APPLICATION OF RESILIENT MODULUS TEST EQUIPMENT AND PROCEDURES FOR SUBGRADE SOILS

Part 1

MATERIALS TESTING

FINAL REPORT

HP&R Study: 083-5158

by

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16. Abstract
This report is the first part of a two-part series. It describes the techniques involved in, and the results from, resilient modulus testing of subgrade soils that are typically found in Oregon. Two methods of testing were investigated: the triaxial and diametral repeated load procedures. Subgrade soils obtained from two projects were tested. One project was a new alignment construction project in the Willamette Valley (Salem Parkway) for which there were two distinct subgrade soils (AASHTO classifications A-7-6 and A-4), the other was an overlay project in Central Oregon with a pumiceous subgrade soil (AASHTO classification A-1-b). All other materials occurring in each pavement were tested at their in situ compositions, such that sufficient resilient modulus data was obtained for analyses and designs to be accomplished for each project.

It was found that the diametral testing procedure was adequate for use with cohesive soils, typical of those occurring in the Willamette Valley, but it is not recommended for use with the noncohesive volcanic soils occurring in Central Oregon. For such soils the triaxial testing mode is recommended. The major advantage of the diametral test for treated materials is its simplicity compared to the triaxial test. However, the necessity to consider the effects of confining pressure for untreated soils diminishes this advantage, and with cohesionless soils the test is no simpler than the triaxial test, which is preferable for modeling the in situ stress regime.

The second part of this report (Part 2) presents procedures for analysis and design of flexible pavements, utilizing the results of the materials testing reported in Part 1. The two projects investigated in this study were used to show how the current procedures used for design of new pavements and overlays in Oregon, can be supplemented by analytically based procedures.

17. Key Words
Resilient Modulus Test Equipment, Subgrade Soils, Repeated Load Triaxial Test, Repeated Load Diametral Test

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ABSTRACT

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DISCLAIMER

1.0 INTRODUCTION

1.1 Problem Statement

Highways in Oregon, as well as in other states, are constructed using a wide variety of subgrade materials, and pavements designed using standard procedures, such as the Hveem or CBR-based methods, often do not perform satisfactorily. To attempt to more accurately predict pavement performance, analytical procedures based on multilayer elastic theory, in conjunction with suitable failure criteria, can be employed. This approach requires a knowledge of the mechanical properties of each pavement component under repeated load test conditions, typically the dynamic Young's Modulus (Resilient Modulus, M_R) and Poisson's Ratio (ν).

1.2 Objectives

The specific objectives of this study were:

1) To evaluate the use of the diametral and resilient modulus test equipment and procedures for subgrade soils.
2) To recommend procedures for routine use of the diametral test for soils evaluation and pavement design.
3) To recommend procedures for implementation of mechanistic analysis and design methods by Oregon State Highways Division.

1.3 Scope

This report presents the results of a study to examine the use of two repeated load testing procedures, the diametral and triaxial devices. A variety of soil types and treated materials from two projects were tested, such that recommendations for the use of the appropriate testing technique could be made, and so that the results could be used to demonstrate the implementation of analytical procedures.

This report is divided into two parts. This part (Part 1), describes the materials testing procedures and results, and presents recommendations for use of the triaxial and diametral methods of repeated load testing. Part 2 presents procedures for analysis and design of flexible pavements, utilizing the results of the materials testing reported in Part 1.

2.0 EXPERIMENTAL PROGRAM

2.1 Project Locations and Descriptions

Two project sites were selected for this study. The first site is a new alignment construction project in the Willamette Valley, which will be referred to as the Salem Parkway project, and the second is an overlay design project east of the Cascades which will be referred to as the U.S.-97 project. The precise location of both the projects is shown in Figure 2.1. Cross sections of both pavement structure sites are shown in Figure 2.2.

2.2 Testing Program

The test program undertaken in the study for the base and subgrade materials is illustrated in Figure 2.3. The program consists of five major phases as follows:

1) Standard indicator tests,
2) Test specimen preparation,
3) R-value tests,
4) Repeated load diametral modulus tests, and
5) Repeated load triaxial modulus tests.

For the bound materials, i.e., asphalt concrete, cement-treated base (CTB), and cement-modified soil (CMS), the test program consisted of a series of resilient modulus tests using the repeated load diametral test apparatus.

The standard indicator tests were performed for basic identification of base and subgrade materials, as shown in Table 2.1. The R-values data are used in the design procedure of flexible pavements by Oregon Department of Transportation as described in Part 2 of this report. The resilient modulus (M_R) of the asphalt concrete cores, base and subgrade materials were deter-

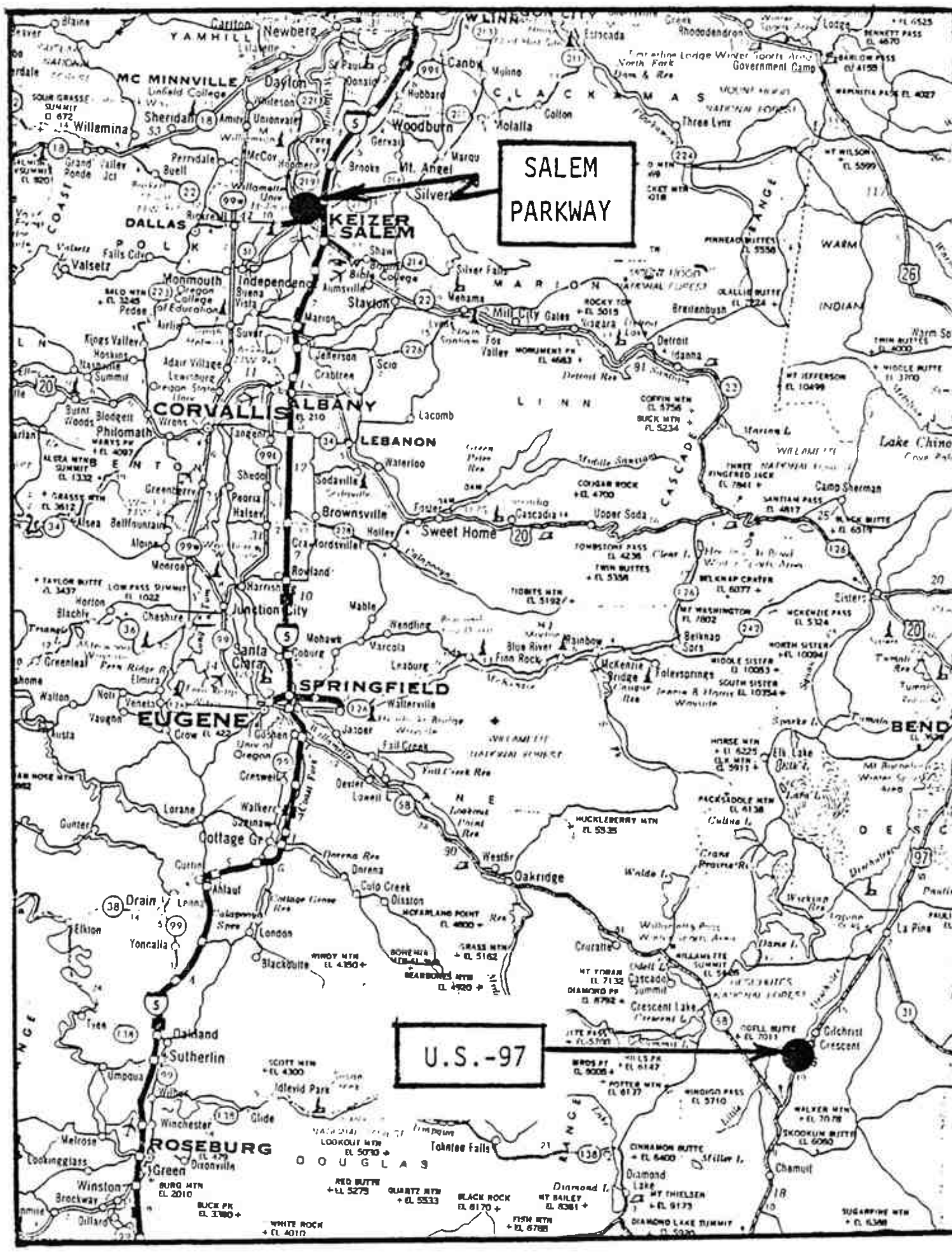


Figure 2.1 - Location Map - Salem Parkway Project and US-97 Projects

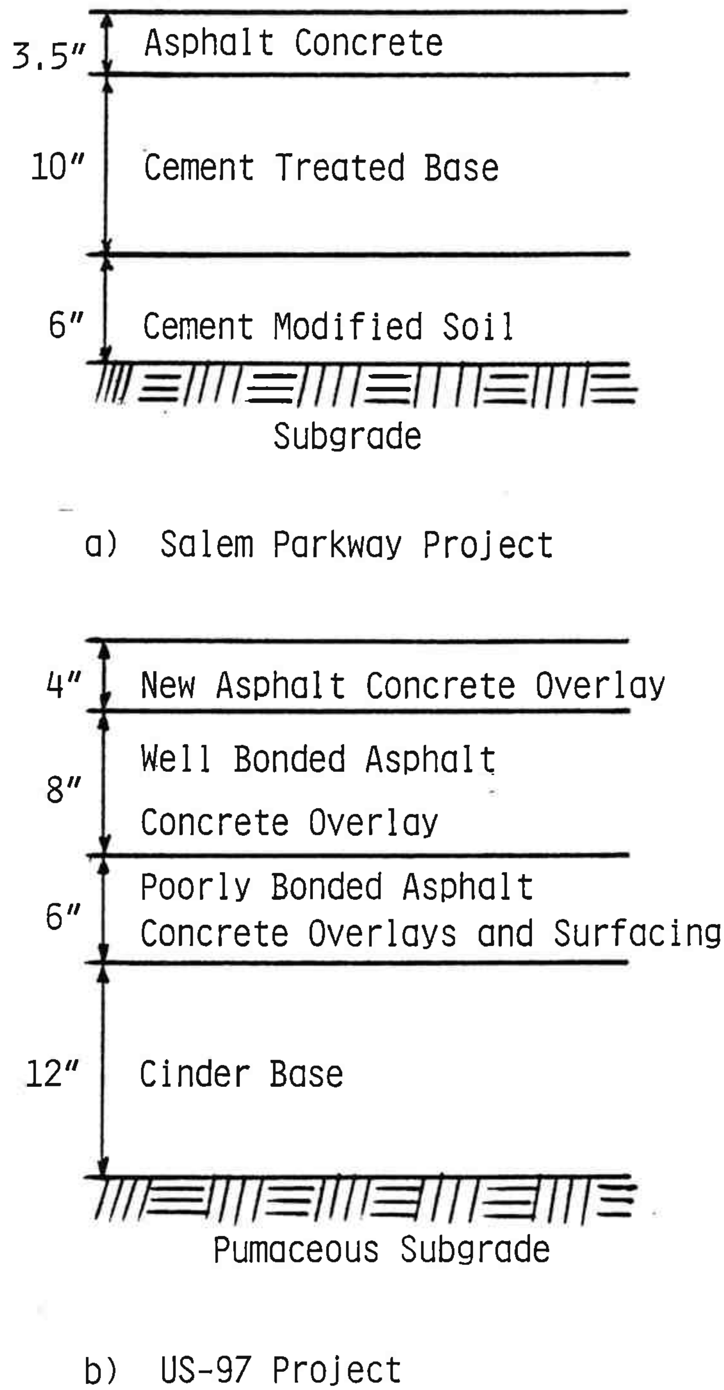


Figure 2.2 - Cross Sections of Pavement Structures for Both Projects

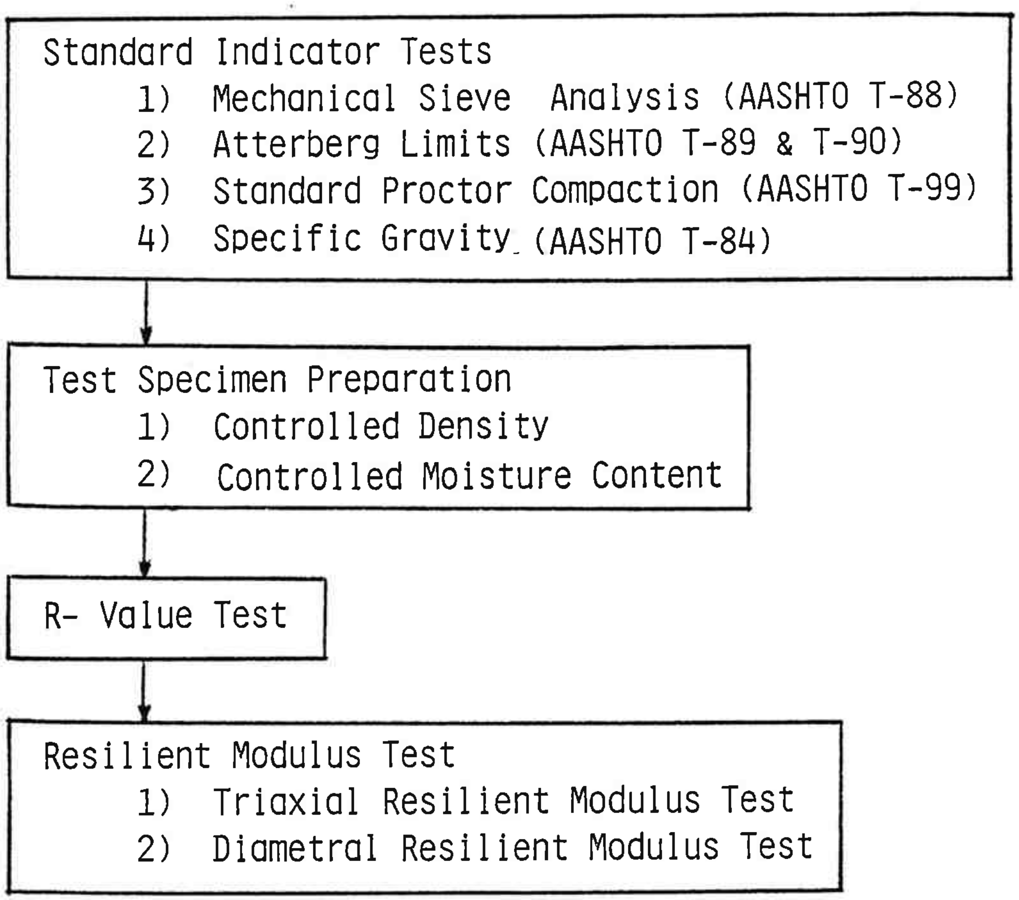


Figure 2.3 - Flow chart of test program for base and subgrade materials.

Table 2.1

Material Properties, Standard Indicator Test

	% Passing			
	Salem - New Parkway		U.S.-97	
	Subgrade			
Particle Size	1	2	Subgrade	Base
38.1 mm (1 - 1-1/2")				100
25.4 mm (1")			100	97.6
19 mm (3/4")			99.8	94.3
12.7 mm (1/2")			98.2	88.3
9.5 mm (3/8")			95.8	83.0
6.4 mm (1/4")			91.1	74.6
4.75 mm (No. 4)	100	100	87.6	69.7
2.00 mm (No. 10)	99.9	99.9	66.1	56.1
0.425 mm (No. 40)	98.9	99.7	32.1	34.7
0.175 mm (No. 60)	96.2	99.5	26.4	27.1
0.074 mm (No. 200)	73.1	33.1	17.3	13.3
Liquid Limit, % (AASHTO T-89)	48	23	NP	NP
Plasticity Index, % (AASHTO T-90)	20	NP	NP	NP
Specific Gravity	2.70	2.72	2.20	2.79
AASHTO Soil Classification	A-7-6	A-4	A-1-b	A-1-b
Maximum Density (pcf) (AASHTO T-99)	90.45	107	45*	100*
Optimum Water Content, % (AASHTO T-99)	25	18	60*	9.1

1 kN/m^3 = 6.369 pcf

*Used for testing

mined for input into layered elastic analyses and design procedures for the two projects, as described in Part 2 of this report.

In situ properties, i.e., density and water content, of base and subgrade materials were also determined. Table 2.2 is a summary of these in-place properties.

2.2.1 Standard Indicator Tests and Hveem Stabilometer Resistance Value

The standard indicator tests and Hveem stabilometer resistance value (R-value) tests were performed at the Oregon Department of Transportation, Highway Division, Material Section, Salem. The standard indicator tests included Atterberg limits (AASHTO T-89 and T-90), sieve analysis (AASHTO T-88), specific gravity (AASHTO T-84), and standard Proctor compaction (AASHTO T-99).

Results of standard indicator tests, summarized in Table 2.2, show that the subgrades occurring along the Salem Parkway project were a clay and a silty sand material, which classified as A-7-6 and A-4 (AASHTO soil classification), respectively. These soils will be referred to as subgrade 1 and subgrade 2. A volcanic pumiceous material, which classified as A-1-b, occurred as the subgrade for the second project (U.S.-97). Figures 2.4 and 2.5 present the results of standard Proctor compaction tests performed on the subgrade and base materials for each project. The results of the standard proctor compaction tests for the U.S.-97 materials are variable. This is due to the nature of the pumice-type volcanic material which absorbs moisture and retains a high moisture content. Based on the tests performed on these materials and experience in the use of the pumice material (3,6), a maximum density of 45 pcf and 60% optimum moisture content was used for the subgrade testing. The base material was tested at 100 pcf and 9.1% moisture.

Table 2.2

In-Place Material Properties

Location and Material	In-Place Water Content, %		In-Place Density, pcf	
	Subgrade 1	Subgrade 2	Subgrade 1	Subgrade 2
Salem - New Parkway				
Subgrade	23.5	14.2	83.1	103.9
Dalles - California Hwy. (U.S.-97)				
Base	9.0		*	
Subgrade	76.1		*	

*No in-place tests were conducted.

Subgrade 1 = clayey soil (AASHTO classification A-7-6)

Subgrade 2 = silty soil (AASHTO classification A-4)

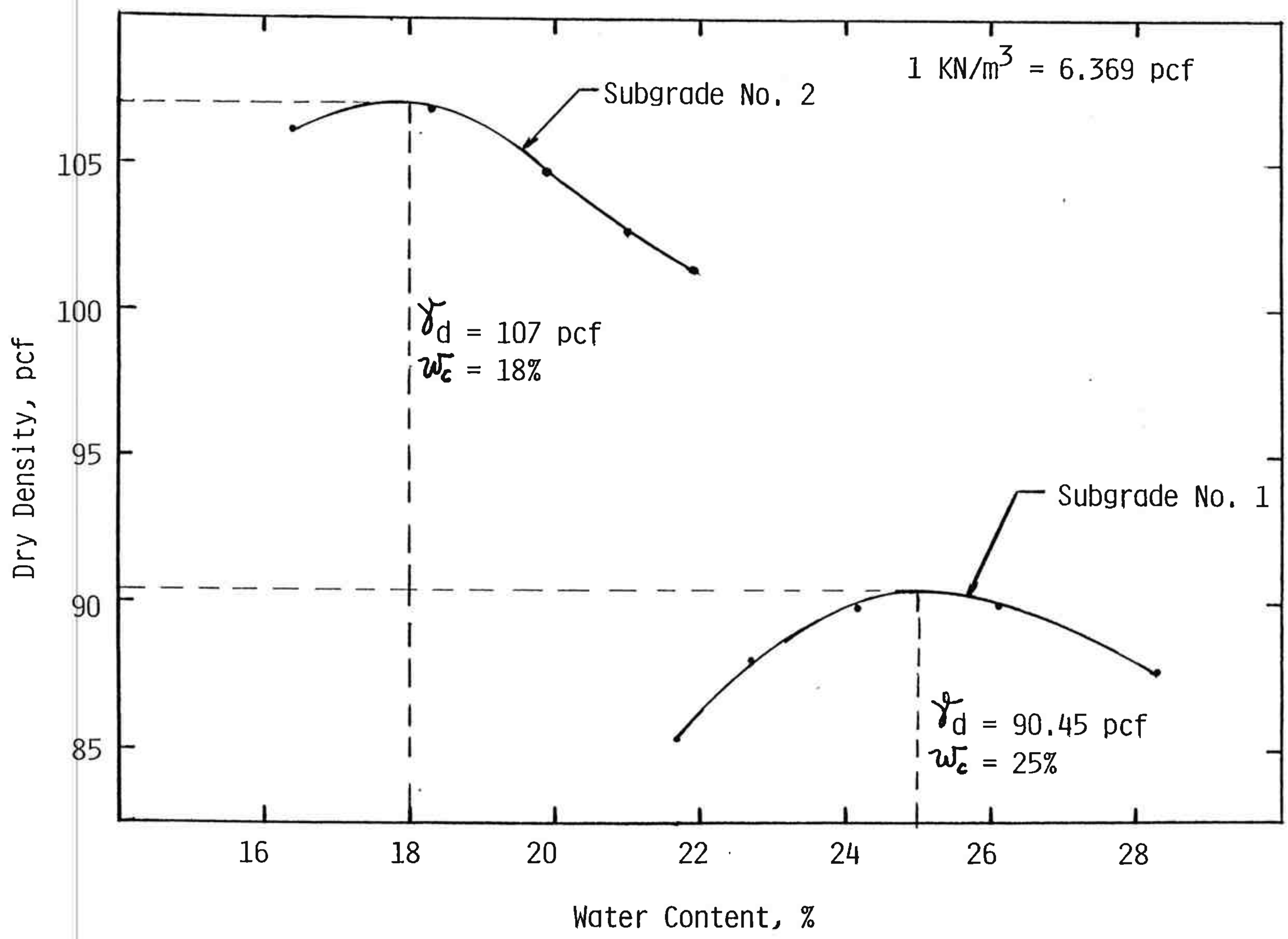
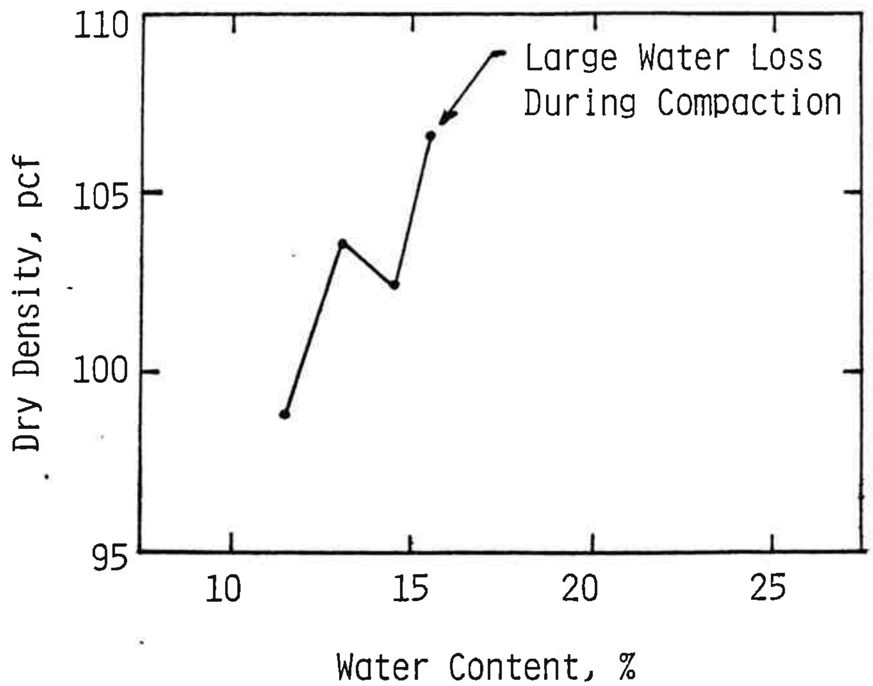
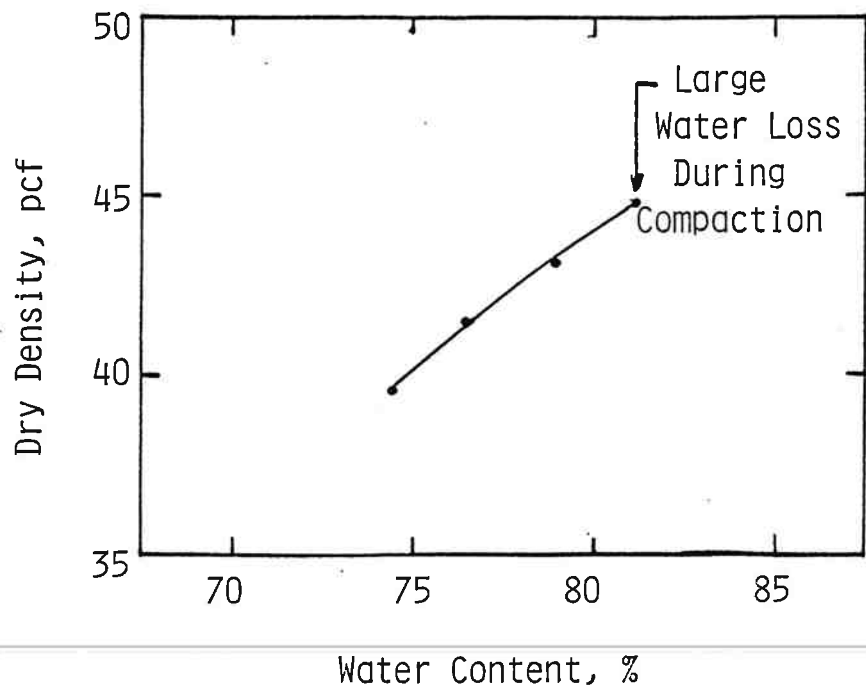


Figure 2.4 - Moisture - Density Curve, Salem Parkway Project.
For both Subgrade Soils



a) Base Material

1 KN/m^3 = 6.369 pcf



b) Subgrade Material

Figure 2.5 - Moisture-Density Curve, US-97 Project Base and Subgrade Materials

Table 2.3 presents the results of R-value (AASHTO T-190) tests on materials from both projects.

2.2.2 Specimen Preparation

The desired water contents and densities for the subgrade and base materials used in the repeated load tests were determined by choosing moisture contents above and below optimum and at maximum dry density obtained from the standard AASHTO compaction test (T-99), and at 100% and 95% of the AASHTO T-99 maximum dry density such that the range of test conditions encompassed those occurring in each project. For each density the water content was to be at optimum and ± 4% of optimum, except for the volcanic material, where the range was ± 20%. This type of material retains a lot of water. Figure 2.6 shows the combination of moisture and density for which resilient modulus tests (M_R) were conducted. Due to the limited time available for this study, the comparison between the diametral and triaxial testing modes was only possible for subgrade 2 from the Salem Parkway project and for the subgrade from the U.S.-97 project. In addition, due to the high level of saturation at 100% relative compaction and wet of optimum condition, tests were not successful at this combination. For similar reasons, tests at 95% relative compaction and wet of optimum were not successful for subgrade 2 from the Salem Parkway. In summary, duplicated samples were tested using the repeated load triaxial and diametral test devices at five different combinations of moisture content and density.

The triaxial test specimen preparation was based on procedures used by Filz (1), Hull, et al (2), and Kidwai (3). Appendix A describes the details of sample preparation and the test procedure.

Table 2.3

Hveem Stabilometer Resistance Tests Data (R-value)

<table>
<tr><th rowspan="4">Location and Material</th><th colspan="6">R-Value</th></tr>
<tr><th colspan="2"></th><th colspan="4">Laboratory Specimen</th></tr>
<tr><th colspan="2">Field Condition
Subgrade</th><th colspan="2">= 95% of max
Subgrade</th><th colspan="2">Exud. Pres. = 300 psi
Subgrade</th></tr>
<tr><th>1</th><th>2</th><th>1</th><th>2</th><th>1</th><th>2</th></tr>
<tr><td>Salem - New Parkway Subgrade</td><td>48**</td><td>65**</td><td>20</td><td>15</td><td>28</td><td>61</td></tr>
<tr><td>Dalles - California Hwy. (U.S.-97)</td><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td>Base</td><td colspan="2">-</td><td colspan="2">-</td><td colspan="2">-</td></tr>
<tr><td>Subgrade</td><td colspan="2">86</td><td colspan="2"></td><td colspan="2">63</td></tr>
</table>

Subgrade 1 - clayey soil (A-7-6)

Subgrade 2 - silty sand (A-4)

**Exudation press, psi = 800

Material used in the laboratory as received from the field.

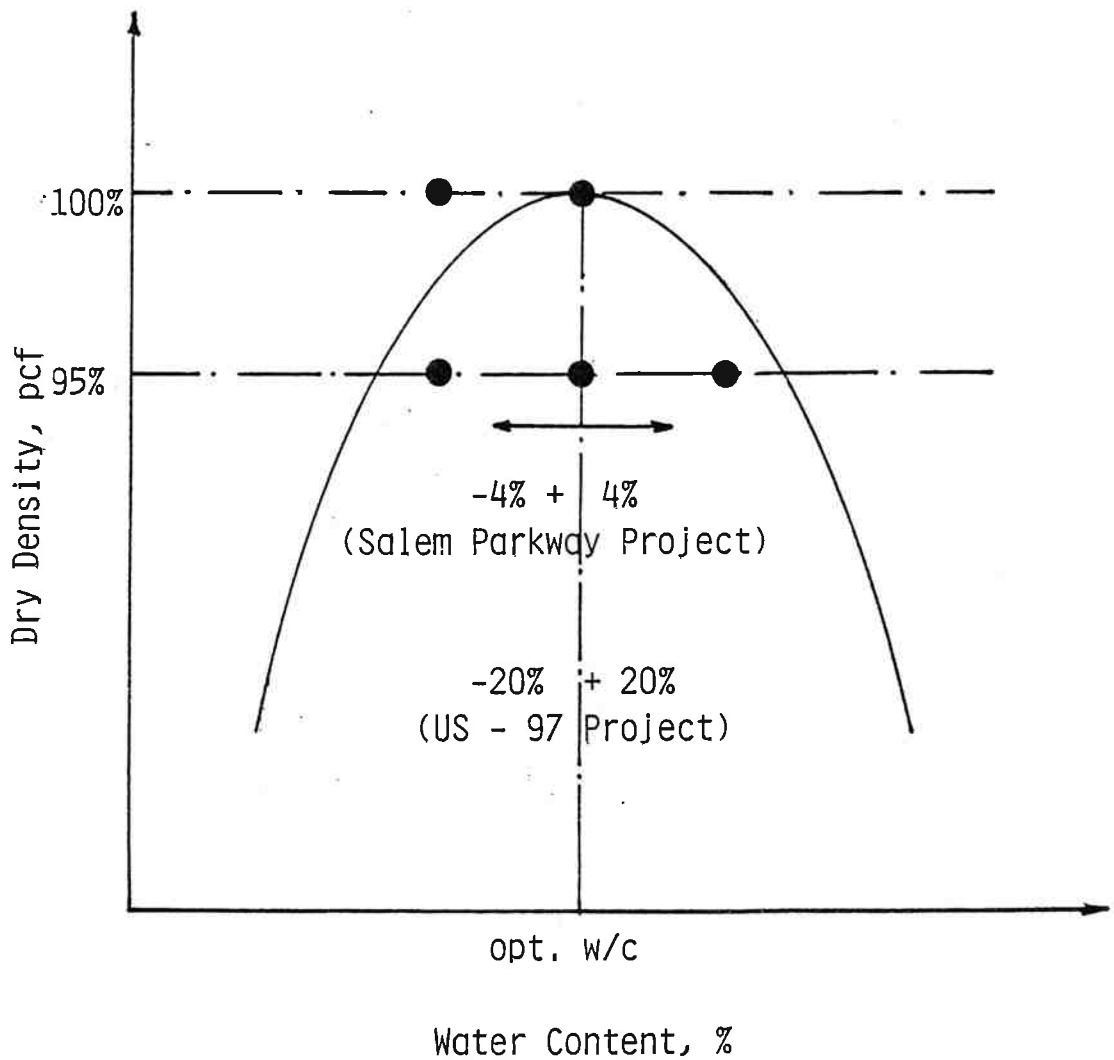


Figure 2.6 - Combination of Moisture and Density for Resilient Modulus Subgrade Testing Program

In summary, the test specimens were prepared by adding a predetermined amount of water to the sample and allowing equilibrium to be reached (24 hours waiting period). The appropriate soil weight was proportioned to give the desired test specimens density in a mold of known volume. The test specimens were compacted in seven lifts (triaxial modulus specimen) and two lifts (diametral modulus specimen). A 2.5 kg (5.5 lbs) hammer dropped 30.5 cm (12 inches) was used in compacting the specimens. Trial and error procedures were used to determine the number of blows per lift to reach the desired density. Appendix A presents in detail the compaction procedure.

Cement-modified soil specimens at 5% cement and optimum water content were also prepared for diametral modulus testing. The same compaction procedure explained above was used in the preparation of these.

Asphalt concrete and cement-treated base cores, 10.4 cm (4 inches) in diameter, were obtained at the fields. From these cores, 6.4 cm (2.5 inch) high specimens were obtained for the repeated load diametral modulus tests.

2.3 Test Equipment and Testing Procedure

A brief description of the test equipment and procedures for the repeated load triaxial and diametral modulus is presented in this section. See Appendix A for operation details.

2.3.1 Repeated Load Triaxial Resilient Modulus Test

The repeated load triaxial test device employed in this study is shown in Figure 2.7. This triaxial equipment consists of: 1) triaxial cell, 2) loading system, 3) timing device, and 4) suitable readout equipment for the type of loading and deformation monitoring devices which are incorporated.

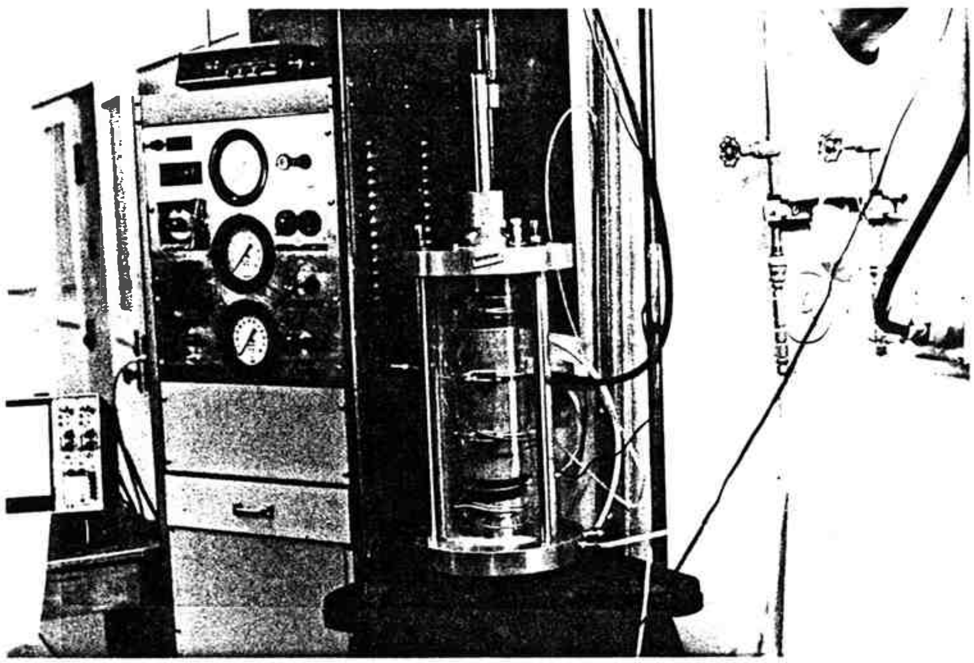


Figure 2.7 - Repeated Load Triaxial Test Apparatus

The test procedures employed are essentially the same as used in previous studies (1,2,3,4). The load on the test specimen is measured by a load cell and vertical displacements are measured by two linear-variable differential transformers (LVDT's). The output from the load cell and LVDT's are input to a strip chart recorder.

The results from repeated load triaxial tests are expressed in terms of a resilient modulus, M_R. The resilient modulus is defined as:

$$M_R = \frac{\sigma_d}{\varepsilon_a} \qquad (2.1)$$

in which,

σ_d = cyclic deviator stress (σ_d = P/A),

ε_a = recoverable axial strain,

P = axial load, and

A = horizontal specimen area.

The test specimen, 10.4 cm (4 inches) in diameter by 25.4 cm (10 inches) in height, were enclosed in rubber membranes, achieved by fitting the rubber membrane in the split mold prior to compaction. After the mold is removed, LVDT clamps are attached to the specimen-rubber membrane. A 10.4 cm (4 inch) gage length is set between the clamps and finally, the triaxial cell is assembled, placed in the load frame, and the cyclic load is applied. A load duration of 0.10 seconds at a rate of 30 repetitions per minute was chosen in this study.

The resilient modulus for the base and subgrade materials was evaluated over a range of stresses. Table 2.4 presents the stress level, sequences, and stress ratio used in the test program. The stress sequence for base and

Table 2.4

Stress Level and Sequence for Stress Ratio Used for Repeated Load Testing of Untreated Soils

		Deviator Stress, psi						
		Base				Subgrade		
Confining Pressure, psi		2	4	6	8	2	4	6
Stress Ratio, σ_1/σ_2	1.5	1.0	2.0	3.0	4.0	1.0	2.0	3.0
	2.0	2.0	4.0	6.0	8.0	2.0	4.0	6.0
	2.5	3.0	6.0	9.0	12.0	3.0	6.0	9.0
	3.0	4.0	8.0	12.0	16.0	4.0	8.0	12.0
	3.5	5.0	10.0	15.0	20.0	5.0	10.0	15.0

1 psi = 6.9 kN/m^2

subgrade materials is in accordance with recommendations made by Kalcheff and Hicks (4), and these stress levels were chosen to encompass those likely to occur in the field.

Before resilient modulus was measured, the sample was preconditioned (1,2,3) to eliminate the effects of the interval between compaction and loading and initial loading versus reloading. The specimens were preconditioned with 1000 load repetitions at a combination of confining pressure and deviator stress which produced the greatest deflection of the sample to insure removal of any permanent deformation. The conditioning started by applying 200 repetitions at maximum confining pressure and minimum deviator stress, then increasing the deviator stress every 200 repetitions keeping constant the confining pressure until 1000 repetitions and maximum deviator stress were achieved. Once the sample has been conditioned, it is only necessary to subject the sample to 100-150 stress repetitions at each combination of confining pressure and deviator stress before measuring the resilient modulus.

2.3.2 Repeated Load Diametral Resilient Modulus Test

The repeated load diametral test system has been used extensively at Oregon State University for bituminous mixture characterization (5) and the one used in this study is the same as the system employed by Hsu, et al (6) for soils. This system and the procedures used are very similar to those described in ASTM D4123-82 for bituminous mixtures. Figure 2.8 shows a repeated load diametral system and Figure 2.9 presents the modifications required for soils testing.

The repeated load diametral test unit includes the same type of loading and deformation monitoring devices as the repeated load triaxial test unit. The vertical diametral load is measured with a load cell, horizontal deforma-

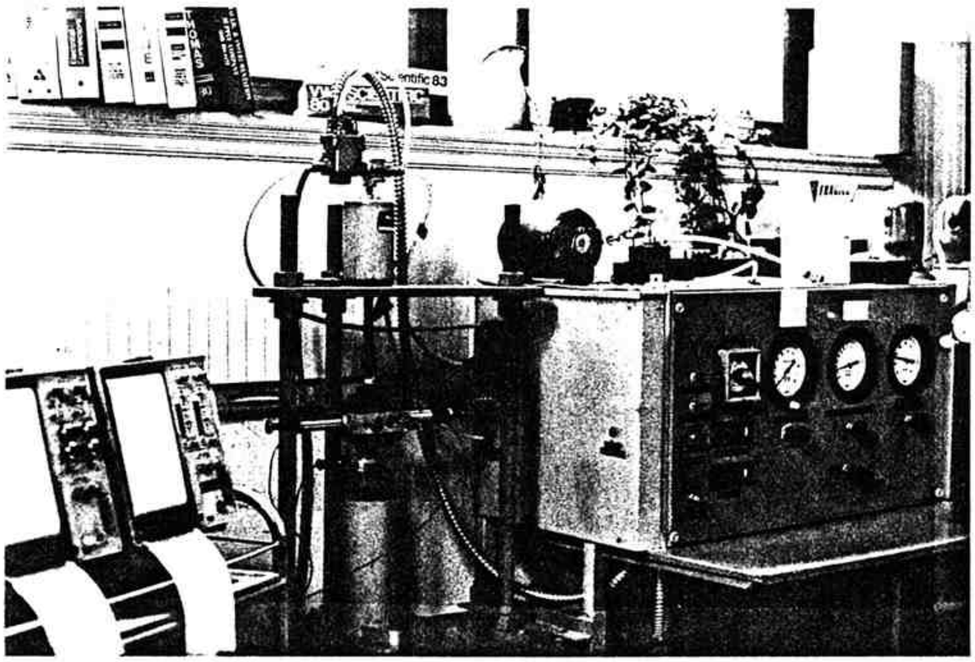


Figure 2.8 - Repeated Load Diametral Test Apparatus

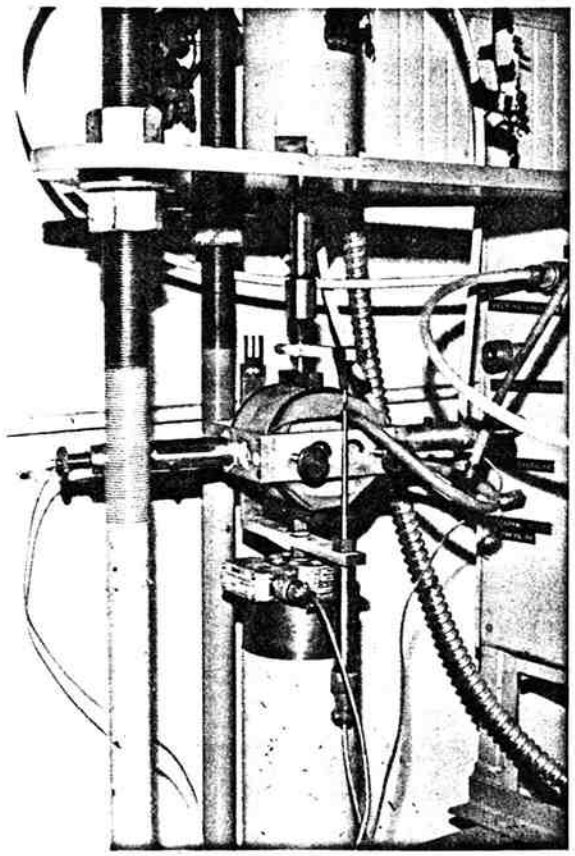


Figure 2.9 - Repeated Load Diametral Test Equipment Modified for Vertical Deformation Reading and Confining Pressure Applications

tions are measured with two horizontally mounted transducers. The vertical deformation was measured with a gage head LVDT. The output from the load cell, transducers and LVDT are recorded with a two-channel strip chart recorder.

The results from repeated load diametral tests are expressed in terms of a Poisson's ratio (ν_{RI}) and resilient modulus (M_R). Equations developed by Kennedy (7) provide the formulas which permit the calculation of Poisson's ratio and modulus, as follows:

Instantaneous resilient Poisson's ratio:

$$\nu_{RI} = \frac{DR\ (0.0673) - .8954}{DR\ (-.2494) - .0156} \tag{2.2}$$

Instantaneous resilient modulus:

$$M_R = \frac{P}{H_{RI} \times t} \left(0.2692 + .9974\ \nu_{RI} \right) \tag{2.3}$$

where,

$DR = \frac{V_{RI}}{H_{RI}}$ = deformation ratio,

H_{RI} = instantaneous resilient horizontal deformation,

V_{RI} = instantaneous resilient vertical deformation,

P = diametral load $\left(P = \frac{\sigma_d \cdot t \cdot \pi \cdot d}{6} \right)$,

t = thickness,

σ_d = deviator stress, and

d = diameter of specimen.

The test specimens, 10.4 cm (4 inches) in diameter by 6.4 cm (2.5 inches) in height were compacted and transferred to a split mold and fitted with a rubber membrane. The specimen was enclosed between two aluminum plates, two teflon sheets and the rubber membrane (see Appendix A for details). A vacuum was applied to confine the specimen. The specimens were preconditioned following the same pattern used with the repeated load triaxial test. Also, the resilient modulus was evaluated over the same range of stresses used on the subgrade material tested with repeated load triaxial test equipment.

For the treated materials the levels of stress used were different. The asphalt concrete cores were tested at 50, 75, 100, and 125 microstrain. The CTB cores were tested at 150, 200, and 300 lbs deviator load. Finally, the cement-modified subgrade specimens were tested at 50, 75, 100, and 200 lbs deviator load.

3.0 TEST RESULTS

This chapter presents a summary of the repeated load triaxial and diametral resilient modulus tests. For the Salem Parkway project the results for 95% of maximum density at optimum and -4% of optimum water content are presented for the subgrade 2 soils. For subgrade soil 1, only the triaxial results for 95% relative compaction and optimum moisture are presented. For the U.S.-97 project, results for 95% of maximum density and 40, 60, and 80% moisture contents are presented. The results obtained at 100% of maximum density and different moisture contents are presented in Appendix B. The results of the diametral repeated load tests of all treated materials is also presented. Finally, the comparison between repeated load triaxial and diametral modulus test results for the subgrade materials is presented.

3.1 Triaxial Resilient Modulus Results

Triaxial resilient modulus test results for the untreated soils from both projects are presented below.

3.1.1 Base Layer

The U.S.-97 base material was tested using only the repeated load triaxial test system. Figure 3.1 shows the effect of the confining pressure, and principal stress, (θ) on the resilient modulus. This figure shows that the resilient modulus increased with an increase of the confining pressure. Also, the resilient modulus increased with an increase of the principal stress. A range of modulus from 6160 psi at σ_3 = 2 psi to 18072 psi at σ_3 = 8 psi was obtained when 95% of maximum density and 9% water content, (base field water content), testing condition were used. Detailed example calculations of the results are given in Appendix A.

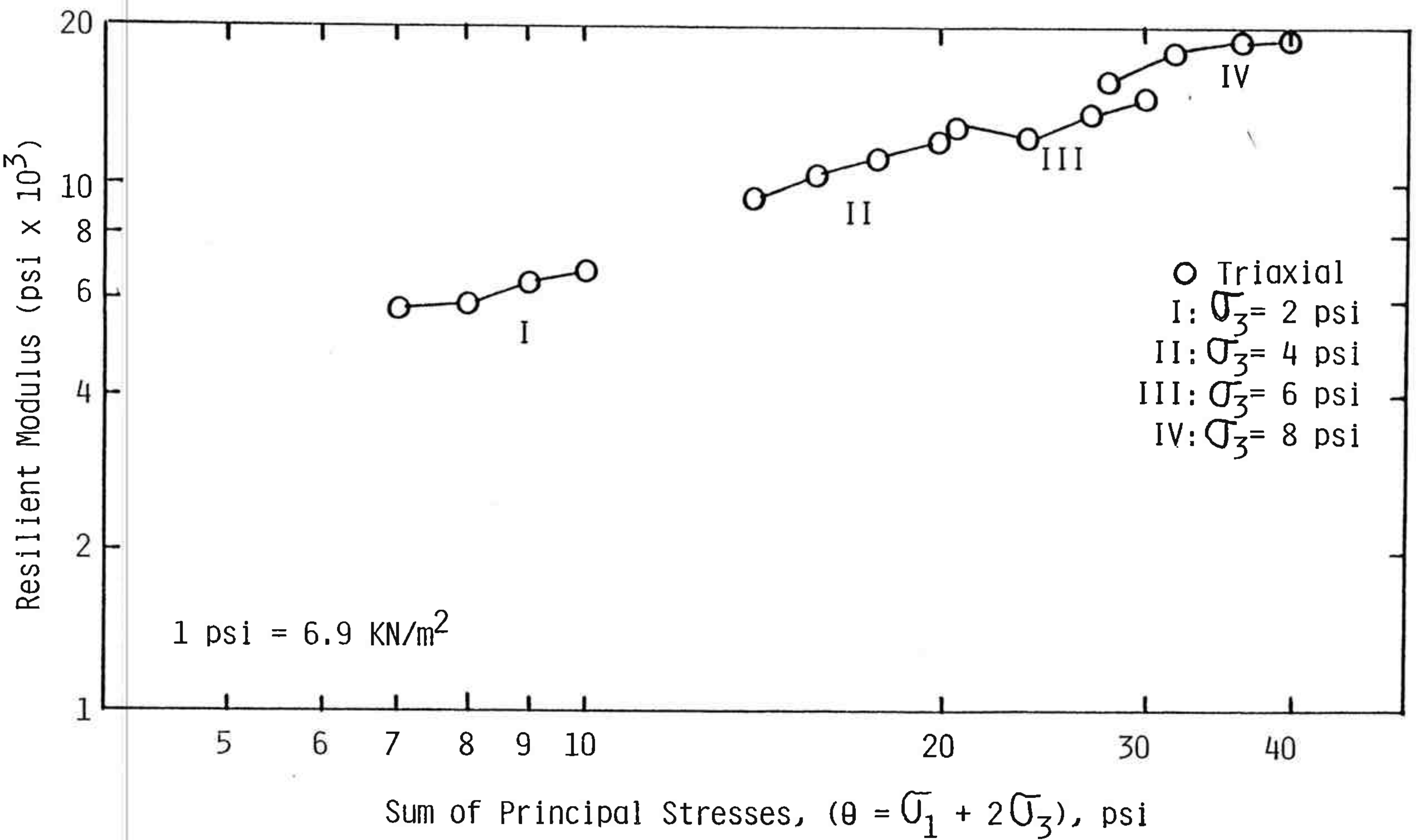


Figure 3.1 - Sum of Principal Stresses vs. Triaxial Resilient Modulus, 95% Compaction, 9% Water Content, Cinder Base, US-97 Project

3.1.2 Subgrade Layer

Subgrade material from both projects was tested using the repeated load triaxial system. The results for the Salem Parkway project for both subgrade soils are given in Figures 3.2, 3.3, and 3.4, respectively. These figures show the effect of the confining pressure (σ_3) and deviator stress (σ_d) on the resilient modulus. Both soils exhibited the usual behavior found with fine-grained soils, viz, the modulus increased with an increase in the confining pressure and decreased to a minimum with an increase of the deviator stress. For subgrade 2, further increase in deviator stress caused a slight increase in modulus. A range of modulus of 4560 psi at σ_3 = 2 psi to 8860 psi at σ_3 = 6 psi, for the subgrade 2, and 7600 psi at σ_3 = 6 psi to 9500 psi at σ_3 = 2 psi for the subgrade 1, were obtained.

The resilient modulus results for the U.S.-97 subgrade are presented in Figures 3.5, 3.6, and 3.7, respectively. These figures show the effect of the confining pressure, (σ_3) and sum of principal stresses (θ) on the modulus. For this soil the triaxial resilient modulus increased with an increase in the confining pressure and increased with an increase of the sum of the principal stresses. A range of modulus of 1650 psi at σ_3 = 2 psi to 7360 psi at σ_3 = 8 psi was obtained. Detailed example calculation of the results are given in Appendix A.

3.2 Diametral Resilient Modulus Test Results

Diametral resilient modulus test results for asphalt concrete, cement-treated base, cement-modified soil, and subgrade material of both projects are presented below.

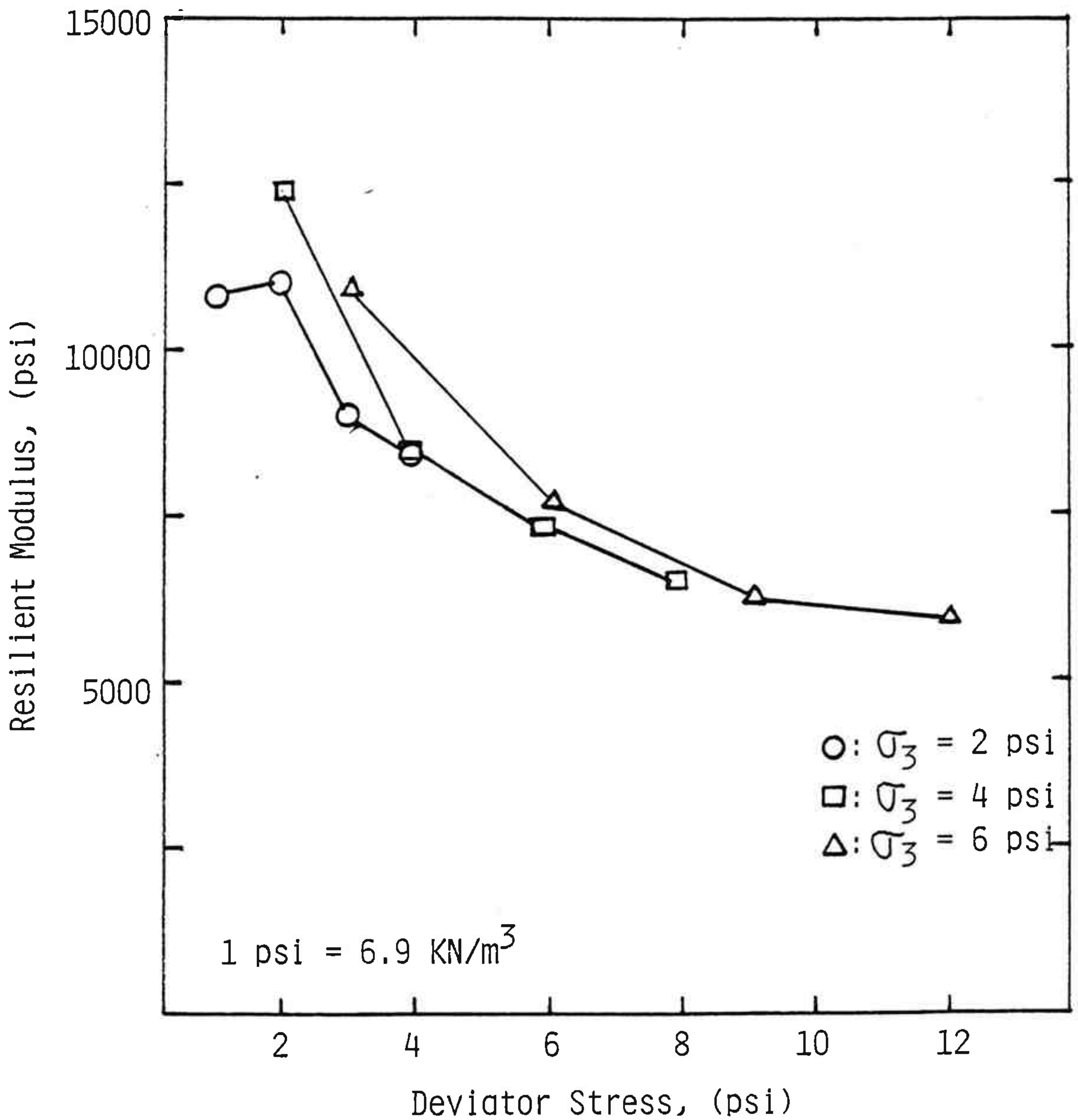


Figure 3.2 - Triaxial Resilient Modulus vs. Deviator Stress
Salem Parkway Project, Subgrade 1,
95% Compaction, 25% Water Content

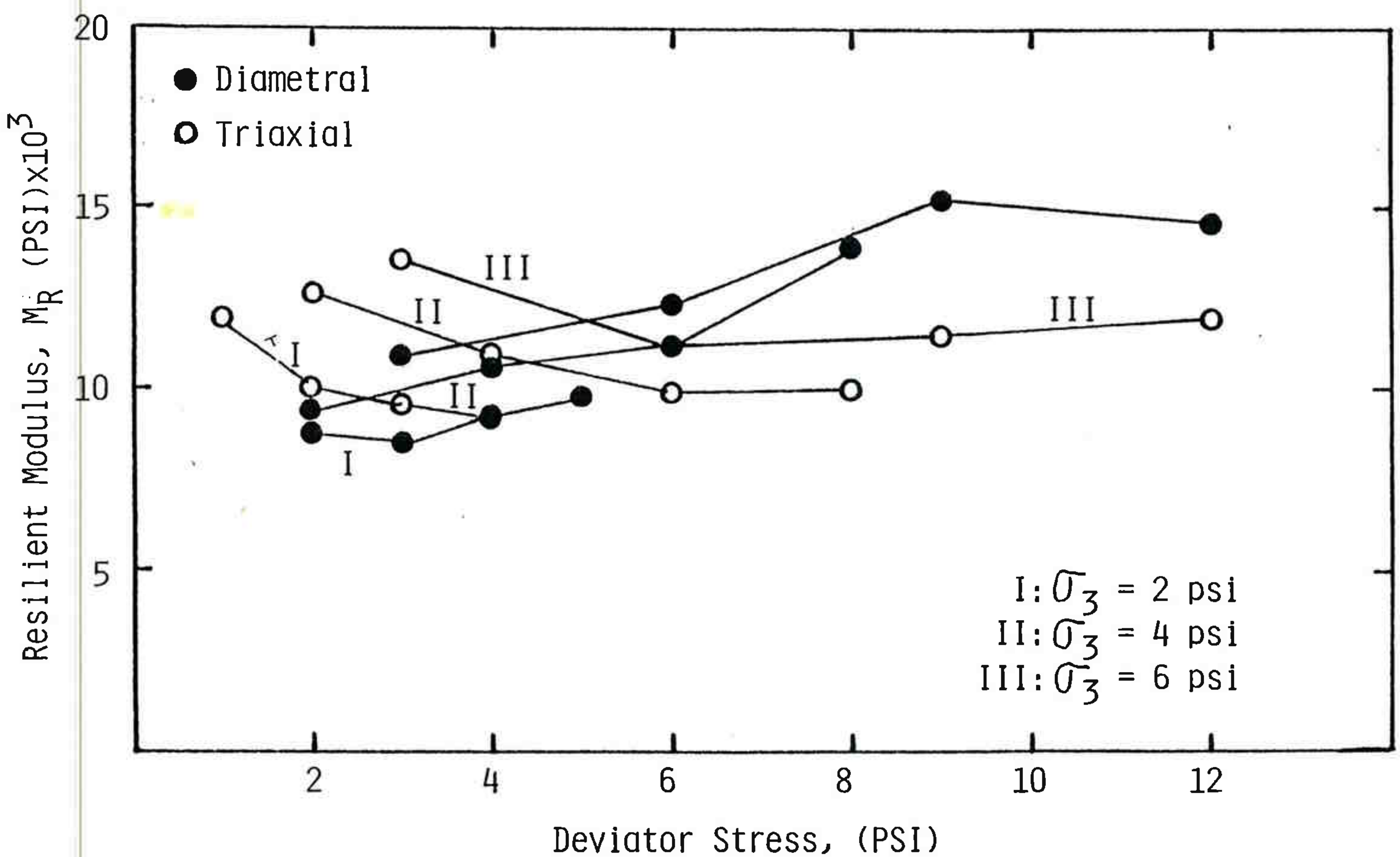


Figure 3.3 - Comparison of Triaxial and Diametral Resilient Modulus Results, Salem Parkway Project, Subgrade 2, 95% Compaction, 14% Water Content

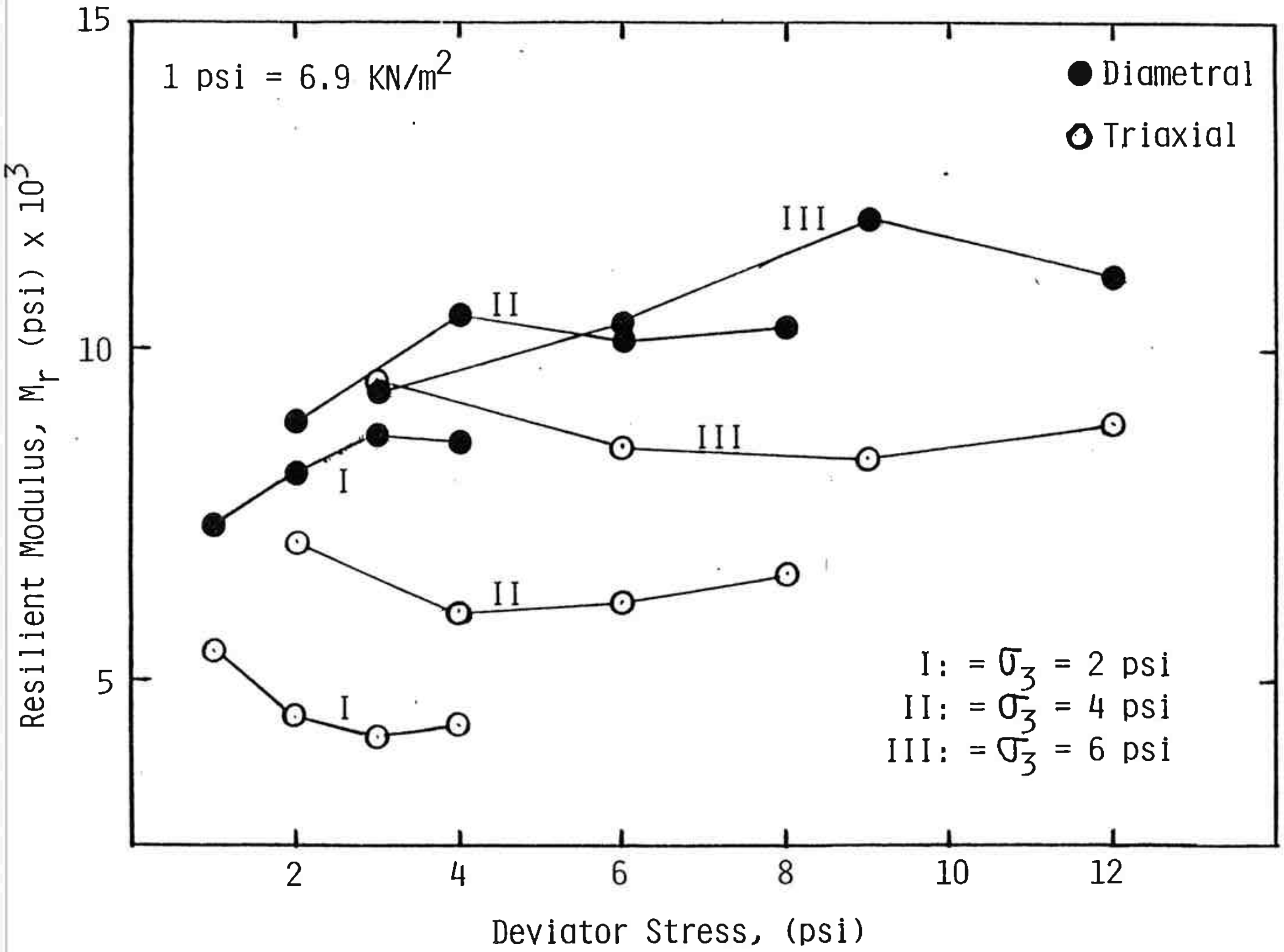


Figure 3.4 - Comparison of Triaxial and Diametral Resilient Modulus Results, Salem Parkway Project, Subgrade 2, 95% Compaction, 18% Water Content

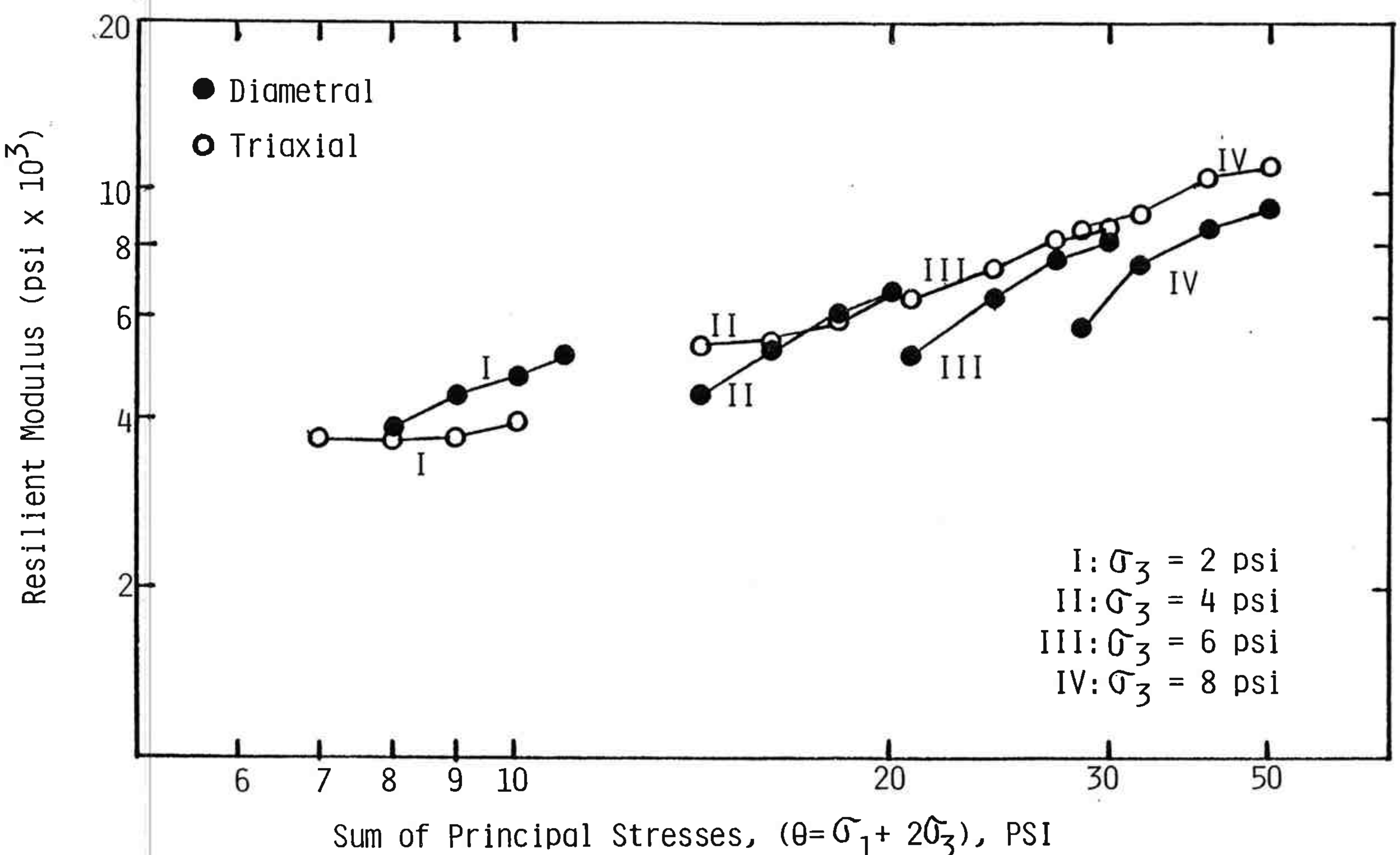


Figure 3.5 - Comparison of Triaxial and Diametral Resilient Modulus, US-97 Project, Subgrade Soil, 95% Compaction, 40% Water Content

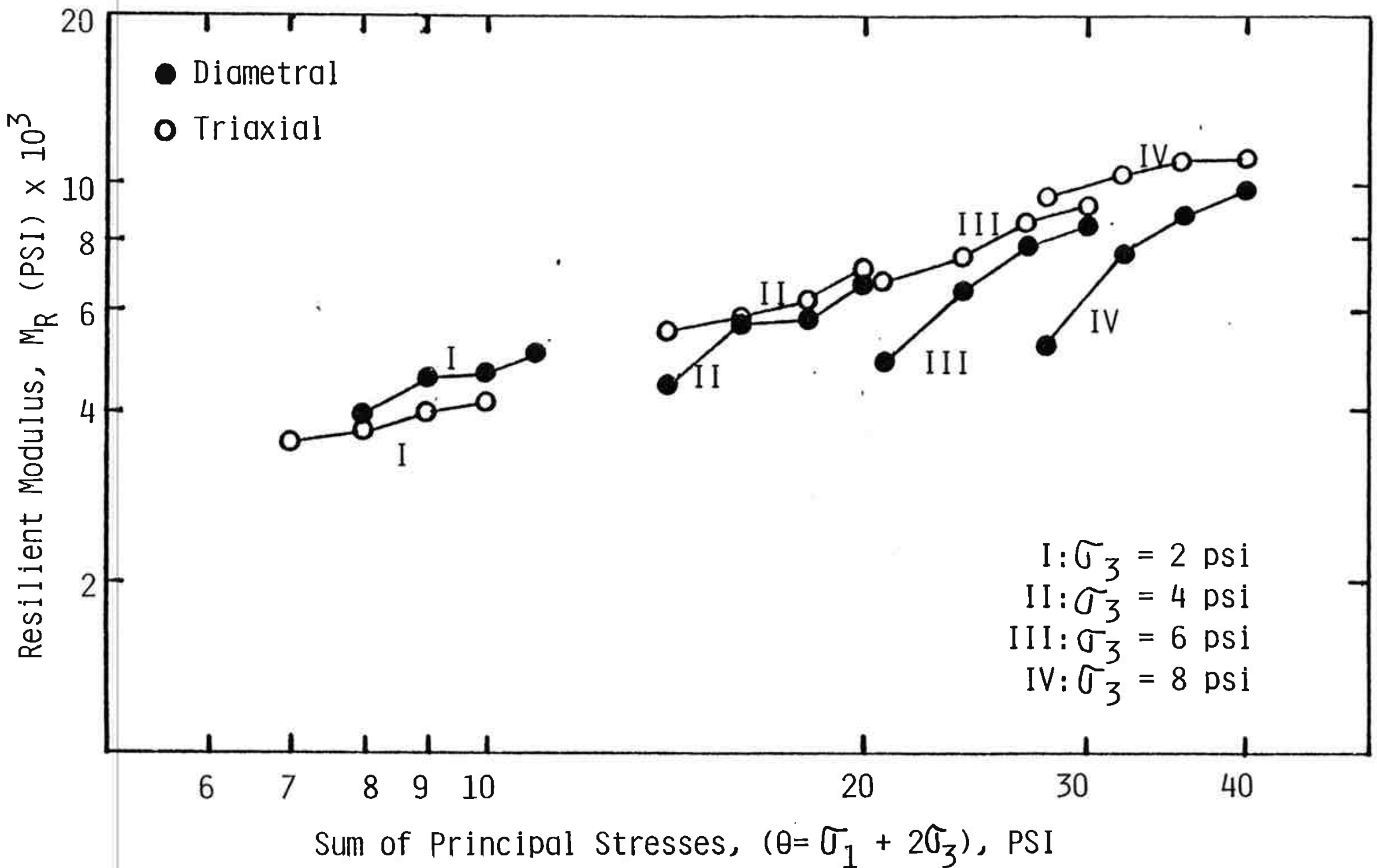


Figure 3.6 - Comparison of Triaxial and Diametral Resilient Modulus, US-97 Project, Subgrade Soil, 95% Compaction, 60% Water Content

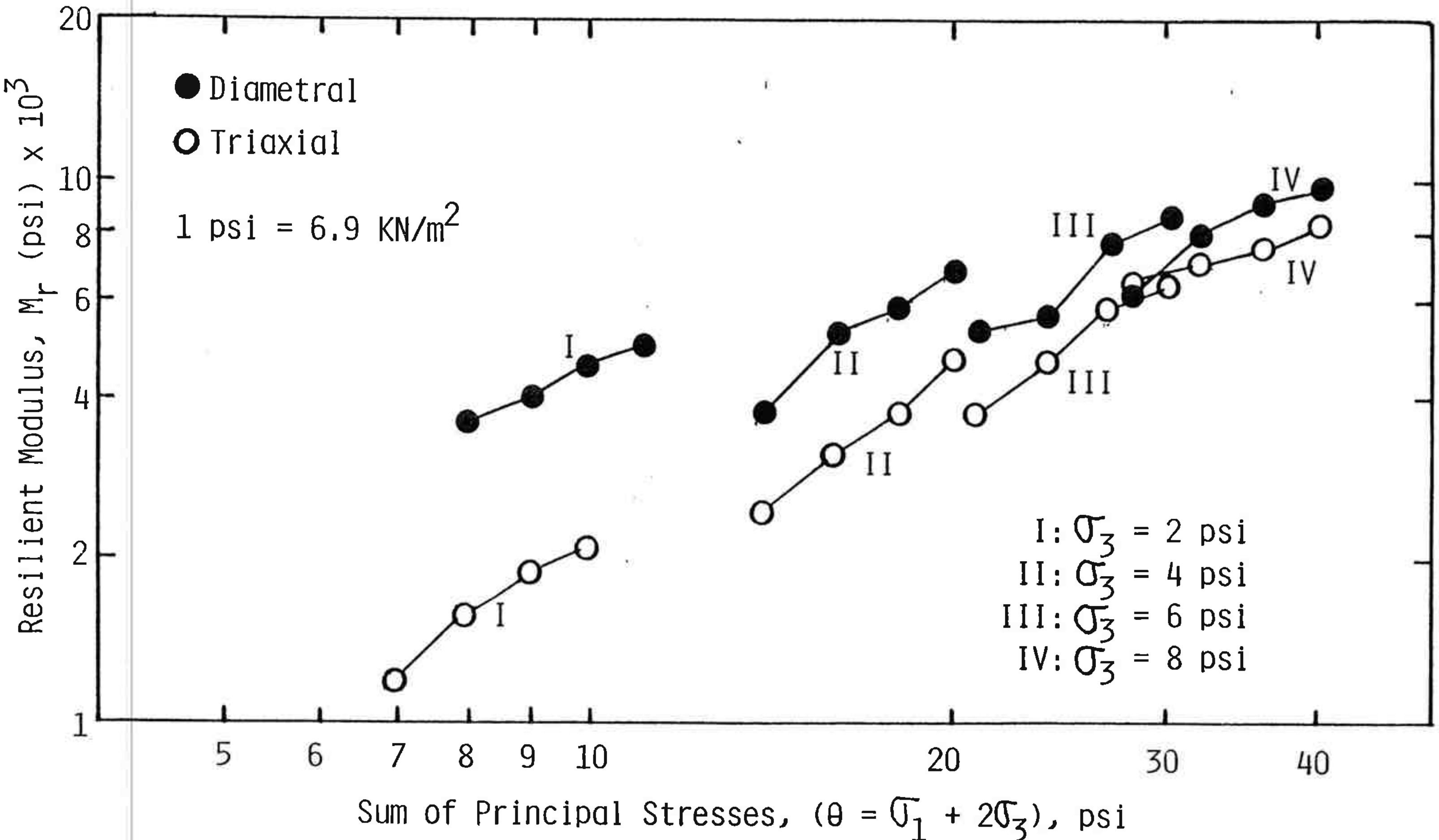


Figure 3.7 - Comparison of Triaxial and Diametral Resilient Modulus, US-97 Project, Subgrade Soil, 95% Compaction, 80% Water Content

3.2.1 Asphalt Concrete Layer

Field asphalt concrete cores from both projects were tested using the repeated load diametral test system. Table 3.1 summarizes the results for the Salem Parkway project. Samples corresponding to the two subgrade types were tested with average resilient modulus of 172,320 and 139,200 psi, respectively.

Table 3.2 summarizes resilient modulus results for the asphalt concrete cores for the U.S.-97 project determined with the diametral apparatus. The cores tested from this site included the new overlay (0-4") and the old pavement (4"-18"). An average resilient modulus of 448,140 psi, from the new overlay (0-4") pavement, and an average resilient modulus of 728,360 psi (4"-10") and 1,006,720 psi (10"-12"), from the old pavement were obtained. The tests on the asphalt concrete cores were conducted at an average temperature of 20°C. A Poisson's ratio of 0.35 was used for the resilient modulus calculations.

3.2.2 Cement-Treated Base and Cement-Modified Soil Layers

Cement-treated base cores from two sites of the Salem Parkway project were tested. Table 3.3 summarizes the diametral resilient modulus test results. Average resilient modulus of 2,100,000 and 2,320,000 psi were obtained, respectively, for the two sets of cores tested.

Cement-modified subgrade (CMS) specimens (5% cement and optimum water content) for both subgrades from the Salem Parkway project were prepared in the laboratory. Resilient modulus diametral test results for subgrades 1 and 2 are given in Tables 3.4 and 3.5, respectively. An average resilient modulus of 729,000 psi at seven days curing time for subgrade 2 were obtained and 180,000 psi at seven days curing time for subgrade 1. A Poisson's ratio of 0.20 (CTB) and 0.22 (CMS) were used for the resilient modulus calculation.

Table 3.1

Summary of the Asphalt Concrete Cores
Diametral Resilient Modulus Test
Salem Parkway Project

Sample ID	Height cm	Height in	Resilient Modulus, x10^3 psi 50 μs	75 μs	100 μs	125 μs	Average Resilient Modulus x10^3, psi
1-1	6.39	2.52	293.08	186.69	182.67	184.84	
1-2	6.26	2.46	165.07	159.28	161.58	161.83	172.32
1-3	5.78	2.28	166.65	169.20	167.24	169.71	± 11.38
2-1	5.60	2.20	157.31	164.83	159.01	158.81	
2-2	5.46	2.15	123.67	120.75	122.27	123.18	139.20
2-3	5.67	2.23	132.66	137.27	135.44	135.23	± 16.41

Test temperature = 20°C

1 psi = 6.9 kN/m^2

1 in. = 2.54 cm

Table 3.2

Summary of Asphalt Concrete Cores
Diametral Resilient Modulus Test
U.S.-97 Project

Sample ID	Depth in.	Height cm	Height in	Resilient Modulus x10^3 (psi) 50 μs	75 μs	100 μs	125 μs	Average Resilient Modulus x10^3, psi
1-1	0-4	6.15	2.42	480.51	455.47	451.40	440.40	448.14
2-1	0-4	5.84	2.30	433.70	446.33	446.12	430.90	± 15.49
1-3	4-10	5.81	2.29	573.47	579.09	566.69	564.42	728.26
2-3	4-10	6.41	2.52	911.44	900.78	868.57	861.64	± 169.01
1-4	10-12	5.28	2.08	909.58	880.01	868.12	835.56	10006.72
2-4	10-12	5.91	2.33	1173.4	1159.2	1124.1	1103.8	± 145.52

Test temperature = 20°C

0-4 in. = New Overlay

4-18 in. = Old Pavement

Table 3.3

Summary of the Cement-Treated Base Cores
Diametral Resilient Modulus Test
Salem Parkway Project

Sample ID	Height cm	Height in	Resilient Modulus, x10^6 (psi) Load = 150 lb. (68 kg)	Load = 200 lb. (91 kg)	Load = 300 lb. (136 kg)	Average Resilient Modulus x10^6, psi
1-8-1	6.24	2.46	1.815	1.853	1.977	2.10
1-8-2	6.12	2.41	2.224	2.347	2.381	± 0.25
2-8-1	6.45	2.54	2.241	2.392	2.380	2.32
2-8-2	6.20	2.44	2.203	2.363	2.362	± 0.08
Average Deviator Stress (psi)			29.13	38.81	58.22	
Average Tensile Strain (x10^{-6} in.)			7.267	9.216	13.517	

1 psi = 6.9 kN/m^2

1 in. = 2.54 cm

Table 3.4

Summary of the Diametral Resilient Modulus Tests
Cement-Modified Subgrade Soil 1
Salem Parkway Project

Sample ID	Height cm	Height in.	Curing Time (days)	Resilient Modulus, (MR) 10^5, psi Load=50 lb (22.73 kg)	Load=75 lb (34.1 kg)	Load=100 lb (45.46 kg)	Load=200 lb (90.91 kg)	Average Modulus (MR)10^5
7	6.41	2.52	2	1.77	1.80	1.74	1.71	1.66
8	6.41	2.52	2	1.57	1.59	1.55	1.51	± 0.11
7	6.41	2.52	7	1.89	1.87	1.86	1.83	1.80
8	6.41	2.52	7	1.76	1.76	1.75	1.671	± 0.08

1 psi - 6.9 kN/m^2

5% cement

Table 3.5

Summary of Diametral Resilient Modulus Tests
Cement-Modified Subgrade Soil 2
Salem Parkway Project

Curing Time	Load (lbs)	Resilient Modulus $x10^5$ (psi)	Average Resilient Modulus $x10^6$ (psi)
2	48.8	3.83	
2	78.0	5.17	4.83 ± 0.88
2	97.5	5.50	
7	78.0	7.16	
7	97.5	7.27	7.29 ± 0.14
7	146.3	7.25	
7	195.0	7.48	

1 psi = 6.9 kN/m^2

5% cement

3.2.3 Subgrade Layer

Subgrade material from both projects were tested using the repeated load diametral test equipment. However, only subgrade soil 2 (AASHTO A-7-6) was tested for the Salem Parkway project, due to the extensive testing required. The results for this soil are shown in Figures 3.3 and 3.4, which show the effect of the confining pressure, σ_3 and deviator stress, σ_d on the resilient modulus. An average diametral resilient modulus of 8,250 psi at σ_3 = 2 psi to 10,740 psi at σ_3 = 6 psi was obtained. Diametral resilient modulus test results, for the U.S.-97 subgrade soil are shown in Figures 3.5, 3.6, and 3.7 which show the effect of the confining pressure, σ_3 and principal stress, θ on the diametral resilient modulus. A range of modulus of 3030 psi at σ_3 = 2 psi to 8270 psi at σ_3 = 8 psi was obtained. Detailed example calculation of the results are given in Appendix A.

3.3 Comparison of Triaxial and Diametral Resilient Modulus Test Procedures and Results

3.3.1 General

The comparison of the triaxial and diametral resilient modulus test procedures is based on the results for subgrade materials from both projects and previous work by Hsu, et al (6).

The resilient modulus for a homogeneous, isotropic, linear elastic material, whether determined with a triaxial system or determined with a diametral test system should be identical. Soils are generally recognized as highly nonlinear, anisotropic, heterogeneous materials. The diametral loading response undoubtedly differs from the triaxial loading response owing to these factors alone.

The comparison between resilient moduli determined with triaxial and diametral test systems may be examined assuming (6):

1) the initial state of stress of the test specimens are identical both in the triaxial and diametral test systems, and
2) the state of biaxial deviator stress of the diametral test specimen does not affect the resilient modulus and Poisson's ratio, i.e., assuming the diametral test specimen is an idealized homogeneous, isotropic and linear elastic material.

Based on these assumptions, the comparisons between triaxial resilient modulus and diametral resilient modulus may be examined in terms of comparable states of stress. Specifically, the triaxial test results are assessed in terms of the axial compressive deviator stress ($\sigma_d = P/A$), and the diametral test results are assessed in terms of the compressive deviator stress at the center of the specimen, ($\sigma_d = 6P/t\pi d$).

Hsu, et al (6) stated that in the diametral resilient modulus test, the deviator stresses are not distributed uniformly either along the vertical or horizontal diameter of the specimen. Equations (2.2) and (2.3) employed in this study to compute resilient modulus and Poisson's ratio are based upon linear elasticity for an idealized material (7). The values of resilient modulus and Poisson's ratio should be constant for a homogeneous, isotropic and linear elastic material. But, the values of resilient modulus and Poisson's ratio for unbound materials would not be constant owning, in part, to the nonlinear and heterogeneous properties associated with unbound materials. Based on this fact, Hsu, et al (6), suggested that the diametral test results should be termed "equivalent" diametral resilient modulus and "equivalent" diametral Poisson's ratio to emphasize that these values are determined and computed based upon linear elasticity and do not take account of nonlinear and heterogeneous properties associated with unbound materials.

3.3.2 Comparison of Test Procedure

The triaxial test equipment and procedure is a very straightforward test. The compaction of the triaxial test specimen is done on the test equipment base which avoids the disturbance of the specimen after the compaction is completed. Data obtained using this equipment can be reproduced if the same testing conditions are used, and can be used for routine determination of the soil properties required for implementation of improved design methods.

The diametral test equipment and procedure for unbound materials is in its preliminary developmental stages. The test is very simple, but it needs skill and knowledge of the testing equipment. After the compaction of the specimen is done a transfer of it to the split mold-rubber membrane is carried out and this may produce disturbance and loss of particles from the specimen. The reproduction of the data were not constant even though the same testing conditions were used.

3.3.3 Comparison of Test Results

Comparison of triaxial and diametral test results for two subgrade soils are shown in Figures 3.3 to 3.7.

The effect of the axial deviator stress (σ_d) and confining pressure (σ_3) on the triaxial and diametral resilient modulus for the subgrade 2 material from the Salem Parkway project is shown in Figures 3.3 and 3.4. The triaxial resilient modulus increased with an increase in the confining pressure and decreased to a minimum and then increased with an increase of the deviator stress. The confining pressure and deviator stress effects on the diametral resilient modulus are about the same as for the triaxial test results. In general, the diametral resilient modulus increased with increasing confining pressure and increased slightly with increasing deviator stress.

The effect of the sum of principal stresses (θ) on the triaxial and diametral resilient modulus for the subgrade from the U.S.-97 project is shown in Figures 3.5, 3.6, and 3.7. As shown, the triaxial and diametral resilient modulus increases with an increasing sum of the principal stresses. Judging from Figures 3.5, 3.6, and 3.7, it can be deduced that at low levels of stress the diametral resilient modulus is higher than the triaxial resilient modulus, but at high levels of stress there is not a particular trend.

In summary, due to the differences and inconsistency of the resilient modulus values (M_R) no general statement can be made about the triaxial and diametral resilient modulus. Results obtained with diametral equipment were more variable, but average values were not consistently higher or lower than triaxial results. From the results obtained in this study, it appears that the relationship between moduli obtained using both devices is a function of soil type in addition to differences in equipment and testing procedures.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

Repeated load diametral and triaxial tests were conducted to determine the resilient moduli of all materials, but particularly the subgrade materials for two projects, one new alignment project in the Willamette Valley (Salem Parkway), and one overlay project in Central Oregon (U.S.-97).

The resilient moduli for the subgrades were measured over a range of density, moisture content, and level of stress. However, for the purpose of comparing the diametral and triaxial test procedures, only the results corresponding to the 95% of the maximum density were used. A summary of the significant findings are presented below:

1) For the subgrade 1 and 2 materials from the Salem Parkway project, the triaxial resilient modulus increased with an increase in the confining pressure and decreased to a minimum and then increased with an increase of the deviator stress. The diametral resilient modulus increased with increasing confining pressure and increased slightly with increasing deviator stress.

2) The triaxial and diametral resilient modulus increased greatly with an increase in the sum of the principal stresses (θ).

3) For all subgrade soils the resilient modulus increased with an increase in the level of compaction, but decreased with an increase in the water content.

4) The diametral resilient modulus results tended to be higher, at low stress levels, than the triaxial resilient modulus results, but at high stress levels there was no particular trend.

5) The equations employed in the diametral resilient modulus and Poisson's ratio calculation are based upon linear elasticity for an idealized material. The value of resilient modulus and Poisson's ratio should be constant for a homogeneous, isotropic and linear elastic material, but the values obtained are not constant due to the nonlinear and heterogeneous properties associated with the unbound material tested.

6) The repeated load triaxial test is straightforward to conduct and it produces repeatable results for all pavement materials.

7) The repeated load diametral test is very well established for treated materials, but for untreated materials, particularly cohesionless soils, the results obtained tend to be variable. To conduct the test requires high skill and knowledge of the equipment being used.

8) Enough information was developed using repeated load testing for all materials for use in the design and analysis of the pavement examined in this study.

4.2 Recommendations.

Based on the results of this study and a previous one conducted by Hsu, et al (6) the following recommendations are proposed:

1) For cohesive soils, the repeated load diametral test can be used for determination of the resilient properties. However, the results of this study show that the repeated load triaxial procedure is preferable.

2) For untreated cohesionless material the repeated load triaxial test should be used for routine determination of the soil properties required for implementation of improved design methods.

3) For treated materials the repeated load diametral test can be used with confidence. It is easy and fast to perform.

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APPENDIX A

APPENDIX A

OPERATION DETAILS FOR REPEATED LOAD TESTING EQUIPMENT

This appendix describes in detail the method of performing resilient modulus tests on soils with the Repeated Load Device developed at Oregon State University. It contains sections on operation of each unit, calibration of both the load cell and the Linear Variable Differential Transducers (LVDT's) to the HP (Model 7402) recorder and description of the resilient modulus test procedure.

OPERATION

Operation of the Repeated Load Device requirements:

1. Set the appropriate connections on the control cabinet and load apparatus.
2. Calibrate the recorders for the appropriate measurement devices (load cell, triaxial LVDT's).
3. Place the sample in the load cell and position it under the load apparatus.
4. Adjust, using the recorders, load intensity and duration, and confining pressure to achieve the desired test conditions.

For convenience, the operation of each unit will be presented separately in this section.

TRIAXIAL LOAD SYSTEM

The load system consists of the testing apparatus and the control cabinet. Figure A-1 shows the control panel and the cabinet electrical and pneumatic outputs. The inputs to the cabinet are the air pressure quick-connect, located on the left side panel and the electric socket, located on the back panel.

All electrical and pneumatic controls are accessible from the control panel. The main air valve and the main electrical power switch should both be shut off before connecting the machine both to the supply lines and to the testing apparatus.

From the bottom to the top, the three bellofram regulators located above the main air valve control the pulse load, the static load and the triaxial cell confining pressure. A precision air pressure gauge gives the output pressure in pounds per square inch for each regulator.

Electrical controls are grouped on the left side of the panel. The timers control the pulse intervals and the pulse deviation. Before testing, these timers should be set to the desired values. The pulse duration timers control the dynamic load duration and the counter. Calibration of the pulse duration timer requires the use of the HP recorder. Calibration of the pulse interval timer is done using a stop watch.

The "fatigue-modulus switch" controls the operation mode of the Repeated Load Device. The "off" position is used during testing as a convenient way to disconnect the timers, and therefore, to interrupt the dynamic load. The "modulus" position is the normal testing mode. In this position, the timers, and therefore both the counter and the dynamic load, are activated.

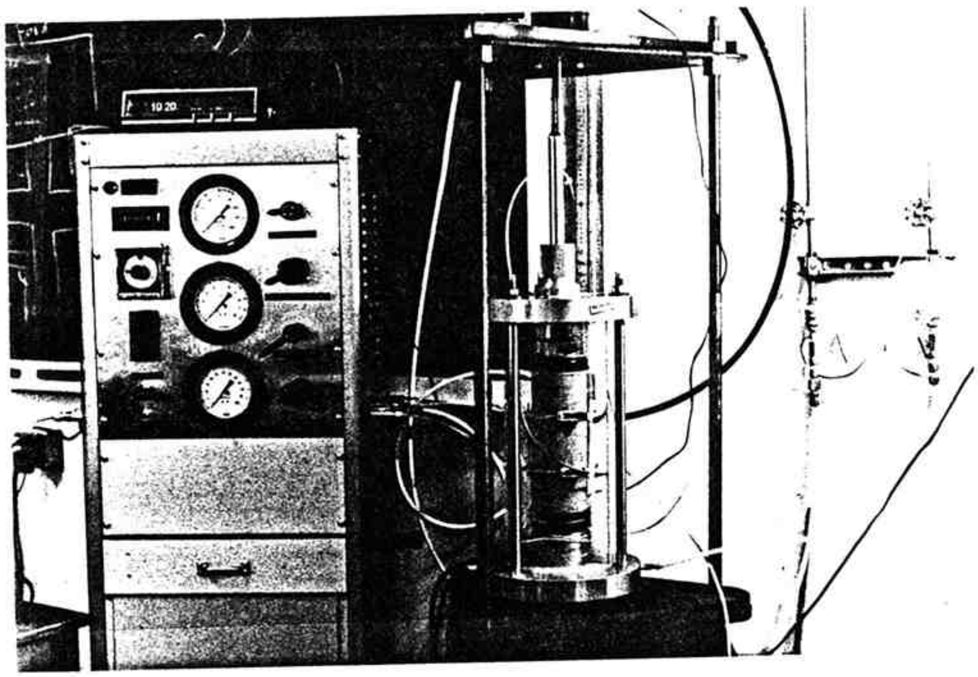

Figure A-1 - Shows the Control Panel and Testing Apparatus

The testing apparatus is shown in Figure A-1. The Mac valve and the shuttle valve control the air flow to the bellofram air cylinder. The Mac valve is a normally closed electrical valve which opens only when the pulse timer is activated. The shuttle valve allows the line of highest pressure to flow in the air cylinder. When the Mac valve is closed, the static load line is under higher pressure than the Mac valve line, and the static load line is connected to the air cylinder. When this Mac valve is open, since the pulse load pressure is normally higher than the static load, the shuttle valve shuts down the static load line and corrects the air cylinder with the Mac valve line.

The triaxial "confining" quick-connect should be connected when a confining pressure is required. The "pulse load" and "static load" should be connected to the corresponding quick-connects on the testing apparatus. It is important to adjust the position of the top plate so that the air cylinder piston is approximately in the center part of the cylinder.

Triaxial Cell

The triaxial cell is composed of a Plexiglas cylinder held in place between a top and a bottom plate by three rods bolted to the plates. An air-tight bearing in the top plate allows the load rod to transmit the load to the sample with minimum function. The quick-connect on the top plate is used to apply confining pressure inside the triaxial cell. Electrical connections between the sample and the recorders are possible through the bottom plate.

This unit was designed to run unchained dynamic triaxial tests on 4-inch (~ 10 cm) by 10-inch (25.4 cm) cylindrical specimens. A pair of LVDT's with clamp are used to measure the sample's vertical deformation under an axial dynamic load. The load applied, the confining and the sample's vertical deformation are then recorded and used to compute the sample's resilient modulus.

Recorder

A two-channel Hewlett-Packard Model 7402A oscillographic recorder, with two HP17403A A/C carrier preamplifier is used for the triaxial test LVDT's. Detailed information on both the oscillographic recorder and the A/C preamplifier are presented in the Operating and Service Manuals. A separate calibration of the A/C preamplifier is required for each set of transducers or LVDT's. Calibration of the transducers and LVDT's require using the appropriate calibrators. LVDT's are very accurate instruments and their calibration is necessary only when some modifications have been done to the wiring system connecting the LVDT to the HP recorder. The equipment required for the calibration is a small-size screwdriver and a calibrator with a resolution of 0.0005 inches (0.0127 mm) (such as Schaevitz Model No. 42M). For the calibration of the transducers, a micrometer with a socket-type end fitting and the transducer casing are needed. A resolution of 0.1 x 10^{-6} inches (2.54 x 10^{-6} mm) is recommended for the micrometer.

Calibration procedure for the A/C preamplifiers should be done by carefully following the steps detailed in the A/C preamplifier manual; paragraph 3-12 through 3-18. Both the LVDT's and the transducers are full bridge devices. The attenuator switch should be set--and remain--at 100. The LVDT's core should be out of the LVDT during the first part of the calibration process. Similarly, no load should be applied to the tip of the transducers.

After calibrating the A/C preamplifiers the calibrators can now be used to determine the relationship between the LVDT's and transducers output and chart reading. It is preferable to run this calibration at a rather high sensitivity (5 of 10 mV/V/FS) and then determine the calibration factors for other sensitivities by multiplying the measured calibration factor by the ratio: desired sensitivity/sensitivity at which calibration was done. (It is recommended to take several readings and use the average value.)

Calibrating the Load Cell

To calibrate the load cell the following steps are suggested:

1. Connect the load cell to one channel of the HP recorder through the triaxial cell base.
2. On the HP recorder, set the following control on the load channel:

SENSITIVITY	= OFF
FILTER	= 50 (to filter electronic noise at 50 Hz)
OPR-BAL	= OPR
ZERO SUPPRESSION POLARITY	= OFF
CAL	= 0.0
OFFSET	= 0.0
BRIDGE	= FULL
ATTENUATOR	= 1

3. With the chart speed at 1 mm/second, use the PEN POSITION control to set the chart pen to the center of the paper.
4. Set the OPR-BAL switch to BAL.
5. Adjust the C BAL and R BAL controls:
   a. Increase the SENSITIVITY until the pen just deflects off the charge paper. Turn the control back one step so the pen is back on the paper.
   b. Adjust the C BAL control for minimum pen deflection from zero.
   c. Adjust the R BAL control for minimum pen deflection from zero.

d. Repeat a, b and c until the SENSITIVITY control is on 0.1 mV/V/FS and the pen is as close to zero as possible.

e. Set the OPR-BAL switch to OPR.

f. With the SENSITIVITY still at 0.1, adjust R BAL until the pen is exactly on zero.

6. Calibrating the load cell:

a. With the SENSITIVITY control on OFF, use the PEN POSITION control to set the pen 5 mm from the right-hand edge of the chart paper.

b. Set the load cell, which is still connected to the recorder, on the floor and stack approximately 400 pounds of weights on the cell.

c. With the SENSITIVITY control on 1 m/V/V/FS, adjust the vernier control until the pen deflects 1 mm to the left for every 10 pounds of weight on the load cell.

d. To verify the accuracy of the load cell, load weights from 100 to 700 pounds by 100-pound increments on the cell and read the pen deflection on the chart, using an appropriate SENSITIVITY setting. A linear regression may be run between the known weights on the load cell and the chart pen deflection.

Calibrating the LVDT's

To calibrate the LVDT's the following steps are suggested:

1. On the HP recorder set the following controls on the LVDT channel:

SENSITIVITY	= OFF
FILTER	= 50
OPR-BAL	= OPR
ZERO SUPPRESSION POLARITY	= OFF
CAL	= 0.0

OFFSET	= 0.0
BRIDGE	= FULL
ATTENUATOR	= 1

2. With the chart speed at 1 mm/second, use the PEN POSITION control to set the chart pen to the center of the paper.
3. Set the OPR-BAL switch to BAL.
4. Preliminary adjustment of the C BAL and R BAL controls:
   a. Disconnect the LVDT's from the HP recorder.
   b. Increase the SENSITIVITY until the pen just deflects off the chart paper. Turn the control back one step so that the pen is again on the chart.
   c. Adjust the C BAL control for minimum pen deflection from zero.
   d. Adjust the R BAL control for minimum pen deflection from zero.
   e. Repeat steps b, c, and d until the SENSITIVITY control is on 0.1 mV/V/FS and the pen is close to zero as possible.
5. Connect the LVDT's to the recorder through the test base.
6. Mount the LVDT's and their cores on the Schaevitz calibration mounts (Figure A-2).
7. With the micrometer, insert the core of one LVDT into its LVDT toward the neutral point. The neutral point has been reached when, with the OPR-BAL switch on BAL, the pen has minimum deflection from the chart zero line. These micrometers have a little push-pull slop, so it is necessary to approach all measurements from a consistent direction. Find the neutral point for this LVDT by adjusting the micrometer and increasing the SENSITIVITY until it is 0.1 mV/V/FS. Note the micrometer reading at the neutral point.

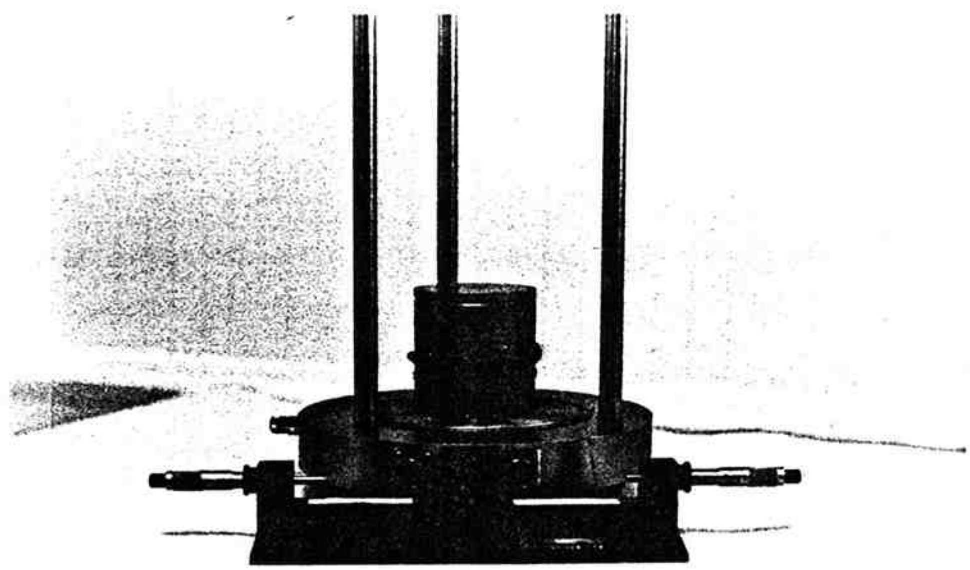

Figure A-2 - LVDT Cores Mounted in Schaevitz Micrometers for Calibration

8. Repeat step 7 for the other LVDT and its core. Remember to decrease the SENSITIVITY before beginning to insert the second LVDT core.

9. Final adjustment of the C BAL and R BAL controls:

    a. Without moving the LVDT cores from their neutral points, perform steps 4b through 4e.

    b. Set the OPR-BAL switch to OPR.

    c. With the SENSITIVITY control still at 0.1 mV, adjust R BAL until the pen is on the chart paper zero.

10. Calibrating the LVDT's:

    a. Set the SENSITIVITY control on OFF.

    b. Use the PEN POSITION control to set the pen on the left-hand edge of the chart paper.

    c. Set the SENSITIVITY control to 5 mV/V/FS.

    d. Move the LVDT cores in 0.05 inches from their neutral points.

    e. Use the VERNIER control to adjust the pend position until it is exactly on the right-hand edge of the chart paper. Thus, on the 5 mV scale, 5 centimeters equals 0.05 inches of displacement of the LVDT's.

    f. The LVDT's have now been calibrated to the HP recorder. Once calibrated, it is important to avoid switching the two LVDT cores.

11. To verify the LVDT calibration, check the pen displacement, on appropriate SENSITIVITY scales, corresponding to different core displacements from the neutral points. Do not exceed ± 0.1" from the neutral points, as this is the limit of the linear range for the LVDT's.

TRIAXIAL RESILIENT MODULUS TEST

Procedure

The following steps are suggested for sample preparation:

1. Prepare the soil at the desired water content and store in the humidity room.
2. Prepare the mold to receive the soil:
    a. Place the membrane on the test base with approximately 3/4" (1.9 cm) of the membrane extending down onto the stand.
    b. Roll the rubber O-ring up to its notch.
    c. Place about six wraps of black plastic tape around the membrane and O-ring at the top of the test base. By increasing the diameter of the membrane with the black tape, the two-piece mold will clamp more securely onto the test base.
    d. Clamp the two-piece mold around the membrane. The mold should set on top of the taped O-ring. While putting the mold on the base, try to keep wrinkles from developing in the cloth near the bottom of the mold.
    e. After the mold has been firmly clamped in place with the two C-clamps, stretch the membrane over the top of the mold and tape it in place.
    f. Place a fillet of vacuum grease in the crack between the mold and the taped O-ring around the top of the test base. A small amount of vacuum grease may be required where the two mold halves contact.
    g. Place the three wooden blocks under the mold (Fig. A-3). This insures the proper height and level for the mold.

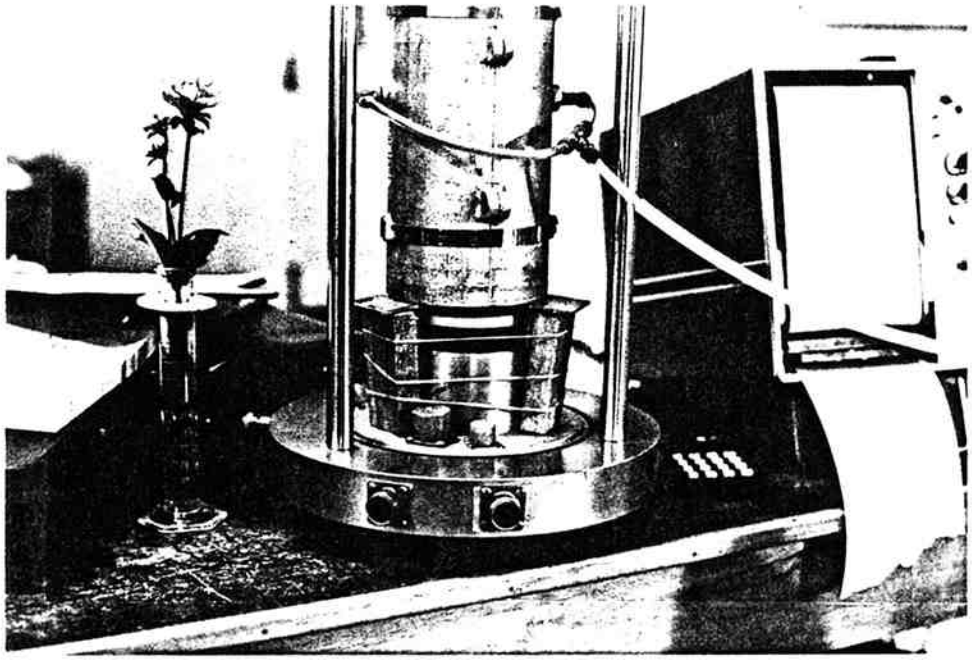

Figure A-3 - Wood Blocks in Place, Insuring Proper Height and
Level of Sample

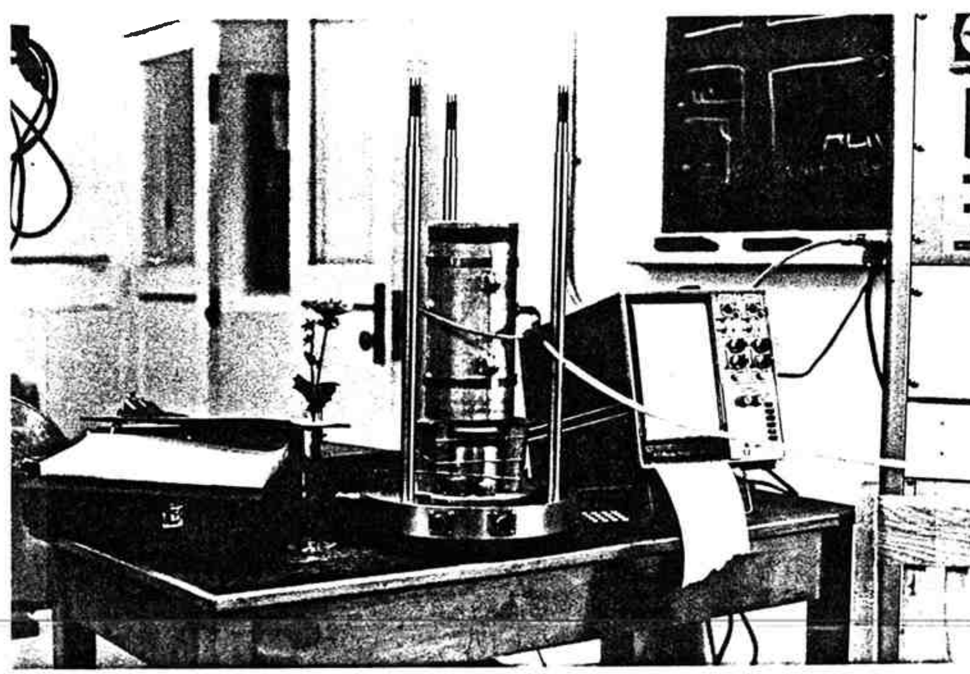

Figure A-4 - Assembled Mold with Two Lifts of Compacted Soil

h. Apply a vacuum to the mold and check to make sure the membrane is pulled out against the mold.

i. Drop two or three filter papers down into the mold to cover the vacuum hold in the center of the test base.

3. Compact soil into mold:

a. Remove enough soil from the humidity room to perform the test.

b. Weight a sample of the soil (200-500 g) to use for a moisture determination.

c. Weight seven equal batches of the soil such that each batch has the necessary weight to provide the required dry density when compacted into a 1.43-inch (3.62 cm) lift, i.e.:

$$\text{weight/batch} = (\gamma_d)\ (1 + w)\ (\pi r^2(H/7"))$$

where γ_d = dry density of soil

w = water content

r = radius of test base ($\simeq$ 2")

H = height of the sample

d. With a scale, measure the distance from the top of the test base to the top of the mold ($\simeq$ 10-1/8").

e. Carefully place the first batch of soil into the mold.

f. Use a 5.5-pound (2.5 kg) hammer to compact the soil into the mold and measure the thickness of the layer this compactive effort produces (Fig. A-4). Continue compacting the soil until a 1.43-inch (3.62 cm) layer is achieved.

g. Compact the next five lifts into the mold. Do not, however, rely on the compactive effort used to achieve the 1.42-inch (3.63 cms) thickness in the first layer. Use less time than the first layer required and measure the accumulated soil column height. Then use more blows until the required thickness is achieved.

h. Tape the extension collar on top of the mold (Fig. A-5).

i. Compact the final layer of soil into the mold until it is just below the top of the two-piece mold.

j. Remove the extension collar.

k. Use the finishing plate and one or two more blows of the impact hammer to finish the surface of the soil.

l. Place the load cell on top of the soil, remove the tape holding down the rubber membrane, pull the membrane up around the load cell, and roll the rubber O-ring down into its notch on the load cell.

m. Switch the vacuum from.the mold to the base.

n. Remove the three wooden blocks, remove the two C-clamps, and use a screwdriver in the filed notch on the mold to separate the mold halves (Fig. A-6). Remove the mold.

o. Wipe the vacuum grease off the tape over the O-ring.

4. Mounting the LVDT's

a. Apply a vacuum to the soil.

b. Place the LVDT clamp system about three inches up from the bottom of the sample. Use a rubber band to hold the clamp in place.

c. Connect the cable from the LVDT's to the test base. Connect the cable from the HP recorder to the test base. Set the ZERO SUPPRESSION POLARITY switch to OFF, the OFFSET control to 10.0, and the CAL control to 0.00.

d. Put one of the LVDT's into its hole in the LVDT clamp so the core is near the middle of the LVDT. With the recorder SENSITIVITY control

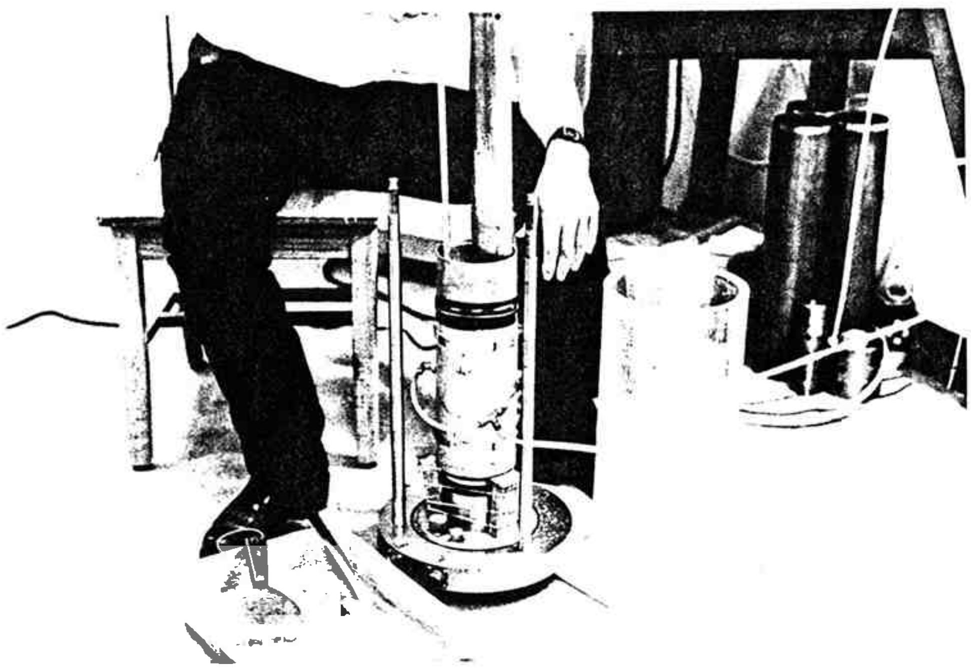

Figure A-5 - Extension Collar Attached for a Final Lift Soil

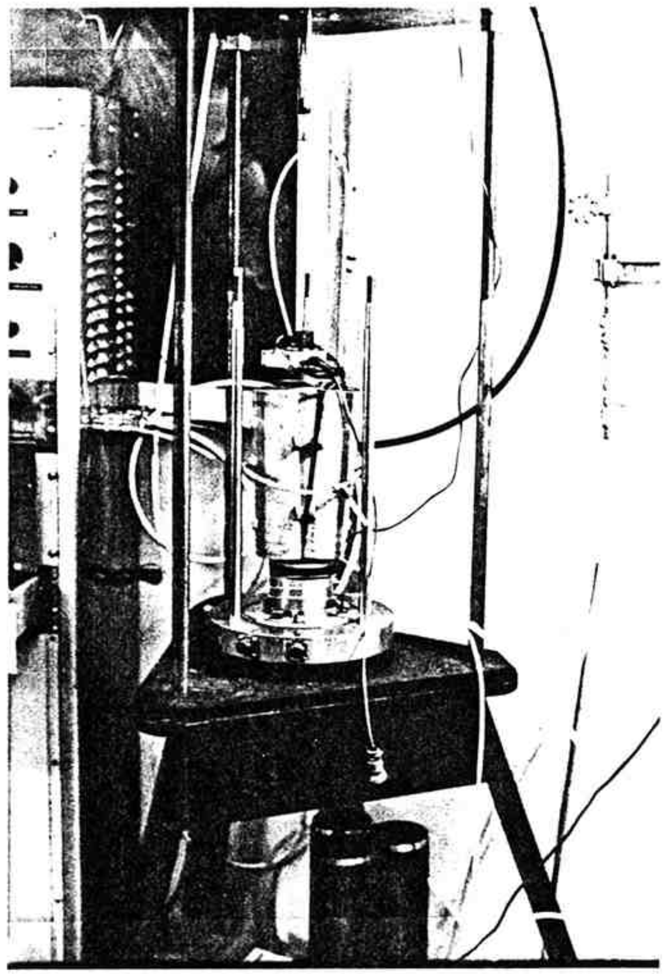

Figure A-6 - Separation of the Split Mold on OFF, zero the pen on the center of the chart using the PEN POSITION control. Using the 1 mm/set chart speed and gradually increasing the SENSITIVITY control, move the LVDT so it is 0.01" below its neutral point. This will correspond to a chart pen deflection of 12.5 mm to the left of center on the 2 mV/V/FS SENSITIVITY control setting, since only one LVDT is being used. Firmly secure the LVDT in its proper position, making sure the LVDT is not twisted and thus hindering the free movement of the core.

e. Repeat the last step for the remaining LVDT. Now the total chart pen deflection should be 25 mm to the left of zero on the 2 mV/V/FS SENSITIVITY setting. This corresponds to a distance of 0.01" from the neutral point, since both LVDT's are being used.

DIAMETRAL RESILIENT MODULUS TEST

Test Apparatus

1. Loading system capable of testing with a load pulse over a range of frequencies from 0.5 to 7 Hz and a duration from 0.02 to 1.0 sec (see Figure 2.8).
2. Strain set 1000-pound capacity load cell.
3. Displacement transducing cells, Statham Instrument Model UC 3 connected with yoke.
4. Load recorder, HP Strip-Chart Recorder, Model No. 7402, with Bridege Amp Model 17404 A.
5. Horizontal deformation recorder, HP Strip-Chart Recorder Model 7402 A with carrier preamp Model No. 17403 A.

6. Vertical displacement transducing gage, Schaevitz GCA-121-250 gage head LVDT.
7. Vertical deformation recorder, HP Strip-Chart Recorder Model No. 7402, with Bridege Amp Model 17404 A.
8. Two aluminum plates, two teflon sheets and a rubber membrane (as shown in Figure A-7).
9. Split mold as shown in Figure A-8, and
10. Two 1/2 by 1/2-inch, 3-inch long steel loading strips.

Test Procedure

The following steps are suggested for sample preparation:

1. Prepare the soil at the desired water content and store in the humidity room.
2. Compact soil into mold. Use the same compacting procedure than the one used for the triaxial specimen. When compacting the diametral specimen only two 3.18 cm (1.25-inch) lifts are used.
3. Place the rubber membrane on the split mold.
4. Apply a vacuum to the mold and check to make sure the membrane is pulled out against the mold.
5. Transfer the specimen to the split mold with the rubber membrane and release the vacuum.
6. Place the teflon sheets and the aluminum plates.
7. The specimen enclosed between two aluminum plates, two teflon sheets and the rubber membrane is placed in the yoke and connected with two horizontal displacement transducers.
8. The specimen and the yoke is placed on the loading plate with two loading strips on opposite sides along the vertical direction.

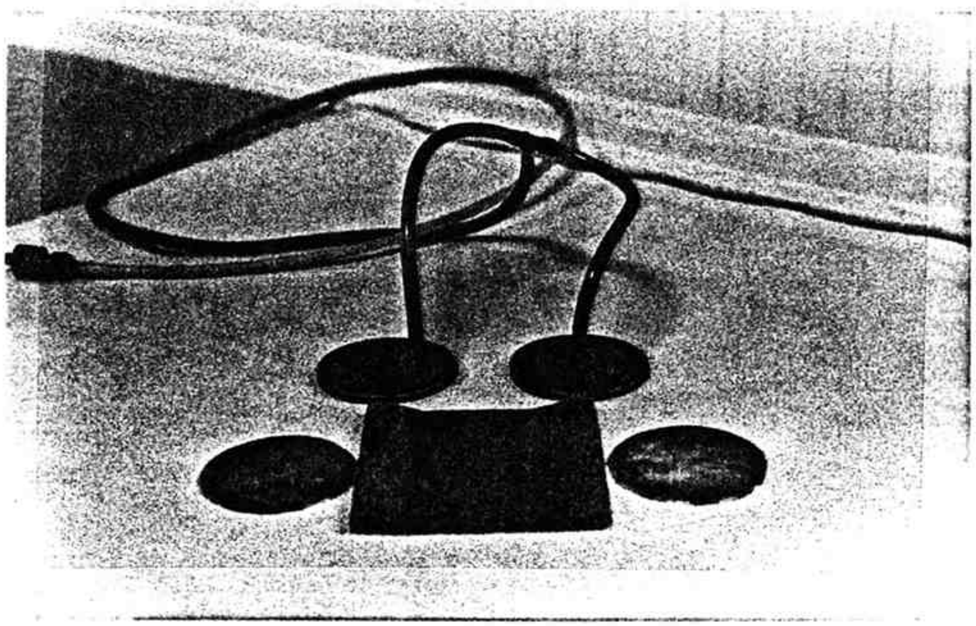

Figure A-7 - Membrane, Aluminum Plates and Teflon Sheets used in Diametral Resilient Modulus Test

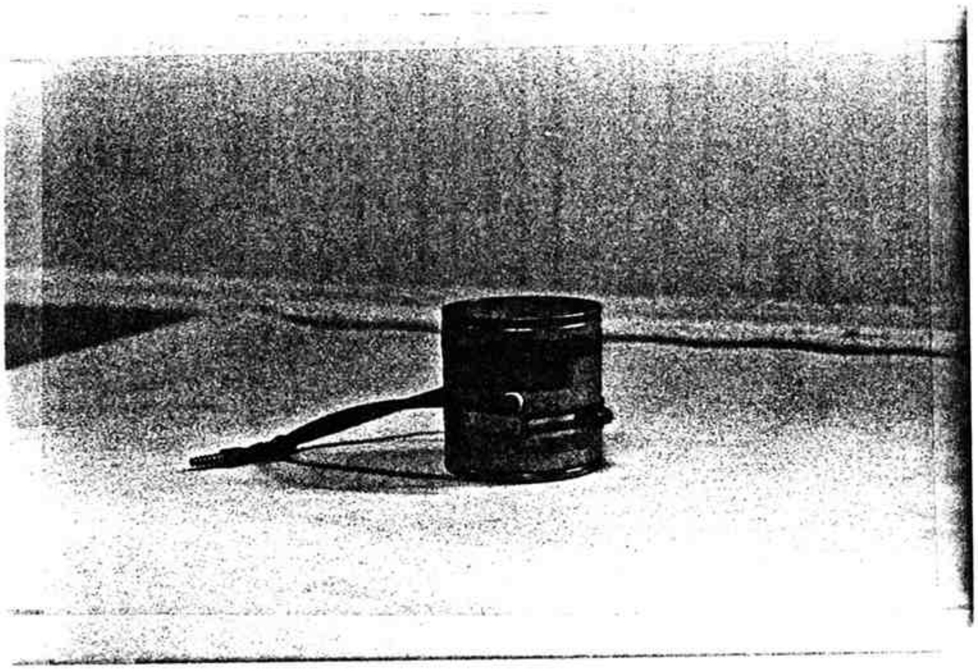

Figure A-8 - Split Mold used in Diametral Resilient Modulus Test

9. The vertical displacement transducer (gage head LVDT) is connected.
10. A seating load of 10% of the diametral load is applied.
11. The transducers are adjusted and attached to the specimen.
12. A vacuum is applied to bring the specimen to the desired confining pressure.
13. A load duration of 0.1 sec at a rate of 30 repetitions per minute is applied.
14. A preconditioning procedure of the specimen is done as stated in the triaxial procedure.
15. After the conditioning of the specimen is completed, the applied load, horizontal and vertical deformations are recorded.
16. For unbound material the resilient modulus is computed using the Eqs. (2.2) and (2.3).
17. For bound material the vertical deformation is not required. A value of the Poisson's ratio is assumed and the resilient modulus is calculated using Eq. (2.3). Use a Poisson's ratio of 0.35 for asphalt concrete, 0.20 for cement-treated base, and 0.22 for cement-modified soil.

Example Calculation

Triaxial Resilient Modulus

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

Load Cell Output = 7.3 mm. at sensitivity .2.

Vertical deformation LVDT = 5.1 mm at sensitivity .2.

Calibration of Load Cell for sensitivity .2 is 1.778 lbs/mm.

Load = 7.3 x 1.778 = 12.98 lbs.

Area of specimen - 12.98 in^2.

σ_d = Deviator stress = 12.98/12.98 = 1 psi.

Calibration of vertical LVDT for sensitivity .2 is 1.950 x 10^{-6} in/mm.

Vertical deformation = 5.1 x 1.950 x 10^{-6} =

= 994.50 x 10^{-6} in.

$$\varepsilon_a = \text{axial strain} = \frac{994.50 \times 10^{-6}}{4} = 248.63 \times 10^{-6} \text{ in./in.}$$

Then,

$$M_R = \frac{1.0}{248.63 \times 10^{-6}} = \underline{4022 \text{ psi}}$$

Diametral Resilient Modulus

Asphalt Concrete Modulus

ν = 0.35

$$M_R = \frac{P}{H_{RI} \times t} (.2692 + .9974 \times 0.35) =$$

$$M_R = \frac{0.6183 \; P}{H_{RI} \times t}$$

Load cell = 7.7 mm at a sens. .2 calibration of load call at .2 sens. = 20.461 lb/mm.

Load = 7.7 x 20.461 = 157.55 lbs.

Horizontal transducer 1 = 8.0 mm at a sens. 1

Horizontal transducer 2 = 6.7 mm at a sens. 1

Calibration at sens. 1 of the transducer 1 and 2 are 6.445 x 10^{-6} in/mm and 6.885 x 10^{-6} in/mm.

Deformation (H_{RI}) = 8.0 x 6.445 x 10^{-6} + 6.7 x 6.885 x 10^{-6} = 97.690 x 10^{-6} in.

t = thickness = 5.84 cm. = 2.3 in.

$$M_R = \frac{0.6183\ (157.55)}{97.690 \times 10^{-6} \times 2.3} = \underline{433.550\ \text{psi}}$$

Cement-Treated Base (ν =0.20)

$$M_R = \frac{0.4687\ P}{H_{RI} \times t}$$

Load cell = 14.7 mm at a sensitivity .1

Calibration of load cell at a sensitivity .1 = 10.23 lbs/mm.

Load = 10.23 x 14.7 = 150.4 lbs.

Horizontal transducer 1 = 6.6 mm at a sensitivity .2

Horizontal transducer 2 = 5.3 mm at a sensitivity .2

Calibration of transducers for a sensitivity .2 are 1.289 x 10^{-6} in/mm and 1.377 x 10^{-6} in/mm.

Deformation (H_{RI}) = 6.6 mm x 1.289 x 10^{-6} in/mm +

+ 5.3 mm x 1.377 x 10^{-6} in/mm =

= $\underline{15.806 \times 10^{-6}\ \text{in}}$

t = thickness = 6.24 cm = 2.46 in.

$$M_R = \frac{0.4687 \times 150.4}{15.806 \times 10^{-6} \times 2.46} = \underline{1{,}813{,}000\ \text{psi}}$$

Unbound Material

$$\nu_{RI} = \frac{DR\ (0.0673) - .8954}{DR\ (-.2494) - .0156}$$

$$M_R = \frac{P}{H_{RI} \times t}\ (0.2692 + .9974\ \nu_{RI})$$

$$DR = \frac{V_{RT}}{H_{RT}}$$

Load Cell = 9.63 mm at a sensitivity of .5

- Calibration of load cell at a sensitivity of .5 = 4.384 lbs/mm.

Load = 4.384 x 9.63 = <u>42.22 lbs</u>

Horizontal transducer 1 = 17.6 mm at a sensitivity

Horizontal transducer 2 = 23.4 mm of .2

Calibration of horizontal transducers at a sensitivity of .2 are 1.656×10^{-6} in/mm and 1.597×10^{-6} in/mm.

H_{RI} = horizontal deformation =

$$= 1.656 \times 10^{-6} \times 17.6 + 1.597 \times 10^{-6} \times 23.4 =$$

$$= \underline{665.15 \times 10^{-6} \text{ in.}}$$

Vertical LVDT = 10.0 mm. at a sensitivity of .5.

Calibration of vertical LVDT at a sensitivity of .5 = 6.31×10^{-4} in/mm.

V_{RI} = Vertical deformation = $6.31 \times 10^{-4} \times 10 =$

$$= 6310.0 \times 10^{-6} \text{ in.}$$

t = thickness = 6.4 cm = 2.52 in.

$$DR = \frac{6310.0 \times 10^{-6}}{665.15 \times 10^{-6}} = 9.487$$

$$\nu_{RI} = \frac{9.487\ (0.0673) - .8954}{9.487\ (-.2494) - .0156} = 0.108$$

$$M_R = \frac{42.22\ (0.2962 + (.9974 \times 0.108))}{665.15 \times 10^{-6} \times 1.52} =$$

$$= \underline{9{,}490 \text{ psi}}$$

APPENDIX B

APPENDIX B

SUMMARY OF RESILIENT MODULUS TESTING RESULTS

This appendix contains tables and figures of resilient modulus test results for subgrade and base course materials from both projects.

Table B-1

Triaxial Test Results
Base Material
U.S.-97 Project (Volcanic Cinders)
95% Compaction, 9% Water Content

Confining Pressure, σ_3 (psi)	Deviator Stress, σ_d (psi)	Principal Stress (psi)	Resilient Modulus γ = 95 pcf W_c = 9% (psi)	Average Resilient Modulus (psi)	Resilient Modulus γ = 100 pcf W_c = 9% (psi)	Average Resilient Modulus (psi)
2	1.0	7	5,700	6,160	8,550	8,618
2	2.0	8	5,860		8,370	
2	3.0	9	6,410		8,430	
2	4.0	10	6,670		9,120	
4	2.0	14	9,540	10,743	18,650	15,050
4	4.0	16	10,520		13,910	
4	6.0	18	11,190		13,380	
4	8.0	20	11,720		14,520	
6	3.0	21	12,310	13,200	19,850	17,688
6	6.0	24	12,190		16,910	
6	9.0	27	13,990		16,480	
6	12.0	30	14,310		18,230	
8	4.0	28	15,780	18,072	27,350	24,660
8	8.0	32	18,030		22,480	
8	12.0	36	18,940		24,130	
8	16.0	40	19,540		24,680	

1 psi = 6.9 kN/m^2

1 kN/m^3 = 6.369 pcf

Tests were performed at field water content.

Table B-2

Triaxial Resilient Modulus Test Results
Salem Parkway Project
*Subgrade Soil 1
95% Compaction, 25% Water Content

Confining Pressure, σ_3 (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.5	1.0	10,796	9,500
2	2.0	2.0	11,088	
2	2.5	3.0	9,050	
2	3.0	4.0	8,373	
4	1.5	2.0	12.432	8,500
4	2.0	4.0	8,373	
4	2.5	6.0	7,155	
4	3.0	8.0	6,361	
6	1.5	3.0	10,796	7,600
6	2.0	6.0	7,597	
6	2.5	9.0	6,237	
6	3.0	12.0	5,903	

No diametral test in this material

*Subgrade 1 = Clayey soil

Table B-3

Triaxial and Diametral Resilient Modulus Test Results
Salem Parkway Project
Subgrade Soil 2
95% Compaction, 14% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.5	1.0	12,320			
2	2.0	2.0	10,130		8,740	
2	2.5	3.0	9,660	10,360	8,570	9,100
2	3.0	4.0	9,340		9,350	
2	3.5	5.0			9,760	
4	1.5	2.0	12,800		9,430	
4	2.0	4.0	10,800		10,600	
4	2.5	6.0	9,840	10,850	11,100	11,230
4	3.0	8.0	9,950		13,800	
6	1.5	3.0	13,600		10,900	
6	2.0	6.0	11,100	12,000	12,300	13,280
6	2.5	9.0	11,400		15,200	
6	3.0	12.0	11,900		14,700	

1 psi = 6.9 kN/m^2

Table B-4

Triaxial and Diametral Resilient Modulus Test Results
Salem Parkway Project
Subgrade Soil 2
100% Compaction, 14% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.5	1.0	14,000			
2	2.0	2.0	12,400		10,250	
2	2.5	3.0	11,500	12,250	9,100	10,300
2	3.0	4.0	11,100		10,000	
2	3.5	5.0			11,250	
4	1.5	2.0	16,700		11,350	
4	2.0	4.0	13,500		11,290	
4	2.5	6.0	12,000	13,650	12,190	11,905
4	3.0	8.0	12,400		12,790	
6	1.5	3.0	17,400		13,120	
6	2.0	6.0	13,900		12,860	
6	2.5	9.0	11,400	14,900	14,120	13,435
6	3.0	12.0	14,500		13,640	

1 psi = 6.9 kN/m^2

Table B-5

Triaxial and Diametral Resilient Modulus Test Results
Salem Parkway Project
*Subgrade Soil 2
95% Compaction, 18% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.5	1.0	5,430	4,560	7,420	8,250
2	2.0	2.0	4,420		8,150	
2	2.5	3.0	4,110		8,740	
2	3.0	4.0	4,280		8,690	
4	1.5	2.0	7,030	6,490	8,930	10,000
4	2.0	4.0	6,020		10,560	
4	2.5	6.0	6,240		10,090	
4	3.0	8.0	6,660		10,410	
6	1.5	3.0	9,560	8,860	9,380	10,740
6	2.0	6.0	8,550		10,480	
6	2.5	9.0	8,430		12,020	
6	3.0	12.0	8,900		11,090	

1 psi = 6.9 kN/m^2

*Subgrade 2 = Silty sand

Table B-6

Triaxial and Diametral Resilient Modulus Test Results
Salem Parkway Project
Subgrade Soil 2
100% Compaction, 18% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.5	1.0	7,730			
2	2.0	2.0	6,640		5,290	
2	2.5	3.0	5,940	6,520	7,000	6,810
2	3.0	4.0	5,770		7,430	
2	3.5	5.0			7,520	
4	1.5	2.0	9,530		5,810	
4	2.0	4.0	7,580		8,060	
4	2.0	6.0	7,020	7,900	9,020	8,142
4	3.0	8.0	7,480		9,680	
6	1.5	3.0	11,290		8,490	
6	2.0	6.0	8,980		8,490	
6	2.5	9.0	8,790	9,590	12,300	10,620
6	3.0	12.0	9,310		11,700	

1 psi = 6.9 kN/m^2

Table B-7

Triaxial and Diametral Resilient Modulus Test Results
U.S.-97 Project
Subgrade Soil
95% Compaction, 40% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Deviator Stress (psi)	Principal Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.0	7.0	3,540			
2	2.0	8.0	3,540		3,740	
2	3.0	9.0	3,600	3,630	4,310	4,440
2	4.0	10.0	3,850		4,690	
2	5.0	11.0			5,010	
4	2.0	14.0	5,260		4,390	
4	4.0	16.0	5,400		5,330	
4	6.0	18.0	5,810	5,745	5,980	5,580
4	8.0	20.0	6,510		6,620	
6	3.0	21.0	6,410		5,040	
6	6.0	24.0	7,110		6,410	
6	9.0	27.0	8,130	7,610	7,540	6,790
6	12.0	30.0	8,790		8,160	
8	4.0	28.0	8,640		5,780	
8	8.0	32.0	9,120		7,450	
8	12.0	36.0	10,750	9,890	8,710	7,830
8	16.0	40.0	11,050		9,390	

1 psi = 6.9 kN/m^2

Table B-8

Triaxial and Diametral Resilient Modulus Test Results
U.S.-97 Project
Subgrade Soil
100% Compaction, 40% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Deviator Stress (psi)	Principal Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.0	7.0	3,660			
2	2.0	8.0	3,830		3,740	
2	3.0	9.0	3,940	3,880	5,070	5,380
2	4.0	10.0	4,100		5,760	
2	5.0	11.0			6,220	
4	2.0	14.0	5,960		5,250	
4	4.0	16.0	6,080		6,500	
4	6.0	18.0	6,620	6,450	7,220	6,750
4	8.0	20.0	7,130		8,020	
6	3.0	21.0	7,600		6,500	
6	6.0	24.0	7,690		8,270	
6	9.0	27.0	8,710	8,410	9,810	9,630
6	12.0	30.0	9,650		9,940	
8	4.0	28.0	10,390		7,660	
8	8.0	32.0	11,010		9,350	
8	12.0	36.0	12,010	11,380	10,780	9,880
8	16.0	40.0	12,110		11,720	

1 psi = 6.9 kN/m^2

Table B-9

Triaxial and Diametral Resilient Modulus Test Results
U.S.-97 Project
Subgrade Soil
95% Compaction, 60% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Deviator Stress (psi)	Principal Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.0	7.0	3,580			
2	2.0	8.0	3,700		3,920	
2	3.0	9.0	4,030	3,880	4,570	4,575
2	4.0	10.0	4,200		4,660	
2	5.0	11.0			5,150	
4	2.0	14.0	5,700		4,420	
4	4.0	16.0	5,890		5,860	
4	6.0	18.0	6,330	6,260	5,880	5,710
4	8.0	20.0	7,130		6,690	
6	3.0	21.0	6,800		4,930	
6	6.0	24.0	7,420		6,510	
6	9.0	27.0	8,480	7,965	7,870	6,920
6	12.0	30.0	9,160		8,370	
8	4.0	28.0	9,450		5,340	
8	8.0	32.0	10,320		7,540	
8	12.0	36.0	11,240	10,680	8,920	7,920
8	16.0	40.0	11,720		9,870	

1 psi = 6.9 kN/m^2

Table B-11

Triaxial and Diametral Resilient Modulus Test Results
U.S.-97 Project
Subgrade Soil
95% Compaction, 80% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Ratio σ_1/σ_3	Deviator Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.0	7.0	1,170			
2	2.0	8.0	1,490		3,610	
2	3.0	9.0	1,820	1,650	3,920	3,030
2	4.0	10.0	2,100		4,570	
2	5.0	11.0			4,910	
4	2.0	14.0	2,380		3,970	
4	4.0	16.0	3,010		5,290	
4	6.0	18.0	3,730	3,440	6,000	5,530
4	8.0	20.0	4,650		6,840	
6	3.0	21.0	3,620		5,170	
6	6.0	24.0	4,550		6,600	
6	9.0	27.0	5,350	4,920	7,750	7,030
6	12.0	30.0	6,150		8,580	
8	4.0	28	6,190		6,100	
8	8.0	32	7,050		8,000	
8	12.0	36.0	7,680	7,360	9,200	8,270
8	16.0	40.0	8,520		9,790	

$\theta = \sigma_1 + 2\sigma_3$

1 psi = 6.9 kN/m^2

Table B-12

Triaxial and Diametral Resilient Modulus Test Results
U.S.-97 Project
Subgrade Soil
100% Compaction, 80% Water Content

			Triaxial Results		Diametral Results	
Confining Pressure (psi)	Deviator Stress (psi)	Principal Stress (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)	Resilient Modulus (psi)	Average Resilient Modulus (psi)
2	1.0	7.0	3,420			
2	2.0	8.0	3,180		3,060	
2	3.0	9.0	3,120	4,255	3,580	3,630
2	4.0	10.0	3,300		3,810	
2	5.0	11.0			4,080	
4	2.0	14.0	4,660		3,410	
4	4.0	16.0	4,910		4,220	
4	6.0	18.0	5,570	5,260	4,830	4,485
4	8.0	20.0	5,900		5,480	
6	3.0	21.0	5,720		4,110	
6	6.0	24.0	5,930		4,970	
6	9.0	27.0	6,590	6,420	5,700	5,120
6	12.0	30.0	7,450		5,700	
8	4.0	28.0	8,210		4,150	
8	8.0	32.0	9,120		5,490	
8	12.0	36.0	9,470	9,140	6,690	6,030
8	16.0	40.0	9,770		7,780	

1 psi = 6.9 kN/m^2

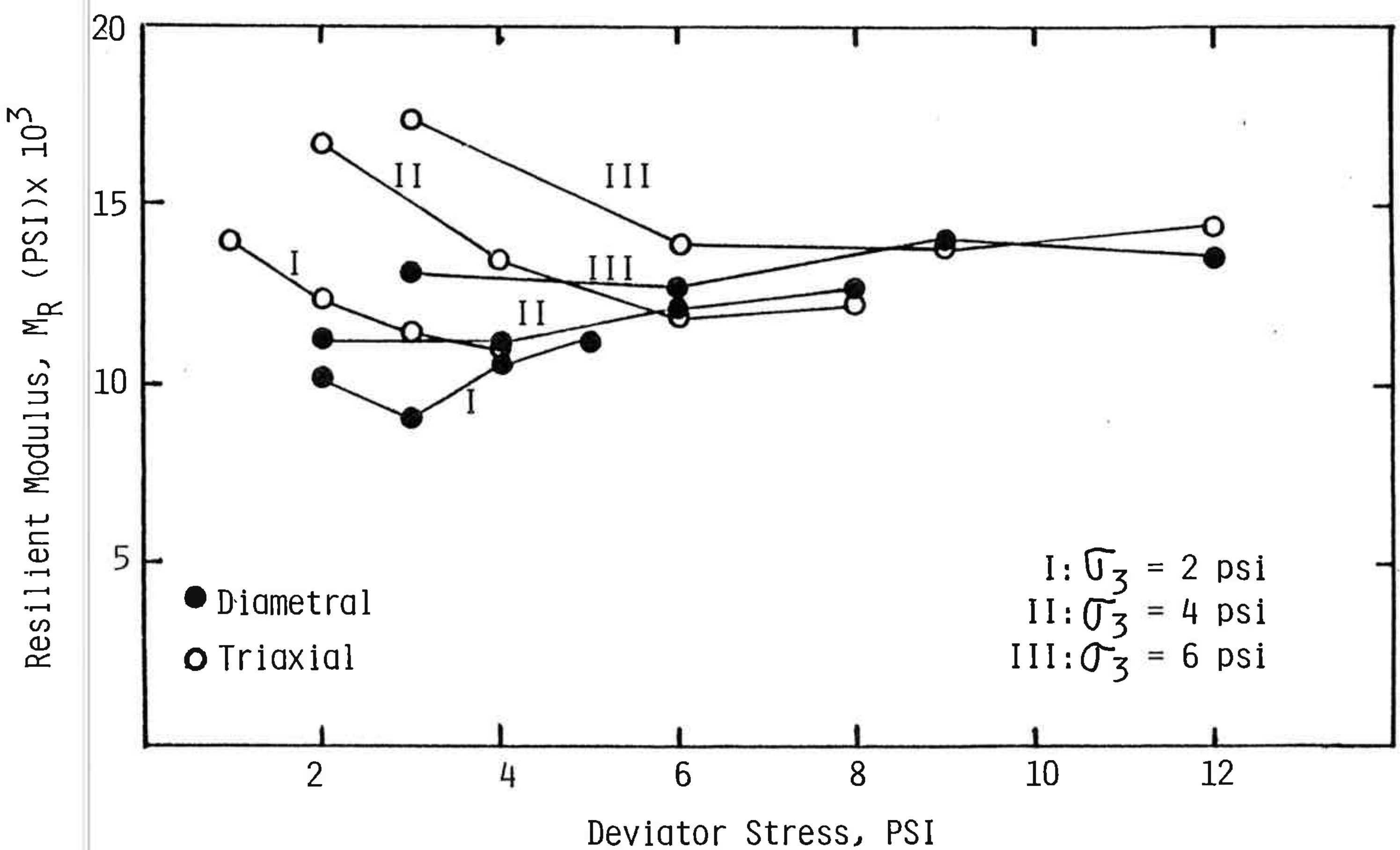

Figure B-1 - Comparison of Triaxial and Diametral Resilient Modulus Results, Salem Parkway Project, Subgrade 2, 100% Compaction, 14% Water Content

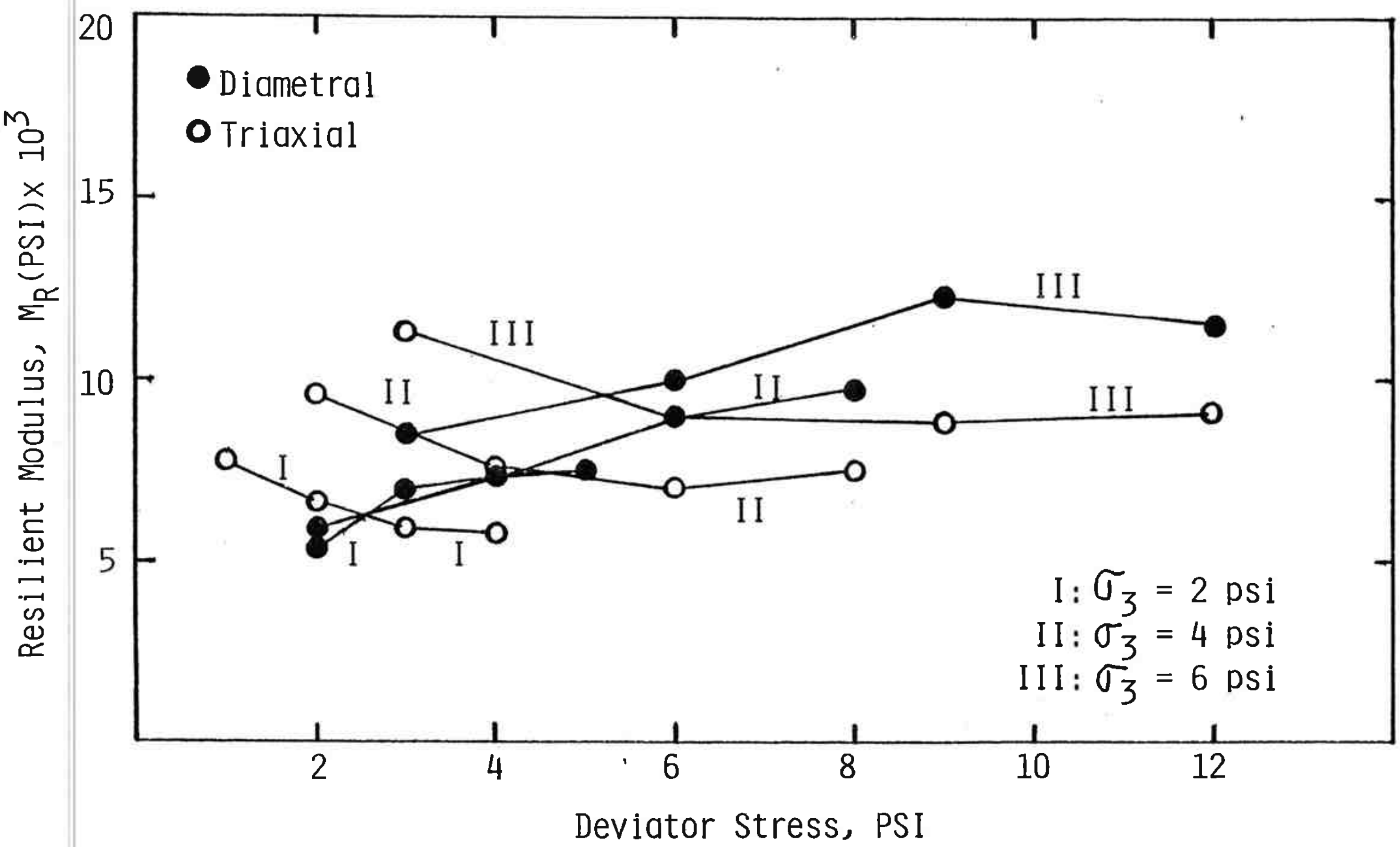

Figure B-2 - Comparison of Triaxial and Diametral Resilient Modulus Results, Salem Parkway Project, Subgrade 2, 100% Compaction, 18% Water Content

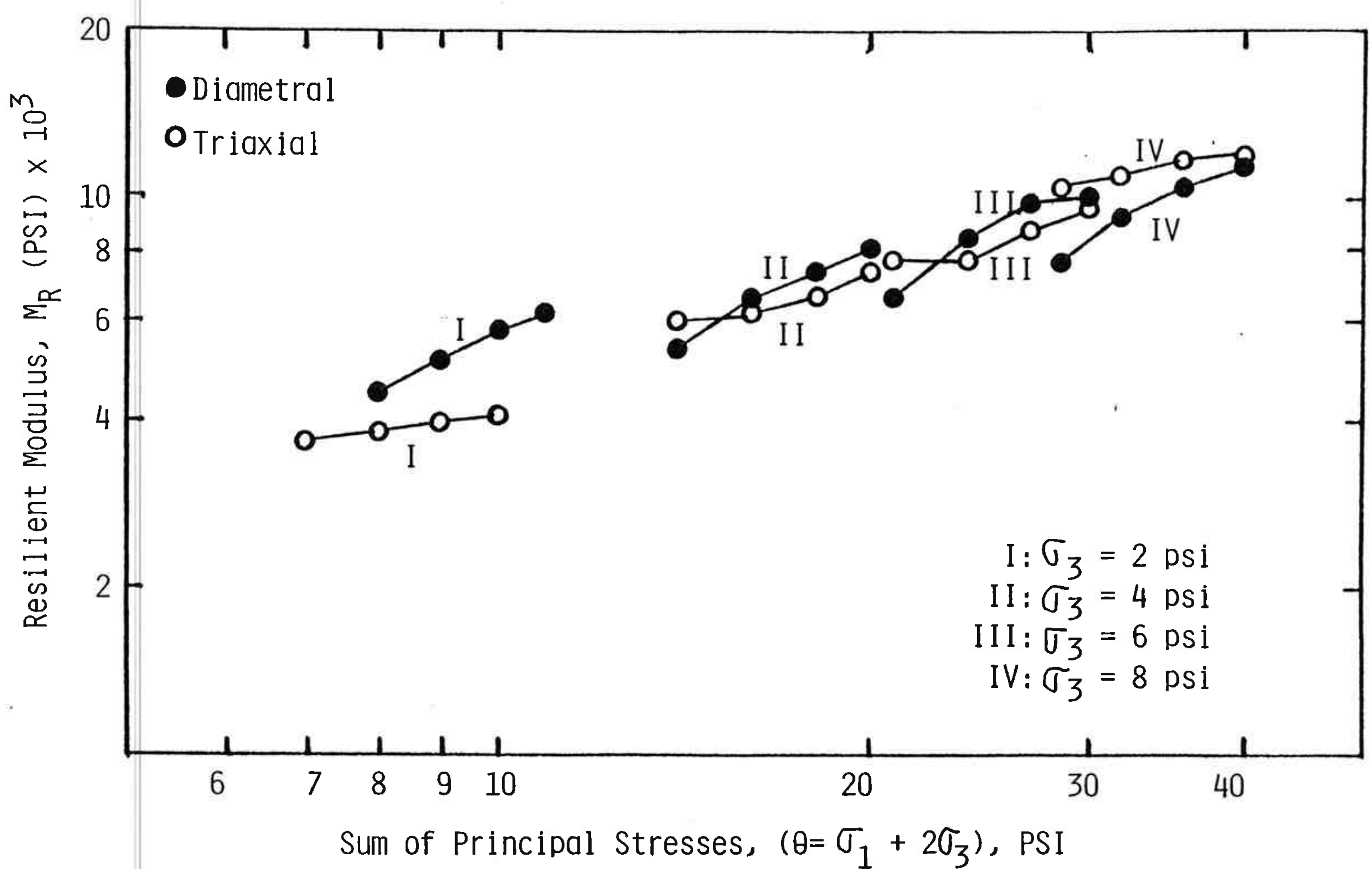

Figure B-3 - Comparison of Triaxial and Diametral Resilient Modulus
US-97 Project, Subgrade Soil, 100% Compaction, 40% Water Content

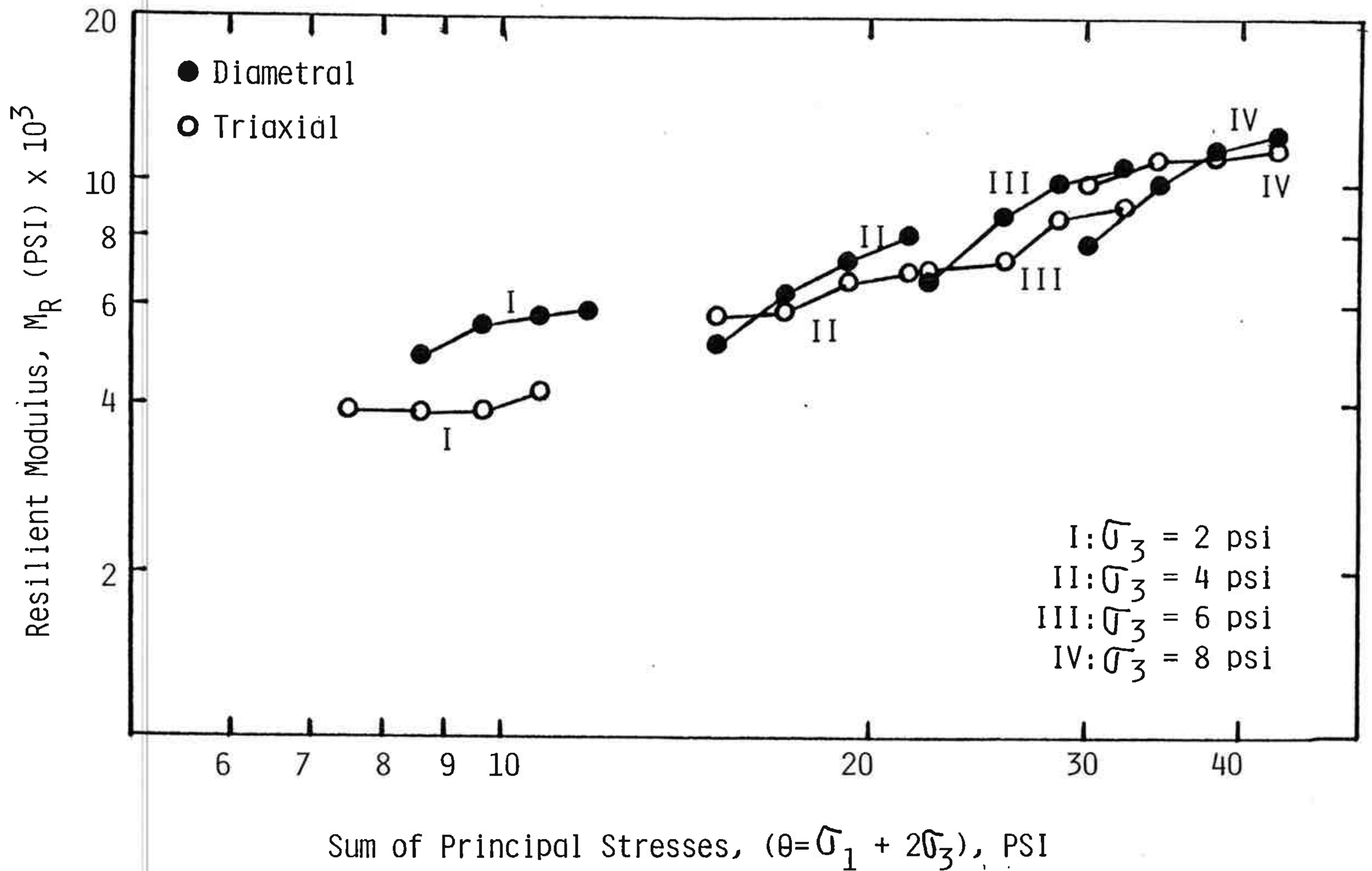

Figure B-4 - Comparison of Triaxial and Diametral Resilient Modulus Results, US-97 Project, Subgrade Soil, 100% Compaction, 60% Water Content

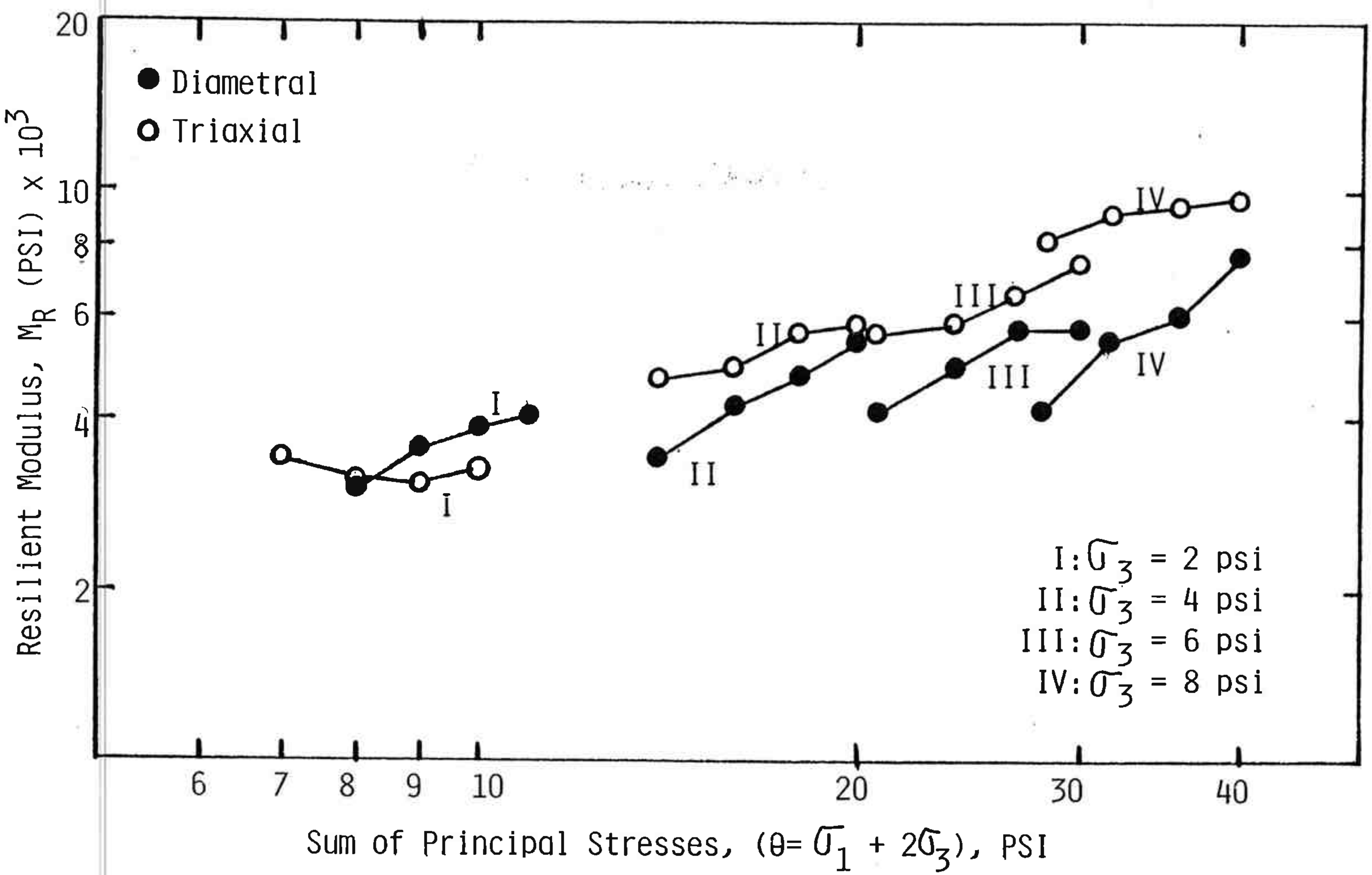

Figure B-5 - Comparison of Triaxial and Diametral Resilient Modulus Results, US-97 Project, Subgrade Soil, 100% Compaction, 80% Water Content

APPLICATION OF RESILIENT MODULUS TEST EQUIPMENT AND PROCEDURES FOR SUBGRADE SOILS

Part 2

APPLICATION OF RESULTS IN PAVEMENT ANALYSES AND DESIGNS

FINAL REPORT

HP&R Study: 083-5158

by

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15. Supplementary Notes

16. Abstract This report is the second part of a two-part series. Part 1 describes the techniques involved in, and the results from, resilient modulus testing of subgrade soils that are typically found in Oregon. Two methods of testing were investigated, the triaxial and diametral repeated load procedures. Subgrade soils obtained from two projects were tested. One project was a new alignment construction project in the Willamette Valley and the other an overlay project in Central Oregon, east of the Cascades. It was found that the diametral procedure was adequate for use with cohesive soils, typical of those occurring in the Willamette Valley, but that it was not suitable for use with the noncohesive volcanic soils occurring east of the Cascades.

This second part of the report (Part 2) presents procedures for analysis and design of flexible pavements. It is shown how the results of the materials testing, reported in Part 1, can be used in conjunction with field deflection measurements to evaluate pavements. It is also shown how the current procedures used for the design of new pavements and overlays in Oregon, can be supplemented by analytically based procedures. Methods of evaluating and analyzing pavement structures using layered elastic theory are presented. Both "exact" and "approximate" procedures are considered, and their use in mechanistic pavement design procedures is presented.

Recommendations for the implementation of mechanistic methods of analysis and design of pavements are presented. To be fully effective for pavement evaluation, these methods need accurate deflection and materials data. It is strongly recommended that deflection basins, rather than just maximum deflections, should be measured, and that more information on cement-treated and volcanic materials should be obtained.

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ABSTRACT

This report is the second part of a two part series. Part 1 describes the techniques involved in, and the results from, resilient modulus testing of subgrade soils that are typically found in Oregon. Two methods of testing were investigated, the triaxial and diametral repeated load procedures. Subgrade soils obtained from two projects were tested. One project was a new alignment construction project in the Willamette Valley and the other an overlay project in Central Oregon, east of the Cascades. It was found that the diametral procedure was adequate for use with cohesive soils, typical of those occurring in the Willamette Valley, but that it was not suitable for use with the noncohesive volcanic soils occurring east of the Cascades.

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1.0 INTRODUCTION

The subgrade soils from two paving projects in Oregon were tested to determine their resilient moduli by triaxial and diametral repeated load procedures. Part 1 of this report describes the test procedures, and presents the results and recommendations for use of the most appropriate test procedure. In addition to testing of the subgrade soils, all the other materials occurring in the two pavements were tested to determine their resilient moduli, such that each pavement could be analyzed by layered elastic procedures. One project was a new alignment construction project situated in Salem, Oregon and the other was an overlay project situated on U.S.-97, in Central Oregon, east of the Cascades. The results of Benkelman beam deflection measurements were also available, and were used in addition to the laboratory-determined resilient moduli in the analysis of the pavements and design calculations.

The remainder of this chapter summarizes the information that is presented in Part 1 of this report, and presents the deflection data. Subsequent chapters present the results of analyses and designs and present recommendations for implementation of the procedures used.

1.1 Project Descriptions

1.1.1 The Salem Parkway Project

The Salem Parkway Project was a new construction job in the Willamette Valley. The project is approximately three and a quarter miles long, and passes over two distinct subgrade soils, AASHTO classifications A-7-6 and A-4. In future sections these are referred to as subgrade soils 1 and 2, respectively. The current as-built cross section for this project is shown in Figure 1.1, and it should be noted that the same cross section was used for each subgrade soil. The project was completed in the fall of 1982.

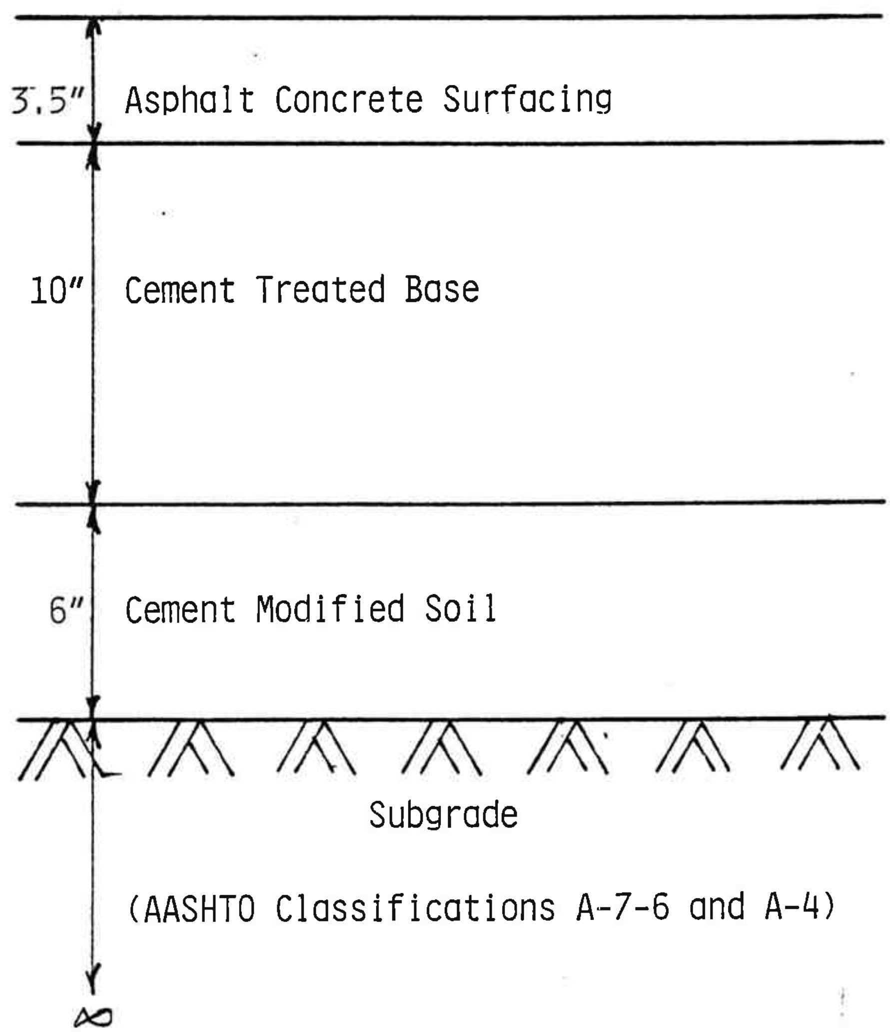


Figure 1.1 - Pavement Cross-Section, Salem Parkway Project

It should be noted that stage construction was planned for this project. At the design stage it was planned that the asphalt concrete should be applied in two 2-inch lifts. To date (July, 1983), the second lift has not been applied, and the first lift was increased to 3.5 inches following some problems with the cement-treated base (CTB) during construction. Hence a total thickness of 5.5 inches of asphalt concrete will occur eventually.

1.1.2 The U.S.-97 Project (Hackett Drive to Crescent, The Dalles to California Highway)

This project was an overlay project, approximately seven miles long, located in Central Oregon, east of the Cascades and south of Bend. It will be referred to only as the U.S.-97 project. The subgrade for this project is a pumiceous material, AASHTO classification A-1-b. This section of U.S.-97 was originally built in 1942, apparently with a fairly thin asphalt surface (approximately 2 inches) and a 12-inch cinder base. Examination of cores from this project indicate that subsequent overlays of approximately 2 inches thick, and totaling 12 inches, were added before the most recent overlay of 4 inches in the summer of 1982. Also, the upper 8 inches of the previous overlays were well bonded, whereas the lower 4 inches plus the original surface were poorly bonded. The cross section for this project is shown in Figure 1.2.

Pavements in this area suffer from the harsh environmental conditions in addition to heavy traffic (approximately 600 standard axle loads per day). Thermal cracking of asphalt-treated layers is common, and also frost damage due to susceptible subgrade soils.

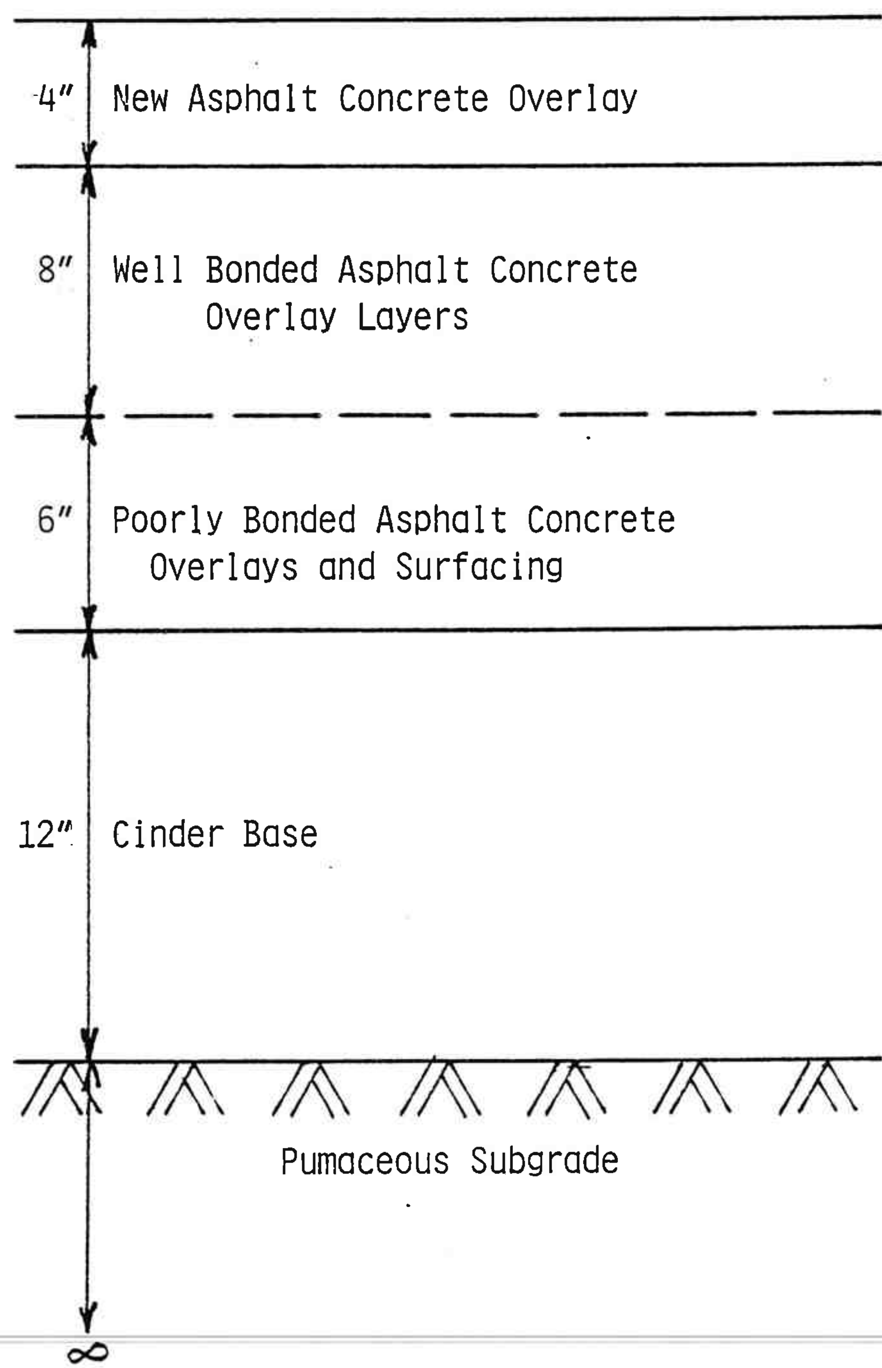


Figure 1.2 - Pavement Cross-Section, US-97 Project

1.2 Resilient Modulus Data

The full set of resilient modulus data for all the pavement materials is presented in Part 1 of this report. Essential data is presented below.

1.2.1 Salem Parkway Project

The results for both subgrades required for the analyses presented in Chapter 2 of this part of the report, are given in Figures 1.3 and 1.4, which show moduli determined by triaxial and diametral testing devices, when both were used. The results presented are for those test conditions closest to the in situ density and moisture content.

The resilient moduli for the asphalt concrete (at 20^{o}C) and CTB were determined by the diametral device on samples obtained from cores. The average values for each material are shown in Table 1.1. The moduli for the cement-modified soil, (CMS) for each of the subgrade types, were determined on laboratory manufactured samples, after a 7-day cure time. The 7-day moduli are presented in Part 1 of this report. There is no available information to project such 7-day moduli to a representative fully cured value. For the purpose of this study each CMS was assumed to have a modulus of 200,000 as shown in Table 1.1.

1.2.2 U.S.-97 Project

The resilient modulus results for the subgrade and base are shown in Figures 1.5 and 1.6 respectively. Since the subgrade is a noncohesive soil, the modulus is shown plotted versus the sum of the principal stresses rather than versus the deviator stress, the usual parameter used for cohesive soils. Also, Figure 1.5 shows both the results obtained by the diametral and triaxial devices, for the test conditions closest to the in situ density and moisture contents.

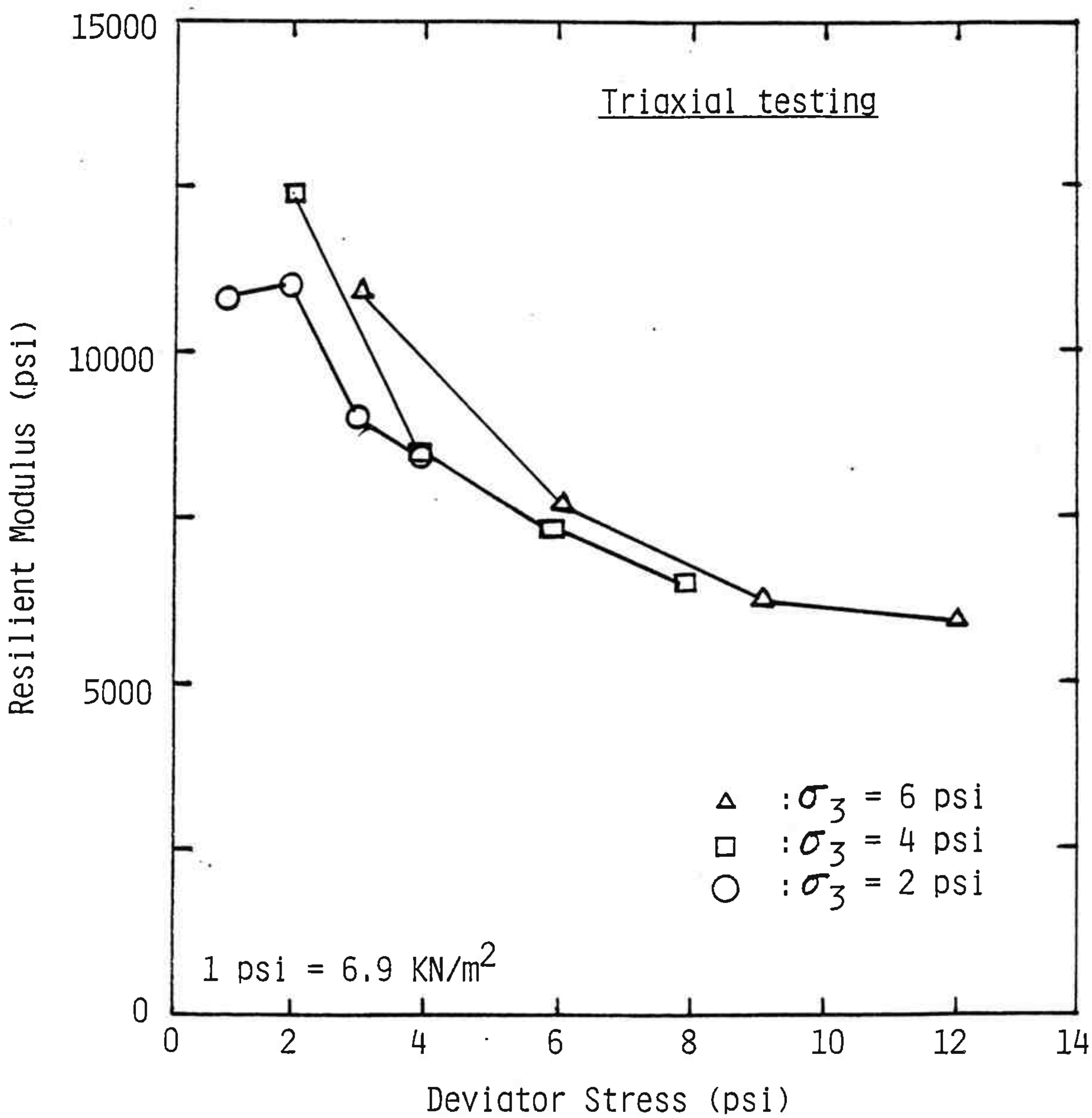


Figure 1.3 - Resilient Modulus Variation, 95% Compaction, 25% Water Content, Salem Parkway, Subgrade Soil 1

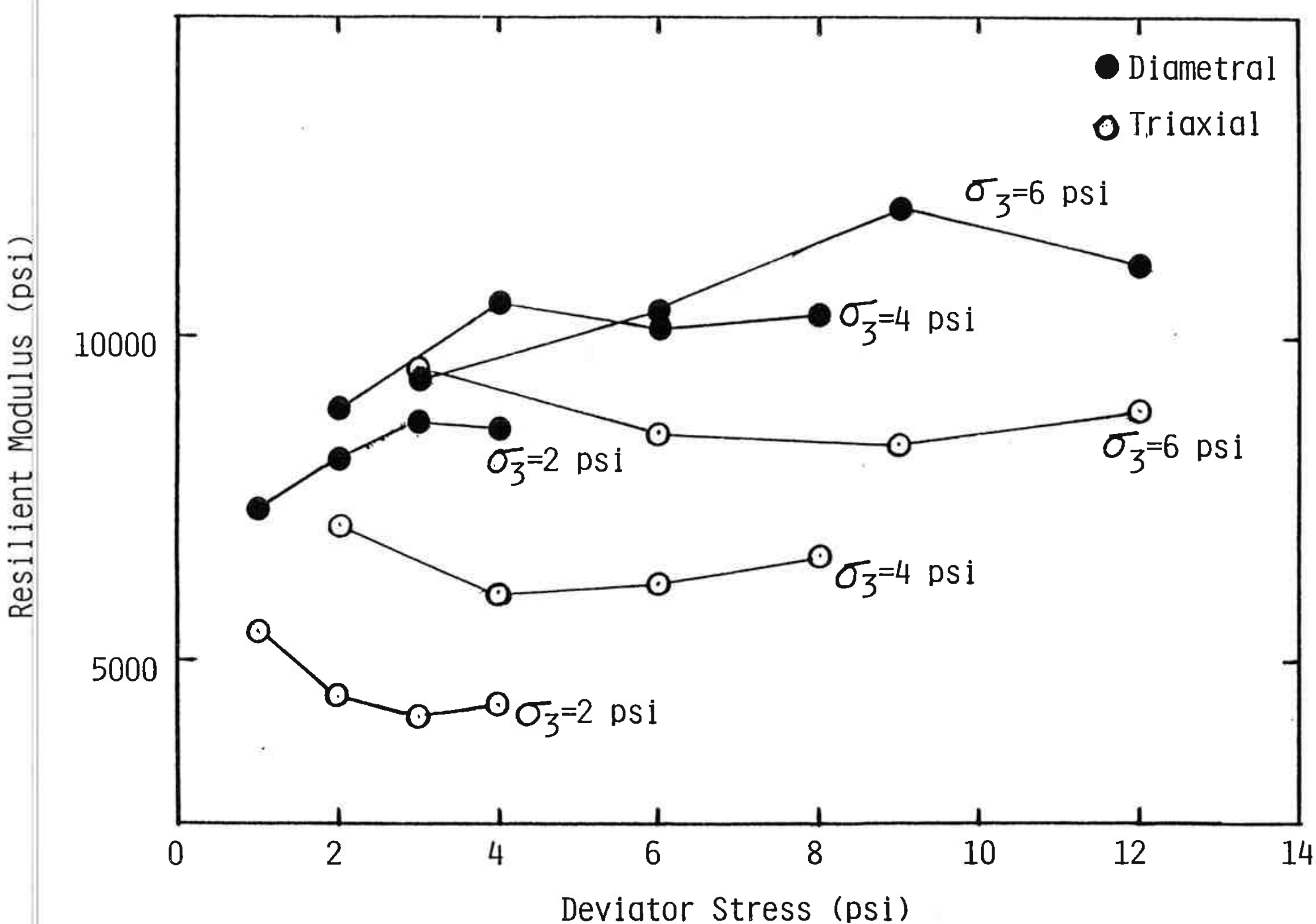


Figure 1.4 - Resilient Modulus Variation, 95% Compaction, 18% Water Content, Salem Parkway Project, Subgrade Soil 2

Table 1.1

Summary of Resilient Modulus Results
Salem Parkway Project
All Treated Layers

Location in Pavement (in.)	Material Description	Resilient Modulus (psi)
0-3.5	Asphalt Concrete Surfacing	155,000*
3.5-13.5	Cement-Treated Base	2,200,000
13.5-19.5	Cement-Modified Soil	200,000

*Determined at 20°C (68°F)

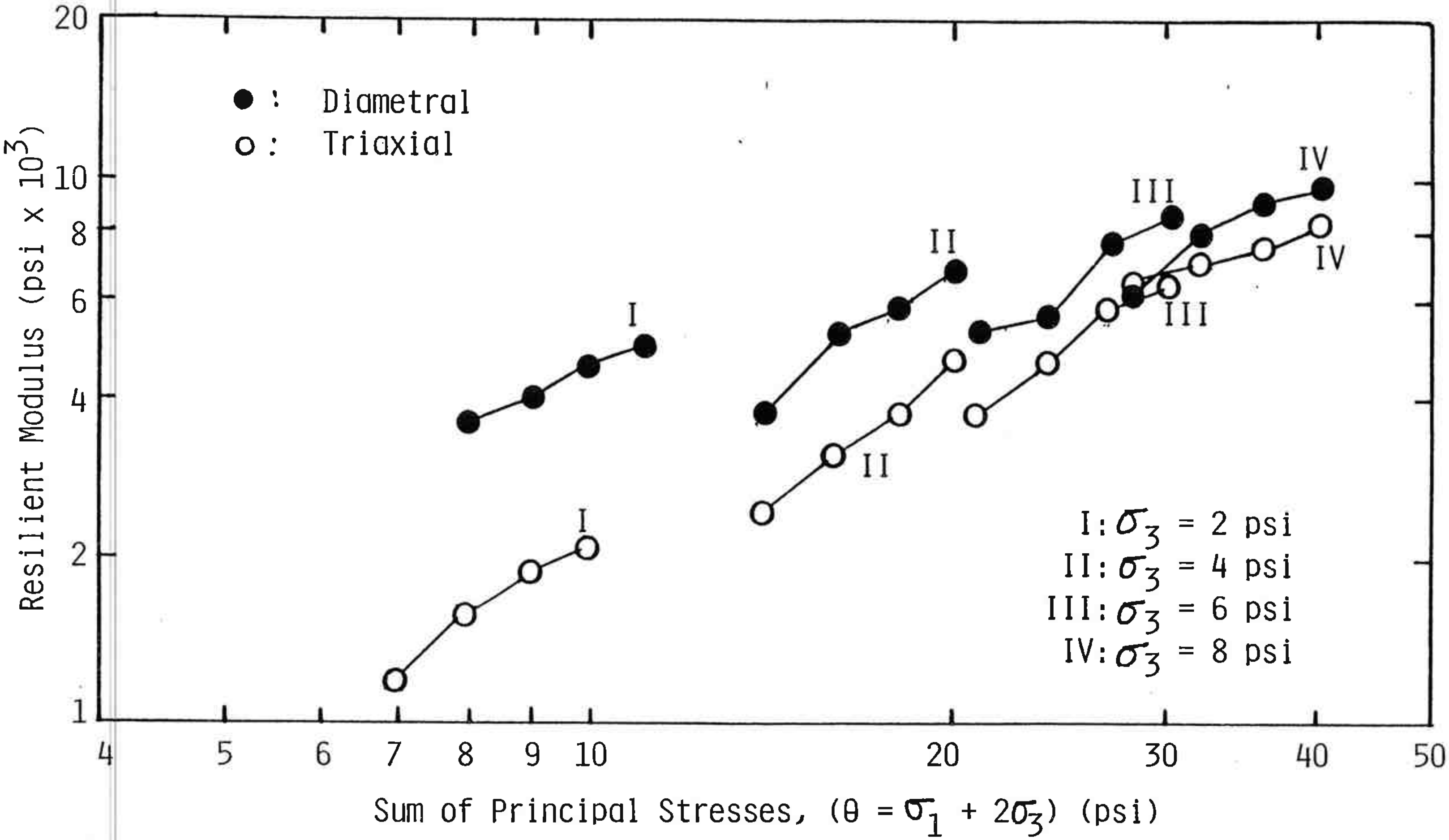


Figure 1.5 - Resilient Modulus Variation, 95% Compaction, 80% Water Content, US-97, Subgrade Soil

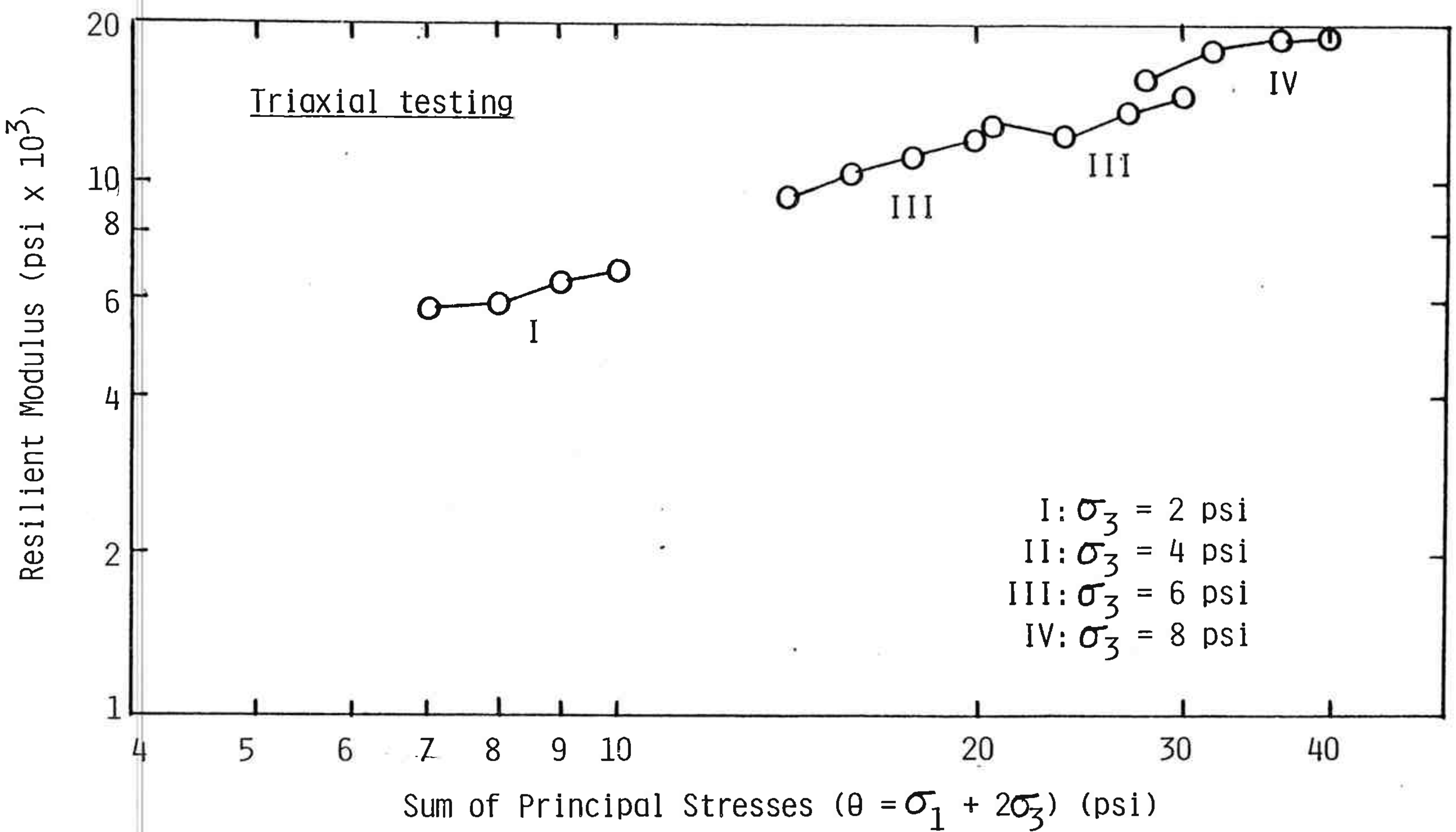


Figure 1.6 - Resilient Modulus Variation, 95% Compaction, 9% Water Content, US-97 Project, Base Material

The moduli determined for the asphalt concrete by tests on samples obtained from cores, using the diametral test, are shown in Table 1.2. This table shows results obtained by Oregon State University (at 20°C) and Oregon State Highways Division (at 25°C).

1.3 Benkelman Beam Deflection Data

All the deflections presented in this section were measured with a Benkelman beam using a standard axle load (18,000 pounds single axle with dual wheel arrangement). Where appropriate the deflections are corrected to a standard temperature of 70°F according to Figure A.2, the chart used in the Oregon overlay design procedure (see Appendix A).

1.3.1 Salem Parkway Project

The deflections measured at various locations along the project are presented in Table 1.3. Measurements were made after construction of the CTB, and also after construction of the first lift of the asphalt concrete surface (3.5 inches). Hence the difference between the two sets of deflections is a function of the curing of the CTB and the addition of the base course. The 80 percentile value is presented, which is calculated from:

$$\delta_{80} = \delta_{mean} + 0.84\ (\overline{\sigma}) \qquad (1.1)$$

where,

δ_{80} = 80 percentile value

δ_{mean} = mean measured deflection

$\overline{\sigma}$ = standard deviation

The deflections were not corrected for temperature difference, because of the use of a CTB.

Table 1.2

Summary of Resilient Modulus Results
U.S.-97 Project
Asphalt Concrete Layers

Location in Pavement (in.)	Material Description	Resilient Modulus (psi)	
		OSU (20^{o}C)*	OSHD (25^{o}C)**
0-4	New Overlay	450,000	220,000
4-12	Old Overlay	870,000	420,000
12-16	Old Overlay	870,000	250,000
16-18	Old Surfacing	870,000	200,000

*20^{o}C = 68^{o}F

**25^{o}C = 77^{o}F

Table 1.3

Benkelman Beam Deflection Measurements
Salem Parkway Project

Location	Subgrade Soil*	Pavement Layer	Mean Deflection (in.x10^{-3})	Standard Deviation	80th Percentile Deflection (in.x10^{-3})
STA 233+00	1	CTB	8	2.8	11
STA 213+00	2	CTB	9	2.5	11
STA 211+50	2	CTB	8	2.0	10
STA 229+50	1	Surface	8	2.6	10
STA 238+50	1	Surface	9	3.3	12
STA 213+00	2	Surface	6	1.2	7
STA 210+00	2	Surface	6	0.8	7

*Soil 1 is classified by AASHTO as A-7-6
Soil 2 is classified by AASHTO as A-4

1.3.2 U.S.-97 Project

The deflections were measured at one location in the inside wheel track (IWT), outside wheel track (OWT) and between the wheel tracks (BWT), both before and after construction of the overlay. The results are given in Table 1.4. The deflections measured before construction of the overlay are shown corrected for temperature. The Oregon overlay design procedure requires a temperature correction for pavements with less than 6 inches of asphalt concrete. The asphalt concrete was about 14 inches thick before overlay, and the deflections were temperature corrected as if a 6-inch layer had occurred, presumably because of low temperatures at the time of measurement. However, for the deflections taken after the overlay, no correction was applied.

Table 1.4

Benkelman Beam Deflection Measurements
U.S.-97 Project
Before and After Overlay Construction

Date	Location	Wheel Track	Mean Deflection (in.x10^{-3})	Standard Deviation	80th Percentile Deflection (in.x10^{-3})
		BEFORE OVERLAY CONSTRUCTION			
10/25/78 *	Milepost 182.3	inner	43	5.7	48
		outer	46	6.4	51
		between	45	6.3	50
		AFTER OVERLAY CONSTRUCTION			
9/14/82 **	Milepost 182.5	inner	25	2.4	26
		outer	26	1.8	27
		between	26	2.1	27

*Air and pavement temperatures not available, but typically the average values for late October are similar and 45-50°F.
A temperature correction factor of 1.07 was applied.

**Measured pavement temperature = 60°F
Measured air temperature = 44°F
No temperature correction was applied.

2.0 PAVEMENT EVALUATION, ANALYSIS, AND DESIGNS

This chapter presents the designs used by Oregon State Highways Division (OSHD) for the two projects examined in this study. The data obtained from the laboratory testing program are then used, in conjunction with the deflection data, to analyze the two projects using layered elastic theory and evaluate the OSHD designs. The analysis and design procedures are presented in a form readily implementable, and details of all procedures used are given in the Appendices.

2.1 Oregon State Highways Division Designs

The design procedures used by OSHD for design of flexible pavements and overlays are essentially the same as those used by Caltrans, the California Department of Transportation (1,2) with modifications to suit Oregon's soil, traffic and climatic conditions. The essential information for use of the OSHD procedures is presented in Appendix A.

2.1.1 Salem Parkway Project

The pavement design developed by OSHD is summarized in Figure 2.1. It should be noted that the same design was used for the entire length of the project utilizing the lower R-value (R=9) resulting for the two soil types occurring along the project. The cores received at OSU, which were obtained after construction indicated that the as-built pavement (Figure 1.1) was different to the design due to the use of stage construction, and due to a problem with the CTB resulting in an increase in the first asphalt concrete stage to 3.5 inches and a decrease in the CTB to 10 inches. Furthermore, the design called for a lime-modified subgrade, whereas a cement-modified layer was preferred when constructed.

OREGON STATE HIGHWAY DIVISION
Location Unit
Surfacing Design Group

REVISED

Section: Chemawa Int. - Pine St. N.E. (Salem)	Date December 8, 1981
Highway: FAU 1525 ("Old" I-305)	Twenty Year Traffic Coefficient 10.3
County: Marion	18 kip s.a. per day 394
Prefix: 324-1941	R Value 9
Res. Engr. Loren Weber	Frost Penetration ----
	CBE Total Requirement 33.0"

STRUCTURAL SECTION
NEW WORK

ACTUAL THICKNESS	COMPONENT	CREDIT
4.0"	A.C.	8.0"
11.0"	C.T.B.	20.0"
6.0"	L.T.S.	6.0"
21.0"		34.0"

1.0"	Asphaltic Concrete Wearing Surface & Base	=	2.0" Aggregate Base
1.0"	Cement Treated Base	=	1.8" Aggregate Base
1.0"	Plant Mix Bituminous Base	=	1.8" Aggregate Base
1.0"	Emulsion Treated Wearing Surface & Base	=	1.8" Aggregate Base
1.0"	Oil Mat	=	1.8" Aggregate Base
1.0"	Cement Treated Existing Roadway Material	=	1.5" Aggregate Base
1.0"	Lime or Cement Treated Subgrade	=	1.0" Aggregate Base
1.0"	Aggregate Subbase	=	0.8" Aggregate Base

Above Factors apply to materials that comply with Standard Specifications and Special Provisions.

Figure 2.1 - Oregon State Highway Pavement Design, Salem Parkway Project

2.1.2 U.S.-97 Project

The design for the overlay developed by OSHD is summarized in Figure 2.2. The total thickness of asphalt concrete indicated for the existing roadway (i.e., the overlay) is 5.5 inches, however, the final stage of 1.5 inches has not yet been programmed. It should be noted that the total of 5.5 inches is less than that required for a traffic coefficient of 10.8 (corresponding to five million 18 kip standard axles) according to Figure A.3 in Appendix A, when the 80th percentile deflection of 0.050 inches is used. A 4-inch overlay corresponds to a traffic coefficient of 8.3 (about 0.5 million standard axles), according to Figure A.3.

2.2 Pavement Evaluation and Analysis Using Layered Elastic Theory

2.2.1 Introduction

A knowledge of the elastic properties of the materials in the various layers of a pavement enables an analysis of the pavement to be accomplished. The simplest form of analysis requires the Young's modulus of elasticity (E) and Poisson's ratio (ν) determined at loading conditions similar to those existing for an in-service pavement. Hence, repeated load testing devices can be used to determine appropriate values of E and ν and the word "resilient" is often used because of the dynamic loading situation. The analysis enables stresses, strains and deflections to be calculated at critical locations in the pavement, and these may be compared with allowable values in the design process.

For this study, the resilient properties of the various pavement materials were selected from the test results and deflection measurements summarized in Chapter 1. There are a variety of computer programs available for

OREGON STATE HIGHWAY DIVISION
Location Unit
Surfacing Design Group

Section: Hackett Dr.-Crescent	Date June 22, 1979
Highway: The Dalles-California #4	Twenty Year Traffic Coefficient 10.8
County: Klamath	18 kip s.a. per day 618.6
Prefix: 18-1943	R Value 16
Res. Engr.	Frost Penetration 30"±
	CBE Total Requirement 32.0"

STRUCTURAL SECTION
NEW WORK

ACTUAL THICKNESS	COMPONENT	CREDIT
1.5"	A.C. (Final Stage)	3.0
2.0"	A.C.W.S.	4.0
8.0"	A.C.B.	16.0
9.0"	A.B.	9.0
20.5"		32.0"

EXISTING ROADWAY

1.5"	A.C. (Final Stage)
4.0"	A.C.O.L. w/ 1.0" extra allowance for leveling.

1.0"	Asphaltic Concrete Wearing Surface & Base	=	2.0" Aggregate Base
1.0"	Cement Treated Base	=	1.8" Aggregate Base
1.0"	Plant Mix Bituminous Base	=	1.8" Aggregate Base
1.0"	Emulsion Treated Wearing Surface & Base	=	1.8" Aggregate Base
1.0"	Oil Mat	=	1.8" Aggregate Base
1.0"	Soil Cement (Existing Roadway Material)	=	1.5" Aggregate Base
1.0"	Lime or Cement Treated Subgrade	=	1.0" Aggregate Base
1.0"	Aggregate Subbase	=	0.8" Aggregate Base

Above Factors apply to materials that comply with Standard Specifications and Special Provisions.

Figure 2.2 - Oregon State Highway Pavement Design, US-97 Project

pavement analysis, and the application of those which use the theory of elasticity has been thoroughly described by Hicks (3). These methods will be referred to as "exact" for the purposes of this report, and it is emphasized that this refers to the calculation procedure rather than to the correctness of the results of the analyses. The commonly used computer programs are:

a) CHEV5L

b) ELSYM5

c) BISAR

d) PSAD

e) PSAD2A

The programs CHEV5L, ELSYM5, and BISAR will only consider layered elastic behavior for any pavement layer, whereas PSAD (also known as CHEV5L with iteration) and PSAD2A will consider nonlinear elastic behavior of a granular base layer. CHEV5L and PSAD will only consider a single wheel load, whereas the other programs will consider at least a dual wheel load arrangement. Currently, each of the five programs listed requires use of a fairly large computer, or a sophisticated small computer system. A recent study by Hsu and Vinson (4) utilized analyses by ELSYM5 and PSAD2A, for pavements with asphalt-bound and granular bases respectively. For this study, the two projects considered contained substantial thicknesses of treated materials, and therefore only ELSYM5 was used, since use of PSAD2A may only be necessary for pavements with thin treated layers and thick granular layers. Essential information for the use of ELSYM5 is given in Appendix B. For the use of the other exact programs, reference (3) should be consulted.

In addition to "exact" methods of analyses, various "approximate" methods are available. Some of these are in chart (5) or tabular form (6,7), and have been obtained from "exact" solutions. Other solutions based on approximate theory may be accomplished by hand calculations or computer programs. One approximate approach based on the use of Boussinesq equations was developed by Ullidtz (8), and its use has been described in detail by Bell (9). This approach is easily implementable on any small computer with limited internal storage, and on some hand-held computers. It is valid for most pavements and produces comparable results to the "exact" methods, particularly with regard to deflection calculations. The major drawback to the use of the modified Boussinesq approach is for pavements with thin treated layers or with a thick granular base. Use of "approximate" methods is justified because of the many assumptions involved in any method of analysis and subsequent design procedures, and they are certainly valid for preliminary analyses to more "exact" approaches with which more pavement technologists are familiar. The use of the modified Boussinesq approach is described in detail in Appendix C.

As mentioned above, to use an elastic analysis requires a knowledge of Young's modulus (E) and Poisson's ratio (ν) for each pavement layer, and also the layer thickness. The modulus varies considerably with material type and various factors such as temperature (for asphalt-treated layers), density, and moisture content (for soils). Poisson's ratio will also vary but within a fairly narrow range and it is usually sufficient to assume a typical value. It is preferable to have measured values of modulus. However, there are various techniques available for estimating appropriate values, and these are presented in Appendix D. For this project, measurements of moduli were available from the laboratory test program, and measurements of pavement deflection were available from each project which can be used to ensure that the moduli assigned to each pavement layer match the performance of the whole pavement.

2.2.2 Salem Parkway Project Evaluation

Moduli determined by the diametral device for the treated pavement layers were used in conjunction with values selected from the tests on the subgrade soils. The results for each of the two subgrade soils, for the same density and moisture content as the in situ soils (Figures 1.3 and 1.4), were examined. As a lower bound to the results, a modulus of 4,000 psi was selected to represent the subgrade response, and an upper bound of 12,500 psi. These moduli, one intermediate value, and appropriate Poisson's ratios were used to calculate the surface deflection between a dual wheel loading arrangement, representing an 18 kip standard axle load, the same as used in the Benkelman beam deflection measurements. The pavement properties and loading arrangement are summarized in Figure 2.3. It should be noted that the measured deflections are corrected for temperature according to Figure A.2, Appendix A, when the pavement temperature is higher or lower than 70^{o}F (21^{o}C), although, for pavements with cement-treated bases no correction is applied. However, for the purposes of checking the Benkelman beam deflections, it is necessary to use an asphalt concrete modulus corresponding to 70^{o}F (21^{o}C). Since the testing temperature for the laboratory determined moduli was 68^{o}F (20^{o}C), no adjustment was made.

A plot of surface deflection versus subgrade resilient modulus is shown in Figure 2.4. The analysis was accomplished by use of the ELSYM5 computer program, and a Boussinesq approximate analysis (DEFL) was also used. Only the total pavement structure was analyzed. Both sets of results are shown in Figure 2.4, and it may be seen that there is excellent comparison. Comparison of the measured deflections (given in Table 1.3) with the calculated values, enables an estimate of the subgrade modulus to be made. Table 2.1 shows the

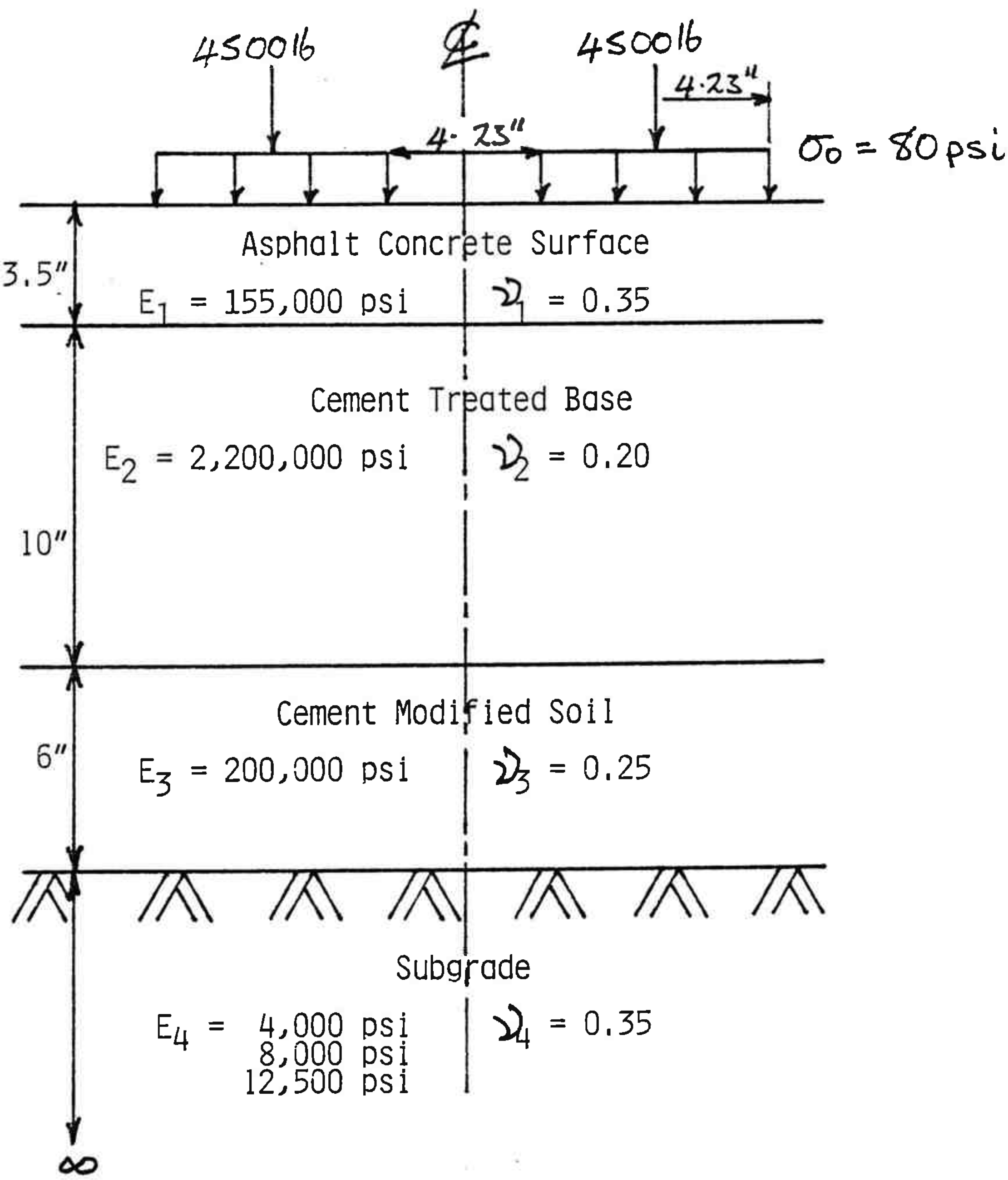


Figure 2.3 - Loading Arrangement and Material Properties for Analysis, Salem Parkway Project

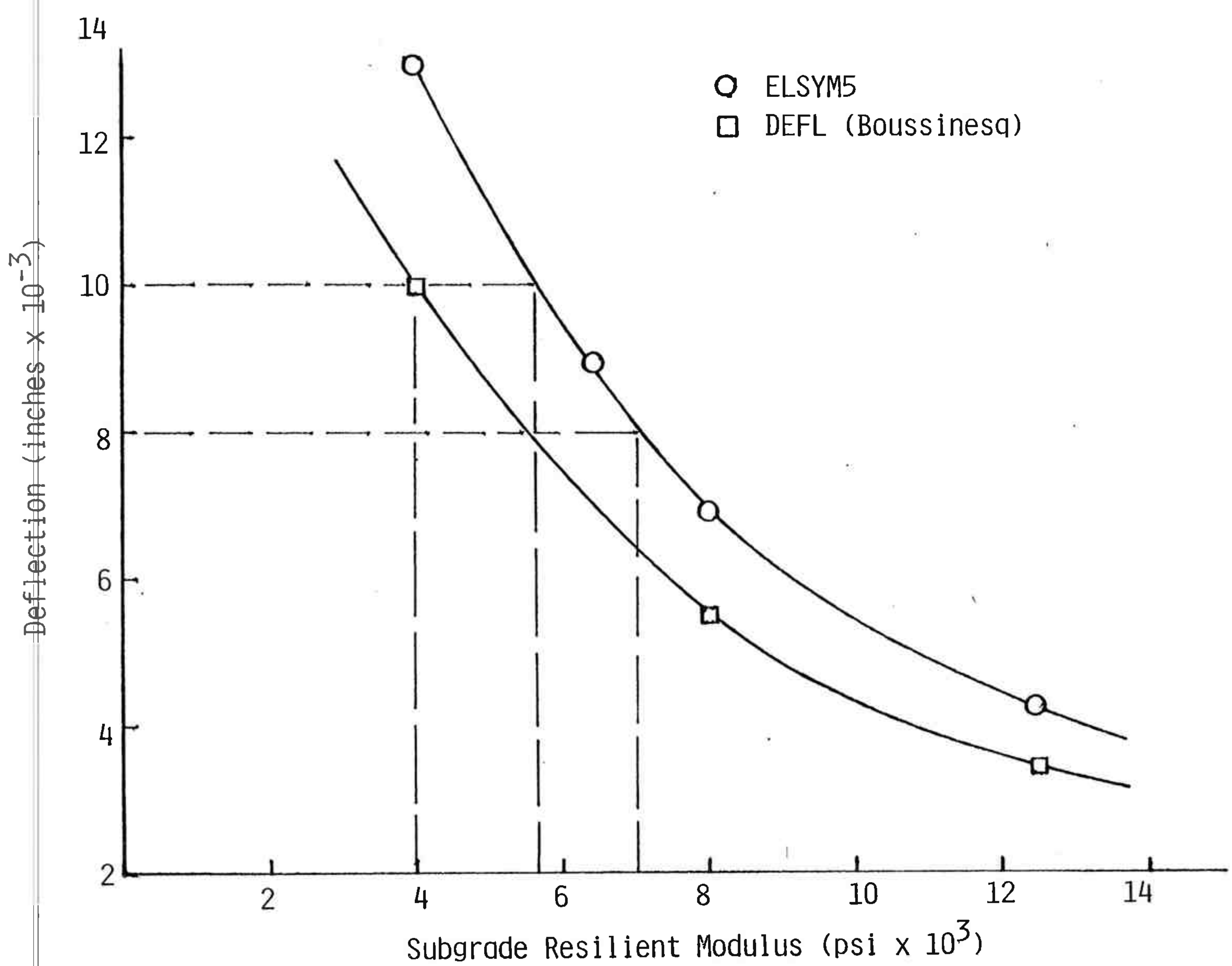


Figure 2.4 - Variation of Deflection with Modulus, Salem Parkway Project

Table 2.1

Subgrade Moduli Corresponding to Measured Deflections
Salem Parkway Project

Location	Subgrade Soil*	80th Percentile Deflection (in.x10^{-3})	Corresponding Subgrade Modulus (psi) ELSYM5	DEFL
STA 229+50	1	10	7,000	5,500
STA 238+50	1	12	5,500	4,000
STA 213+00	2	7	11,000	9,000
STA 210+00	2	7	11,000	9,000

*Soil 1 is classified by AASHTO as A-7-6

Soil 2 is classified by AASHTO as A-4

subgrade modulus corresponding to the 80th percentile measured deflections. The 80th percentile deflection was chosen rather than the mean value, because these higher values correspond to lower values of subgrade modulus, which are suggested by the low values of confining pressure and deviator stress resulting in the subgrade with either analysis. It should be noted that Figures 1.3 and 1.4 show the highest value of modulus at low deviator stress for subgrade soil 1, and, the lowest modulus for subgrade soil 2 at the lowest confining pressure. Hence, the values of modulus indicated by the deflection studies are not strictly in keeping with the results, the value of about 10,000 psi for subgrade soil 1 (A-7-6) corresponds to a deviator stress of about 3 psi, as does the value of about 6,000 psi for subgrade soil 2 (A-4). It is interesting that the R-value for subgrade soil 2 was higher than for subgrade soil 1 (see Part 1 of this report).

2.2.3 U.S.-97 Project Evaluation

As with the Salem Parkway project, the results of the resilient modulus tests were used to assign appropriate values to the various pavement layers to analyze the pavement structures before and after the overlay construction and to compare the deflections predicted with those measured by Benkelman beam.

An overlay project, particularly for a fairly old project is difficult to analyze, because cores of treated materials do not necessarily reflect the condition of the various pavement layers, or the bonding between layers. Before proceeding with the analysis it was noted that there was a very large difference in deflections before and after construction of the overlay (see Table 1.4). Ordinarily, the addition of 4 inches of asphalt concrete, to a pavement already having 14 inches of asphalt-treated layers, would not reduce the deflection from about 0.050 to 0.025 inches. An extremely weak or resilient subgrade is implied by such deflections, and/or cracked or poorly bonded layers of treated material. Photographic evidence taken before and after the overlay (Figure 2.5) shows there was not severe alligator cracking, and this is supported by engineers reports of regular but not closely spaced thermal cracking. However, examination of pavement cores did indicate that the lower 6 inches of asphalt-treated material was poorly bonded, as indicated in Figure 1.2. It is also possible that in cooler weather, microcracks in the treated layers will open up due to thermal contraction and further exaggerate non-composite behavior.

With the above background, it was decided to investigate the effect of varying the subgrade modulus and asphalt-treated layers modulus. All 14 inches of treated material was combined, and a constant ratio of granular base modulus to subgrade modulus of 2:1 was adopted. The loading arrangement and range of properties used to analyze the before overlay situation are shown in Figure 2.6, and the results of the deflection analyses using DEFL (the Boussinesq-based analysis) are shown in Figure 2.7. The loading arrangement and range of properties used to analyze the after overlay situation are shown in Figure 2.8, and the results of the deflection analysis using DEFL are shown in Figure 2.9. In this case, a range of subgrade modulus from 1000 to 3000 psi was investigated, since this corresponded to the possible range indicated for the before overlay situation (Figure 2.7), and a constant overlay modulus of 500,000 psi was used. The overlay modulus was estimated from the results of the laboratory tests (Table 1.2) and utilizing the Shell estimation procedure outlined in Appendix D, and corresponds to the pavement temperature of $60^{o}F$ measured at the time of the deflection determination. The Boussinesq-based analysis (DEFL) was used since many combinations of layer properties

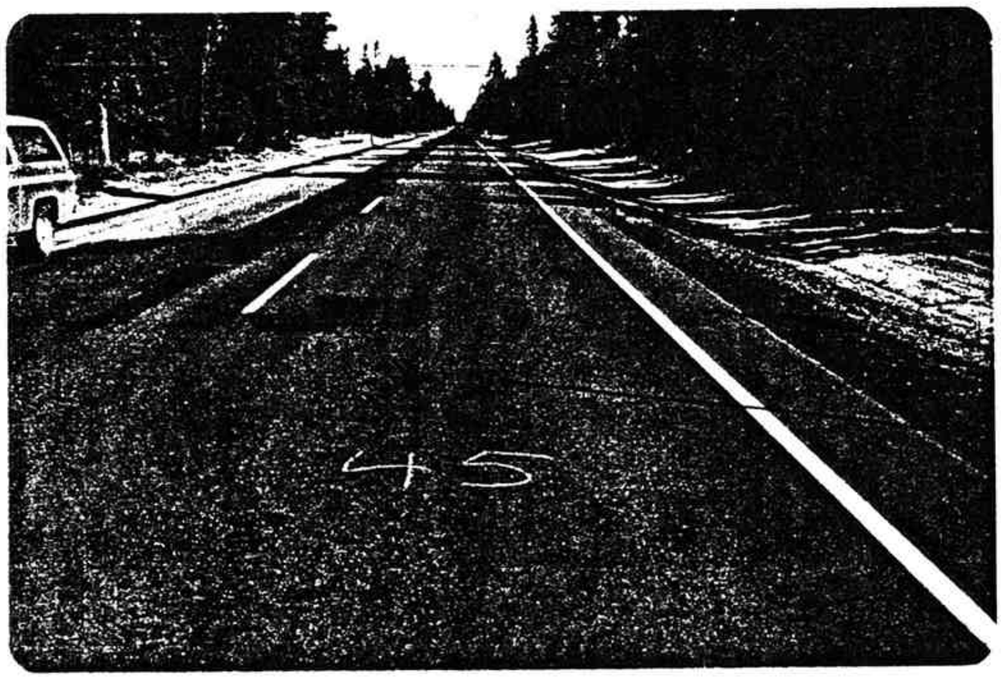


Figure 2.5 - US-97 Project before Overlay

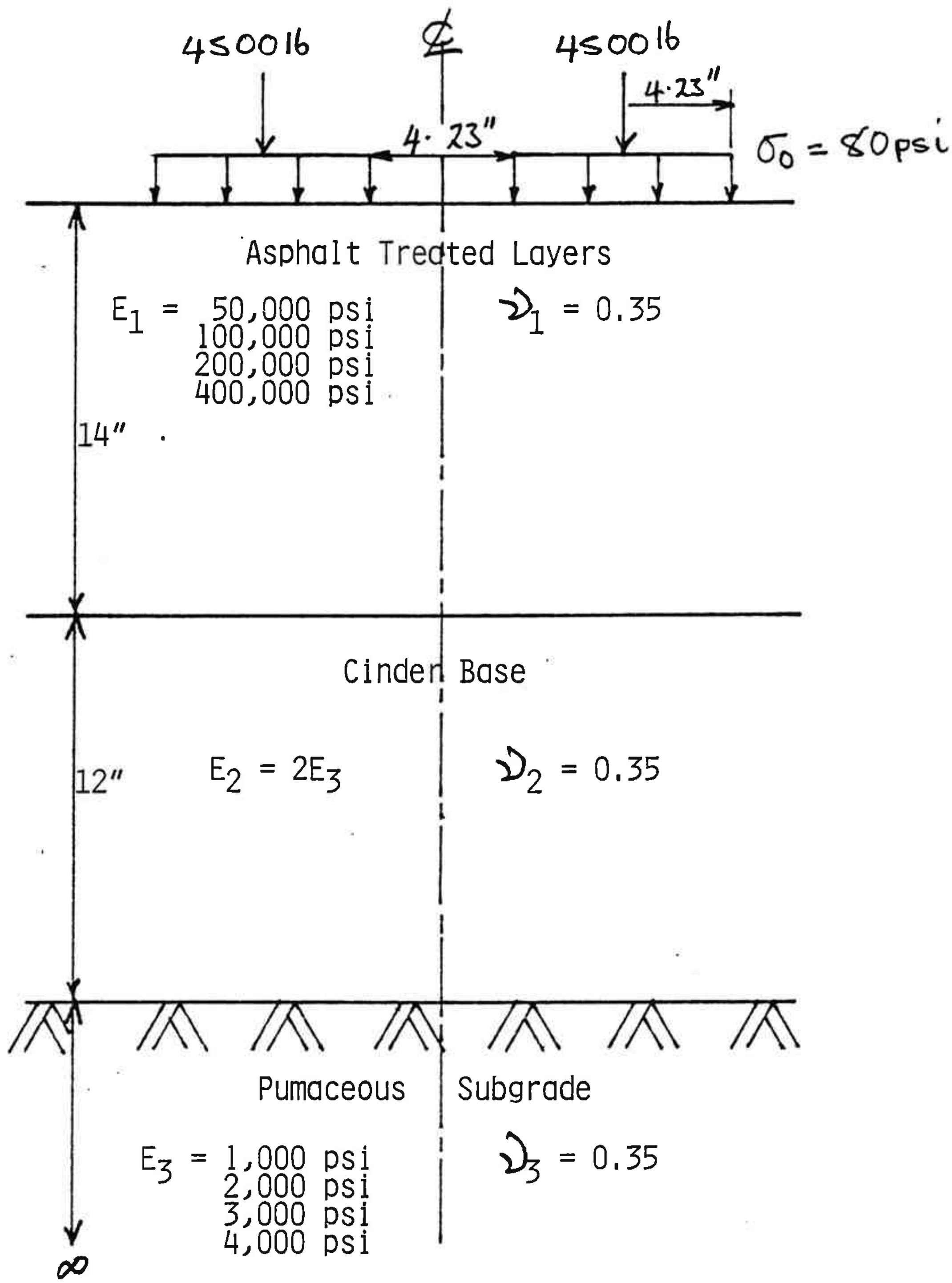


Figure 2.6 - Loading Arrangement and Material Properties for Analysis, US-97 Project, before Overlay

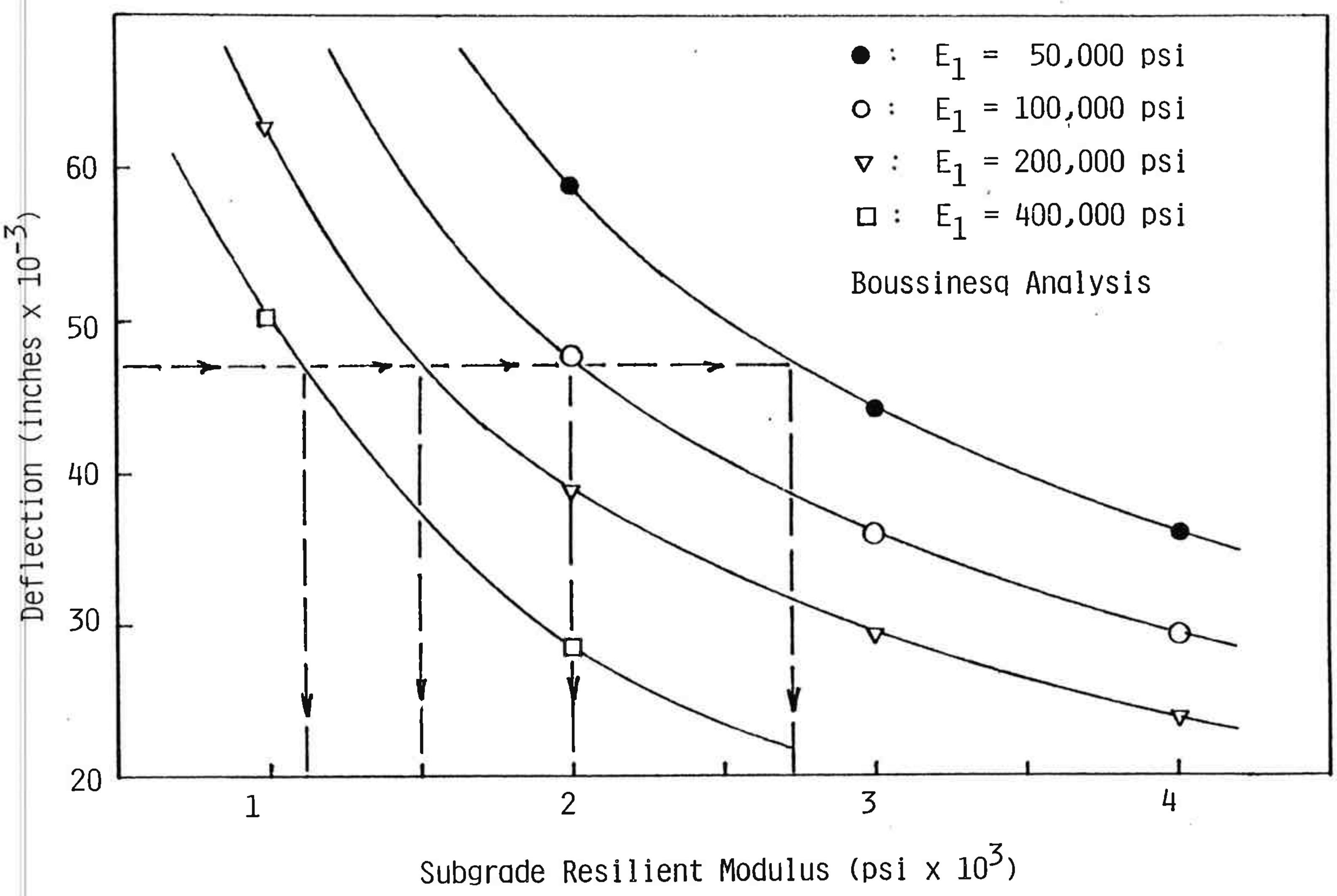


Figure 2.7 - Variation of Deflection with Subgrade Modulus, US-97 Project before Overlays

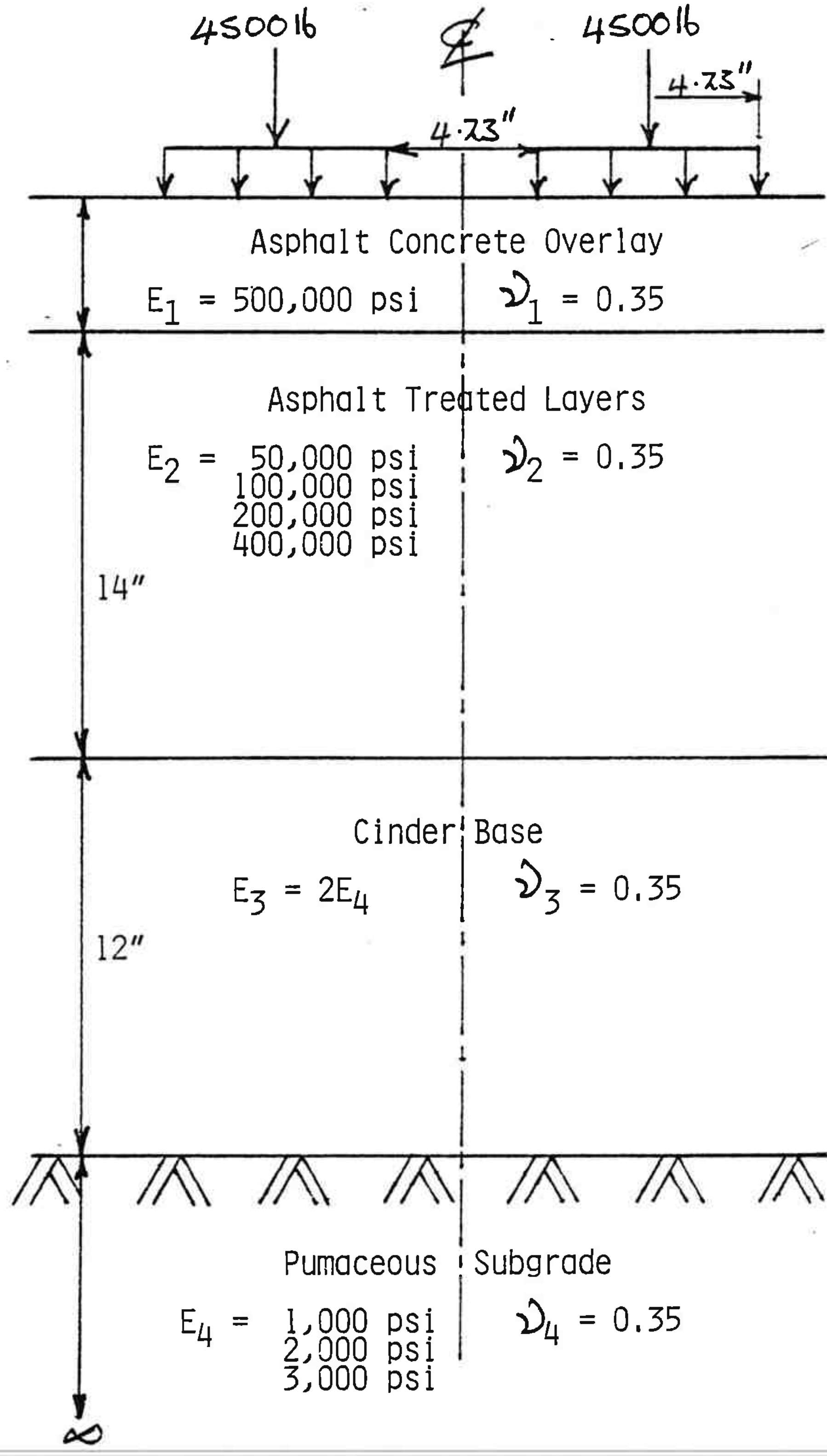


Figure 2.8 - Loading Arrangment and Material Properties for Analysis, US-97 Project, after Overlaying

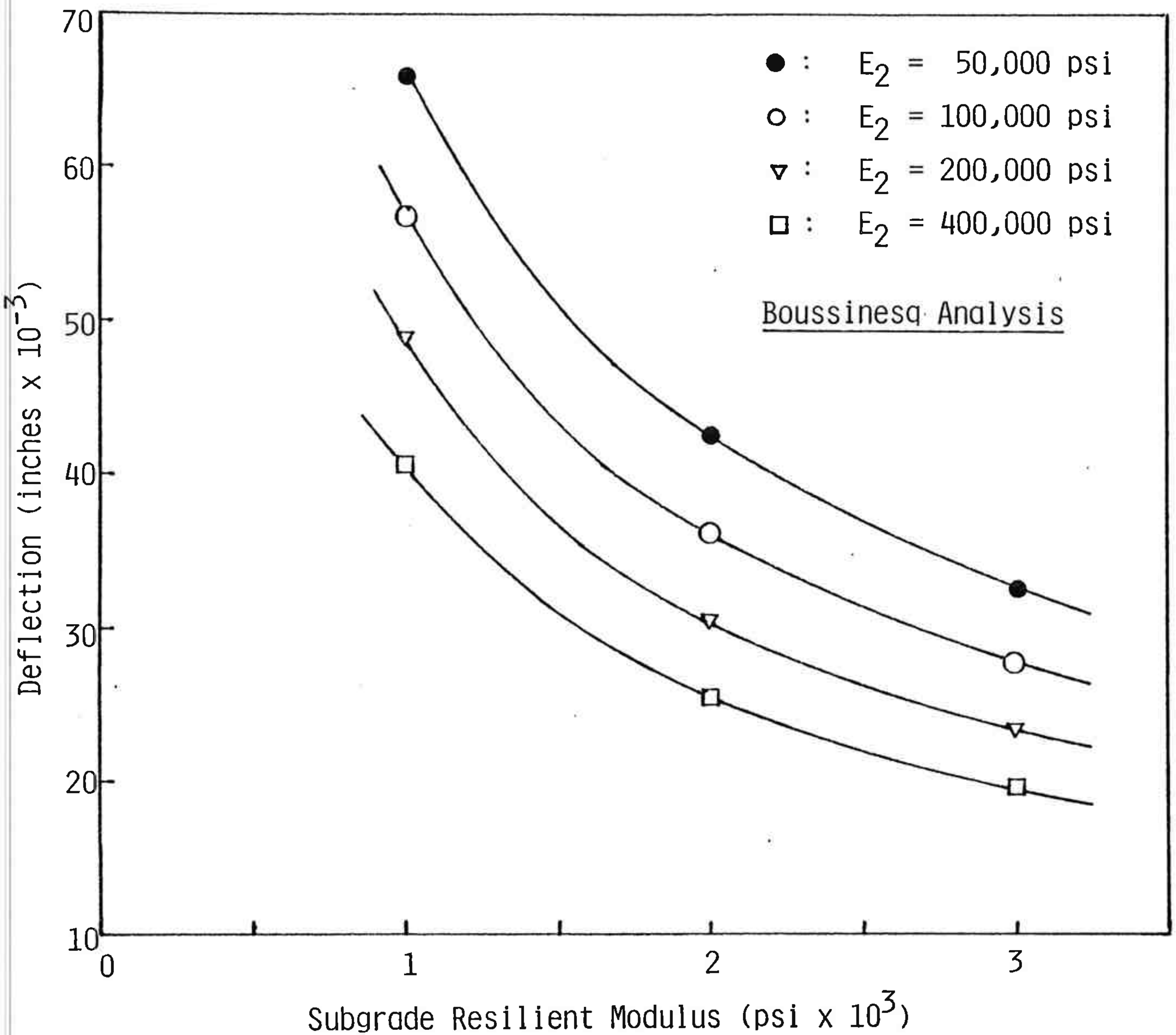


Figure 2.9 - Variation of Deflection with Subgrade Modulus, US-97 Project after Overlaying

were analyzed and because the approximate knowledge of the range of properties did not justify a more sophisticated method.

The results of the analyses described above show that the two sets of results are not compatible - none of the combinations of properties corresponding to a deflection of 50 milli-inches will produce a deflection of about 25 milli-inches after the overlay, each is too high by about 10 milli-inches. It is implied that the overlay is stiffer than initially estimated and that the subgrade modulus is close to 2000 psi. It is also possible that the subgrade was in a weaker condition when the deflections were measured prior to overlaying, since they were taken in late October rather than mid-September as in the after overlay deflections. If this is assumed to be the case a subgrade modulus of 1500 psi and asphalt-treated layer modulus of 200,000 psi would produce the before overlay deflections, and an increase in subgrade modulus to 2500 psi would combine with 200,000 psi for the treated layers and 500,000 for the overlay to produce the after overlay deflection. Such a situation is most likely, since the asphalt-treated layers would probably be less stiff when it was warmer at the time of the after overlay deflection measurements. Hence, a subgrade modulus of about 2,000 psi is likely for average year round conditions, and this corresponds well with the value obtained by triaxial testing of this soil at low deviator and confining stresses as shown in Figure 1.5. Similarly, a cinder base modulus of about 4,000 psi corresponds with that measured by triaxial testing at low confining and deviator stresses as shown in Figure 1.6.

2.3 Pavement Designs Using Analytical Methods

2.3.1 Introduction

As discussed in the previous section, an analysis of a pavement structure using layered elastic analysis enables stresses, strains and deflections to be calculated at critical locations in the structure. The ability to calculate deflections enables existing pavements to be evaluated by comparing measured and calculated values, as demonstrated in the previous section. The resulting estimates of the material properties can be modified or supported by laboratory repeated load tests, and estimates made regarding the adequacy for the original design and remaining pavement life. In order for this to be achieved, the various load-induced failure modes of the pavement are examined by comparing the critical levels of stresses or strains with allowable values. In other words the damage susceptibility of the various pavement components must be known. A proposed scheme for pavement design based on analytical methods is outlined below.

2.3.2 General Analytical Design Approach

The general approach is outlined in Figure 2.10, the specific components of the procedure will be discussed below. It should be emphasized that pavement design is a dynamic process, procedures are (and should be) regularly modified in the light of observations and measurements of pavement performance and changes in construction methods, as shown in Figure 2.11. The various components of the design approach are outlined below:

Estimation of Traffic As with the majority of pavement design procedures, the total number of equivalent 18,000 pound standard axle loads (EAL) should be estimated by any acceptable method.

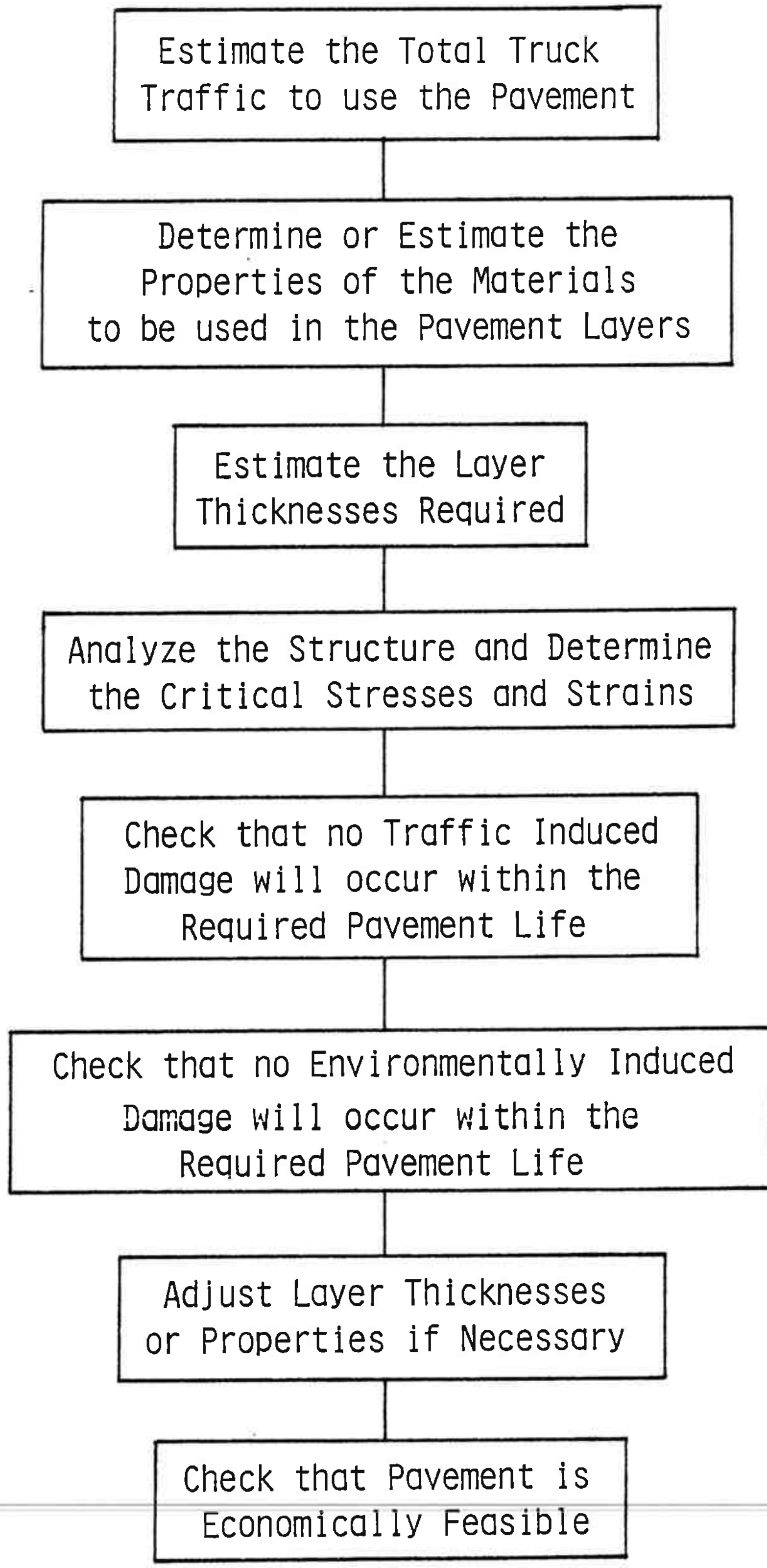


Figure 2.10 - Outline of Analytical Design Procedure

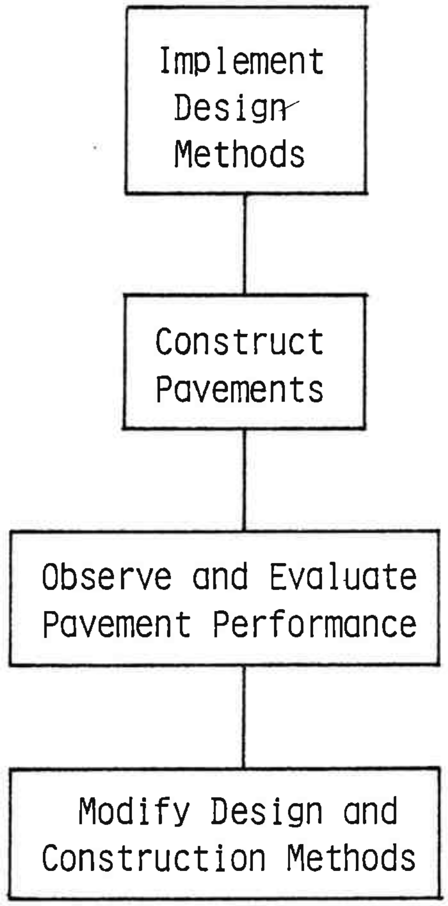


Figure 2.11 - Dynamic Nature of Pavement Design

Determine Material Properties. Material properties may be determined by measurement of the resilient properties by laboratory repeated loading devices, or by estimation using the techniques outlined in Appendix D.

Estimate Layer Thicknesses. Layer thicknesses for new designs may be estimated from experience or from an alternative design method with consideration of practical maximum and minima. For overlay design or evaluation, existing layer thicknesses should be determined from coring or from construction records.

Analyze the Structure. The pavement structure may be analyzed by any suitable layered elastic analysis (see Reference (3)). However, for routine applications a routine computer program such as ELSYM5 (Appendix B) is sufficient, or within the restrictions given in Appendix C, approximate Boussinesq-based analyses could be used.

Traffic Induced Damage. Pavements may perform poorly due to excessive permanent deformation occurring in any or all of the pavement layers and resulting in rutting at the surface. Additionally, fatigue of treated layers may occur due to repeated texture at loads lower than that which would cause ultimate failure. It is common for both failure modes to develop at the same time. Fatigue damage results in alligator cracking which may be distinguished from other modes of cracking since it will occur only in the wheel tracks. Fatigue-life relationships can be established by laboratory repeated load tests, but there is difficulty relating the results to in-service performance.

The procedures adopted in the new published Asphalt Institute design method for flexible pavements (10), for controlling fatigue of asphalt-treated layers and permanent deformation of the entire pavement will be adopted herein. The fatigue criterion for asphalt-treated layers was originally proposed by Finn et al (11), it is described by the following equation:

$$N = 18.4 \{C\} [0.00432 (\varepsilon_t)^{-3.29} (E)^{-0.854}] \tag{2.1}$$

where, N = number of allowable 18,000-pound equivalent axle loads (EAL) and C reflects the mix components,

$C = 10^M$, and

$M = 4.84 (A - 0.69)$

$$A = \frac{V_b}{V_v + V_b}$$

where, V_b = volume of asphalt in mixture (percent),

V_v = volume of air voids (percent),

ε_t = tensile strain repeatedly applied (in/in), and

E = resilient modulus of asphalt mixture (psi)

The deformation criterion was originally proposed by Santucci (12). It is expressed by the following equation:

$$N = 1.36 \times 10^{-9} (\varepsilon_c)^{-4.48} \tag{2.2}$$

where ε_c = vertical compressive strain at subgrade surface.

Figure 2.12 shows a typical fatigue life plot constructed from Eq. (2.1) and, Figure 2.13 shows Santucci's relation for deformation life.

To date, there is no well-accepted criterion for fatigue of cement-treated materials. Their performance may be judged based on local experience. In Oregon, providing the design thickness is sufficient, fatigue cracking seems not to be a problem, and shrinkage cracks are widely spaced, such that layers of CTB will act compositely. Additionally, repeated load tests can be used to determine laboratory fatigue life plots, since similar tests of asphalt-treated materials tend to underestimate lives, use of such plots should lead to conservative design.

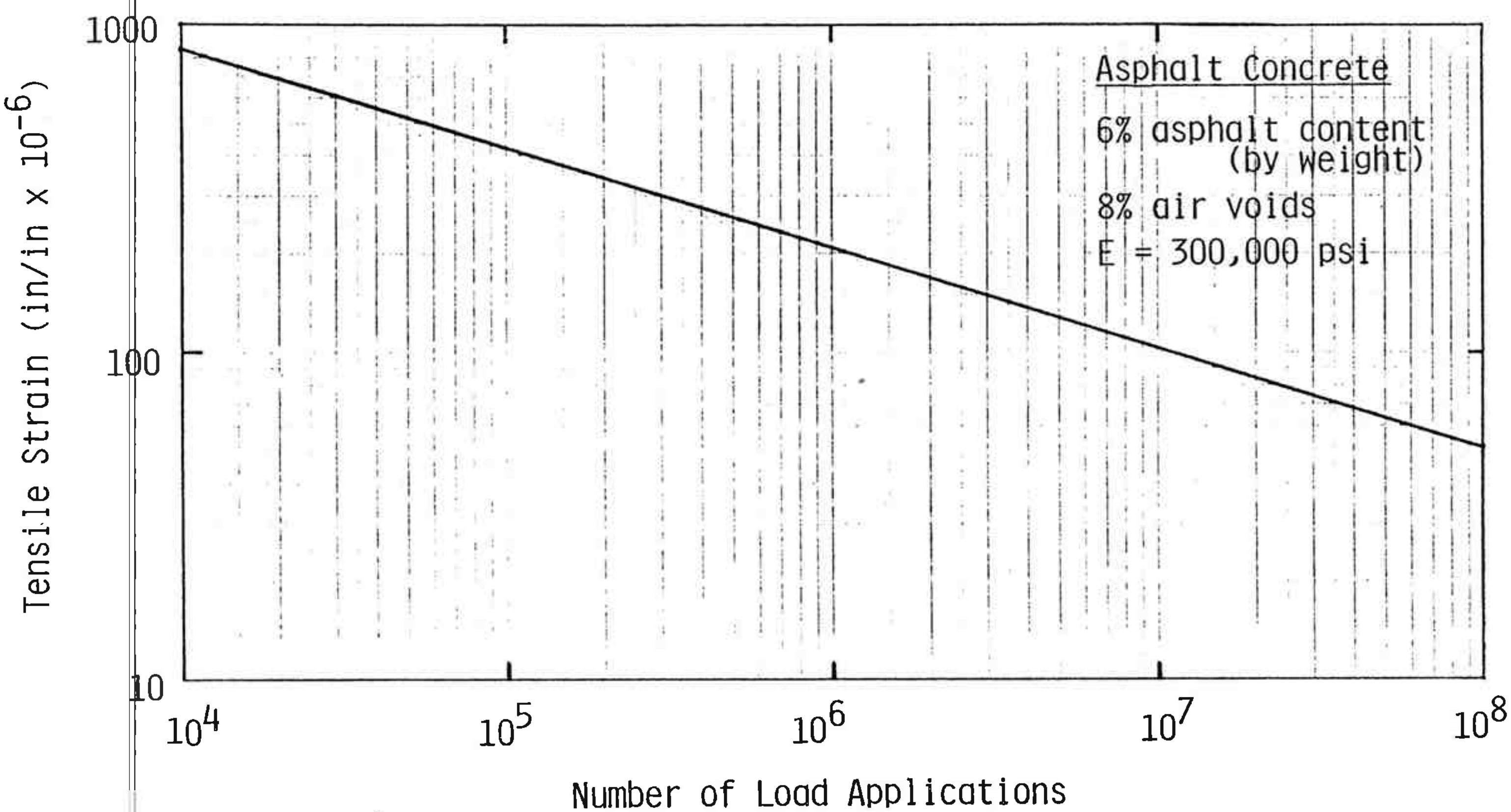


Figure 2.12 - Typical Fatigue - Life Plot

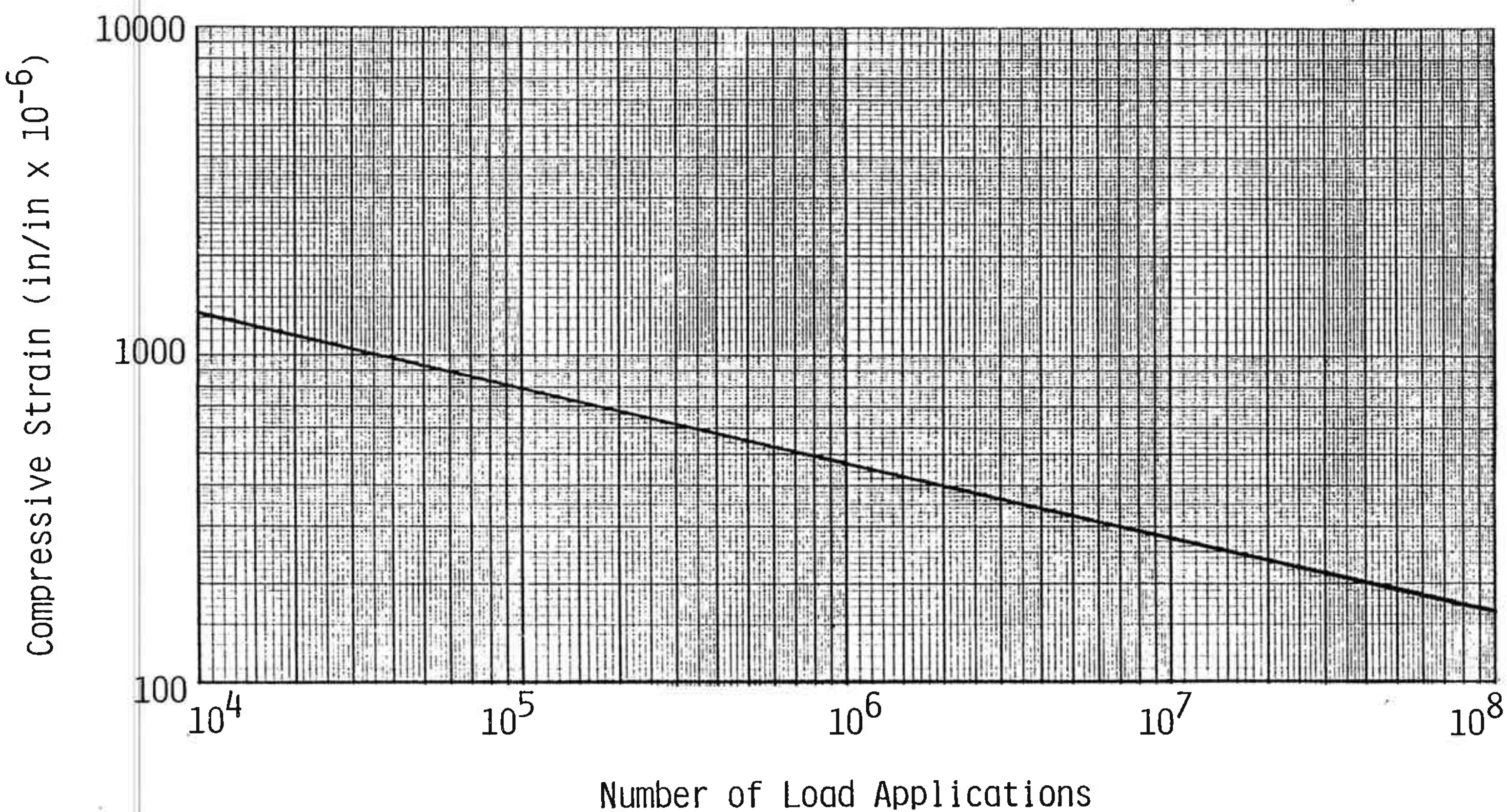


Figure 2.13 - Plot to Determine the Allowable Subgrade Strain (after ref. (12))

Environmentally Induced Damage. Existing methods for controlling frost damage and ensuring adequate drainage may be used. The recently published NCHRP synthesis 96 (12) should be referred to for design of subsurface drainage. Also, where low-temperature cracking of asphalt pavements is a problem, a recent Asphalt Institute publication (14) should be consulted.

Adjust Layer Thicknesses. The design is iterative, layer thicknesses should be adjusted until a balanced design is achieved, considering all possible failure modes.

Economy. Some designs may be more expensive than others depending on material availability, and current price trends for various methods of construction. The optimum design should be chosen to give the best performance at lowest total cost (including maintenance).

2.3.3 Design for Salem Parkway Project

The design of this project will be carried out according to Figure 2.10, for the subgrade soil 2 (AASHTO A-4) which had the lower modulus of 6000 psi. It will be assumed that this modulus represents an average yearly condition, and also that the thicknesses of the CTB and CMS will be fixed at the values shown in Figures 1.1 and 2.3 at 10 inches and 6 inches respectively. Hence, the only variable will be the thickness of the asphalt concrete surface. It will be assumed that the laboratory determined moduli for all three treated materials represent typical in-service conditions. This is reasonable for both cement-treated layers, whose moduli are not temperature dependent, and for the asphalt concrete where the average yearly pavement temperature will be approximately that of the test conditions (68^{o}F). Hence, the loading arrangement and material properties shown in Figure 2.14 were utilized. The design requires that 2.9 million EAL's should be provided for in a 20-year life (see Figure 2.1).

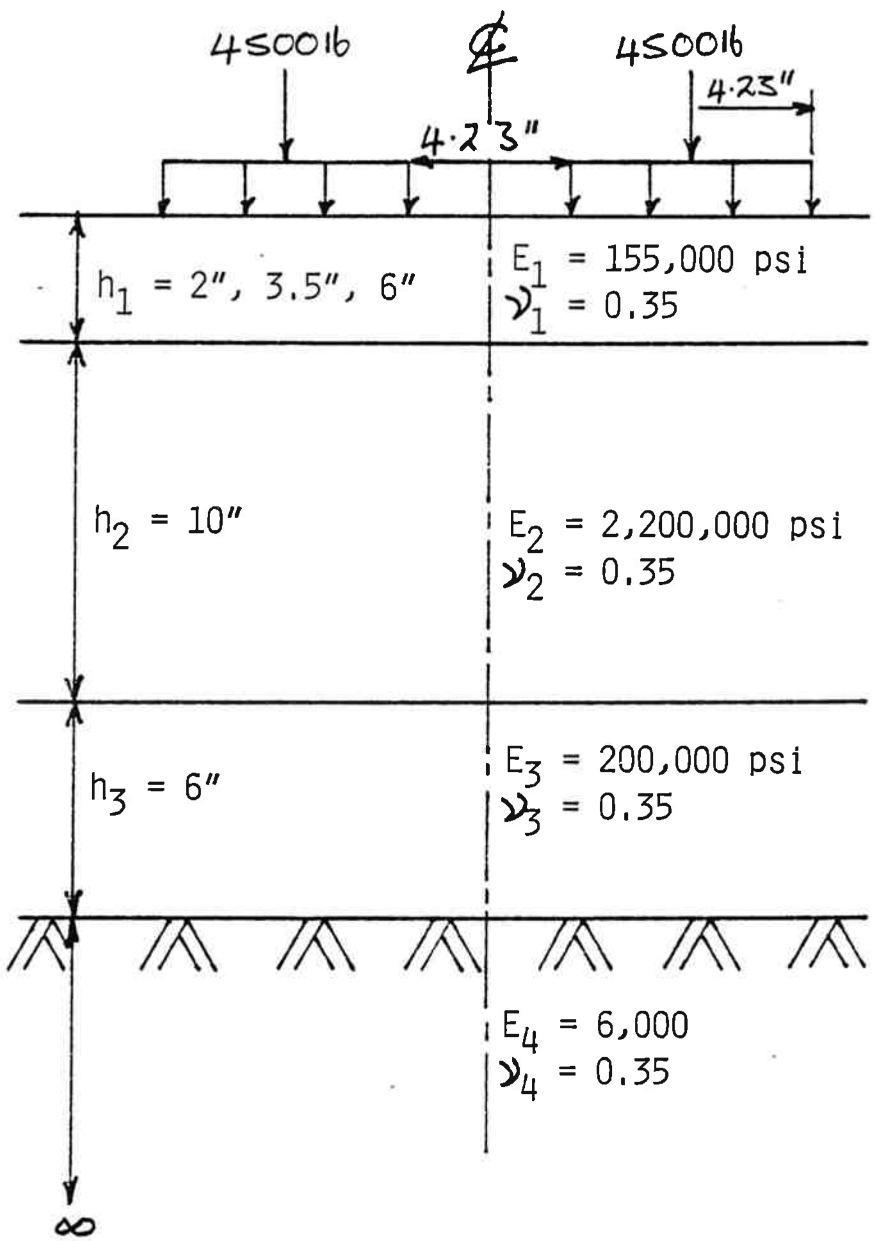
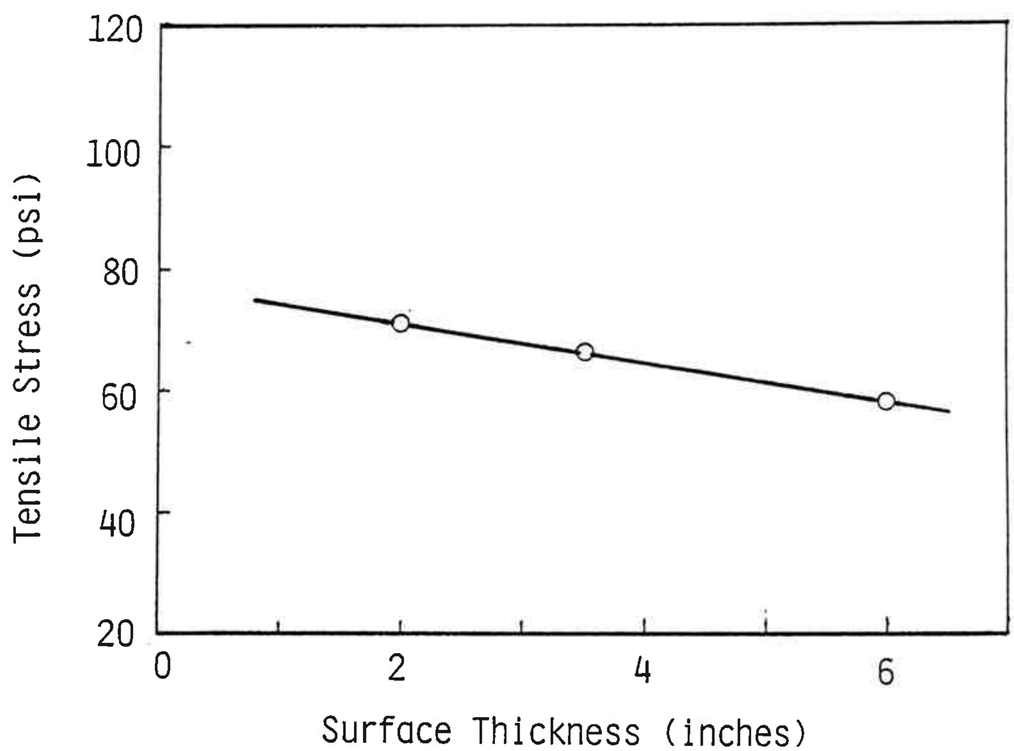


Figure 2.14 - Loading Arrangement and Material Properties for Design of Salem Parkway Project (no cracking of CTB and CMS)

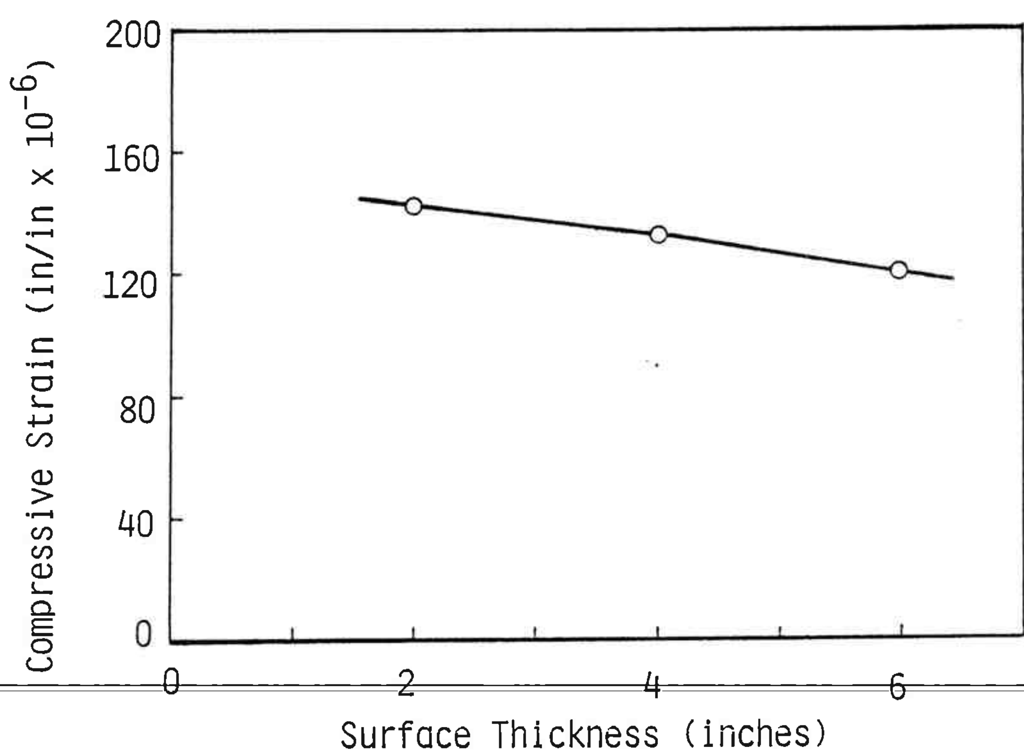
The results of an ELSYM5 analysis are shown in Figure 2.15. The maximum tensile stress in the CTB and the maximum vertical compressive strain in the subgrade are plotted against thickness of asphalt concrete surfacing. Figure 2.13 is used to determine the maximum permissible compressive strain to give a particular life in terms of EAL's, and Figure 2.16 to determine the maximum permissible tensile stress for the CTB. This relationship was derived from the results of fatigue tests on 28-day-old laboratory-made samples of the CTB, tested using the diametral repeated load equipment. The asphalt concrete will not be subject to any tensile strains as long as the CTB does not fail in fatigue.

The values of the tensile stress in the CTB were all less than the maximum permissible value (74 psi, from Figure 2.16) to achieve the design EAL of 2.9 million 18 kip equivalent axles. Similarly, the values of vertical compressive strain on the subgrade were all much less than the maximum permissible value of 370 microstrain obtained from Figure 2.13, as would be expected with a thick CTB base. Hence, the existing thickness of asphalt concrete should be sufficient to ensure no load-associated failure of the pavement will occur.

A rigorous design would evaluate the possibility of load-associated cracking in the CMS also. However, no criteria are available to enable such an evaluation to be accomplished, and it was beyond the scope of the laboratory component of this study to produce such data. However, the calculated maximum tensile stress (σ) for the CMS (always less than 20 psi) was very low, compared to its probable tensile strength (modulus or rupture, MR) which is probably greater than 50 psi. It should be noted that such materials will generally suffer no fatigue when the stress ratio (σ/MR) is less than 50%.



a) Tensile stress in CTB vs. Surface Thickness



b) Compressive Strain in Subgrade vs. Surface Thickness

Figure 2.15 - Variation on Tensile Stress in the CTB and Compressive Strain in the Subgrade with the Surface Thickness

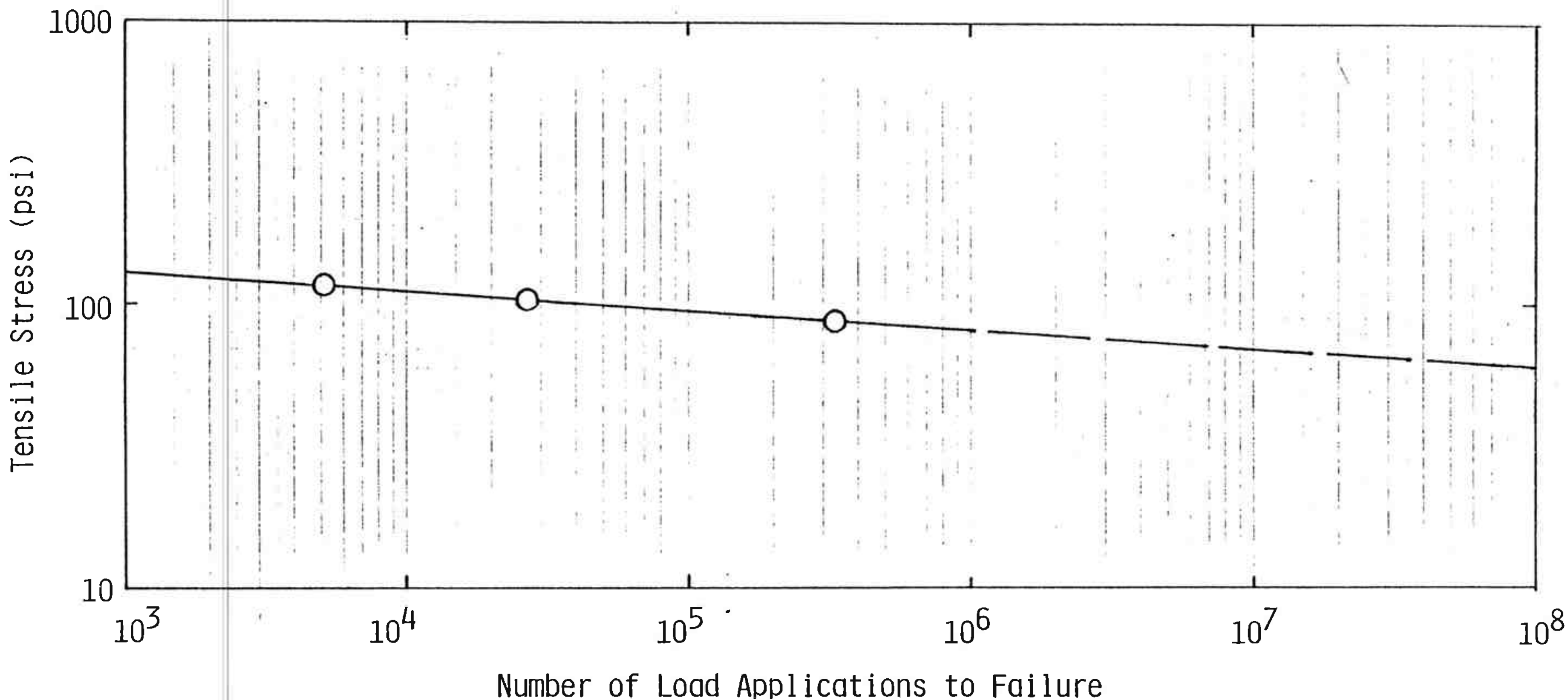


Figure 2.16 - Fatigue-Life Relationship for the Cement Treated Base, Salem Parkway Project

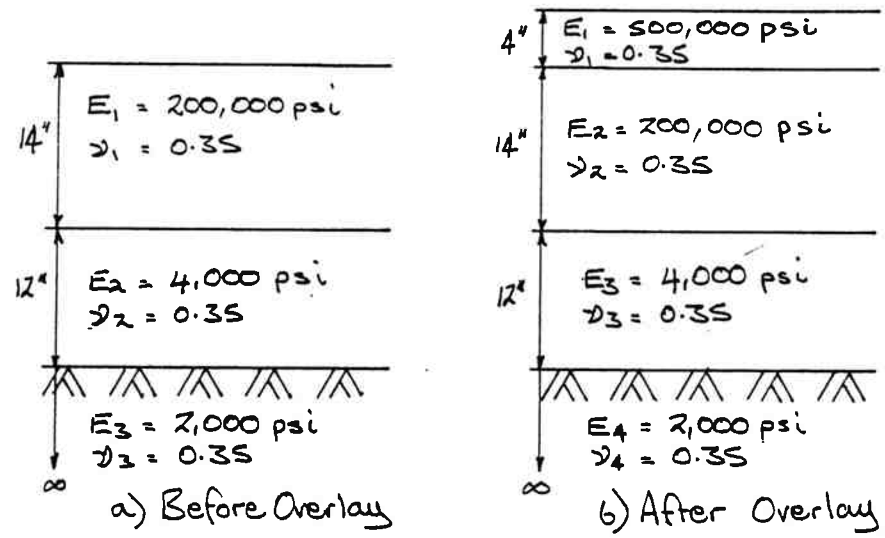
In summary, the OSHD design is supported by the design calculations based on analytical procedures. The results of the design calculations presented in this study may lead to a conclusion, that a minimum asphalt concrete surfacing thickness would suffice, since the analysis indicates that 2 inches of surfacing is more than sufficient to ensure no fatigue cracking in the CTB. However, the fatigue relation used for the CTB has not been validated, and the performance of both the CTB and CMS is so uncertain that some safety factor should be provided. The current 3.5 inches of surfacing should ensure good performance for a number of years, and it may be preferable to delay the application of the final stage of 2 inches more, until serviceability decreased noticeably.

2.3.4 Designs for U.S.-97 Project

The design for this project was carried out according to Figure 2.10, for the material properties determined in Section 2.2.3 The original structure was evaluated, as was the new structure using an ELSYM5 analysis. The results of both design problems are shown in Figure 2.17. The fatigue lives are obtained from Figure 2.18 which show plots based on Eq. (2.1), using air void contents obtained from mixture analyses conducted by OSHD with the values shown in the figure.

It should be noted that to date the pavement on the U.S.-97 project has suffered mainly environmental damage, some frost damage, and some low-temperature cracking. It was due to low serviceability caused by these factors that the overlay was applied. For this reason the pavement design process presented was only a check on the structural adequacy of the pavement rather than a more complete design as presented for the Salem Parkway Project.

Pavement Structures Analyzed:



Results of Analyses:

a) Before Overlay

Traffic:	Current yearly traffic		= 80,000 EAL
	∴ Maximum possible 20yr traffic		= 1.6×10^6 EAL
Critical Strains:	In asphalt layers (ε_t)		= 185 microstrain
	In subgrade (ε_c)		= 477 microstrain
Predicted Life:	For fatigue (Figure 2.18)		= 2.5×10^6 EAL
	For deformation (Figure 2.13)		= 0.9×10^6 EAL

∴ % fatigue life used = 1.6/2.5 = 64%

b) After Overlay

Traffic:	Projected 20 year traffic	= 4.5×10^6 EAL
Critical Strains:	In asphalt layers (ε_t)	= 109 microstrain
	In subgrade (ε_c)	= 287 microstrain
Predicted Life:	For fatigue (Figure 2.18)	= 15×10^6 EAL
	For deformation (Figure 2.13)	= 8×10^6 EAL

∴ % fatigue life required = 4.5/15 = 30%

∴ Total Fatigue life required = 64 + 30 = 94%

∴ ASPHALT LAYERS SHOULD NOT FAIL

Figure 2.17 - Summary of Analytical Design, US-97 Project

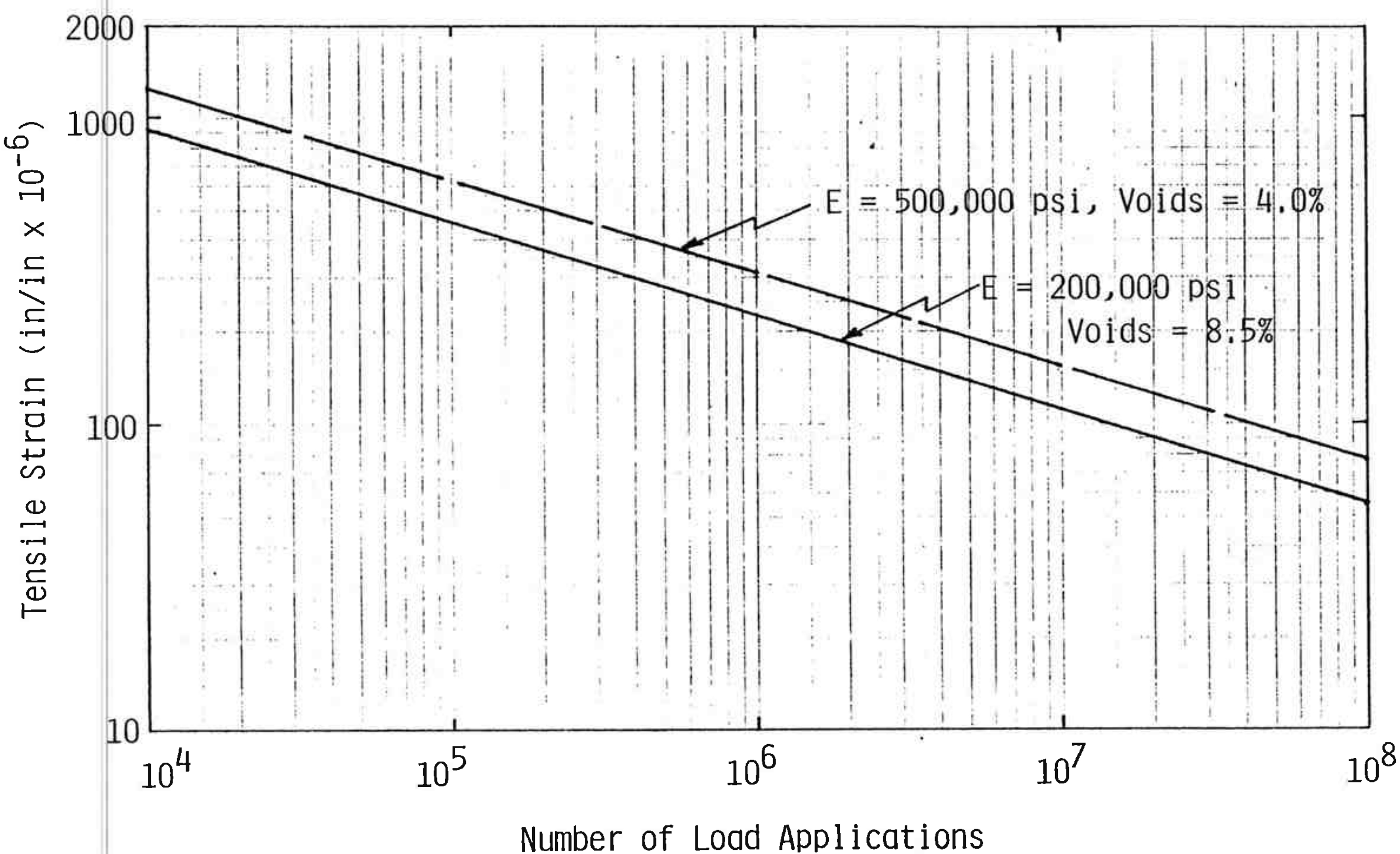


Figure 2.18 - Fatigue Life Plots, Asphalt Treated Materials, US-97 Project

Before Overlay. It may be seen that the life of the pavement before overlay was governed by the subgrade strain criterion (Figure 2.13) and assessed at 0.9 million EAL's. The annual one-way traffic prior to the overlay was approximately 80,000 EAL's, and assumming that flow in previous years was lower, and that the asphalt concrete was previously of high quality, then it is reasonable to deduce that failure had not occurred prior to overlaying. The appearance of the pavement before overlaying also supports this (Figure 2.5), as do site reports from the time of overlaying. Figure 2.17 also shows that the fatigue life of the pavement before overlaying was estimated to be about 2.5 million EAL's. Even assuming that the pre-overlay traffic of 80,000 EAL's per year had occurred for the previous 20 years (total of 1.6 million EAL's), no fatigue would have occurred, as indicated by Figure 2.5 and observations, and a maximum of 64% of the fatigue life would have been used.

After Overlay. Again, the deformation life was lower than the fatigue life, and in this case the required life is considerably exceeded according to either criterion. So far as deformation is concerned the overlaid pavement may be regarded as new. However, so far as fatigue of the original asphalt-treated layers is concerned, only 36% of the fatigue life is remaining. According to the design (Figure 2.17) a further 30% of the fatigue life will be used, and therefore failure will probably not occur. If failure did occur, there will be some additional life for the overlay.

Hence, it is unlikely that the pavement will fail due to load-associated damage. So far as frost penetration is concerned, the OSHD design (Figure 2.1) indicates a value of 30 inches. This is the exact thickness of pavement above the subgrade and there should, therefore, be no frost damage to that material. Protection against thermal cracking should have been achieved by the use of a low viscosity grade asphalt (AR-2000) in the overlay and achieving high mix density. Both were achieved with this project.

In summary, the OSHD design was verified by the analytically based procedure.

3.0 DISCUSSION OF RESULTS

This chapter will discuss each of the major factors influencing the application of the resilient modulus results as reported in this part of the report (Part 2).

3.1 Projects Studied

The projects considered were the best available during the period of the study, and met the requirements that one should be a new construction project (Salem Parkway) and the other an overlay project (U.S.-97). Each project contained typical materials used in the Western U.S.A., but, each contained materials whose performance is poorly quantified, namely cement-treated materials in the Salem Parkway project, and volcanic granular soils in the U.S.-97 project. Since reasonable agreement was obtained between the regular Oregon State Highways Department designs and those using analytical methods, there should be confidence in applying analytical methods to more straightforward pavements, such as those using asphalt-treated or crushed rock bases.

3.2 Resilient Modulus Data

The results of the resilient modulus tests are presented and discussed in Part 1 of this report. It was concluded that the triaxial repeated load test equipment is preferable for noncohesive soils, and that the diametral equipment is suitable for use with cohesive soils and treated materials. It was apparent, from the analyses of the two projects studied, that the levels of stress applied during the majority of the soils tests were much higher than was necessary, and this was due to the substantial layers of treated material overlying these soils. The stress levels used were appropriate for modeling the behavior of soils used in granular bases and for subgrades underlying granular bases. For soils in subbase layers and in subgrades underlying thick treated layers, it is appropriate to test at lower stress levels, possibly with zero confining pressure if possible. This is supported by the fact that the analyses for both of the pavements studied resulted in deviator stresses of about 1.0 psi and confining stresses approaching zero. Only an increase in the levels of both stresses due to overburden pressure (not included in the analysis) raises the induced stresses to levels corresponding to the lower stress levels.

3.3 Deflection Data

Deflections available for use in this study were obtained using the Benkelman beam. Considerable difficulty was experienced in correlating the deflections measured for the U.S.-97 project with theoretically calculated values. This was largely due to the uncertainty of the condition of the soils at the times that the deflections were measured. Although the subgrade was sampled and in situ measurements of density and water content were obtained, the time of sampling did not correspond with the time when the deflections were measured.

A better evaluation of any pavement can be accomplished by measuring the deflection basin rather than just the maximum deflection. If the basin can be sufficiently defined, a much more accurate estimate can be made of all the component layer properties, particularly if laboratory resilient modulus data is available. Considerable research has been directed towards use of devices that can measure deflection basins because of this potential, and three devices are commonly used: Dynaflect, Roadrater, and the Falling-Weight Deflectometer (FWD). The FWD is regarded as that producing a pavement response closest to that occurring under a moving wheel (15). Oregon State Highways

Division utilizes Dynaflect on a regular basis, which can measure deflection basins and could be used to obtain a better estimate of performance for many pavements.

3.4 Oregon State Highways Division Designs

The designs for both projects were produced using the regular OSHD procedures which are based on those developed by Caltrans (1,2). The procedures are easily utilized, and are also based on substantial field experience. Use of the design procedure required significant laboratory testing to determine appropriate material properties. It would be no more time consuming to determine resilient moduli for utilization in analytically based procedures, which could be used in chart form (for examples, see references (4) and (10)).

3.5 Pavement Evaluation and Analysis Using Layered Elastic Theory

The results of the Benkelman beam deflection measurements were compared with deflections calculated from elastic theory, utilizing a range of material properties indicated to be appropriate from the laboratory resilient modulus test program. From these evaluations, it was possible to produce a set of material properties that were compatible with both the measured deflections and laboratory results. However, this was extremely time consuming and costly in terms of computer time when ELSYM5 was used. An alternative is to use a Boussinesq-based approximate analysis (programs PLOAD and DEFL are used for stress/strain and deflection analysis, respectively), but there are restrictions on its use (see Appendix C). These procedures have a useful application in preliminary analyses, or when available data is very approximate and more detail is not justified.

The moduli for the asphalt-treated layers resulting from the pavement evaluations correspond to the pavement temperature at which the deflections were measured. The resilient modulus of asphalt-treated layers is extremely temperature dependent, with a difference of about a factor of two being typical for a $10^{o}F$ (approximately $5^{o}C$) change in temperature. The results of the laboratory tests shown in Table 1.2 are typical. Therefore, the resulting modulus may not be the same as that which should be used in design calculations, which should correspond to an average annual pavement temperature.

3.6 Pavement Design Using Analytical Methods

The procedures presented are flexible, in that they are not dependent on one method of analysis, and the procedures can be modified as experience is gained. One area where considerable improvement could be achieved is in dealing with cement-treated and cement-modified materials. It is imperative that comprehensive information on field performance of such materials should be obtained before designs can take full advantage of these materials. Particularly, little information is available quantifying the extent that such materials crack due to curing and temperature effects, and their fatigue performance is not defined.

Another area where knowledge could be improved is in modeling the behavior of the volcanic soils occurring in Eastern Oregon which are extremely resilient. As indicated by the evaluations for the U.S.-97 project, the volcanic materials in the base and subgrade performed as if they had very low moduli. This results in high calculated compressive strains at the subgrade surface which leads to a poor pavement life prediction based on Santucci's criterion (12) which is representative of most currently used criteria. This does not appear to correspond to observed performance, and it would be appropriate to develop a criterion applicable to pavements constructed on such soils.

A further additional improvement would be to include prediction of permanent deformation in asphalt-treated layers based on the results of simple creep tests (16). Such an approach is gaining popularity (17) and is fairly easily implementable.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

The following conclusions can be drawn from those aspects of the study presented in this part of the report (Part 2):

1) A framework for the use of analytically based design procedures has been presented.

2) Designs produced by the regular Oregon State Highway Division procedures and by the analytical procedures compared favorably.

3) Pavements containing cement-treated materials are difficult to design due to the uncertainty of the performance of such materials.

4) Pavements containing volcanic cohesionless soils are also difficult to design due to the uncertainty of the performance of such materials.

5) The deflection basin of a pavement should be measured rather than the maximum deflection, for the most rigorous evaluation of pavement layer properties.

6) The stress conditions used for determination of resilient moduli of untreated soils, should be compatible with those likely to occur in pavements where they are used. Such conditions are different for pavements with thick layers, treated materials and those with thin layers.

7) Approximate methods of analysis can be used with confidence for the analysis of some types of pavements. At worst they can be used as a means of making preliminary calculations prior to a more exact analysis. In some circumstances the accuracy of the input data may not justify more exact methods.

4.2 Recommendations

1) Analytically based design procedures should be used by Oregon State Highways Division to supplement their existing methods for design of new pavements and overlays.

2) Deflection basins should be measured in routine pavement evaluations. The use of the FWD is recommended.

3) Criteria should be established for predicting the performance of cement-treated materials and volcanic soils. This could be achieved by a thorough survey of pavements containing such materials and by laboratory repeated load tests.

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