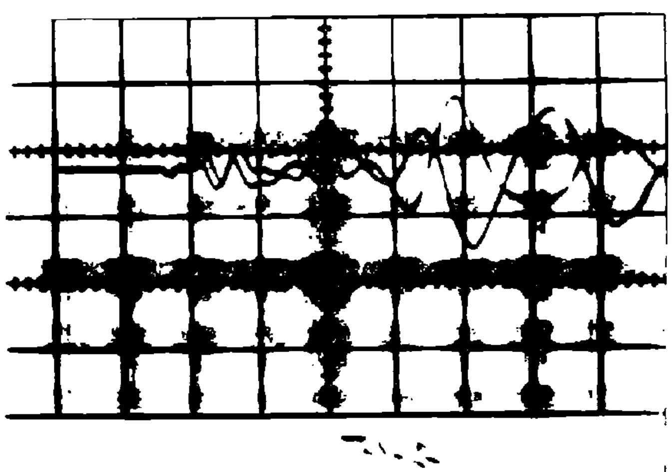
Plan

source — 17.9' — geophone



Plan

source — 17.9' — geophone



Photos 245 & 247 superimposed

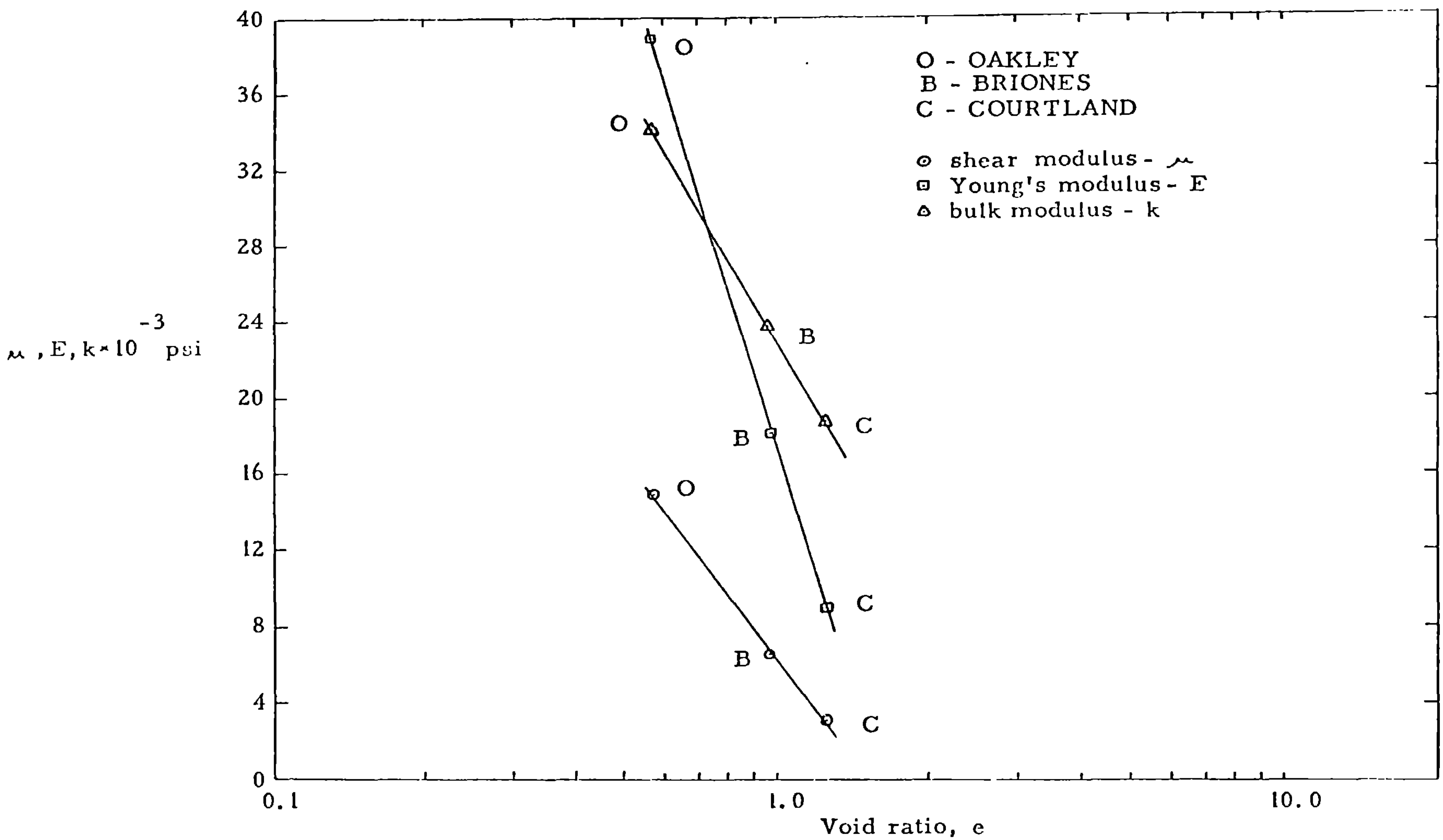


FIG. [redacted] 2

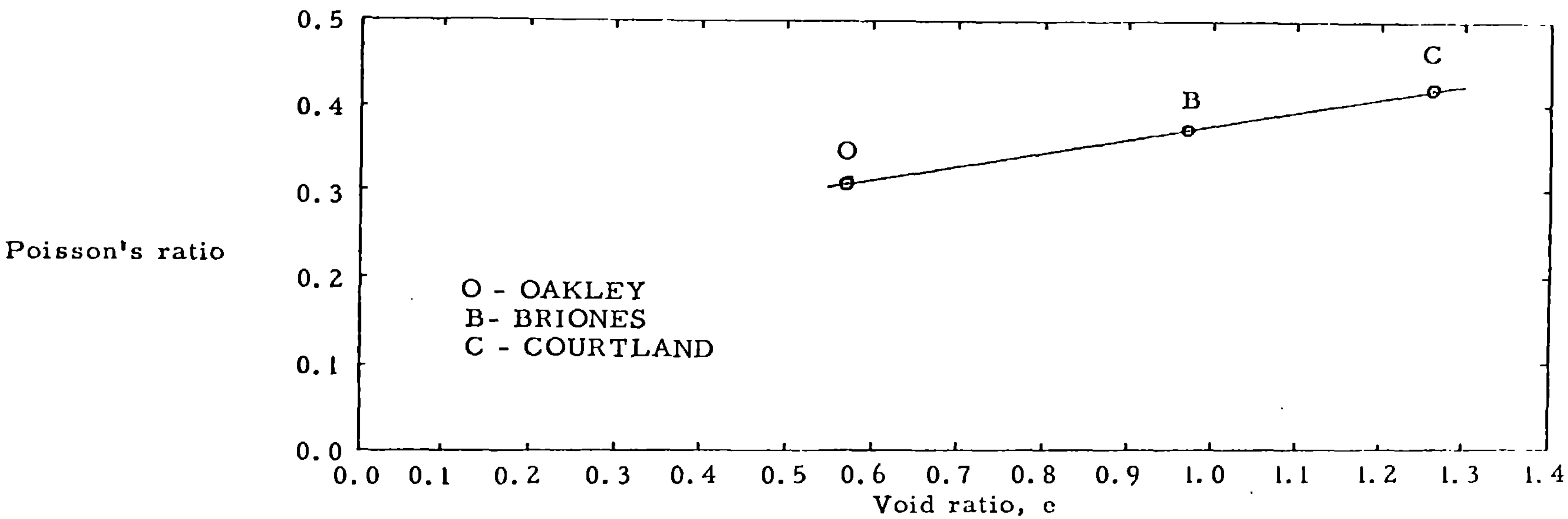
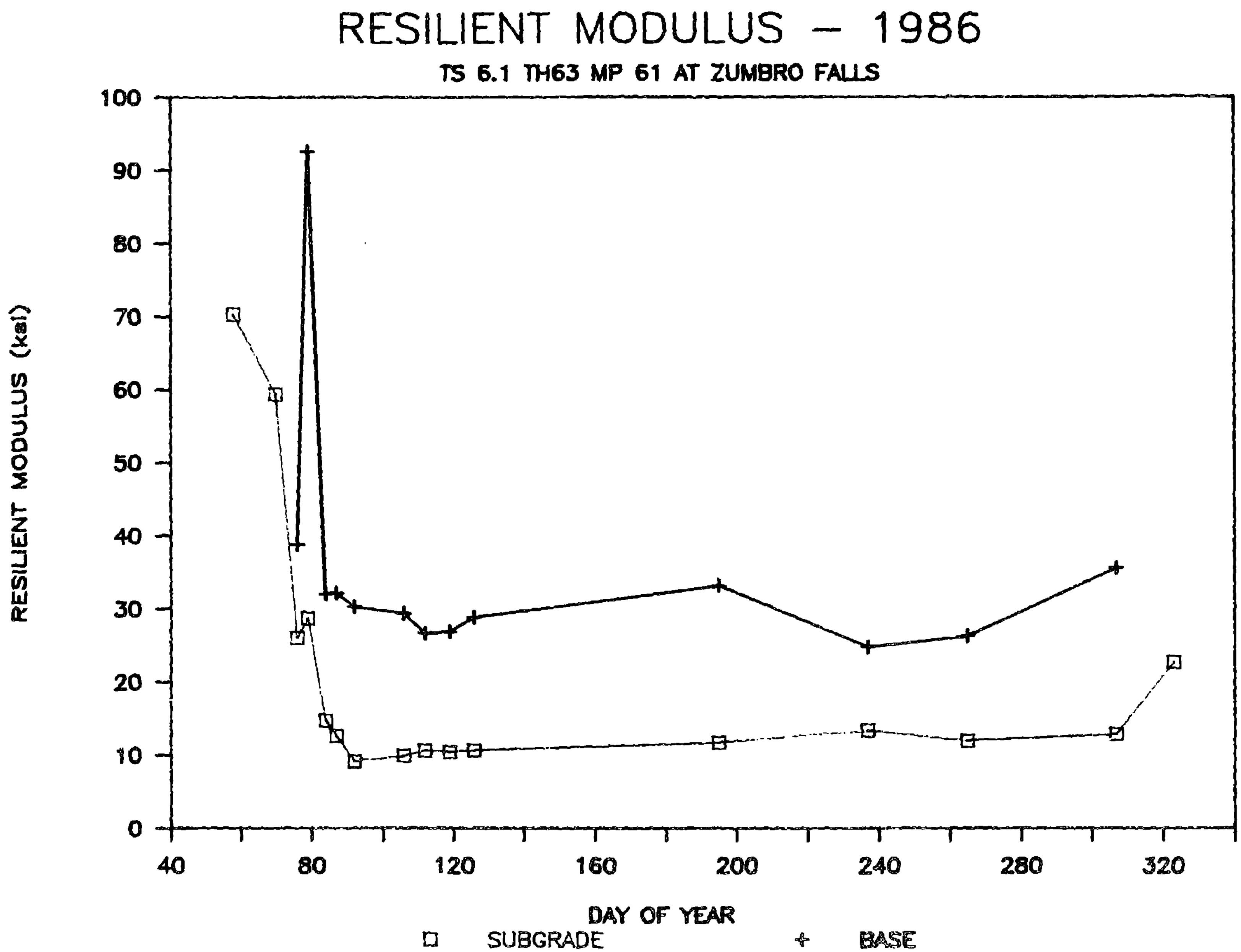

Poisson's ratio
0.5
0.4
0.3
0.2
0.1
0.0
O
B
C
O - OAKLEY
B- BRIONES
C - COURTLAND
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4
Void ratio, e

FIG. 3

F W D


RESILIENT MODULUS – 1986
TS 6.1 TH63 MP 61 AT ZUMBRO FALLS
RESILIENT MODULUS (ksi)
100
90
80
70
60
50
40
30
20
10
0
40
80
120
160
200
240
280
320
DAY OF YEAR
SUBGRADE
BASE

S, CL

Fig 4

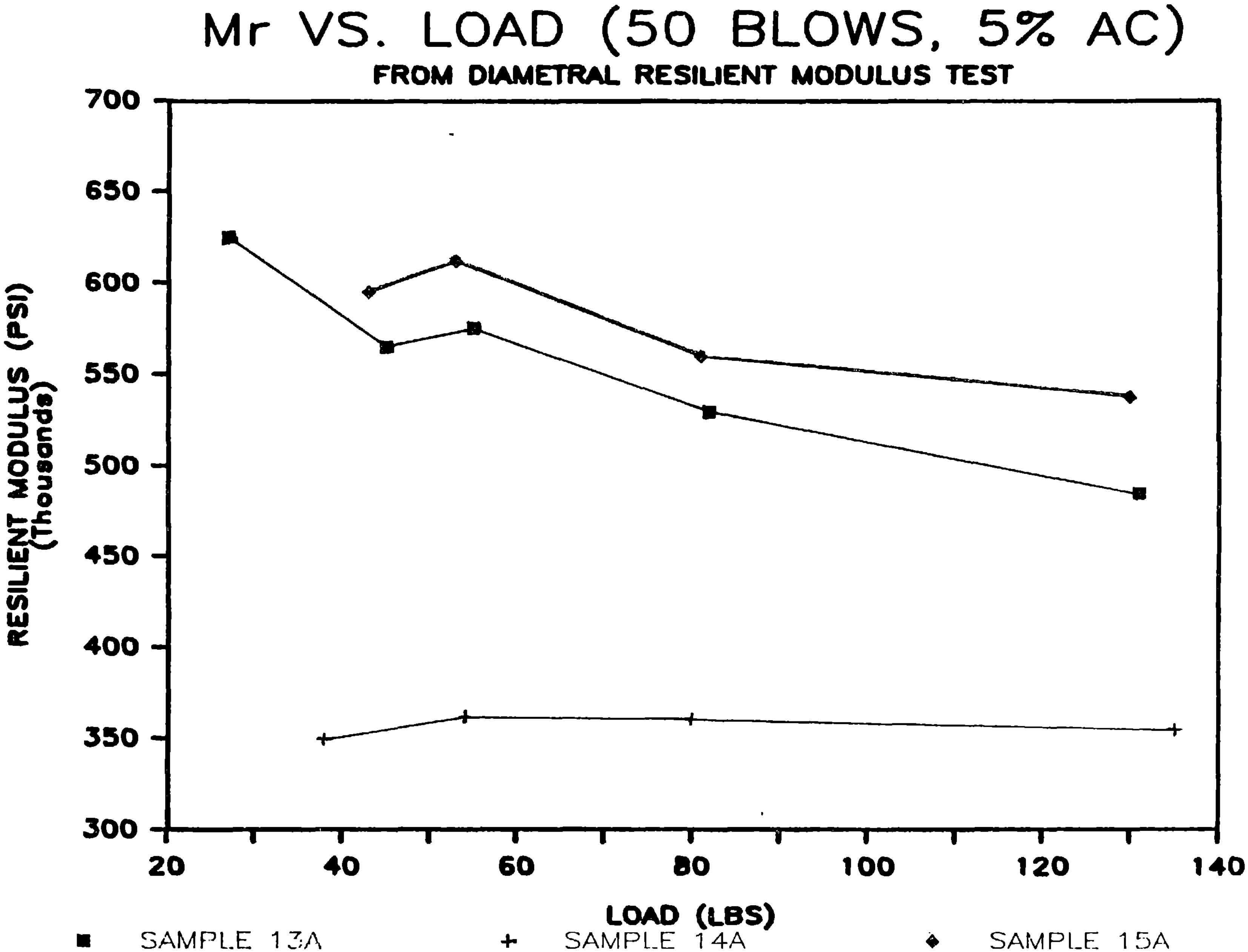

Mr VS. LOAD (50 BLOWS, 5% AC)
FROM DIAMETRAL RESILIENT MODULUS TEST
RESILIENT MODULUS (PSI)
(Thousands)
700
650
600
550
500
450
400
350
300
20
40
60
80
100
120
140
LOAD (LBS)
SAMPLE 13A
SAMPLE 14A
SAMPLE 15A

Fig. 5

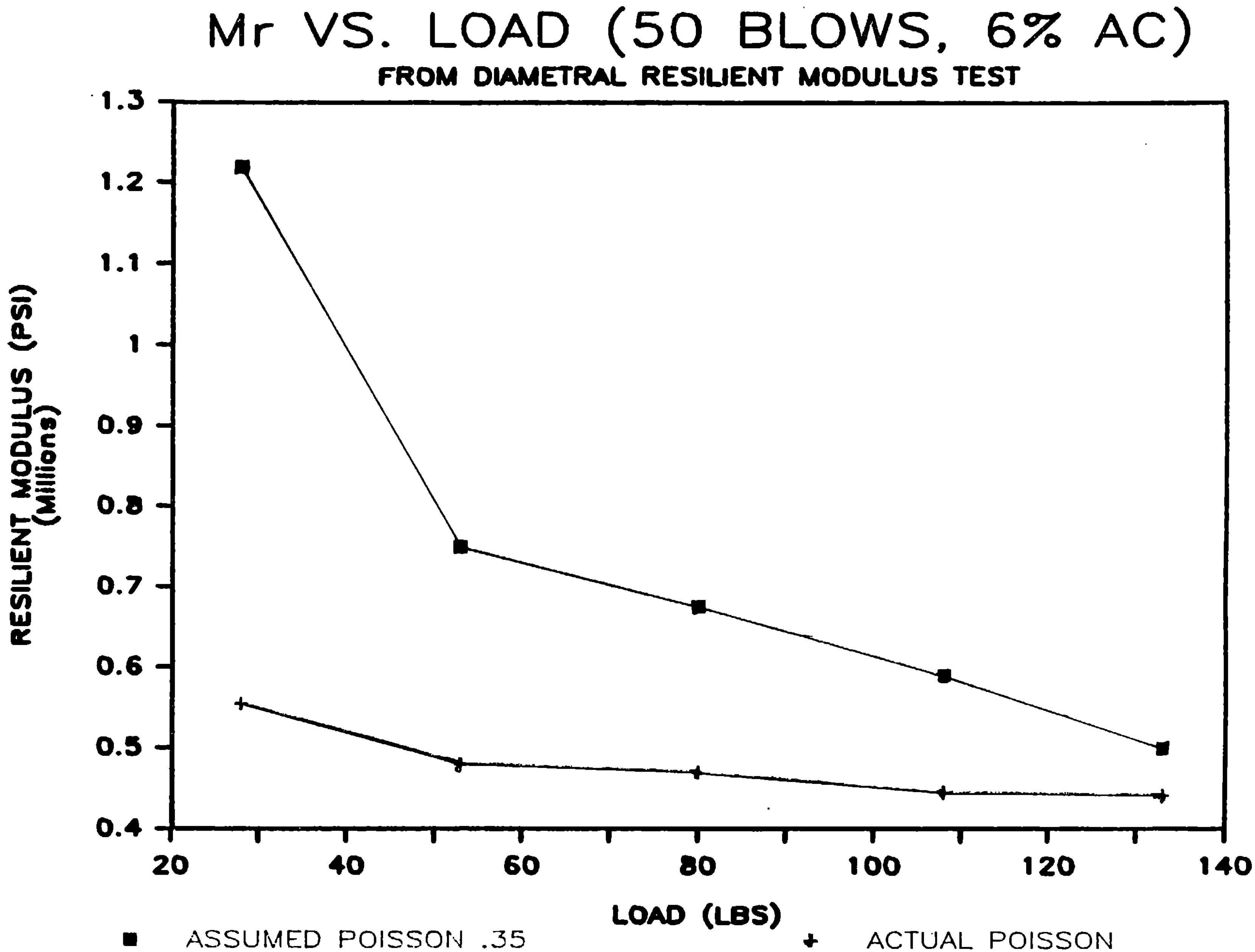

Mr VS. LOAD (50 BLOWS, 6% AC)
FROM DIAMETRAL RESILIENT MODULUS TEST
RESILIENT MODULUS (PSI)
(Millions)
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
20
40
60
80
100
120
140
LOAD (LBS)
ASSUMED POISSON .35
ACTUAL POISSON

Fig. 5 a

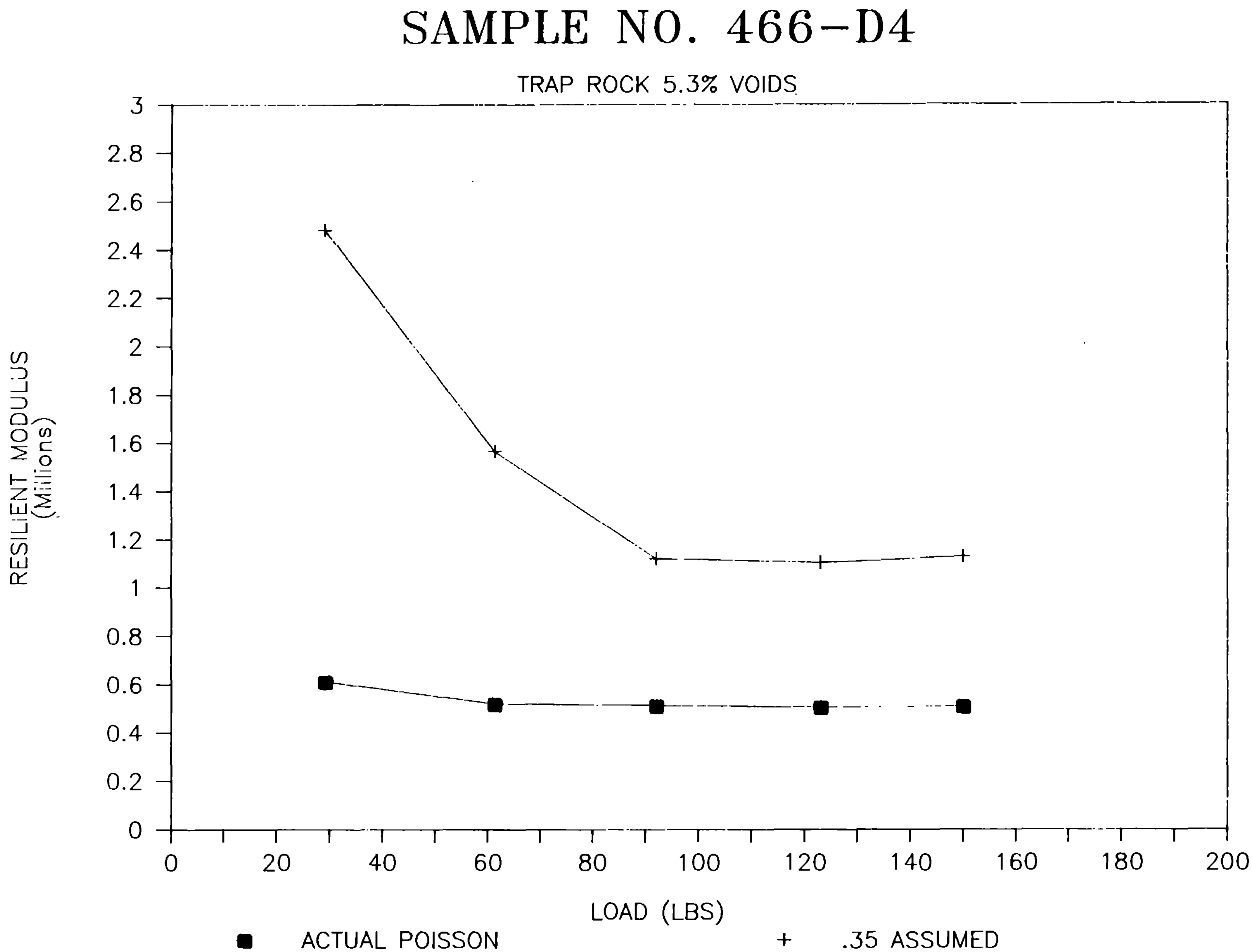

SAMPLE NO. 466-D4
TRAP ROCK 5.3% VOIDS
RESILIENT MODULUS
(Millions)
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80
100
120
140
160
180
200
LOAD (LBS)
ACTUAL POISSON
+ .35 ASSUMED

Fig. 6

SAMPLE NO. 467-D5

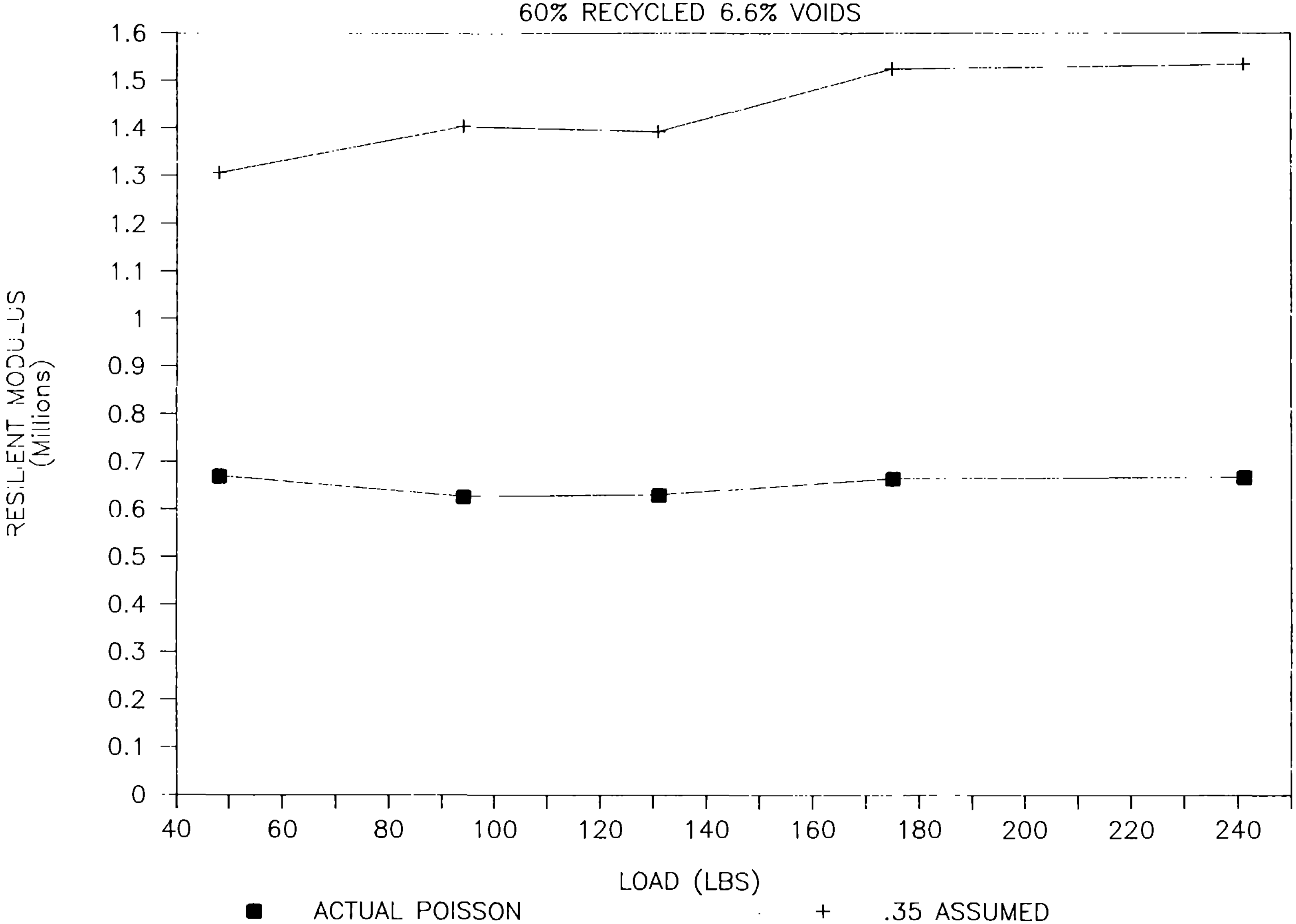


Fig 7

SAMPLE NO. 467-C4

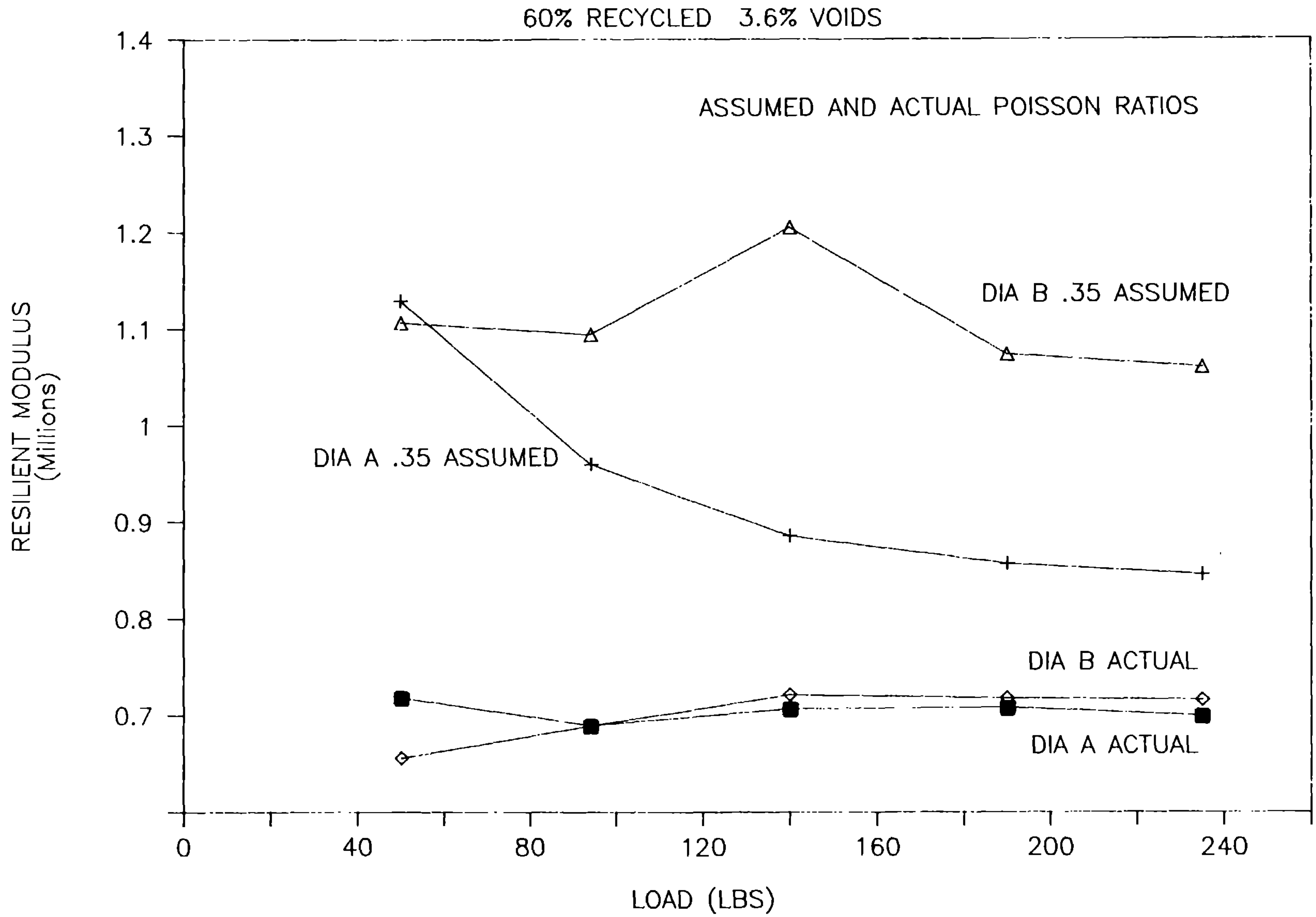
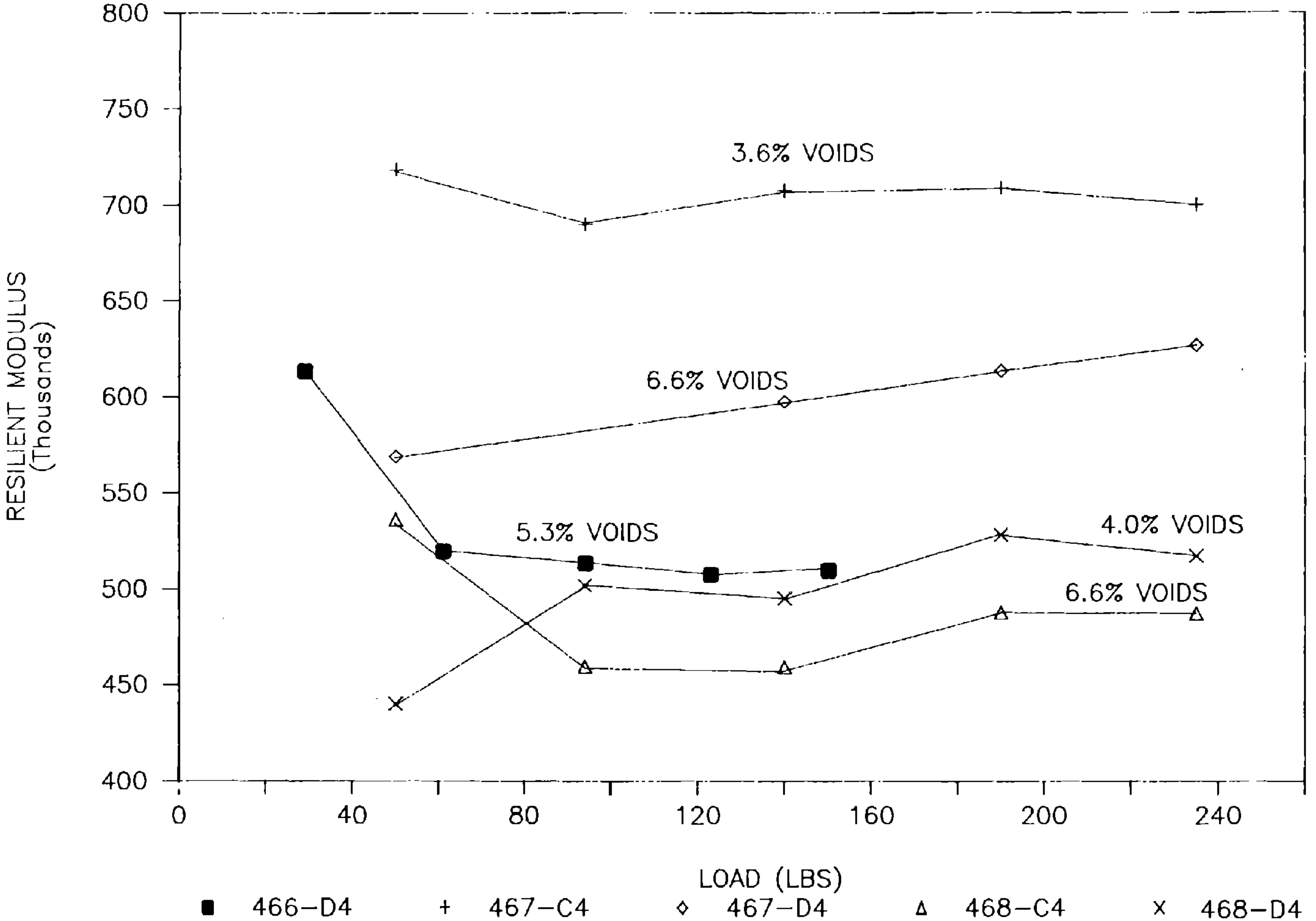


Fig. 8



Virgin 60% Recycled 40% Recycled

Fig. 9

ASPHALT RESILIENT MODULUS VS. APPLIED LOAD

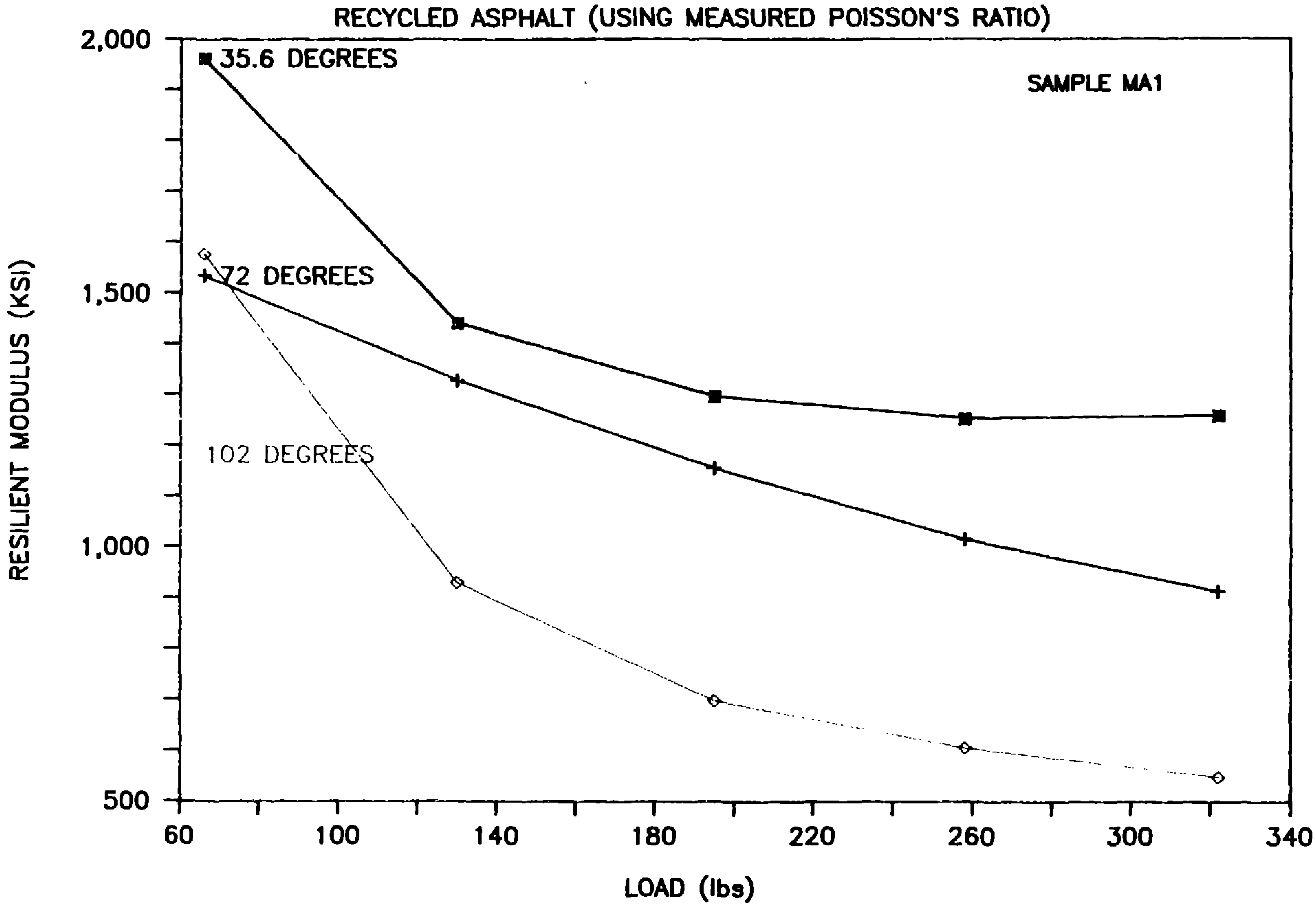


Fig. 10

ASPHALT RESILIENT MODULUS VS. APPLIED LOAD

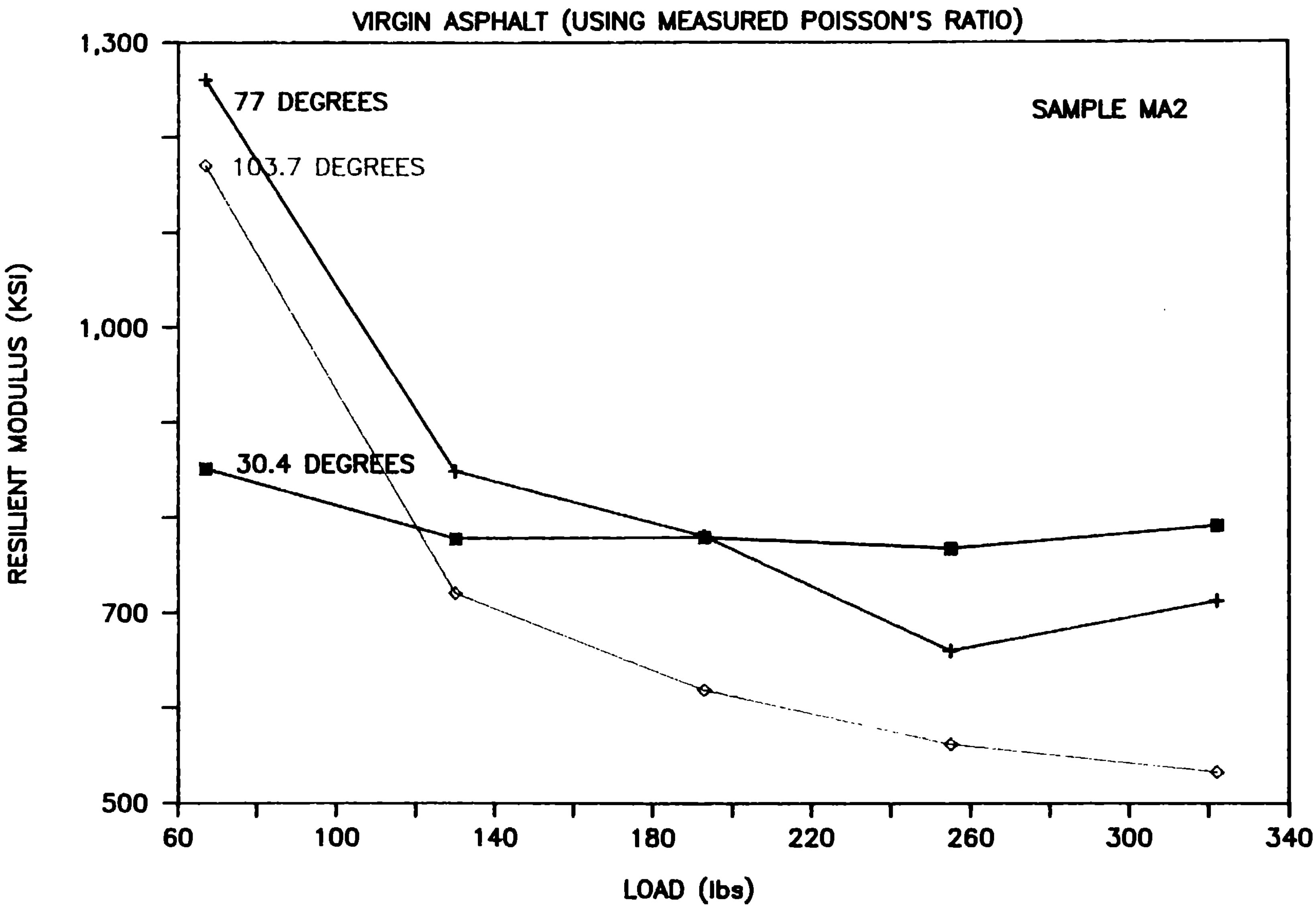


Fig. 11

ASPHALT RESILIENT MODULUS VS. APPLIED LOAD

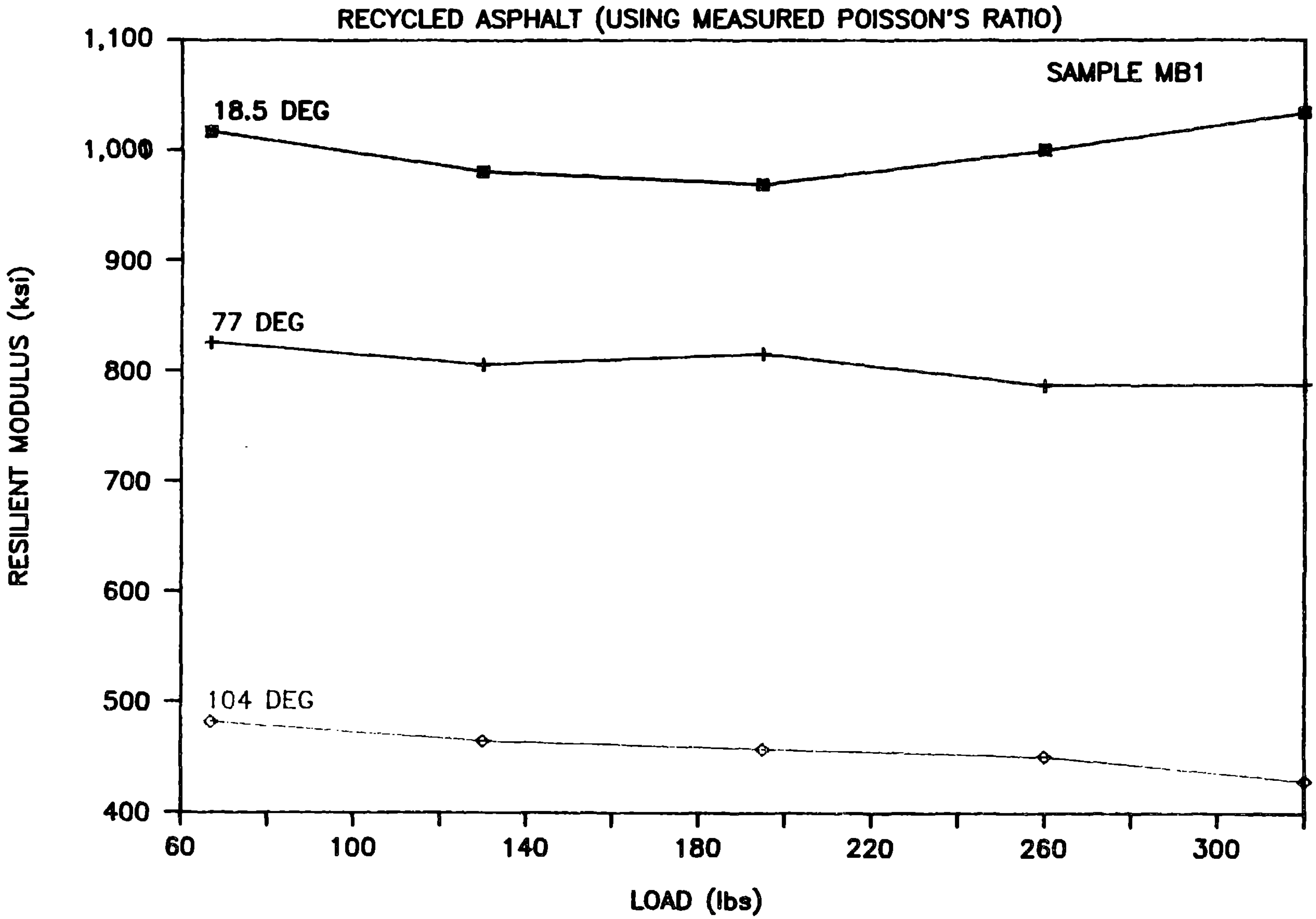


Fig. 12

ASPHALT RESILIENT MODULUS VS. APPLIED LOAD

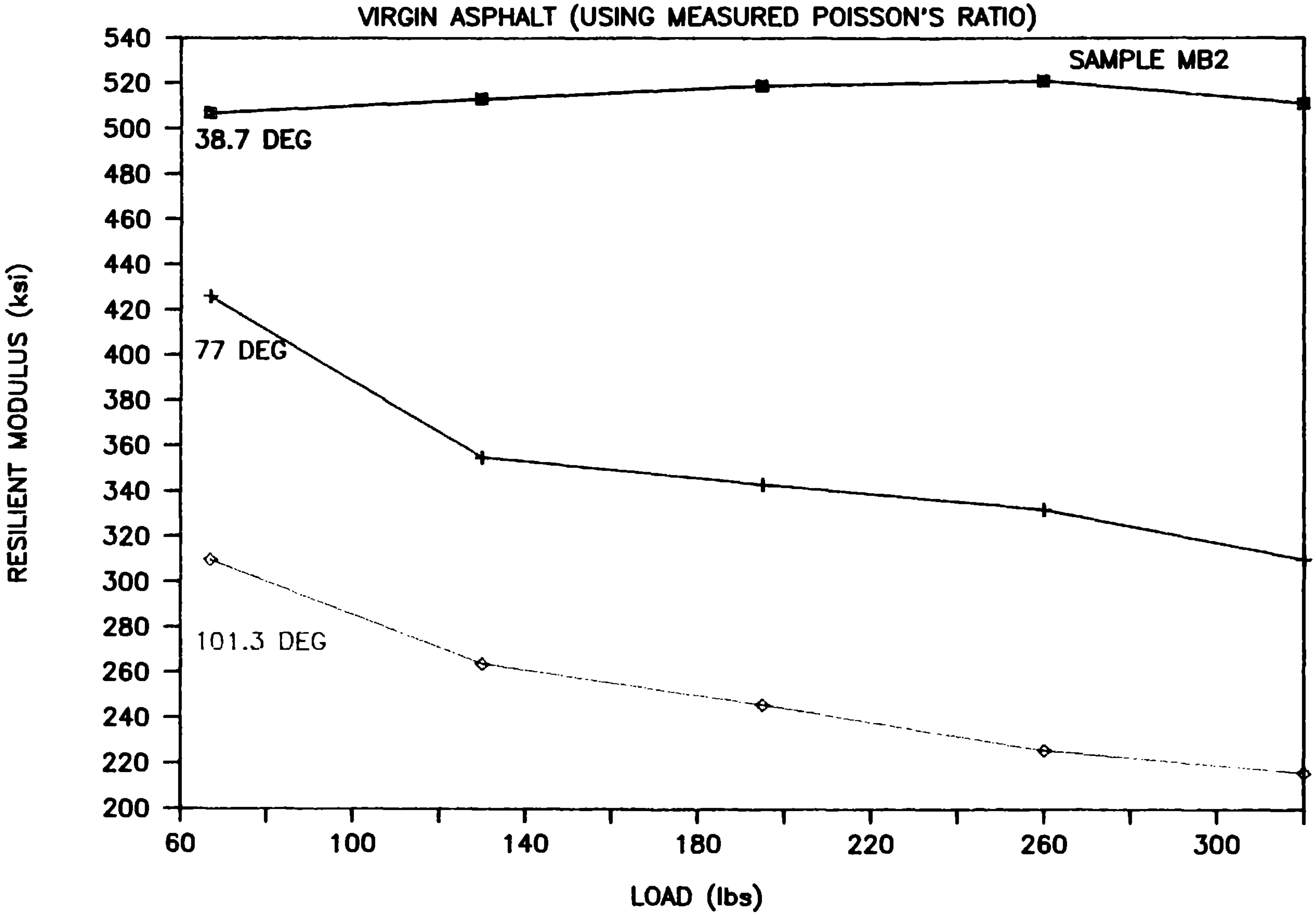


Fig. 13

MEASURED POISSON'S RATIO VS. APPLIED LOAD

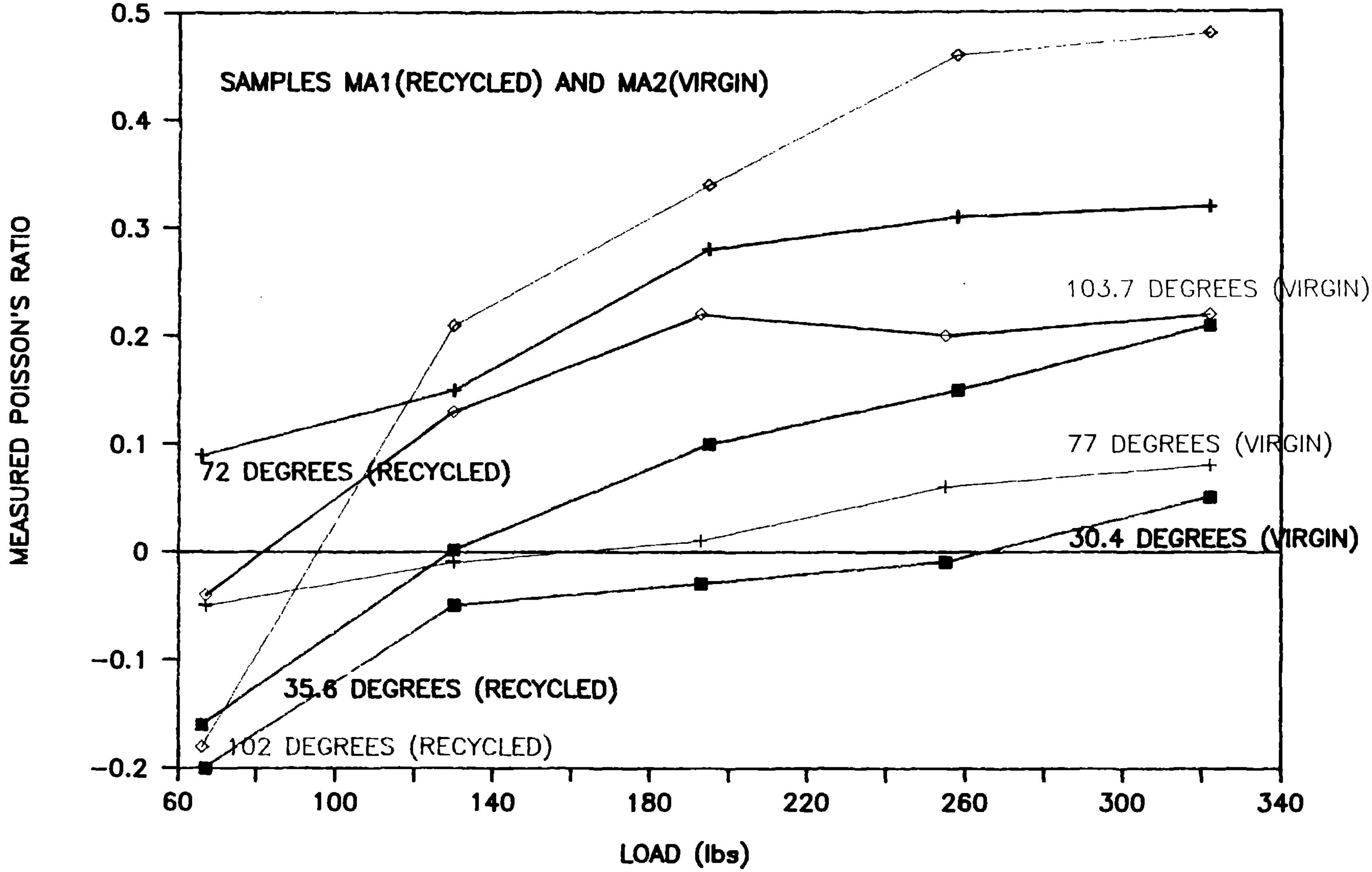


Fig. 14

RESILIENT MODULUS OF RECYCLED ASPHALT

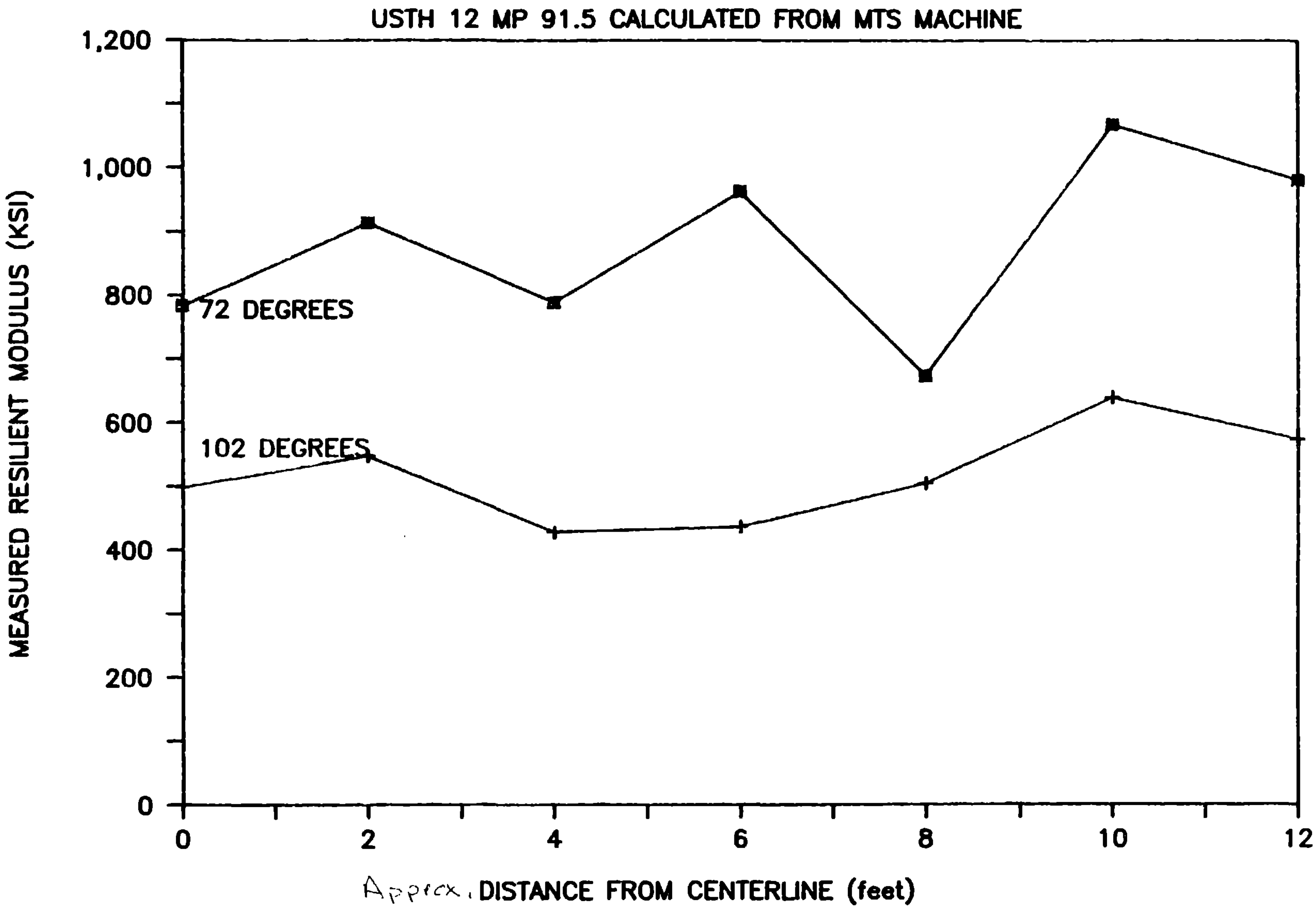


Fig 15

RESILIENT MODULUS OF VIRGIN ASPHALT

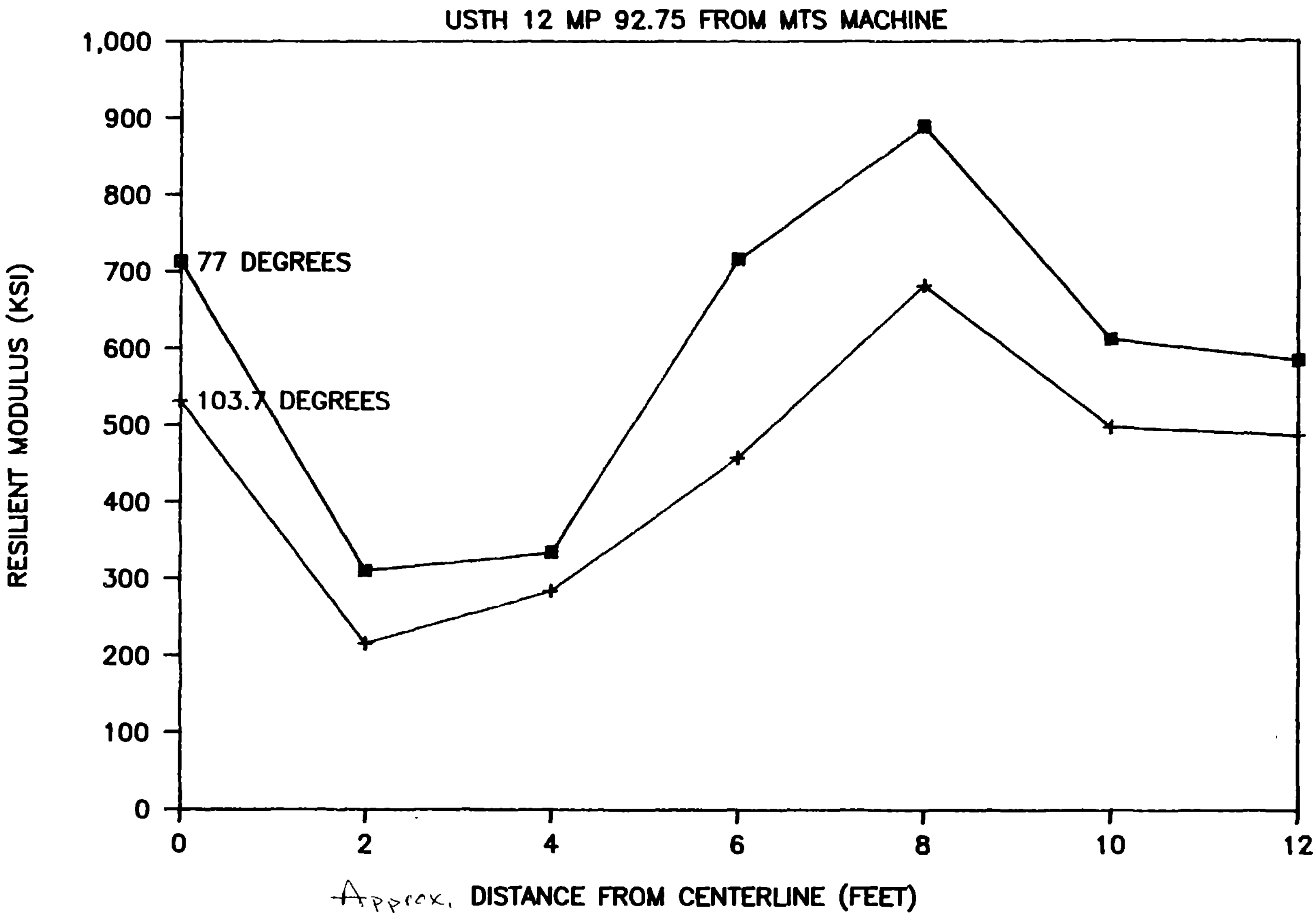
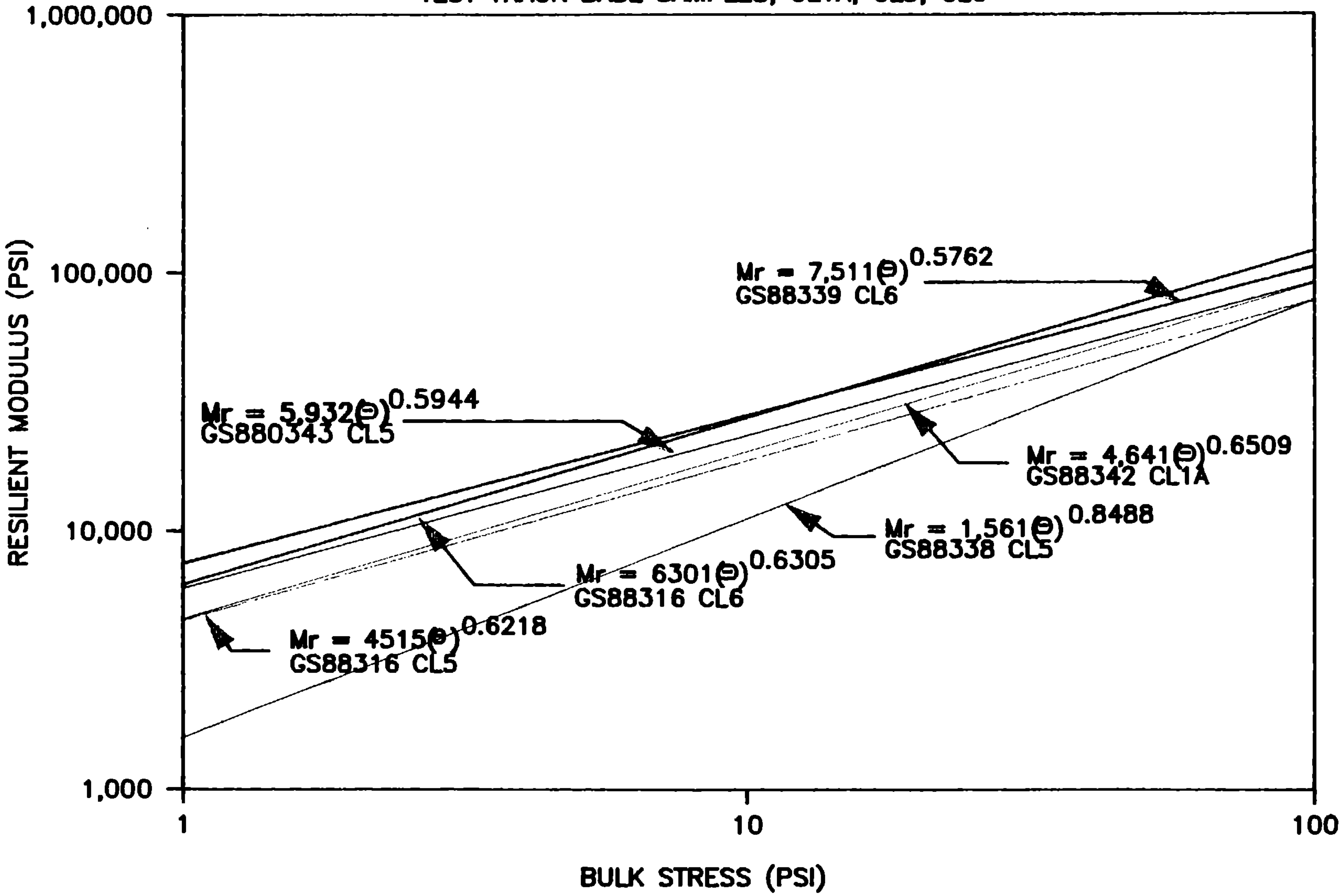


Fig. 16

RESILIENT MODULUS VS. BULK STRESS

TEST TRACK BASE SAMPLES, CL1A, CL5, CL6



Fi. 17

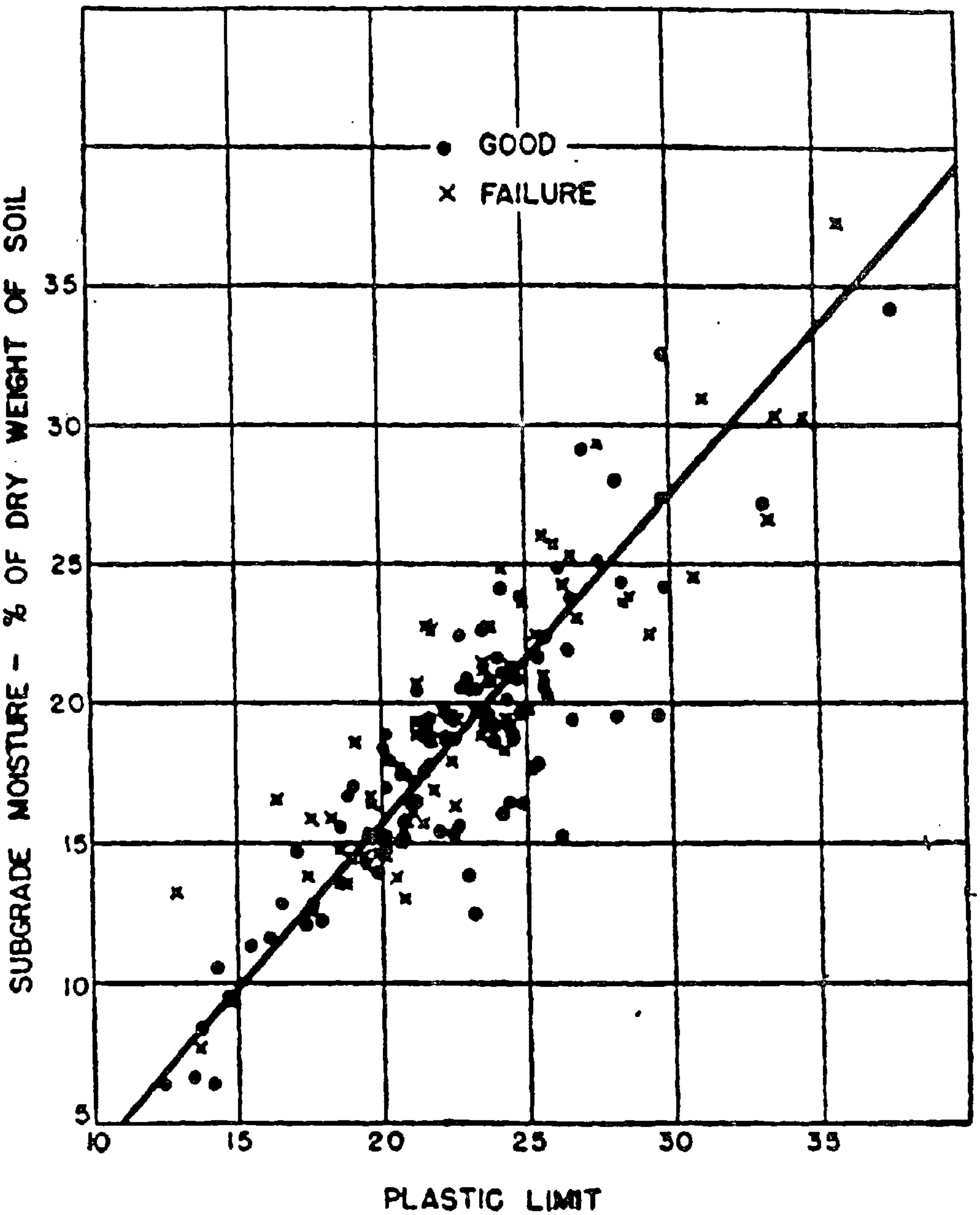
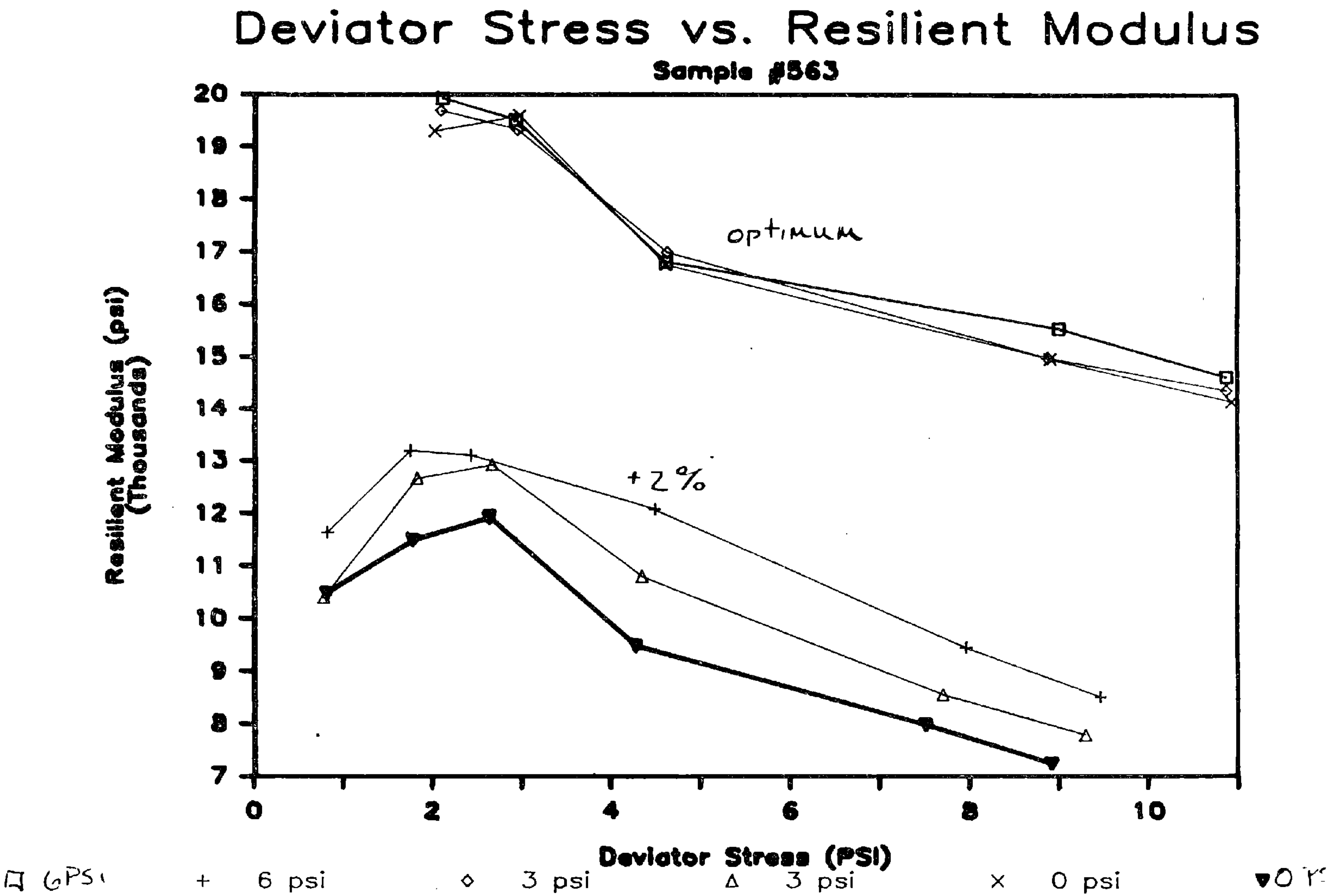


Figure 18. Relation between Subgrade Moisture and Plastic Limit, 1943 Survey

10



Confining Stress

Fig. 19

Deviator Stress vs. Resilient Modulus

Sample #565

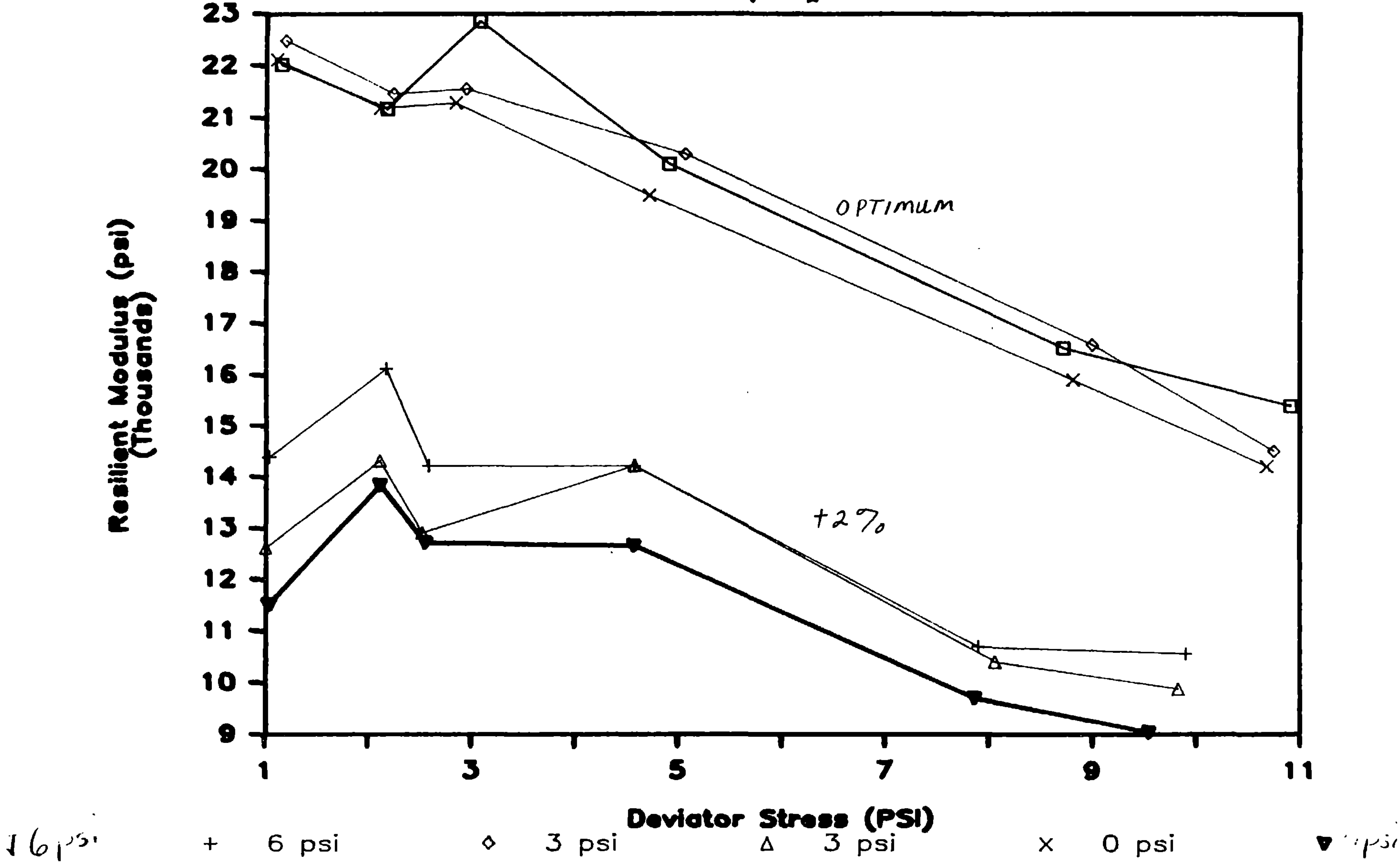


Fig. 20

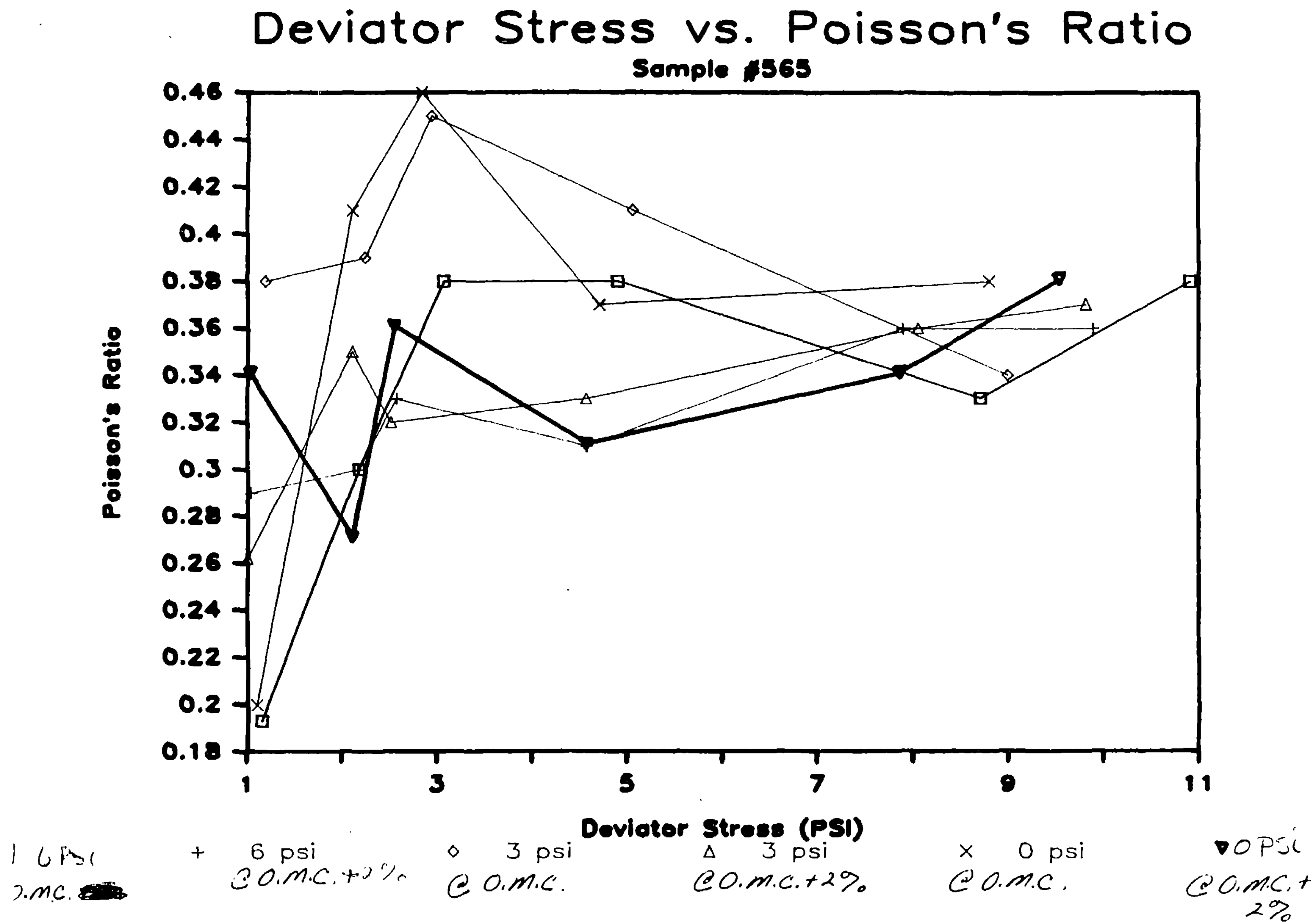

Deviator Stress vs. Poisson's Ratio
Sample #565
Poisson's Ratio
0.46
0.44
0.42
0.4
0.38
0.36
0.34
0.32
0.3
0.28
0.26
0.24
0.22
0.2
0.18
1
3
5
7
9
11
Deviator Stress (PSI)
+ 6 psi @O.M.C.+2%
◇ 3 psi @ O.M.C.
△ 3 psi @O.M.C.+2%
× 0 psi @O.M.C.
▼0 PSI @O.M.C.+ 2%

Fig. 2.1

LABORATORY MR INVESTIGATION OF 2331 MIX

Neil McGee
Foundation Lab Chief
Mn/DOT

<u>Topic</u>

General Investigation on Marshall briquets using mix 2331 which has aggregate from Barton at Scandia.

<u>Scope</u>

The General Investigation will cover the following - Resilient Modulus test to be run on three sets of bituminous Marshall briquets to show effects of different blows and asphalt content.

1. The <u>first set</u> of briquets have an asphalt content of 4 percent A.C. and will have nine briquets. Three briquets were hammered out at 35 blows with the Marshall hammers. There are three different Marshall hammers, and there will be one briquet each hammered out by each of the Marshall hammers. Three briquets will also be hammered out at 50 blows and another three briquets at 75 blows using the same procedure as the 35 blow briquets.

2. The <u>second set</u> of briquets have an asphalt content of 5 percent and was hammered out at 35, 50 and 75 blows, the same as the hammering procedure in 1.

3. The <u>third set</u> of briquets have an asphalt content of 6 percent and was hammered out the same as the hammering procedure in 1.

Procedure used -
The Mn/DOT Indirect Tensile Test Apparatus, which is a copy of a design by Gilbert Baladi of Michigan State University and Michigan's Department of Transportation, was used for the Resilient Modulus test. ASTM Specification D4123-82 was used except for the following:

1. Only one frequency was run and that was at 1 HZ. (The reason for running only one frequency was that from previous testing with this apparatus at room temperature, all three specified frequencies run for resilient modulus fell on top of one another, so all values were the same.

2. Only room temperature was used because we have not finished our new environment cabinet that will fit over this newly made apparatus.

General Description of Testing

1. The Marshall briquet is measured for thickness and two diameters are marked on the briquet, one diameter called DIA. A, the other called DIA B.

2. The briquet was placed in the apparatus which has a one-half inch width loading strip on top and bottom of the sample made with a curvature to fit the Marshall briquets diameter.

3. All transducers are then zeroed in on a range of the oscilloscope used for measuring the output millivolts of the vertical and horizontal strain transducers and the load cell.

4. The Indirect Tensile Apparatus sits on the base of an electro-hydraulic repeated loading testing system manufactured by M.T.S. The loads are applied to the apparatus from the M.T.S. machine in the form of an haversine wave with a .1 second duration and 1 HZ frequency at room temperature.

5. Millivolt readings from horizontal and vertical transducers are converted to inches and the load cell millivolts are converted to pounds.

Calculations

1. Total recoverable horizontal strain deflections in inches
$$\Delta H_T = \frac{\text{MV Horizontal}}{\text{203,500 MV/inch}}$$

2. Total recoverable vertical strain deflections in inches
$$\Delta V_T = \frac{\text{MV Vertical}}{83{,}150}$$

3. Total resilient poisson ratio $\nu_{RT} = 3.59 \dfrac{\Delta HT}{\Delta V_T} - .27$

4. $$\text{LOAD} = \frac{\text{MV LOAD}}{\text{2500 MV/Full Scale}} \times 500 \text{ lb.} = P$$

5. E_{RT} = Total resilient modulus of elasticity (PSI) =
$P\ (\nu_{RT} + .27)/t\ \Delta H_T$

   Where P = LOAD
   t = Sample Thickness
   ΔH_T = Total Recoverable Horizontal Deformation
   ν_{RT} = Total Resilient Poison Ratio

6. E_{RT} using assumed poisson ratio of .35
$$E_{RT} = P\ \frac{(.35 + .27)}{t\ \Delta H_T}$$

7. Graphs were drawn of Modulus vs. LOAD at different blows and A.C.

Analysis of Results

1. Analysis of the Modulus vs. LOAD graphs at different blows (see Graph #1) for 4 percent asphalt content.

   a. Graph Procedure - Two of the three Marshall briquets hammered out by the three Marshall hammers were tested for each of the 35, 50 and 75 blow briquets. The samples were tested on two diameters--one called the A DIA., the other called the B DIA. They were loaded with loads of approximately 25 lbs., 50 lbs., 80 lbs., 100 lbs. and 130 lbs. on each diameter. A resilient modulus was calculated for each load using actual poisson calculated values and graphed (the first graph). The second graph going up vertical from the bottom of graph paper is the average of the two DIAMETERS, DIA. A and DIA. B graphs. The third graph at the top of the graph paper are the graphs of the resilient modulus using an assumed poisson ratio of .35.

b. Graph Analysis -

1b. Looking horizontally across the graphed results of the average of the A and B diameter graphs using actual calculated poissons (middle row of graphs) we see approximately a 12 percent increase in modulus strength between the 35 and 50 blow briquets and there is approximately another increase of 12 percent in modulus strength between the 50 blow briquets and the 75 blow briquets. Total strength increase from 35 blow to 75 blow briquets was approximately 25 percent.

2b. The bottom roll of graphs on graphs No. 1 and 2 show that the tests were fairly well repeatable when run on two diameters varying from about 2 percent to 10 percent variance except on the very lightest loads. (The light loads are quite difficult to measure because of the small strain measurements.)

3b. The top row of graphs on graphs No. 1 and 3 represent the resilient modulus using the assumed poisson of .35. When the actual poisson is of a low value, the modulus is up in value and, when the poisson increases, the modulus begins to decrease. The graph shows much more variance when using assumed poisson values than when using actual values.

4b. It can be noted from these graphs certain observances such as the following:

4b-A. There is a slightly larger modulus on the lightest loading.

4b-B. There is an increase of poisson ratio as the asphalt content increases (see all three graph sheets #1, #2, #3 and look below the horizontal line, the average poisson of DIA. A and B is there, except on sheet 2, and there each poisson of each sample is listed).

4b-C. The poisson ratio increases in value as the loadings increase.

4b-D. The 4 and 5 percent asphalt content showed little difference in modulus strength, whereas the 6 percent showed a somewhat marked decrease in strength from the 4 and 5 percent asphalt samples.

5. Three plots of Poisson's Ratio vs. Load were graphed to show the change of poisson's ratio with increased load. These graphs are labeled #4, #5 and #6.

Conclusions

1. Modulus strength was approximately the same for the 4 percent and 5 percent asphalt samples with a slight edge given to the 5 percent samples (could be within testing error of one another). The 6 percent asphalt content samples showed a more marked decrease in modulus strength.

2. Poisson ratio continues to show an increase in value with larger loading as evident in these tests and previous tests.

3. Poisson ratio increased with the higher asphalt content particularly the 6 percent asphalt samples.

4. Increased blows changed the modulus strength in the neighborhood of approximately 25 percent from the 35 blows to 75 blows samples.

5. Assumed poisson ratio graph curves show more of a scatter in modulus values and run higher than the actual calculated poisson values that were graphed because our actual poissons were lower than .35 in most cases. Previous assumed poisson curves showed us some modulus values that were lower than the actual tested poisson values. This was when the actual poisson was higher than .35. Of course, the only poisson that will match the assumed poisson is when the actual tested poisson equals .35.

6. There was seemingly more creep in the 6 percent asphalt samples than the 4 and 5 percent as evident during testing.

7. There was a higher poisson at low blows in the 6 percent samples.

8. The test's horizontal millivolt reading from one cycle to another showed more inconsistency than the vertical millivolt readings, particularly in the light loadings where the aggregate structure is not compressed together in a more solid body structure as when there is heavier loads applied. The readings are also small and harder to measure.

LABORATORY MR INVESTIGATION
OF RECYCLED BITUMINOUS MIXES

Neil McGee
Foundations Lab Chief
Mn/DOT

DIFFERENT ADMIXTURES

TOPIC: To make a comparison of Resilient Modulus values from asphalt concrete marshall briquets of different admixtures.

SCOPE: To find out what effect different admixtures added to asphalt concrete would make on Resilient Modulus and Poisson Ratio values.

Procedure used:

The Mn/DOT Indirect Tensile Test Apparatus, which is a copy of a design by Gilbert Baladi of Michigan State University and Michigan's Department of Transportation, was used for the Resilient Modulus test. ASTM Specification D4123-82 was used except for the following:

1. Only one frequency was run and that was at 2 HZ. (The reason for running only one frequency was that, from previous testing with this apparatus at room temperature, all three specified frequencies run for Resilient Modulus fell on top of one another, so all values were the same.)

2. Two diameters were run on the samples, as ASTM specifications require, except for the samples containing trap rock. These samples were run on only one diameter because the surfaces were rough and the tester epoxied the sides where the horizontal transducers make contact.

3. Only room temperature tests were run.

Calculations

1. Total recoverable horizontal strain deflections in inches

$$H_T = \frac{\text{MV Horizontal}}{203{,}500 \text{ MV/inch}}$$

2. Total recoverable vertical strain deflections in inches

$$V_T = \frac{\text{MV Vertical}}{83{,}150}$$

3. Total resilient Poisson Ratio $\nu_{RT} = 3.59 \dfrac{\Delta HT}{\Delta V_T} - .27$

4. $\text{LOAD} = \dfrac{\text{MV LOAD}}{2500 \text{ MV/Full Scale}} \times 500 \text{ lb.} = P$

5. E_{RT} = Total resilient modulus of elasticity (PSI) =
   $P\,(\nu_{RT} + .27)/\tau \Delta H_T$

   Where
   P = LOAD
   τ = Sample Thickness
   ΔH_T = Total recoverable horizontal deformation
   ν^*_{RT} = Total resilient Poisson Ratio

6. E_{RT} using assumed Poisson Ratio of .35

$$E_{RT} = P \frac{(.35 + .27)}{\tau \Delta H_T}$$

7. Graphs were drawn of modulus vs. LOAD and Poisson Ratio vs. LOAD.

Graph Analyses by the Tester:

Some observations of the graphed results will follow:

1. Mix #2361 with trap rock samples are the #466 samples and ran as follows:

   a. The trap rock with control samples (466D4 & DS), Resilient Modulus ran approximately 500,000 psi+.

   b. The trap rock with latex additive (polysar) samples (466E), Resilient Modulus ran approximately 400,000 psi+.

c. The trap rock with neoprene additive samples (466F4 & F5), Resilient Modulus ran approximately 400,000 psi. –

d. Poisson ratios for all #466 samples containing trap rock ran low. The Poisson Value ($3.59 \frac{\Delta x}{\Delta y} - .27$) shows that there is a fairly small Δx movement to Δy movement in these trap rock samples. It would seem suggestive that this material would cause less of a rutting problem as compared to samples that show a higher Poisson ratio value at a given Resilient Modulus value.

e. The trap rock samples show an adequate to good Resilient Modulus.

2. Mix #2332 containing 60 percent recycled millings and 40 percent Barton of Scandia Aggregate with and without additives are the samples #467. These samples showed good Resilient Modulus Strength.

   a. The control samples (467C4 & C5) Resilient Modulus strength ran approximately 700,000 psi.

   b. The samples with the latex additive (polysar) ran just slightly lower in Resilient Modulus strength than the control and ran approximately 650,000 psi.

   c. The samples with the neoprene additive had the lowest Resilient Modulus strength of the #467s samples and ran approximately 600,000 psi.

   The Poisson Ratios of the #467 samples was somewhat varied and ran from a fairly low value to near an average value for Poisson Ratio.

3. Mix #2332 containing 40 percent recycled millings and 60 percent Barton

of Scandia aggregate with and without additives are the samples #468. These samples ran only fair to good in Resilient Modulus strength.

a. The control samples 468C4, C5, D4 and D5 Resilient Modulus strength ran approximately 500,000 to 550,000 psi. The C4 and C5 samples had 2.7% + add oil while the D4 and D5 samples had 3.4% + add oil. The 3.4% + add oil ran slightly stronger in Resilient Modulus strength.

b. The samples with the latex additive (polysar) #468 E4, E5 and E6 had a lower Modulus than the control samples and ran approximately 350,000 psi.

c. The samples with the neoprene additive #468 F4, F5 and F6 ran just slightly lower than the latex samples and ran approximately 330,000 to 340,000 psi.

Poisson ratios for the #468 samples showed a higher Poisson Ratio for the control samples than the additive samples.

Conclusion:

It seemed to this tester that the main governing factor as to Resilient Modulus strength and Poisson Ratio values was the aggregate body structure of the sample. There seemed to be a definite influence by additives on Resilient Modulus strength, but the additive influence on Resilient Modulus strength was not as pronounced as was the type of aggregate mixture the samples body structure was made of.

Some of the samples show quite low Poisson Ratio values. It seems suggestive that this might be an indication of samples that might have good resistance to rutting but, of course, samples of approximately the same Resilient Modulus values would have to be compared in order for this assumption to be considered.

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
466-D4 Dia.A	2361	Trap Rock (Control)	5.02%	5.3%	2008	2.519	85-100	-.12	29.1	613,710	2,483,400
"	"	"	"	"	"	"	"	-.06	61.2	520,190	1,566,850
"	"	"	"	"	"	"	"	.01	92.0	514,030	1,121,620
"	"	"	"	"	"	"	"	.02	122.8	507,970	1,103,140
"	"	"	"	"	"	"	"	.01	150.0	510,220	1,129,500
466-D5 Dia.A	2361	Trap Rock (Control)	5.02%	5.3%	2008	2.519	85-100	-.07	28.6	793,580	2,465,814
"	"	"	"	"	"	"	"	-.002	62.0	566,194	1,309,090
"	"	"	"	"	"	"	"	.07	90.4	515,630	935,280
"	"	"	"	"	"	"	"	.01	124.6	528,020	1,171,930
"	"	"	"	"	"	"	"	.12	151.4	517,000	816,033
466-E4 Dia.A	2361	Trap Rock with Latex	5.5%	5.5%	1828	2.514	85-100	-.15	22.0	532,550	2,818,530
"	"	"	"	"	"	"	"	-.11	40.4	453,610	1,725,280
"	"	"	"	"	"	"	"	-.07	59.8	445,695	1,392,960
"	"	"	"	"	"	"	"	-.05	77.6	410,670	1,136,190
"	"	"	"	"	"	"	"	.02	97.2	416,190	889,480
466-F4 Dia.A	2361	Trap Rock w/Neoprine	5.5%	2.4%	2909	2.599	85-100	-.06	30.7	525,270	1,551,390
"	"	"	"	"	"	"	"	.04	60.8	434,487	860,290
"	"	"	"	"	"	"	"	.10	90.9	414,590	693,990
"	"	"	"	"	"	"	"	.15	118.4	364,300	532,400
"	"	"	"	"	"	"	"	.11	148.8	370,260	598,140
466-F5 Dia.A	2361	Trap Rock w/Neoprine	5.5%	2.4%	2909	2.599	85-100	-.06	31.4	490,650	926,390
"	"	"	"	"	"	"	"	.13	61.6	424,026	660,870
"	"	"	"	"	"	"	"	.16	91.6	394,680	565,630
"	"	"	"	"	"	"	"	.10	121.6	395,990	658,940
"	"	"	"	"	"	"	"	.12	149.2	380,770	609,490

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
467-C4 Dia.A	2332	60% Rcycld Millings 40% Barton of Scandia (Control)	2.0% + add oil	3.6%	2397	2.435	200-300	-.12	48.6	718,470	1,129,310
"	"	"	"	"	"	"	"	.18	93.9	689,800	960,050
"	"	"	"	"	"	"	"	.23	142.1	707,470	885,890
"	"	"	"	"	"	"	"	.24	187.8	708,850	857,190
"	"	"	"	"	"	"	"	.24	235.0	699,980	846,020
467-C4 Dia.B	"	"	"	"	"	"	"	.10	49.8	655,980	1,106,880
"	"	"	"	"	"	"	"	.12	94.2	689,980	1,094,450
"	"	"	"	"	"	"	"	.10	141.4	721,740	1,204,750
"	"	"	"	"	"	"	"	.14	189.0	718,240	1,073,540
"	"	"	"	"	"	"	"	.15	236.4	716,470	1,060,090
467-C5 Dia.A	"	"	"	"	"	"	"	.26	50.4	733,730	856,050
"	"	"	"	"	"	"	"	.19	99.0	707,920	951,810
"	"	"	"	"	"	"	"	.23	147.0	696,680	860,970
"	"	"	"	"	"	"	"	.24	195.6	685,230	830,570
"	"	"	"	"	"	"	"	.24	243.2	669,510	809,960
467-C5 Dia.B	"	"	"	"	"	"	"	.26	50.4	632,490	956,390
"	"	"	"	"	"	"	"	.19	99.0	629,590	934,180
"	"	"	"	"	"	"	"	.23	147.0	624,180	763,370
"	"	"	"	"	"	"	"	.24	195.6	632,850	743,800
"	"	"	"	"	"	"	"	.24	243.2	608,940	728,270

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
467-D4 Dia.A	2332	60% Rcycld Millings 40% Barton of Scandia (Control)	1.3% + add oil	6.6%	3083	2.407	200-300	-.03	52.0	568,960	1,472,050
"	"	"	"	"	"	"	"	.05	147.0	597,490	1,170,380
"	"	"	"	"	"	"	"	.06	195.8	613,420	1,160,120
"	"	"	"	"	"	"	"	.05	243.6	626,800	1,228,980
467-D4 Dia.B	"	"	"	"	"	"	"	-.05	50.6	602,810	1,718,900
"	"	"	"	"	"	"	"	-.09	98.8	575,770	2,013,760
"	"	"	"	"	"	"	"	-.03	147.2	574,080	1,470,710
"	"	"	"	"	"	"	"	-.02	195.6	575,380	1,423,840
"	"	"	"	"	"	"	"	.001	245.5	600,570	1,374,120
467-D5 Dia.A	"	"	"	"	"	"	"	.05	47.6	671,050	1,306,840
"	"	"	"	"	"	"	"	.01	94.0	627,060	1,404,220
"	"	"	"	"	"	"	"	.01	131.0	631,440	1,392,520
"	"	"	"	"	"	"	"	.0004	175.6	665,080	1,524,600
"	"	"	"	"	"	"	"	.00006	241.6	667,820	1,533,880
467-D5 Dia.B	"	"	"	"	"	"	"	.08	48.8	829,070	2,753,990
"	"	"	"	"	"	"	"	.002	93.6	742,715	1,697,870
"	"	"	"	"	"	"	"	-.06	135.2	745,260	2,215,140
"	"	"	"	"	"	"	"	-.02	177.4	650,699	1,608,980
"	"	"	"	"	"	"	"	-.06	233.6	655,490	1,977,460

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
467-E4 Dia.A	2332	60% Rcycld Millings 40% Barton of Scandia with Latex (polysar)	2.06% + add oil	3.9%	3275	2.447	200-300	.05	48.1	635,210	1,212,710
"	"	"	"	"	"	"	"	.12	95.0	610,360	958,070
"	"	"	"	"	"	"	"	.11	134.0	586,050	945,020
"	"	"	"	"	"	"	"	.08	171.8	604,390	1,082,870
467-E4 Dia.B	"	"	"	"	"	"	"	-.04	48.4	708,320	1,877,350
"	"	"	"	"	"	"	"	.06	94.0	693,410	1,298,610
"	"	"	"	"	"	"	"	.19	135.2	637,140	852,180
"	"	"	"	"	"	"	"	.21	171.6	640,640	824,080
"	"	"	"	"	"	"	"	.18	232.8	639,780	882,620
467-E5 Dia. A	"	"	"	"	"	"	"	-.01	47.6	593,120	1,422,720
"	"	"	"	"	"	"	"	.05	94.8	613,290	1,174,860
"	"	"	"	"	"	"	"	.04	135.2	612,760	1,226,730
"	"	"	"	"	"	"	"	.02	172.8	658,340	1,416,164
"	"	"	"	"	"	"	"	.07	235.0	631,470	1,148,140
467-E5 Dia.B	"	"	"	"	"	"	"	.01	47.2	765,320	1,713,070
"	"	"	"	"	"	"	"	.18	94.0	657,170	909,770
"	"	"	"	"	"	"	"	.24	133.4	667,500	816,660
"	"	"	"	"	"	"	"	.26	175.0	688,370	801,080
"	"	"	"	"	"	"	"	.31	238.0	684,000	737,380

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
467-F4 Dia.A	2332	60% Rcycld Millings 40% Barton of Scandia w/Neoprene	2.06 + add oil	4.3%	3237	2.435	200-300	.06	95.2	678,030	1,270,400
"	"	"	"	"	"	"	"	.04	132.8	664,920	1,346,830
"	"	"	"	"	"	"	"	.06	172.4	715,850	1,334,690
"	"	"	"	"	"	"	"	.07	231.4	670,320	1,214,916
467-F4 Dia.B	"	"	"	"	"	"	"	.01	47.2	627,140	1,407,920
"	"	"	"	"	"	"	"	-.004	93.8	599,980	1,398,970
"	"	"	"	"	"	"	"	.13	133.6	605,360	928,050
"	"	"	"	"	"	"	"	.15	174.6	609,350	903,450
"	"	"	"	"	"	"	"	.17	234.0	601,280	856,580
467-F5 Dia.A	"	"	"	"	"	"	"	-.05	48.6	606,950	1,698,940
"	"	"	"	"	"	"	"	.16	95.0	584,290	844,810
"	"	"	"	"	"	"	"	.12	133.6	554,340	890,050
"	"	"	"	"	"	"	"	.08	174.8	558,150	984,490
"	"	"	"	"	"	"	"	.05	235.2	545,000	1,045,790
467-F5 Dia.B	"	"	"	"	"	"	"	-.02	47.8	628,570	1,563,170
"	"	"	"	"	"	"	"	.14	92.4	622,630	936,730
"	"	"	"	"	"	"	"	.09	136.0	613,430	1,060,560
"	"	"	"	"	"	"	"	.10	176.8	653,220	1,092,900
"	"	"	"	"	"	"	"	.16	239.6	644,230	920,080

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STABILITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
468-C4 Dia.A	2332	40% Rcycld Milling 60% Barton of Scandia (Control)	2.7% + add oil	6.61%	2000	2.371	200-300	.14	35.2	536,320	812,000
"	"	"	"	"	"	"	"	.16	71.4	459,790	658,830
"	"	"	"	"	"	"	"	.10	107.2	459,380	766,250
"	"	"	"	"	"	"	"	.16	142.8	487,760	2,684,110
"	"	"	"	"	"	"	"	.11	177.8	487,190	784,640
468-C4 Dia.B	"	"	"	"	"	"	"	-.07	38.4	440,410	1,392,000
"	"	"	"	"	"	"	"	.01	74.0	448,870	988,290
"	"	"	"	"	"	"	"	.06	109.2	467,060	865,920
"	"	"	"	"	"	"	"	.09	144.4		
"	"	"	"	"	"	"	"	.12	174.0	469,710	754,740
468-C5 Dia.A	"	"	"	"	"	"	"	-.05	42.0	641,350	1,290,870
"	"	"	"	"	"	"	"	-.07	81.2	538,900	1,294,070
"	"	"	"	"	"	"	"	-.04	109.0	524,340	968,590
"	"	"	"	"	"	"	"	.02	135.2	515,050	861,410
"	"	"	"	"	"	"	"	.01	163.2	528,920	903,500
468-C5 Dia.B	"	"	"	"	"	"	"	.04	41.6	641,350	1,290,870
"	"	"	"	"	"	"	"	-.01	80.8	538,900	1,294,070
"	"	"	"	"	"	"	"	.07	111.2	524,340	968,590
"	"	"	"	"	"	"	"	.10	138.8	515,050	861,410
"	"	"	"	"	"	"	"	.09	165.6	528,920	903,500

ADMIXTURE DATA RESULTS

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
468-D4 Dia.A	2332	~~60%~~ Rcycld Millings ~~40%~~ Barton of Scandia (Control)	3.4% + add oil	4.0%	2400	2.419	200-300	.02	48.1	440,140	941,910
"	"	"	"	"	"	"	"	.14	95.0	502,080	755,760
"	"	"	"	"	"	"	"	.20	138.4	495,260	658,550
"	"	"	"	"	"	"	"	.28	189.2	528,400	590,980
"	"	"	"	"	"	"	"	.27	236.4	517,290	595,850
468-D4 Dia.B	"	"	"	"	"	"	"	-.03	49.2	528,120	1,391,650
"	"	"	"	"	"	"	"	.09	93.9	528,690	902,050
"	"	"	"	"	"	"	"	.13	140.3	526,140	821,060
"	"	"	"	"	"	"	"	.15	186.3	512,190	752,400
"	"	"	"	"	"	"	"	.21	238.2	508,170	662,720
468-D5 Dia.A	"	"	"	"	"	"	"	.34	35.2	729,880	743,430
"	"	"	"	"	"	"	"	.20	68.0	556,310	733,370
"	"	"	"	"	"	"	"	.19	101.2	572,660	777,230
"	"	"	"	"	"	"	"	.20	135.1	569,030	752,530
"	"	"	"	"	"	"	"	.26	168.8	566,200	668,460
468-D5 Dia.B	"	"	"	"	"	"	"	.17	36.4	599,730	838,670
"	"	"	"	"	"	"	"	.25	68.6	553,800	656,080
"	"	"	"	"	"	"	"	.25	102.0	550,100	654,460
"	"	"	"	"	"	"	"	.20	136.4	532,560	698,380
"	"	"	"	"	"	"	"	.26	169.2	523,720	608,260

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
468-E4 Dia.A	2332	40% Rcycld Millings 60% Barton of Scandia with latex (polysar)	3.4% + add oil	4.6%	2415	2.401	200-300	-.06	68.6	392,250	1,148,440
"	"	"	"	"	"	"	"	-.04	98.2	375,580	996,350
"	"	"	"	"	"	"	"	.02	133.2	359,980	777,890
"	"	"	"	"	"	"	"	-.04	160.6	354,990	960,224
"	"	"	"	"	"	"	"				
468-E4 Dia.B	"	"	"	"	"	"	"	-.14	41.6	434,390	2,089,300
"	"	"	"	"	"	"	"	.03	68.0	398,630	813,141
"	"	"	"	"	"	"	"	-.03	98.6	385,950	990,410
"	"	"	"	"	"	"	"	-.03	138.0	366,890	936,599
"	"	"	"	"	"	"	"	.01	160.2	366,560	804,580
468-E5 Dia.A	"	"	"	"	"	"	"	.04	68.0	391,136	1,072,813
"	"	"	"	"	"	"	"	-.08	98.4	379,669	1,241,939
"	"	"	"	"	"	"	"	-.06	134.8	342,980	1,000,800
"	"	"	"	"	"	"	"	-.05	162.8	350,808	1,002,319
468-E5 Dia.B	"	"	"	"	"	"	"	-.01	41.2	386,750	1,386,660
"	"	"	"	"	"	"	"	-.03	88.6	376,430	962,030
"	"	"	"	"	"	"	"	-.06	98.0	377,020	1,111,810
"	"	"	"	"	"	"	"	-.05	136.8	351,920	972,730
"	"	"	"	"	"	"	"	-.04	160.6	325,580	871,820
468-E6 Dia.A	"	"	"	"	"	"	"	-.02	67.2	351,992	885,688
"	"	"	"	"	"	"	"	-.03	97.8	344,340	890,580
"	"	"	"	"	"	"	"	00	134.8	325,760	790,140
"	"	"	"	"	"	"	"	.02	160.8	308,920	691,120

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
468-E6 Dia.B	"	"	"	"	"	"	"	-.04	67.2	393,980	782,700
"	"	"	"	"	"	"	"	-.01	97.8	347,140	831,900
"	"	"	"	"	"	"	"	-.05	134.8	354,730	688,020
"	"	"	"	"	"	"	"	-.03	160.8	331,010	680,760
468-F4 Dia.A	2332	40% Rcycld Millings 60% Barton of Scandia w/neoprene	3.4% + add oil	4.2%	2837	2.401	200-300	-.07	41.2	383,930	1,215,240
"	"	"	"	"	"	"	"	-.01	68.1	333,520	803,470
"	"	"	"	"	"	"	"	-.04	95.6	308,920	826,501
"	"	"	"	"	"	"	"	-.06	135.6	328,620	985,430
"	"	"	"	"	"	"	"	-.10	164.4	314,310	1,161,070
468-F4 Dia.B	"	"	"	"	"	"	"	.07	43.7	383,250	1,184,470
"	"	"	"	"	"	"	"	00	68.6	362,250	819,010
"	"	"	"	"	"	"	"	.06	97.0	364,170	685,060
"	"	"	"	"	"	"	"	00	138.4	328,830	746,220
"	"	"	"	"	"	"	"	-.02	166.8	319,610	781,670
468-F5 Dia.A	"	"	"	"	"	"	"	-.01	43.7	435,240	1,632,920
"	"	"	"	"	"	"	"	.01	69.6	407,730	888,850
"	"	"	"	"	"	"	"	.02	95.6	386,630	831,470
"	"	"	"	"	"	"	"	.02	135.2	354,630	766,310
"	"	"	"	"	"	"	"	-.04	162.0	350,010	939,320
468-F5 Dia.B	"	"	"	"	"	"	"	-.19	42.5	360,250	2,679,880
"	"	"	"	"	"	"	"	.01	69.6	349,650	780,214
"	"	"	"	"	"	"	"	-.04	109.0	320,120	872,780
"	"	"	"	"	"	"	"	-.04	148.8	318,980	862,780
"	"	"	"	"	"	"	"	-.01	189.2	311,880	739,860

SAMPLE NO.	MIX NO.	MATERIAL TYPE	ASPHALT CONTENT	AIR VOIDS	STA-BIL-ITY	BULK SPG.	PEN	POISSON RATIO	LOAD (lbs)	Mr (Actual Poisson Used)	Mr (Assumed Poisson of .35 used)
468-F6 Dia.A	"	"	"	"	"	"	"	-.07	68.0	371,960	1,154,060
"	"	"	"	"	"	"	"	-.05	107.6	360,490	996,070
"	"	"	"	"	"	"	"	00	147.2	330,280	757,030
"	"	"	"	"	"	"	"	-.03	190.0	341,910	871,500
468-F6 Dia.B	"	"	"	"	"	"	"	-.07	67.8	371,660	1,150,660
"	"	"	"	"	"	"	"	-.06	104.0	321,560	962,740
"	"	"	"	"	"	"	"	00	148.0	324,560	753,530
"	"	"	"	"	"	"	"	-.04	192.4	326,760	890,540

LABORATORY INVESTIGATION OF FIELD CORES CONTAINING RECYCLED BITUMINOUS MIX

Neil McGee
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Mn/DOT

Topic

A comparison investigation of the Resilient Modulus values of the Recycled Mix 2332 and the Controlled Bituminous Mix 2331 from T.H. 12.

Scope

The comparison investigation will involve comparing resilient moduli of the recycled and controlled mixes placed in T.H. 12. The mixes were placed in the roadway at the same period of time. The samples tested were drilled field cores. They were placed about 12 years ago.

Features of the Testing

1. The sample numbers will have meanings as below:

   M will stand for cores taken from the mid-depth of the asphalt concrete of the roadway; e.g., MC1: all samples were taken from the mid-depth of the asphalt concrete of the roadway. The second letter will stand for the location of the sample between the centerline and the shoulder of the roadway. The 1 will stand for samples that are of the recycled mix. The 2 will stand for the controlled mix.

2. The samples were calculated and graphed with the number 1's (recycled mix) placed directly above or below the number 2's (controlled mix). The number 1's and number 2's graphed on the same sheet are at the same location with respect to the centerline and shoulder of the roadway.

3. Both an actual poisson Resilient Modulus value and an assumed .35 poisson value were used in the Resilient Modulus calculations and graphing.

4. The samples were tested in our homemade environmental box. Some rubbing of the loading piston on the hole through the box was noted on some of the samples, but the tester could not determine if any noticeable undesirable effects resulted, as the loads should have been high with respect to any friction.

Procedure used -

The Mn/DOT Indirect Tensile Test Apparatus, which is a copy of a design by Gilbert Baladi of Michigan State University and Michigan's Department of Transportation, was used for the Resilient Modulus test. ASTM Specification D4123-82 was used except for the following:

1. that only one frequency was run and that was at 1 HZ; (The reason for running only one frequency was that, from previous testing with this apparatus at room temperature, all three specified frequencies run for Resilient Modulus fell on top of one another, so all values were the same.)

2. that only one diameter was tested;

3. that the sample was plastered with dental plaster to evenly distribute the load under the loading strips, as the samples are rough cut from the field;

4. that the horizontal transducers are approximately .12 inches off center of the sample due to the testing apparatus being made strictly for 4-inch diameter samples and the field cores are approximately 4.2 to 4.25 inches in diameter. This lends to the fact that now the poisson values will be

slightly lower than they should be and it also lends to the fact that the Resilient Modulus values will be slightly higher than they should be.

Calculations

1. Total recoverable horizontal strain deflections in inches
   $$\Delta H_T = \frac{\text{MV Horizontal}}{203{,}500 \text{ MV/inch}}$$

2. Total recoverable vertical strain deflections in inches
   $$\Delta V_T = \frac{\text{MV Vertical}}{83{,}150}$$

3. Total resilient poisson ratio $\nu_{RT} = 3.59 \dfrac{\Delta HT}{\Delta V_T} -.27$

4. $\text{LOAD} = \dfrac{\text{MV LOAD}}{2500 \text{ MV/Full Scale}} \times 500 \text{ lb.} = P$

5. E_{RT} = Total resilient modulus of elasticity (PSI) = $P\ (\nu_{RT} + .27)/t\ \Delta H_T$

   Where P = LOAD
   t = Sample Thickness
   ΔH_T = Total recoverable horizontal deformation
   ν_{RT} = Total resilient poisson ratio

6. E_{RT} using assumed poisson ratio of .35
   $$E_{RT} = P\ \frac{(.35 + .27)}{t\ \Delta H_T}$$

7. Graphs were drawn of modulus vs. LOAD and poisson ratio vs. LOAD.

Design Criteria

Table 2 summarizes the initial trial mix data using 50/50 and 60/40 ratios of salvaged bituminous and virgin aggregate. The recycled mix was produced at the ratio of 60 percent salvage and 40 percent virgin material throughout the project.

TABLE 2 INITIAL TRIAL MIX DATA

	Recycled Base & Binder		Recycled Shoulder Wear		Conventional Shoulder Wear
Proportion (Salvage/virgin)	50/50	60/40	50/50	60/40	0/100
% Asphalt Mixed	3.1	2.6	4.3	4.0	7.5
Density (lb/cu.ft.)	136.6	138.2	139.9	141.1	138.6
Stability (lbf)	2064	2402	1770	1939	922
Percent Voids	6.9	8.2	4.2	3.4	5.1
CWA* Loss (%)	10.8	4.9	4.3	3.9	4.5
% Asphalt Recommended	3.3	3.0	4.3	3.8	7.3

* = Cold Water Abrasion
1 lb./cu.ft. = 16.02 Kg/m^3
1 lbf - 4.448N

The initial trial mix recommendations were superseded by several updated designs because of changes in the virgin aggregate gradation.

The virgin aggregate was required to meet the standard gradation and quality requirements for bituminous aggregate. The mix design was based on guidelines that were added to the special provisions to establish the amount of additional asphalt required.

Marshall Stability (50 blows):	3,000 lbf max.
	500 lbf min.
Voids in Mix:	6% base and binder
	4% shoulder wear
Cold water Abrasion Loss:	15% max.

The samples were taken mostly from the base region of the roadway. The controlled mix was 2331 and the recyled was 2332.

Graph Analysis by the Tester:

1. The samples designated with a "1" on the recycled samples showed more sensitivity to temperature change, as far as Resilient Modulus is concerned. Both the recycled (number 1) and the controlled (number 2) samples showed far less Resilient Modulus change due to changing temperature than fresh asphalt concrete would show. This is probably primarily due to the aging process.

2. The samples of both the recycled and controlled mixes showed good modulus strength. There was no real noticeable plastic off-setting, even at the 320 lb. loadings. The modulus values are, of course, slightly higher than they should be because the apparatus was designed for 4-inch diameter samples. The number 1 samples showed Resilient Modulus strength of approximately 300,000 psi to 500,000 psi greater strength than the number 2 samples, except for ME1 and ME2 which had a reversed result.

3. Many of the number 2 designated samples had a rough-textured surface, probably due to the aggregate shattering loose during the coring operation. It has been pointed out to me that perhaps the controlled mix had a little less asphalt when layed down.

4. Resilient Modulus values calculated with an assumed .35 showed more scattering than the Resilient Modulus values calculated by using actual determined poisson values.

5. Poisson values continue to show an increased value as loadings are increased; as in previous testing, until approximately the 255 lb. loading, in most cases. At the heavier loadings, the poisson starts to level out or take a small dip in value. The poisson showed more scattering than in some of the previously tested laboratory samples and did not always hold true to form.

6. Poisson values in most cases changed with temperature, being lower at low temperature and higher at high temperatures.

7. Mr Values (Resilient Modulus Values) tested are listed on the following pages.

TEMP.	SAMPLE NO.	POISSON RATIO	Mr. ACT.	Mr. ASSUMED	LOAD (LBS)
35.6°F	MA1 Recycle	-.16	1,962,380	11,165,140	66.4
	MA1	.002	1,440,660	3,283,960	130.2
	MA1	.1	1,297,460	2,176,980	194.2
	MA1	.15	1,252,770	1,833,067	258
	MA1	.21	1,259,460	1,614,240	323.2
72°F	MA1	.09	1,534,170	2,615,370	67.4
	MA1	.15	1,329,080	1,984,160	129.8
	MA1	.28	1,155,560	1,293,460	200
	MA1	.31	1,016,310	1,093,680	258
	MA1	.32	913,360	966,260	321.8
102°F	MA1	-.18	1,576,800	11,165,130	66.4
	MA1	.21	930,755	1,208,840	131.8
	MA1	.34	698,850	705,500	193
	MA1	.46	606,470	514,900	256.2
	MA1	.48	548,214	451,180	320.2
30.4°F	MA2 Virgin	-.2	851,550	7,201,130	67.6
	MA2	-.05	777,610	2,226,730	129.6
	MA2	-.03	779,740	2,030,250	194.4
	MA2	-.01	768,180	1,818,040	256
	MA2	.05	792,420	1,534,160	322.6
77°F	MA2	-.05	1,260,790	3,611,220	67.8
	MA2	-.01	849,030	2,030,250	129.6
	MA2	.01	780,790	1,744,130	193.2
	MA2	.06	660,340	1,225,570	232.4
	MA2	.08	713,100	1,262,640	322.4
103.7°F	MA2	-.04	1,171,180	3,186,080	65.8
	MA2	.13	721,350	1,121,950	130.6
	MA2	.22	619,390	790,870	196
	MA2	.20	562,340	746,840	256.6
	MA2	.22	532,560	669,113	321.6
18.5°F	MB1	.01	1,017,630	2,254,270	66.6
	MB1	.01	981,170	2,152,460	131.6
	MB1	.15	969,440	1,439,380	194.4
	MB1	.07	1,000,320	1,810,230	256.4
	MB1	.10	1,033,610	1,743,930	325.2
77°F	MB1	.16	825,637	1,199,440	66
	MB1	.19	805,670	1,080,325	131
	MB1	.18	816,146	1,113,730	195.2
	MB1	.20	786,650	1,038,810	260.4
	MB1	.23	788,420	980,150	323.6

TEMP.	SAMPLE NO.	POISSON RATIO	Mr. ACT.	Mr. ASSUMED	LOAD (LBS)
104°F	MB1	0	482,140	1,114,430	67
	MB1	.39	465,330	438,570	129.6
	MB1	.31	457,980	485,440	194.4
	MB1	.27	450,620	513,510	256.4
	MB1	.34	428,600	439,150	320.4
38.7°F	MB2	.1	507,130	859,530	67.2
	MB2	.25	513,180	608,370	130.8
	MB2	.27	518,860	595,460	193.2
	MB2	.28	520,690	583,160	257.2
	MB2	.28	511,305	576,742	322
77°F	MB2	.24	426,280	514,680	67.4
	MB2	.32	355,400	373,430	127
	MB2	.33	342,720	351,760	193.2
	MB2	.39	332,190	312,590	256
	MB2	.44	310,150	272,180	317
101.3°F	MB2	.42	309,808	276,780	66
	MB2	.62	263,520	183,730	130
	MB2	.91	245,972	128,970	193.6
	MB2	.76	226,170	135,580	255.2
	MB2	.96	216,180	109,190	318
35°F	MC1	-.11	755,210	3,015,530	66.6
	MC1	.04	813,060	1,611,230	129.4
	MC1	.12	903,960	1,429,730	195.2
	MC1	.18	918,070	1,274,247	258.4
	MC1	.16	954,280	1,362,490	322.8
77°F	MC1	-.02	825,660	2,060,730	66.2
	MC1	.1	798,870	1,354,100	130.5
	MC1	.25	788,820	938,095	194
	MC1	.27	795,140	909,230	257.4
	MC1	.29	783,420	866,900	322
104°F	MC1	.28	611,010	691,190	68
	MC1	.34	562,170	572,990	130
	MC1	.24	458,880	560,560	192.8
	MC1	.42	513,080	460,750	254.4
	MC1	.38	499,750	475,760	320
30.4°F	MC2	-.21	417,450	4,441,250	66.1
	MC2	-.07	318,070	984,600	127.7
	MC2	.0022	336,640	766,880	192.4
	MC2	.05	352,920	680,910	254.8
	MC2	.08	370,540	653,170	320.8
77°F	MC2	-.08	353,100	1,172,200	64.8
	MC2	.02	336,010	720,410	130.2
	MC2	.05	324,340	625,410	196.8
	MC2	.09	320,980	550,774	258
	MC2	.12	334,450	536,680	325.2

TEMP.	SAMPLE NO.	POISSON RATIO	Mr. ACT.	Mr. ASSUMED	LOAD (LBS)
104°F	MC2	-.1	316,120	1,148,950	68.4
	MC2	.04	267,970	531,150	131
	MC2	.06	280,660	527,420	194
	MC2	.13	284,500	436,930	256.4
	MC2	.17	284,670	404,990	318.6
28°F	MD1	-.23	1,497,670	20,839,774	128.5
	MD1	-.18	1,357,250	9,450,370	195.6
	MD1	-.13	1,279,354	5,850,480	266.4
	MD1	-.05	1,236,632	3,508,070	334.8
77°F	MD1	-.02	1,144,820	2,868,140	130.6
	MD1	.05	1,096,620	2,095,790	195.2
	MD1	.09	1,015,970	1,756,780	261.8
	MD1	.08	961,780	1,704,380	331.6
104°F	MD1	.10	1,065,910	1,793,015	66.8
	MD1	.17	664,700	944,660	131
	MD1	.17	540,286	762,040	194
	MD1	.19	478,630	649,730	258
	MD1	.20	437,770	573,330	320.4
40°F	MD2	-.05	922,870	2,640,780	65.9
	MD2	.05	938,760	1,824,710	129.6
	MD2	.23	940,650	1,455,547	195.2
	MD2	.16	953,430	1,382,090	260
	MD2	.10	936,230	1,553,180	322
77°F	MD2	-.12	755,490	3,190,160	128.6
	MD2	-.01	729,390	1,712,930	194
	MD2	.02	724,913	1,543,670	257.8
	MD2	.01	716,600	1,587,640	320
105°F	MD2	-.06	579,270	1,719,110	66
	MD2	.22	535,100	682,440	131
	MD2	.33	511,840	533,000	194.4
	MD2	.26	483,790	566,860	256.8
	MD2	.28	458,030	520,940	320
33°F	ME1	.07	664,540	1,209,480	64.9
	ME1	-.02	714,720	1,745,680	128.8
	ME1	.04	716,330	1,429,780	195
	ME1	.05	775,340	2,210,510	257
	ME1	.02	792,150	1,715,570	322.2
77°F	ME1	-.03	460,750	1,202,020	64.5
	ME1	-.01	579,000	1,375,870	129.2
	ME1	.04	623,570	1,230,610	192.6
	ME1	.04	640,810	1,273,960	259.2
	ME1	.07	672,640	1,220,500	322

TEMP.	SAMPLE NO.	POISSON RATIO	Mr. ACT.	Mr. ASSUMED	LOAD (LBS)
101°F	ME1	.14	558,220	834,890	67.2
	ME1	.08	512,490	913,790	132.8
	ME1	.1	499,090	837,760	194.8
	ME1	.1	512,230	864,930	257.2
	ME1	.1	505,340	839,920	323
33°F	ME2	-.14	1,213,050	5,648,890	65.6
	ME2	-.01	1,159,292	2,768,470	128.6
	ME2	.03	1,148,420	2,340,620	194.8
	ME2	.06	1,140,130	2,163,330	259.6
	ME2	.11	1,121,090	1,824,800	321.4
77°	ME2	-.16	1,203,030	7,026,670	68
	ME2	.10	1,059,430	1,791,110	130
	ME2	.08	1,027,914	1,826,180	194.4
	ME2	.29	964,350	1,071,140	255
	ME2	.12	889,930		322
100°F	ME2	.23	841,870	1,052,710	65.2
	ME2	.22	769,960	971,330	131.6
	ME2	.13	733,530	947,550	194.4
	ME2	.22	709,340	892,300	255.6
	ME2	.22	682,410	856,490	321.6
28°F	MF1	.29	2,107,220	2,342,870	67.6
	MF1	.15	1,375,150	2,047,960	130
	MF1	.25	1,288,150	1,542,270	195.8
	MF1	.25	1,197,690	1,441,400	263.4
	MF1	.23	1,181,700	1,457,380	333.6
77°F	MF1	.38	2,003,970	1,911,960	66.2
	MF1	.36	1,439,170	1,416,640	130
	MF1	.36	1,228,360	1,216,250	201.2
	MF1	.33	1,121,690	1,161,040	268
	MF1	.37	1,067,860	1,029,830	332.8
103.3°F	MF1	.4	2,227,158	2,055,010	67.2
	MF1	.13	1,213,830	1,818,040	140.2
	MF1	.64	1,051,270	715,850	191.4
	MF1	.38	677,768	642,340	257
	MF1	.42	640,720	579,120	318.6
28°F	MF2	-.08	1,355,930	4,442,270	67.4
	MF2	-.1	755,200	2,840,680	129.3
	MF2	-.05	710,660	1,993,090	194.4
	MF2	-.04	648,330	1,742,330	256.8
	MF2	-.04	605,930	1,662,960	324.4
77°F	MF2	-.02	1,014,898	2,363,601	66.6
	MF2	.05	732,460	1,403,400	130.8
	MF2	-.02	627,346	1,488,670	193.6
	MF2	.09	611,008	1,119,760	322.8

TEMP.	SAMPLE NO.	POISSON RATIO	Mr. ACT.	Mr. ASSUMED	LOAD (LBS)
103°F	MF2	-.07	828,500	2,518,280	131
	MF2	.03	545,780	1,139,780	195
	MF2	.11	522,393	843,640	256
	MF2	.16	497,828	716,680	328
32.7°F	MG1	-.19	1,903,145	15,033,400	67.2
	MG1	-.15	1,252,930	6,482,660	130.4
	MG1	.05	1,135,274	2,170,000	194
	MG1	-.01	1,038,020	2,463,680	258
	MG1	.04	1,006,880	2,015,890	324.4
77°F	MG1	.17	1,748,202	2,453,370	131.6
	MG1	.03	1,246,600	2,552,940	194
	MG1	.19	1,186,620	1,610,720	259.2
	MG1	.24	980,460	1,195,100	323.2
101.5°F	MG1	.14	1,327,030	1,984,088	128.6
	MG1	.21	809,800	1,052,250	195.2
	MG1	.33	660,540	686,940	256.4
	MG1	.35	574,300	576,840	323.6
31.5°F	MG2	-.17	1,175,375	7,560,340	69.2
	MG2	.002	812,170	1,848,900	132
	MG2	.05	740,030	1,418,840	194.8
	MG2	.1	718,728	1,203,640	260
	MG2	.1	702,900	1,189,150	320
77°F	MG2	.12	1,522,640	2,418,140	66.4
	MG2	.14	792,920	1,210,890	133
	MG2	.32	677,970	714,400	201.4
	MG2	.36	606,709	597,280	264.4
	MG2	.43	584,617	520,690	329.8
104°F	MG2	.19	1,951,013	2,606,475	66.8
	MG2	.31	744,830	791,480	130.4
	MG2	.46	603,730	514,750	196
	MG2	.53	522,506	403,613	258.6
	MG2	.60	486,920	345,430	320.6

LABORATORY MR TESTS
ON TEST TRACK BASE AGGREGATES

Neil McGee
Foundation Lab Chief
Mn/DOT

Topic: Resilient Modulus tests run on test track base materials. Samples of base materials run for Resilient Modulus Values were:

1. GS88133 Special No. 1 (Pit 86079)
2. GS88133 Special No. 6 (Pit 86079)
3. GS88234 (Source 173006 Meridian at Waite Park)

Scope: To run a Resilient Modulus test on the above listed samples to aid in the determination of what materials to use for bases in the test track. The Resilient Modulus value will also aid in the determination of the bases thicknesses to be used in the test track.

Procedures: AASHTO T274-2(86) specifications were used, except for the following:

1. The compaction process was based on the T180 proctor test. An electric vibratory hammer was used to densify the samples. The densities were right on to very close to the T180 proctor values. 7 lifts with a vibration time of 15 seconds per lift was used.

2. An additional seating load was used to extend the seating load to 30 p.s.i., instead of just 20 p.s.i. and a 30 p.s.i. load was used at the 20 p.s.i. confinement pressure test. The tester thought it would be good to see if the samples would take an upward turn in modulus values at an increased loading such as the 30 p.s.i. loading.

3. The transducers were placed at 1/3 pts., rather than 1/4 pts. due to the length of the transducer probes.

Description of the Testing:

The Resilient Modulus testing was done with a Materials Testing System (MTS machine). A repeated loading haversine loading wave was used to apply the loads to the samples. A one second load duration and a total cycle of two seconds was used in the loading process. The samples were 6 inches by approximately 12 inches in size. The transducer holder rings holds two transducers of good sensitivity.

Results:

The results are shown on the graph and the work sheet.

LABORATORY MR TESTS ON TEST TRACK SUBGRADE SOILS

Neil McGee
Foundation Lab Chief
Mn/DOT

COLD REGION FACILITY - SUBGRADE SOILS

BY NEIL MCGEE 5/27/88

Topic: Resilient Modulus Test run on test track subgrade soils No. 563, 564, 565 and 566.

Scope: Resilient modulus tests were run on the new Mn/DOT test track subgrade soils at Optimum Moisture Content and Optimum Moisture Content plus 2 percent. The tests are to give parameters that will aid in the design of the track and will also show variation of Resilient Modulus between and O.M.C. and an O.M.C. plus 2 percent increase in water content.

Procedures: AASHTO T274-2(86) specifications were used in the testing procedure, except for the following:

1. The compaction process was based on a T99 proctor and the impact hammer method was used.

    1a. 5 lifts were employed in the compaction of the 2.8 inch diameter samples at 7 blows/lift.

    1b. The number of lifts and blows were equated to the energy of the T99 proctor.

Description of the Testing: The resilient modulus testing was done with a Materials Testing System (MTS) machine. It applied the repeated loading haversine wave. A .1 second load duration and a total cycle of 2 seconds was used in the loading process. The clamps are of a hinged-type holding three LVDT's. The clamps were aligned parallel to one another and perpendicular to the loading axis by means of machined guide rods with tabs to place the clamps at approximately the quarter points on the samples. The readout was taken on a Nicollet Oscilloscope. A non-contacting horizontal deflection measurement system was used to determine Poisson ratios.

Test Results: The samples had a maximum variation from one another of 6.1% clay content difference between the leanest and fattest of them and three of the samples SiL content were primarily the same with one sample having from 12-14 percent greater SiL content than the other three samples.

The Resilient Modulus Tests run at O.M.C. and their values taken from the 3 p.s.i. loadings ranged from approximately 11,500 p.s.i. to 21,000 p.s.i. on the four samples.

The Resilient Modulus Test values of the O.M.C. plus 2 percent water content samples were approximately 80% to 200% less in value than the O.M.C. samples.

Slightly higher strength were shown in all of the samples due to variation of the confinement pressures. This difference was small, as compared to the increase of water content effect on Resilient Modulus.

Poisson ratio values were determined on some of the samples. The Poisson values were somewhat scattered and unpredictable on the 1 p.s.i. loadings, but after heavier loadings, such as the 3 or 4 p.s.i. loadings, they fit a fairly tight pattern on both the O.M.C. and O.M.C. + 2% samples. The Poissons varied in most cases not much more than .05 Poisson value, and many of the loadings much less. Values of .3 to .4 Poisson was shown on the heavier loadings of the O.M.C. samples, whereas near a .5 was shown on the O.M.C. + 2% of the Sample No. 564.

ALASKA DOT/PF APPLICATIONS OF RESILIENT MODULUS TEST RESULTS FROM H&V EQUIPMENT

by

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ALASKA DOT/PF APPLICATIONS OF RESILIENT MODULUS TEST RESULTS FROM H&V EQUIPMENT

Thomas L. Moses, Jr., P.E.*

Introduction

The Highway Research Section of the Alaska Department of Transportation and Public Facilities purchased a repeated load test device in 1981. This equipment has been used in a research efforts to provide a general characterization of the resilient modulus values of asphalt pavement, crushed aggregate base, and subbase samples throughout Alaska. This equipment has also been used on numerous design projects to determine the resilient modulus values of the asphalt pavement, base course, subbase, embankment, and crushed pavement. In addition, indirect tensile tests have been run on pavement that did not meet specification to assess the long term performance.

Equipment

The H&V repeated load pneumatic test system is capable of measuring indirect tensile resilient modulus of bituminous mixtures as described in ASTM Test Method D-4123 and resilient modulus of unbound materials under triaxial loading as described in AASHTO Test Method T-274.

The testing frame is bench mounted and has a maximum capacity of 2,500 lbs. The load and control system can apply pulsed dynamic loads to 1,000 lbs. and operates on 15 cfm at 120 psi air supply. The frequency control ranges from 10 to 60 cycles per minute and the duration of applied load is adjustable within the range of .05 to 1 second in increments of .05 seconds. The seating loads, dynamic loads and confining pressures are manually adjusted. The equipment is furnished with a fatigue shutoff switch with visual readout of number of cycles to failure when the machine is used in the fatigue testing mode.

The triaxial cell can accept loads up to 1,000 lbs. and samples up to 4-inch in diameter and 8-inch in length. The triaxial cell uses linear variable differential transformers with specimen clamps for 4-inch diameter samples. The diametral yoke has displacement transducers for 4-inch diameter by 2.5-inch thick asphalt concrete specimens.

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The original recording device was a two channel Hewlett-Packard Model 7402 A Oscillographic recorder with two HP 17403A A/C carrier preamplifiers. This recorder has since been replaced with an IBM PC compatible computer.

Test Procedure

In general the dynamic diametral tests are run in accordance with ASTM Test Method D-4123. Due to the soft grades of asphalt (AC 1.75 & AC 2.5) used in Alaska, the 4-inch diameter by 2-inch thick asphalt cores are tested in an environmental chamber at 50° F. Prior to testing the cores are precooled for 12 to 18 hours before testing at 50° F. For samples of AC-5 or AC-10 the samples are run at 50° F. and 72° F. The sample is pulsed for .1 second at a load to obtain an initial strain at the start of the test of 200 M-strains. The frequency of the pulse is 1 second. In many cases the load has to be reduced to obtain 100 M-strain due to deformation of the sample.

In general the resilient modulus of unbound materials is tested in accordance with AASHTO Test Method T-274. The samples are placed in the membrane in 3 equal lifts. Each lift is compacted with a vibratory hammer for 1 minute. A confinement pressure of 4 pounds is applied to the chamber to contain the sample. The sample is conditioned by applying a deviator stress of 15 psi at a confining pressure of 7.5 psi for 200 repetitions. The sample is pulsed at 0.1 second with a frequency of one second. Two hundred repetitions of each of the following deviator stress and confining pressure conditions are applied to the sample:

Deviator Stress (psi)	Confining Pressure (psi)
3	3
5	5
7.5	7.5
10	10
20	20
6	3
10	5
15	7.5
20	10
40	20
9	3
15	5
22.5	7.5
30	10
60	20

Research

The first indirect tensile testing of bituminous materials was part of the Alaska DOT/PF research project "Use of Layered Theory in the Design and Evaluation of Pavement Systems". The primary purpose of this project was to evaluate the CHEV5L, CHEV5L with iteration, BISAR, ELYMSM5 and PSAD2A computer programs for use in mechanistic pavement designs. As part of that project indirect tensile tests were run on several asphalt concrete cores in the Fairbanks area. Cores were taken from both old (15-20 yr.) and new (less than 2 yr.) pavements were tested. These samples were tested at varying temperatures with the following results:

TABLE 1

Resilient Modulus Properties of Alaskan Asphalt Concrete

	Resilient Modulus (PSI)	
Temperature (° F.)	Minimum (New)	Maximum (Aged)
70	36,000	410,000
60	110,000	620,000
50	220,000	920,000
40	400,000	1,300,000
30	610,000	1,700,000
20	980,000	2,200,000
70*	74,100	82,900
70**	58,100	98,600

* recycled mix with AC 2.5 as additive (laboratory sample)
** recycled mix with AC 1.75 as additive (laboratory sample)

General Properties:
Asphalt Content = 5% - 6%
Air Voids = 2% - 4%
Pavement Age = 2 - 10 years

A second research project was the comparison of resilient modulus values of asphalt cores taken in test sections of Shaw Creek project on the Alaska Highway. Indirect tensile tests were run on asphalt cores of AC 2.5, AC 1.75 with chemcrete, and AC 1.75. Ten samples were tested in each group. The resilient modulus values of the AC 2.5 varied between 175 ksi to 403 ksi with an average of 385 ksi. The resilient modulus values of the AC 1.75 ranged from 46 ksi to

255 ksi with an average of 164 ksi. The samples of AC 1.75 with chemcrete had resilient modulus values that ranged from 82 ksi to 306 ksi with an average of 252 ksi. These samples were tested at 50° F.

Design

Indirect tensile tests of bituminous pavement and resilient modulus tests of unbound materials have been done on several design projects. Laboratory tested resilient modulus values have been useful in comparing various pavement design alternatives and evaluating the properties of unusual types of materials such as crushed pavement and cold recycled asphalt.

The following three projects are examples where mechanistic pavement designs based on laboratory tested resilient moduli were instrumental in evaluating various pavement design alternatives.

Seward Highway - Girdwood to Ingram Creek - The Seward Highway project from Girdwood to Ingram Creek was reconstructed and paved in 1986. The pavement had developed severe fatigue cracking.

Several pavement design alternatives were considered including hot recycling, cold recycling, and crushing the existing pavement prior to overlaying. The mechanistic design for each alternative was based on laboratory tested moduli for HAP (422ksi), hot recycled asphalt (453ksi), cold recycled asphalt (200ksi), and crushed pavement (63ksi). The base course and embankment moduli were based on back calculating falling weight deflectometer data.

The most economical design was to crush the existing pavement and overlay with 3-inch of HAP.

Sterling Highway MP 117-157 - The Sterling Highway project from MP 117 to MP 157 was originally constructed and paved in the mid 1960's. The road between MP 133 to MP 157 has developed extensive fatigue cracking.

Reconstruction is scheduled for 1989. To eliminate reflective cracking the pavement between MP 133 and MP 157 will be rotomilled and overlaid with 2.5-inch HAP. The section between MP 117 and MP 133 will be overlaid with 2.5-inch of HAP.

A mechanistic pavement design was done for both sections. Design moduli for each section was based on back calculations of falling weight deflection data. These values were checked with laboratory resilient modulus tests.

Cold Bay Airport - The main runway at Cold Bay was reconstructed in 1988 by removing the existing 4-inch pavement and placing 6-inches of hot recycled asphalt pavement. This 10,443-foot long runway was originally constructed during World War II and overlaid in the mid-1970's. Fatigue cracking developed in the wheel paths requiring reconstruction.

The two pavement designs considered were crushing the existing 4-inch pavement and overlay with 2-inch of HAP and removing the existing pavement and placing 6-inch of hot recycled asphalt pavement in the 100-foot wide keel strip.

The resilient moduli for the HAP and the recycled asphalt pavement were based on laboratory resilient modulus tests. The resilient modulus values for the base course, crushed pavement, and embankment were based on previous resilient modulus testing done on similar samples.

Construction

Indirect tensile tests of asphalt cores have been conducted for two projects to assess the future performance of new pavement.

On the Parks Highway MP 71-104 project, check Marshall Stability tests of the asphalt pavements did not meet the minimum 1500 lbs stability requirement. Three pavement cores were tested from this project and from an adjacent project (Parks Highway MP 35-42) that used the same grade of asphalt (AC-2.5) and had passing check Marshall Stability tests. The samples were tested at 50° F. and 72° F.

The tested resilient modulus values of the asphalt pavements did not compare favorably. However, a mechanistic design using the tested resilient modulus values indicated that the pavement would have a sufficient performance life and the pavement was accepted.

Project	50° F.	72° F.
Parks MP 71-104	233 ksi	40 ksi
Parks MP 35-42	360 ksi	69 ksi

One section of the asphalt pavement on the Boniface Parkway Project had clay balls. Indirect tensile tests were run on four asphalt cores taken from the section with clay balls and without clay balls. These samples were tested at 50° F. and 72° F. There was a significant difference in resilient modulus values and the pavement in the section was overlaid.

	50° F.	72° F.
Clay Balls	407 ksi	49 ksi
w/o Clay Balls	559 ksi	77 ksi

Future Uses

The Alaska Department of transportation and Public facilities is in the process of purchasing a second repeated load test device for the Central Materials Laboratory in Anchorage. This equipment will be used primarily for design and construction projects.

The primary benifit will be to measure the resilient moduli of existing pavements for overlay pavement design. In addition, laboratory tested resilient moduli for crushed asphalt, hot recycled asphalt, and cold recycled asphalt will be used for the mechanistic pavement design.

This equipment will also be used for comparing the resilient moduli of asphalt mixes using the current Alaska DOT Marshall mix design method and the mix design method recommended by teh FHWA Technical Advisory TA 5040.27. The two major differences between the two mix design methods are the inclusion ov a VMA requirement and limiting the amount of natural fines.

TWO AXIS COMPUTERIZED DYNAMIC TESTING SYSTEM

by

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March 1989

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Mr. James Thissen and Mr. Robert Heinen, both Engineering Technicians, were helpful in setting up the equipment and implementing the testing procedures.

NOTICE:

The authors and the State of Kansas do not endorse products or manufacturers. Trade or manufacturers names appear herein solely because they are considered essential to the object of this report.

INTRODUCTION

Over the past few years, several highway and research agencies have developed laboratory methods whereby pavement materials could be dynamically tested. Dynamic testing could simulate in the laboratory the stress conditions found in the field. Traditionally the static method of testing was used in characterizing highway material properties. Dynamic testing could provide a greater insight as to how highway pavements react to stresses under different conditions. These stresses could be either axial or radial in nature. They could be changed for a period of time such that each pulse could simulate separate wheel pass. The pulses could then vary in magnitude, duration, and shape as compared to the variations in wheel loads in magnitude, speed, and pavement depths.

In 1984, the Research Unit for the Kansas Department of Transportation (KDOT) acquired a dynamic testing machine (or simply referred to as the press). The purpose was to produce different stresses under various loading conditions whereby new pavement mixes (with and without additives) could be analyzed. Old mixes could also be tested as a basis of material reference to the new mixes. The objective of KDOT's Research Unit, is to develop better, longer lasting, and hence more economical pavements.

In addition to the research efforts, the new Pavement Management System (PMS) required dynamic stress and strain data to develop stress prediction models. The equipment can even be used as a design tool in conjunction with the field dynaflect equipment now on hand in the PMS system.

The system was purchased and put together as a component system. Competitive bids were initially issued and received in September 1981, for the main closed-loop servo-hydraulic testing system as well as the strip chart recorder and dual channel oscilloscope. Due to a request to modify the bids after the closing date and the availability of an equivalent digital (computer controlled) system at a comparable price, new bids were solicited with consideration given to an equivalent digital system. The final bid selection for the closed-loop servo-hydraulic testing system was made in November 1981.

The specifications under which the final bids were issued, and how the hydraulic testing system met those specifications is presented in Table 1.

AMI Consultants in Sparks, Nevada was finally awarded the contract, who in turn sub-contracted to the actual manufacturer, Cox and Sons, Inc., Colfax, California. The oscilloscope was not required for the newer computer system but other components were

TABLE 1. Specification for Dynamic Testing Machine.

SPECIFICATION FOR DYNAMIC TESTING MACHINE		ACTUAL
1. DYNAMIC MATERIAL TESTING LOAD FRAME (1)		
Static Capacity, lbs.	50,000 min.	50,000
Dynamic Capacity, lbs.	25,000 min.	30,000
Spring Rate, lbs/in.	2.6×10^6 min.	3.5×10^6
Max. Frame Deflection @ Rated Capac. in.	0.02 max.	0.014
Horizontal clearance, in.	25.0	30 (25 usable)
Vertical Clearance, in.	50.0	45
Hydraulic Lifts and Locks	Required	Yes
2. SERVO CONTROLLED HYDRAULIC ACTUATOR(1)		
Mounting	Crosshead	Yes
Dynamic Rating, lbs.	11,000 min.	20,000
Stroke, in.	4.0 min.	10.0
An LVDT (linear to 0.5% of total travel) should be mounted on the hollow core of the piston rod. It must be positioned from the grip mounting end to provide for convenient adjustment of the null position. The actuator piston should have seal supports to prevent twist or roll of the seal during high frequency cyclical operation. The actuator piston must have an internal thread for mounting specimen grips and plattens.		Yes Meets Specifications
3. PERFORMANCE PACKAGE - HYDRAULIC POWER SUPPLY(1) AND SERVO VALVE(S)		
Low level switch	Required	Yes
Filter-micron	3	10
Noise rating, dba @ 3 ft.	80 max.	
Reservoir Temperature, °F	125 max.	
Temperature and Pressure Indicators		
Service Manifold as required		
The hydraulic power supply and servo valve(s) should be matched, so the static capacity of the system is at least 11,000 lbs. Also, the system must be able to maintain a loading of 3,000 lbs. at 20 Hz, ± 0.1", without affecting the static characteristics of the system. The hydraulic power supply should be rated at 3,000 psi. and should have a rating of at least 5 gpm. The system must provide for an interlocked bumpless transfer system to change control modes of the actuator, while hydraulic pressure is applied, without manipulation of supression or offset controls.		Yes Meets Specifications

4. FATIGUE RESISTANT LOAD CELL(1)

Rating, lbs. (same as static rating of system)	11,000 min.	2,500 & 10,000 lb.
Compensated Temperature Range, °F	30 to 150	150°F. (Max.)
Usable Temperature Range, °F	-50 to 200	
Cyclic Life, rating from full rated tension to full rated compression, cycles	10^8	10^8
Mounting	Table	Actuator
Thread	Internal	Internal

5. COMPRESSION PLATENS(2)

Rating, lbs.	25,000	
Diameter, in.	4.0	2.0 & 4.0
Thread - enternal, to match internal threads on load cell and actuator		Ball bearing

6. ELECTRONIC CONTROL SYSTEM

(Note: The following specifications were used in the qualification of the final contract recipient).

Control of the system will be achieved through a central processor unit, (CPU) with 16 bits of processing power and minicomputer instruction set. This instruction set should greatly simplify and speed programming effort resulting in substantial savings in programming time and memory utilization.

The digital controller must include conditioning equipment for load, strain and stroke. It should manage all the operational aspects of the testing system including the servohydraulic system and the closed loop operation under load, and/or stroke, and/or strain control. The system must also have capabilities for data acquisition, data analysis, and report generation. Output will be directed to the CRT and/or an optional printer-plotter. The interface for the printer-plotter must be supplied with the system even if the printer-plotter option is not taken.

The system should have enough memory space so user programs can be stored, as well as data acquired during testing. Additionally, the system should have a bulk data storage medium (disk, cassette or paper tape).

7. OTHER

7a. Consoles

Enough console space must be provided to house the electronic control system plus one strip chart recorder and one oscilloscope which will be supplied by the KsDOT. Back mounting space will be approximately 15 inches (vertical) for the recorder and 6 inches (vertical) for the oscilloscope. Controls should be located in a manner that will provide easy access by the operator. The console(s) must contain a bin with a DC power supply large enough to drive all the modules and interconnections required for the system. All the connections should be made using the back panels. — Meets Specification

7b. Keylock Switch(1)

A keylock switch to prevent unauthorized use of the system.	Meets Specifications

7c. Interconnecting Electrical Cables(1 lot)

Include all necessary interconnecting cables.	Meets Specifications

7d. Hydraulic Hoses(1 lot)

Include all necessary hydraulic hoses.	Meets Specifications

7e. Operation and Maintenance Manuals.		2 Sets Supplied

7f. System Service.

Includes installation assistance and interconnection of all the supplied items, and on-site training of KsDOT personnel in the operation and maintenance of the complete system.

7g. Maintenance.

Includes maintenance during the first year (under warranty). A proposal for maintenance contract should be submitted for consideration by the KsDOT.

7h. Warranty.	1 Year	1 Year
7i. Delivery.	16 wks or less	Due to modifications the 16 week period was exceeded

needed and selected on a competitive bid basis. The cost of the close-looped hydraulic testing system and the various components are listed in Table 2. The manufacturers are also listed for most of the equipment.

EQUIPMENT

The equipment consists of a hydraulic pump or power pack, load frame, computer control package, and a cabinet for the accessories. The actual and final layout is shown in Figure 1. The power package supplies hydraulic pressure from a 208 volt, three phase electric motor operated pump. A heat exchanger uses water to cool the hydraulic fluid but a fail safe temperature switch is also supplied. A low oil level switch is supplied to shutdown the system if the hydraulic oil level in the reservoir drops too low. The capacity of the reservoir is approximately 80 gallons.

The load frame contains the hydraulic actuator, servo valves, filters, accumulators, and necessary column clamps needed to raise and lower the column. The hydraulics makes a complete loop from the previously mentioned power package through the load frame. Basically the hydraulic pump drives the actuator through a servo valve, which in turn is electrically controlled from an analog signal generated from the control system. The load frame is the location at which the samples are actually tested.

The next location is a cabinet that houses some of the accessories such as the strip chart recorder and LVDT conditioning equipment. There was extra space allowed for future expansion or mounting of additional equipment.

The last cabinet contains the electronic systems that operates and controls the machine.

1. The central processor unit (CPU) is a Texas Instrument module (TM 9901). It contains the microcomputer, memory, parallel digital input and output (I/O) and two serial data ports (RS 232). The actual computer language used is a version of Engineering Logic FORTH (E. L. FORTH) called ELF. ELF is a simple but powerful language for machine control and data acquisition. It differs from conventional language by using Hewlett Packard calculator type notation. Values are entered into the machine before the machine is instructed to use them.

   ELF is also a stack oriented language just as some calculators are stack oriented. When numbers are typed into the system, they go onto the stack. Different operations use the data off of the stack and sometimes put the results back on it.

Table 2. Summary of Equipment Manufacturers and Costs.

DESCRIPTION	MANUFACTURER	COST
Closed Loop Hydraulic Testing System	Cox & Sons	57,320.00
Environmental Cabinet	B-M-A Inc.	4,997.00
Triaxial Cell & Panel	Research Engineering	12,666.00
Split Mold	Research Engineering	900.00
Strip Chart Recorder	MFE Corporation	3,645.00
Printer	C. Itoh & Co., LTD	559.00
LVDT Conditioning Eq.	Daytonic	2,615.65
LVDT's	Schaevitz	1,904.40
LVDT Calibrator		198.00
Load Cells	Strainset	968.50
		85,773.55

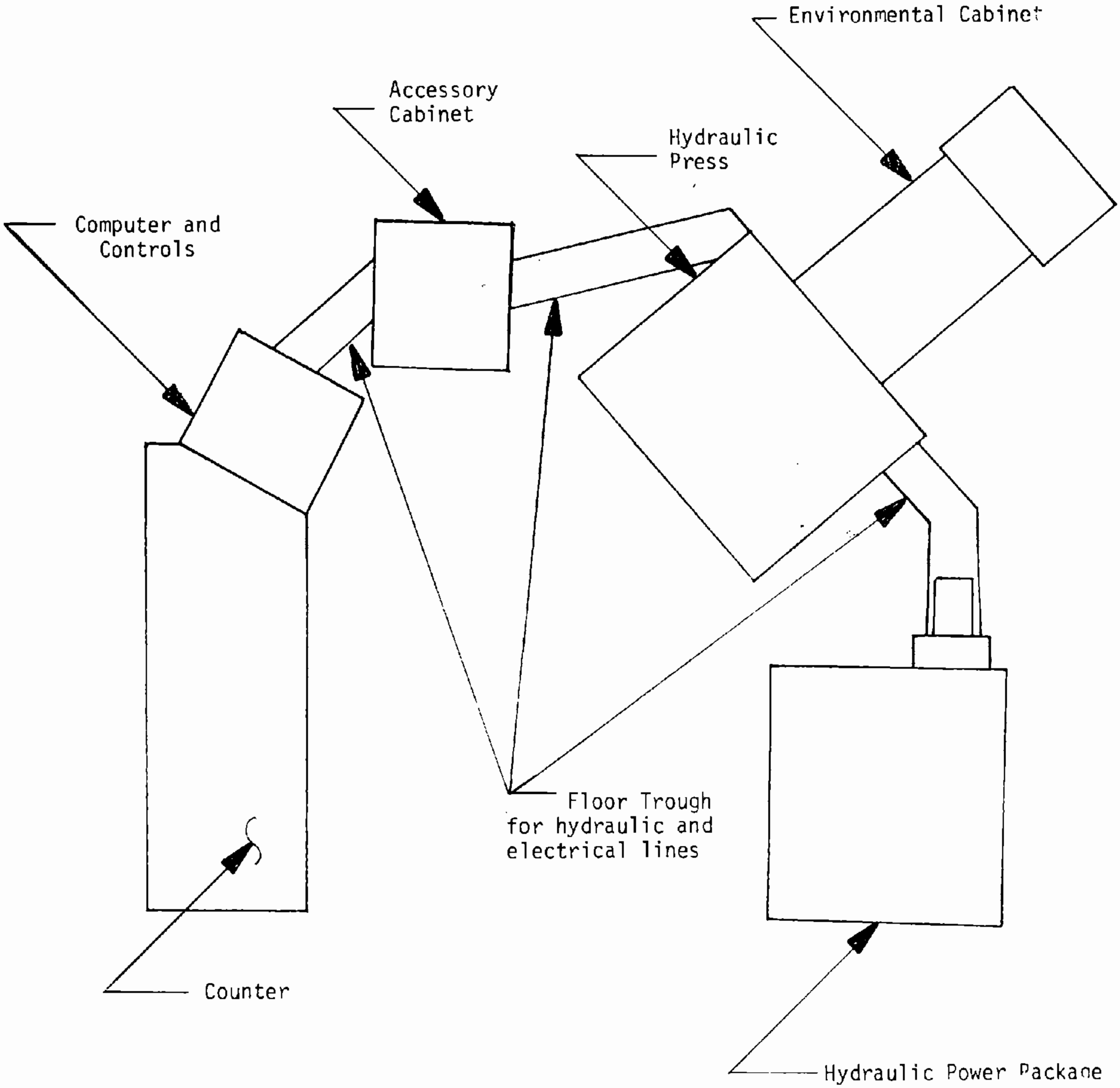

Figure 1. Dynamic Testing Machine Layout.

2. The Memory Expansion Module contains additional random access memory (RAM) and programmable read only memory (PROM). PROM contains the system software and the machine control software. Additional test control, data reduction and report writing routines can be loaded from the floppy disk.

3. The analog interface is a 16 channel, fully differential, analog to digital converter board. All inputs are read at least 25 times/sec. Ten additional channels are used for scanning maximum and minimum values at a 500 Hz rate. This data is then available to the operator through a variety of paths. The operator may choose to write his own routines to access and act upon the analog data. A listing of the analog channels are presented in Table 3.

4. A dual 8" floppy disk recorder may be used to store programs for machine control, data reduction, or data analysis. It may also be used as a data acquisition storage device.

5. The video display terminal consists of a typewriter type keyboard and a video display monitor. The monitor has a 12" screen with green phosphorus. The top seven lines of the screen are used for realtime data display (Analog channels, major axes display, parameters display, or peak detector display). The middle 17 lines (80 characters) may function as a scrolled terminal for system control, an editor for developing disk resident programs, a diagnostic tool for system checkout, an analog scaling value monitor, or a computer program editor. The bottom line on the video monitor allows control programs to efficiently output information to the operator.

6. A printer located beside the control cabinet, is used as an interface to the system. It may be used to print data acquisition reports, to print data program listings, or to print operator defined outputs.

7. The front panel is used to select the operation mode of the system and for manual positioning. It contains all of the controls used during actual testing. This includes four rotary switches, a series of 24 lighted control buttons, an ON/OFF button, and an EMERGENCY reset button.

8. Signal conditioning is supplied for the transducers used in machine control. Additional signal conditioning modules were made available for LVDT's and pressure transducers. These modules scale the analog inputs to the standard ± 10 volt range used as inputs to the analog interface.

Table 3. Analog Data Channels.

ANALOG DATA CHANNELS

Channel	Function
1	Z Axis Stroke error signal
2	Z Axis Load error signal
3	Y Axis error signal
4	Z Axis Load feedback - filtered
5	Z Axis Storke feedback
6	Z Axis Load - not filtered
7	LVDT Set #1
8	LVDT Set #2
9	Y-Axis feedback (temperature)
10	Load Cell (Triaxial)
11	Triaxial Pressure Transducer #1
12	Triaxial Pressure Transducer #2
13	Triaxial Pressure Transducer #3
14	(Not used)
15	(Not used)
16	Limit module analog input
17	Max Peak - Z Axis Load feedback - filtered
18	Min Peak - Z Axis Load feedback - filtered
19	Max Peak - Z Axis Stroke feedback
20	Min Peak - Z Axis Stroke feedback
21	Max Peak - Z Axis Load feedback
22	Min Peak - Z Axis Load feedback
23	Max Peak - Peak scanned/LVDT Set #1
24	Min Peak - Peak scanned/LVDT Set #1
25	Max Peak - Peak scanned/LVDT Set #2
26	Min Peak - Peak scanned/LVDT Set #2

9. Data may be acquired by any of four methods. First, it may be stored on the disk during the test. Disk storage is useful for high speed data acquisition and to collect data in machine readable form for later data reduction and analysis. Second, it may be printed out on the system printer. The printer is useful for collecting data at any rate compatible with its speed. It gives immediate output as the test progresses, and can be used in conjunction with the floppy disk if both immediate output and machine readability are required. The third method is from the strip chart recorder tracings. And the fourth and last method, the min/max channels can be manually copied from the video monitor.

10. Electrically Erasable Programmable Read Only Memory (EEPROM) is used as modifiable storage for the scaling valves and engineering units of the analog inputs. This allows calibration of certain analog signals as well as storage of the correct units.

11. The system also provides positive fail-safe limits for both an overstroke or an overload condition. A separate analog controller works in parallel with the computer controlled system. It has a separate power supply and is isolated from the computer controlled system to the maximum degree practical. The separate controller monitors the feedback signals for pulses or voltages beyond the operator selected limits. If a feedback signal gets outside the accepted range, the analog controller takes control and places the machine in another control mode that is pre-selected by the operator. Once the analog controller takes control the actuator moves to either a pre-selected position or to a pre-selected load. The hardware limits may be set on both sides of the feedback signals.

12. The two axis analog controller controls the main actuator (z-axis) in either <u>load</u> or the <u>stroke</u> mode. In the load mode of operation the actuator will position itself so that a pre-selected load is applied to a specimen through a load cell. The load cell can register loads in compression or tension. In the stroke position, the actuator is directed to a pre-selected position. This position is determined by a sonic transducer. A "bumpless" transfer between load or stroke may be executed under either manual or programmed control. The z-axis may be directed in either load or stroke mode to hold, ramp, perform cyclic functions, run gated functions, or to run from a computer program. The cyclic and gated functions are sine, haversine, triangle, and rectangle. The analog controller also controls the secondary axis (Y-axis) in the temperature mode. However, the control capabilities are limited to a few basic functions.

APPLICATIONS

At the present time, there are five tests that this equipment can accomplish. Most of these tests are computerized and fully automatic. Data acquisition from these particular tests is always printed by the printer even though the video screen, strip chart recorder, and possibly the floppy disk can be used as a backup.

Several types of test can be run on bituminous samples. First, the samples must either be manufactured in the laboratory (kneading compactor, etc.), or obtain from the field as asphalt concrete cores. After obtaining the samples, aluminum clamps are glued around each core with super-glue. The normal core size is four inches in diameter and approximately eight inches in length. Gage lengths have varied down to one or two inches, but the accuracy of the test will be sacrificed. LVDT's are placed on each side of the sample. One half of the total deflection is recorded on each LVDT and the sum of both LVDT's is added at the signal conditioner. The voltage signal from the LVDT conditioner is then fed into one of two analog inputs for data actuation as previously described. Dynamic Tests on these samples can be accomplished at various temperatures. To achieve the desired temperatures, the bituminous samples are placed in an environmental cabinet where the whole sample can be brought to a temperature between $0^{O}F$ to $100^{O}F$. The temperature is controlled by either the Y-axis computer controller or manually set.

Once the clamps were glued on the sample, LVDT's mounted, and proper temperature achieved, the dynamic modulus can be determined. The procedure will not be explained but can be obtained by referring to ASTM D-4123.

Dynamic Modulus testing can also be accomplished by the indirect method. This procedure will be the same as that found in ASTM D-4123.

Bituminous plugs (Marshall size) can also be tested by the split tension method. Tensile strength can then be calculated. The test can be accomplished in the environmental cabinet at most temperatures. Procedures are also outlined in ASTM D-4123. Possibly, the only exception is the two inches/minute loading rate of the hydraulic ram (z-axis).

Creep tests are also accomplished on these asphalt mixtures. Clamp and LVDT arrangements are the same as previously described but the test method is different. A compressive load is applied to a core or laboratory sample in about 1 or 2 seconds. LVDT deflection readings are taken as long as the load remains on the sample. Computer programs are set up so that the appropriate data channels are activated, and the compressive loads automatically applied at specific lapse time intervals.

Other tests accomplished by this equipment for the Kansas

DOT are resilient modulus determinations of soil and granular samples. AASHTO T-274 describes the procedures. The triaxial cell is used and chamber pressures up to 100 psi can be achieved.

The chamber is manufactured of a thick transparent material and can withstand pressures up to 300 psi. Air is used as a pressure medium. Deflection measurements are again recorded through LVDT's mounted in clamps that in turn are friction mounted on the outside of the specimen membrane. Loading schedule is a haversine type loading accomplished in 0.1 second with a 0.9 second rest period. All data channels and controls are computer controlled through programs that are stored on disk floppies.

SUMMARY

The dynamic testing system as described has been in operation over four years. The equipment has been extensively used.

It is estimated that the equipment averages approximately 10 man-hours for a 40 hour week. As of this date, downtime has been relatively low with only one major repair. All other repairs have been relatively minor and should be considered routine maintenance. The system support from Cox and Sons, Inc. has been excellent. Parts are still available and interested individuals may contact Glenn Fager or the Manufacturer for further information.

MODULUS TESTING BY COLORADO DOT

by

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Paper prepared for the

Workshop on Resilient Modulus Testing
Oregon State University
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Modulus Testing by Colorado DOH

Dick Hines*

Purchase

Colorado DOH (CDOH) purchased a resilient modulus testing device in October 1974 for $5,935. It was designed by Roger Schmidt and manufactured by Retsina Co. The device was obtained to examine the possibility of using modulus to determine strength coefficients of asphalt mixes and to compare modulus results to other mix properties.

Operation

The device applies a load across the vertical diameter of a 4" diameter, 2.5" thick specimen of asphaltic concrete. A solenoid valve allows pressurized air from a surge tank to enter a bellofram resulting in the load. The resulting deformation is measured by two transducers mounted in a yoke suspended across the horizontal diameter of the specimen.

Controls for load application and deflection readout are contained in a control box. Load is set by adjusting the pressure in the surge tank using a pressure regulator. The surge tank pressure is monitored by a pressure transducer in the control box. Deflection can by determined either 0.05 or 0.1 seconds after the load is applied.

Calibration

The transducers are calibrated using a differential translator. A differential translator looks like a micrometer and works in a similar manner. The differential translator moves the transducer tip a known amount and the deflection is read on the meter on the control box. Gain is adjusted until the meter readout agrees with the differential translator.

Load is calibrated to applied air pressure using a proving ring. A graph of deflection versus load for the proving ring was supplied with the device. Also included is a dead weight applicator so that the graph can be checked using a known dead load.

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Lottman Research

From 1975 to 1980 the device was used in a multi-agency research project headed by Dr. Robert Lottman of the University of Idaho. This research was the field verification phase of the development of a test system to predict moisture susceptibility of asphalt mixes.

The test system currently used by CDOH begins with the compaction of two replicate specimens of the asphalt mix. One specimen is conditioned by vacuum saturation, freezing for 16 hours, and soaking in a 140^0 F water bath for 24 hours. The conditioned specimen is then transferred to a 77^0 F water bath for 2 hours in preparation for testing. The dry specimen is stored in air at 77^0 F until tested. The specimens are tested by loading to failure across a vertical diameter (split tensile test). The ratio of the tensile strengths of the conditioned specimen to the dry is determined.

For the research project, resilient modulus ratio was determined in addition to the tensile strength ratio. Also, test temperatures were 73^0 F and 55^0 F. The test system over predicted moisture damage for the five year study period. However it was felt that the test system would be an accurate predictor for longer term damage. The tensile strength was chosen over modulus as the standard test because it was the more accurate predictor of moisture damage. A testing temperature of 77^0 F was found to be more convenient than those used in the study.

Current Use

Resilient modulus is currently used as a specification for one grading of asphalt mix. For this grading Hveem stability did not yield consistent results. Since the device was purchased resilient modulus has been determined for all asphalt mixes and has been considered in design. In a few months CDOH plans to use resilient modulus to determine asphalt mix strength coefficients.

Problems

The device has had few mechanical problems in 14 years of continuous operation. The transducer advance screws were replaced because of wear. The off-on switch was broken by mishandling and was replaced.

Past and Future

The Retsina device has been dependable and provided consistent results. It is simple to operate and fast. CDOH will soon be using resilient modulus for strength coefficient determination and later creep may be used to determine rut resistance.

SESSION III - RESILIENT MODULUS TESTING

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AN OVERVIEW
OF
RESILIENT MODULUS TEST SYSTEMS

by

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AN OVERVIEW OF RESILIENT MODULUS TEST SYSTEMS

Andrew M. Brickman*

1.0 INTRODUCTION

Since development of the 1986 AASHTO *Guides for Design of Pavement Structures*, the apparatus for testing diametral and triaxial resilient modulus has emerged from the university research lab to the production lab arena of road building agencies and private consulting firms.

This transformation has imposed demands on the equipment designer to not only simplify and streamline the apparatus, but to also:

- increase reliability, accuracy, and efficiency of equipment
- increase user convenience while minimizing chance of error in set-up, calculations, and reporting data
- implement PC or microprocessor-based controllers to achieve the above goals

This paper attempts to define the current criteria and design constraints of a repeated load system. Specifically, the paper treats:

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- Loading systems; air versus oil power, open and closed loop control.
- Instrumentation; transducers and signal conditioning.
- Data acquisition and control; structure of signal capturing and load control.
- Future developments; what to expect in system improvements and changes.

It is expected the reader will gain a clearer understanding of the various test system options available to user agencies.

2.0 LOAD APPLICATION

Both AASHTO T-274 and ASTM D-4123 prescribe a load waveform which is either a sinusoid or a pulse, with frequency of 0.33, 0.5, or 1 Hz and duration of 0.1 to 0.4 sec. Load magnitudes can be as small as 10 lbs. for soft soils in the triaxial test to over 2,000 lbs. for stiff bound materials in the diametral test. Equipment manufacturers have gone exclusively to fluid power to apply repeated loads in both triaxial and diametral testing. Mechanical testers employing cams, levers, gear or screw drive, while suitable for static or slow displacement testing, prove too cumbersome for repeated load, especially in a load-controlled mode. Electromagnetic drive systems, while well suited for metal fatigue testing in resonant-drive machines at frequencies much higher than 10 Hz are not suitable for resilient modulus testing. The high currents needed to produce repeated loads at 1 Hz and below create a noisy environment to nearby electronic instrumentation.

Fluid power options open to the designer include:

- fluid medium: air or hydraulic oil
- control mode: open or closed loop

A brief discussion of advantages and disadvantages of each follows.

2.1 Fluid Medium

2.1.1 Compressed air is a logical choice as a source of load power. It is non-messy, non-toxic, and readily available in most labs. As an additional advantage, the noisy compressor can be placed remote from the test lab. At normal line pressures, however, (100-125 psi) upper load limits are 1,000 - 2,000 lb for reasonably sized actuators. Also, since air is highly compressible, considerable energy is expended to cycle high loads continuously (e.g., as in an hours-long fatigue test). Compressibility also places a limit on the quickness of load application (rise time to attain full load).

2.1.2 Hydraulic Fluid (oil) really shines as a fluid power performer. For example, at working pressures of 2,500 or 3,000 psi, it can be very quick in load rise time, and a 10-kip, 2-in. stroke actuator can be smaller than two Marshall specimens.

First cost of hydraulic systems is high, though, and these systems are more complex than pneumatic systems. Additionally, the pump unit usually resides closed to the test apparatus, and may require external cooling means as well as noise reducing cabinetry. Oil leakage also is all too often a problem.

2.2 Control Mode

2.2.1 Open Loop Loading Systems respond to a command input without regard to current output status of load or displacement of the actuator. A good example is a constant rate triaxial load frame (older style without servo-motor). The command input is a setting to a constant speed; once started, the platen moves until shut off. No self adjusting takes place to maintain speed (Figure 1).

Repeated load modulus systems of the open loop variety use a source of constant pressure to derive their load pulses. Typically, the actuator cylinder is toggled by a valve between a high pressure source and a low pressure source to gain the desired train of load pulses. The chief advantages are simplicity, reliability, and low cost. The valves used are

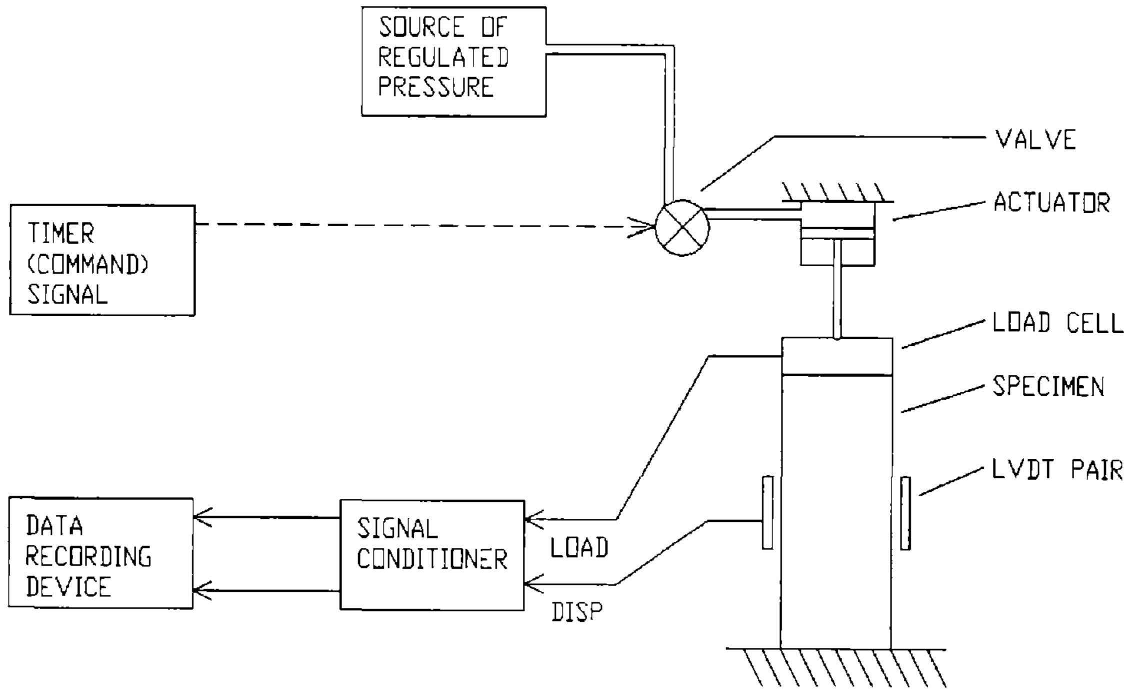


Figure 1 - Open Loop Load Control of Test System

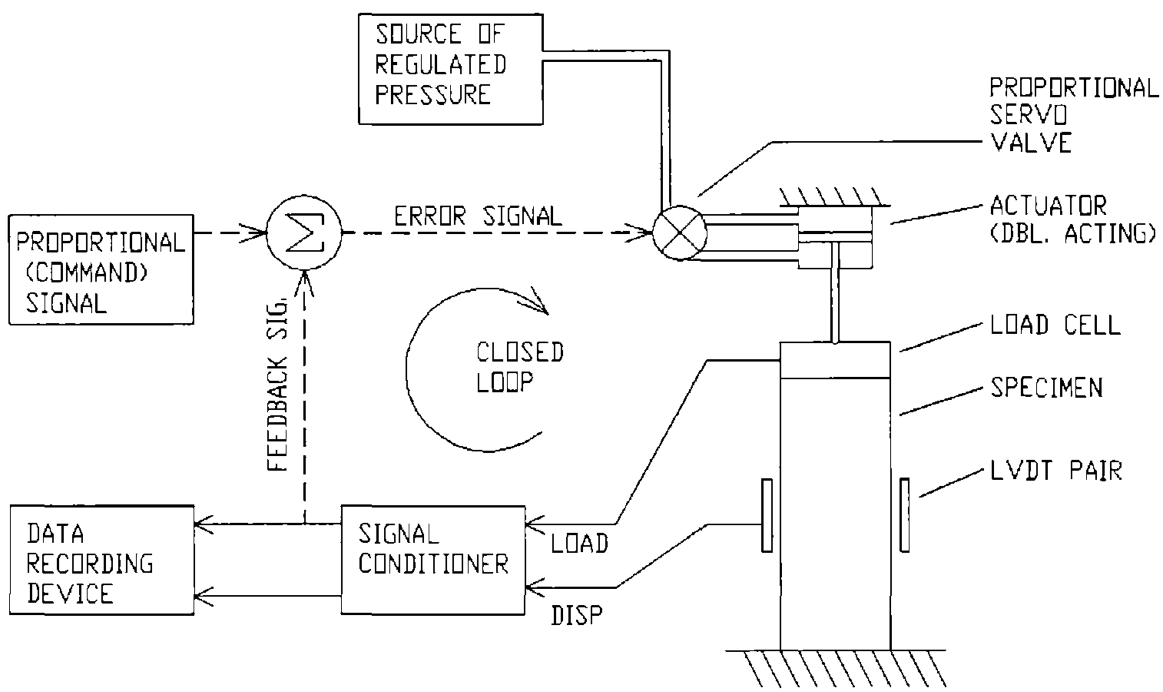


Figure 2 - Closed Loop Load Control of Test System

rugged on/off devices which are easy to service or replace, and the actuator can be single acting (unidirectional). Pressure regulators with output gauges can supply the high and low pressure; the gauges give the operator a rough idea of applied loads.

2.2.2 Closed Loop Loading Systems employ a sensor at the actuator output that can monitor the desired variable, either load or displacement. That signal which reports the current output status is called the feedback signal. It is compared to another signal, input command, at a summing point. The difference between the input command and output status is the error and is used to drive the actuator control valve to rapidly minimize error (Figure 2).

The chief advantage of closed loop control is its ability to follow command signal input changes, within the speed and amplitude capabilities of the actuator. Indeed, a large industry has evolved in the field of structural response testing, both destructive and nondestructive, based on the capabilities of closed loop controlled actuators to simulate field phenomena. However, closed loop systems are inherently more complex and expensive than open loop systems. The actuator must be double-ended (bi-directional) and the fluid must be ported by a double acting servo valve. This is a proportional, electrically driven metering valve manufactured to fine tolerances. A servo amp drives the servo valve; dynamic response of the complete system with feedback must be optimized or "tuned" for the materials and load frame used. Performance of an improperly adjusted system can range from sluggish to wildly unstable.

3.0 SYSTEM INSTRUMENTATION

Resilient modulus testing of diametral and triaxial specimens requires load and displacement measurement by electronic means. Also to be reported are temperature of specimens and confining pressure (triaxial test). A transducer is a device designed to convert a measurable variable (load, pressure, strain, etc.) into some sort of electrical signal. A signal conditioner accepts the signal from its type of transducer, then applies amplification, zero offset, and linearizing as necessary to provide an output voltage signal which varies linearly with the input measured quantity, and spans a specified full range (e.g., 0 to 10 volts, -5 to +5 volts, 0 to 1 volt, etc.).

3.1 Transducers

3.1.1 Load Monitoring is most often achieved by strain gauge load cells in resilient modulus systems. There is a wide selection of cells varying in profile, ruggedness, environment capability and mounting, and of course, price. Since a specimen must be 'pushed on' to be stressed, it is not difficult for the designer to find a place in the 'load line' to place a load cell.

3.1.2 Displacement measurement most commonly is carried out by the LVDT (Linear Variable Differential Transformer) which features little or no hysteresis, 'infinite' resolution, good stability and ruggedness[1].

Other sensors which can be used especially in the diametral test, but which may tend toward the disadvantages shown are:

- strain gauges applied directly to specimen: expensive and time consuming to apply.

1 *Handbook of Measurement and Control*, E.E. Herceg, Schaevitz Engineering, 1972.

- strain gauge "clip-on" extensometers: may lack needed sensitivity for diametral test (10 micro-inches), or may vibrate excessively during loading cycles.
- strain gauge "arm" sensors touching specimen from external stationary mounting: may respond to load frame flexing in addition to specimen deformation; tendency of specimen to shift laterally with each load pulse may mask deformations.

Both diametral and triaxial testing invariably use two LVDTs whose outputs may be summed in the signal path, and sometimes a third or fourth to read other deflections for Poisson's ratio or permanent deformation information.

The most convenient form of LVDT is the gauge head, which packages body, spring loaded core and tip all in one unit (Figure 3). Small gauge heads with precision ball bearings are available from several suppliers.

3.1.3 Pressure indication for triaxial testing may employ Bourdon gauges or mercury manometers for

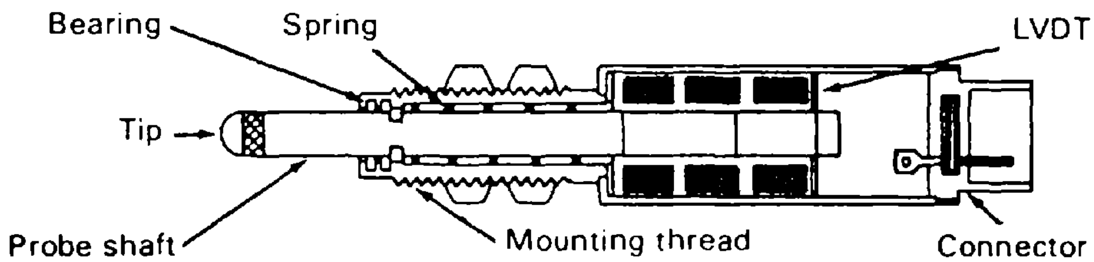


Figure 3 - Cross Section of Typical LVDT Gauge Head (after E.E. Herceg)

high or low cell pressures, respectively. Alternately, transducers of the variable reluctance or strain gauge type may be employed with suitable signal conditioning and readout.

3.1.4 Temperature monitoring of asphalt concrete specimens can be carried out quite successfully with traditional thermocouple or thermistor probes in contact with the specimen surface. Even if the specimen is in an environmental cabinet (constant temperature air bath), temperature should be reported using such a surface probe.

3.2 Signal Conditioning

This presents a real challenge to the system designer. The operator, probably not a researcher or instrumentation specialist, must make electronic adjustments to get meaningful dynamic data. In the case of the LVDT channels, interactive mechanical and electronic adjustments are usually necessary. Any design that blends convenience with operator confidence will increase efficiency. Calibration should be made easy to carry out, whether a quick confidence check or full periodic comparison against lab standards.

3.2.1 Options for the designer are many in the signal conditioning market. Manufacturers can supply conditioning in the following packaging:

- stand-alone cabinet
- multi-channel cabinet configurable with plug-in modules
- modules to be installed in users' cabinet
- printed circuit cards requiring mounting and power supply

On the other hand, the designer is free to specify and have built his own custom conditioning system. This could involve significant development time as well as some liability on the part of the equipment manufacturer to provide some guarantee of long-term performance.

4.0 DATA ACQUISITION AND CONTROL

Most would agree that the days of the strip chart recorder and clipboard are numbered as the exclusive data acquisition (DA) devices of resilient modulus testing. The PC or microprocessor-based DA unit, just a few years ago so expensive and clumsy to implement, now boasts these characteristics:

- faster sampling rate (frequency response)
- higher accuracy
- easier to reconfigure for different sampling modes
- more convenient storage and retrieval of records
- lower cost
- computational and control capabilities

It is this last feature that enables the PC to become the control center of a very powerful and configurable laboratory data system. Hardware and software have proliferated in recent years; each month brings new offerings in the DA field.

4.1 Data Acquisition

4.1.1 The basic elements of an automated DA system (Figure 4) are:

- time-varying signals of interest which have been signal-conditioned to a uniform scale, usually several volts magnitude
- a multiplexer or scanner mechanism which can switch among the signals on command
- an analog-to-digital converter (A to D or A/D) to convert ("digitize") the sampled voltage levels to binary words (bytes)
- a buffer or short-term memory of adequate size to hold the rapidly sampled set of voltage values
- a controller with clock to provide necessary timing commands to the A/D converter, multiplexer, and buffer

These DA systems are available to the designer in a large variety of configurations; most commonly they are employed using a host PC (IBM or compatible, Apple, HP or other), and comprise either (Figure 5):

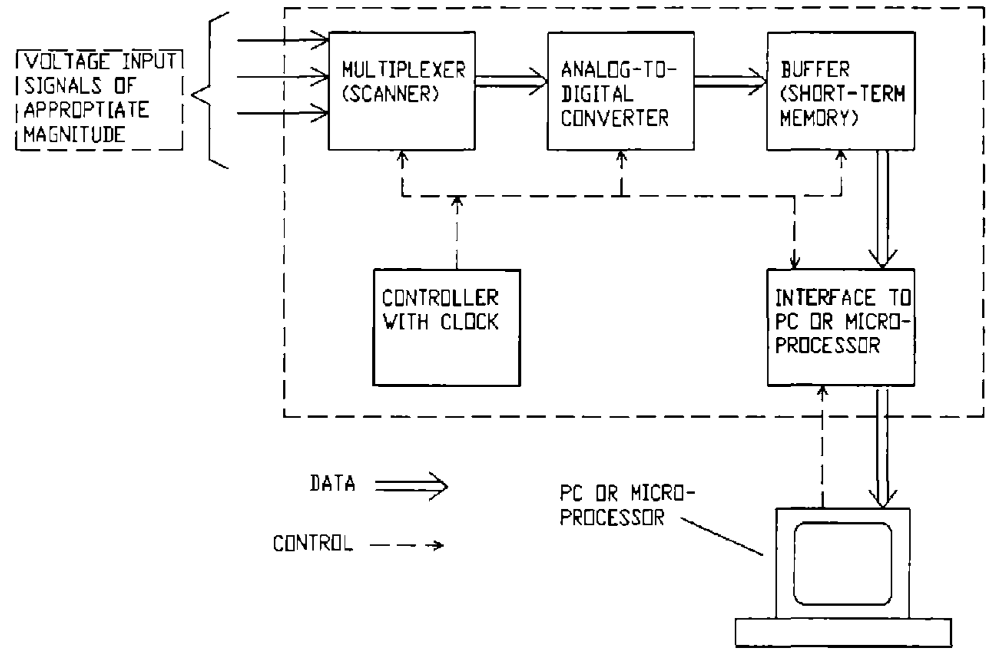
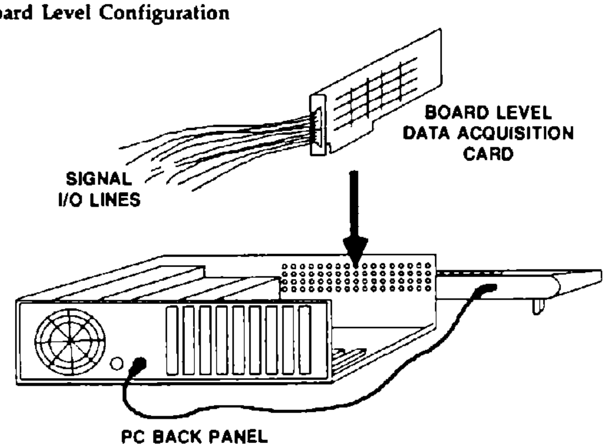
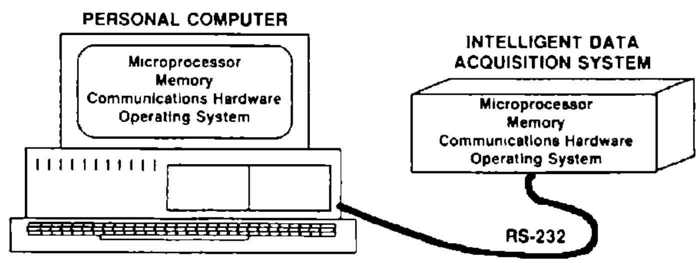


Figure 4 - Basic Elements of Automated Data Acquisition System

Board Level Configuration



Intelligent Data Acquisition System Architecture



Typical Expansion Chassis Configuration

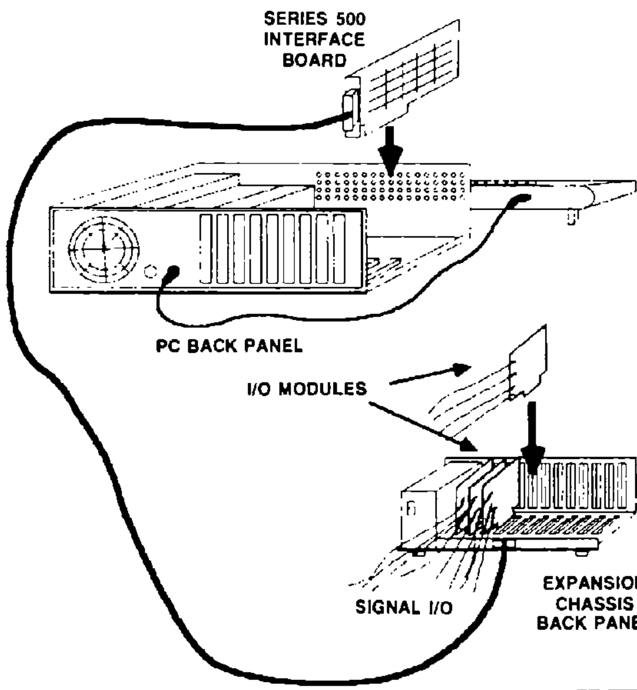


Figure 5 - PC-Based Data Acquisition Arrangements
(After Keithley Inst. Corp.)

- a card to fit in a PC an expansion slot, or
- a module cabinet intended to communicate via a cable data link to the PC (Figure 5).

Sampling rates in excess of 1,000 samples/sec are quite adequate for modulus testing, and are widely available in DA add-ons for PCs at an economical price. Full-scale resolution of 12-bits (1 part in 4,096) or 16-bits (1 part in 65,536) provide ample resolution of the sampled signal.

4.1.2 The Host Computer or microprocessor controls the DA section of the system, and can also perform the following functions:

- graphical display of sampled dynamic load and displacement waveforms
- initial data processing of sampled values to obtain preliminary results
- file generation, recording, and retrieval
- on-screen status reports and prompts to operator
- report generation at end of test sequence

Indeed, there is really no reason the host computer cannot be interactive with the operator at every step of the test process, for example, even prompting the new trainee in the set-up of the specimen or reminding personnel of need for a scheduled calibration.

4.2 Control of Test System

It is now well within the capabilities of the faster PCs (286- and 386-based) to be used directly in control of closed loop servo feedback systems. To achieve this, one of these higher speed machines can be programmed to perform the following sequence in a fraction of a millisecond:

- scan several analog input channels (one of which is the intended variable, say load) and convert to digital data (fast A- to -D processing).
- compare this most recent load data to the most current value of intended load (the load command signal)
- output a correcting analog error signal, sized appropriately to minimize the next error signal (this analog output is from a D- to -A convertor)

This corrective analog signal would then be used to drive the closed loop servo valve (Figure 6). Thus, the faster PCs could be applied directly in the closed loop itself, as well as doing all the data acquisition.

Several variations on this theme of PC control are possible, with the PC tied either directly to actuator control

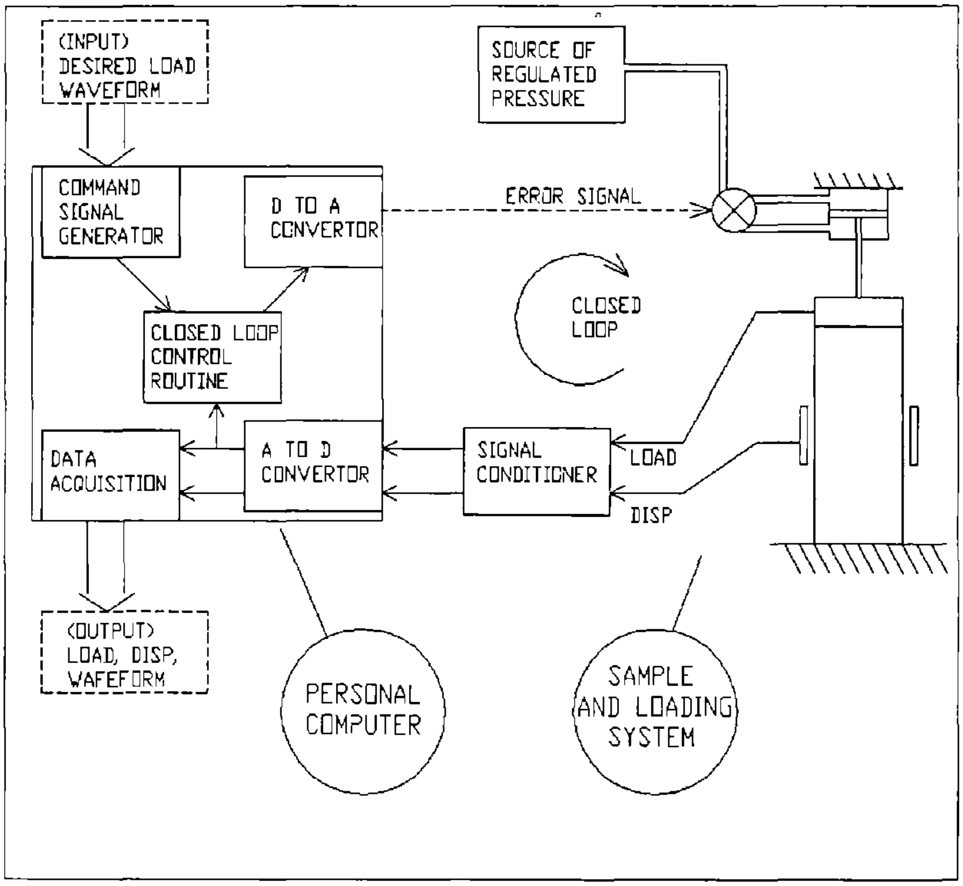


Figure 6 - Using a PC Directly for Closed Loop Control

duty (inner closed loop control), or with the PC indirectly commanding and monitoring an analog closed loop controller (Figure 7).

5.0 FUTURE DEVELOPMENTS

The complexity of equipment used for resilient modulus testing, especially in the triaxial test set up, can intimidate and frustrate new users. This author sees the bulk of future developments aimed at reducing complexity, not only to ease the training burden for new operators, but also to increase reliability and through-put of the testing process.

5.1 The Role of the PC

PCs will steadily gain ground as the central instrument of measurement, control, turnkey operation, and even training. A prolific amount of A to D and D to A hardware is already available. Now it is the programmer's task to structure software for the resilient modulus tests, both diametral and triaxial. Expect to see more use of menus, graphics, and interactive screen prompting.

5.2 More Use of Closed Loop Control

This is likely for both pneumatic and hydraulic systems. At least one manufacturer offers an air servo valve and matching servo amplifier valve driver (Dynamic Valves, Inc., Palo Alto, CA, (415\494-2333).

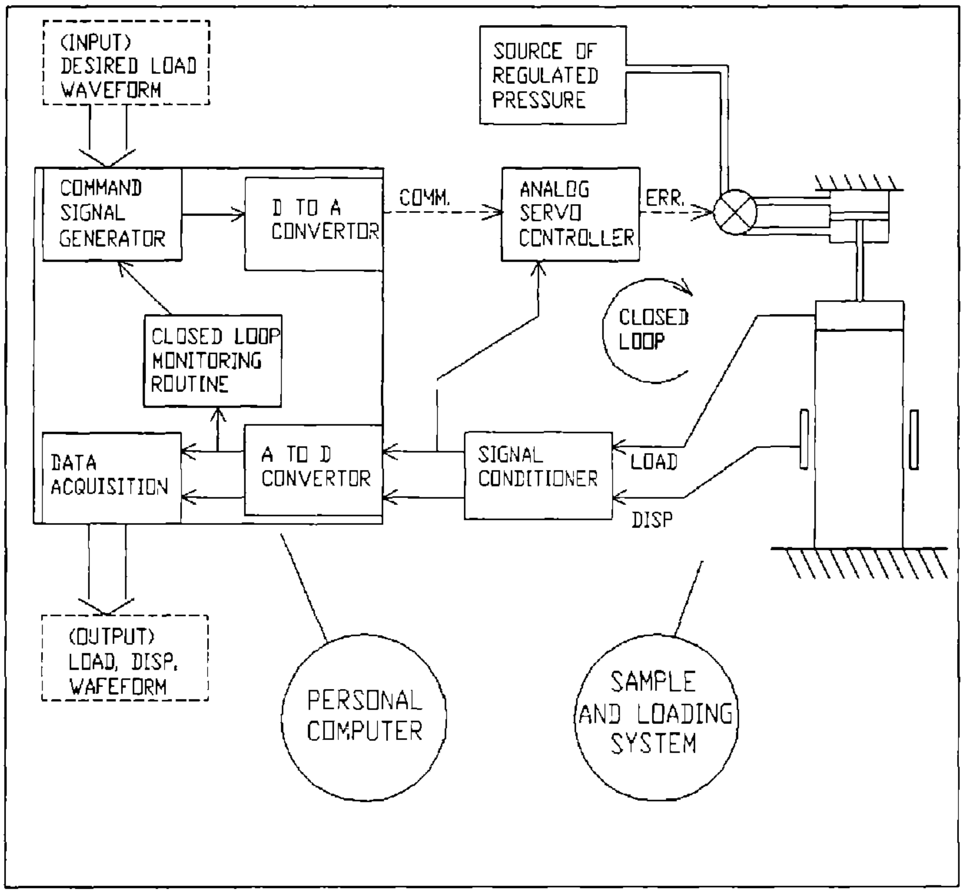


Figure 7 - Using a PC to Monitor an Analog Closed Loop Controller

5.3 Better Fixtures

Better Fixtures to hold diametral specimens and LVDTs as well as place them accurately in the load line are needed. Accuracy and uniformity could be improved, and efficiency increased.

5.4 Non-Contact Displacement Transducers

These may be sufficiently evolved in a few years to meet the stringent demands of the diametral test. Required resolution of 10 micro-inches and response to 10 to 50 Hz is taxing on LVDTs and associated signal conditioning, but laser displacement technology has made recent strides in these directions, as well as in size and price reductions.

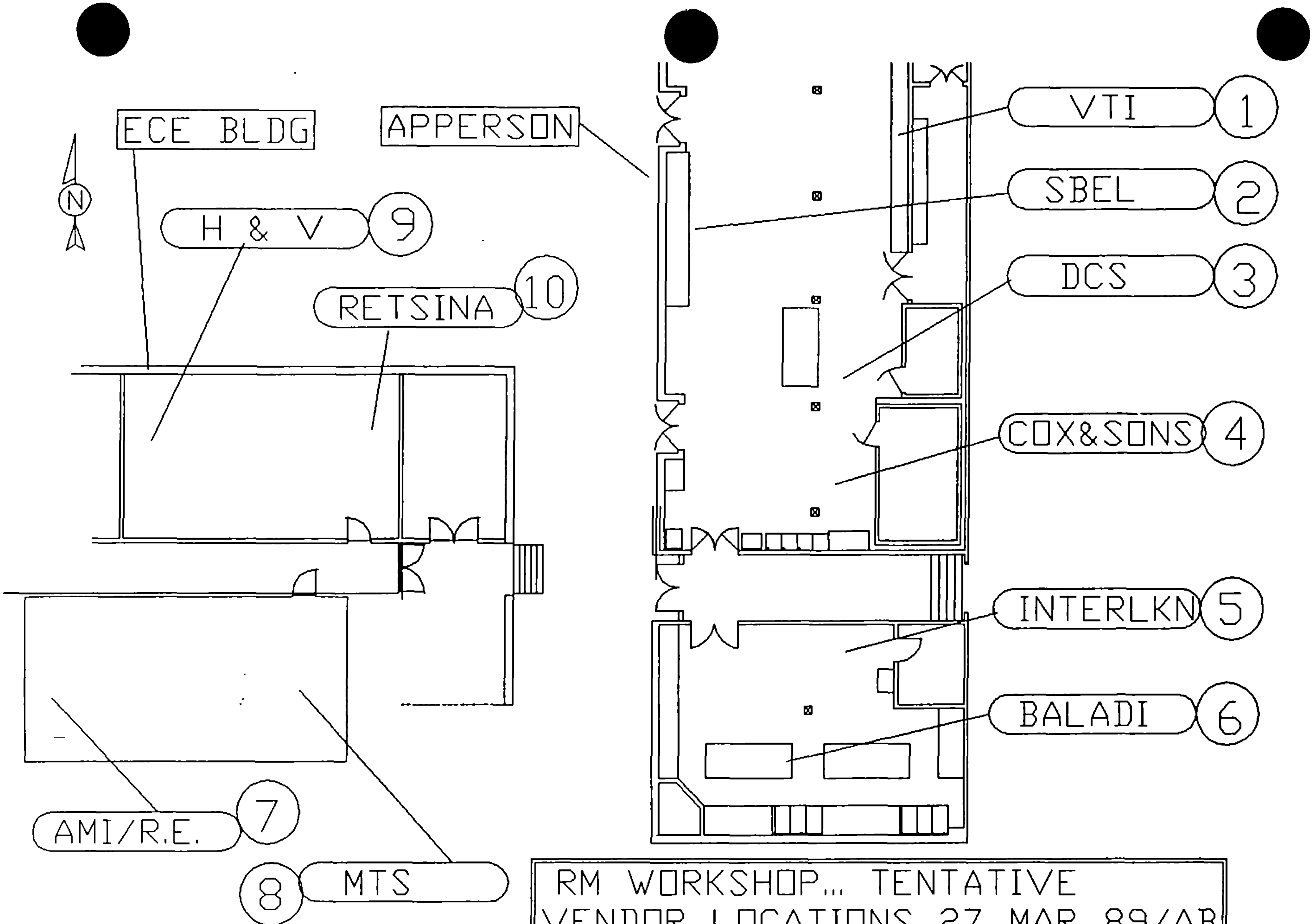
6.0 CONCLUSIONS

Recent improvements to the Repeated Load Resilient Modulus Test equipment mostly have involved the data acquisition portion of the system. These improvements have been significant and worthwhile with many designs exploiting the power of the personal computer for rapid data capture and increased operator interaction. Further efforts to refine the system should concentrate on the goal common to the design of any quality measuring instrument:

- accuracy and repeatability
- ease of use
- ruggedness and dependability
- reasonable cost

- ease of maintenance

As more of these systems are fabricated and then applied in daily lab use, it is hoped that all of these attributes can be achieved through diligent design and application of the most up to date measurement technology.



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INFLUENCE OF METHOD OF SAMPLE PREPARATION ON RESILIENT MODULUS OF ASPHALT CONCRETE

by

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March 1989

TEXAS TRANSPORTATION INSTITUTE

A COMPARATIVE EVALUATION OF LABORATORY COMPACTION DEVICES BASED ON THEIR ABILITY TO PRODUCE MIXTURES WITH ENGINEERING PROPERTIES SIMILAR TO THOSE PRODUCED IN THE FIELD

By

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and HAROLD VON QUINTAS

PRESENTED AT

THE 68th ANNUAL MEETING
TRANSPORTATION RESEARCH BOARD
WASHINGTON, D.C.

JANUARY 1989

TEXAS TRANSPORTATION INSTITUTE
THE TEXAS A&M UNIVERSITY SYSTEM
COLLEGE STATION, TEXAS

A COMPARATIVE EVALUATION OF LABORATORY COMPACTION DEVICES BASED ON THEIR ABILITY TO PRODUCE MIXTURES WITH ENGINEERING PROPERTIES SIMILAR TO THOSE PRODUCED IN THE FIELD

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August 1988

ABSTRACT

A major objective of the Asphalt Aggregate Mixture Analysis System (AAMAS), sponsored by the National Cooperative Highway Research Program (NCHRP), is to insure that laboratory-molded specimens will be fabricated in a manner which will adequately simulate field conditions and consequently yield reliable engineering properties. This paper describes a field and laboratory study which evaluates the ability of four compaction devices to simulate field compaction.

The compaction devices evaluated were selected based on their availability, uniqueness in mechanical manipulation of the mixture, and potential for use by agencies responsible for asphalt mixture design. The devices evaluated are: 1) the Mobil steel wheel simulator, 2) the Texas gyratory compactor, 3) the California kneading compactor, and 4) the Marshall impact hammer.

The ability of the four laboratory compaction devices to simulate field compaction is based on the similarity between engineering properties (resilient moduli, indirect tensile strengths and strains at failure, and tensile creep data) for laboratory-compacted samples and field cores. Five projects were selected for this study. Project locations were in Texas, Virginia, Wyoming, Colorado, and Michigan. The field compaction procedure used at the sites was the standard procedure used by the state highway departments responsible for the highways involved.

Overall, the Mobil steel wheel simulator demonstrated the ability to produce mixtures with engineering properties nearest those determined from field cores. The Texas gyratory and California kneading compactors ranked second and third, respectively, but with very little difference between the two. The Marshall mechanical hammer ranked a distant last.

INTRODUCTION

In the last few years, the highway community has acknowledged the need to improve existing asphalt concrete paving mixture design methods. Heavier traffic loads, higher contact pressures and a greater awareness of environmental effects have made highway engineers aware of the need to develop a design system which is able to optimize the selection, proportioning and manufacturing of asphalt binders and aggregates in order to produce pavements resistant to all forms of distress.

The National Cooperative Highway Research Program (NCHRP) is sponsoring the development of an asphalt aggregate mixture analysis system (AAMAS) for the laboratory evaluation of asphaltic mixtures. A major objective of the study is to insure that laboratory specimens will be fabricated in a manner which adequately simulates field compaction and consequently will yield reliable engineering properties.

Brent Rauhut Engineers, Inc. (BRE), is the prime contractor of the AAMAS project. Under this research program, BRE and their subcontractors are developing a design system based on performance-related criteria that will account for a wide range of distress mechanisms, e.g., fatigue cracking, thermal cracking, permanent deformation, moisture damage, age hardening, etc.. The evaluation system will also set standards for the preparation of test specimens, conditioning of the specimens, testing of the specimens, and criteria for mixture selection.

The Texas Transportation Institute (TTI), under the direction of Brent Rauhut Engineers, Inc., was responsible for the implementation of an experimental program to evaluate the elements of laboratory sample preparation necessary to duplicate field conditions closely enough to yield realistic engineering properties of the asphalt concrete mixtures.

The specific objective of the research conducted by TTI was to evaluate a variety of laboratory compaction methods which are widely used and/or have the potential to simulate field compaction. The study was to select the compaction technique best able to achieve material and engineering properties (percent air voids, strength, stiffness, etc.) similar to those of the material placed in the field using standard compaction practices.

Field sites were selected by research team members for the study. Sites were selected so that a wide range of aggregate sizes and gradations would be represented; yet, the ability and probability of maintaining close control of field variables was also a requirement.

Four compaction devices were selected for the study: 1) Texas gyratory shear compactor, 2) California kneading compactor, 3) Marshall mechanical compactor, and 4) Mobil steel wheel simulator. These devices were selected because of the unique compaction techniques produced by each, e.g., gyratory action, kneading action, impact, and simulated rolling wheel, respectively (2,3,4,5,6,7).

Two other compaction devices were initially considered in this study. The Arizona vibratory kneading compactor (8) and the Waterways Experiment Station (WES) gyratory compactor. Limited testing with the Arizona device indicated that it did not produce mixtures which closely simulate field engineering properties. The WES gyratory (3), was not made available for this study; however, it is the intent of AAMAS to evaluate this device as part of an extended program.

RESEARCH APPROACH

The first step in the evaluation was to collect field cores and samples of asphalt, aggregate and loose mix from the drum plants and transport them in sealed containers from the field projects selected for this study to the laboratory. Specimens were manufactured by reheating the loose mix in the laboratory oven and then compacted at or near the same average air void content that was produced in the field by a traditionally-used field compaction method.

Compaction curves (compaction energy versus air void content) were developed for each material and for each compaction device in order to select the energy required for each device to produce the target air void content established by the field cores. Once laboratory samples were fabricated with each compaction device at the target air void content for each site, triplicate field cores and laboratory-prepared samples were tested for indirect tensile strength, indirect tensile creep, and diametral resilient modulus. Each test was performed at 77°F.

The laboratory compaction methods were prioritized based on their ability to produce samples with engineering responses in close agreement with those measured from field cores.

The full-scale test matrix consisted of five field projects (which will be discussed subsequently), four compaction devices, one test temperature (77°F), and three replicates per test cell. The mean and variance of the laboratory specimens compacted with the different devices were computed and compared with field core statistics to determine whether or not a significant difference existed between the field cores and the laboratory-fabricated samples for each engineering property, i.e., tensile strength, tensile strain, resilient modulus, and tensile creep. Secondly, mean squared error (MSE) was used as a criterion by which to rank each laboratory compaction device with respect to the way it predicted field compaction results based on resilient modulus and indirect tension testing. Indirect tensile creep data were compared and evaluated based on power law curves which best fit the accumulated strain versus time of loading for the mean response from three replicate samples. The power curves for the laboratory mixtures which most closely approximated the power curve for the field cores were ranked highest. The comparison of creep curves was based on a t-test statistic for the slope of the log-log plot of the compliance versus time of loading. Indirect tensile creep testing was performed in lieu of compressive creep testing because of size (thickness) limitations from field cores. Since many of the field mixtures were placed as relatively thin overlays, cores were not thick enough for compressive creep testing.

Selection of Field Projects

Field sites were selected based partly on the potential to exercise control over construction and material variables influencing compaction (1). The first criterion for selection was that the project must meet certain minimum standards which insured that the variability of the following factors could be adequately controlled: compaction process, aggregate consistency and gradation consistency, base placement temperature, mixture placement temperature, consistency of the mixing plant, and air void content. Other variables which were considered in project selection included: asphalt type and grade, aggregate size and type, and mix plant type.

Five field projects were selected. Each project possessed unique characteristics which are discussed below:

Colorado: The site selected was a section of a two-lane rural highway, designated as State Route 9. The goal of this project was to extend pavement life by means of an overlay. The process began with a leveling course averaging 1.5 inches in thickness. On top of the leveling course, a nonwoven geotextile (Trevira) was placed. This was followed by the placement of two lifts, each 1.5 inches thick, of a dense-graded (" inch top size aggregate), surface course mix. The surface course layer was evaluated in this study.

Michigan: This project was an overlay for a rural two-lane highway designated as State Route M21. In order to assure that cores were at least 1.5 inches in height, the state and contractor agreed to increase the mat thickness in the area of the test sections to 1.75 inches.

Texas: The Texas project was located on Highway 21. This was a major reconstruction project converting an existing two-lane roadway into a four-lane divided highway. The thickness of the asphalt concrete lifts vary transversely across the roadway from 2 to 3 inches.

Virginia: This was a reconstruction project of a two-lane highway designated State Highway 621. It consisted of placing 4 inches of an asphalt concrete base on top of an untreated aggregate base course. An asphalt concrete binder and surface course were to be placed on top of the asphalt concrete base mix.

Wyoming: This project consisted of an overlay of a four-lane divided interstate highway, designated as IH-80, with a recycled mixture. Four inches of the existing asphalt concrete pavement in the driving lane in each direction was removed by cold planing. This material was recycled back into the asphalt concrete mix on a basis of 40 percent reclaimed material and 60 percent new aggregate.

More detailed characteristics of these projects are depicted in Tables 1 and 2. These tables refer to the aggregate blends and asphalt types. Several other considerations should be noted - all mixtures were fabricated in drum mix plants and placed at temperatures ranging from 275 to 310°F. The compaction trains utilized in the field projects included: a) vibratory rolling for breakdown compaction followed by static rolling for finish compaction, b) static rolling for breakdown compaction followed by pneumatic rolling for intermediate compaction and static rolling for finish compaction, and c) pneumatic rolling for breakdown compaction followed by static rolling for finish compaction. All projects were constructed during the summer of 1988 so that the base placement temperature varied only from 90° to 100°F.

Material Handling and Specimen Preparation

The sampling of asphalt concrete mixtures for laboratory specimen preparation was performed in such a way as to insure random selection of trucks and to prevent segregation of mixtures. Properly sealed containers were used to transport mixtures and great care was taken to provide full mixture documentation and temperature histories.

The sampling of asphalt concrete cores from the roadway was performed in accordance with the following sequence:

1. Drill cores from pavement test section,
2. Allow cores to cool and dry,
3. Identify cores by test section and subset, and
4. Wrap cores with clear tape (for protection during transportation) and place them in zip-lock bags (impermeable to air and water).

Figure 1 depicts the flow chart of the compaction study developed for specimen preparation.

Approximately 25 field cores were drilled from each field project the next day after rolling compaction. These cores were then collected and stored in the laboratory for testing. Nine of the 25 cores were tested for resilient modulus (3 replicates), indirect tensile stress and strain at failure (3 replicates), and indirect tensile creep (3 replicates). Since material properties of these field cores were determined on sets of three replicate samples, it was necessary to arrange field cores in sets in a manner that would minimize the variance of air void content within a set and yet so that there would be no statistical difference between the mean of air voids in any set and the mean of air voids in the overall project. Laboratory-compacted samples were prepared at a void content approximately equal to the mean void content of all field cores from a selected project.

Table 3 presents air void content summaries for all field projects. The field section selected was that which used a compaction procedure prescribed by the state agency in which the project was located. The program for preparation and compaction of laboratory specimens was as follows:

1. Reheat the loose mix to the same compaction temperature as was used in the field.
2. Determine compactive effort (7,9,10), by trial and error, for each compaction device required to produce the mean air void content derived from field cores for the project in question.

Table 4 presents a summary of the compactive efforts required for each compaction device and for each project to equal the mean air void content of field cores.

Testing Methods

Once the compactive efforts were determined, a series of specimens were prepared with the same air void content (± 0.5 percent air voids) as the related field project. Sets of three samples per test, whose means were not significantly (statistically) different from the overall mean air void content of the field cores, were prepared for testing at 77½F. The tests performed were the diametral resilient modulus test, indirect tensile creep, and the indirect tensile strength test.

Indirect tensile strength tests were performed by BRE in accordance with Test Method TEX-226-F of the Texas State Department of Highways and

Public Transportation at 77½F, at a loading rate of 2.0 inches per minute. This test was performed on mixes and field cores from 3 projects: Michigan, Texas, and Virginia. The repeated load indirect tensile test (resilient modulus) was performed in accordance with ASTM standard D4123-82. It was conducted by applying compressive loads of a haversine waveform (11). The resulting horizontal deformation of the specimen was measured and used to calculate the resilient modulus. The load applied to the specimens was determined on the basis of the indirect tensile strength (IDT) test results. Ten percent of the stress to failure in the IDT was the stress applied to the specimens to produce deformation in the elastic range without damaging the sample. Table 5 shows the indirect tensile strength test results at 77½F for field cores.

The indirect tensile creep was performed in the same way as the resilient modulus except that a static load, in lieu of a cyclic load, was continuously applied for 60 minutes and then removed. Deformation was measured during the loading and recovery periods (12).

RESULTS

Results of resilient modulus testing at 77½F are presented in Table 6. A review of these data demonstrates that only the values boxed, prepared by Marshall impact compaction, are statistically different from the field cores data ($\alpha = 0.05$) (13).

The mean squared error (MSE) was used to rank-order the abilities of the various compaction devices to produce samples which produce engineering properties similar to those produced by field compaction. The MSE is defined as:

$$MSE = (\bar{x} - \nu)^2 + s^2$$

where: $\bar{x}$ = mean of the test value,

ν = target value or the mean of the field test data, and

s^2 = variance of the laboratory test values.

Table 7 presents MSE data derived from resilient moduli data from laboratory-prepared samples compared to field cores. Of course, a lower MSE, indicates that the compaction device in question better simulates field conditions as measured by resilient moduli at 77½F. The results are presented by project and in summary form. The gyratory and steel wheel compactors produce samples closest to field conditions. Next, is the kneading compactor followed distantly by the Marshall compactor.

Table 8 presents stress and strain data from the indirect tensile test (IDT). The data which are significantly different from the field core data are boxed in Table 8. Table 9 presents MSE values comparing laboratory-prepared samples with field cores computed from stress and strain data from the IDT test. Based on the average MSE rankings, samples compacted by the gyratory compactor are most similar to field cores followed by those compacted by the steel wheel simulator, the kneading compactor and the Marshall hammer.

Table 10 is a summary of average difference between field cores and laboratory-compacted specimens based on data from resilient modulus and IDT testing (stress at failure and strain at failure). In this table, the absolute difference represents the accumulated difference between field cores and laboratory-prepared specimens based on the test values.

Figure 2 through 6 present plots of the accumulated permanent strain versus time of loading from the indirect tensile creep test. These data were analyzed as to the difference between data from field cores and laboratory-prepared samples by statistically comparing the slopes and intercepts for each relationship (produced from laboratory-compacted samples) with that produced from field data. Table 11 summarizes these data.

In the computation of the t-test statistics (14), which was used to evaluate the difference among tests, the null hypothesis was:

$$H_o : \beta = \beta_o$$

where: β_o = the slope of the log-log plot of accumulated permanent strain versus time of loading for tests on field cores,

β = slope of the log-log plot of accumulated permanent strain versus time of loading for tests on laboratory-compacted specimens.

The t-test statistic was computed as:

$$T_{(n-2)} = \frac{\beta - \beta_o}{S_b}$$

where: S_b = standard error of the slope for the laboratory-compacted method in question.

The experiment has n-2 degree of freedom, where n = number of samples tested in the experiment.

The t-test statistic is presented in the last column of Table 11. Only the boxed entries represent statistically significant differences between the creep results for the compaction procedure in question and the field core data at the $\alpha = 0.05$ level of significance.

Although the results vary from project to project, the average rankings based on the t-test statistics are summarized at the bottom of Table 11. Once again, the steel wheel simulator is ranked first. The kneading compactor is now second followed by the gyratory compactor with the Marshall, a distant last. In this ranking, the difference between the steel wheel and kneading compactor was not significantly different ($\alpha = 0.05$). All other comparisons were statistically valid ($\alpha = 0.05$).

Tables 12 and 13 present a summary of the evaluation. Table 12 presents the overall average ranking for all tests, the percentage of the time the results were ranked first and the percentage of time they were ranked either first or second. A high ranking indicates high relative similarity between engineering test results from lab and field cores.

Table 13 shows how often the difference between a test (mean of three replicates) from a specified laboratory comparison procedure was not significantly different from the data from field cores for the specified project.

DISCUSSION

The Mobil steel wheel simulator most closely simulates field compaction based on all mixture properties: resilient modulus, indirect tensile strength and strain at failure, and indirect tensile creep. However, this high correlation may be partially influenced by the disturbance effect involved in the core drilling operation required to obtain both field cores and specimens compacted with the steel wheel. No other laboratory compaction processes require coring. Additionally, the potential to use the Mobil steel wheel simulator as a standard laboratory procedure is hampered by the difficulty of using the device. The device prepares samples in the form of a 6" x 12" x 4" prism. To produce a standard 2.5 inch x 4-inch cylinder requires a large core drill to core a specimen from the compacted prism. Substantial modifications to the system would be required to produce cylindrical specimens capable of being tested in compressive creep or for axial or diametral resilient modulus. Two persons are typically required to perform the compaction procedure.

The Texas Gyratory Shear Compactor proved to be very effective with traditional mixes (top size less than 3/4 inches) but did a poorer job with coarse mixes such as those from the Virginia and Wyoming projects. However, because of its simplicity of operation and the potential to use the larger gyratory models capable of fabricating large-size specimens, and thus accommodating large-sized aggregate, the Texas gyratory seems to be the most prudent choice as the compaction device to be used for future preparation of specimens for mixture design/analysis.

Although the kneading compactor ranked third in its ability to replicate field conditions, its level of simulation of field mixes based upon their engineering properties is not greatly different from the Texas Gyratory and the Steel Wheel Simulator. Furthermore, the California Kneading Compactor is the only laboratory device capable of fabricating any size of cylindrical specimen as well as rectangular beams. This capability makes the kneading compactor an appealing choice for compaction purposes.

The Marshall mechanical hammer did the poorest job of simulating field conditions. The absence of kneading effect during the compaction operation due to the uniform impact type of load applied by the mechanical version of the Marshall hammer is probably the major reason behind the poor correlation shown by this compaction device. A manual compactor, is expected to perform somewhat better because the tamping imparted by the operator will not always fall in the same portion of the specimen, thus providing some kind of rearrangement of aggregate particles after every blow.

CONCLUSIONS

The Mobil steel wheel simulator closely simulates field compaction based on a comparison of selected engineering properties of mixtures from five field sites. However, this device cannot be broadly implemented because of its inability to produce specimens of the varied geometries which will be required by AAMAS.

The Marshall impact hammer does not adequately simulate field compaction based on the results of this study and should not be used for samples compaction in the AAMAS study.

The leading candidates for laboratory compaction based on the criteria of acceptable ability to simulate field compaction and utility of use are the Texas gyratory compactor and the California-type kneading compactor. The Texas gyratory demonstrated a slight superiority in its ability to produce samples with material properties which simulated field conditions.

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Table 1. Summary of Aggregate Blends Used in Field Projects.

Aggregate Blend	Colorado		Texas		Michigan		Virginia		Wyoming	
Coarse	Pit run gravel	30%	3/4" crushed limestone	35%	Pit run gravel	39%	Trap rock #56	60%	RAP -	40%
			3/8" crushed limestone	33%			Trap rock #8	5%	Coarse gravel	40%
Fine	Pit run	70%	Limestone screenings	15.1%	Concrete sand	25%	Crushed fines #10	20%	Fine gravel	20%
			Field sand	16.9%	Blend sand	16%	Natural sand	15%		
					#CS sand	20%				

Table 2: Type and Percentage of Asphalt and Additives Used at the Job Site.

Material	Colorado		Michigan		Texas		Virginia		Wyoming	
Asphalt	Sinclair AC-10	5.5%	Marathon (85-100)	5.6%	Exxon AC-20	5.5%	Chevron AC-20	4.5%	Sinclair AC-20	2.75%
Additive	Pave bond	0.4%	----		----		ACRA 1000	0.6%	Hydrated Lime	1.00%

Table 3. Summary of Air Void Information from Field Cores.

Project	Statistical Data	Section (Compaction Train)		
		VB/SS	SP/SS	PB/SS
Colorado	Mean	8.19		
	Standard Deviation	0.936		
Michigan	Mean			4.21
	Standard Deviation			0.63
Texas	Mean		8.75	
	Standard Deviation		0.966	
Virginia	Mean	5.85		
	Standard Deviation	1.193		
Wyoming	Mean	5.77		
	Standard Deviation	0.688		

Note: VB/SS - Vibratory roller for breakdown compaction followed by a static steel wheel roller for finish compaction.
SP/SS - Static roller for breakdown compaction followed by a pneumatic roller for intermediate compaction and a static steel wheel roller for finish compaction.
PB/SS - Pneumatic-rubber tired roller followed by a static steel wheel roller.

Table 4. Summary of Compactive Efforts Required to Compact Laboratory Samples Using Different Compaction Devices to Simulate Air Voids in Field Cores.

1.	Marshall Hammer	(Blows per Face)
	• Colorado	20
	• Michigan	23
	• Texas	18
	• Virginia	47
	• Wyoming	34
2.	California Kneading Compactor	(Number of Tamps and Tamping Pressure) (psi)
	• Colorado	20 (250)
	• Michigan	25 (500)
	• Texas	20 (250)
	• Virginia	50 (500)
	• Wyoming	20 (250)
3.	Texas Gyratory	(Gyration Pressure, Number of Gyrations and Leveling Pressure - psi)
	• Colorado	25-3-250
	• Michigan	50-3-500
	• Texas	25-3-0
	• Virginia	100-3-2500
	• Wyoming	50-3-250
4.	Mobile Steel Wheel Simulator	(Number of Cycles or Coverages)
	• Colorado	28
	• Michigan	16
	• Texas	15
	• Virginia	175
	• Wyoming	38

Table 5. Summary of Indirect Tensile Strengths Data for Field Cores

Project	Indirect Tensile Strength (psi)
Colorado	
Mean	90
Standard Deviation	9.18
Michigan	
Mean	90
Standard Deviation	8.01
Texas	
Mean	119
Standard Deviation	35.20
Virginia	
Mean	224
Standard Deviation	16.30
Wyoming	
Mean	143
Standard Deviation	5.30

Table 6. Summary of Resilient Modulus Test Data at 77°F.

Project	Marshall	Kneading Compactor	Texas Gyratory	Mobile Steel Wheel Simulator	Field Cores
Colorado					
Mean (psi)	$\boxed{1,060,086}$ *	488,909	538,543	408,617	609,628
Standard Deviation	469,911	70,557	77,925	74,160	160,932
Coefficient of Variation (%)	44.3	14.43	14.47	18.15	26.40
Michigan					
Mean (psi)	422,829	458,974	499,710	330,519	358,911
Standard Deviation	24,384	151,078	307,908	27,037	64,029
Coefficient of Variation (%)	5.77	32.92	41.61	8.18	17.84
Texas					
Mean (psi)	$\boxed{1,308,290}$	799,658	743,792	756,678	697,233
Standard Deviation	167,120	28,309	21,882	22,404	35,362
Coefficient of Variation (%)	12.77	3.54	2.94	2.96	5.07
Wyoming					
Mean (psi)	933,121	607,062	997,155	819,403	758,836
Standard Deviation	55,173	43,578	169,271	181,862	171,717
Coefficient of Variation (%)	5.91	7.18	16.98	22.20	22.63
Virginia					
Mean (psi)	1,713,667	646,000	758,333	734,333	924,667
Standard Deviation	1,195,166	67,480	155,307	72,141	211,666
Coefficient of Variation (%)	69.74	10.44	20.48	9.82	22.89

* Boxed cells are those which are significantly different from field cores.

Table 7. Summary of MSE Values (in ksi) for Resilient Modulus Tests at 77°F - Ranking of Compaction Devices.

Project	Marshall	Kneading Compactor	Texas Gyratory	Mobile Steel Wheel Simulator
Colorado	423,400 (4)*	19,682 (2)	11,125 (1)	45,877 (3)
Michigan	4,672 (2)	32,301 (3)	63,145 (4)	1,513 (1)
Texas	401,210 (4)	11,393 (3)	2,693 (1)	4,084 (2)
Wyoming	33,301 (2)	25,040 (1)	85,205 (4)	36,724 (3)
Virginia	625,546 (4)	82,330 (3)	51,914 (2)	41,665 (1)
Average Ranking	3.2	2.4	2.4	2.0
1 rating	0	20%	40%	40%
1 or 2 rating	40%	40%	60%	60%

* ( ) = Number in parenthesis corresponds to the ranking of compaction devices.

Table 8. Summary of Mean and Variance Values of Tensile Strength and Strain at Failure - Laboratory-Compacted Specimens at 77°F and Field Cores.

Project		Marshall	Kneading Compactor	Texas Gyratory	Mobile Steel Wheel Simulator	Field Cores
Michigan						
IDT	Mean	$\boxed{103}$ *	94	84	89	90
(psi)	Variance	$\boxed{48}$	2.34	2.99	43	64
Strain at	Mean	$\boxed{9.19}$	$\boxed{9.01}$	14.56	15.67	14.56
Failure (mils/in.)	Variance	$\boxed{0.36}$	$\boxed{3.38}$	0.81	0.94	7.57
Texas						
IDT	Mean	$\boxed{172}$	$\boxed{147}$	129	172	119
(psi)	Variance	$\boxed{100}$	$\boxed{324}$	54.3	161	19
Strain at	Mean	$\boxed{3.81}$	7.37	9.01	7.00	9.01
Failure (mils/in.)	Variance	$\boxed{0.09}$	3.94	0.63	0.305	1.17
Virginia						
IDT	Mean	$\boxed{153}$	$\boxed{127}$	$\boxed{114}$	$\boxed{149}$	224
(psi)	Variance	$\boxed{219}$	$\boxed{80.3}$	$\boxed{112}$	$\boxed{26.3}$	266
Strain at	Mean	$\boxed{5.55}$	8.32	$\boxed{7.97}$	9.12	10.00
Failure (mils/in.)	Variance	$\boxed{0.69}$	2.16	$\boxed{0.09}$	0.50	0.13

* Boxed cells are those which are significantly different from field cores.

Table 9. Summary of Mean Squared Errors for Tensile Strength and Strain at Failure - Laboratory-Compacted Specimens at 77°F and Field Cores.

Project		Marshall	Kneading Compactor	Texas Gyratory	Mobile Steel Wheel Simulator
Michigan	IDT	217 (4)*	18 (1)	39 (2)	43 (3)
	Strain at Failure	29.1969 (3)	34.18 (4)	0.812 (1)	2.17 (2)
Texas	IDT	2,909 (3)	1,108 (2)	154 (1)	2,970 (4)
	Strain at Failure	27.13 (4)	6.63 (3)	0.63 (1)	4.35 (2)
Virginia	IDT	5,260 (1)	9,489 (3)	12,212 (4)	5,651 (2)
	Strain at Failure	20.49 (4)	4.98 (3)	4.21 (2)	1.27 (1)
Average Ranking		3.17	2.67	1.83	2.33
1 rating		16.17%	16.70%	50.00%	16.70%
1 or 2 rating		16.67%	33.33%	83.33%	66.70%

* ( ) = Number in parenthesis corresponds to the ranking of compaction devices.

Table 10. Summary of Average Differences Between Field Cores and Laboratory-Compacted Specimens at 77°F.

Compaction Device	Indirect Tensile Strength			Strain at Failure			Resilient Modulus		
	Rating	Absolute	Arith	Rating	Absolute	Arith	Rating	Absolute	Arith
Marshall Hammer	4	-45.67	-1.67	4	5.01	-5.01	4	417,744	417,744
California Kneading	3	-43.00	-21.67	3	2.96	-2.96	3	150,730	-69,734
Mobile Steel Wheel	2	-43.00	-7.67	2	1.33	-0.59	1	107,950	-59,945
Texas Gyratory	1	42.00	-35.33	1	0.68	-0.68	2	132,619	376,520

Average Ranking: Marshall (4), California Kneading (3), Steel Wheel (1.67), Texas Gyratory (1.33).
% Ranking 1st place: Texas Gyratory - 67%, Steel Wheel - 33%
% Ranking 1st or 2nd place: Texas Gyratory - 100%, Steel Wheel - 100%.

Table 11. Summary of Power Equations, Regression Data - Indirect Tensile Creep at 77°F.

Project	R-squared	Slope	Intercept	Std. Error of Slope	Std. Error Intercept	t-test (Slope)
Virginia						
Texas Gyratory	0.891563	0.576438	-1.27847	0.034476	0.215940	$\boxed{1.9062}$ (3)
Kneading Compactor	0.857157	0.552637	-1.22469	0.031285	0.239987	1.3398 (2)
Steel Wheel	0.889208	0.516630	-1.42876	0.031274	0.195884	0.1889 (1)
Marshall Compactor	0.803910	0.619924	-0.39349	0.042458	0.325696	$\boxed{2.5720}$ (4)
Field Cores	0.880600	0.510721	-1.33436	0.047001	0.208163	—
Colorado						
Texas Gyratory	0.948048	0.690015	-1.47302	0.022843	0.172820	$\boxed{3.5048}$ (3)
Kneading Compactor	0.931948	0.703549	-1.42503	0.027159	0.200192	$\boxed{2.4475}$ (1)
Steel Wheel	0.865860	0.657510	-1.33212	0.036970	0.276357	$\boxed{3.0448}$ (2)
Marshall Compactor	0.900131	0.557329	-1.42231	0.025994	0.199377	$\boxed{29.0678}$ (4)
Field Cores	0.880600	0.510721	-1.33436	0.047001	0.208163	—
Michigan						
Texas Gyratory	0.906760	0.729422	-1.30219	0.035669	0.239692	1.9150 (3)
Kneading Compactor	0.913237	0.679691	-1.42797	0.029336	0.222171	0.5830 (1)
Steel Wheel	0.927148	0.714616	-1.35888	0.028049	0.213949	0.6360 (2)
Marshall Compactor	0.851964	0.551779	-1.27532	0.0311896	0.244700	$\boxed{4.5460}$ (4)
Field Cores	0.880600	0.510721	-1.33436	0.047001	0.208163	—

Table 11. Summary of Power Equations, Regression Data - Indirect Tensile Creep at 77°F. (continued)

Project	R-squared	Slope	Intercept	Std. Error of Slope	Std. Error Intercept	t-test (Slope)
Texas						
Texas Gyratory	0.908856	0.687949	-1.39331	0.031444	0.230748	5.0675 (1)
Kneading Compactor	0.916190	0.600636	-1.44643	0.026784	0.185879	9.2090 (3)
Steel Wheel	0.935618	0.670726	-1.59638	0.024637	0.188536	7.1666 (2)
Marshall Compactor	0.880858	0.510343	-1.49552	0.027090	0.205865	12.4380 (4)
Field Cores	0.929432	0.847290	-1.58408	0.037384	0.235043	—
Wyoming						
Texas Gyratory	0.926571	0.590475	-1.59832	0.023051	0.230748	3.1770 (3)
Kneading Compactor	0.908457	0.585501	-1.38552	0.026025	0.199614	3.0040 (2)
Steel Wheel	0.924937	0.613214	-1.39623	0.024704	0.189197	2.0438 (1)
Marshall Compactor	0.890229	0.443410	-1.51394	0.021802	0.166715	7.1935 (4)
Field Cores	0.912511	0.663703	-1.35273	0.029358	0.216923	—

Average Ranking: Steel Wheel (1.6), Kneading (1.8), Texas Gyratory (2.6), Marshall (4).
% Ranking 1st Place: Steel Wheel (40%), Kneading Compactor (40%), Texas Gyratory (20%).
%Ranking 1st or 2nd: Steel Wheel (100%), Kneading Compactor (80%), Texas Gyratory (20%).

Table 12. Overall Ranking of Compaction Devices.

Compaction Device	Average Ranking	% 1st	% 1st or 2nd
Steel Wheel	1.90	32.42%	81.68%
Texas Gyratory	2.04	44.25%	60.83%
Kneading Compactor	2.47	19.17%	38.33%
Marshall Compactor	3.59	4.04%	14.20%

Table 13. Percentage of Cells That Are Not Significantly Different (a = 0.05) from Field Core Data.

Compaction Device	Percent
Marshall	12.5%
California Kneading	62.5%
Texas Gyratory	62.5%
Steel Wheel	75.0%

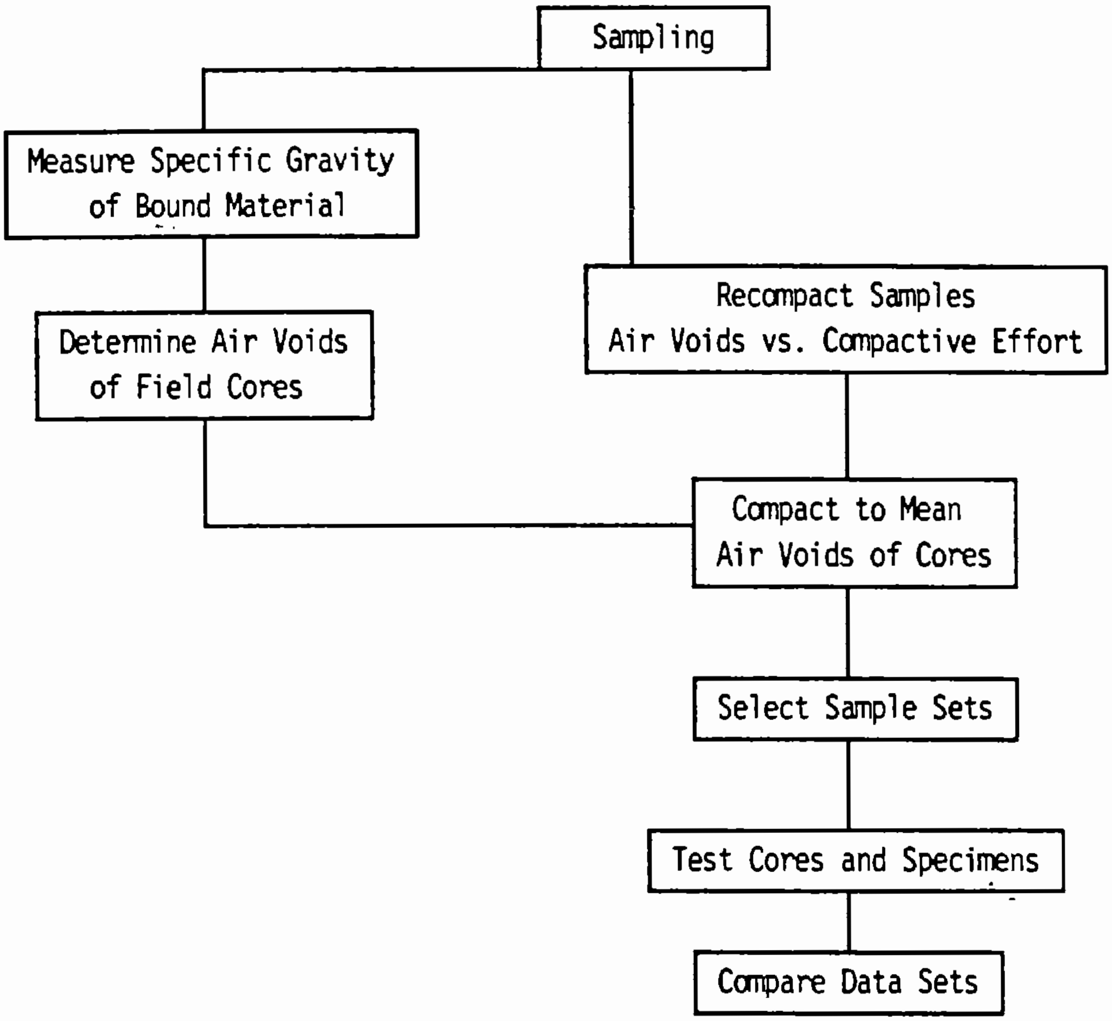

Figure 1. Flow Chart of Compaction Study.

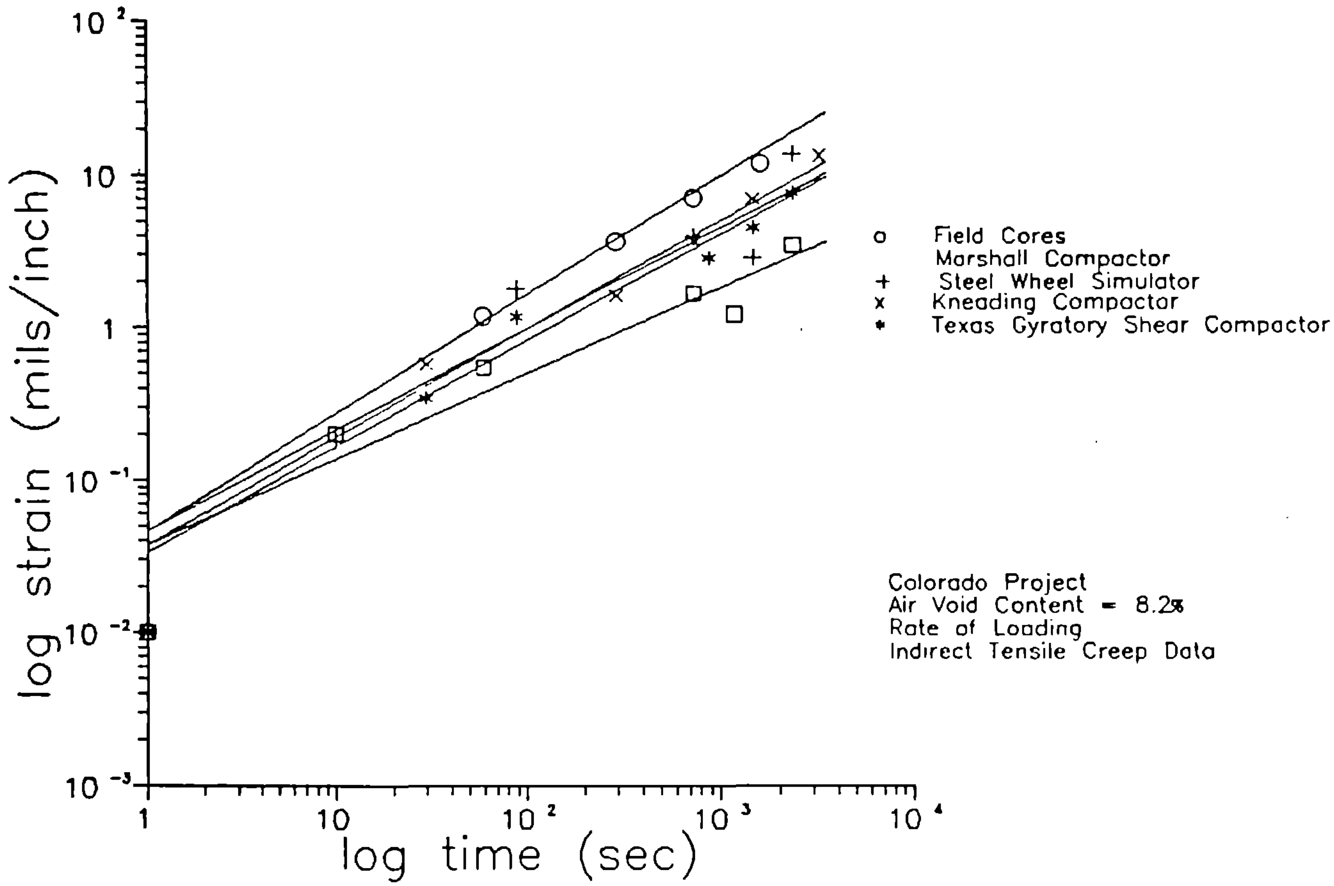

Figure 2. Comparative analysis of power curves for field cores and laboratory-compacted samples – Colorado project.

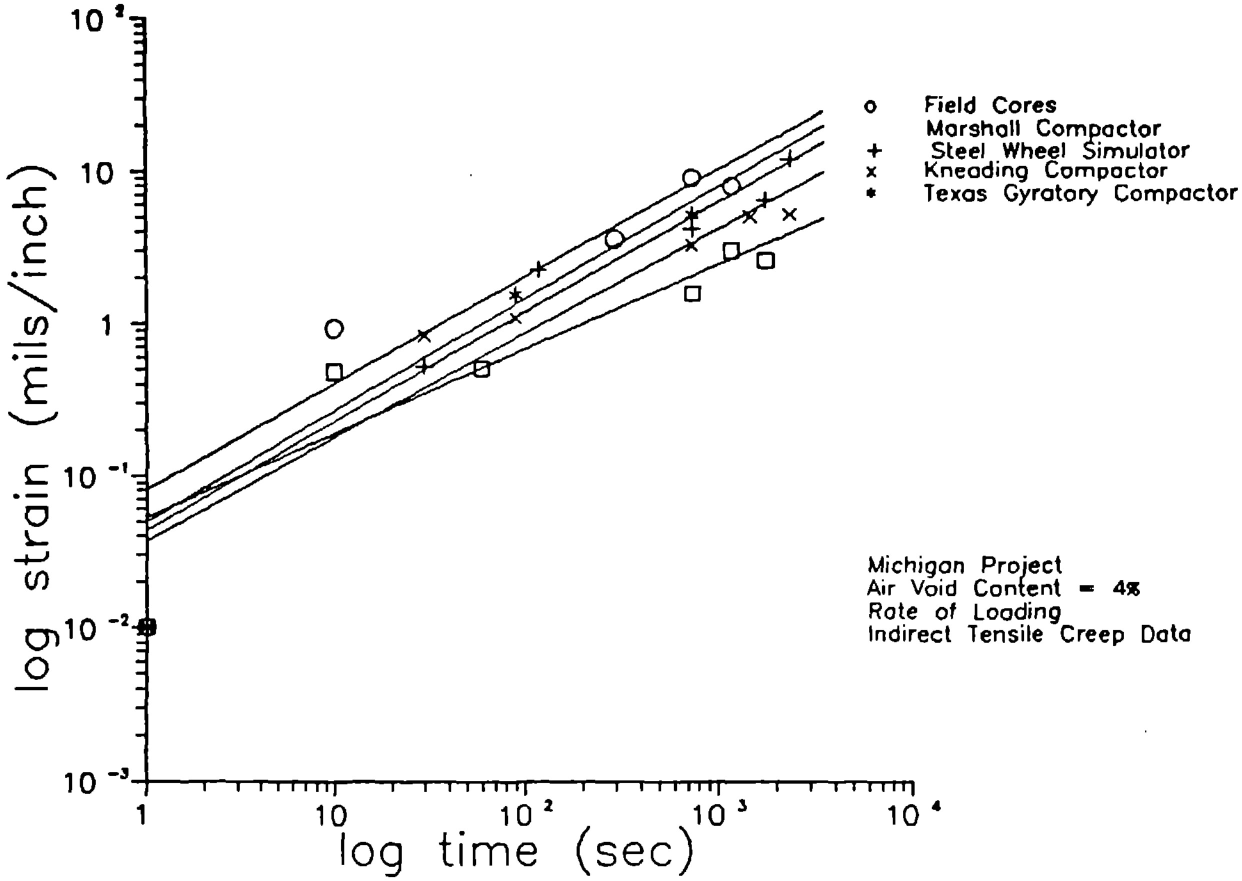

Figure 3. Comparative analysis of power curves for field cores and laboratory-compacted samples – Michigan project.

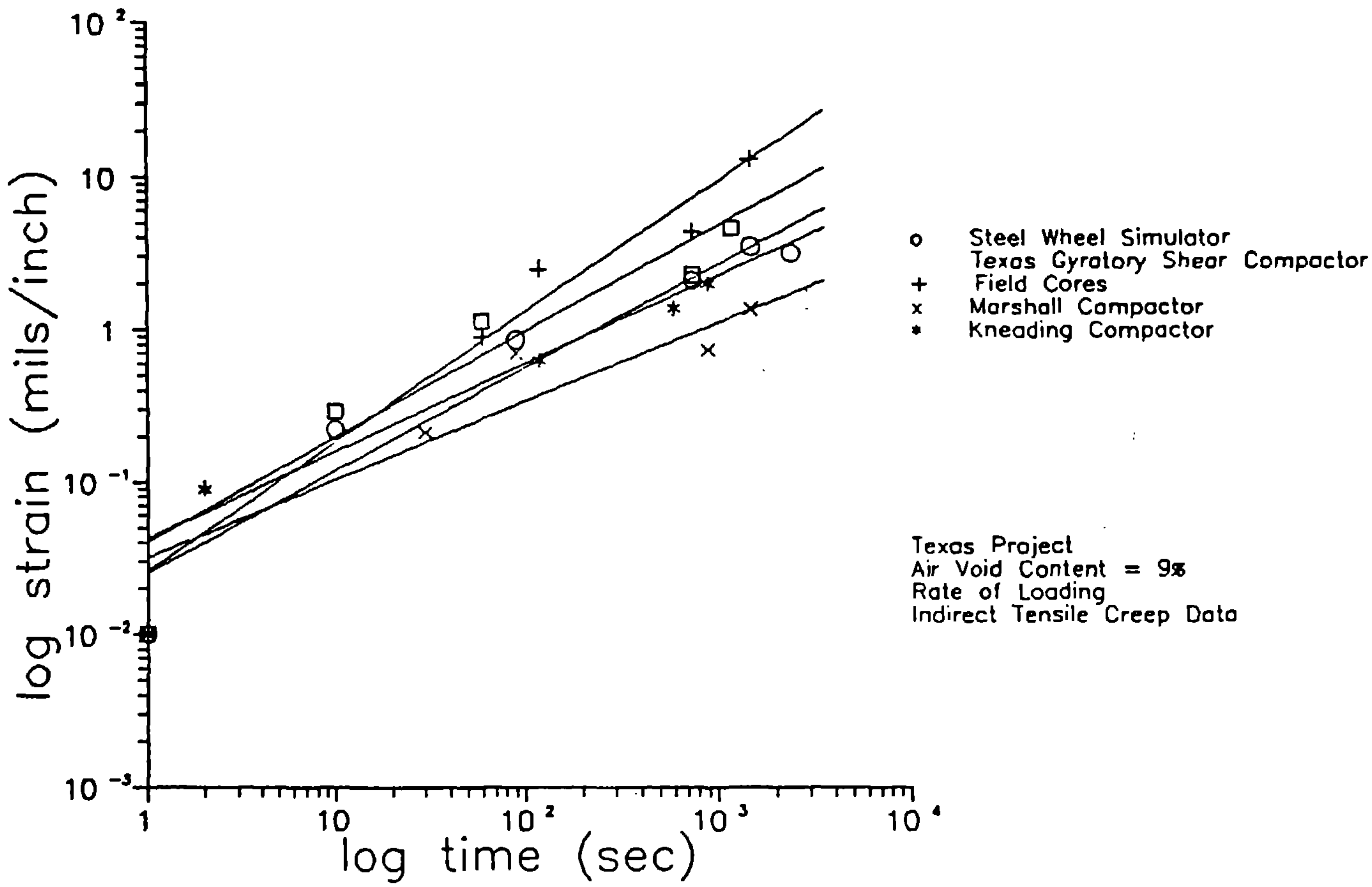

Figure 4. Comparative analysis of power curves for field cores and laboratory-compacted samples – Texas project.

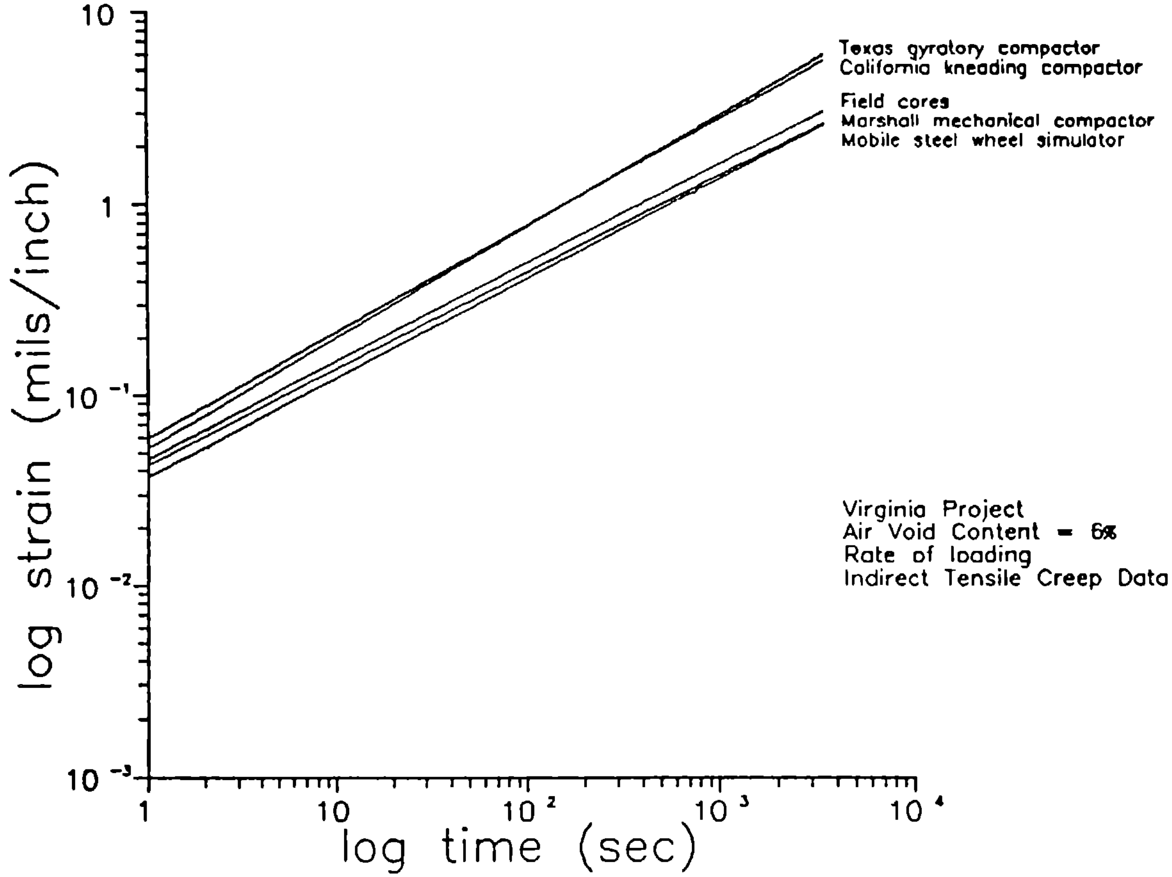

Figure 5. Comparative analysis of power curves for field cores and laboratory-compacted samples — Virginia project.

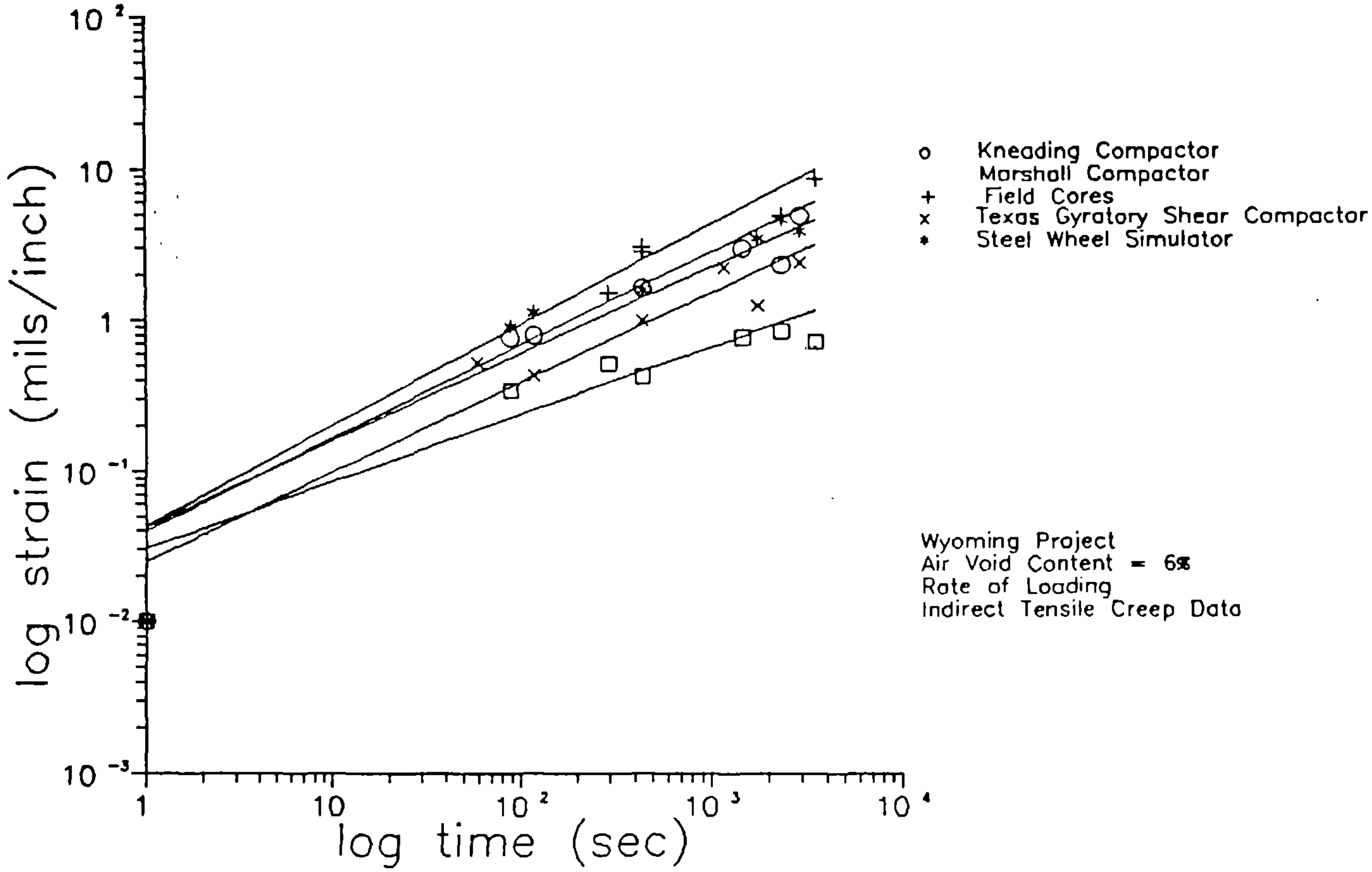

Figure 6. Comparative analysis of power curves for field cores and laboratory-compacted samples – Wyoming Project.

RESILIENT MODULUS

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RESILIENT MODULUS

1.0 GENERAL

The design of flexible pavements has rapidly evolved from empirical and semiempirical procedures to design methods based on elastic and/or viscoelastic theories (1 - 7). Today, many highway agencies use such methods in one form or another for the design of new, reconstructed, and/or overlaid asphalt pavements. In addition, the 1986 AASHTO design guide replaces the soil support value (SSV) of the roadbed soil by its resilient modulus and it suggests the use of the resilient modulus of the other pavement layers to estimate the values of the layer coefficient. Hence, the use of mechanistic design procedures and/or the 1986 AASHTO procedure requires (to various degrees) a thorough knowledge of the basic structural properties (fatigue life, resilient characteristics, and plastic deformation) of the pavement layers (8 - 13).

Several tests and test equipment have been developed and employed in various laboratories to evaluate the structural properties of the pavement layers (14 - 27). It was found that (regardless of the complexity of the tests, test procedures, and test equipment) different tests yielded different results and that the test results are difficult to reproduce (12). Further, existing asphalt concrete mix design procedures are based on parameters that do not necessarily have any relationship to the structural design of asphalt pavements and their fatigue lives (12, 15).

Structural properties of the pavement materials have a direct bearing on pavement performance under traffic loading and environmental conditions (25 - 33). The pavement surface response to a wheel load (whether the wheel is moving or stationary) is deflection (deformation) which is the sum of the deflections of each individual pavement layer. For a moving wheel load, the magnitudes of these deflections are different from those of static loads (parked vehicles or a wheel load moving at a creep speed) and they are functions of the magnitude and speed of the wheel load, the axle type and configuration, the types of the materials in the pavement and their mechanical and physical properties, and the pavement temperature.

Unlike static loads, a moving wheel imparts a dynamic load pulse to all pavement layers including the roadbed soils as shown in figure 1. For each moving wheel load, the magnitude of the dynamic load pulse (at a point on or under the pavement surface) increases from zero, when the wheel load is far from that point, to a peak value, when the wheel load is almost over that point. It decreases back to zero as the wheel load departs. The time interval between the two zero values is related to the speed of the vehicle. After the departure of the first wheel load, the point under investigation will experience a relaxation period (a period of zero dynamic load) before a second wheel load approaches that causes the dynamic load cycle to be repeated (32, 33).

The above scenario implies that, in the laboratory, the proper test procedure for material evaluation should be capable of simulating the dynamic load pulse imparted to the pavement by a moving wheel load. This test procedure is typically referred to as cyclic load test, repeated load test, or

2. Since the resilient modulus test is not a strength test, the peak magnitude of the applied cyclic load is typically equal to a small fraction of the strength of the specimen. Hence, the test specimen is not loaded to failure.

3. During the test, specimen deformations are continuously monitored (as shown in figure 2) during the load-unload cycle and during the relaxation period.

For cohesionless base and subbase materials as well as for cohesive roadbed soils, the test is conducted on cylindrical specimens confined in a triaxial cell as shown in figure 3. The system allows varying confining pressures to be applied to the specimens to model the in situ lateral stresses. A suitable loading system is used to apply a repeated load pulse of a fixed magnitude and fixed time duration to the cylindrical specimen. During the test, the confining pressure can be monitored using a pressure gauge; the cyclic load is generally recorded using an electronic load cell; and the specimen deformation is continuously recorded for analysis (as shown in figure 2) using linear variable differential transducers (LVDT). Nevertheless, in order to obtain accurate and representative data, the following conditions must be satisfied:

1. The ratio of specimen length to diameter must be at least two.

2. The triaxial test results depend upon the friction between the upper and lower loading platens and the upper and lower ends of the specimen. High friction causes restraints at the specimen ends and, hence, a rotation of the principle planes (the planes along which the shear stresses are zero).

3. The minimum specimen diameter to the maximum aggregate (particle) size ratio should be 4.

3.1 Data Analysis

Although most resilient modulus tests are conducted using a sinusoidal wave form, the test can also be conducted using other wave forms such as square and impulse. In addition, the cyclic load can be applied with or without a relaxation period. Since, as of this time, no standard definition of the resilient modulus has been established, these variations in the test mode make the task of comparing and/or sharing the test data between different laboratories and/or researchers extremely difficult. These difficulties stem from the way the resilient modulus is calculated and consequently reported in the literature. For example, in reference to figure 2, the total deformation of the specimen (D_T) is the sum of three deformation components: elastic (D_E), viscoelastic (D_V), and plastic deformation (D_P). The former two components are recoverable while the last one is irrecoverable. Each of these components can be divided by the original length of the specimen to obtain the respective strain (equations 1 through 3). The applied cyclic load, on the other hand, is divided by the cross-sectional area of the specimen to calculate the deviatoric cyclic stress (the difference between the major and minor principle stresses) using equation 4. It should be noted that the minor

example, figure 2 shows that the peak specimen deformation lags behind the peak cyclic load by a time period (Δ). This is typically called the phase angle, and it is a measure of the energy dissipation characteristics of the material.

3.2 Models for Resilient Modulus of Cohesive Soils

The basic model describing the stress dependency of the resilient modulus of fine grained cohesive soil is given in the following equation (28, 32, 33)

$$M_R = K1*(s_d)^{K2} \tag{5}$$

where M_R = resilient modulus (psi);
s_d = deviator stress, psi;
K1, K2 = material constants.

It should be noted that the values of K1 and K2 are dependent on the physical and mechanical characteristics of the fine soil such as density, water content, plasticity, and compressibility.

3.3 Models for Resilient Modulus of Cohesionless Soils

Two basic models were developed to express the resilient modulus of coarse-grained soil in terms of the applied stresses. The first expresses the resilient modulus as a function of the applied confining pressure. The second in terms of the bulk stresses (the sum of the principle stresses). The second model tends to be more accurate and is expressed in the following equation

$$M_R = K_3(\theta)K_4 \tag{6}$$

where θ = the sum of the principal stresses ($s_1 = 2s_2$) with s_1 = major principle stress, and s_2 = minor principle stress (the confining pressure; and
K3, K4 = material constants.

Comparing equations 5 and 6, it can be seen that the stress sensitivity of coarse-grained non-cohesive soils is opposite to that exhibited by the fine-grained roadbed materials. that is the The resilient modulus of cohesionless soils increases as the sum of the principle stresses increases.

Typical values of K3 and K4 for various coarse-grained soils are presented in table 1.

4.0. RESILIENT MODULUS OF COMPACTED ASPHALT MIXES

The resilient modulus of compacted asphalt mixes can be obtained using either repeated load triaxial test or repeated load indirect tensile test. The procedure for triaxial test is similar to that for cohesive and cohesionless soils. The indirect tensile test and the repeated load indirect tensile test are described below.

performance in cold climates. Mixes that are brittle at cold temperatures will fail with very low tensile strains. Typical stress and strain values at low temperatures are shown in figure 5 for several asphalt concrete specimens illustrating different low-temperature behavior (32, 33, 35, 36).

4.2 The Repeated Load Indirect Tensile Test (ASTM D4123)

The test equipment for determining the resilient modulus using the repeated load indirect tensile test includes an indirect tensile test apparatus, a repetitive loading mechanism (typically a closed loop hydraulic system), a test chamber for maintaining constant temperature during the test, and a measurement system (e.g., load cell and deformation sensors). The test is conducted on disc-shaped specimens (typically Marshall-size specimens) placed on its side on a curved loading strip (the curvature of the loading strip is the same as that of the specimen). The cyclic load is applied along the vertical diameter of the specimen and the resulting deformations along the vertical and horizontal diameters are measured. The ASTM D4123 standard permits measurement of the deformation along the horizontal diameter only. For this case, a value of Poisson's ratio has to be assumed prior to the analysis of the data (31, 32, 33).

In a recent development, it was found that existing indirect tensile test apparatus produces inconsistent results due to several problems including rocking motion of the loading head during the test, which distorted the measurement of the specimen deformations. Figure 6 shows typical variations in the values of the resilient modulus obtained using existing apparatus. Further, indirect tensile test apparatus that do not allow measurement of the deformation along at least two directions (i.e., along the vertical and horizontal diameters), which necessitate the assumption of the value of Poisson's ratio as stated in the ASTM D4123, produced misleading and inconsistent results as shown in figure 7 (32). Using the ASTM standard, the resilient modulus can be calculated using the following equation:

$$M_R \text{ or } E = (P)(U + 0.2734)/(D_H)(L) \qquad (10)$$

where
- P = the magnitude of the cyclic load (pounds);
- V = an assumed value of Poisson's ratio;
- D_H = deformation along the horizontal diameter (inch);
- L = specimen thickness (inch).

It should be noted that if the values of U and D_H of the equation represent the instantaneous Poisson's ratio and the instantaneous recoverable deformation, then the value of M_R represents the instantaneous resilient modulus. On the other hand, if the values of U and D_H of the equation represent the total Poisson's ratio and the total recoverable deformation then the value of E represents the total resilient modulus.

The ASTM recommends a value of Poisson's ratio of 0.35. A magnitude of cyclic load of 40, 50 and 60 pounds with a load duration of 0.1 second. The test is generally conducted at three temperatures, 41, 77 and 104 degrees F to generate design values over the range of temperatures normally encountered in the field.

Theoretically (for homogeneous, isotropic, and linear materials) the values of the resilient modulus calculated using the vertical deformation and Poisson's ratio (second equation) should be exactly the same as those calculated using the deformation along the thickness of the specimen and Poisson's ratio (third equation), (28 - 31, 39). Asphalt mixes are heterogeneous and anisotropic Due to this and measurement errors, differences between the two calculated values should be expected. Nevertheless, the second equation can be utilized if the deformation along the thickness of the specimen is not measured. If deformations in all three directions are measured, then the values of the calculated resilient modulus from both equations should be compatible (a maximum difference of 5 percent). A better procedure, however, is to calculate the values of the resilient modulus and Poisson's ratio using the measured deformations in all three directions. This procedure is presented in the following equations (31):

$$U = \{(0.225127)(H^2) - (0.269895)(V^2) - (0.0447676)(A^2) + (3.570975)(H)(V) + (0.086136)(A)(H) + (1.145064)(A)(V)\}/D \quad (14)$$

$$M_R \text{ or } E = \{(0.253680)(H) + 3.9702876)(V) - (0.0142874)(A)/D \quad (15)$$

where $D = (1.105791)(H^2 + V^2 + A^2) - [H - (0.0627461)(V) + (0.319145)(A)]^2$;
$H = (D_H)(L)/P$;
$V = (D_V)(L)/P$;
$A = D_L/P$; and
M_R, E, D_V, D_H, D_L, L, and P are as before.

Based on tests conducted using the new indirect tensile test apparatus where the specimen deformations were measured in all three directions, and on the test results, the following equations were developed to respectively estimate the values of the instantaneous and total resilient modulus of asphalt mixes (28, 31):

$$\ln(M_R) = 16.092 - (0.03658)(T) - (0.1401)(AV) - (0.0003409)(CL) + (0.04353)(ANG) + (0.0008793)(KV) \quad (16)$$

$$\ln(E) = 16.385 - (0.04529)(T) - (0.1549)(AV) - (0.0003339)(CL) + (0.04258)(ANG) + (0.0008364)(KV) \quad (17)$$

where:
- ln = natural logarithmic operator;
- M_R = instantaneous resilient modulus (psi);
- E = total resilient modulus (psi);
- T = temperature (°F);
- AV = the percent air voids in the asphalt mix (AV = 1, 2,..);
- CL = magnitude of the applied cyclic load (pounds);
- ANG = aggregate angularity (ANG = 4 for crushed aggregate, = 2 for rounded river deposited aggregate; and
- KV = kinematic viscosity of the asphalt at 275 °F (AASHTO T 201), (centistokes).

Figure 13 depicts the measured and estimated values of the resilient modulus using the last equation (31).

5.3 Viscoelastic Properties

If the resilient modulus test is conducted properly (i.e., using loading and relaxation periods), the test results will yield information relative to the elastic as well as the viscoelastic properties of the test materials.

5.4 Permanent Deformation Characteristics

The resilient modulus test procedure also lends itself to the analysis of permanent deformation. Continual monitoring of the resilient modulus test over an extended period of time provides indications of the permanent deformation potential of the roadbed soil. One way to accomplish this (using a strip chart recorder) is to greatly decrease the speed of the strip chart for the period of interest, while allowing the pen of the recorder to continuously record the deformation traces. A schematic example of a deformation trace produced by this technique is presented in Figure 14. The permanent deformation during the time of interest is designated by D_p in the figure.

6.0 ADVANTAGES OF THE CYCLIC LOAD INDIRECT TENSILE TEST

In addition to the above noted advantages of the resilient modulus test relative to the static and strength tests, the cyclic load indirect tensile test will yield additional information that cannot be obtained from the cyclic load triaxial test. Recall that the test specimen in the cyclic load indirect tensile test is subjected to tension along the vertical diameter of the specimen. This will result in a plastic tensile strain. The magnitude of this strain will increase as the number of load application increases. When the plastic strain along the vertical diameter reaches a certain value, a crack along the vertical diameter will develop indicating that the specimen has exhausted its fatigue life. Hence, fatigue curves in terms of the applied cyclic stress and the tensile plastic strain can be obtained. Such fatigue curves are shown in figure 15. These curves can be employed to investigate the effects of the different asphalt mix constituents (aggregate type and gradation, asphalt type and content, test temperature, and the percent air voids) upon the fatigue life of the mix. These curves and the corresponding fatigue life equations can also be used (after a careful calibration using field data) to predict the fatigue life of asphalt pavements.

7.0 ROLE OF THE RESILIENT MODULUS IN PAVEMENT DESIGN

The resilient modulus of the roadbed soil, base, subbase, and/or asphalt mixes play a major role in both mechanistic and the 1986 AASHTO design procedures. For example, the resilient modulus of the roadbed soil is a direct input for most mechanistic pavement design programs that use elastic layer theory. It is also a direct input to the 1986 AASHTO Design Guide where it replaces the old Soil Support Value (S). In the AASHTO design guide, the resilient modulus of each pavement layer material (including the roadbed soil) exerts an influence on the structural coefficient of that layer and an extremely strong influence on the thickness and/or the structural requirements of the overlying layer. In the mechanistic procedures, it exerts strong influence on the overall pavement performance.

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Table 1. Summary of K_1 and K_2 statistics for various soils and aggregates.

Soil/Aggregate type	Number of data points	K_3			K_4		
		Mean	Std. dev.	Range	Mean	Std. dev.	Range
Silty sands	8	1620	780	710- 3830	0.62	0.13	0.36-0.80
Sand gravel	37	4480	4300	860-12840	0.53	0.17	0.24-0.80
Sand/aggregate blends	78	4350	2630	1880-11070	0.59	0.13	0.23-0.82
Partially crushed gravel	8	5967	2800	2156-4,119	0.52	0.12	0.40-0.75
Crushed stone	115	7210	7490	1705-56670	0.45	0.23	-(0.16)-0.86
Limerock	13	14030	10240	5700-83860	0.40	0.11	0.00-0.52
Slag	20	24250	19910	9300-92360	0.37	0.13	0.00-0.52
All data	271	7391	8250	710-92360	0.52	0.16	-(0.16)-0.86

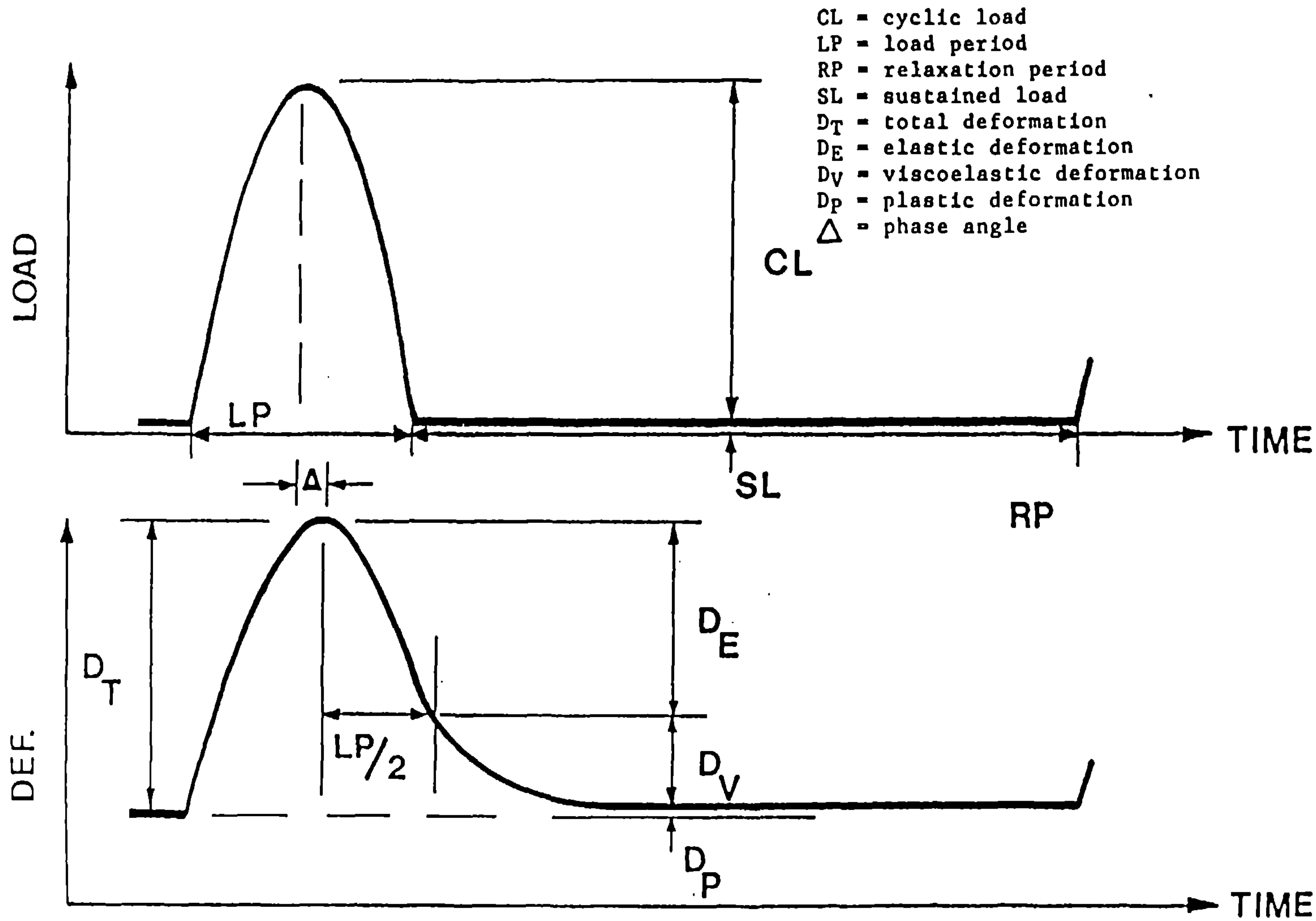

Figure 2. Typical l load and deformation cycles with 0.1 second loading time and 0.4 second relaxation period.

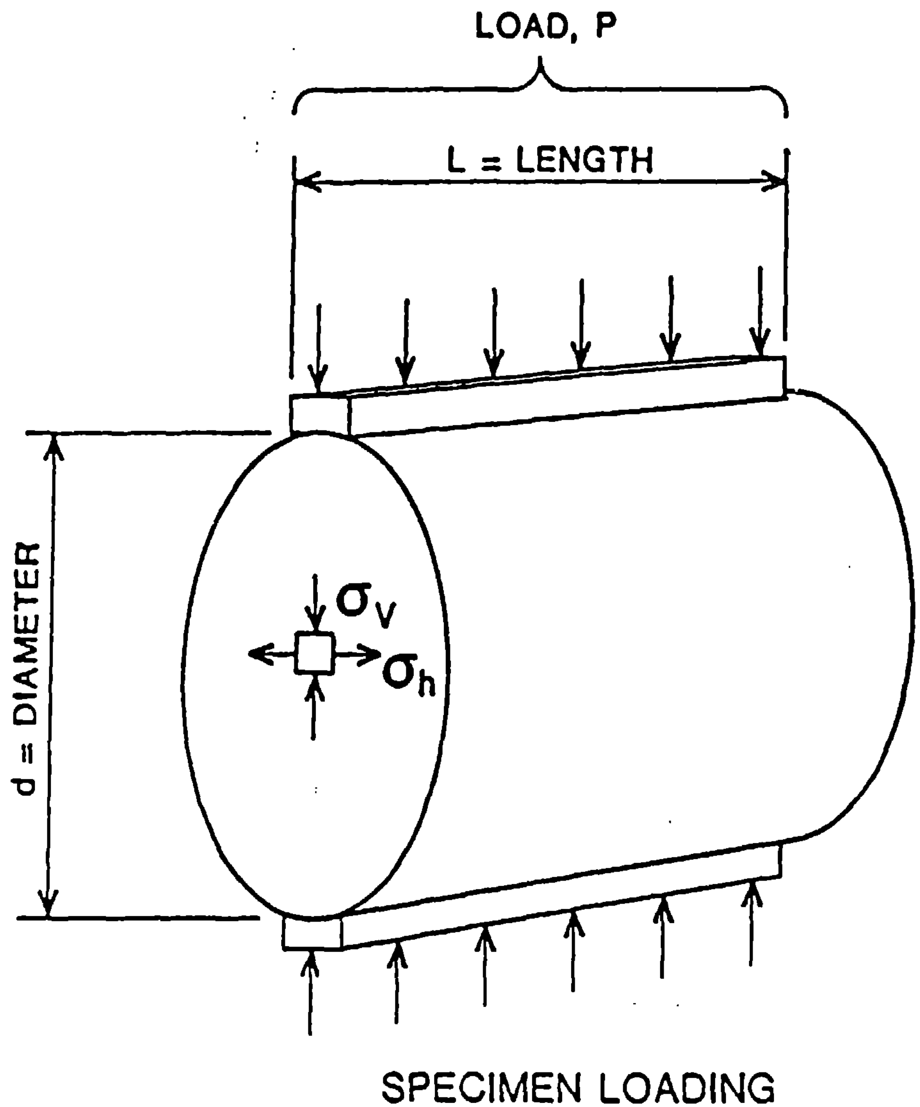

Figure 4. Schematic respresentation of the compressive and tensile stress, indirect tensile test.

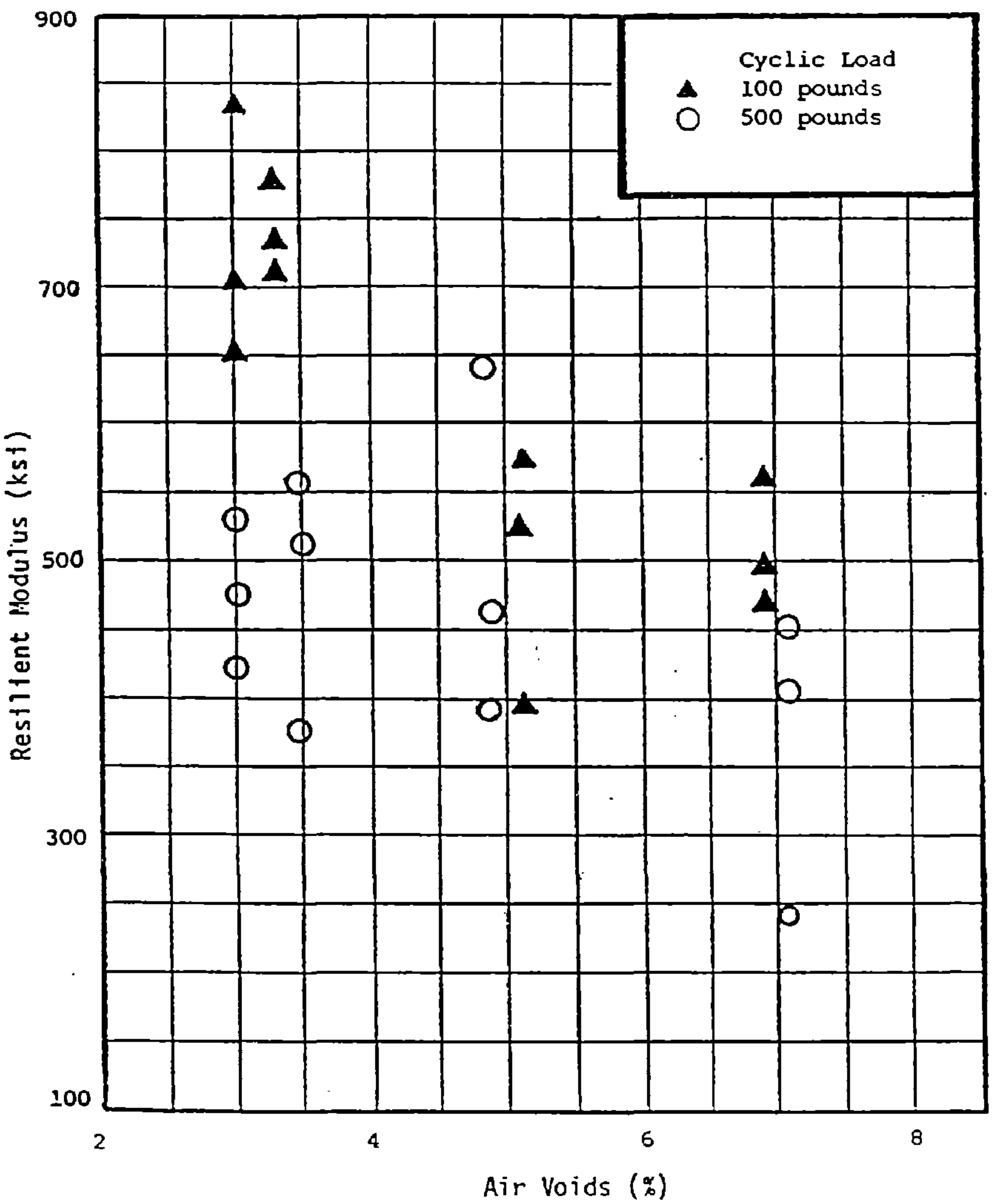

Figure 6. Resilient modulus versus the percent air voids for asphalt mix specimens tested using existing indirect tensile test apparatus.

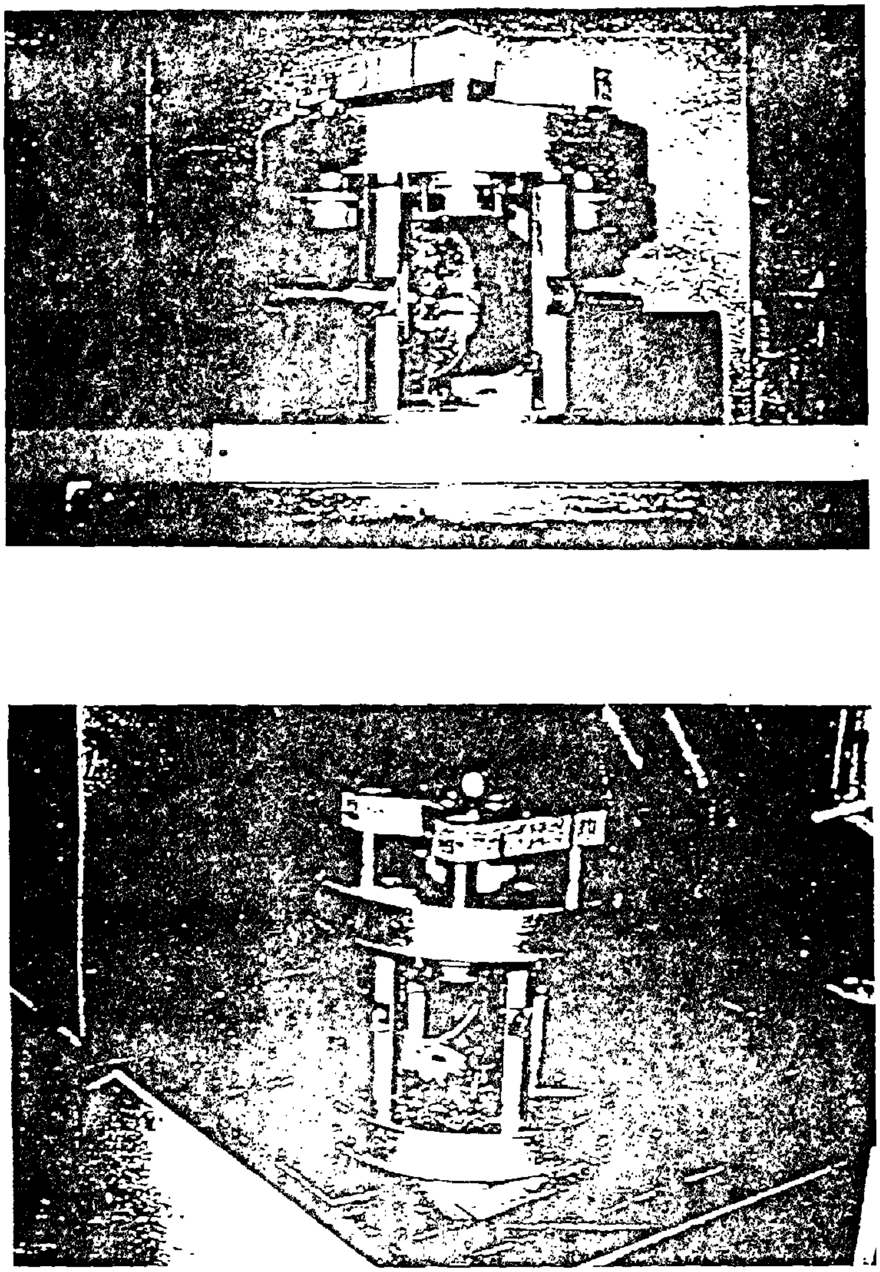

Figure 8. General views of the new indirect tensile test device.

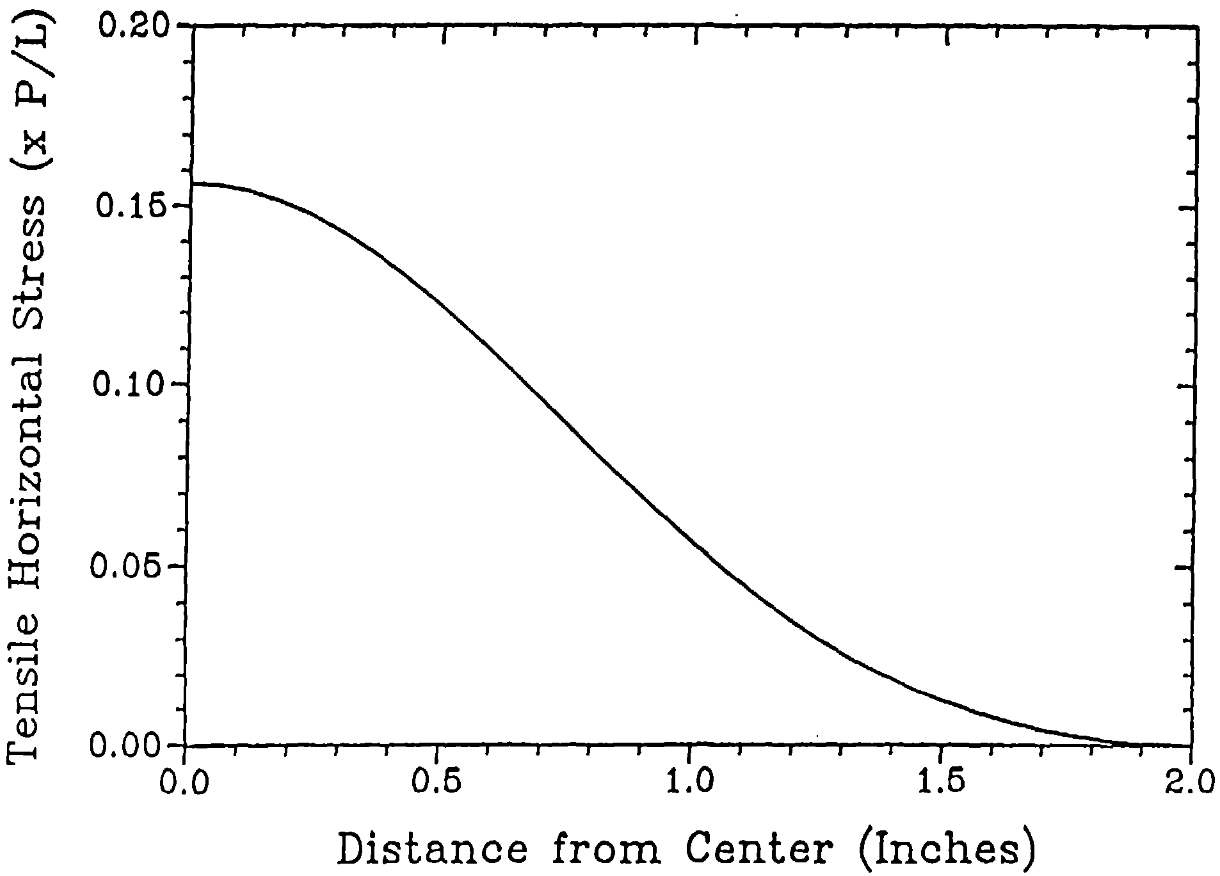

Figure 10. Variations of horizontal stresses along the horizontal diameter of an indirect tensile test specimen.

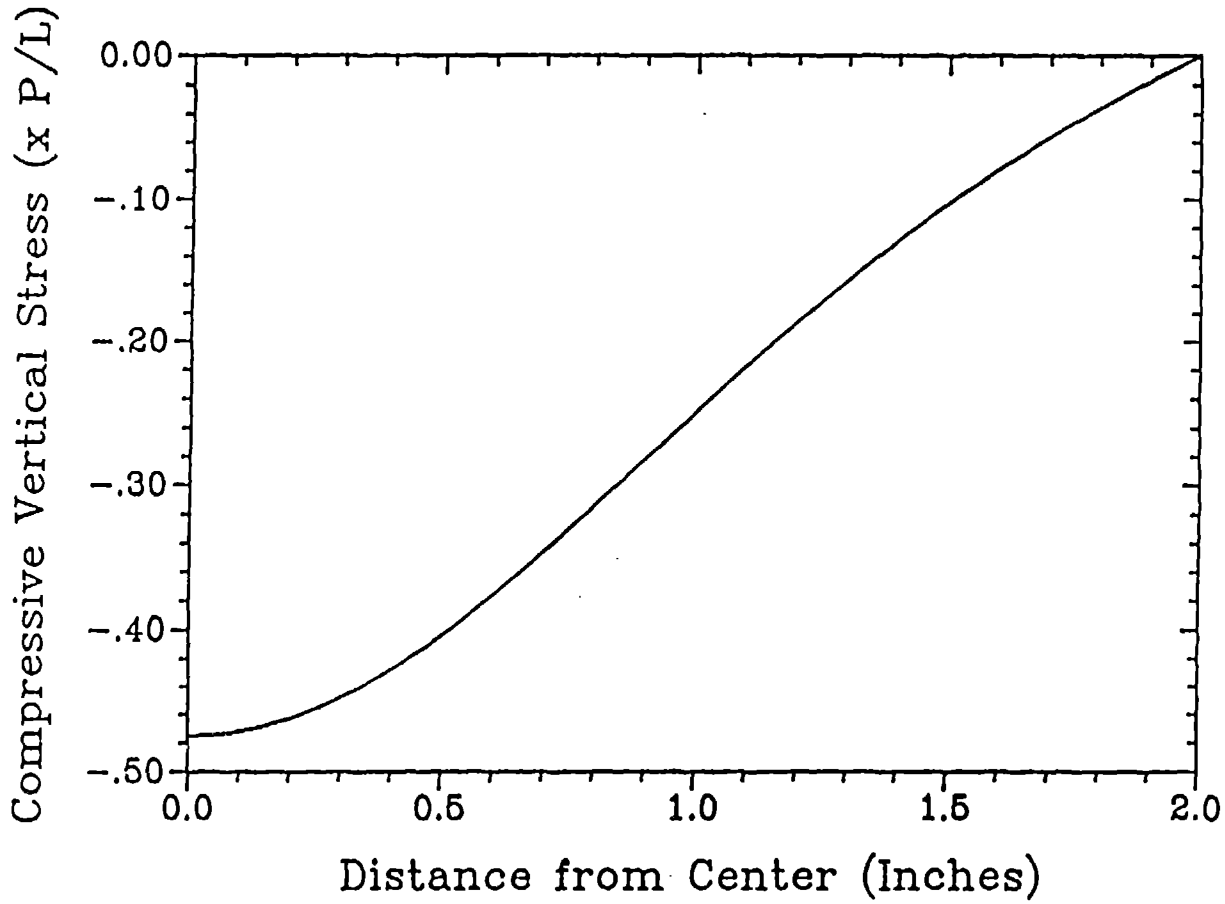

Figure 12. Variations of vertical stresses along the horizontal diameter of an indirect tensile test specimen.

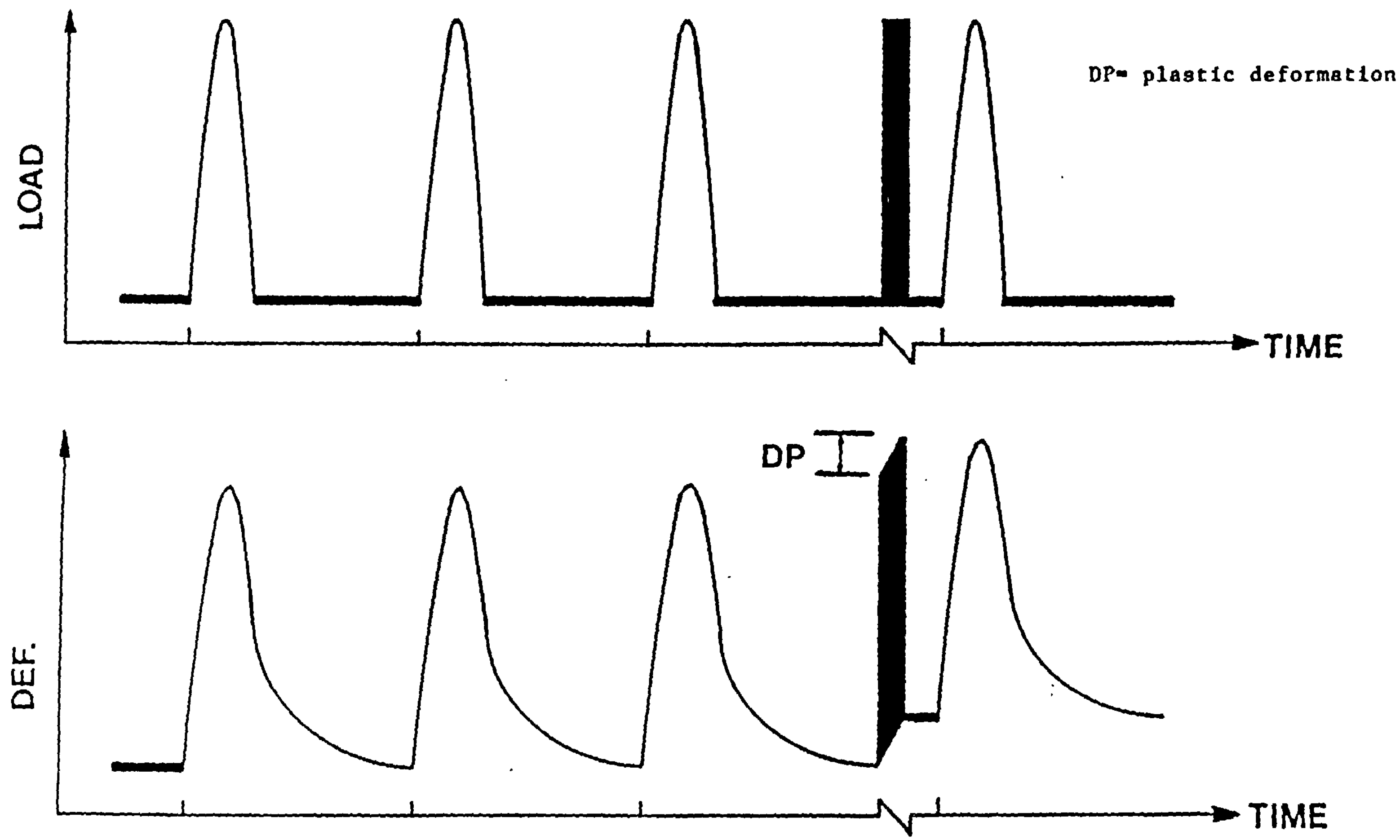

Figure 14. Typical load deformation cycles with 0.1 second loading time and 0.4 second relaxation period.

NO. OF LOAD CYCLES	250	500
AVG. CYCLE LOAD	503.6932	509.9026
RESILIENT MODULUS	360735.9	351388.9
TOTAL MODULUS	238634.7	329289.0
RESILIENT POISSON'S RATIO	0.161399	0.112115
TOTAL POISSON'S RATIO	0.530121	0.258199

VERTICAL DEFORMATION

TIME LAG	3	-1
RESILIENT	0.001924	0.001995
VISCOELASTIC	0.000000	0.000041
PLASTIC	0.000176	0.000318
TOTAL	0.001924	0.002036

HORIZONTAL DEFORMATION

TIME LAG	26	24
RESILIENT	0.000139	0.000137
VISCOELASTIC	0.000024	0.000019
PLASTIC	0.000051	0.000095
TOTAL	0.000164	0.000156

RADIAL DEFORMATION

TIME LAG	22	3
RESILIENT	0.000826	0.000663
VISCOELASTIC	0.002992	0.000694
PLASTIC	0.000082	0.000157
TOTAL	0.003818	0.001357

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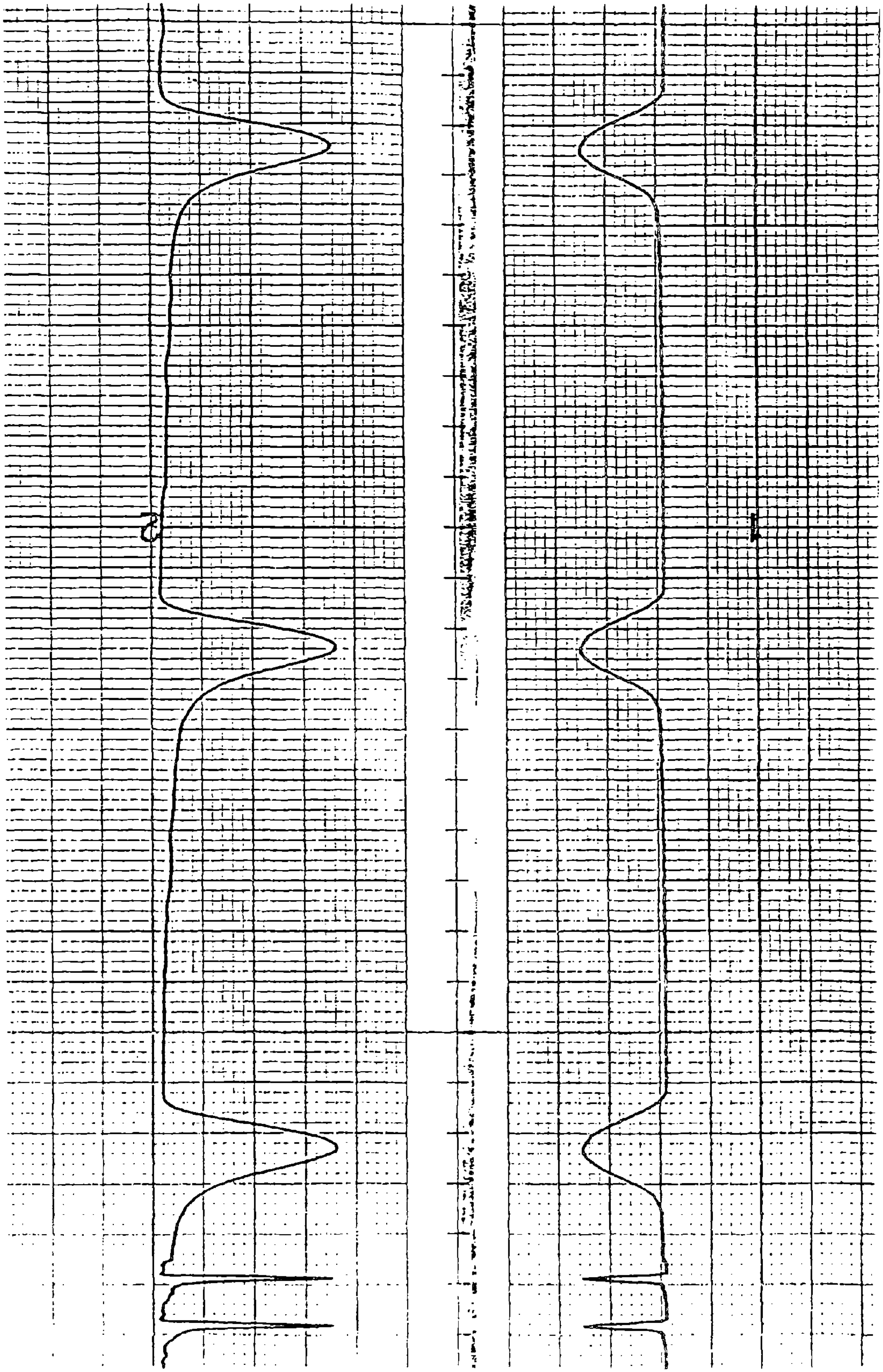


AFTER SPECIMEN ROTATION

NO. OF LOAD CYCLES	250	500
AVG. CYCLE LOAD	508.1169	508.0763
RESILIENT MODULUS	348765.8	366355.6
TOTAL MODULUS	243137.3	230637.1
RESILIENT POISSON'S RATIO	0.143034	0.170355
TOTAL POISSON'S RATIO	0.775383	0.683670

VERTICAL DEFORMATION

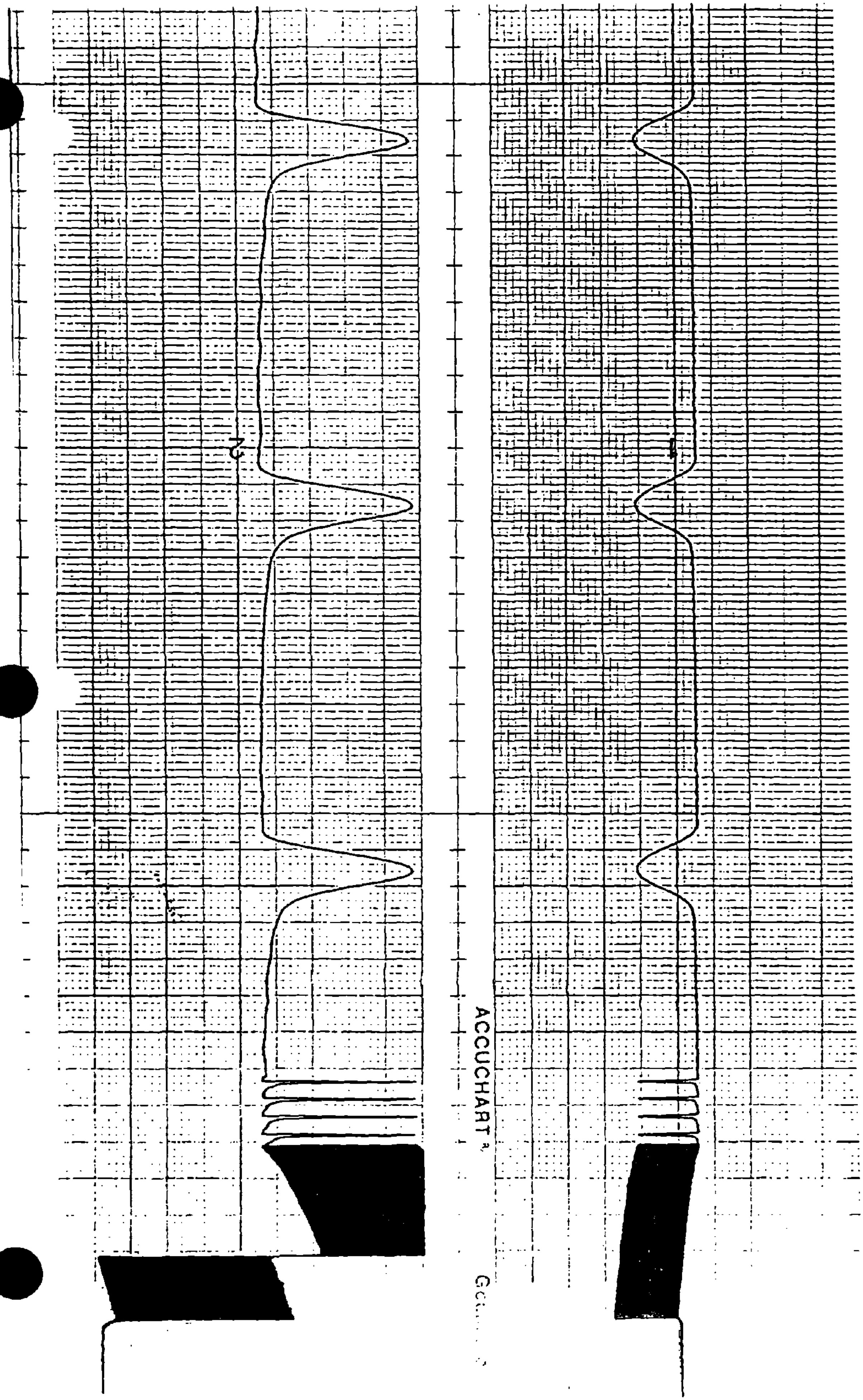
TIME LAG	-2	-3
RESILIENT	0.002015	0.001908
VISCOELASTIC	0.000020	0.000183
PLASTIC	0.000761	0.001198
TOTAL	0.002035	0.002091

HORIZONTAL DEFORMATION

TIME LAG	19	20
RESILIENT	0.000138	0.000137
VISCOELASTIC	0.000250	0.000182
PLASTIC	0.000127	0.000158
TOTAL	0.000389	0.000319

RADIAL DEFORMATION

TIME LAG	18	21
RESILIENT	0.000831	0.000874
VISCOELASTIC	0.002998	0.003156
PLASTIC	0.000341	0.000489
TOTAL	0.003829	0.004029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STRAIN AND TEMPERATURE EFFECTS ON RESILIENT MODULI

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STRAIN AND TEMPERATURE DEPENDENCY OF THE IRM_r
by John Heinicke and Ted S. Vinson

Introduction

A laboratory test program was conducted to: 1) develop relationships between the resilient modulus and the tensile strain level and temperature, and 2) establish the significance of these relationships on the determination of the IRM_r[1]. As previously noted, the current test procedure (ASTM D 4123) specifies a temperature tolerance of ±1.8°F (±1°C), and there is no requirement to perform all tests at a specific level of tensile strain.

The asphalt concrete specimens used in the test program were fabricated by the Materials Division of the Oregon Department of Transportation (ODOT). Six 4-inch diameter by 2.5-inch long specimens were prepared using the ODOT Class "C" mix design for heavy traffic (Sullivan et.al., 1986). The specimens were compacted with a Hveem kneading compactor, and air voids contents of approximately 4.6% and 6.8% (three specimens, each) were obtained by varying the the number of blows at the 500 psi level from 150 to 50 blows, respectively.

The test procedure involved the following steps:

1) The bulk specific gravity of each specimen was determined to obtain the air voids contents; the following groups were identified:

   Group 1 ==> 4.6% ±0.4% air voids content

   Group 2 ==> 6.8% ±0.2% air voids content

2) The specimens were allowed to cure for two days.
3) The specimens were placed inside an environmental cabinet

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[1] IRM_r = Index of Retained Resilient Modulus

$$= \frac{\text{MR of conditioned Specimen}}{\text{MR of Dry (control) Specimen}}$$

and the temperature was stabilized at the lowest test temperature [36°F (2.1°C)].

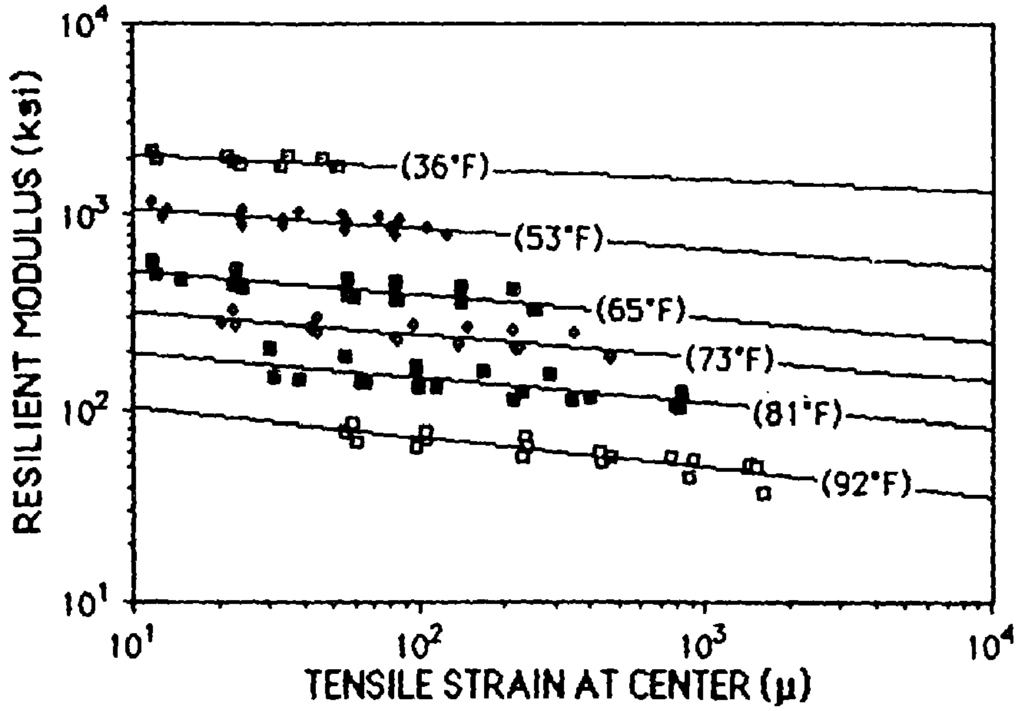
4) The resilient modulus test was performed along a randomly selected axis; each test was conducted using a load frequency of 1 Hz and a load duration of 0.1 second; the test temperature was maintained within ±0.2°F (±0.1°C) of the target temperature and monitored using thermistors attached to the sides of the specimens; the load was continually increased, allowing tensile strain and modulus values to be measured at six levels.
5) The specimen was rotated 90° and the procedure was repeated.
6) After testing each specimen, the temperature was increased to the next level and allowed to stabilize overnight.

Steps 3 through 5 were performed using the following temperature levels: 36°F, 53°F, 65°F, 73°F, 81°F, and 92°F (2.1°C, 11.7°C, 18.8°C, 22.8°C, 27.2°C, and 33.1°C). Approximately 300 load repetitions were applied at each level.

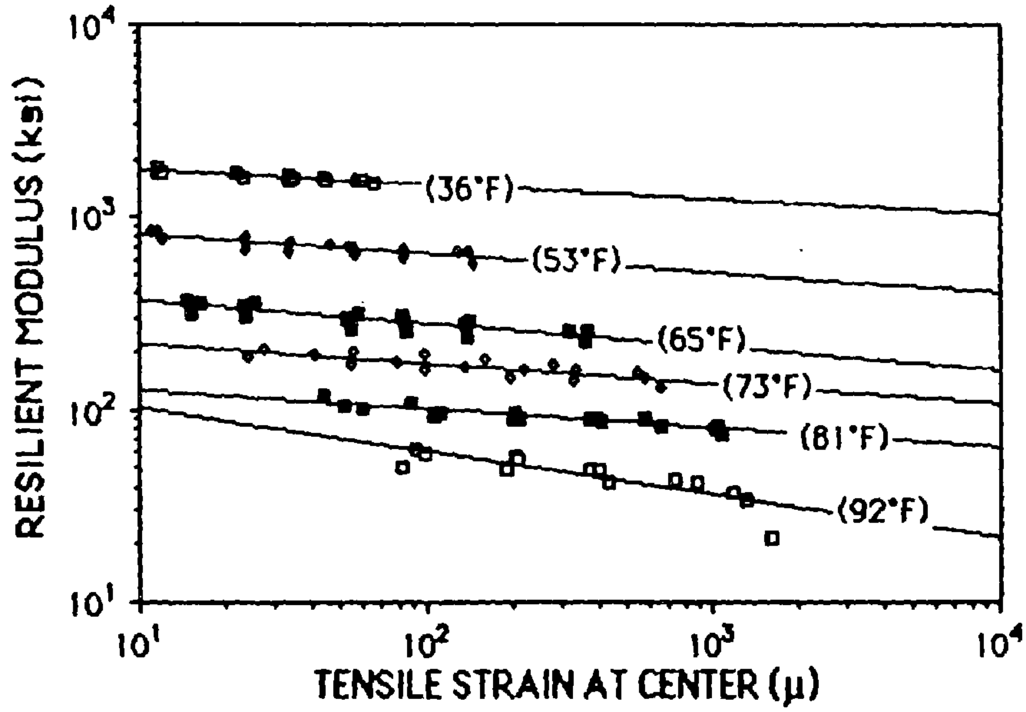
Test Results

The results of the test program are plotted in Figure 1. Figure 1a presents the log tensile strain vs. log resilient modulus relationship for all tests performed on Group 1. The data for the three specimens are combined corresponding to the six test temperatures. Similarly, Figure 1b shows this relationship for Group 2.

The regression line equations for each set of data take the general form:



(a) 4.6% Air Voids Content



(b) 6.8% Air Voids Content

FIGURE 1. Resilient Moduli vs. Tensile Strain for the Temperature Range of 36°F to 92°F.

$$M_r = K_1 \, \varepsilon_t^n \qquad (1)$$

where K_1 = a constant evaluated at $\varepsilon_t = 1\mu$ (ksi)

n = a constant representing the slope of the log-log regression line

The values of K_1, n, and R^2 for the regression lines are given in Table 1

Discussion of Results

As noted previously, the tensile strain vs. resilient modulus data points were obtained at six strain levels. However, fewer data points were obtained at the temperature extremes, namely 36°F and 92°F. At 36°F, the specimens were so stiff that the load limit of the test equipment (i.e., a 1000 lb. load cell) was reached at approximately 50µ to 80µ. At 92°F, permanent deformations were visible at strain levels greater than 1500µ, and one of the Group 2 specimens displayed slight cracking at 1600µ.

The coefficients of determination (R^2 in Table 1) range from 0.59 to 0.91 for all data. However, if separate regression lines are calculated for each specimen, the R^2 values range from 0.91 to 1.00, with the majority above 0.95. Further, the correlation is better for the specimens of Group 2, which have less variation of air voids contents. Therefore, most of the scatter at each temperature level can be attributed to the variation of the air void contents in each group.

The data display two characteristic trends:

1) The slopes of the regression lines, with minor exceptions, become steeper as the temperature increases. This may simply indicate that the relationship between the tensile strain and the resilient

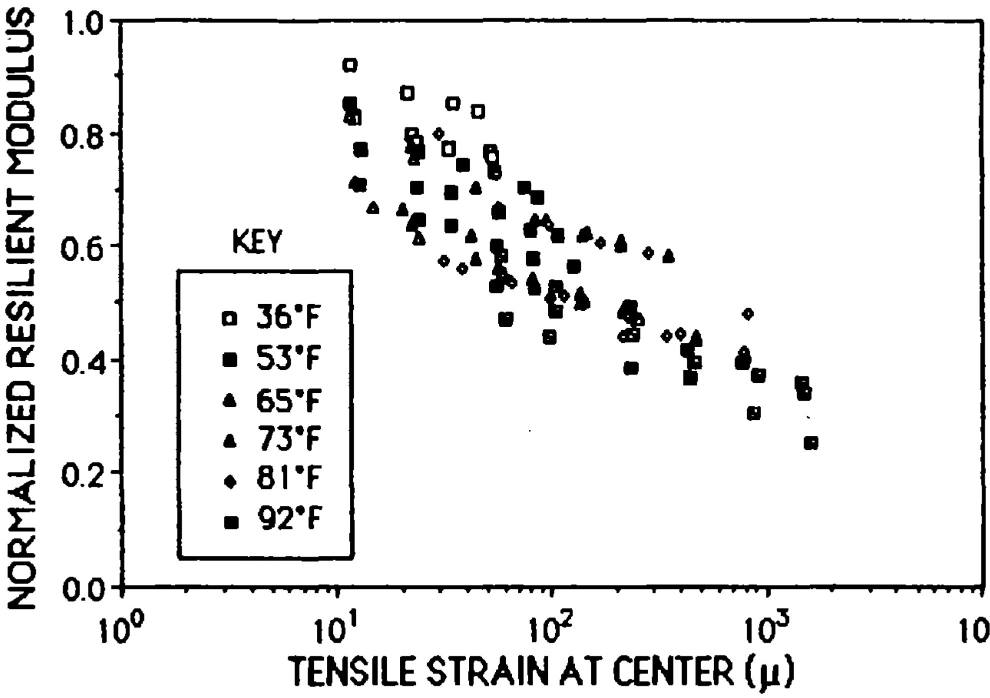
TABLE 1. Regression Constants for Log Tensile Strain vs. Log Resilient Modulus Relationships.

(1) GROUP NUMBER	(2) TEMPERATURE (°F)	(3) K (ksi)	(4) n	(5) R^2
1 (4.6% air voids)	36	2369	-0.063	0.59
	53	1384	-0.104	0.73
	65	700	-0.126	0.81
	73	424	-0.122	0.77
	81	258	-0.128	0.75
	92	146	-0.155	0.80
2 (6.8% air voids)	36	2075	-0.074	0.90
	53	1038	-0.103	0.83
	65	475	-0.116	0.87
	73	276	-0.100	0.83
	81	159	-0.099	0.91
	92	125	-0.166	0.89

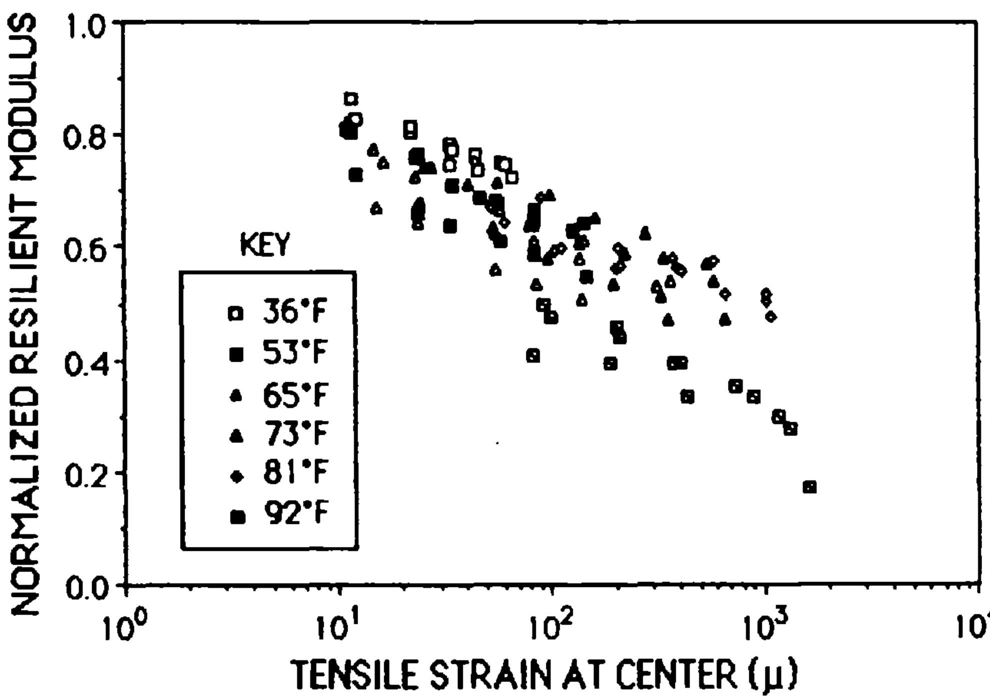
modulus is likewise dependent upon temperature. However, the strain levels increase with temperature, implying that the strain dependency of the resilient modulus is greater at higher strain levels.

2) The regression lines of Groups 1 and 2 are generally parallel, with Group 2 data displaced downward. Using the regression equations to obtain resilient moduli at the 100μ strain level, the Group 2 moduli are from 71 to 83% less than the Group 1 moduli. Thus, increasing the air voids content by approximately 2% results in a 75% reduction of resilient moduli.

The resilient modulus data can be normalized to illustrate the general trend of the tensile strain vs. resilient modulus. Normalizing the moduli at each temperature level also eliminates the effects of stress history from previously performed tests. The normalized moduli are obtained by dividing the resilient modulus at any given strain and temperature level by the corresponding resilient modulus evaluated at ε_t = 1μ (i.e., the constant K_1). The normalized resilient moduli vs. tensile strain are plotted in Figure 2. It may be noted that the results shown in Figure 2 resemble the relationship between normalized dynamic moduli and shear strain shown in Figure 2. Although Figure 2 represents only one particular asphalt concrete mixture at two air voids contents, a similar relationship may exist for all asphalt concrete mixtures. Such a characteristic relationship would permit the resilient modulus of any asphalt concrete mixture to be estimated at a standard tensile strain by determining the modulus at any other strain level. Figure 3 shows the



(a) 4.6% Air Voids



(b) 6.8% Air Voids

FIGURE 2. Normalized Resilient Moduli vs. Tensile Strain.

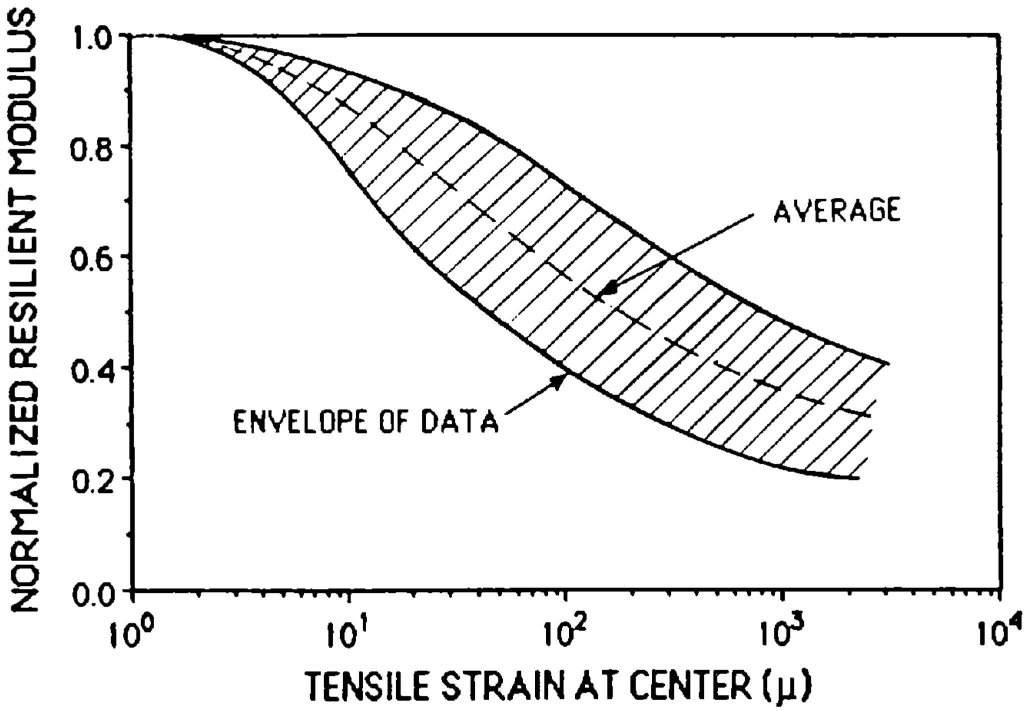


FIGURE 3. Envelope and Average Relationship for Normalized Resilient Moduli vs. Tensile Strain.

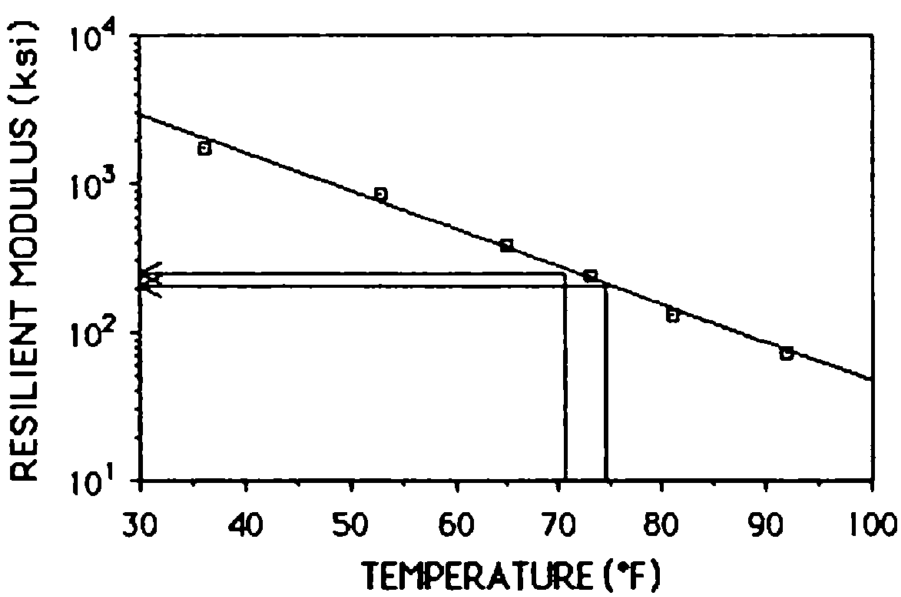


FIGURE 4. Resilient Modulus vs. Temperature at a Tensile Strain of 100 μ.

envelope and the average relationship between the normalized resilient modulus and the tensile strain for both air void contents.

Figure 3 can be used to illustrate the significance of the resilient modulus vs. strain relationship on the IRM_r through the following example. Consider two values of resilient moduli such that the resulting IRM_r = 0.70. Let

M_r(control) = 250 ksi, evaluated at $\varepsilon_t = 100\mu$;

M_r(conditioned) = 175 ksi; and

t = 2.5 inches.

Further, consider that the moduli are evaluated under constant load. Then,

H(control)[2] = $(100 \times 10^{-6}/0.52 = 0.192 \times 10^{-3}$ inch

Substituting H(control) into

P(control) = $(250{,}000)(0.192 \times 10^{-3})(2.5)/0.62$=194lbs.

P(control) represents the load that would also be used for the test on the conditioned specimen. The value of horizontal deflection for the conditioned specimen may be obtained from

H(conditioned) = $(0.62)(194)/(175{,}000)(2.5) = 0.275 \times 10^{-3}$ inch

The corresponding strain level may be computed from

ε_t(conditioned) = $(0.52)(0.275 \times 10^{-3}) = 143\mu$

Entering Figure 3 at the two tensile strain levels identified above, the appropriate normalized resilient modulus ratios (use average value represented by the dashed line) may be obtained,

for ε_t (control) = 100μ ==> 0..57

2 The equations suppporting this calculation are given in the paper by T. Vinson in the Proceedings of the Workshop on Resilient Modulus Testing, Oregon State University, March 1989.

for ε_t (conditioned) = 143μ ==> 0..52

The resulting IRM_r is thus determined as,

$$IRM_r = (175/250)\ (0.57/0.52) = 0.77 \quad (\neq 0.70\ !)$$

Note that this does not represent the worst case. For the nominal value of IRM_r = 0.70 determined with the control specimen at ε_t = 50μ and the conditioned specimen at 150μ, the actual value of IRM_r = (0.70)(0.65/0.50) = 0.91. Clearly, the failure to perform both resilient modulus tests at the same tensile strain level results in the misinterpretation of the resulting IRM_r.

The significance of the resilient modulus vs. temperature relationship on the IRM_r can be demonstrated by plotting the resilient modulus vs. temperature at a specific value of tensile strain. Such a plot for ε_t = 100μ is shown in Figure B.4. The allowable temperature extremes for the commonly used test temperature of 73°F (22.8°C) are represented by two vertical lines. Using the regression line as a turning point, the corresponding values of the resilient moduli are 206 and 256 ksi. Obviously, the ratio of any two equivalent moduli must be identically one. However, the ratios of these values (206/256 = 0.81 and 256/206=1.24) identify a range of ±20%. Thus, the failure to conduct each modulus test at the same temperature (but within the test specifications) may result in a ±20% error in the resulting IRM_r.

Thoughts on AASHTO T-274-82

RESILIENT MODULUS OF SUBGRADE SOILS

by

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March, 1989

WSDOT Materials Lab
Report No. 200

According to AASHTO T-274, the resilient modulus test is used to provide "a constitutive relationship between stress and deformation of pavement construction materials for use in structural analysis of layered pavement systems".

The basic structure of the test is quite simple. In the test, a repeated axial deviation stress is applied to a cylindrical soil specimen. The recoverable (resilient) axial strain response of the specimen is measured and used to calculate a stress-dependent resilient modulus.

For those of you who have had to run T-274, the inherent simplicity of the test procedures ends right after one reads the basic definitions. The description of the preparation procedures are difficult, at best, while the description of test sequence is nothing short of a maze.

We at the Washington State Department of Transportation (WSDOT) Materials Laboratory, like most western states, have used the Hveem Stabilometer R value test to characterize subgrade soil stiffness. The Hveem Stabilometer R value test has been a very stable, repeatable test. With a reasonable production sequence, we can usually test four samples a day four out of five days a week with one operator.

With this background, we obviously had quite a few concerns as we looked at adopting T-274 as our primary test for characterizing subgrade soil stiffness. Some of our more basic concerns were the following:

1. The test process has distinctly different processes between cohesive and non-cohesive soils. We have a lot of silty sands to sandy silts with some of either characteristic.

2. Very large number of conditioning and test sequences for non-cohesive soils.

3. Most test stages for non-cohesive soils far exceed the stresses expected in any pavement section.

4. When testing cohesive soils, we broke many samples at 0 psi confining pressure. This is probably because we have largely silty soils with very little clayey soils.

5. We question that there is a need to condition all samples at the full range of deviatior and confining stresses. The conditioning sequence takes as much, if not more, operator attention and time as the test sequence.

6. No mater how sophisticated we become in using the stress dependent relationship of the subgrade soils, we still think of the qualitative stiffness value as a single number. In no prior test procedures have we produced a test value that is an exponential equation dependent on a stress

state. How many people have a good feel for a soils stiffness when it's expressed as $4{,}000\ \theta^{.350}$?

We purchased a resilient modulus test device several years ago and began performing resilient modulus tests in our soils lab as part of a combined research project with Dr. Joe Mahoney at the University of Washington to develop a mechanistic pavement design procedure. Twenty-one pavement test sites were established throughout the state to quantify seasonal layer stiffness and fatigue responses for the project. Extensive resilient modulus testing was performed on the different surfacing materials and subgrade soils from all of the test sites. At the time we set up the test procedures for this study, we were privy to the test procedures tentatively recommended for ASTM D-18 Committee on Soil and Rock. Those recommendations basically called for a single set of test stages using 1, 2, 5 and 10 psi deviator stress at 6, 3 and 1 psi confining stress for all materials. The conditioning stages were reduced somewhat from that described in T-274.

For the mechanistic design research project, we decided to perform the test at 1, 2, 4, 6, 8 and 12 psi deviator stress and 1, 2, 4 and 6 psi confining pressure. Sample conditioning was accomplished with 1200 repetitions @ 8 psi deviator stress at 6 psi confining stress.

After completion of the tests for the research project, we reviewed the test results and convinced ourselves that on the majority of the tests we obtained the same stress relationships with or without the higher stress levels. Reasonably repeatable results were obtained performing the test at 1, 2, 4, 6 and 8 psi deviator stress and 1, 2 and 4 psi confining stress. Conditioning was again accomplished using 1200 cycles of 8 psi deviator stress and 6 psi confining pressure.

After again reviewing these test procedures, we have had some concerns that we reduced the test series to stress states well below that which we commonly experience in our roadway sections. To better cover the range of stress found in our roadway section, we now use the following sequence:

Conditioning sample at 8 psi deviator stresses and 6 psi confining stress -

1, 2, 4 and 6 psi deviator stress @ 1 psi confining stress
1, 2, 4 and 6 psi deviator stress @ 2 psi confining stress
1, 2, 4, 6 and 8 psi deviator stress @ 4 psi confining stress
1, 2, 4, 6, 8, 10 & 12 psi deviator stress @ 6 psi confining stress

This pattern covers the range of stresses found in most of our pavement sections which vary from gravel with bituminous surface treatments to full depth asphalt concrete pavement.

FIGURE 1

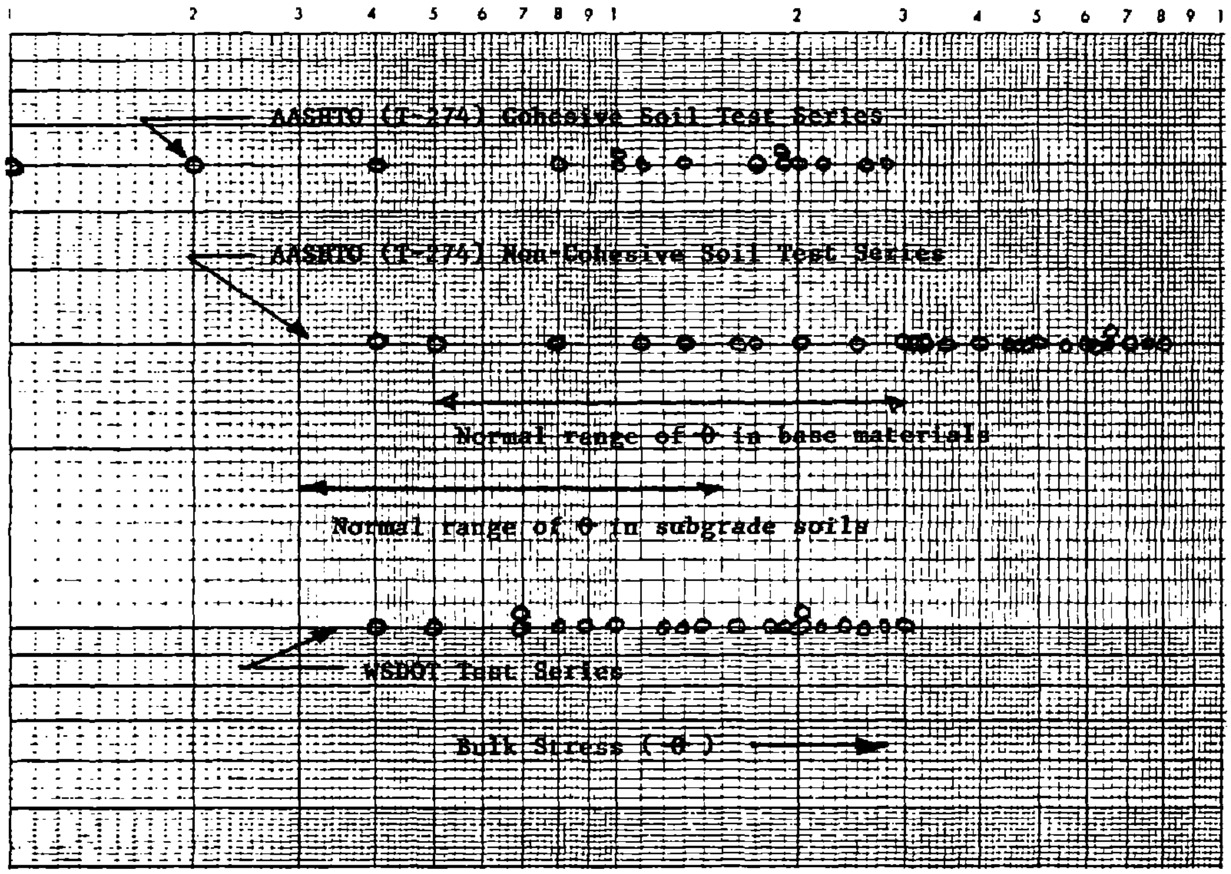

Figure 1 shows the relative position of the test series for T-274 and WSDOT. We prefer this test series because:

1. It covers the bulk stress range found in most of our pavement sections.

2. The conditioning series is such that the operator can make better use of his time while the sample is being conditioned. We can now produce two modulus tests per day with one operator.

3. We do not fail as many samples as we did using T-274.

4. The test sequence is the same for all soils.

In addition to the stress state changes, we have added the following changes to the test procedures that we also consider necessary:

1. We have established a regression procedure to eliminate operator error in fitting a line through the test points. ① The regression statistics are also helpful in judging a given test result. A user-friendly program has been developed to make this process easier.

2. The results of the test are not reported in the form of a stress dependent equation to our field personnel. We have established a convention to report only a single modulus value to our field personnel. The modulus value for a granular material is reported at a bulk stress of 25 psi. The modulus value for fine grained soils (minus exponent) is reported at 10 psi deviation stress.

The modifications we have described reflect our crude attempts to make the test more rational to us. We are aware of many peoples' concerns with the test procedure. We strongly encourage and support a national effort to refine the test procedures to make them more rational and efficient.

1 Joe P. Mahoney, "Regression Analysis for WSDOT Material Applications", Final Report Research Project GC 8286 Task 3, February 1988.

NJ-T-274

RESILIENT PROPERTIES OF SUBGRADE SOILS

by

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Paper prepared for the

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Oregon State University
Corvallis, OR

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TRANSPORTATION ENGINEERING JOURNAL

Resilient Properties of Subgrade Soils

By Marshall R. Thompson[1] and Quentin L. Robnett,[2] Members, ASCE

(Reviewed by the Highway Division)

Introduction

For given traffic and environmental conditions, the most significant factor influencing the design thickness of a flexible pavement is subgrade soil support. This influence is most pronounced at low soil support values, values that are unfortunately typical for the majority of Illinois soils.

It has been well documented that the "resilient deformation" (rebound deformation from repeated load applications) of a flexible pavement structure is responsible for fatigue-type failures in asphalt concrete surface courses. For a typical flexible pavement structure, the subgrade significantly contributes to the total deflection of the pavement system. Recognition of the importance of the resilient behavior of flexible pavements is reflected by the fact that many current flexible pavement thickness design philosophies incorporate "limiting deflection" or "limiting asphalt concrete radial strain" criteria.

The general objectives of the research summarized in this paper were to identify and quantify those soil properties that control the resilient behavior of Illinois soils. Even though this study was conducted with Illinois soils, the results should be applicable to pedologically similar soils.

Soil Sampling and Evaluation

Sampling Program.—A field sampling program, based on pedologic soil series, was implemented to collect typical fine-grained Illinois soils. Soils developed from the major parent materials of the state (loess, Wisconsinan till, and Illinoian till) were included. Emphasis was primarily placed on collecting those soils of predominant occurrence.

It was deemed desirable to examine and compare the resilient properties of not only the predominantly occurring soils of the landscape, but also the soils

Note.—Discussion open until June 1, 1979. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Transportation Engineering Journal of ASCE, Proceedings of the American Society of Civil Engineers, Vol. 105, No. TE1, January, 1979. Manuscript was submitted for review for possible publication on March 3, 1978.

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that are closely associated on the landscape. For this reason, where possible, the other soils of the catena in which the predominant soils occur were also sampled.

A total of 50 individual soil samples was considered in the program. Twenty seven pedologic sites representing approx 39% of the land area of Illinois were sampled. Based on the laboratory data obtained for the soils included in the sampling program, reasonable estimates of the resilient properties of pedologically similar soils can be made.

TABLE 1.—Soil Property Determination

Property (1)	Test procedure (2)
Liquid limit (LL), as a percentage	AASHTO T 89
Plasticity index (PI), as a percentage	AASHTO T 90
American Association of State Highway and Transportation Officials (AASHTO) group index (new procedure)	AASHTO M 145
Grain size analysis	AASHTO T 88
percentage passing No. 200	
percentage silt (0.05 mm–0.002 mm)	
percentage clay (<0.002 mm)	
Soil activity	PI/percentage < 2 μ clay
Maximum dry density, in pounds per cubic foot	AASHTO T 99
Optimum moisture content, as a percentage	AASHTO T 99
California bearing ratio (unsoaked), as a percentage	AASHTO T 193
California bearing ratio (soaked 96 hr), as a percentage	AASHTO T 193
California bearing ratio swell, as a percentage	AASHTO T 193
Specific gravity	AASHTO T 100
pH	1:1 soil water suspension pH meter
Organic carbon, as a percentage	Wet combustion
Static stress-strain	—[a]

[a]Static stress-strain tests were conducted with samples compacted at 95% of AASHTO T 99 maximum dry density. Two nominal moisture contents, optimum (AASHTO T 99) and optimum plus 2% were considered. The unconfined compressive strength was calculated for the ultimate applied load. The static modulus of deformation was determined as an "initial tangent" modulus. Three specimens were used to determine the average response for each test.

Evaluation of Soil Properties.—Selected soil properties were determined. Those properties considered included the more common soil index properties as well as such properties as organic carbon content and pH. Engineering property data were also developed. Table 1 lists the various properties determined and the test procedures utilized.

Presentation of Soil Property Data.—A complete summary of the soil sampling and soil property data is available in Ref. 4. Abbreviated data [liquid limit, plasticity index, percentage of silt, percentage of clay, American Association of State Highway and Transportation Officials (AASHTO) classification, Unified classification, and U.S. Department of Agriculture (USDA) classification] are summarized in Table 2.

Resilient Testing Program

General.—A realistic repeated load testing procedure for evaluating the resilient characteristics of fine-grained soils has been developed at the University of Illinois (3). During the developmental stage, repeated load testing techniques that had been used or proposed for use and information concerning the factors that significantly influence the resilient characteristics of fine-grained soils were considered.

Specimen Preparation.—The factors deemed to be of significance relative to the preparation of resilience test specimens included: (1) Specimen size; (2) compaction moisture and density; and (3) type of compaction.

A specimen size of 5 cm (2 in.) in diameter and 10 cm (4 in.) high was chosen because of the ease and speed of preparation, the small quantity of material required, and the uniformity obtained. For fine-grained soils of the type considered in this project, the specimen size selected is adequate relative to maximum soil particle size consideration.

Compaction moisture content and density are widely recognized as having an important influence on the resilient characteristics of fine-grained soils. Illinois Department of Transportation compaction specifications for fine-grained subgrades require a minimum of 95% of AASHTO T-99 density. Therefore, a substantial amount of the subgrade should be placed at densities ≥ 95%. Consequently, 95% AASHTO T-99 density was chosen as the primary compaction level for test specimen preparation. Forty one (14 A horizons, 27 BC horizons) of the soils were also tested at 100% AASHTO T-99 density.

A study (3) of placement moisture contents for a variety of soils from two interstate highway sections in Illinois, each approx 10 miles (16 km) in length, revealed that sizeable quantities of the subgrades were placed wet of optimum. It was noted that the compaction moisture content averaged 97.2% of optimum with a standard deviation of 15.1% of optimum for the 1,213 observations. If the moisture contents are assumed to approximate a normal distribution, this would indicate that approx 43% of the soil embankment was placed wet of optimum. More specifically, the data indicated that approx 20% of the subgrade material was placed at 110% of optimum or wetter, and approx 7% of the subgrade material was placed at 120% of optimum or wetter.

Based on the facts that large quantities of subgrade may be placed wet of optimum and flexible pavement distress is most pronounced during the wet spring period, resilience test specimens were prepared at and above the optimum moisture content (AASHTO T-99). Specimen moisture contents at preparation were approximately optimum, optimum plus 1%, and optimum plus 2%. In some instances, additional moisture contents were utilized. Specimens compacted at 100% density (AASHTO T-99) were prepared only at optimum plus 2%.

Information in the literature and University of Illinois pilot studies (3) indicated that kneading-type compaction, a procedure that develops substantial shearing strains during the compaction operation, produces specimens in which resilient

TABLE 2.—Soil Classification Data

Soil (1)	Liquid Limit (2)	Plasticity index (3)	Percentage less than 2 μ clay (4)	AASHTO class (5)	Unified class (6)	USDA class (7)
AASHO	25	11	25	A-6(6)	CL	sil
Alvin B	21	2	15	A-4(8)	ML	sl
Ava B	33	11	23	A-6(11)	CL	sil
Bluford A	28	7	6	A-4(5)	CL-ML	sil
Bluford B	47	20	39	A-7-6(20)	CL	sicl
Bryce A	51	22	10	A-7-6(16)	ML	sil
Bryce B	46	24	34	A-7-6(18)	CL	cl
Bryce C	48	26	47	A-7-6(22)	CL	c
Catlin A	42	14	12	A-7-6(15)	ML	sil
Catlin B	47	19	29	A-7-6(22)	ML	sicl
Cisne A	25	4	18	A-4(2)	CL-ML	sil
Cisne B	53	29	42	A-7-6(29)	CH	sic
Clarence C	53	28	72	A-7-6(32)	CH	c
Clinton A	34	9	16	A-4(9)	ML	sil
Clinton B	43	20	38	A-7-6(23)	CL	sic
Drummer B	49	26	38	A-7-6(28)	CL	sic
Elliot A	33	10	13	A-6(5)	CL	sil
Elloit B	45	22	40	A-7-5(19)	CL	sic
Elloit C	34	15	35	A-6(12)	CL	sicl
Fayette A	43	11	20	A-7-5(15)	ML	sil
Fayette B	43	21	31	A-7-6(23)	CL	sicl
Fayette C	32	9	18	A-4(9)	ML	sil
Flanagan A	49	19	24	A-7-5(24)	ML	sil
Flanagan B	49	22	36	A-7-6(25)	CL	sic
Hamburg	25	1	9	A-4(1)	ML	sl
Herrick A	41	12	13	A-7-6(14)	ML	sil
Herrick B	46	21	37	A-7-6(23)	CL	sic
Hosmer A	34	5	3	A-4(6)	ML	si
Hosmer B_2'	47	22	30	A-7-6(25)	CL	sicl
Hosmer B_2	36	9	23	A-4(10)	ML	sil
Hoyleton A	31	6	13	A-4(6)	ML	sil
Hoyleton B	44	19	39	A-7-6(20)	CL	sic
Huey B	44	28	28	A-7-6(14)	CL	sicl
Illinois Till B	28	14	22	A-6(5)	CL	l
Illinois Till C	22	8	12	A-4(1)	CL	l
Ipava A	49	14	28	A-7-5(19)	ML	sicl
Ipava B	56	31	41	A-7-6(36)	CH	sic
Muscatine A	47	17	20	A-7-6(20)	ML	sil
Muscatine B	48	26	32	A-7-6(29)	CL	sicl
Richview A	29	4	11	A-4(3)	ML	sil
Richview B	36	11	29	A-6(10)	ML	sicl
Sable A	57	29	26	A-7-6(33)	CH	sicl
Sable B	51	27	31	A-7-6(30)	CH	sicl
Stoy A	31	8	6	A-4(7)	ML	sil
Stoy B	47	24	28	A-7-6(26)	CL	sicl
Tama A	45	17	32	A-7-6(20)	CL	sicl

TABLE 2.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Tama B	46	24	32	A-7-6(27)	CL	sicl
Wisconsin Till	27	10	15	A-4(4)	CL	l
Wynoose A	26	1	12	A-4(1)	ML	sil
Wynoose B	50	24	36	A-7-6(28)	CL	sic

characteristics are accentuated. Kneading-type compaction, described in Ref. 3, was therefore used to prepare the test specimens.

A complete summary of specimen preparation data (moisture content, dry density, degree of saturation, and volumetric water content) is available in Ref. 4 for the moisture contents of optimum, optimum plus 1%, and optimum plus 2%.

After preparation, the specimens were wrapped in plastic and placed in a constant temperature room at 77° F (25° C) to cure for a minimum of 7 days prior to resilience testing. This curing period was used to minimize any effects that "thixotropic" strength gain might have on the determination of resilient moduli.

Resilience Testing

Testing Equipment.—A pneumatic loading apparatus, capable of applying repeated dynamic loads of controlled magnitude and duration, was used in the study. The typical load pulse duration was 0.06 sec. Resilient (recoverable) deformation (over the entire length of the specimen) was measured with a LVDT. Details of the testing apparatus are presented in Ref. 3.

Specimens were tested with no lateral confining pressure, i.e., $\sigma_3 = 0$. A number of factors were considered in adopting the $\sigma_3 = 0$ testing procedure: (1) Simplicity and ease of testing; (2) it can be shown (e.g., finite element and, elastic layer theory that the magnitude of confining pressure that exists in the upper regions of the subgrade of a typical flexible highway pavement is very low, normally less than 5 psi (34.5 kN/m^2); (3) numerous examples were found in the literature where no lateral confining pressure was used during examination of the resilient behavior of fine-grained soils; (4) results obtained during earlier University of Illinois laboratory studies (3) indicated that small magnitudes of confining pressure [up to $\sigma_3 \simeq 5$ psi (34.5 kN/m^2)] had no significant effect on the resilient behavior of the fine-grained soils examined; and (5) recent studies by Fredlund, Bergan, and Wong (2) indicated that the resilient modulus is not significantly influenced by confining pressure effects if the fine-grained soils are compacted on the "wet side" of optimum water content.

Testing Procedure.—After the curing period, four specimens (given soil, moisture content, density) were prepared for the resilience test. A rubber membrane was placed on each specimen to prevent moisture loss during testing and plexiglas loading caps were placed on each end of the specimen to insure uniform loading.

The specimens were then placed in the resilience device and conditioned with 1,000 axial stress applications (unconfined compression) of a predetermined magnitude. The magnitude of axial stress applied during conditioning was equivalent to the vertical subgrade stress produced by a 9-kip (40-kN) dual

TABLE 3.—Average Soil Property Data

Property (1)	Average (2)	Standard deviation (3)
Liquid limit	40.3	9.8
Plasticity index	16.3	8.4
Group index	16.5	9.9
Percentage passing No. 200	89.7	12.5
Percentage silt	60.9	14.0
Percentage clay (<2 μ)	25.8	13.0
Activity (PI/percentage of clay)	0.68	0.37
γ_D max, in pounds per cubic foot (AASHTO T-99)	101.9	8.2
w_{opt}, percentage of (AASHTO T-99)	19.7	3.6
California bearing ratio (immediate)	8.6	3.0
California bearing ratio (soaked)	3.8	2.1
California bearing ratio swell, as a percentage	1.7	1.13
Specific gravity	2.67	0.06
pH	5.9	1.1
Organic carbon, as a percentage	0.83	0.83

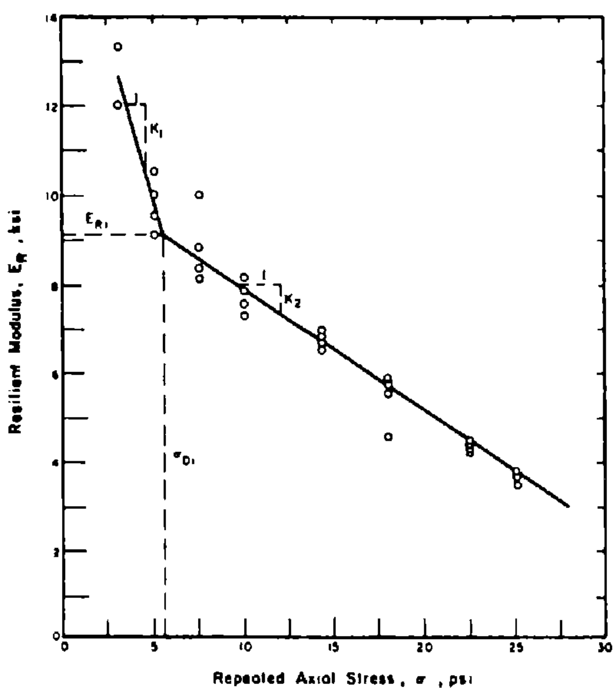

FIG. 1.—Typical Resilient Response Plot (1 ksi = 6.89 MN/m²; 1 psi = 6.89 kN/m²)

wheel load [80-psi (552-kN/m²) tire pressure] placed on a typical Class III (less than 1,000 vehicles for average daily traffic) flexible pavement designed by the Illinois Department of Transportation flexible pavement design procedure (1).

Following specimen conditioning, the resilient behavior of the four specimens was determined at various levels of axial stress. Resilient deformations were measured using an axially mounted LVDT connected to a high-speed recorder. At each incremental stress level, approx 10 stress applications were applied to the specimen and the resilient deformations recorded. Ten stress applications were sufficient to develop a "repeatable" deflection output on the high-speed recorder but yet did not introduce significant "number of load application" effects on the resilient response of the soil. The stress level was incrementally increased, approx 3 psi-5 psi (21 kN/m^2-34.5 kN/m^2) increment, until a substantial amount of permanent deformation developed, making it impossible to further accurately record the resilient behavior.

Resilient Modulus Calculation.—The resilient modulus, E_R, was calculated by dividing the repeated axial stress (equal to the deviator stress for no lateral confinement) by the resilient or recoverable strain.

$$E_R = \frac{\sigma}{\epsilon_R} \quad \text{(1)}$$

in which E_R = resilient modulus; σ = repeated axial stress; and ϵ_R = resilient (recoverable) strain.

Resilient Response.—A special procedure was devised to characterize resilient response. A plot of resilient modulus, in kips per square inch, versus repeated deviator stress, in pounds per square inch, was prepared using the response data for all of the specimens. A typical plot is shown in Fig. 1. The plots typically displayed a "break point" deviator stress where there was a substantial change in the slope of the E_R-σ_D relation. Linear regression analyses were conducted using the data for deviator stresses less than and greater than the break point deviator stress. From the two linear regression equations it was possible to determine their point of intersection. The resilient modulus and deviator stress corresponding to the intersection point were noted as E_{Ri} and σ_{Di}, respectively, as indicated in Fig. 1. The slopes of the two linear equations were designated as K_1 and K_2 as in Fig. 1. Note that the units of K_1 and K_2 are kips per square inch and pounds per square inch, respectively.

Summary resilience plots for all of the soils are presented in Ref. 4. Only the resilient response data for optimum, optimum plus 1%, and optimum plus 2% were used in the statistical analyses for this paper since data for other moisture contents were not developed for all soils.

Effect of Resilient Behavior on Pavement Response.—To provide a proper perspective, the influence of subgrade resilient behavior on the structural response of flexible pavements must be considered. Thompson and Robnett (4) conducted a sensitivity analysis using a stress-dependent elastic layer structural model. The pavement section considered was a 2-in. (51-mm) asphalt concrete surface course and a 10-in. (250-mm) crushed stone base course. The factors varied were E_{Ri} and K_1 of the subgrade. The E_{Ri} values of 3 ksi, 6 ksi, and 15 ksi (21 MN/m^2, 42 MN/m^2, and 104 MN/m^2) and K_1 values of 500 psi/psi, 1,100 psi/psi, and 2,000 psi/psi (3.45 MN/m^2/MN/m^2, 7.6 MN/m^2/MN/m^2, and 13.8 MN/m^2/MN/m^2) were included in the analysis.

Pavement surface deflection, a valid indicator of flexible pavement performance, was used to study the E_{R_i} and K_1 effects. The E_{R_i} effects were most pronounced and primarily controlled surface deflection. Surface deflection is fairly insensitive to K_1 variations except at very low E_{R_i} values. It was apparent that the consequences of the errors associated with estimating E_{R_i} were more critical for the lower E_{R_i} values. In most of the subsequent data analyses, emphasis is placed on the E_{R_i} response parameter.

Data Analyses

Average Values.—Soil property and resilient response data were analyzed. Tables 3 and 4 are summaries of the average values and the standard deviations based on all of the soils. Tables 5 and 6 are similar summaries considering the soils as either A or the combination of B and C horizons.

TABLE 4.—Average Soil Resilient Property Data

Property (1)	Average (2)	Standard deviation (3)
(a) Optimum Moisture		
K_1, in kips per square inch per pound per square inch	−1.21	0.6
K_2, in kips per square inch per pound per square inch	−0.186	0.09
E_{R_i}, in kips per square inch	8.94	3.4
σ_{D_i}, in pounds per square inch	6.2	1.0
q_u, in pounds per square inch	24.4	8.4
E_{static}, in pounds per square inch	2,677	1,374
Degree of saturation, as a percentage	71.7	5.6
(b) Optimum Moisture + 1%		
K_1, in kips per square inch per pound per square inch	−1.135	0.60
K_2, in kips per square inch per pound per square inch	−0.171	0.11
E_{R_i}, in kips per square inch	7.36	3.4
σ_{D_i}, in pounds per square inch	5.9	1.3
Degree of saturation, as a percentage	76.3	5.8
(c) Optimum Moisture + 2%		
K_1, in kips per square inch per pound per square inch	−1.011	0.60
K_2, in kips per square inch per pound per square inch	−0.170	0.10
E_{R_i}, in kips per square inch	6.22	3.2
σ_{D_i}, in pounds per square inch	6.17	1.14
q_u, in pounds per square inch	19.5	6.6
E_{static}, in pounds per square inch	1,767	1,788
Degree of saturation, as a percentage	80.3	5.8

Note: 1 psi = 6.89 kN/m^2; 1 ksi = 6.89 MN/m^2.

Simple Correlation.—Simple correlation procedures were used to determine the relation among the soil properties and resilient response characteristics. Tables 7 and 8 summarize the simple correlation data. Only those correlations significant at $\alpha = 0.05$ are shown.

Regression Equations.—Based on the preliminary data obtained from the simple correlation analysis, selected commonly determined soil properties (those with the higher correlation coefficients from the simple correlation analyses) were used to develop simple and multiple linear regression equations for predicting the resilient response parameter, E_{Ri}, for conditions of optimum moisture content and 95% AASHTO T-99 density. Table 9 is a summary of the better correlations developed.

Analysis of Variance—Soil Classification Effects.—The soils were grouped according to the AASHTO, Unified, and USDA classification systems. In some

TABLE 6.—Average Soil Property Data for A Horizons and B + C Horizons

Property (1)	Averages: A horizons (18) (2)	Averages: BC horizons (32) (3)
Liquid limit	37.41	41.28
Plasticity index	10.59	18.88
Group index	11.00	18.94
Percentage passing No. 200	89.65	89.53
Percentage silt	70.06	55.75
Percentage clay ($<2\,\mu$)	15.12	31.44
Activity (PI/percentage of clay)	0.83	0.59
γ_d max, in pounds per cubic foot (AASHTO T-99)	99.49	103.54
w_{opt}, as a percentage (AASHTO T-99)	20.99	18.84
California bearing ratio (immediate)	7.65	9.08
California bearing ratio (soaked)	3.76	3.88
California bearing ratio swell, as a percentage	1.76	1.67
Specific gravity	2.60	2.70
pH	5.66	6.06
Organic carbon, as a percentage	1.62	0.34

instances, the groups included more than one soil type. Analysis of variance procedures and Duncan's Multiple Range Test were utilized to determine if there are significant differences in resilient response behavior among the various soil groupings. The resilient response parameter, E_{Ri}, was considered for approximate moisture contents of optimum, optimum plus 1%, and optimum plus 2%.

Tables 10, 11, and 12 are summaries of the analyses. There are no significant differences among the means underscored by the same line.

Linear Regression—Degree of Saturation Effects.—The simple correlation analyses indicated a highly significant ($\alpha = 0.01$) relation between E_{Ri} and degree of saturation. Fig. 2 includes the plot (95% AASHTO T-99 density) showing the relation between E_{Ri} and degree of saturation. The regression equation shown in Fig. 2 is highly significant ($\alpha = 0.01$).

TABLE 6.—Average Soil Resilient Property Data for A Horizons and B + C Horizons

Property (1)	Water content (2)	Average: A horizons (18) (3)	Average: BC horizons (32) (4)
Water content, as a percentage	O	21.04	18.80
	O + 1	22.09	19.86
	O + 2	23.17	20.96
Degree of saturation, as a percentage	O	75.39	69.79
	O + 1	79.14	74.72
	O + 2	83.06	78.87
K_1, kips per square inch per pound per square inch	O	−0.880	−1.387
	O + 1	−0.854	−1.268
	O + 2	−0.646	−1.186
K_2, kips per square inch per pound per square inch	O	−0.126	−0.218
	O + 1	−0.113	−0.199
	O + 2	−0.093	−0.209
E_{Ri}, kips per square inch	O	5.54	10.72
	O + 1	4.77	8.71
	O + 2	3.80	7.46
σ_{Di}, in pounds per square inch	O	6.23	6.21
	O + 1	6.07	5.86
	O + 2	6.12	6.24
q_u, pounds per square inch	O	17.8	27.8
	O + 2	15.1	21.8
E_{static}, pounds per square inch	O	1,658	3,225
	O + 2	960	2,198

Note: O = Optimum moisture content, as a percentage; 1 psi = 6.89 kN/m^2; 1 ksi = 6.89 MN/m^2.

TABLE 7.—Significant Simple Correlation Coefficients

Resilient response property (1)	LL, as a percentage (2)	PI, as a percentage (3)	Group index (4)	Silt,[a] as a percentage (5)	Clay,[a] as a percentage (6)	California bearing ratio immediate (7)	California bearing ratio soaked (8)	Swell,[b] as a percentage (9)	Specific gravity (10)	Organic carbon, as a percentage (11)
(a) Optimum[c]										
K_1		−0.274[d]		0.433[e]	−0.356[e]	−0.296[d]			−0.290[d]	0.272[d]
K_2		−0.337[d]	−0.278[d]	0.274[d]	−0.494[e]	−0.430[e]			−0.536[e]	0.365[e]
E_{Ri}	0.330[d]	0.581[e]	0.434[e]	−0.450[e]	0.600[e]	0.397[d]			0.597[e]	−0.519[e]
σ_D						−0.310[d]				
(b) Optimum + 1%[c]										
K_1				0.295[d]	−0.395[e]				−0.291[d]	
K_2	−0.365[d]	−0.418[e]	−0.367[e]		−0.478[e]	−0.478[e]		−0.278[d]	−0.366[e]	0.282[d]
E_{Ri}	0.461[e]	0.610[e]	0.508[e]	−0.383[e]	0.587[e]	0.450[e]	−0.321[d]	0.423[e]	0.437[e]	−0.446[e]
(c) Optimum + 2%[c]										
K_1	−0.331[d]	−0.425[e]	−0.353[d]	0.283[d]	−0.519[e]	−0.349[d]			−0.315[d]	
K_2	−0.465[e]	−0.594[e]	−0.502[e]	0.371[e]	−0.566[e]	−0.532[e]		−0.345[d]	−0.480[e]	0.312[d]
E_{Ri}	0.459[e]	0.597[e]	0.500[e]	−0.354[e]	0.555[e]	0.484[e]	−0.318[e]	0.422[e]	0.440[e]	−0.351[d]

[a] USDA grain size terminology.
[b] CBR swell procedure.
[c] Moisture content relative to AASHTO T-99 optimum.
[d] $\alpha = 0.05$, $r \geq 0.273$ (indicates significance at $\alpha = 0.05$).
[e] $\alpha = 0.01$, $r \geq 0.354$ (indicates significance at $\alpha = 0.01$).

Fig. 2 is based on all of the soils included in the study. Similar regression analyses were made for the soils grouped according to the AASHTO, Unified, and USDA classification procedures. Further subdivision within each group was made based on soil horizon as indicated in Table 13.

TABLE 8.—Significant Simple Correlation Coefficients

Resilient response property (1)	Degree of saturation, as a percentage[a] (2)	Unconfined compressive strength, in pounds per square inch[b] (3)	Static modulus of elasticity, in pounds per square inch[b] (4)
K_1	0.169[c]	−0.472[d]	−0.290[d]
K_2	0.413[e]		−0.351[d]
E_{Ri}	−0.641[e]	0.684[d]	0.732[d]

[a]Based on 149 observations.
[b]Based on 100 observations.
[c]$\alpha = 0.05$, $r \geq 0.159$ (indicates significance at $\alpha = 0.05$).
[d]$\alpha = 0.01$, $r \geq -0.254$ (indicates significance at $\alpha = 0.01$).
[e]$\alpha = 0.01$, $r \geq 0.208$ (indicates significance at $\alpha = 0.01$).
Note: 1 psi = 6.89 kN/m^2.

TABLE 9.—Summary of Regression Equations including Soil Properties

		Regression Coefficient b							
Equation number (1)	Intercept, a, in kips per square inch (2)	Clay, as a percentage (3)	PI (4)	Organic carbon, as a percentage (5)	Silt, as a percentage (6)	Group index (7)	Liquid limit (8)	Correlation coefficient R (9)	Standard error of estimate, S_E, in kips per square inch (10)
1	4.88	0.157						0.600[a]	2.76
2	5.12		0.235					0.581[a]	2.80
3	10.71			−2.14				0.519[a]	2.95
4	15.59				−0.109			−0.450[a]	3.08
5	6.46					0.150		0.434[a]	3.10
6	4.32						0.114	0.330[b]	3.25
7	4.46	0.098	0.119					0.630[a]	2.70
8	6.90	0.0064	0.216	−1.97				0.757[a]	2.30
9	9.97	−0.0178	0.222	−1.88	−0.043			0.772[a]	2.26
10	6.37	0.034	0.450	−1.64	−0.0038	−0.244		0.796[a]	2.18
11	8.58	0.0586	0.1397		−0.0561			0.611[a]	2.68
12	3.63	0.1239	0.4792		0.0031	−0.3561		0.721[a]	2.49

[a]Significant at $\alpha = 0.01$.
[b]Significant at $\alpha = 0.05$.
Note: Regression equation of the form: $E_{Ri} = a + b_1 X_1 + b_2 X_2 + b_n X_n$ in which E_{Ri} is in kips per square inch. 1 ksi = 6.89 MN/m^2.

Certain of the soils were tested at conditions of 100% AASHTO T-99 density and optimum plus 2% water content. Fig. 2 shows a plot of E_{Ri} as a function of degree of saturation for 100% compaction. A highly significant ($\alpha = 0.01$) regression equation relating E_{Ri} and degree of saturation is also shown in Fig. 2.

Analysis of Results

General.—The average data summaries, Tables 3 and 4, indicate that typical fine-grained Illinois soils display a wide range of resilient behavior for given compaction conditions and also provide extensive general information concerning soil properties and resilient response behavior. It is apparent that a wide spectrum of soils was considered in the program.

It is important to note in Table 4 that σ_{Di}, the stress level corresponding to E_{Ri}, is approx 6 psi (41.4 kN/m^2) and has a small standard deviation regardless of compaction moisture content. For many purposes, it may be satisfactory to assume a stress level of 6 psi (41.4 kN/m^2) for σ_{Di}.

TABLE 10.—Analysis of Variance Results—AASHTO Classification

Horizons (1)	Moisture content, as a percentage[b] (2)	Average (3)	Group Averages[a] A (4)	B (5)	C (6)	D (7)	Calculated F (8)
ABC	O	8.93	6.10	6.27	10.49	11.03	29.58[c]
ABC	O + 1	7.34	4.62	4.97	7.16[d]	9.34[e]	23.61[c]
ABC	O + 2	6.23	3.74	3.92	5.46[d]	8.20[e]	37.97[c]
BC	O	10.72	7.19[f]	10.33[g]	11.63	11.64	26.15[c]
BC	O + 1	8.71	5.19[f]	7.35[d]	7.89[g]	10.26[e]	15.69[c]
BC	O + 2	7.46	3.84[f]	5.58[d]	6.31[g]	9.18[e]	168.25[c]
A	O	5.77	3.28	5.59	6.89	7.97	26.03[c]
A	O + 1	4.77	2.44	4.81	5.83	6.18	—
A	O + 2	4.05	2.03	3.90	4.87	5.11	7.40[c]

[a]For averages indicated in the manner 7.16[d], d denotes proper grouping.
[b]O = optimum moisture content, AASHTO T-99.
[c]Significant at $\alpha = 0.05$.
[d]Group A-6 = D.
[e]Group A-7-6 = C.
[f]Group A-4 = B.
[g]Group A-7-5 = A.
Note: Variable – E_{Ri} is in kips per square inch; 1 ksi = 6.89 MN/m^2. Underscoring indicates that there are no significant differences among the means.

Table 4 indicates substantial variability in the E_{Ri} and K_1 resilient response properties. It is apparent that soil properties substantially effect a soil's resilient response characteristics.

Resilient behavior is influenced by many factors. Various important factors are considered in more detail in the following sections.

Soil Properties.—Soil properties that significantly correlated with resilient behavior are indicated in Table 7. Those properties that tend to contribute to low resilient moduli (low E_{Ri}) are low plasticity (LL, PI), low group index, high silt content, low clay content, low specific gravity, and high organic carbon contents.

Soil Classification Effects.—The analysis of variance soil classification data (Tables 10, 11, and 12) indicates that the resilient behavior (E_{Ri} response parameter) of the various groups in the classification systems is not, in general, significantly different. Thus, classifying the soil in the AASHTO, Unified, or USDA system is not sufficient for the purpose of placing fine-grained soils into distinctive resilient behavior groups.

Strength Properties.—The correlation data in Table 7 indicate that certain strength related properties also significantly relate to E_{Ri}. Immediate California

TABLE 11.—Analysis of Variance Results—Unified Classification

Horizons (1)	Moisture content, as a percentage[b] (2)	Average (3)	Group Averages[a] A (4)	B (5)	C (6)	Calculated F (7)
ABC	O	8.93	6.52	10.65	10.86	33.47[c]
ABC	O + 1	7.34	5.48	8.45	11.14	12.81[c]
ABC	O + 2	6.23	4.56	7.22	8.83	9.38[c]
BC	O	10.72	8.91	11.26[d]	11.33[e]	3.28
BC	O + 1	8.71	7.35	8.88	11.14	1.76
BC	O + 2	7.45	6.30	7.67	9.01	1.01
A	O	5.77	5.05	7.13	9.67	41.55[c]
A	O + 1	4.77	4.32	6.24	—	6.85[c]
A	O + 2	4.04	3.48	4.82	8.27	53.70[c]

[a] For averages indicated in the manner 11.26[d], d denotes the proper grouping.
[b] O = optimum moisture content, AASHTO T-99.
[c] Significant at $\alpha = 0.05$.
[d] Group CH = C.
[e] Group CL, ML-CL = B.
Group ML, MH = A; Note: variable E_{Ri} is in kips per square inch; 1 ksi = 6.89 MN/m^2. Underscoring indicates that there are no significant differences among the means.

bearing ratio (CBR) correlates positively with E_{Ri}, but for optimum and optimum plus 2% moisture contents, soaked CBR (the value normally used in pavement design) was *negatively* correlated with E_{Ri}. A similar negative correlation was reported in an earlier University of Illinois report (3). The positive correlation between E_{Ri} (for optimum plus 1% and optimum plus 2% moisture contents) and percentage of CBR swell are probably related to the fact that clay content is positively correlated with swell and plasticity index.

Strong positive correlations with E_{Ri} (Table 8) were obtained for the static stress-strain data (unconfined compressive strength and static modulus). Regres-

TABLE 12.—Analysis of Variance Results—USDA Classification

Horizons (1)	Moisture content, as a percentage[b] (2)	Average (3)	Group Averages[a] A (4)	B (5)	C (6)	D (7)	Calculated F (8)
ABC	O	8.94	6.27	7.30	10.00	11.82	11.15[c]
ABC	O + 1	7.34	4.96	5.53	8.36	10.81	23.14[c]
ABC	O + 2	6.23	4.41	4.57	7.00	9.49	18.36[c]
BC	O	10.72	6.27	10.62	10.76	10.81	3.63[c]
BC	O + 1	8.71	4.96	7.15	8.79	10.81	4.97[c]
BC	O + 2	7.45	4.41	6.16	7.31	9.49	3.79[c]
A	O	5.77		5.46	7.31		2.93
A	O + 1	4.78		4.67	5.58		0.49
A	O + 2	4.05		3.73	5.64		4.30

[a]For averages indicated in the manner—1.32 (C), C denotes proper grouping.
[b]O = optimum moisture content, AASHTO T-99.
[c]Significant at $\alpha = 0.05$.
Note: Groups A = sandy loam; B = silt loam, loam, silt; C = silty clay loam, clay loam; D = silty clay, clay; variable = E_{Ri}, in kips per square inch; 1 ksi = 6.89 MN/m^2. Underscoring indicates that there are no significant differences among the means.

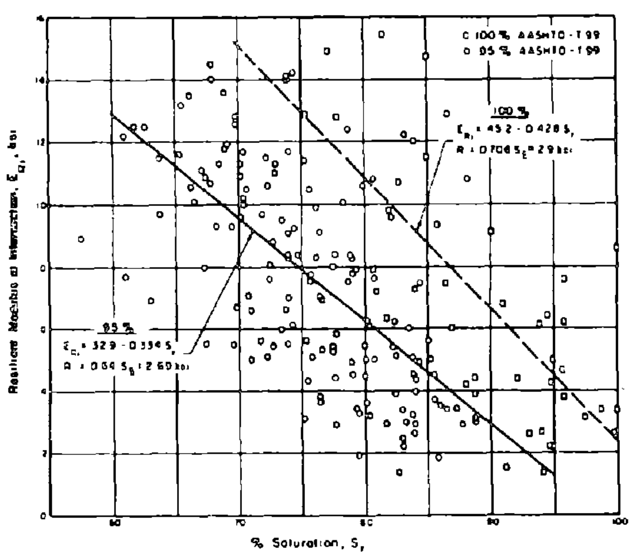

FIG. 2.—Resilient Modulus—Saturation Relations, as a Percentage, for 95% and 100% Compaction (1 ksi = 6.89 MN/m^2)

sion equations relating E_{Ri} and unconfined strength and E_{Ri} and static modulus are

$$E_{Ri} = 3.46 + 1.9\,E; \quad R = 0.73; \quad S_{\hat{x}} = 2.43 \qquad (2)$$

$$E_{Ri} = 0.86 + 0.307\,q_u; \quad R = 0.684; \quad S_{\hat{x}} = 2.61 \qquad (3)$$

in which E_{Ri} is in kips per square inch; E = static modulus, in kips per square

TABLE 13.—E_{Ri}—Degree of Saturation Regression Equations (Soil Classification Effects)

Group (1)	Horizons (2)	a, in kips per square inch* (3)	b* (4)	Standard error of estimate, in kips per square inch (5)	Correlation coeffi- ient, R (6)
		(*a*) USDA			
sicl, c, sic	ABC	27.59	−0.248	2.21	0.620
	BC	25.60	−0.217	2.10	0.595
sil, l, si,	ABC	31.14	−0.325	2.68	0.572
	BC	33.90	−0.345	3.59	0.541
		(*b*) AASHO			
A-7-5	ABC	39.83	−0.453	1.81	0.831
	BC	27.54	−0.266	1.57	0.711
A-4	ABC	17.33	−0.158	1.22	0.619
	BC	16.76	−0.146	1.64	0.549
A-7-6	ABC	31.22	−0.294	2.28	0.652
	BC	24.65	−0.196	2.05	0.529
A-6	ABC	36.15	−0.362	3.32	0.645
	BC	35.67	−0.354	3.67	0.639
		(*c*) Unified			
CL, ML-CL	ABC	31.89	−0.312	2.59	0.623
	BC	32.13	−0.311	2.56	0.637
CH	ABC	21.93	−0.151	1.84	0.594
	BC	23.02	−0.161	1.87	0.650
ML, MH	ABC	31.39	−0.331	2.15	0.670
	BC	29.01	−0.284	2.78	0.577

*Equation of the form $E_{Ri} = a + b\,S_r$.
Note: E_{Ri} is in kips per square inch; S_r = degree of saturation, as a percentage; 1 ksi = 6.89 MN/m^2.

inch; q_u = unconfined compressive strength, in pounds per square inch; $S_{\hat{x}}$ = standard error of estimate; and R is significant at α = 0.01. Note that E_{Ri} is substantially larger than the static modulus of elasticity.

Degree of Saturation Effects.—Degree of saturation is a factor that reflects the combined effect of density and moisture content. The value of E_{Ri} is strongly correlated with degree of saturation as shown in Table 8. The regression equations shown in Fig. 2 indicate that E_{Ri} can be estimated based on degree of saturation. The E_{Ri} degree of saturation regression equations differ for 95% AASHTO T-99 and 100% AASHTO T-99 compaction. Compaction of 100% provides higher E_{Ri} for a *given degree of saturation*. The difference in E_{Ri} values for 100% and 95% AASHTO T-99 compaction is reduced at increased degrees of saturation.

The combined effects of compaction moisture content and density are easily discerned using the E_{Ri} degree of saturation relations. For soils substantially wet of optimum, high degrees of saturation, and low E_{Ri} values are characteristic, regardless of level of compaction.

The fact that the E_{Ri} degree of saturation relations are valid for the wide range of fine-grained soils considered is particularly significant. Since moisture content and density are used in compaction control testing, the specific gravity of the soil solids is the only additional datum needed to calculate the degree of saturation and subsequently predict E_{Ri}.

TABLE 14.—Analysis of Variance Results—Percentage of Compaction Effects

Grouping (1)	Average Resilient Modulus, in kips per square inch			Calculated F (5)
	Repeated deviator stress, in pounds per square inch (2)	95% density[a] (3)	100% density[a] (4)	
A horizons	3	5.94	5.86	0.021
	5	4.46	4.46	0.0
	8	3.87	3.89	0.005
	16	2.82	2.98	0.519
B + C horizons	3	11.73	13.36	6.24[b]
	5	8.86	10.14	9.96[b]
	8	7.50	8.72	10.82[b]
	16	5.5	6.7	10.07[b]

[a]AASHTO T-99.
[b]Significant at $\alpha = 0.05$.
Note: 1 psi = 6.89 kN/m^2; 1 ksi = 6.89 MN/m^2.

Additional E_{Ri} degree of saturation regression equations were developed for various soil classification groups. The equations developed (Table 13) can be used to predict E_{Ri} for different soil groups. The regression coefficient, b, is indicative of moisture sensitivity. It is interesting to note that for the BC horizons those classifications with higher clay contents and increased plasticity tend to be less sensitive to changes in degree of saturation. For example, the CH group in the United system and the silty clay loam, clay, and silty clay groups in the USDA system have regression coefficients substantially less than the other groups.

Compaction Effects.—In addition to the E_{Ri} degree of saturation linear regression analyses (Fig. 2), direct comparisons of the effect of 95% and 100% AASHTO T-99 compaction on resilient behavior were also made. Various soils (14 A horizons 27 B and C horizons) were compared at a water content of approximately optimum plus 2%. Thus, the only variable was percentage of compaction. Compaction conditions and the resilient moduli data for various repeated deviator stress levels [3 psi, 5 psi, 8 psi, and 16 psi (21 kN/m^2, 34.5 kN/m^2, 55.2 kN/m^2, and 110.4 kN/m^2)] have been summarized in Ref. 4.

Analysis of variance procedures were used to determine if percentage of compaction was a significant factor when moisture content was maintained at approximately the same value. Separate analyses were conducted for A horizon soils and the combined B + C horizon soils. The results are summarized in Table 14.

The effect of increased compaction was *not* significant for A horizon soils, but was significant at all stress levels for the BC horizon soils. It is important to note that the differences, though statistically significant, were not large. The maximum resilient modulus increase (Table 14 data) achieved by increased compaction was about 1.6 ksi (11 MN/m^2) and the minimum increase was 1.2 ksi (8.3 MN/m^2). As shown in Fig. 2, a degree of saturation change of approx 4% or 5% (about a 1% change in gravimetric water content) would effect approximately the same result.

Horizon Effects.—The average data in Table 6 indicate the substantial effect of horizon (A or BC). Note in particular that for A horizons: (1) The degrees of saturation are higher; (2) K_1 response parameters are lower; (3) E_{Ri} values are lower; and (4) q_u and E_{static} are lower.

It is apparent that, in general, A horizon soils are inferior to BC horizon materials. The use of A horizon materials in subgrade construction and associated problems of "top soil stripping" should be carefully considered.

Procedures for Predicting Resilience

General.—Results from this study have clearly demonstrated that the resilient properties of fine-grained Illinois soils range over a wide spectrum. Degree of saturation effects account for a substantial portion of the variability in resilient properties. The resilient response parameter, E_{Ri}, is the most significant subgrade input relative to predicting the structural response of a flexible pavement section. For practical purposes, a K_1 value of 1.1 ksi/psi (1.1 $kN/m^2/N/m^2$) may be used and σ_{Di} may be taken as 6 psi (41 kN/m^2).

The development of procedures for predicting E_{Ri} would be of great value in the analysis and design of new or existing flexible pavements. Various prediction procedures are described in the following sections.

Soil Property Based Procedure.—The soil property based regression equations presented in Table 9 can be used to predict E_{Ri} for the conditions of 95% AASHTO T-99 compaction and optimum moisture content. Selection of the best equation to use depends on the nature of the soil property data available and a consideration of the standard error of estimate for the particular equation. The equation with the lowest standard error of estimate should be used.

To correct E_{Ri} for different moisture content conditions (compaction still at 95%), adjust E_{Ri} 0.334 ksi (2.3 MN/m^2, the regression coefficient in Fig.

2 for 95% compaction) for each 1% change in percentage of saturation (decrease E_{Ri} for increased saturation and increase E_{Ri} for decreased saturation). Extrapolation of the relation to moisture contents significantly dry of optimum is not recommended.

Degree of Saturation Procedure.—The value of E_{Ri} can be predicted from degree of saturation data for either 95% or 100% compaction conditions based on the regression equations shown in Fig. 2. If soil classification data are available, more precise estimates for 95% compaction can be made with the regression equations for the various soil classifications summarized in Table 13.

Soil Classification Procedure.—General estimates of the E_{Ri} resilient response parameter can be made based on the data presented in the analysis of variance results (Tables 10, 11, and 12). The values of E_{Ri} can be corrected for different moisture contents (95% compaction) using the technique described under "Soil Property Based Procedure." It is important to note that these data are "average values" and no "standard errors" of estimate are related to the data as is the case with a regression equation.

Adequacy of Prediction Procedures.—Based on the various regression equations developed, E_{Ri} can be predicted with a certain degree of accuracy. The standard error of estimate, $S_{\hat{y}}$, shown for the various regression equations is a measure of the error associated with the prediction. The E_{Ri} value will be within $\pm$ 1 $S_{\hat{y}}$ of the predicted value 68% of the time and $\pm$ 2 $S_{\hat{y}}$ 95% of the time.

Typical standard errors of estimate are in the range of 1.5 ksi-3.5 ksi (10.3 MN/m^2-24.1 MN/m^2). Considering that field moisture content and density conditions (thus degree of saturation) and soil type along a project location are quite variable, an $S_{\hat{y}}$ of 2.0 ksi-3.0 ksi (13.8 MN/m^2-20.7 MN/m^2) seems adequate for many pavement design and analysis considerations. This is especially true considering the inability of current technology to predict subgrade moisture content and moisture changes as a function of time.

Summary.—Several procedures have been described for estimating resilient properties. If the estimated resilient properties are not sufficiently accurate for the particular application, the laboratory resilient testing procedures used in this study can be used to evaluate the resilient properties of the soil for desired conditions of moisture and density.

Summary and Conclusions

An extensive and comprehensive study of the resilient properties of Illinois soils was conducted. The range of soils included in the program are characteristic of a substantial percentage of the fine-grained soils encountered in pavement construction throughout the state and other areas of the United States with pedologically similar soils.

Illinois soils display a wide range of resilient properties. Natural soil characteristics and compaction conditions, particularly moisture content, are shown to primarily control resilient behavior.

Significant correlations and regression relations were developed between resilient behavior and such factors as soil properties, degree of saturation, and static strength and modulus data. The regression equations can be used to predict the probable resilient properties of a soil.

Current classification procedures do not group fine-grained soils into classes with distinctive resilient properties. However, certain classification procedures provide a general indication of expected resilient behavior and the soil's sensitivity to moisture change.

The influence of increased compaction on resilient properties (95%–100% of AASHTO T-99) was demonstrated. Based on E_R, degree of saturation relations, increased compaction effected higher resilient moduli. Direct comparison of resilient moduli for 95% and 100% compaction (optimum plus 2% moisture content) indicated no significant difference for A horizon soils but a significant difference for the BC horizon grouping. Resilient moduli were higher for 100% compaction, although the increase was quite small, on the order of 1.2 ksi–1.6 ksi (7.6 MN/m^2–11 MN/m^2).

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ROUND ROBIN TESTS AND USE OF TEST RESULTS IN PAVEMENT DESIGN

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ROUND ROBIN TESTS AND USE OF TEST RESULTS IN PAVEMENT DESIGN[a]

By Ira J. Huddleston[1] and Haiping Zhou[2]

INTRODUCTION

Resilient modulus tests are used to characterize the structural behavior of component materials in mechanistic pavement design procedures and in the 1986 AASHTO pavement design procedure. Because resilient modulus is a relatively new test for many agencies, the degree of repeatability in a "production" lab is not well understood. This study was undertaken to evaluate 1) the variability of resilient modulus test results obtained from different agencies for three common pavement materials, and 2) the effect of the variability on actual pavement designs performed in accordance with the 1986 AASHTO Guide and mechanistic procedures.

Test Design

Seven sets of pavement samples were prepared in the Oregon state Highway Department (OSHD) Laboratory. Each set included three asphalt concrete samples for diametral indirect tension testing, a prepared 4.0-in x 8.0-in cohesive soil sample for triaxial testing and a bulk granular base sample for triaxial testing. Five of the sets were sent to different agencies for testing. The triaxial tests were to be performed in accordance with AASHTO T-274 and the diametral in accordance with ASTM D4123. Not all of the agencies were able to complete their testing in time for the results to be included in this paper.

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[a] Presented at the Workshop on Resilient Modulus Testing at Oregon State University, Corvallis, Oregon.
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Soil and Base Materials

The cohesive soil samples were prepared at maximum density and optimum moisture content using a standard proctor compactive effort (AASHTO T-99). The moisture density curve and soil classification information is shown in Figure 2.

The granular base course samples were shipped bulk. Each agency was asked to prepare and test these samples in a loose and dense state. Actual sample preparation procedures were left up to the agencies.

TEST RESULTS

Diametral Modulus

Diametral test results were received from five agencies. Table 2 summarizes these test results. Due to the variations associated with the aging, not all these results can be compared directly. The results of tests performed by more than one agency on the same set of samples are shown in Table 3. The test results are plotted on Figure 3. Generally, the test results are fairly comparable.

Triaxial Modulus

Triaxial test results were received from three agencies. The granular material was tested at different moisture and density conditions. Tables 4 and 5 summarize the test results. Differences between the test results are illustrated in Figure 4. Test results for the pre-compacted cohesive soil are summarized in Table 6 and illustrated in Figure 5. The results show a considerable amount of variability for the granular material. The cohesive soil test results compare more favorably.

Additional assumptions used to compute the design are listed below:

a. Traffic = 1.0 million 18-kips ESAL's.

b. Reliability = 50%

c. Standard deviation = 0.45 (for flexible pavement).

d. Initial PSI = 4.2; terminal PSI = 2.5.

e. Drainage coefficient = 1.0.

The resulting pavement designs are summarized in Table 9.

<u>Mechanistic Design</u>

In a mechanistic design, the resilient modulus of each layer is used in a layered elastic pavement model (ELSYM5) to predict critical stresses and strains. For this example, the locations are shown in Figure 6. To compare each set of test results, the layer thicknesses were fixed. The layer thickness used were 4.0 inches of asphalt concrete and 10.0 inches of aggregate base. Material properties for mechanistic analysis are summarized in Table 10. Based on these layer thicknesses and the predicted layer moduli, the expected repetitions to failure were estimated using failure criteria for fatigue (Finn's fatigue relationship) and rutting (Chevron relationship) as shown in Figures 7 and 8. Table 11 summarizes the results of the mechanistic analysis. Figure 9 shows a comparison using the test data from the three agencies on predicted pavement fatigue and rutting performance.

DISCUSSION OF RESULTS

Based on the results from the tests performed for this study, the following comments are appropriate:

1. The asphalt concrete modulus varied in a relatively small range. The most significant finding for the asphalt concrete testing may be

Table 1 Asphalt Concrete Samples Prepared for Resilient Modulus Workshop

Sample #	Hour FABR'D	OSHD MR (ksi)	Average of 3	Bulk SP.GRAVY.
1	2.0	430		2.42
2	2.0	455	433	2.42
3	2.0	414		2.42
4	2.5	458		2.42
5	2.5	559	462	2.43
6	2.5	368		2.40
7	3.0	459		2.41
8	3.0	540	492	2.42
9	3.0	476		2.42
10	3.5	445		2.41
11	3.5	526	505	2.43
12	3.5	544		2.42
13	4.0	642		2.41
14	4.0	583	651	2.41
15	4.0	729		2.42
16	4.5	580		2.40
17	4.5	672	655	2.41
18	4.5	712		2.42
19	5.0	747		2.41
20	5.0	702	743	2.42
21	5.0	781		2.42

Table 3 Summary of Diametral Test Results on Identical Samples

Sample I.D.	Modulus (ksi) *	Average	Variance
13	529, 619, 642, 650	610	48
14	497, 568, 583, 600	497	39
16	451, 552, 553, 580	451	49
5	397, 559, 571	509	79
8	365, 540, 575	493	92
12	372, 544, 575	497	89
3	414, 428	421	7
6	368, 387	378	10
11	526, 560	543	17

* Tested at 77 °F

Table 5 Summary of Granular Soil Test Results (Loose)

Test Agency	K1	K2	R^2	Water Content	Wet Density	Dry Density
A	4874	0.2563	0.26	6.0	100.67	94.98
B	3133	0.5004	0.83	4.8	116.35	111.02

Table 7 Summary of Unit Weight Test Results for AC Mix

Test Agency	Sample I.D.	Unit Weight (pcf)	Average (pcf)	Variance
A	5 8 12	150.94 147.94 149.92	149.60	1.25
B	1 9 10	149.33 149.57 148.46	149.12	0.48
C	13 14 16	150.68 150.34 149.90	150.31	0.32
D	13 14 16	150.60 150.90 149.80	150.43	0.46
E	3 6 11	150.70 150.60 151.20	150.80	0.26

Table 9 Pavement Structures Design Using AASHTO Method

Test Agency	Layer	Thickness (inch)	Design SN	Required SN	Subgrade Modulus (ksi)
A	AC BC	4.5 9.5	1.89 0.47	2.36	11.6
B	AC BC	4.5 13.0	1.89 0.39	2.28	12.7
C	AC BC	4.5 5.5	1.89 0.50	2.38	11.3

Note: AC= Asphalt Concrete; BC= Base Course

Table 11 Summary of Mechanistic Analysis Results

Test Agency	AC Mr	BC Mr	Subgrade Mr	ε_{ac}	N_{ac} (10^5)	ε_{sg}	N_{sg} (10^5)
A	730 620 510	12.1 12.4 12.7	11.6 11.6 11.6	313 346 389	3.0 2.0 1.5	586 626 675	5.0 3.3 2.4
B	730 620 510	9.1 9.4 9.7	12.7 12.7 12.7	330 367 415	2.2 1.8 1.1	526 567 616	7.0 5.2 3.5
C	730 620 510	19.6 20.2 20.8	11.3 11.3 11.3	276 301 332	5.0 3.5 2.5	606 641 682	4.0 3.0 2.2

Note: Moduli are in ksi; strain in μ-strain

ε_{ac} = tensile strain in AC; ε_{sg} = vertical strain on top of subgrade

Nac = Number of repetitions of an 18-kip ESAL (based on Finn's fatigue relationship)

Nsg = Number of repetitions of an 18-kip ESAL (based on Chevron's failure relationship)

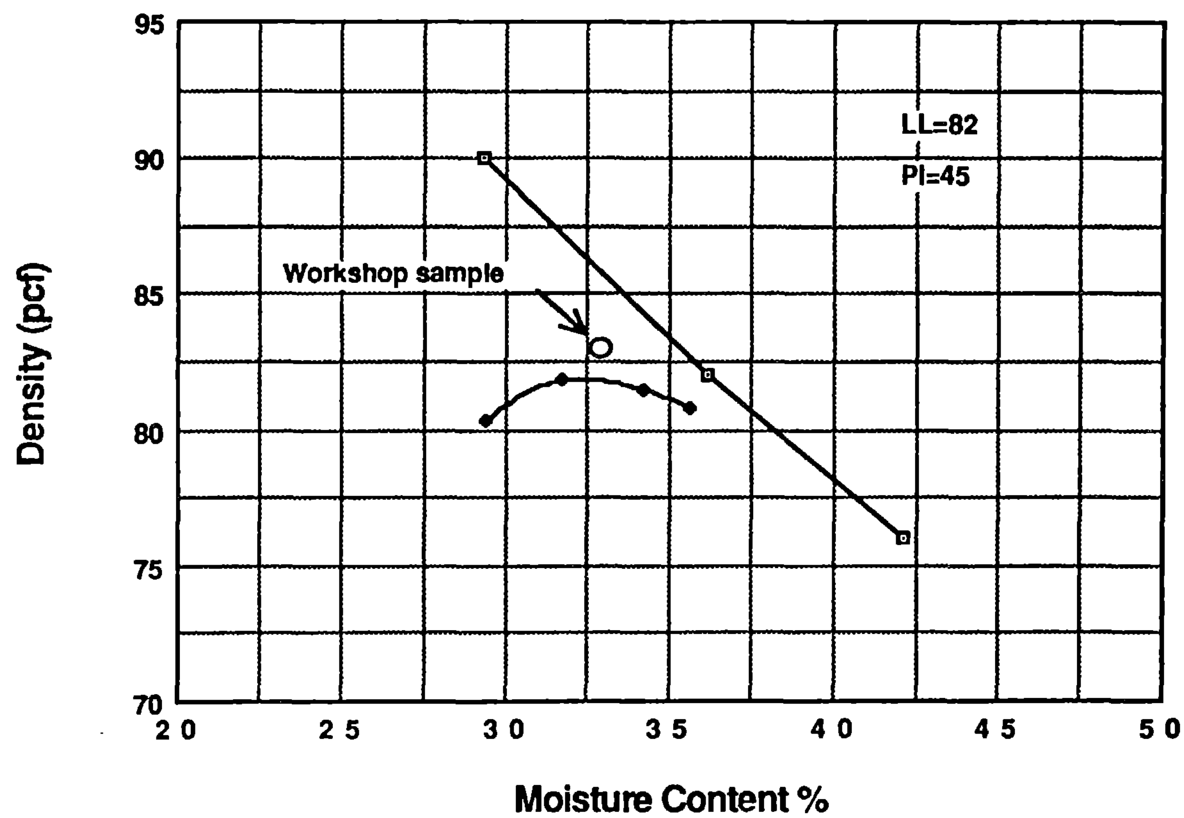
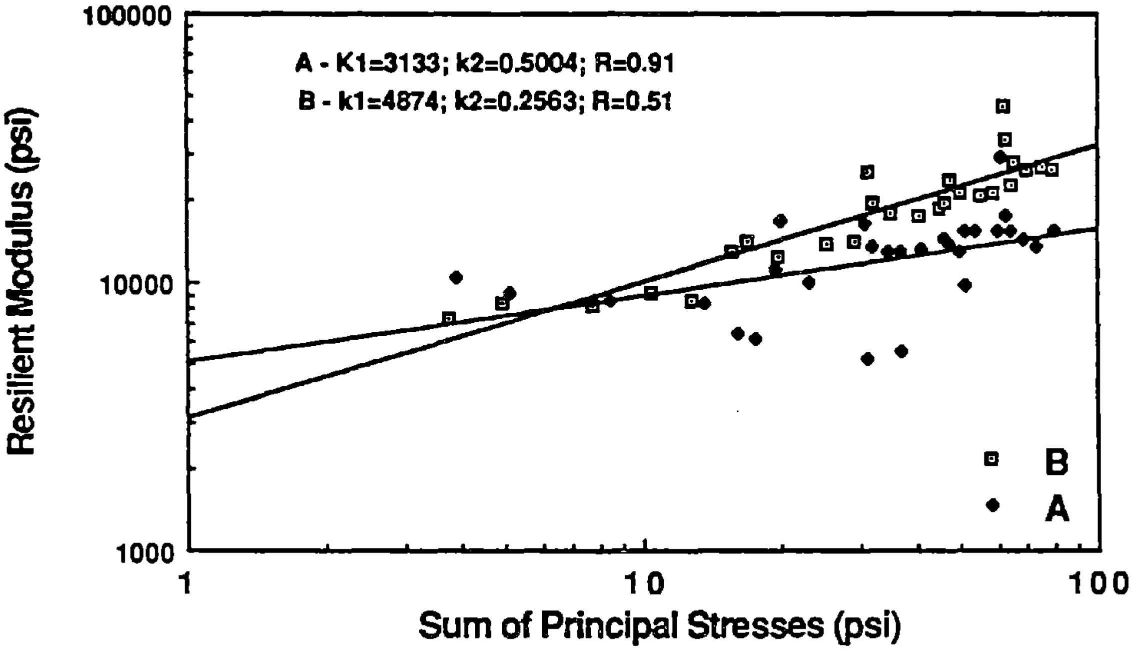
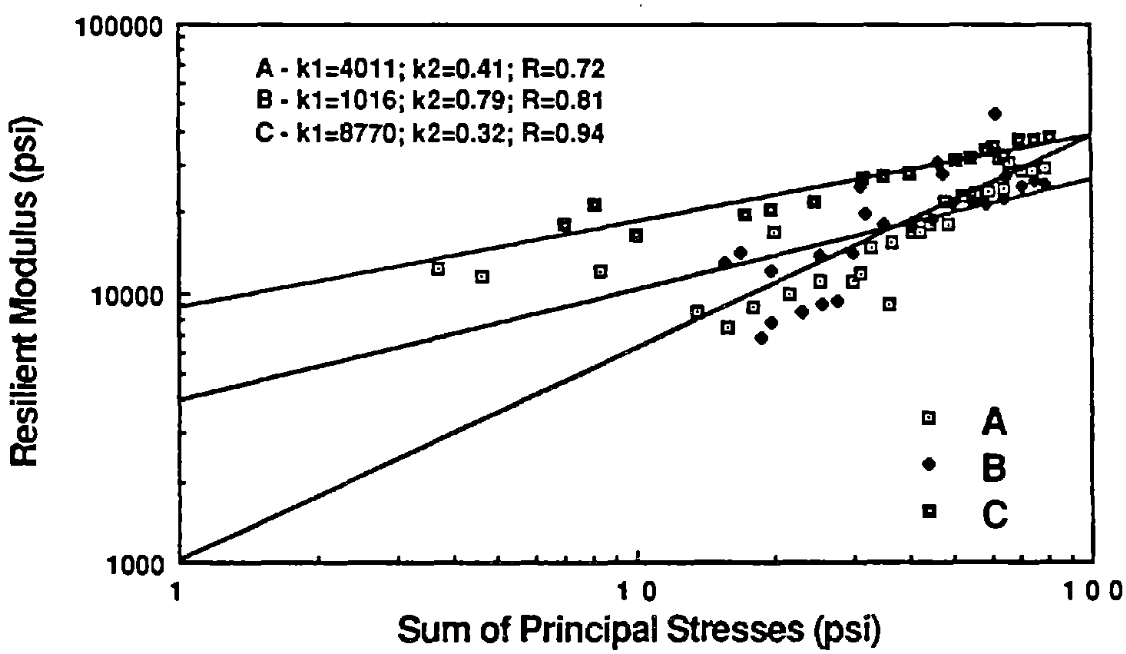


Figure 2 Density versus Moisture Content



a) Loose Condition



b) Dense Condition

Figure 4 Resilient Moduli of Granular Material Tested by Different Agencies

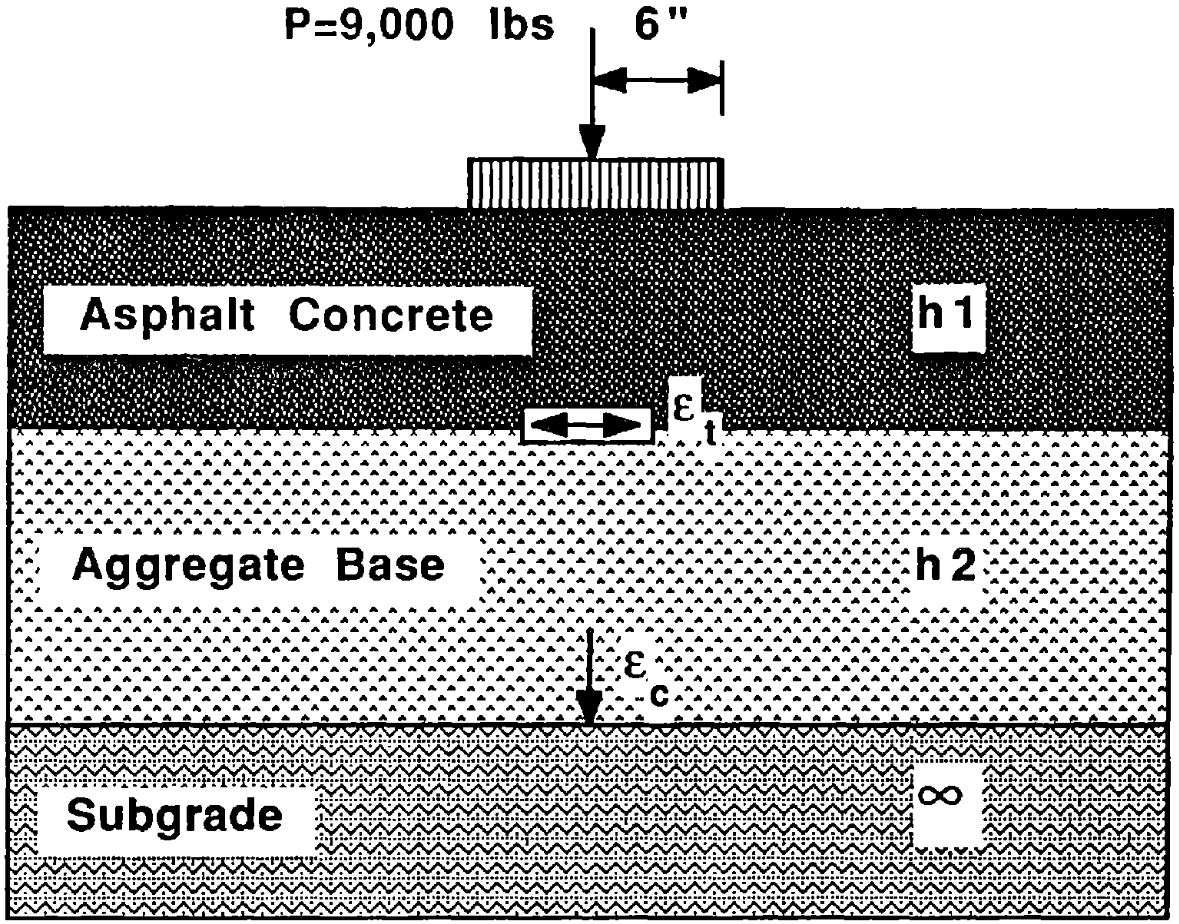


Figure 6 Pavement Structure Used for the Design Example

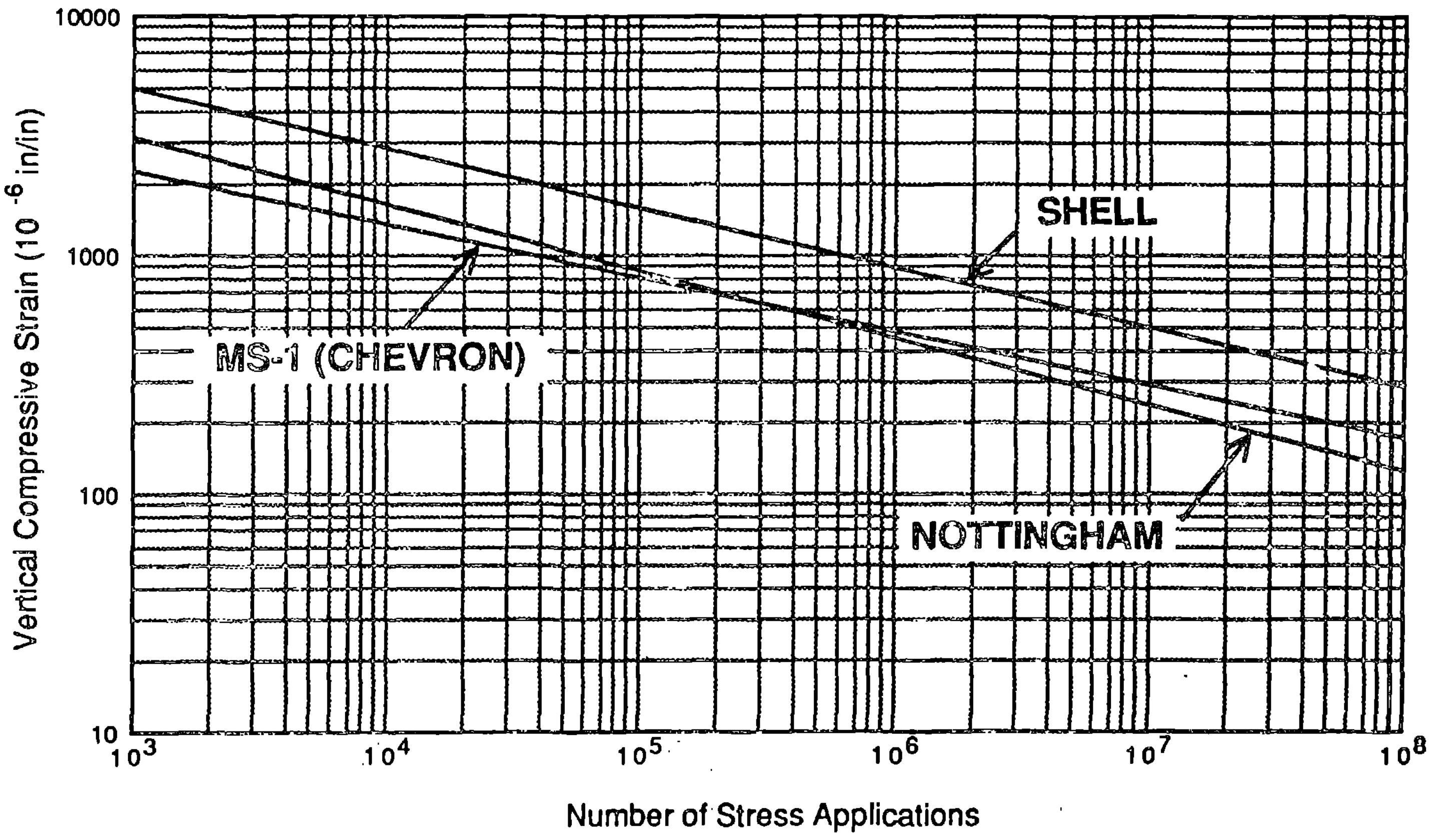


Figure 8 Vertical Compressive Strain versus Number of Stress Repetitions (3)

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Resilient Modulus Testing:

Interpretation of Laboratory Results for Design Purposes

by

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Paper prepared for Workshop on Resilient Modulus Testing to be held at Oregon State University, Corvallis, Oregon, March 28-30, 1989.

INTRODUCTION

In recent years, engineers have increasingly made use of multilayer elastic or viscoelastic analyses to estimate the response of pavement structures to both load and environmental influences, particularly to estimate the potential for fatigue cracking, permanent deformation in the form of rutting, and thermal cracking (e.g., References 1, 2, and 3).

Solutions for stresses, strains, and deflections in the layered systems representing pavement structures require the use of linear elastic characteristics (e.g., Young's modulus and Poisson's ratio) or linear viscoelastic characteristics (e.g., creep compliance and Poisson's ratio) for the materials comprising these systems. It must be recognized, however, that many pavement materials do not strictly satisfy the assumptions of linearity. Accordingly, ad hoc simplifications of materials response must be used.

One such simplification is to define the elastic characteristics of materials through repeated load testing and to develop relationships between applied stress and recoverable strain termed resilient moduli, the subject of this workshop. This paper attempts to provide a summary of some of the available information on the resilient characteristics of the various materials comprising pavement structures and the use of this information in pavement analyses.

Some guidelines are suggested for conditioning materials to insure that representative modulus values are used in specific analyses. Emphasis is placed on the resilient characteristics of fine grained soils and granular materials although some discussion of the resilient properties of asphalt bound and other treated materials is included.

STIFFNESS CHARACTERISTICS

In this section a summary of some of the available information on the stiffness characteristics of paving materials useful for pavement analyses is included. While emphasis is placed on the resilient characteristics of fine-grained soils and untreated granular materials as noted above, some information is included for asphalt mixtures and portland cement and lime treated materials.

Fine-grained soils

The stiffness characteristics of fine-grained soils are dependent on dry density, water content, soil structure, and stress level. For a particular condition, the stiff-

1. Modulus dependent on confining pressure (7):

$$M_R = K\sigma_3^n \tag{2}$$

where:

M_R = resilient modulus determined from repeated load test and equal to the quotient of σ_d and ϵ_a.

σ_3 = confining pressure

σ_d = deviator stress, $\sigma_1 - \sigma_3$

ϵ_a = recoverable axial strain

K, n = experimentally determined coefficients.

2. Modulus dependent on first stress invariant (7):

$$M_R = k_1 \theta^{k_2} \tag{3}$$

where

θ = $(\sigma_1 + \sigma_2 + \sigma_3)$ or $(\sigma_d + 3\sigma_3)$ in triaxial compressive test

k_1, k_2 = experimentally determined coefficients

3. Modulus dependent on mean normal stress, p, and deviator stress, σ_d or q (8):

$$M_R = F(p, q) \tag{4}$$

where (in triaxial compression):

$$\sigma_d \text{ or } q = \sigma_1 - \sigma_3$$

$$p = (\sigma_1 + 2\sigma_3)/3$$

$$\text{or } (\sigma_d + 3\sigma_3)/3$$

At this time it is recommended that the relationship

$$M_R = k_1 \theta^{k_2} \tag{3}$$

be used for design purposes; Fig. 8 illustrates test results from a triaxial compression repeated load test plotted using this relationship (9).

ness (termed resilient modulus herein) is dependent on the applied stress, i.e.,

$$M_R = f(\sigma_d) \tag{1}$$

where

σ_d = repeatedly applied deviator stress in triaxial compression test.

An example of this dependency is illustrated in Fig. 1 for a fine-grained soil.

The general dependency of soil modulus on water content and dry density (and presumably structure) for a specific subgrade material tested in the as-compacted condition is illustrated in Fig. 2.

Moduli of soils are also dependent on soil moisture suction. Data from tests on undisturbed specimens illustrating relationships between resilient moduli measured in laboratory repeated load tests and soil moisture suction are shown in Figs. 3 and 4 (4). The data in Fig. 4 illustrate that laboratory prepared specimens exhibit the same stiffness characteristics as field compacted specimens when specimens are compared at the same values of suction.

Freeze-thaw action also affects the stiffness characteristics of fine-grained soils. When the soil is frozen, the stiffness increases; upon thawing the stiffness is reduced substantially even though the water content has not changed. This type of response is illustrated in Figs. 5 (5) and 6 (6). (Note that in some situations it would be expected that the soil would become weak on thawing due to accumulation of ice lenses and the associated increase in water content). Such effects have been incorporated in the Asphalt Institute's design procedure as shown in Fig. 7 (2). It should be noted that a similar pattern of soil modulus change can also occur in areas without freezing and thawing but with seasonal fluctuations in precipitation.

Poisson's ratio for fine-grained soils generally varies in the range 0.3 to 0.5; the higher values are associated with degrees of saturation approaching 100 percent. For design purposes, a value of 0.35 to 0.4 appears reasonable.

<u>Untreated Granular Materials</u>

The stiffness characteristics of untreated granular materials are dependent on the applied stresses. A number of different expressions have been proposed to represent this stress dependency:

Mix stiffness can be measured by one of the techniques illustrated in Fig. 11 or estimated using the Shell (1) or Asphalt Institute (2) methodologies; alternatively standard values representative of specific mixes can be incorporated directly into design methodologies (e.g., Shell design procedure (1)).

When using repeated load equipment to determine moduli some consideration must be given to the loading conditions (time of loading and time interval between load applications). Resilient moduli (Fig. 11e) were determined on 4 in. diameter by 8 in. high specimens using the equipment shown in Fig. 12. The results were compared with dynamic moduli (Fig. 11d) determined on hollow cylindrical specimens 18 in. high, 9 in. outside diameter and 1 in. wall thickness (17). In both instances the computer is used to control the load; Fig. 12 illustrates this schematically for the repeated load testing. In this instance the computer is used to control the duration and frequency of the load, record the magnitude of the load and the recoverable deformation, and determine the resilient modulus, M_R defined by the expression:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{6}$$

where:

σ_d = axial stress repeatedly applied,

ϵ_r = recoverable axial strain.

Resilient modulus data obtained with this equipment are shown in Fig. 13 for tests at 11°C and 25°C. Modulus data obtained from tests on two specimens are shown to illustrate the repeatability of the test results. In this figure it will be noted that the stiffness modulus is dependent on time of loading for a fixed ratio of time that the load is applied to the time that the load is removed. For example, for a time of loading of 0.1 sec and the ratio of time off to time on of 10, a load would be applied every 1.1 sec (approximately 55 repetitions per minute).

When comparing the moduli determined by pulse loading (repeated loading) with moduli determined by sinusoidal loading, consideration must be given to both the time of loading and to the ratio of time off to time on in the repeated loading test. The latter ratio defines the amount of recoverable strain used to determine the modulus. The smaller the ratio, the less the recoverable strain and, therefore, the larger the values of M_R. This is illustrated in Fig. 14 for tests conducted at 25°C. It should be noted that the dependence of M_R on the recovery period

Table 1 contains a summary of the results of repeated load triaxial compression tests wherein the results were interpreted according to equation (3). More detailed information is available in Reference (10).

Reference (16) contains a summary of modulus values used in a number of design procedures around the world. These are summarized in Table 2. The data generally reflect the stress sensitivity characteristics of granular materials. For example, the modulus of untreated granular layers over "cemented" layers are larger than over untreated layers due to the higher stress state in the materials when stiffer layers are placed below them. Also, the values of the NITRR indicate that a higher modulus will be obtained in a granular layer directly below a surface treatment (chip seal) than in the same material beneath a layer of asphalt concrete since the stress state (from the wheel load) is larger in the former case.

Asphalt-Bound Materials

Whereas the stiffness characteristics of fine-grained soils are dependent on stress state (for a given condition of water content and dry density) the stiffness (modulus) characteristics of asphalt-bound materials (containing asphalt cements are dependent on time of loading and temperature and for a range of conditions can be considered sensibly independent of stress state[1]. This stiffness can be represented by an equation of the form:

$$S_{mix}(t, T) = \frac{\sigma}{\epsilon} \tag{5}$$

where:

$S_{mix(t, T)}$ = mixture stiffness for a particular time of loading, t, and temperature, T .

σ, ϵ = applied stress and resultant strain.

Fig. 9 illustrates the dependence of S_{mix} on both time of loading and temperature; Fig. 10 indicates ranges in stiffness which might be expected in different environments in the United States under moving wheel loads.

For mixes with dense-graded aggregates, limiting values of stiffness at short loading times and cold temperatures are about 4 to 5 × 10^6 psi.

[1]At higher temperatures (e.g., above 25°C) it is likely that the stress state will have an influence on the stiffness characteristics of these materials, this influence becoming more pronounced as the binder is less stiff.

$E_{T,t}$ = stiffness modulus at temperature T, and curing time, t.

$E_{T,f}$ = stiffness modulus in fully cured state at Temperature T.

$E_{T,i}$ = stiffness modulus in uncured (initial) state at temperature T.

RF_t = reduction factor representing the amount of cure at time t and defined as shown in Fig. 17 for a six-month cure period.

The stiffness modulus for the various stages of cure can be plotted as intermediate parallel lines between the initial and fully cured states.

Poisson's ratio for dense-graded asphalt concrete mixtures will vary from about 0.25 at cold temperatures and short loading times to 0.5 at high temperatures and/or long loading times.

Portland Cement and Lime-Stabilized Materials

Moduli of portland cement and lime-stabilized materials can be determined in direct compression, bending, or the diametral modes of loading. The values used for design purposes are those based on some degree of curing, e.g., 28 days.

There are a number of categories of portland cement stabilized materials. Each have distinct stiffness characteristics associated with them. The categories include: 1) lean concrete; 2) cement-treated aggregate; and 3) soil-cement (both granular and fine-grained soils). Table 3 lists representative values for both stiffness modulus and Poisson's ratio, as reported in the literature.

The data reported by Williams (20) in this table were determined by electrodynamic measurements. It should be noted that these stiffness values are associated with small strains and are about 10 to 15 percent higher than would be obtained with conventional static testing.

The data reported by the NITRR of South Africa (24) reflect two different phases of the performance of cement-bound materials, one where the material is intact (pre-cracked) and the other where the material has been cracked from repetitive loading (post-cracked). The second phase is further subdivided into the conditions where either a treated material or an untreated material exists above the cement-treated material, the latter condition being representative of a construction procedure used in South Africa in which the cement-bound layer is placed under a granular is also temperature dependent with the dependence decreasing as the temperature is reduced.

Comparisons of the moduli determined in dynamic loading and in repeated loading are given in Fig. 15. Essentially the same results are obtained at 11°C whereas at 25°C some differences exist in the moduli determined from the two modes of loading. For repeated loading data, reducing the ratio of time off to time on would increase the modulus values (as seen in Fig. 14) and bring these data closer to the values determined dynamically. Other investigators have addressed the problem as well, e.g., Brown and Cooper (18). These data emphasize the importance of selecting appropriate loading conditions to insure that reasonable stiffness values for analysis and design purposes will be obtained from laboratory tests.

Stiffness moduli for asphalt emulsion treated mixtures utilized in pavement structures, particularly as treated base courses, can also be determined from repeated load tests as recommended by Chevron Research (19). A special chart has been developed to provide a linear relationship between mix stiffness and temperature, Fig. 16.

For design purposes Chevron Research has developed stiffness data for three groups of emulsion treated materials; the data have been incorporated into the design procedure of the Asphalt Institute (2) for the following categories of emulsion-treated base:

- Type I. Emulsified asphalt mixes with processed dense-graded aggregates.
- Type II. Emulsified asphalt mixes with semi-processed, crusher-run, pit-run, or bank-run aggregates.
- Type III. Emulsified asphalt mixes with sands or silty sands.

Stiffness vs. temperature data are shown in Fig. 16 for Mix Type I.

It is possible to consider the effects of the emulsion mix on pavement performance. In Fig. 16 is also plotted stiffness data for the mix in the uncured state. To obtain the stiffnesses at intermediate stages. the following expression can be utilized:

$$E_{T,t} = E_{T,f} - (E_{T,f} - E_{T,i})(RF t)$$

where

TESTING CONSIDERATIONS

In general, it is important to obtain representative samples of the specific pavement materials to: (1) prepare them to conditions which will be representative of those existing in the proposed pavement structure; (2) measure their response under representative loading conditions; and (3) obtain some measure of the variability which can be anticipated in these characteristics by conducting a sufficient number of tests at the estimated conditions.

This section contains a brief discussion of some of these considerations for fine-grained soils, untreated granular materials, and asphalt-bound materials.

Fine-Grained Soils

Random sampling of subgrade materials for test purposes is recommended (28). Classification tests are useful to identify sections for the detailed testing as well as to identify potential problems. For example, the Atterberg limits are useful to identify whether or not a fine-grained soil may be expansive. Research by Seed, et al (29) provides a relatively simple guide to identification of soils with potential volume change problems by means of the Plasticity Index. Swelling potential under low surcharge (approximately one psi) according to their study is as follows:

Degree of Expansion	Swelling Potential - percent	Plasticity Index PI
Low	0 to 1.5	less than 10
Medium	1.5 to 5	10 - 25
High	5 to 25	25 - 45
Very High	greater than 25	greater than 45

Pressure applied by typical pavement structures will be in the range one to three psi.

The Atterberg limits (for soil classification) together with a measure of the grain size distribution (percent finer than 0.02 mm) also provide an indication of the potential for frost effects (e.g., Reference (30).

An understanding of soil compaction is extremely important for the designer to ensure that soils will be properly conditioned in the laboratory for pavement design testing. In general, soils are compacted to achieve improvement in one or a number of soil properties including: 1) compreslayer (which is very well compacted) and a comparatively thin layer of an asphalt-bound material is used as the surfacing.

It should be noted that a considerable range in stiffnesses is possible for cement-treated materials. As presented by Mitchell (25) the moduli may range from the order of 10,000 psi to several million psi, depending on soil type, treatment level, curing time, water content, and test conditions.

Poisson's ratio is generally in the range 0.1 to 0.2 for cement treated granular materials (although the NITRR recommends a value of 0.35 for design, Table 3) and in the range 0.15 to 0.35 for cement-treated fine-grained soils.

Like portland cement stabilized materials, the stiffness characteristics of lime stabilized materials are dependent on soil type, treatment level, curing conditions, etc. There are two general categories of lime treated materials used for pavements: 1) fine-grained soils treated with hydrated lime or quick lime, termed soil-lime; and 2) granular materials treated with a mixture of lime and flyash (approximately four parts of flyash to one part of lime) termed lime-flyash.

Moduli can be determined by repeated load testing or estimated from other measurements for design purposes. For example, for cured soil-lime mixtures, the compressive stiffness modulus, E_c is related to unconfined compressive strength. Thompson has suggested the following general form (26):

$$E_c(\text{psi} \times 10^3) = k_1 + k_2 \text{ (UC)} \tag{8}$$

where:

k_1 and k_2 = laboratory determined coefficients and equal to 9.98 and 0.124 respectively.

Poisson's ratio for soil lime is in the range 0.1 to 0.25 and a value on the order of 0.15 to 0.20 is recommended for design.

For lime-flyash mixtures, moduli are primarily dependent on aggregate gradation (fines, content), curing conditions, and mixture proportions. Available data indicate that for fully cured mixtures, the moduli range from 0.5×10^6 psi to 2.5×10^6 psi with the lower values associated with higher fines contents (27). Poisson's ratio is typically in the range 0.1 to 0.15.

a soil stiffness modulus such as the resilient modulus, M_R. Dispersed structures will exhibit more plastic and resilient (recoverable) or "elastic") strain under repetitive loading than flocculated structures. Thus, laboratory compaction procedures used to prepare specimens for M_R testing should be selected in accordance with field compaction conditions.

As stated above, knowledge of the field compaction conditions are required. Generally, any compaction process which causes shearing deformations in cohesive materials whose degrees of saturation are greater than about 80 percent (i.e., to the right of the line of optimums) will result in materials with <u>dispersed structures</u>. As noted earlier, a material with a dispersed structure will exhibit greater elastic (recoverable) deformation than would the same material under identical conditions of water content and dry density with a <u>flocculated</u> structure when the materials are tested under the same stress conditions. Fig. 2 illustrates the general variation of resilient modulus with initial compaction conditions. Note in Fig. 2 that in order to obtain a given value for the resilient modulus (M_R) the dry density must increase as the molding water content increases.

Some general criteria which can be used to guide selection of the appropriate compaction conditions for clay soils are as follows:

(1) If field compaction conditions will be at a water content less than 80 percent of the saturation water content and the in-service water content is expected to remain less than this value, then any of the standard compaction tests, including impact, kneading, static, or gyratory may be used to simulate the in-service condition.

(2) If field compaction conditions will be at a water content corresponding to more than 80 percent of the saturation water content and the in-service water content is expected to remain at least at this value, then the compaction process must be one which induces shearing deformation during the compaction process-impact, kneading, or gyratory.

(3) If field compaction conditions will be at a water content less than 80 percent of saturation water content and the in-service conditions are expected to exceed this value, then static compaction should be used to simulate the soil structure (flocculated).

sibility; 2) strength; 3) volume change (swell and shrinkage) characteristics; 4) resilient response; and 5) frost heave potential.

For granular materials, compaction requirements are relatively straightforward. To achieve the most desirable situation with respect to the properties listed above requires that as high a density as possible be obtained in the material in the field. For fine-grained soils, however, such is not the case since proper compaction may involve more than a requirement for minimum acceptable density.

Fig. 18 illustrates a general relationship between dry density, compaction water content, and compactive effort for fine-grained soils. For a particular compaction method (e.g., laboratory impact compaction, AASHTO T180), the locus of points of maximum dry density and optimum water content is termed the line of optimums. This line usually lies between the 80 and 90 percent degree of saturation lines, as seen in Fig. 18. Location of the line of optimums is useful, as will be seen subsequently, in defining compaction conditions for pavement design purposes. It should be noted that compaction of the same soil in the field by pneumatic-tired rollers will produce a line of optimums slightly to the right (1.5 to 2 percent water content increase) of the laboratory determined line of optimums using impact compaction.

For fine-grained soils, the structure of the soil, as well as water content and dry density, affect its response. Fig. 19 provides an indication of the arrangement of the plate-like clay particles depending on compaction conditions (31). At water contents dry of optimum for a particular compactive effort (i.e., at A and E) the particles are arranged in a random array termed a "flocculated" structure. Wet of the line of optimums, provide shearing deformation are induced during compaction, parallel particle arrangements which are termed "dispersed" structures (i.e., at C and D) are developed. These dispersed and flocculated compacted soil structures lead to differences in mechanical properties for specimens assumed to be at the same water content and dry density.

If a soil is determined to be expansive, e.g., as measured by the Plasticity Index, compaction of the soil to the right of the line of optimums will tend to reduce its propensity for expansion (32) because the dispersed structure produced tends to result in less expansion than the flocculated structure (obtained to the left of the line of optimums) with increase in water content.

These considerations are important in preparing samples of fine-grained soils for laboratory testing to measure

$$M_R = A\sigma_d{}^b \tag{10}$$

which is one form of the general relationship introduced as equation (1).

If the soil conditions can be reasonably defined (e.g., based on suction estimates) a relatively limited range in water contents and dry densities can be used for stiffness testing As shown schematically in Fig.22, if the range of compaction conditions and the range of in-service conditions are known, an appropriate laboratory compaction method can be selected and test specimens prepared within the in-service range shown in Fig. 22.

On the other hand, if the conditions are not clearly defined, specimens should be prepared and tested over a substantial range in dry densities and water contents. Results can be presented as shown in Fig. 2, and the resilient modulus data can be used in conjunction with other characteristics, e.g., swell considerations, to select the range of field placement conditions (and, of course, the design modulus.

<u>Untreated Aggregates</u>

As with subgrade materials, it is important to obtain representative samples of the untreated aggregates to be used as base and subbase layers for laboratory determinations of stiffness. In addition, estimates must be obtained of the dry density and water content conditions anticipated in-situ since the stiffness moduli characteristics of granular materials are dependent on these parameters, Fig. 23.

The stiffness modulus characteristics of untreated granular mixtures, as seen earlier, are dependent on applied stress; equation (3) represents a useful form in which the results of stiffness determination can be presented.

While method of compaction is important for fine-grained soils because of soil structure considerations, the primary factors affecting the stiffness characteristics of granular materials are water content (degree of saturation) and dry density. Accordingly, any method of compaction which produces the desired dry density is suitable. Vibratory compaction has, for example, been used successfully (e.g., Reference (<u>11</u>)).

To develop the influence of stress state on stiffness modulus, specimens should be tested at confining pressures in the range 1 to 20 psi. For each confining pressure, a

As an illustration, consider the situation for a sample prepared by kneading compaction and soaked to a condition representative of that expected at some subsequent time after placement. Resilient response is shown in Fig. 20. If the designer were to prepare the sample to the same initial conditions by kneading compaction to save time in the laboratory (since it takes considerable time for a fine-grained material to become saturated), a different result would be obtained. On the other hand, if the soil was prepared by static compaction to the same conditions, essentially the same result would be obtained as for the situation where the sample was prepared "dry" by kneading compaction and soaked to the particular state. In this case, static compaction wet of the line of optimums creates essentially the same structure as kneading dry of the line of optimums.

When testing fine-grained (cohesive) subgrade soils (e.g., AASHTO T274), the stress conditions in-situ must be taken into account. As seen earlier, the resilient moduli of such soils are dependent on applied stress, Fig. 1.

Confining pressure has a relatively small influence on the resilient modulus and will normally be in the range 1 to 5 psi. This pressure is due to the overburden (pavement thickness) and the applied loads.

The overburden pressure, σ_3, can be estimated as follows:

$$\sigma_3 = k_o \cdot \gamma_{ave} \cdot z \tag{9}$$

where:

k_o = earth pressure at rest, usually 0.5,

γ_{ave} = average unit weight of material

z = depth to point under consideration.

The horizontal stress resulting from load is usually small (usually less than one psi).

The applied or deviator stress varies depending on the load. It is recommended that testing be done in the range 0.5 to 15.0 psi. Stress conditioning, as called for in AASHTO T274, is important prior to the repeated load testing for modulus determination.

Test results can be plotted as shown schematically in Fig. 21. At times the logarithmic form of plotting may be more convenient to permit definition of the coefficients A and B in the expression:

Thus, in analyses where variability in material properties are considered, Table 4 provides at least a general indication of what to expect, depending on the degree of control exerted during the construction process.

Reliability

Reliability may be defined as the probability that the pavement will perform satisfactorily for the expected traffic and environmental conditions for which it was designed.

In the analytically-based approach to design, it is possible to consider reliability in the design process by recognizing variability in material properties such as those summarized in Table 4. In addition, it is possible to consider different acceptable values for various distress criteria, depending upon the importance of the highway facility (e.g., as indicated by traffic intensity). Essentially, by adopting more conservative values for a particular parameter, larger pavement thicknesses will be required -- with the result that a higher probability of carrying some specific amount of traffic will be achieved.

For example, in The Asphalt Institute design procedure (2) the value of subgrade modulus is selected depending on traffic level. If P_o = percent of project overdesigned and P_u = percent of project underdesigned, then:

$$P_o = 100 - P_u. \tag{11}$$

Fig. 24 illustrates how P_o varies with traffic intensity. For example, at a traffic level equal to 10^6 repetitions of an 18,000 lb axle load, a value for P_o = 87.5 percent is selected. Thus, it would be expected that only 13.5 percent of the project would be underdesigned, since the design value would be obtained from:

$$\begin{matrix}\text{Design value for} \\ \text{subgrade modulus}\end{matrix} = \begin{matrix}\text{Mean value} \\ \text{of modulus} \\ \text{(from tests)}\end{matrix} - A\begin{pmatrix}\text{standard} \\ \text{deviation}\end{pmatrix} \tag{12}$$

where:

A = adjustment factor based on normal probability theory and obtained from Fig. 25.

In general, for heavier traffic volumes where the consequences of early distress could be costly, the higher the value for p_o which is used for design purposes.

range in deviator stresses should be used varying from 1 to 5 times the confining pressure. As with fine-grained soils, stress conditioning prior to determination of stiffness modulus values is important. Research by Hicks (11) has shown that one specimen can be used for the entire stress sequence so long as proper conditioning has been accomplished. Results of a typical set of tests have already been presented in Fig. 8.

Asphalt-Bound Materials

To measure the stiffness characteristics of asphalt concrete for design purposes, the method of specimen compaction for laboratory testing should produce deformations in the compaction process similar to those obtained by compaction in-situ. Accordingly, for laboratory specimen preparation it is recommended that one of the following methods of compaction be utilized:

> For cylindrical specimens: Kneading or gyratory compaction (e.g., ASTM D3496).
>
> For beam specimens: Kneading compaction (34) or preparation of slabs using rollers and sawing specimens from slabs (35).

References (35) and (36) provide substantiating data for these recommendations.

VARIABILITY AND RELIABILITY CONSIDERATIONS

Variability

Variability must be recognized in the design process, including variability in:

(1) traffic estimates,

(2) layer thicknesses due to construction limitation, and

(3) material properties (due to inherent variability and that resulting from the construction process).

Relative to material property and layer thickness variability, a number of researchers have examined available data to provide useful guidelines (e.g., (37, 38). Witczak, et al (38) have evaluated considerable data for portland cement concrete, asphalt concrete, subgrade materials, and cement-treated materials; Table 4 summarizes recommendations for coefficients of variation for material properties contained in Reference (38).

SUMMARY

In this paper a summary of the stiffness characteristics of a number of materials comprising pavement sections has been presented. The importance of insuring that traffic loading conditions (e.g., time of loading and stress state) and environmental influences be considered when selecting the actual stiffness values for pavement design and analysis purposes has been stressed. Moreover, emphasis has been placed on insuring that the stress vs deformation characteristics of laboratory prepared specimens are representative of the materials in-situ.

For fine-grained soils this requires that soil structure as well as water content and dry density correspond to in-situ conditions when laboratory stiffness modulus determinations are made. Similarly, asphalt concrete mixtures require that a compaction procedure (e.g., kneading) which will simulate the movement of particles, asphalt film thicknesses at points of aggregate contact, and grain arrangement, be utilized so that specimens with stress vs deformation characteristics similar to the same materials in-situ will be produced.

With these characteristics as well as appropriate distress characteristics for specific components of the pavement sections, it is possible to estimate the propensity for pavement distress such as fatigue and rutting in the asphalt-bound layers using analytically based procedures. Moreover, this type of approach provides increased reliability in performance estimates.

While simplifications may appear desirable at times, the engineer should be cautious in adopting single values for material parameters as a means to control specific forms of distress (39). For example, to control rutting in the asphalt concrete, some investigators have suggested a minimum stiffness value for asphalt concrete mixtures at a specific temperature, time of loading, and stress level. Without other controls on the mixtures, such limiting stiffness values may not guarantee satisfactory performance.

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TABLE 1 - SUMMARY OF REPEATED LOAD TRIAXIAL COMPRESSION LABORATORY TEST DATA FOR UNTREATED GRANULAR MATERIALS.

Investigator(s)	Material(s)	k_1	k_2
Hicks (11)	Partially crushed gravel; crushed rock	1,600 - 5,000	0.57 - 0.73
Hicks, Finn (12)	Untreated base - San Diego Test Road	2,100 - 5,400	0.61
Allen (13)	Gravel, crushed stone	1,800 - 8,000	0.32 - 0.70
Kalcheff and Hicks (14)	Crushed stone	4,000 - 9,000	0.46 - 0.64
Boyce, Brown, Pell (15)	Well graded crushed Limestone	8,000	0.67
U.C. Berkeley (9)	In service base and subbase materials	2,900 - 7,750	0.46 - 0.65

$M_R = k_1 \theta^{k_2}$ where M_R and θ are in psi units.

TABLE 2 - REPRESENTATIVE MODULI UNTREATED GRANULAR MATERIALS

Organization	Material		Modulus, ksi
Belgium	Stone base Subbase		72.5 29.0
Czechoslovakia	Subbase		21.8
Italy	Granular material		36.3
U.S.A., FHWA	AASHO Base: Spring Other seasons		 30.0 40.0
	AASHO Subbase Spring other		 15.0 20.0
South Africa, NITRR	Overlaying Cement Stabilized Layer		
	High quality crushed stone	Range	36 - 130
		Design value	65
	Crushed stone/ Natural gravel	Range	29 - 116
		Design value	51
	Gravel base	Range	25 - 102
		Design value	51
	Gravel subbase	Range	21 - 65
		Design value	36
	Overlaying Untreated or Cracked Stabilized Layer		
	High quality crushed stone	Range	25 - 87
		Design value	29*
	Crushed stone/ Natural gravel	Range	15 - 65
		Design value	29*
	Gravel base	Range	15 - 65
		Design value	29*
	Gravel subbase	Range	11 - 58
		Design value	29*
	Gravel subbase (lower quality)	Range	7 - 44
		Design value	21*

* The values shown are for bases or subbases under asphalt concrete. For base courses directly under surface treatments higher values are used; e.g., in the case of the crushed stone base a design modulus of 36 ksi (vs. 29 ksi) is recommended (reference (16)).

TABLE 3 - STIFFNESS CHARCTERISTICS OF PORTLAND CEMENT STABILIZED MATERIALS

	Material	Unconf. comp. str. at 7 days psi	Modulus of rupture at 28 days psi	Stiffness modulus, psi: Range	Stiffness modulus, psi: Recommended design value	Poisson's ratio
Williams - Great Britain (20)	Lean concrete		250	4.0 to 5.5 x 10^6	4.75 x 10^6	
			225	3.75to5.25 x 10^6	4.50 x 10^6	
	Cement-treated granular matl		175	2.0 to 3.75x10^6	2.75 x 106	
	Soil cement		175	0.5 to 3.0 x 10^6		
Belgium Road Research (21)	Lean concrete		220		2.2 x 10^6	
Bolk - Netherlands (22)	Sand cement				2.2 x 10^6	0.25
Autret, et al, France (23)	Cement-treated granular matl				2.9 x 10^6	0.25
	Sand cement				1.75 x 10^6	0.25
NITRR, South Africa (24)					Precracked Phase	
	Cement-treated:					
	Crushed stone	870-1740			2.0 x 10^6	0.35
	Stone/gravel	435-870			1.2 x 10^6	0.35
	Gravel (base quality)	220-430			0.87 x 10^6	0.35
	Gravel (sub-base quality)	110-215			0.50 x 10^6	0.35
					Post-cracked Phase	
					a) under treated (bound) layer	
	Crushed stone				0.22 x 10^6	
	Stone/gravel				0.14 x 10^6	
	Gravel (base quality)				0.11 x 10^6	
	Gravel (sub-base quality)				0.073 x 10^6	
					b) under untreated layer	
	Crushed stone				0.17 x 10^6	
	Stone/gravel				0.11 x 10^6	
	Gravel (base quality)				0.073 x 10^6	
	Gravel (sub base quality)				0.044 x 10^6	

TABLE 4 - VARIABILITY RECOMMENDATIONS*

	Coefficient of Variation (Percent)		
	Low	Average	High
1. SUBGRADE MATERIALS			
Modulus of Subgrade Reaction	10 - 20	20 - 35	35 - 50+
Resilient Modulus	10 - 20	20 - 35	35 - 50+
CBR	15 - 22	23 - 31	32 - 40+
2. CEMENT-TREATED MATERIALS			
Thickness	6 - 10	11 - 15	16 - 19+
Modulus of Elasticity	53 - 63	63 - 73	73 - 83
3. ASPHALT CONCRETE MATERIALS			
Thickness	1 - 5	5 - 10	10 - 15
Dynamic Modulus			
Temp: 40°F	8 - 10	11 - 13	14 - 16+
70°F	10 - 12	13 - 15	16 - 19+
100°F	18 - 20	21 - 22	23 - 24+
Base Modulus of Elasticity	25 - 35	35 - 45	45 - 55+
Flexural Stiffness			
Temp: 40°F	15 - 20	20 - 25	25 - 28+
68°F	20 - 23	24 - 26	27 - 30+
Poisson's Ratio	35 - 48	49 - 62	63 - 75
4. PCC MATERIALS			
Thickness	1 - 3	4 - 6	7 - 9+
Modulus of Elasticity	20 - 30	30 - 40	40 - 50+
Poisson's Ratio	8 - 12	13 - 16	17 - 20+
Modulus of Rupture	10 - 13	14 - 17	18 - 20+

*After Witczak, et al (38).

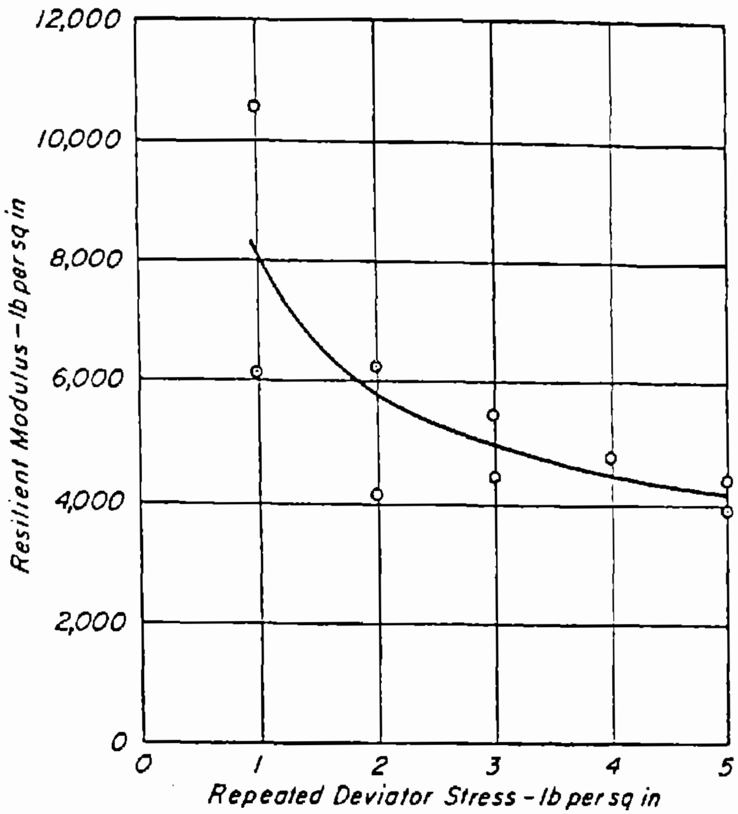


Fig. 1 - Results of repeated load tests, subgrade soil.

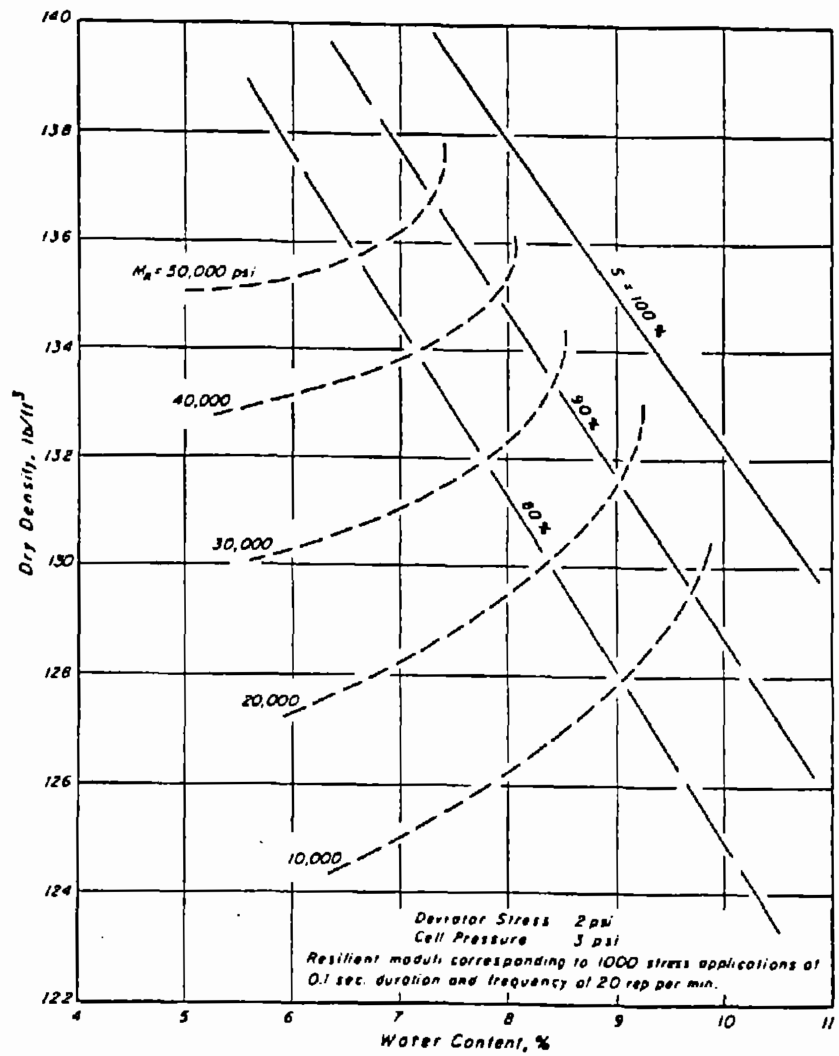
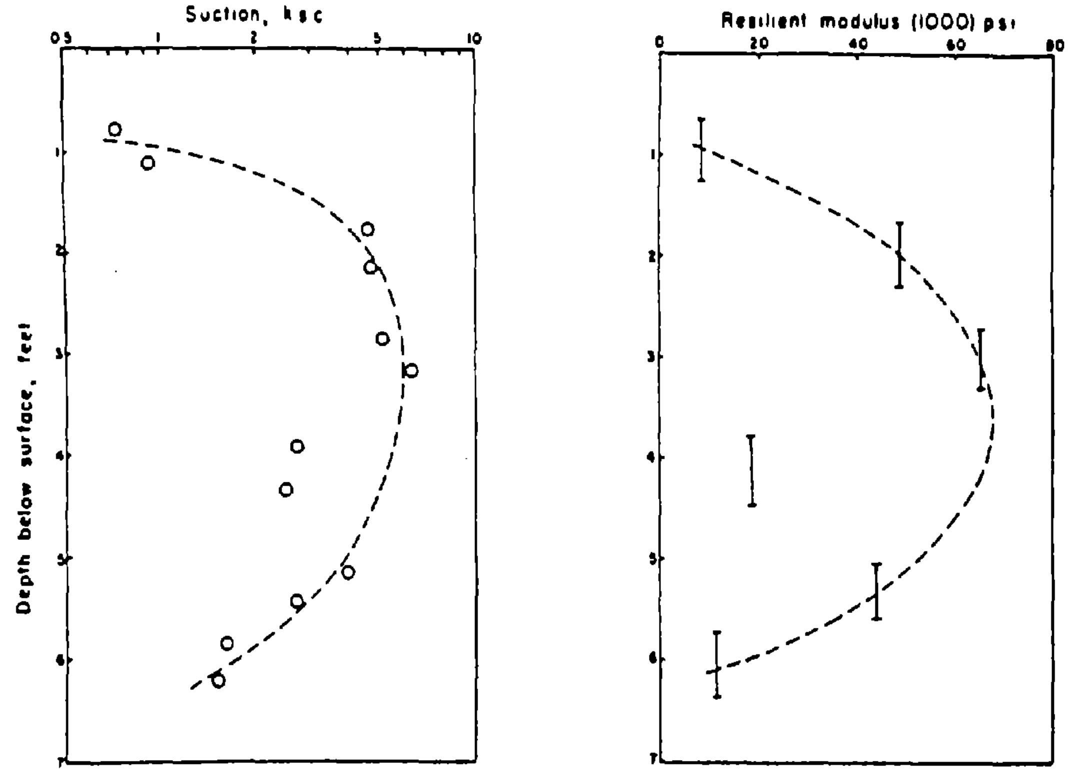
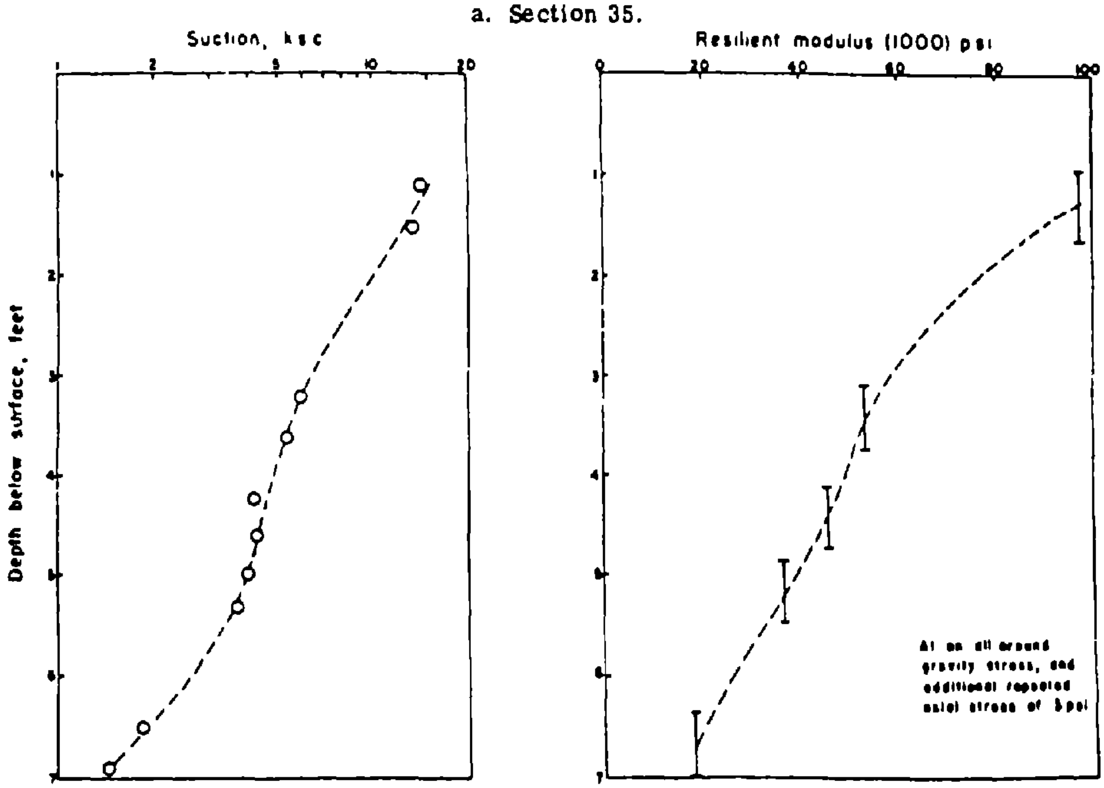


Fig. 2 - Water content - dry density - resilient modulus relationship for subgrade soil.



a. Section 35.



b. Section 2.

Fig. 3 - Relationships between suction and resilient modulus (Reference (4)).

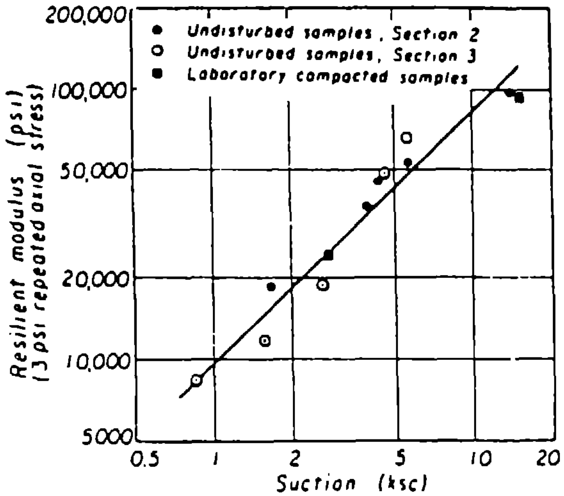


Fig. 4 - Relation between resilient modulus and suction, subgrade soil.

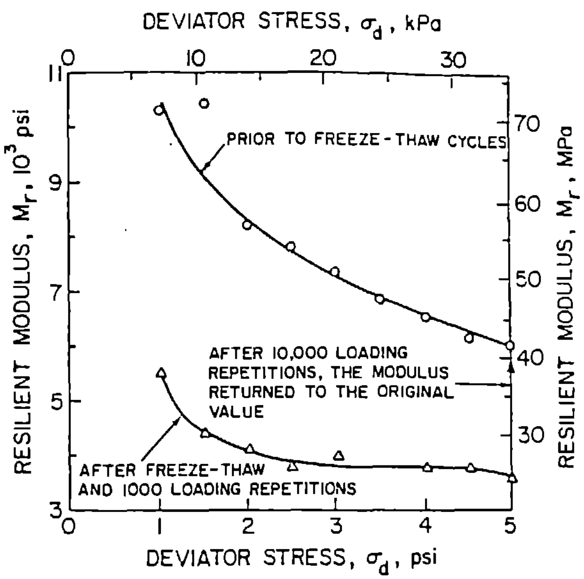


Fig. 5 - Resilient modulus test results before and after freeze-thaw for undisturbed Regina clay (Reference (5)).

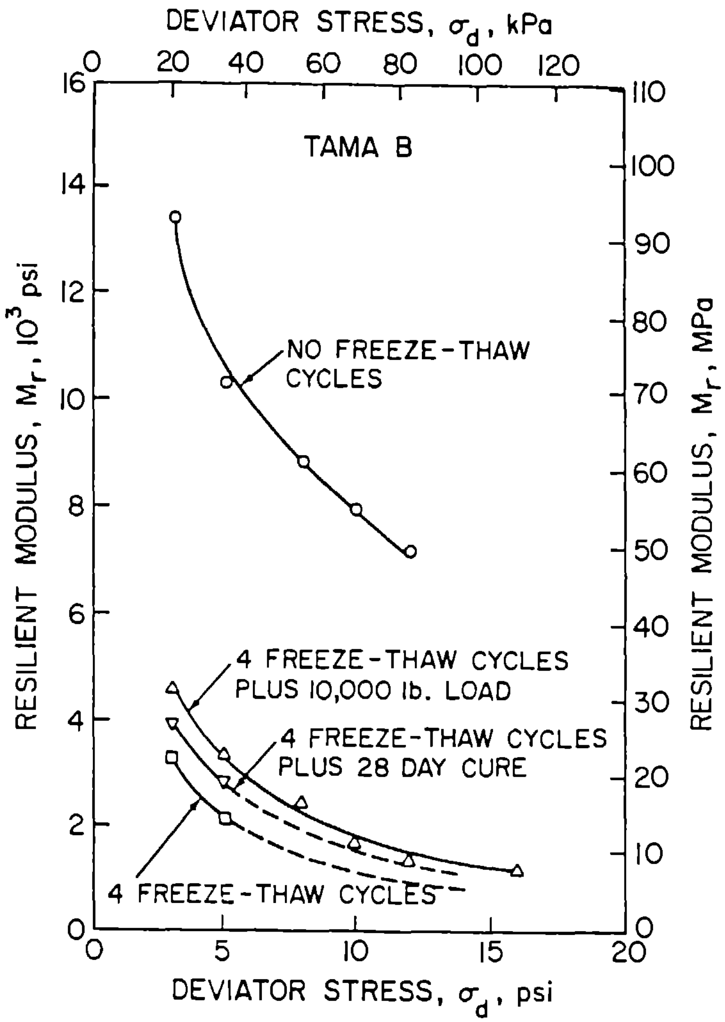
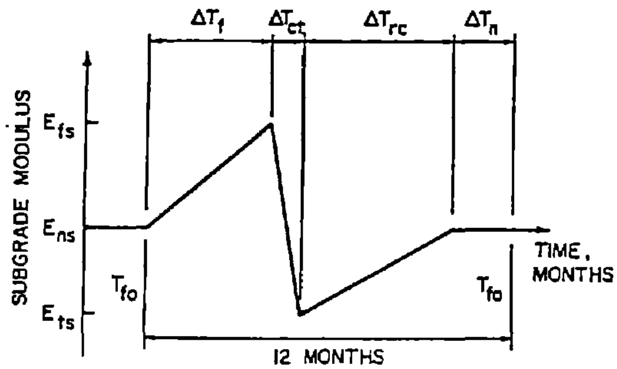


Fig. 6 - Effect of freeze-thaw, additional loading, and additional curing on resilient response of a natural Tama B soil (Reference (6)).



$E_{ts} = r_t E_{ns}$

E_{fs} = Frozen subgrade modulus

E_{ns} = Normal subgrade modulus

E_{ts} = Thaw (reduced) subgrade modulus

r_t = Thaw reduction factor

T_{fo} = Month freeze started

ΔT_f = Time of freeze

ΔT_{ct} = Time of critical thaw

ΔT_{rc} = Time of thaw recovery

ΔT_n = Time-normal subgrade condition

Fig. 7 - Representation of subgrade stiffness (Modulus) variations throughout year (Reference (2)).

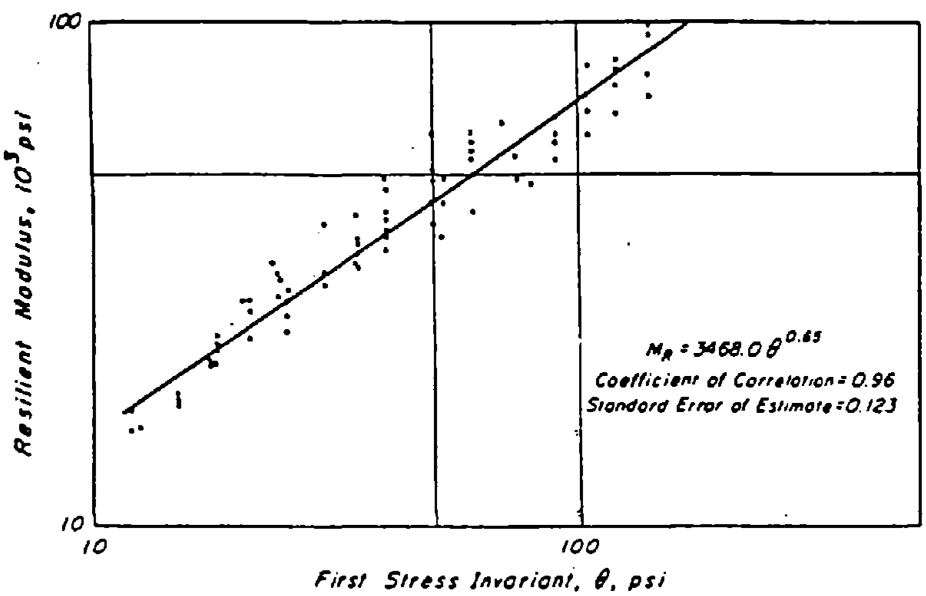


Fig. 8 - Individual test results of modulus vs sum of principal stresses, base course aggregate (Reference (9)).

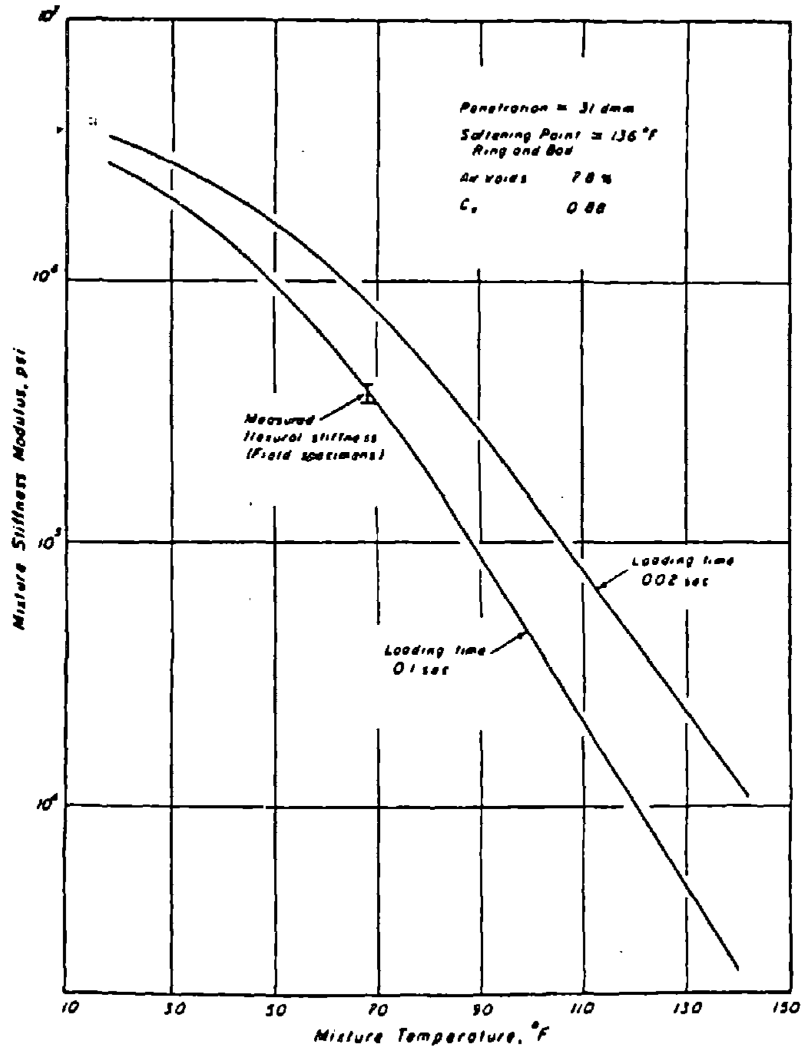


Fig. 9 - Computed relationships between mixture stiffness and temperature - asphalt concrete.

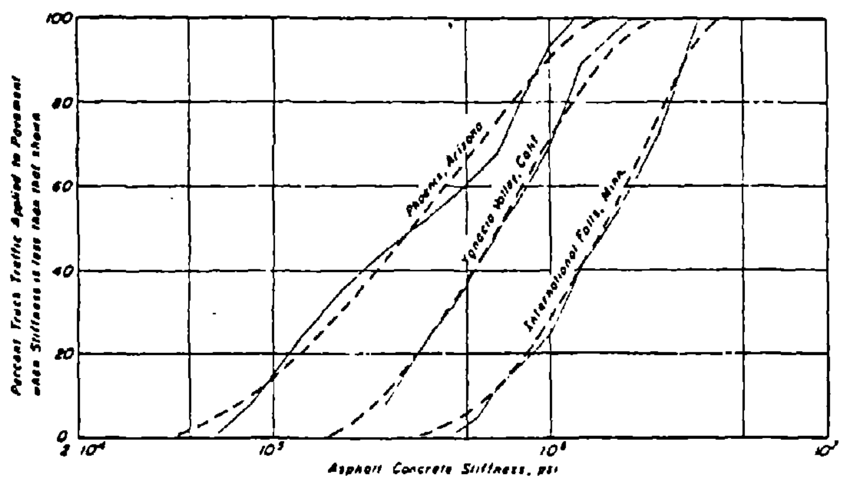


Fig. 10 - Stiffness variations with traffic applications - 12-in. thick asphalt concrete layer.

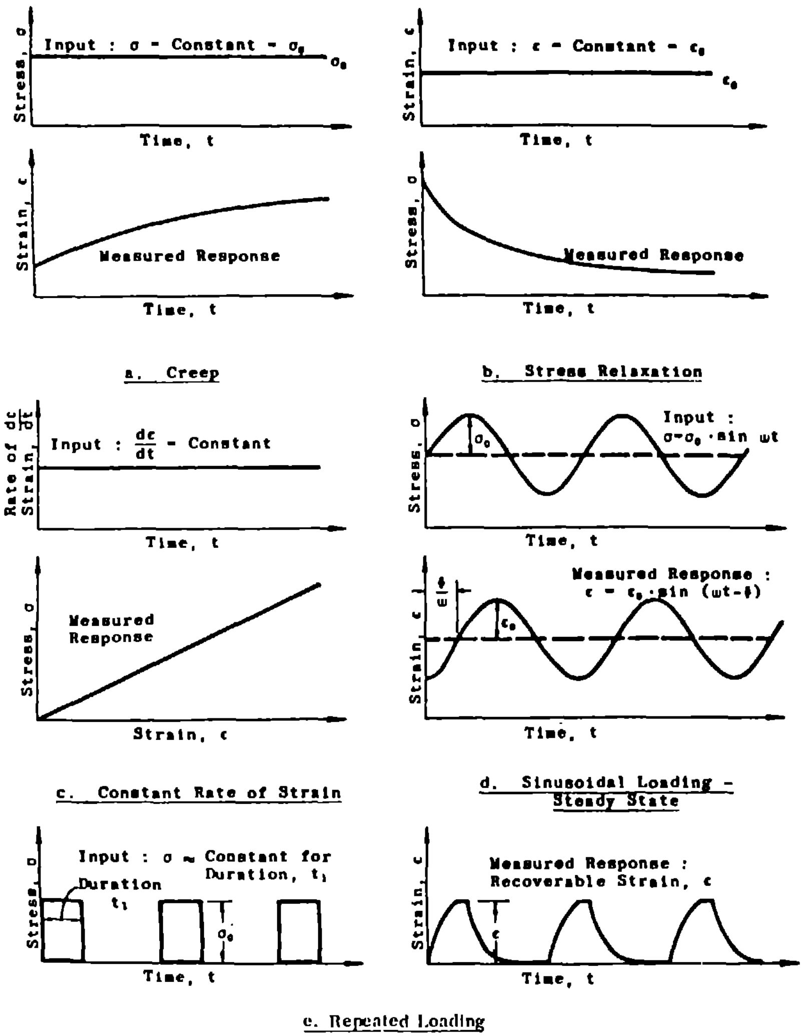


Fig. 11 - Types of loading to mesure stiffness characteristics of asphalt mixtures.

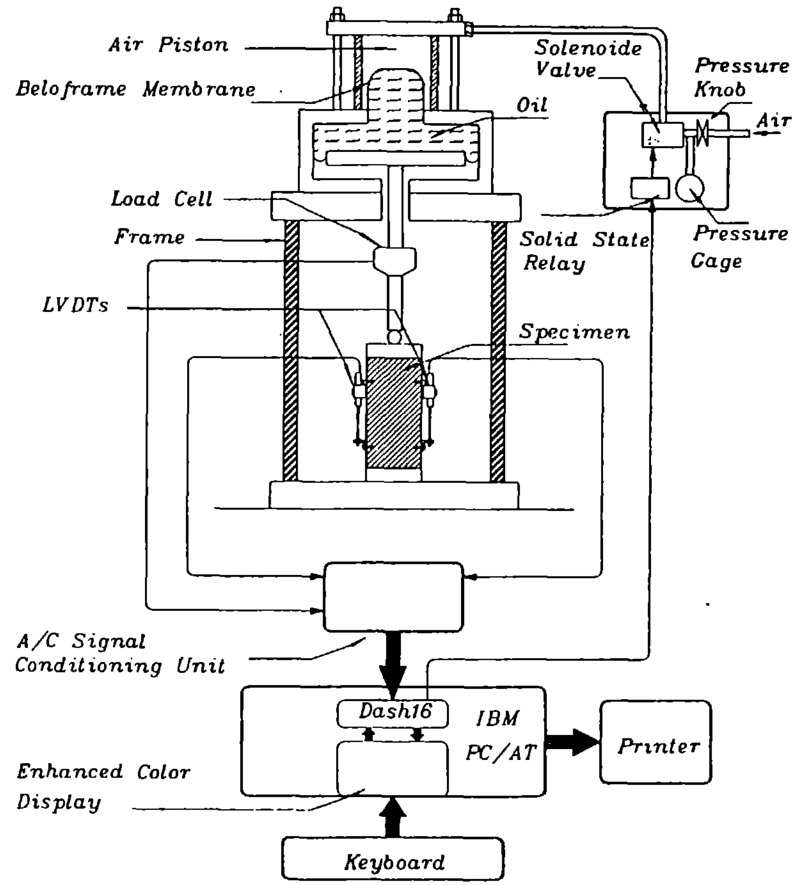


Fig. 12 - Test system for creep and repeated load testing.

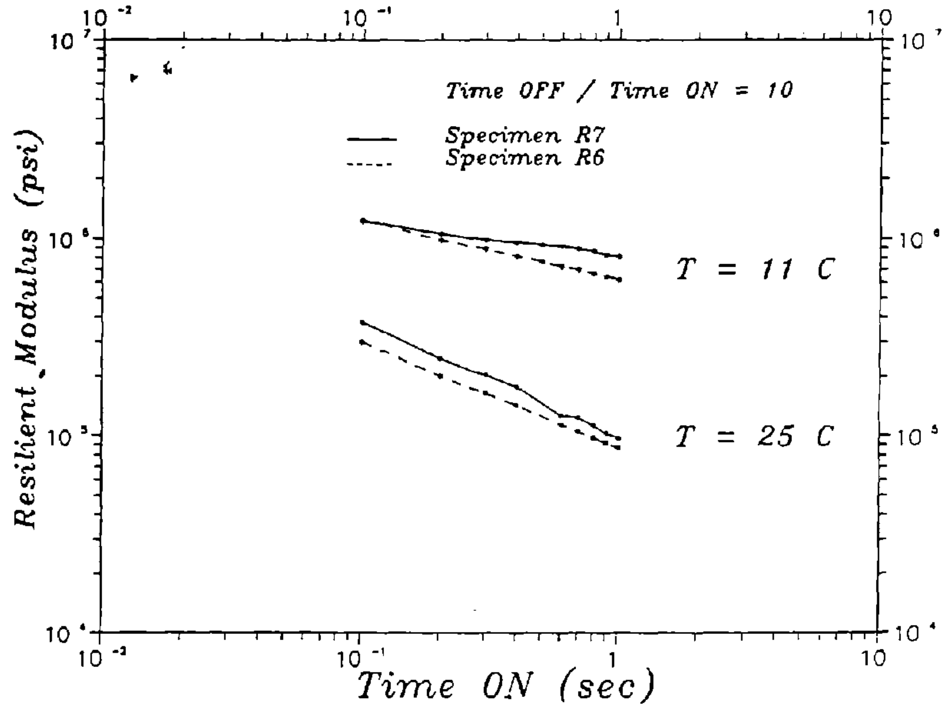


Fig. 13 - Influence of duration of pulse length on resilient modulus determined in repeated loading in compression.

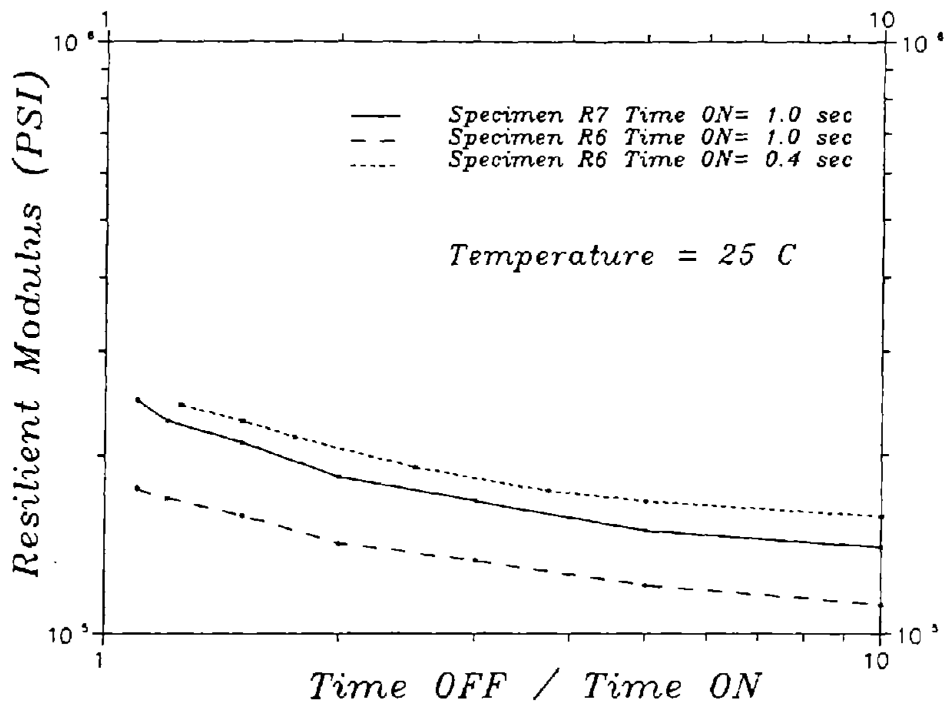


Fig. 14 - Influence of time interval between load applications on resilient modulus determined in repeated loading in compression.

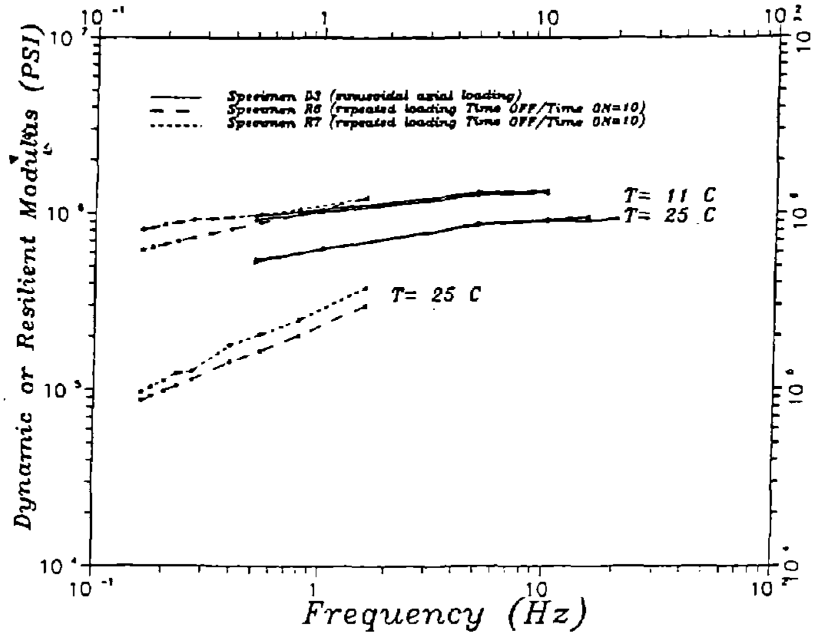


Fig. 15 - Comparison of stiffness moduli determined in sinusoidal and repeated load compression testing.

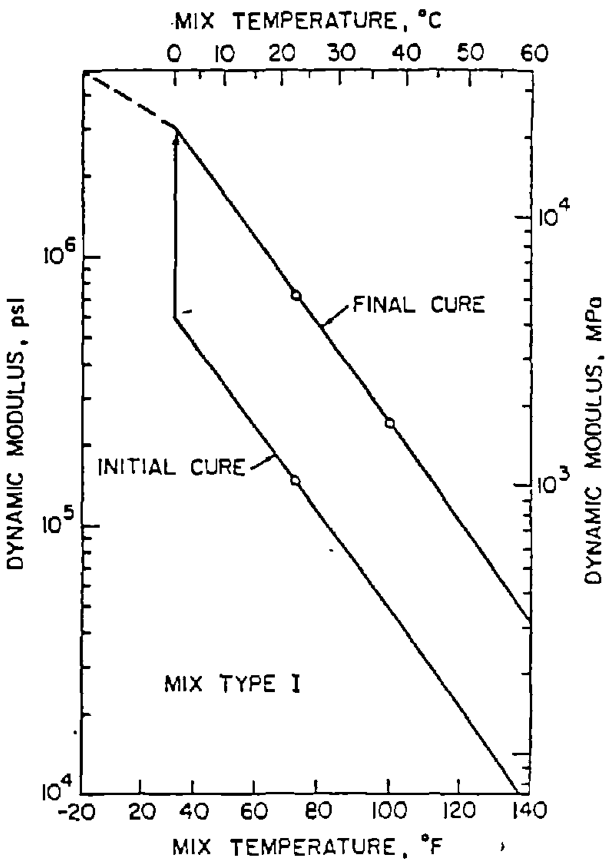


Fig. 16 - Modulus-temperature relationship for asphalt emulsions (Chevron scale) (Reference (2)).

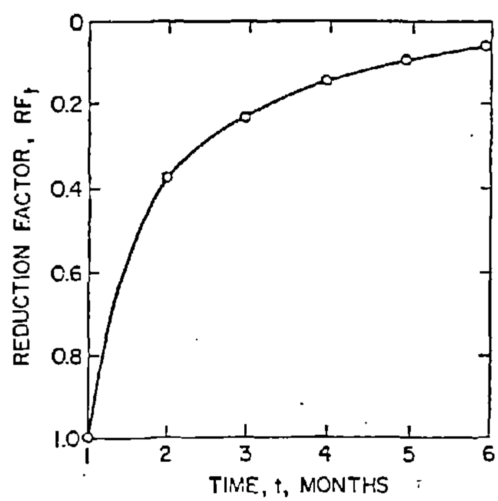


Fig. 17 - Reduction factor vs. time for 6 month cure period (Reference (2)).

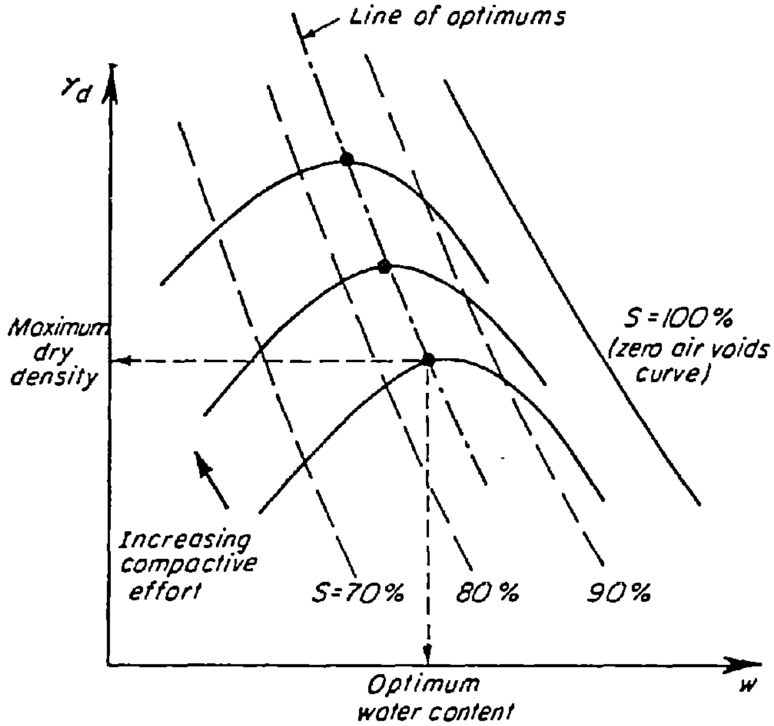


Fig. 18 - Relationships between water content, dry density, compactive effort and degree of saturation.

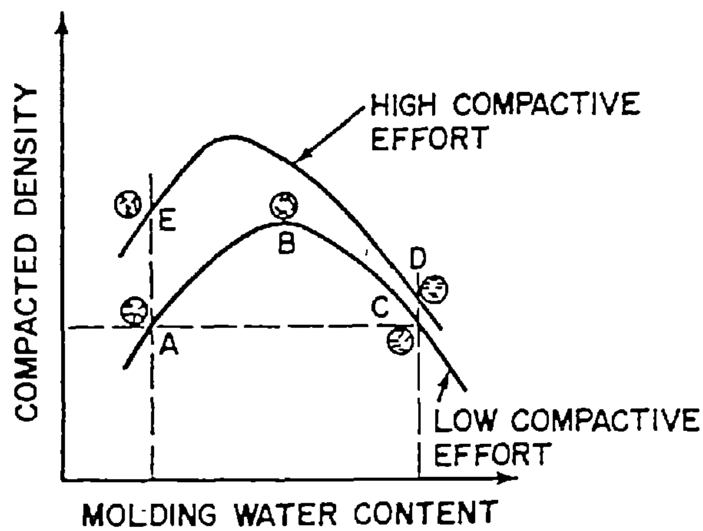


Fig. 19 - Influence of molding water content on soil structure (Reference (31)).

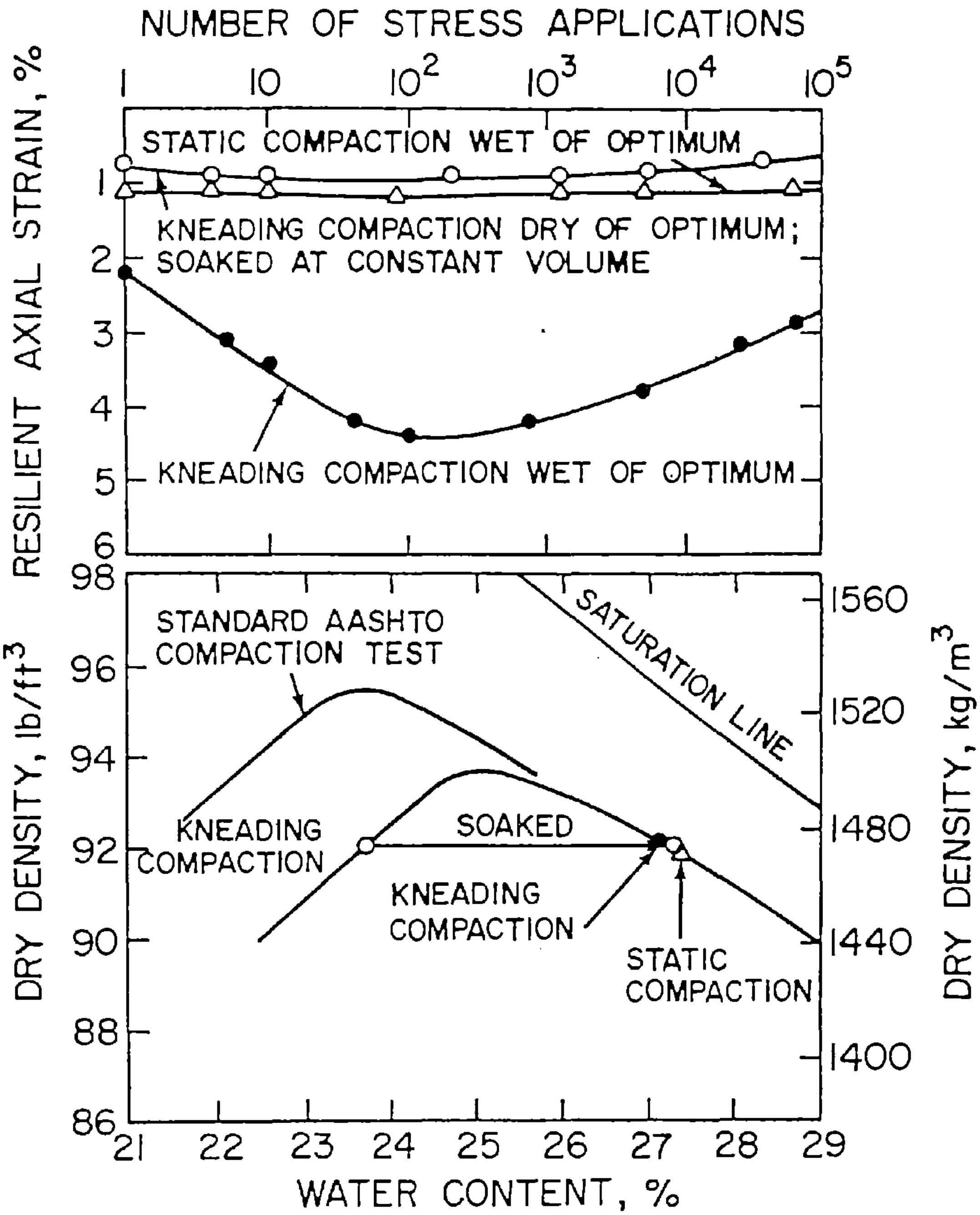


Fig. 20 - Influence of sample preparation procedures on resilient axial strain (Reference (33)).

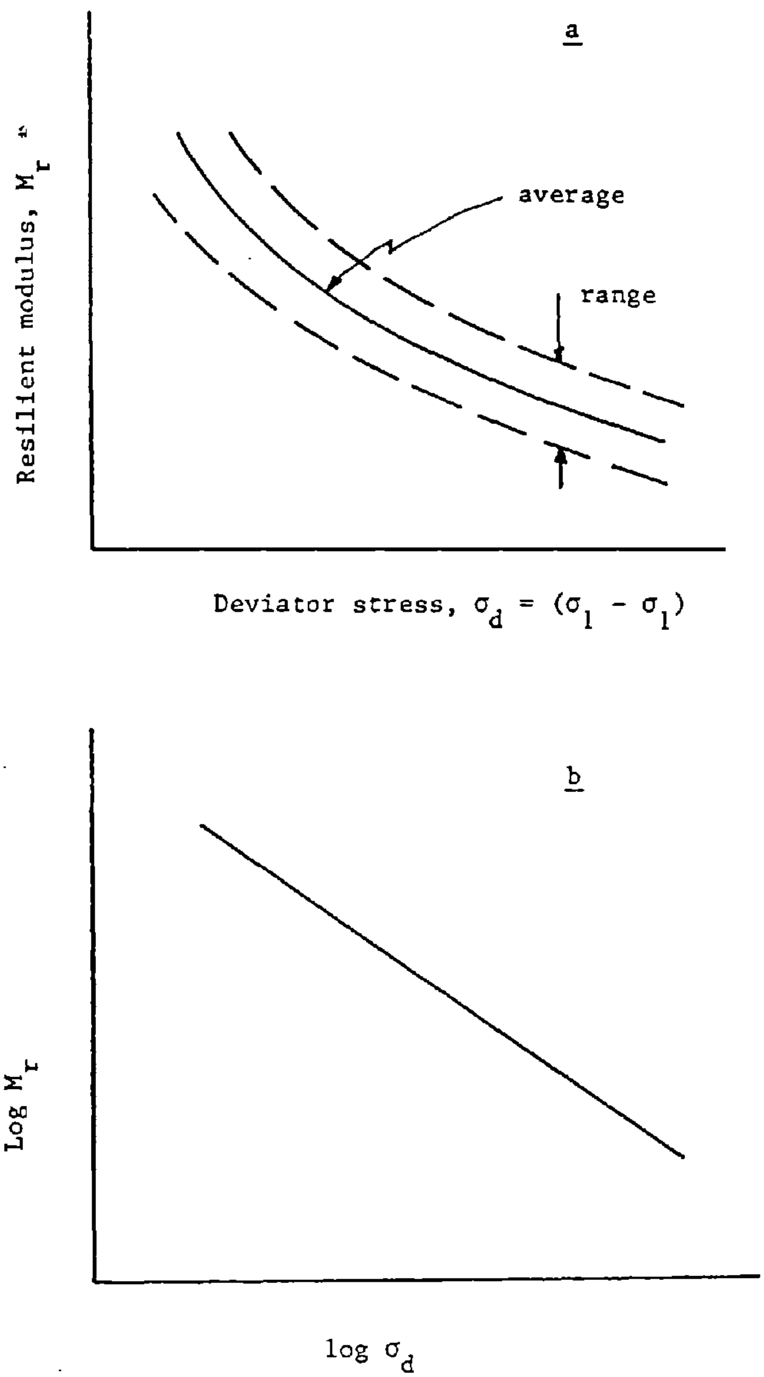


Fig. 21 - Relationship between resilient modulus and applied deviator stress for cohesive soil; a) arithmetic plot; b) logarithmic plot.

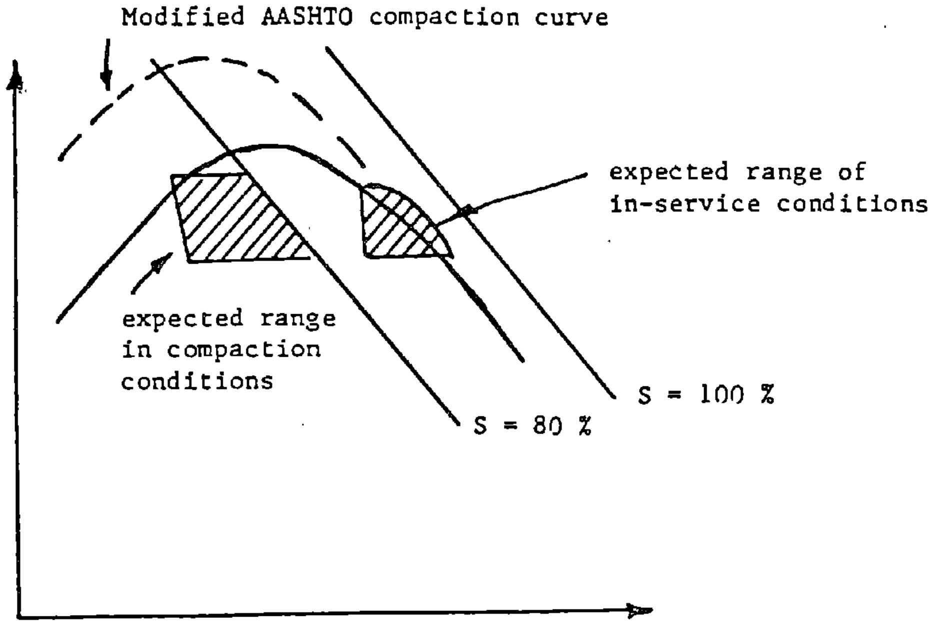


Fig. 22 - Dry density and water content considerations in laboratory specimen preparation.

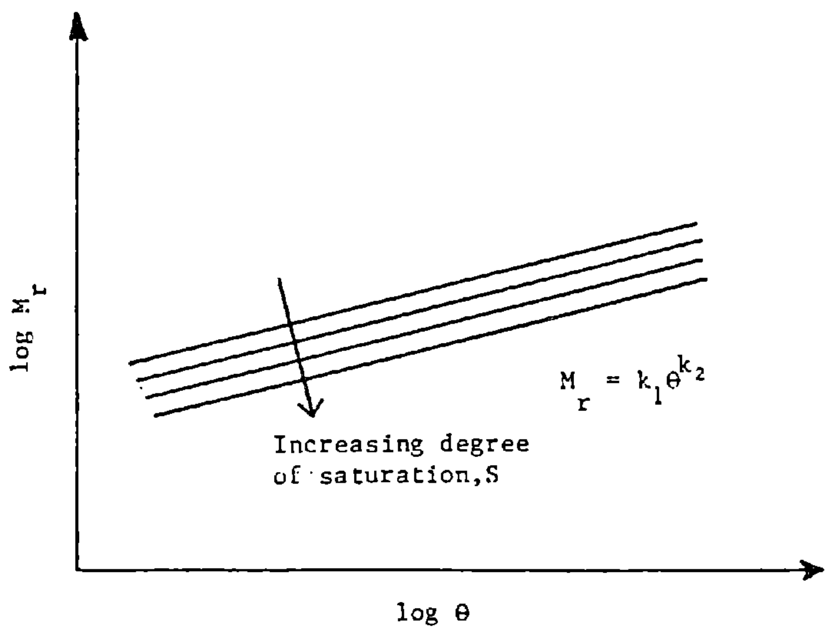


Fig. 23 - Influence of degree of saturation on stiffness characteristics of untreated granular material.

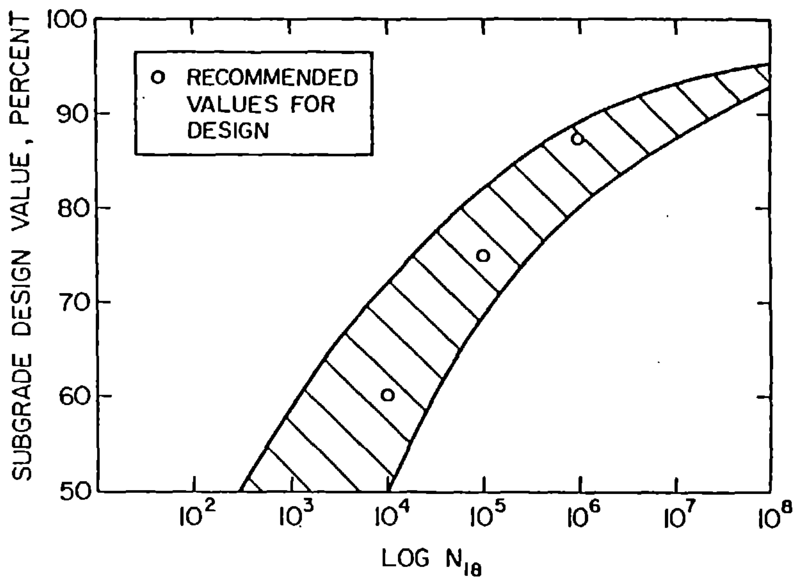


Fig. 24 - Recommended values to be used for selecting subgrade design value (Reference (2)).

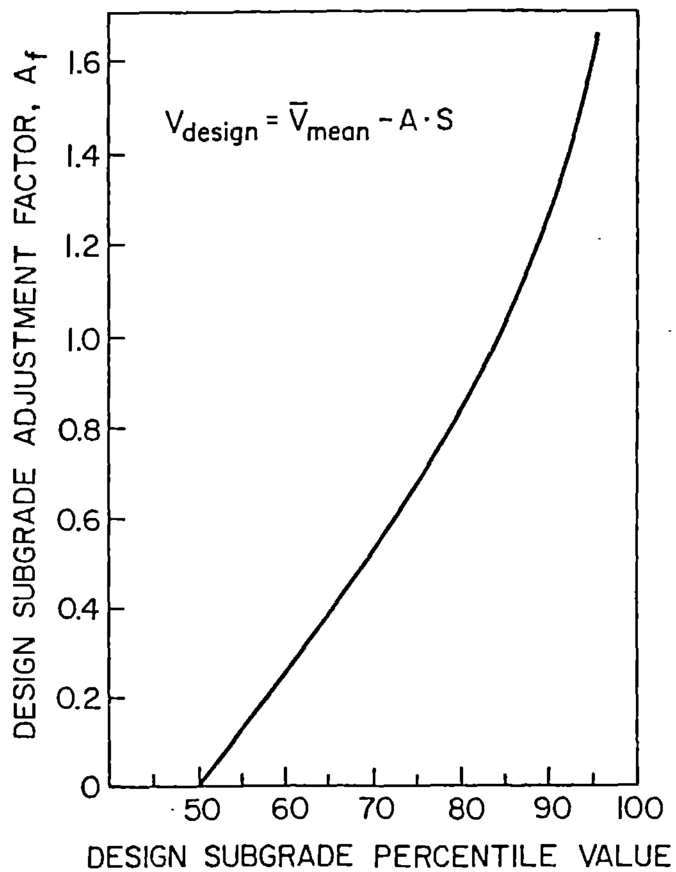


Fig. 25 - Adjustment factor for design subgrade percentile value (Reference (2)).

ROUND ROBIN TEST PROGRAM DISCUSSION AND QUESTION/ANSWER PERIOD

by

J. Sorenson, Regional Pavement Engineer
Federal Highway Administration

INTRODUCTION

The purpose of this session is to allow time for the audience to discuss the results of the Round Robin Test Program as well as any other concerns members might have. As moderator, I have collected a number of questions from the audience and will have one of our distinguished panelists answer each of the questions posed.

The participants include:

1) Carl Monismith, University of California, Berkeley
2) Jim Huddleston, Oregon Department of Transportation
3) Newton Jackson, Washington Department of Transportation
4) Marshall Thompson, University of Illinois
5) Ron Terrel, Transportation Research Institute, Oregon State University
6) Jim Wilson, AMI

The following section presents the transcriptions of the questions and answers. Unfortunately, some of those who asked questions were not properly identified. I hope this will not offend any of the participants.

QUESTIONS/ANSWERS

J. Sorenson: The pressing question that wasn't answered or wasn't even asked is: **Why Resilient Modulus?** Marshall, do you want to try to tackle that one? We'll have the panel follow.

M. Thompson: I think that early on, as soon as we got into a situation where we were able to measure moving wheel deflections and compare them with, say, static or slowly moving Benekleman beam deflections, it became very apparent that a moving wheel load response of the pavement section and the static deflection of that pavement section are different.

With the advent of the resilient modulus work in the early '60s, it became obvious that resilient moduli values weren't the same as those we achieved from static-type testing. It became obvious that if we wanted to predict the behavior of pavements under moving wheel loads, we needed to be testing in a different manner. From that day on, I think everyone concurred that the moduli values that we need for pavement analysis and design need to be determined based on "repeated loading." In that process, we learned to differentiate between a "stress-hardening" and "stress-softening" material.

Technology has advanced considerably, and at this time we have correctly identified the major inputs. Some of the current structural models are able to handle resilient modulus inputs. I would concur with Carl (Monismith) that we have to recognize the GIGO principal, "garbage in, garbage out." If you have lousy input, you're going to get lousy output. We need to put a lot of effort into how to develop these inputs so that we can characterize those materials and take into account seasonal effects, etc.

The flip side of the chart is transfer functions (fatigue criteria, how to avoid permanent rutting, permanent deformation, etc., in the section). Do we need modulus testing? A final parting shot -- I don't think resilient modulus is the way we ought to characterize granular materials for pavement design in terms of an AASHTO coefficient. Modulus is a property interest, but <u>not</u> <u>the</u> <u>one</u> I think relates to the performance of a material in a pavement section.

J. Sorenson: **Jim (Huddleston), why resilient modulus from the State perspective?**

J. Huddleston: Well, we're primarily interested in resilient modulus for two reasons: One is for implementation of the AASHTO Guide. We have adopted and use the AASHTO Guide in Oregon for pavement design. We use modulus testing primarily to characterize subgrade response and support. The second reason for modulus testing is that we foresee a move in the future to more rational design procedures that will incorporate modulus as the primary input. Our current experience with modulus testing provides an opportunity for our staff and laboratory people to gear up and prepare for making the move to rational design procedures.

J. Sorenson: Ron wasn't sitting with us last night when we went through all the questions, so we'll just hit him "cold turkey" with some of them. **Ron, what's your perspective on resilient modulus?**

R. Terrel: I really think the resilient modulus is a compromise from the ideal. It's something that we can probably measure if we get a little bit better at it, perhaps after a few of these workshops, but I think it does represent the real world for transient loads. This is probably as close as we're going to get, somewhere in-between long-term loading, such as creep, and some of the more dynamic tests. I think it also represents what we see in the field in terms of materials, but what we cannot characterize as well as in the lab in terms of its performance.

J. Sorenson: Newt has a few hundred tests that he's run in Washington. **Let's see what his perspective is on MR.**

N. Jackson: We have more questions than answers in some ways, but we have started to adopt a mechanistic design procedure in the State of Washington. We've done this because we have some data that most people don't have. We have a Pavement Management System (PMS) that allows us to look at the downstream end of the design process, that is the damage functions. We can look back and see <u>when</u> most of our pavements failed and <u>how</u> they failed.

We did take a step a little bit ahead of everybody else in trying to step into mechanistic design and, at the same time, recognized that with that step we could also do the AASHTO design with modulus values. Dynamic testing is necessary for mechanistic design. Though Jim, I know the highway industry is starting to look at it as far as the AASHTO Guide, the AASHTO Guide in the near future will be mechanistic design. It is a very rational design procedure as opposed to empirical processes and allows you to extend your understanding in material properties beyond always looking back at what you've tested in the past in empirical processes. It's something that is here, and it's something we're trying to be able to get a handle on.

J.Sorenson: **Dr. Monismith, do you want to address the question: Why Resilient Modulus?**

C. Monismith: I look at the pavement design process as an iterative activity. Years ago, the CBR and stabilometer procedures were used and pavement design was predicated on the premise of preventing excessive shear deformations. In 1956, Francis Hveem presented one of the classic papers in the pavement area entitled "Pavement Deflections and Fatigue Failures." It demonstrated that while the then current design methodologies might provide pavements that were satisfactory against rutting, they did not preclude fatigue cracking. From Hveem's efforts, we recognized that we would have to do something better. Efforts like those presented here are a step forward.

We have to realize that there are problems with the way we measure material properties, but this is the next iteration in the loop. Perhaps fifty years from now there'll be something better because pavements will be utilized for many years to come. In effect, we can look at resilient modulus testing as an iteration in the design process, and I think it's a very important step.

J. Sorenson: Thank you. I think we've heard a couple of different perspectives. Marshall has a firm feeling on aggregate testing. I think we're hearing overall that resilient modulus is going to give us some better materials characteristics than those that we've seen from static laboratory testing. **Does anyone else want to answer the question: Why Resilient Modulus?** (No volunteers) The first question we'll direct to Marshall, and we'll see what the panel wants to add to it. **Marshall, how many load repetitions are necessary to remove the softening effects of a freeze-thaw cycle?**

M. Thompson: Carl (Monismith) showed a slide from some work Robnett and I did several years ago. Repeated loading by itself applied shortly after the freeze-thaw cycle will not increase the modulus. It takes a while for the soil to condition in a closed system where we aren't experiencing a moisture content increase during the freezing. Basically, in a freeze-thaw cycle, the suction goes kaput and you come up with near zero suction. What happens is, as that specimen sets, it cures; and the moisture films are realigned. The suction comes back up in an unsaturated soil, and the modulus increases. The philosophy, then, is that you are married to that softened material until the moisture suction condition is reestablished.

Del Fredlund in Canada has done some interesting work in this area. I know the folks at CRREL also have considered the topic. Repeated loading won't restore that rascal; it is a time-dependent thing. Ultimately, the modulus will come back. I think the most discouraging thing about freeze-thaw effects on cohesive soils is that it only takes one freeze-thaw cycle to drop your modulus considerably. Whether there's one cycle, three cycles, or four cycles, you're going to take a "beating" with just one. That doesn't get you out of t he woods, so to speak, just because you live in an area that only infrequently experiences a freeze-thaw cycle.

Luke over there (in Minnesota), freezes up and stays frozen, so he only gets one cycle. But in southern Illinois, you know, those folks don't like those freeze-thaw cycles, but they do catch them. We have problems with reduced modulus values in the South during the thaw period.

J. Sorenson: Thank you. Another questions here is: **What is the appropriate stress level that we should be defining as the reference resilient modulus?** Newt, could you try to field this one?

N. Jackson: I think I got picked on because I offered that up yesterday as having some typical modulus value or a standard by which we reference moduli. We had made an attempt early on to model a couple of different roadway sections and to look at those roadway sections, which were primarily gravel-thickness materials picked at fairly high bulk stress in the middle of the layer. Looking back, I almost wished I hadn't tossed the numbers up for the last year. I thought I had set them too high.

If we want to adopt a standard by which to give a single modulus value, that standard ought to be based on the typical 5,000 lb. wheel load, the normal PSI pressure of the tire of 100-110 PSI, and then we should look exactly at what the bulk stress in a typical roadway section. Then, we should come to some consensus as to what that bulk stress is at which you adopt a standard based on the use.

J. Sorenson: The next one I'll throw out here, we'll direct to Ron Terrel: **In many states rutting of asphalt pavements is a primary concern. Have there been any successful attempts to correlate resilient modulus to the mix's rutting potential?**

R. Terrel: There have been quite a few attempts, of course. As for successful, I don't know what you could call success. But I think that the shift factor that you would obtain from a modulus in the laboratory compared with what you might get in the field, or expect in the field, can be variable. I think we get comfortable if we do it ourselves. You get a good feeling when you get the field values to match up, and then you find someone else who has used your numbers and can't get them to match at all, so you begin to lose confidence. I think that probably the State of Kentucky has done some of the better work in trying to do this through the years, and Washington as well. Other states are trying it, but many of them have not applied the information in a design system yet.

J. Sorenson: Perhaps, Carl (Monismith), you'd want to jump in on this one and maybe also address: **What future you see in creep testing, such as the Shell Method, as a means of characterizing the rutting potential?**

C. Monismith: I think one has to be careful about assigning one value for a modulus as a criterion to mitigate rutting. For example, the Europeans are considering a creep modulus in which a creep modulus is specified at 40°C and at a time of loading of 60 or 100 minutes. One has to be careful about this. Certain mixes may provide a satisfactory creep modulus at 40°C, but when the temperature is increased to 60°C, the specimen fails.

In our experience, for example, some rounded gravels, when tested in creep at 40°C, performed better than a crushed granite tested at the same temperature. However, when the temperature was increased to 60°C, the granite proved to be far superior. If you are measuring moduli for rutting predictions as embodied in the Shell procedure, you have to be careful and not just think in terms of one modulus as a criterion. You must define creep moduli at different temperatures to permit estimates of rutting to be made at the higher temperatures that may occur in the upper parts of the pavement sections on hot summer days.

M. Thompson: An interesting reference that you may want to look up when you get back home was published by Khedr in the January 1986 Transportation Engineering Journal of ASCE. He presents a rutting model that embodies both repeated deviator stress and resilient modulus that seemed to account fairly well for some laboratory data. We've since played some games with that model on some test sections, and the general format of the model seems to hang together darn well. It does include a modulus concept, as well as a repeated stress state concept, and from those you predict a "rutting rate." It's an intriguing piece of work. I think it needs some fleshing out, but it's one of the more attractive schemes that have hit the streets in the last few years based on what we've been doing lately in our NCHRP 1-26 Project.

J. Sorenson: The panel doesn't have all the answers. A lot of you in the audience are all part of this team that's pulling together this technical area, so please don't hesitate to jump in. I know that there are some differing perspectives on some of this, and we want to hear from all of you because we may not resolve these issues, but we're going to get them down in writing.

R. Terrel: One of Carl's slides reminded me of the effect on modulus of curing an emulsion mix with time. Some of the earlier work in this area has shown, for example, that if you're concerned about rutting, you must be aware of the effect of time, particularly with the emulsion-treated bases like we use in the Northwest. We've found that the curing period in the field can be up to two years from the time of construction until it actually cures out to a final ultimate modulus value. During that two-year period, you could have considerable rutting occurring because of that treated base and the effect of heavy traffic.

J. Sorenson: Thank you. **Marshall, should compaction pre-stresses be accounted for in selecting the appropriate stress level for resilient moduli of nonlinear material, and do you think that the compaction pre-stresses are removed by load repetitions (i.e., traffic)?**

M. Thompson: I've heard many times that Burmister always talked about this effect, even back in the 1940's. Under compaction, you build in pre-stresses, etc., and residual confining pressure. There's been some activity. Ernie Selig at the Univerisity of Massachusetts at Amherst has done some things in the rail system area where he's trying to measure residual stress. Even under the most extreme circumstances, all Ernie ever measured was a couple of PSI, at most, in terms of a residual confining pressure.

I would suggest that to base a pavement design on the assumption that those small stresses are going to be there and stay there would not be a prudent thing to do at this time. I'm not saying they are not there; I'm saying that it seems that at this time they're fairly small. Ray Seed has done some interesting things at Berkeley, in terms of compaction-induced residual stress. I think that's an emerging area that deserves our attention. I would say that's about where we were with resilient modulus testing in the middle 1960's. I think its time may come, but it hasn't arrived.

J. Sorenson: Jim Wilson represents the manufacturer's perspective. He's been dealing with the equipment and trying to make heads or tails of some of the mechanical/electrical side of this for many years. One of the questions we have here, Jim, is: **From your perspective, is it necessary to lubricate the end platens in the triaxial repeated load tests, and do you think it's really being done?**

J. Wilson: Well, to some degree. There are attempts to lubricate them, using sheets of teflon with the lubricant in-between a silicon grease. This attempts to reduce end effects. The attempt is to minimize them. I think probably the more significant problems with the end effects were actually in the preparation, that is, the platens are not completely flush with the end of the specimen and stress concentrations are developed. That probably attributes more to poor measurements in the measuring of axial deformation rather than induced strength.

J. Sorenson: This question is directed to someone who has worked with some non-cohesive soils. I think maybe this one Newt would want to field. **In larger samples the effects of gravity would become a problem, especially in fine-grain, non-cohesive soils. How can**

the test be performed without causing the water in the sample to flow to the bottom during the test itself?

N. Jackson: If you've got that much water, there are a multitude of things. I'm not too sure I'm very confident that we've put together silt-type materials in our modulus tests and get from them numbers that I believe too well. We've done quite a bit of work in some of the loess silts in remolding a sample, setting it aside for almost a month at a time, and then testing it again. We see it almost double in strength just from stabilizing after the mixing process. We do make sure that we temper the stuff overnight pretty well to disperse some of that fluid. You may have to let the sample set and drain for awhile to stabilize some of the pore pressures that Marshall was talking about. I think if you have those kind of problems, you have to work around them by compacting on the dry side and letting them soak for awhile so that you stabilize those films, the way they are out in the field. I don't know that I've solved the problem at all.

J. Sorenson: **Marshall, do you think that the same amount or number of conditioning repetitions are needed for the cohesionless soils, as might be required for cohesive soils?**

M. Thompson: I think the conditioning process relates back, perhaps, to the idea of seating the caps properly on the specimen (as Jim Wilson referred to), and I think it's essential in a conditioning process to test at a stress state that's reasonable for field conditions.

During conditioning, you experience a lot of early up-front permanent deformation. When you drop down to reduced stress states, you can do resilient testing with a reduced number load reps without the problem of picking up additional permanent strain. Permanent strain accumulation is very stress-history dependent. If you condition at some reasonable stress state, well below the ultimate strength of the material, you can then drop back and do some testing without having this "permanent strain accumulation" problem. As you move up to the higher stress states, if you desire to do so, you might pick up some additional permanent deformation.

I think the two criteria: (1) seating the specimen properly and (2) getting some of the permanent deformation are going to take 1,000 load reps or so. A semi-log plot is reasonable, so that what you get from 1 to 10 is what you get from about 10 to 100, is about what you get from 100 to 1,000, etc. At 1,000 reps, a big chunk of that permanent deformation is out of the way.

J. Sorenson: Thank you Ron (Terrel), I know you weren't there to defend yourself last night, but you may want to try to elaborate on this question, and we might get some feedback on this one. **Do you think that it's reasonable to combine the repeated load test for plastic deformations (we've talked about this a little bit this morning) with the resilient modulus testing?**

R. Terrel: I think it's a practical way to go. If I'm understanding your question right, I think you should be measuring the resilient, as well as the permanent deformation in any testing you do. It should become part of your data base because I think this is our basis for our understanding of the behavior of the material. Somehow, the resilient portion may be more significant or less significant, depending on where it is in the structure, for example, or the conditions under which you test. I think that there is no problem at all with combining these, particularly in treated material. It's really kind of a help to know, and it can't hurt any.

J. Sorenson: **I guess if you were going to go to the expense of running the test, we might as well try to get all the data out of it we can.**

R. Terrel: Well, I think so. I think that with the capability that we've been shown by the equipment people this week, we now have the ability to accumulate this data and store it for future reference, building a data base that we could not do in the early days when I was a graduate student. We were looking at dial gauges and writing down numbers all day and all night and this was rather tedious, but we're getting away from that. Why not use this capability?

J. Sorenson: Marshall, you may want to try to field this one, and maybe Jim Wilson could add to it. **Do you think that you need the same number of conditioning reps when you use**

the internal clamp on LVDTs as are necessary when you use the external LVDTs to measure subgrade material deformations?

M. Thompson: I think the same philosophy applies there. The idea is to seat the end plates, and that sort of thing, and get some early permanent strain out of the way during the conditioning phase. Later on, you drop back to a lower stress state and start measuring resilient responses. The same criteria would apply. Some folks have difficultly keeping those clamps (LVDTs) hanging together for extended periods. That needs to be a concern. Start looking at how many conditioning reps you put on, but I would suggest we could use the same number and that would be appropriate.

J. Sorenson: **Jim, do you have a perspective on internal vs. external, and how effective are they?**

J. Wilson: Well, the internal clamps are much more difficult to use, depending on the type of material. For instance, if it's a subgrade material, it's not subject to the irregularities on subbase material. Some of the difficulties experienced with third point measurements are the friction over the gauge, like the friction of the coil, and the core problem. I always like to have the ability to look at each LVDT independently while the specimen is being conditioned and to look at the response of each one. Sometimes one LVDT will not be working at all, and the other one may be going through some fairly large excursions. That leads you to believe that there's something wrong, and it gives you a chance to do something with it.

As to the number of cycles, the platen to platen vs. the third point, I like to look at when the test specimen comes to some steady state, or nondeformation. In other words, you'll see that the third points will not exhibit the amount of deformation the total LVDT gives you for the platen to platen. In fact, I have some handouts here today to show the ratio of the resilient modulus of platen to platen vs. third point. This is on a range of specimens for resilient modulus. I threw one in that was not a resilient modulus test, but it was on a specimen of rock that was abutment wall material from a dam in Southern California that experienced some damage in the San Fernando earthquake.

We made some third-point measurements not using LVDT third-point clamps. This was with strain gauges, but it gives you a feel for the difference in measurements between platen to platen and third point.

J. Sorenson: I think the overhead works there, since you're on the subject, if you want to throw up a couple of viewgraphs. Marshall?

M. Thompson: May I make a plea? If anyone in the audience has data where you have well-defined samples and measurements based on both external and internal LVDT arrangement, that information is very helpful and vital to what's going to be happening in some programs. For example, how are we going to do SHRP modulus tests? Please forward that information to me. I'd be glad to accumulate and analyze the data.

R. Terrel: Jim (Wilson), before you start, you might remember the work that you did with us at the University of Washington. We had 4 x 8 in. samples where we attached a whole range of strain gauges.

J. Wilson: Okay, this first set of data is for a resilient modulus test. This was performed on some material from Alaska, and we have the density and moisture content for the two listed. If you look at the confining pressures, we started at 2, 4, 6, and 8 PSI. The number of cycles over here were measured at 10 and 100 cycles. The cyclic stress, third point strain, microstrain, total strain, and the modulus values were computed at 6% water content. The ratio of the third point to the platen to platen one, this was at 8 PSI, was in the order of 2.3 to 1, and as the confining pressure was dropped, the ratio dropped. You can see the corresponding values for the modulus. This same material was remolded and performed at 9% water content, and you can take a look at the ratios again. The modulus dropped, as you would expect, and the ratios between the two dropped. With confining pressure, they dropped down to about 1.78 to 1.

The next set of data was for a different project. These tests were oriented toward AASHTO T-274. We just had developed this procedure. This is at cycle 100, 200, and 300, and this test was with some equipment from the days when we were using open loop machines so the cyclic stress, the amplitude, was a little more difficult to control. This one started with the pressures at 2, 4, and 8. The effect of confining pressures, and again you can see the differences, were 1.35 to 1 to a little over 2 to 1. The last one was not a resilient modulus test, but I just threw this in to give you a feel for how the difference in third point vs. platen to platen can vary depending on the modulus of the material. The stiffer the material, the greater the divergence in the two measurements (re: Exhibit 1, page IV-73).

J. Sorenson: Thank you, Jim. Are there any questions from the audience?

M. Thompson: Jim, ask Al Bush to comment on that. Is Al back there?

J. Sorenson: You want to summarize that, Marshall?

M. Thompson: I think what Al was suggesting is that the effect is more accentuated for the higher modulus material. Several years ago, we did a study where we had an optical scanner and traced targets on the membrane and compared that back to total axial. In that scheme (those were open graded materials with few dense graded), we found a fair comparison. At times they were almost equivalent. **That's a critical issue; it really is.** It is difficult to place clamps on a broad range of materials. Imagine trying to clamp on something like an AREA #4 ballast with a top size of two inches and nothing passing the #4 sieve! That's tough! How about impossible? If we want a range of data, it may be in the intermediate period. The concept of doing it both ways is something we're going to have to live with. When a clamp arrangement dies in the middle of a test, there are a lot of very unhappy, distraught lab techs. Golf comes later in the day then!

J. Sorenson: Marshall hasn't had to work out here in the Northwest with me too long. I really do believe in this teamwork concept, so it doesn't bother me to put the Professionals and the States that I work with on the spot and try to get as much out of them. Newt says I always try for two bucks out of two nickels. Being a Fed, we are out here to help, and we try our darndest.

I want to put a pitch in for our manufacturers who are here. There's a lot of technical expertise here. I noticed yesterday morning that there was good discussion in the groups. There was a lot of conversation. Last night those labs were open, and again tonight many of the manufacturers and vendors are still going to be available. We're only covering a few isolated questions in the technical area. Most of us are Civil Engineers; we're delving into some of the electronics and the computerized systems where we don't necessarily have the technical expertise.

I encourage all of you to go over there and bend those folks' ears, corner them, tie them down somewhere if you have to, and try to get some of the answers to the questions that are plaguing you within the highway departments. We do have a unique opportunity here this week, and I'd like to see our highway departments and our users out here get the most out of this workshop, either personally or professionally.

Since you did so well on that, Marshall, maybe you'd like to touch on this one. **What is the AASHTO test at bulk stress levels which far exceed those that typically found in our highway situations?**

M. Thompson: I think the stress states that are in AASHTO T-274 at this time probably were established and included in that standard well before we had good fixes on what those bulk stresses might well be. I think, in retrospect, we would now probably change those. I wasn't involved in preparing that spec, by the way. I want to make sure we understand that.

I think that the theta values need to be representative. My philosophy on bulk stresses for resilient modulus testing, in granular materials in particular, is that we need to have stress states that bump us up into the working region and a bit beyond. We need enough data so that we can properly define the K-theta to the n relationship, but on the other hand, the values should be realistic.

It is possible to fail a lot of granular materials at deviator, overconfining pressures of 45 over 15. You bang on that 1,000 - 5,000 times during conditioning and you may "kill" your specimen. We've had several materials where we had to drop back to 30 over 15 to get them to live through 1,000 to 5,000 load reps. I think those stress states need careful reconsideration. I would suggest that's something to definitely be considered by some group, such as AASHTO. It is true, and Jim points this out, and Newt too.

If you're using thin surface pavements or bituminous surface treatments over granular base, you have some bulk stresses hanging around up in the high numbers. You're up there frequently. You figure you get 100 PSI radial tire on the topside, throw in a few psi confining pressure on top, and you're easily banging around well over 100. You have to be. We know we are. We need some numbers up there in some cases. On the other hand, bury that granular material under asphalt concrete with stiffness values of 1,000 ksi, and the bulk stresses are quite low. We have to bracket the whole range in many cases to do a good job with those materials and conduct proper analysis and design.

C. Monismith: I just wanted to emphasize that you must consider the pavement structure that you're dealing with. If the pavement contains a thick asphalt concrete layer, to define the stiffness of granular layers, you must utilize one set of stress conditions. If you're evaluating a thin pavement surfacing, you use another. Again, I call your attention to Table 2 in my paper; reference is made to the work of the South Africans. In their design methodology, they recognize these considerations, and they select the modular values depending on the thickness of the asphalt bound layer, which is what Marshall is saying as well.

This brings up an important point that I hope someday personnel in our highway departments can reconcile. That is, the people who are doing the materials testing should not be in one location and the people who do the pavement design in another, for example, at separate ends of a building. Rather, the people should be together so that the pavement designer and the materials evaluator can actually work together to design the pavement system because you can't divorce the materials design from the pavement design.

J. Sorenson: I'd like to have Jim Huddleston try to answer this one. I know it's something that's come up from ODOT before, so he's probably the best one to field it. Jim, when

testing nonplastic, sandy silts and low PI clay silty soils, we've experienced lab compacted moduli values that are significantly lower than the backcalculated values. Do you want to try to explain why?

J. Huddleston: We typically sample existing subgrades by taking thin-walled (Shelby) samples. These are usually taken in the top two to three feet of subgrade. We then try to represent or correlate the results of tests of those samples to backcalculation results from NDT data, which take into account a much larger depth. One explanation would be that some stratification is present in the subgrade. We are testing the worst case in the lab. However, the NDT results take into account increased confinement and density with depth. Also, from what I've heard at this conference and considering some of the results Newt has had, the fine, silty soils tend to be very sensitive to small changes in moisture content. A lot of the scatter and low lab values might be caused by minor changes in moisture content.

However, from my point of view as a Pavement Design Engineer, the best way to validate your results is to look at the road that you are doing your backcalculations from. If you are backcalculating 10-15,000 and measuring 2-5,000 in the lab, you certainly should see some reflection of that in the condition of the pavement that you are testing. If the road has performed well, and you cannot explain that good performance with the lower lab values, then you should place more reliance on the backcalculation results.

J. Sorenson: Would anybody like to echo any more on that?

J. Sorenson: Newt was nodding.

N. Jackson: Sounds good. Just one of many reasons, I think, they're difficult to handle. I have a lot more confidence in what we find with in situ testing than anything we could do with the lab with some of those materials. Jim (Huddleston) makes a good point. In most of our work, we're dealing in overlays and existing roadways, and you should look at those existing roadways and see what they're telling you. If the performance of that roadway does not match the numbers you've been getting out of the lab, you should suspect what you get out of lab.

J. Sorenson: Marshall?

M. Thompson We have a lot of high PI granular materials that sometimes slip into our roadways in Illinois. We have worked with several of those to see what could be done with them. Frequently, we lime-treat them to shape them up and put them back down. Many of those materials, though, are so moisture-sensitive that a ½-1% increase in moisture over optimum would drop your shear strength by a factor of a half. Here you are pounding around on that specimen at some high stress state, and the thing just dies. We actually start pumping fines out the exhaust vent on the triaxial cell! We have to put a bucket under the exhaust port; obviously, things aren't in good shape. I think Dave Allen's analogy of liquifaction may be appropriate. Those high PI and fines material just flat go unstable! They generally show large permanent strain accumulation with a few number load reps. Dry them up, and they look a lot better.

M. Thompson: I was responding to Dave Allen. We've had tube samples of cohesive soils from the field representing, say, early spring conditions, where ERI values (modulus at a state of 6 psi) are a thousand psi or less.

N. Jackson: I think we were talking about subgrade materials, and I think the question fit more like we see it, and that is, in backcalculations for subgrades, we see them stronger than what we can do at the lab. But in base, yes, backcalculation. Our procedures don't look too representative of what we would expect that material to be.

M. Thompson When you start looking at a 10 or 12-inch granular base under a few inches of asphalt concrete, the actual amount of deformation accumulated in that base is very small. Here we are thumping on that rascal with an NDT device, trying to sort out small changes in asphalt concrete modulus. I don't think, in many instances, we do that very well, and we get some funny numbers kicking around. I'd say in modeling the granular materials, we really aren't doing a very good job structurally. I'd be the first to acknowledge it. We're making some progress. The next generation of models to handle that sort of thing has to be developed. It is going to be awhile before those even get close to an application stage. I concur with your thought, Gary.

R. Terrel: I would be interested in getting some feedback from some of you out there with regard to what I call an equilibrium water content in the upper layers. No matter where I've seen pavement sampled, it seems like there's always water at the base of the asphalt surface course. This obviously has an influence on the modulus. Even in desert areas where we haven't had rain for many months, you'll find water there, and this obviously has an influence. I'm wondering if maybe the backcalculation moduli values are related to this phenomenon. Perhaps we should first of all identify the presence of moisture so that we can do a little better job of determining the appropriate values in the lab.

J. Sorenson: I think Ron brings up a good point, and even if you don't want to jump in here and put your two cents worth in, we need to find answers to some of these qquestions. We need to find some experience in these areas. If you don't ask him now, the question's going to be put down.

M. Thompson: Jim, why don't you have Bob Lytton comment about the FHWA study you folks are funding on climatic effects?

J. Sorenson: Last time I asked that question, I almost was thrown out of a classroom. Bob, is that going to take away part of your presentation or do you want to touch on that?

J. Sorenson: I guess the bottom line is, how long is it going to take to get it out of your shop and through Federal Highways and out to the real people? I want to touch on an equipment question that cropped up two or three times in here. I don't have any hesitation in putting Jim Wilson on the spot on this. I know he's worked with many manufacturers over the years, and he's always been up front with his responses and professional in dealing with us.

One of the basic questions that we keep getting asked is: What's the difference between electrohydraulic and electropneumatic? Many of you may have asked some of those questions yesterday. Why do we need such expensive equipment to run the tests? **What are the differences in the wave form considerations? How much do you have to pay for this test versatility?** Without really getting into the numbers, I'd like to ask Jim to touch base on this subject for a few minutes and really end it with his perspective on **whether or not you can get good results with some of the less costly equipment?**

J. Wilson: Well, first, electrohydraulic and electropneumatic. I've worked with both, and there are advantages and disadvantages to each. I guess we could talk about the electrohydraulic first. Electrohydraulic systems are responsive at higher frequencies. You don't have the problem of the compression of air in the cylinder. That's one advantage, although the tests that we're doing with the resilient modulus are not severe tests. It has a period of a tenth of a second, which is equivalent to a frequency of 10 hertz, but there are almost two seconds between the events, so it's not a severe problem for that type of machine. The electrohydraulic is a little more expensive; the hydraulic power supply costs a little more, and there's more of a heating problem. You have to have a heat exchanger and cool the oil. As for the wave form and the performance from either system, there's really not any difference at the frequency and the amplitudes that we're testing these specimens under. Again, it's not a severe condition.

The electropneumatic has some forgiveness in the system. You want to think about the two systems. You have to realize that since this is a forced control application, that means the test specimen is directly in the control loop. With the specimen in the control loop, you have to take into consideration the spring constant of the test specimen, the electronic gain of the control system, and the gain of all of the components. That is, the compliance of the whole system. And another factor that comes into it, in the electrohydraulic system, is the bulk modulus of the oil. It provides some damping, but not as much damping in the system or in the system actuator as a pneumatic system. In some cases, you can do some playing with the system, and you can tune the pneumatic system to respond by just changing the length of the lines between the actuator and the servo valve itself. You can get more damping in the system, which aids in increasing the electronic gain.

But there are trade-offs in all of these things, and probably by the time you get to the cost of electrohydraulic vs. electropneumatic, you put a dryer in the system to condition the air because the valve has to be kept very clean. It has to be free from moisture. It has to be lubricated or you have to put an oiler in the system. The cost of the hydraulic power supply may be a little bit higher, but not much. It's probably just about a wash. It depends on the kind of testing you're going to do in your laboratory. If you're going to do something other than resilient modulus, then you may want to look at an electrohydraulic system. It may be more versatile for you in the long run, but that's another application. Strictly speaking, for the type of testing we're doing, the electrohydraulic and pneumatic are comparable.

J. Sorenson: **How about some of the different wave forms that we see being used?**

J. Wilson: Either the electrohydraulic or electropneumatic are proportional closed-loop control. You can virtually program any wave form you like. You can program a square wave, a sine wave, etc. The problem with most systems is that the sine wave or a triangular is bipolar. It's not unical or it can be made that way with a function generator or from a computer where you can offset the wave form, but most of these start at the zero plane and cycle plus and minus. That's difficult to do with the test specimen unless it's confined, and you increase the minor principal stress high enough so that you can cycle bipolar.

My experience with wave form has been that this square wave that you program into the system with a fairly high rise time on viscoelastic materials can cause some strain rate sensitivity problems. The difference between the sine and the square wave produces a difference in the strain that you measure at the same stress. You will generally measure a higher modulus with the square wave than with a sine wave. That's dependent on the water content of the material and the kind of material. As to the effectiveness of less expensive systems, I've done that too. We started with a basic open-loop system with servo valves and a timer and performed resilient modulus tests with that system back in 1965. We used that system for about 1½ years, and with that particular system, you could get good results. There wasn't any problem with getting good results. The problem was that it was operator-oriented. The problem never was getting good results; it was just how much trouble it was to get them, and who was going to operate the machine.

I solved that problem in 1966 or early 1967, I think. We bought an MTS machine, and I worked with MTS on developing a modification to their function generator so that we could develop the gated sine. That system and one at the University of California provided two systems operating for many years. What they really bought was getting rid of the operator technique in the program. You've eliminated at least one of the variables. Probably, in the long run, it's worth it. I was just thinking about the economics of it.

For awhile, 1967 was the benchmark for the inflation factor. A few years ago, they were talking about the inflation from 1967 to 1980, whatever it was, '84 or '85, being about 3.2. I recall building that system, that open-loop system, when the electronics cost about $1,000-$2,000 a channel and an oscillograph cost about $7,000. The designing and the manufacturing of prototype equipment was probably another $15,000, so we were probably into the system for about $25,000-$30,000 back then. That was an open-loop system. It was not terribly sophisticated. You could not do anything but program square waves, and we did some things to modify the wave form a little bit, but given that inflation factor, you're probably looking at about a $90,000 system or the equivalent. Today's systems, even the closed-loop system, is probably not too bad a buy. You're looking at systems, closed-loop systems, in the range of $50,000, computer-coupled, so that you don't have to set and scale all of the data from the oscillograph traces and have somebody to verify all of the measurements and verify all of the calculations. It's a labor-intensive thing. Given what we did then, and given what there is now, I don't think it's really all that expensive.

J. Sorenson: From a public agency perspective, I'd like to look at pavement design and testing like on a 10-mile paving job. How much of an inch of pavement do we have to shave off to save $50,000-$100,000? It doesn't take long to pay for a little bit of preliminary engineering; whether it's in equipment or hardware or just further testing. I think that we don't want to lose our perspective on what good engineering skills and equipment needs are when we look at our total program needs. That, again, is what I think resilient modulus is tending to push us toward. But $50,000-$100,000 isn't much, even on one major job, much less what it is toward the cost of 250 jobs a year.

J. Sorenson: Marshall ended up with a couple of overheads. We'd like him to touch base on a question. **How do you explain or interpret resilient modulus results based on these view graphs?**

M. Thompson: I showed these the other day. These data are the result of the analysis that Witczak did several years ago. He pulled data together from the literature. As you can see, there is a relationship between K and n. If you have high K's, you have low n's; if you have low K's, you have high n's. My thought is that if we can do a reasonable job on establishing n, we can estimate K.

J. Sorenson: Thanks, Marshall. Jim Wilson, we've touched on this external vs. internal already today, but from your perspective again: **What's the significance of the external vs. internal LVDTs and the load cells on the AASHTO T-274?**

J. Wilson: As I showed in this data, which I want to hand out, the ratio between the total displacement and the axial is -- I've never measured it ever to be at unity. It's always been, even on bay muds, which are marine clays, very soft. You expect to have the measurements close to exactly the same number.

If you stop and think about it, there are problems with the platen to platen versus third point measurements. The platen to platen has two problems; one of them you can solve by conditioning. That eventually makes the specimen conform to the platens to some degree. The other problem is, to have a specimen to put in your cell, you have to trim the specimen, so you induce disturbance at the ends, and that's another kind of end effect. By trimming the ends of the specimen, you've changed the soil fabric and changed the density at the ends. The total LVDT displacement essentially just integrates all of the deformation from the ends and through the whole specimen. That's why you get a larger deformation number. There's really no way to get around trimming the specimen and putting it into the test frame. You can get rid of the mismatch between the sample and the end platens by conditioning, but you really see these problems when you're testing rock cores.

We went to a lot of trouble to prepare rock cores. We made a special fixture, put this test specimen in the fixture, put the fixture in a lathe with a four-jaw chuck, and used a dial indicator to dial indicate it in. It was now perfectly true. Then we machined the ends of the specimen with a diamond tip cutter, each end, and turned it around and indicated it in again so that we had both ends parallel to each other and perfectly flat. That helped a lot, but it still didn't totally solve the problem. We also used LVDTs to measure the platen to platen, not connected to the piston, but connected directly from the upper platen to the lower platen, so that essentially the only compression was between the specimen and the platen, the measurable compliance. I used two LVDTs 180° apart, plus third points, and we still had the differences there.

J. Sorenson: Maybe Newt will rebound on this a little differently, but I know he rose to this one last night. **Would you like to address the adjustment of the compactive effort on the bottom layer to come up with a more uniform sample specimen?**

N. Jackson: That's only because I got up and said: What do you do, just lean on it a little bit less? I think the question centers on the fact that if you want a uniformly dense specimen, you start preparing your specimen on the bottom lifts; you have more resistance at the bottom so that you get higher densities. Normally, you apply the same amount of effort all the way through the lifts. Obviously the way to solve that, or make those a little more uniform, is to back off just a little bit on the efforts you put on that bottom lift because you have more resistance at that point. We ease off a touch, but that's about it, with nothing too well defined. Anybody else want to comment on that?

J. Sorenson: I think some of the panel discussed trying to find the working range in advance of running the test and the frustrations that there are when you get halfway through the test and have it bomb out on you because you've misjudged that working range. Carl, did you want to respond?

C. Monismith: George Dehlen, a number of years ago, made an extensive study of granular materials in repetitive loading and developed a constitutive relationship assuming these materials to be non-linear elastic in response. He measured deformations with the LVDT holders attached to the specimens and made an extensive study of the confinement resulting from using a finite element idealization. His conclusion was that there was no significant influence in terms of the confining effect as far as the deformation measurements are concerned.

I personally would urge that when testing granular materials, deformations be measured by attachments to specimens because of the problem that Jim (Wilson) and other people have discussed. I think there are ways of getting around the adjustment problem. Systems can be developed to permit adjustments to be made outside of the cell rather than having to take apart the cell and adjust the LVDTs. These, to my way of thinking, are minor instrumentation problems that can be handled. I would encourage people to make their measurement on the specimens.

J. Wilson: I have one comment about that, Carl. We had a prototype system that was designed for NCHRP 1-10. I think you were on that committee, and we did just that. I designed a system so that you could, under pressure, release the lock nut and go inside and turn an allen wrench and readjust the LVDTs continuously under confining pressure. That can be done. It's a little bit of a nuisance, but it can be done.

(Comment from audience)

No, what I'm talking about is mechanically repositioning the core with respect to the coil back to mechanical zero. It's not a zero suppression electronically. You're right, that can't be done.

J. Sorenson: You may want to pursue this a little further later on. Newt, you liked that compactive effort question, we'll just give you another one. This all relates back to sample preparation. I think Newt's run quite a few samples, as a few of the other states have. **Do you want to touch on the use of various types of automatic compactors such as the Reinhardt automatic compactor with the rotating base, and the use of sheeps foot (tamping foot) for AASHTO T-274? And you may even want to expand on sample preparation a bit there.**

N. Jackson: I don't know why I got hung with this one simply because we tried a multitude of ways. I think if you build a sample that has a moisture content and the densities that you need, you can do it a variety of ways. The Reinhardt should work fairly well, it would seem. We do not have one like that, but we have tried a kneading compactor. Actually, I was trying to get this deferred to Marshall because he's done a lot more work than we have and done it more rigorously and academically than we have. Marshall, do you have some comments on that area, too? I think you did last night.

M. Thompson: I said almost everything I wanted to say yesterday. I think, on granular materials in particular, you can use several procedures as long as you get the same moisture and density with the exception of materials with "flaky particles." You're going to get a little different orientation of that stuff depending on full-faced vibratory compaction as opposed to any sort of an impact or kneading compaction. Once again, Carl emphasized it this morning, and I mentioned it to you yesterday afternoon. With cohesive materials, we really don't know what the heck we've normally got in the field. If you want to be conservative, either a "kneading" or an "impact-type compaction" will give you low-ball numbers that probably are going to keep you in reasonable shape in terms of assigning a modulus.

J. Sorenson: **Well, that leads right into another question on treated samples. Marshall (Thompson), cement, lime or fly ash; do you think that AASHTO T-274 is applicable in its present form?**

M. Thompson: I think T-274 is strictly for granular materials and cohesive soils. I would not recommend using that for high strength materials such as cement-treated aggregates, soil cement, stabilized base, etc. The critical thing in the design of those sections, in a real-world sense, is the strength of that material. You can ball-park the modulus and do a heck of a job on design. We currently are using a mechanistic-based concept for looking at high-strength stabilized base and design, which is material with a compressive strength over 500 psi. We estimate the modulus (in ksi) as 500 plus the compressive strength (in psi). Now the critical thing is strength as a function of time. That's where you really need to be concerned. I'd suggest measuring compressive strength and estimating modulus, which is the less critical part of the system. The strength is the critical thing in terms of fatigue behavior.

Those would be my thoughts, but there are schemes for measuring the modulus of cured high-strength materials. All of those work well. If you try to do it with diametral modulus testing, you get down to total displacements of a few micro-inches, and I get uptight about trying to measure something that small. We've used that scheme, but we strain-gauged the specimen diameter and assumed that the average strain times the gauge length is a good indication of displacement. We are very happy, pleased, and comfortable with that procedure.

J. Sorenson: Well, Marshall (Thompson), you're kind of on a roll, so we'll just keep throwing them at you. Maybe Carl (Monismith) or even Gary (Hicks) would like to add to this one. I personally would like to see this one put to rest. **What's the extent of establishing correlation values between M_r, CBR, and R values?**

M. THompson: I guess it's a matter of if you were to wander around on a modulus vs. deviator stress relation, and appropriately pick all of the numbers (not at the same deviator stress), you might get a solid correlation. That procedure, I think, is a reasonable "ball-park" estimate, but don't expect to get precise numbers. The TRRL group, in their Lab Report 1132 on flexible pavement design, has a relationship, but it's not 1500 times the CBR; it's something times CBR to a power. They recognize that it's not a linear relationship. I think those are ball-park estimates, and I would suggest you need to confirm those as opposed to picking one up out of the literature. We never had a lot of luck in doing that correlation.

J. Sorenson: It may stimulate some more response from these other two. **It's in the AASHTO Guide. The conversion's there.**

C. Monismith: One must be careful with using such correlations. If you study the original Shell data, you will note that 1500 times the CBR relationship is the mean relationship, and the values can vary from 750 to 3000 times the CBR; that is, plus or minus a factor of two. Similarly, with the R value, there are correlations. I view those as sort of last resorts. I would rather see you measure the moduli than estimate them. However, if you have nothing better, and you wish to use a mechanistic analysis, then the modulus vs CBR relation provides a start. But recognize, however, that there can be considerable variation.

For example, a number of years ago, we only had some CBR values for the subgrade soil at the airport in Salt Lake City. Some deflection measurements also were available with the result that a factor of 2200 or 2300 times the CBR was required to predict the modulus of the subgrade. You have to realize that the 1500 value is not a magic number and therefore should be treated with care.

J. Sorenson: I think we talked this morning about the difference between a static test and the condition we're measuring at one point, meaning R values to CRBs vs. something that's a moving test. I think that has a lot of bearing on why we can't get that correlation. We can't just draw off a straight graph like that. Maybe someone in the audience wants to kick in on this one, and we'll perhaps try to get Carl (Monismith) or Marshall (Thompson) to start this. **I'd like to see some discussion on the issue of Poisson's ratio. Should it be measured or should it be assumed, do we have the right test procedure equations to evaluate it accurately, and we'd like to hear some comment on the negative values that are also frequently obtained.** Gilbert (Baladi), we can hear you better if you're down front, if you get a chance to kick in on it. Do you want to try to touch that, Carl?

C. Monismith: One can measure a Poisson's ratio in a triaxial compression test by measuring both the axial and radial strains. I don't believe that Poisson's ratio can be determined from measurements in a diametral test, if you're measuring vertical and horizontal and other deformations, since the stress state is much more complex. If you're using a resilient modulus test, I think it's more realistic to assume a value for Poisson's ratio.

A number of years ago, as part of the project that Jim Wilson referred to, Dr. K. Nair performed fundamental measurements of Poisson's ratio; these have been published in an NCHRP report. From these measurements, Poisson's ratio varies from about 0.3 at 0°C to 0.5 at 60°C. At 20-25° a value in the range of 0.3 to 0.4 would appear reasonable. You also have to be careful when measuring Poisson's ratio to ensure that the deformation level is not too high. At large deformations at high temperatures, values greater than 0.5 will be obtained. Heukelom presented such data a number of years ago, and Jorge Sousa confirmed this from measurements on hollow cylindrical specimens at high temperatures and large deformations. If you obtain negative values, there's something wrong with your measurement system.

As noted earlier, in one resilient modulus diametral a complicated stress state exists. At higher temperatures asphalt concrete is not an elastic material. Under these conditions, the stress state will influence the deformations that you measure. Under these circumstances, the best thing to do is to compute a modulus by measuring the radial strains and assuming a value of Poisson's ratio.

J. Sorenson: Well, Ron (Terrel), down that same line on Poisson's ratio. Poisson's ratio has been and still is being backcalculated from the results obtained in the indirect tension test, horizontal and vertical deformations. The values for AC in 77° are reported at this workshop based on the results of indirect tensile test, and those are as follows: The Resilient Modulus is about 15 hundreds by two values and the total modulus is around 73 hundreds. I think we gave you a viewgraph on this, if you'd like to use that.

Could you or other members of the panel comment on the use of the indirect tensile test results to backcalculate modulus in the discrepancies noted above?

R. Terrel: Jim, since you did ask me to begin, I'd like to comment on this too. I think that Carl Monismith is right in that we are probably doing ourselves a disservice by thinking we have to do exhaustive testing on all these properties. With the given materials, they have base courses we are familiar with or a conventional class "B" Asphalt Concrete like we use on the West Coast. You can do this once, or even rely on other people's data as we just saw and probably make a better estimate than you can by testing it yourself because of the complications involved.

J. Sorenson: I think the point here is that there is still a lot to learn about resilient modulus and even about some of the basic inputs to our testing and test procedures. Dr. Baladi brought that up the other day. There were several questions in here about it and even with the experts. We need to come to some point of standardization, and we need to come to some point of conclusion. That's what AASHTO and ASTM do.

We have about 20-25 more questions here. What I'd like to do now is to document all of the questions we received. We're not guaranteeing answers to those we haven't addressed, but we'll assure you that AASHTO and ASTM will see the questions that were raised. We'll let those astute panels try to come up with some positions on some of them in their evaluation of the test procedures and on the direction that we're going in the resilient modulus area.

Unless there's a burning question that's been generated here, I'd like to give the panel one heck of a hand because I think they have taken quite a bit for us to throw a lot of these questions at them. Let's also give the audience a hand for its participation, and I thank you all.

Jim Wilson
AMI Consultants

Typical Resilient Modulus Test Data

No. of Cyc	Cyc Stress PSI	1/3 Pt. Micro Strain	Total Micro Strain	Modulus 1/3 Pt. PSI	Modulus Total PSI	Ratio 1/3 Pt. / Total	Effect Sigma 3 PSI
10	7.76	217	500	35760	15520	2.30	8.0
100	7.92	233	500	33990	15840	2.15	8.0
10	7.76	258	556	30080	13960	2.15	6.0
100	7.84	258	578	30390	13560	2.24	6.0
10	7.84	292	644	26850	12170	2.21	4.0
100	7.84	292	656	26850	11950	2.25	4.0
10	7.76	392	833	19800	9320	2.12	2.0
100	7.76	400	822	19400	9440	2.06	2.0

Note: Dry Density=125 PCF Moisture=6.0% Clayey silty sand SC

No. of Cyc	Cyc Stress PSI	1/3 Pt. Micro Strain	Total Micro Strain	Modulus 1/3 Pt. PSI	Modulus Total PSI	Ratio 1/3 Pt. / Total	Effect Sigma 3 PSI
10	7.92	267	578	29660	13700	2.16	8.0
100	7.84	267	578	29360	13560	2.17	8.0
10	7.84	308	622	25450	12600	2.02	6.0
100	7.92	325	611	24370	12960	1.88	6.0
10	7.84	383	689	20470	11380	1.80	4.0
100	7.84	375	689	20900	11380	1.84	4.0
10	7.84	483	822	16230	9540	1.70	2.0
100	7.84	475	844	16510	9290	1.78	2.0

Note: Dry Density=125 PCF Moisture=9.0% Clayey silty sand SC

No. of Cyc	Cyc Stress PSI	1/3 Pt. Micro Strain	Total Micro Strain	Modulus 1/3 Pt. PSI	Modulus Total PSI	Ratio 1/3 Pt. / Total	Effect Sigma 3 PSI
100	4.39	425	578	10330	7600	1.36	2.0
200	4.55	238	433	19120	10510	1.82	4.0
300	5.09	157	339	32420	15010	2.16	8.0

Note: Dry Density=102.5 PCF Moisture=6.6% Fine silty sand SM

This test is not a typical resilient modulus test

Cyc Stress PSI	1/3 Pt. Micro Strain	Total Micro Strain	Modulus 1/3 Pt. Million PSI	Modulus Total Million PSI	Ratio 1/3 Pt. / Total	Effect Sigma 3 PSI
488.5	179	1136	2.729	.430	6.3	142
740.9	258	1500	2.872	.494	5.8	142
971.5	320	2050	3.036	.474	6.4	142
1267.8	400	2560	3.169	.495	6.4	142
1525.8	462	2900	3.249	.519	6.4	142
2074.6	562	3690	3.688	.562	6.6	142
2590.5	660	4450	3.925	.582	6.7	142
3150.0	785	5200	4.013	.606	6.6	142

Note: This test was performed on rock using a steady state unpolar sinewave at .5 Hz. The 1/3 point measurements were made with resistive bridge strain gauges and the specimen density was 162.2 PCF.

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May 1, 1989

Mr. Jim Sorenson
Federal Highway Administration
Room 312 Mohawk Building
708 SW Third Avenue
Portland, OR 97204

Dear Mr. Sorenson:

Enclosed with this letter are some observation concerning round robin testing that I had planed to present at the conference and never got around to. I thought it may be of some interest to you, since you will most likely be involved in some of these types of test programs with state and federal agencies.

If you have any questions or need any additional information please contact Don Curphey or me.

Sincerely,
AMI Consultants, Inc.

James H. Wilson

James H. Wilson

enclosures

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ROUND ROBIN TEST MEASUREMENT CONSIDERATIONS

The following are some of the important factors that must be given consideration if the test results between laboratories are going to be compared:

A. Sample preparation and the type of compaction used static, impact, or kneading. It is important for all the laboratories to use the same methods of compaction due to the soil fabric differences exhibited by clayey soils. Static compaction produces a random soil structure while kneading compaction produces a dispersed structure and impact will produce a soil fabric between static and kneading compaction. The closest method to actual field conditions would be the kneading. It models the sheep foot better than the other methods.

B. Density gradients between layers in the sample. This potential problem can cause large one-third point measurement errors. For example, consider a case where the upper and lower LVDT clamps with a gauge length of 2 inches are placed one inch above and below a compaction joint. The surface of the lower portion of the joint may have a higher density than the portion above the joint if the sample preparation is not carefully performed. This will cause large displacements to be measured resulting in a low modulus value calculation for the sample.

C. The selected waveform should be the same for all laboratories. Many of the test results reported in the literature state load duration and the relaxation time or the period of the cycle without reporting the waveform. Is it a haversine, triangle or square wave? It is important to know when dealing with visco-elastic soils which are strain rate sensitive. The modulus measured using a square wave may very from a haversine or other waveforms.

D. Platen to platen (total displacement) and one-third point measurements. The ratios of the platen to platen to one-third point strains measured at the midpoint of the test specimen have a range and depend on the following factors. First, how well is the test specimen coupled to the platens?

If the ends of specimen are not perfectly flat, there will be stress concentrations that will cause larger strains between the ends of the specimen and the platens.

Second, sample disturbance of the exists at the ends due to trimming. To what distance from the ends of the sample is the disturbance, what are the effects of the new density gradient and how do the effects of trimming alter the natural soil fabric? In general, the higher the modulus of the test specimen the more pronounced these errors become. The summation of these factors help to explain why the resilient modulus as measured at the sample one-third points is always higher than the resilient modulus as determined from platen to platen measurements. For low modulus specimens, 2 to 15 KSI, the ratio may be on the order of 1.25 to 1.75 : 1. Specimens with a modulus of 15 to 35 KSI may have a ratio of 1.75 to 2.3 : 1, and specimens with a modulus on the order one-million PSI will have resilient modulus ratios as determined by the two methods of measurement of 5 to 10 : 1.

E. Calibration of the test equipment. One of the basic problems with test equipment is the frequency of calibration. The calibration of the equipment should be done as a complete measurement SYSTEM, not as components. There should also be, as a minimum, annual calibration of all control and measurement sub-systems. This calibration should be traceable to at least a secondary transfer standard certified by the National Bureau of Standards (NBS). Most calibrations are performed under static conditions. It would be advisable to check the test system under actual dynamic loading conditions with an oscilloscope or oscillograph to be sure of its dynamic performance.

F. Last remember, <u>you will always get a signal</u> from the system which <u>can be translated into a modulus number</u>! The question is, how valid is the number given the quality of the measurement. The signal from which the modulus number is derived is an integration of the test data, mechanical response of the transducers, electrical cables, signal conditioning, filters, or the measurement portion of a computer analog to digital interface.

The engineer in charge of the system, should have a good understanding of how to make measurements with electronic devices, both analog and digital. This person should also have a very good understanding of the properties of materials, such as linearity, elasticity, isotropy, and visco-elastic material behavior. This will be of great help in the understanding of the test data and their validity.

Nondestructive Testing Equipment and Back-Calculation Applications

Robert L. Lytton

OUTLINE OF PRESENTATION

Introduction

A. Pavement layer properties that are needed for evaluation, design, and management of pavements

   1. Layer thickness, binder content, elastic stiffness, density

   2. Fatigue and permanent deformation properties

B. Measurement methods

   1. Near field measurements

      a. Static or slowly moving loads
      b. Vibratory loads
      c. Impulse loads

   2. Far field measurements

      a. Vibratory loads
      b. Impulse loads

Analysis Methods

A. Historical methods

   1. Scrivner, Swift, Yih Hou

   2. Equivalent layer methods

B. Microcomputer methods

   1. Measured deflection

   2. Layer thickness and load

   3. Seed moduli

   4. Deflection calculation

   5. Error check

   6. Constitutive relations

7. Stress and strain level corrections

8. Search for new moduli

9. Controls of the range of moduli

C. Systems identification methods

D. Impulse methods for near-field measurements

Corrections

A. Non-linear stress strain curves

1. Load level

2. Strain level

B. Temperature

C. Frequency

Errors and Their Reduction

A. Random errors (reduce by repeating measurements)

1. Instrument error

2. Spatial variation of material properties

B. Systematic error (reduce by realistic selections)

1. Deflection calculation method

2. Constitutive relations

3. Stress and strain level corrections

4. Seed moduli selection

5. Closure tolerances

6. Interpreting anomalous results

Conclusions

A. Laboratory tests are models of the real thing, the in-situ moduli, and are as much in need of critical assessment and reduction of systematic and random errors as NDT measurements

B. Nondestructive testing of pavements in the field

1. Requires an expert or an expert system to control the size of random and systematic errors

2. Offers promise of providing reliable and repeatable measurements of pavement layer properties

3. Back-calculation method chosen should match the analysis method used in design

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SESSION V - FUTURE OF RESILIENT MODULUS TESTING **Page**

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REPEATED LOAD TESTS ON UNTREATED SOILS
A FLORIDA EXPERIENCE

Robert K. H. Ho
Soils Materials Engineer
Florida Department of Transportation

Workshop on Resilient Modulus Testing
Oregon State University, Corvallis, Oregon
March 28-30, 1989

Abstract

Two granular subgrades and one base material were tested for resilient modulus values using three methods. They are AASHTO T-274-82, ASTM(draft) method and a so-called Modified method in which the conditioning stresses are applied statically. All samples were 4-inch diameter by 8-inch high and were prepared by tamping compaction equivalent to modified proctor energy. Internal and external LVDTs were used to measure resilient deformation of the samples. Behavior of samples during conditioning stages were observed and the resilient modulus values computed from the three methods using internal and external LVDTs were compared. The repetitive plate load test facility at the Central Laboratory of the Florida Department of Transportation was introduced and future plate load tests may be correlated with triaxial tests using relationships established by Burmister and Lee (Seed et al 1967).

Acknowledgement

The writer gratefully appreciates the assistance of his staff in the testing of soil materials, preparation of the figures and reviewing of the manuscript. He is particularly grateful to J. Stewart, K. Rao and M. Nelson who worked under tremendous pressure to meet the published deadline.

SI Units Conversion Table

1 inch	=	2.540×10^{-2} meter (m)
1 foot	=	3.048×10^{-1} meter (m)
1 pound	=	4.448 newton (N)
1 psi	=	6.895×10^{3} pascal (Pa)

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REPEATED LOAD TESTS ON UNTREATED SOILS A FLORIDA EXPERIENCE

Robert K.H. Ho*

Introduction

The new AASHTO guide for Design of Pavement Structures requires the use of resilient modulus values (M_R) in place of soil support values. In response to this need, the Florida Department of Transportation Materials Office had been addressing the problem of resilient modulus testing of untreated soil materials.

In 1976, the Florida Department of Transportation(FDOT) Materials Office acquired an MTS electrohydraulic closed loop test system which consisted of separate Soils, Bituminous and Structural loading frames with independent control consoles. Florida was involved in the VESYS Program with Federal Highway Administration, Washington, D.C. in the late seventies characterizing soils and asphaltic concrete properties. In the soils area, base and subgrade materials were tested.

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Test Equipment and Sample Preparation

The MTS soils testing system is being used to conduct resilient modulus tests on untreated soils. Cyclic loading functions of sine, haversine, triangle, square and ramp waves can be programmed with frequencies between .00001 hertz and 990 hertz. It can be preprogrammed for a given number of cycles by a counter panel. The system has the capability of being operated in load, stroke or strain control selectable by simple push buttons. A 4-channel strip chart recorder registers load, stroke, and two strain signals from transducer conditioners. Each strain signal represents the average of two LVDT readings.

The sample size is 4-inch in diameter and 8-inch high. Samples were prepared by compacting soil material in a 4-inch by 8-inch mold with a collar using equivalent modified proctor compaction energy (AASHTO T-180). This requires 8 layers with 27 blows per layer. The material is compacted at or about 1% dry of optimum. After compaction, the sample is extruded from the mold and placed on the pedestal of the triaxial cell. Two internal LVDTs are mounted on clamps on the middle half of the sample. Two external LVDTs are also mounted on the piston to record resilient deformation of the entire 8-inch long sample. The drainage valve to the specimen is left open during testing.

Triaxial Test Procedures

Since Florida started resilient modulus testing over 10 years ago, several different test procedures have been encountered. Their differences generally lie in the magnitude and sequence of conditioning stresses as well as testing stresses and number of cyclic loads. Three test methods were used in this study - AASHTO T-274-82, ASTM(draft) and a so called Modified method. AASHTO T-274-82 requires different stress conditioning levels for cohesive and cohesionless soils with 200 repetitions for each stress condition. The draft ASTM method requires only one conditioning stress with 1000 repetitions for both cohesionless and cohesive soils. AASHTO T-274 subjects the sample to considerably higher conditioning stresses than the proposed ASTM method. The Modified method subjects the sample to static conditioning stresses equal to the testing stresses for three 10-minute cycles before repetitive loads up to 10,000 cycles are applied. The conditioning and testing stress sequences are tabulated below for the three test procedures.

Table 1

AASHTO T-274-82 Method

Conditioning and Testing Stress Sequences

Cohesionless

Conditioning

σ_3	σ_d	Reps
5	5, 10	200 ea.
10	10, 15	
15	15, 20	

Testing

σ_3	σ_d	Reps
20	1, 2, 5, 10, 15, 20	200 ea.
15	1, 2, 5, 10, 15, 20	
10	1, 2, 5, 10, 15	
5	1, 2, 5, 10, 15,	
1	1, 2, 5, 7.5, 10	

Cohesive

Conditioning

σ_3	σ_d	Reps
6	1,2,4, 8, 10	200 ea.

Testing

σ_3	σ_d	Reps
6, 3, 0	1	200 ea.
6, 3, 0	2	
6, 3, 0	4	
6, 3, 0	8	
6, 3, 0	10	

All stresses are in psi.

Table 2

ASTM METHOD (DRAFT)

Conditioning Stresses

σ_3	σ_d	Reps
6	1	1000

Testing Stresses

σ_3	σ_d	Reps
6	1,2,5,10	200 each
3	1,2,5,10	
1	1,2,5,10	

Same for both cohesionless and cohesive soils. All stresses are in psi.

Table 3

Modified Method

Conditioning and Testing Stresses

σ_3	σ_d	Reps
1	2	10,000 max.
2	2	10,000 max.
2	4	10,000 max.
5	2	10,000 max.
5	5	10,000 max.

One sample is used for each stress combination.

All stresses in psi.

Axial Deformation Measurements

AASHTO T-274-82 recommends that materials with resilient modulus greater than 15,000 psi be measured by LVDTs attached to the sample inside the triaxial chamber. Materials with resilient modulus less than 15,000 psi may be tested with LVDTs clamped to the piston rod outside the test chamber. This brought up the question of deciding which system to use if the estimated modulus of a soil material is around 15,000 psi. For comparison purposes, it was decided to test preliminary samples using both internal and external LVDTs.

To calibrate the internal and external LVDTs, a solid 4-inch diameter, 8-inch high dummy polyethylene cylinder was used. Two external LVDTs were mounted on the piston and two internal LVDTs were mounted on the top and bottom platen of the triaxial cell. A load of about 13 psi (150lbs) was applied and both LVDTs readings were taken. The top clamp of the internal LVDT was moved to mid height of the dummy and readings were taken under the same load. Results were tabulated below:

External LVDT	Internal LVDT	Gage Length
.0033"	.0030"	8"
.0025 (8")	.0012 (4")	

The readings seemed to indicate that compression between the piston and top platen is negligible. For this solid dummy sample, the LVDT readings confirmed that the resilient deformation is proportional to the gage length of the sample.

The next step involved the testing of two soil material types, limerock and subgrade with both internal and external LVDTs to compare the test results under static and dynamic conditioning and varying stress levels. The two soil types were chosen in an attempt to differentiate between the stronger limerock and the weaker subgrade material.

Repetitive Plate Load Tests

To digress a moment from the triaxial test, the FDOT had been using the repetitive rigid plate load test to evaluate new sources of base materials for over 20 years. This testing facility consists of a test pit with testing surface area of 8 feet wide by 12 feet by 8 feet deep (fig. 1). Another newer test pit measures 8 feet by 24 feet. A sump with interconnecting reservoirs surrounds the test pit and the water table can be raised or lowered to saturate or drain the soil material from below. The bottom of the pit is filled with about 12 inches of gravel with filter fabric separating it from the one foot of builders sand and 2 feet of fine subgrade sand. On top of the subgrade is the base material under evaluation. The base

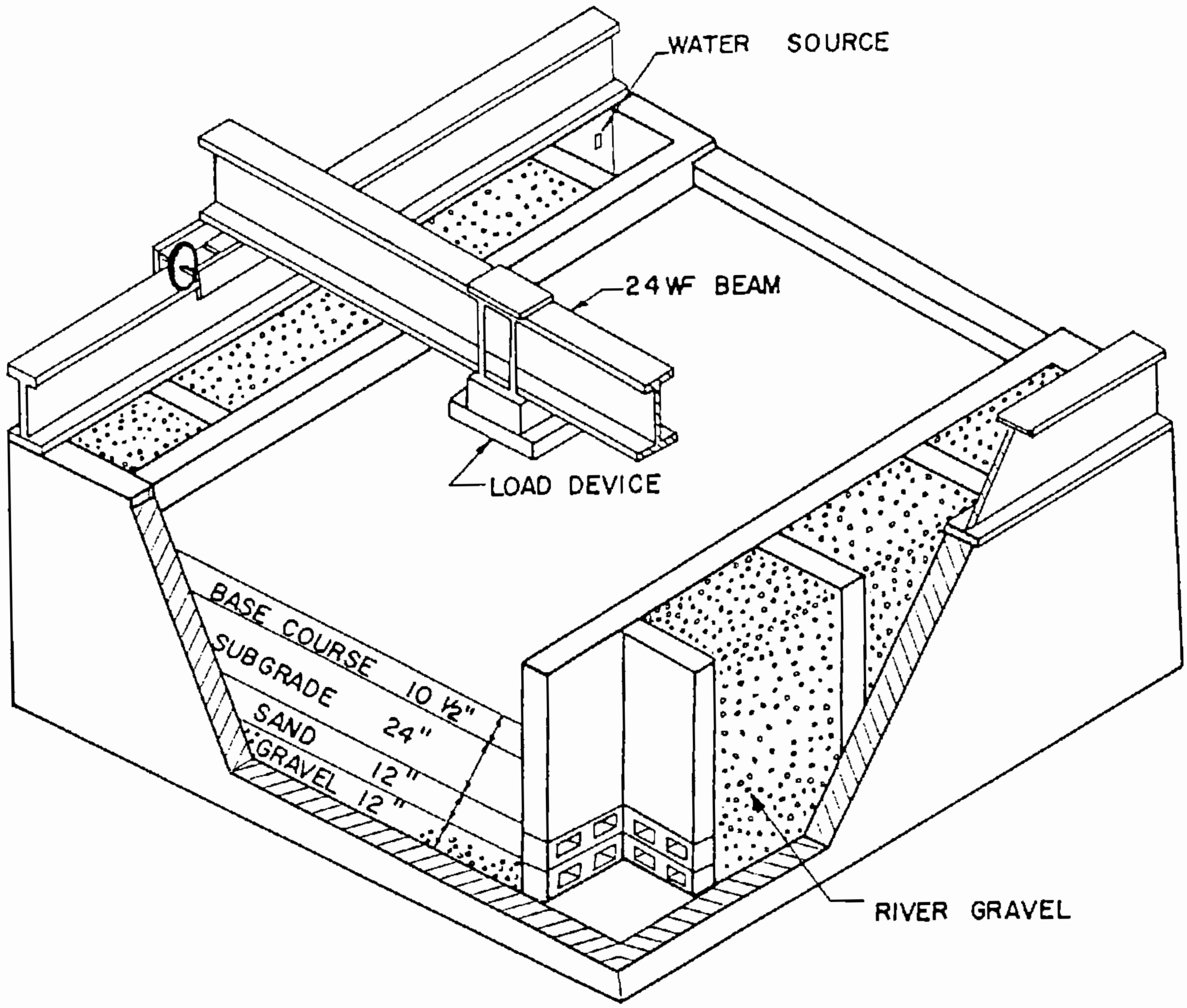


FIGURE 1. A SECTIONAL VIEW OF THE TEST-PIT

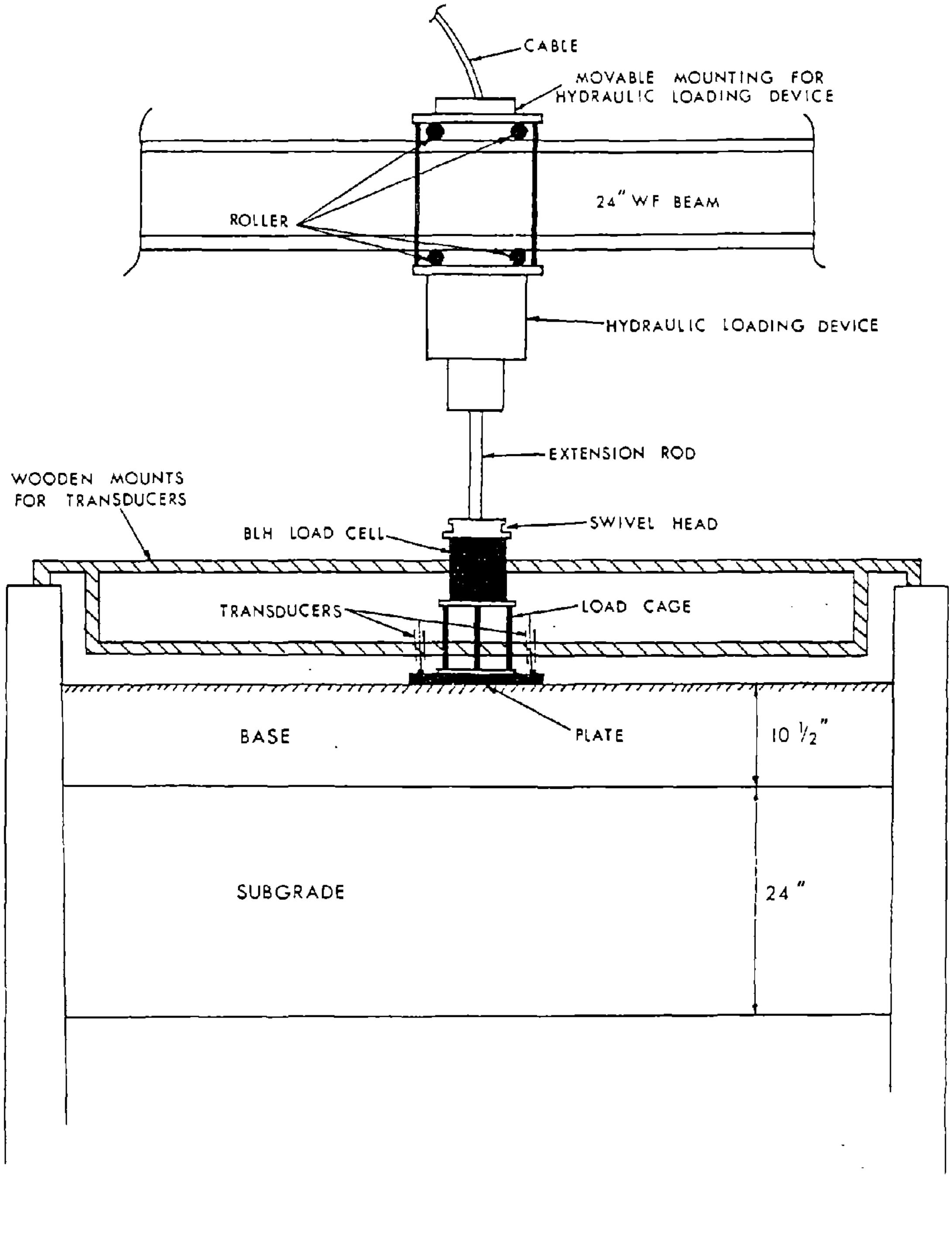


FIGURE 2. A SCHEMATIC DIAGRAM OF THE LOADING SYSTEM

material is generally 10 1/2 inches thick and placed in two lifts with each layer compacted to a minimum 98% of maximum modified proctor density. Load is applied through a 12-inch diameter rigid plate with 2 LVDTs to monitor total and resilient deflections (fig.2).

The loading system was made by MTS with similar controls as the triaxial MTS system. The base material is evaluated under three moisture conditions: as compacted (optimum), saturated and drained. Total and resilient deformations are recorded in terms of number of cyclic loads. 10,000 cycles can be applied in a normal 8-hour working day.

The Boussinesq's equation for a rigid circular plate with a uniform pressure gives

$$\Delta = 1.18 \frac{pr}{E} \qquad \text{for } u=.5 \qquad (1)$$

where Δ = deflection

p = applied pressure

r = radius of plate

E = modulus of elasticity

u = Poisson's ratio

Since the base and subgrade soils are not homogenous, the concept of equivalent resilient modulus E_{eR} and resilient deformation Δ_{eR}, was introduced and the above equation is re-written as (Cox, 1971)

$$\Delta_{eR} = 1.18 \frac{pr}{E_{eR}} \quad (2)$$

Hence the equivalent resilient modulus computed from these tests is a composite of the base and subgrade moduli of the pavement system.

Theories relating results of laboratory triaxial compression tests to field plate load tests have been presented by Skempton and Burmister and extended by Lee to include resilient deformation (Seed et al, 1967). For the same moduli, the applied stresses of the triaxial and plate load tests are related by

$$\sigma_d = 0.29 \sigma_p \quad (3)$$

where σ_d = deviator stress in triaxial test

σ_p = uniform load on plate

Future repetitive plate load tests using minimum two feet thick subgrade or limerock base material may be performed to correlate with triaxial tests using the above stress relationship (equation 3).

Soil Materials and Properties

Two soil material types were tested in this program - stabilized subgrade soil and limerock.

Stabilized subgrade consists of A-2-4 or A-3 silty sand or fine sand stabilized with shell or limerock. The specification requirement for stabilized subgrade is to have a minimum LBR value of 40.

Limerock (not a rock) is actually weathered limestone and is commonly used in Florida as a base material under flexible pavements. It is a granular, non-plastic, well graded material. Its carbonate content is greater than the required 70% and generally goes over 90%. Approved limerock base material has to have Limerock Bearing Ratio values (LBR) greater than 100. (Note: Numerically LBR = 1.25 CBR and the LBR test is a modification of the CBR test to suit Florida soil conditions.)

Typical subgrade and limerock base material tested include:

1) 20849-S Dark grey sand with limerock
LBR = 40
Max. Density = 112.5 pcf
Optimum Moisture = 10.5%
Minus 200 = 9%

2) 20891-S Grey sand with shell

LBR = 45

Max. Density = 116.0 pcf

Optimum Moisture = 10.5%

Minus 200 = 14%

3) 21077-S Limerock

LBR = 148

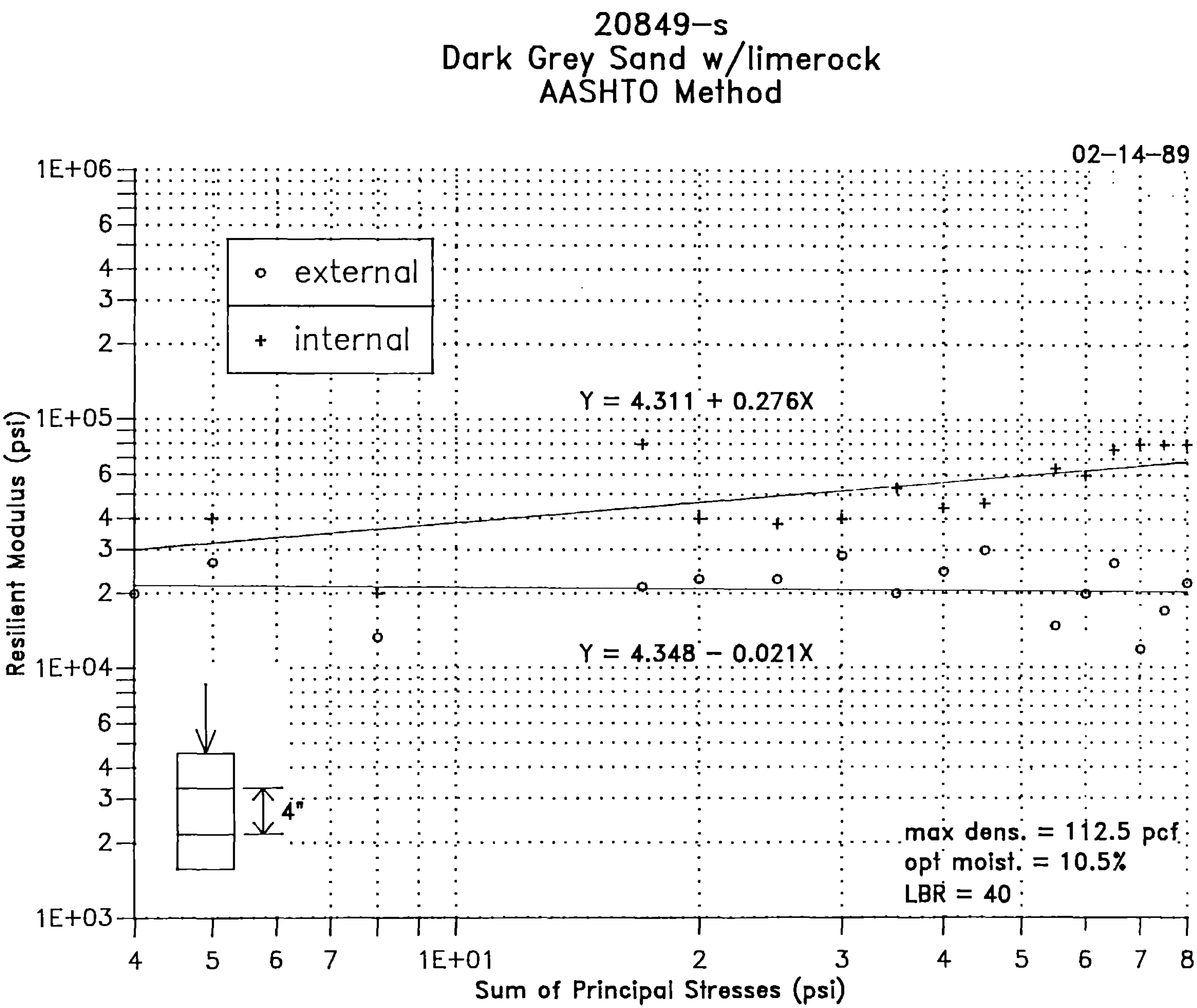
Max. Density - 116.8 pcf

Optimum Moisture = 12.3%

All tests results were compacted as close to maximum density and optimum moisture as practicable.

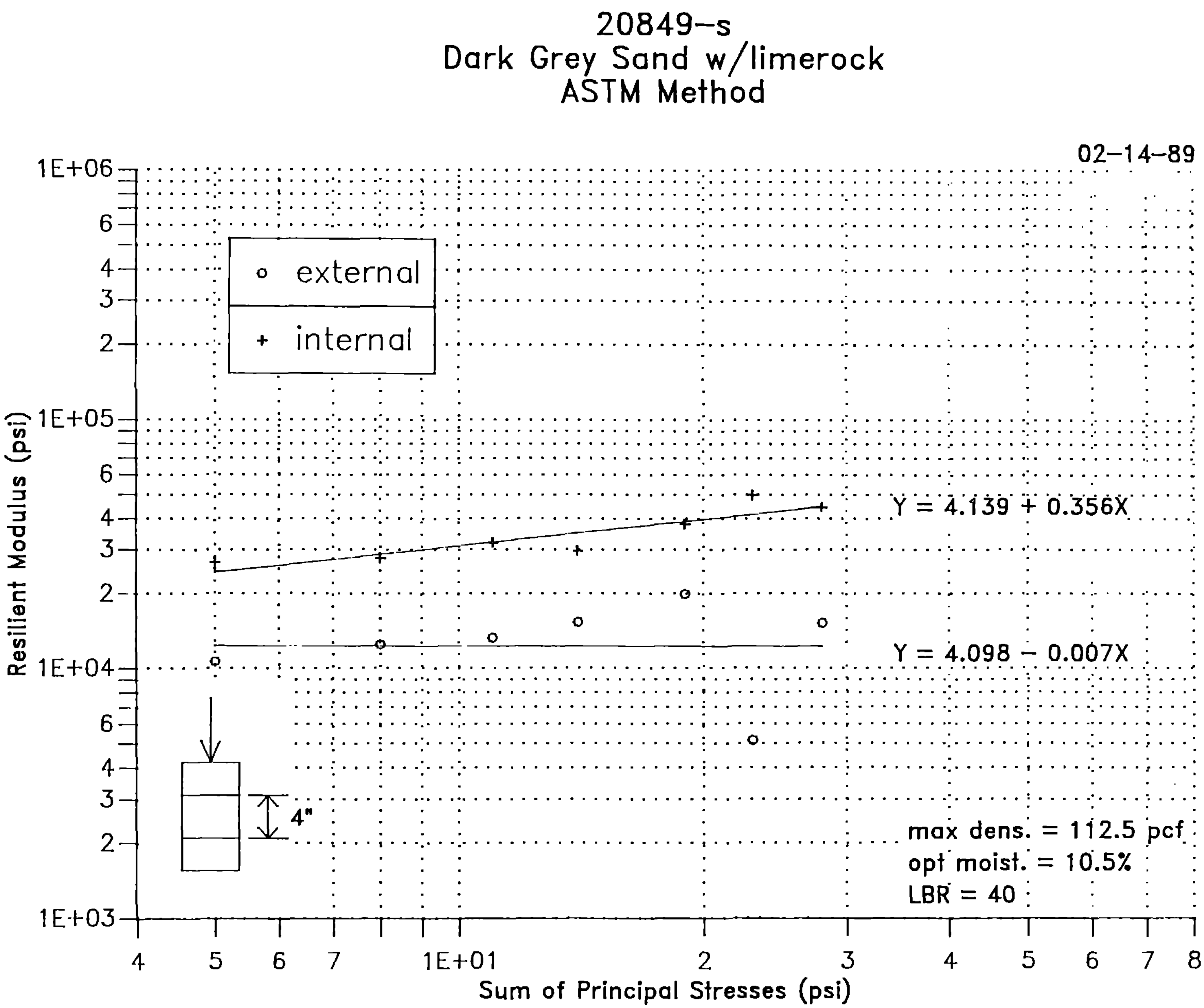
Triaxial Test Results

Typical log-log plots of resilient modulus vs sum of principal stresses are shown for each material. (Sum of principal stresses equals $\sigma_1 + \sigma_2 + \sigma_3 = \sigma_d + 3\sigma_3$). A separate plot is included for each test method for subgrade material 20849-S. (Figures 3, 4 & 5) The subgrade (20849-S) failed under $\sigma_3 = 5$ psi, $\sigma_d = 5$ psi during Modified method. The test data for this subgrade are summarized in tables 4 and 5 for both AASHTO and ASTM (draft) methods respectively.



20849-s
Dark Grey Sand w/limerock
AASHTO Method
02-14-89
external
internal
Y = 4.311 + 0.276X
Y = 4.348 - 0.021X
Resilient Modulus (psi)
Sum of Principal Stresses (psi)
4"
max dens. = 112.5 pcf
opt moist. = 10.5%
LBR = 40

FIGURE 3



20849–s
Dark Grey Sand w/limerock
ASTM Method
02–14–89
external
internal
Y = 4.139 + 0.356X
Y = 4.098 – 0.007X
4"
max dens. = 112.5 pcf
opt moist. = 10.5%
LBR = 40
Resilient Modulus (psi)
1E+06
1E+05
1E+04
1E+03
Sum of Principal Stresses (psi)
1E+01

FIGURE 4

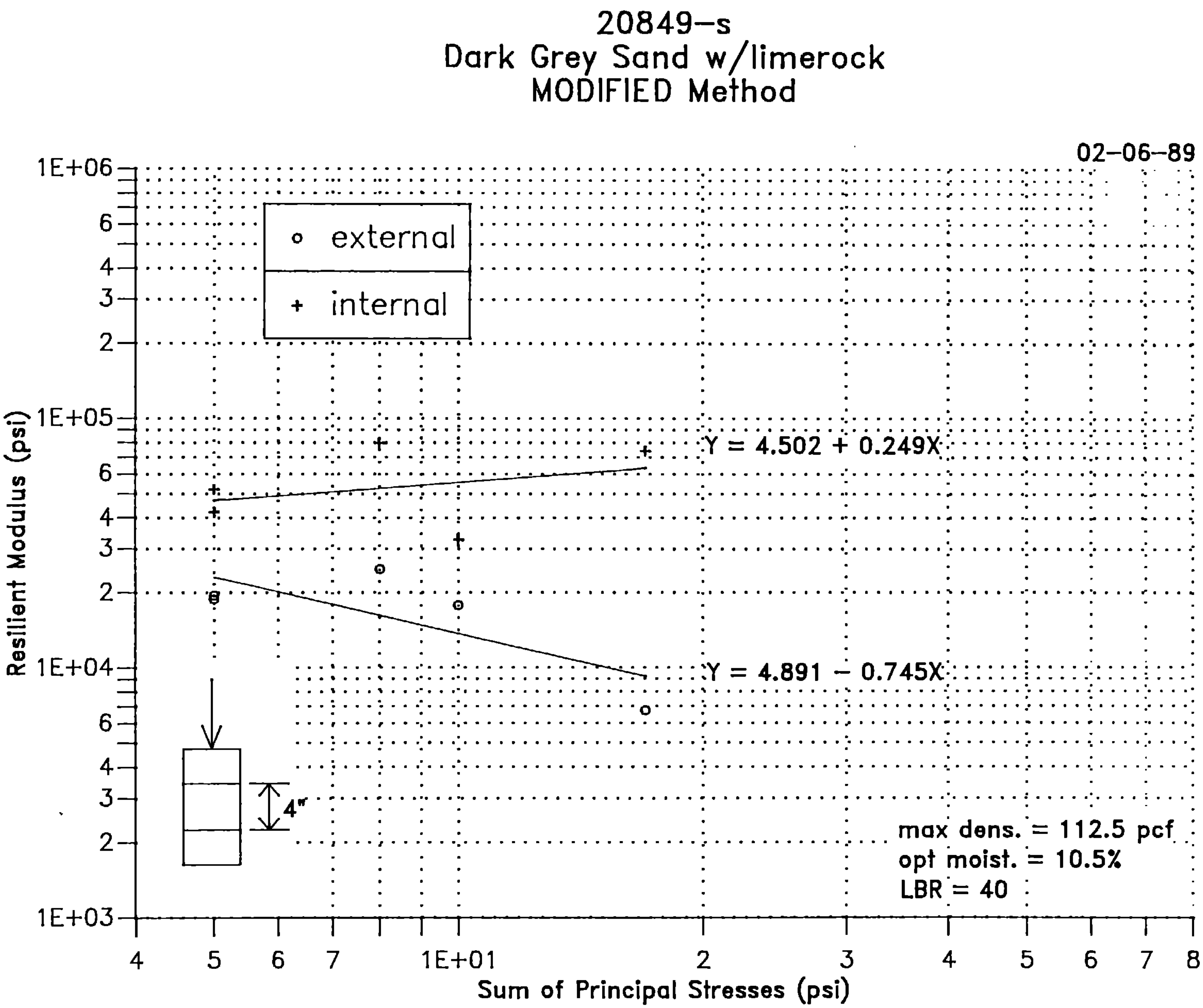

20849–s
Dark Grey Sand w/limerock
MODIFIED Method
02–06–89
external
internal
Y = 4.502 + 0.249X
Y = 4.891 – 0.745X
4"
max dens. = 112.5 pcf
opt moist. = 10.5%
LBR = 40
Resilient Modulus (psi)
Sum of Principal Stresses (psi)
1E+06
1E+05
1E+04
1E+03
1E+01

FIGURE 5

TABLE 4

TEST TYPE: AASHTO SOIL DESCRIPTION: Gray Sand w/rock

SOIL TYPE: Stab. Subgrade

DATE: 2-14-89

SAMPLE # 3

LAB # 20849-S

CONDITIONING INFORMATION:
LOAD TYPE =DYNAMIC
OF REPETITIONS = 200

STRESS LEVELS

CONFINING(psi)	DEVIATOR(psi)
5	5
5	10
10	10
10	15
15	15
15	20

INTERNAL LVDT'S (at 4" apart)

CHAM. PRESSURE (psi)	DEV. STRESS (psi)	CHART READ. (div)	SCALE (in/div)	REC. DEFORM. (in)	# REPS	STRAIN (in/in)	RES. MODULUS (psi)
20	2	0.10	0.0005	0.000050	200	0.000013	160000
20	5	0.25	0.0005	0.000125	200	0.000031	160000
20	10	1.00	0.0005	0.000500	200	0.000125	80000
20	15	1.50	0.0005	0.000750	200	0.000188	80000
20	20	2.00	0.0005	0.001000	200	0.000250	80000
15	2	0.10	0.0005	0.000050	200	0.000013	160000
15	5	0.40	0.0005	0.000200	200	0.000050	100000
15	10	1.25	0.0005	0.000625	200	0.000156	64000
15	15	2.00	0.0005	0.001000	200	0.000250	60000
15	20	2.10	0.0005	0.001050	200	0.000263	76190
10	2	0.10	0.0005	0.000050	200	0.000013	160000
10	5	0.75	0.0005	0.000375	200	0.000094	53333
10	10	1.80	0.0005	0.000900	200	0.000225	44444
10	15	2.60	0.0005	0.001300	200	0.000325	46154
5	1	0.10	0.0005	0.000050	200	0.000013	80000
5	2	0.20	0.0005	0.000100	200	0.000025	80000
5	5	1.00	0.0005	0.000500	200	0.000125	40000
5	10	2.10	0.0005	0.001050	200	0.000263	38095
5	15	3.00	0.0005	0.001500	200	0.000375	40000
1	1	0.20	0.0005	0.000100	200	0.000025	40000
1	2	0.40	0.0005	0.000200	200	0.000050	40000
1	5	2.00	0.0005	0.001000	200	0.000250	20000
1	7.5	FAILED	0.0005	ERR	200	ERR	ERR
1	10	FAILED	0.0005	ERR	200	ERR	ERR

EXTERNAL LVDT'S (at 8" apart)

CHAM. PRESSURE (psi)	DEV. STRESS (psi)	CHART READ. (div)	SCALE (in/div)	REC. DEFORM. (in)	# REPS	STRAIN (in/in)	RES. MODULUS (psi)
20	2	5.50	0.0010	0.005500	200	0.000688	2909
20	5	6.75	0.0010	0.006750	200	0.000844	5926
20	10	6.75	0.0010	0.006750	200	0.000844	11852
20	15	7.00	0.0010	0.007000	200	0.000875	17143
20	20	7.25	0.0010	0.007250	200	0.000906	22069
15	2	4.00	0.0010	0.004000	200	0.000500	4000
15	5	4.60	0.0010	0.004600	200	0.000575	8696
15	10	5.40	0.0010	0.005400	200	0.000675	14815
15	15	6.00	0.0010	0.006000	200	0.000750	20000
15	20	6.00	0.0010	0.006000	200	0.000750	26667
10	2	1.00	0.0010	0.001000	200	0.000125	16000
10	5	2.00	0.0010	0.002000	200	0.000250	20000
10	10	3.25	0.0010	0.003250	200	0.000406	24615
10	15	4.00	0.0010	0.004000	200	0.000500	30000
5	1	0.50	0.0010	0.000500	200	0.000063	16000
5	2	0.75	0.0010	0.000750	200	0.000094	21333
5	5	1.75	0.0010	0.001750	200	0.000219	22857
5	10	3.50	0.0010	0.003500	200	0.000438	22857
5	15	2.10	0.0020	0.004200	200	0.000525	28571
1	1	0.20	0.0020	0.000400	200	0.000050	20000
1	2	0.30	0.0020	0.000600	200	0.000075	26667
1	5	1.50	0.0020	0.003000	200	0.000375	13333
1	7.5	FAILED	0.0020	ERR	200	ERR	ERR
1	10	FAILED	0.0020	ERR	200	ERR	ERR

TABLE 5

TEST TYPE: ASTM

SOIL DESCRIPTION: Dark Grey Sand With Limerock

SOIL TYPE Stab. Subgrade

DATE: 2-14-89

SAMPLE # 2

LAB # 20849-S

CONDITIONING INFORMATION:
LOAD TYPE =DYNAMIC
DEV. STRESS = 1 psi
CHAM. PRESSURE = 6 psi
OF REPETITIONS = 1000

INTERNAL LVDT'S (at 4" apart)

CHAM. PRESSURE (psi)	DEV. STRESS (psi)	CHART READ. (div)	SCALE (in/div)	REC. DEFORM. (in)	# REPS	STRAIN (in/in)	RES. MODULUS (psi)
6	1	0.15	0.0001	0.000015	1000	0.000004	266667
6	1	0.25	0.0001	0.000025	200	0.000006	160000
6	2	0.40	0.0001	0.000040	200	0.000010	200000
6	5	4.00	0.0001	0.000400	200	0.000100	50000
6	10	9.00	0.0001	0.000900	200	0.000225	44444
3	1	1.00	0.0001	0.000100	200	0.000025	40000
3	2	2.50	0.0001	0.000250	200	0.000063	32000
3	5	6.75	0.0001	0.000675	200	0.000169	29630
3	10	5.25	0.0002	0.001050	200	0.000263	38095
1	1	0.60	0.0002	0.000120	200	0.000030	33333
1	2	1.50	0.0002	0.000300	200	0.000075	26667
1	5	3.60	0.0002	0.000720	200	0.000180	27778

EXTERNAL LVDT'S (at 8" apart)

CHAM. PRESSURE (psi)	DEV. STRESS (psi)	CHART READ. (div)	SCALE (in/div)	REC. DEFORM. (in)	# REPS	STRAIN (in/in)	RES. MODULUS (psi)
6	1	8.00	0.0005	0.004000	1000	0.000500	2000
6	1	8.00	0.0005	0.004000	200	0.000500	2000
6	2	12.50	0.0005	0.006250	200	0.000781	2560
6	5	7.75	0.0010	0.007750	200	0.000969	5161
6	10	5.25	0.0010	0.005250	200	0.000656	15238
3	1	1.00	0.0010	0.001000	200	0.000125	8000
3	2	1.20	0.0010	0.001200	200	0.000150	13333
3	5	2.60	0.0010	0.002600	200	0.000325	15385
3	10	4.00	0.0010	0.004000	200	0.000500	20000
1	1	0.80	0.0010	0.000800	200	0.000100	10000
1	2	1.50	0.0010	0.001500	200	0.000188	10667
1	5	3.20	0.0010	0.003200	200	0.000400	12500

The second subgrade (20891-S) could not be conditioned under AASHTO method. The sample failed under initial dynamic conditioning level of σ_3 = 5 psi and σ_d = 5 psi. An attempt was made to statically condition the material under the AASHTO conditioning levels. This was to be followed by the dynamic AASHTO test procedure. Again, the sample failed at the initial conditioning level of σ_3 = 5 psi and σ_d = 5 psi. Using the proposed ASTM procedure, the material did not fail until the final stress level, σ_d = 5 psi and σ_3 = 1 psi. The Modified method was performed at σ_3 = 1 psi, σ_d = 2 psi and σ_3 = 5 psi, σ_d = 2 psi. Results of the draft ASTM and Modified methods are presented on the same graph for subgrade 20891-S (fig. 6).

Limerock was tested using the AASHTO and Modified methods. The draft ASTM method was not performed because it was felt that the low confining pressures of this method are inappropriate for base material. Modified method was run only at stresses comparable to higher levels of AASHTO procedure. Results of the two methods are shown on the same plot (fig. 7).

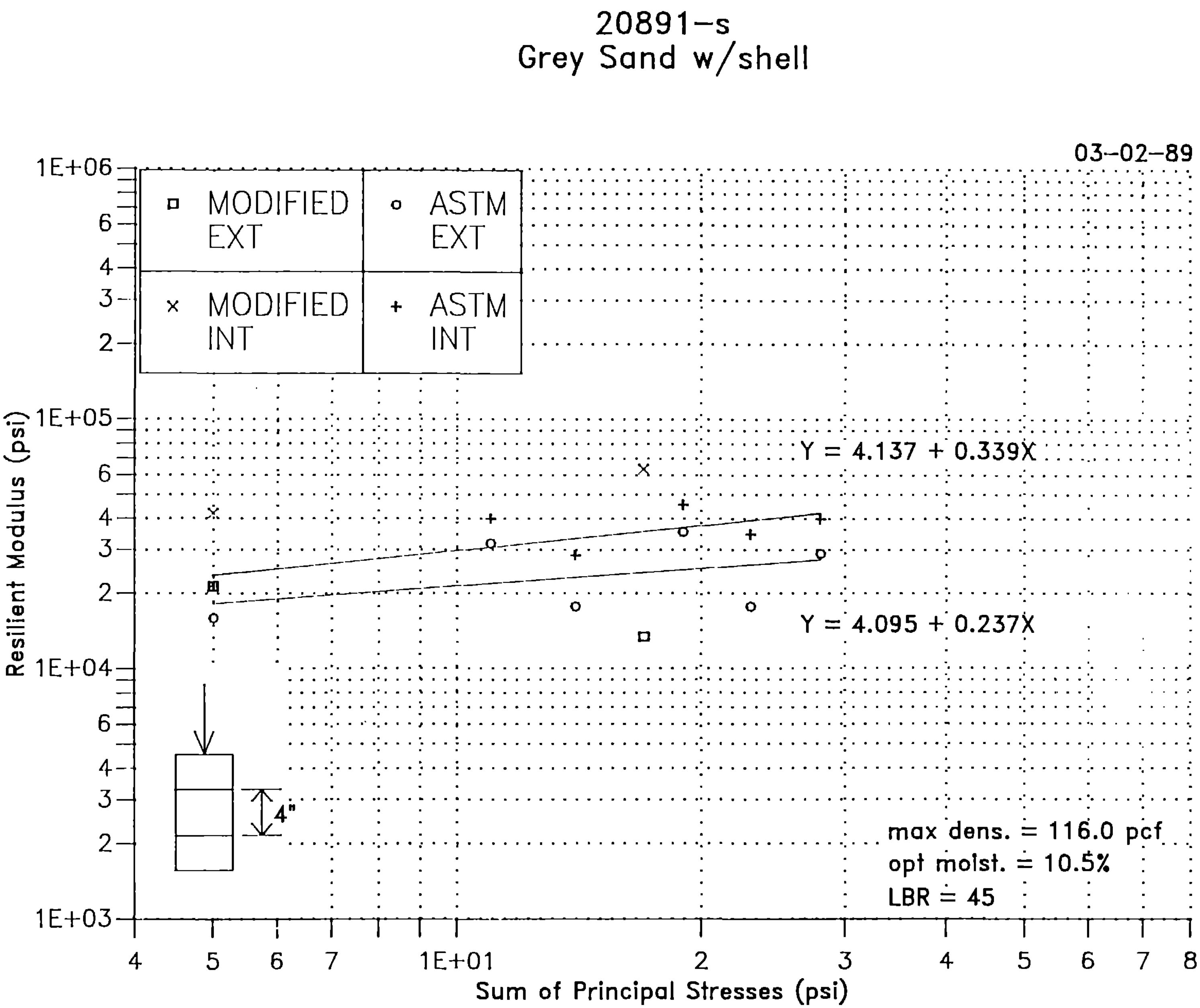

20891-s
Grey Sand w/shell
03-02-89
MODIFIED EXT
ASTM EXT
MODIFIED INT
ASTM INT
Y = 4.137 + 0.339X
Y = 4.095 + 0.237X
max dens. = 116.0 pcf
opt moist. = 10.5%
LBR = 45
4"
Resilient Modulus (psi)
1E+06
6
4
3
2
1E+05
6
4
3
2
1E+04
6
4
3
2
1E+03
4
5
6
7
1E+01
2
3
4
5
6
7
8
Sum of Principal Stresses (psi)

FIGURE 6

21077-s
Limerock

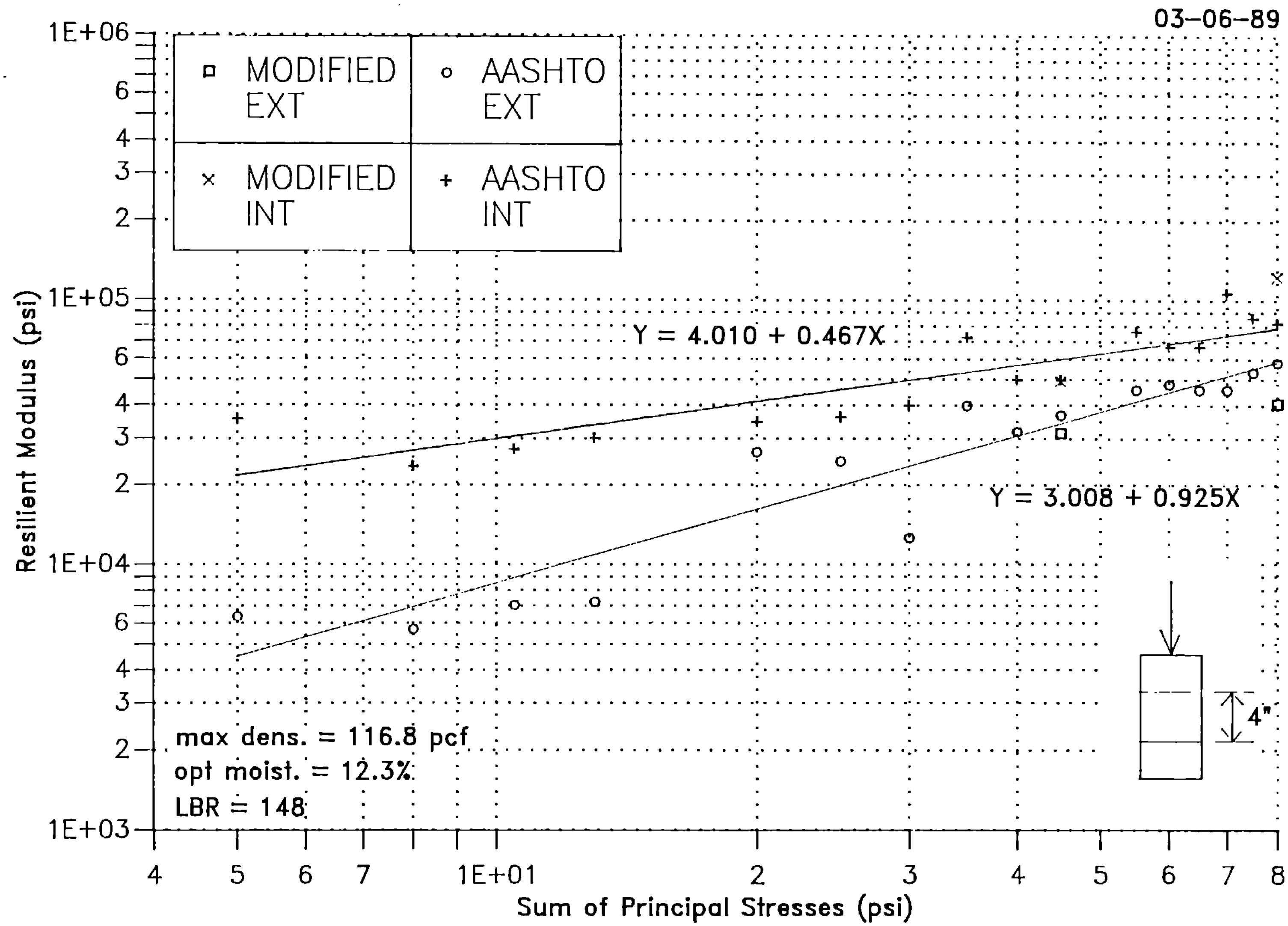


FIGURE 7

Discussion of Test Results

The results obtained for subgrade materials seem to indicate that portions of the AASHTO procedure are inappropriate for the types of granular material typically found as subgrade in Florida. The conditioning procedure is too severe. Additionally, the higher confining pressures of 15 and 20 psi seem unrealistic for subgrade materials.

From the Modified method, the modulus of subgrade materials generally appears to be independent of the number of repetitions (fig 8). For limerock, however, results indicate a tendency for the resilient modulus to increase as the number of repetitions increases (fig. 9). The results of the Modified method which are plotted on log-log plots represent 200 repetitions. This was done to facilitate comparison with results from other test methods.

In all cases, internal measurements result in higher modulus values (lower resilient deformation) than external measurements. This is to be expected since external measurements include end effects. However, it is unclear how much of the difference is due to end effects and how much is due to method of compaction, stress ratio, sample size, or other factors. In an initial attempt to better define the difference between internal and external measurements, a

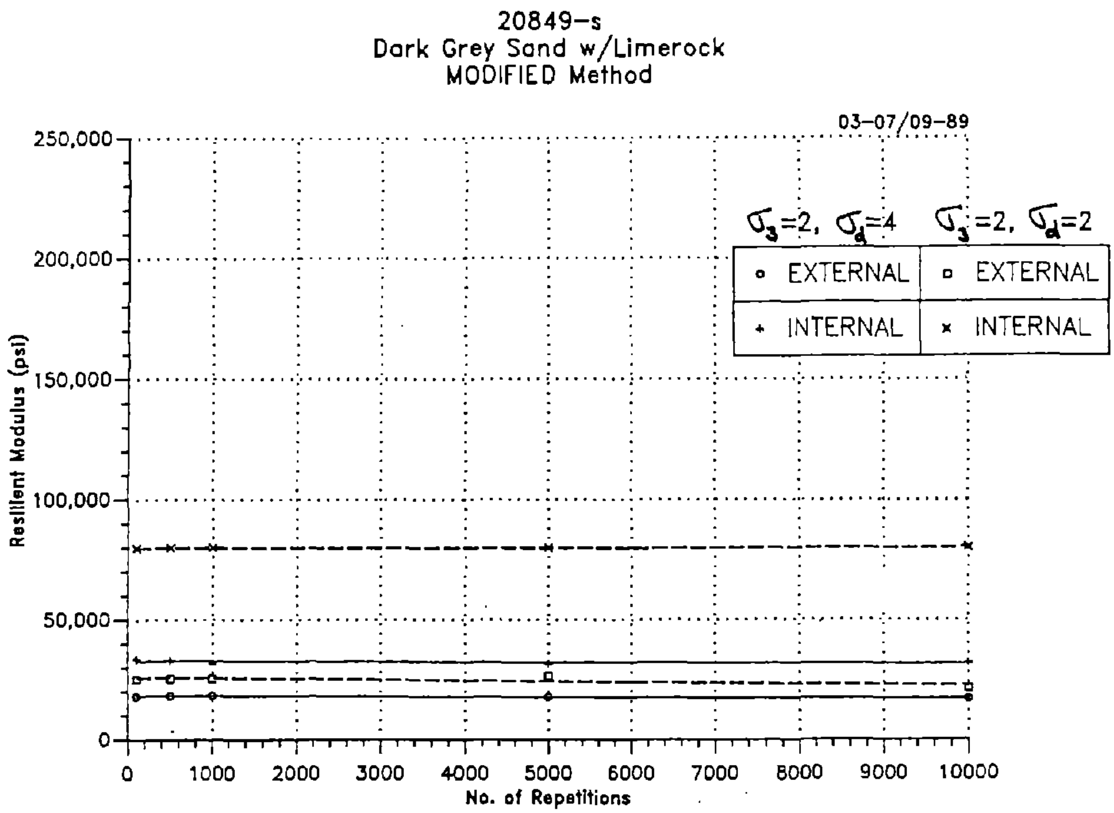


FIGURE 8

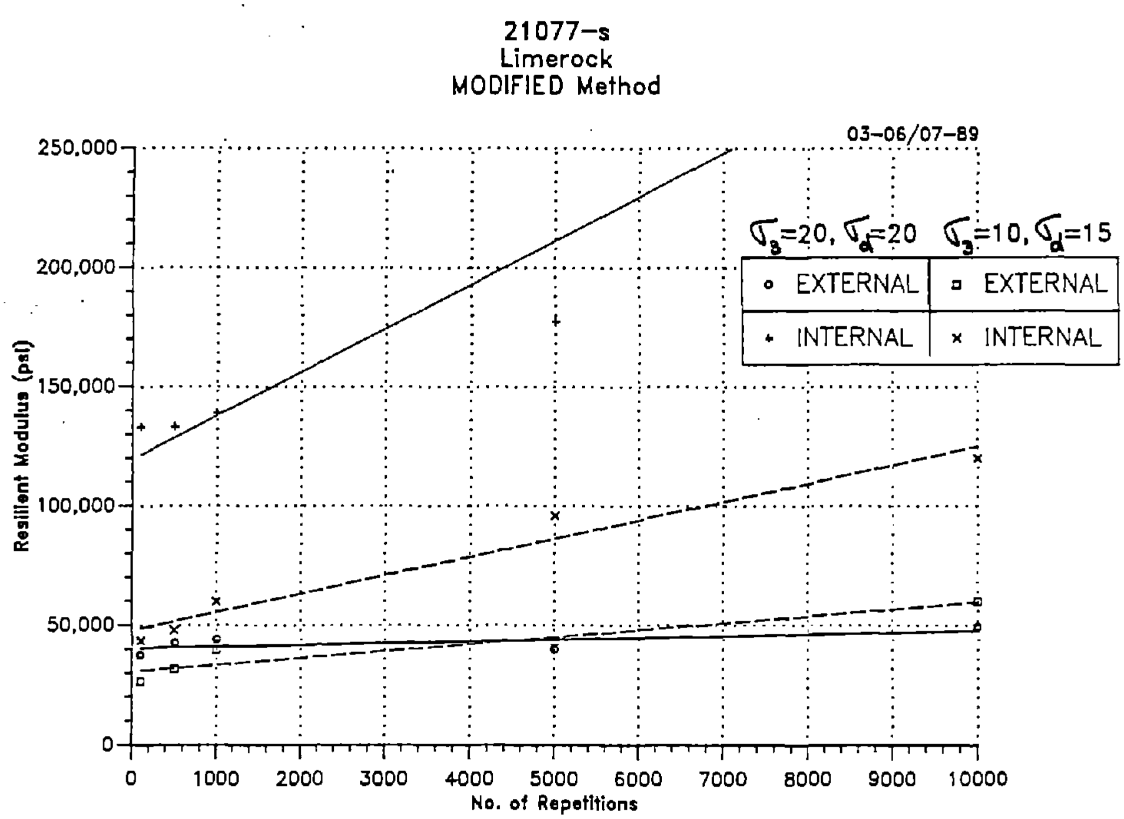


FIGURE 9

limerock sample was tested with internal LVDT's placed to measure strain over the bottom 4 inches of the sample (fig. 11). The results were then compared with those from a test on equivalent material in which strains were measured internally over the middle 4 inches (fig. 10). AASHTO method was used for both tests. While external measurements were within the same range, the difference between external and internal measurements was significantly reduced when considering resilient deformation for the lower half of the sample.

AASHTO T-274-82 specifies that ring clamp LVDT holders can only be used on minimum 4-inch diameter samples. Therefore, for the same subgrade material, modulus values computed from external LVDT readings on 2.8-inch diameter samples could be much less than those obtained with internal LVDT's on 4-inch diameter samples because of possible end effects.

In all three methods and for both subgrade and limerock base materials, low deviator stresses (1 & 2 psi) applied to samples under high confining pressures (low σ_1/σ_3 ratio) do not appear to stress the specimens sufficiently to yield realistic modulus values. For the limerock samples, deviator stresses of 1 and 2 psi often did not produce any measurable strain. Generally, test results have been disregarded if the principal stress ratios were less than about 1.5.

20125 #1
Limerock

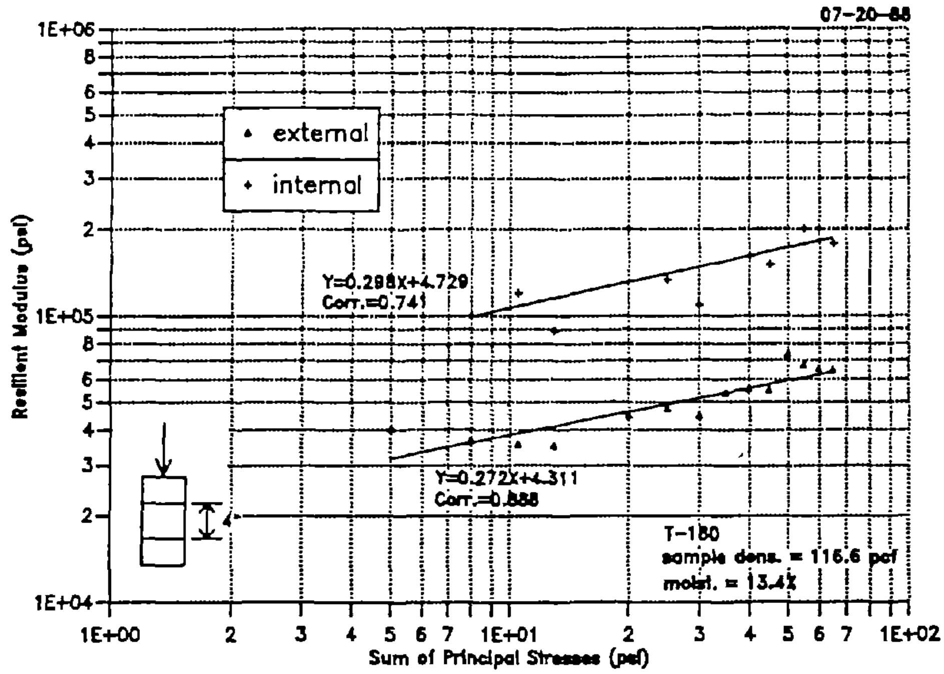


FIGURE 10

20125 #2
Limerock

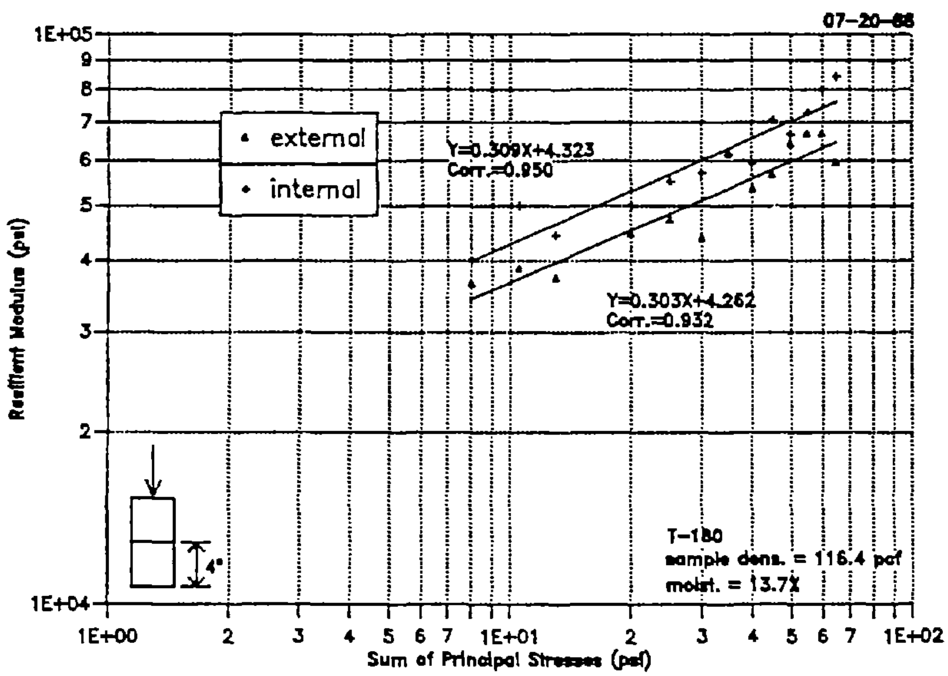


FIGURE 11

Summary

Salient points of the FDOT study to date can be summarized as follows:

1) At, or close to, optimum moisture of the soils tested, the M_R values of the subgrade material seem to be independent of the number of repetitions (up to 10,000). While the limerock base data indicated an increase in M_R values with the number of repetitions, the internal LVDT readings yielding much higher rate of M_R increase than those from external LVDT readings.
2) M_R values computed from external and internal LVDT readings can be significantly different.
3) Conditioning stresses based on AASHTO T-274-82 can be too severe for Florida subgrade soils.
4) Low deviator stresses of 1 and 2 psi or σ_1/σ_3 ratios less than 1.5 yields unrealistic high M_R values which should be neglected from consideration.

Currently proposed future work includes:

1) Additional testing on various materials.
2) Definition of a test method and measurement device suitable for Florida soils.
3) Comparison with test pit results.

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RESILIENT MODULUS TESTING IN KENTUCKY
(PRESENT AND FUTURE)

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Workshop on Resilient Modulus Testing
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RESILIENT MODULUS TESTING IN KENTUCKY (PRESENT AND FUTURE)

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INTRODUCTION

The behavior of asphalt bound layers, unbound aggregate bases, and foundation soils (subgrade) can be affected by such variables as gradation, asphalt and/or moisture content, type of aggregate, density, method of compaction, temperature, magnitude of loading, duration of each load cycle, and other less significant factors. The complex interaction of all these variables will yield a composite behavior for a particular pavement structure that can manifest itself in some form of distress or possibly even failure.

Flexible pavements are known to be susceptible to rutting. Rutting is the result of a large percentage of wheel passes that occur within a relatively narrow path on the pavement surface.

In the early 1970's, the Kentucky Department of Transportation initiated a research study with the general objective of determining the rutting characteristics of the more common paving materials used in Kentucky. This study was the impetus for purchasing the equipment and developing the techniques for the dynamic testing of paving materials.

DESCRIPTION OF EQUIPMENT

To determine the permanent deformation (plastic) characteristics of paving materials, as well as the resilient modulus, a dynamic testing machine was purchased from Structural Behavior Engineering Laboratories in Phoenix, Arizona. It is identified as a Model STD-1000. The unit is capable of loading to 1,250 pounds (5.560 kN). Stress intensity, duration of stress, and frequency can be varied. Load and unload times (for this particular machine) can be varied in a continuous spectrum from 0.1 second to 10 seconds. A photograph of the equipment is shown in Figure 1.

Strains were measured by two DC-DC LVDT's. These were mounted on opposite sides of the specimen. The LVDT's were held in place by two aluminum rings that were mounted on the specimen. This method of mounting the LVDT's did not work

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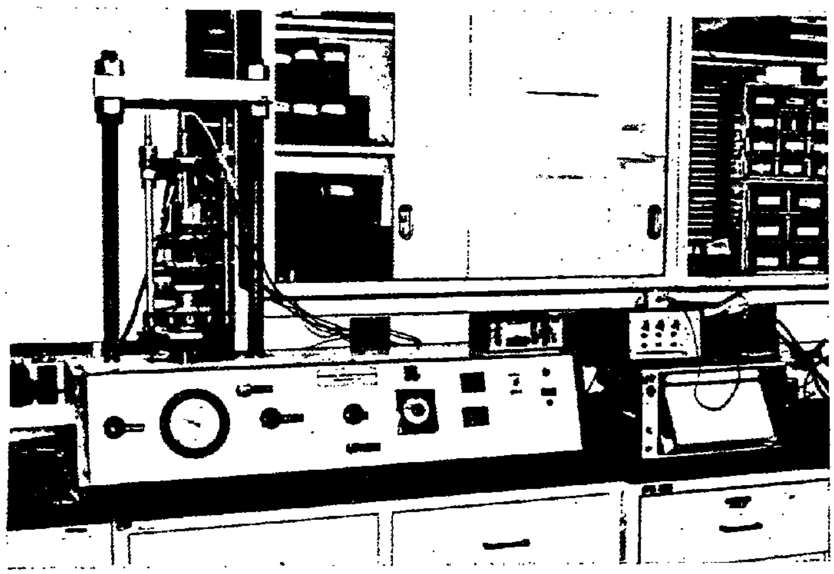

Figure 1. Repeated-Load Equipment.

well for very soft soils, because the soil tended to deform under the weight of the mounting rings and the LVDT's. The strains were recorded on a dual-pen, Hewlett-Packard strip chart recorder. A Hewlett-Packard DC voltage power supply was used to excite the LVDT's.

Load was monitored by a bonded strain gage load transducer, with a 3,000-pound capacity. The particular transducer used was manufactured by Transducers, Incorporated. Signal conditioning and recording of the load was accomplished by a Sanborn oscillographic recorder.

The specimens were tested in a modified triaxial chamber, manufactured by Karol-Warner, Inc. The chamber permitted the unbound aggregate bases and the soils to be tested with confining pressure. Also, when testing asphaltic concrete specimens, a heating or cooling coil was placed in the chamber to provide temperature control. A circulating temperature bath was used to circulate water at the appropriate temperature through the coil in the chamber. Temperature was monitored by a thermistor in the chamber.
In the last 5 years, data acquisition from this equipment

has been accomplished with an IBM PC-XT, with a Techmar data acquisition board. Control of the loading sequence on the specimen is still from the internal trigger of the STD-1000.

TESTING PROCEDURES

Largely for convenience and the time constraints on running a large number of tests to many cycles of loading, the loading program that has normally been used in testing is 1.0 second of load dwell time with 1.0 second of unloading time. It is recognized this may not be sufficient recovery time for highly viscoelastic materials such as asphaltic concrete at higher temperatures.

Figure 2 illustrates an "idealized" strain-time curve obtained from this equpment, using the load-unload time sequence just described. The resilient strain is defined as the strain that is recovered in the "rest" period between load cycles. Therefore, the resilient modulus is calculated as the dynamic stress divided by the resilient strain. Usually an average of several values, calculated at different locations during the test, is reported. The first 100 to 200 cycles are never used in the calculation. This is usually considered a conditioning period.

The plastic or nonrecoverable portion of strain, illustrated in Figure 2, is used to develop rutting models for the paving materials.

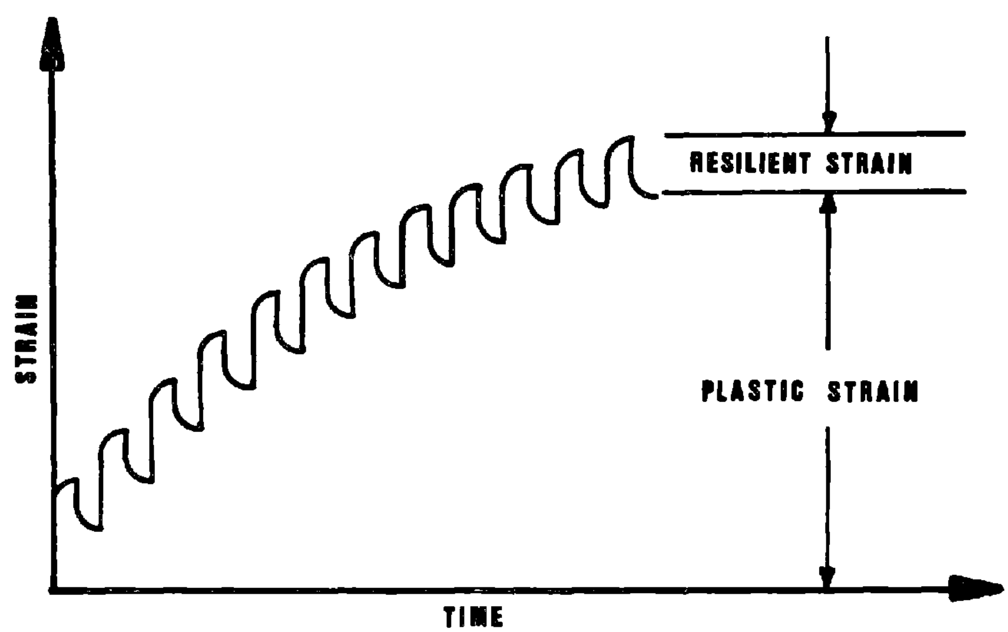

Figure 2. Strain-Time Curve From Repeated-Load Test.

Asphaltic Concrete Testing - The asphaltic concrete specimens were cylindrical in shape. They were 8.0 inches (203 mm) in height and 4.0 inches (101 mm) in diameter. They were placed on the base plate of the triaxial chamber. The LVDT's were mounted on the specimen, and the top platen was placed on the specimen. The copper heating/cooling coil was then placed over the specimen and LVDT's and connected to the base of the triaxial chamber. It was then connected by a flexible tube to the outlet fitting in the top cap of the triaxial chamber. The top cap of the chamber was bolted in place, and the load ram was inserted into the chamber. The chamber was placed in the loading frame, and the circulating water bath was connected to the heating/cooling coil fittings. The load cell was placed between the end of the load ram and the cross head of the loading frame. All electrical connections were then made to their respective excitation and recording devices. The temperature control fluid was allowed to circulate for at least 24 hours, to ensure the specimen had reached the testing temperature. Most specimens received 10,000 cycles of loading; however, for some fatigue specimens, they were tested to complete failure.

Aggregate and Soil Testing - Procedures for testing aggregates and soils were similar to those for asphaltic concrete. The major difference is no temperature control was used for aggregates and soils, but confining pressure was used. Aggregates and soils were tested at various levels of confining pressure and various degrees of saturation. When the specimens were tested at anything less than 100 percent saturation, then a total stress analysis was used. If an effective stress analysis was desired, then the specimens were saturated, and a pore pressure transducer was used to measure pore pressure.

The load transducer was placed inside the triaxial chamber for the aggregate and soil tests. This was done because the chamber's o-ring seal around the load ram caused about four pounds of additional load to be read by the load transducer, to overcome the friction of the seal. This load was considered significant for aggregates and soils. The load sequence was the same as that used for the asphaltic concrete specimens.

The aggregate specimens were 4.0 inches (101 mm) in diameter and 8.0 inches (203 mm) in height. The soil specimens were 6.0 inches (152 mm) in height and 2.8 inches (71 mm) in diameter.

RUTTING MODELS

To develop rutting models, large numbers of laboratory specimens of asphaltic concrete, dense-graded aggregate, and a number of Kentucky soils were compacted and repeated-load tests were performed. The asphaltic concrete was identified as a "typical" Kentucky Class I base course. Tests on the base material were performed at three test temperatures and three stress levels, using procedures previously discussed. Typical results from those tests are shown in Figure 3. An empirical model of the following form was developed to describe rutting in the asphaltic concrete.

$$\log e_p = C_0 + C_1(\log N) - C_2(\log N)^2 + C_3(\log N)^3$$

where e_p = permanent strain (change in length/initial length),

N = number of stress repetitions,

C_3 = 0.00938,

C_2 = 0.10392,

C_1 = 0.63974,

C_0 = a function of temperature and stress,

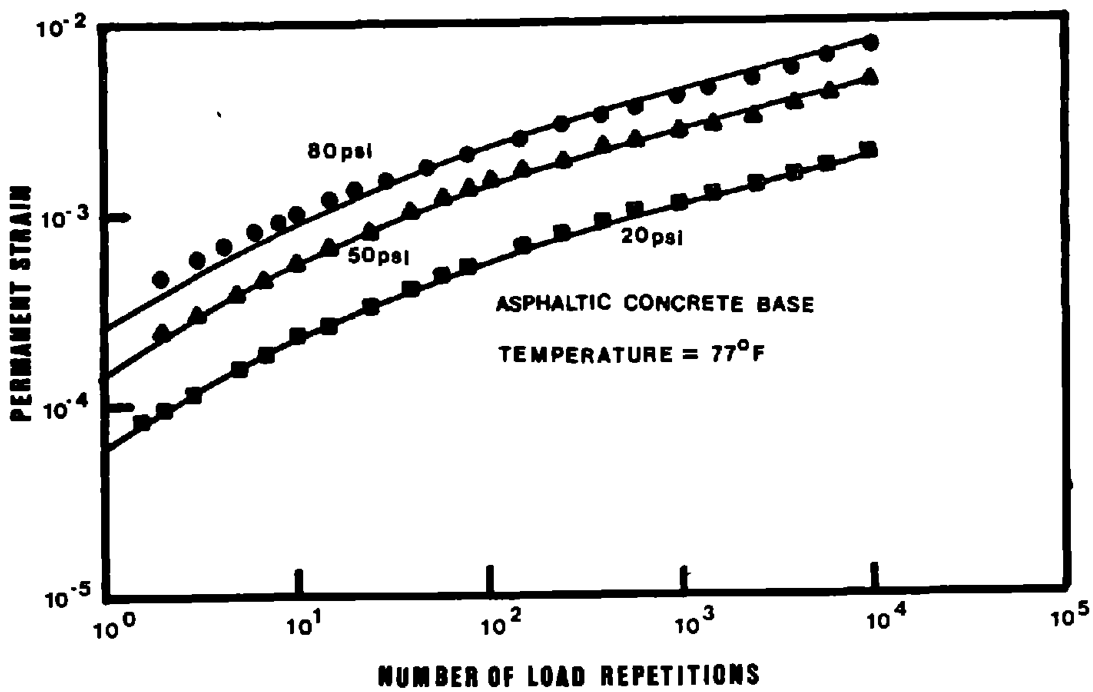

Figure 3. Permanent Strain as a Function of Load Repetitions.

Details of testing procedures and development of these models have been previously reported by Allen and Deen (1980) in the proceedings of the Association of Asphalt Paving Technologists (1).

Models describing the rutting behavior of the dense-graded aggregate and the soils were also developed from the laboratory repeated-load tests. These tests were performed at three longitudinal stress levels, three confining pressures, and three degrees of saturation. Typical data are shown in Figure 4. The empirical models that were developed for the dense-graded aggregate and the soils are as follows:

$$\log e_p = C_0 + C_1(\log N) + C_2(\log N)^2 + C_3(\log N)^3$$

where e_p = permanent strain,

N = number of load repetitions,

C_1, C_2, C_3 = constants that are functions of moisture content,

C_0 = a constant that is a function of moisture content, confining pressure, and longitudinal stress.

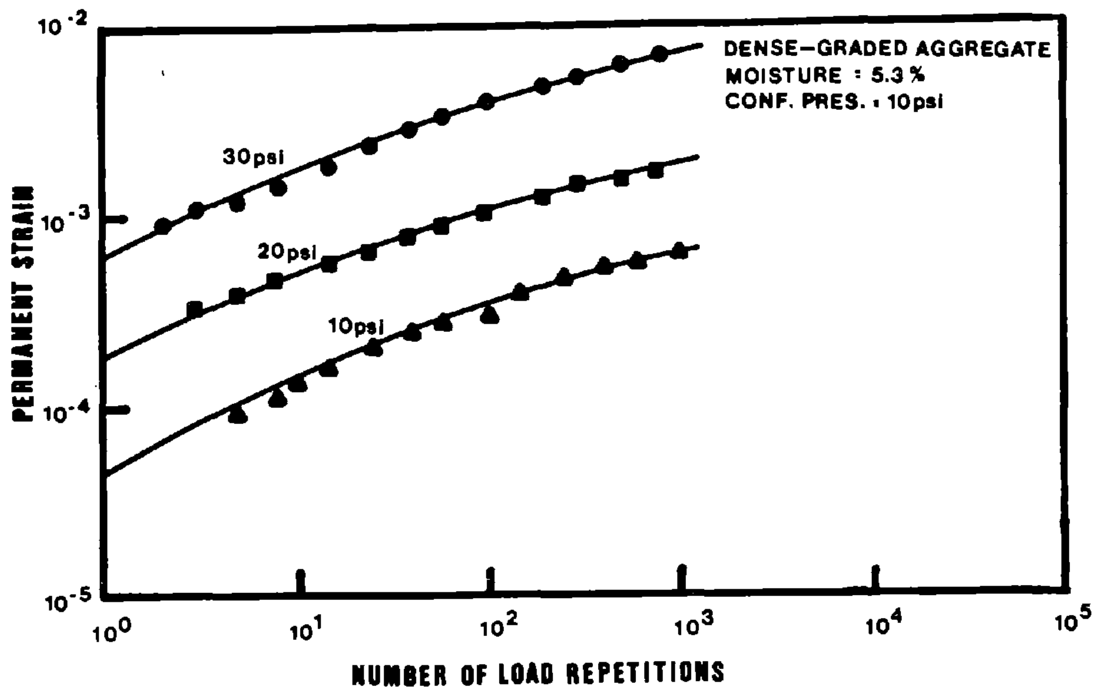

Figure 4. Permanent Strain as a Function of Load Repetitions.

These models, along with environmental and traffic models, were compiled into a computer program entitled PAVRUT and published by Allen and Deen (1986) in the Transportation Research Record of the Transportation Research Board (2).

RESILIENT MODULUS TESTING

Resilient modulus has been obtained as previously described, and as illustrated in Figure 2. Figure 5 shows the relationship obtained between resilient modulus, confining pressure, and moisture content. The data shown in Figure 5 was developed from tests where the dynamic longitudinal stress was 30 psi (207 kPa). An empirical model was developed from Figure 5 and is of the following form.

$$\log M_r = (5.46 - 2.73\log W) + (0.18 + 1.19\log W)(\log S_3)$$

where M_r = resilient modulus (psi),

W = moisture content (percent), and

S_3 = confining pressure (psi).

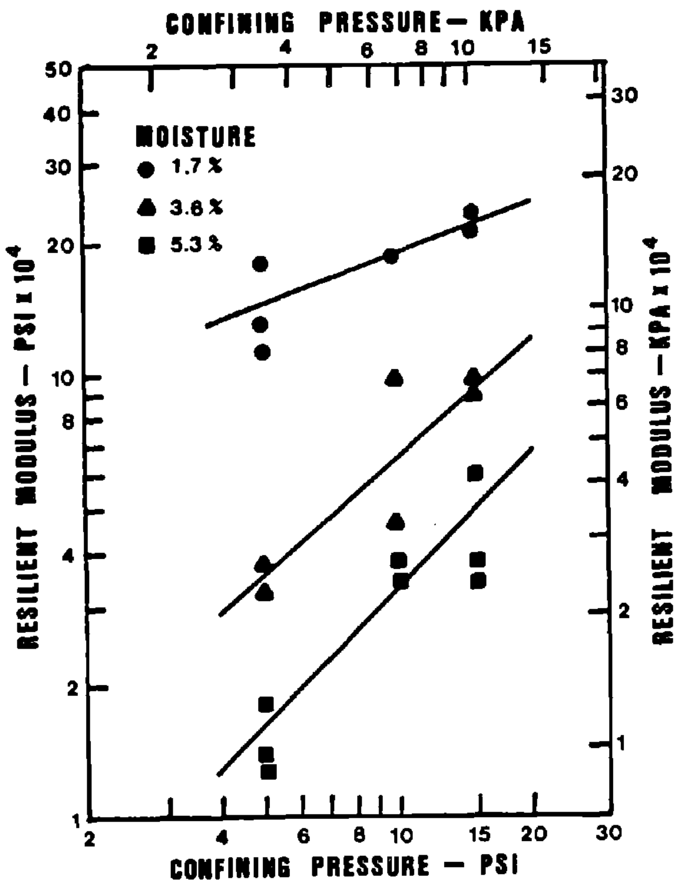

Figure 5. Resilient Modulus as a Function of Confining Pressure.

A similar model was developed for the soils tested in the study, and both resilient modulus models are used in the computer program PAVRUT to calculate layer moduli for the dense-graded aggregate base and the soil subgrade.

In recent years, Kentucky has increasingly used the resilient modulus test as a referee test to compare different mix designs. This has been used, in addition to the Marshall stability value, on special design projects. However, at present, all of this testing has been performed by the Kentucky Transportation Center at the University of Kentucky, as the Kentucky Department of Highways presently does not have this capability.

The resilient modulus test has also been used recently in Kentucky as a tool in overlay design. In cases where Road Rater data were not available, resilient modulus tests were performed on cores obtained from the in-service pavement, and these modulus values were used in the overlay design procedure. Also, in some cases, the resilient modulus test has been used to help verify back calculated moduli values obtained from the Road Rater data.

FUTURE OF THE RESILIENT MODULUS TEST IN KENTUCKY

This author anticipates an expanded role for and use of the resilient modulus test in Kentucky. It is a very real possibility that the test may become a part of the normal series of specifications for state acceptance of a particular mix design, assuming a standard procedure for performing the test can be developed. Furthermore, it is probable that the test will be used routinely in future overlay designs and new pavement designs. Recommendations have recently been made to Kentucky pavement designers that all overlay projects on major highways be cored and tested as a part of the design process.

For further research and development of more standardized testing procedures, the Kentucky Transportation Center at the University of Kentucky recently purchased a dynamic loading frame manufactured by MTS Corporation of Minneapolis, Minnesota (see Figure 6). This equipment will be used to help implement the resilient modulus test in the Kentucky testing and design procedures.

ILLINOIS' EXPERIENCE WITH RESILIENT MODULUS

by

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ILLINOIS' EXPERIENCE WITH RESILIENT MODULUS

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Abstract: Information presented in this paper has been adopted from many research studies conducted by the University of Illinois in cooperation with Illinois DOT and FHWA. Illinois DOT fabricated equipment similar to the University of Illinois and has been evaluating resilient properties of fine-grain soil since 1984.

Soil properties that control the resilient behavior of many Illinois typical fine-grain soils were identified and quantified. Regression equation for estimating resilient modulus at optimum water content and 95 percent maximum density was developed based on soil characteristics (% clay & PI).

The mechanistic pavement design procedure utilizing the stress-dependent resilient properties of subgrade soils was developed and implemented effective July 1988 for hinge-jointed plain PCC and full-depth asphalt concrete pavements.

Benefits of the working platform for the construction of pavements were established. A relationship between the in-situ subgrade strength and remedial thickness of backfill/lime modification was also presented.

Testing under various moisture-density conditions will continue to establish resilient modulus input for conventional flexible pavement.

INTRODUCTION

The Illinois Department of Transportation has decided to implement Mechanistic Pavement Design for the state system effective July 1, 1988. At present, this procedure has been developed for hinge-jointed plain Portland Cement Concrete and full-depth asphalt concrete pavements. The future plans call for this new pavement design procedure to be extended to conventional PCC and flexible pavements.

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These procedures use the actual stress, strains, and deflections experienced by the pavement to determine its fatigue life. They were developed using structural mechanical analysis, and computer modeling as well as actual performance and response of in-service pavement sections. A major portion of the information presented herein has been adopted from many Illinois highway research studies. Many of these studies were conducted by the University of Illinois in cooperation with Illinois DOT and FHWA. This collaboration has worked well for us and has been beneficial in developing the needed equipment, background information, and implementation of mechanistically based pavement design procedure.

For given traffic, materials and environmental conditions, the most significant factor influencing the design thickness of a flexible pavement is subgrade soil support. For a typical flexible pavement structure, the subgrade significantly contributes to the total deflection of the pavement system. Many Illinois DOT engineers have come to question the use of our Illinois Bearing Ratio (IBR) test for evaluating subgrade soil support capacity in flexible pavement design. The basis of this concern is repeated instances where the IBR values indicated an adequate subgrade but under the repeated loading that occurs on in-service pavements, the pavement failed prematurely.

RESILIENT MODULUS

During the last 5-10 years, significant advancement has been made in the method of subgrade soil testing. These methods include a procedure for evaluating soils in the laboratory for their resilient properties. In this procedure a reasonable simulation of wheel loading is produced. This is accomplished by laboratory modeling of the physical conditions and stress states of subgrade material beneath the flexible pavement. Under IHR-603 study, Thompson and Robnett (1) developed equipment and laboratory testing procedures for evaluating the resilient properties of fine-grained soil. Fifty typical Illinois soils, representing 24 pedologic soil series, were evaluated. It was found that moisture-density conditions and degree of saturation has significant influence on the resilient properties of these soils. Under IHR-508 field studies of pavement response were conducted to validate the procedures used to incorporate resilient properties into the flexible pavement design process. From these research studies, regression equations, based on the soil characteristics and pavement deflection, were developed and presented herewith.

Determination from Testing

We have fabricated equipment similar to the University of Illinois and have been evaluating resilient properties since 1984. As with any new equipment and testing procedure, we have gone through a "debugging" process.

Specimens for resiliency testing can be obtained from Shelby tube sampling or from laboratory remolded samples. Shelby tube and remolded specimens are trimmed to a nominal 2-inch diameter by 4-inch height.

A summary of the Illinois resilient modulus testing procedure is as follows:

1. Verify that the electronics are functioning properly. Check also that the load duration is set at 0.60 - 0.100 seconds and the cycle duration is set at 3 seconds.

2. Specimens are conditioned and tested without lateral confining pressure ($\sigma_3 = 0$).

3. Place the specimen in the resilience device (Figure 1) and condition it with 200 axial stress applications (deviator stress σ) of 6 psi. Some material may be too fragile or too soft for this deviator stress so a reduction to 4 psi may be necessary. Record the ending height.

4. Apply ten axial stress applications of 2 psi. Repeat using 4, 6, 8, 10, 14, and 18 psi. Some testing may have to be terminated at 10 psi or lower due to excessive height deformation. Record the ending height to the nearest 0.01 inch for each deviator stress.

5. Measure the resilient deformations by the use of an axially-mounted LVDT connected to a high-speed recorder.

6. For each deviator stress (σ), determine the amount of recovered strain (in inches) from the chart recorder traces. A typical chart trace developed by the resilience testing eauipment is shown in Figure 2.

7. Divide the recovered strain by the ending sample height for each load increment. This will become the resilient strain (E_r).

8. Divide the deviator stress (σ) by the resilient strain (E_r) to determine the resilient modulus (Mr) for any load increment. Mr = σ/E_r.

9. Choose the modulus at 6 psi for the resilient modulus of the sample. Typically, the graph of a sample should have a break point at 6 psi. This point is referred to as the modulus at intersection (Mri). Some graphs follow this trend, but many will have a break point at stresses other than 6 psi. Some have little or no break at all with virtually no downward slope. Others may even have an upward slope. Nevertheless, the resilient modulus in Illinois is still determined at 6 psi. A typical plot is shown in Figure 3.

To date, our testing has indicated that the following should be closely monitored:

1. Calibration of the air piston with a load cell is essential, especially at lower deviator stresses. It is important also to have the air piston compressed to the point at which there is only 0.01 - 0.02 inches of play.

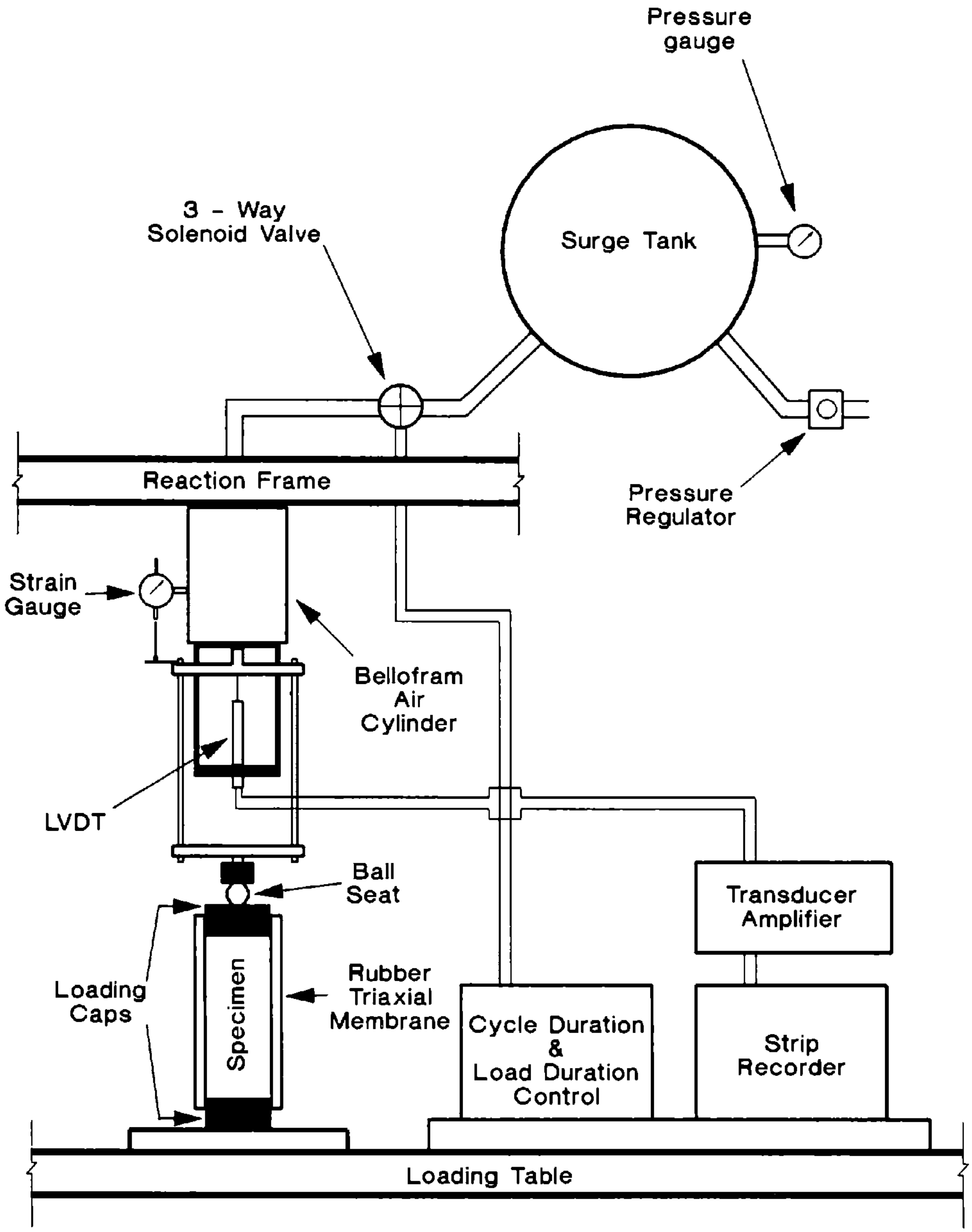

Figure 1. Schematic Diagram of Resilience Testing Equipment

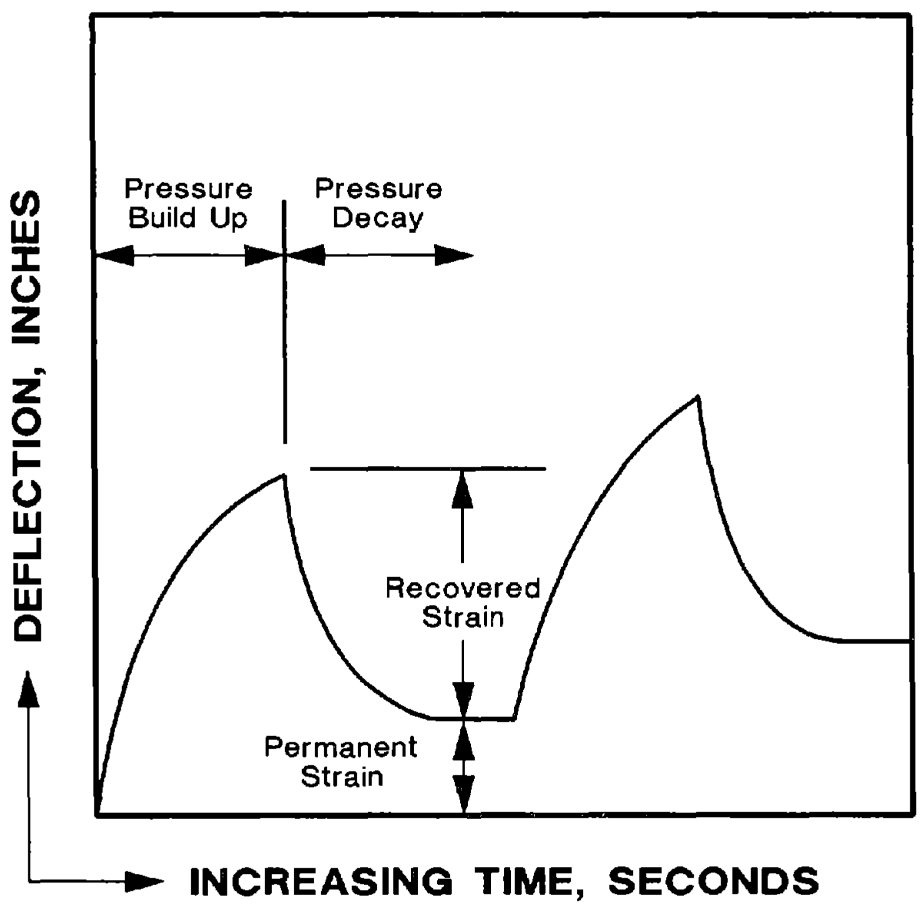

Figure 2. Typical chart trace developed by the resilience testing equipment (not to scale)

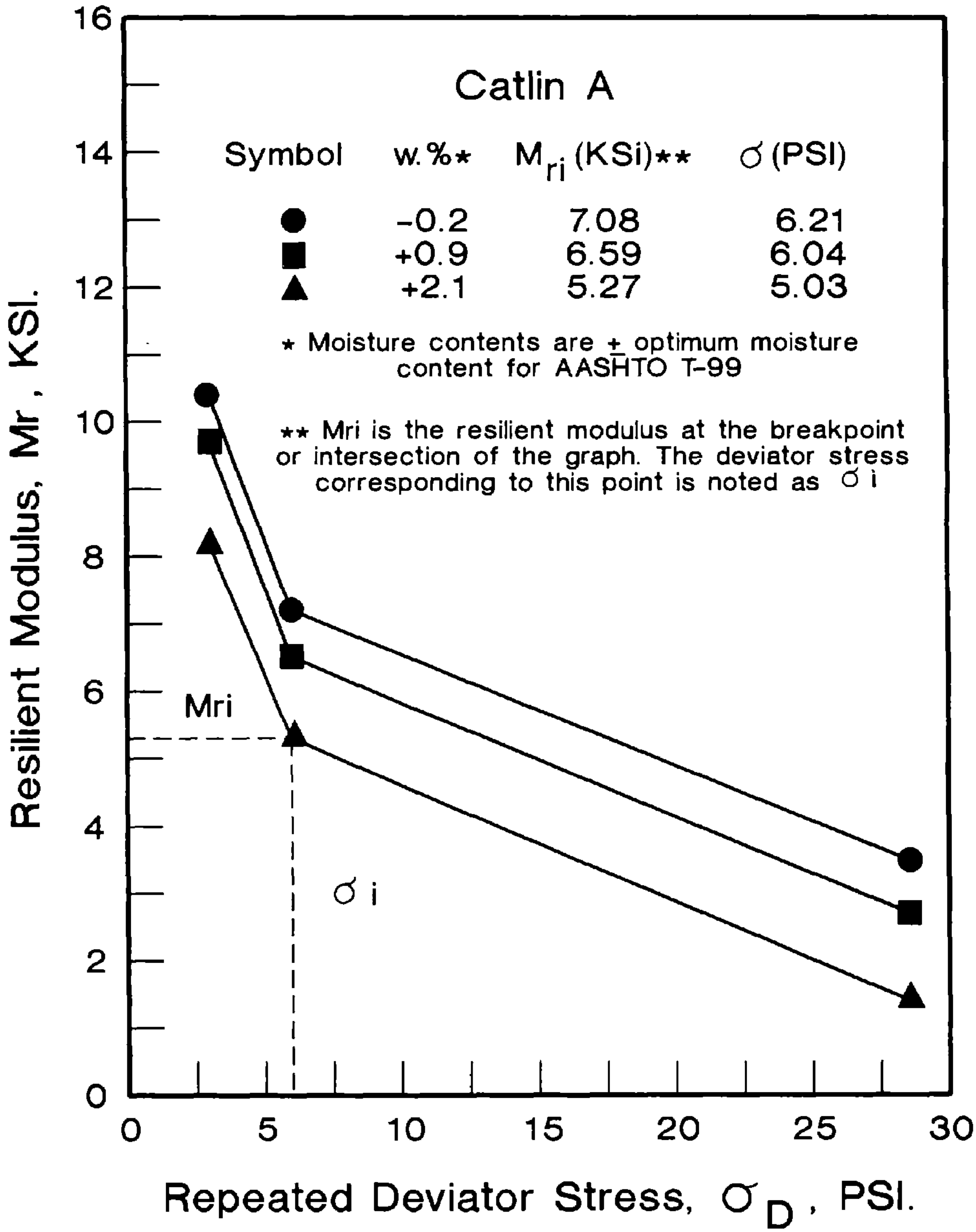

Figure 3. Typical Summary Plot for Resilient Responce Data.

2. If the LVDT transducer amplifier has noise filters, be sure they do not hinder the rise time of the strip recorder. Check for poor electrical connections between components which will cause malfunctions. Check especially all ground connections. Poor grounding will cause excessive and uncontrollable recorder pen movements.

3. If spacer blocks are used to raise the sample to a desired height, be sure that these spacers are flat and smooth. A wobbly base will cause incorrect readings.

Estimation from Soil Index Tests

From many Illinois research studies it has been indicated that even though laboratory testing of subgrade soil can be performed, it may not be an effective means (in terms of cost, time, and accuracy) for determining the yearly variability of a given soil series. Extensive testing is probably only justified for large/research projects where samples can be obtained of the major series and tested under various moisture/density conditions. With this in mind, Thompson and LaGrow (2) developed the following regression eouation for predicting resilient modulus at optimum water content and 95 percent maximum density (AASHTO T-99).

Mr (Opt) = 4.46 + 0.098 (% clay) + 0.119 (PI)
Mr (Opt) - Resilient modulus at optimum water content and has ksi units
Clay = Particle finer than 2 micron
PI = Plasticity Index (AASHTO T-90)

Back-Calculation from Pavement Deflection

Another method of determining/estimating the subgrade Mr is from the Falling Weight Deflectometer (FWD). The Department uses a Dynatest 8002 FWD. In our test we use a 9000 lb. drop force and measure the deflection 36 inches away from the center of the 12-inch diameter loading plate. This deflection is designated (D-3). Back-calculation of Mr values are achieved by several ILLIPAVE algorithms which vary depending upon the pavement type and thickness. The algorithms are as follows (3):

1. Surface treatment plus granular base

   Mr (ksi) = 24.2-5.71 (D3) + 0.35 $(D3)^2$

2. Asphalt Concrete (3+ inches) plus Granular Base

   Mr (ksi) = 25.0-5.25 (D3) + 0.29 $(D3)^2$

3. Asphalt Concrete (any thickness) plus Granular Base (any thickness)

   Mr (ksi) = 24.1-5.08 (D3) + 0.28 $(D3)^2$

4. Full Depth Asphalt

   Mr (ksi) = 24.7-5.41 (D3) + 031 $(D3)^2$

5. Stiff Pavement

$$Mr\ (ksi) = 25.7 - 7.28\ (D3) + 0.53\ (D3)^2$$

<u>Selecting for Design</u>

To determine the effect of Mr variance on estimated pavement life, various Mr seasonal progression was used in the ILLIPAVE algorithms developed by Thompson and LaGrow (2). It was concluded that the reauired full depth asphalt pavement thickness is not sensitive to the majority of the soils in Illinois (Er 2 to 5 ksi). These two values resulted in only a 0.2- and 0.5- inch difference in reauired pavement thickness. Spring rainfall and freeze-thaw in Illinois contribute to a high water table. Conseauently, the subgrade soil has a high degree of saturation. In this condition the subgrade has low modulus and strength. These factors were considered in selecting design Mr inputs in the pavement design procedures.

Illinois mechanistic design procedures for rigid pavements use a subgrade strength parameter known as the modulus of subgrade reaction "k" (psi/in.). For full depth asphalt pavements the subgrade strength input is the resilient modulus Mr (psi). The mechanistic pavement design procedure was developed for three types of subgrade support ratings (SSR) normally found in Illinois. These included the SSR designations of "poor", "fair", and "granular". Comparable CBR, Mr, and "k" values for each of SSR are as follows:

Subgrade Support Rating (SSR)	Poor	Fair	Granular
Resilient Moduli			
Mr (psi)	2	5	*
Modulus of Subgrade			
Reaction "k", (psi/in.)	50	100	200
Estimated CBR	2	3-5	6-10

*Mr for granular material is "stress dependent" so a "single value" cannot be assigned.

In Illinois subsurface exploration is an essential part of the engineering survey for location and design of a transportation system. It includes investigation, sampling, testing, identification of material type, and distribution with respect to the vertical and horizontal alignment of the highway. The soil report becomes a design document and is furnished to the project design engineers. It is the responsibility of the geotechnical engineer to provide the SSR in the soil report based on grain-size and application of Figure 4. The SSR should represent the average/majority rating classification within the design section. Figure 4 assumes a high water table and appropriate frost penetration in the subgrade soil.

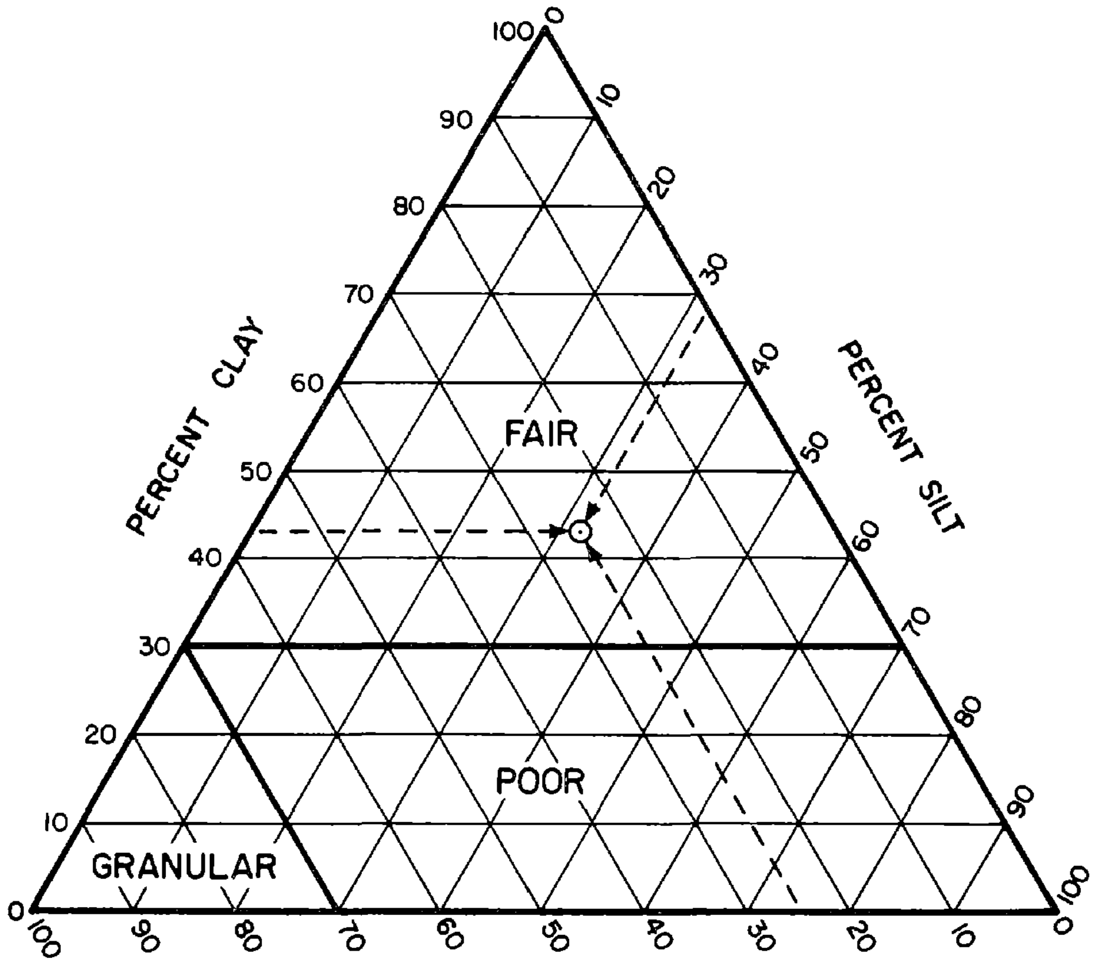

Figure 4. Subgrade Support Rating

<u>Particle-Size Limits</u>

Sand 2.000 - 0.075 mm

Silt 0.075 - 0.002 mm

Clay finer than 0.002 mm

<u>Example</u>

For soil having 25% sand, 32% silt, and 43% clay the SSR is FAIR.

IMPROVED SUBGRADE LAYER

It was established by policy that the subgrade must provide a stable working platform for construction of all pavements. Benefits of the working platform include resistance to moisture-related problems and rutting which produces smooth pavements, and more efficient and effective construction.

The stable working platform (12-inch thick) may require soil modification or removal and replacement with granular material. Modifying agents include lime, by-product lime, cement, or other approved materials. It was decided that the proposed subgrade treatments should be presented in the Project Soil Report so that a complete economic evaluation can be made by the designer.

In some situations, such as rock cuts and fills, the working platform thickness may be reduced. However, in other situations, a 12-inch improved subgrade layer will not provide an adequate working platform (if in-situ IBR's of 4 or less are encountered during construction). The IDOT Subgrade Stability Manual (3) is followed during construction to establish a remedial treatment thickness greater than the required 12-inch improved subgrade layer.

In this Manual a scheme, based on the Corps of Engineers' unsoaked CBR design approach, was adopted to develop the remedial thickness of granular backfill/improved subgrade layer (see Figure 5). The required thickness above the subgrade from Figure 5 reduces the maximum subgrade stresses to about 75 percent of the soil's shear strength (4). Most soil can withstand 500-1000 repeated stresses of this magnitude without experiencing permanent deformation strain in excess of 1-1.5 percent.

In Illinois, when subgrades are improved to provide a stable working platform, change in the pavement design SSR is not allowed. For mechanistic pavement design the SSR of the soil prior to its improvement is used.

SUMMARY

From many research studies, we in Illinois, have established that laboratory testing is not cost effective or justified for determining the yearly variability of a given soil series. In most cases we are/will be using regression equations or the results developed from Figure 4 for the hinge-jointed plain concrete and full-depth asphalt concrete pavements. Illinois DOT will continue testing for large/research projects where samples can be obtained and tested under various moisture/density conditions. Information from these projects will be used to establish resilient modulus input for conventional flexible and PCC pavements.

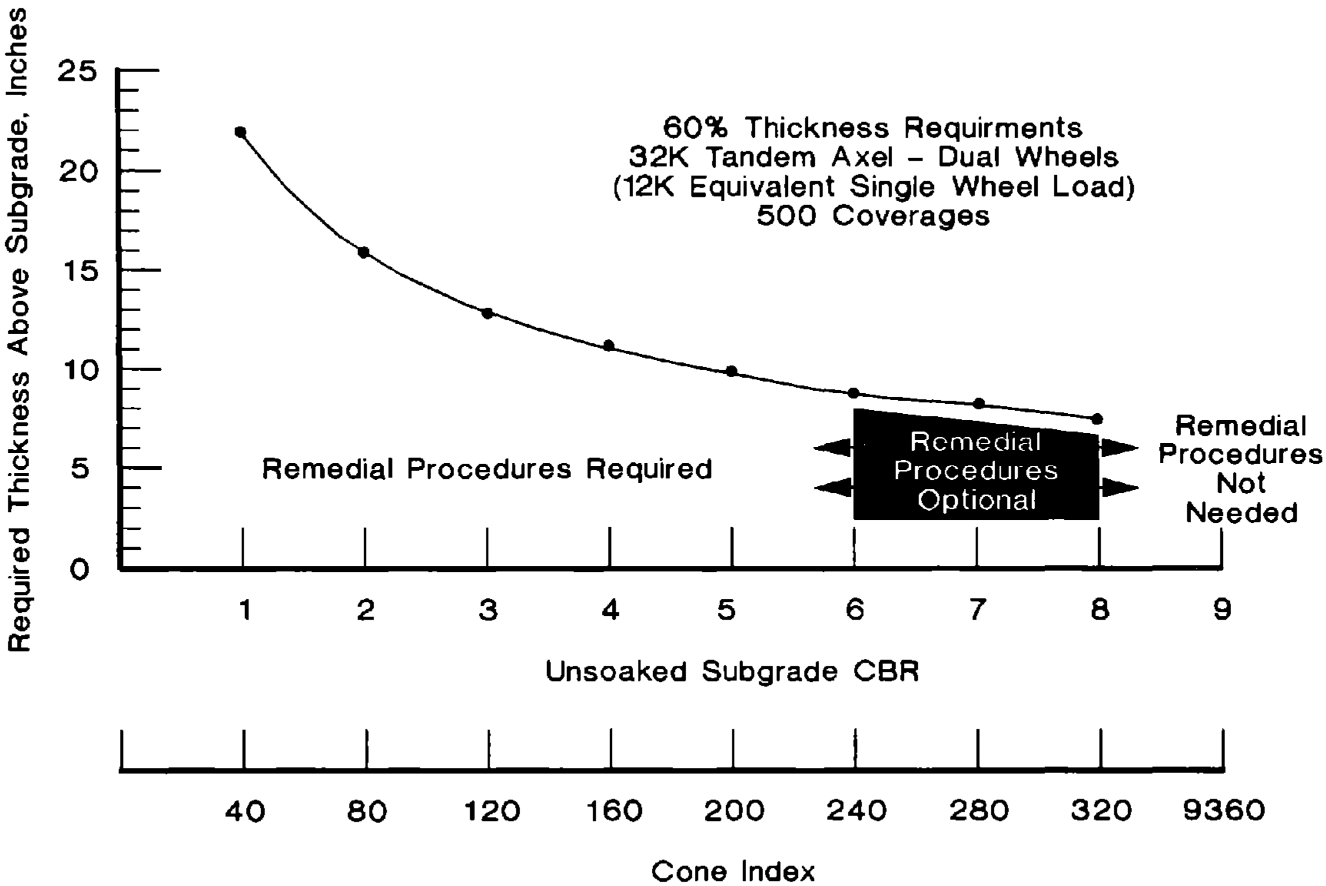

Figure 5. Unsoaked CBR - based thickness design procedure for granular backfill/admixture modified soil

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RESILIENT MODULUS TEST IN CALIFORNIA DOT - YESTERDAY, TODAY, AND TOMORROW

by

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RESILIENT MODULUS TEST IN CALIFORNIA DOT - YESTERDAY, TODAY, AND TOMORROW

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Since the early 1970's, the California Department of Transportation (Caltrans) has incorporated resilient modulus testing in several research studies to observe the test values. This paper covers a brief review of Caltrans' experience with the resilient modulus test. Concerns regarding the application of the resilient modulus criteria to flexible pavement design and/or asphalt mix design procedures are discussed. Some reasons for Caltrans' interest in the resilient modulus testing are presented along with possible future uses.

The use of resilient modulus testing by Caltrans may not expand beyond research purposes in the immediate future. If, however, improved test equipment provides acceptable repeatability, accuracy, and correlation with other design parameters, the resilient modulus of paving materials may be adopted as an additional mix design property. The resilient modulus test may be used when unique materials will be incorporated into pavement structural sections. Currently, the only proposed application of the resilient modulus is for special analytical problems (i.e., overload/permits) in conjunction with a multilayer elastic system computer program. It is expected that the use of resilient modulus for structural evaluation and analysis will increase in the future at Caltrans.

INTRODUCTION

Several agencies have used various testing methods to measure the modulus of pavement structural section materials such as asphalt mixtures (non-aged/aged, as-compacted/conditioned, virgin /recycled, and cores), non-treated aggregates and soils, and treated aggregates and soils (with lime or portland cement, etc). Three test procedures to measure the modulus of flexible pavement structural section materials have been standardized by AASHTO or ASTM: Resilient Modulus of Subgrade Soils (AASHTO T 274, hereafter called repeated load triaxial test), Dynamic Modulus of Asphalt Mixtures (ASTM D 3497), and Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (ASTM D 4123, hereafter called repeated load diametral test).

The resilient moduli of the pavement structural section materials are some of the essential input data for the mechanistic-based pavement design procedure such as the method described in the AASHTO Pavement Design Guide [1]. According to this Design Guide, the soil support number is replaced by the resilient modulus in the flexible pavement design procedures. In addition, the layer coefficients for the various materials are

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defined in terms of resilient modulus as well as standard strength criteria such as CBR or R-value.

Since the early 1970's, the California Department of Transportation (Caltrans) has incorporated resilient modulus testing in several research studies to observe the test values. Studies performed mainly by the California Transportation Laboratory (Translab) are reviewed briefly in the following section. The Retsina Mark II repeated load diametral test equipment has been used to measure the resilient modulus of asphalt concrete mixes. But some doubts regarding the accuracy of data obtained by the Mark II model occurred in 1988 (Note: This Mark II model is almost 15 years old). Repeated load triaxial resilient modulus test equipment has been used for measuring the resilient modulus of soils, aggregates, asphalt treated permeable base materials, and open graded asphalt concrete mixes. However, these resilient modulus tests have been used for research purposes and information only.

This paper presents a brief review of Caltrans' experience with the resilient modulus tests (both repeated load triaxial and diametral test). Concerns regarding the application of the resilient modulus criteria to asphalt mix design and/or flexible pavement design are discussed. Finally, some reasons for Caltrans' interest in resilient modulus testing are presented along with possible future uses by Caltrans.

RESILIENT MODULUS TESTING BY CALTRANS

As mentioned in the previous section, Caltrans has used the Retsina Mark II repeated load diametral test equipment to measure the resilient modulus of dense graded asphalt concrete (DGAC) mixes and repeated load triaxial resilient modulus test equipment for soils, aggregates, and asphalt treated permeable base (ATPB) materials, and open graded asphalt concrete (OGAC) mixes. However, these tests have been performed for research purposes and information only.

A. Repeated Load Diametral Test

In 1978, Doty and Scrimsher [2] performed a small research study in order to determine relationships between conventional properties of DGAC mixes (e.g., Hveem stability, cohesion, and specific gravity) and resilient modulus. Another purpose of this study was to determine the feasibility of incorporating the repeated load diametral test into the Caltrans design procedure for asphalt concrete pavements.

Asphalt grades AR-2000 and AR-4000 from three different sources were combined with three different aggregate gradations. Test specimens 2.5" high by 4" diameter were used. The kneading compactor was used to obtain uniformity and conformance to the standard method of laboratory compaction used in California.

During the test, a pulsing load (60 lbs) of 0.1 second duration repeated 20 times a minute was applied. The test temperature ranged from 72 to 76 F. After the diametral test, and within 48 hours, the specimens were heated to 140 F and tested for Hveem stability, cohesion, and specific gravity.

Doty and Scrimsher discussed the effect of mix components, compaction method (static, Marshall and Kneading), compaction temperature, temperature susceptibility of asphalt cement, and test temperature on the resilient modulus of test specimens. In addition, the authors discussed briefly the relationship between resilient modulus of mixture and fatigue resistance. The results of this study showed that resilient modulus of mixes increased with Hveem stability and/or specific gravity. However, they did not find any direct relationship between cohesion and resilient modulus.

The authors concluded that the resilient modulus value cannot be used to reliably determine optimum asphalt content from the standpoint of voids, Hveem stability or cohesion. Also, the authors concluded that the resilient moduli values measured at 72-76 F does not provide an accurate measure of asphalt concrete pavement fatigue resistance.

In the early 80's, Kemp and Predoehl [3] completed a study on the durability of asphalt concrete mixes. They tried to use a resilient modulus test to evaluate, in part, the properties of non-aged and aged specimens. However, there was no definite conclusion about the relationship between resilient moduli of test specimens and degree of aging.

One study supported by the FHWA, which is titled "Mix Design Modification for Dense Graded Mixes (DGAC) to Improve Asphalt Concrete Durability," is presently being performed. This study will seek to develop a correlation between pavement performance and the Hveem stability, Marshall stability and resilient modulus of asphalt concrete mixtures in part. Figure 1 illustrates the diametral test equipment used by Caltrans.

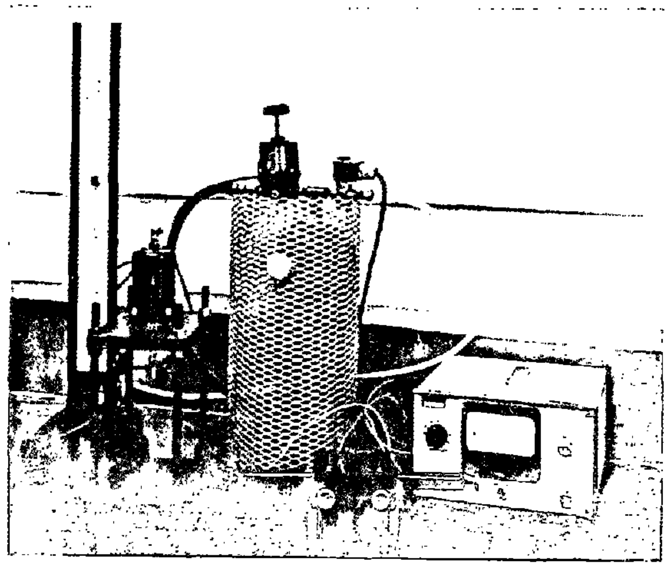

Figure 1. The Repeated Load Diametral Testing Machine (Retsina Mark II Model).

B. <u>Repeated Load Triaxial Test</u>

In 1974, a study on the reproducibility of the triaxial resilient modulus test was performed by Svetich et al. [4]. One of the major objectives of this study was to compare test results of a basement soil (from a Caltrans' full depth asphalt concrete project on Route 101) when determined by different agencies using their resilient modulus testing machines. Two agencies - the Oregon State Highway Division (OSHD) and the University of California, Berkeley (UCB)- completed the testing along with Translab. This study also included using the resilient moduli values determined by three agencies in an actual design of a full depth asphalt concrete pavement.

In Translab, the specimen was placed in a chamber where a confining pressure was applied to the specimen. As an axial haversine load was applied, the radial and axial deformation of the specimen were measured by means of linear variable differential transformers (LVDTs) which were attached to the specimen by means of clamps (for radial readings) and LVDTs placed between these clamps for axial readings. A load duration of 0.1 second at 3 second intervals was used with the testing machine shown in Figure 2.

It was originally intended to run an analysis of variance on the test results that would include both the repeatability for each agency and the reproducibility among the agencies. However, since only three agencies completed the testing and a different test procedure was used by each agency, a statistical analysis of the data was not made. Each agency performed the tests using different stress ratios for the various confining pressures. Two agencies placed the LVDTs used to measure the axial deformation between clamps attached directly to the specimen, while the other

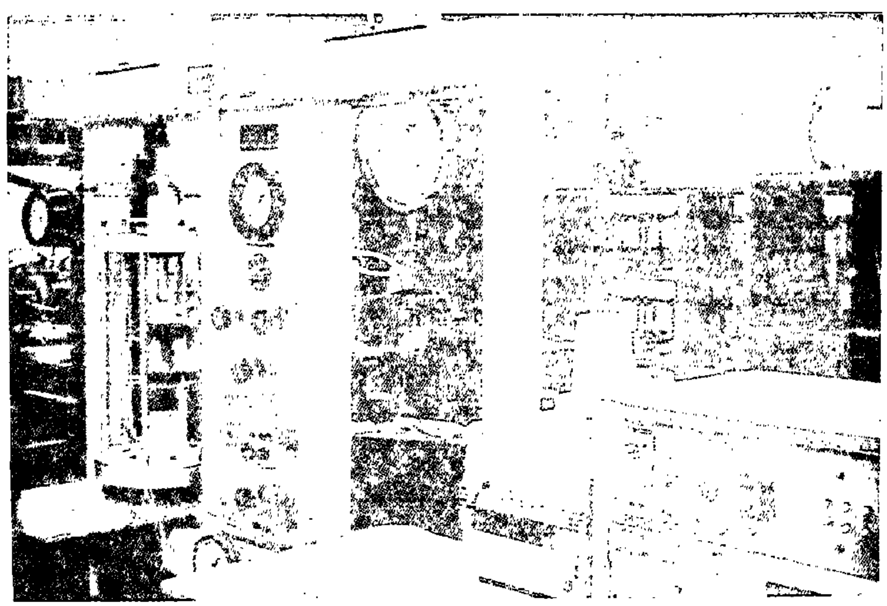

Figure 2. The Repeated Load Triaxial Testing Machine.

agency placed the deformation measuring equipment outside the triaxial cell cover plate and measured the movement of the loading piston.

In the final report for this study, the authors recommended that a more definitive method of testing and measuring the resilient modulus be established. The authors concluded that the repeated load triaxial resilient modulus test should not be adopted as a standard method for use in asphalt concrete pavement design at that time.

Translab recently introduced a new design concept called "incremental design" for flexible pavement [5]. Forsyth et al. compared the results from the incremental design method to those from the 1986 AASHTO Design Guide procedure, the current Caltrans standard (R-value) procedure, and mechanistic methods. For this study, the resilient modulus of the subgrade soil, aggregate base material, and ATPB material was measured in the laboratory. Diametral testing was used for the modulus of the DGAC surface mixes. One of their conclusions is that the structural section thickness determined by the recently modified AASHTO procedure [1] for a 10 year design was similar to that resulting from the incremental procedure. This was probably due to drainage coefficients of structural layers. R-value and mechanistic designs had similar overall thickness that were larger than the incremental design. More detailed information on incremental design is available in Reference 5.

Caltrans uses asphalt treated or cement treated permeable base (ATPB and CTPB)to provide drainage within the structural section of the roadbed. Thus, the repeated load triaxial test has been used to evaluate the characteristics of the ATPB materials with 4" diameter by 8" high specimens. The current standard specifications for ATPB are presented in Table 1.

Table 1. Standard Specifications for ATPB (After Ref.6).

Sieve Size	Percentage Passing
1"	100
3/4"	90 - 100
1/2"	35 - 65
3/8"	20 - 45
No. 4	0 - 10
No. 8	0 - 5
No. 200	0 - 2

Tests	California Test	Requirements
Percentage of Crushed Particles (Min.)	205	90%
Los Angeles Rattler Loss at 500 Rev. (Max.)	211	45%
Cleanness Value (Min.)	227	57
Film Stripping (Max.)	302	25%

Moore et al. [7] attempted to determine the gravel factor (G_f) of ATPB and OGAC using the R-value test and the repeated load triaxial resilient modulus test. The ATPB was also evaluated using Dynaflect deflection measurements. The results of the R-value test showed the ATPB to have less strength than aggregate base, whereas, the resilient modulus tests showed the ATPB, as well as OGAC, to have greater strength. Deflection measurements showed the ATPB to have similar strength to DGAC. The resilient modulus of ATPB ranged from 100 to 200 ksi with an average of 140 ksi. A load duration of 0.1 second and frequency of one cycle per second was used for this study. Additional data and the actual levels of applied confining pressure and deviator stresses used for this study are presented in Reference 7.

The researchers concluded that more credence should be given to the results of the resilient modulus test rather than the R-value test. However, since no direct correlation between resilient modulus and G_f exists, the G_f of ATPB was determined by deflection values measured in the field before and after construction rather than by the resilient modulus.

CONCERNS REGARDING APPLICATION OF THE RESILIENT MODULUS TEST

Even though further research effort and experience in resilient modulus testing are necessary and currently occurring, Caltrans has several concerns regarding application of resilient modulus testing based upon previous experience. Those concerns are discussed in this section.

A. Repeatability/Reproducibility

Sometimes the repeatability of the resilient modulus test was found to be one of major problems in the past Caltrans' experience. In addition, the precision of the diametral resilient modulus test is not established in ASTM D 4123. Because pavement structural section materials are non-homogeneous and anisotropic, relatively wide variance in test results can be expected. However, the level of variance due to different test equipment should be minimized. A study of test repeatability/reproducibility must be conducted and the magnitude of test repeatability/reproducibility must be acceptable.

B. Poisson's Ratio

For the diametral test, the resilient modulus is a function of the dimension of the specimen, horizontal deformation, applied loading, and Poisson's ratio as shown in the following equation;

$$E = P(\nu + 0.27)/t\ H_T$$

where:

E = resilient modulus of elasticity, psi (or MPa),
H_T = recoverable horizontal deformation, in.(or mm),
P = repeated load, lbf (or N),
t = thickness of specimen, in.(or mm), and
ν = Poisson's ratio.

In general, it is recommended to use 0.35 as a Poisson's ratio of asphalt mixes. Also, it is indicated in ASTM D 4123 that a value of 0.35 for Poisson's ratio has been found to be reasonable for asphalt mixtures at 77 F (25 C). However, the Poisson's ratio of asphalt mixes may vary with temperature, specimen condition, and component materials such as rubber material. Thus, it seems to be desirable that the diametral test equipment have the capacity to measure the Poisson's ratio of the test specimen accurately.

C. Association with Pavement Distress

Flexible pavement structural section materials including roadbed soils show various distress type and complex failure behavior. The resistance to failure depends on the properties of the paving materials, the stress field, the environmental conditions and time. Further, it is often difficult to detect a point of failure of pavement.

Although numerous researchers have developed models to predict fatigue and permanent deformation of flexible pavement during the last three decades, the degree to which resilient modulus is associated with pavement distress (e.g., fatigue cracking, rutting, thermal cracking, etc.) is not yet clearly determined. As stated in the AASHTO Design Guide, stabilized base materials and asphalt concrete surface materials may be subject to cracking under certain conditions. Thus, because of the complexity of flexible pavement failure, the resilient modulus may not be the sole indicator of pavement distress likelihood even though it can provide valuable information.

Although previous Caltrans' experience has revealed some shortcomings of the resilient modulus test, Caltrans still has some interest in this test procedure for the following reasons.

REASONS FOR INTEREST IN THE RESILIENT MODULUS TEST

Since the values of resilient moduli can be used to evaluate the relative quality of materials, the resilient modulus test, along with conventional laboratory tests, may help to differentiate between materials performing well and those performing poorly. Also, the resilient modulus test provides a means of evaluating paving materials under a variety of environmental conditions and stress state that realistically simulate the conditions that exist in pavements subjected to moving wheel loads.

A. Dynamic Test

As contrasted with many conventional laboratory tests, the resilient modulus test utilizes the dynamic (cyclic) load application, therby permitting varying the load duration and load frequencies depending on the traffic conditions (volumes, speed, loading, etc). The test also can be used to study the effects of loading rate and rest periods under different sample conditions (degree of saturation and temperature, etc.).

B. <u>Nondestructive Test</u>

Since the resilient modulus test is nondestructive, this test can be incorporated with any destructive laboratory tests. The properties of a test specimen (e.g., split tensile strength, Hveem or Marshall stability, specific gravity, cohesion, creep stiffness, etc.) can be obtained after a resilient modulus measurement. In addition, tests can be repeated on a specimen to evaluate the curing effect on emulsion mixes or recycled mixes and conditioning as with temperature or moisture. Thus, considerable time and materials can be saved.

C. <u>Use in Mechanistic Design</u>

The resilient modulus test provides the basic constitutive relationship between load and deformation of flexible pavement construction materials for use in structural analysis of layered pavement systems. Since moduli values and Poisson's ratios of pavement structural materials are required in mechanistic procedures, the values of resilient modulus can be used to generate input for pavement design or pavement evaluation and analysis.

D. <u>Supplement to Conventional Test</u>

The conventional laboratory test procedures may not be applicable to some new pavement materials. The resilient modulus test may be useful when considering new materials and/or unusual situations for which no experience is available (i.e., empirical approach cannot be used) such as ATPB, CTPB, fiber-modified or rubber modified asphalt concrete, and high tire pressures and/or high axle loads.

FUTURE USE OF THE RESILIENT MODULUS TEST BY CALTRANS

Some questions often arise as to the values of using the resilient modulus test for asphalt mix design and/or flexible pavement design. This tendency is particularly strong when the presently used mix design or pavement structure design procedure appear to be very sound.

However, several changes have occurred during the last 10 years in asphalt pavement materials, asphalt paving technology, and traffic characteristics including truck volume, tire pressure, speed, and permissible gross vehicle and axle loads. These changes may require modifications and/or additions to existing mix design criteria and test procedures to properly evaluate mix properties. In addition, new methods may be needed to replace or supplement existing procedures to simulate in-service conditions.

Along with these changes, nationwide emphasis has been placed on the development of more mechanistic design procedures in both mix design and pavement design. The use of the resilient modulus test to characterize structural section component materials seems to be gaining acceptance.

A. Mix Design

Resilient modulus testing has not been used in the asphalt mix design process by Caltrans. The optimum binder content is determined based upon the Hveem stability, visual observation and air voids of mixes. At Caltrans, the resilient modulus test has been performed for research purposes and information only, not for routine design and evaluation. Although the accuracy of data obtained with existing diametral test equipment has been unsatisfactory, it is expected to purchase of an improved test equipment.

There are some possibilities for applying the resilient modulus test to the evaluation of paving materials in the future. One area where some potential exists is to evaluate the possibility of moisture-induced damage to asphalt concrete mixes. The resilient modulus test incorporated with a conditioning method such as AASHTO T 283 (Resistance of Compacted Bituminous Mixture to Moisture Induced Damage) may provide valuable information.

Also, the resilient modulus test can be helpful in the evaluation of the effects of additives on properties of asphalt concrete and emulsion mixes. With new paving materials and technology ,such as pavement recycling, the application of the resilient modulus test can be expanded for evaluation and possibly mix design procedures.

Even though routine use of the resilient modulus test by Caltrans is not foreseen in the immediate future, the opportunity to incorporate resilient modulus into the Caltrans' mix design may become a distinct possibility if improved test equipment provides acceptable repeatability, accuracy and correlation with other design parameters. Only then will it be possible to adopt the resilient modulus criteria as an additional mix design property.

B. Pavement Structure Design, Evaluation, and Analysis

The resilient modulus is not used yet for pavement structural section design purposes. For pavement design, Caltrans exclusively follows the California design method based upon the R-value of the basement soil, gravel factors (G_f) and the Traffic Index (TI), which is calculated from the projected number of 18 kip equivalent single axle load applications in one direction of travel.

Current asphalt concrete construction and rehabilitation projects have often used various pavement materials such as rubber powder, crumb rubber, or other binder modifiers. In order to take full advantage of these materials for better pavement performance and to reflect the advantages in the design of pavements, studies on the characteristics of these materials are essential. As mentioned in the previous section, the Caltrans' procedure for the design of flexible pavement structural sections involves the use of an empirical formula that includes gravel factors (G_f). However, the G_f of new pavement materials cannot be determined according to the existing procedures. The resilient modulus test, performed in conjunction with field performance studies, might be helpful in determining the gravel factor (or layer coefficient) of new pavement materials. Thus, the resilient modulus test may be useful when unique materials will be incorporated into structural sections.

Currently, the only Caltrans application of the resilient modulus test is for special analytical problems employing a multilayer elastic system computer program [8]. When the evaluation of a pavement structure is required for special load conditions (i.e., overloads or high tire pressures), deflection measurements will be used with a layered elastic analysis with assumed or measured resilient modulus of pavement materials. Resilient modulus will be used for routine evaluation of the effect of overload if an interim procedure proves satisfactory. Thus, it is expected that the use of resilient modulus for structural evaluation and analysis will increase in the future at Caltrans.

SUMMARY

Caltrans' experience with the resilient modulus test (both repeated load triaxial and diametral test) has been reviewed briefly. While the use of resilient modulus for structural evaluation and analysis is growing, the application of the resilient modulus test for mix design is not likely.

Although widespread routine use of the resilient modulus test by Caltrans is not foreseen in the immediate future, the resilient modulus test may be used when unique materials will be incorporated into pavement structural sections. This will require modification of Caltrans pavement and/or overlay design methods. It is expected that the use of resilient modulus for structural evaluation and analysis for special situations will increase in the future at Caltrans.

DISCLAIMER

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PROFESSIONAL SOCIETIES/AGENCIES

PANEL DISCUSSION

Summarized by
J. Sorenson, Regional Pavement Engineer
Federal Highway Administration

INTRODUCTION

The purpose of this session is to allow representatives from ASTM, AASHTO, NCHRP and SHRP to present their views on the status and future of modulus testing from the perspective of their respective organizations. To accomplish this, we have assembled representatives (or surrogate) of each organization to address these issues. The panel includes:

1) Gilbert Baladi - ASTM
2) Robert Ho - AASHTO
3) Daniel (Bill) Dearasaugh - NCHRP
4) Marshall Thompson - SHRP

ASTM Viewpoint

G. Baladi: Two ASTM subcommittees are working on topics related to resilient modulus. I am Chairman of D18.10, working in cooperation with D-189, Soil Dynamics, and members of D4.39, Nondestructive Testing. We are trying to come up with a definition of resilient modulus. That activity is now in its fourth year; we still do not have a standard definition of resilient modulus. Right now, Mr. Richard May is in charge of predrafting the standard, rewriting the standard for another fellow. We still are getting some negative votes. Just to give you a little background, ASTM policy is that the standard will not go on the book if there is a single negative against the standard. All negatives should be satisfied or withdrawn.

The other committees and D4 are in charge of D41.23, the standard for the indirect and soil test. This standard is in the process of being rewritten or modified to include what we learned over the last few years from laboratory and in situ soil tests. The main objection of the subcommittee is the assumption of .35 Poisson's ratio. That's the standard specified to assume .35, regardless of what test temperature you're at. You're testing at 40°F, 77°F, or 104°F, the standard specifies low load on the sample.

That standard is being examined. Unfortunately, I didn't have the time to redo anything since last June, so I'm planning to write a draft revision for that standard by the next ASTM meeting, which is June 1989.

J. Sorenson: If there is a question for any of the speakers, we encourage you to ask it. Thank you, Gilbert. Jim (Brown), you've just gone through what AASHTO is doing, maybe you could give us a little perspective on T-274.

J. Brown: I have submitted a written contribution to the workshop (re: Exhibit 1, page V-85).

<u>AASHTO Viewpoint</u>

J. Sorenson: Wes Moody is on the committee on T-274 and Dave Seim works for Wes, but he's not really involved in it. Dr. Ho, do you have any comments on that committee?

R. Ho: At the last annual meeting of AASHTO in August, 1988, in Orlando, Florida, there was discussion about this resilient modulus test. Questions came from the various states about how the test was run and what equipment to buy. At that time, many people hadn't run the test. Some states had run the test and expressed their concern about the T-274 method and its complexity. Florida was one of them; we presented some of the data at that time. Then there was a recommendation from Mr. Smith, Chairman of Technical Section 1b, to form a task force to look into this matter. That's how it was formed. I think there are about seven or eight states represented in the task force. Mr. Moody, who wasn't present at the meeting, was elected Chairman of the Task Force. That is the situation right now. Recently, Mr. Moody has written a letter to all the members of the task force to request experiences from all of the states so that he could get all of the information together to see what they can do with T-274. The last time I spoke to him, he said nobody had replied to his letter yet.

<u>NCHRP Viewpoint</u>

J. Sorenson: I talked to Wes Moody when we were putting this program together. John Strada, WSDOT, is out here and he's also working with Wes. All of the input from this 3-day workshop is going to that task force. As I understand it, it will be a stepping-off point so that we can make some forward progress with that task force initiative. I think we've all contributed toward that; we're going to give them something to start with. How about the NCHRP activities, Bill (Dearasaugh)?

B. Dearasaugh: Thank you for the invitation to be a part of this panel. I've been with the Transportation Research Board for a year; I'm still the new kid on the block there. It takes about a year to get through the entire process of the NCHRP activities. We're a part of the National Research Council, which is part of the National Academy of Sciences. I'm in Division D of the Transportation Research Board, which is the National Cooperative Highway Research Program Division. Division A is the division that visits the states and puts on the annual meeting, whereas the NCHRP is the research arm of AASHTO. Our funds come directly from the states. One and one-half percent of federal funds to the states are for HP&R, and 5.5 percent of that 1.5 percent is contributed voluntarily by the states to the NCHRP program. Those contributions result in an approximate funding level of $8 million a year.

One of our fiscal year '90 projects is a brand-new research effort toward resilient modulus testing. It is NCHRP Project 1-28: Laboratory Determination of Resilient Modulus for Flexible Pavement Design. At their September meeting, the AASHTO Standing Committee on Research (SCOR) decided that this would be one of the new projects for this year and allocated the funds. Actually, the project was submitted by Michigan and Florida. SCOR thought the two submittals were so close that they wanted us to combine them into one project.

The next step in the process was forming a panel to prepare the project statement. That panel will eventually make an agency selection based on the proposals that are submitted. The people on the panel do most of the work. They are Gale Page (Florida), the Chair of the Committee; Don Anderson (South Dakota); Ron Terrel (Washington); and Kevin Stewart (FHWA). All of these panel members are attending this workshop. Also on the panel are: Roger Green (Ohio); Freddy Roberts (Auburn NCAT); Joe Hannon (California); Bill Hadley (Louisiana Tech); Adrien Peltzner (SHRP); and Jay

Jayaprakash (TRB). The panel recognized one big concern right off the bat, and that was the fact that we didn't tackle this thing about two years ago. It is time to do something about resilient modulus testing. The panel prepared the project statement, based on the two problem submittals from Michigan and Florida. This first panel meeting took place March 16 & 17, just two weeks ago, so this is hot off the press. Here is the project statement that I brought along with me (re: Exhibit 2, page V-91). The objective, really, of my presence here is to present this to you and to solicit your interest in this project. I hope to receive some proposals from some of the people who are here and who are interested in doing research for the NCHRP and for AASHTO.

The submittal from Florida focused on structural coefficients for the AASHTO guide. There was concern about that, however, questioning why we should spend a lot of research money developing structural coefficients for the AASHTO equation when we're moving in the direction of mechanistic design concepts in the future. There is a portion of this project that addresses Florida's concerns. In accordance with some interpretations of the AASHTO guide, someone could take his product, like an additive, run a resilient modulus, come up with a huge structural coefficient, and then say: you have to use this. Structural coefficients are not completely based on resilient modulus alone. I know that the guide doesn't really say that, but it could be misinterpreted to say that; some people have been doing just that. At least this project will look at that aspect and possibly come up with some rewording of particular paragraphs in the guide to alleviate that situation, to make it clear that structural coefficients do not directly correlate only with resilient modulus.

The primary objective of the research in Project 1-28, however, is to develop and recommend laboratory test procedures for determining resilient moduli of component materials in a flexible pavement structure. Procedures must account for varying field conditions such as temperature of the asphalt surface layer and moisture content of the subbase -- all of those factors that we've been talking about for the last three days. The procedures must take these factors into consideration and be able to account for them.

The total funding allocated for Project 1-28 is $425,000, and the time allotted is 33 months. Proposals are due on June 2, 1989. The next meeting of the panel to select an agency is in July, and research is anticipated to begin in early 1990. I really do encourage your interest in this project. I think the project will go a long way toward improving the testing for resilient modulus. I'm very proud to be a part of the NCHRP program, and I do what I can to try to have successful projects that will benefit all of the states, who are the sponsors of the program. Other NCHRP Projects that are ongoing that affect this project are: 1-26, a Mechanistic Design project by the University of Illinois; 9-6(1), the AAMAS project by Brent Rauhut Engineering in Austin; 1-27, Non-Destructive Testing by Bob Lytton at Texas A & M; and 10-26A, which is being done by Dave Anderson at Penn State, and has to do with Performance Related Specifications. All of these projects tie together. I believe there's really a bright future for pavements. The work being done by SHRP, by AASHTO, and by ASTM all fit together, and the NCHRP efforts are just one part of it. I think this project's going to be a good one.

SHRP Viewpoint

J. Sorenson: Thank you, Bill. I'd take that TEAMWORK concept back to headquarters with you and just keep the thing alive; we will get there one of these days. If there are no questions, we have Marshall Thompson to give us some background on SHRP. I know Jim Brown mentioned his involvement in some of the concepts with Long-Term Pavement Performance (LTPP). SHRP, $150 million, .25 percent for the next 5 years. There will be a lot of money going into that. There are questions being kicked around; even the questions from the audience here had points to consider. They couldn't understand why we don't have standardization even among the resilient modulus devices that the four SHRP regions will be purchasing and using. I don't know how we're going to get that coordination. Marshall, you might want to tackle this.

M. Thompson: Thank you, Jim. First of all, I want to preface all of my remarks with a caveat: "I am not any sort of an official, unofficial, or even peripherally related representative of the

SHRP program." I'm basically serving as the volunteer leader of a group to establish a protocol testing scheme for conducting AASHTO T-274-type testing.

Basically, the documents that went out in the RFPs for the regional contractors to do the lab testing of soils and bituminous materials indicated that AASHTO T-274 is to be used "for the testing of unbound granular base/subbase materials and subgrade soil samples." The unbound base materials obviously will be recompacted samples from bulk materials obtained from test pits. They will be, according to the guides, prepared at approximate field moisture and density conditions. For subgrade cohesive soils, supposedly there will be tube samples. If tube samples cannot be obtained, those also will be "recompacted samples" to approximate field moisture and density conditions.

A group was asked to review T-274 with the idea of developing a protocol that could be used by the regional testing contractors. John Lynch from Law Engineering in Atlanta, Jim Shook from ARE (the Washington office), and Harold von Quintus from Brent Rauhut Engineering were members of that group. To my recollection, Lynch was the only one who attended the meeting where this group was put together. We all were "volunteered" and then Amir Hannah said: "Would you folks mind doing this?" Obviously one can't turn down a request like that for such an important issue as resilient modulus testing. So we did it. We examined the T-274 procedure and provided recommendations to SHRP. At this time, there has not been any follow-up on that, but my latest input from the SHRP people (Adrian Pelzner) indicates things are happening. There will be an up-front version of that testing protocol. I am certain that it will not be T-274 "as is." To what extent it will be modified is not established at this time. I think if you've looked at T-274, you would agree that it's not a production test. Adrian Pelzner indicated that there will be something like 6,000 resilient modulus tests in one form or another conducted in the SHRP program to the tune of several million dollars worth of activity. Four regional contractors will be doing that testing.

It is absolutely essential that we compare data and numbers; that's the thrust of LTPP. Jim mentioned that we're going to have a nice data base and a lot of good performance data. I would suggest that we make sure that our resilient modulus values are reasonably comparable. I didn't say identical -- reasonably comparable. I don't have any great hopes for getting identical data on a set of samples shipped around to these labs on any sort of a resilient modulus test. I was pleased to see the data that Jim Huddleston pulled together and discussed yesterday afternoon. I think that's what we're going to experience in SHRP. We are encouraging SHRP to include a round-robin testing procedure with the protocol ultimately adopted. We will have samples of granular materials and subgrade materials going out to these labs, and they will be running the test by the protocol -- multiple samples of the same material. Then we'll pull those numbers back together to see whether or not we can get reproducibility and agreement. If not, we are just blowing smoke at one another, and there's too much of that going on already.

If we can't get similar modulus values on the same materials that go out in the round-robin program, we must seriously consider just what it is that we can come up with to get reproducibility. My personal opinion is that we're a lot better off with a simple-minded, reproducible test that incorporates some aspects of resilient modulus than with a sophisticated test that will not provide us any degree of duplication and agreement between labs. Now that's just my own prejudiced opinion. Jim Brown says he doesn't make hard-core statements. As a prof, I always use the phrase: "It seems to me..." That means this is a personal opinion. I think if we want to use these data in terms of LTPP, (correlating mechanistic analysis with performance and other similar types of activities), we have to have some numbers that match up on the same material, or else we just don't have a chance of doing that.

The SHRP protocol is still evolving. I talked to Adrian Pelzner right after he came back from Bill Dearasaugh's panel meeting. Adrian said: "My gosh, Marshall, they're going to spend $425,000 so that we might be able to decide what to do within the next few weeks on the SHRP testing." I think that's a good statement. We've got a minimum of $425,000 worth of unanswered questions out there concerning resilient testing. It's not all that bad, really, but we do need to pay careful attention to what we ultimately decide to do in the SHRP program. Once again, nothing definitive has been established yet; information will be coming out shortly. Some of the regional contractors are here. Many of the Braun Engineering people are here. I know for sure there's a minimum of two or three different types of equipment in their labs. The Law people have an Instron, and I think Luke's getting an Interlocken system for the Braun testing lab. I'm not sure what ARE is coming up with.

J. Sorenson: I don't think our vendors and manufacturers would be against having equipment conform to a standard if we could make up our minds.

M. Thompson: Well, that may well be. The only requirement of the regional testing contractors, was that they would have an electrohydraulic-type machine. That was the only limitation, I think, of any consequence that showed up there.

J. Sorenson: We have, through SHRP and LTPP, asked our 50 states to go out there and think about whapping the pavement any time they take an FWD around so that you could backcalculate and correlate. I don't know how many times you're going to get any correlation with all the different types of FWDs running around out there. It seems to me we're going to have a lot of confusion if we don't get on top of it in a hurry.

M. Thompson: A point I'd like to make, once again strictly as a disinterested, uninformed observer: It seems to me that if we are going to place a great deal of effort on refining falling-weight deflectometer testing and schemes of that sort for thumping these pavements, we are obligated to direct some attention to resilient testing. Otherwise, we're back to this position of not paying equal attention to these things. I mean, it's one of those things where we had to fight to get FWD testing halfway tied in with the field sampling and testing. Al Bush was chairman of the Expert Task Group on FWD testing. That wasn't even in the original FWD plan. We finally got that resolved to some extent, and there will be significant linkage in that respect. Consequently, we need to also get some tender love and care directed to the modulus testing program, or else that thump data is of limited value. Once again, I am not an official SHRP representative.

General Discussion

G. Baladi: Excuse me, Jim. Al Bush also is the chairman of D-439, which is to establish several standards on destructive testing. I don't know if he's willing to address the activities of ASTM.

J. Sorenson: Al, do you want to volunteer to talk about what the ASTM Non-Destructive Testing Standards Committee has been doing?

A. Bush: As Gil said earlier, he is chairman of a committee on D-18, Soil and Rock, on D18.10. I'm chairman of D4.39, Non-Destructive Testing of Pavement Structures. Gil's committee worked primarily on static- and cyclic-load testing. We concentrated on impulse devices and a general test procedure. Right now, in the 1989 manual, we have two standards, one on the impulse test device (D4694), and a guide for general pavement testing (D4695). The latter covers the type of data that should be collected, as well as how many tests should be run. It applies to all devices, such as Road Raters, dynaflects or falling-weight deflectometers.

Our goal right now is to alleviate some of those problems Jon Epps was talking about earlier in the week and to come up with a precision and bias statement for the impulse-load device. We have some data that will be presented at our meeting in June on the SHRP testing at Purdue, where they tested the four SHRP devices and the Purdue device. I'm presently doing a study for all the people that control the airports, the Army, Navy, and the FAA, to look at all of the different types of NDT devices. I think we have a data base now that will enable us to come up with a pretty good precision and bias statement to strengthen that particular standard. Gilbert has a standard on the books on cycling load devices.

G. Baladi: Standard guide.

A. Bush: Standard guide, right. Our impulse-load device is a standard test method; we're trying to get some meat into it. The standard guide is for deflection measurements, and it gives you a guide. We're trying to get some teeth into our standard on falling weights.

J. Sorenson: Thank you. Are there any other questions pertinent to this testing that anyone would like to ask? Is there anything you're aware of that's going on that you think we ought to discuss? We'd like to thank the panel and the audience for their presentation and participation. It's obvious from the discussion that there is a lot of work underway in each of the professional/technical organizations. At some point in the future, there will appear a need to compare notes for the benefit of all.

Summary Evaluation

I think we have heard from the professional societies/agencies represented that our pavement community is aware of the concern for M_R testing, standardization, and the need for some conclusion in this area of pavement design in the relatively near future.

The SHA's are making a considerable investment through SHRP and the LTPP studies. SHRP is aware of our testing concerns and is moving ahead toward standardized test procedures for both laboratory (M_R) and field (FWD) testing.

The comments and concerns of this workshop should prove as useful to the rest of the highway community as they were to those of us attending. On behalf of the FHWA, I wish to thank OSU and ODOT for all of their hard work in organizing and hosting this technical workshop and for all of the excellent Northwestern hospitality. I believe we also should express appreciation to all of the manufacturers and vendors who provided the equipment, the demonstrations, and the frank discussions. Thank you all for participating.

RESILIENT MODULUS AND PAVEMENT DESIGN PRACTICE

James L. Brown, P.E.*

These few brief remarks are being made solely for the purpose of clarifying the AASHTO Joint Task Force on Pavement Design's position relative to resilient modulus testing. More specifically, this paper attempts to clarify the use of resilient modulus in pavement design by reviewing its introduction into the 1986 AASHTO Pavement Design Guide and by predicting future usage by AASHTO members. (Ref 1)

The AASHTO Interim Guide for the Design of Pavement Structures was developed soon after the AASHO Road Test, a large "field" experiment primarily designed to determine the relative effects of a variety of different sized trucks on a few different pavement structures. (Ref 2) A secondary objective was to evaluate a few design variables, i.e., thickness of paving layers. Critically important design variables; subgrade, environment, and time were not included in the experiment. However, politically it appeared desirable to produce a "Design Guide", albeit, quite incomplete. (The Blatnik Committee of the Federal Congress was leaning heavily upon FHWA officials to check the pavement designs of the various states.) Arbitrary "soil support", "regional factors", and performance coefficient scales were established in the "Interim Guides". (Ref 3)

The states were encouraged to establish so-called satellite studies to better quantify the missing elements in the Guide. (Ref 4, 5) Additionally research was undertaken using the National Cooperative Highway Research Program to better characterize paving materials and to further evaluate the Interim Guides. (Ref 6, 7) Implied, if not specific, recommendations from these projects were that some form of mechanistic-empirical design analysis (which used a resilient modulus materials characterization) should be adopted for any revised Guide.

In the meantime, however, most state highway agencies had "made their peace" with local FHWA officials and were using their adapted version of the Interim Guide or some other method of pavement design suitable to FHWA. Changes did not occur until Texas, having to undertake early rehabilitation of thin underdesigned continuously reinforced pcc pavements, obtained a revision to the Guide in 1981. (Ref 8) This revision to the rigid pavement portion of the Guide triggered unexpected responses by the pcc industry. These responses required AASHTO to update the Guide in its entirety.

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The updating was undertaken by the Task Force on Pavements; a task force of the Subcommittee on Design. The Task Force was upgraded to a Joint Task Force so that Materials, Maintenance, and Construction could be represented. A team of academic consultants was hired to do the work. Many of those on the team had been involved with the previously mentioned NCHRP studies. The new team recommended adoption of resilient modulus to characterize both paving materials and subgrade soils even though a mechanistic analysis was not to be employed in the new Guide. The eight reasons cited below were given for recommending resilient modulus:

1. Not identified with any specific agency
2. Fundamental engineering property
3. Techniques currently available for characterizing resilient modulus using NDT
4. Resilient modulus now a standard test procedure
5. If initial equipment investment is too high, possible to use correlation with other laboratory test
6. Favorable comparisons with other laboratory tests (U.S. Forest Service Study)
7. Resilient modulus test is not too complex; familiarity and experience should reduce current problems with application
8. Reservoir of information

In addition to these reasons, the following ideas have probably influenced researchers in recommending resilient modulus as a material property that can be substituted for a performance coefficient:

1. Stiffer paving materials reduce stresses or strains in underlying layers.
2. Stiffness usually correlates well with strength of a material.

Most paving researchers realize that this simplistic view, i.e. a performance coefficient, has many pitfalls and only at best grossly represents reality.

What is pavement design-pavement performance prediction reality? It would seem that only the naive, geniuses, or the grossly egotistical would attempt to predict pavement performance. (The author readily admits to the latter.) The pavement designer must forecast weather, traffic, and the results of a low bid contractor that uses such precise tools as bulldozers and draglines. The traffic forecast must include not only how many trucks but must include size of load and vehicle configurations, including tire pressures and types. Construction materials include those processed by Mother Nature (subgrades) and those semi-processed by the lowbid contractor (base and subbase materials). The properties of these materials and the future loadings need to be known twenty-four hours per day, three-hundred and sixty-five days per year for so far into the future that most pavement designers will retire before the design life has been reached!

For design purposes, how accurately do we need to know the resilient modulus of paving materials and subgrades? Not very accurately. If one examines the variance components in the overall variance of the design process as was done in the new Guide-see Volume 2, Appendix EE or if one merely thinks about items mentioned above, it should be evident that for performance prediction purposes resilient modulus can not, and need not be very precise. Can not be precise because the variation over space and time within the project that has not yet been built will be large. Need not be precise because the other factors of loading, environment, construction, and maintenance have large variance components.

Please note that while the author may sound disparaging about the ability to predict performance, he is only suggesting that we not over emphasize any aspect of design prediction. Additionally, note that it is our ability to predict dis-performance (pre-mature failure) that is most important. For example, it is well known that too much asphalt in a mix will result in rutting or bleeding or that a three inch pcc pavement will not carry heavy trucks. We can predict the premature failure much more accurately than we can predict life.

What about the future use of resilient modulus in pavement design? Two areas need to be considered in answering this question. First, how will the designer get the number, the "resilient modulus", and secondly, what will he do with it?

Using Texas' experience as a basis, it is predicted that the resilient modulus values for subgrade and unbound base materials characterization will come from back calculation from falling weight deflectometer deflections. Design values for asphalt materials will come from laboratory tests. For materials bound with hydraulic cements either back calculated moduli from deflections at cracks or laboratory moduli adjusted for future shrinkage cracks will be used.

In the late '60's Professor Scrivner convinced the Texas DHT that in-situ characterization was the only practical method to handle the wide range in climate encountered in Texas. (Ref 9) The Texas DHT has been using back-calculated pseudo-elastic moduli since 1972 with fair success. Three important problems appear resolvable in the near future. The problem of non-linearity should be reduced with the heavier load applied by the FWD. More importantly, one should be able to adjust subgrade moduli for "depth to bedrock" in the near future. Finally, by using a model that accepts an elastic modulus instead of the empirical stiffness coefficient developed by Scrivner, Texas can use laboratory resilient moduli values for paving materials. It has been quite difficult to separate paving layers when back-calculating from deflections.

It is predicted that the AASHTO Joint Task Force will follow a similar tact. Marshall Thompson and Ernie Barenberg in NCHRP Project 1-26 have been charged with developing calibrated mechanistic models for the Task Force to use to supplant the AASHO Road Test equations. SHRP has a fairly good start on a data base that can be used to obtain regional factors for that model.

Finally, let the author get on a soap box once again. (Ref 10) We appear to be in fairly good shape relative to new pavement design and with our ability to analyze the stresses, strains, and displacements in un-damaged pavements. This is not so for pavements needing rehabilitation. SHRP has been only able to devote a token effort to rehabilitation performance and the mechanistic-empirical methods hold little hope analyzing the rehabilitation techniques. Full scale field studies to evaluate such items as fabrics, crack and seat, SAMI, etc.

In summary, resilient modulus as a design input has a long history associated with the AASHTO Guides. However, for design input we do not need much precision in individual tests but we may need many tests. Back-calculation for unbound materials and laboratory values for other materials seems most optimum. At sometime in the not too distant future, the AASHTO Joint Task Force will try to adopt a mechanistic-empirical method which can more correctly utilize resilient moduli then the AASHO Road Test algorithms.

APPENDIX A

1. AASHTO, "AASHTO Guide for Design of Pavement Structures 1986", Joint Task Force on Pavements, Highway Sub-committee on Design.

2. AASHO, "AASHO Interim Guide for the Design of Flexible Pavement Structures." Committee on Design (1961).

3. "The AASHO Road Test--Proceedings of a Conference held May 16-18, 1962, St. Louis, Mo." HRB Spec. Rep. 73 (1962).

4. Irick, P. E., "An Introduction to Guidelines for Satellite Studies of Pavement Performance." NCHRP Report 2 (1964).

5. Irick, P. E., and Hudson, W. R., "Guidelines for Satellite Studies of Pavement Performance." NCHRP Report 2 A (1964).

6. Hudson, W. R., and McCullough, B.F., "Flexible Pavement Design and Management-Systems Formulation." NCHRP Report 139 (1973) 64 pp.

7. Van Til, C. J., McCullough, B. F., Vallerga, B. A., and Hicks, R. G., "Evaluation of AASHO Interim Guides for Design of Pavement Structures." NCHRP Report 128 (1972) 111 pp.

8. AASHTO, "AASHTO Interim Guide for Design of Pavement Structures, 1972 Chapter III Revise, 1981", Task Force on Pavements of the Highway Sub-committee on Design.

9. Scrivner, F. H., and Michalak, C. H., "Flexible Pavement Performance Related to Deflections, Axle Applications, Temperature and Foundation Movements." Res. Rept. 32-13, Texas Trans. Inst. (1969).

10. Brown, James L., "The LTPP Experiment of SHRP - An Evolving Process", Proceedings of Roads and Traffic Safety.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Transportation Research Board
National Research Council

FY '90

Project Statement

Project Number: 1-28

Research Project Title: Laboratory Determination of Resilient Modulus for Flexible Pavement Design

Specific Problem Area: Pavements

Research Problem Statement:

The resilient modulus of pavement materials and subgrades is an increasingly important input item for design of flexible pavement structures. Existing laboratory test procedures for determining resilient modulus vary in approach; they appear complex and ambiguous, and require a major investment in time and equipment, while providing questionable results. Even the term "resilient modulus," as used by highway design practioners, differs from the "modulus of resilience" used in other engineering diciplines. Laboratory test procedures do not adequately simulate field conditions, and considerable differences exist between field-determined moduli and laboratory test results.

Resilient modulus is an essential input variable for pavement design using mechanistic concepts. In addition, the emperical design procedures presented in the 1986 AASHTO Guide for Design of Pavement Structures require the resilient modulus of the subgrade as a design input in place of the "soil support value" used in the previous editions. However, in the AASHTO Guide, an undue emphasis may be placed on the use of resilient modulus in determining structural coefficients. This may lead to the misinterpretation that resilient modulus is the only property of importance in this determination.

Objectives:

The primary objective of this study is to develop and recommend laboratory test procedures for determining resilient moduli of component materials in a flexible pavement structure. These procedures are intended for use in design of both new pavements and rehabilitation of existing pavements. The procedures must be able to account for varying field conditions, such as temperature of the asphalt surface layer and moisture content of a subbase or subgrade layer.

Another objective is to assess the applicability and constraints of using the resilient modulus to establish structural coefficients for the flexible pavement design procedure in the 1986 AASHTO Guide.

Accomplishment of these objectives will require, as a mimimum, the following tasks:

Task 1. State of the art. Review state-of-the-art procedures and equipment for laboratory resilient modulus testing and their interrelationships with current and emerging practices for design of flexible pavements.

Task 2. Candidate procedures. From the information obtained in Task 1, identify test procedures and equipment for further development under Task 4.

Task 3. Interim report. Submit an interim report within 6 months after initiation of the research. The interim report shall summarize the accomplishments of Tasks 1 and 2, and include a detailed plan for the laboratory work to be performed under Task 4. NCHRP approval of the interim report and the proposed plan will be required before commencing with the remaining tasks.

Task 4. Test procedures. Develop detailed laboratory test procedures for determining resilient modulus values suitable for use in flexible pavement design. This task may include either modifications of existing equipment and methods or development and fabrication of new equipment or both. The procedures should encompass the normal range of load and environmental factors and material characteristics, and should be suitable for testing both laboratory specimens and field samples. The validity and suitability of the test procedures should be confirmed with sufficient testing of materials encompassing the range of characteristics normally encountered in highway design. The goal of this task is to recommend laboratory test methods that are easily performed and yield consistent and realistic material characteristic values. Recommended test procedures should be in a format suitable for adoption by AASHTO or ASTM. At the conclusion of this task, submit a second interim report containing the recommended test procedures and a detailed plan for the validation and study required in Tasks 5 and 6. NCHRP approval of this second interim report and the proposed plan will be required before proceeding with the remaining tasks.

Task 5. Multi-lab validation. Perform a validation analysis of the recommended test procedures through multi-lab testing. (It is not envisioned that a full "round-robin" laboratory test validation will be accomplished in this project, however, proposals should indicate the extent of validation anticipated under this task.) The results of the validation analysis shall be used to refine test procedures.

Task 6. Field study. Conduct a limited study to compare and analyze field-determined modulus obtained by commonly used nondestructive testing devices and back-calculation procedures with laboratory-determined modulus using validated test procedures. The purpose of this study is to provide an indication of the magnitude of the difference between field and laboratory resilient modulus values.

Task 7. AASHTO Guide. Review the 1986 AASHTO Guide for the Design of Pavement Structures with particular emphasis on Chapter II, paragraphs 2.3.3 and 2.3.5. Assess the applicability and constraints of using resilient modulus values to establish structural coefficients for use in the flexible pavement design procedure. Recommend any revisions as appropriate.

Task 8. Final report. Prepare a final report documenting the research effort and the research findings.

SPECIAL NOTES:

A. This research effort will require continuing knowledge of ongoing activities of AASHTO, ASTM, and SHRP that are related to this project, as well as other NCHRP projects, such as:

NCHRP Project 1-26,	"Calibrated Mechanistic Structural Analysis Procedures for Pavements"
NCHRP Project 9-6(1),	"Development of Asphalt-Aggregate Mixtures Analysis System"
NCHRP Project 10-27,	"Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing"

B. It is recognized this project is equipment-intensive, and an important aspect of proposal evaluation will be consideration of equipment and facilities.

C. Proposal shall include the total costs for each task.

CLOSING REMARKS

by

R. G. Hicks

It gives me pleasure to make the closing remarks of the first workshop on resilient modulus testing. It should be noted that this was the first major effort to focus on resilient modulus testing, a test evaluation method that has been used in pavement research since the 1970's. However, it wasn't until the development of the 1986 AASHTO guides for Pavement Design that resilient modulus was officially adopted as a test method for evaluating soils and materials tests in lieu of the California Bearing Ratio (CBR) and the Hveem Stabilometer (R value). Because of this, most state highway agencies are now trying to "gear up" to measure modulus so they can use it in the 1986 Guide for Design of Pavement Structures and, subsequently, for use in mechanistic design procedures.

At the beginning of the conference, Jim Huddleston outlined a number of questions that were to be addressed as a part of this conference. These questions include the following:

1) What types of equipment are available, and where can these be obtained?
2) What factors affect modulus?
3) What are the limitations of modulus tests?
4) Are tests between equipment types repeatable?
5) How can modulus results be used in pavement design?
6) What does the future of materials testing look like?

Throughout the conference, there has been an excellent response from both speakers and participants in an effort to address these questions. In fact, it's worthy to note that we had more than 170 attendees representing 34 states and one foreign country, Sweden. All of the participants have made contributions, in one form or another, to addressing one or more of the questions.

How did we do in terms of the questions addressed? Let's look at them one at a time. First, what types of equipment are available and where can these be obtained? The answer to this question is conference by demonstrating equipment exhibiting a variety of sophistication. Most of the equipment manufacturers indicated that the hardware and software could range from simple to complex where the equipment is driven by a computer. The equipment ranged from air-operated pneumatic test equipment to air-servo equipment, to sophisticated electro-hydraulic test systems.

Second, what factors affect modulus? It was made clear in papers by Thompson, Monismith, and Epps that a number of factors affect the properties of materials tested for modulus. Perhaps the most important properties are identified for the following materials:

a) Asphalt concrete: temperature, rate and frequency of loading, type of stress pulse;

b) Granular materials: water content, type of aggregate, confining pressure;

c) Soils: water content, deviator stress, soil type

All of these factors need to be carefully controlled if tests are to be repeated either within or between laboratories.

Third, what are the limitations of modulus tests? It was obvious in the discussion that there are a number of limitations with the various modulus tests. The most important limitation is that the modulus test alone is probably not sufficient to obtain layer coefficients for use in pavement design. That is, modulus testing alone is not the whole story. Another major limitation brought out in the discussions is that there are significant differences between the lab-measured versus the field backcalculated modulus values. Before modulus values can be used extensively in pavement design and in pavement evaluation, these differences need to be resolved.

Fourth, are tests between equipment types repeatable? It was also pointed out in the discussions, in particular with the round robin test results, that tests are not always repeatable. Errors come about through operator differences because different agencies measure different properties (total or instantaneous modulus); errors occur because of the black box approach in terms of not really knowing the true stress pulse; and errors arise because of differences in temperature, differences in equipment, etc. As we walk away from the conference, we recognize that errors do exist and that considerable effort is still needed to try to resolve their effect on the differences in modulus values. However, it is also clear that if the various agencies were to follow standard procedures to the letter, these differences would tend to be minimized.

Fifth, how can modulus results be used in pavement design? Throughout the conference it was clear that the modulus test could be used in not only the 1986 AASHTO Guide for determining soil

Funds Available: $425,000

Contract Time: 33 months (including 2 months for review of each interim report, and 3 months for final report review and revisions)

Authorization to Begin Work: Early 1990 - Estimated.

Submit Twenty-Five Single Bound Copies of Proposals to:

PROPOSAL - NCHRP
ATTN: Dr. Robert J. Reilly
Director, Cooperative Research Programs
Transportation Research Board
2101 Constitution Avenue, NW
Washington, D.C. 20418

IMPORTANT

Proposals must be accompanied by an executed, unmodified copy of the Liability/Insurance statement found on the final page of the Project Statement. Proposals submitted without this unaltered statement will be rejected.

Proposal Deadline: **Proposals are due not later than 4:00 p.m. June 2, 1989**

This is a firm deadline, and extensions simply are not granted. In order to be considered, all 25 copies of the agency's proposal **accompanied by the executed, unmodified Liability/Insurance statement** must be in our offices not later than the deadline shown, or they will be rejected. When using the U.S. Postal Service, Federal Express, Emery, Purolator, DHL, or any other **paid** messenger service, use the above address. When **personally** delivering the package, take it directly to room 300 of the Cecil and Ida Green Building, 2001 Wisconsin Avenue, NW, in Washington, D.C. **Do not use the Wisconsin Avenue address for mailing.**

Note 1. According to the provisions of Title 49, Code of Federal Regulations, Part 21, which relates to nondiscrimination in federally assisted programs, all parties are hereby notified that the contract entered into pursuant to this announcement will be awarded without discrimination on the grounds of race, color, religion, sex, or national origin.

Note 2. The essential features required in a proposal for research are detailed in the January 1989 National Cooperative Highway Research Program brochure entitled, "Information and Instructions for Preparing Proposals." **Proposals must be prepared according to this document, and attention is directed specifically to Section IV for mandatory requirements. Proposals that do not conform with these requirements will be rejected.** Requests for the brochure should be addressed to:

Brochure NCHRP
Transportation Research Board
2101 Constitution Avenue, NW
Washington, D.C. 20418

or call (202) 334-3224 for immediate response. In the interest of saving paper, reduced mailing costs, and ease of handling, it is desired that proposal pages be printed on both sides using the lightest bond weight permitting such practice and maintaining margins of less than 1 inch.

Note 3. Proposals are evaluated by the NCHRP staff and a project panel consisting of individuals collectively very knowledgeable in the problem area. Selection of an agency is made only by the project panel and in consideration of: (1) the proposer's demonstrated understanding of the problem; (2) the merit of the proposed research approach and experiment design; (3) the probability of success in meeting the projects objectives; (4) the successes ("track record") in the same or closely related problem area; and (5) the adequacy of the facilities. The total funds available are made known in the Project Statement and line items of the budget are examined to determine the reasonableness of the allocation of funds to the various tasks.

If the proposed total cost exceeds the funds available, the proposal is rejected.

Note 4. Mr. D.W. (Bill) Dearasaugh, has responsibility for surveillance of this project. He can be reached at (202) 334-3236 to answer inquiries.

Note 5. All proposals become the property of the National Cooperative Highway Research Program. Final disposition will be made according to the policies thereof, including the right to reject all proposals.

Note 6. It is not necessary for recipients of this project statement to notify the NCHRP that they do not intend to submit a proposal but that they wish to remain on our mailing list. Until we are otherwise notified, the addressee will remain on our mailing list and automatically receive all future project statements.

IMPORTANT NOTICE

Potential proposers should understand clearly that the research project described herein is tentative. The final content of the FY '90 program depends on the level of funding made available through States' agreements for financial support of the NCHRP in FY '90. Nevertheless, to be prepared to execute research contracts as soon as possible after sponsors' approvals, the NCHRP is assuming that the tentative program will become official in its entirety and is proceeding with requests for proposals and selections of research agencies.

support or layer coefficients, but also in the new, to-be-developed, mechanistic design approaches that are evolving through NCHRP efforts, such as Project 1-26. Furthermore, the modulus values could be used in better modelling the response of pavements under applied loads in terms of fatigue and permanent deformation. The work underway in SHRP Project A-003A should provide assistance in this effort.

Finally, what does the future of materials testing look like? It is clear that the repeated-load testing systems described in this conference will be around for some time, and their use will be enhanced as more and more people begin to use mechanistic design approaches. It also is obvious that both lab and backcalculated moduli will be used in the future. Certainly, through the efforts of AASHTO, ASTM and NCHRP, more repeatable and reliable test procedures will be developed. However test procedures are developed, it appears that they will be used in conjunction with personal computers and possess data acquisition systems.

In summary, I'd like you all to leave this conference feeling that we've had a successful workshop, that we've provided an excellent background for agencies gearing up for modulus testing, and that we have identified the concerns for those who will be in a position to influence the future of modulus testing. Once again, I'd like to thank all of the speakers and the participants for their involvement in the workshop. You all have contributed to the success of the workshop. Thank you for attending, and have a safe journey home.

Appendix A Resilient Modulus Workshop Representatives

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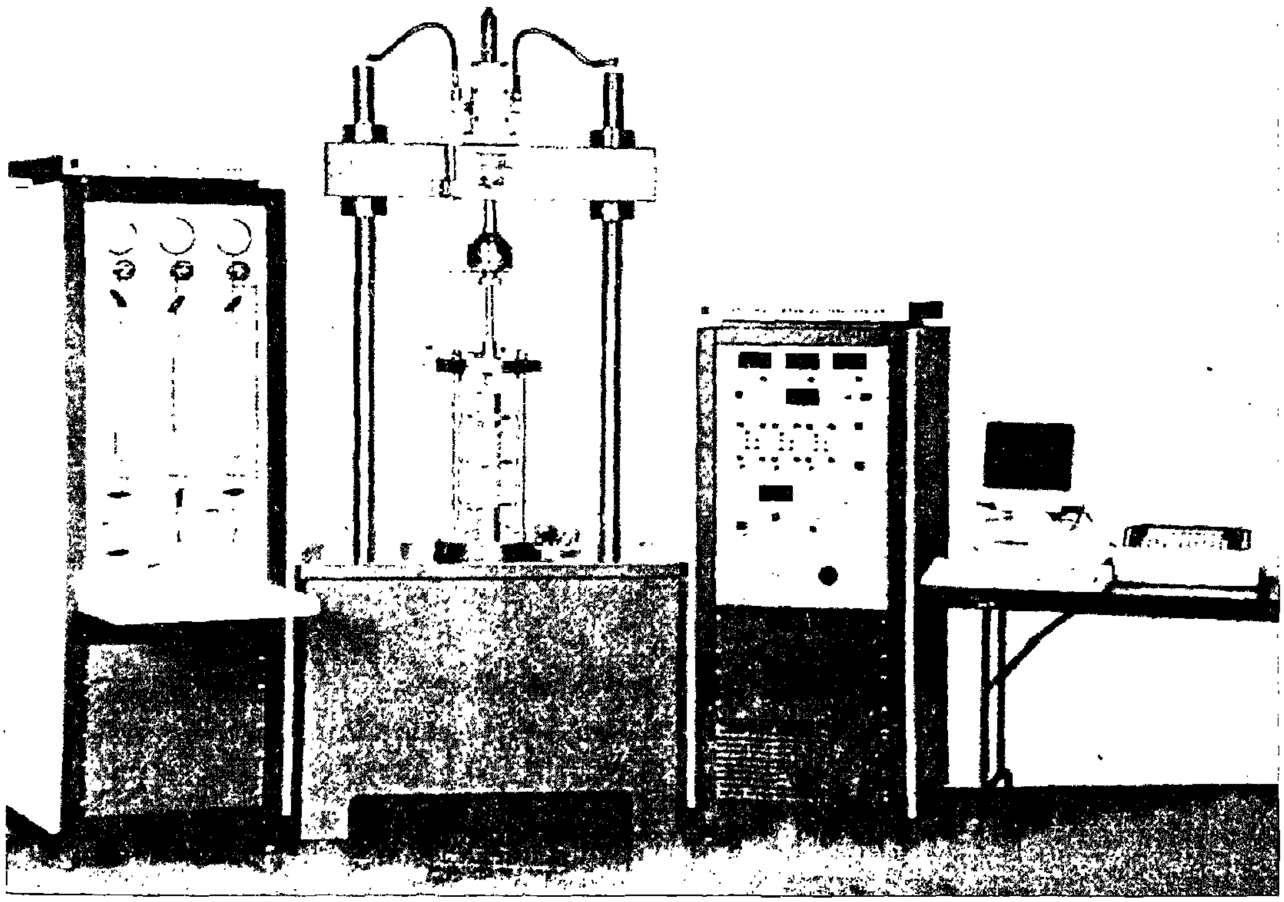
Appendix B Resilient Modulus Test System

Model SM-5400

RESILIENT MODULUS TEST SYSTEM



- CLOSED-LOOP ELECTRO-HYDRAULIC SYSTEM.
- INTERCHANGEABLE LOAD CELLS FROM 500 LB. TO 16000 LB.
- SPECIMEN SIZES UP TO 6 INCHES DIAMETER AND 12 INCHES HEIGHT.
- MENU DRIVEN SOFTWARE FOR AUTOMATED TEST CONTROL AND REPORT GENERATION ACCORDING TO AASHTO 274-82 (1986).
- OPTIONAL ADAPTERS FOR PERFORMING MARSHAL TEST AND INDIRECT TENSION TEST.
- SYSTEMS BUILT TO CUSTOMER SPECIFICATIONS.

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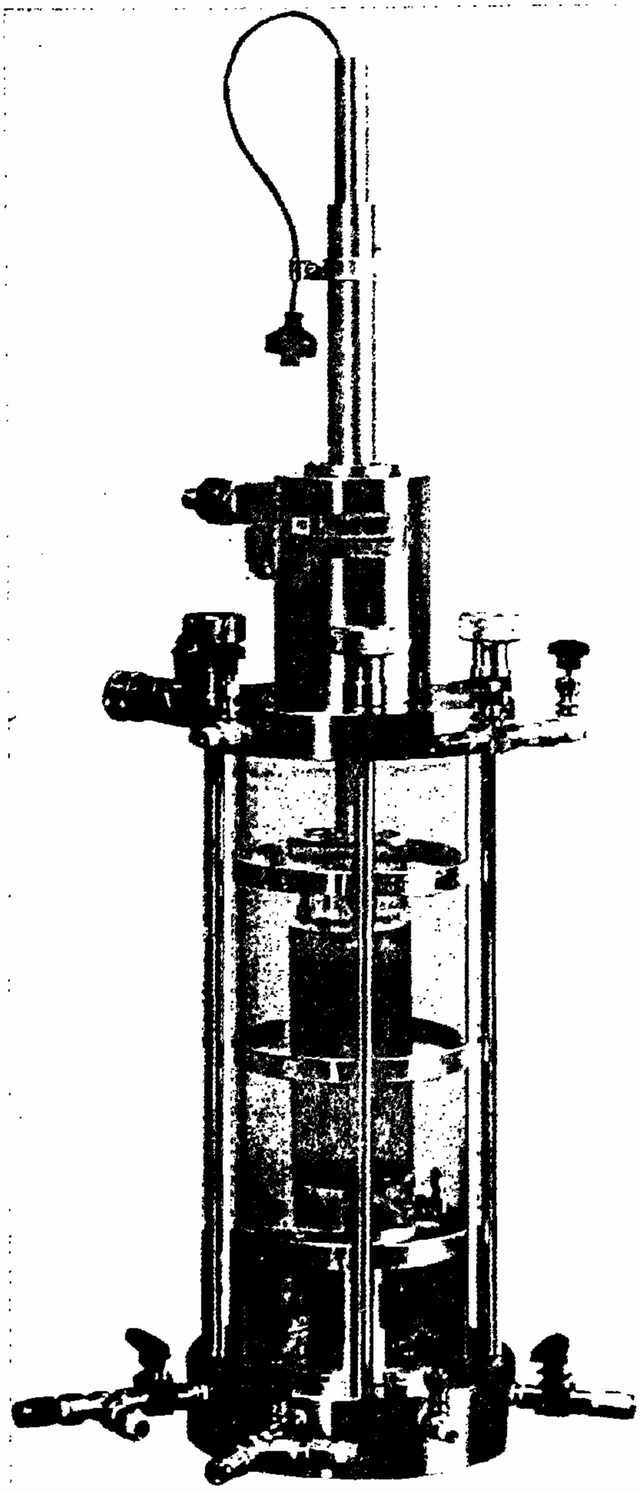
THE HX-100 TRIAXIAL CELL

AND 604 SERIES SERVO CONSOLES



- CYCLIC AND STATIC TESTS
- LIQUEFACTION AND CYCLIC DEGRADATION OF STRENGTH TESTS

A MULTIPLE TEST APPARATUS FOR

- Triaxial compression
- One-dimensional consolidation
- Resilient Modulus
- Unconfined compression

- Top mounted actuators
- Static and cyclic, axial, lateral and pore water controls
- Closed loop servo control
- No loading frames required
- Internal load cells
- Computer Interactive
- Multiple waveforms to 10 Hz

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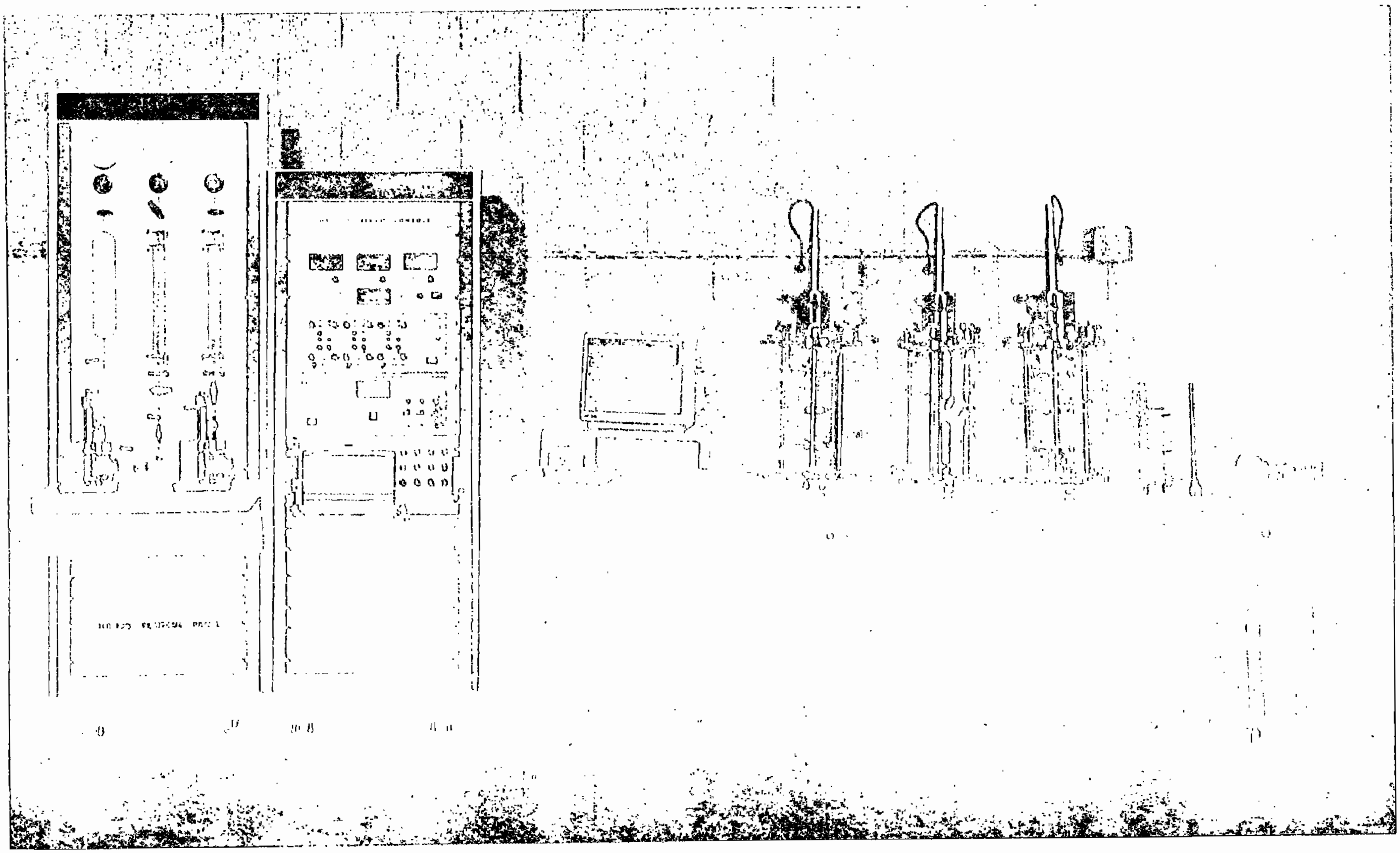
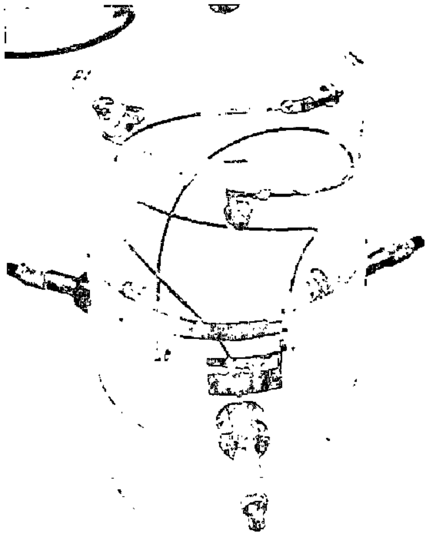
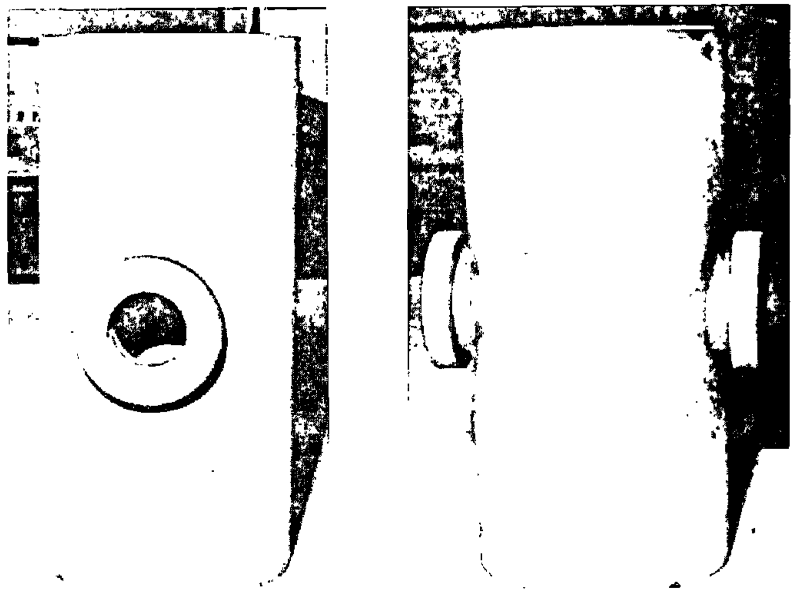
HX-100 CELLS WITH 604-3 CONSOLE

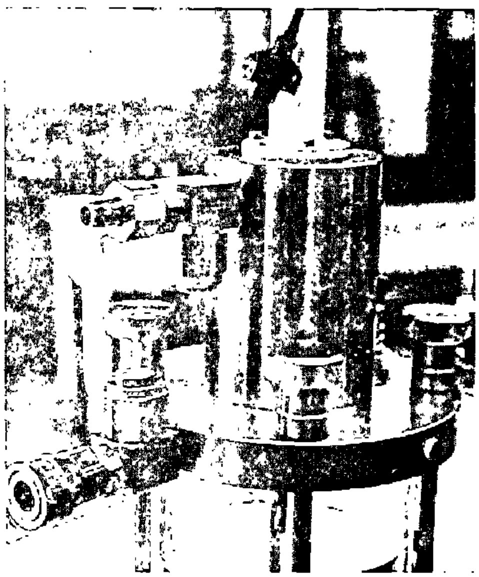




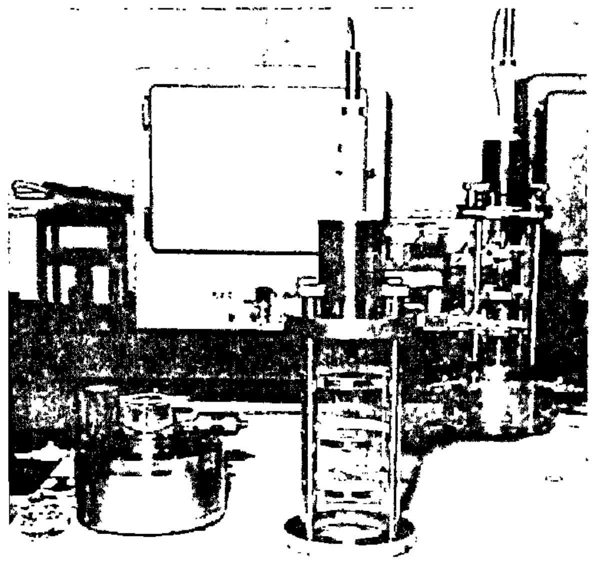
Non-Contacting Strain Transducers



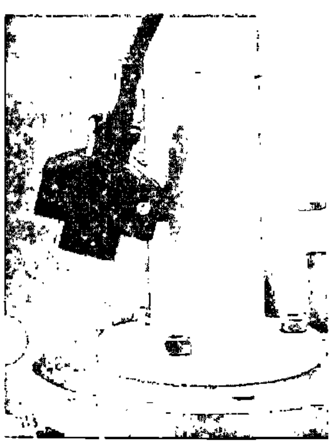
25 Kgm/Cm² Metal Wall



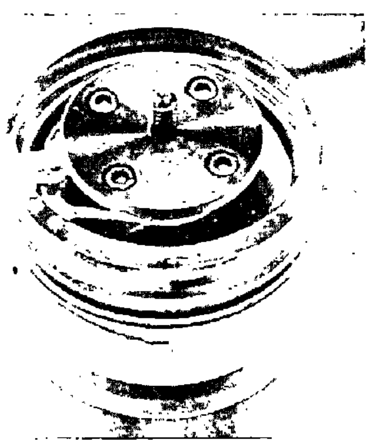
Pneumatic Input Ports to Actuator



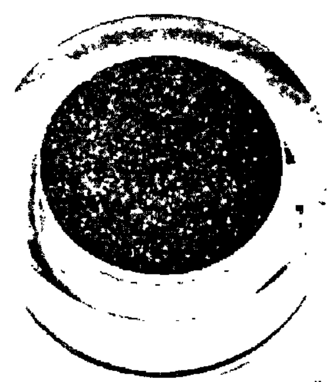
Cell Wall Removed (Assembled)



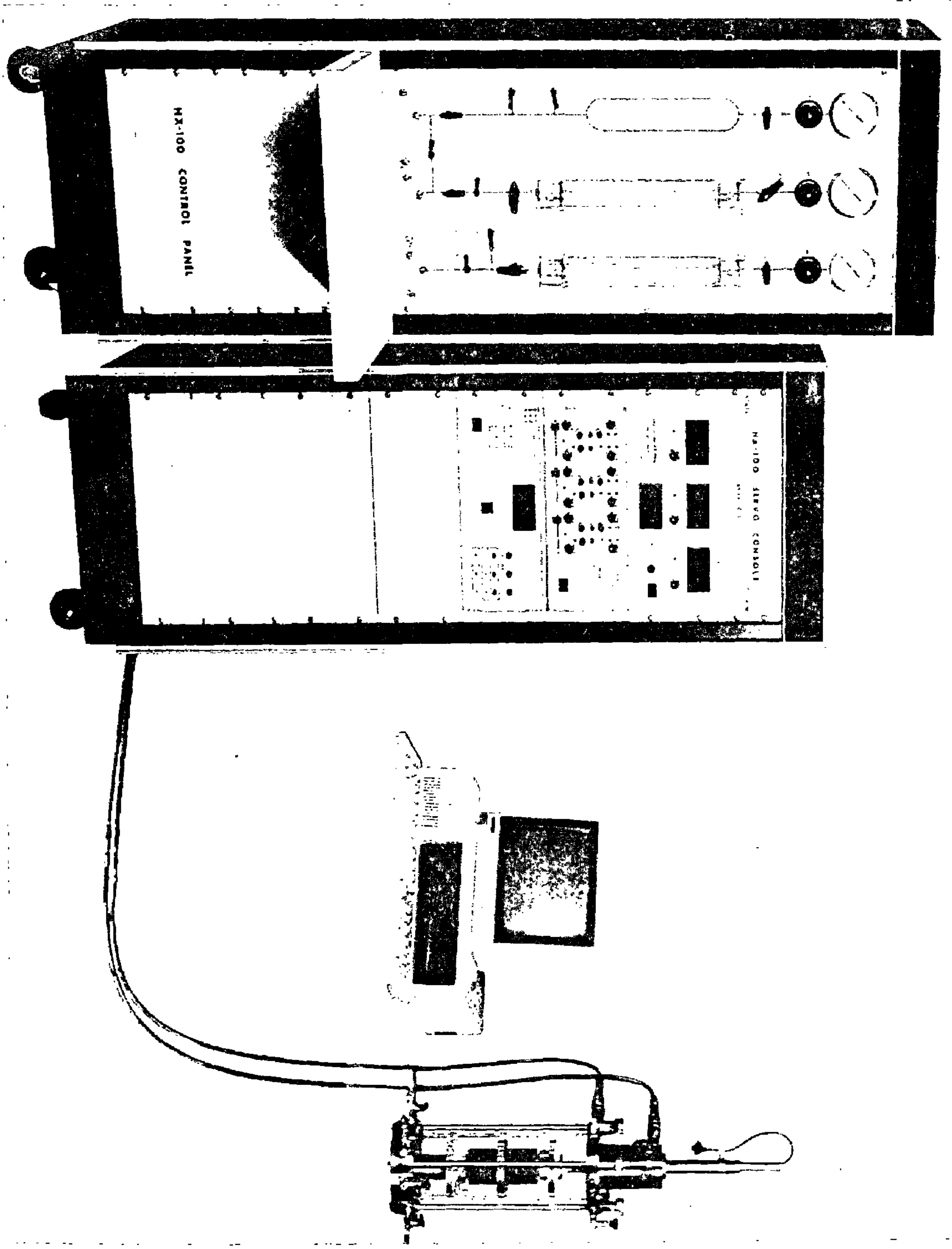
LVDT Transducer
Cable Connector

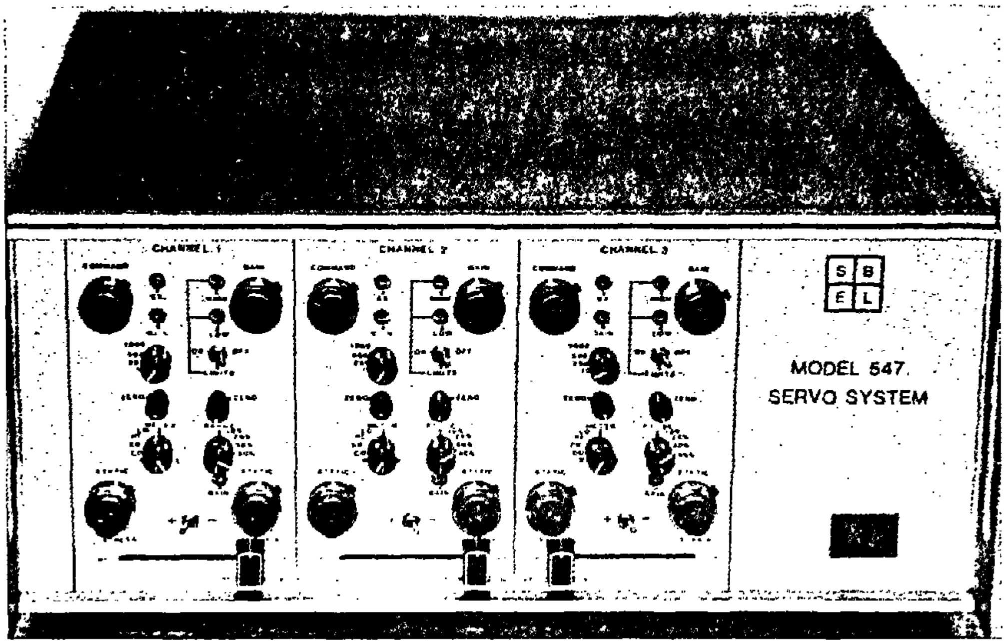


Stress Reversal
Platen



Recessed Porous
Stones


MX-100 CONTROL PANEL
MX-100 SERVO CONSOLE



THE SBEL 547 SERVO CONTROLLER (Dual-Mode) FOR USE WITH

ELECTRO-HYDRAULIC AND ELECTRO-PNEUMATIC CLOSED LOOP SERVO SYSTEMS

Accepts command signals from Function Generators, Computers, and Magnetic Tapes

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GENERAL DESCRIPTION

The Model 547 is a new generation of servo controller. The SBEL Model 547 Servo System is a complete dual mode controller designed to provide the user with optimum state-of-the-art performance. All of the integrated circuit electronics required for full control of a complex servo system are included. The features of the 547 Servo Controller System are:

Full conditioning of the command signal and AC/DC transducer dual feedback signals.
Selectable parameter for limit shutdown.
Multifunction readout monitoring.
Dual full scale static command bias.
Manual or computer programmed feedback control transfer.

Single or multiple channel control systems may be combined for more complex test applications. The 3 channel system can be used for any combination of load control or strain control or pore or lateral control tests.

FUNCTIONAL DESCRIPTION

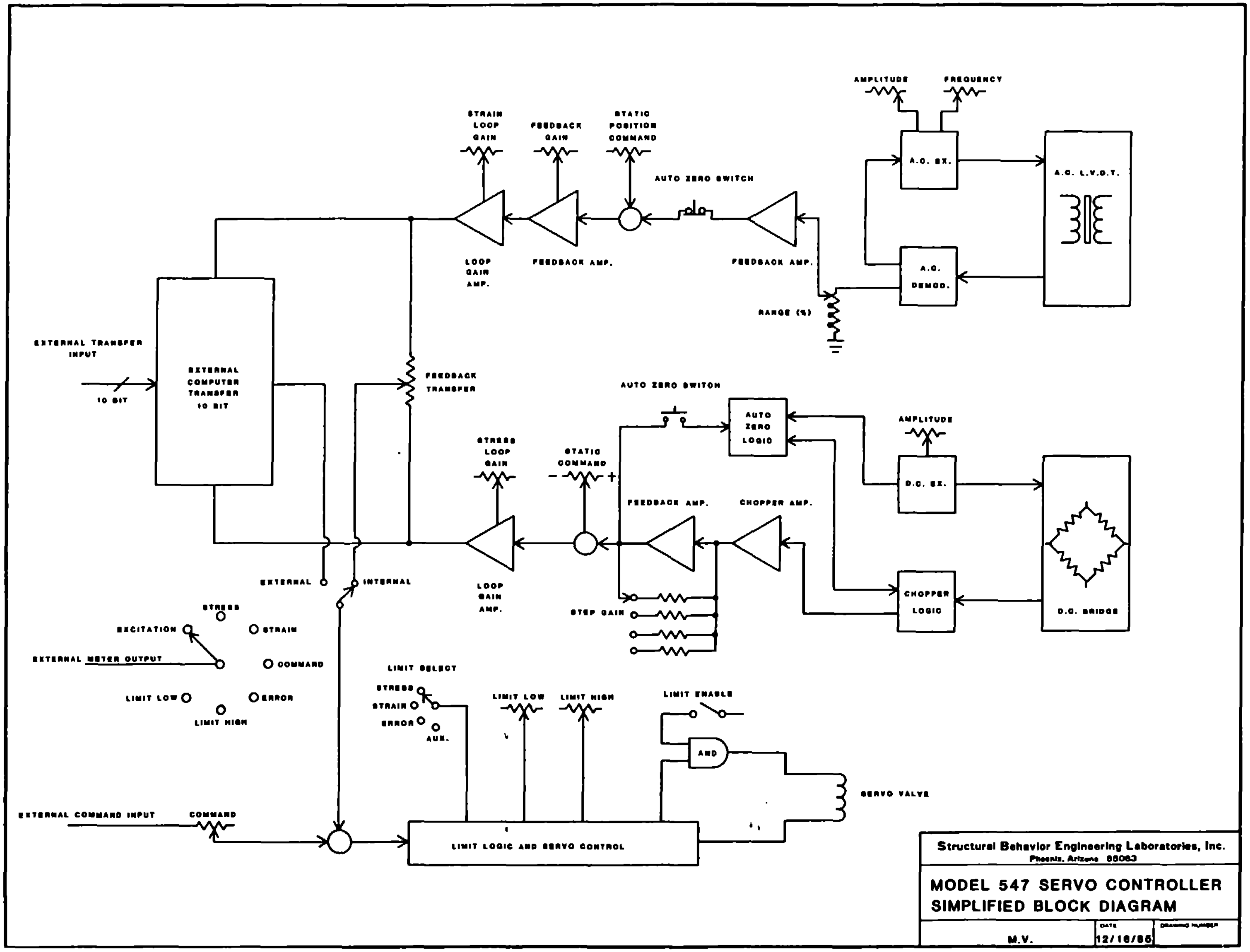
The 547 Servo Controller has dual feedback modes. The Stress feedback transducer is a DC bridge type transducer. Stress control mode employs an automatic null circuit to remove transducer tare imbalance or other offset bias signal. The Strain transducer is an AC LVDT. This mode may be made more sensitive to displacement with the use of a range switch. Both feedback modes have individual static bias potentiometers to apply a static command signal for preload or initial strain.

A dynamic command source signal is input to the 547 Controller from a function generator, magnetic tape input, or computer generated waveform. The computer generated command signals must be limited to $\pm$ 5.0 Volts. The command signal enters the controller thru the command pot attenuator (0 to 100%). The modified dynamic command signal is then summed with the static bias signal. The resultant composite command signal is fed to the outer loop summing junction where it is algebraically added to the AC or DC transducer feedback control signal. The difference or error signal is amplified and used to correct the output so the error is reduced toward zero. When there is information feedback from the output the servo system loop is closed.

The servo system is designed to interface with a computer generated programmable logic signal for feedback control transfer or the feedback control parameter may be selected through the use of a manual slide potentiometer. The choice of transfer method is switch selectable. Stress or strain feedback control may be chosen or control may be transfered during test procedures. The proportion of feedback control will be assigned by the position of the potentiometer slide. This is a smooth feedback control transfer method sometimes refered to as "bumpless transfer." A multi-pin connector and internal/external switch is incorporated so that the computer can program this transfer with a 10 bit binary signal.

During servo control operations limit detection circuitry continuously monitors the limit parameter. The limit detection parameter is switch selectable. Internal limit may be input from stress (LD), strain (SN), or error (ER). The circuit detects when the chosen parameter exceeds the limits preset by the upper and lower limit adjustment potentiometers which are front panel mounted. The adjustment of these limits may be monitored through the meter monitor switch (HI or LO). Should a limit be exceeded, a system shut down will occur via a relay output. The limit detection system may be disabled with the off/on switch. The 547 System provides detection accuracy of the limit mode signals to within ±0.02 volts. An external limit may be chosen to shut down the system. The external limit input (EX) is a contact closure from the computer or other devices.

Other features include an individual servo loop gain adjustment, a switchable meter monitor, an output (BNC) connector for monitoring of adjustments, and BNC connectors for individual feedback output monitoring. The error signal is available at the meter monitor to be used with additional signals and processing to provide null pacing or program interrupts to ensure constant program level integrity.


AMPLITUDE
FREQUENCY
STRAIN LOOP GAIN
FEEDBACK GAIN
STATIC POSITION COMMAND
AUTO ZERO SWITCH
A.C. EX.
A.C. L.V.D.T.
A.C. DEMOD.
LOOP GAIN AMP.
FEEDBACK AMP.
FEEDBACK AMP.
RANGE (%)
EXTERNAL TRANSFER INPUT
10 BIT
EXTERNAL COMPUTER TRANSFER 10 BIT
FEEDBACK TRANSFER
AUTO ZERO SWITCH
AUTO ZERO LOGIC
AMPLITUDE
D.C. EX.
STRESS LOOP GAIN
STATIC COMMAND
FEEDBACK AMP.
CHOPPER AMP.
LOOP GAIN AMP.
STEP GAIN
CHOPPER LOGIC
D.C. BRIDGE
EXTERNAL
INTERNAL
STRESS
EXCITATION
STRAIN
EXTERNAL METER OUTPUT
COMMAND
LIMIT LOW
ERROR
LIMIT HIGH
LIMIT SELECT
STRESS
STRAIN
ERROR
AUX.
LIMIT LOW
LIMIT HIGH
LIMIT ENABLE
AND
SERVO VALVE
EXTERNAL COMMAND INPUT
COMMAND
LIMIT LOGIC AND SERVO CONTROL
Structural Behavior Engineering Laboratories, Inc.
Phoenix, Arizona 85083
MODEL 547 SERVO CONTROLLER
SIMPLIFIED BLOCK DIAGRAM
M.V.
DATE
12/16/86
DRAWING NUMBER

547 SERVO CONTROLLER

FEATURES

One, two, or three channels of dual feedback servo control.

State-of-the-Art electronics control and modern packaging techniques for high performance and reliability.

Dual AC/DC feedback signal conditioning for load, pressure or strain.

Push button transducer automatic zero offset.

Manual or computer control of proportional transfer of feedback control mode.

Multi signal selectable limit detector with independently adjustable high/low threshold, shutdown control and limit override switch.

Selectable outputs to meter and for monitoring feedback transducer outputs.

DC bridge shunt calibration jacks.

Current protected drive to servo valve.

Plug in integrated circuits for ease of maintenance.

SPECIFICATIONS

AC power required	100-240 VAC 47-63 Hz 130 watts
DC power	± 12 VDC @ 1.7 AMP.
SERVO valve	Wired series or parallel Maximum servo valve current ± 200 ma Current limited
DC Bridge Exct.	+6.5 VDC – +10.5 VDC
DC Bridge Feedback Ampl.	A differential input amplifier with rated output to ± -10.00 volts Input impedence 10 Meg ohms Non-linearity ± .01% Bridge supply temp. coefficient .01 %/C Ampl. gain temp. coefficient ± 50 PPM/C
Bridge step gain	100 - 250 - 500 - 1000
AC LVDT Excit.	Excitation 5 – 10 Vpp Frequency 2.1 – 3.2 KHz Distortion < 0.01% AGC controlled
LVDT range steps	10% - 25% - 50% - 100%
Feedback control transfer	Slide Potentiometer STRAIN — STRESS 10 bits binary signal 1023 binary, Stress feedback 0 binary, Strain feedback
Meter output switch	EX = DC bridge excitation LC = DC bridge signal conditioned DC output DF = AC LVDT deformation DC output signal CO = Servo Command input ER = Error signal to servo valve HI = Limit high setting +12.0 - 0 Volts LO = Limit low setting 0 - –12.0 Volts
COMMAND potentiometer	10 turns counting pot 10K ohms .25% linearity
STATIC STRESS pot	10 turns counting pot 10K ohms .25% linearity
STATIC STRAIN pot	10 turns counting pot 10K ohms .25% linearity
STRAIN GAIN pot	10 turns counting pot 10K ohms .25% linearity

	Pin	Function
Stress feedback transducer connector	B	Transducer output
	C	DC Excitation
	D	DC Excitation
	E	Transducer output
Strain feedback transducer connector	B	Secondary output
	C	AC Excitation
	D	AC Excitation
	E	Secondary output
	F	Secondary
	A	Secondary
Servo valve connector	A	Drive signal
	B	Drive signal
	C	
	D	

SBEL SERVO-CONTROLLER SOFTWARE

DESCRIPTION

This program is intended to be used along with the 547 SERVO CONTROLLER SYSTEM. The program is self documented (user friendly) and all that is needed to operate it is to follow the screen instructions as you proceed.

The program generates 3 different command outputs simultaneously to drive up to three channels on the 547 SERVO SYSTEM. Each one of these signals are user definable with a variety of wave forms to choose: ramps, sine waves, random waves, etc. In case none fit your needs there is also an option for reading the command from a file, i.e. an earthquake response record. These command outputs can be scaled and/or converted to engineering units. Complicated defined commands that will be used regularly may be stored in a disk file to be used later, thus saving time in defining such command every time.

Another feature is the automatic data acquisition. There are 16 channel inputs for reading the different transducers used in a test. Channel data is displayed in user defined units and plotted at real time on screen. The program can store a maximum of 8000 data values for each channel. This data is automatically saved into a disk file. Different files may be used for each stage of a test.

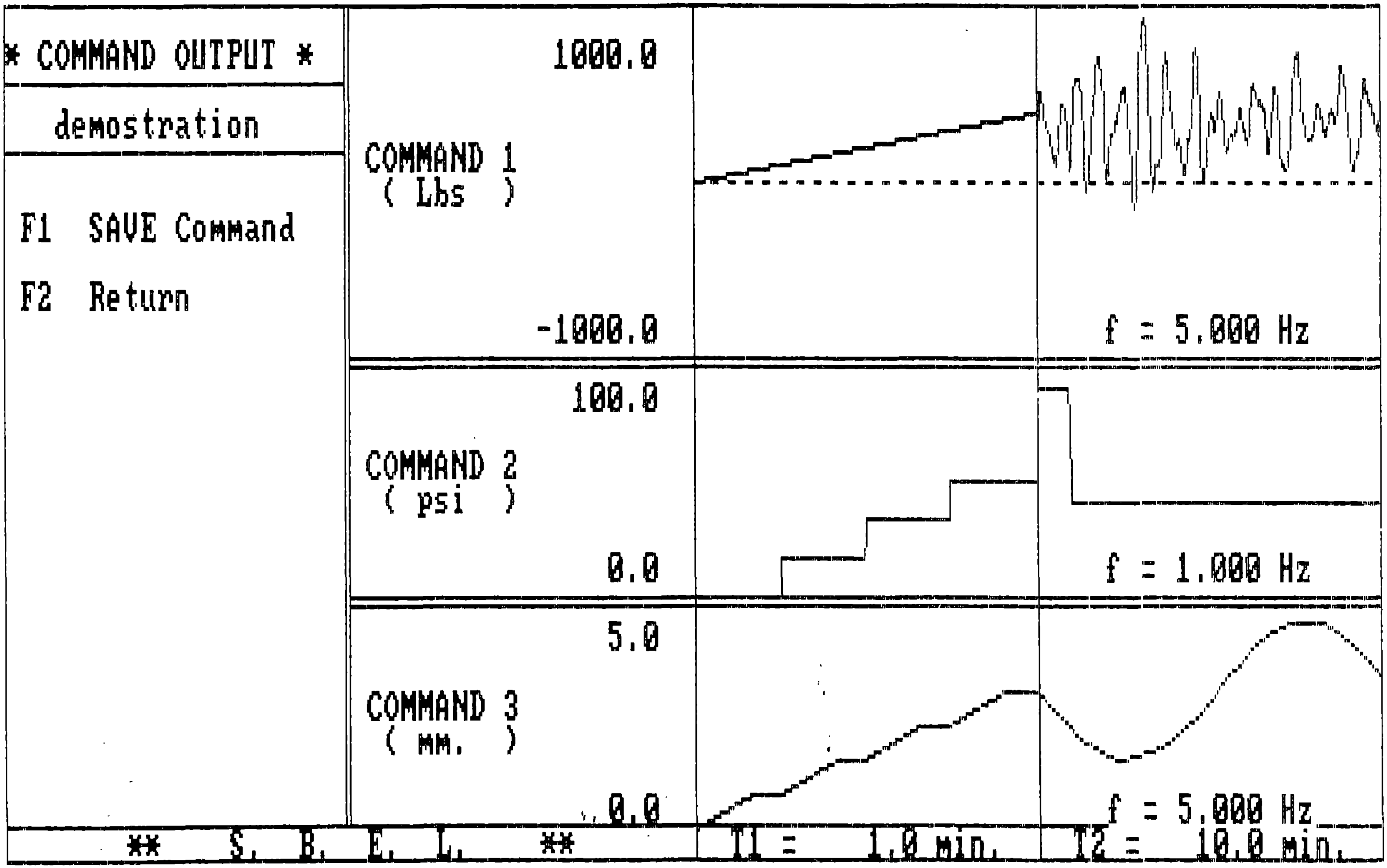
Also provided in this program is a general graphics option to plot data that has been acquired or stored in a disk file. The program lets you specify a formula to be plotted for each axis. These formulas may include data from several channels (to plot averages for example), corrections or conversion factors, etc. Data may be plotted in a linear-linear, linear-log, log-linear, or log-log scales. Filtering, zooming, and editing capabilities are also included.

COMPUTER HARDWARE REQUIREMENTS

—IBM PC/XT or AT compatible computer running MS-DOS or PC-DOS version 2.0 or later
—640 K RAM available memory
—2 floppy disk drives (1 floppy and a hard disk recommended)
—IBM color graphics adaptor or compatible
—Color RGB monitor or monochrome composite monitor
—1 available expansion slot
—Graphics printer

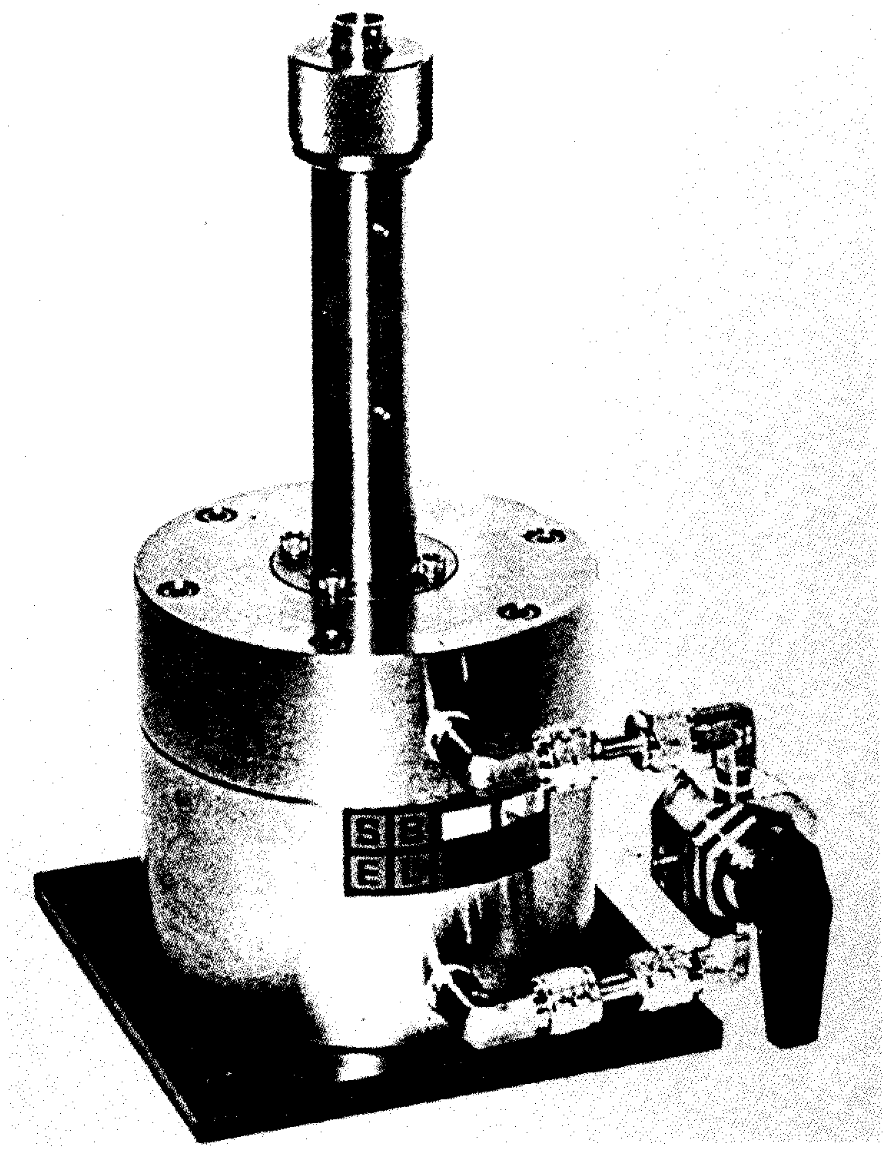
The 8087/80287 math coprocessor is now supported by this program, although it's not necessary.

***IMPORTANT: Specify your computer hardware configuration when ordering.**



TYPICAL COMPUTER WAVEFORMS

VOLUME CHANGE DEVICE SOIL MECHANICS



The VC-25A Volume Change Device supercedes our previous Model VC-25 Apparatus. Unlike the previous model the new unit does not require an A.C. Signal Conditioning System, merely a stable ± 15VDC excitation.

As an optional item, we offer the SCS-25DC Signal Conditioning System which supplies ± 15VDC excitation to the tranducer and also incorporates an electronic zero shift or null feature.

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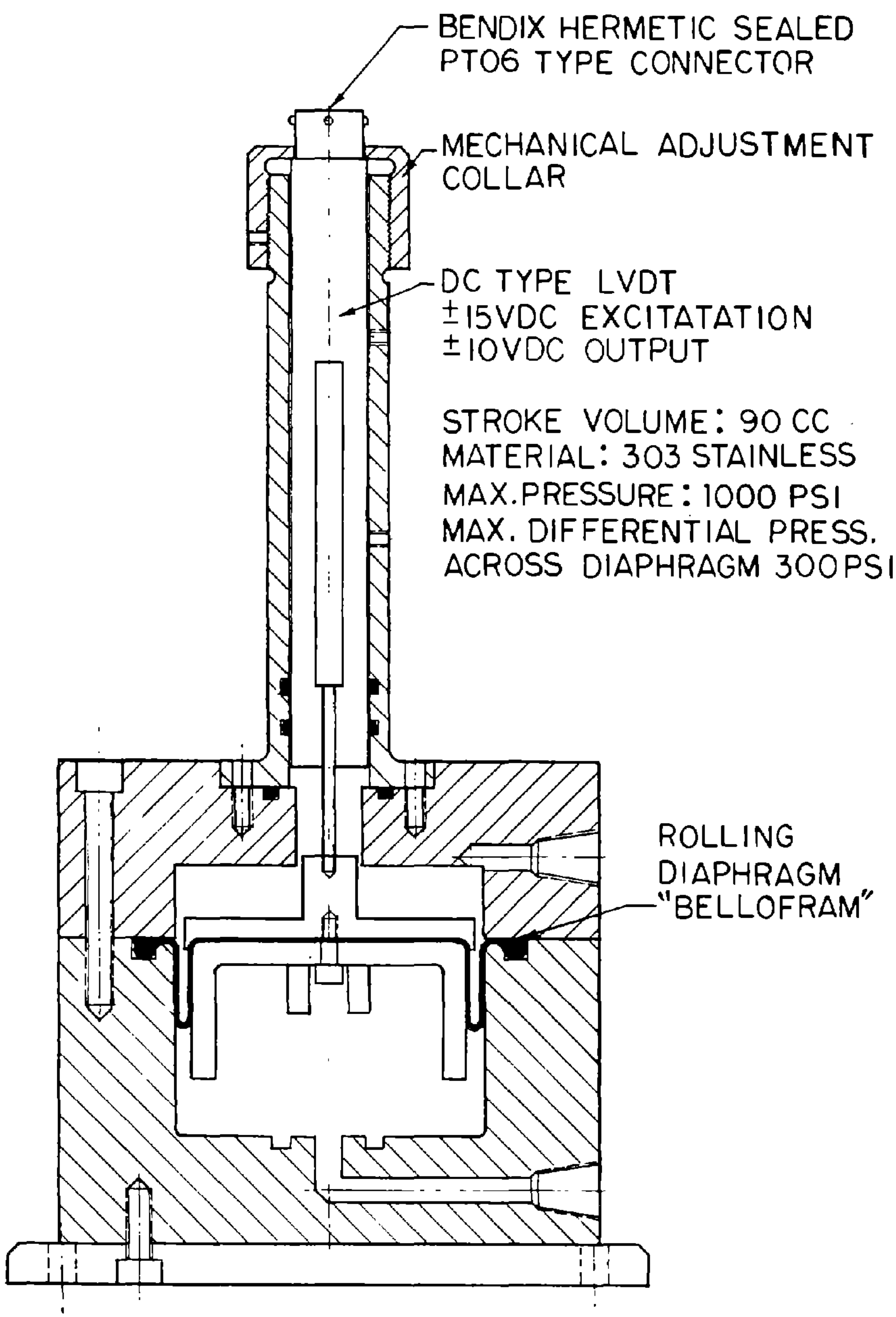
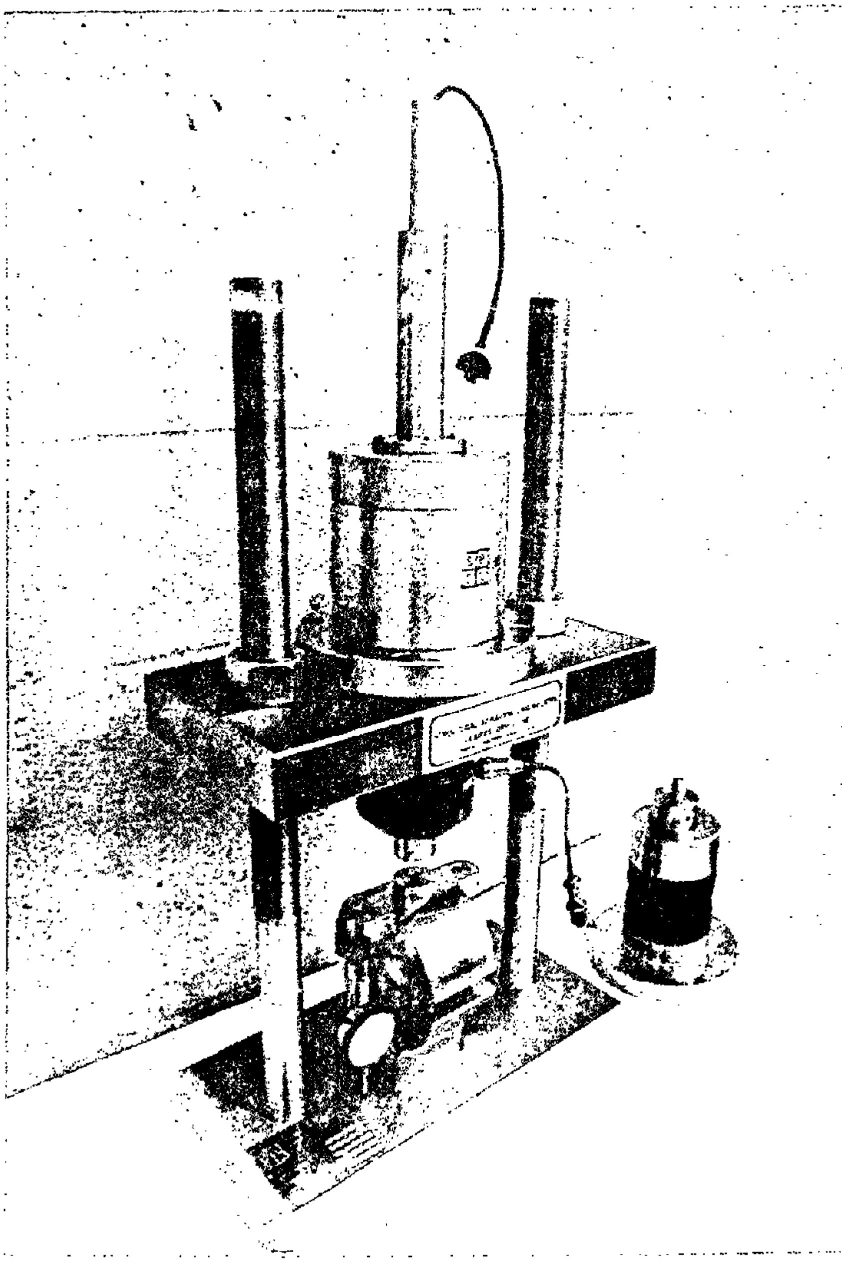
Phoenix, Arizona 85063
Telephone: (602) 272-0274
John O. Hess, President


BENDIX HERMETIC SEALED
PT06 TYPE CONNECTOR
MECHANICAL ADJUSTMENT
COLLAR
DC TYPE LVDT
±15VDC EXCITATATION
±10VDC OUTPUT
STROKE VOLUME: 90 CC
MATERIAL: 303 STAINLESS
MAX. PRESSURE: 1000 PSI
MAX. DIFFERENTIAL PRESS.
ACROSS DIAPHRAGM 300 PSI
ROLLING
DIAPHRAGM
"BELLOFRAM"



SERVO CONTROLLED 5500 LB. LOADING FRAME WITH MARSHALL MOLD AND LOADING PLATENS

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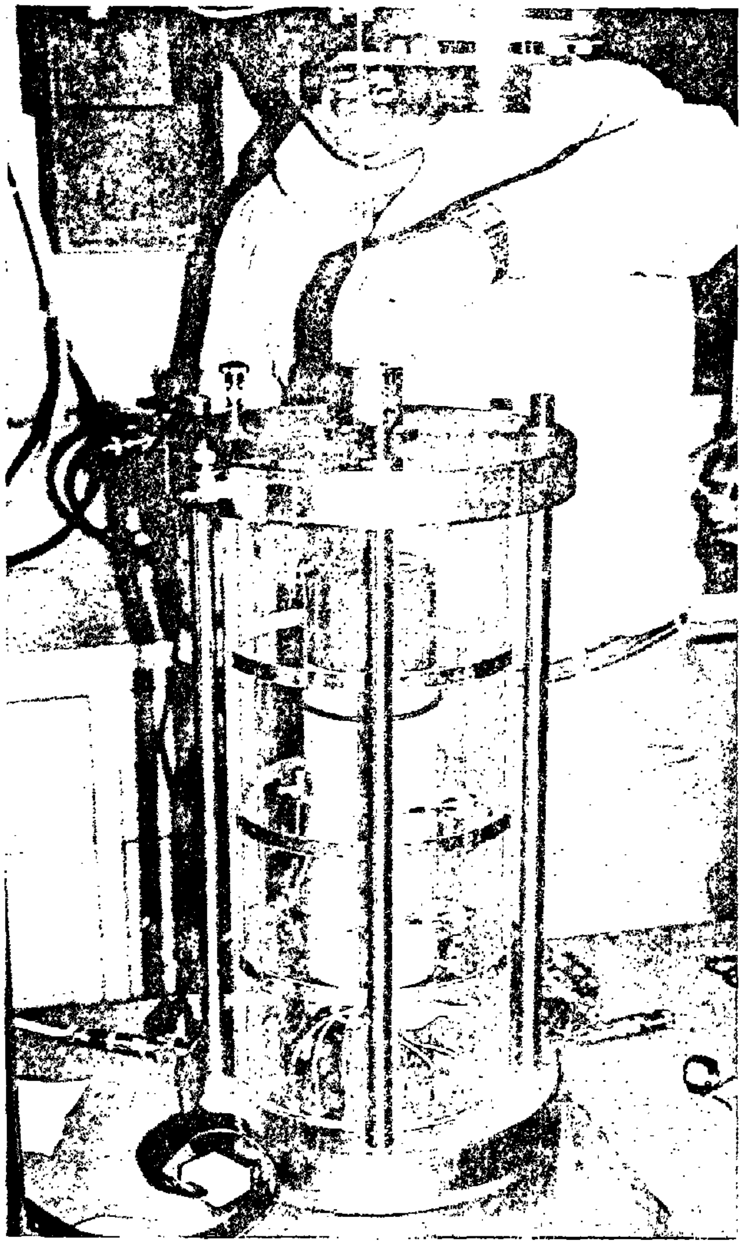

TRIAXIAL CELL WITH AXIAL AND RADIAL DEFORMATION CLAMPS

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Appendix C Resilient Modulus Repeated-Load Test System
H & V Materials Research and Development, Inc.

Resilient Modulus Repeated Load Test System

H & V Materials Research and Development, Inc.
P.O. Box 1708
Corvallis, OR 97339

March 1989

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APPENDICES

A: Resilient Modulus Diametral Test Theory

B: Resilient Modulus Calculations Based on Repeated Load Triaxial Test Results

1.0 INTRODUCTION

The Repeated Load Test System described herein was designed to conduct dynamic diametral and triaxial tests with maximum reliability and ease of operation. The operator can control, within a wide range, the variables involved in the repeated load diametral and triaxial tests as follows:

1) Amplitude of applied load
2) Load duration and frequency
3) Static load
4) Confining pressure (triaxial only)

The use of a separate signal output channel for the load and displacement transducers and a large selection of signal conditioning sensitivities provide the operator with a good indication of sample behavior during testing.

The test device is a simple, durable unit requiring little maintenance. The choice of a pneumatic-powered system avoids complicated mechanical, hydraulic and/or electrical devices. Most parts do not require lubrication and need only be cleaned periodically.

In case of malfunction, this manual provides the operator with all pertinent information needed to repair or replace any part using basic tools available in most laboratories. A parts list with name and phone numbers of suppliers has been included at the end of this manual in the event a part needs to be replaced.

2.0 DESCRIPTION

The Repeated Load Test System is shown in Figure 1. It consists of three basic components: (1) load system, (2) testing accessories, and (3) signal condition and recording system. These units are described below.

2.1 LOAD SYSTEM

The load system is shown in Figure 1. It includes an air-powered load frame and a control cabinet from which dynamic and static load can be controlled. Figure 2 shows the electropneumatic system used to apply loads. It consists of a Bellofram air cylinder, a shuttle valve, and a MAC valve. Operation of the MAC valve requires a 110-volt trigger signal, a pilot air supply, and a main air supply. The Bellofram air cylinder can be activated either by the MAC valve line or the static load line. The shuttle valve controls air flow to the Bellofram air cylinder and is designed to allow the line of higher pressure to flow into the air cylinder. Since the MAC valve is normally closed, the static load line is connected to the Bellofram air cylinder when the MAC valve is not activated by an electrical signal. If the MAC valve is activated, the shuttle valve closes the static load line and opens the MAC valve line to the Bellofram air cylinder. Static and dynamic load pressure lines and electrical signals to the MAC valve originate from the control cabinet.

The control cabinet, shown schematically in Figure 3, is composed of an electropneumatic system able to supply air to the Bellofram air cylinder and an electrical signal to the MAC valve. Precision air regulators and pressure gauges provide good control of the static, dynamic, and confining air pressure lines. A dual timer controls the electrical signal to the MAC valve (pulse interval and pulse duration), and a counter records the number of load pulses. The control cabinet also houses the signal conditioning system.

An important feature which controls the operation mode of the Repeated Load Test System is the MODULUS-FATIGUE SWITCH. The OFF position disconnects the timer. This setting is particularly useful during the conduct of a test to move a sample in or out of the testing apparatus. The MODULUS position is the normal test mode. In the FATIGUE position, an electrical circuit must be plugged into the fatigue shut-off socket (right side panel) to activate the dynamic load and the counter. If this circuit is open, the timer is disconnected and testing is automatically stopped. Figure 4 shows an application of this system to the fatigue testing of a standard asphalt concrete sample. A metallic foil tape placed around the sample is used to complete the closed circuit. When the sample deformation exceeds a limiting value, the foil tape breaks, the circuit is opened, and the test is terminated.

2.2 TESTING ACCESSORIES

A diametral yoke shown in Figure 5 is required to conduct repeated load diametral tests. The yoke is used to measure the horizontal deformation of cylindrical samples subjected to dynamic vertical loading. The horizontal deformation of the sample is measured by two transducers. The dynamic load is measured using a load cell.

Triaxial tests require a triaxial cell (Figure 6a). The triaxial cell is a plexiglas cylinder held in place between a top and bottom plate by three rods bolted to the plates. Electrical connections between load and displacement transducers inside the cell and the signal conditioners are made through the bottom plate. The bottom plate consists of a load pedestal on which the sample is positioned. The load applied to the sample is measured by a load cell attached to the load cap which is positioned on top of the sample (Figure 6b). A pneumatic line with a quick connect on the top plate of the cell is used to apply confining pressure. The vertical deformations of the sample are measured with two Linear Variable Differential Transformers (LVDTs). The LVDTs may be placed inside or outside of the cell. The LVDTs inside the cell are held by clamps which are attached at the quarter points of the sample to provide a 4-in. gauge length. The clamps hold the LVDTs on opposite sides of the sample (Figure 6b). An air-tight bearing on the top plate allows the piston load rod to transmit the load to the sample with minimum friction. If appropriate, vertical deformations may be measured outside the cell using the LVDT load rod clamp. For this type of measurement, the gauge length is equal to the height of the sample.

2.3 SIGNAL CONDITIONING AND RECORDING SYSTEM

The signal conditioning system allows the LVDTs and load cell to step through 12 gain settings each. The range of settings allows tests to be conducted on very soft to very stiff materials. The LVDTs must both be near their null setting during a test. The signal conditioning system allows the null position of each LVDT to be established separately. The output signals from the LVDTs and load cell are recorded and manipulated with a personal computer (PC) and the Resilient Modulus (RM) program. The load and displacement traces may be displayed on the PC screen monitor either separately or together, and may be saved to disk and subsequently analyzed. The resilient modulus of the sample, in either the diametral or triaxial test mode, is calculated from this data set.

3.0 INSTALLATION

A 110-volt electrical power source and a reliable air pressure supply are required to operate the Repeated Load Test System. The recommended air system should supply a continuous flow of up to 10 cfm at a pressure between 100 and 150 psi. Below 100 psi, high loads would not be attainable; above 150 psi, the air regulators are not reliable.

The air supply is connected to the left side of the control cabinet. The four air lines located on the right side of the control cabinet are connected to the load apparatus as follows:

1. The <u>TRIAXIAL CELL AIR</u> (3/8-in. O.D. air hose) is connected to the quick connect on top of the triaxial cell.
2. The <u>STATIC LOAD AIR</u> (3/8-in. O.D. air hose) is connected to the shuttle valve opposite the MAC valve.
3. The <u>PULSE LOAD AIR</u> (½-in. O.D. air hose) is connected to the MAC valve.
4. The <u>PILOT AIR</u> (¼-in. O.D. air hose) is connected to the MAC valve.

Although an efficient muffler reduces the Bellofram air cylinder exhaust noise to a comfortable level for the operator, it is preferable that the Repeated Load Test System be installed in a location away from people to reduce disturbance from equipment noise.

4.0 OPERATION OF TEST SYSTEM

Operation of the Repeated Load Test System can be summarized in the following steps:

1. Attach the appropriate air and electrical connections on the control cabinet to the load apparatus (see Section 3.0).

2. Calibrate the load and displacement transducers following the manufacturer's recommendations, or using the calibration module of the RM program. (Note: Calibration need only be done periodically.)

3. Place the sample in the diametral yoke (or in the triaxial cell) and position it in the load apparatus.

4. Adjust the static load and pulse load pressure, pulse interval, and pulse duration to achieve the desired test conditions.

5. Collect and process test data using the RM program.

The specific operation of the Repeated Load Test System is presented below.

4.1 LOAD SYSTEM

All electrical and pneumatic controls of the load system are accessible from the control panel. The main air valve and electrical power switch should both be in the off position before connecting the machine to the supply lines and the testing apparatus.

There are four air regulator controls on the front panel (Figure 3). The main air supply must be opened before air is supplied to the remaining three regulators. The three air regulator controls, which are adjacent to the pressure gauges, direct air to supply: (1) the pulse load, (2) the static load, and (3) the triaxial cell confining pressure. The precision air pressure gauges measure the output pressure in psi for each regulator. At low confining pressures for the triaxial cell (less than 10 psi), the output pressure may be measured with the mercury monometer attached to the side of the control cabinet.

Electrical controls are grouped on the left side of the panel. The timer controls the pulse interval and the pulse duration. These should be set to the desired values before testing. The pulse duration and interval may be adjusted during the Collect Data scan mode of the RM program.

Operation of the load system requires that the appropriate air pressure and electrical connections are made between the control cabinet and the load system (see Section 3.0 and Figure 2).

The load frame accommodates either the triaxial cell or the diametral test sample, depending on the position of the top plate. In either case, it is important to adjust the top plate so that the Bellofram air cylinder piston is in the retracted position to minimize air consumption and to decrease the rise-time of the load pulse.

The PULSE LOAD AIR quick-connect on the control cabinet is a full-flow quick-connect without an end shut-off. To avoid noisy air leaks through this quick-connect when connecting the control cabinet to the main air supply, it is recommended to keep the PULSE LOAD AIR hose connected to the MAC valve at all times.

4.2 DIAMETRAL TEST

Samples to be tested in the diametral yoke should have a 4-in. diameter and be between 2- to 4-in. thick. To measure the horizontal deformation of a sample under repeated load, the following procedure is recommended:

1. Place the diametral yoke (with LVDTs inserted in their collars) on its stand.

2. With the LVDT tips retracted, place the sample inside the yoke and clamp the yoke to the sample using the four thumbscrews.

3. Secure the bottom load strip to the load cell and place the assembly beneath the piston load rod of the Bellofram.

4. Remove the sample with the diametral yoke attached from the stand and place the assembly on the bottom load strip of the load cell assembly; make sure the sample central axis is parallel to the bottom platen.

5. Place the top load strip on the sample making sure that both load strips are parallel and that the plane formed by the strips is perpendicular to the plane in which the LVDTs measure the sample horizontal deformation. Move the sample to align the piston load rod with the center of the top load strip. Apply a static load (approximately 5% of the anticipated dynamic load) to hold the sample in place.

6. Using the LVDT collar thumbscrews, move the LVDT tips until they are in contact with the sample. Referring to Figure 7, which shows the lower signal conditioning panel of the control cabinet, select a middle-range gain setting for LVDT channels A and B. Select LVDT A as the output signal. Move LVDT A with the thumbscrew collar toward the sample until the red vertical light for LVDT A on the lower panel is in the center of the display. Increase the gain setting and move LVDT A until the red light is again in the middle of the display. Continue to increase the gain setting while recentering the light display. Select LVDT B as the output signal and repeat the procedure for LVDT B. At the conclusion of this operation, both LVDTs are in the null position at their highest gain setting.

7. The RM program (Section 4.4) is now used to continue the adjustment process. Select the Collect Data Option from the DATA

submenu. Answer the "type of test?" as "diametral" (press D or Enter) and type in your file name. Next, select "3" to display both load and deformation. With the pulse load pressure at zero, turn the MODULUS-FATIGUE switch to MODULUS. The pulse load signal is now sent to the MAC valve to produce a "clicking" sound. Gradually increase the pulse load pressure to apply a pulse load to the sample. Select LVDT A Output and observe the load and displacement waveforms on the screen. If the displacement waveform is off the screen, the LVDT is not in its null position. To correct this problem, use the thumbscrew adjustment to move the LVDT and cause the waveform to be centered on the screen or use the voltage offset for LVDT A. Next, select LVDT B output and make all adjustments necessary to ensure the displacement waveform is centered on the screen. With both LVDT outputs centered on the screen, select LVDT A+B as the output signal.

8. Apply 50 to 100 load repetitions to the sample at a suitable load to eliminate early plastic flow and to achieve good contact between the sample and the load strips. The pulse load duration and interval timers may be adjusted to obtain the desired load characteristics. The load cell readout is generally the sharpest trace and should be used to set the pulse duration and interval. The adjustment is made during the Collect Data scan mode of the RM program. The resilient modulus is now determined using the RM program (Section 4.4). The strain amplitude and the sample temperature should always be reported with the resilient modulus.

   If a poor waveform is displayed it may be associated with one (or more) of the following conditions:

   a) The static load is too small to prevent bounce of the sample.

   b) The yoke screws are not tight enough.

   c) The load strips are not parallel.

   d) The load strips are not parallel to the sample central axis.

   e) The sample vertical axis is not aligned with the load piston.

   f) The Bellofram piston is well below the half stroke position, causing slow rise time of the load pulses, as well as excess air consumption.

The equations used to calculate resilient modulus and horizontal tensile strain from the results of a diametral test are given in Appendix A. The ASTM Standard Method of Indirect Tension Test for Resilient Modulus of Bituminous Mixtures (D-4123) is the recommended procedure for determiing resilient modulus.

4.3 TRIAXIAL TEST

The triaxial cell is designed to accommodate drained or undrained dynamic triaxial tests on 4x10-in. cylindrical samples. Two LVDTs are used to measure the vertical deformation of the sample under a dynamic axial load. The applied load, the confining pressure, and the vertical deformation of the sample are recorded and used to compute the resilient modulus of the sample. Samples can be tested either confined or unconfined. Unconfined tests do not require the use of the plexiglas cylinder. The sample is therefore placed directly between the bottom and the top platen.

Confined tests require that an appropriate rubber membrane be used around the sample. The rubber membrane should be fit to the sample. The membrane is sealed to the load pedestal with an O-ring. The top part of the membrane should be sealed in the same way to the load cap with an O-ring. Two molds are included with test system. A double split mold with a porous liner is used to prepare cylindrical samples of cohesionless soil (e.g., base course); a single split mold is provided to compact cohesive soils. After compaction the membrane stretcher is used to place the rubber membrane on the sample.

After the sample is in the cell with the rubber membrane sealed to the pedestal and load cap, the following procedure is recommended to conduct a triaxial test with the internal sample clamps:

1. Assemble the sample clamps, spacer bars, and LVDTs. The adjustable targets should not touch the LVDT tip.

2. Place the entire assembly on the sample centering the assembly on the sample in the vertical position and secure the sample clamp to the sample using elastic bands. Some minor adjustment of the position may be required to ensure a stable seating. Remove the spacer bars.

3. Lower each LVDT tip target until it comes into contact with the tip (refer to Section 4.2, step 6 to establish the LVDT null position).

4. Check for and remove any dust or sand on the O-ring seal of the triaxial cell cylinder. Position the triaxial cell cylinder on the base and place the top plate on the cell.

5. Move the piston load rod through the air-tight ball bushing assembly on the top plate to make contact with the top cap. NOTE: The load rod socket must be secured to the load cell.

6. Seal the triaxial cell using the three tension rods and place the triaxial cell under the load apparatus.

7. Apply a small static load to the piston load rod and confine the sample. NOTE: Applying a confining pressure before applying the static load can result in the ejection of the loading rod.

The RM program (Section 4.4) is now used to continue the adustment process. Select the Collect Data Option from the DATA submenu. Answer the "type of test?" as "triaxial" (press T) and type in your file name. Next, select "3" to display both load and deformation. With the pulse load pressure at zero, turn the MODULUS-FATIGUE switch to MODULUS. The pulse load signal is now sent to the MAC valve to produce a "clicking" sound. Gradually increase the pulse load pressure to apply a pulse load to the sample. Select LVDT A Output and observe the load and displacement waveforms on the screen. If the displacement waveform is off the screen the LVDT is not in its null position. To correct this problem, use the voltage offset for LVDT A. Next, select LVDT B output and make all adjustments necessary to ensure the displacement waveform is centered on the screen. With both LVDT outputs centered on the screen, select LVDT A+B as the output signal. (Note: If the LVDT outputs cannot be centered on the screen, the user must disassemble the triaxial cell and readjust the LVDT tip target.)

8. Apply 50 to 100 load repetitions to the sample to eliminate early plastic flow and ensure good contact between the sample and the caps (refer to Section 4.2, step 8 for a procedure to adjust pulse duration and interval times). The resilient modulus is now determined using the RM program.

Contact between the sample and the caps should be as "flush" as possible. This is generally not a problem with soil samples. If testing difficulties appear to be related to this type of problem, it is recommended that (1) the top and bottom of the asphalt concrete sample be capped, and (2) soil samples be retrimmed to ensure the ends are perfectly "square."

The equations used to calculate resilient modulus from the results of a triaxial test are given in Appendix B. The AASHTO Standard Method of Test for "Resilient Modulus of Subgrade Soils" (T-274) is the recommended procedure for determining resilient modulus.

4.4 RESILIENT MODULUS PROGRAM

4.4.1 Hardware/Software Installation

Installation of the program, Resilient Modulus (RM), begins with the assembly of all components of the computer, including a monitor, printer, keyboard, and a dual disk drive configuration in your IBM-PC or close compatible. A math coprocessor chip is not required. However, the MetraByte DASH-8 card must be installed in your computer. The MetraByte card is preset to an I/O address location of Hexidecimal 300 (&h300). Additional information on the MetraByte card is contained in the documentation that supports the card. To insure that the installation is complete, proceed by inserting the program disk. The disk is self-booting and includes a copy of DOS and GRAPHICS.COM which supports your computer.

4.4.2 Setup

Install. After the disk is booted, the program title screen is displayed. Pressing any key displays the Resilient Modulus **MAIN MENU**. The installation of the program continues by selecting **Install** from the **SETUP** submenu. Press **Enter** or press I and an **Install** menu will appear on the screen. Choose the **INSTALL DASH-8 BOARD** by pressing **1**. An explanation of the I/O base address is given. More information on the I/O base address may be found in the MetraByte manual. Press any key to continue. The desired base address will be requested on the screen. The recommended base address from the MetraByte manufacturer is &h300. Press **Enter** after you have chosen your base address. You will be asked whether you want to generate your address file or not. Press **Y** to save the file and you will be back to the **MAIN MENU**. Pressing **N** will return you to the **MAIN MENU** without having saved the file. NOTE: Selection of a base address needs only to be done once or any time the base address is physically changed on the DASH-8 board.

The second choice under the **Install** menu is **CALIBRATE LOAD CELL**. By pressing **2**, an explanation of the load calibration cell appears on the monitor. Calibration proceeds by establishing the unloaded (zero load) case. Make sure the load cell is unloaded and press **Enter**. Place a known load on the load cell and **Enter** the amount of the load in pounds. Next, increment the load by a known amount and **Enter** the total load (the sum of the two loads). Repeat this process by incrementing the load and **Entering** the total load. You can **Enter** up to 20 load increments after establishing the unloaded case or terminate the process at any time by pressing **Esc**.

When the process has been terminated, the data are analyzed via least square adjustment (i.e., linear regression). The data (Load in pounds and Voltage) are then displayed along with the calibration factor and the coefficient of determination (r^2).

The calibration factor is the slope of the best fit line through the data points when load is plotted as a function of voltage. The coefficient of determination is a measure of how well the data fit a straight line. A value of r^2 = 1.0000 means the data follow a straight line whereas r^2 = 0.0000 means you have meaningless results. As a general rule of thumb, data having r^2> = 0.9500 should be considered acceptable. Print the calibration screen by pressing **P** (you will need the calibration factor later). Press **Esc** to return to the **Install** menu.

The third choice **(CALIBRATE LVDTs)** is almost identical to the load cell calibration procedure. Press **3** and an explanation of the LVDT calibration is displayed. Calibration proceeds by establishing an initial position of the LVDTs.

Remove each LVDT from the diametral or triaxial yoke and place them in the calibration stands. Switch the LVDT selector on the control box to A and establish the initial position for LVDT A (it is recommended that the initial position be set to -2 Volts as indicated at the bottom of the screen display). Switch the LVDT selector to B and establish the initial position for LVDT B (-2 Volts). Switch the LVDT selector to A+B. The voltage readout at the bottom of the screen display should be approximately equal to the sum of the voltages for LVDT A and LVDT B (if not, repeat the above procedure to establish of the null position of the LVDTs). When the voltage display is approximately

-4 volts in the LVDT A+B switch position, press **Enter.** Now, displace each LVDT an equal amount (say 0.005-in. each) and **Enter** the total displacement (i.e., the sum of the two displacements: 0.010-in. for this example). Displace each LVDT an equal amount (say 0.005-in. each) again and **Enter** the total displacement (0.020-in. for this example). Repeat this process by displacing each LVDT equal amounts and **Entering** the total displacement. You can **Enter** up to 20 displacements after establishing the initial position of the LVDTs or terminate the process at any time by pressing pressing **Esc.**

As with the load cell calibration, when the process has been terminated the data are analyzed via least squares adjustment. The data (Displacement in inches and Voltage) are then displayed along with the calibration factor and the coefficient of determination. Print this screen by pressing **P** and then press **Esc** to return the **Install** menu. To Exit from the **Install** menu press **Esc.**

Setup Options. You can select the **Setup Options**, **DOS**, or **Quit** options from the **SETUP** submenu by moving the cursor to the option and pressing **Enter** or pressing **S**,**D**, or **Q**, respectively.

Under **Setup Options**, the second selection in the **SETUP** submenu, you can input the minimum and maximum voltage or default values on Channel 1 and Channel 2. By convention, load is Channel 1 and displacement is Channel 2. The default value for the maximum voltage (for the Y axis) is +5, and the minimum value (for the Y axis) is -5. This represents the maximum range of 10 volts that can be accommodated by the MetraByte card. You may input the default values by pressing **Enter** (without any other keystroke). Alternatively, you may select a narrower range of voltages. This has the effect of magnifying the load or displacement trace. This represents a "zoom" feature of the software.

Generally, when you start a test, you will want to use the default values of ±5 volts. When you are conducting a test you may wish to execute the zoom capability. To do so, you must return to the **Setup Options** menu and change the values of minimum and maximum voltage to "zoom" the data on the screen. The minimum and maximum voltages you select must encompass the waveform or the portion of the waveform you wish to view. You may execute the zoom feature on a previously stored data set in the same manner (see **Graph Data Sets** for more details).

Under **Setup Options**, you also input the calibration scale for load and deformation. If you have executed the **CALIBRATE LOAD CELL** and **CALIBRATE LVDTs** routines under the **Install** menu, **Enter** the calibration factors given by the routines. For a diametral test you may select Poisson's ratio and sample thickness or accept the default values of 0.35 and 2.5-in., respectively. A 4.0-in. diameter sample is assumed in the calculation of the diametral resilient modulus.

For a triaxial test, you may select the sample diameter and gage length of the triaxial yoke or accept the default values which, for both cases, is 4.0-in. If an LVDT is attached to the piston loading rod outside the triaxial cell, then the gage length is the sample length. Again, to accept the default value, simply press **Enter**. If you enter a value different than the default value, you must input the value and then press **Enter**. When you are finished establishing values under **Setup Options**, you can save these values in **RM.CNF** by pressing S. If you save these values in this manner, the next time you execute **RM** it will look for this file and the values saved will be established as the default values. Subsequently saving values established under **Setup Options** overwrites previous values saved.

Pressing R allows re-entry of values, while pressing Esc returns you to the **MAIN MENU**.

DOS. When you return to the **MAIN MENU** you may elect to move the cursor block down to **DOS** or press **D**. The **DOS** feature is beneficial when you want to copy or erase a data file, or execute other **DOS** commands. If you select **DOS**, you will be shelled to the **DOS** environment. In this environment, you may execute any **DOS** command. It is always a good idea to check the directory of the disk you will use to collect your data. For example, if you do not have enough disk storage available when you try to store a data set, an error will result and the data will not be stored (If your disk is nearing its capacity, either replace it with a new formatted disk or use the DOS commands **DEL** or **ERASE** to delete unwanted files.).

During the DOS shell, the **RM** program is resident to the computer. NOTE: This is only a shell: the **RM** program remains resident in memory. You can restore **RM** from the **DOS** shell (environment) by typing "**Exit**" and pressing **Enter**.

Quit. The **Quit** option under **SETUP** is executed to exit the program in an orderly fashion. Unlike the DOS shell, **Quit** returns control exclusively to DOS.

4.4.3 Data Acquisition

Collect Data. Following installation of the program as identified in the previous paragraphs, move the cursor block to the **DATA** submenu of the **MAIN MENU**. This is accomplished by pressing the **Tab** key while in the **MAIN MENU**. The **Tab** key jumps the cursor block from one submenu to another in the **MAIN MENU**. Select the **Collect Data** option from the **DATA** submenu by pressing **Enter** or **C**. The initial question asked in the **Collect Data** routine is the type of test: diametral or triaxial? Press **D** for diametral or press **T** for triaxial. NOTE: Pressing **ENTER** is equivalent to pressing **D**. The next question asked is the name of the data file. This may be any name you wish to assign up to eight characters long and three characters after a period as an extension (e.g., FILENAME.DAT). The file name must follow the same convention given in the **DOS** manual.

The next question that is asked relates to the screen display. If you select **1**, only load will be displayed; if you select **2**, only deformation will be displayed; if you select **3**, both load and deformation are displayed. Initially, it is a good idea to display both load and deformation. Therefore, select **3** and press **Enter**. The limits of the load and deformation screens reflect the maximum and minimum voltage levels and calibration scales established under **Setup Options**. If you selected triaxial **(T)** as the type of test, you are asked for the confining pressure (the cell pressure) for the data set. **Enter** the confining pressure in pounds per square inch (psi).

The next question asked under the **Collect Data** routine is the desired sample rate. Three sampling rates are available: 300, 500, and 1000 samples per second. The next question asked is the duration of scan. You may select a duration of scan (DS) in the range 0<DS<=5 seconds.

Together, the sampling rate and duration of scan define the accuracy with which the load and deformation waveforms are collected and, thus, the resolution of the waveforms displayed on the screen. For example, a sampling rate of 500 samples per second with a duration of scan of 1 second gives a more accurate waveform than 300 samples per second at the same duration of scan.

The default values for the sampling rate and the duration of scan are 500 samples per second and 1 second, respectively, and can be accepted by pressing **Enter** (without any other keystroke). These values result in good waveforms that are displayed rapidly. Increasing either value slows program execution with respect to displaying the waveforms and decreasing either value speeds up program execution. NOTE: The duration of scan should be equal to the inverse of the pulse load frequency (e.g., for a 1 Hz loading frequency, use a duration of scan of 1 second; for a loading frequency of 1/3 Hz use a duration of scan of 3 seconds).

After you have entered the duration of scan, a graphics screen will appear on your monitor. The screen is shown on Figure 8. You will see **SCANNING** appear intermittently in the lower left corner if the program is functioning properly. In the scanning mode, the data from your test are collected over the duration scan interval with the number of points you have previously selected. You may note irregular output from your load and/or LVDT channel even when you are not conducting a test. This is associated with outside electrical interference or noise in your laboratory. If the noise as portrayed on your screen is excessive, it could result in erroneous data collected during your test. Should this be the case, you will have identify ways to isolate (ground) the test system from the surrounding environment.

Additional key functions that appear on your graphic display at the bottom of the screen are as follows: Esc, to abort data collection and return you to the **MAIN MENU**; Space, to write data last scanned to a disk; **F10**, to suspend data collection and observe the waveform(s); and S, to return you to **Setup Options** (from **Setup Options** you are returned to data collection). When you press **F10** and freeze the data, you are getting a representation of the data set that you have collected over the duration of scan time interval. Following the freeze of a data set, you may press any key to continue scanning. If you elect to save the data following a freeze of a data set, you must press any key to continue followed by the space bar. (To accomplish this, it is best to simply press the space bar twice in rapid succession.) NOTE: Only those data sets with the waveform near the middle of the display window should be saved. Data sets with waveforms too near the left side of the display window cannot be properly analyzed.

<u>View Data.</u> Press the Esc key to return to the **MAIN MENU**. If you next select **View Data** from the DATA submenu, you will be asked the question, "Name of View Data File to View." You will only be able to view data that you have saved under the **Collect Data** option. The **View Data** option will present data in column form with time, Channel 1-load and Channel 2-deformation. The units shown in the **View Data** option columns represent the voltage outputs of the transducers employed for load and deformation. The time is in seconds. As you press any key, the data set is scrolled (displayed page-by-page) until the end of the data set. You may press Esc to return to the **MAIN MENU** at any time during the scroll operation.

<u>Print Data.</u> Selecting the **Print Data** option will result in the question, "Name of Data File to Print." This refers to a data file which was saved under the **Collect Data** option. If you input the data file name to be printed and the error message appears in the upper right corner of the screen, it indicates the printer has not been turned on, is not on-line, etc., or the file name was not found. The printed data are in the column forms of time, Channel 1-load, and Channel 2-deformation.

<u>Transfer Data.</u> The last option under the **DATA** submenu is **Transfer Data**. Once you access the **Transfer Data** option, the first question is "File Name to Transfer From." This question refers to the file name identified in the **Collect Data** option. The "File Name to Transfer To" is typically different than the file name identified in the **Collect Data** option and may differ with respect to the characters following the period (extension) in that a binary file is created in the **Collect Data** mode whereas **Transfer Data** creates an ASCII file. Consequently, the last three characters past the period may be "ASC," to identify the fact that an ASCII file is to be created. Following the transfer of the file (which may take several seconds), you may access the transferred file with respect to further analysis. The transfer operation is complete when "Press Any Key to Continue" appears on the screen. The ASCII file can be imported into a spreadsheet package (e.g., LOTUS, Symphony) for further analysis.

4.4.4 Data Analysis

<u>Analyze Data Sets.</u> Data analysis is accomplished through the **Analysis** option. You are first asked the type of test: diametral or triaxial? Press D (or **Enter**) for diametral or press T for triaxial. To analyze the resilient modulus of a bound diametral sample subjected to a repeated load identify the name of the data file name to be analyzed. The default data file that is displayed is the most recent data file that was saved under the **Collect Data** option. The screen display will also give the average load and average deformation. You may print this screen by pressing P or return to the **MAIN MENU** by pressing Esc. NOTE: Be careful to press P only once. If you hold down **P**, you will get multiple prints of the screen display.

If you selected triaxial (T) as the test type, you are asked "Material Type: Coarse or Fine-grained (C/F)?" Press C for coarse-grained materials or press F for fine-grained materials.

The classification of coarse vs. fine-grained material must be made by the user. However, it is important that you understand how **RM** treats each material type. By selecting **F** (for fine-grained materials), **RM** assumes the soil (material) will follow the relationship:

$$M_R = f(\sigma_d),$$

where:

M_R = resilient modulus, and
σ_d = the deviator stress imposed on the sample.

Conversely, by selecting **C** for coarse-grained materials, **RM** assumes the soil will follow the relationship:

$$M_R = k\theta^n$$

where:

M_R = resilient modulus,
θ = the sum of the principal stresses imposed on the sample, and
k,n = regression constants determined by least squares analysis.

The calculation of the resilient modulus is identical for both material types but the way **RM** treats the results of each type is substantially different (see below).

Once the material type has been selected, you are asked "Number of Data Files for this Material Type." Unlike analysis of a diametral test sample, the triaxial analysis can accommodate analysis of multiple data files. Herein lies the importance of specifying the test type (i.e., coarse- or fine-grained). If you **Enter** 1, there is no difference in the analysis of a coarse-grained material and a fine-grained material. However, if you **Enter** a number greater than one, **RM** will treat the results of the analysis of the material types differently in two ways:

1. A least squares analysis will be performed on the results of coarse-grained materials and you are given an equation of the form $M_R = k\theta^n$, whereas for fine-grained materials a least squares analysis is not performed; and

2. At your option, coarse-grained material results are plotted (graphed) with log M_R as a function of log θ, whereas fine-grained material results are plotted with M_R as a function of σ_d.

Once the number of data files is **Entered**, you are prompted for the data file name(s). **Enter** the file name(s) for each file to be analyzed. The results for multiple data files are displayed in tabular form and, if you analyzed data files (more than one) specified as coarse-grained materials, you are also given an equation for the resilient modulus. The equation given pertains only to the material analyzed. The coefficient of determination is given as well. Again, the coefficient of determination is a measure of how well the data fit the line calculated by the least squares analysis. In this particular case, the coefficient of determination is a measure of the level of reliability you can place on the equation given for the resilient modulus.

A word of caution is warranted in the use of this equation to estimate the resilient modulus. Several factors affect the resilient modulus of coarse-grained materials. Some of these include confining pressure (which is reflected in the equation), density, moisture content, material type, etc. Therefore, the equation should only be applied to materials equivalent to those used in the test.

You can print the screen display by pressing **P** or you can return to the **MAIN MENU** by pressing Esc. You are also given the option of obtaining a graph of the results (for two or more files analyzed). Press **G** to view the graph for either coarse- or fine-grained materials. NOTE: The analysis of one data file will not give you the option of obtaining a graph of the results. The output of the results is also quite different (not in tabular form). To obtain a graph of the results, you must analyze two or more data files.

Once you are in the graph mode, you can show the legend by pressing **S**. The legend defines the symbols that accompany the data points on the graph. Data points are delineated by the symbols according to the confining pressure (in psi) that existed during the test that the data points represent. The legend can be subsequently hidden by pressing H. The graph can be printed by pressing **P**, while pressing Esc returns you to the **MAIN MENU**.

<u>Graph Data Sets.</u> Finally, the **Graph Data Sets** option allows you to view a data set saved under the **Collect Data** option. You are first asked for the name of the data file to be graphed, then you may select 1-load vs. time, 2-displacement vs. time, or 3-load and displacement vs. time. The graph of the waveform(s) displayed on the screen may be printed by pressing **P**. NOTE: Press **P** only once. Holding down **P** will give you multiple prints of the screen display.

The zoom feature may be executed under the **Graph Data Sets** option when viewing either load or deformation independently. To view a "zoomed" waveform, first view the data set in the dual display mode (load and deformation together) and approximate the voltage range for each waveform. In the dual display mode, the horizontal grid line divisions on the display screen represent a two volt increment. The bottom grid line corresponds to the minimum voltage (-5 volts) for the deformation channel; the fifth (middle) horizontal grid line corresponds to the maximum voltage (+5 volts) for the deformation channel and the minimum voltage (-5 volts) for the load channel; and the top grid line corresponds to the maximum voltage (+5 volts) for the load channel.

Next, return to Setup Options (press Esc and then S) and **Enter** the approximated voltage ranges as the maximum and minimum volts for each channel. Finally, return to the **Graph Data Sets** option to view the "zoomed" waveform(s); select either load or deformation. A zoomed waveform can only be viewed in the single display mode (load **or** deformation). The dual display mode is unaffected by the maximum and minimum voltages selected under **Setup Options**.

You may print the display screen by pressing **P**, return to the **MAIN MENU** by pressing **Esc**, or graph another data set (or the same one) by pressing any other key (e.g., by pressing the space bar you are once again prompted for the name of the data file to be graphed with the data file last graphed as the default file name).

4.5 MAINTENANCE

The Repeated Load Test System is designed for continuous operation as, for example, when conducting a fatigue test. Maintenance requirements are therefore reduced to a minimum. Most of the maintenance is related to the air supply, which should be perfectly clean and free of water and oil. The Bellofram regulators are particularly sensitive to dust and dirt which decrease their precision and can cause severe damage. The air filter shown in Figure 9 provides very good protection for the machine in the event oil or dirt are present in the air supply. This filter should be checked regularly and if any abnormal amount of dirt, dust, water, or oil is present in the filter, it is recommended that the air supply system be checked.

To perform routine maintenance:

1. Check the air filter prior to the conduct of a test.

2. Clean the MAC and shuttle valves (Figures 10 and 11) every three months or when irregularities show up on the load or transducer recordings. Use silicon grease for lubrication of these valves.

3. Disassemble and clean the Bellofram regulators if they do not keep the set pressure or if the increase (or decrease) in pressure is not linear with the regulator setting (Figure 12).

4. Use car wax to protect the non-painted parts of the loading apparatus from rust.

5. Drain any accumulated water from the points indicated in Figure 8.

4.6 REPAIR MANUAL: ELECTRICAL AND AIR DIAGRAM, PARTS LIST

This section contains the electrical and air flow diagrams useful for trouble-shooting and parts replacement. Also included is a parts list giving the part name and source. See Tables 1 and 2 and Figures 13 to 15.

5.0 SIGNAL CONDITIONER OPERATION

The Signal Conditioning Unit is a high quality two-channel signal conditioner to be used with the Repeated Load Test System. Intended signals are load and displacement. This section will familiarize the user with the Signal Conditioner and guide the user through the set-up and operation of the unit in the repeated load test system.

5.1 DESCRIPTION

5.1.1 General

The Signal Conditioning Unit contains three AC carrier amps; one for load and one for each of the two LVDT gage heads. The outputs of the LVDT carrier amps are summed to give total displacement. Full-scale output for the unit is ±7 volts. A Light Emitting Diode (LED) bar-graph gives quick visual indication of the output of the displacement channel (to aid in setting the null of the two LVDTs). A 500-to-1 range of gains is provided in a 1-2-5 step sequence and zero suppression is controlled by 10-turn potentiometers, one for each carrier amp.

The output of the unit can be monitored with a PC-based data acquistion unit, strip chart recorder, storage oscilloscope, or any device capable or recording signals of +7 to -7 volts at frequencies of zero to approximately 50 Hz.

5.1.2 Front Panel Controls

Controls (Figure 16) are generally arranged so the upper ones are load, and the lower ones are displacement. These are described below:

1. Gain

   - A rotary switch controls gain in six steps from 1 (lowest) to 6 (highest)

   - A toggle switch selects a gain of "times one" (x1) or "times ten" (x10)

Note that, although there are twelve possible combinations of Gain settings, the x1 and x10 ranges overlap so there are in fact nine different gain steps (see Table 3). These steps follow a "1-2-5" multiplier sequence to give three gain steps per decade over a 500-to-1 range.

2. Zero

   Each transducer has a zero offset control allowing an offset adjustment of approximately ±10% of full scale. The control is a 10-turn potentiometer and is convenient for repositioning each channel's trace on the screen of the video monitor.

3. LVDT Output Select

This control selects either LVDT A signal, or LVDT B, or the sum of A+B to be sent to the output connector and the LED (Light Emitting Diode) Bargraph. This facilitates adjustment of the LVDT tips to optimum position for the type of testing required.

4. LVDT Output "Bargraph:

This is a Light Emitting Diode (LED) moving-dot display of the currently-selected LVDT output voltage A, B, or A+B. Mid-scale is zero volts.

5. Inputs and Outputs

Three connectors accept signals from the two LVDTs and the Load Cell. The Output provides a connection for the data gathering/monitoring instrument to read load and displacement signals.

5.2 INSTALLATION

5.2.1 General

The Signal Conditioning Unit is contained on a standard 7x19-in. panel and is intended to reside in the lower front portion of the Control Cabinet (Figure 17). Power for the unit is derived from a regulated DC power supply mounted inside the cabinet. A standard duplex 120-volt outlet supports and energizes the DC supply.

5.2.2 Retrofit

Retrofitting to an existing H&V test system is not difficult; however, it should be carried out **only** by **qualified technical personnel**. A 120-volt duplex outlet must be installed in the Control Cabinet; location is not critical (Figure 16). Power can be obtained from the bottom terminals of the fatigue sensor control box. Be sure the outlet is properly grounded to the cabinet and mounted in a metal "handy box" with a suitable metal cover.

Install the Signal Conditioning Unit in place of the blank panel at the bottom of the Control Cabinet and put the DC power supply in place on the duplex outlet. Route the power supply cable down and forward and insert the cable plug into the jack on the Signal Conditioning Unit.

5.3 OPERATION

5.3.1 Start-Up

Connect input cables and transducers to input connectors:

a) Load cell:	blue tag on cable
b) LVDT A	one red tag on cable
c) LVDT B	two red tags on cable

Connect output cable to a PC-based data acquisition system or other suitable monitoring/recording system.

5.3.2 Verification

Verification that the transducers and signal conditioner are working properly can be accomplished as follows:

1. Load cell: set GAIN to 6 and x10, then manually apply load while you watch the PC screen or other external monitoring device. NOTE: The load application can be as simple as just pressing or standing on the active part of the load cell.

2. LVDTs: set GAIN to 1 and x1. Set LVDT OUTPUT SELECT switch to LVDT A, then actuate the tip of LVDT A while you watch the LED bar graph. The dot should show almost full excursion of the bar graph; furthermore, you should be able to center the excursion of the dot by setting the LVDT A ZERO control to its mid-range. Repeat this procedure for LVDT B.

5.3.3 Calibration

Calibration of the unit involves for each of several Gain Settings:

1. Applying a known load (or displacement) in steps.

2. Recording output voltage or other indication for each load (or displacement) step.

3. Determining the "best fit" linear relationship between input and output.

4. Extrapolating as necessary these linear relationships to other Gain Settings.

A specific calibration procedure must be tailored by the user to fit the needs of the lab and equipment.

A convenient calibration program has been developed and is supplied within the RM software to guide the user through the calibration procedure and calculate the "best fit" calibration constant. A manual procedure is outlined below.

A digital voltmeter (DVM) of 1% or better accuracy should be used to monitor the output. A cable terminated in three "banana plugs" can be supplied for the signal conditioning unit to connect the output easily to a DVM. The steps to do this are as follows:

1. Set the signal conditioning unit GAIN to 6x10 (this is the most sensitive setting). Use dead weights or a calibrated compression testing machine to apply load to the load cell until the signal conditioner output voltage reaches +7 volts (7 volts is the maximum linear output). Note the approximate load related to the 7 volt output. Choose a convenient fraction of this as a load increment so the full-scale load spans about five to eight increments. Next, apply the load increments and record the corresponding output voltages. The calibration value can be calculated from these data points using a linear regression process either on a hand-held calculator, personal computer, or by plotting on paper. Alternatively, the calibration subroutine from the **RM** program may be used.

   Example:

   - Load Cell Gain Setting: 6x10
   - Weight producing 7 volts output: 130 lbs
   - Choose load increment of 20 lbs
   - Obtain the following data:

Load (Y)	Output Voltage (X)
0	0.200
20	1.238
40	2.220
60	3.260
80	4.290
100	5.270
120	6.310

   - Use calculator or other means to obtain linear regression (load = Y values, voltage = X)

     Slope = 19.25 lbs/volt
     Y - intercept = -16.2 lbs

   Thus, the load cell calibration constant for

     Gain = 6 (x10): 19.25 lbs/volt

Note that for purposes of getting a calibration constant for the load cell signal conditioner combination, the slope is the calibration constant and the Y-intercept value is not needed.

2. Extrapolating Calibration Constants to Other Gain Settings. There is a 1-2-5 sequence to the GAIN steps of the Signal Conditioner. If calibrations are performed for several adjacent GAIN Settings this 1-2-5 sequence will appear and is generally predictable to within about 1.0%. For example, some calibration constants obtained for an LVDT were:

Gain Setting	Cal. Constant (x 1E - 6 in/v)	Sequence Multiplier
1x1	10,930	500
2x1	4,374	200
3x1	2,195	100
4x1	1,101	50
5x1	441.0	20
6x1	215.5	10

This relationship suggests that only several Gain Settings need to be calibrated and the others may then be extrapolated.

3. Displacement Channel Calibration. Place LVDT A in the LVDT calibration stand and set the GAIN control to 1x1 (least sensitive setting). Monitor the output voltage using the DVM and set the micrometer ("mike") head adjuster to give a signal output of approximately -7 volts. Then set the mike head to its next exact small division on the rotating barrel. Note that each small division on the rotating barrel of the mike head is 0.001" and one full revolution is 0.025". Now rotate the mike head barrel and displace the LVDT tip enough to change output approximately 2 volts (i.e., to -5 volts) so as to arrive at an exact scale division. The goal is to increment the tip of the LVDT about six or seven equal increments and cause the output voltage to go from approximately -7 to +7 volts.

Suggested tip displacement increments are:

Gain	Tip Increments (x 0.001")
1x1	20
2x1	8
3x1	4
4x1 (1x10)	2
5x1 (2x10)	1

Because LVDT sensitivity is so great at higher gain settings, calibration constants for these settings must be extrapolated (see section 4.3.2.2).

You are now ready to apply transducers to the sample and proceed with testing.

5.3.4 Use During Test

Use of the Signal Conditioner is as follows:

1) Diametral. Place diametral yoke with LVDTs on the sample, align the sample into position in load frame, and a apply seating load for stability (approximately 5% of the expected pulse load). Set the GAIN to 4 (x10) to start and LVDT OUTPUT SELECT to A. Then adjust LVDT A inward on the sample until the LED bar graph indicates the output is active around zero (i.e., the dot is at center of the scale). Repeat this process for LVDT B by turning LVDT OUTPUT SELECT to B and centering the dot on the LED bar graph (the LVDT OUTPUT SELECT switch allows a quick check of the status of either LVDT at any time). Next, turn the switch to A+B to sum the LVDTs and start the load pulses. If you "lose the signal" at any time on the recording device, use the LVDT OUTPUT SELECT switch with appropriate ZERO control and mechanical LVDT adjustment to bring the signal back into range.

2) Triaxial. Place the triaxial clamps with LVDTs on the sample; then align the sample into position in load frame and apply seating load for stability. NOTE: For soft materials, a low GAIN must be used to accommodate large displacements. Furthermore, since there is no access to LVDTs inside the triaxial cell, testing must start with LVDTs more or less fully extended to accommodate permanent deformation. For these reasons on soft materials, the operator may choose to place a single LVDT external to the triaxial cell. Next, set GAIN to 1 (x1) and the LVDT

output select to A and adjust LVDT A to the center of the LED display. Repeat this for LVDT B, then back out both LVDT adjusters equally to move the LED dot near the negative (bottom) end of the bar graph display. Testing may proceed when this is accomplished.

5.4 THEORY

The signal conditioner utilizes two AC carrier amplifier "cards" (PC boards), one for single channel of load and one for the two channels of LVDTs necessary for diametral and triaxial testing. The latter two-channel card is equipped with a precision summing circuit so the user can select channel A, B, or A+B for monitoring. The cards provide AC excitation to their respective transducers. Gain switches and zero control is provided at the front panel; an output switch allows monitoring of LVDT A, B, or A+B for easy set-up of LVDTs. A low-pass filter of 50-Hz cut-off frequency minimizes undesirable noise. The LED bar graph allows constant monitoring of LVDT output. A regulated DC power supply energizes the cards and bar graph; nominal voltage is ±12 volts.

CONTROL CABINET

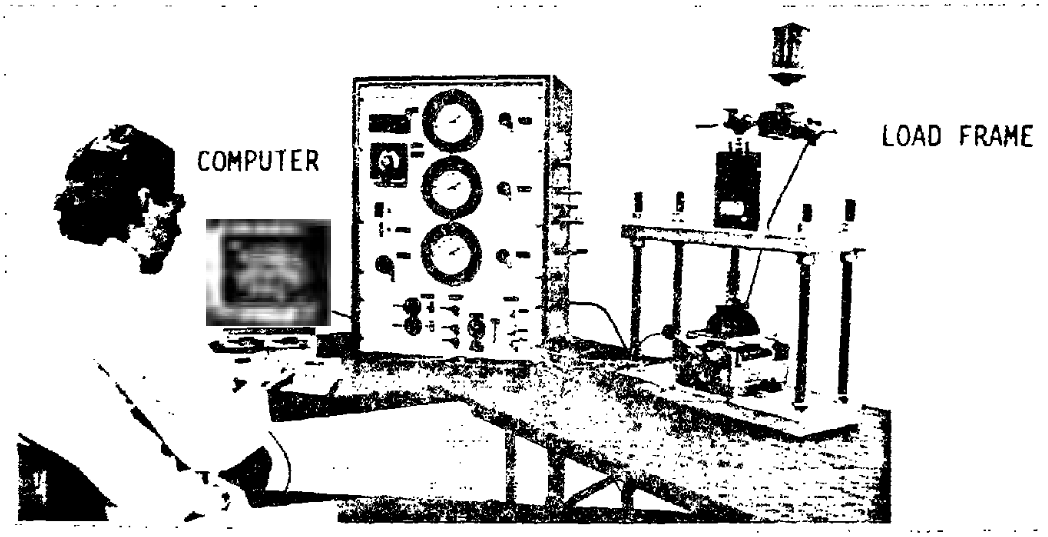


Figure 1 - Repeated Load Test System

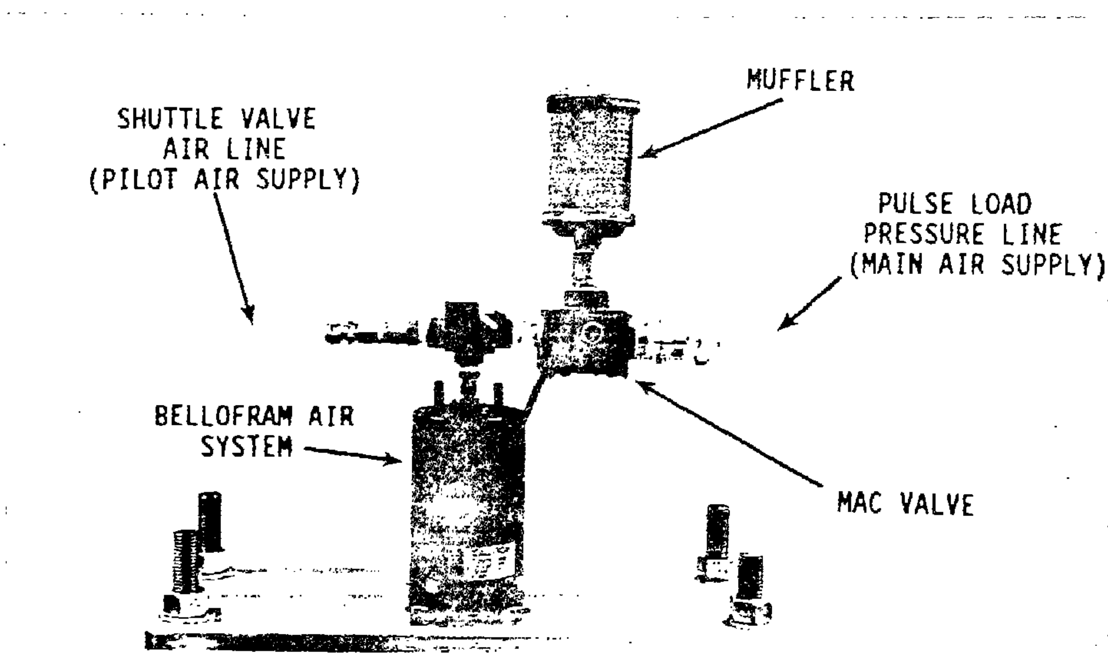
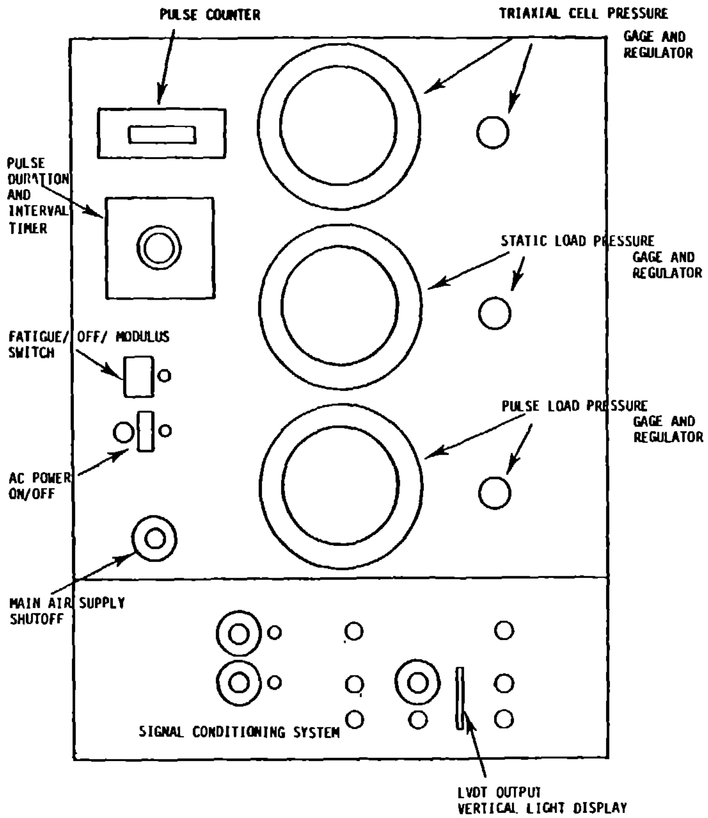
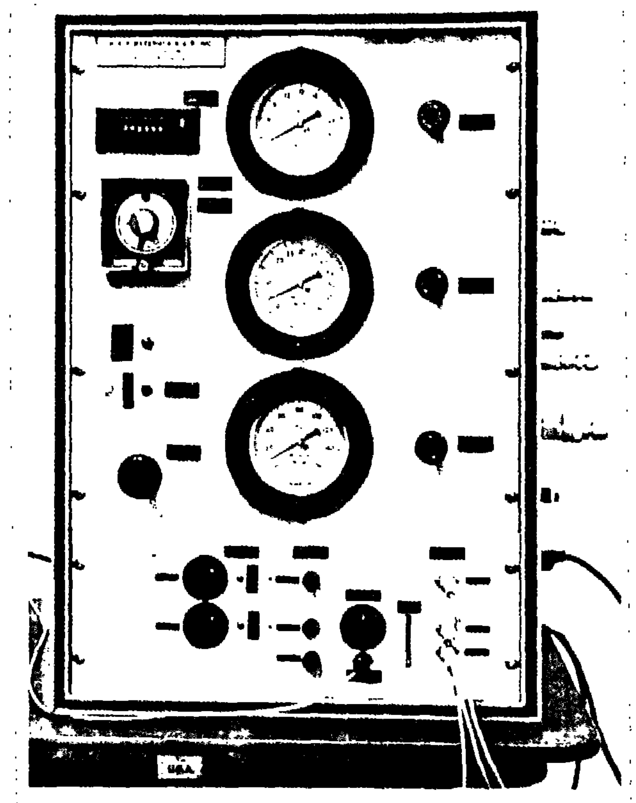


Figure 2 - Electropneumatic Load Applicator



a) Schematic



b) Photo

Figure 3 – Schematic of Control Cabinet

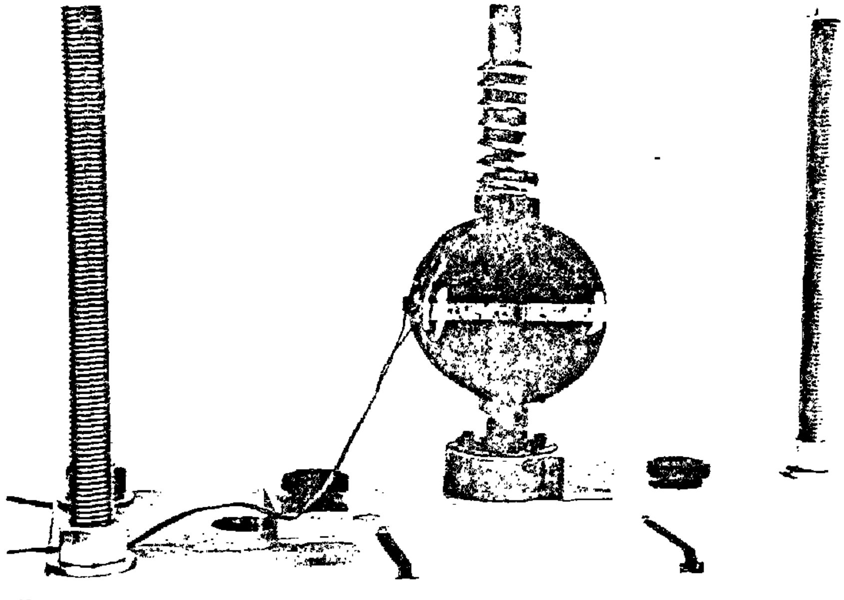


Figure 4 – Example Application of the Fatigue Test Automatic Shut-Off Device

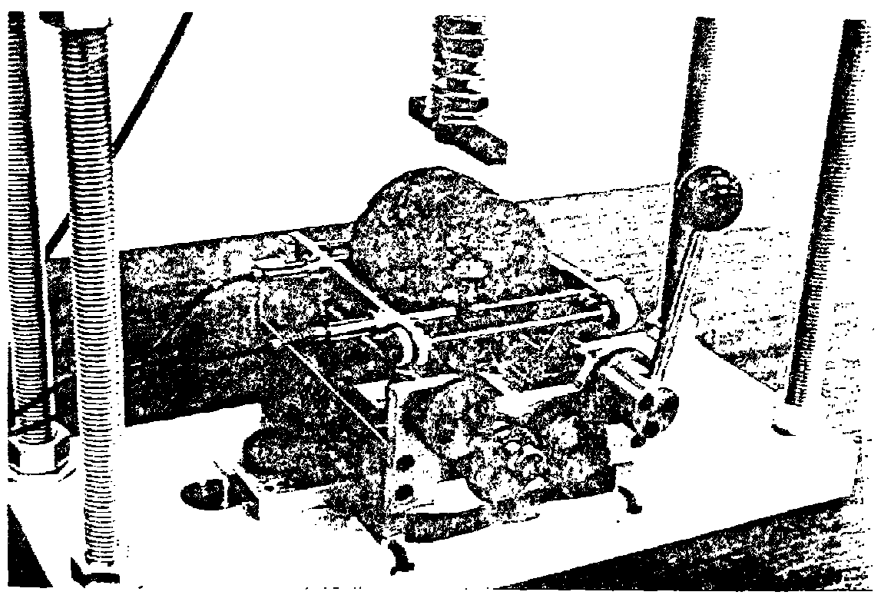
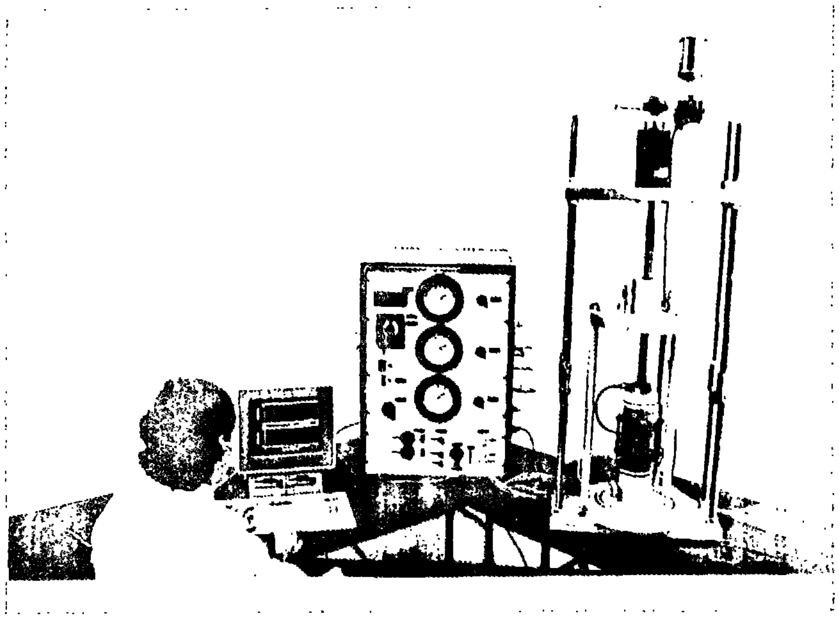
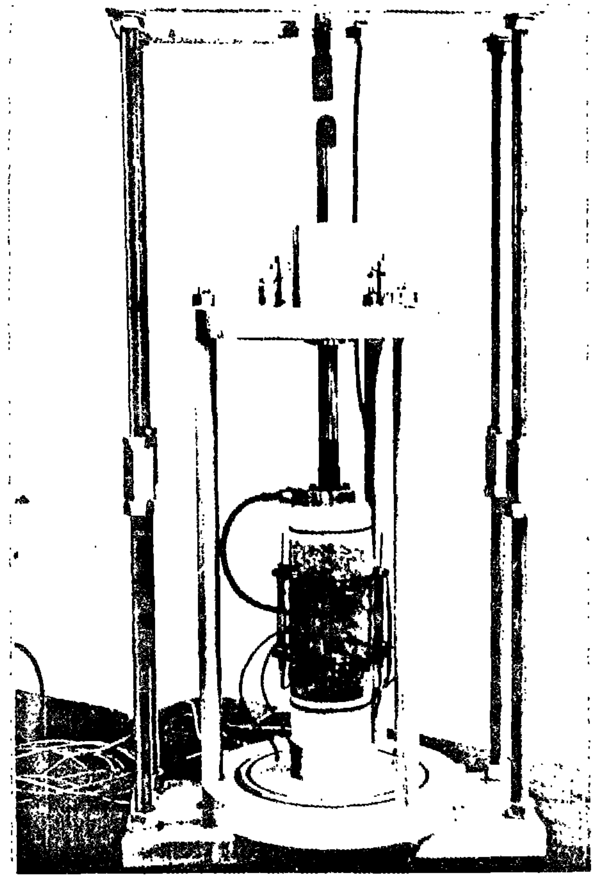


Figure 5 – Sample with Diametral Yoke



a) Overall View of Triaxial Cell Assembled



b) Sample with Load Cell and LVDTs

Figure 6 – Triaxial Test Equipment and Accessories

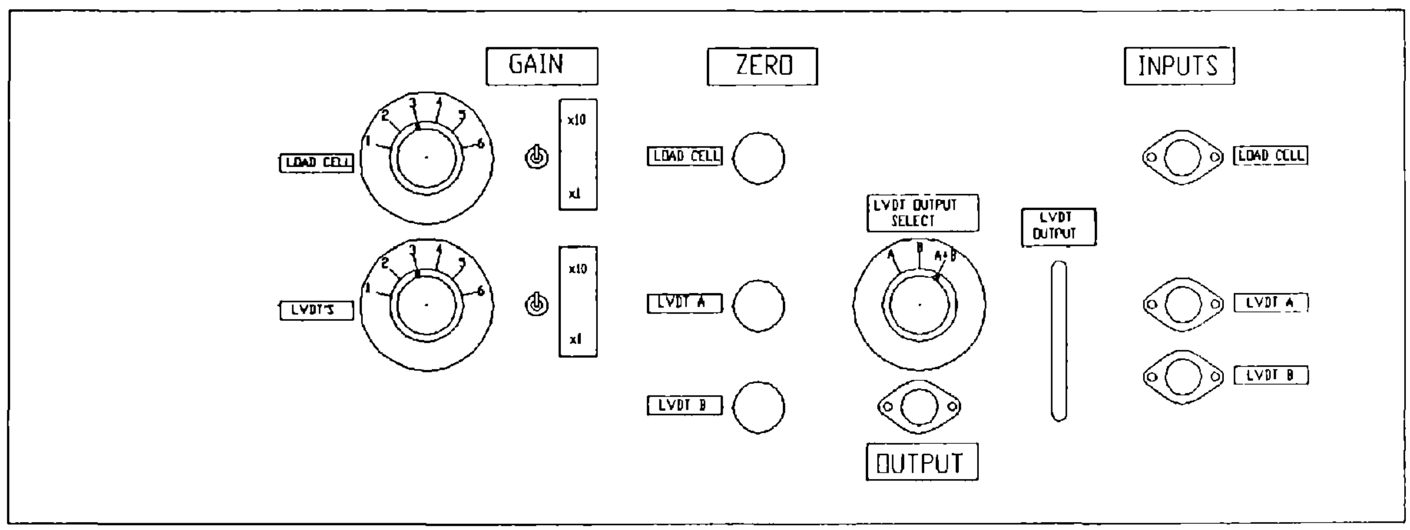


Figure 7 – Schematic of Signal Conditioning Panel

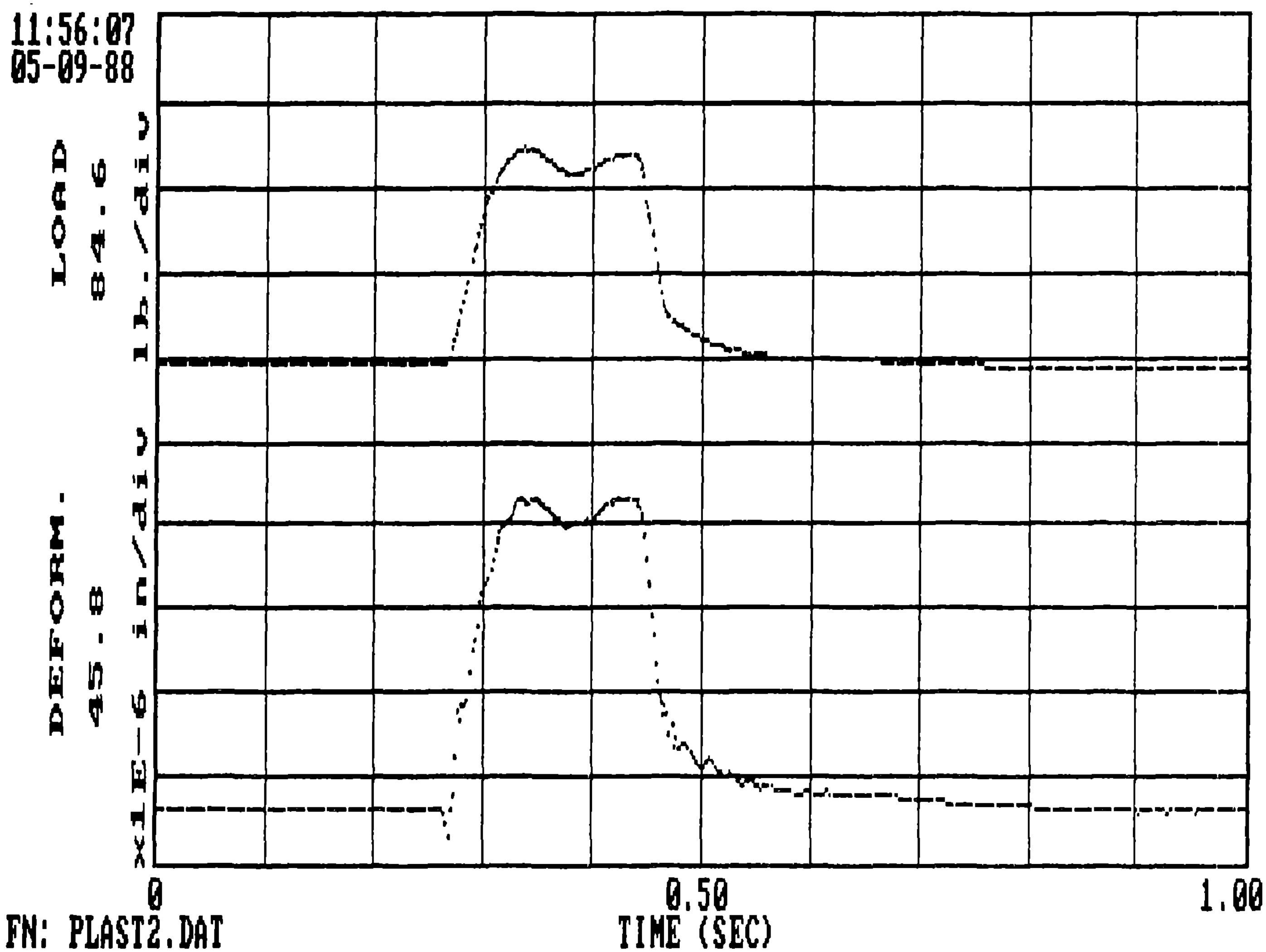


Figure 8 – Load and Deformation Trace from Repeated Load Test

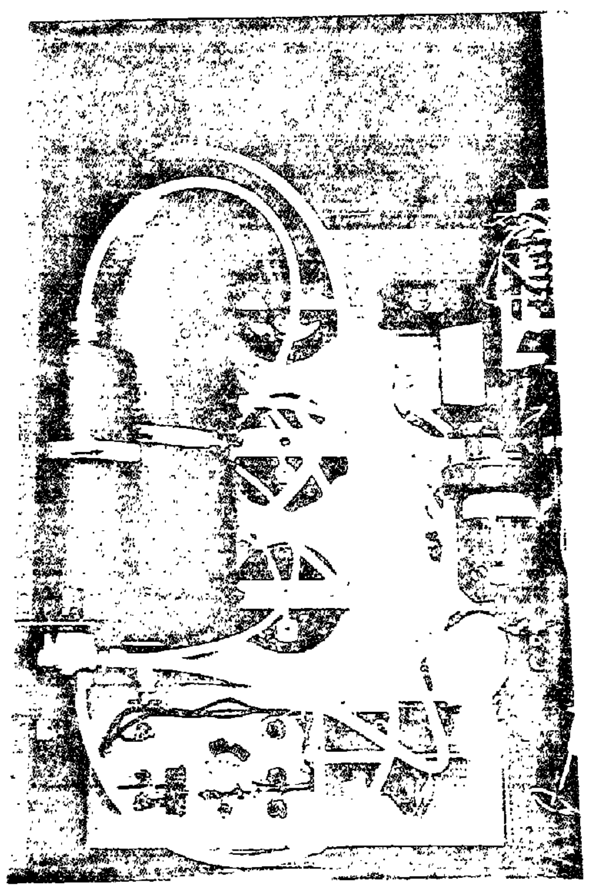


Figure 9 – Repeated Load Test Device, Inside View

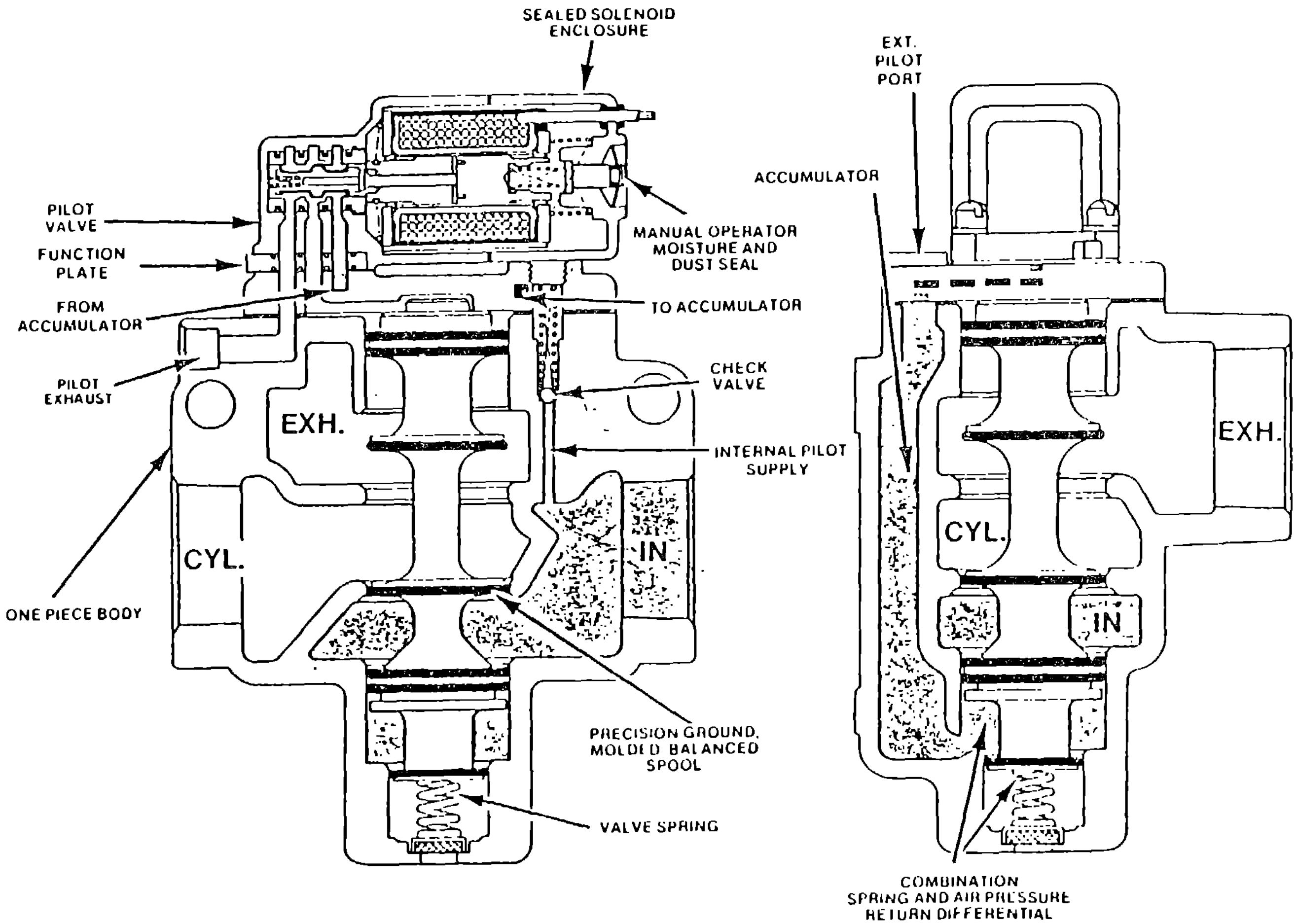


Figure 10 - Cross-Section View of MAC Valve

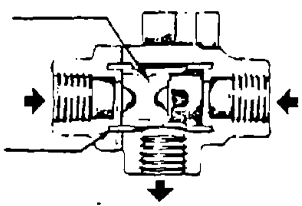


Figure 11 - Cross-Section of Shuttle Valve

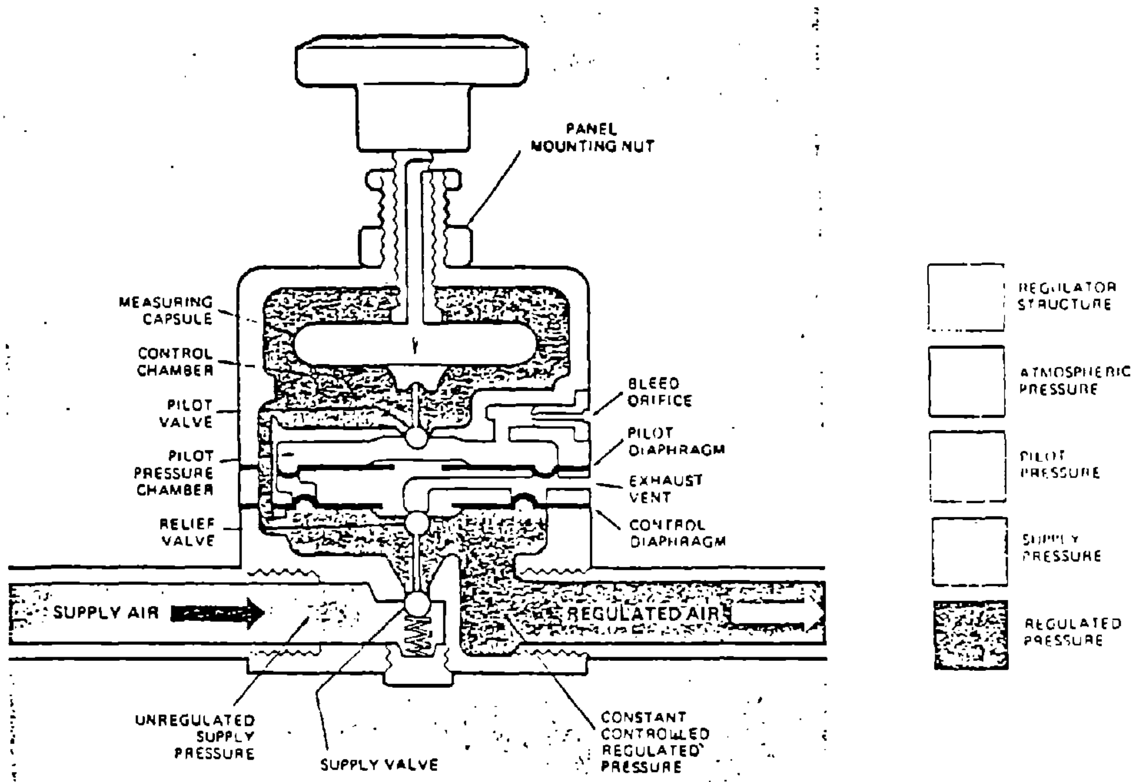
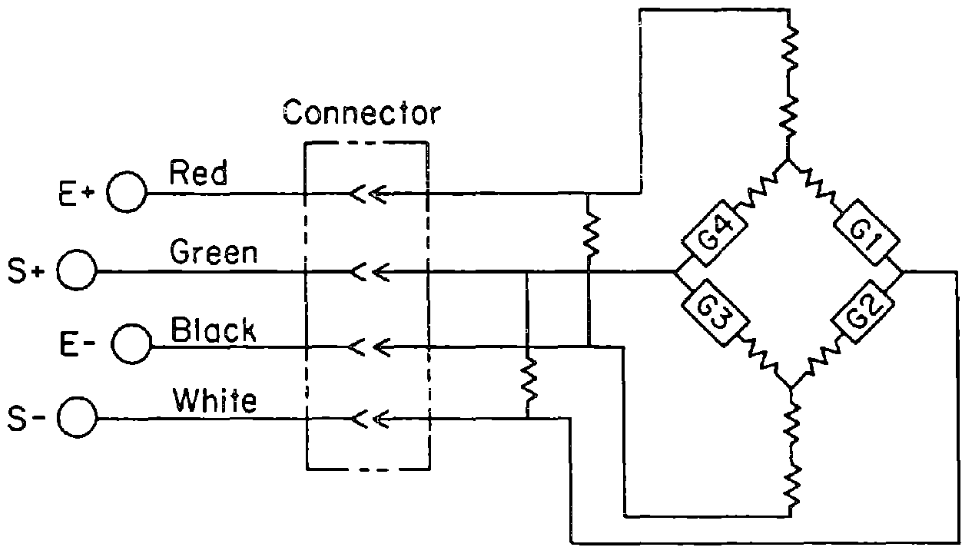


Figure 12 - Cross-Section of Bellofram Air Regulator



LEGEND

Strain Gages: G1, G2, G3, G4

Excitation: E+, E-

Signal: S+, S-

Figure 13 – Load Cell Schematic Diagram

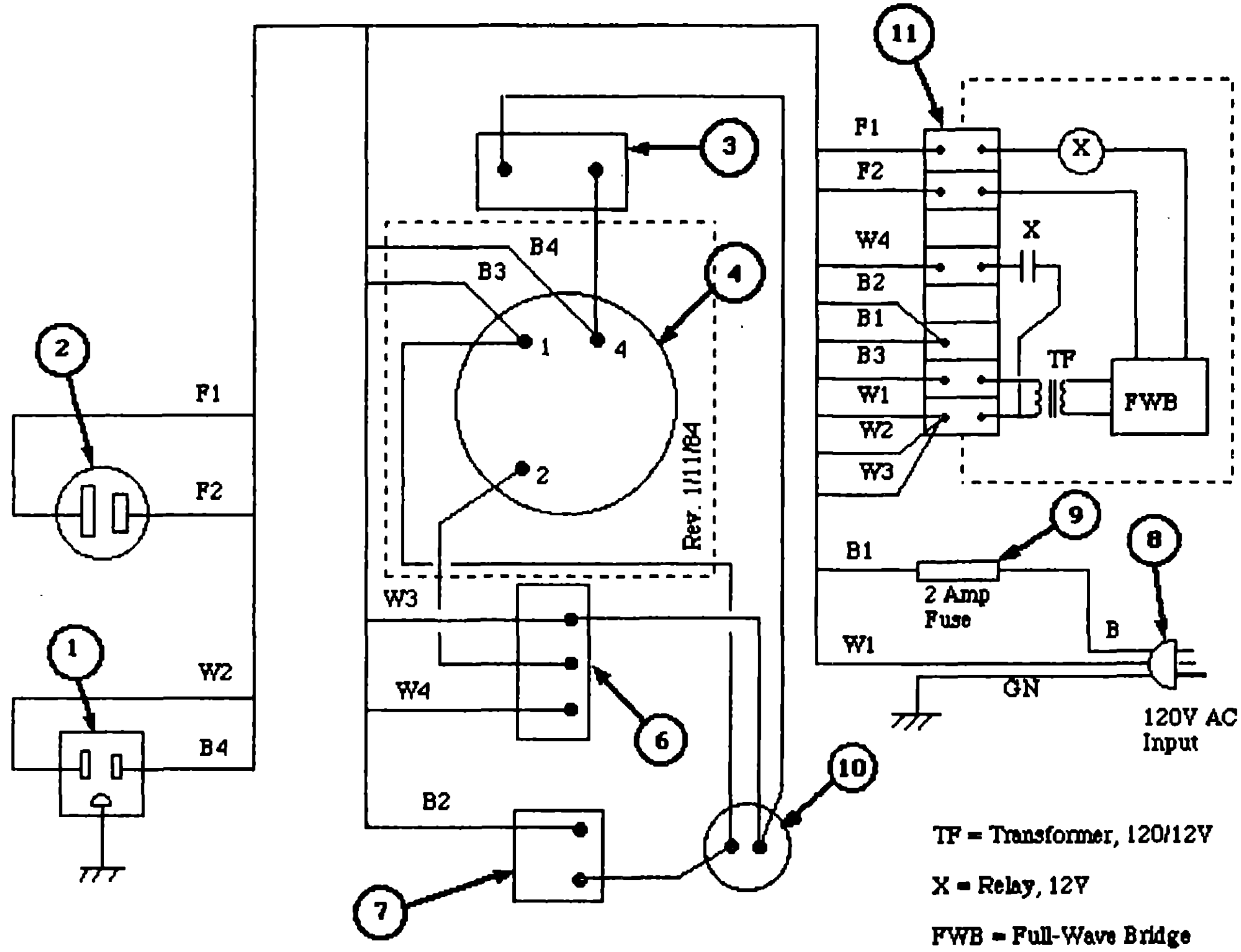


Figure 14 - Repeated Load Device, Wiring Diagram (See Table 1 for Key to Components)

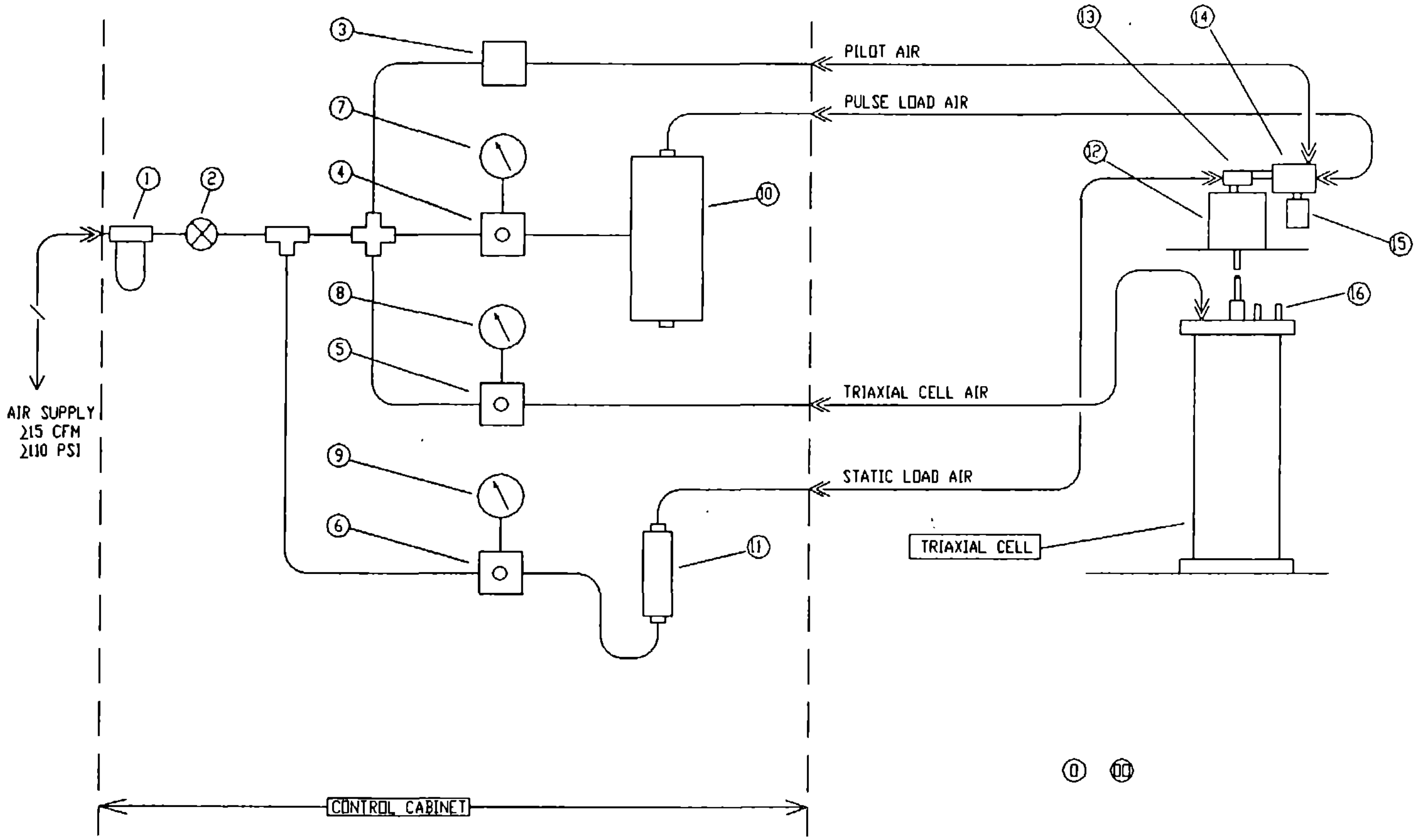
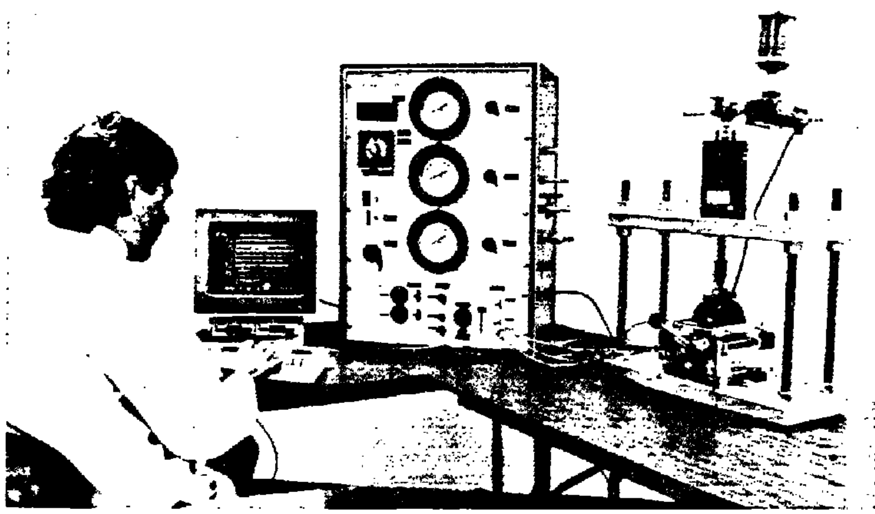
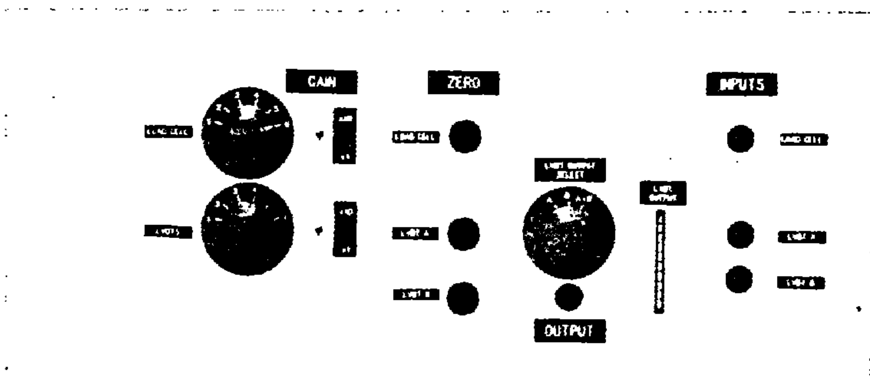


Figure 15 – Repeated Load Test Device, Pneumatic System Diagram (See Table 2 for Key to Components)



a) Overall View of Diametral Set-up



b) Close-up of Signal Conditioning Unit

Figure 16 – Layout of Front Panel Controls

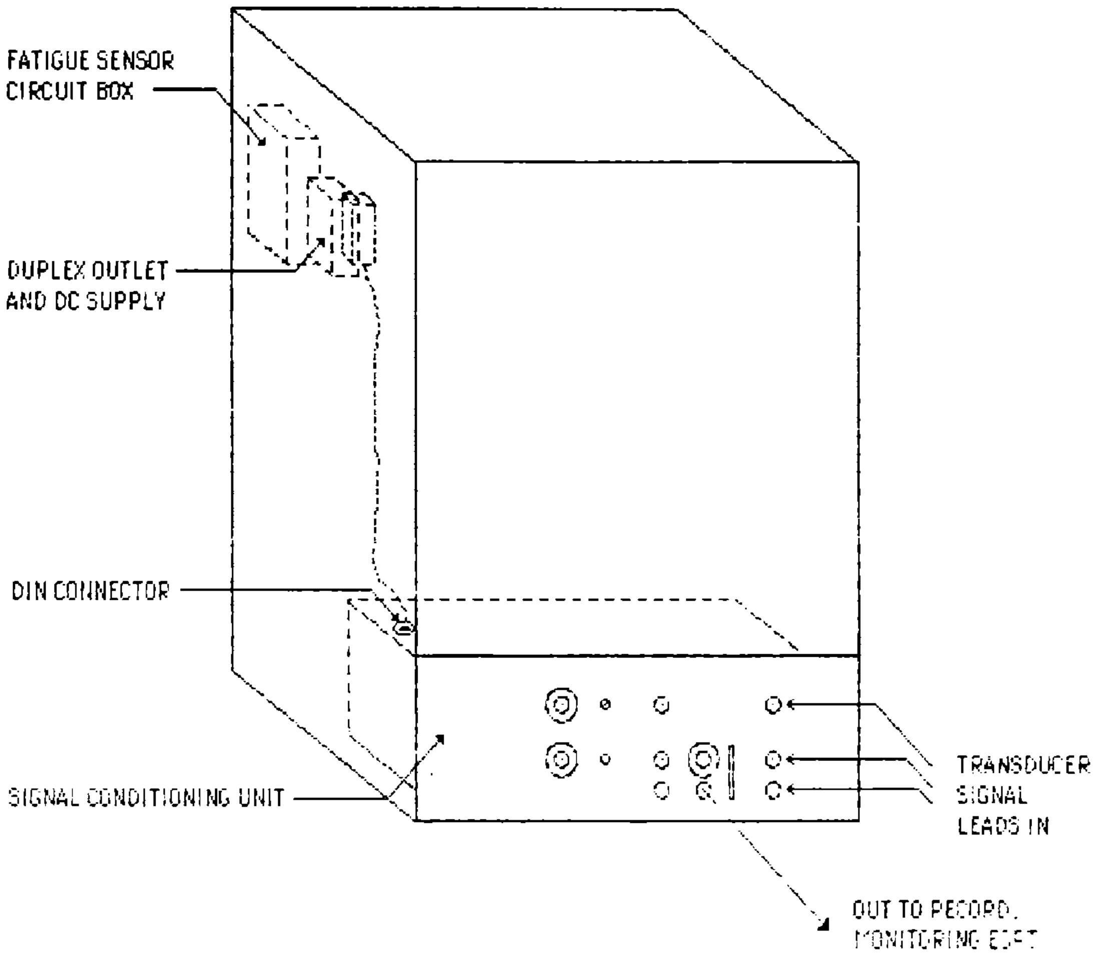


Figure 17. Repeated Load Control Cabinet Showing Location of Sig Con Unit and Power Supply.

Table 1. Electrical Parts List

Key	Description	Manufacturer	Manufacturer's Part Number	Source
1	120-V AC Receptacle	Eagle	34207	1
2	2-Prong Socket	Beau	S-3302-FP	1
3	120-V AC Counter	Eagle	PCC-7	2
4	Duration and Interval Timer	Eagle	CA3030A603	2
6	2PDT Toggle Switch	Arrow-Hart	82600	1
7	SPST Toggle Switch	Arrow-Hart	82608	1
8	120-V AC Line Cord	Belden	17238B	1
9	Fuseholder	Littlefuse	342012	1
10	Pilot Light	Leecraft	32R211T (Similar)	1

Source	Name	Location	Phone
1	Newark Electronics	Portland, OR	(503) 257-0741
2	Branon Instruments	Portland, OR	(503) 283-2555

Table 2. Pneumatic System Parts List

Key	Description	Manufacturer	Manufacturer's Part Number	Source
1	Air Filter	Watts	604-3	2
2	Shut-off Valve	Whitey	B-1-VS-6	3
3	Regulator, Fixed	Arrow	1612 FK	5
4	Regulator, 0-125 psi	Bellofram	221-960-071	1
5	Regulator, 0-25 psi	Bellofram	221-960-011	1
6	Regulator, 0-60 psi	Bellofram	221-960-017	1
7	Gauge, 0-160 psi	Marsh	EFLSH-1242	1
8	Gauge, 0-30 psi	Marsh	EFLSH-1246	1
9	Gauge, 0-60 psi	Marsh	EFLSH-1252	1
10	Reservoir	Bendix	225000	4
11	Reservoir	Bendix	224220	4
12	Bellofram Actuator	Bellofram	SS9FSMUM	1
13	Shuttle Valve	Bendix	278614	4
14	Air Valve	MAC	56C-33-111CS	2
15	Silencer	Alwitco	MO-5	2
16	Pressure Release	Whitey	B-4CPA-2-3-DC	3

Source	Name	Location	Phone
1	Branom Instrument Co.	Portland, OR	(503) 283-2555
2	Buchanan Fluid Systems	Portland, OR	(503) 226-7868
3	Portland Valve & Fitting	Portland, OR	(503) 234-0866
4	Fluid Air Components	Portland, OR	(503) 244-9393
5	Air-Oil Products	Portland, OR	(503) 234-0866

Resilient Modulus Diametral Test Theory

APPENDIX A: RESILIENT MODULUS DIAMETRAL TEST THEORY

The resilient modulus diametral test theory is summarized in this appendix.

Hadley, et.al. (1970) developed equations to evaluate the material properties (i.e. elastic modulus, E, and Poisson's ratio, υ) of diametrically loaded cylindrical specimens. These equations have subsequently been applied to the elastic response of specimens subjected to repeated loads. The following assumptions are made in the development of the equations:

1. The material is elastic, thus Hooke's Law is valid.
2. The material is homogeneous and isotropic, allowing the use of a single value for the modulus and Poisson's ratio.
3. Plane-stress conditions exist and, therefore, the problem can be modeled as two-dimensional.
4. The x- and y- axes are principal planes. This assumption follows from the stress analysis, in which $\tau_{r\theta} = 0$ along these axes.

The stress analysis of a perfectly elastic, homogeneous, isotropic, and weightless, circular element with a diametrically applied compressive strip was performed by Hondros (1959), using a Fourier series. The definition sketch for the theoretical development appear in Figure A.1. Notation is given in Appendix B. Figure A.2 illustrates the unit stress distributions that result from Hondros' analysis for a 4-inch diameter disk with a 0.5-inch loading strip width.

Any of the four stress distributions ($\sigma_{\theta x}$, σ_{rx}, $\sigma_{\theta y}$, σ_{ry}) can be written

$$\sigma = \pm(2P/\pi at)[f(r,R,\alpha)] \qquad (A.1)$$

If the width of the loading strip is fixed (a = 0.5-inch for this case), Equation

APPENDIX A: RESILIENT MODULUS DIAMETRAL TEST THEORY

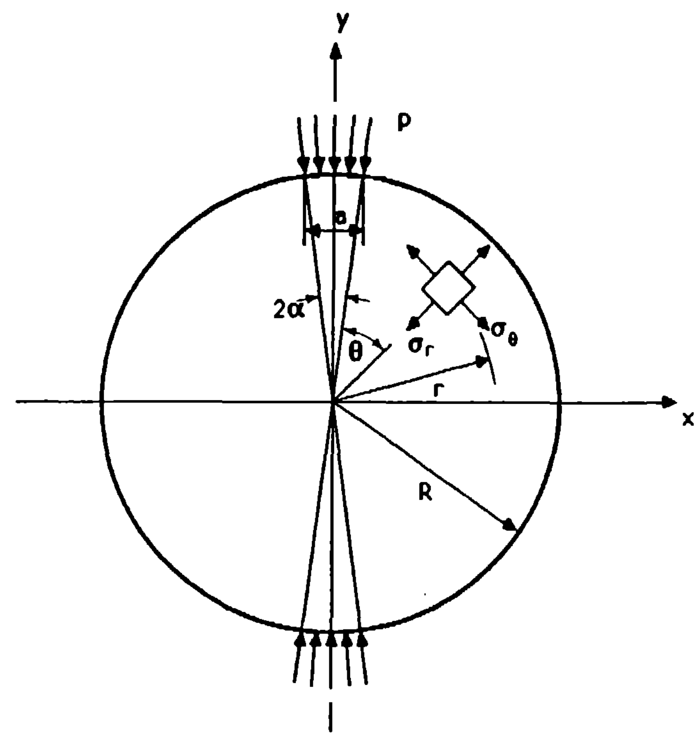
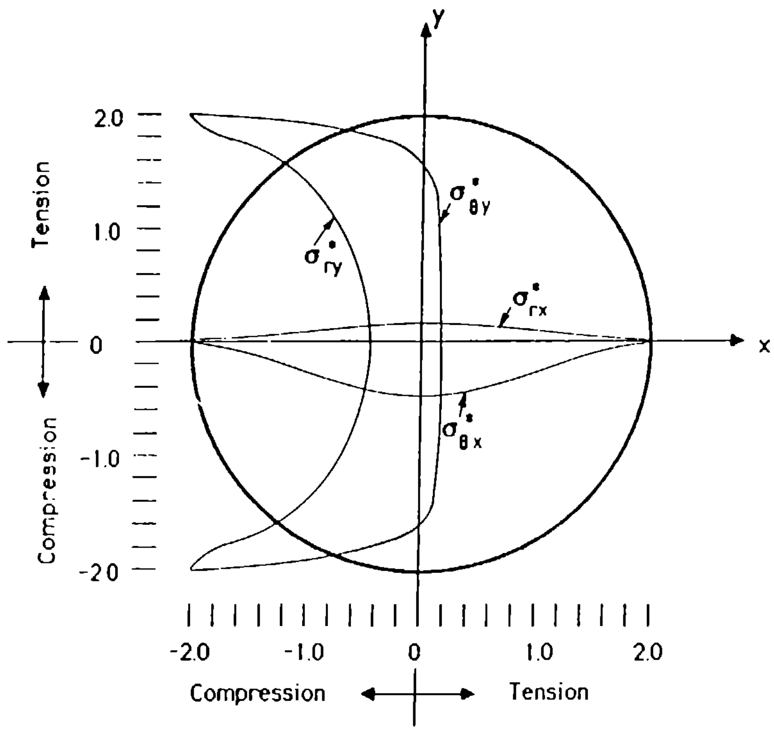


FIGURE A.1. Definition Sketch of a Diametrally Loaded Circular Element.



σ^* = unit stress = $(2/\pi a)(F[r,R,\alpha])$

FIGURE A.2 Unit Stress Distribution Along the Vertical and Horizontal Axes for a Diametrally Loaded 4- inch Diameter Circular Element with a 0.5-inch Strip Load (after Hondros, 1959).

APPENDIX A: RESILIENT MODULUS DIAMETRAL TEST THEORY

A.1 can be rewritten

$$\sigma = (P/t)\{2/\pi a\,[\,f(r,R,\alpha)]\} = (P/t)\,\sigma^* \qquad \text{(A.2)}$$

where σ^* = unit stress

For any differential element along the x-axis, Hooke's Law is expressed as

$$\varepsilon_x = 1/E\,(\sigma_{rx} - \upsilon\,\sigma_{\theta x}) \qquad \text{(A.3)}$$

In terms of the total horizontal deflection,

$$H = \int_D \varepsilon_x = \int_D 1/E\,(\sigma_{rx} - \upsilon\,\sigma_{\theta x})$$

from which

$$H = 1/E\,\left(\int_D \sigma_{rx} - \upsilon \int_D \sigma_{\theta x}\right) \qquad \text{(A.4)}$$

where $\int_D$ = integral over the diameter

Next, the previously defined expression for unit stress is substituted into Equation A.4. Solving for E,

$$E = P/Ht\,\left(\int_D \sigma_{rx}^* - \upsilon \int_D \sigma_{\theta x}^*\right) \qquad \text{(A.5)}$$

Performing the same operations along the y-axis results in

$$E = P/Vt\,\left(\int_D \sigma_{ry}^* - \upsilon \int_D \sigma_{\theta y}^*\right) \qquad \text{(A.6)}$$

Equations A.5 and A.6 can now be equated to solve for υ,

$$\upsilon = \frac{\int_D \sigma_{ry}^* - \frac{V}{H}\int_D \sigma_{rx}^*}{\int_D \sigma_{\theta y}^* - \frac{V}{H}\int_D \sigma_{\theta x}^*} \qquad \text{(A.7)}$$

An equation for tensile strain can now be obtained by first expressing Hooke's Law for the deflection over a finite length at the center of the specimen,

$$H_l = \int_l \epsilon_x = \int_l 1/E\,(\sigma_{rx} - \upsilon\,\sigma_{\theta x}) \qquad \text{(A.8)}$$

where $\int_l$ = integral over a finite length at the center

By the definition of strain,

$$\epsilon_{xl} = H_l/l = 1/El\,(\int_l \sigma_{rx} - \upsilon \int_l \sigma_{\theta x}) \qquad \text{(A.9)}$$

Expressing Equation A.9 in terms of unit stress and solving for E,

$$E = (P/tl\epsilon_{xl})\,(\int_l \sigma_{rx}^{*} - \upsilon \int_l \sigma_{\theta x}^{*}) \qquad \text{(A.10)}$$

The modulus, E, has now been expressed in terms of the total horizontal deflection (Equation 2.5A) and in terms of tensile strain (Equation 2.10A). Equating these two expressions results in

$$\epsilon_{xl} = \frac{H}{l}\left[\frac{\int_l \sigma_{rx}^{*} - \upsilon \int_l \sigma_{\theta x}^{*}}{\int_D \sigma_{rx}^{*} - \upsilon \int_D \sigma_{\theta x}^{*}}\right] \qquad \text{(A.11)}$$

The Mean Value Theorem can now be applied to the expressions $\int_l \sigma_{rx}^{*}/l$ and $\int_l \sigma_{\theta x}^{*}/l$ to arrive at

$$\epsilon_{xl} = H\left[\frac{\sigma_{rx}^{*}\big|_{r=0} - \upsilon\,\sigma_{\theta x}^{*}\big|_{r=0}}{\int_D \sigma_{rx}^{*} - \upsilon \int_D \sigma_{\theta x}^{*}}\right] \qquad \text{(A.12)}$$

where $\big|_{r=0}$ indicates "evaluated at r = 0"

The expressions for Poisson's ratio, resilient modulus (M_r = E), and tensile strain at the center ($\epsilon_t = \epsilon_{xl}$) can be solved by numerically integrating the unit stress over the diameter of the specimen and solving the unit stress at the origin:

$$\upsilon = \frac{-3.59 - 0.27\,(V/H)}{-0.063 + (V/H)} \qquad \text{(A.13)}$$

APPENDIX A: RESILIENT MODULUS DIAMETRAL TEST THEORY

$$M_r = (P/Ht)\,(0.27 + \upsilon) \qquad \text{(A.14)}$$

$$\epsilon_t = \left[\frac{0.16 + 0.48\,\upsilon}{0.27 + \upsilon}\right] H \qquad \text{(A.15)}$$

Assuming tha Poisson's ratio is equal to 0.35 results in the commonly used equations:

$$M_r = 0.62\,(P/Ht) \qquad \text{(A.16)}$$

and

$$\epsilon_t = 0.52\,H \qquad \text{(A.17)}$$

NOTATION

a = width of loading strip

E = elastic modulus

H = total recoverable horizontal deflection

H(int) = horizontal deflection of interior node of finite element model

H_l = horizontal deflection over the length l in the center of the test specimen

IRM_r = index of retained resilient modulus

l = length in the center of the specimen over which the strain is measured

M_r = resilient modulus

n, K_1 = regression coefficients

P = repeated load

r = radial distance from the origin of the test specimen

R = radius of the test specimen

R^2 = coefficient of correlation

t = thickness of test specimen

TSR = tensile strength ratio

V = total recoverable vertical deflection

2α = angle at origin subtended by the width of the loading section

ϵ_t = tensile strain in the center of the specimen

ν = Poisson's ratio

μ = microstrain

σ_{rx} = tangential stress along the horizontal axis

$\sigma_{\theta x}$ = radial stress along the horizontal axis

σ_{ry} = tangential stress along the vertical axis

$\sigma_{\theta y}$ = radial stress along the vertical axis

σ_{rx}^* = tangential unit stress along the horizontal axis

$\sigma_{\theta x}^*$ = radial unit stress along the horizontal axis

σ_{ry}^* = tangential unit stress along the vertical axis

$\sigma_{\theta y}^*$ = radial unit stress along the vertical axis

Resilient Modulus Calculations Based on Repeated Load Triaxial Test Results

APPENDIX B

Resilient Modulus Calculations
Based on Repeated Load Triaxial Test Results

This appendix presents the equations used to calculate stresses, strain and resilient modulus from the results of triaxial tests. The inputs required are the dynamic load applied to the sample and the corresponding sample elastic deformation. In the case of the triaxial test, the confining pressure applied to the sample must also be included in the calculations.

The resilient modulus M_R is defined as the ratio of the repeated axial deviator stress, σ_d, to the recoverable axial strain, ϵ_a.

$$M_R = \frac{\sigma_d}{\epsilon_a} \qquad \text{E-1}$$

Deviator stress is calculated from the area of the specimen and the load cell output displayed on the recording equipment. The deviator stress, σ_d, is then expressed as follows:

$$\sigma_d = \frac{P}{A} \qquad \text{E-2}$$

where:

P = repeated load, lbs

A = cross-sectional area of sample, in^2

σ_d = deviator stress, psi

The cross-sectional area is usually assumed to remain constant. Since damaging stresses are avoided during a dynamic triaxial test, this assumption is reasonable.

Specimen axial strain is calculated from the LVDT output signals. The elastic strain, ϵ_a, is determined by dividing axial displacement by the gauge length. For the internal LVDT clamps the gauge length is 4-in; for the external LVDT clamp the gauge length is equal to the length of the sample.

Appendix D Questions for Resilient Modulus Workshop

QUESTIONS FOR RESILIENT MODULUS WORKSHOP

1. How many load repetitions are necessary to remove the softening effect of a freeze-thaw cycle(s)?

2. How many repetitions of load are typically applied to a freeze-thaw softened material (subgrade) before defining its resilient modulus?

3. What is the appropriate stress level at which we should be defining the "reference" resilient modulus?

4. Should compaction prestress be accounted for in selecting the appropriate stress load for resilient modulus of mon-linear materials?

5. Do you think compaction prestresses are removed by load repetitions or traffic?

6. How many data points (stress combinations) are necessary for an acceptable determination of the K_1, K_2 parametrics?

7. Is it necessary to lubricate the end plates in triaxial repeated load tests? Is it mainly done in practice?

8. Do you think the same amount (numbers) of conditioning repetitions are needed for cohesionless soils as might be required for cohesive?

9. Do you think you need the same number of conditioning repetitions when you use internal (clamp on) LVDT's as you do if you use external LVDT's to measure subgrade materials deformations?

10. How many repetitions do you think you should apply in order to define the plastic deformation behavior of a cohensive subgrade soil? (at a given σ_3, σ_d combination)

11. Do you think it is reasonable to combine the repeated load test for plastic deformations (rutting) with the resilient modulus test?

Mr. Jim Hardcastle from University of Idaho.

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1. Please comment on the effect of resilient modulus of soils?

   a. number of layers of compaction
   b. curing time after compaction
   c. membranae penetration
   d. number of tamps (for constant density)

2. What to do about segregation of granular samples during specium construction? (especially with coarse samples)

Mr. D. Elton from Auburn University

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1. Marshall Thompson - Because of the minor effects of some compositional properties on the M_R of soils, it would appear that the M_R of soils may not be a very useful property to measure. Would the total modulus be more useful? If only resilient deformations are used to calculate a modulus, does the modulus have anything to do with subgrade deformations?

2. Gilbert Baladi - Of what use is the resilient modulus? There was no conclusion to the theory presented.

3. Anyone testing asphalt mixtures especially in the diametric mode. -Often a load such as 500 lbf needed to produce modulus at low temperatures, such as 41^{O}F, with quickly fail a specimen at a high temperature, such as 104^{O}F. The specimen does not respond like a confined pavement. So what loads or stresses do I use? Maybe Bud Furble discussed this, but I missed his talk.

Mr. Kevin D. Stuart from Federal Highway Administration

How can we use M_R test for asphalt concrete mix design purposes?

Mr. Ok-Kee Kim from Caltrans

---

1. What is the basis for assuming that there is a relationship between M_R and layer coefficients? Is there any evidence to support such an assumption?

2. Relative to retained modulus (test for moisture susceptibility) what is the basis for the sample conditioning process i.e. freezing and then soaking at 140^{O}F? Does this scheme represent field conditions? Is there any data to support this process?

Mr. Joe Head from West Virginia University

It seems like everyone uses a different magnitude of load or strain when testing asphalt concrete in indirect tension. Should the load or strain be standardized and approximately what magnitude of values should we be considering?

Mr. Bill Maupin from Virginia Transportation Research Council

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In your handout on the indirect tensile test, there are two Poisson's Ratio calculated; one is the resilient Poisson's Ratio and the other the Total Poisson's Ratio can you explain the difference between these two ratios? Also, can you explain why the total poisson's ratio exceeds 0.5 in the results after specimen rotation?

Mr. Vincent Janoo from USACRREL

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1. Since M_R of soils is also stain dependent why not to use strain based relationships to increase the statistical fit of M_R and reduce the scatter of the experimental data?

2. Do you think that it is possible to compare soil modulus obtained with M_R test and resonant column for the same level of strain?

3. Do you think that strain distribution on the soil sample is uniform and it is more convenient to use external LVDT's instead of internal LVDT's?

4. Why not to use the State of stresses on average pavements layer to test the sample in the laboratory instead of the set σ_d and σ_3 of T274 which are not realistic?

5. Do you think the load duration and local cycle affect the soils M_R?

Mr. German Claros from UT Austin

Do you see a need to incorporate the shear strength of the pavement layer materials in the mechanistic design procedure?

Mr. V. Jamoo from USACRREL

Apart from M_R = 1500 CBR, has any work been done to get better correlation between CBR & M_R? I feel that 1500 CBR is too conservative. One of the problems we have is checking our back calculated value with lab CBR tests. Unfortunately, resilient modulus testing is still not common place.

Mr. Margot Yapp from ARE Inc

---

1. Are the lower M_R values obtained from testing, on specimens prepared by kneading methods, more consistent than those obtained from specimens prepared by static methods?

2. Will any of the following items have a significant effect on M_R values of subgrade soils:

   a. changes in duration of load applications (cohesive and granular test specimens)

   b. rearrangement of the sequence in which stresses are imposed on test specimens (cohesive and granular)

3. What do you feel are the major areas of AASHTO T-274 test which most need re-evaluation and possible revision?

Dave Seim of New Yorks DOT

When you run a triaxial load test on cohesive materials what are the importance of hold periods between the load pulses? Will there be a big difference in M_R if you run the test with a sinusoidal loading curve? The effect on permanent deformation?

Krister Yorevik

Has anyone compared test results using a frame containing three LVDT's with two LVDTs (for axial deformation in soil sample)? It seems to me that if one LVDT is not good enough, then two are not a lot better and that a minimum of three LVDTs are required to accurately determine the correct values for strain calculations.

Jim Blacklock from University of Arkansas, Little Rock, Arkansas

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Are Triaxial resilient modulus values dependent upon the shape of the waveform of the applied load (deviator load)?

If so, how can M_R values be used reliably in the AASHTO design equations?

Keith Johnston of ODOT Materials and Research Section

Poisson's ratio has (and still is) being back calculated from the results obtained in an indirect tension test (horizonal and vertical deformations). Values for an AC at 77^{O}F reported at this workshop based on the results of an indirect tension test are as follows:

resilient M $\approx$ .15 (are of 2 values)
total M $\approx$ .73 (are of 2 values)

Yoder and Witczeak (see attached) suggest:

M .35 - .40
AC at 77^{O}F

Would members of the panel please comment on (1) the use of indirect tension test results to back calculate M_R and (2) the discrepenancy in the results as noted above.

Vincent Janao/Ted Vinson of USACRREL/OSU

I would like you to address the issue of Poisson's ratio. Should it be measured? Should it be assumed? Do we have the right test procedure, equations to evaluation it accurately? Also, comment on the negative values obtained frequently.

Louay Mohammad of L.S.U.

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1. What is the significance of external versus internal LVDT's and load cell's in AASHTO-274?

2. Should one adjust the compactive effort on the bottom layer to come up with a more uniform sample specimens.

3. For cement, lime, and fly ash treated samples is the AASHTO T-274 applicable in its present form?

4. What are the anticipated changes to the AASHTO T-274 and when would they likely appear in the standard?

5. Explain the details of kneading-type compaction in your session III paper?

6. Would a Rainhart Automatic Compactor with rotating base and mold and with the sheeps foot tamping foot be acceptable in AASHTO T-274?

Jim Nevels of Oklahoma Department of Transportation, Soils and Foundation Engineer

---

1. Why is the electric-hydro system preferred by the SHRRP staff over the pneumatic system?

2. Most equipments available are rather expensive. Is a home-made device considered acceptable for research work? for production work?

3. In selecting M_R equipment what are the key considerations?

   a. Electro-Hydro versus Electro-Pneumatic.
   b. Wave form considerations.
   c. Test versatility.

4. Is cost of the system really representative of test accuracy?

5. Can you get good results with less costly equipment?

Ali Selim of South Dakota State University.

What is the extent of establishing correlation values between M_R and CBR or R-values to verify or refine conversion formula given in AASHTO Guide?

Emery Marko of Washington County, Oregon.

---

1. When testing non-plastic sandy silt and low P.I. clayey silt soils, we have experienced lab compacted modulus values that are significantly lower than the back calculated values. Can you tell me why?

2. How do modulus test results from undisturbed samples compare with lab prepared samples on non-plastic, fine grained soils?

3. AASHTO T274 defines cohesive soils as being A-2-6, A-2-7, A-6, and A-7. Assumably everything else is non-cohesive. Do you feel this criteria is appropriate? Also, are separate criteria for cohesive and non-cohesive soils really necessary?

4. Why does AASHTO test at bulk stress levels far exceeding those found in typical highway situations?

Roger Miles of Oregon State Highway Division.

In larger samples the effects of gravity become a problem especially in fine grain non-clay soils. How can the test be performed without causing the water in the sample to flow to the bottom during the test?

Jim Blacklock of the University of Arkansas, Little Rock.

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1. In many States, rutting of asphalt pavement is a primary area of concern. Have there been any successful attempts to correlate M_R results to a mix's rutting potential?

2. What future do you see for creep testing (shell method) as a means of characterizing rutting potential?

John Adam of the Iowa Department of Transportation

---

1. We have experienced modulus test results which increase with increasing plasticity of the soil. Is this typical? With the "R"-value test, we typically got the opposite response, is there an explanation?

2. How does the method of compaction affect test results for various soil types (granular, fine grained plastic, fine grained non-plastic)?

3. Should the stress conditions for which moduli are selected be "standardized" for typical highway loading conditions?

4. How would you explain/interpret the attached resilient modulus results?

Jeff Gower of the Oregon Department of Transportation.

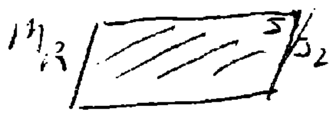
CRREL has developed some resilient modulus equasions for thawing sandy materials. The equasion is similar to those presented here and is in the form of $M_R = K_1 (F(\sigma))^{K_2}$

where K_1, K_2, are constants and $f(\sigma)$ can be either J_1 or J_2/T_{oct}

J1 = 1st stress invariant - text - octha hedral shew strength
J_2 = 2nd stress invariant - " " " " " " " " " " "

K_1 was found to be strongly dependent on the moisture tension in the soil. It was also found that if J_1 was used to describe the stress state, then M_R was laos a function of σ_1/σ_2 (fig 1).

However when J_2/T_{OCT} was used, then the σ_2/σ_3 ratios all collapsed into 1 line (fig 2)



J_1
Fig 1

J_2/T_{oct}
Fig 2

My Question is whether anyone of you looked at J_2/T_{oct}. J^2 and T_{oct} are not difficult to calculate as you still use σ_1, σ_2, and σ_3 in the calculations?

Vincent Jonoo of USACRREL

---

1. For field cores and/or variable diameter samples, do you agree with flexible or "floating" upper loading strip? This is an FHWA modification to your device as I understand it.

   My most sincere compliments to you for putting your device (or its design) into the public domain. A noble gesture, certainly one that helps all of us that have the opportunity to use the device.

Bill Whysaveh of Brown Engineering Testing, Inc.

Remembering that we write from left to right and subtract from right to left why do we load AASHTO resilient modulus samples from low values to higher values? Why not load samples with a random sequence of σ_d values and σ_3 values? Real traffic does not increase from low to high

suggested sequence:

old	new	or	new	or	new	
1 psi	2		4		1	
2 psi	1		2		4	
4 psi	4		8		2	
8 psi	10		1		10	
10psi	8		10		8	etc.

Jim Blacklock of the University of Arkansas, Little Rock.

What is the status of the ASTM equivalent to the AASHTO T-274?

Jim Nevels of Oklahoma Department of Transportation Soils and Foundations Engineer.

---

1. Who do we talk to, to lobby for changes (streamlining) of the M_R test procedures?

2. Does AASHTO plan to offer reference samples for either one or both of the resilient tests being discussed? (soils - asphalt concrete)

3. Your paper tends to suggest that a square or rectangular wave form is an adequate approximation to the haversine Waveform. Is this a correct interpretation of what you are saying?

   If so, what is the basis for this conclusion?

4. More generally, what characteristic make one waveform "better" than another?

Bill Wysauch of Brown Engineering Testing, Inc.

In your presentation on the influence of specimen preparation on M_R you only covered one method of granular specimen preparation. I am aware that different preparation methods (such as raining sand; static compaction; vibratory compaction, etc.) produce different structures and therefore different M_R. have you looked into other lab compaction method and how do they relate to the field? Also, how do other lab compaction methods affect the M_R of granular materials.

Vicent Janoo of USACRREL

Appendix E Workshop on MR Testing: Evaluation and Remaining Areas of Concern

WORKSHOP ON M_R TESTING
EVALUATION AND REMAINING AREAS OF CONCERN

Specific Comments and Questions Remaining

1. I cannot over emphasize the importance of getting efforts of AASHTO, ASTM, NCHRP and SHRP coordinated NOW, not after each completes their work. Also, it appears that "production" v.s. "research" must be considered. Remember, AASHTO involves both joint Task Force on Pavements and Subcommittee on Materials Technical Section. JTF for design guide, TS for standard.

2. Workshop provided excellent balance of viewpoints, theory, implementation, etc. Clearly one of the best workshops I ever attended. Q&A session was a great feature; highly informative in an informal atmosphere. I would really appreciate a follow-up workshop dedicated to exactly how the tests should be run. What I mean is the sort of "hands on, how-to" tips needed to get accurate, reliable results right from the start as opposed to spending 3-6 months trying to learn all the tricks. Perhaps this is where FHWA or the Asphalt Institute could step in.

3. Presentations by equipment manufacturers excellent. Being able to see all varieties of soil/AC testing equipment will be very useful in making future purchases. Topics that were addressed and need to be addressed by the appropriate committees are:

   AC: Reference standard (e.g. teflon, TIVAR, etc.) that can be used by all manufacturers, so (e.g.) 50 micro-inch @ .1 second load on zone machine = 50 micro-inches @ .1 second load on machine two.

   Load on versus load off ratio needs to be consistent between research programs.

   M_R needs to be defined for a specific strain range:

   (i.e.) adjust load to obtain 40-60 micro-inch strain instead of setting load and measuring resulting strain.

   Soils:

   Revamp test method -

   a. Preload (i.e. conditioning) for each Stress State - is it recess.

   b. Number of cycles - can they be reduced?

   c. Limit stress states.

   Need to define range of compaction methods for various in-situ conditions, i.e. condition for static or kneading.

   If its going to be generally used - make is production oriented!

4. This workshop needs to be held about a year from now to bridge the gap from research to full production use. This workshop should prove extremely beneficial to implementation of the AASHTO Pavement Design Guide.

5. Please provide more information on Poisson's Ratio for various modifiers, temperatures, etc.

6. Need more time to ask questions. Too little attention paid to sample preparation and equipment/sample configuration. The data collection aspect was very good. I never did get a handle on how much testing error is allowable before it significantly affects the designs. Good coverage on the full range of topics related to Resilient Modulus. Good effort to get all the experts in one place. Very valuable to get the manufacturer's equipment here. Unbelievable that this many resources could be assembled in one place regarding M_R. Every opportunity was here to get answers to our questions regarding modulus, if answers existed. Good job Jim Sorenson and Jim Huddleston!

7. We should have some kind of yearly conference supported by FHWA for highway agencies on asphaltic concrete pavement design since this seems to be a significant percentage of the monies spent on highways, especially one which the public feels the most. This is also evident, since much of this workshop involved not only resilient modulus, but other factors in pavement design. I especially liked the discussions on the State's experiences, theory versus practice.

8. If possible, it would have been nice to have some of the papers to read before the conference. Also the discussion/panel groups and question period was very good, but could have been longer.

9. Another workshop in 9 to 12 months as a follow-up would be beneficial. I'm sure there would be a lot of interest, particularly after participants have had time to absorb the information presented in this workshop. EXCELLENT WORKSHOP!

10. M_R symposium was very informative and enjoyable. GREAT JOB!

11. The more knowledgeable and experienced individuals from the presenters at this workshop should be brought together to develop the framework for revised specifications. This workshop should be viewed as the beginning of the development of the state-of-the-practice and should be followed-up by a more structured workshop that has the goal of achieving the needed background for development of needed specification(s) for appropriate test procedures.

12. The AASHTO T-274 should include a statistical analysis of the M_R data for soils which allows you to analyze the linear regression residuals and the elimination of outlyer through the USC test as cook distance on others because it is important to eliminate some bad data from the regression equation.

13. Excellent Program - Excellent Speakers - Excellent Content. Those involved in planning/arranging are to be commended. Would have appreciated more bias or additional session data on "production" type testing.

14. I would think it is imperative that ones in responsibility to stop arguing on the issues and decide on a direction and go forward. Let the research catch up and modify the test procedure accordingly.

15. I would like to obtain a copy of video: Resilient Modulus Lab Tests.

16. Workshops like this in each of the FHWA Regions is suggested. I suggest the State Design and Material personnel should be invited.

17. I did not appreciate the unprofessional manner in which Monosmith conducted himself. To offer opinions and comments is one thing, however, to give the impression of belittling and disregarding someones work and research is completely uncalled for.

    The conference was excellent. Good overall perspective. Please consider having an annual schedule for the conference so everyone can keep apprised of new developments.

18. FHWA (via Jim and Mark) has provided a major contribution to the profession via this conference. Why didn't AASHTO or ASTM fulfill their role and take the lead?

    I need 10 copies of the proceedings to cover my State highway agencies that didn't attend.

    Thanks again for all the courtesies extended to make our stay so enjoyable.

19. I believe the moisture equilibrium and suction should be considered in Resilient Modulus testing of subgrade. I have done research work on that with results available in TRB Paper #1121 (1987), entitled, "An Evaluation of Design High-Water Clearances for Pavements." I will be happy to give more details on that if needed. Please contact Mohamed K. Elfino.

20. Any follow-up should absolutely, positively include Marshall Thompson. He has done a tremendous amount of work in this area. He has in my opinion demonstrated a superior handle on the whole range of M_R testing, from sample preparation (moisture, compaction, etc.) to test procedures, to interpretation of results. He effectively bridges the gap from theory to practice.

21. Excellent conference, well organized. The laboratory demonstrations added greatly to the workshop. It was brought up by several speakers that both the AASHTO and ASTM test procedures are inadequate or require modification. Rewriting and/or modifying these procedures should be a top priority, with new test procedures approved ASAP.

22. A summary of the available equipment listing capabilities and approximate cost would be quite useful even though it may be out-dated rather quickly. Some future comparison of tests on some specimens by different equipment would be helpful. This could be done at a later date and the results sent to the conference attendees.

23. The equipment demos were the best part. Having everyone in one place to compare hardware and software was an excellent idea. This should be done again in 1-2 years.

24. I believe the industry is missing the boat by evaluating only axial strain when performing resilient modulus testing of soils. By measuring only the axial strain we have no knowledge of how consolidation, pore pressures, dilation or radial strains relate to the values we obtain for resilient modulus.

25. Obviously AASHTO T-274 is not adequate for the majority of the users. This theme was mentioned by a number of different presenters throughout the program.

26. From a technical viewpoint, it was good to see that our testing has been for the most part in line with other agencies with regard to AASHTO T-274. Standardizing these procedures as much as possible, with few restrictions in the existing T-274 will help out. I.E. new procedures for AASHTO T-274 will help everyone stay closer together instead of branching out on our own in modification. Was helpful seeing new and different equipment.

27. I plan on doing some prospecting for diamonds in pavement using back calculation. Every once in a while I find 170,000,000 psi surface modulus.

    M_R et. al. is a very useful tool. Successful use requires

    - rational attention to details.
    - learning what "non conformance" of various expected material properties means. Lack of fit is information.

28. How about a round robin sponsored by AMRL National Bureau of Standards? This could be for soils and asphalt mix.

29. There is a need for a "state-of-the-art" paper with practical recommendations on production testing and design.

30. As compared to others workshops, this was as good as any that I have attended and much better than most.

31. What is the exact definition of resilient modulus.

32. There is an obvious need to standardize test procedures, keeping the end user's application in mind. Excessive testing with unclear conclusions is of little help in actual design guidance.

33. The best thing about the conference is the variety of groups represented. The only thing missing is the deflection equipment people.

34. It seems premature at this point for FHWA to mandate resilient modulus as a design method without clearing up a lot of the remaining questions on testing and interpretation of results. There appears to be no data on comparative performance of pavements under different design methods. (None were presented.)

35. Overwhelmed by size, expertise and experience of this assembled group.

I was hoping to leave here with a better handle on where to proceed with pavement design based on M_R. Now I realize that this process is a lot more complex than I originally thought. I do not have time to become an expert in M_R, but I still need a reasonable procedure to follow where I can feel comfortable that I am getting representative results.

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