LOTTMAN

FINAL REPORT

October 1976

PRACTICAL LABORATORY MEASUREMENT AND APPLICATION OF STIFFNESS OR RESILENT PROPERTIES OF SOILS AND GRANULAR BASE MATERIALS FOR IDAHO FLEXIBLE PAVEMENT DESIGN PROCEDURES

Prepared For

Division of Highways
Idaho Transportation Department
Boise, Idaho 83707

University of Idaho
Robert P. Lottman
Principal Investigator
Department of Civil Engineering
Moscow, Idaho 83843

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ACKNOWLEDGMENTS

Roger Bloomfield, MSCE, developed and tested the resilient modulus test device used in this project. Robert Howard, MSCE, applied the test device to specific test procedures necessary for accurate resilient modulus determination.

Lou Chase, Civil Engineering technician, fabricated the resilient modulus device and integrated the electro-mechanical parts. Mike Winkleman, BSME, and Fernando Mora, M.M.E., designed the test frame for the device. Winkleman also fabricated the frame for the Idaho Transportation Department. Jim Maurer, BSEE, designed and built the signal generator for the regulation of the repeated load functions.

Steve Junkersfeld, BSCE, and Dick Mally, BSCE, assisted in the comparative pavement thickness analysis using resilient modulus. Steve Junkersfeld also applied the computer design procedures for this analysis.

Moscow Materials Laboratory, Idaho Division of Highways, performed the Idaho R-value tests on selected subgrade soils. Leif Erickson, former Materials and Research Engineer and Bob Smith, Soils Engineer, Idaho Division of Highways, with assistance and cooperation of the District Materials Engineers, obtained the selected subgrade soils used in this project.

James Hill, Research Engineer, and Bob Smith, Soils Engineer, Idaho Transportation Department, encouraged the emphasis of the practical implications of the project's findings in the Final Report.

The accomplishment of this project is mainly based on the efforts and cooperation of all the above persons.

Robert P. Lottman
Principal Investigator

ORGANIZATION OF REPORT

This final report is divided into five sections:

A. Introduction,

B. Background Testing,

C. Soil Specimen Preparation, Testing and Resilient Modulus Determination,

D. Application of Resilient Modulus to Flexible Pavement Thickness Design, and

E. Recommendations and References.

Sections C and D are specifically organized for practical implementation and may be copied and bound separately by the Idaho Division of Highways if so desired.

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SECTION A

INTRODUCTION

Background

Idaho's asphalt surfaced highways have shown a tendency to crack prematurely in many locations around the State. This has been noticed especially in the dense-graded asphalt concrete pavements that have been built to take a lot of traffic. Although the cause may be due to cold weather thermal contraction or stripping, many of the crack patterns suggest fatigue cracking.

It is thought that the fatigue cracking was due to excessive pavement deflection resulting from excessive subgrade soil deformation. It is also thought that some or all of the fine-grained soils, comprising the roadbed of most of the Idaho highway mileage, were resilient or lacked stiffness. The problem was to determine how resilient the subgrade soils may be and to evaluate this resilience on the basis of how it affects pavement thickness designs.

In July 1974 the Idaho Transportation Department awarded an applied research contract with the University of Idaho to evaluate the effects of subgrade soil resilience and to develop a laboratory procedure that could be used by the Department to provide data for future supplemental pavement thickness design methods.

The resulting project's findings, contained in this Final Report, were considered to be the third phase of projects carried out by the Department. The first two phases undertaken by the Department were concerned with

Idaho R-value modification and measurement of pavement deflection on pavements that showed premature cracking, and the analyses of the causes of cracking and high pavement deflection. The results of the first two phases can be summarized as follows:

1. The Idaho laboratory determination of R-value of subgrade soils was modified to measure R-value when the soil is less compacted and wetter than in the previous Idaho method --- giving lower R-values and hence thicker pavements for some fine-grained soils, and
2. Pavement deflection measurement and analysis were accomplished on several pavements but, because of financial, time and manpower restrictions imposed by tight budgets, the data that could be gathered were not sufficient to specify definite pavement deflection limits for various soils in order to minimize fatigue cracking. Complete sets of data for I-15 near Malad City were obtained and were helpful. Subgrade soil changes and seasonal pavement deflections were not analyzed around the State to the extent desirable.

In 1974 when the University of Idaho started this project, measurement of the engineering soil stiffness property (inverse of resiliency) known as resilient modulus, M_R, was underway here and there around the U.S. Several agencies were experimenting with the repeated load triaxial test. The equipment was expensive and time consuming for routine use by laboratory personnel, but the procedures formed a basis for simplification. University of Illinois had just developed a simpler repeated load device and was finding M_R of subgrade soils for the Illinois Highway Department. The University of Idaho then developed a prototype device after the

Illinois device but modified it and the soil specimen preparation to coincide with current Idaho Division of Highways laboratory procedures as much as possible.

The Idaho prototype device was put together during the winter of 1974-75 and the first M_R determinations of Idaho soils began. The University of Idaho project then consisted of finding M_R, modifying specimen preparation procedures, further testing of M_R, using M_R results in pavement thickness analytical approaches (elastic layer analysis) and, finally, building a prototype device for use by the Idaho Division of Highways.

Assumptions Used in Project - Testing and Analysis

1. The resilient modulus, M_R, is an elastic rebound modulus of the subgrade soil and is representative of the stiffness of the subgrade soil under the pavement layers and under traffic loading after several hundred cycles of repeated load. The use of M_R in pavement thickness design procedures is a growing trend in the U.S., currently in the development-field evaluation stage.

2. M_R can vary over a pavement length depending on the normal changes of soil type, soil density and moisture content to at least the variation amount shown for R-value and CBR.

3. The soil specimens used for M_R testing are at Idaho "design" water content, density and soil grain orientation as produced by the R-value method at the specified current exudation pressure used in Idaho.

4. The 7-day curing of soil specimens for M_R produce internal soil structures representative of field conditions.

5. The height of 4-inch diameter soil specimens for M_R testing, about 5 inches high, does not appreciably affect M_R results from tests using 8-inch high specimens.

6. Soil specimen variations and test equipment variations do not exceed 10%.

7. Elastic deformations in the subgrade soil occur in the pressure bulb or depth of significant stress: from the subgrade-base interface downward to a depth where the stress is 1/10 of the interfacial stress. The average stresses in this depth occur at a distance of 1/3 of this depth downward from the subgrade-base interface (average depth). The vertical stress from traffic loading at this average depth is calculated by elastic analysis and is called the deviator stress. The confining stress produced at this average depth is calculated by pavement and soil overburden weight and is called the confining stress. M_R is dependent on the magnitude of these stresses.

8. Elastic layer theory is satisfactory for calculation of deviator and confining stresses as well as for calculation of pavement deflection, stresses and strains which affect pavement performance.

9. WASHO 15-kip deflection is equal to 1/2 Benkelman Beam 18-kip rebound deflection.

SECTION B

BACKGROUND OF TESTING

This section briefly describes some of the procedures and analyses that needed to be evaluated before a test system procedure could be specified for determination of M_R.

The Soil Testing Device

Device Improvements and Modifications

The original prototype test device shown in Figure 1 was designed to transmit cyclic compressive loads ranging from 5-20 psi to test specimens of various heights and to record the applied load and total specimen deformation monitored by electrically operated measuring components. Difficulties in varying the loading frame height necessitated construction of a more mechanically efficient one, which is easily transportable. The improved test frame allows a technician to readily make changes in the device to allow for various specimen heights if necessary.

The pressure solenoid which regulates air pressure to the Bellofram air cylinder was also replaced to provide lower driving pressures from 0 to 5 psi as well as the higher pressures to 60 psi.

Triaxial Test Equipment

A conventional triaxial cell was adapted to the resilient modulus device to provide confining pressure to soil specimens during testing for M_R. The vertical stress applied to soil specimens is induced via the triaxial cell shaft with compressed air producing regulated confining pressure on the specimen.

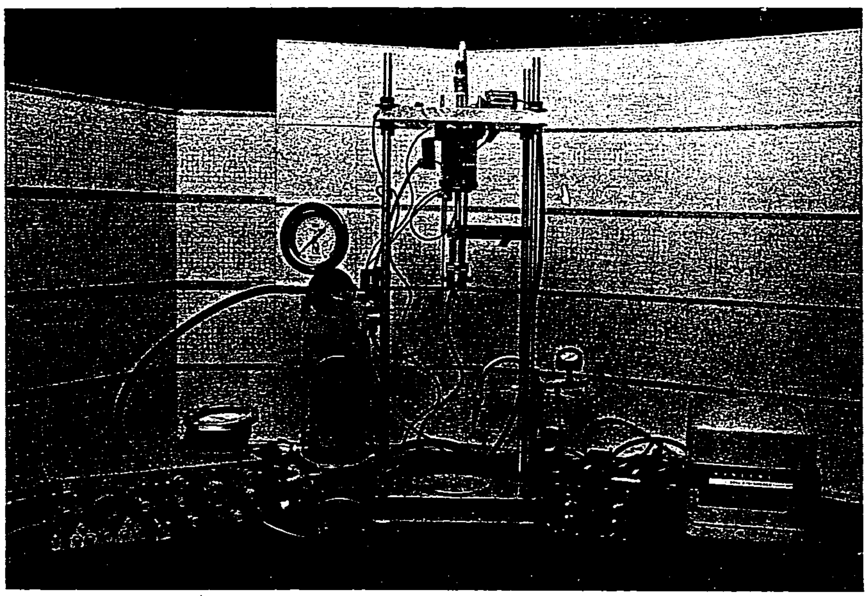


Figure 1. Resilient Modulus Test Device Showing Wavetek Generator Signal Conditioners, and Visicorder.

The triaxial cell cap limits the lateral movement of the cell shaft that connects the Bellofram to the loading plate on the soil specimen. This ensures that only a vertical force is transmitted to the soil specimen, eliminating non-uniform specimen creep that in previous unconfined tests caused specimens to slant from horizontal.

Soil Testing

Only subgrade soils were tested since it appeared that 65-75 percent of the total elastic deformation of a pavement surface is from subgrade soil deflection. The soils used for this research project were sampled from subgrades of existing pavements in Idaho. Locations for each subgrade soil are shown in Table 1.

TABLE 1

Subgrade Soil Locations

Soil	Location	County
B	U.S. 95, White Bird Area	Idaho
C	I-15-W, 7 miles east of junction with I-80 North	Cassia
F	U.S. 20, 8 miles east of Caldwell	Ada
G	I-15, 4-9 miles north of Malad City	Oneida

Table 2 lists the Idaho R-Value water content and dry density of each subgrade soil.

TABLE 2

Water Content and Dry Density of Soil Used in Specimens with Idaho R-Value.

Soil	Water Content, %	Dry Density, pcf	R-value
B	16.6	107	63
C	18.5	103	71
F	31.7	84	67
G	33.9	86	11

Table 3 identifies the subgrade soil by ASSHTO classification and Atterberg Limits.

TABLE 3

ASSHTO Classification and Atterberg Limits of Tested Soils

Soil	L.L.	P.L.	P.I.	ASSHTO Classification
B	--	--	--	A-3 Fine Sand
C	--	--	--	A-3 Fine Sand
F	37.3	33.1	4.2	A-4 Low Compr. Silt
G	58.5	29.2	29.3	A-7-5 High Compr. Si-Clay

M_R of soil specimens is affected considerably by one or more of the following (1)(7)(10):*

1. Specimen dry density,
2. Specimen water content,
3. Specimen height,

---

* Note: Numbers underlined in parentheses refer to the References (Section E).

4. Specimen curing time, and
5. Specimen disturbance during membrane placement and handling before testing.

The specimen density, water content, and curing time are most critical and therefore were closely watched to ensure that they remained relatively constant for every specimen for each soil tested.

The deviator or cyclic pressure and confining pressure greatly affect soil M_R; the effect of confining pressure on M_R becomes greater as clay content of the soils decreases (8)(10). Consequently testing pressures were of primary concern and will be subsequently discussed in detail.

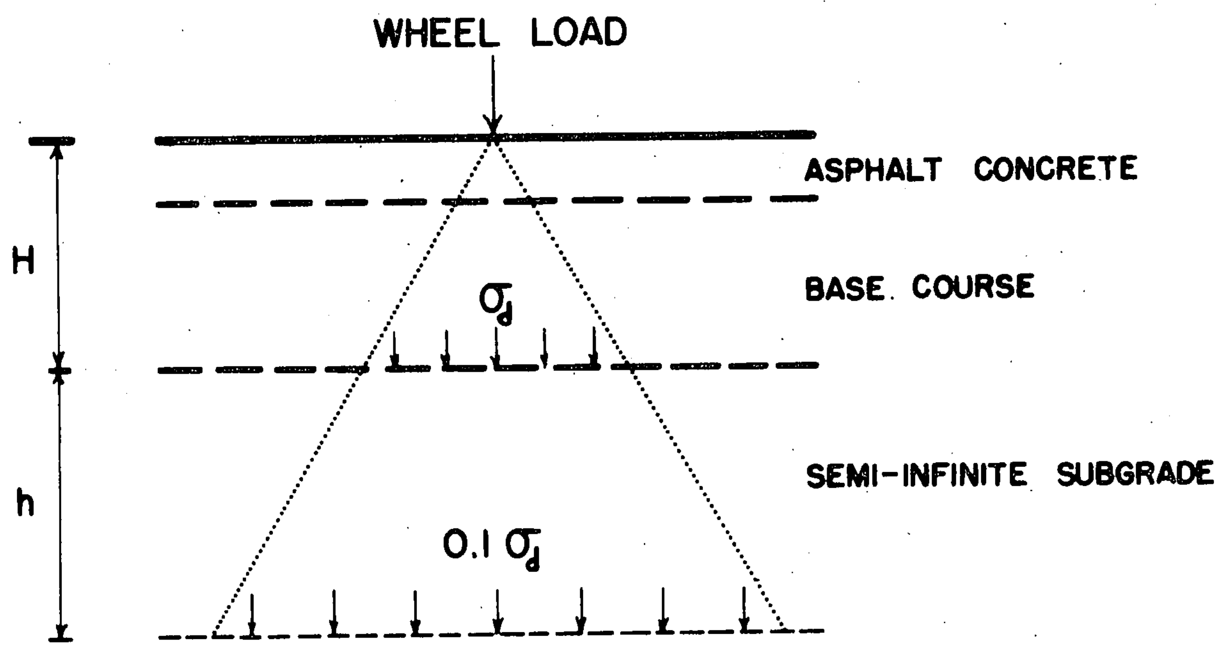
Soil Stress Analysis

In-Situ Subgrade Pressures

Pressures that are present in a pavement structure occur in two forms; 1) static or gravity stress from the existing overburden pavement materials and 2) dynamic stresses induced by cyclic loading of the pavement by vehicle traffic. Laboratory triaxial testing of soil specimens attempted to reproduce the stress conditions by applying confining pressure on all specimen surfaces to simulate gravity stresses and a cyclic, deviator pressure on the specimen ends to reproduce the dynamic loading.

Predicting Static and Dynamic Subgrade Stress

The deviator stress, σ_d, that a soil subgrade layer experiences from dynamic loading of the pavement surface decreases with increasing soil depth as it distributes through the layer until the influence of σ_d becomes 0.1 of its original value (4). The point where this occurs is termed "The Depth of Significant Stress," h, as shown in Figure 2. The average



1. σ_d = STRESS AT BASE-SUBGRADE INTERFACE
2. $0.1\,\sigma_d$ = LIMIT OF SIGNIFICANT STRESS RANGE = DEPTH OF SIGNIFICANT STRESS
3. A.D.S.S. = $h/3$, A.D.S.S. RELATIVE TO ASPHALT CONCRETE SURFACE = $H + h/3$
4. AT A.D.S.S., THE SUBGRADE RADIAL AND VERTICAL CYCLIC STRESSES ARE OBTAINED FROM COMPUTER OUTPUT

Figure 2. Average Depth Of Significant Stress (A.D.S.S.)

depth of significant stress, A.D.S.S., is then found by dividing h by 3. With respect to a road pavement surface, it is equal to H + h/3, where H is equal to the depth of the asphalt concrete and base course layers and h/3 is equal to the depth from the base-subgrade interface to A.D.S.S. The estimated stress measured at the A.D.S.S. provides good approximations of the average pressure present from dynamic loading in the influenced subgrade layer.

Figure 3 shows deviator and confining stress definitions.

With respect to confining stresses (or confining air pressures) used on the test specimens in the triaxial cell, accurate simulation in testing requires σ_1 to be present on the specimen top and σ_3 to support or confine its sides while σ_d is applied in a cyclic manner. Triaxial testing used in this project only supplied one pressure, that being σ_3, to test specimens thereby creating a deficiency in the required pressure balance (σ_1-σ_3) on the vertical axis. However, it was thought that the application of a lower than required σ_1 (= σ_3) will cause a specimen to be, perhaps, more resilient, (M_R is lower) which provides an immediate factor of safety to test data that are being used for design purposes. Also, the applied σ_d stresses are low enough to perhaps minimize the actual vs. theoretical confining stress differences.

Estimating Deviator Stress Intensities Using Chevron 5-L Program

The A.D.S.S. approach was used to approximate deviator stress intensities and the depths where they occur using Chevron 5-L computer output

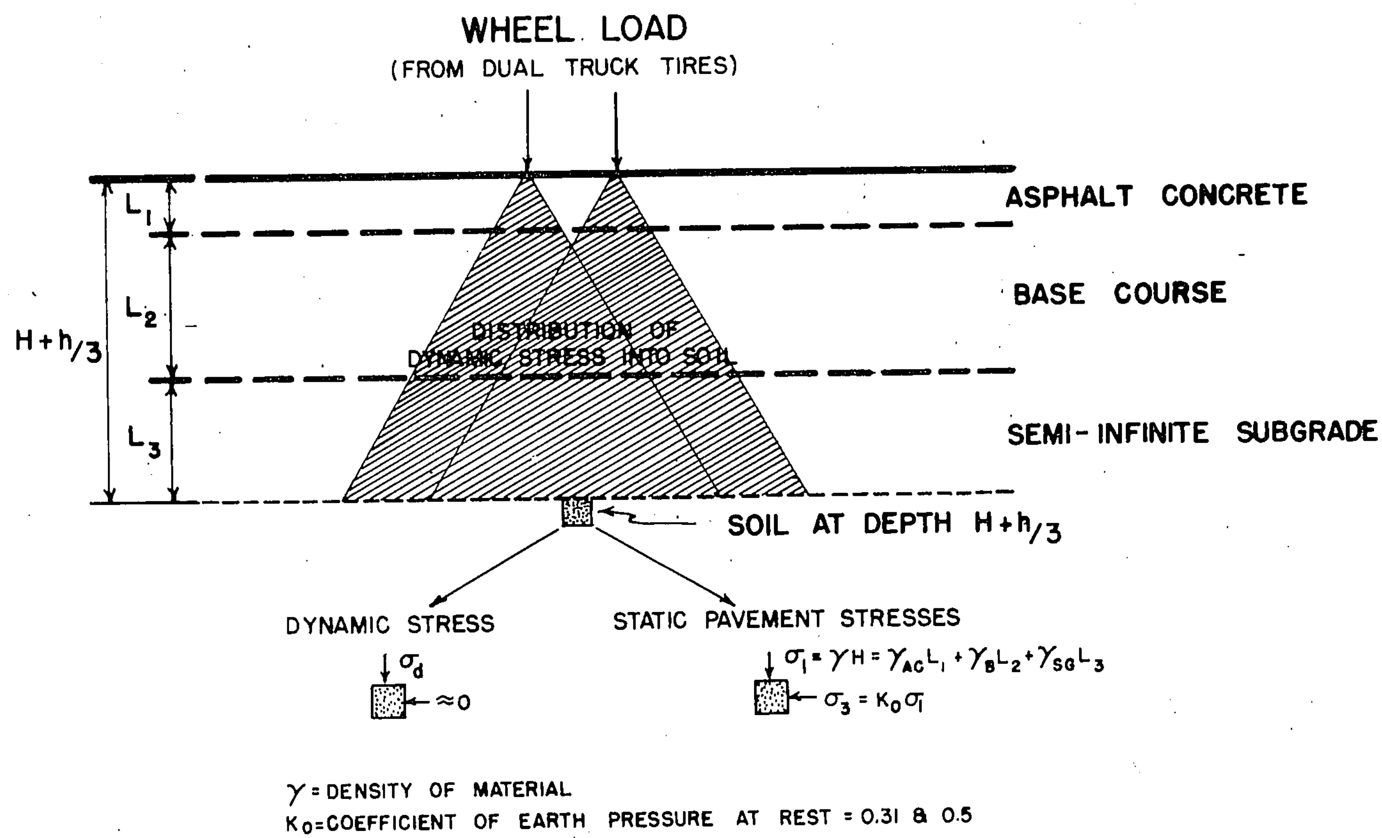


Figure 3. Pressure Present In Subgrade Soil From Dynamic And Static Loading.

(using information example of Table 5 from input data of Table 4). The static stresses from overburden pavement materials at these depths were then computed as shown in Figure 3, their range being the basis for selection of the confining pressures, $\sigma_3 = k_o\sigma_1$, used in repetitive load triaxial testing.

The Chevron 5-L program calculates stresses, strains, and deflections for specified points in individual layers that compose an asphalt concrete pavement system which is subjected to surface loading. The following data are needed for analysis:

1. Thickness of the pavement system layers,
2. M_R of the pavement system layers,
3. Poissons Ratio of the pavement system layers, and
4. Tire load and pressure on the pavement surface.

Table 5 shows the variables that were input in various combinations into Chevron 5-L to estimate the range of cyclic vertical and radial stresses possible in pavement subgrade layers at average depths of significant stress.

TABLE 4

Chevron 5-L Data Input

Layer	Thickness-(in.)	Resilient Modulus (psi)	Poissons Ratio
A.C.	3.6 - 10.0	150,000-750,000	0.35
Base Course	0 - 15	15,000 & 35,000	0.35
Subgrade	semi-infinite	3,000-15,000	0.40

<u>TABLE 5</u>

Example of Stress Information
from Chevron 5-L Computer Output

```
********************* ANALYSIS OF A.C. PAVEMENT DESIGN LAYERED SYSTEM          ********************* PAGE 1
                                THE PROBLEM PARAMETERS ARE

        TOTAL LOAD..          9000.00  LBS
        TIRE PRESSURE..         70.00  PSI
        LOAD RADIUS..            6.40  IN.

        LAYER 1 HAS MODULUS   350000.   POISSONS RATIO 0.350   AND THICKNESS  10.00 IN.
        LAYER 2 HAS MODULUS    35000.   POISSONS RATIO 0.350   AND THICKNESS   6.00 IN.
        LAYER 3 HAS MODULUS     9000.   POISSONS RATIO 0.400   AND IS SEMI-INFINITE.
```

			STRESSES					DISPLACEMENT	STRAINS		
	R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK	VERTICAL	RADIAL	TANGENTIAL	BULK
Asphalt	0.0	5.0	-4.228E 01	-7.638E 00	-7.638E 00	0.0	-5.756E 01 *	1.567E-02 *	2.810E-05	2.810E-05	-4.934E-05
Concrete	0.0	-10.0	-8.047E 00	6.971E 01	6.971E 01	0.0	1.314E 02 *	1.508E-02 *	1.375E-04	1.375E-04	1.126E-04
	0.0	10.0	-8.047E 00	3.071E 00	3.071E 00	0.0	-1.904E 00 *	1.505E-02 *	1.375E-04	1.375E-04	-1.632E-05
Base	0.0	13.0	-4.957E 00	4.155E 00	4.155E 00	0.0	3.404E 00 *	1.432E-02 *	1.274E-04	1.274E-04	2.917E-05
	0.0	-16.0	-3.446E 00	6.389E 00	6.389E 00	0.0	9.332E 00 *	1.366E-02 *	1.531E-04	1.531E-04	7.999E-05
1)>	0.0	16.0	-3.446E 00	-6.916E-04	-6.916E-04	0.0	-3.448E 00 *	1.366E-02 *	1.531E-04	1.531E-04	-7.661E-05
Subgrade	0.0	18.5	-2.935E 00	-3.437E-02	-3.437E-02	0.0	-3.003E 00 *	1.278E-02 *	1.281E-04	1.281E-04	-6.674E-05
	0.0	20.0	-2.691E 00	-4.440E-02	-4.440E-02	0.0	-2.780E 00 *	1.232E-02 *	1.166E-04	1.166E-04	-6.178E-05
	0.0	25.0	-2.094E 00	-5.221E-02	-5.221E-02	0.0	-2.199E 00 *	1.102E-02 *	8.959E-05	8.959E-05	-4.886E-05
	0.0	30.0	-1.695E 00	-4.605E-02	-4.605E-02	0.0	-1.787E 00 *	9.997E-03 *	7.226E-05	7.226E-05	-3.971E-05
	0.0	35.0	-1.407E 00	-3.842E-02	-3.842E-02	0.0	-1.484E 00 *	9.158E-03 *	5.997E-05	5.997E-05	-3.247E-05
3)>	0.0	40.0	-1.189E 00	-3.250E-02	-3.250E-02	0.0	-1.254E 00 *	8.455E-03 *	5.367E-05	5.367E-05	-2.786E-05
	0.0	45.0	-1.019E 00	-2.862E-02	-2.862E-02	0.0	-1.075E 00 *	7.854E-03 *	4.337E-05	4.337E-05	-2.391E-05
	0.0	50.0	-8.830E-01	-2.634E-02	-2.634E-02	0.0	-9.356E-01 *	7.343E-03 *	3.749E-05	3.749E-05	-2.079E-05
	0.0	55.0	-7.737E-01	-2.511E-02	-2.511E-02	0.0	-8.234E-01 *	6.895E-03 *	3.269E-05	3.257E-05	-1.830E-05
	0.0	60.0	-6.834E-01	-2.444E-02	-2.404E-02	0.0	-7.323E-01 *	6.503E-03 *	2.874E-05	2.874E-05	-1.627E-05
	0.0	65.0	-6.093E-01	-2.399E-02	-2.300E-02	0.0	-6.573E-01 *	6.135E-03 *	2.540E-05	2.543E-05	-1.461E-05
	0.0	70.0	-5.478E-01	-2.352E-02	-2.352E-02	0.0	-5.949E-01 *	5.844E-03 *	2.278E-05	2.278E-05	-1.322E-05
	0.0	75.0	-4.954E-01	-2.284E-02	-2.284E-02	0.0	-5.422E-01 *	5.565E-03 *	2.054E-05	2.054E-05	-1.205E-05
	0.0	80.0	-4.532E-01	-2.205E-02	-2.205E-02	0.0	-4.973E-01 *	5.312E-03 *	1.867E-05	1.867E-05	-1.105E-05
	0.0	85.0	-4.166E-01	-2.093E-02	-2.093E-02	0.0	-4.585E-01 *	5.080E-03 *	1.712E-05	1.712E-05	-1.019E-05
	0.0	90.0	-3.854E-01	-1.959E-02	-1.959E-02	0.0	-4.245E-01 *	4.866E-03 *	1.582E-05	1.582E-05	-9.435E-06
2)>	0.0	95.0	-3.586E-01	-1.804E-02	-1.804E-02	0.0	-3.947E-01 *	4.668E-03 *	1.474E-05	1.474E-05	-8.772E-06
	0.0	100.0	-3.355E-01	-1.633E-02	-1.633E-02	0.0	-3.691E-01 *	4.493E-03 *	1.382E-05	1.382E-05	-8.181E-06
	0.0	105.0	-3.153E-01	-1.449E-02	-1.449E-02	0.0	-3.443E-01 *	4.309E-03 *	1.305E-05	1.305E-05	-7.651E-06
	0.0	110.0	-2.975E-01	-1.257E-02	-1.257E-02	0.0	-3.227E-01 *	4.145E-03 *	1.239E-05	1.239E-05	-7.171E-06

Procedure to Compute A.D.S.S. and Vertical Cyclic Stress

1) Vertical Compressive Stress from Wheel Load at Base-Subgrade Interface (σ_I=3.446 psi.).

2) Point Where σ_I is 0.1 its Original Value ($0.1\sigma_I$=.3446 psi., Depth of Significant Stress=98"-16"=82").

3) a. A.D.S.S. at Base-Subgrade Interface = h/3 = 82"/3 = 27.3".
   b. A.D.S.S. with Respect to the Pavement Surface = H + h/3 = 16" + 27.3" = 42.3".
   c. A.D.S.S. = 42.3", Corresponding Vertical Cyclic Stress, σ_d = 1.1psi.(Values Interpolated).

The range of computed deviator stresses varied from approximately 0.5 psi to 6.0 psi while radial stresses were never larger than 0.06 psi. Figure 4 shows the ranges obtained. Consequently the cyclic deviator stress, σ_d, during repeated load triaxial testing varied in increments from 0.5 to 6.5 psi.; cyclic radial stresses, being small in relation to the static confining pressures, were considered to be zero for testing purposes.

Poissons ratio was not varied because: 1) pavement response appeared comparatively insensitive to small changes in this parameter, and 2) it is relatively constant for the range of triaxial cell test pressures that were used (10) (11).

Test Pressures and Their Sequence

Results of triaxial testing projects described in the literature show that the testing pressure sequence of cyclic or deviator stress, σ_d, and confining pressure (stress), σ_3, does not affect soil M_R values, especially at low testing stresses (2) (11). Since small dynamic and confining stresses were used during this test program, test stresses could be applied in any sequence which simplified and reduced the running time of tests. The following testing stress sequence was used:

1. $\sigma_d \simeq 1.0, 2.0, 3.5, 4.5, 6.5$ psi
2. $\sigma_3 = 0.0, 1.0, 2.0, 3.0$ psi

Each specimen's testing was initiated at a confining stress of 3 psi, and a series of deviator stresses then applied in the sequence shown above,

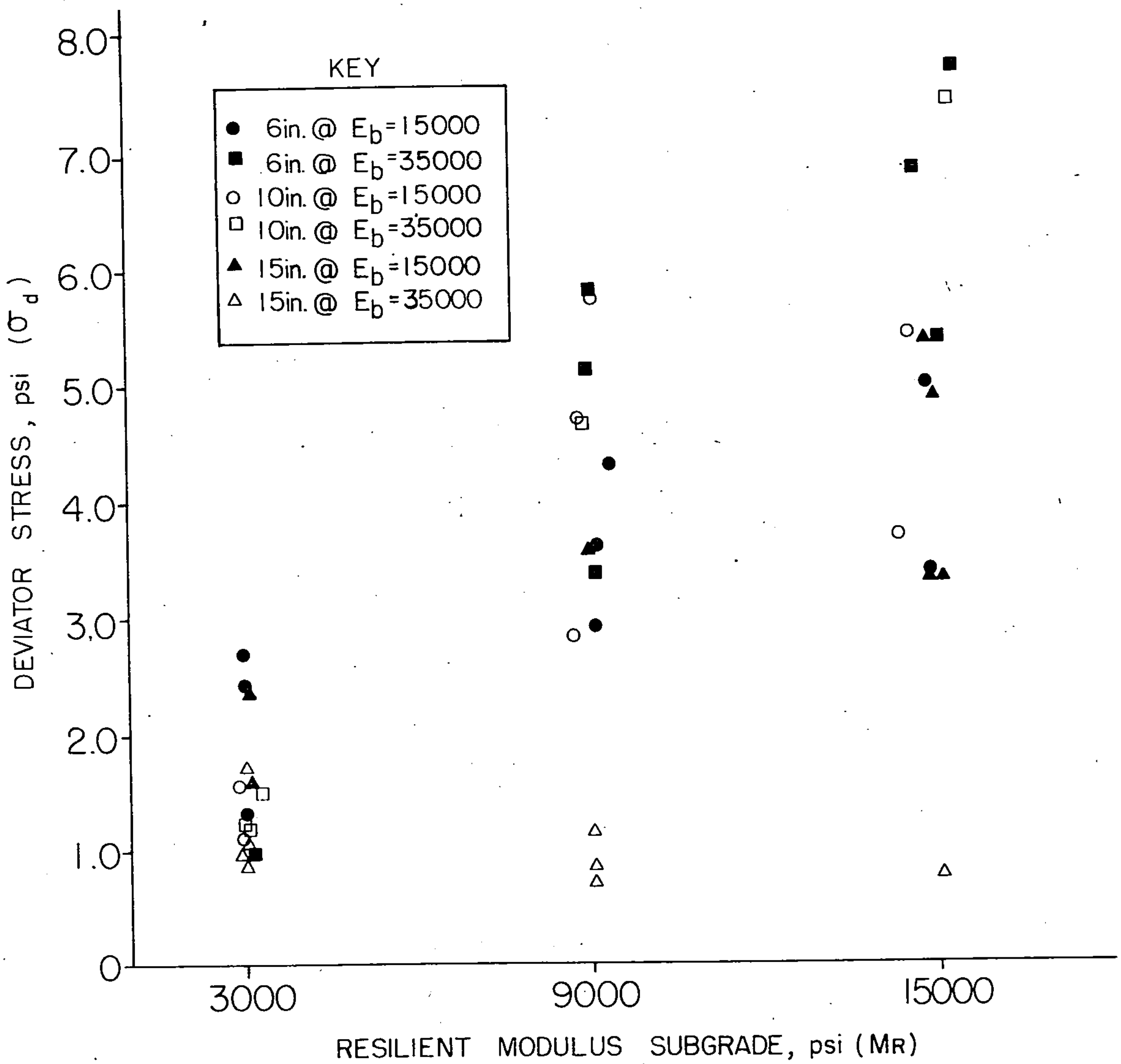


Figure 4. Chevron 5L Output: Deviator Stress Vs. Subgrade Resilient Modulus.

i.e. 1.0 to 6.5 psi. (The number of load cycles and testing procedures are discussed in Section 3). Next, the confining stress was decreased to 2.0 psi and another sequence of deviator stresses applied. Confining stress was subsequently decreased to 1.0 and then to 0.0 psi, with the sequence of applied σ_d maintained at each confining stress.

Specimen Size

Four-inch diameter specimens were compacted to approximately 5 inches high since the Idaho kneading compactor used for 4-inch diameter specimens cannot fabricate specimens higher than six inches without modification. R. A. Bloomfield (1), using lubricated loading plates at the ends of each test specimen to eliminate or minimize end friction, concluded that soil specimens of height $\geq$ 5.0 inches with a diameter of 4.0 inches could be used in M_R tests without appreciable effect on a soil's M_R value, whereas some other research programs, probably not using lubricated end plates, have required that a specimen height to diameter ratio be $\geq$ 2.0. The height to diameter ratio of $\geq$ 2.0 has been required primarily to eliminate specimen end effects that are caused by friction between specimen ends and unlubricated loading plates. The result of end effects is the alteration of values of dynamic Poissons ratio and soil shear strength. Consequently the practice of testing specimens of height around 5.0 inches was continued since only M_R was being evaluated. Specimen size is also discussed later in Section C.

SECTION C

SOIL SPECIMEN PREPARATION, TESTING AND RESILIENT MODULUS DETERMINATION

Resilient Modulus Device and Equipment Calibration

The prototype resilient modulus device as shown in Figure 5, has a Bellofram air cylinder mounted to it that applies the deviator stress pulses to soil specimens. The air cylinder is triggered by the use of a solenoid valve and triac switch; air pressure is controlled by an air regulating valve. Laboratory line air pressure is used.

Specimen total deformation is measured by an LVDT. Since specimen deformation is small, an LVDT that must be calibrated in a travel range of 0.25 inches will provide accurate readings.

The load cell, which measures the load applied to a specimen, is located between the Bellofram and triaxial cell shaft.

Figure 6 shows a Wavetek signal generator used to control the loading duration and frequency. With this generator, the loading time and no-load time are dependent. Calculations of vehicle load times at soil A.D.S.S. and consultation in the literature gave some average values. Therefore, the load duration time and no-load time per cycle for the project was set at 0.15 seconds and 1.85 seconds, respectively.

Differential alternating current signals from the load cell and LVDT are fed into signal conditioners, shown in Figure 7, and linearly transformed to differential dc signals that are transmitted to a Visicorder. The signal conditioners should be checked periodically for signal linearity and calibration drift to prevent erroneous data.

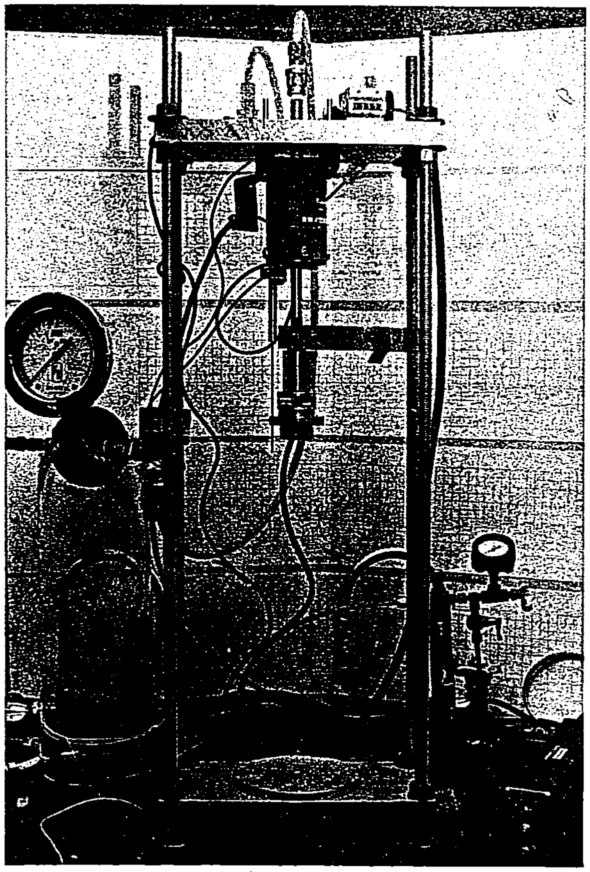


Figure 5. Resilient Modulus Test Device with Adjustable Loading Plate Showing Bellofram, LVDT, and Load Cell.

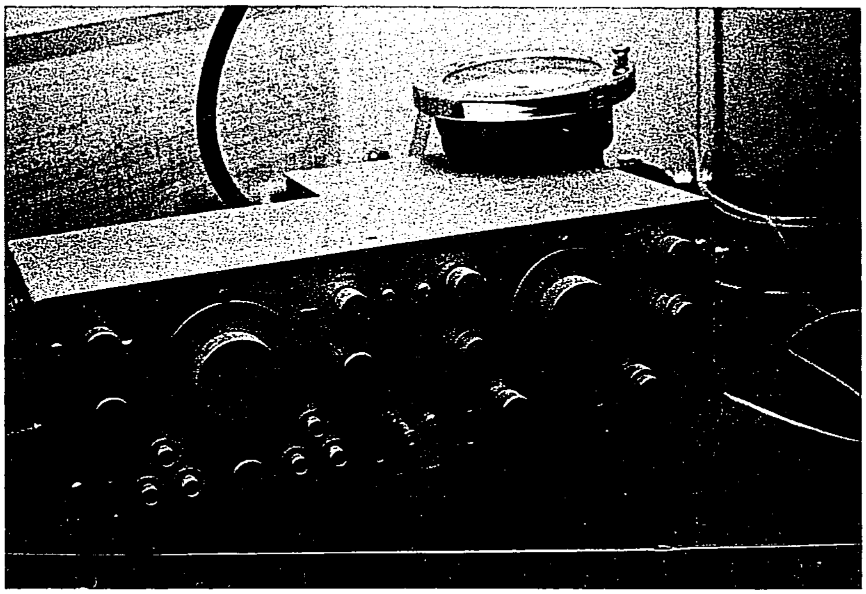


Figure 6. Wavetek Generator for Producing Load Pulses.

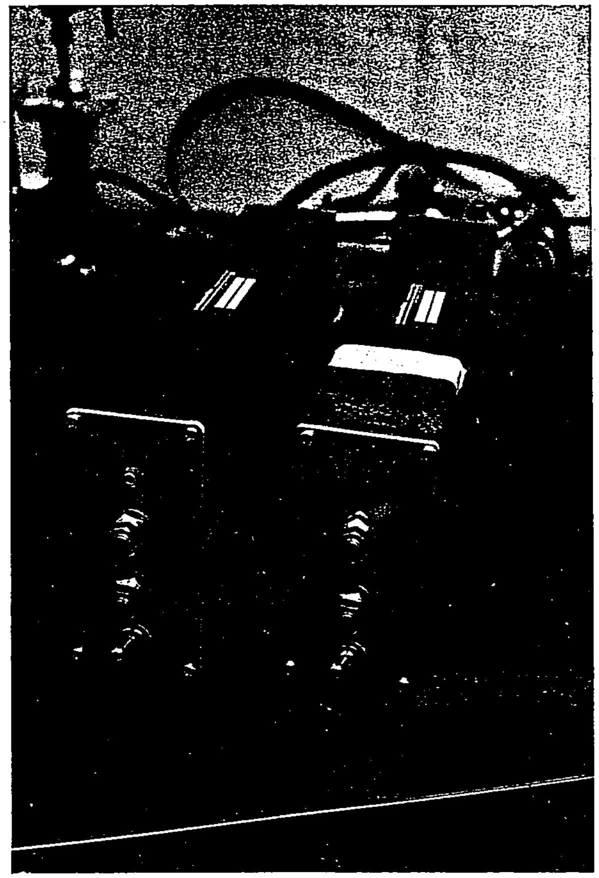


Figure 7. Signal Conditioners Which Convert Signals from the LVDT and Load Cell from ac to dc for Oscillograph Recordings.

The linear dc load and deflection signals are traced by means of a Visicorder light beam on light sensitive tracing paper as shown in Figure 8. The trace lights are calibrated as follows:

1. A proving ring with a dial gage reading to 10^{-4} in. is placed in the device, and a repeated load is applied as in Figure 9.
2. The maximum amount of deformation (dial reading) is read from the dial gage at the full magnitude of the repeated load pulse.
3. A desirable scale is chosen for the deformation trace on the chart paper in the Visicorder. (Example: 1 vertical div. = 0.0002 in.)
4. The number of divisions needed on the chart paper is found by dividing the dial reading by 0.0002 in. Example: 0.010 in/ 0.0002 = 50 divisions.
5. The gain control on the signal amplifier is adjusted such that the Visicorder traces the desired number of divisions for the deflection trace, eq. 50 divisions.
6. Next, the maximum amplitude of the repeated load is then determined from the calibration curve for the proving ring, the load being a function of deformation obtained in Step 2.
7. Steps 3, 4, and 5 are carried out for the load trace calibration on the chart paper. Periodic calibration checks should be run on the device to ensure accuracy over a testing period, and they should be done at different repeated load magnitudes in order to establish a calibration curve.

Since reading accuracy of tracings is important a scale for deformation traces should be chosen that considerably augments the specimen deformation and at the same time will not exceed the tracing paper width at the highest deformation expected.

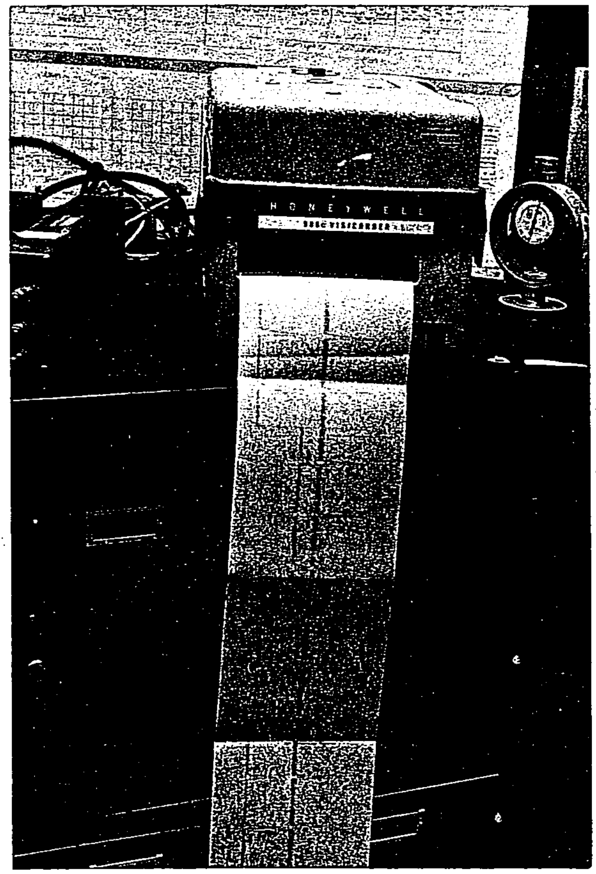


Figure 8. Load and Deflection Amplitudes Printed by Visicorder Oscillograph.

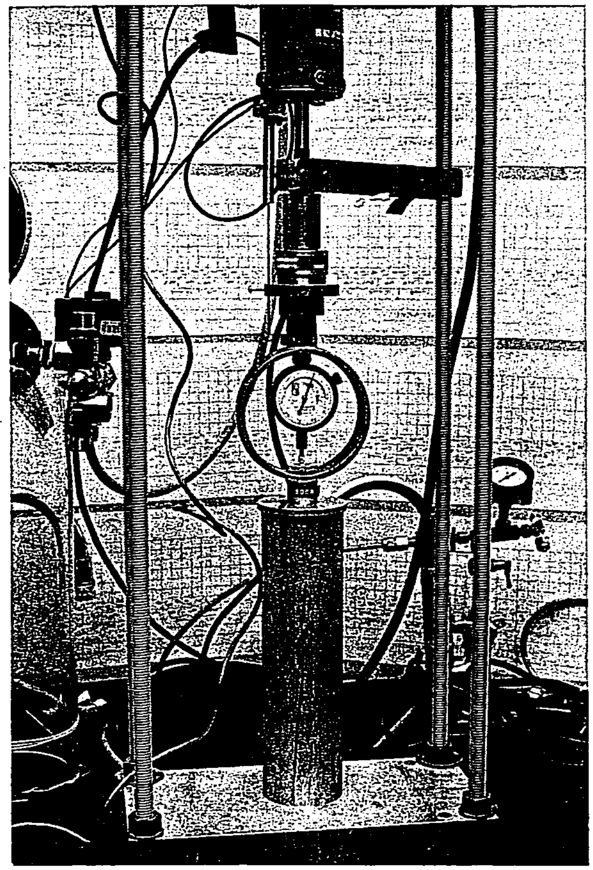


Figure 9. Calibration of Test Device for Load and Deflection Using Proving Ring.

Specimen Preparation

A newly attained sample of subgrade soil should be "room dried". Since specimens 5 in. high and 4 in. diameter are required, the total weight of soil required to produce a specimen this size can be calculated for each soil using data from Table 2 and the following relationship,

Tot. Wet Wt. of Soil Required = (Vol. of mold) X (R-Value Wet Density of Soil).

Preparing soil for compaction should be similar to the following:

1. Select a batch of "room-dried" wet weight of soil and determine the amount of water contained in it.
2. Select the required R-Value water content.
3. The percent water to be added to the soil in (1) to achieve R-Value water content (2) is determined.
4. Add the water content determined in (3) to the soil in (1), and mix:
   (a) for low plasticity soil - use an electric mixer. The mixing should be performed quickly (approximately 45 seconds) to prevent drying of the soil.
   (b) for high plasticity soil - the soil should be mixed as well as possible by hand, adding the water in increments. (Mixing time should be short.) Highly plastic soils are mixed by hand because their cohesive characteristics prevent efficient mixing of water and soil using an electric mixer.

   After mixing is complete a damp cloth can be placed over the wet soil in the mixing bowl to prevent water evaporation prior to compaction.
5. Batch out enough of the soil mix in (4) to be equal to the

total wet weight of soil required for a specimen. Note: The soil batch weight should be adequate to fill the 4-inch diameter soil compaction mold to the 5-inch net height after compacted to the R-Value density by kneading compactor.

Fabrication of Soil Specimens

Compaction of Soil

To compact the soil, a split mold (6 in. high and 4 in. in diameter), shown in Figure 10, is coated with a spray lubricant or light oil on its inside surface. The mold is placed on two 1/4 in. spacers, centered under a kneading compactor, and bolted in to place to prevent movement during compaction, as shown in Figure 11. The procedure to use for compaction is then as follows:

1. place approximately 1/2 of the prepared soil in the tray next to the compactor and cover the remainder.
2. the soil is added, as in Figure 12, to the mold in small increments while compacting until a total of 75 blows at 75 psi. are reached. This process is referred to as "precompaction".
3. add the last half of prepared soil to the tray and repeat step 2.
4. the spacers are then removed and the sample is compacted for 50 blows at 75 psi. With the spacers removed the mold will remain approximately 1/4 in. above the base of the compactor due to internal pressure being applied both from the compaction foot and the reaction at the base of the machine. This helps to produce a fairly constant density throughout the height of the sample. This is referred to as the "compaction" stage.

The compaction process should yield specimens after extraction with densities within 2.0 pcf of the target required R-Value density. Specimens that do not meet this requirement and whose water contents are not within 0.3% should not be used for M_R testing.

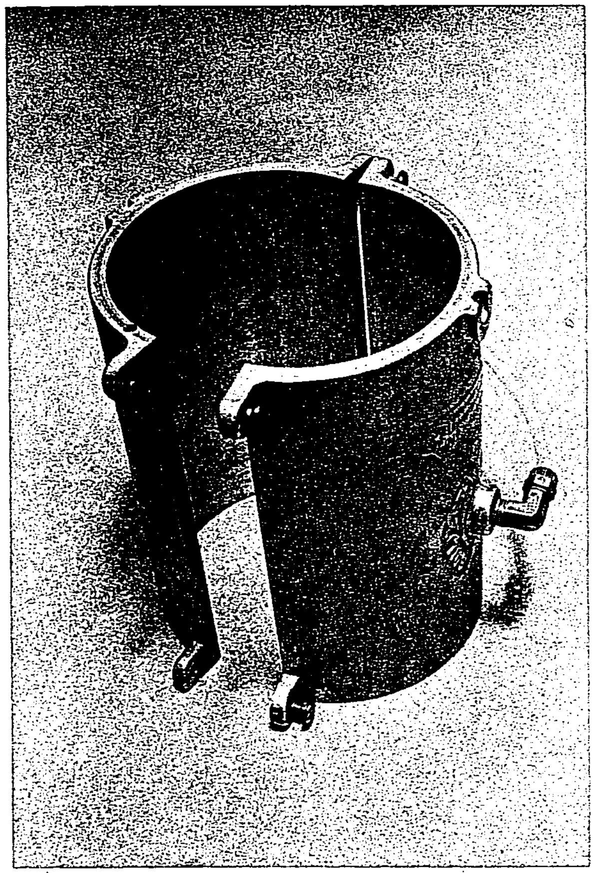


Figure 10. Split Mold Used for Compaction (Open Position).

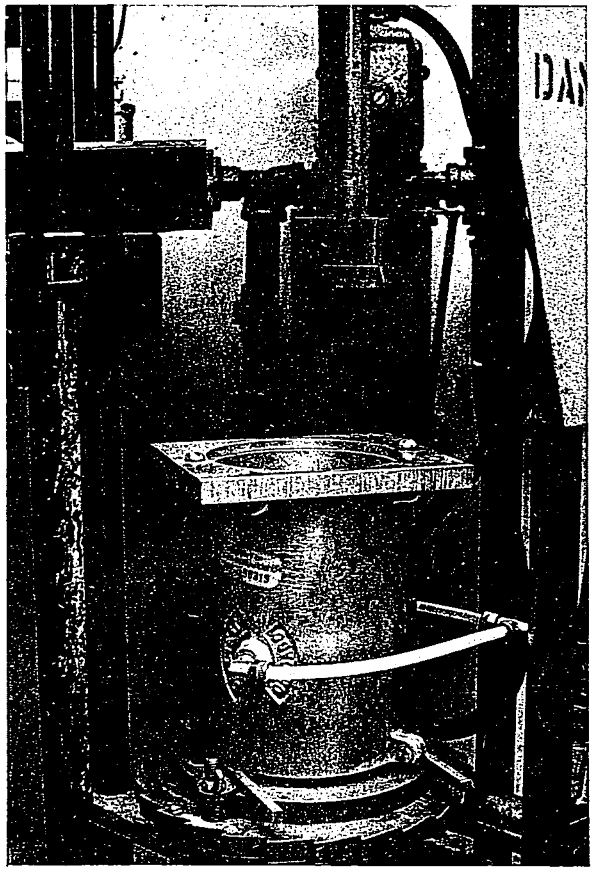


Figure 11. Split Mold Mounted in the Kneading Compactor.

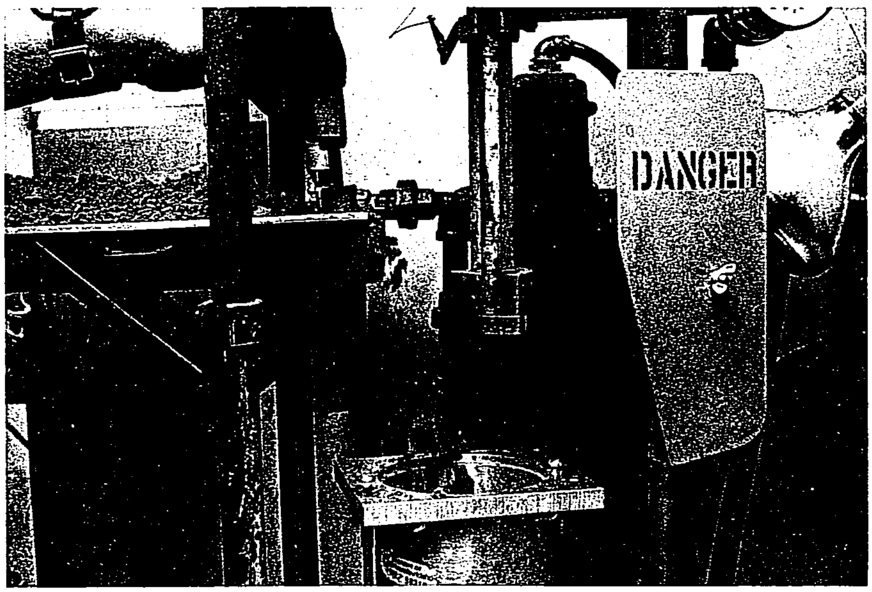


Figure 12. Soil Being Added to the Split Mold During the Compaction Process.

Specimen Extraction

Specimen extraction from the mold may be accomplished in one of two ways depending on the plasticity of the soil:

1. for non plastic soil - the split mold is loosened and the sample is tapped loose from the sides of the mold by tapping the mold ends on a table top. The mold can then be easily pulled apart.
2. for plastic soils - the mold is loosened and the specimen is hydraulically jacked out of the mold as shown in Figure 13.

The lightweight oil or spray lubricant which is applied to the inside surface of the mold before a specimen is made will provide for smoother specimen extraction as evidenced by the smoothness of the specimen in Figure 14.

Preparation for Curing

The soil specimen, shown in Figure 15, is immediately weighed and measured, and its density calculated. If R-Value target density is met, the specimen is then prepared immediately for curing by wrapping it in a double layer of saran wrap and placing it in 2 to 3 plastic bags, shown in Figure 16, and sealing each bag.

Curing Specimens

The curing or "aging" of specimens is a simple but necessary procedure. Two processes may occur during the curing period:

1. The water content stabilizes throughout the sample. Although the soil and water are thought to be well mixed before compaction, there may be areas of the sample containing more moisture than other areas. The sample should have a homogeneous water content at the time of testing.

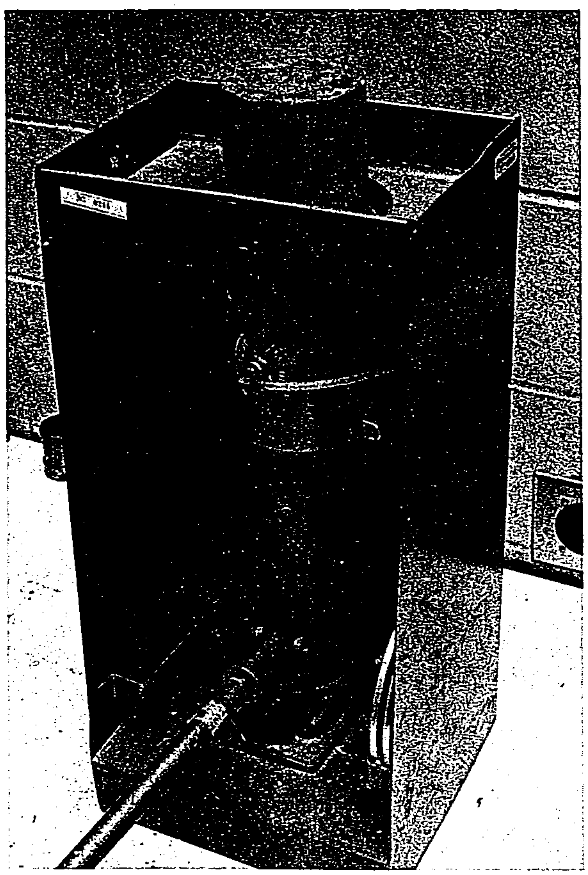


Figure 13. Extraction of a Compacted Soil Specimen from the Mold Using a Hydraulic Jack.

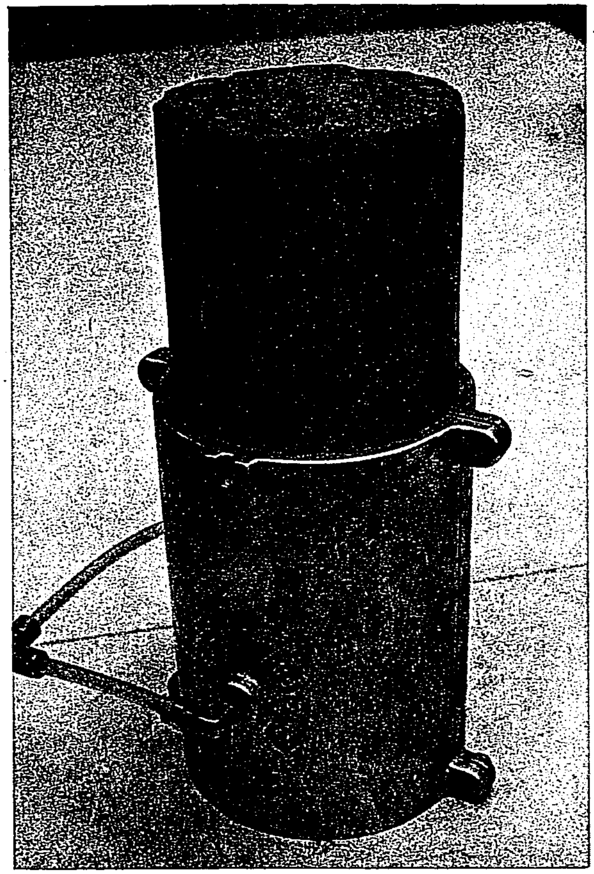


Figure 14. Compacted Soil Specimen Before its Complete Extraction.

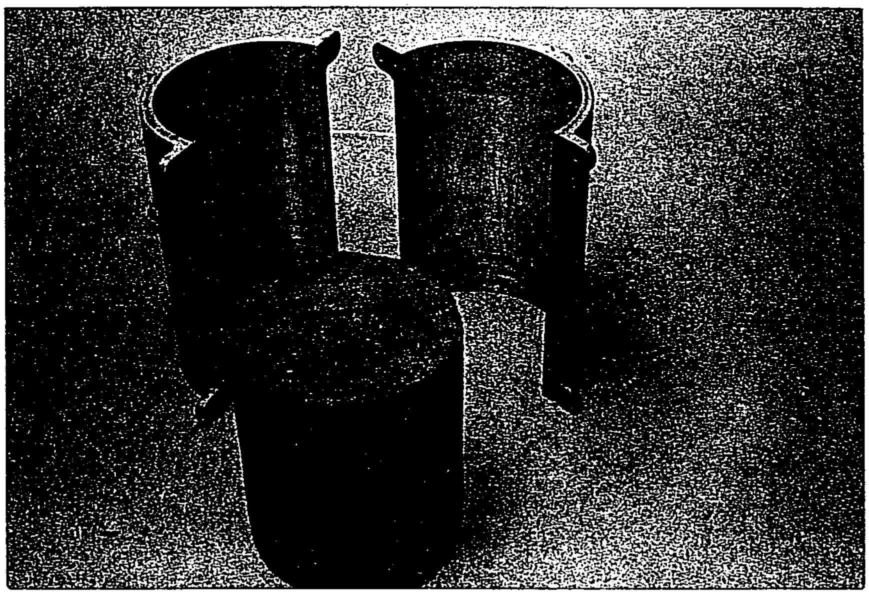


Figure 15. Compacted Soil Specimen Immediately After Extraction from the Split Mold.

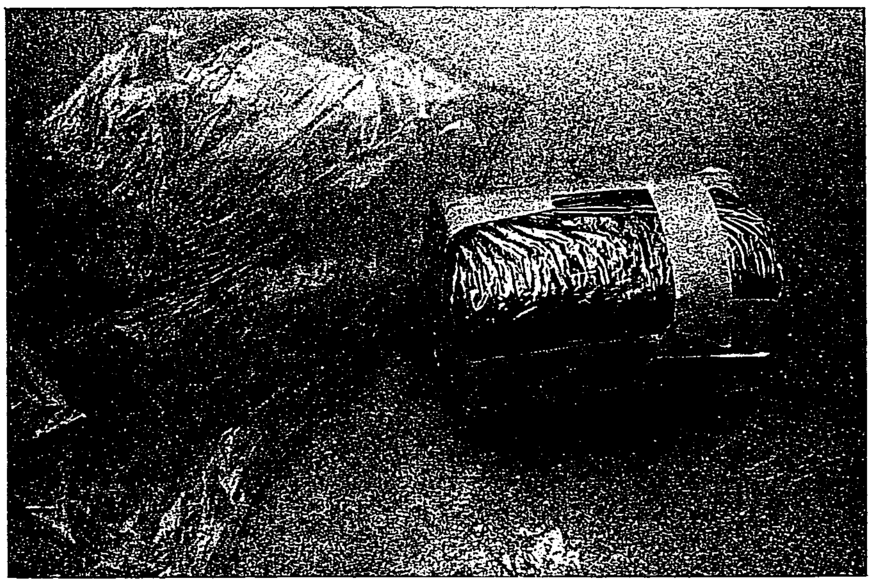


Figure 16. Compacted Soil Specimen Wrapped in Saran Wrap Showing One of the Bags it is to be placed in and Sealed for Curing.

2. Soils which show plastic characteristics (P.I. > 0) exhibit a thixotropic strength gain. During kneading compaction soil particles are strained into a dispersed configuration. With time, soil particles reorient and regain a portion of the strength that was lost from compaction.

Wrapped specimens are placed horizontally on a soft pad in a concrete moisture room for at least one week before testing. A curing period of one week is thought to be sufficient to develop most of the thixotropic strength gain effects on the resilient modulus for plastic soils and will allow for most of the water dispersion through specimens (1) (9).

Testing Procedure

Specimen Preparation for Testing

When the curing period is complete, the specimen should be unwrapped, weighed to ensure there was no moisture loss, and its ends trimmed to level, as shown in Figure 17.

Figure 18 shows the attachment pieces used to prepare a specimen for repeated load triaxial testing. The loading caps are lubricated with a thin film of silicone grease and a thin rubber membrane is placed bewteen the specimen and each of the lubricated ends of the loading caps. This is done to minimize the effect of end friction on the specimen during test.

A .0014 in. thick rubber membrane coated with talc powder is pulled over the specimen (Figure 19) without damaging the specimen ends. If the specimen has little plasticity, a load cap should be placed on it and the membrane pulled over the specimen and cap to prevent specimen end damage.

The membrane is pulled down to where its end is only an inch above the cap surface and two "O" rings are then stretched around the cap. The cap and specimen are turned over as in Figure 20, and the membrane is then pulled tightly over a second cap placed on top of the specimen, with "O" rings being placed as before. Figure 21 demonstrates how a specimen prepared for testing should appear. Excess lengths of membrane are folded back over the "O" rings and along the specimen sides.

Placing the membrane over the specimen prevents air migration through it during testing to prevent moisture loss, and allows the compressed air surrounding it to provide the required triaxial cell confining pressure.

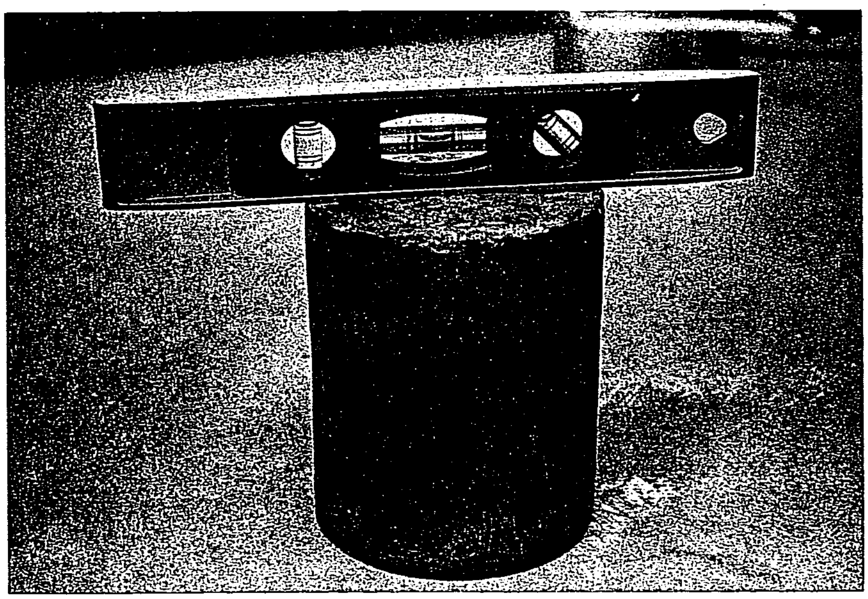


Figure 17. Leveling of the Compacted Soil Specimen Ends After Being Trimmed for Level.

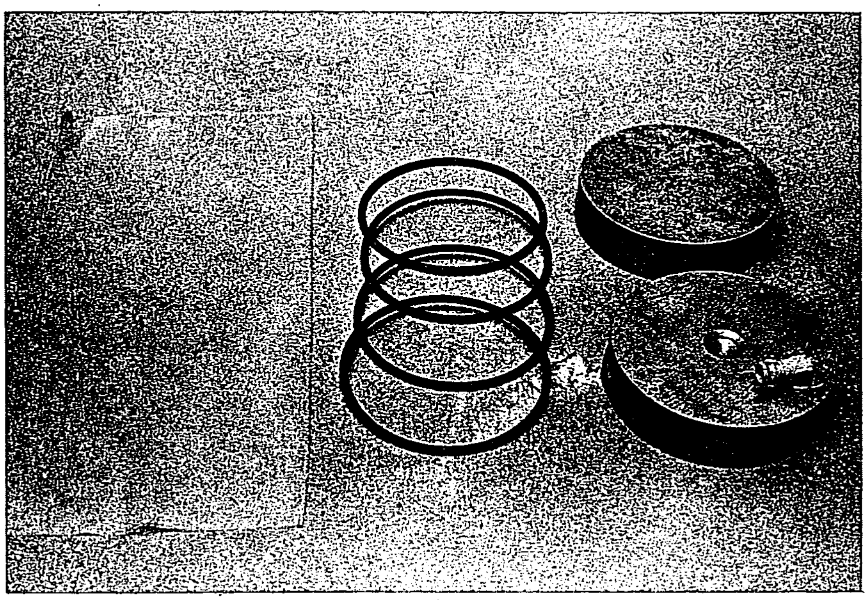


Figure 18. Parts Used for Covering the Compacted Soil in Preparation for Confining Pressure: 1) .014" Membrane, 2) "O" Rings, 3) Load Caps.

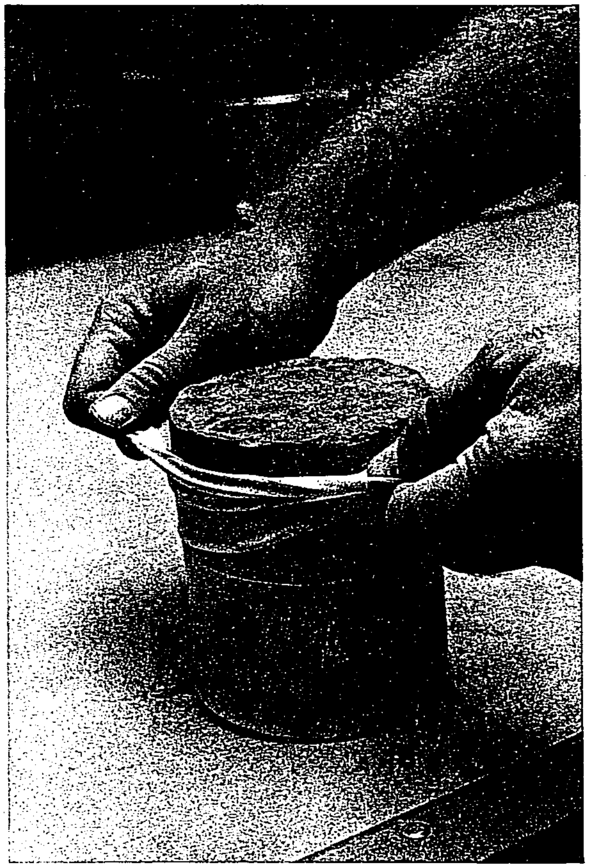


Figure 19. Placing the Membrane Over the Compacted Soil Specimen, Showing Minimum Disturbance of Specimen Ends.

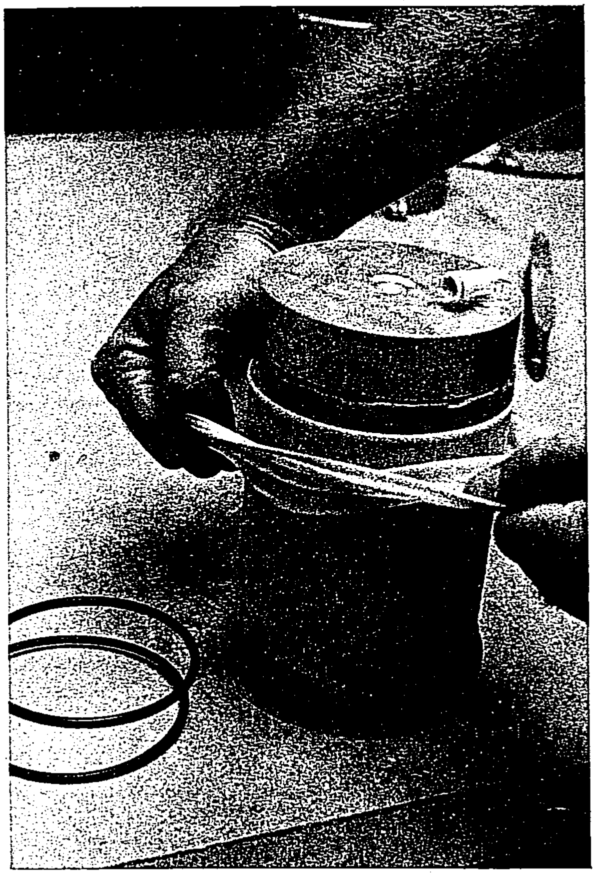


Figure 20. Compacted Soil Specimen Partially Prepared for Placement in Triaxial Apparatus.

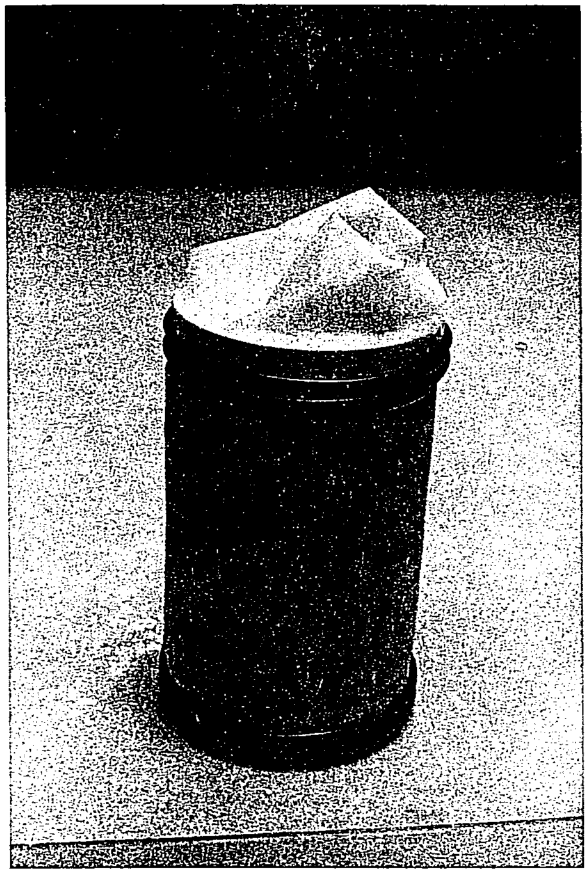


Figure 21. Compacted Soil Specimen Completely Prepared for Placement in Triaxial Apparatus.

Enclosing the Specimen in the Triaxial Cell

The prepared specimen is placed on the triaxial cell base and a flexible tube is connected to the top loading cap as shown in Figure 22. A small hole (1/32" diameter) has been drilled through the cap, the flexible hose connected to a fitting that is countersunk around the hole, and the hose is then attached to a fitting in the cell base which is connected by a reamed crosscut to a turnstop. This provides an open line from the specimen interior to atmosphere and serves two purposes:

1. to determine if there is an air leak from the triaxial chamber through the specimen.
2. to eliminate any internal pressures which could develop from compression by the confining cell pressure.

An air leak can be a result of any one of the following:

1. a hole in the membrane caused from previous testing or simply a defect.
2. improper placement of the "O" rings. They should be approximately 1/4 in. apart.
3. voids between the load cap sides and the membrane that cannot be eliminated by the "O" rings because of trapped soil particles. After each test the loading caps sides should be cleaned and a small amount of silicone grease applied to them to ensure a tight fit of the membrane to the caps sides.

A steel ball is placed in an indent reamed into the top load cap. Its function is to provide a strictly vertical transfer of load from the triaxial cell rod to the load cap.

The plastic triaxial cylinder (Figure 23) is lowered on to the base,

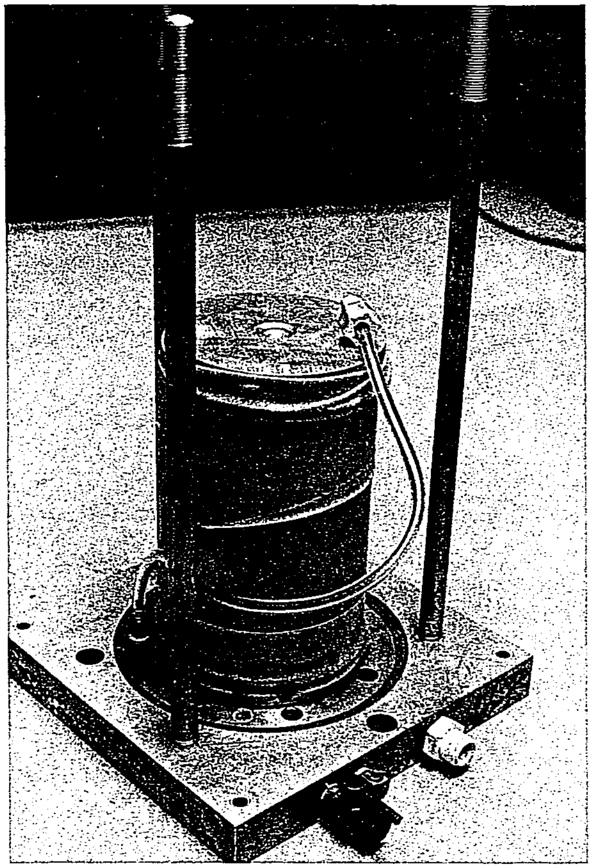


Figure 22. Compacted Soil Specimen on the Triaxial Apparatus Base Showing the Tube That Connects the Base to the Specimen and Atmosphere for Detecting Membrane Leaks.

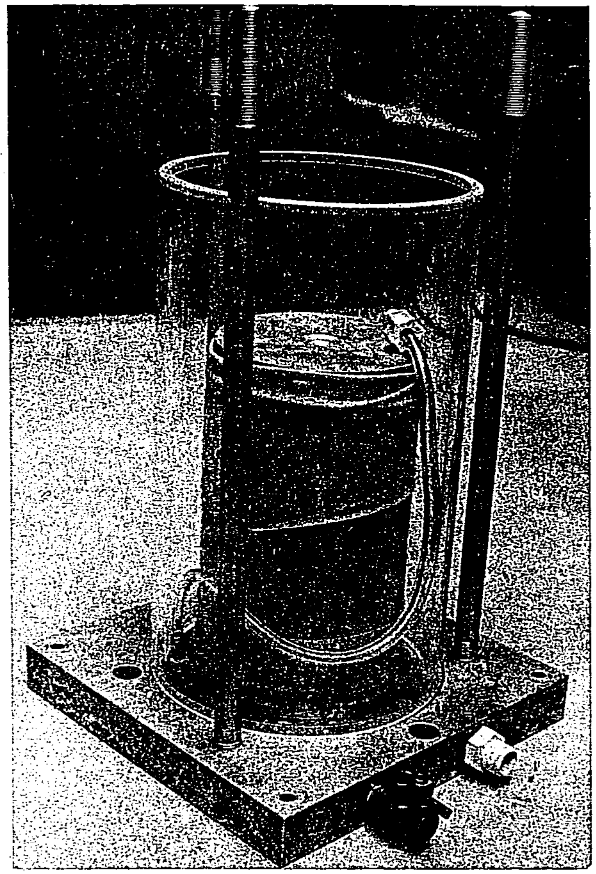


Figure 23. Plastic Triaxial Cylinder Placed Around the Compacted Soil Specimen.

the cell cap placed on it, and secured by bolted rods as shown in Figure 24. A centering plate is located between the ball and triaxial cell rod. It provides a vertical load transfer between the ball and loading rod when one is not centered to the other. The triaxial rod is also lubricated with a light oil to keep friction between the rod and its bushing at a minimal, thereby providing accurate specimen load readings.

The specimen is now ready for testing and may be placed in the resilient modulus test device and connected to it.

Testing Pressures

Figure 25 shows the triaxial apparatus, specimen, and test device prepared for testing. First, air pressure into the triaxial cell is regulated to produce 3 psi, then all connections and joints are checked for leakage. The confining pressure, σ_3, of 3 psi is maintained and a series of deviator stresses, σ_d, of approximately 1.0, 2.0, 3.5, 4.5, and 6.5 psi are applied to the specimen in ascending order. The confining pressure is then decreased systematically to 2, 1, and 0 psi when a complete series of deviator stresses, σ_d, has been run for each confining pressure, σ_3.

For each σ_d-σ_3 combination, a prescribed number of loading cycles must be applied to the specimen before M_R readings can be obtained. Resilient modulus may increase as the number of load cycles increases, M_R becoming constant after a certain number of cycles have been applied. It is therefore recommended that the following number of cycles be applied before readings are taken:

1. high to medium plastic soils - 600 cycles
2. low and non-plastic soils - 400 cycles

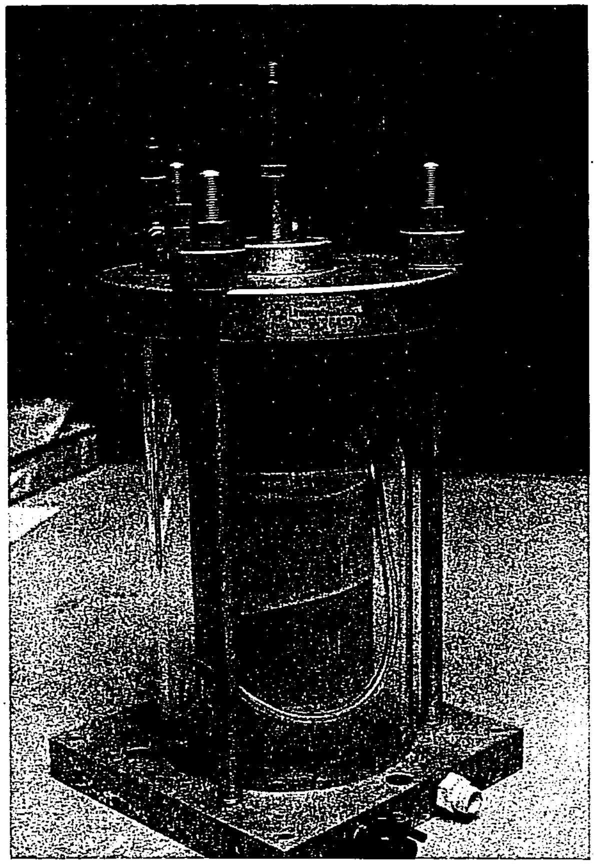


Figure 24. Triaxial Cell Apparatus with a Compacted Soil Specimen at Beginning of Resilient Modulus Testing.

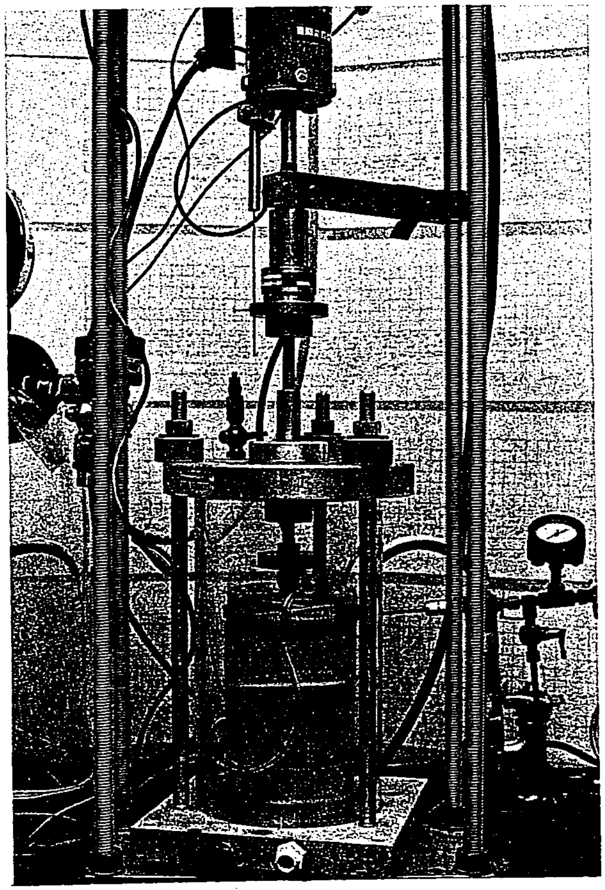


Figure 25. Resilient Modulus Test Device with Triaxial Apparatus.

Load and deformation readings are first taken when the recommended number of cycles are reached, and M_R is then calculated. After an additional 200 cycles, readings are then taken again to verify that M_R is not changing appreciably. If change is still occurring, an additional 200 cycles should again be added and readings taken, the process continuing until M_R is constant for the particular σ_d-σ_3 combination.

The load pulse duration time and total individual cycle time should be characteristic of the actual expected conditions. For this project, a pulse time of 0.15 seconds and a cycle time of 2.0 seconds was used (with approximately 1.85 seconds at no load or rest).

To record data, the Visicorder chart drive is turned on and three or more deformation and deviator load pulse cycles are recorded. The pulse amplitudes are averaged for deformation and for deviator load for the cycle set, and are used to calculate M_R at the particular σ_d-σ_3 combination. Averaging a pulse amplitude cycle set minimizes any errors present from data acquisition.

Calculation of Resilient Modulus, M_R

From each combination of σ_d and σ_3, a set of load and deformation readings is taken. A sample trace of one set is shown in Figure 26. In this example, the load trace has been calibrated such that each vertical division is equal to 5 lbs. The actual specimen deformation has been calibrated to the curve shown in Figure 27 and is a function of measured deformation read from Visicorder traces. (Figure 27 will be discussed later).

$$M_R = \frac{\text{Stress Amplitude}}{\text{Strain Amplitude}}$$

where,

stress amplitude = load/cross sectional area of specimen

strain amplitude = actual specimen deformation/original specimen height (actual deformation is read from Figure 27).

From the sample trace:

load = 46 lbs.

measured deformation = .00125 in.

actual deformation (from Figure 27) = .0018 in.

thus,

stress amplitude = 46 lbs./13.156 in^2 = 3.50 lbs/in^2 (= σ_d)

strain amplitude = .0018 in./4.94 in. = 3.64×10^{-4}

$$M_R = \frac{3.50 \text{ psi}}{3.64 \times 10^{-4}} = 9607 \text{ psi.}$$

Note: These data are for a 4-inch diameter x 4.94-inch high specimen tested at a confining pressure of σ_3 = 3.0 psi.

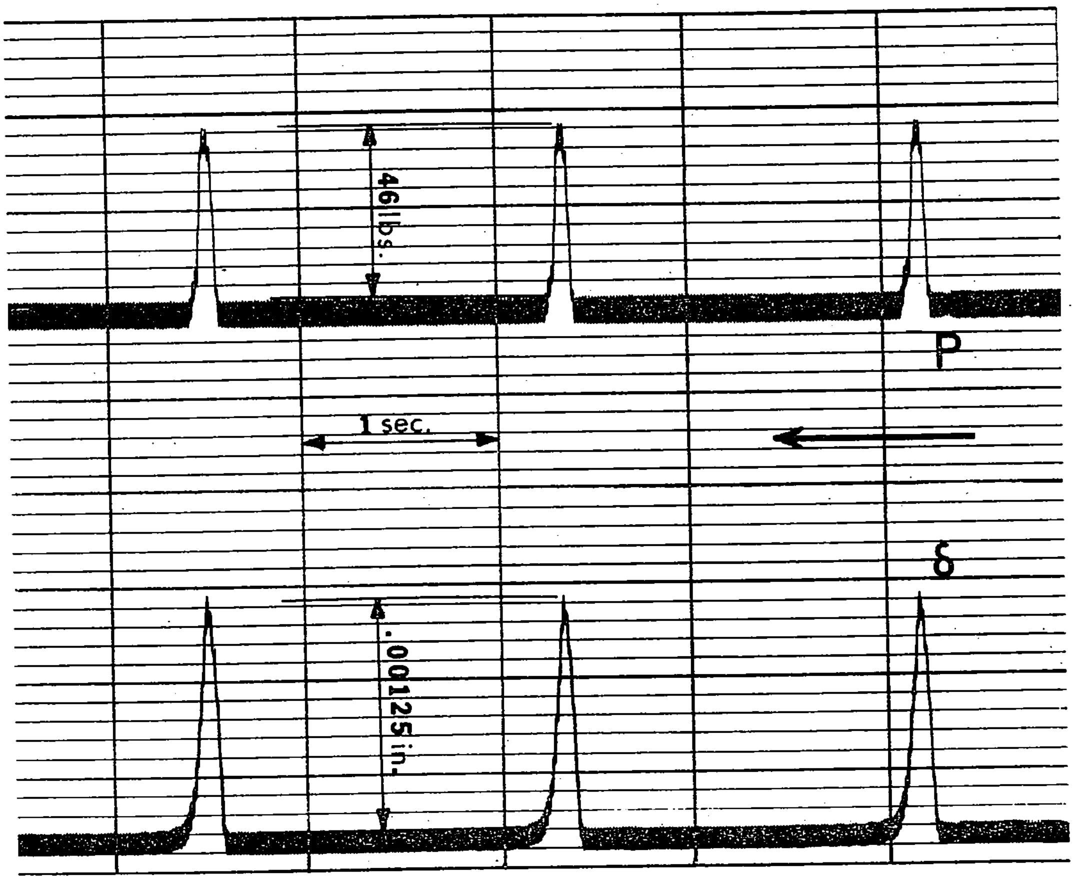


Figure 26. Visicorder Trace Showing the Measured Magnitudes of Load, P, and Deflection, δ.

The calculated M_R is plotted as a point corresponding to the σ_d-σ_3 test conditions in order to develop a plot such as the ones shown in Figures 30, 31, 32, and 33 (shown later).

M_R Determination: Test System Characteristics and Findings

Test System Inconsistencies

Test system inconsistencies were a primary source of M_R fluctuation. The LVDT signal conditioners that were used exhibited non-linear, inconsistent response. Therefore a calibration chart (Figure 27), was graphed to correct test data and average calibration fluctuation. Any error from signal conditioner calibration was increased by 1) error in plotting and then reading the chart, and 2) constant fluctuation of the graph between the dashed lines, altering readings as much as $\pm$ 10%. Points in the curve shown in Figure 27, were found by: 1) inducing a small load and reading the deformation on the dial gage shown in Figure 9, 2) setting the deformation signal conditioner to produce a Visicorder trace amplitude equal to the deformation shown on the dial gage. (At completion of 2 do not readjust signal conditioner), 3) picking various loads to produce sets of dial gage deformations and Visicorder traces and, 4) graphing the measured deformation from the Visicorder trace (ΔL_M) versus the dial gage reading (ΔL_C) for each set. A similar procedure was used for load calibration, however, the signal conditioner that was used showed linear response for the range of applied loads, enabling actual load readings to be read directly from the Visicorder tracer using the linear calibration factor.

System response during preliminary testing altered specimen deformation data. It is required that conditioner response from LVDT signals must be immediate of complete measurements of deformation (and load) will not be transmitted to the Visicorder.

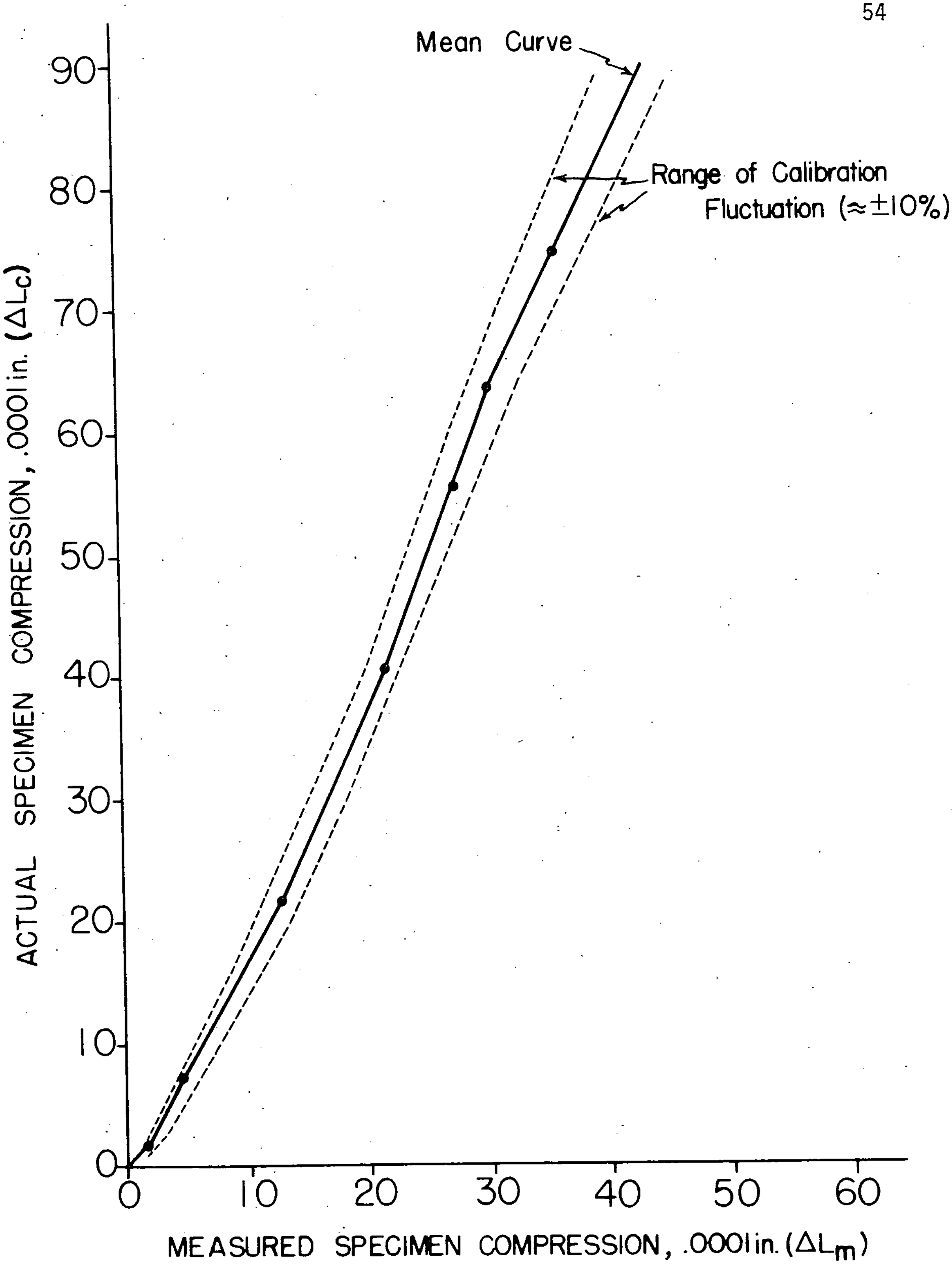


Figure 27. Measured Vs. Actual Specimen Compression Under Cyclic Deviator Stress (U. of Idaho Prototype M_R Device).

Incorrect positioning of the LVDT post in its core causes non-linear LVDT output; it therefore should be centered or "nulled" with respect to the core center when measurements are made, the nulling to be performed by measuring the point at which the LVDT indicates zero voltage output on a voltmeter.

Pulse Duration Time

The duration of the vertical stress pulse can greatly effect M_R values. The Transportation Research Board (2) and The Asphalt Institute (3) recommend pulse times of 0.1 seconds while Thompson and Allen (11) have used 0.15 second stress pulses. This project used 0.15 seconds for a load pulse time to simulate the traffic loading reaction time periods for the subgrade soil at depth A.D.S.S.

The effect of varying the load duration pulse for the Idaho soils tested is displayed in Figure 28. M_R increases rapidly as the load duration pulse becomes increasingly smaller than 0.25 seconds, but shows no appreciable change at time durations from 0.25 to 0.50 seconds. The effect of load duration also doesn't appear to favor one particular soil type. Also, Figure 28 only reflects soil response for σ_d = 4.5 psi and σ_3 = 1.5 psi, the same response trends in each curve could be expected using other load-confining pressure combinations.

Effect of Sample Height

As previously discussed, sample height of 5 inches or more was determined as not greatly affecting M_R. This is demonstrated for a soil B specimen, reduced from 7.3 to 5.3 inches, and tested at σ_3 = 2 and 0 psi for values of σ_d ranging from 1.0 to 6.5 psi. Figure 29 shows there was

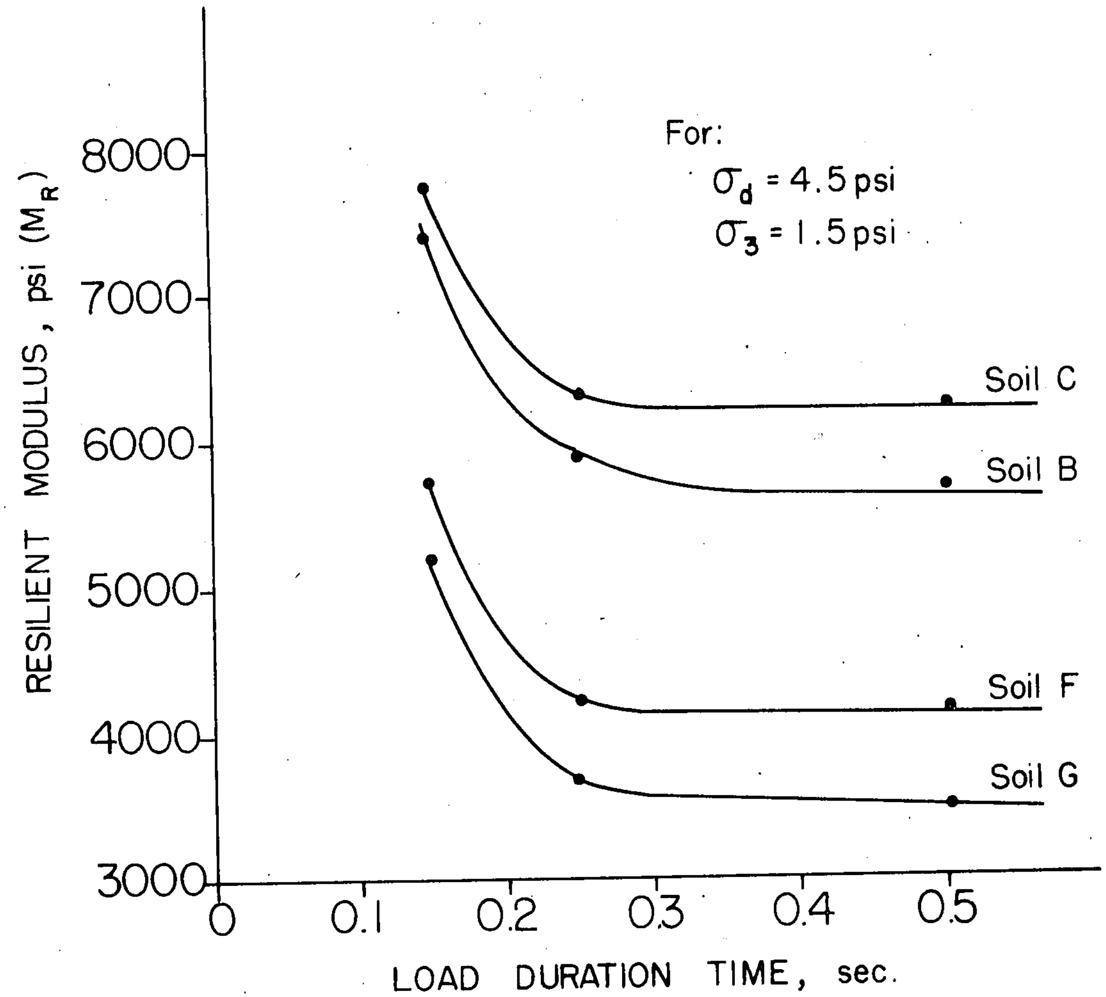


Figure 2 8. Dependence Of Resilient Modulus On Time Duration Of Cyclic Vertical Loading.

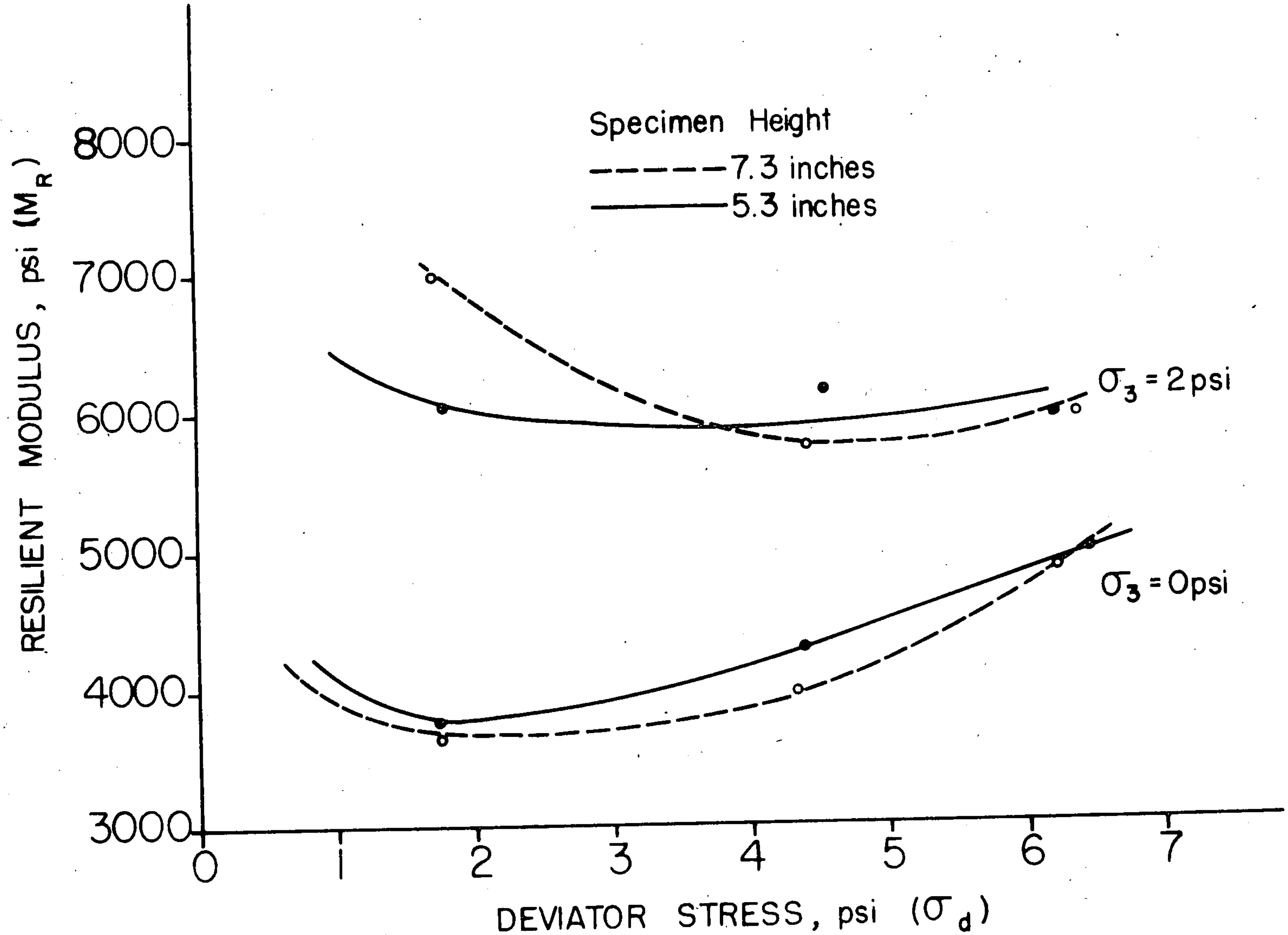


Figure 29. Variation Of M_R With Specimen Height For Soil B.

little effect on M_R when the specimen's height was reduced.

Resilient Modulus Data

The range of M_R for soils B, C, F and G are shown respectively in Figures 30, 31, 32, 33. M_R is dependent on the intensity of repeated deviator stress, $\sigma_d \simeq 1.0$ to 6.5 psi, and constant confining pressure, $\sigma_3 = 3, 2, 1, 0$ psi.

Dependence of M_R on σ_d

The plastic soils, F and G, are influenced more by changes in σ_d than the cohesionless soils B and C.

Each graph of M_R in Figures 32 and 33 spans a range of approximately 3000 psi, while M_R for each graph in Figures 30 and 31 spans a range of only 1000 psi. Each soil shows an increase in M_R as σ_d decreases; M_R of soils F and G increases at a more rapid rate than for soils B and C. For deviator stresses less than 0.75 psi., each graph of M_R exhibits a drastic upward trend. This appears to be common in the literature for similar test data, somewhat due to the limitations of output of the test device or may be due to the relatively small strain of the soil specimens. The small deviator stress results from very thick pavements and/or small wheel loads.

Dependence of M_R on σ_3

Each graph in Figures 30 - 33 indicates that the cohesive and cohesionless soils tested are influenced by confining pressure. Decreasing σ_3 from 3 to 0 psi decreases M_R of soils B and C by 4500 psi consistently across the range of deviator stresses, while M_R for soils F and G decreases by 2000

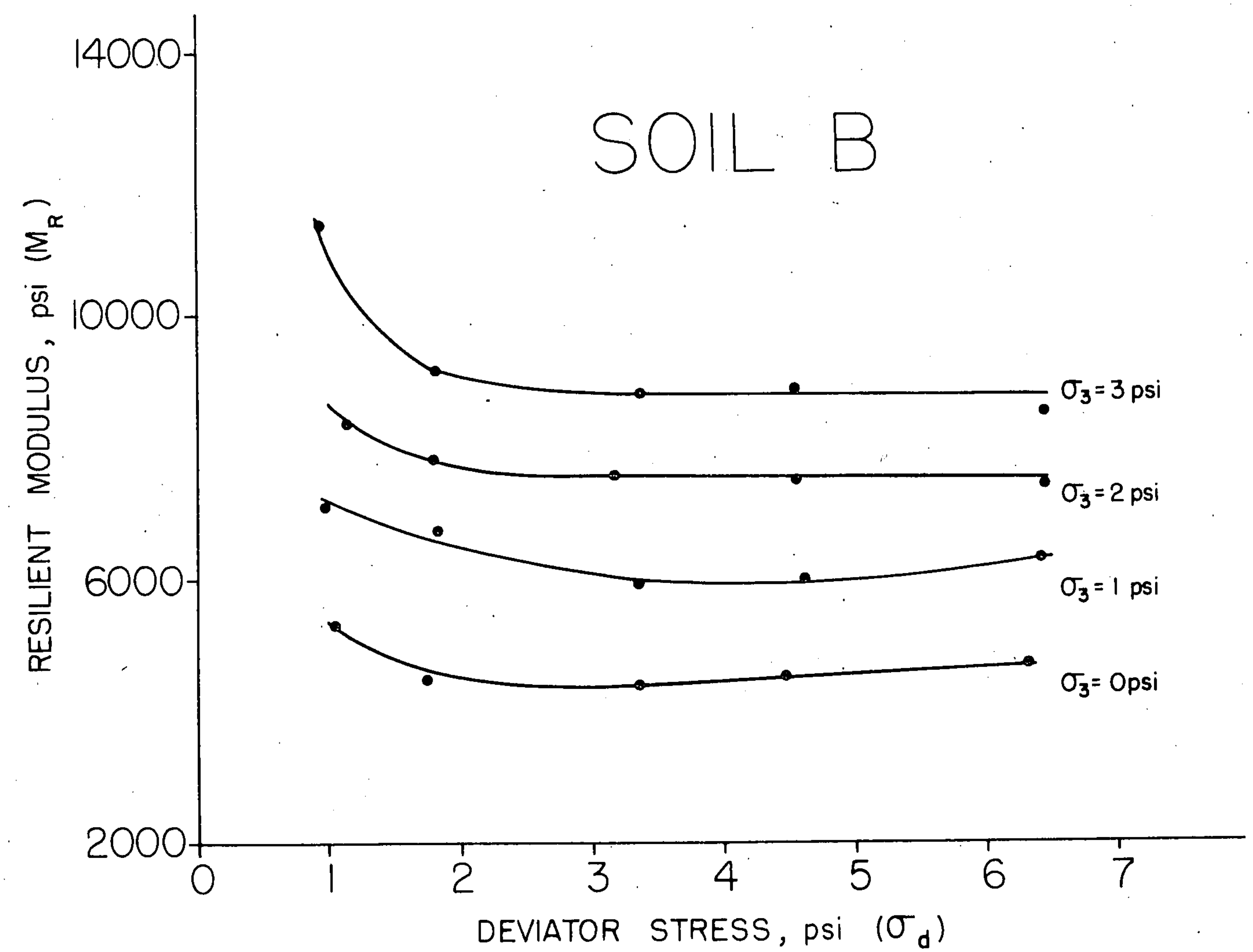


Figure 30. Soil B: Resilient Modulus Vs. Deviator Stress For σ_3, Confining Pressure.

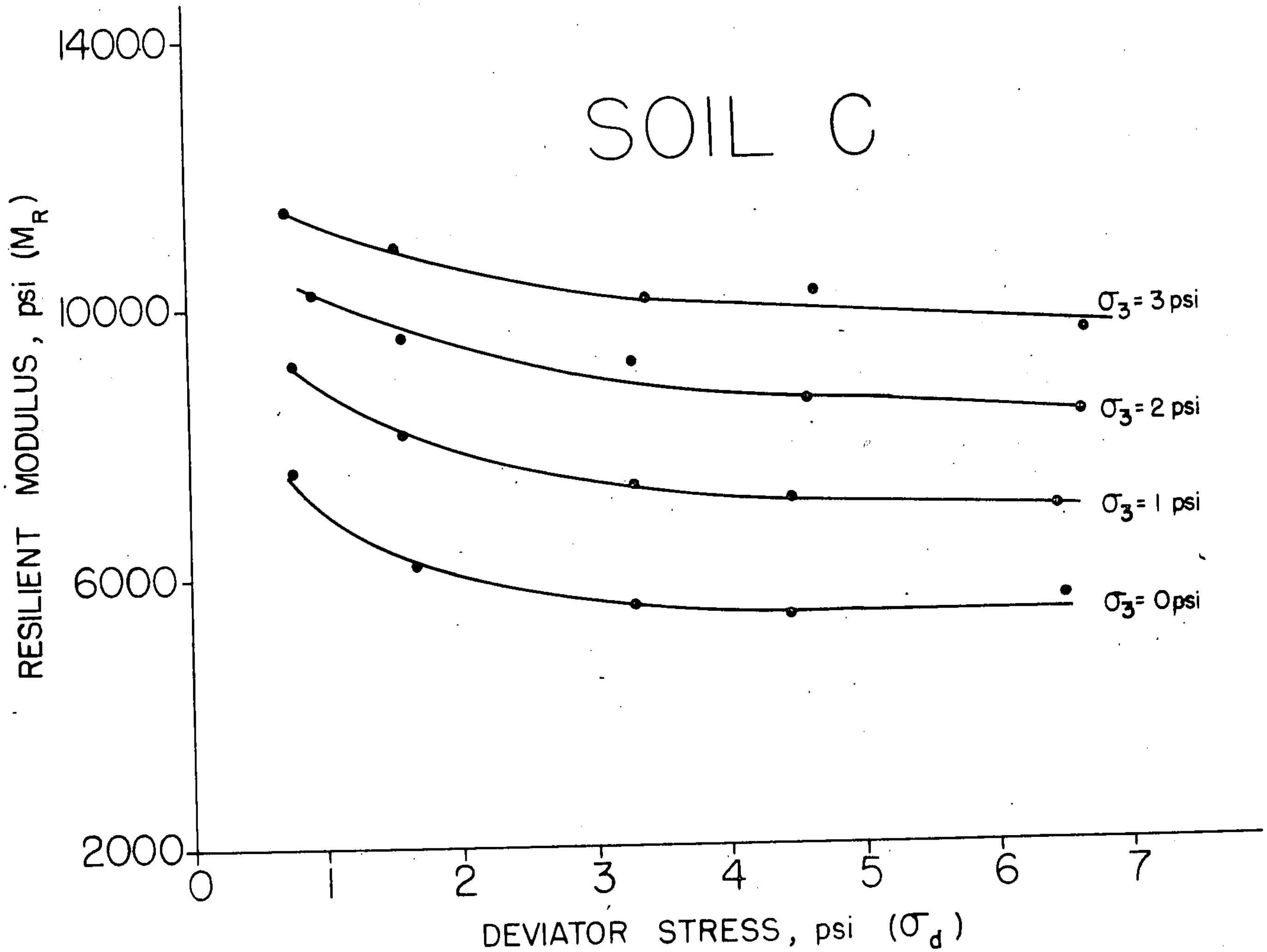


Figure 31. Soil C: Resilient Modulus Vs. Deviator Stress For σ_3, Confining Pressure.

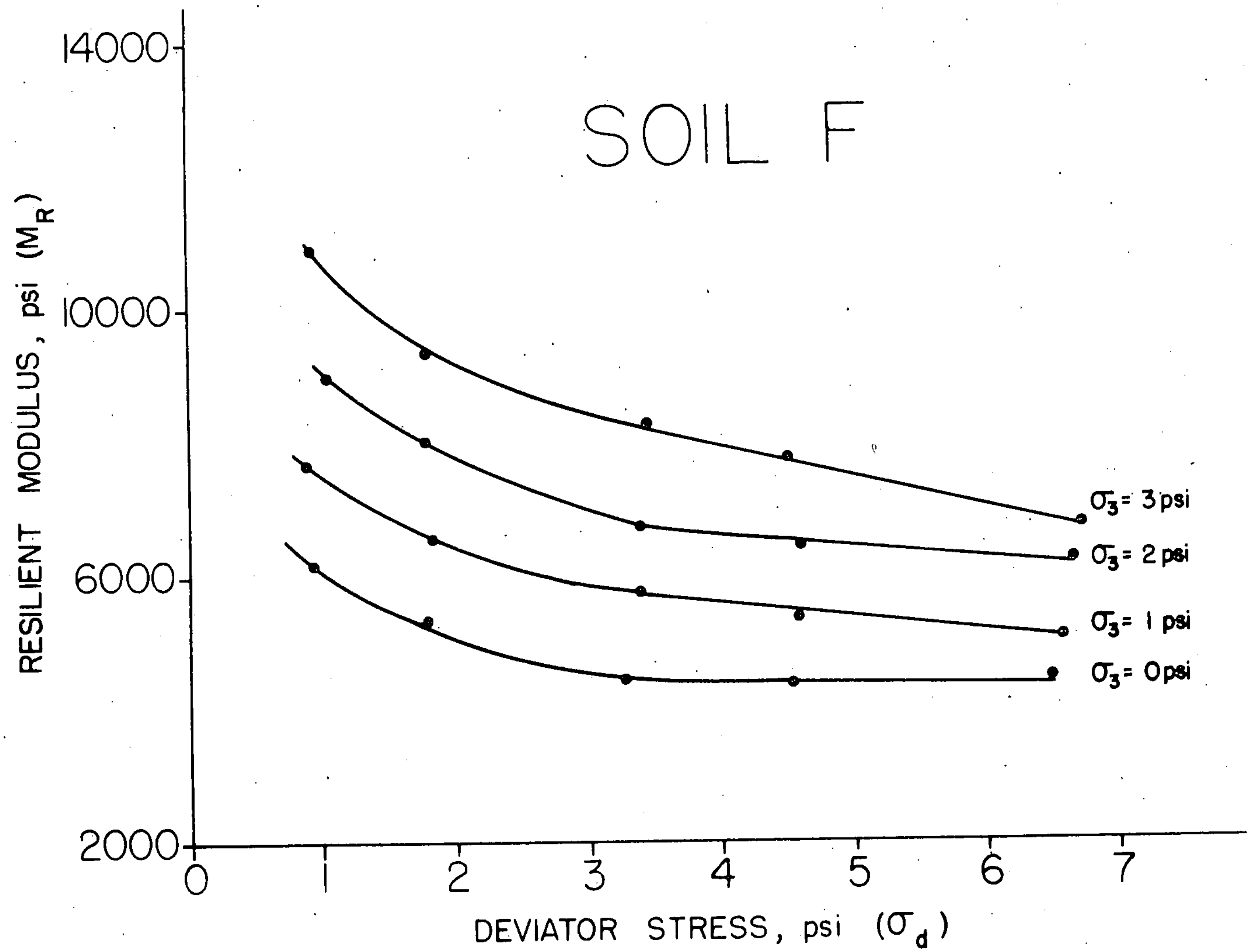


Figure 32. Soil F: Resilient Modulus Vs. Deviator Stress For σ_3, Confining Pressure.

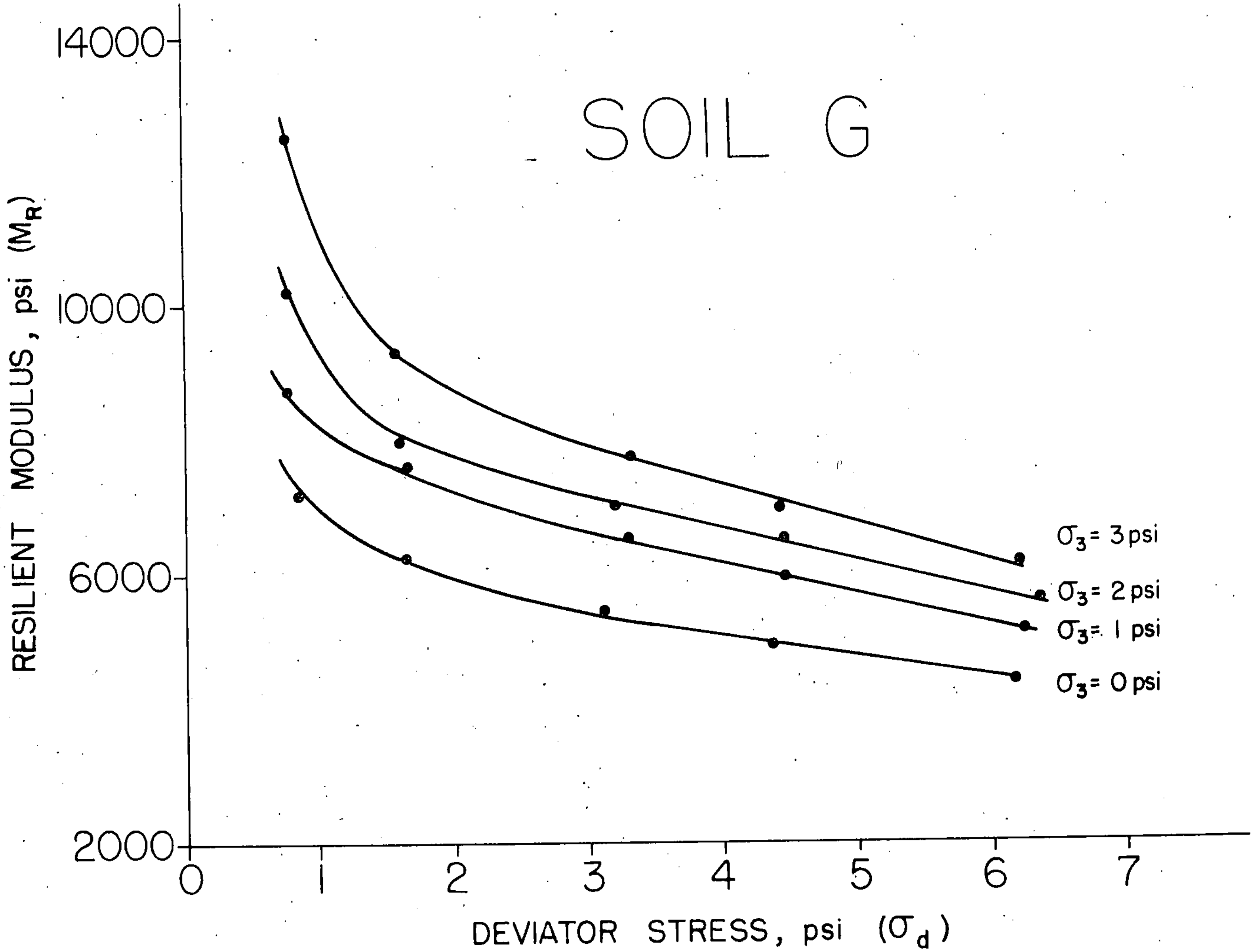


Figure 33. Soil G: Resilient Modulus Vs. Deviator Stress For σ_3, Confining Pressure.

psi at σ_d = 6.5 psi, and 3700 psi at σ_d = 1.5 psi. Cohesive soils become more dependent on σ_3 as deviator stress decreases, while cohesionless soils are predominantly dependent on σ_3; this appears to be a trait that could possibly control a major portion of a pavement system's deformation characteristics.

Final Considerations

M_R of each soil progressively decreases with increasing σ_d, in the range tested, but increases with increasing σ_3.

The validity of the graphical trends of M_R is supported by the fact that the curves are consistent with data obtained by other researchers (12).

One overall observation is that the general M_R magnitude is about the same for all the soils in the σ_d range tested. This illustrates the high probability that cohesionless soils can exhibit as much resilient deformation as the cohesive soils.

Statistical Analysis

The standard deviation, the mean, and the coefficient of variation of individual M_R curves for each soil were calculated and are listed in Table 6. The data reflect how closely specimen fabrication and test conditions were reproduced, and the repeatibility and accuracy of the test system.

S, $\overline{N}$, and V have been calculated from data acquired at σ_d = 1.75 and 4.0 psi for σ_3 = 3, 2, 1, and 0 psi.

The trends are summarized as follows:

1. The standard deviation, S, for all analysis points are generally within $\pm$ 10% of their respective means, and are substantially lower at σ_d = 4.0 psi.

TABLE 6

Standard Deviation, Mean, & Coefficient of Variation of Soil Data

	σ_3-psi	Standard Deviation S(psi) σ_d-psi 1.75	4.0	Mean M_R $\overline{N}$ (psi) σ_d-psi 1.75	4.0	Coefficient or Variation V(%) σ_d-psi 1.75	4.0
Soil B:	3.0	1175	750	9225	8750	12.7	8.6
	2.0	1025	375	7825	7475	13.1	5.0
	1.0	300	225	6650	5875	4.5	3.8
	0.0	375	0	4625	4400	8.1	0.0
Soil C:	3.0	703	544	10,616	9933	6.6	5.5
	2.0	1082	754	9266	8466	11.7	8.9
	1.0	1115	517	7816	6983	14.3	7.4
	0.0	813	263	6083	5433	13.4	4.8
Soil F:	3.0	557	85	9416	7883	5.9	1.1
	2.0	517	671	7866	6516	6.6	10.3
	1.0	616	696	6600	5366	9.3	13.0
	0.0	1063	826	5250	4333	20.3	18.6
Soil G:	3.0	525	150	9025	7250	5.8	2.1
	2.0	650	50	7450	6600	8.7	0.8
	1.0	175	25	7375	6075	2.4	0.4
	0.0	375	100	6225	5000	6.0	2.0

2. The coefficients of variation, V, range from 2.4% to 20.3% at σ_d = 1.75 psi and 0.4% to 18.6% at σ_d = 4.0 psi, the majority of coefficients being less than 14%.

3. V is generally a smaller value at σ_d = 4.0 than at σ_d = 1.75 psi. As confining pressure is reduced, V for all soils decreases.

Larger coefficients of variation, eg. tests of Soil F at σ_3 = 0, are exceptions to the data trends observed and are possibly results of inaccurate testing techniques or data acquisition.

The statistical data do not evaluate resilient modulus corresponding to σ_d greater than 4.0 psi or less than 1.75 psi. It was felt that the majority of M_R lab test measurements will occur in a region between 1.75 and 4.0 psi. in order to match the expected σ_d levels at A.D.S.S. for most pavements.

SECTION C APPENDIX

List of Parts Used in Resilient Modulus Testing System

1.	Wavetek Signal Generator*	Triggered VCG Model 112
2.	Honeywell Visicorder*	Model 906C
3.	Bellofram Air Cyclinder *	D-4-F-SM-0012
4.	Schaevitz Load Cell *	MFTA-1U-500 s/n1927
5.	Schaevitz Signal Amplifier *	SCM 025 (LVDT)
6.	Schaevitz Signal Amplifier*	CAS 025 (Load Cell)
7.	ASCO Solenoid Valve*	Catalog No. 8300C76F
8.	Fisher Governor Regulator	Type 64
9.	Karol Warner Pressure Gage	60 psi.
10.	Relay Switch *	Elec-trol 602-3
11.	Triac Switch *	6 amps
12.	Threaded Rods (4)	3/4 in. x 3 ft.
13.	Frame End Plates (2)	12" x 6" x 2" - bottom 12" x 6" x 1" - top
14.	Loading Plates (2)	4 in. Diameter
15.	Proving Ring (for calibration)	Karol Warner 500r
16.	Triaxial Cell Apparatus	10" High 6" Diameter
17.	Karol Warner Pressure Gage	30psi.
18.	Conflow Air Pressure Regulator	50psi.
19.	Mercury Counter *	Catalog No. BS462UB06
20.	Schaevitz LVDT *	500 MHR, SN 334

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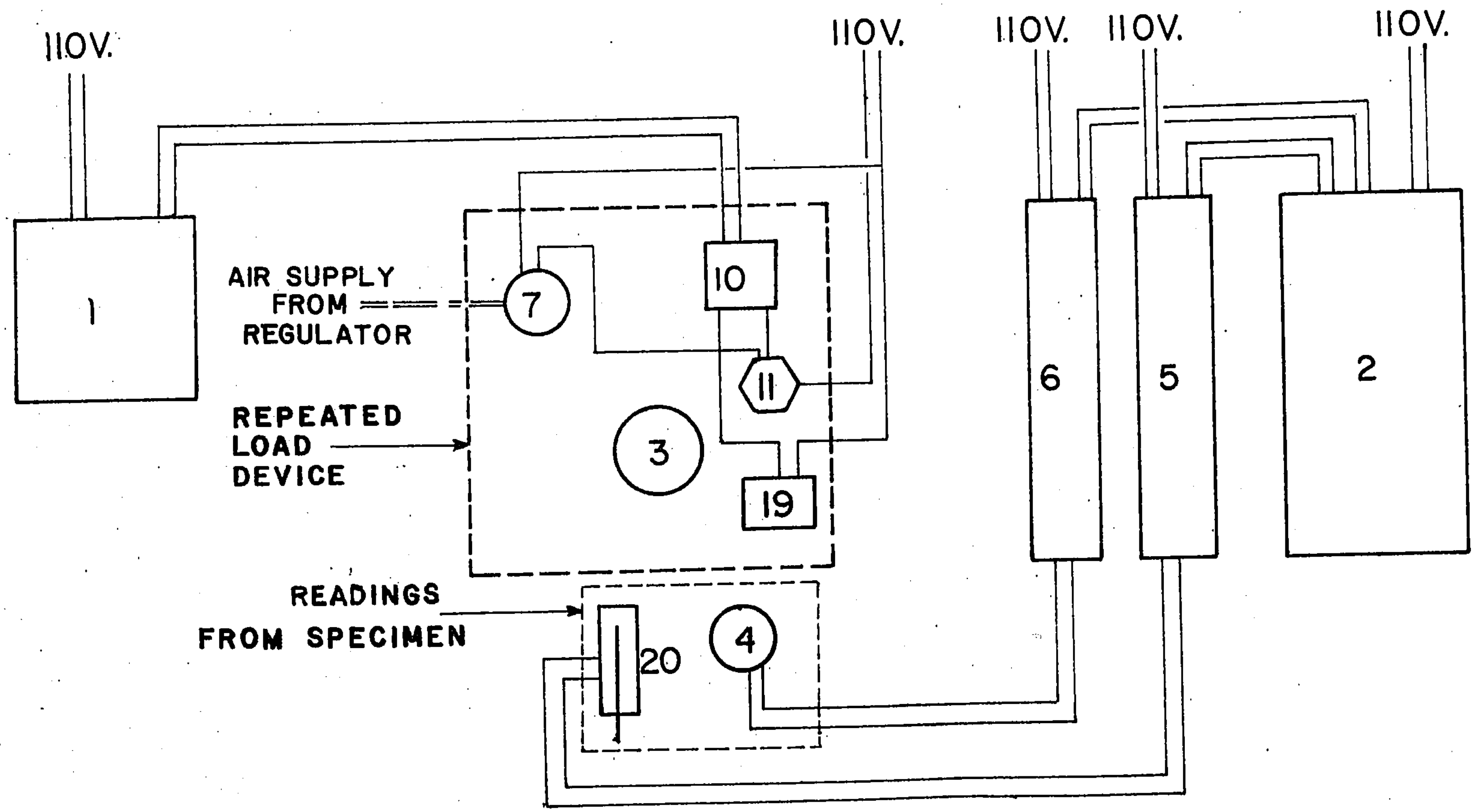
* Note: Asterisks denote equipment shown in the Schematic of Equipment and Wiring on the next page.



SCHEMATIC OF TEST EQUIPMENT AND WIRING

SECTION D

APPLICATION OF RESILIENT MODULUS TO FLEXIBLE PAVEMENT THICKNESS DESIGN

Resilient Modulus Design Considerations and Data Development

Thickness design of flexible pavements using resilient modulus is currently based on limiting surface or vertical subgrade deflection, or limiting subgrade vertical compressive strain and asphalt concrete tensile strain. Thicknesses of asphalt concrete and/or base layers are increased until the magnitudes of the above criteria are equal or below allowable, specified values.

Predicting deflection, stress, or strain in a pavement requires all the pavement materials' stiffnesses and Poisson's ratios to be known. The resilient modulus test gives specific data concerning pavement support material stiffness and is therefore essential for these computations.

Resilient Modulus Data Development

A procedure to utilize resilient modulus data for pavement design was developed in conjunction with existing pavement thickness trends furnished by the Idaho Division of Highways. These pavement thicknesses (and base course depths) were used with the Chevron 5-L elastic layer program output to estimate the range of expected in-situ stresses that would be required for M_R testing (see also Section B). The standard Idaho pavement profile (with constant asphalt concrete thicknesses) was used in the calculation of these in-situ stresses and is shown below.

Interstate		Primary-Secondary
9.6 in.	A.C.	3.6 in.
Base Depth Contingent on Structural Rqmt's	Base	Base Depth Contingent on Structural Rqmt's
	Semi-Infinite Subgrade	

Table 7 lists the in-situ stresses from the Chevron 5-L computer output and data calculated from A.D.S.S. σ_d and σ_3 are the subgrade stresses that were calculated and are assumed to exist in-situ for any given base depth and subgrade resilient modulus.

General Base Thickness Design Iteration Procedure

After a subgrade soil has been tested and graphed, as in Figures 30 - 33, its average M_R may be picked with the anticipated base depth that would be required to fulfill the pavement system structural criteria. To do this, a σ_d and a σ_3 is selected from Table 7, and a corresponding M_R is read from a graph, eg. Figures 30, 31, 32 or 33. An iteration procedure then follows: assume a base thickness -- find the compatible σ_d and σ_3 from Table 7 --- find the compatible M_R for the σ_d - σ_3 combination --- then back to calculating a new base thickness and repeating until the calculated base thickness gives a compatible σ_d - σ_3 combination for the M_R used in the calculation procedure.

TABLE 7

COMPATIBILITY OF CONFINING AND DEVIATOR (DYNAMIC) STRESSES

WITH PAVEMENT BASE DEPTH FOR RESILIENT MODULUS TEST

Asphalt Concrete Depth = 3.6in
(Primary-Secondary Roads)

M_R of Subgrade

Base Depth	3000				6000				9000			
BD	σ_d	σ_{3_1}	σ_{3_2}	ADSS	σ_d	σ_{3_1}	σ_{3_2}	ADSS	σ_d	σ_{3_1}	σ_{3_2}	ADSS
3.6"	3.5	1.0	0.5	23'	4.5	1.0	0.5	22"	6.0	1.0	0.5	20"
6.0"	2.5	1.5	0.5	28"	3.0	1.0	0.5	25"	4.0	1.0	0.5	23"
8.4"	2.0	1.25	1.0	31"	2.5	1.0	1.0	30"	3.5	1.0	0.5	26"
15.0"	1.0	1.5	1.0	43"	1.5	1.0	1.0	40"	2.0	1.0	1.0	36"
24.0"	0.5	2.5	1.5	70"	1.0	2.0	1.5	55"	1.0	2.0	1.5	57"
26.4"	0.5	3.0	2.0	88"	0.5	2.5	1.5	77"	1.0	2.0	1.5	62"

Asphalt Concrete Depth = 10in
(Interstate Rd.)

M_R of Subgrade

	3000				6000				9000			
BD	σ_d	σ_{3_1}	σ_{3_2}	ADSS	σ_d	σ_{3_1}	σ_{3_2}	ADSS	σ_d	σ_{3_1}	σ_{3_2}	ADSS
0"	1.0	1.5	1.0	38"	1.5	1.0	1.0	37"	1.5	1.0	1.0	35"
2.4"	1.0	2.0	1.0	53"	1.0	1.5	1.0	40"	1.5	1.5	1.0	37"
6.0"	0.5	2.0	1.5	58"	1.0	1.5	1.0	50"	1.0	1.5	1.0	43"
15.0"	0.5	2.5	1.5	72"	1.0	2.0	1.5	68"	1.0	2.0	1.5	62"
18.0"	0.5	3.0	2.0	82"	0.5	2.5	2.0	75"	0.5	2.5	1.5	69"
18.7"	0.5	3.0	2.0	82"	0.5	2.5	2.0	76"	0.5	2.5	1.5	71"
26.4"	0.5	3.0	2.0	90"	0.5	3.0	2.0	88"	0.5	3.0	2.0	84"

Note: 1) Average Depth of Significant Stress (ADSS) is Computed Relative to A. C. Surface.
2) Confining Stress in Si-Clay, Silt, Sa-Silt, Loess Subgrades = σ_{3_1}
3) Confining Stress in Sand, Si-Sand Subgrades = σ_{3_2}

Pavement Thickness Results for Soils G, B, C and F

Purpose and Scope

The purpose of this subsection is to compare the required base thicknesses as determined by four independent methods for Interstate and Primary-Secondary highways in Idaho using M_R test data and Idaho R-values for the Idaho subgrade soils G, B, C and F.

The Idaho R-value method of thickness design (the method commonly used) was compared to three other independent design methods which use M_R to determine if there is a significant difference in thickness requirements.

The four independent methods used in this project to determine required base thicknesses were:

1. Idaho R-value
2. Odemark elastic layer
3. Ontario modified Odemark
4. Chevron 5-layer

Design Information

The following constants and assumptions were used in the comparative thickness design methods:

1. Traffic index (Idaho current method only)
   (a) Interstate = 10.5
   (b) Primary-Secondary = 9.5
2. Asphalt concrete thickness (all methods)
   (a) Interstate = 4.8 in.
   (b) Primary-Secondary = 3.6 in.

3. Asphalt treated base (all methods)

   (a) Interstate = 4.8 in.

   (b) Primary-Secondary = 0 in.

4. Soil R-values (Idaho current method)

   (a) Soil F = 67

   (b) Soil C = 71

   (c) Soil B = 63

   (d) Soil G = 11

5. Resilient Modulus (all methods except Idaho)

   (a) Asphalt concrete and asphalt treated base = 400,000 psi (approx. average in Idaho mixes)

   (b) Crushed stone base = 25,000 psi (assumed, from literature values)

   (c) Subgrade soil resilient modulus (Figures 30, 31, 32 and 33)

6. Poisson's ratio (assumed, from literature values) (all methods except Idaho)

   (a) asphalt concrete, asphalt treated base and crushed stone base = .35

   (b) subgrade soil = .40

7. Subgrade soil depth is greater than depth of significant stress (all methods)

8. Truck wheel design loadings are assumed to be a single, 9000 lb. load with 70 psi tire pressure for the Odemark and Ontario modified Odemark methods.

9. Truck wheel design loadings are assumed to be duals 4500 lb. each, 80 psi tire pressure with 13" center-to-center wheel spacing: for the Chevron 5-L program only.

10. No modifications for climate or environment were made. (all methods).

Idaho R-Value Thickness Design

The Idaho R-value method is based on a gravel equivalence which is determined by taking a given R-value and a traffic index and reading a gravel equivalency thickness (in feet) from Fig. 16-231.3 of the Materials and Research Manual (Part 16) (14). From this thickness the adjusted value for asphalt concrete and asphalt treated base is subtracted to determine the granular base requirement. These adjusted values are determined by multiplying the surface layer and base layer thicknesses by appropriate substitution ratios.

SUBSTITUTION RATIOS FOR SURFACING AND TREATED BASE
FOR AGGREGATE BASE MATERIALS

Traffic Index	Plant Mix Pavement (Hot)	Plant Mix Base (Hot)	Road Mix Pavement (Cold) Cement Treat Base Road Mix Base (Cold)	Granular Borrow*
over 7.0	2.0:1	1.75:1	1.50:1	0.75:1
5.5-6.9	2.5:1	2.00:1	1.75:1	0.75:1
Less 5.4	3.0:1	2.5:1	2.00:1	1.00:1

* May include cinder aggregate and selected granular excavation if quality is adequate.

The calculated results for the granular base thickness were:

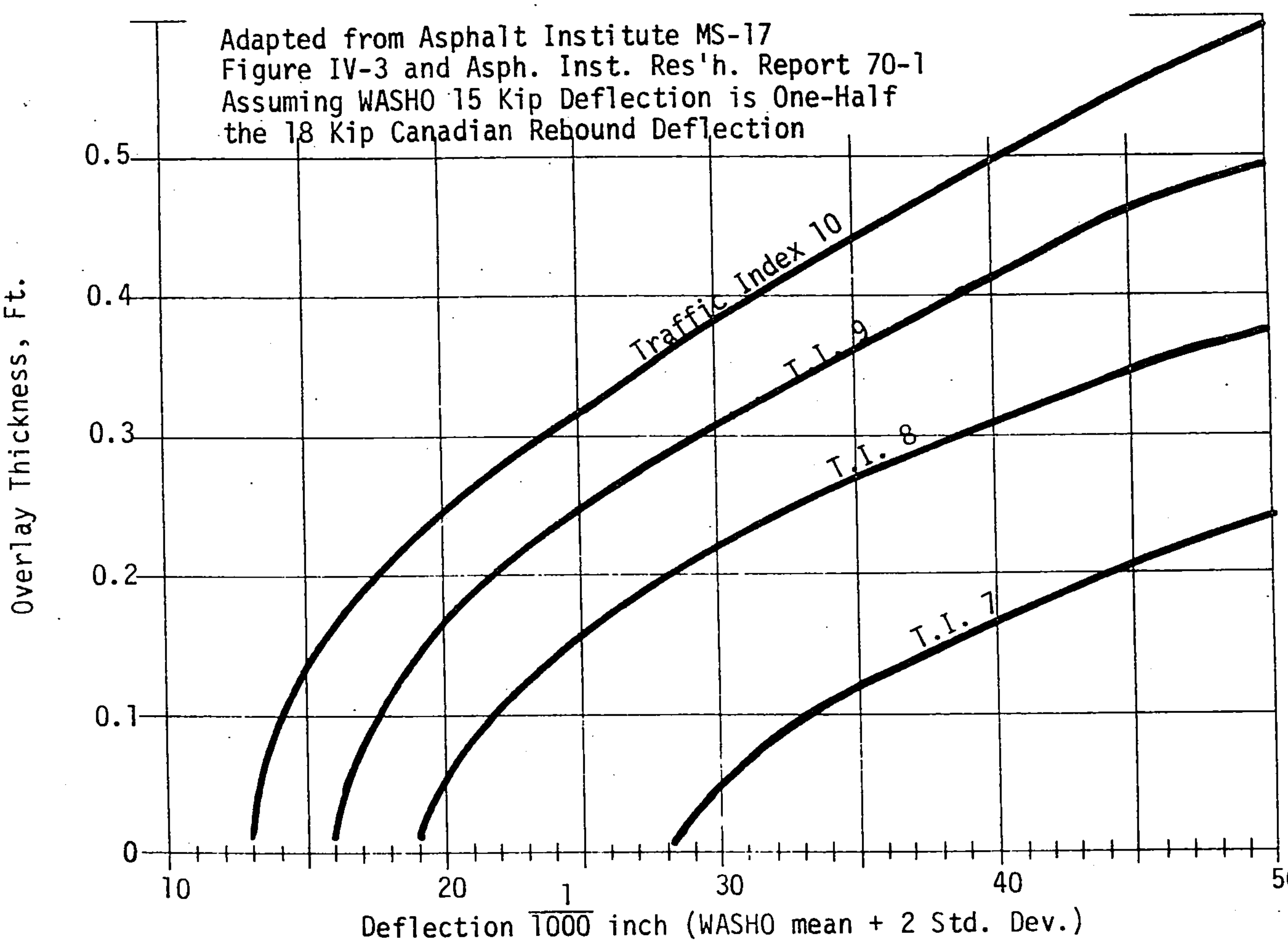
Idaho Soil	Interstate	Primary-Secondary
Soil G	18 in.	25.2 in.
Soil B	-3.6 in. (0)	6 in.
Soil C	6.6 in.	3.6 in.
Soil F	-4.8 in. (0)	4.8 in.

Odemark Elastic Layer Method

The Odemark design method is based on allowable surface deflection of the pavement. The mathematical computations for the solution of the

Odemark method are outlined in research done by Johnnie Sue Green (15), but for the purpose of this project a Fortran computer program was written to solve the equations for base thickness (see Appendix). This program is suitable for any three layer system (or less) and will accommodate various layer moduli and thicknesses.

The design Benkelman Beam (Canadian) rebound surface deflection was determined by assuming a zero overlay thickness and appropriate traffic index in the figure below.



The deflection read from the graph is the WASHO mean plus two standard deviations. This deflection was converted to Benkelman Beam rebound deflection by the following relationship: WASHO Mean x 2 = Benkelman Beam Rebound (Canadian).

Note: two standard deviations, equal to 2 x 10% = 20% of the graph reading, were subtracted from the value obtained from the graph.

Using the above method, allowable deflections were: Interstate = .016 in. and Primary-Secondary = .023 in.

The calculated base thickness results were:

Idaho Soil	Interstate	Primary-Secondary
Soil G	3.5 in.	16.0 in.
Soil B	4.5 in.	17.0 in.
Soil C	0 in.	15.5 in.
Soil F	3.5 in.	17.0 in.

A sensitivity analysis indicated that the base thickness determined by the Odemark method is most sensitive to radical variances in the subgrade modulus and the specified surface deflection.

Ontario Modified Odemark Method

This method uses the basic relationships developed in the original Odemark method with the exception that the design criterion is based on subgrade surface deflection rather than the pavement surface deflection.

The following relationship was developed by the Ontario Dept. of Transportation to correlate the Benkelman Beam rebound (surface) deflection to subgrade deflection (16):

$$\delta = .01 + 87\ W_s^{\ 2}$$

where δ = B.B. rebound deflection

W_s = subgrade deflection

This gave the following allowable subgrade deflections: Interstate = .008 in., Primary-Secondary = .012 in.

A Fortran computer program was developed to determine the design thicknesses for the base (see Appendix).

Idaho Soil	Interstate	Primary-Secondary
Soil G	20.1 in.	23.5 in.
Soil B	20.4 in.	24.3 in.
Soil C	19.3 in.	22.3 in.
Soil F	21.0 in.	23.2 in.

The program was adjusted for a more relaxed subgrade deflection of .011 in. for Interstate and .014 in. for Primary-Secondary, and the base thicknesses for these higher subgrade deflections were:

Idaho Soil	Interstate	Primary-Secondary
Soil G	11.69 in.	21.63 in.
Soil B	13.45 in.	21.36 in.
Soil C	9.63 in.	18.94 in.
Soil F	12.54 in.	21.36 in.

A significant reduction of the base thickness requirement was calculated for the Interstate pavement, but a small, perhaps insignificant reduction was calculated for the Primary-Secondary pavement.

Chevron 5-Layer Method

The Chevron 5-layer method treats the pavement as semi-infinite layers of different materials which interact elastically. The Chevron Research Company of Richmond, California supplied a Fortran program (see Appendix) which was used to calculate the stresses and strains in the layers.

Two strains wree used for design criteria: the vertical strain on the subgrade and the tensile strain in the asphalt concrete. The criteria strains are determined on the basis of number of load repetitions which were calculated using the relationship below (17):

$$W_{18} = 365 \left[\frac{TI}{3.5}\right]^{8.403}$$

where: W_{18} = total number of 18 kip single axle loads planned for the design life, and

TI = traffic index

This relationship gave total 18 kip single axle repetitions of:

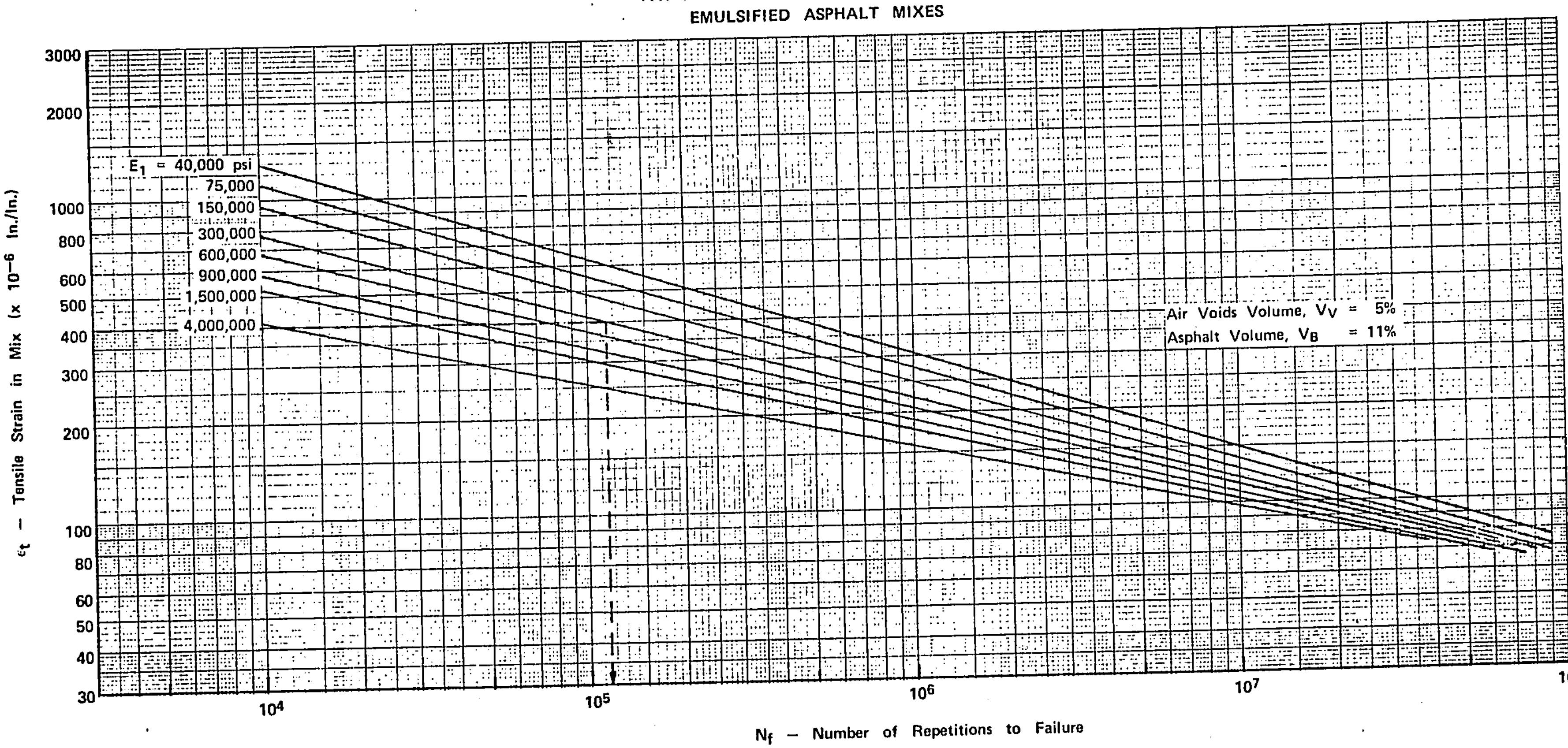
1. Interstate = 3.7×10^6
2. Primary-Secondary = 1.6×10^6

The strain graphs in Figures 34 and 35 (17) gave the following maximum allowable strains for the design 18-kip axle repetitions:

Strain Criterion	Failure Prevention	Interstate	Primary-Secondary
tensile strain in bottom of lower asphalt treated layer	tensile fatigue cracking of asphalt concrete	150 $\mu\varepsilon$	205 $\mu\varepsilon$
vertical strain on top of subgrade	rutting of pavement	350 $\mu\varepsilon$	430 $\mu\varepsilon$

Figure 34

FATIGUE CRITERIA FOR ASPHALT AND EMULSIFIED ASPHALT MIXES

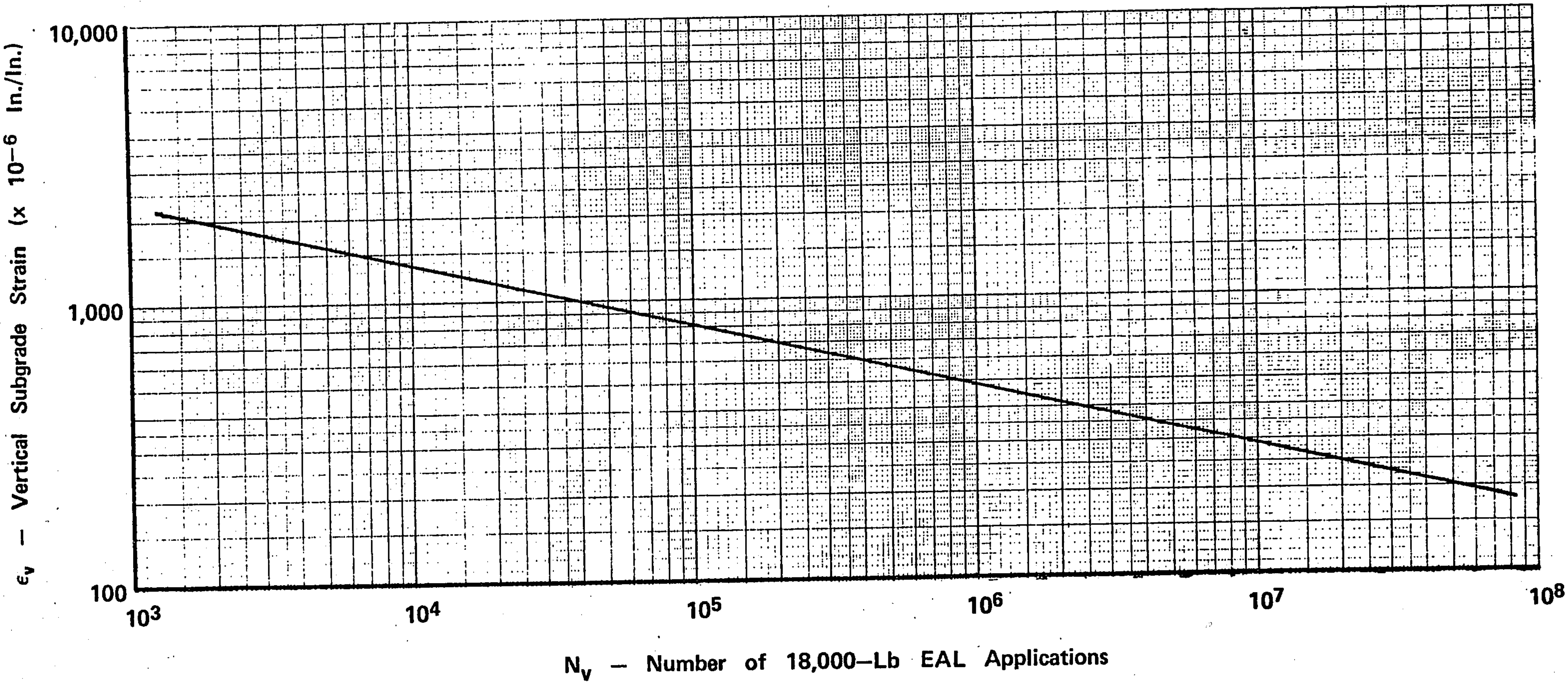


CHEVRON RESEARCH COMPANY
RICHMOND, CALIFORNIA
LES RD 7-3594-3

2-19-75

FIGURE 35

SUBGRADE STRAIN CRITERIA



CHEVRON RESEARCH COMPANY
RICHMOND, CALIFORNIA

9-26-74 LES RE 745035-1

After running the Chevron 5-layer program, the following strains were calculated from the computer output data for the dual wheel loading (see Appendix):

1. Tensile strain in AC $= (R\varepsilon_{r=0}) + (R\varepsilon_{r=13})$

   where $R\varepsilon$ = radial strain measured at appropriate load radius (r) from center of tire print to horizontal stress point of interest and at the interface between the lowest asphalt treated (AC) layer and the top of the granular base layer.

2. Vertical Strain on the top of the Subgrade $= -(R\varepsilon_{r=0}) - (T\varepsilon_{r=0}) - (R\varepsilon_{r=13}) - (T\varepsilon_{r=13}) + (B\varepsilon_{r=0}) + (B\varepsilon_{r=13})$

   where: $R\varepsilon$ = Radial Strain

   $T\varepsilon$ = Tangential Strain

   $B\varepsilon$ = Bulk Strain

These subgrade strains were calculated at the appropriate load radius (r) and at the interface between the bottom of the base layer and the top of the subgrade layer.

The horizontal point of interest is selected where the critical strains occur with respect to the dual wheels and interface depths. For this project, the point was chosen under the center of one of the dual tire prints (r=0). For the second dual tire influence, the point was 13 inches from its tire print center to the center of the other dual tire print (r=13). The strains from both r's (both duals) were added to obtain the total strain.

The base thicknesses determined by this method vary depending on which strain criterion is used for design. The results are listed as follows.

Base thickness determined by

Tensile Strain in AC

Idaho Soil	Interstate	Primary-Secondary
Soil G	0 in.	35 in.
Soil B	0 in.	32 in.
Soil C	0 in.	32 in.
Soil F	0 in.	32 in.

Base thickness determined by

Vertical Strain on Subgrade

Idaho Soil	Interstate	Primary-Secondary
Soil G	4 in.	18 in.
Soil B	5 in.	18.5 in.
Soil C	3 in.	17 in.
Soil F	3.5 in.	18 in.

The base thickness selection can be interpreted as the thickness which satisfies both strain criteria for a given subgrade soil and pavement type. For example, vertical strain on subgrade was found to govern Interstate pavement and tensile strain in the asphalt concrete was found to govern Primary-Secondary pavement thickness.

Comparisons and Implications

The granular (untreated) base thicknesses determined previously by the different methods were compared to each other and are shown in Figures 36 and 37.

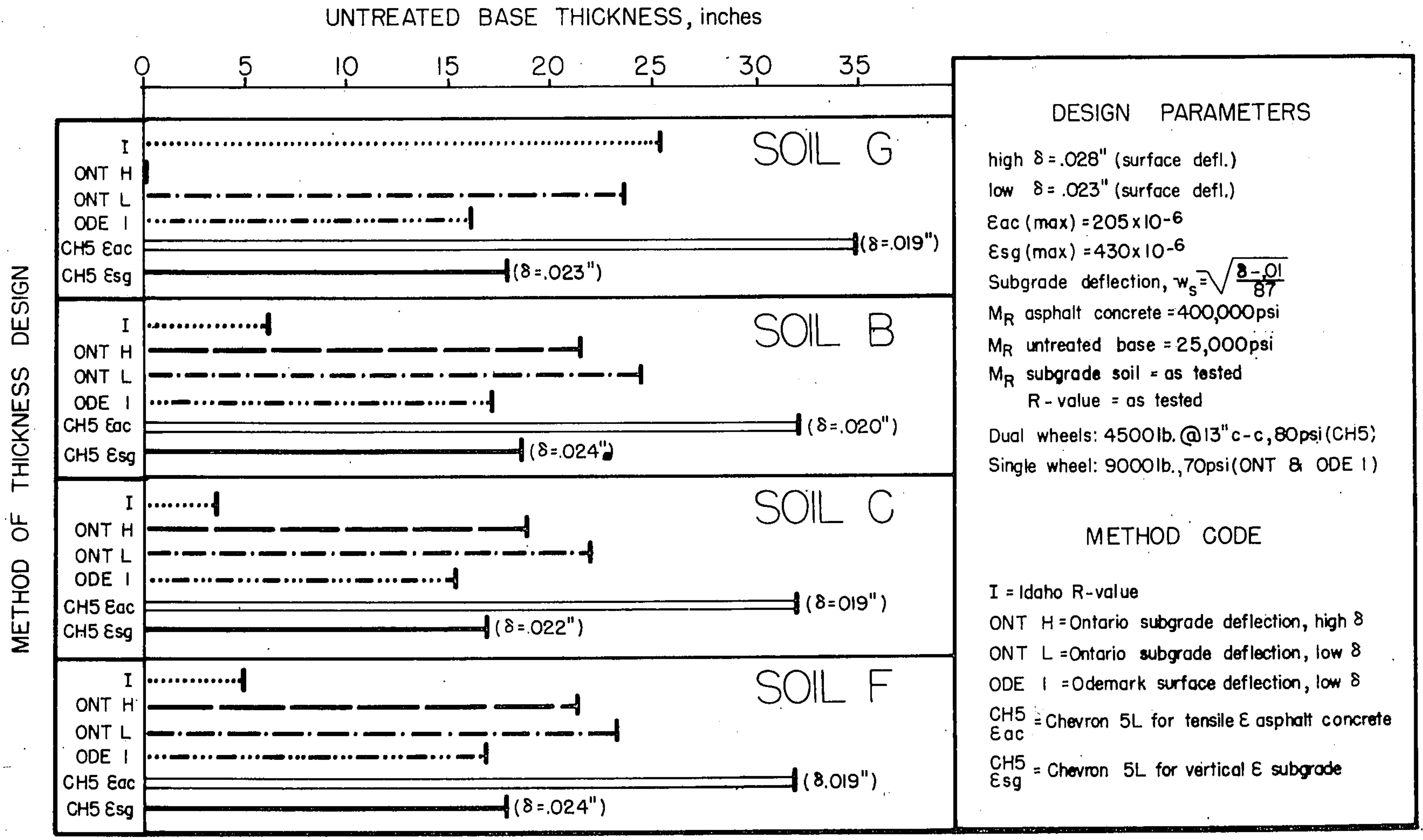


Figure 36. Comparison Of Base Thicknesses Computed By Different Design Method For Secondary - Primary Pavement (3.6 inches asphalt concrete, TI = 9.5).

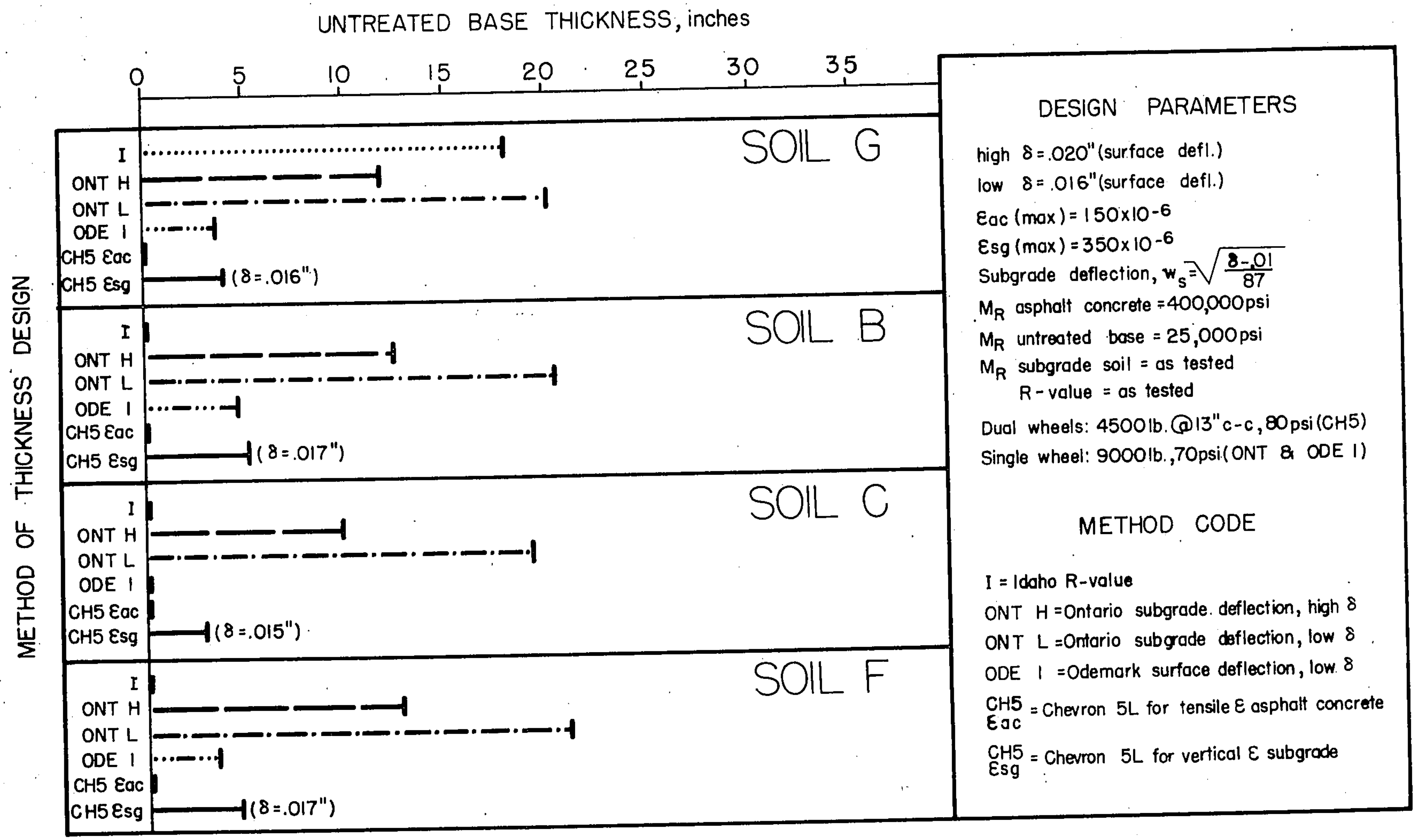


Figure 37. Comparison Of Base Thicknesses Computed By Different Design Methods For Interstate Pavement (9.6 inches asphalt concrete, TI = 10.5).

Although strain criteria were used in the Chevron 5-L method of design, the pavement surface deflection that resulted is shown in parentheses in the Figures.

For the Primary-Secondary pavement (3.6 inches of asphalt concrete), Figure 36, the current Idaho R-value design appears to give adequate base depth for the more plastic subgrade soil G with, perhaps, some premature tensile fatigue cracking as shown by the greater base depth required for the Chevron 5-L tensile strain in the asphalt concrete. For the other, less plastic fine-grained subgrade soils B, C and F, the current Idaho R-value design gives lower base thicknesses than the other methods. According to theory, both premature tensile fatigue cracking and rutting would result.

For the Interstate pavement (9.6 inches of asphalt concrete), Figure 37, the current Idaho R-value design appears to give adequate base depth for the plastic subgrade soil G with no premature tensile cracking or rutting. For the less plastic fine-grained soils, B, C and F, the current Idaho R-value design gives no base requirement at all. Most of the other methods give some base requirement of at least a few inches. The Chevron 5-L strain criteria (and the Odemark Method) indicate some premature pavement rutting could occur if a base depth less than 4 inches was used. The Ontario methods require even greater base depths.

The thicker asphalt concrete pavement of 9.6 inches (Interstate) should minimize or eliminate premature fatigue cracking although some rutting may occur as the accumulated traffic builds up.

The thinner asphalt concrete pavement of 3.6 inches (Primary-Secondary) appears to be damaged primarily by fatigue cracking as the accumulated traffic builds up unless greater base depths (or equivalent structural requirement) are used, especially for the fine-grained slightly plastic to nonplastic soil subgrades. The Chevron 5-L program showed that the untreated base depth for the 3.6 inch asphalt concrete pavement would become very large, out of practical reason, for a traffic index greater than 9.5. If these data are reasonably accurate and higher traffic index is predicted, the Primary-Secondary pavements on these soils would need "beefing up" by either increasing the asphalt concrete depth or by treating the base material to increase its structural equivalency.

APPENDIX

SECTION D

This appendix gives the instructions for running computer programs (Odemark, Ontario Modified Odemark and Chevron 5-L), an example of computer output data for each program and the procedure for finding the base thickness for each program.

The Research Engineer will also receive a separate attachment containing one each of the following:

1. Odemark computer program card deck and program printout,
2. Ontario Modified Odemark computer program card deck and program printout, and
3. Chevron 5-Layer computer program card deck and program printout.

These programs are Fortran and have been set-up for terminal usage with the U. of Idaho IBM 370 computer. It is hoped that only minor modifications will be required by the State computer personnel for adapting the programs to the computer available to the Idaho Transportation Department.

Odemark Computer Program

The elastic, equivalent layer thickness equations developed by Odemark and listed by Green (15) were programmed for the IBM 370 with terminal usage. The program is called "Odemark 1".

The object of the program is to determine the required base thickness (or other layer thicknesses) to satisfy a specified pavement rebound deflection.

To use the program for this project, specified deflections are determined that cannot be exceeded for the pavement surface, moduli and Poisson ratios of the pavement layers are assumed or determined, and the asphalt concrete thickness* is specified. M_R graphs of the subgrade soils are used (M_R vs. σ_d-σ_3) along with a table such as Table 7. The program uses a single tire load.

Refer to Table 8 as an example of the procedure and output for a typical run. The operator loads the program and calls for "run". The computer terminal typewriter then types statements and questions. After each question** the operator types in the correct parameters. After this is completed the computer will calculate the untreated granular base thickness for the parameters used. The typewriter will type a question asking for further runs if desired. Usually one or more additional runs will be necessary until the computed base thicknesses gives σ_d-σ_3 at ADSS compatible with the subgrade M_R chosen. The subgrade soil's M_R vs. σ_d-σ_3 graph (eg. Figures 30, 31, 32, 33) is used with Table 7 for this iteration.

---

* Asphalt concrete thickness can be used as a variable in which base the base thickness, if any, becomes a constant.

** Note: The computer terminal typewriter prints "?" after each question.

TABLE 8

ODEMARK 1 PROCEDURE AND OUTPUT

```
load odemark1
ready

run

ODEMARK1     9:16   09/07/76  TUESDAY

INSURE THAT ALL DATA ARE ENTERED IN THE ORDER ASKED FOR AND THAT ALL ENTRIES ARE SEPARATED BY COMMAS
ENTER THE TITLE OF THIS RUN
?test run

ENTER VALUES FOR ASPHALT MODULUS,BASE MODULUS AND SUBGRADE MODULUS
?400000,25000,3000

WHAT IS THE TIRE LOAD AND TIRE PRESSURE
?9000,70

ENTER THE THICKNESS OF THE ASPHALT LAYER AND THE MAXIMUM ALLOWABLE SURFACE DEFLECTION
?9.6,.023

BASE THICKNESS=12.50 INCHES AND TOTAL DEFLECTION=.0228 INCHES FOR RUN CALLED TEST RUN
DO YOU WISH TO RUN THIS PROGRAM AGAIN YES/NO
?no

STOP
TIME 1 SECS.
```

Ontario Modified Odemark Computer Program

The elastic, equivalent layer thickness equations developed by Odemark were modified by the Ontario Dept. of Transportation (16) to reflect subgrade deflection rather than pavement surface deflection. Ontario's research has claimed that subgrade deflection is more of an indicator of pavement structural performance than surface deflection. Their subgrade deflection relationships were programmed for the IBM 370 with terminal usage. The program is called "Ontmode".

The procedures and output are the same as "Odemark 1"; the only difference is that allowable subgrade deflection is used instead of allowable pavement surface deflection.

Table 9 shows an example of a run. For the parameters used, including a low allowable subgrade deflection, a high untreated base thickness of 78.6 inches is computed. Higher allowable subgrade deflections of .01-.04 inches have been used by Ontario (with resulting greater surface deflections).

TABLE 9

ONTARIO MODIFIED ODEMARK PROCEDURE AND OUTPUT

```
load ontmode
ready

run

ONTMODE      9:14   09/07/76  TUESDAY

INSURE THAT ALL DATA ARE ENTERED IN THE ORDER ASKED FOR AND THAT ALL ENTRIES ARE SEPARATED BY COMMAS
ENTER THE TITLE OF THIS RUN
?test run

ENTER VALUES FOR ASPHALT MODULUS,BASE MODULUS AND SUBGRADE MODULUS
?400000,25000,3000

WHAT IS THE TIRE LOAD AND TIRE PRESSURE
?9000,70

ENTER THE THICKNESS OF THE ASPHALT LAYER AND THE MAXIMUM ALLOWABLE SUBGRADE DEFLECTION
?9.6,.008

BASE THICKNESS=78.60 INCHES FOR RUN CALLED TEST RUN
DO YOU WISH TO RUN THIS PROGRAM AGAIN YES/NO
?no

STOP
TIME 1 SECS.
```

Chevron 5-Layer Computer Program

Elastic layer interactive relationships were computerized a number of years ago by Chevron Research Company. One of the programs is a five pavement layer (or less) program. This program was put on the IBM 370 with terminal usage. The program is called "Pavement".

The procedure for using the program on the terminal was set up to be basically the same as Odemark 1 and Ontmode: the computer asking for values of the parameters and the operator entering them, with finally the main computation and output display being printed. The main exception is that an assumed base thickness is used. An example run is shown in Table 10.

The entering of parameters differs from the Odemark 1 and Ontmode programs in that the operator must enter the following additional parameters: number of layers, (including the base, if any, and thicknesses), Poisson's Ratio for each layer (in addition to M_R), number and magnitude of "r-values" (horizontal distance from tire print center to point of interest), and number and magnitude of "z-values" (vertical distance downward from pavement surface to points of interest). Points of interest are where stresses and strains in the pavement are desired, and the operator will have determined these distances beforehand.

Table 10 shows an output display for two r's and three z's. The two r's are necessary for dual tires, each 4500 lb and 70 psi in this example,* in order to calculate stresses and strains under one tire plus using the superposition influence of the other tire 13 inches horizontally distant. The three z's in this example are: 0 inches in order to calculate surface deflection (as an optional choice), 9.6 inches in order to calculate the

---

* Note: The Chevron 5-L program was run with tire pressures of 80 psi in the base thickness comparisons of this section D: 80 psi is perhaps better than 70 psi, but it is a variable.

TABLE 10

CHEVRON 5-LAYER PROCEDURE AND OUTPUT

```
run pavement
PAVEMENT     9:18   09/07/76  TUESDAY

INSURE THAT ALL DATA ARE ENTERED IN THE ORDER ASKED FOR AND THAT ALL ENTRIES ARE SEPARATED BY COMMAS
ENTER THE TITLE OF THIS RUN
?test run

TYPE IN THE TIRE LOAD,TIRE PRESSURE,AND 0,0
?4500,70,0,0

ENTER THE NUMBER OF LAYERS FOLLOWED BY THE MODULUS AND POISSONS RATIO OF EACH RESPECTIVE LAYER
BEGINNING WITH THE SURFACE LAYER AND WORKING DOWN TO,AND INCLUDING,THE SUBGRADE LAYER
?3,400000,.35,25000,.35,3000,.4

ENTER THE THICKNESSES OF THE LAYERS STARTING FROM THE SURFACE AND WORKING DOWN TO,BUT NOT INCLUDING,THE SUBGRADE LAYER
?9.6,18

HOW MANY R VALUES DO YOU WANT TO CALCULATE AND WHAT ARE THEY
?2,0,13

HOW MANY Z VALUES DO YOU WANT TO CALCULATE AND WHAT ARE THEY
?3,0,9.6,27.6


******************** TEST RUN     ********************  PAGE  1

                         THE PROBLEM PARAMETERS ARE

          TOTAL LOAD..          4500.00  LBS

          TIRE PRESSURE..         70.00  PSI

          LOAD RADIUS..            4.52  IN.

          LAYER  1  HAS MODULUS   400000.  POISSONS RATIO 0.350  AND THICKNESS   9.60 IN.
          LAYER  2  HAS MODULUS    25000.  POISSONS RATIO 0.350  AND THICKNESS  18.00 IN.
          LAYER  3  HAS MODULUS     3000.  POISSONS RATIO 0.400  AND IS SEMI-INFINITE.
```

R	Z	STRESSES VERTICAL	STRESSES RADIAL	STRESSES SHEAR	STRESSES BULK		DISPLACEMENT VERTICAL		STRAINS RADIAL	STRAINS TANGENTIAL	STRAINS BULK
0.0	0.0	-7.000E+01	-9.323E+01	0.0	-2.566E+02	*	1.461E-02	*	-9.033E-05	-9.033E-05	-1.924E-04
SLOW											
0.0	-9.6	-4.628E+00	4.652E+01	0.0	8.841E+01	*	1.390E-02	*	7.964E-05	7.964E-05	6.631E-05
0.0	9.6	-4.628E+00	5.711E-01	0.0	-3.486E+00	*	1.390E-02	*	7.964E-05	7.964E-05	-4.183E-05
0.0	-27.6	-5.525E-01	2.266E+00	0.0	3.979E+00	*	1.202E-02	*	6.665E-05	6.665E-05	4.775E-05
0.0	27.6	-5.525E-01	-3.508E-02	0.0	-6.226E-01	*	1.202E-02	*	6.665E-05	6.665E-05	-4.151E-05
13.0	0.0	3.568E-01	-1.136E+01	-1.368E-05	-3.267E+01	*	1.141E-02	*	-1.144E-05	-4.285E-05	-2.450E-05
SLOW											
13.0	-9.6	-1.445E+00	6.450E+00	-1.267E+00	2.223E+01	*	1.144E-02	*	2.315E-06	3.869E-05	1.668E-05
13.0	9.6	-1.445E+00	-3.264E-01	-1.267E+00	-1.425E+00	*	1.144E-02	*	2.315E-06	3.869E-05	-1.709E-05
13.0	-27.6	-4.328E-01	1.732E+00	-1.048E-01	3.298E+00	*	1.038E-02	*	4.770E-05	6.143E-05	3.958E-05
13.0	27.6	-4.328E-01	-3.042E-02	-1.048E-01	-4.642E-01	*	1.038E-02	*	4.770E-05	6.143E-05	-3.095E-05

```
DO YOU WISH TO RUN THIS PROGRAM AGAIN YES/NO
?no

STOP
TIME 20 SECS.
```

stresses and strains at the bottom "fiber" of the asphalt concrete, and 27.6 inches in order to calculate the stresses and strains on top of the subgrade.

The computer typewriter will first print a display of the problem parameters, followed by a computation pause, and finally followed by the computational output display.

Using the example output of Table 10, the following criteria can be determined (in addition to others not listed below), and then compared to the specified allowable criteria for the base thickness assumed:

1. Surface Deflection: $1.461 \times 10^{-2} + 1.141 \times 10^{-2} = .026$ in.

   (The computer prints "SLOW" for z = 0 because of slow convergence of elastic relationship at z = 0).

2. Tensile Stress at Bottom of Asphalt Concrete:

   $4.652 \times 10^{1} + 6.450 = 53$ psi

   (Note: the computer prints two z depths: -9.6 and 9.6 inches. The -9.6 inch depth denoting a depth just in the lower "fiber" of asphalt concrete and not in the base).

3. Tensile Strain at Bottom of Asphalt Concrete:

   The radial strain is used at -9.6 inches depth.

   $7.964 \times 10^{-5} + 2.315 \times 10^{-6} = 81.96 \times 10^{-6}$, or 82 $\mu\varepsilon$.

   This value is compared to the allowable asphalt concrete tensile strain (eg. Figure 34) in order to provide adequate fatigue life for the design number of 18-kip single axle repetitions.

4. Vertical Strain on the Subgrade

   The vertical strain on the subgrade needs to be calculated from the strain output display at the z depth corresponding to the top of the subgrade.* For this example in Table 10, the z = 27.6 inches. (not z = -27.6).

$(-4.151 \times 10^{-5}) - (6.665 \times 10^{-5} + 6.665 \times 10^{-5}) +$

$(-3.095 \times 10^{-5}) - (6.143 \times 10^{-5} + 4.770 \times 10^{-5})$

$= -31.489 \times 10^{-5} = -314.89 \times 10^{-6}$, or - 315 $\mu\varepsilon$.

The negative sign means compression.

The value is compared to the allowable vertical strain on the subgrade (eg. Figure 35) in order to provide minimum acceptable rutting for the design number of 18-kip single axle repetitions.

Iteration of Chevron 5-L:

Any calculated design criterion from the program output can be compared to a spicified allowable criterion. If the calculated criterion is less than the allowable, for example, base thickness (or whatever) can be reduced and a second run performed with the new subgrade M_R compatible to σ_d-σ_3 in the subgrade. This procedure is repeated until the layer thickness becomes the minimum acceptable.**

In this project, both tensile strain in a.c. and vertical strain on the subgrades were used as design criteria. A usual procedure would be to accept the minimum layer thickness design (based on an economical layer combination) that would satisfy both design criteria.

---

*Note: Bulk strain - (tangential strain + radial strain), calculate for r = 0 and r = 13, and sum algebraically (this is the same relationship as the "multiplied out" relationship shown previously in Section D).

**Note: If the calculated criterion is greater than the allowable, then layer thicknesses need to be increased for the second run, at least.

It is necessary to use a M_R vs. σ_d-σ_3 graph for the subgrade soil as well as a table of depths and σ_d-σ_3 (eg. Table 7) in order to enter a subgrade M_R parameter that is compatible to the σ_d-σ_3 existing for the new trial layer(s) depth in each new run.

Flexibility of the Computer Programs

1. Odemark 1 and Ontmode

Almost any type and quality of surface layer can be used, eg. asphalt emulsion concrete, etc., as long as the specified M_R and thickness for this layer is used. The thickness of this layer can also be varied, but a new run is required for each variable.

The computed base thickness is in terms of untreated quality granular material. Structural equivalencies can then be used to determine thicknesses of treated base materials.

Full-depth asphalt concrete thicknesses can be determined by making computer runs with known asphalt concrete M_R and trial thicknesses until the base thickness finally is computed as 0 inches.

For the above, a maximum surface deflection needs to be specified, ie. Benkelman Beam Rebound (Canadian or similar).

2. Chevron 5-Layer

The flexibility of the Odemark 1 and Ontmode programs also apply to the Chevron 5-L. In addition, the Chevron 5-L can accommodate two additional pavement layers, eg. the investigation of asphalt concrete surfacing which has a different M_R and Poisson's Ratio than the asphalt concrete lower layer(s) with the addition of a treated or nontreated base layer(s) on the subgrade.

More variation of design criteria can be used with Chevron 5-L because of its stress-strain output. Besides the strain criteria example previously shown, for example, allowable tensile split strengths for the pavement can be compared to the calculated tensile stresses at the bottom of the critical layer(s).

SECTION E

RECOMMENDATIONS AND REFERENCES

Recommendations

The results of this project indicate that the fine-grained subgrade soils tested at the Idaho R-value water contents and densities give resiliencies, ie. resilient moduli (M_R), that, when used with the elastic layered computational methods and reasonable design criteria, produce base thickness requirements which can be larger than the requirements from the Idaho R-value method. Especially for the lower plastic, fine-grained soils, the thicknesses appear greater for the M_R method and, unfortunately, would result in greater pavement costs. However, since resiliency methods using M_R are relatively new, their adaption for pavement thickness design would first require careful correlation to test pavement performance in the State in order to evaluate their accuracy and acceptability to warrant the additional costs related to them. Additional costs result from the more elaborate laboratory testing and probable thicker sections that are stipulated. These costs may be offset, however, by more accurate thickness designs and evaluation of contemplated pavement materials and could, on occasions, also give lower thicknesses for some of the coarser materials as compared to the Idaho R-value method. Accordingly, it is suggested that the following could be some of the research and implementation topics that could be pursued by the Division of Highways:

1. Develop laboratory test familarity with the M_R device sent to the Boise Laboratory, and put a computational program on the State computer,

such as Chevron 5-L, for terminal usage from the Boise Laboratory.

2. Obtain M_R data on Idaho subgrade soils and determine locations where pavement thickness requirements by M_R are greater or lower than with the Idaho R-value method.

An evaluation of the design water content and density for M_R subgrade soil specimens needs to be made if it is thought that resiliency design should be done with water contents and densities different than determined by Idaho R-value.

Also, to supplement this step, Idaho would need to determine the kind of design criteria to be used: vertical strain on subgrade, tensile strain in asphalt concrete, surface deflection, etc. (Nationally, the strain criteria are currently popular, perhaps more rational than the surface deflection, but surface deflection is also being used.)

3. Evaluate the field performance results of pavements designed by M_R and elastic layer computation from other recent agency experience, such as Colorado, Illinois, Minnesota, Asphalt Institute, etc.

4. Determine if a "resiliency factor" can be applied (from M_R test data on subgrade soils) to R-values so that the Idaho R-value thickness design method is modified to produce more desired results for certain troublesome fine-grained soils, and if this factor has promise as an iterim procedure.

5. Determine if the thickness requirement using M_R and elastic layer computation is a better overall procedure than the Idaho R-value method and if the computational effort can be reduced by nomographs or graphs such as included in the newest edition of rational thickness design printed by the Asphalt Institute.

Some pavements with test sections could be constructed around the

State designed by both Idaho R-value and M_R methods, where the M_R method section involves about 1000 ft. of pavement. Traffic accumulation, pavement deflection, and current serviceability index, would need to be carefully and continuously monitored over the years of accumulated traffic to the design traffic value in order to ascertain the advantages and disadvantages of the probable additional costs of the thicker M_R sections.

6. Supplemental to above, the laboratory M_R device could be modified to be more versatile. For example, a Statham gage horizontal displacement yoke could be made so that 4 inch x 2.5 inch laboratory specimens of asphalt concrete and other cohesive materials could be tested in the device by indirect tension to find their M_R. The advantage would be that the M_R device would be capable of M_R determination for all kinds of pavement materials.

The University of Idaho would be happy to assist in most of the above recommendations as part of its graduate program when it appears to the Division of Highways that the type of task can best be accomplished at U. of Idaho as a supplement to the larger, overall project at the Division of Highways or the Transportation Department.

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