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Correlation of Subgrade Reaction

SAM I. THORNTON

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16. Abstract

Westergaard's modulus of subgrade reaction (K-value) is used for designing rigid (concrete) pavements. K-value is obtained from an expensive and time consuming field Plate Bearing Test.

In order to investigate the correlation of K-value with laboratory tests, fifteen values of field K ranging from 96 psi/in to 667 psi/in were obtained. The K-values, taken at ten sites, were all fill embankments under construction. For the samples, CBR ranged from 4.8 to 27.6, R-value from 17.9 to 74.2, and M_R from 4,790 psi to 32,500 psi at principle stresses = 15 psi. The samples were classified from A-2-4 to A-6 (AASHTO).

A correlation study between K-value and laboratory test results did not show any correlation between the K-value and CBR, R-value, or M_R, when tests are performed by standard ASTM procedures. However, a relation between K-value and CBR at Field moisture (not optimum moisture) for A-4 and A-6 soils had linear regression of K = 13.8 CBR + 80.6 with a determination coefficient equal to 0.7785. The relations for A-4 and A-6 soils is based on 7 samples taken at 4 sites.

17. Key Words	18. Distribution Statement
Westergaard's modulus, K-value, correlation, rigid pavement design, CBR, R-value, Resilient Modulus	NO RESTRICTIONS

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CORRELATION OF SUBGRADE REACTION WITH CBR, HVEEM STABILOMETER, OR RESILIENT MODULUS

SAM I. THORNTON

IMPLEMENTATION

Arkansas soils classified A-4 or A-6 may now be used in rigid pavement design by determining the CBR at field moisture. Design nomographs modified from the 1972 AASHTO Pavement Design Guide can be used directly with CBR in the same way K-value is used.

Use of CBR at field moisture will save time and cost less than the field plate bearing test.

GAINS, FINDINGS, AND CONCLUSIONS

The following list includes the primary gains and conclusions of this study:

1. A correlation exists between K-value and CBR at field moisture content for A-4 and A-6 soils. The relation is K = 13.8 CBR + 80.6. Care should be used when using this relation because it is based only on seven data points.

2. No relation was found between K-value and R-value or Resilient Modulus.

3. No relation was found between K-value and ASTM standard CBR or between K-value and CBR at field moisture when A-2 soils were included. A-2 soils may have failed to correlate because of the necessary removal of particles over 0.75 inches.

4. A relation between R-value and ASTM standard CBR was found for A-2-4 soils. The relation is CBR = 0.317 R + 3.701.

SUMMARY OF IMPLEMENTATION

Practical Application: Correlation of CBR at field moisture content with K-value provides designers of rigid pavements with an alternative test to the plate bearing test for AASHTO A-4 and A-6 classified soils.

Recommended Procedure: Use of Figures 4.20 and 4.21 are recommended because they are based on the lower 90% confidence line of the correlation and therefore will be conservative 95% of the time. The figures should only be used for CBR values at field moisture content (not at optimum moisture as ASTM requires).

Benefits: The use of CBR at field moisture content takes less time and is less costly than the field plate bearing test.

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INTRODUCTION

Arkansas Highway and Transportation Department uses the "AASHTO Interim Guide for Design of Pavement Structures," 1972, for designing rigid or concrete pavements.

Rigid pavement design requires the evaluation of the terminal serviceability index (P_t), equivalent 18-Kip (80 kn) single-axle loads, the modulus of subgrade reaction (K-value), and the working stress in the concrete (f_t).

Westergaard's modulus of subgrade reaction (K-value) is a critical element in the design of rigid pavements. K-value is determined from the Plate Bearing Test (PBT), an expensive and time consuming test costing from $1,000 to $5,000 per test in 1981. This study attempts to correlate the modulus of subgrade reaction (K-value) with more economical and less time consuming laboratory tests, the California Bearing Ratio (CBR), Hveem Stabilometer (R-value), or Resilient Modulus (M_R).

Chapter 2

LITERATURE REVIEW

Subgrade Reaction, K

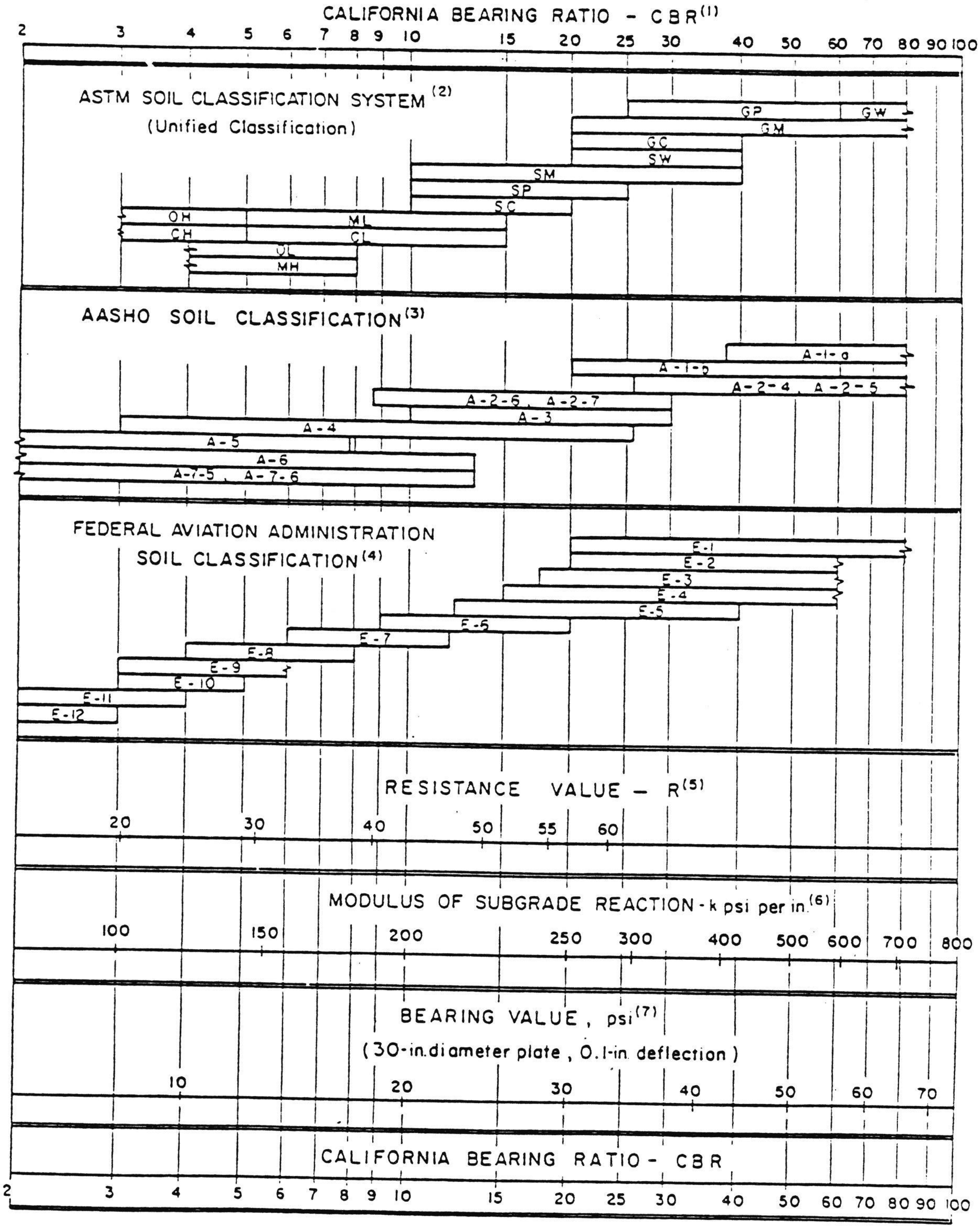
The 1972 AASHTO Interim Guide for Design of Pavement Structures (p. 30) defines Westergaard's modulus of subgrade reaction, K, as "the load in pounds per square inch on a loaded area divided by the deflection in inches of that loaded area". Subgrade reaction in this design guide is determined by plate loading tests performed per AASHTO T222 using a 30-inch diameter plate.

The AASHTO test for K requires forces up to 25,000 pounds. The force may be provided by: two loaded dump trucks with a beam between them, a beam spanning 16 feet between two anchors placed in the soil, or a loaded flat bed truck with sufficient clearance for load measurement equipment. Because of the load requirements, the tests are expensive and time consuming (Oglesby, 1975, p. 653).

In order to save the time and expense of the subgrade test, correlations with K-value or R-value like those shown in Figure 2.1 published by the Portland Cement Association (PCA, 1973, p. 27) are widely employed.

Estep and Wagner (1968, p. 214), however, believed that the PCA chart was constructed by comparing K-value and R-value with California Bearing Ratio (CBR).

Recent attempts were made to permit the use of smaller and therefore more economical plates. Butterfield and Georgiadis (1981, p. 60) propose "a new procedure for interpreting plate-bearing test that allows the complete nonlinear pressure vs. displacement curve to be described in terms of stiffness that are quite independent of the plate size".



(1) For the basic idea, see O. J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, pages 100-136.

(2) "Characteristics of Soil Groups Pertaining to Roads and Airfields," Appendix B, *The Unified Soil Classification System*, U.S. Army Corps of Engineers, Technical Memorandum 3-357, 1953.

(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, 1945, Vol. 25, pages 376-392.

(4) *Airport Paving*, U.S. Department of Commerce, Federal Aviation Agency, May 1948, pages 11-16. Estimated using values given in FAA *Design Manual for Airport Pavements*.

(5) F. N. Hveem, "A New Approach for Pavement Design," *Engineering News-Record*, Vol. 141, No. 2, July 8, 1948, pages 134-139. *R* is factor used in California Stabilometer Method of Design.

(6) See T. A. Middlebrooks and G. E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, page 152. *k* is factor used in Westergaard's analysis for design of concrete pavement.

(7) See item (6), page 184.

Figure 2.1 Approximate interrelationships of soil classifications and bearing values.
(PCA Soil Primer, 1973, p. 27)

Unfortunately, the model requires conventional plate tests on plates of two different sizes.

The AASHTO subgrade reaction test is not usually conducted under the worst possible field condition, i.e. when the soil is saturated. "To simulate the effects of saturation, two samples of the subgrade are subjected to a short term laboratory consolidation test of 10 psi, one in the original condition and one inundated. The ratio of the "as is" settlement to the inundated sample is multiplied by the field K factor to obtain a K value that is corrected for saturation" (Sowers & Sowers, 1970, p. 249).

Resilient Modulus, M_R

The resilient modulus is determined from a Repeated-Load Triaxial-Compression Test. In this test the resilient moduli correspond to recoverable or resilient deformations (Seed, et.al., 1967, p. 20).

$$M_R = \frac{\text{Stress Amplitude}}{\text{Strain Amplitude}} \qquad \text{(Lottman, 1976, p. 50)}$$

where,

stress amplitude = load/area of the specimen
strain amplitude = recoverable deformation/original height

Factors which affect M_R include the following (Table 2.1): load or stress duration, load frequency, grain size, void ratio, degree of saturation, confining pressure, and stress level (Seed, et.al., 1967, p. 24; Majidzadeh, 1978, p. 134; Lottman, 1976, p. 55). In a summary of these factors, Seed, et.al. (p. 24) stated, "The rate of load application, although having an influence, is not of major importance - a reasonable loading rate consistent with moving traffic can be utilized.

TABLE 2.1

Factors Affecting the Resilient Modulus

Stress Duration	M_R increases slightly when time of load application is reduced
Frequency	M_R increases with increased frequency of load application
Grain Size	M_R - moisture and density relation is dependent on soil type
Void Ratio	Granular soil samples tend toward the same void ratio after several hundred load repetitions
Saturation	M_R decreases by a factor of 4 as a result of saturation
Confining Pressure	Increases in confining pressure result in large increases in M_R
Stress Level	Stress level has little effect on M_R so long as the sample has little plastic deformation

Frequency, on the other hand, may influence results significantly, and some indication of the frequency of load applications should be considered. A representative number of repetitions consistent with the field conditions should also be used. The major difficulty is to define the stress condition under which the resilient behavior of the material should be measured".

Hsu and Vinson, 1981, also found that the confining pressure has a significant effect on the resilient modulus associated with cohesionless subgrade soils.

According to Medina and Preussler, 1982, as the compaction moisture content increases, the resilient modulus decreases. Medina and Preussler, 1982, also presented a tentative classification according to resiliency for sandy and clayey soils.

California Bearing Ratio, CBR

CBR is an index of strength and deflection characteristics of a soil that has been correlated with pavement performance (Sowers & Sowers, 1970, p. 249). The test is performed on soil confined in a steel cylinder 6 inches in diameter and 5 inches thick. A 1.9 inch diameter piston is then forced into the soil. The CBR is the percentage of the soil load required to produce a 0.1 inch deflection compared to a standard crushed stone.

A poor correlation between subgrade support and CBR under field conditions was reported by Kondner and Krizek (1972), p. 578). A curve of the relation with data shown is given in Figure 2.2.

The report in 1957 by Nascimento and Simones to the Fourth International Conference on Soil Mechanics was more encouraging (p.

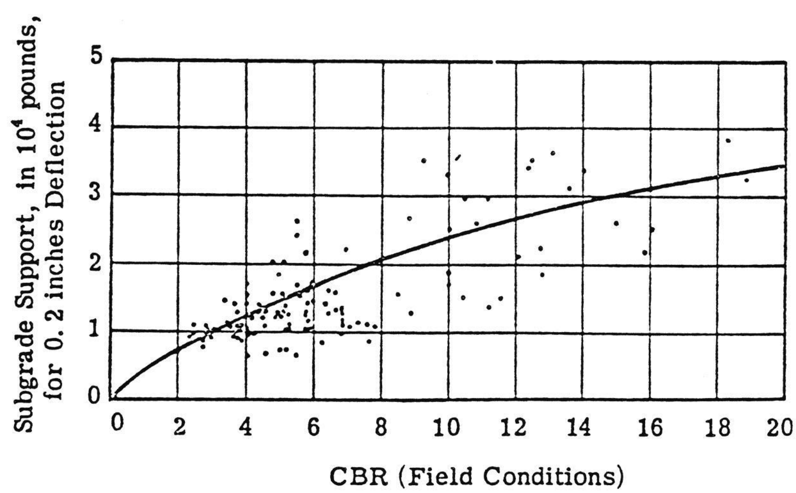


Figure 2.2

Subgrade Support vs. Field CBR
(from Kondner and Krizek, 1962, p. 578)

166-168), "The conclusion is drawn that the modulus of subgrade reaction, K_s, is 1/8 to 1/4 of CBR for soft material and 1/8 to 1/3 of CBR for hard materials".

Hveem Stabilometer, R

The R-value is a number expressing the measure of a soil's ability to resist the transmission of a vertical load in a lateral direction (Hines, 1978, p. 1.). Details of the test method are contained in ASTM D 2844 or AASHTO T-190.

Attempts in California to confirm the relationship of K vs. R-value as shown in Figure 2.1 were reported in 1968 by Estep and Wagner (p. 214). The attempts, however, were not completely successful, "When k-value was plotted vs. R-value, no direct correlation was found as had been predicted. However, we were able to develop a curve which lies at or below the minimum k-values measured for various R-values".

Based on the work of Estep and Wagner, Nielson, et.al. (1969, p. 6) proposed the following relationship between k-value and R-value:

$$K = 0.401 + 2.546R - 0.042R^2 + 0.0008R^3$$

Nielson's equation, based on a least squares fit of a polynomial equation, was accompanied by a figure showing considerable scatter in the data. Nielson's figure is unfortunately not suitable for reproduction.

Attempts have been made to correlate R-value with other tests, especially the resilient modulus M_R. Buu (1980, p. 20) working in Idaho found a good correlation between M_R and R-value for coarse grained soils with the coefficient of correlation r = 0.906. According to the correlation, for coarse grained soils with low plasticity:

$$M_R = 1.455 + 0.057 \text{ (R-value)}$$

The correlation for fine grained soils, however, was poor with a coefficient of correlation $r = 0.330$. For fine grained soils for moderate or high plasticity and R-value higher than 20:

$$M_R = 1.601 + 0.038 \text{ (R-value)}$$

Dr. Gary Hicks along with Dr. Ted Binson are currently (January, 1982) finishing some tests for the Oregon DOT which were begun by Gregg. They reported verbally that no correlation was found between M_R and R-value. Hines (1978), p. 26) in a study for the Colorado Division of Highways reports a good coefficient of correlation; $r = 0.874$ for section averages and $r = 0.768$ for pooled results. Hines' report comments, "However, there is poor agreement with the present correlation. In both cases the regression line is shifted to the left of the present correlation. This could be caused by low laboratory moduli resulting from low confining pressures" (Hines, 1978, p. 26). Figures 2.3 and 2.4 are the figures taken from Hines' report.

In a direct warning that the R-value may not be suitable, Dr. Wayne Heiliger (1971) stated in his dissertation abstract, "An investigation of high strength pavement components disclosed that the R-value test equipment and procedure is not valid for materials which possess high resistance to lateral deformation". This warning is particularly disturbing because the dissertation is titled "Adaptation of the General AASHO Road Test Equation to Arkansas Conditions".

Other Tests

Attempts have also been made to correlate the subgrade reaction K with other tests. The modulus of elasticity was suggested by Vesic

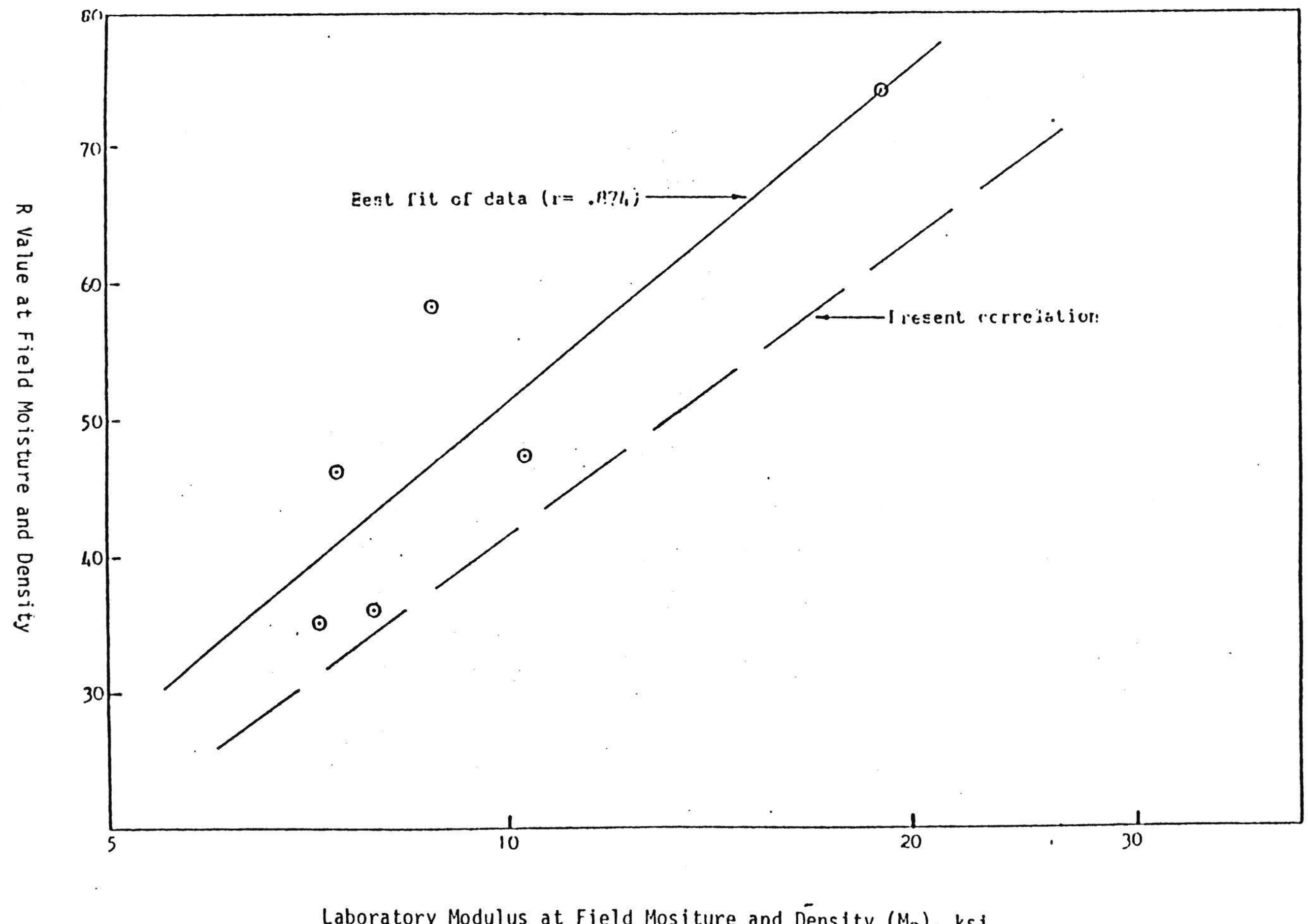


Figure 2.3 R-value vs. M_R (from Hines, 1978, P. 31)

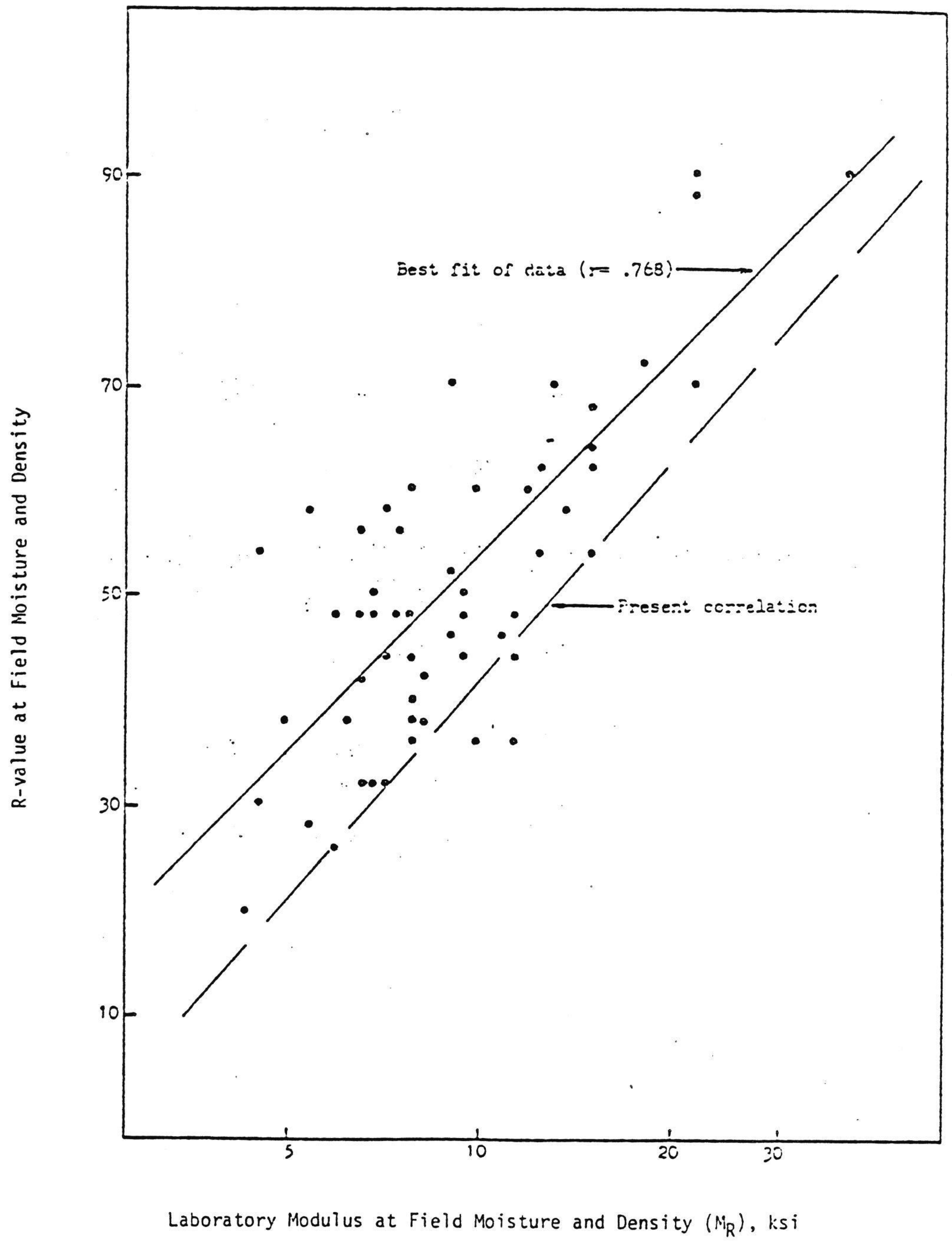


Figure 2.4 R-value vs. M_R (from Hines, 1978, P. 32)

(Winterhorn and Fang, 1975, p. 517) and Carothers (1964, p. 32). Myers and Kinchen (1972, p. 16) suggest a correlation with the Dynaflect test. Butt, et.al. (1968), p. 70) suggest a sphere bearing test.

Chapter 3

FIELD AND LABORATORY INVESTIGATIONS

In November, 1981, the field investigation was started. During the period of late November and early December, 1981, the Plate Bearing Test, Field Density, and sample collection for laboratory investigations were performed in the field.

The laboratory investigations started in early January, 1982, and continued for the following eight months.

California Bearing Ratio, Resilient Modulus, and Hveem stabilometer (Resistance Value) were performed in the laboratory on the compacted samples.

Sites

The research committee decided to conduct the field investigations at ten different sites. The committee also recommended that the sites be representative Arkansas soils as well as variable in strength. The ten sites selected by the Arkansas Highway and Transportation Department (AHTD) are shown in Figure 3.1.

Plate Bearing Test

The Plate Bearing Test was performed by Test Inc., a soil testing firm from Memphis Tennessee.

For the purpose of performing the Plate Bearing Test a trailer truck is needed to provide a reaction load. The AHTD, Arkansas Highway and Transportation Department, furnished a trailer truck with gross weight of 68,000 lbs (Figure 3.2). Test, Inc., furnished the rest of the

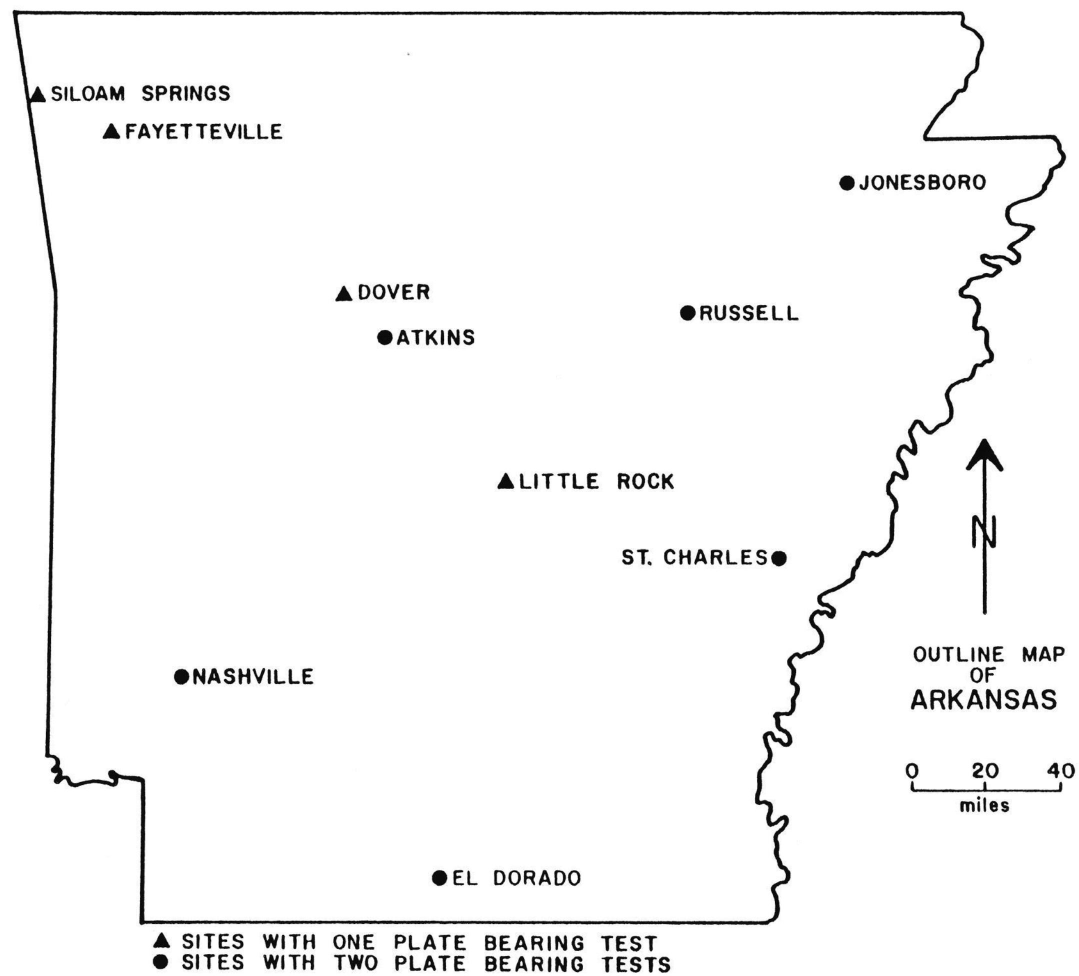


Figure 3.1 Sites of the Plate Bearing Test

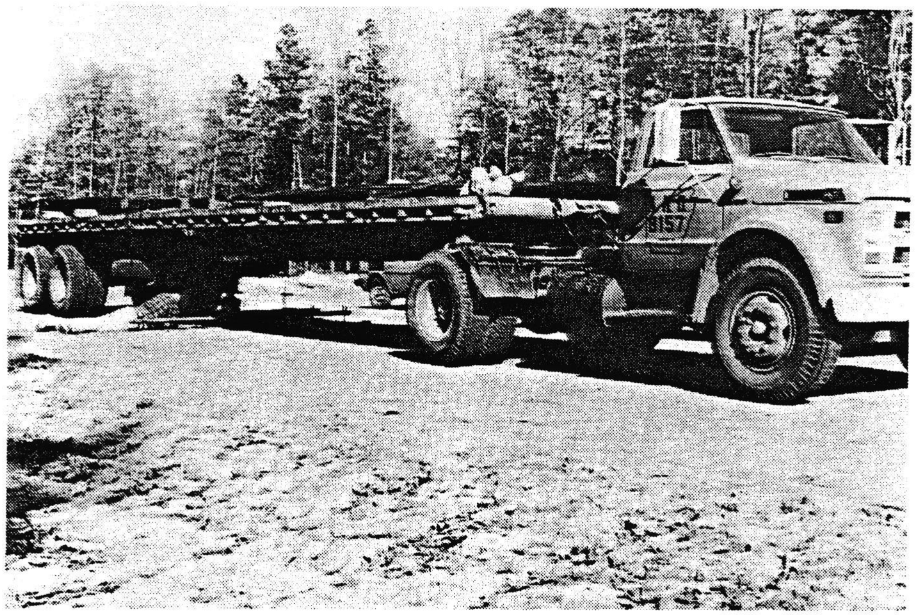


Figure 3.2 Trailer truck used for Plate Bearing Test

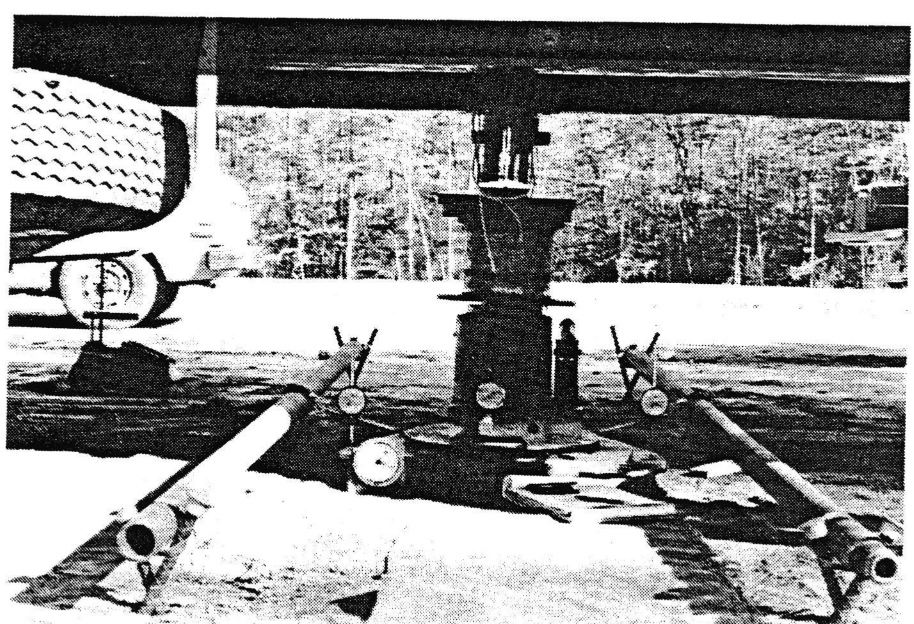


Figure 3.3 Set-up of the Plate Bearing Test

equipment needed to perform the Plate Bearing Test including a 100 ton jack and the 30 in. diameter bearing plate (Figure 3.3).

A total of sixteen Plate Bearing Tests were performed at ten sites. At six of the sites two Plate Bearing Tests were conducted (Figure 3.1) in order to check the variation between the Plate Bearing Test at a site. The second tests were performed approximately 20 to 30 ft. away from the first tests. All of the tests were performed on fill subgrade still under construction.

Except for one modification, the Plate Bearing Test was conducted in accordance with AASHTO T222-66 (1974) (the same as ASTM D1196-61 1971). The AASHTO specifications call for 18 ft. long deflection beams which should rest on supports located at least 8 ft. from the circumference of the bearing plate, nearest wheel, or supporting leg. Wind blowing against the deflection beams caused disturbance in the deflection readings. In order to reduce the disturbance, 10 ft. long deflection beams were used instead of 18 ft. beams. Figure 3.4 shows the Plate Bearing Test in progress using the 10 ft. long beams. Plots of load versus deflection are presented in Appendix A (Figure A-1).

Field density and moisture content were measured by nuclear density gages at 3 ft. from the circumference of the bearing plate. The nuclear density gages were furnished by the AHTD.

Sample Collection and Preparation

Disturbed samples for laboratory investigations were collected at 3 ft. from the circumference of the bearing plate by the AHTD. At the end of the field investigation, the samples were delivered to the Soil Mechanics Laboratory of the University of Arkansas, Fayetteville campus, by the AHTD.

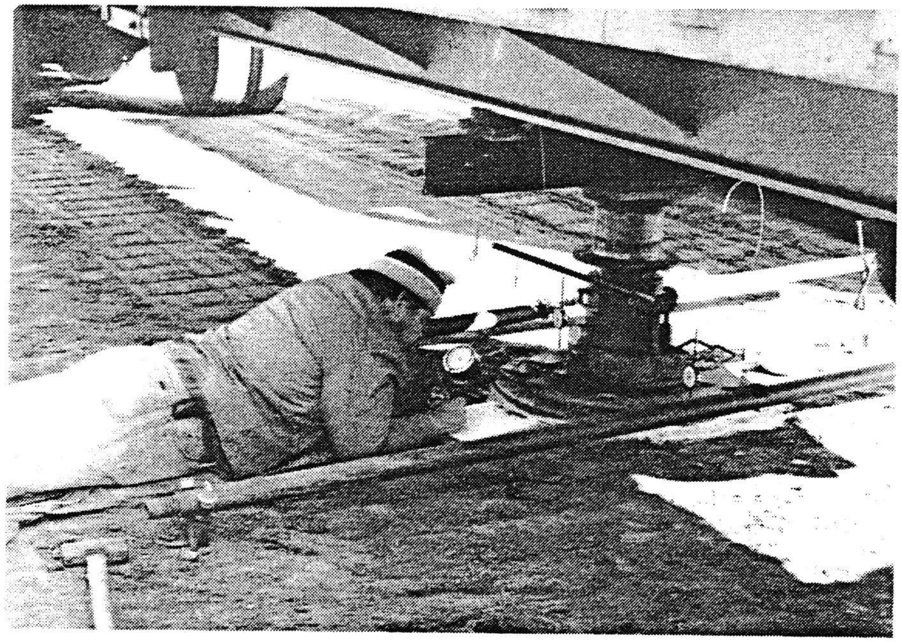


Figure 3.4 Plate Bearing Test in progress

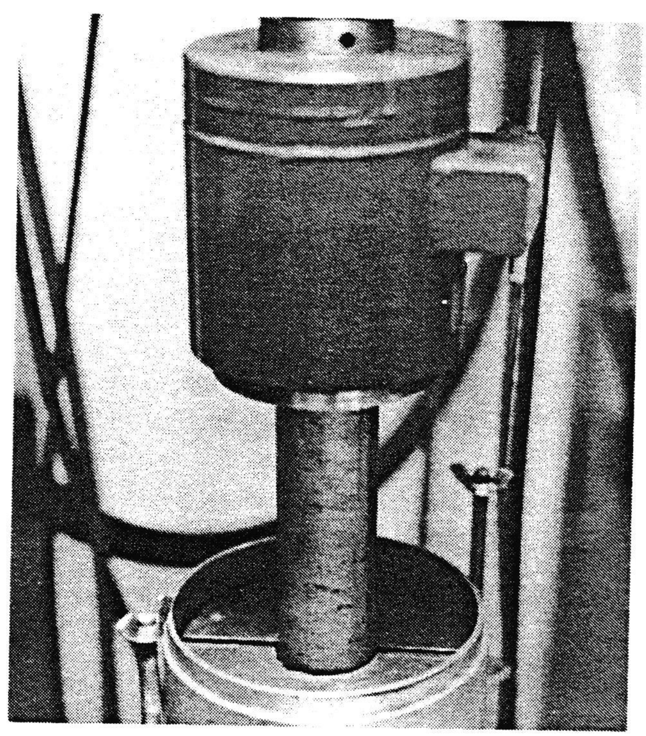


Figure 3.5 Penetration Test on CBR samples

The samples, approximately 120 lbs to 200 lbs each, were oven dried at 140°F. Oven dried samples were disaggregated with a rubber pestle.

Classification

Grain size analysis, liquid limit, and plastic limit were performed in order to classify the soils. Sieve analysis results were adequate for classification, therefore, no Hydrometer Analysis was conducted. The samples were classified in accordance with AASHTO and UNIFIED classification (Table 4.2). Classification ranged from A-2-4 to A-6 by the AASHTO system.

Specific gravity of the material passing the No. 4 (0.187 in.) sieve was obtained for each sample (Table 4.2).

California Bearing Ratio (CBR)

CBR is a bearing ratio determination of the laboratory compacted soil samples to that of a standard material (crushed stone).

The standard proctor compactive effort (5.5 lb hammer/12 in. drop/3 layers) was used to compact the samples in accordance with ASTM D 698-70 (6 in. mold/soil material passing a 3/4 in. sieve). Prior to compaction, the soil samples were mixed with water and stored in a 100% relative humidity moisture chamber for 24 hours. The specimens were compacted using a Rainhart automatic laboratory compaction apparatus with a sectorfaced hammer.

The bearing ratio was determined in accordance with ASTM D 1883-73. This method covers the evaluation of the relative quality of subgrade soils.

To determine the bearing ratio a 1.9 in. diameter piston is pushed 0.5 inch into the specimen at a rate of 0.05 inch per minute. (Figure 3.5).

The bearing ratio is calculated in the following manner: Using the load values (in psi) taken from the load-penetration curve for 0.1 in. and 0.2 in. penetration, the bearing ratios for each is calculated by dividing the loads by the standard loads of 1000 psi and 1500 psi respectively, and multiplying by 100.

The common practice is to soak the CBR specimens for a period of 96 hours in order to saturate the samples and measure the swelling of the soil. But, in order to reproduce the unsaturated field condition at the time of the plate bearing test, the penetration was performed on the unsoaked specimens immediately after the compaction. During the penetration test a surcharge weight of 10 lbs was applied to the specimens (Figure 3.5). Load-penetration curves for three specimens (below, at, and above optimum moisture content) are presented in Appendix A (Figure A-2).

Resilient Modulus (M_R)

The resilient modulus is the ratio of deviator stress to resilient axial strain. The M_R test is a dynamic test.

A standard procedure to conduct the Resilient Modulus test does not exist at the present time. The method used in this study follows, with minor modifications, the "Suggested Method of Test for Resilient Modulus of Subgrade Soils," prepared by the Department of Civil Engineering of the University of Idaho at Moscow, Idaho for the Idaho Transportation Department, Division of Highways, Boise, Idaho. This method, with minor correction, is expected to be approved by the AASHTO Materials Committee. The Resilient Modulus method used is described in Appendix B in detail.

The specimens for M_R tests were compacted by static compaction to the same density and moisture content that existed in the field at the time of the Plate Bearing Test. The method of static compaction used is described in detail in Appendix B.

The mold (2.75 in. inside diameter and 6 in. high) and piston used to prepare the M_R specimens are shown in Figure 3.6. The three layer (2 in. each) specimens were compacted with a constant displacement rate of 0.05 in. per minute (Figure 3.7). Prior to compaction the soils were mixed for the desired moisture and stored in a 100% relative humidity moisture chamber for 24 hours. After compaction the samples were stored in the moisture chamber for 24 hours again prior to the testing.

The soil samples were divided into two categories. The soils with PI of 10 or more (cohesive) and soils with PI of less than 10 (granular).

The various combinations of the chamber pressures (σ_3) and deviator stresses (σ_d) that were applied to the cohesive and granular residual modulus samples are presented in Table 3.1. At each of the chamber pressures, 200 repetitions of σ_d were applied.

A haversquare pulse load (0.1 second load duration) at 30 repetitions per minute was used to apply the deviator stress. Loads were applied with an MTS 810 machine (Figure 3.8). Prior to testing, samples were conditioned with loading applications (Appendix B).

At the 200th repetition the residual, ε_r, was recorded. M_R was calculated by dividing σ_d by ε_r. Plots of M_R versus the sum of principle stresses ($\theta = \sigma_d + 3\sigma_3$) are presented in Appendix A (Figure A-3).

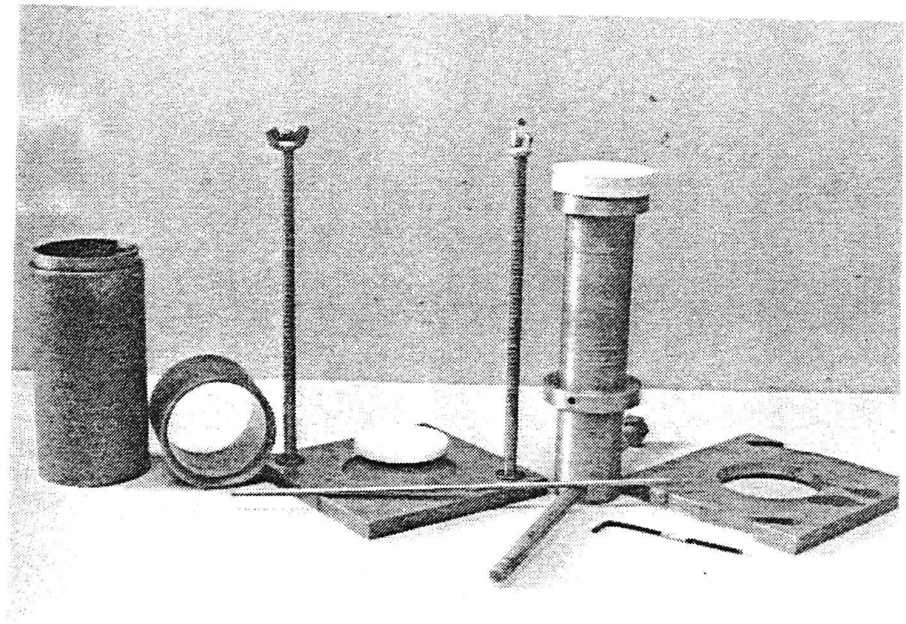


Figure 3.6 Mold and piston used for compaction of Resilient Modulus samples

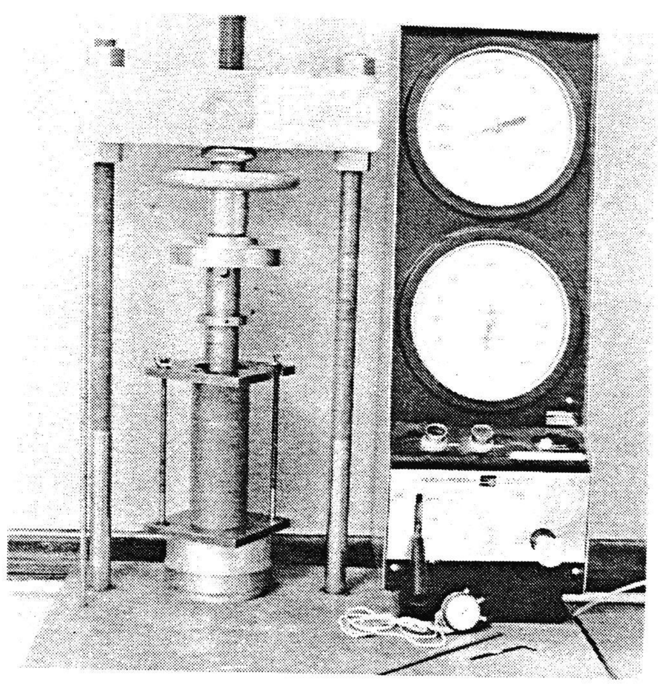


Figure 3.7 Static compaction of Resilient Modulus samples

Table 3.1 Applied Stresses in Residual Modulus Test

Cohesive		Granular	
σ_3, psi	σ_d, psi	σ_3, psi	σ_d, psi
6	1,2,4,8, and 10	20	1,2,5,10, and 20
3	1,2,4,8, and 10	15	1,2,5,10, and 20
0	1,2,4,8, and 10	10	1,2,5,10, and 15
		5	1,2,5,10, and 15
		1	1,2,5,7.5, and 10

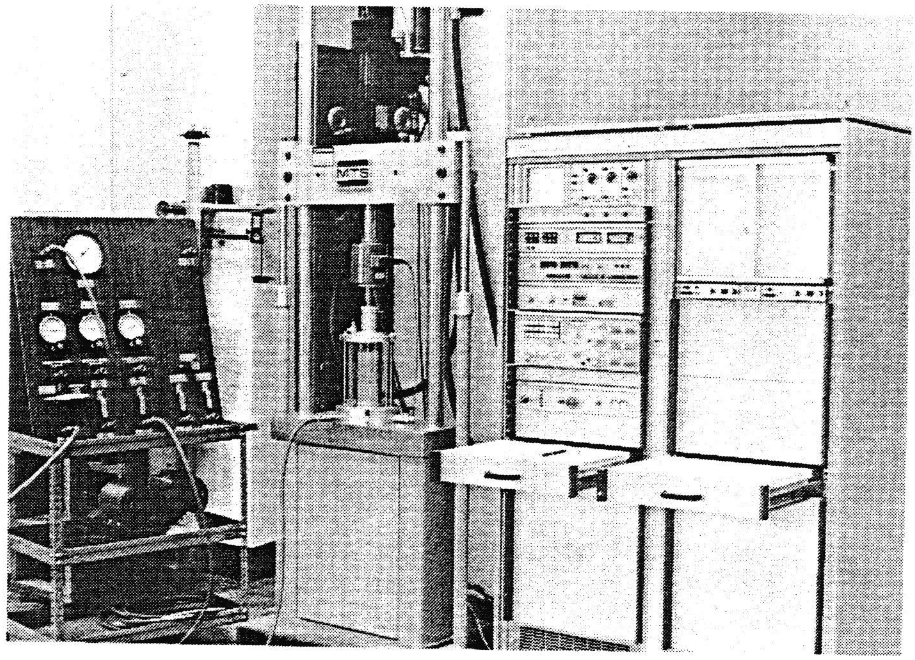


Figure 3.8 Resilient Modulus Test in progress

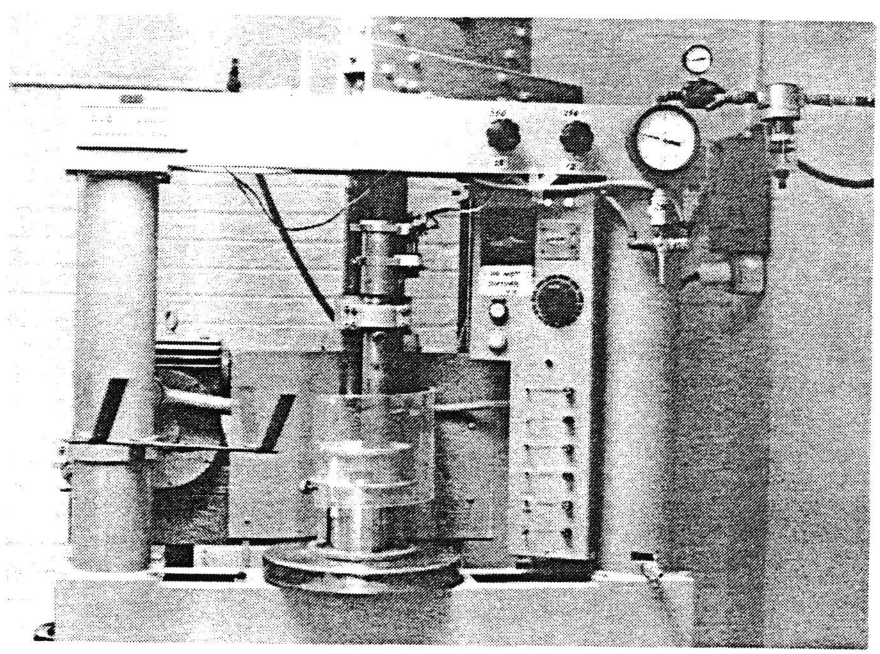


Figure 3.9 LUCAS Kneading Compactor

Hveem Stabilometer (R-value)

The Hveem Stabilometer measures the resistance offered by a soil to transmission of a vertical load in a lateral direction. The resistance or R-value is expressed as the ratio between the lateral transmitted pressure and a vertical pressure of 160 psi which is applied with a testing press (ASTM D 2844-69[75]).

Because the AHTD uses materials which pass the No. 4 sieve for R-value samples, materials passing the No. 4 sieve were used in this study. R-value samples were mixed with water and stored in a 100% relative humidity moisture chamber for 24 hours prior to compaction.

The samples were compacted by means of a mechanical kneading type compactor (Figure 3.9). The mechanical compactor manufactured by GEO. R. LUCAS consolidates the material without static compression or damaging impact; instead a series of individual impressions are made. The kneading ram has a face shaped like a sector of a 4 in. diameter circle. At each application of the ram a pressure of 350 psi is applied over an area of 3.1 square inches. This pressure is maintained for approximately 1/2 second (Grubbs and Roberts, 1966, p. 7).

At least three specimens with different moisture contents were compacted. After compaction the exudation pressure was obtained for each of the R-value samples by use of the exudation indicator device (Figure 3.10).

The R-value samples were forced from the mold into the Hveem Stabilometer (Figure 3.11). Vertical load (P_v) was applied and the produced horizontal pressure (P_h) was determined. Then the lateral displacement (D) of the sample was measured by applying horizontal pressure. The R-value was determined by the following equation:

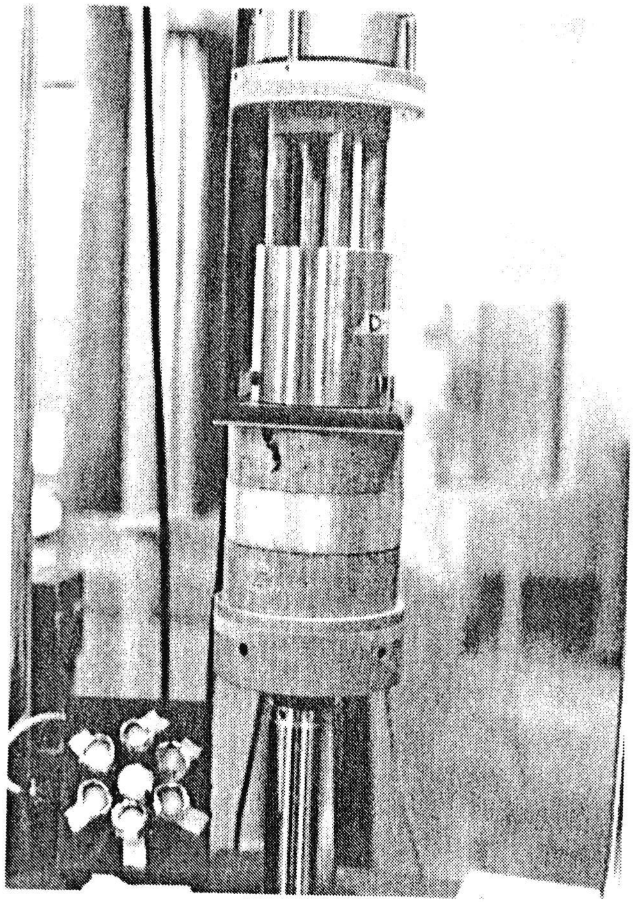


Figure 3.10 Exudation pressure indicator

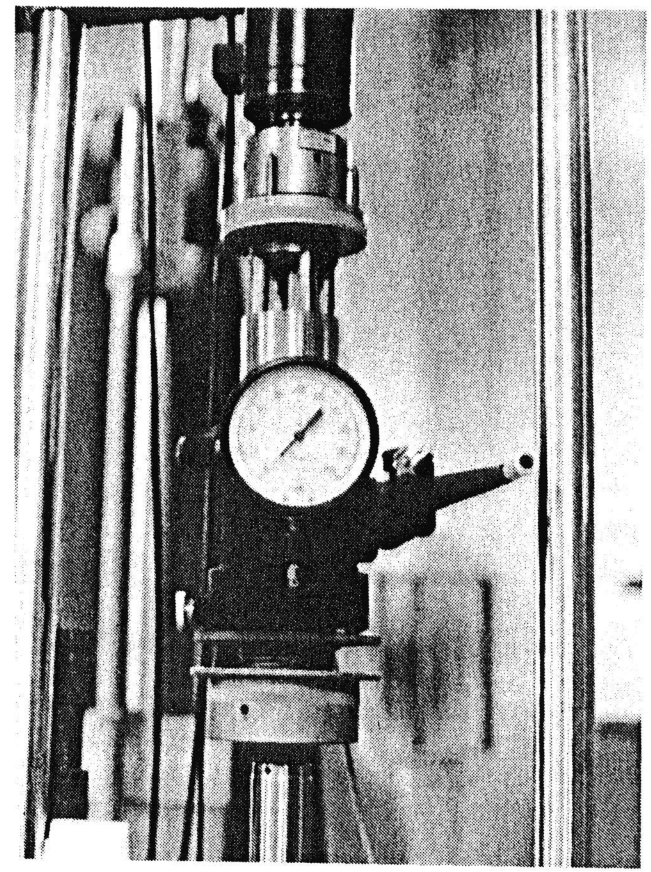


Figure 3.11 Hveem Stabilometer

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_v}{P_h} - 1\right) + 1} \qquad 3.1$$

where: P_v = 160 psi (2000 pounds)

P_h = horizontal pressure at P_v = 2000 pounds

D = Displacement due to horizontal pressure (number of turns)

Plots of R-values versus exudation pressure are presented in Appendix A (Figure A-4).

Chapter 4

RESULTS AND DISCUSSION

Modulus of Subgrade Reaction (K-value)

K-value represents the load in pounds per square inch on a loaded area divided by the deflection in inches of that loaded area (AASHTO, 1972). K-value is obtained from the Plate Bearing Test (PBT), where load up to 25,000 lbs is applied to a 30 in. diameter plate resting on the top of the subgrade soil by jacking against a fixed beam provided by a trailer truck or two dump trucks connected by a beam. The PBT was performed in accordance with AASHTO T222-66 (1974) which is the same as ASTM D1196-64 (1971).

The deflection was recorded by two dial gauges accurate to 0.001 in. set opposite from each other (Figure 3.3). The average of the two dial gauge readings is used for all calculations.

Sixteen Plate Bearing Tests were performed at ten sites. Two tests were performed at six of the ten sites and one test at four sites. At site N-1 (Nashville), the edge of the plate came up when the load was removed. One of the possible explanations is that the jack used to apply the load was not properly centered. Therefore, eccentricity, of the load, might have caused the negative rebound. As a result, sample N-1 was removed from the study.

The K-value, total deflection after rebound, field moisture content, and dry density for all samples are presented in Table 4.1.

The highest K-value obtained is 667 psi/in. for a sandy-clay soil classified as A-2-4 (AASHTO), and the lowest is 96 psi/in. for a

Table 4.1 Results of Field Investigation

Location	Sample	K-value psi/in.	Total deflection after rebound, in.	Field moisture content	Field dry density, pcf
Atkins	A-1	237	0.052	11.9	120.8
Atkins	A-2	279	0.052	14.5	114.4
Dover	D-1	123	0.174	17.5	110.1
El Dorado	E-1	468	0.018	19.9	106.0
El Dorado	E-2	148	0.124	15.4	106.1
Fayetteville	F-1	391	0.023	15.4	114.3
Jonesboro	J-1	357	0.043	7.1	125.4
Jonesboro	J-2	361	0.045	6.6	129.6
Little Rock	LR-1	667	0.024	7.5	131.4
Nashville	N-1	NA*	NA*	9.9	121.8
Nashville	N-2	345	0.046	8.2	112.6
Russell	R-1	301	0.051	16.5	108.7
Russell	R-2	269	0.063	17.7	107.5
Siloam Springs	SS-1	154	0.109	10.4	121.4
St. Charles	STC-1	96	0.148	20.1	104.3
St. Charles	STC-2	109	0.133	21.5	102.6

*Negative rebound at one side of the bearing plate, further investigation terminated.

clay-silt soil classified as A-4 (AASHTO). Generally, higher K-values were obtained at sites with higher density.

The results of the Plate Bearing Tests, in the form of Load-Deflection plots, are presented in Appendix A (Figure A-1). A linear regression was determined and plotted for each of the Load-Deflection results. The slope of this linear regression divided by the cross-section area of the bearing plate (706.80 in^2) is the Modulus of Subgrade Reaction (K-value).

Classification

The soil samples were classified in accordance with AASHTO and UNIFIED Classification Systems (Table 4.2). The classifications ranged from A-2-4 to A-6 (AASHTO) and from SM to CL (UNIFIED) representing a wide range of material.

Specific gravity (G_s) of all the samples of material passing through No. 4 sieve were obtained in accordance with ASTM D854 (Table 4.2).

California Bearing Ratio (CBR)

CBR is the bearing ratio of a laboratory compacted soil specimen which is tested by comparing the penetration load of the soil to that of a standard material (crushed stone with standard loads of 1000 psi at 0.1 in. and 1500 psi at 0.2 in. penetration).

CBR was conducted according to ASTM D1883-73. Several specimens were prepared and compacted in order to determine the maximum density for each soil. Penetration tests were performed on unsoaked specimens immediately after compaction. Although the soaked method of CBR is more commonly used, the test sites were not saturated at the time of the

Table 4.2. Results of Laboratory Investigations

Sample	K-value psi/in.	Classification AASHTO	Classification UNIFIED	G_s -No. 4	CBR @ Penetration 0.1 in	CBR @ Penetration 0.2 in	M_R* @ θ=15 psi	K_1 psi	K_2	R-value
A-1	237	A-4	SC	2.74	11.2	11.5	21,350	38,700	-0.2196	19.0
A-2	279	A-4	SC	2.74	14.2	13.8	19,050	44,500	-0.3132	17.9
D-1	123	A-6	CL	2.75	8.8	8.6	16,890	22,500	-0.1058	18.7
E-1	468	A-2-4	SC-SM	2.67	13.0	13.1	22,790	6,600	0.4576	28.4
E-2	148	A-2-4	SM	2.65	13.5	15.8	16,150	7,400	0.2882	44.0
F-1	391	A-2-4	SC	2.70	14.6	13.2	32,520	86,900	-0.3629	23.5
J-1	357	A-2-4	SM	2.64	20.4	27.6	19,490	11,700	0.1884	74.2
J-2	361	A-2-4	SM	2.65	12.8	14.7	25,960	21,900	0.0628	28.0
LR-1	667	A-2-4	SC	2.68	10.7	11.3	19,600	30,900	-0.1681	17.9
N-2	345	A-2-4	SM	2.67	21.7	27.1	16,230	17,700	-0.0320	71.9
R-1	301	A-6	CL	2.69	13.8	13.2	10,770	16,000	-0.1460	17.9
R-2	269	A-6	CL	2.69	7.6	7.5	4,760	6,300	-0.1038	18.2
SS-1	154	A-2-4	SC	2.70	4.8	6.2	26,490	19,700	0.1093	26.0
STC-1	96	A-4	CL-ML	2.66	17.2	17.9	12,010	5,900	0.2625	19.0
STC-2	109	A-4	CL-ML	2.67	14.2	13.6	9,930	5,100	0.2459	18.5

*$M_R = K_1 \theta^{K_2}$, θ = sum of principal stresses

Plate Bearing Test. Therefore, the penetration test was conducted on the unsoaked specimens in order to more nearly simulate field condition.

The samples from Fayetteville (F-1), Little Rock (LR-1), and Russell (R-1) sites were depleted in preliminary tests. AHTD obtained additional samples needed to continue the laboratory investigations on F-1, LR-1, and R-1 samples. LR-1 and R-1 samples, obtained later, were similar to the original samples collected during the field investigation and classified the same as the original samples. Sample, F-1, however, was a different material than the original F-1 sample collected during the field investigations and was not used in the study. The penetration test for the F-1 sample was performed on the remolded original sample.

Eight of the CBR values obtained showed higher values of CBR at 0.2 in. penetration than 0.1 in. penetration (Table 4.2). The bearing ratio reported for the soil is normally the one at 0.1 in. penetration. ASTM D1883-73 specifies that if the value of CBR at 0.2 in. penetration is greater than CBR at 0.1 in. penetration, a second test should be run. From the results of the second test the higher CBR value (at 0.1 or 0.2 in. penetration) should be reported. At this time the original samples stored in the laboratory were used to the point that a retesting of the CBR with unused soil would not leave any sample to continue the investigation. Therefore, CBR is reported at 0.1 and 0.2 in. penetration. CBR at 0.1 and 0.2 in. penetration for all the samples are presented in Table 4.2.

The plot of the penetration test results for three specimens of each soil sample are presented in Appendix A (Figure A-2).

Resilient Modulus (M_R)

The Resilient Modulus, M_R, is a dynamic test response defined as the ratio of repeated axial deviator stress, σ_d, to the recoverable or resilient axial strain, ε_r, or:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \qquad 4.1$$

(Rada and Witczak, 1981, p. 1)

Resilient Modulus samples were compacted by static compaction. Static compaction provides better control of the sample density than any other method of compaction. Static compaction is described in detail in Appendix B.

Resilient Modulus samples were prepared at the same density and moisture content as existed at the Plate Bearing Test sites.

The resilient deformations generally stabilize before 100 repetitions of load. Therefore, the Resilient Modulus is computed after 200 repetitions (Rada and Witczak, 1981, p. 1). A haversquare pulse load (0.1 second load duration) at 30 repetitions per minute was used for loading the Resilient Modulus sample. The Resilient Modulus is reported in the form of:

$$M_R = K_1 \theta^{K_2} \qquad 4.2$$

where M_R = Resilient Modulus, psi

θ = Sum of principal stresses, psi

K_1 and K_2 = Regression constants

K_1 and K_2 depend upon the material type and physical properties of the specimen during the test (Rada and Witczak, 1981, p. 1).

By varying the chamber pressure, (σ_3), and the deviator stresses, (σ_d), a series of M_R values were obtained for every sample. M_R values were plotted versus θ's on a log-log graph to obtain K_1 and K_2 constants from the regression analysis (Figure A-3). The values of K_1, K_2, and M_R at θ = 15 psi for each sample are presented in Table 4.2.

Rada and Witczak (1981, p. 4) observed that for granular materials a correlation between increasing K_1 and decreasing K_2 values exists (Figure 4.1). The number of M_R tests in this study (15) was low compared to that of Rada and Witczak's, yet the same type of correlation between K_1 and K_2 values was observed (Figure 4.1). Rada and Witczak (1981, p. 6) also state that the range of K_1 (and hence K_2) within a given class of soil appears to be significant.

Generally the plot of M_R versus θ at higher chamber pressures (σ_3 = 15 and 20 psi for granular soils) shows irregular results (Figure A-3). A better relationship exists between M_R and θ at lower chamber pressures. The plots of M_R versus θ indicate that at higher chamber pressures a larger negative slope exists than at lower chamber pressures (Figure A-3). To show the change of the slope at different chamber pressures, three arbitrary curves are plotted at the three lower chamber pressures, σ_3 (σ_3 = 1, 5, and 10 psi for granular soils and σ_3 = 0, 3, and 6 psi for cohesive soils) for all samples (Figure A-3). However, higher M_R values were obtained at lower deviator stresses σ_d regardless of the chamber pressure value (σ_3).

Hveem Stabilometer (Resistance, R-value)

The ability of the soil to resist plastic deformation is measured in terms of R-value (Howe, 1961, p. 5).

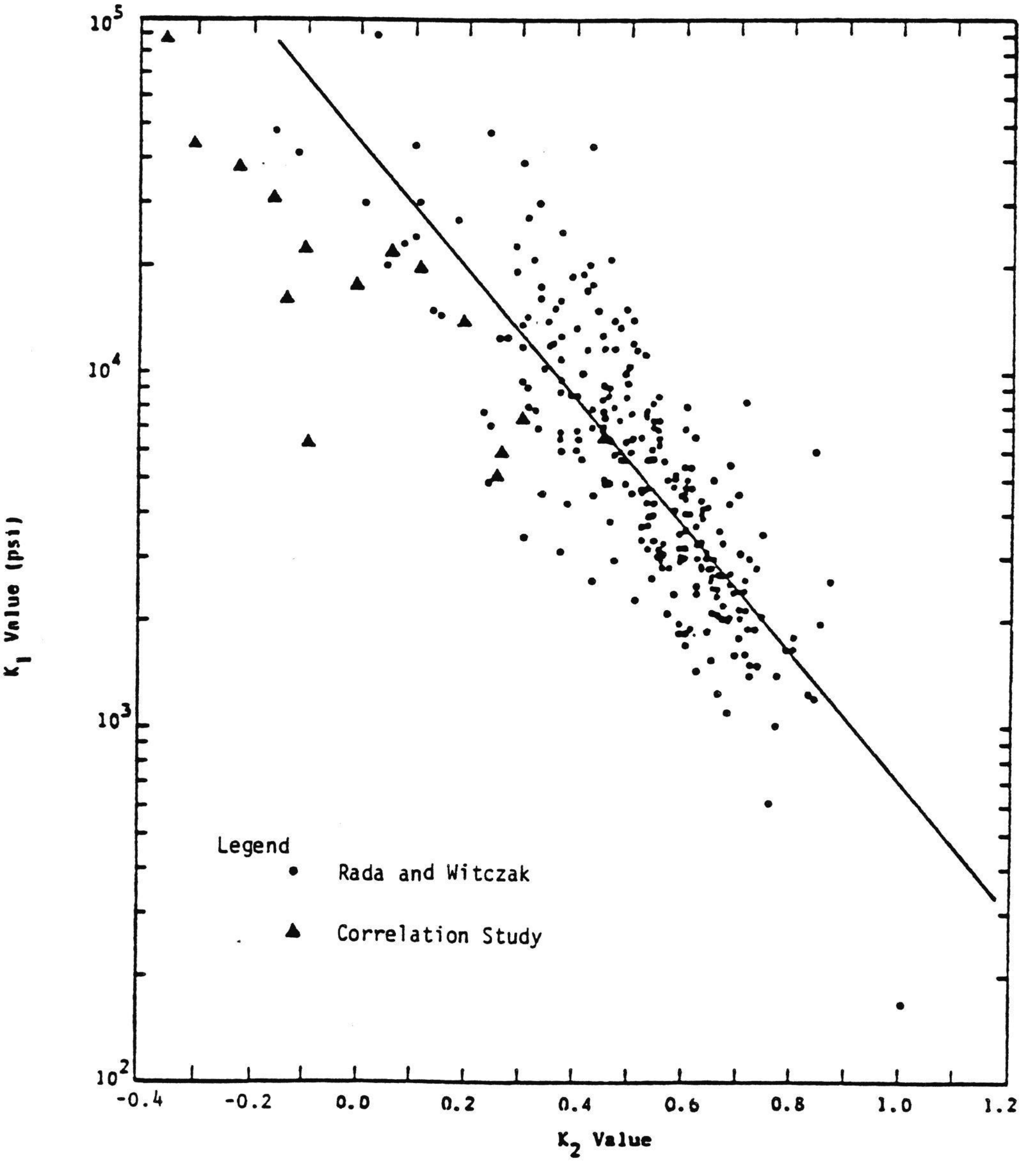


Figure 4.1 K_1-K_2 relationship (from Rada and Witczak, 1981, p. 29)

The R-value test was performed under the guideline given by the ASTM D2844 (same as AASHTO T-190). The samples for the R-value test were prepared by kneading compaction with the material passing No. 4 sieve. At least 3 specimens with different moisture contents were prepared from each sample.

R-value versus exudation pressure curves are presented in Appendix A (Figure A.4). R-value is reported at exudation pressure of 240 psi in AHTD specifications. The 240 psi exudation pressure is used because it was found to agree best with the results obtained at the AASHTO road test (Clements, 1967, p. 9).

Since R-value tests were conducted for the standard 300 psi, some of the 240 psi values had to be obtained by extrapolation. Table 4.2 presents the R-values obtained from the graph of the R-value versus exudation pressure (Figure A-4) at 300 psi exudation pressure. The R-values at 240 psi exudation pressure are presented in Table 4.3. The difference in R-value for samples J-1 and J-2 are similar for 300 psi and 240 psi and cannot be explained by extrapolation.

Howe (1961) showed that the stabilometer primarily reflects the internal friction factor and the effect of clay lubricity (Figure 4.2). Samples D-1, R-1 and R-2, which are classified as CL by UNIFIED classification system, have the lowest R-values obtained in this study (Table 4.2).

DISCUSSION

Results from the Modulus of Subgrade Reaction (K-value), California Bearing Ratio (CBR), Resilient Modulus (M_R) and Hveem Stabilometer (R-value) were analyzed using the Statistical Analysis System at the University of Arkansas Computer Center. The computer made the evaluation and plotting of the results easier and more accurate.

Table 4.3. R-values at 240 psi exudation pressure

Sample	R-value
A-1	16.4
A-2	13.7*
D-1	12.7
E-1	28.2*
E-2	37.2
F-1	17.9
J-1	72.1
J-2	26.8*
LR-1	15.3
N-2	71.7
R-1	12.0
R-2	13.0
SS-1	22.2
STC-1	20.8*
STC-2	16.3

*by extrapolation

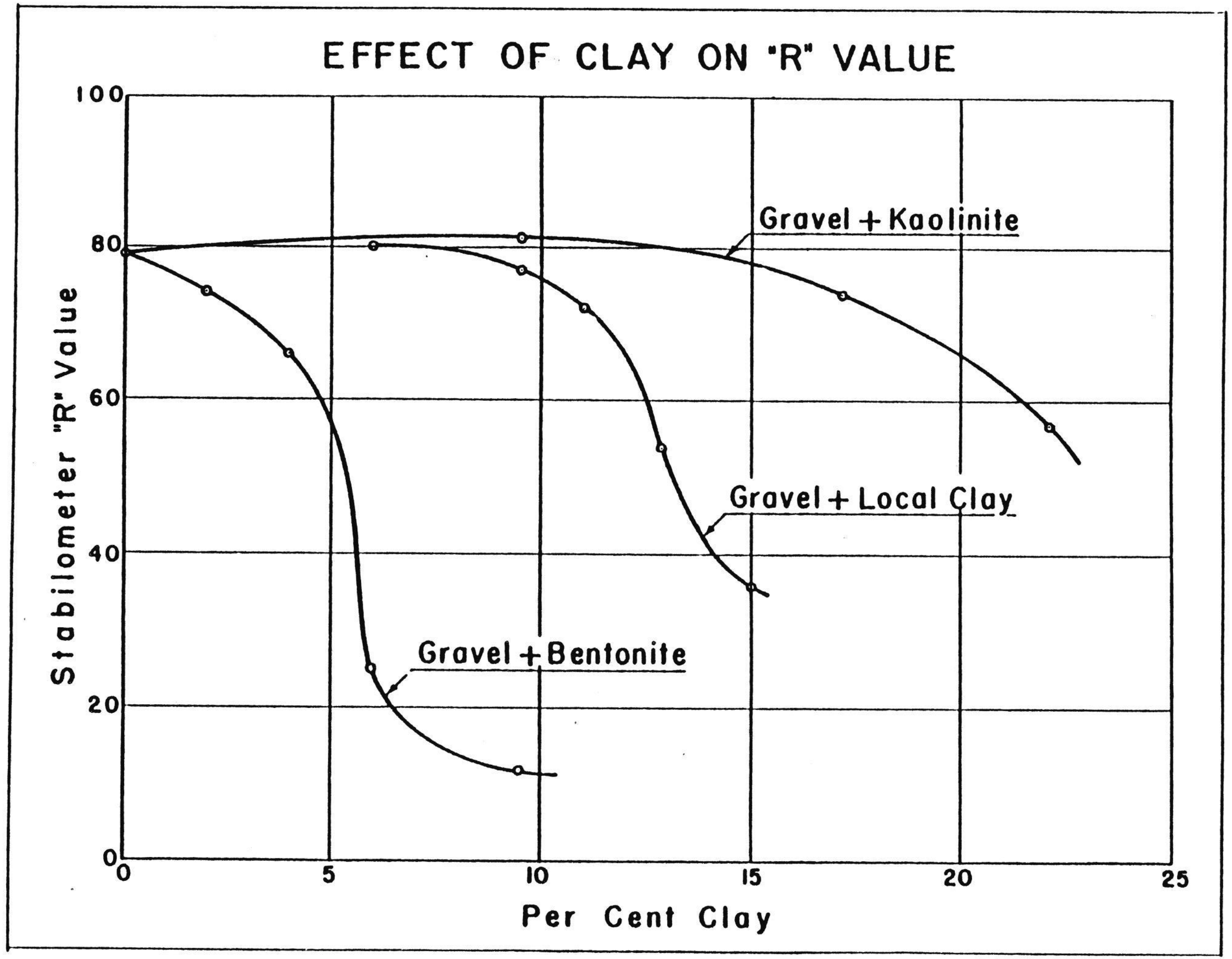


Figure 4.2 Effect of Clay on R-value (from D. R. Howe, 1961, p. 16)

The plots of K-value versus CBR at 0.1 and 0.2 in. penetration (Figures 4.3 and 4.4 respectively), indicate no correlation. The correlation in these Figures is so poor, it is unlikely that the use of remolded sample F-1 would change the results. CBR was determined in accordance with ASTM standard D 1883. No correlation was found between K-value and Resilient Modulus. Resilient Modulus (M_R) is usually presented in the form of $M_R = K_1 \theta^{K_2}$. Figures 4.5, 4.6 and 4.7 show K-value versus K_1, K_2, and M_R at θ = 15 psi, respectively. Fifteen (15) psi was chosen because it is common to all of the samples and M_R at this θ shows the best relation with K-value. In the plot of K-value versus K_1 (Figure 4.5), a concentration of results exists at the lower left hand part of the plot. The K-value versus K_2 plot (Figure 4.6) and K-value versus M_R at θ = 15 psi (Figure 4.7) show similar distribution of results.

The plot of K-value versus R-value at an exudation pressure of 300 psi (Figure 4.8) has no correlation. A plot of K-value versus R-value at an exudation pressure of 240 psi (not shown here) has a similar result distribution. Some R-values at 240 psi exudation pressure were obtained by extrapolation.

Based on observation and straightline regression, no relation between K-value and CBR, M_R, or R-value was found. Sub-relationship between K-value and laboratory test results such as soil classification, liquid limit, plasticity index, percent passing No. 200 sieve, percent passing No. 4 sieve or density of the sample were also investigated. An example of a sub-relation check is a K-value plot versus CBR plot which has contour lines for classification (not shown here). No relationship was found.

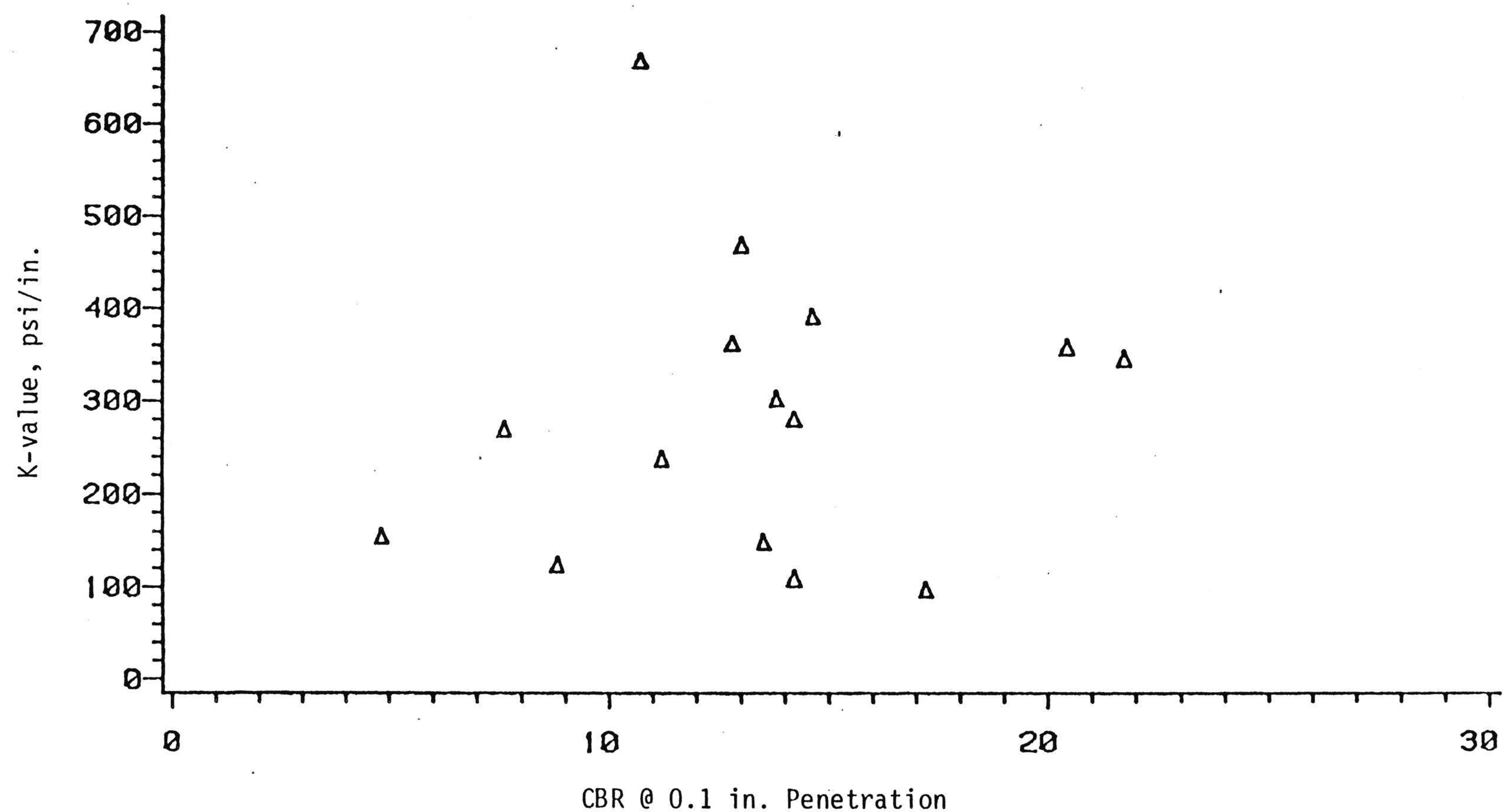


Figure 4.3 K-value, psi/in. vs. CBR @ 0.1 in. Penetration

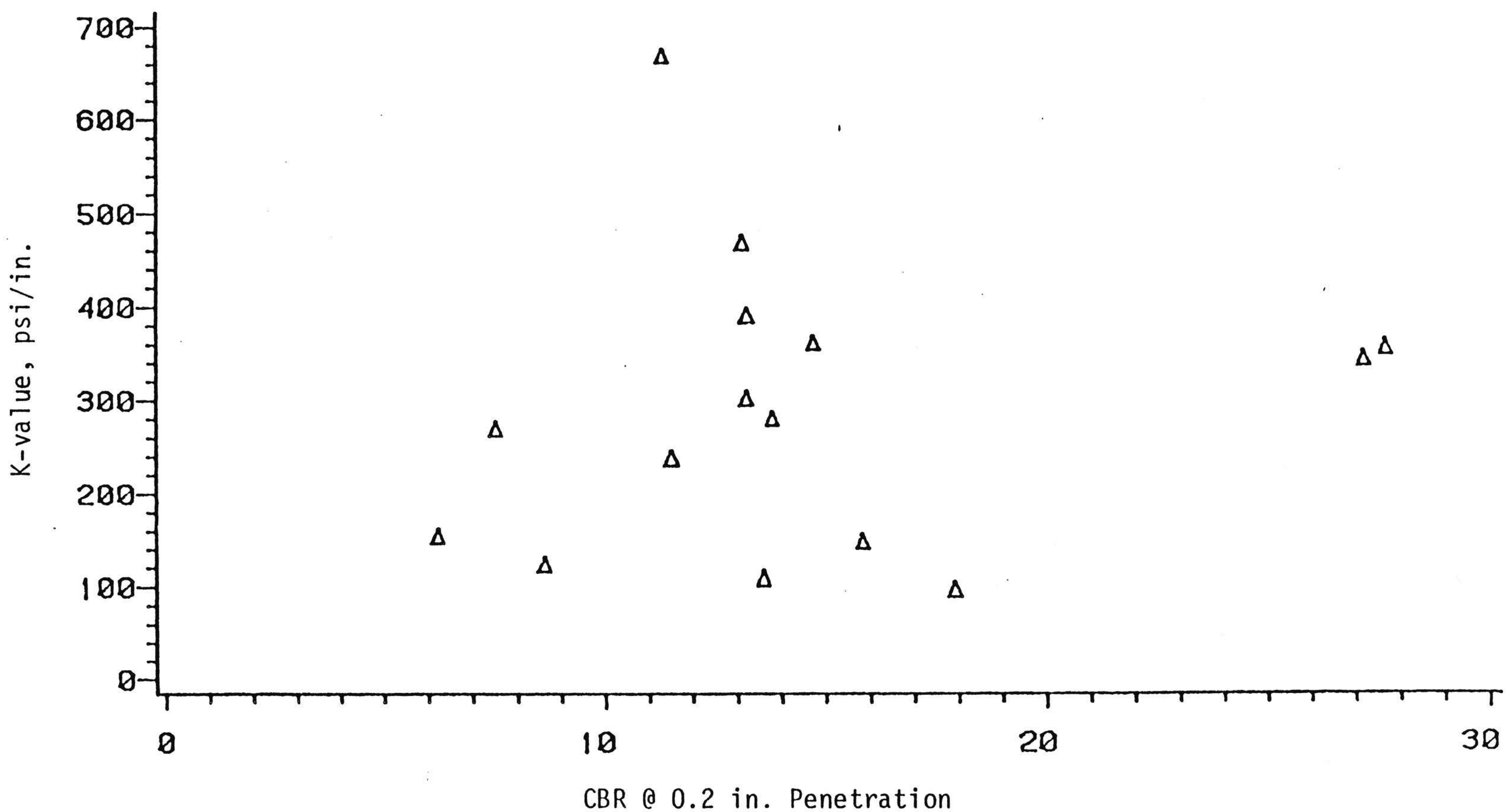


Figure 4.4 K-value, psi/in. vs. CBR
@ 0.2 in. Penetration

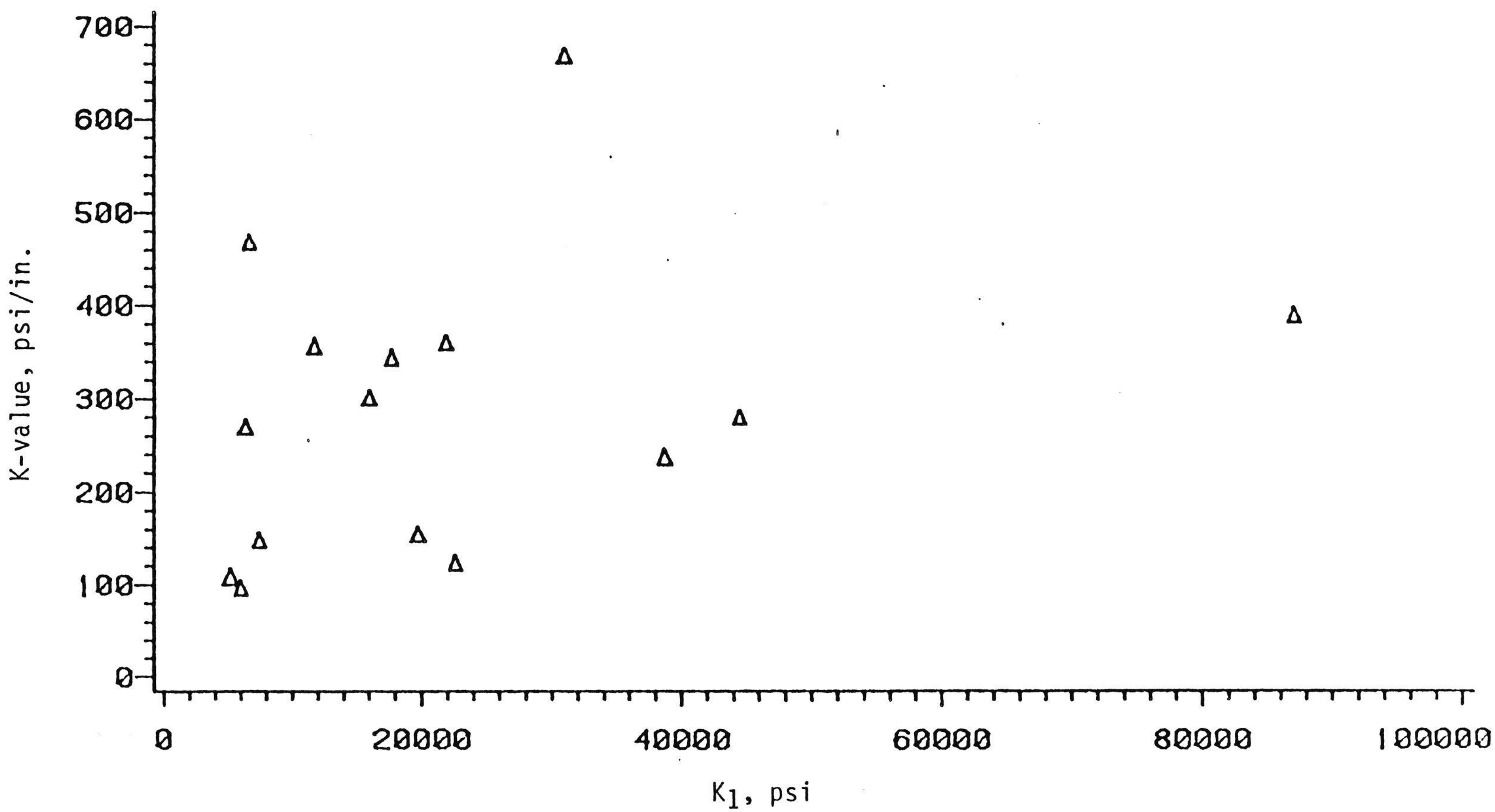


Figure 4.5 K-value, psi/in. vs. K_1 value, psi

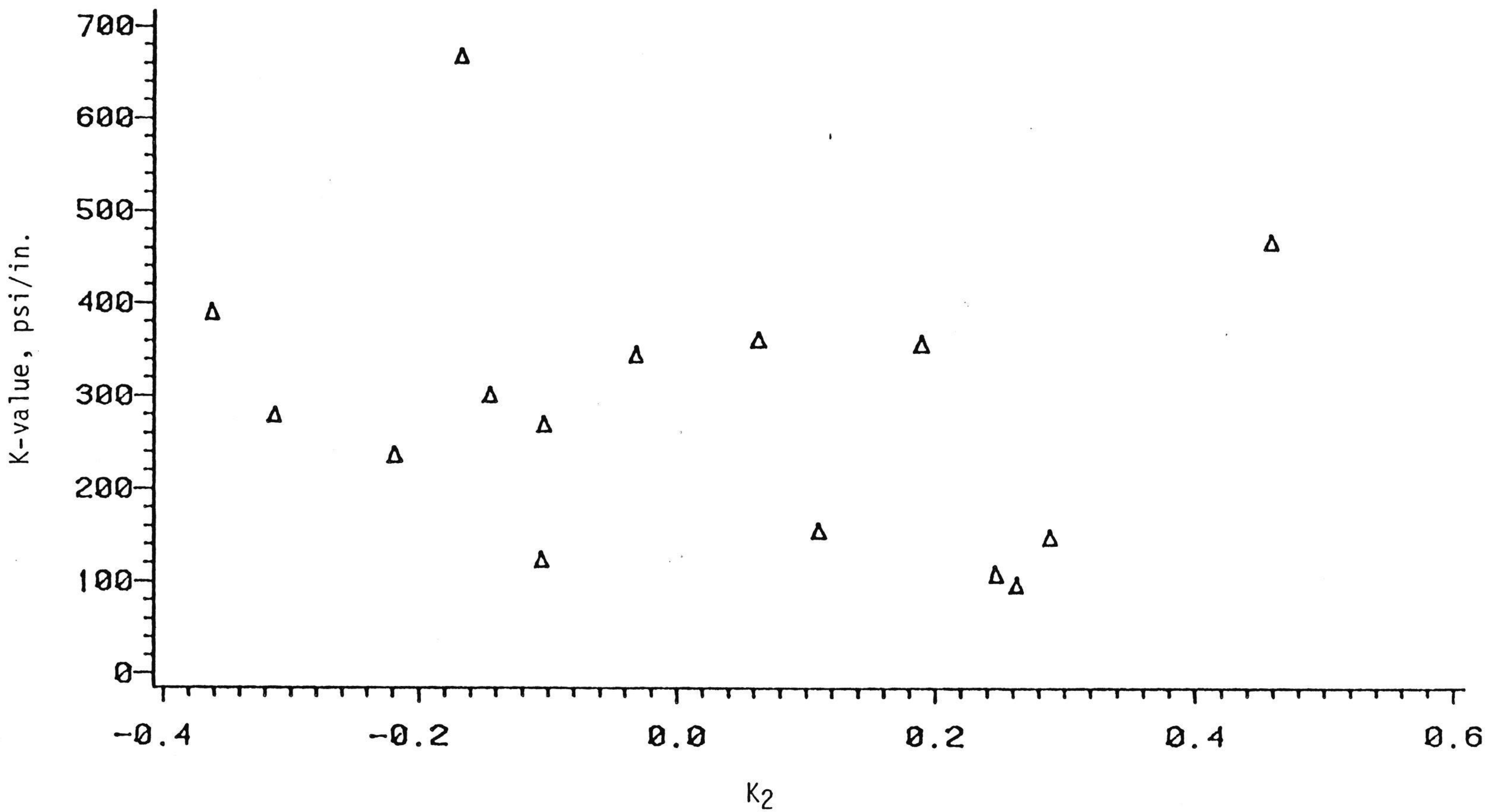


Figure 4.6 K-value, psi/in. vs. K_2 value

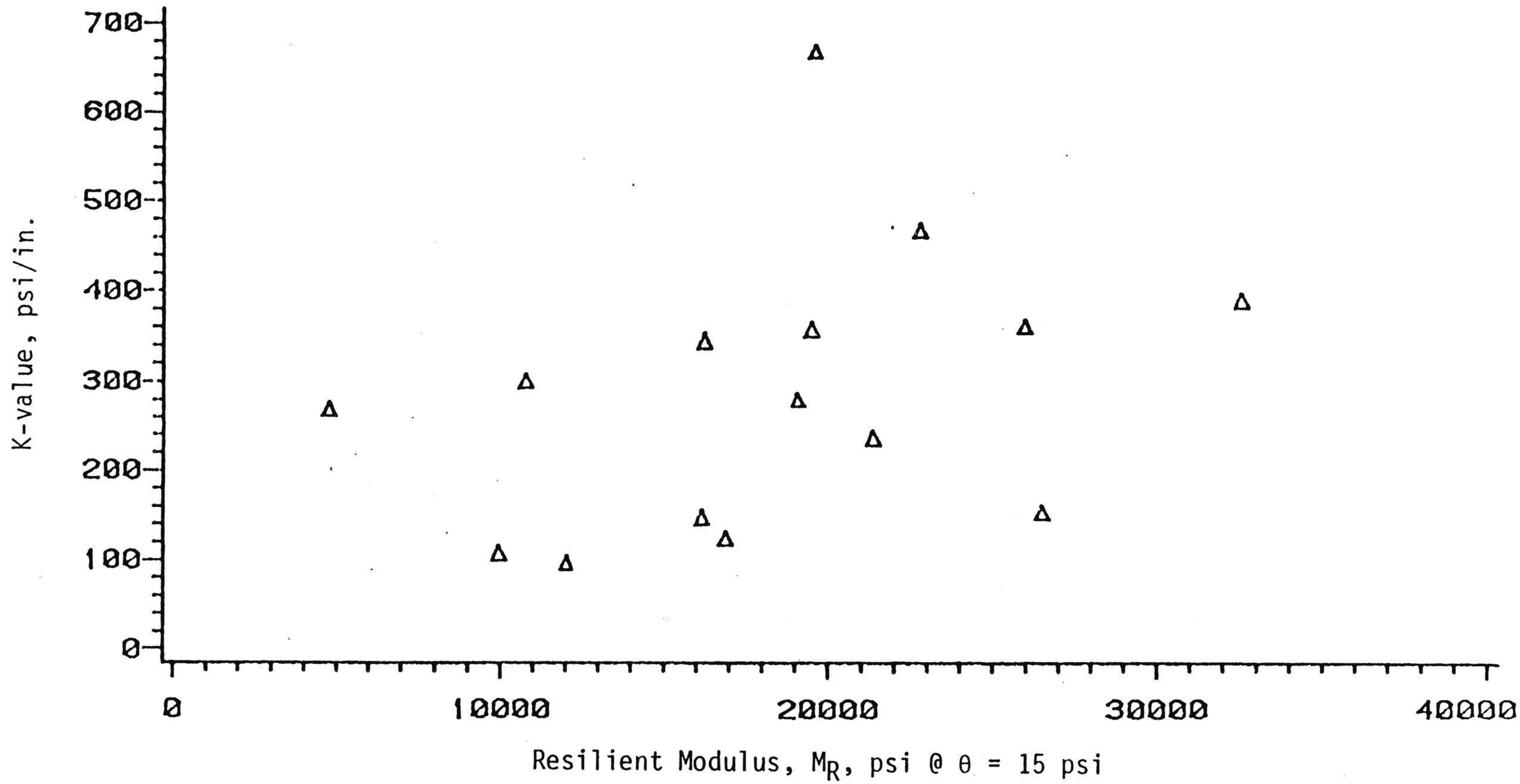


Figure 4.7 K-value, psi/in. vs. Resilient Modulus, psi @ θ = 15 psi

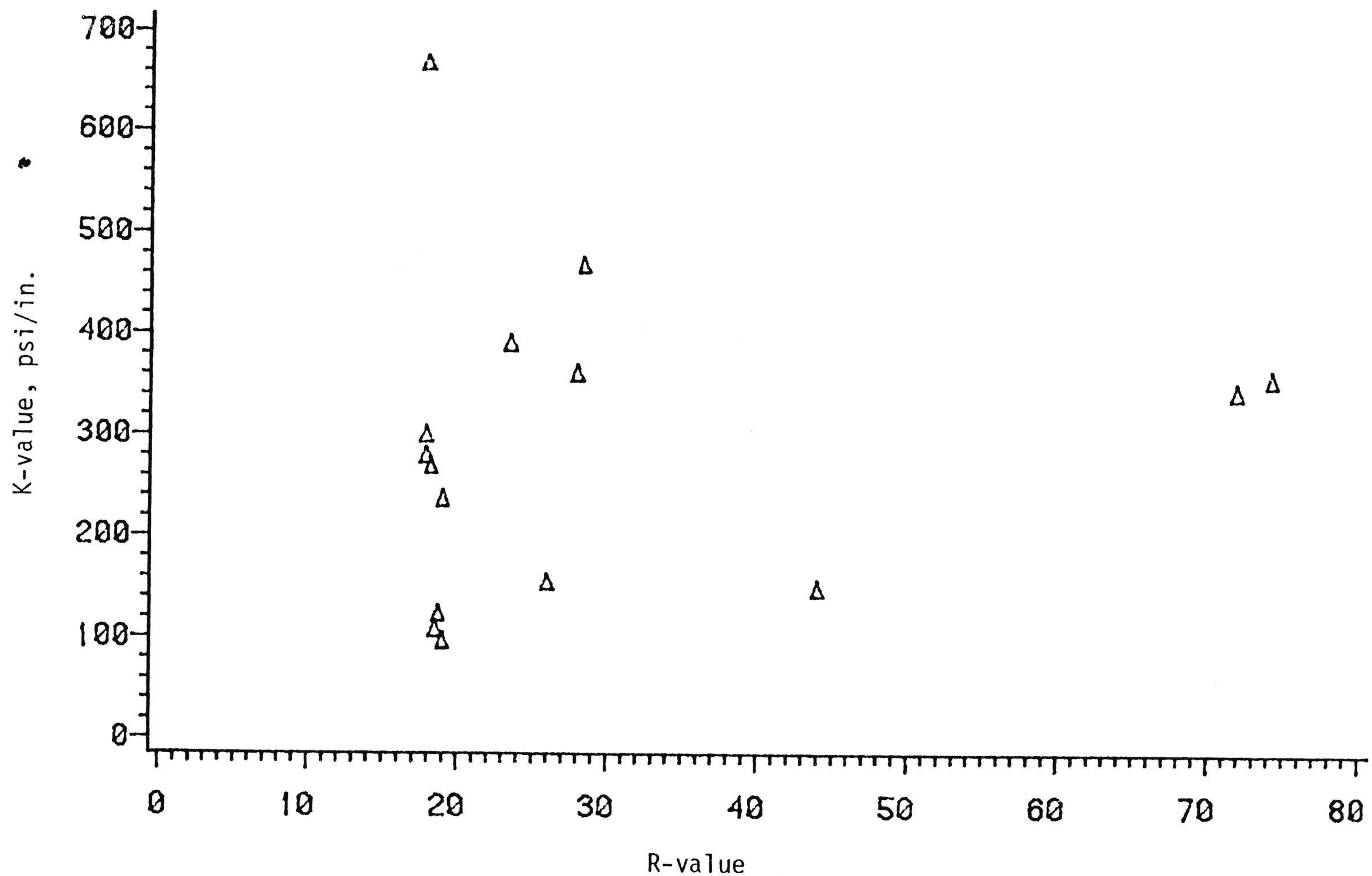


Figure 4.8 K-value, psi/in. vs. R-value

STATISTICAL ANALYSIS

The "Statistical Analysis System" (SAS), a computer package available to computer users at the University of Arkansas, was used to perform the statistical analysis. SAS is a computer system for data analysis developed by SAS Institute. SAS provides: information storage and retrieval, data modification and programming, report writing, statistical analysis, and file handling for the users (SAS User's Guide, 1979, p. 3).

The investigation of any correlations between field and laboratory tests included correlation coefficients. Correlation coefficient, r, is a measure of association between two variables (Cooper, B.E., 1969, P. 206). A correlation usually exists when the squared correlation coefficient (r^2), is 0.7 or greater.

Spearman and Kendall Tau B correlation coefficients were also used to investigate any correlation between K-value and laboratory test results. Spearman's coefficient of rank correlation measures the degree of correspondence between ranking, instead of between actual variate values, but it can still be considered a measure of association between the samples and an estimate of the association between X and Y in the continuous bivariate population (Gibbons, 1971, p. 226). Kendall Tau B is a measure of association between random variables from any bivariate population (Gibbons, 1971, p. 209).

The CORR procedure which SAS provided computes correlation coefficients between variables, including Spearman and Kendall Tau B correlation coefficients and the significance probablity of the correlation (SAS User's Guide, 1979, p. 173). Significance probability provides an intuitive indication of the strength of the evidence against the hypothesis (H) since it is the probability (under H) of getting a value of the test statistics as extreme as or more than the observed value (Lehmann, and D'Abrera, 1975, p. 11).

Standard Tests

Spearman and Kendall Tau B coefficient of correlations for K-value versus CBR at 0.1 and 0.2 in. penetration, M_R at θ = 15 psi, and R-value are presented in Table 4.3. The significance probability of the correlation coefficients for K-value and CBR at 0.1 and 0.2 in. penetration and R-value are high (0.8099, 0.9697 and 0.6418, respectively) and correlation coefficients are low (0.0679, 0.0107 and 0.1309, respectively). Therefore, no significant correlation can be obtained between Plate Bearing Test and CBR, or Hveem stabilometer (R-value) in this study.

The significance probability of Spearman and Kendall Tau B correlation coefficients for K-value versus M_R at θ = 15 psi are low, which means there is a good possibility of a correlation between the two tests. In the case of Spearman, the correlation coefficient (r) is 0.53929. Therefore the determination coefficient (r^2), which defines the percent variability, is 0.29083. A determination coefficient of only 29.083 percent of variability, means this correlation can not be used as a predictive model.

The relationship among the laboratory test results was investigated. The best relationship found, was between CBR and R-values for the samples classified as A-2-4 (AASHTO). CBR at 0.2 in. penetration versus R-value gives a correlation coefficient of 0.8333 (Spearman) with a significance probability of 0.0102 (Figure 4.9). Care should be taken when relating CBR and R-value because the number of test samples are limited and there is no CBR over 28.

Table 4.4. Correlation Coefficients and Their Significance.

Correlation Coefficient (r) / Significance Probability of r	K-value vs. CBR @ Penetration of: 0.1 in.	CBR @ Penetration of: 0.2 in.	K_1	K_2	M_r @ θ =15 psi	R
SPEARMAN	0.06792	0.01072	0.40357	-0.28214	0.53929	0.13094
	0.8099	0.9697	0.1358	-0.3083	0.0380	0.6418
KENDALL TAU B	0.01914	-0.01914	0.29524	-0.20000	0.39048	0.04855
	0.9211	0.9211	0.1250	0.2987	0.0425	0.8035

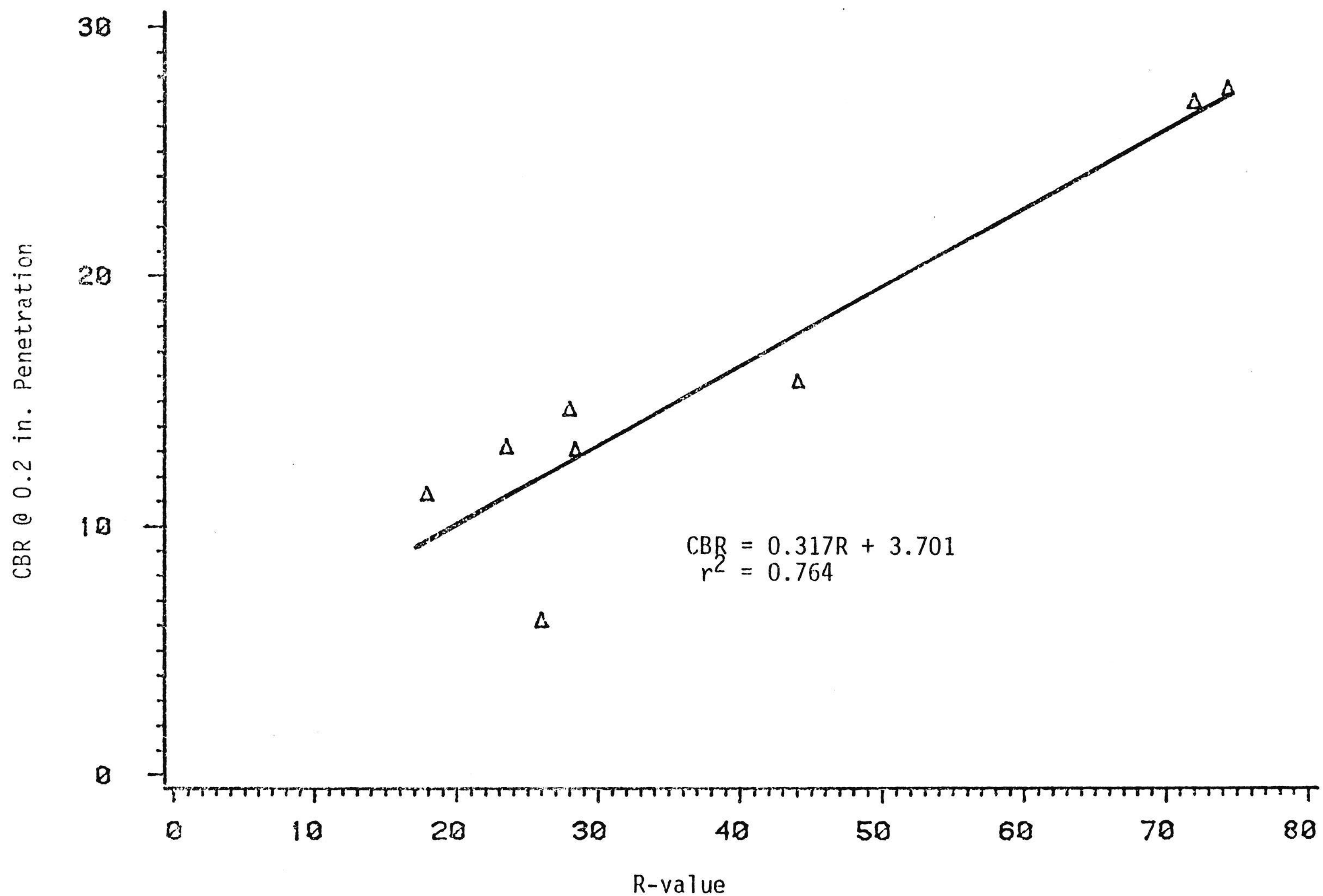


Figure 4.9 CBR @ 0.2 in. Penetration vs. R-Value for AASHTO A-2-4 Soils

Tests at Field Moisture

In this study, resilient modulus samples were formed at field moisture, while the CBR and R-values were formed by ASTM standard procedures. CBR is reported at optimum moisture content. R-value is reported at 300 psi exudation pressure.

In order to check for a correlation at field moisture, values of CBR obtained at various moistures for each sample were plotted versus their corresponding moisture contents (Figures A-2 in Appendix A). From these plots the CBR at field moisture (at 0.1 inch penetration) was obtained for each sample (Table 4.5).

A plot of K-value versus CBR at field moisture (Figure 4.10) indicates a better correlation than K-value versus standard CBR (Figure 4.3). Spearman and Kendall Tau B correlation coefficients were obtained for K-value versus CBR at field moisture (Table 4.6). The determination coefficient (r^2) for K-CBR correlation is less than 0.25. No predictive model can be based on such a low r^2.

However, K-value versus CBR at field moisture for A-4 and A-6 (AASHTO) soil samples (Figure 4.11) gives better correlation coefficients (Table 4.6). The linear regression between K and CBR for A-4 and A-6 soil samples at 0.1 inch penetration ($K = 13.8\ CBR + 80.6$) gives a determination coefficient (r^2) of 0.7785, which is significant. Because of the limited number of points (7), this correlation should be used carefully.

The relation between K-value and CBR at 0.1 inch penetration is reinforced when points at a common site are averaged (Figure 4.12). The determination coefficient is improved to 0.8382.

A correlation between K-value and CBR at field moisture also exists for 0.2 inch penetration (Figure 4.13). The determination coefficient

Table 4.5 CBR and R-value at Field Moisture

Sample	CBR	R-value
A-1	13.8	29.8
A-2	13.0	29.6
D-1	0.5	8.7
E-1	1.6	NA*
E-2	4.6	23.6
F-1	11.5	24.1
J-1	35.5	75.0
J-2	30.0	46.4
LR-1	21.3	31.3
N-2	23.0	68.0
R-1	15.5	17.4
R-2	9.5	15.3
SS-1	18.0	NA*
STC-1	3.0	13.2
STC-2	6.5	7.7

*scatter of data is too large

Table 4.6 Correlation Coefficients for Tests at Field Moisture

Correlation Coefficient (r) / Significance Probability of r	K-value vs. CBR		
	All samples	A-4 & A-6 Samples	R-value
SPEARMAN	0.49643 0.0598	0.78571 0.0362	0.74725 0.0033
KENDALL TAU B	0.39048 0.0425	0.61905 0.0509	0.53846 0.0104

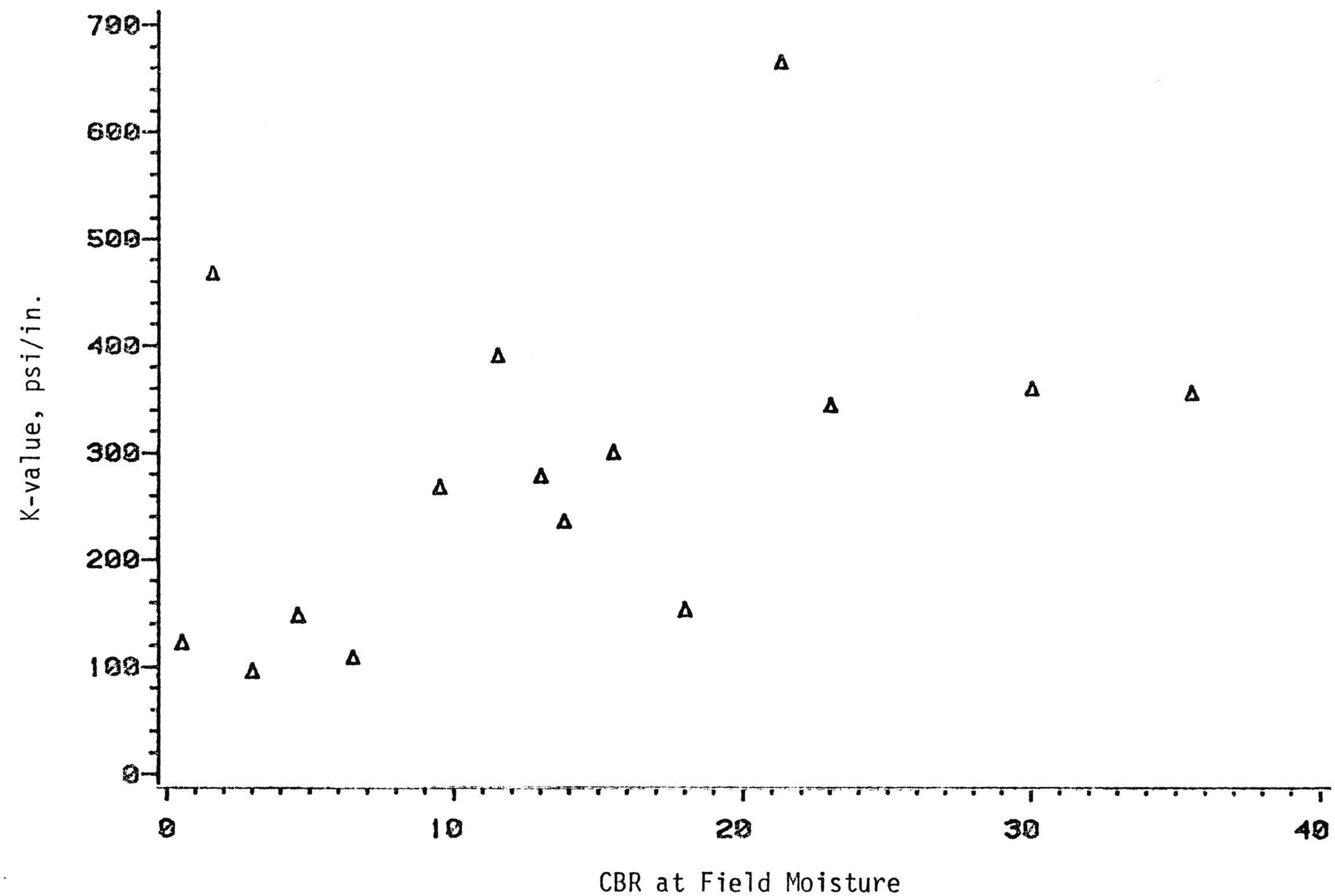


Figure 4.10 K-value, psi/in. vs. CBR @ 0.1 in. Penetration

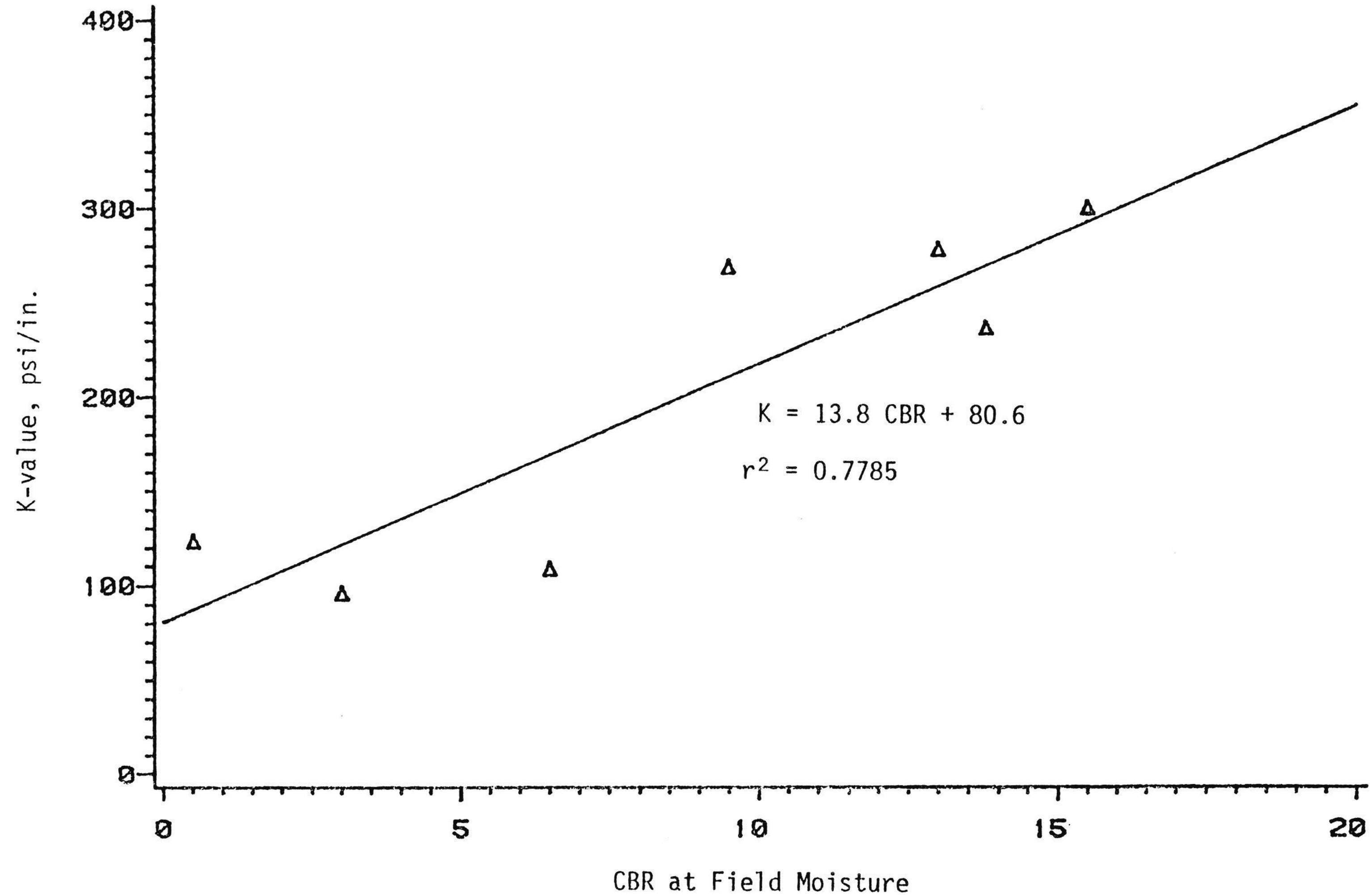


Figure 4.11 K-value, psi/in. vs. CBR @ 0.1 in. Penetration for A-4 and A-6 (AASHTO) Soil Samples

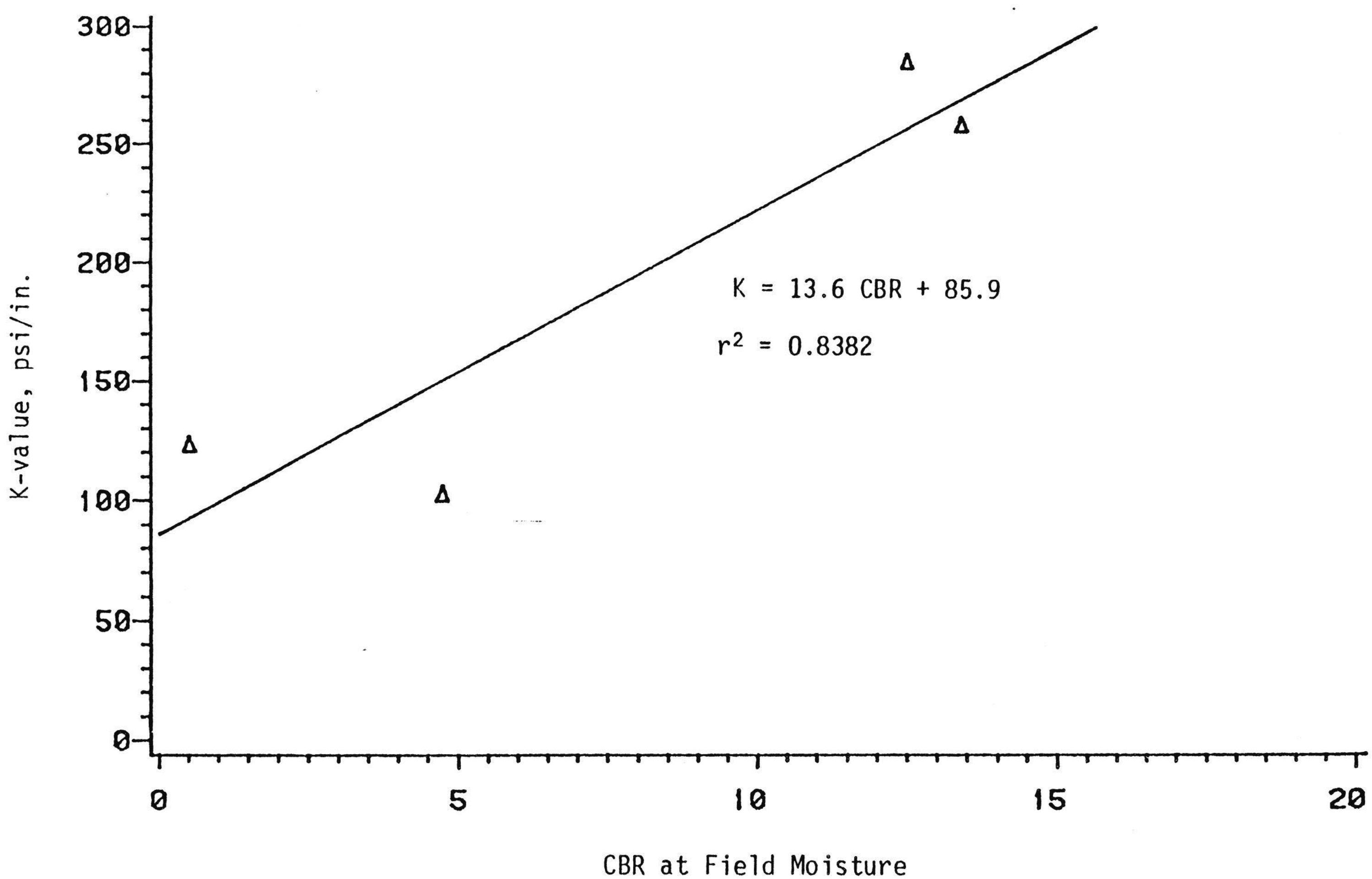


Figure 4.12 K-value, psi/in. vs. CBR at 0.1 in. Penetration for A-4 and A-6 (AASHTO) Sites

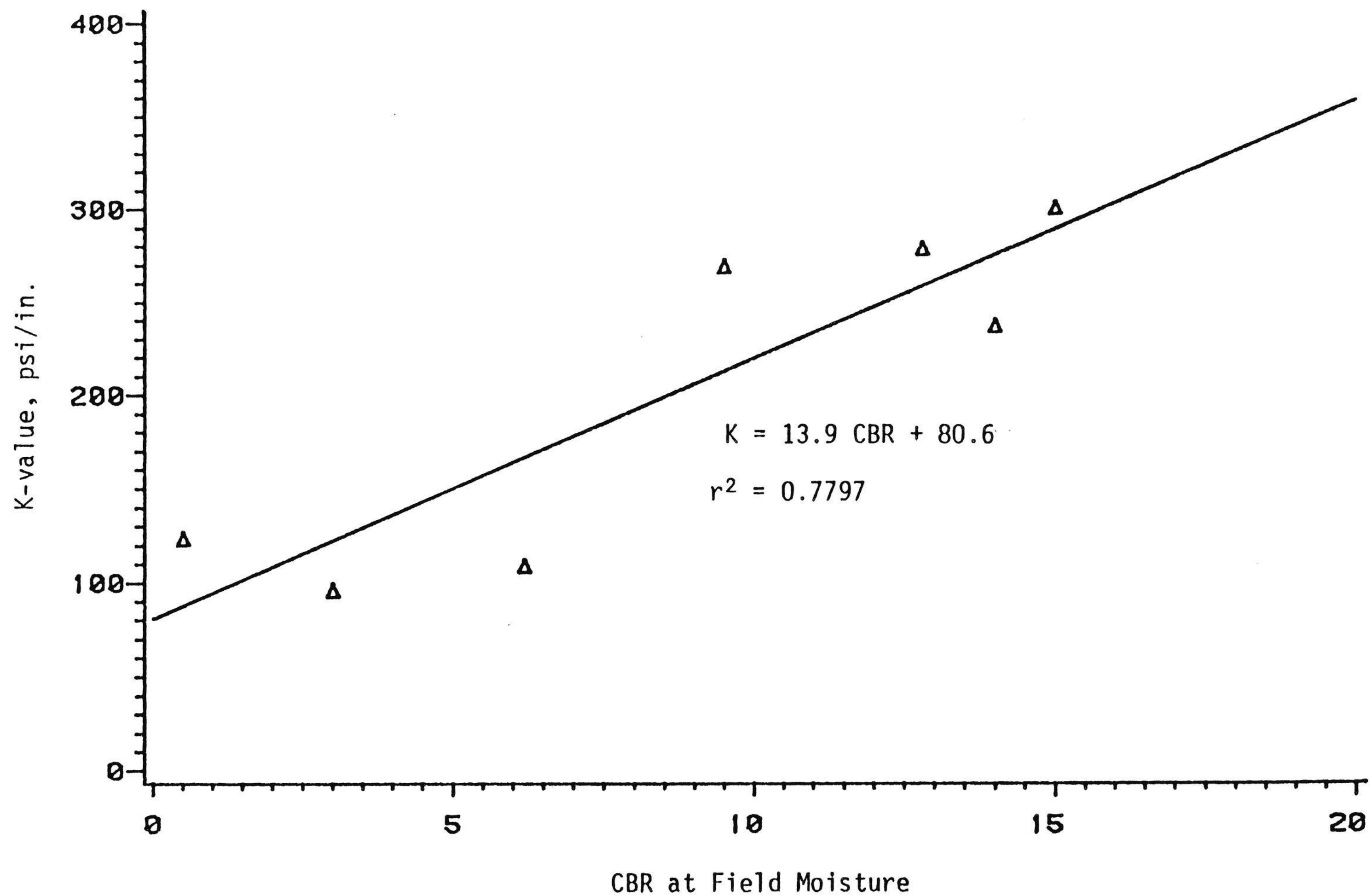


Figure 4.13 K-value, psi/in. vs. CBR @ 0.2 in. Penetration for A-4 and A-6 (AASHTO) Soil Samples

(r^2) of 0.7797 is the same as for 0.1 inch penetration. The plot for points averaged at a site (Figure 4.14) had a determination coefficient of 0.8380 which is the same as for 0.1 inch penetration.

One possible reason for a correlation at field moisture for A-4 and A-6 classified soils only, is that the A-2 soils with larger than 3/4 inch material removed, do not have the same engineering properties as the soils in the field.

R-values for each sample were plotted versus their corresponding moisture contents. R-values at field moisture for each sample were obtained from these plots (Table 4.5). Plots of R-value versus moisture for the E-1 and SS-1 samples were scattered. Therefore, the R-values at field moisture are reported only for thirteen (13) samples (Table 4.5).

The plot of K-value versus R at field moisture (Figure 4.15) shows better correlation than K versus standard R (Figure 4.8). Spearman and Kendall Tau B correlation coefficients for K-R relation are high (Table 4.6). Coefficient of determination, r^2, (Spearman) for K-R relation is 0.5584, which is not highly significant. For a linear regression, r^2 is less than 0.25.

<u>Variance of the Field Test</u>

To investigate the dependibility of the Plate Bearing Test results, Cochran's Test was used. Cochran's test is the testing of the homogeneity of variances (Beyer, W.H., 1968, p. 325).

Based on the experiment a completely random nested model was designed to be tested. This model is:

K-value = MEAN + AASHTO CLASSIFICATION + LOCATION (AASHTO)* + ERROR

*Location is nested in the classification

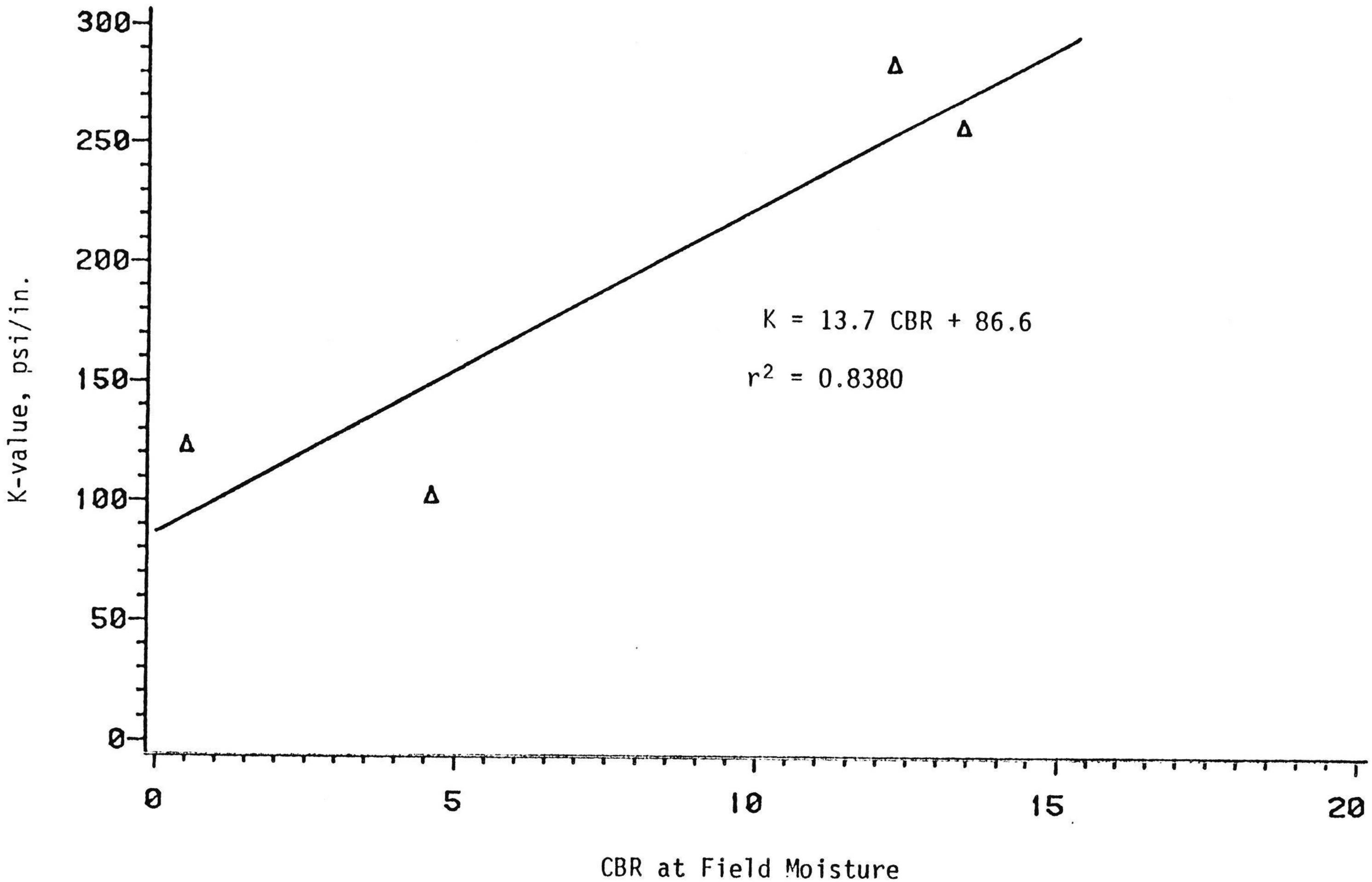


Figure 4.14 K-value, psi/in. vs. CBR at 0.2 in. Penetration for A-4 and A-6 (AASHTO) Sites

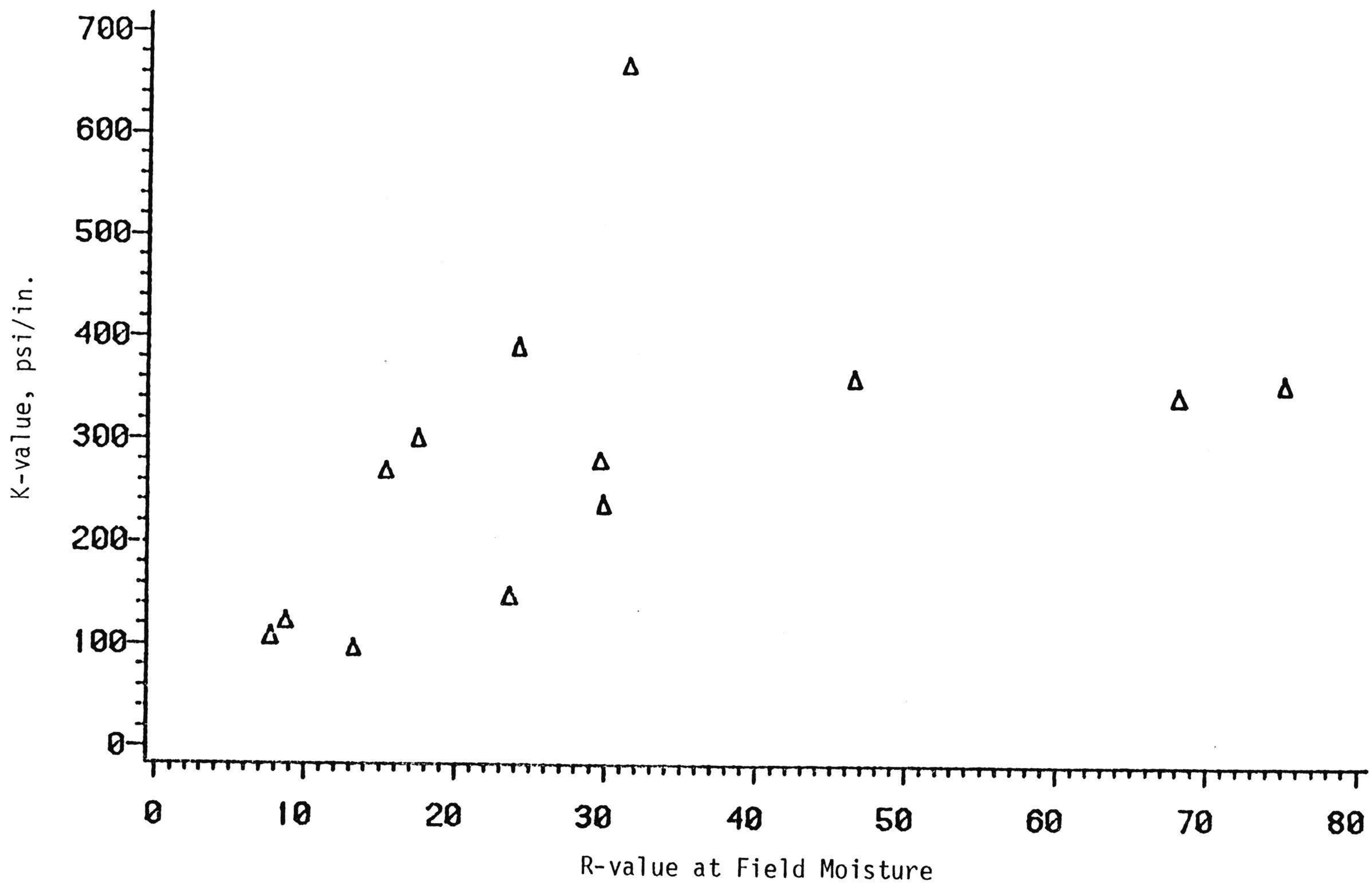


Figure 4.15 K-value, psi/in. vs. R-value at Field Moisture

where MEAN represents the average K-value over all populations; AASHTO CLASSIFICATION represents variability of K from one classification to another; LOCATION (AASHTO) represents variability of K between locations and ERROR represents variability of K within locations. Locations are nested in the classification, which means the levels of location are chosen within the levels of classification (Hicks, C.R., 1982, p. 228).

Variance components are estimated for all fifteen samples. The analysis of the results shows variability within site E (El Dorado) is much greater than within other paired sites. Site E (samples E-1 and E-2) is removed and the variance components are estimated for the remaining thirteen samples. Table 4.5 presents the estimated variances components.

The results from Table 4.5 shows a drastic drop from 40.4% to 1.4% in the variance of error due to removal of E-1 and E-2. The drop of error due to removal of E-1 and E-2 samples makes the K-values obtained for this site questionable. Since the difference in field moisture content (Table 4.1) for the E-1 (19.9) and E-2 (15.4) samples was greater than 4%, it is possible that the Plate Bearing Tests on El-Dorado site were conducted on differently compacted locations.

The results of the variance components for the thirteen samples also show a greater variance among K-values from one site to another (84.2%) than one classification to another (14.4%). The variances within a site is small (1.4%).

After removal of E-1 and E-2 another correlation study was conducted which did not show any better results than the first study.

Table 4.7 Estimate of Variance Components

Dependent Variable: K-value

Variance Component	Variance Components Estimate			
	E-1 & E-2 Included	Percent of Total	E-1 & E-2 Excluded	Percent of Total
σ^2 AASHTO	4256.9	16.3	3730.5	14.4
σ^2 Location (AASHTO)	11306.5	43.3	21889.4	84.2
σ^2 Error	10537.3	40.4	371.6	1.4
Total	26100.7	100.0	25991.5	100.0

Confidence Limits and Design

The designer of a rigid pavement needs the "confidence limits" of the K-value vs. CBR at field moisture relationship in order to use CBR in a conservative design. Figure 4.16 gives the relationship for 0.1 inch penetration at 90% confidence. The figure has three curves. The upper line is the upper 90% confidence line; i.e. 5% or one out of twenty CBR values will fall above the line and 95% or nineteen out of twenty will fall below. The upper confidence line is, therefore, unconservative and should never be used in design. The middle line is the mean or correlation itself (K = 13.8 CBR + 80.6). The mean would overpredict K-value from CBR at field moisture 50% or half of the time. The mean is likely to give the most accurate prediction of K-value. The lower line is the lower 90% confidence line. The lower line would predict K-value safely 95% of the time and therefore would be a conservative design line. The lower 90% confidence line is given in Figure 4.17 to a larger scale for easier reading.

Figures 4.18 and 4.19 are 90% confidence limits and the lower 90% confidence line for CBR at field moisture for 0.2 inch penetration. The curves are similar to those for 0.1 inch penetration.

Using the lower 90% confidence line for 0.1 inch penetration, design values for use in nomigraphs can be chosen. Figures 4.20 and 4.21 are design charts based on values taken from the lower 90% confidence line at 0.1 inch penetration (Figure 4.17). These charts are adapted from the AASHTO Interim Guide for Design of Pavement Structures 1972.

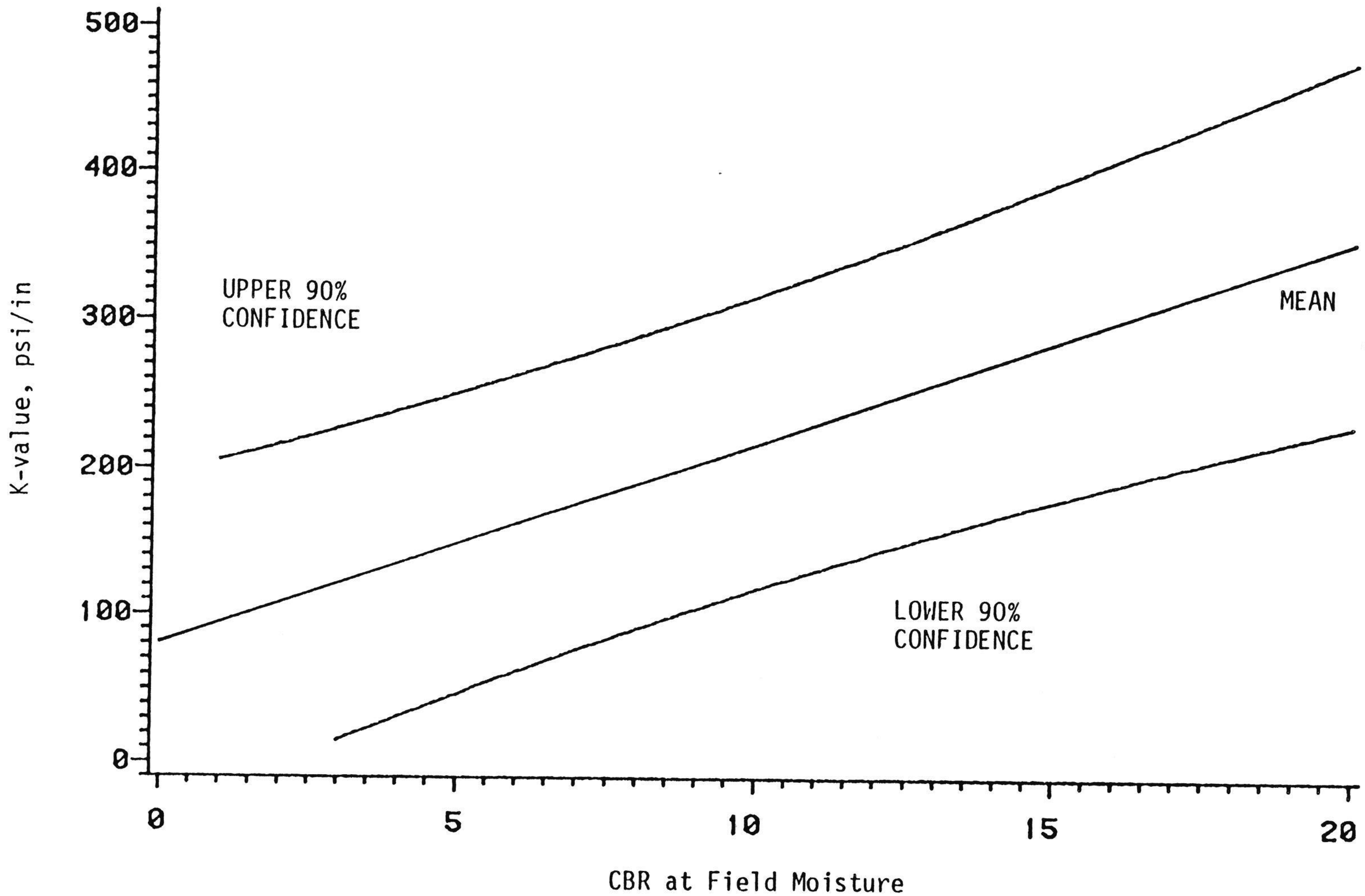


Figure 4.16 Confidence Limits for A-4 and A-6
Samples @ .1 in. Penetration

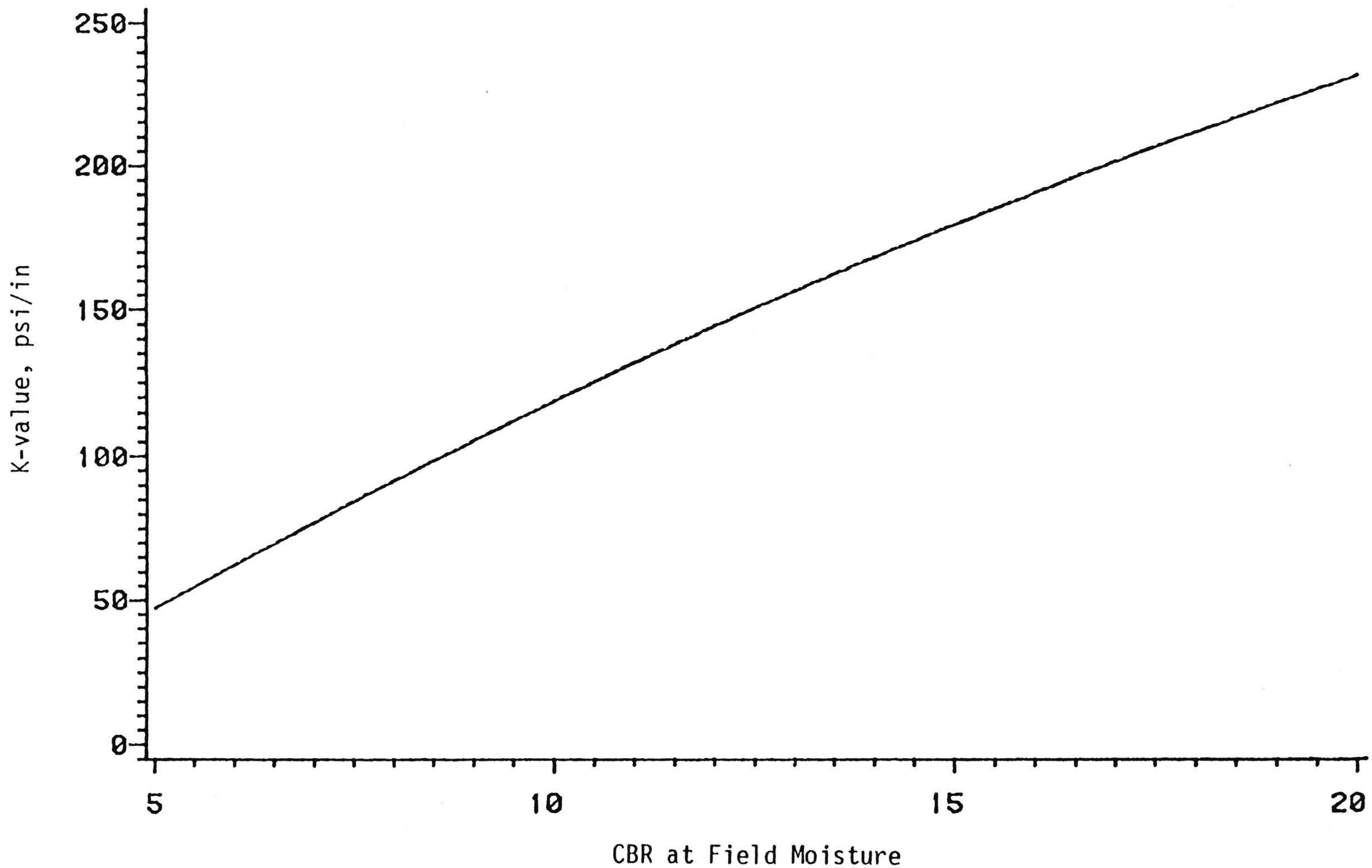


Figure 4.17 Lower 90% Confidence Line
for A-4 and A-6 Samples @ .1 in Penetration

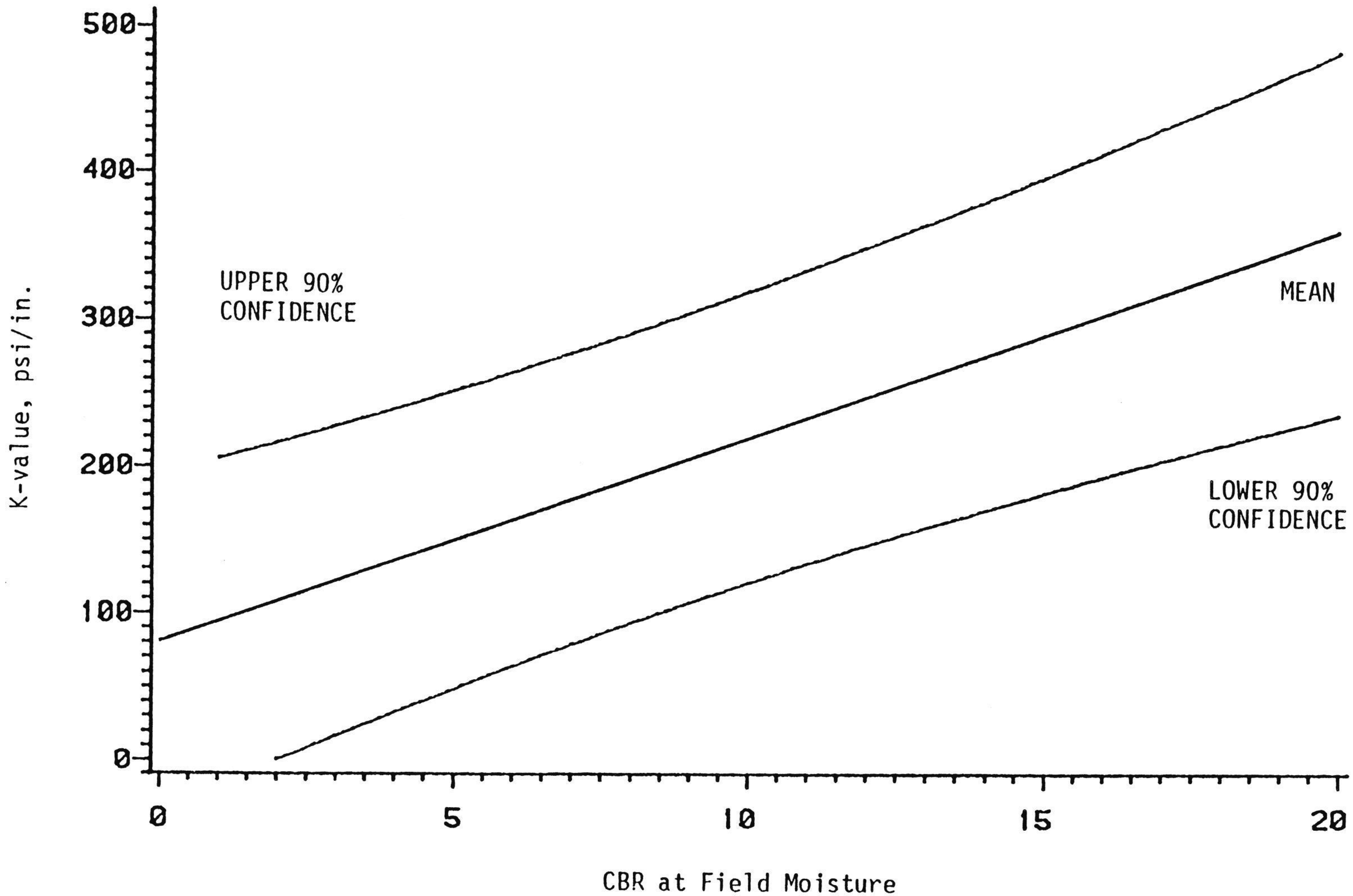


Figure 4.18 Confidence Limits for A-4 and A-6 Samples @ 0.2 in. Penetration

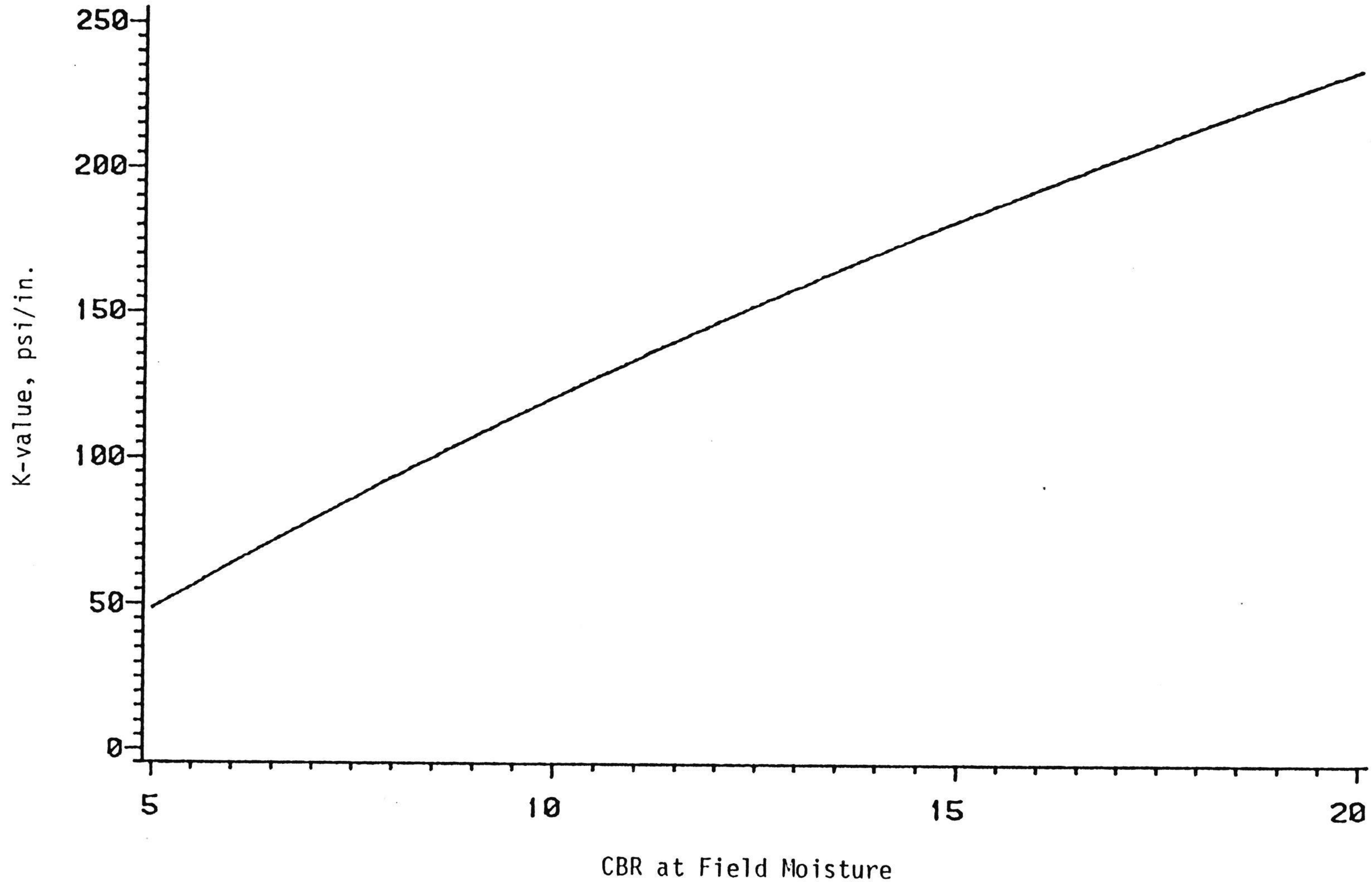
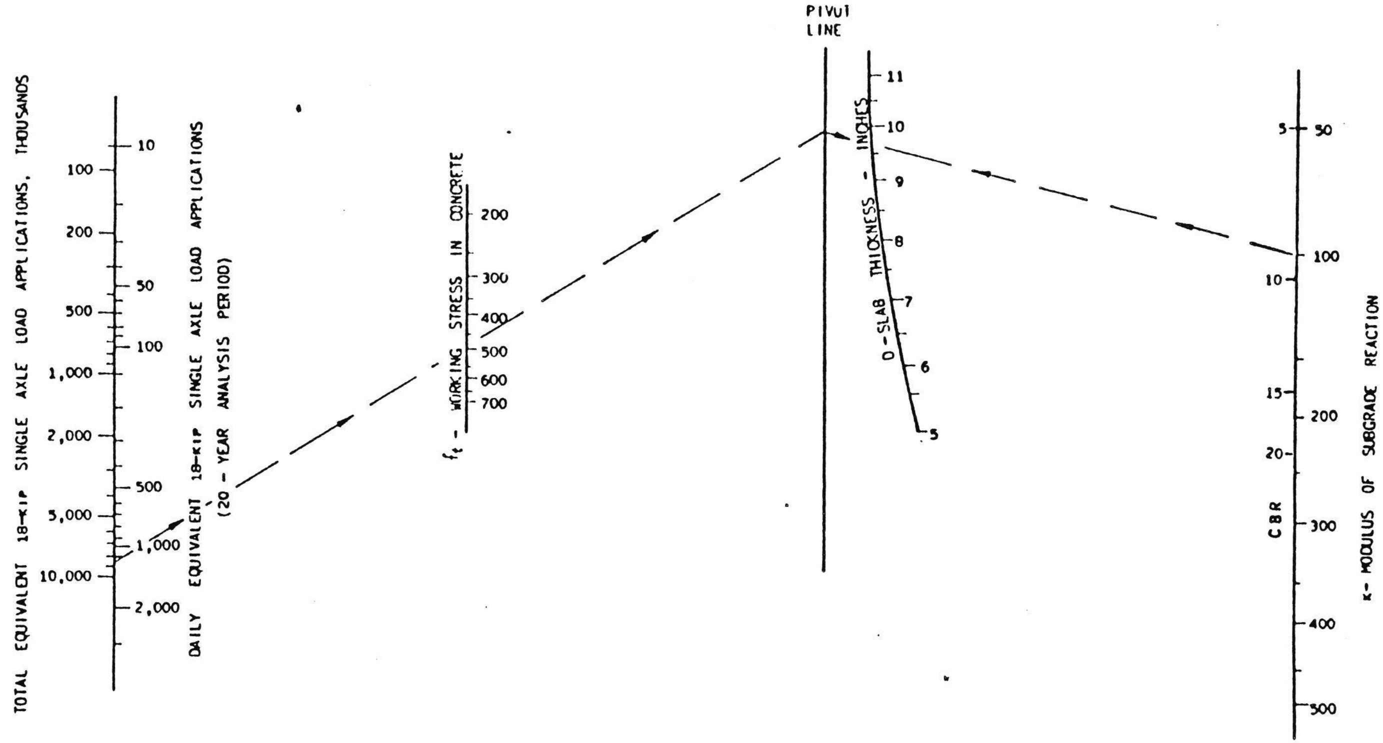
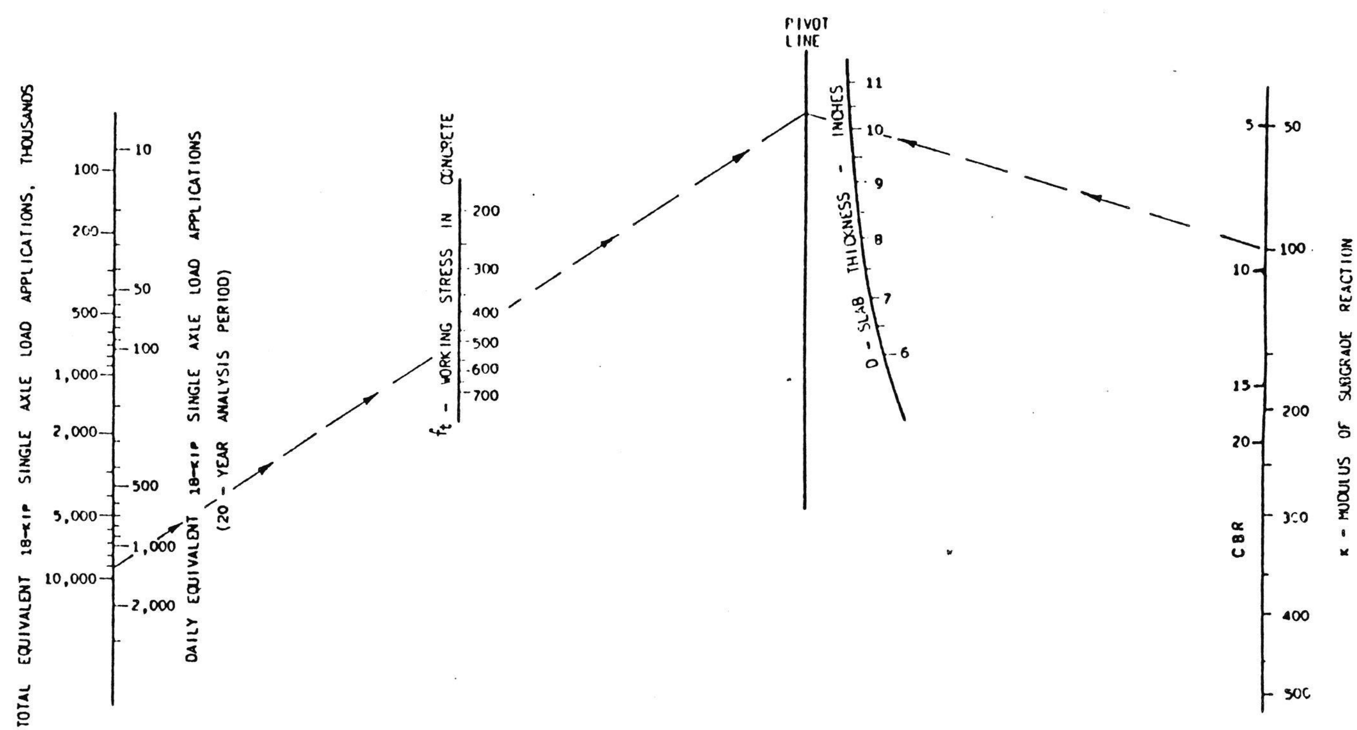


Figure 4.19 Lower 90% Confidence Line for A-4 and A-6 Samples @ 0.2 in. Penetration



Design Chart for Rigid Pavements, $P_t = 2.0$
CBR at Field Moisture for A-4 and A-6 Soils

Figure 4.20



Design Chart for Rigid Pavements, $P_t = 2.5$
CBR at Field Moisture for A-4 and A-6 Soils

Figure 4.21

Chapter 5

CONCLUSIONS

1) No distinct relation exists between Modulus of Subgrade Reaction (K-value), and CBR, Hveem Stabilometer (R-value), or Resilient Modulus (M_R) when tests are performed by standard or ASTM procedures.

2) A linear regression between K-value and CBR at field moisture (not optimum moisture) for A-4 and A-6 AASHTO classified soils has a determination coefficient (r^2) of 0.7785, which is significant.

$$K = 13.8\ CBR + 80.6$$

Because the correlation is based on seven points, it should be used with care. The lower 90% confidence line is suggested for use in designing.

3) A linear regression between R-value and standard CBR for A-2-4 soils is:

$$CBR = 0.317R + 3.701$$

The coefficient of determination for this relation is 0.764.

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APPENDIX A

Plate Bearing Test (K-value) results are included in Figure A-1a through A-1p. The K-value reported is the slope of the linear regression of load-deflection.

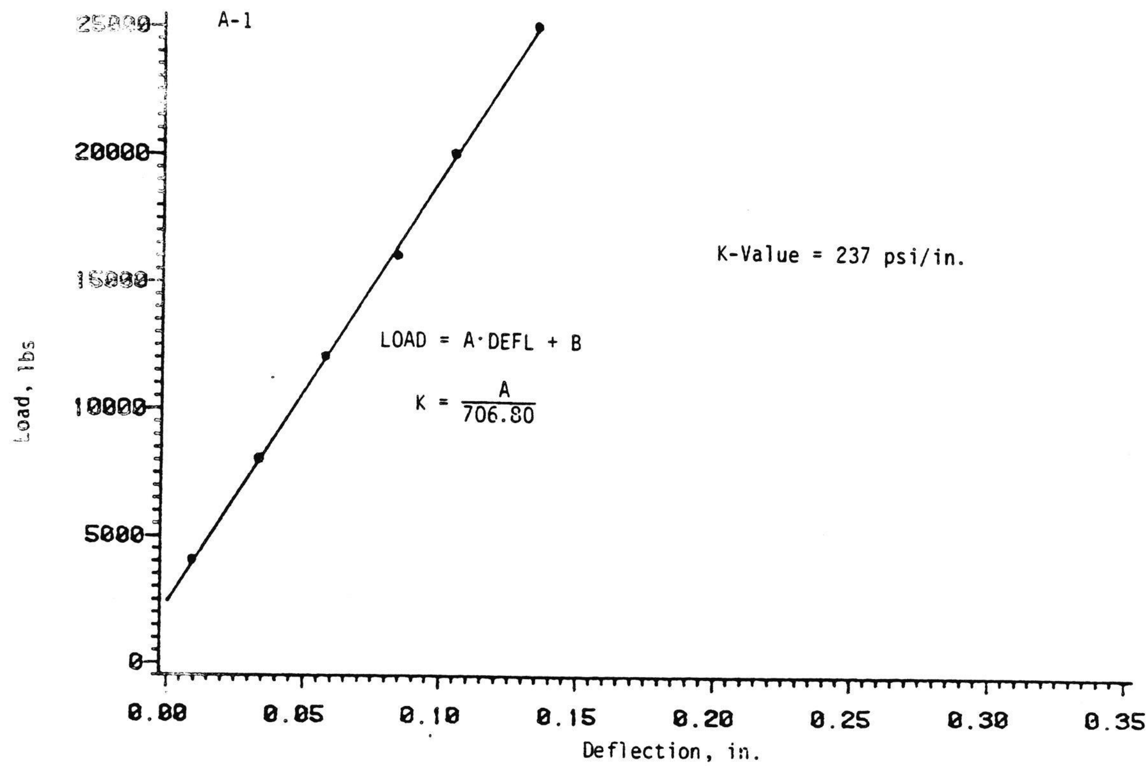
California Bearing Ratio (CBR) results are included in Figure A-2a through A-2o. Load-penetration curves for three specimens (below, at, and above optimum moisture content) are presented for each sample. CBR is the load in psi on the piston divided by the standard load (1000 psi at 0.1 in. penetration and 1500 psi at 0.2 in. penetration).

Resilient Modulus (M_R) results are included in Figure A-3a through A-3o. M_R is reported in the form of $M_R = K_1\ \theta^{K_2}$.

Hveem Stabilometer (R-value) results are included in Figure A-4a through A-4o. R-values at 240 and 300 psi exudation pressure are obtained from R-value-exudation pressure curves for each sample.

Sample no. is shown in upper left hand corner of each graph.

SUPPORTING GRAPHS



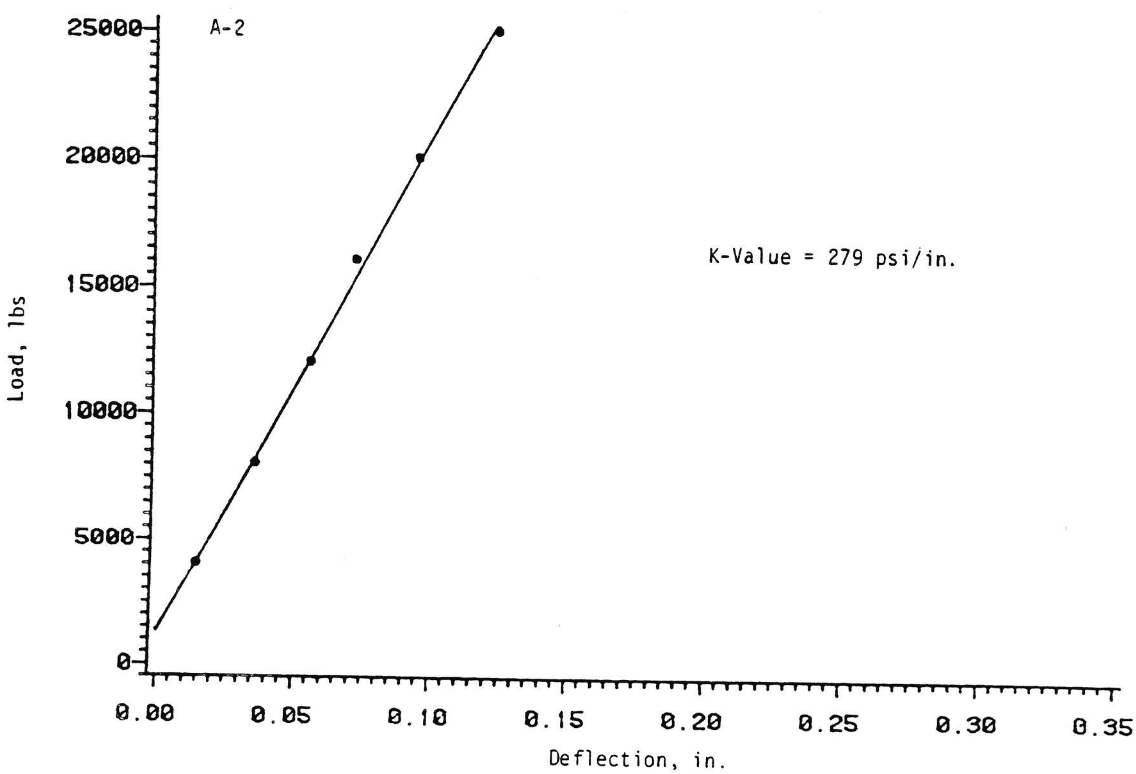


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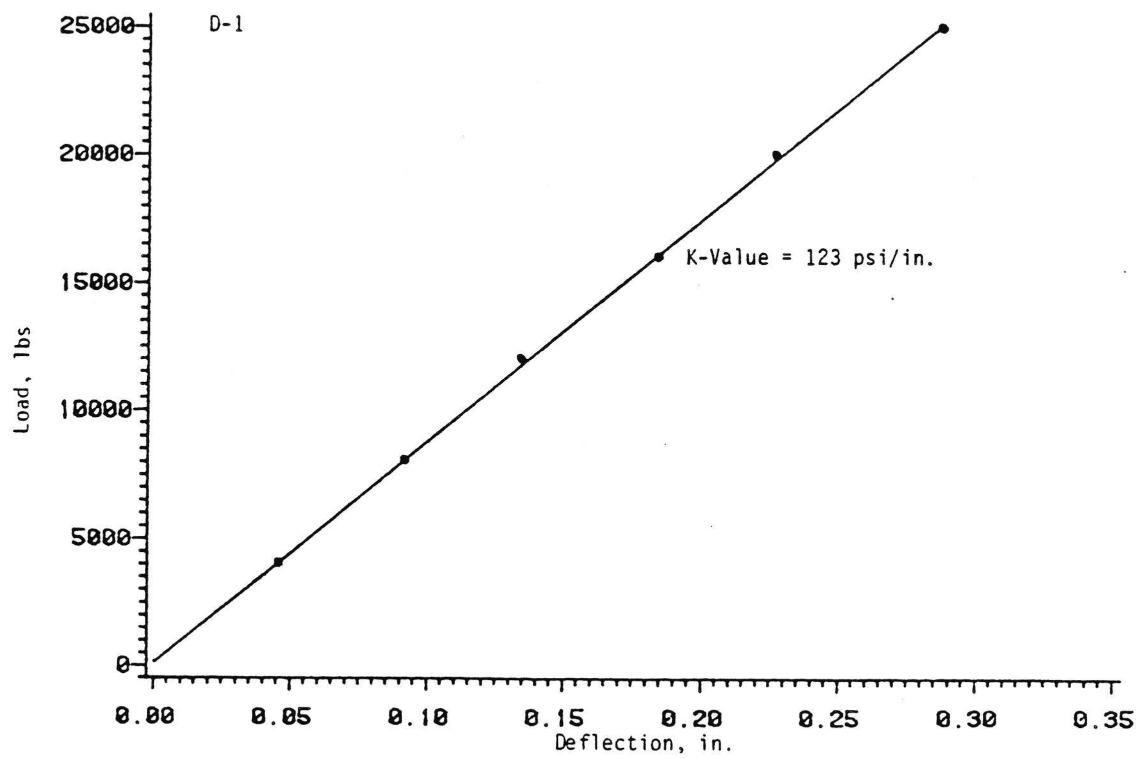
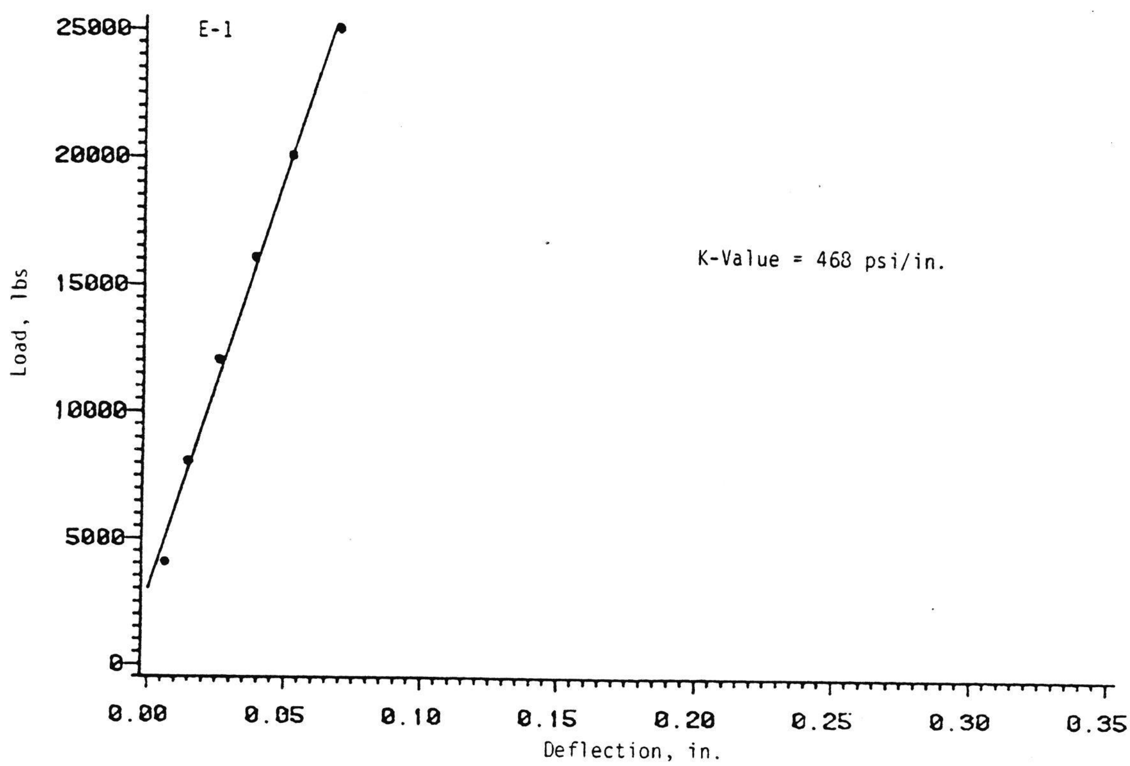
Plate Bearing Test Results





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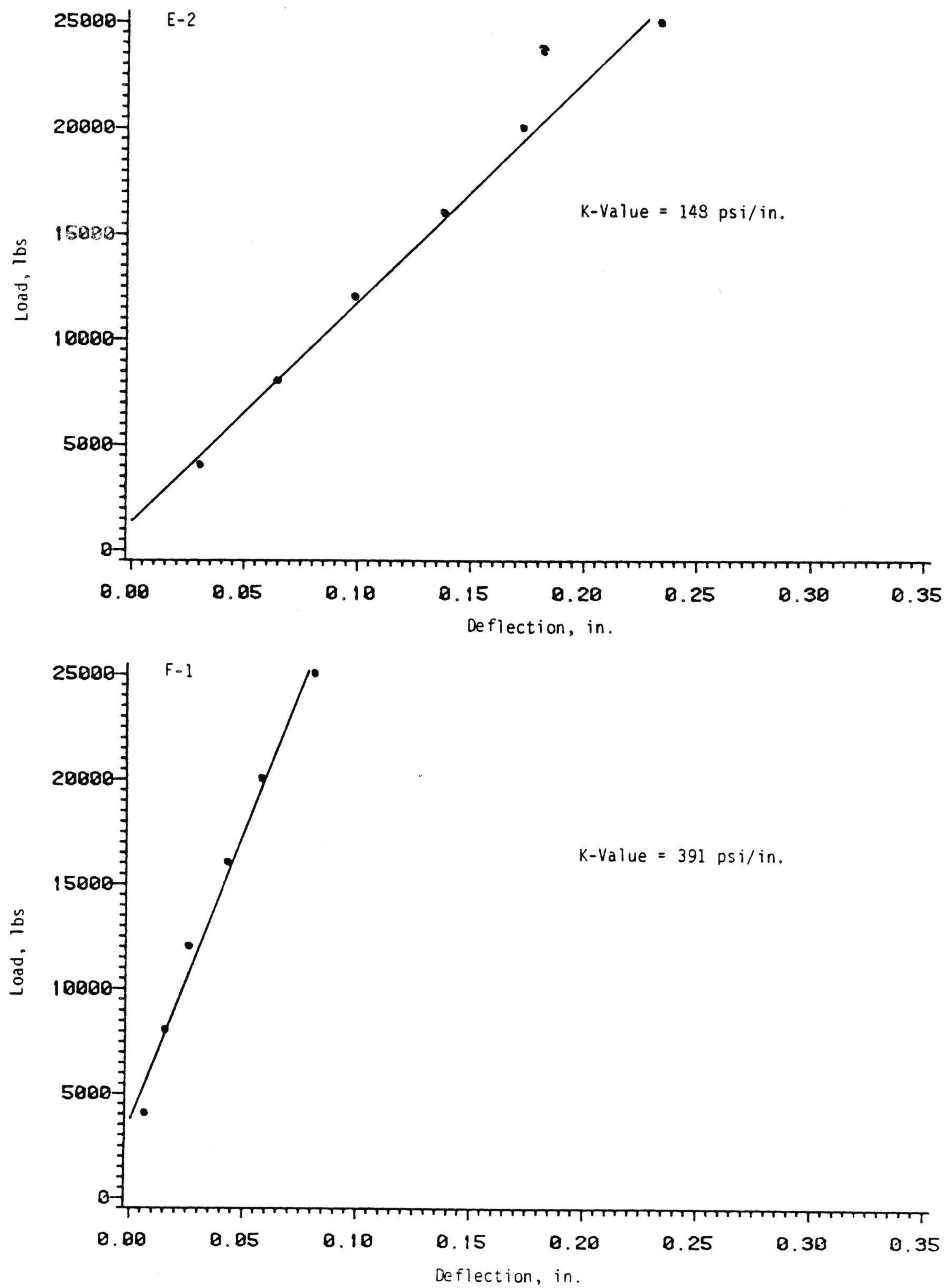

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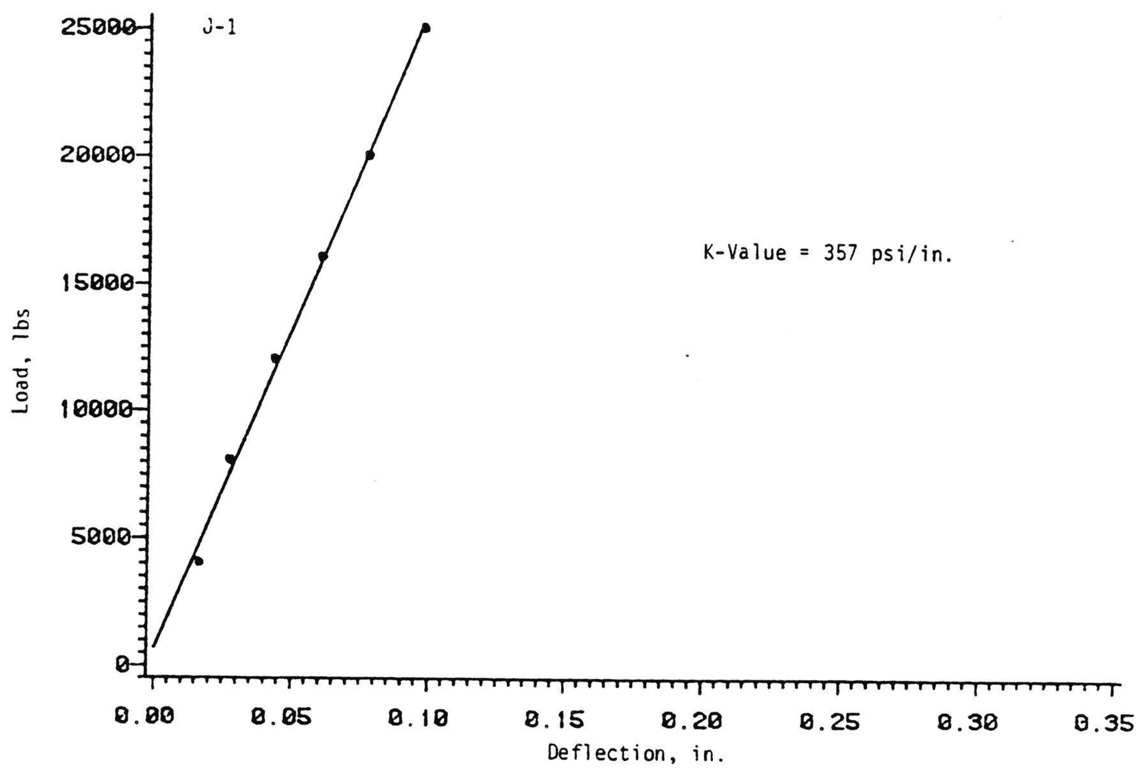

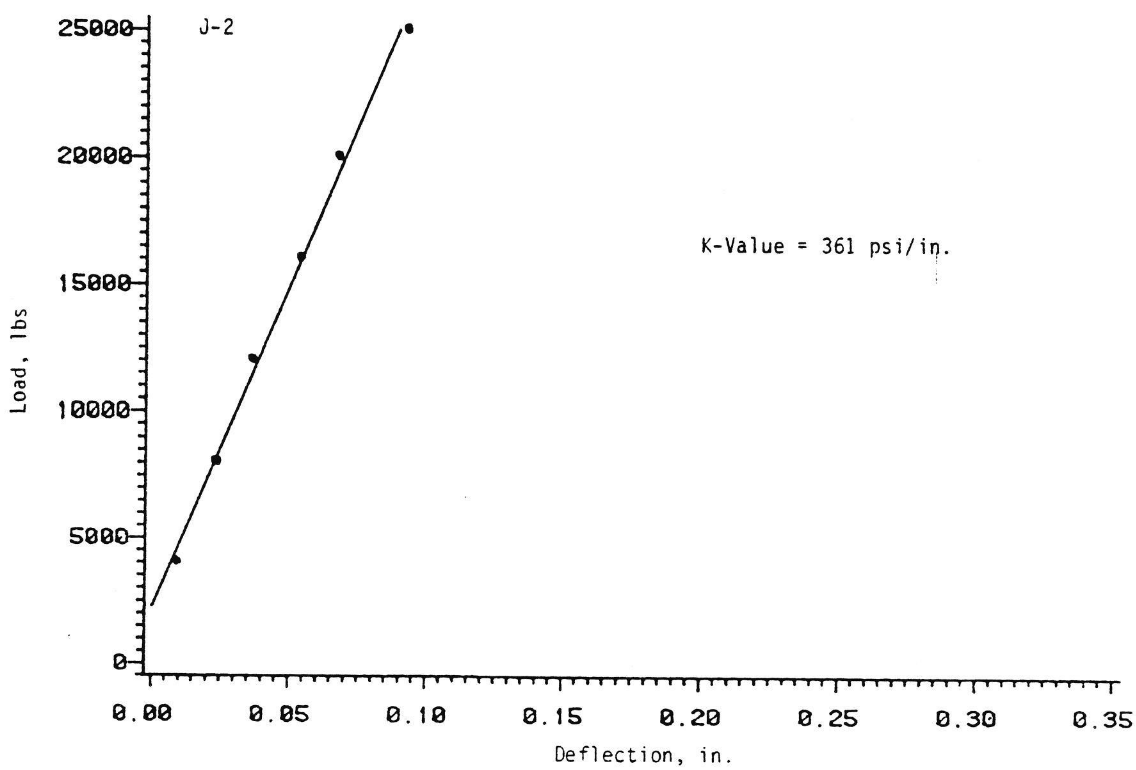

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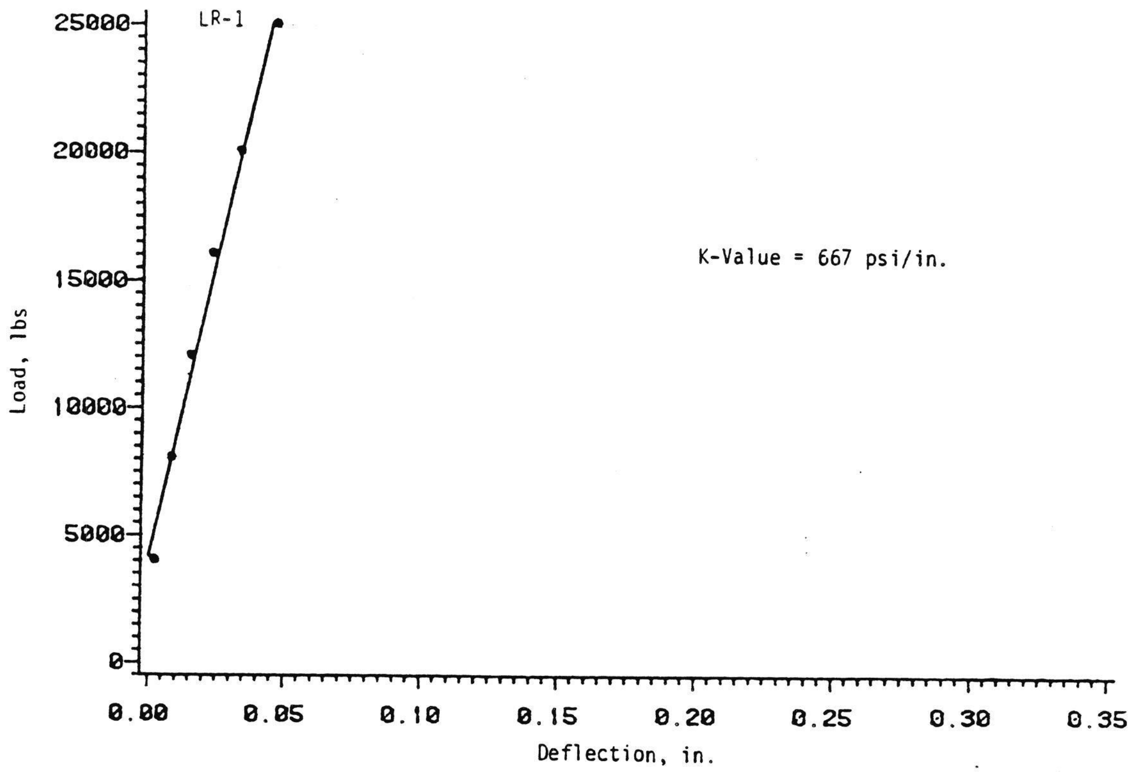

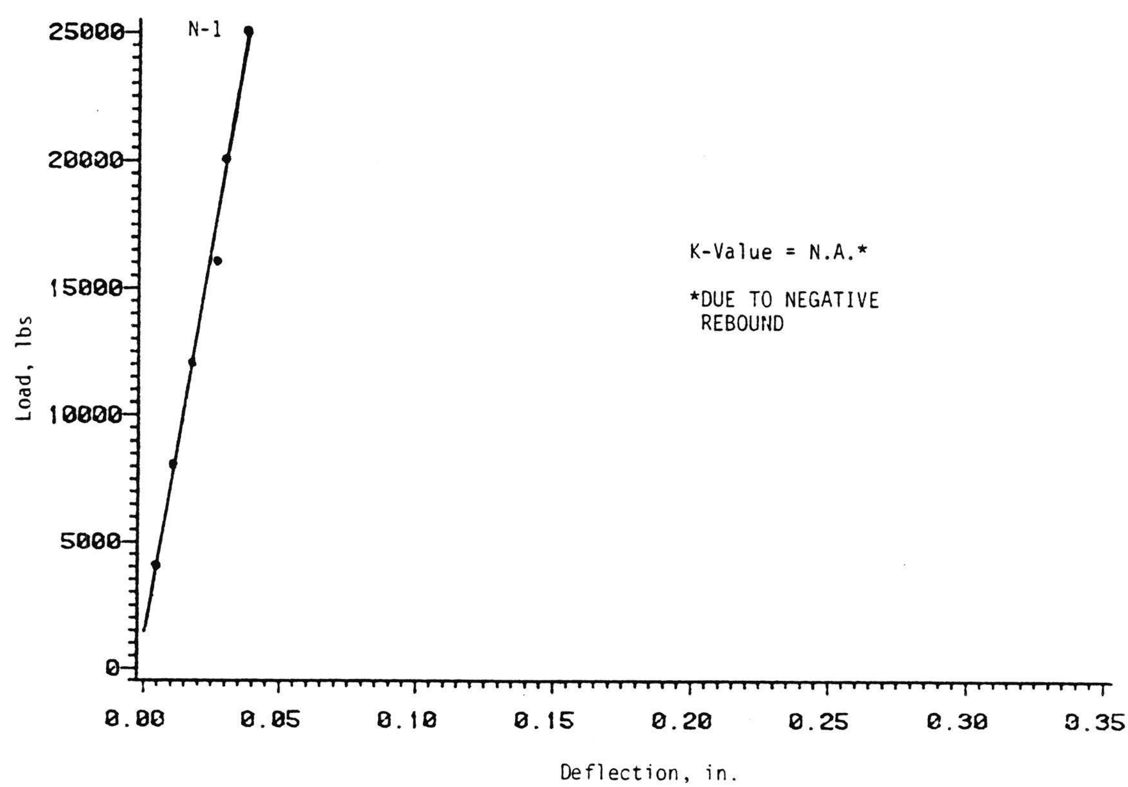

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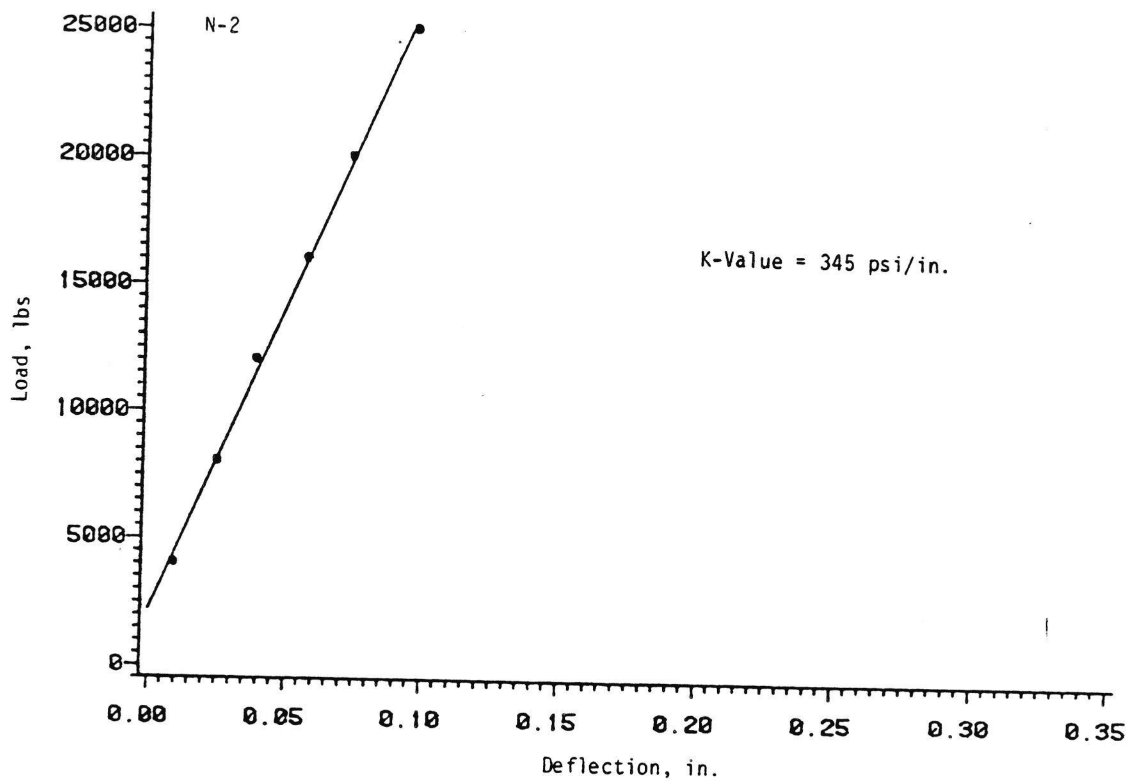

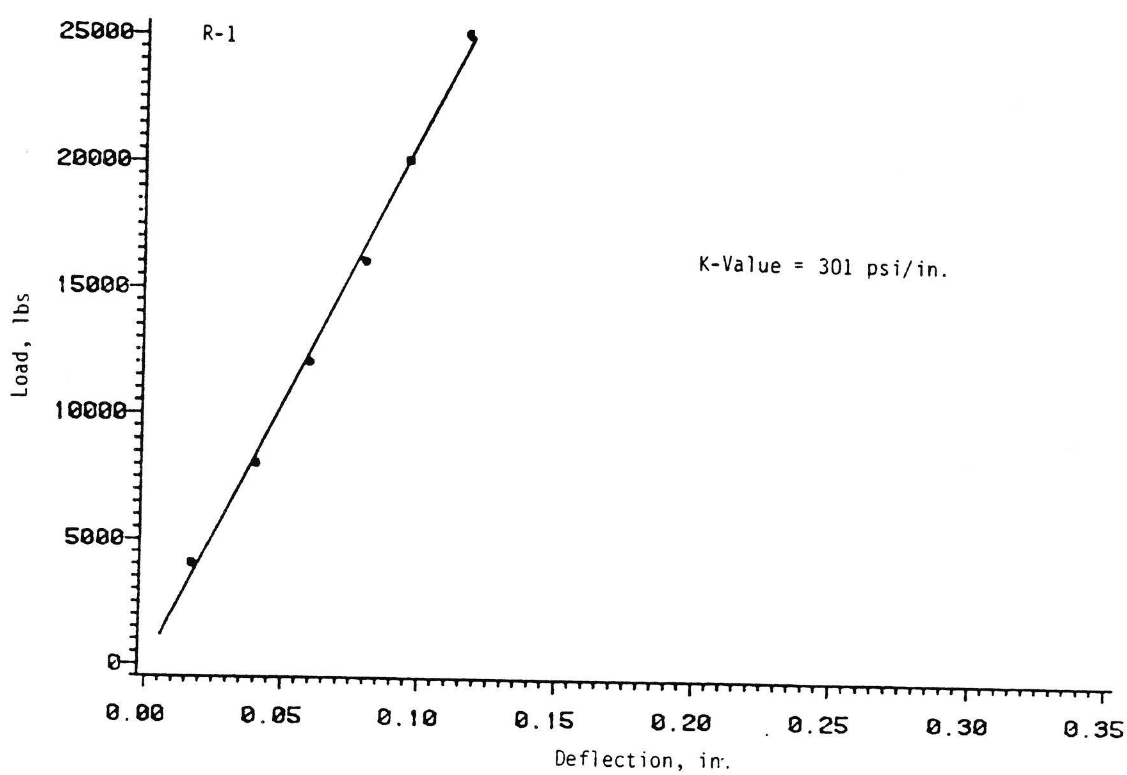

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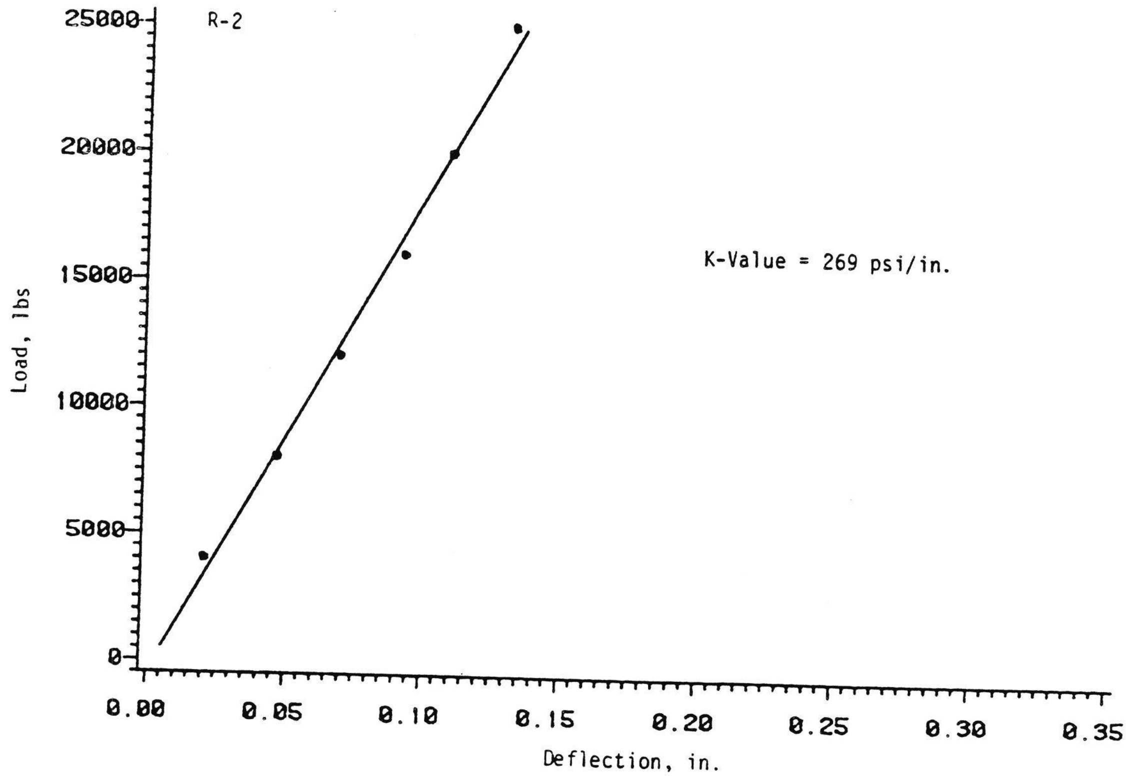

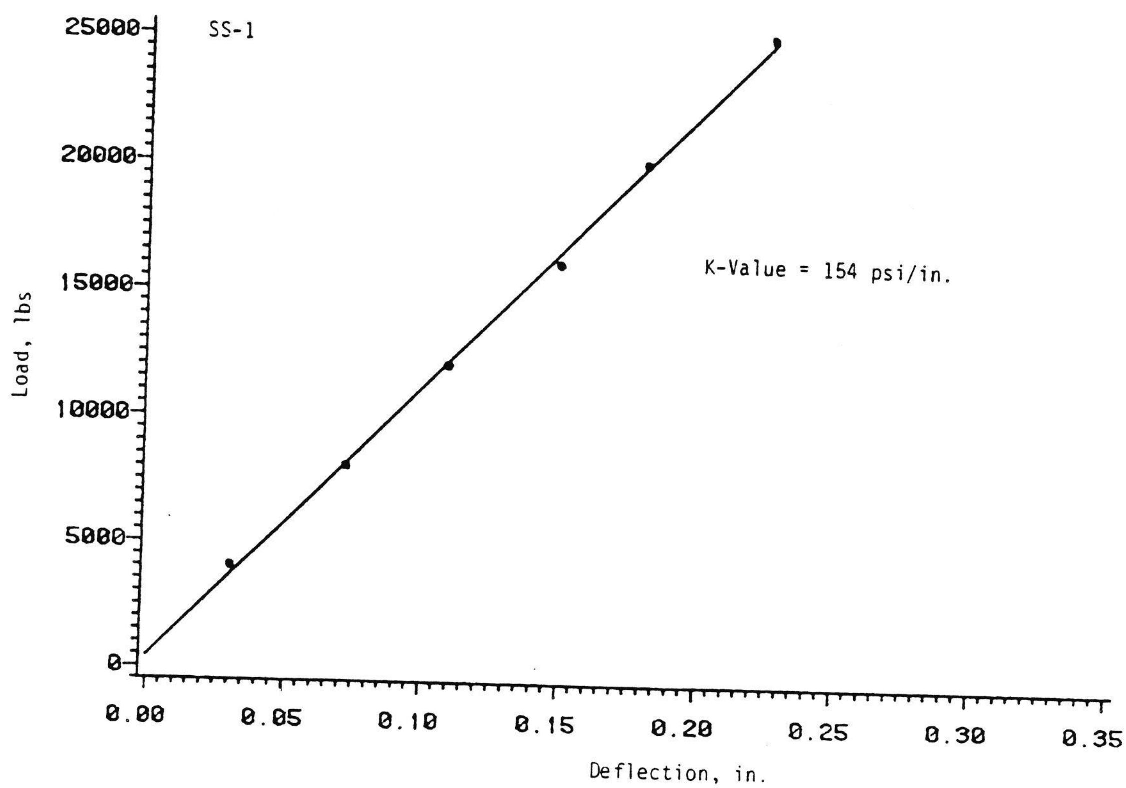

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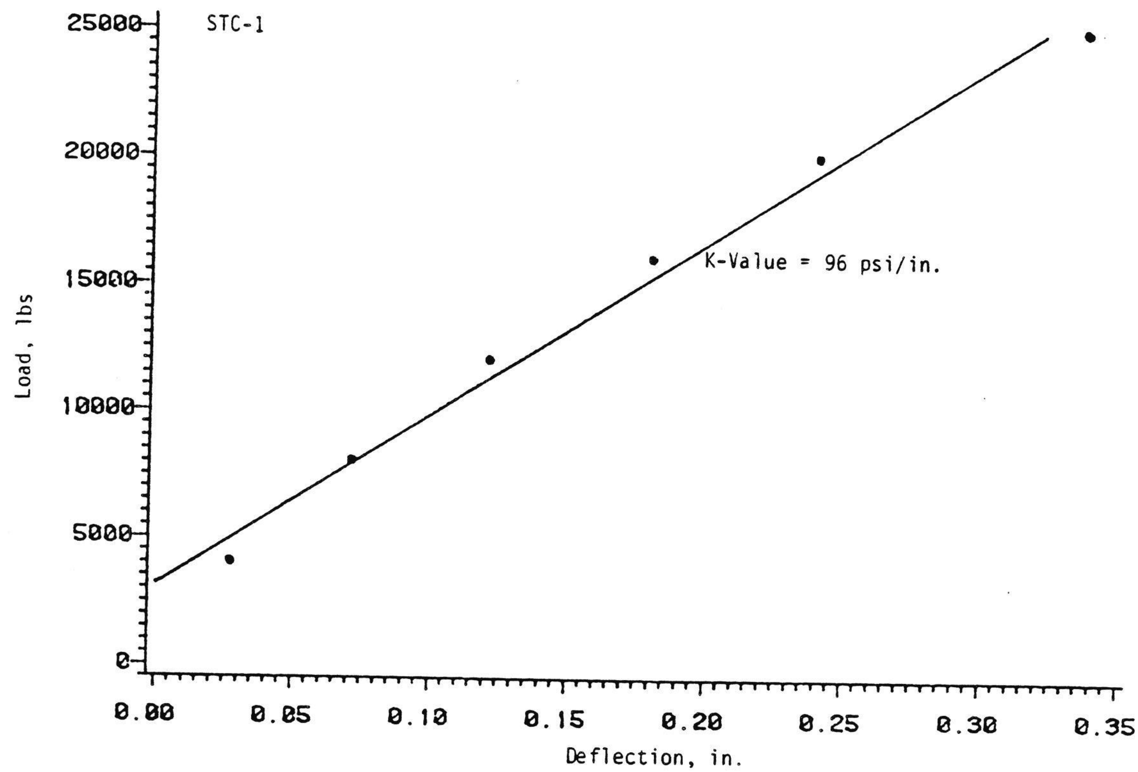

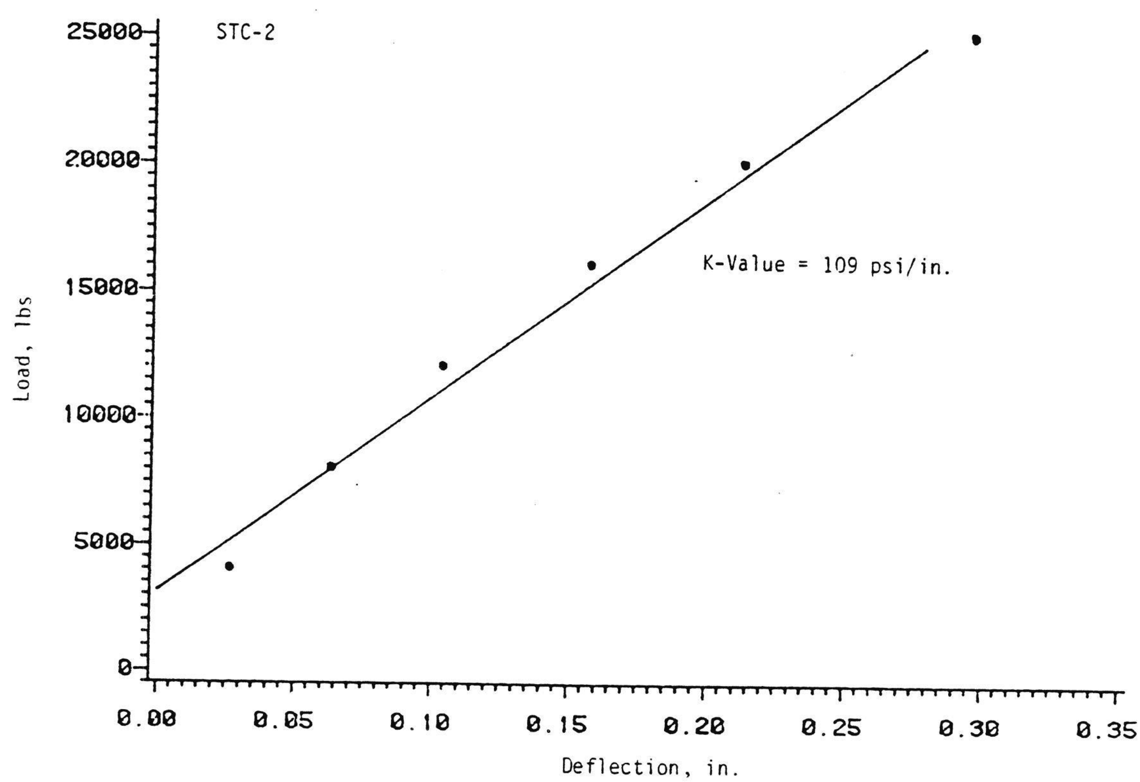

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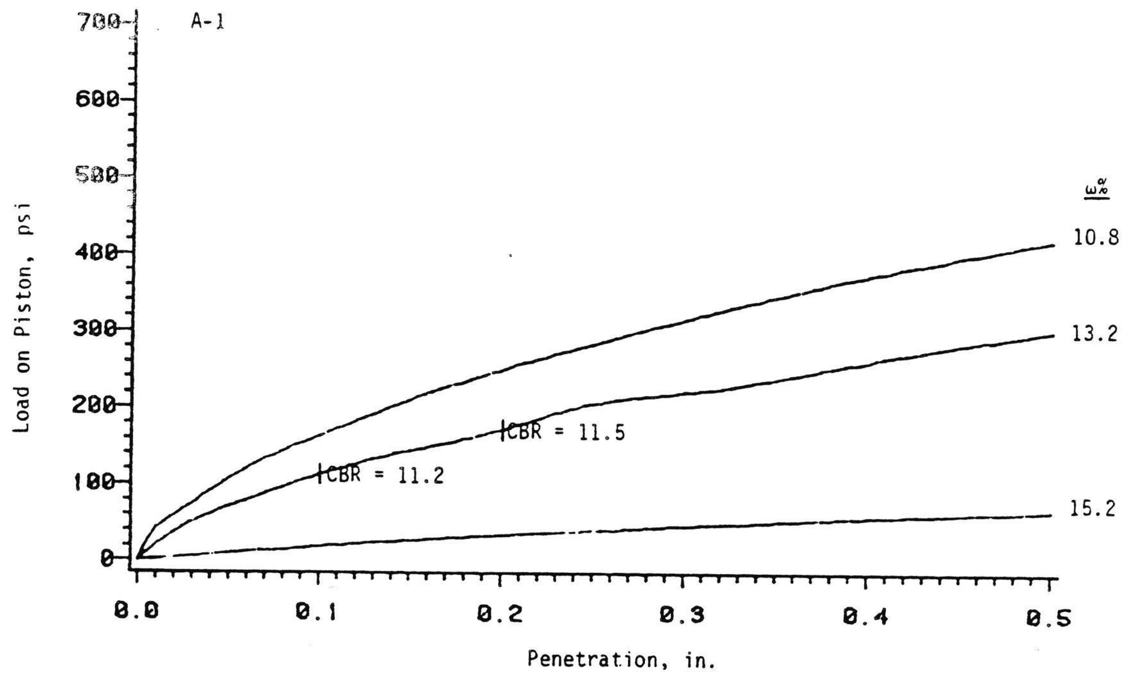

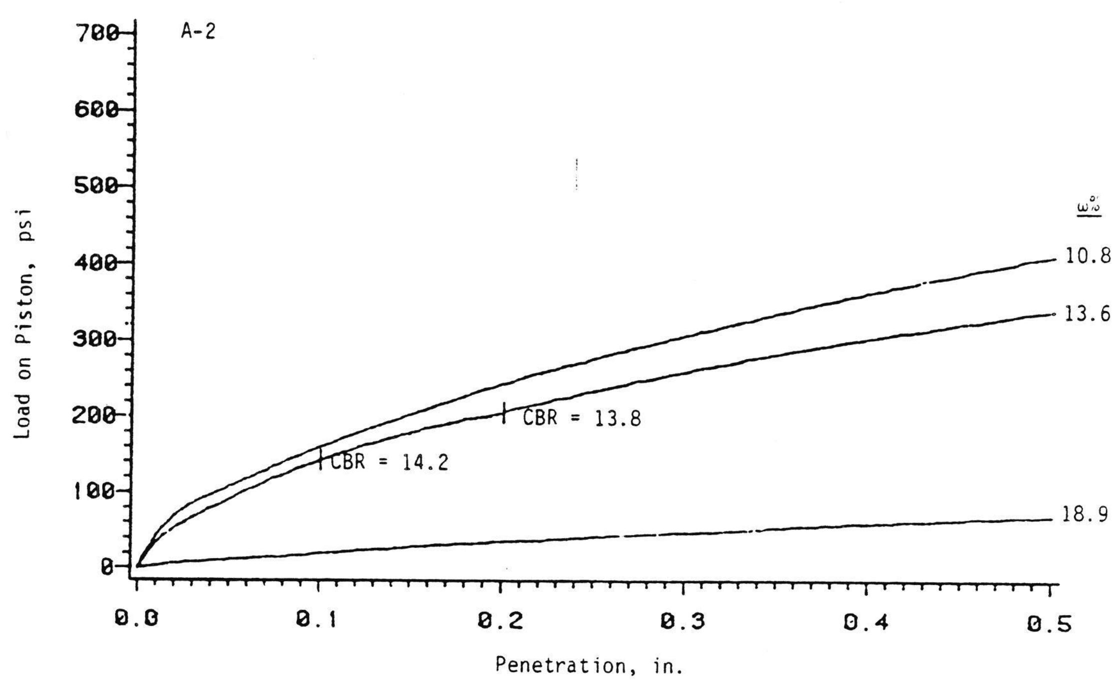

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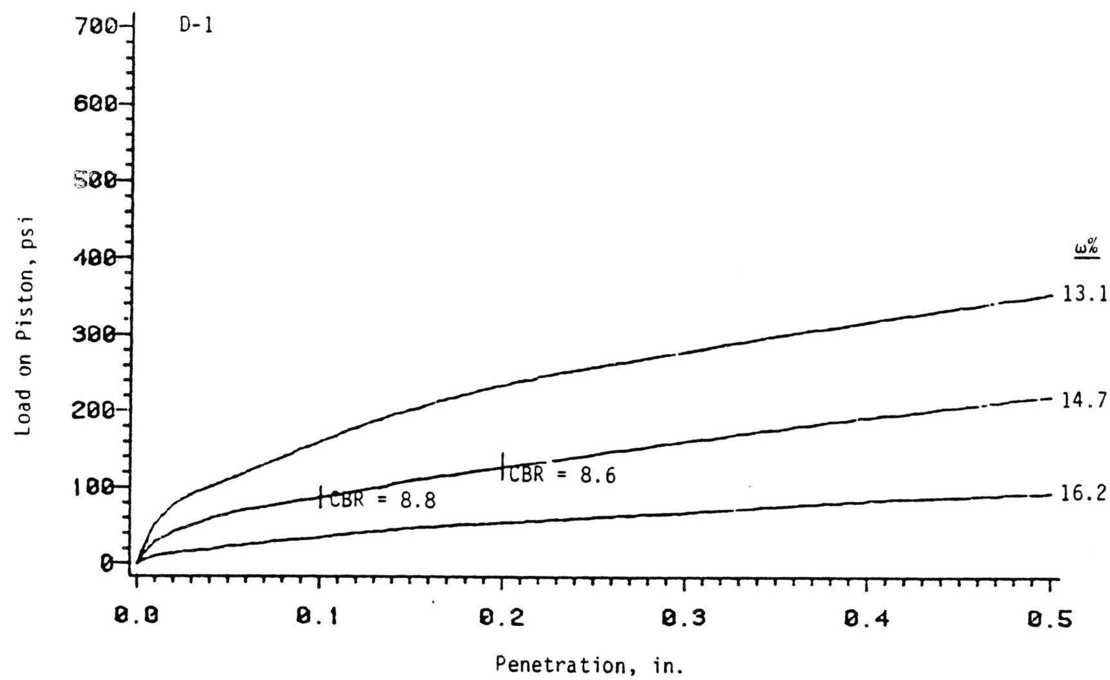

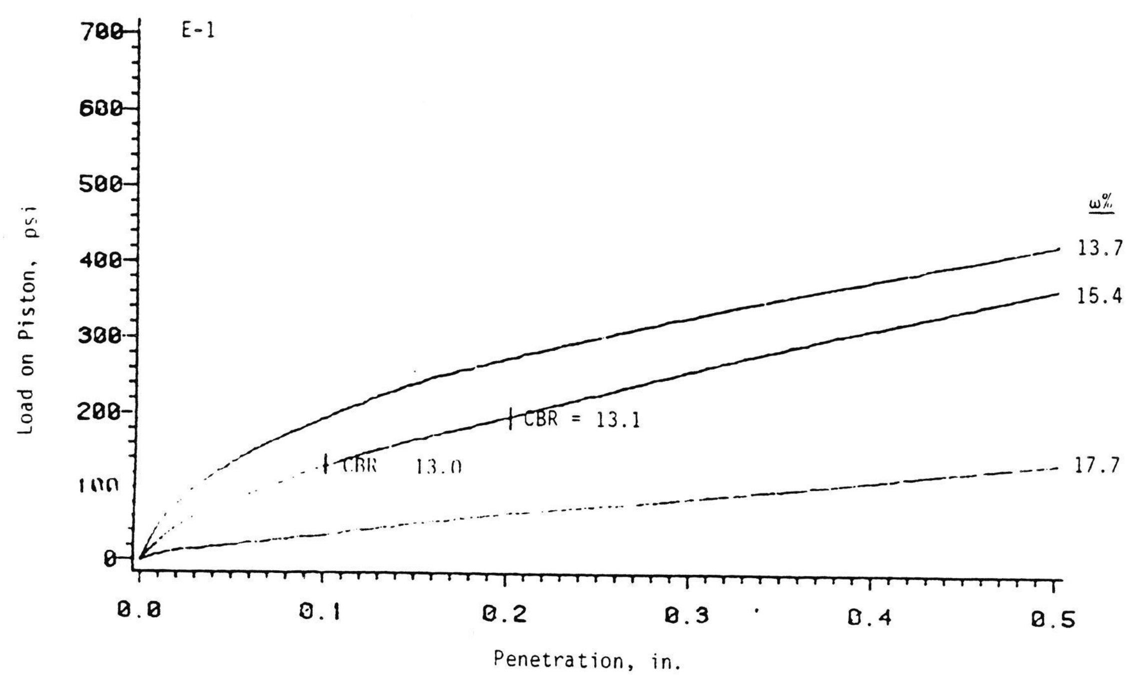

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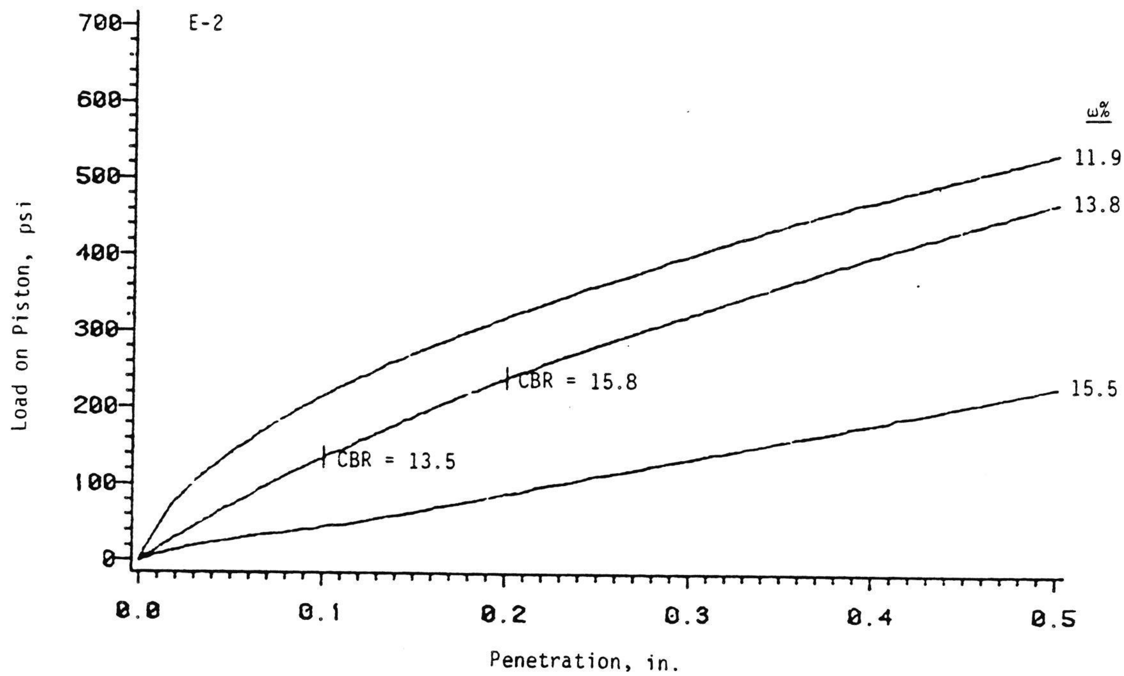

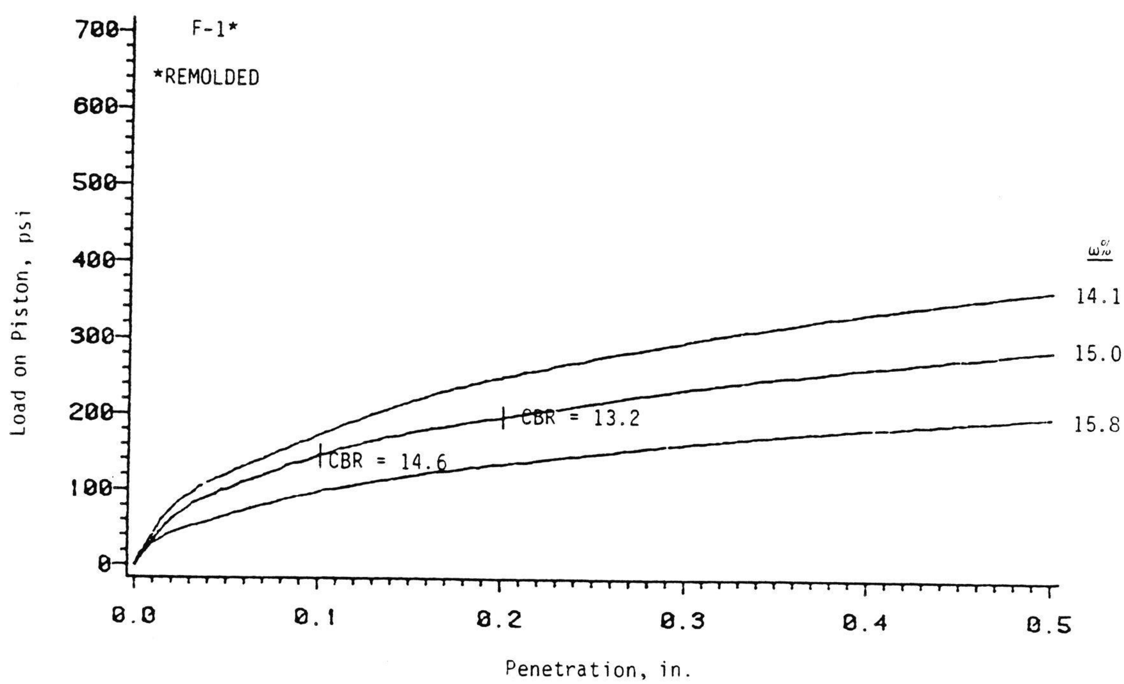

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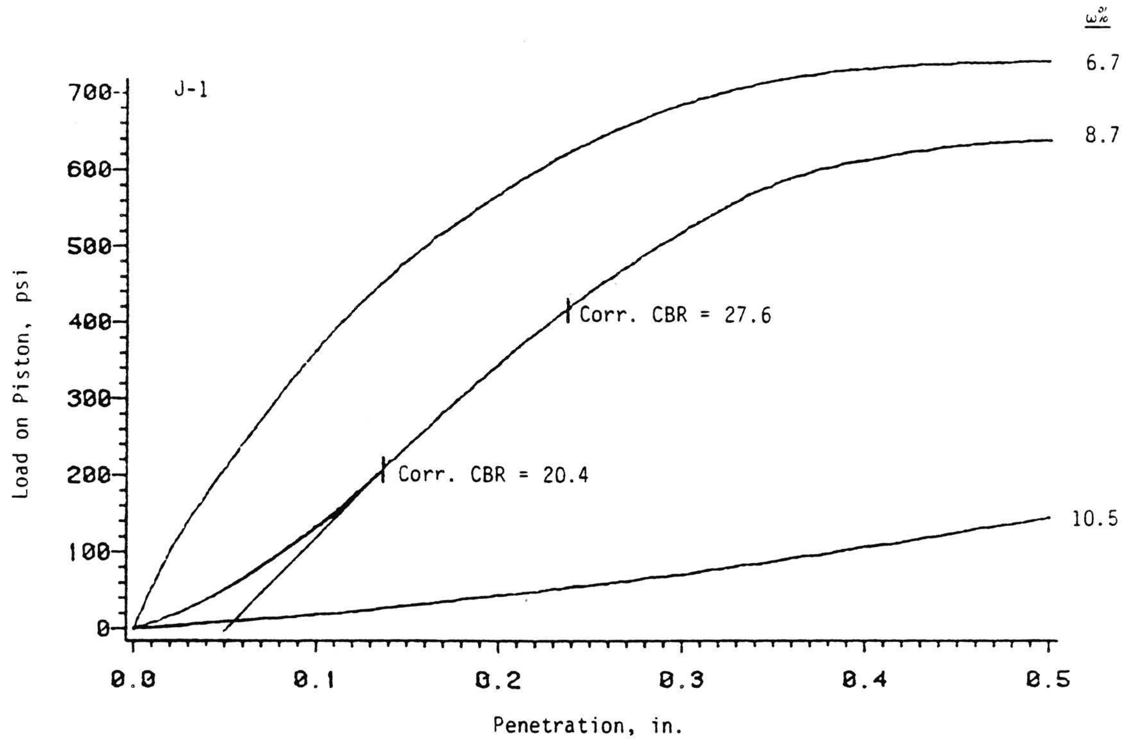

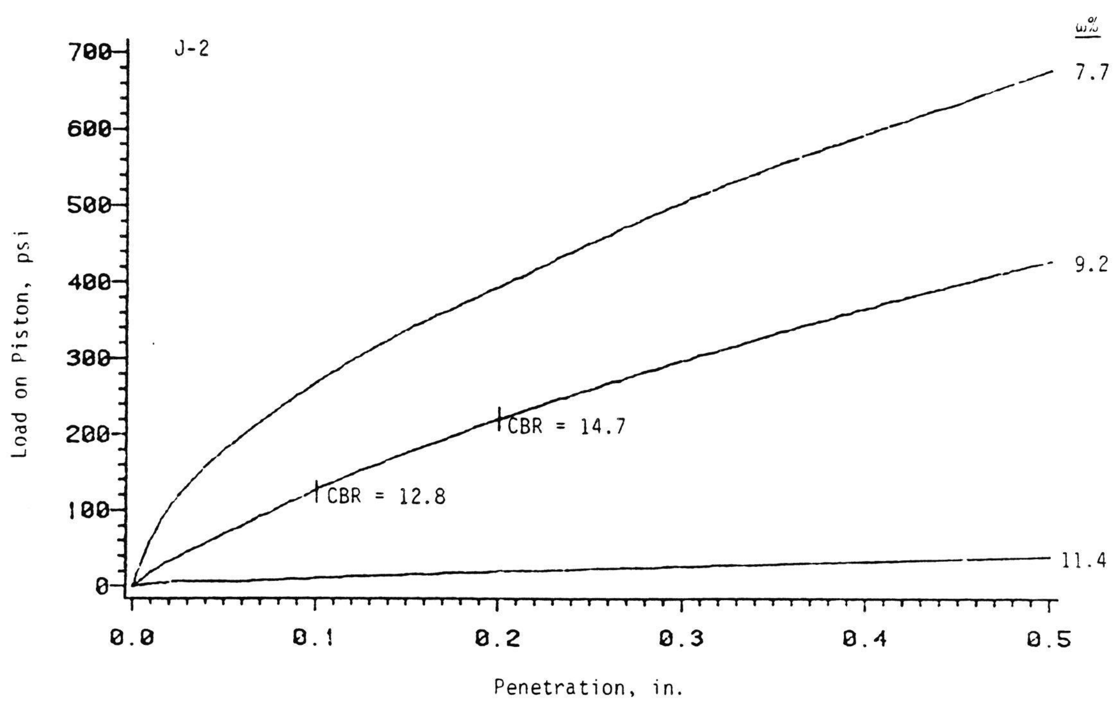

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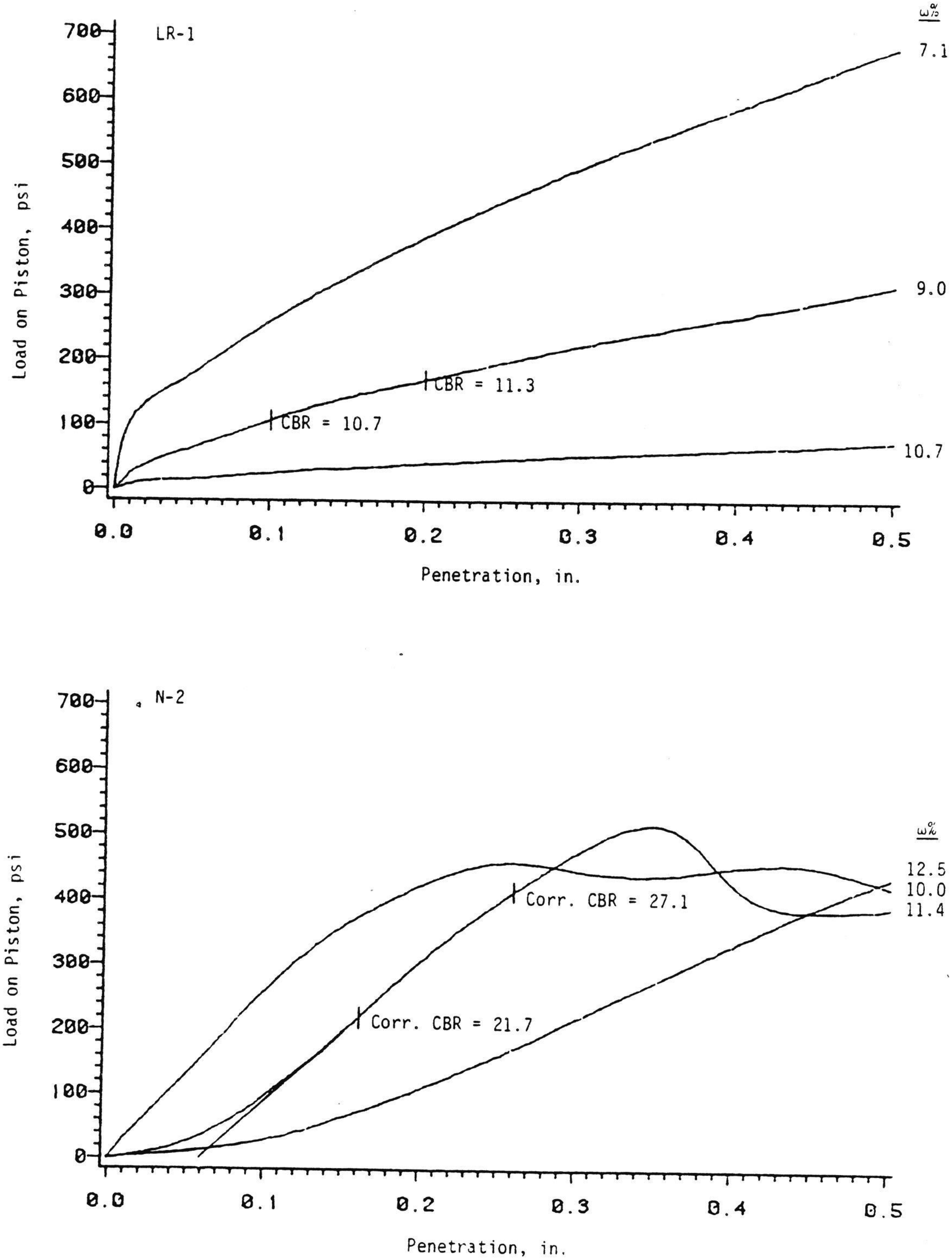

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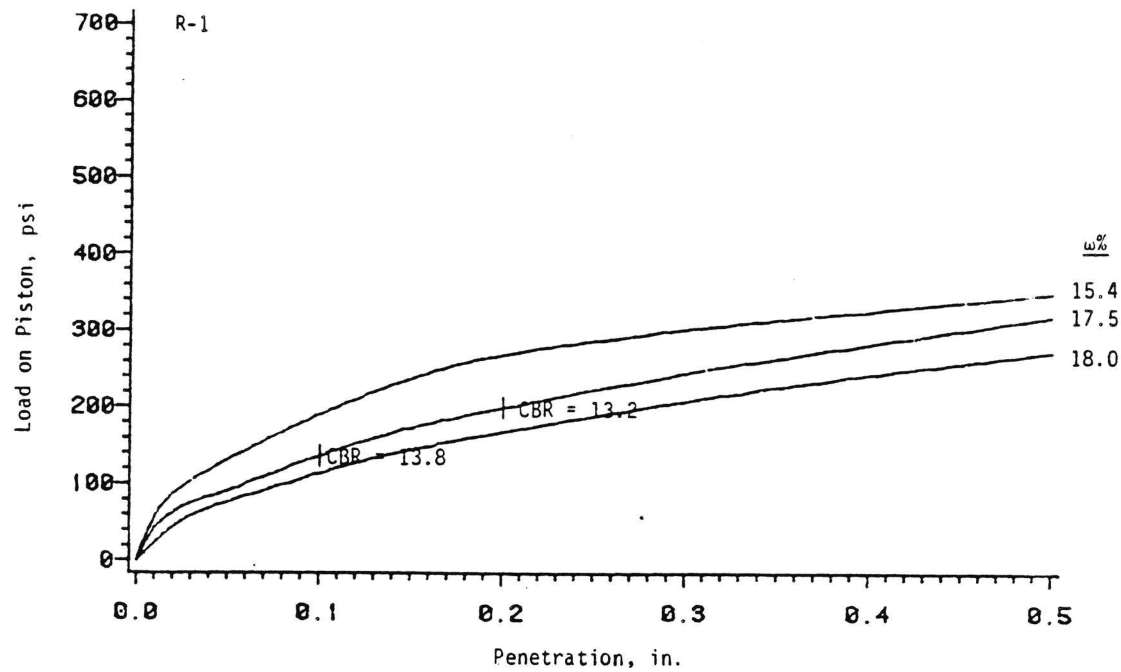
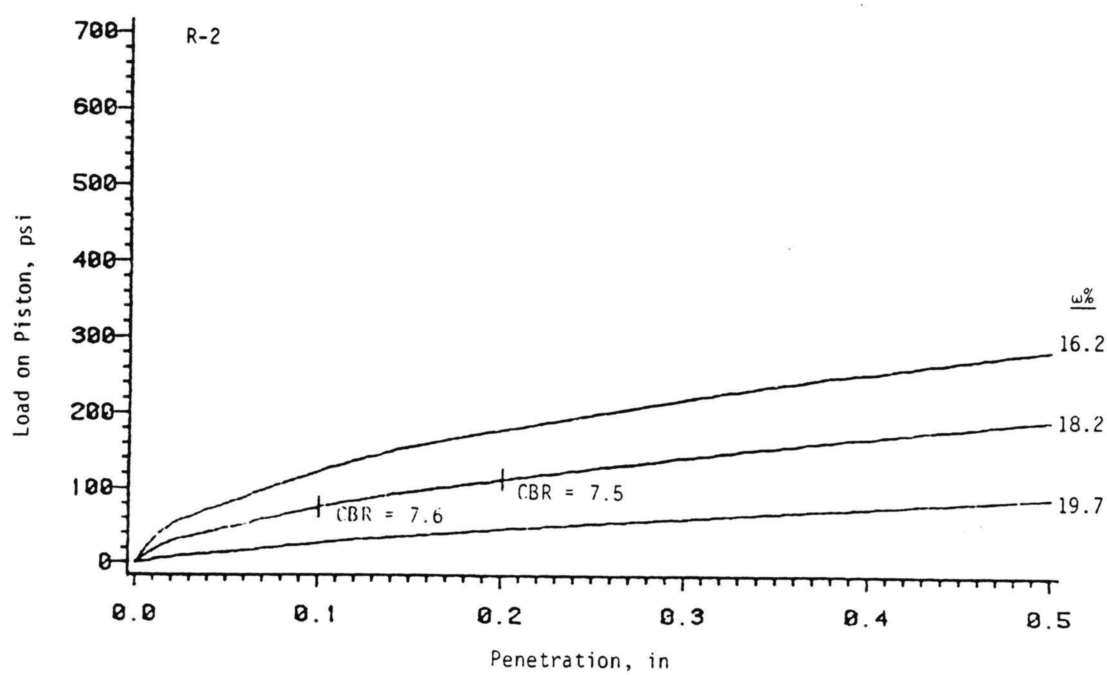
California Bearing Ratio Results





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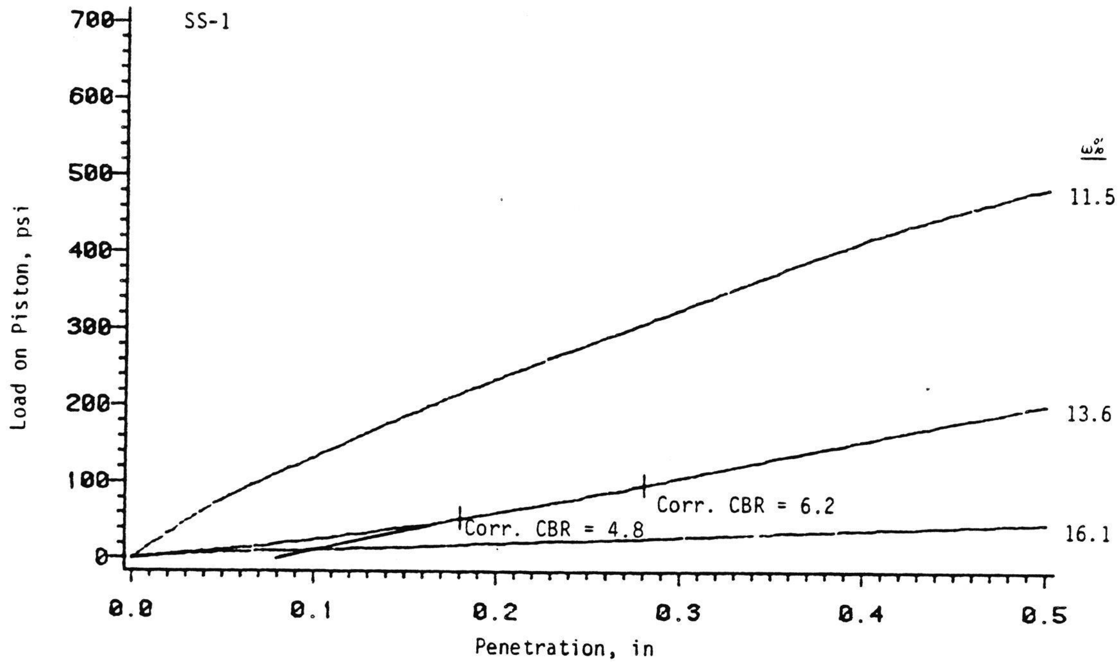

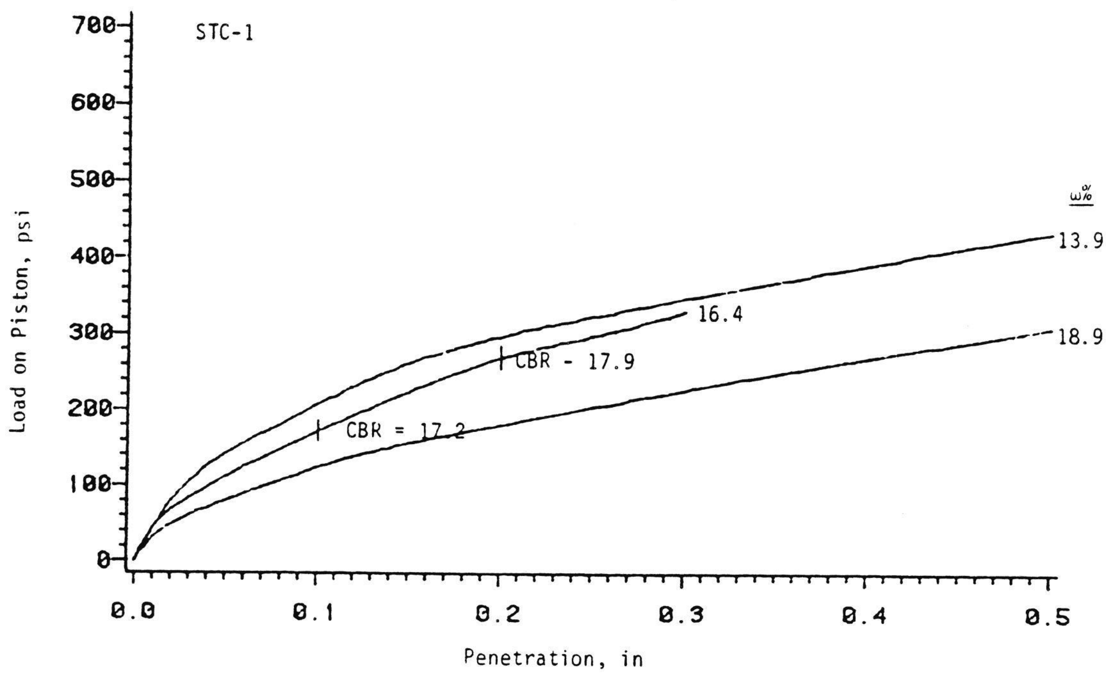

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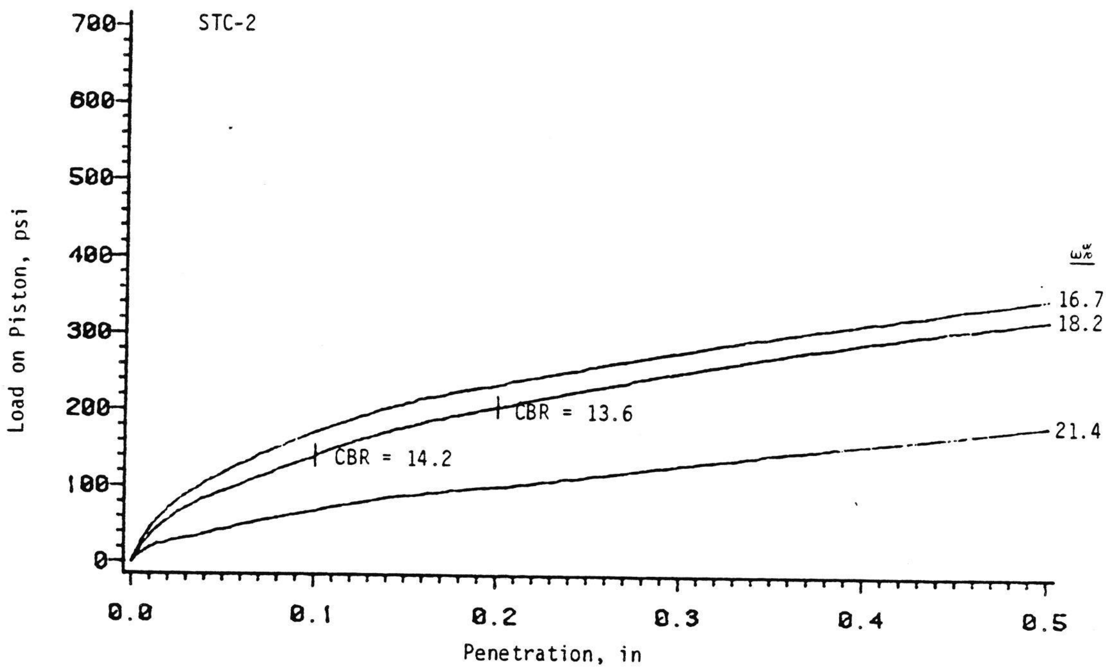
California Bearing Ratio Results



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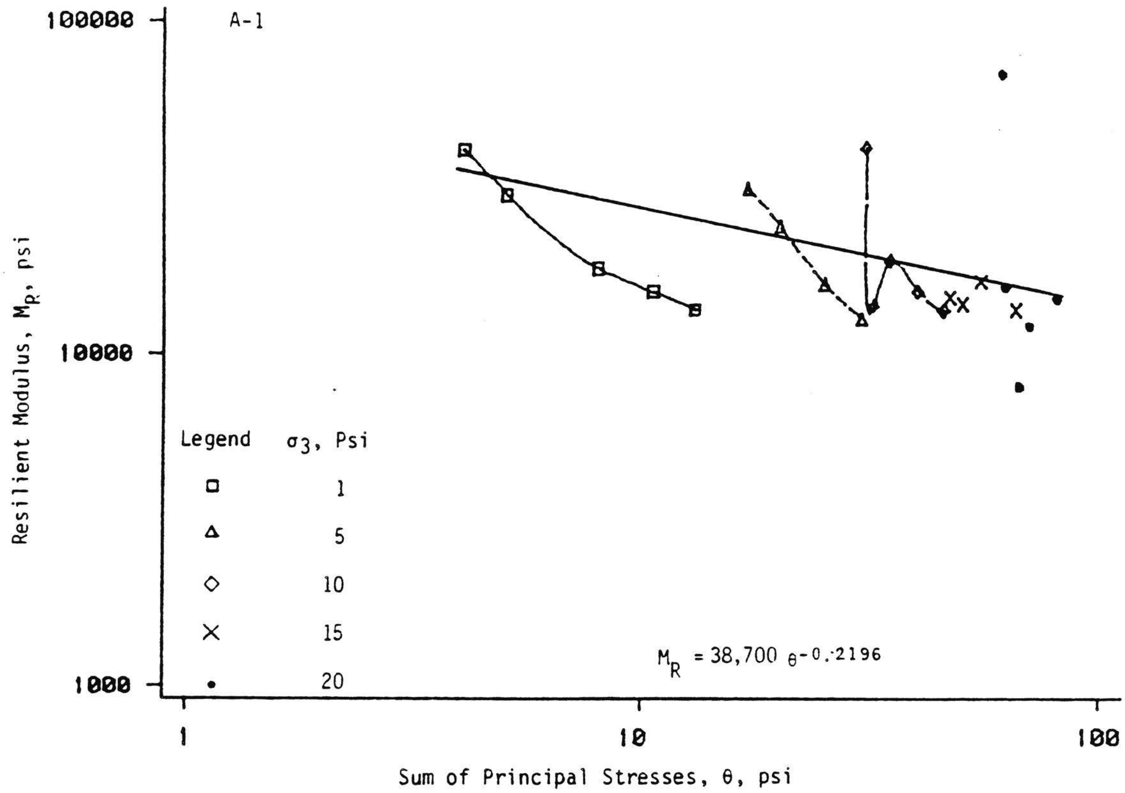
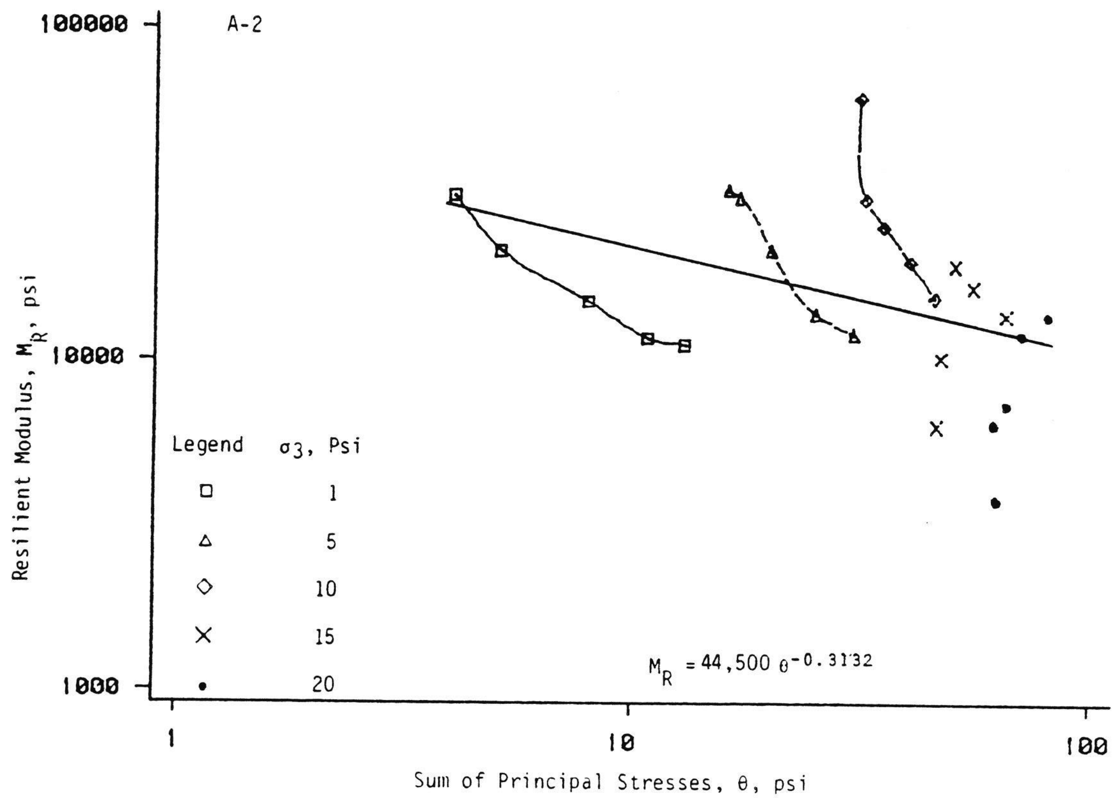
California Bearing Ratio Results





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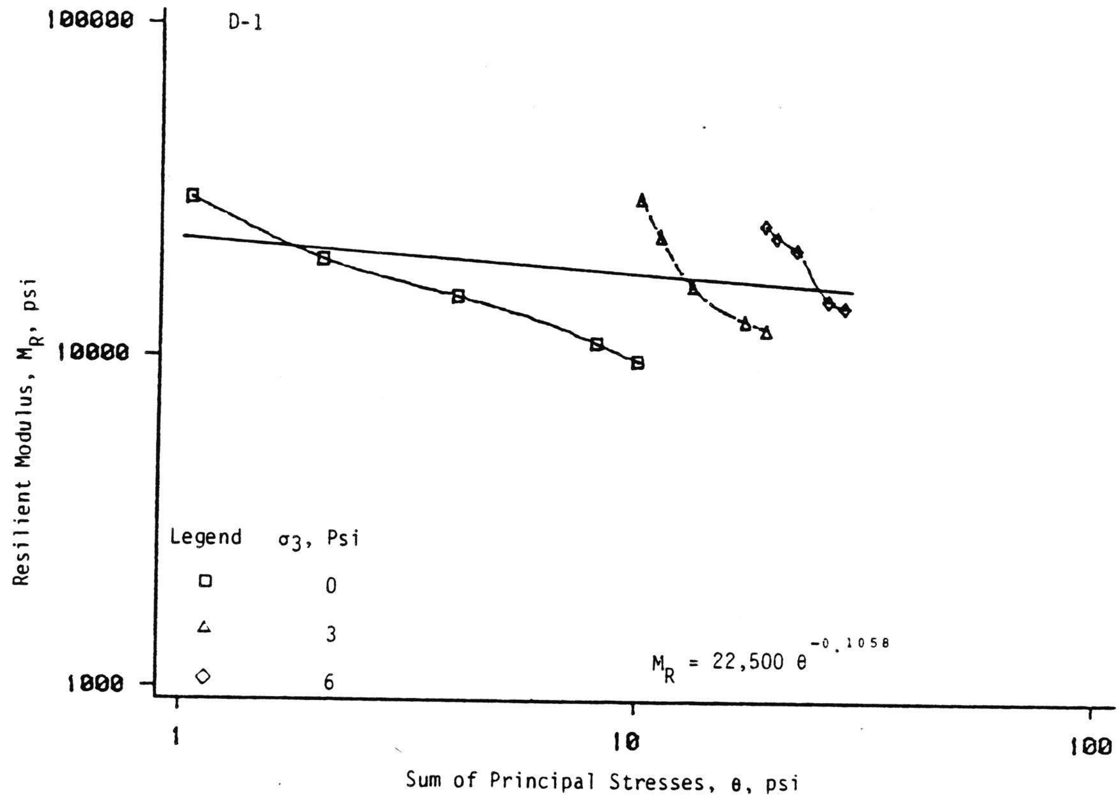
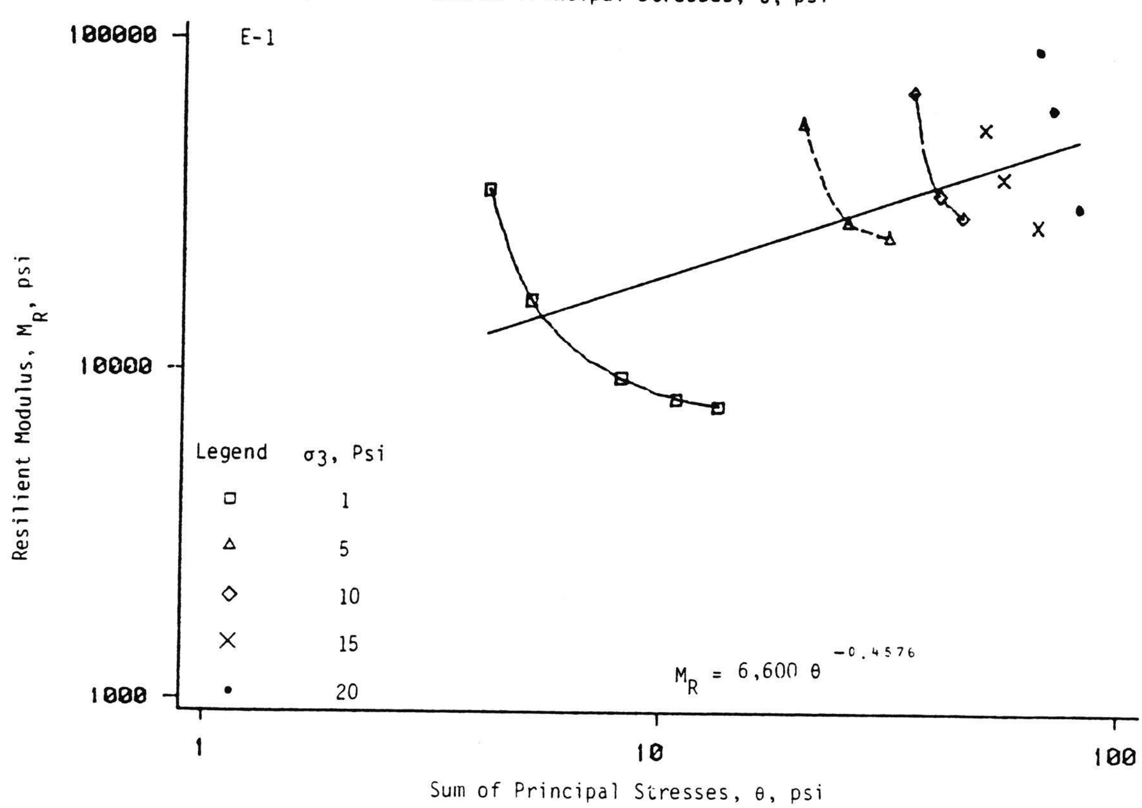
Resilient Modulus Test Results





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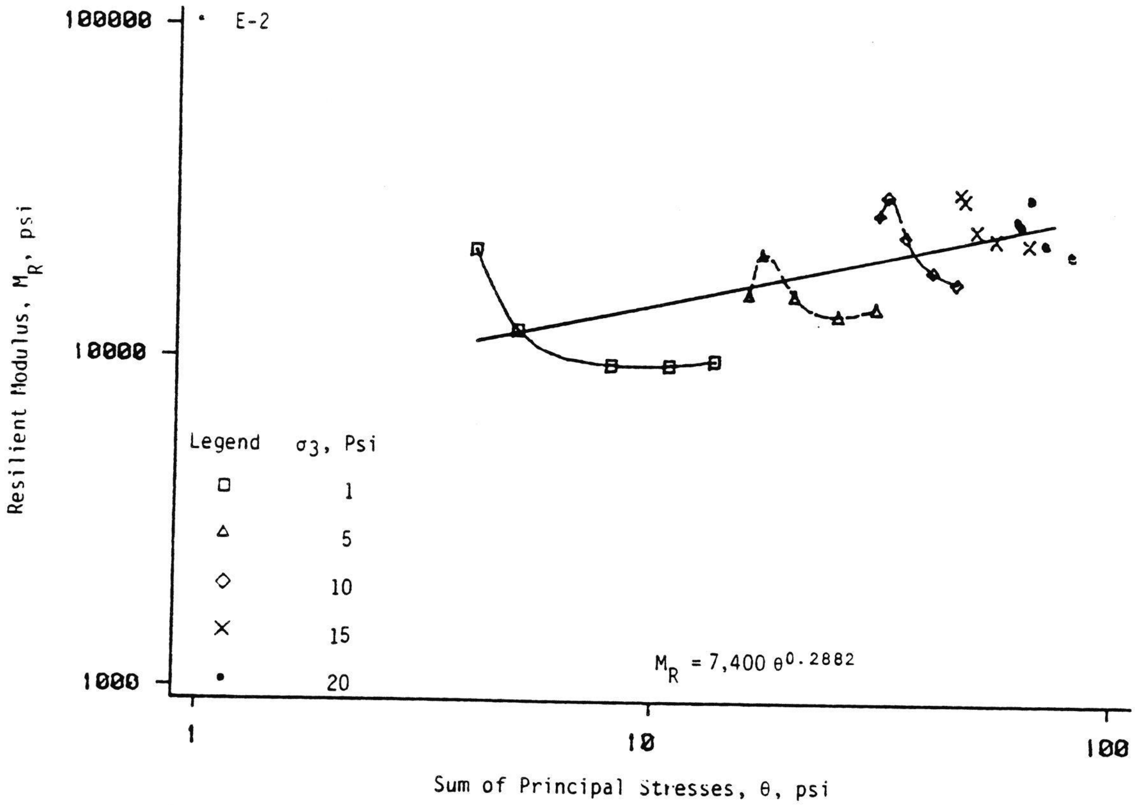

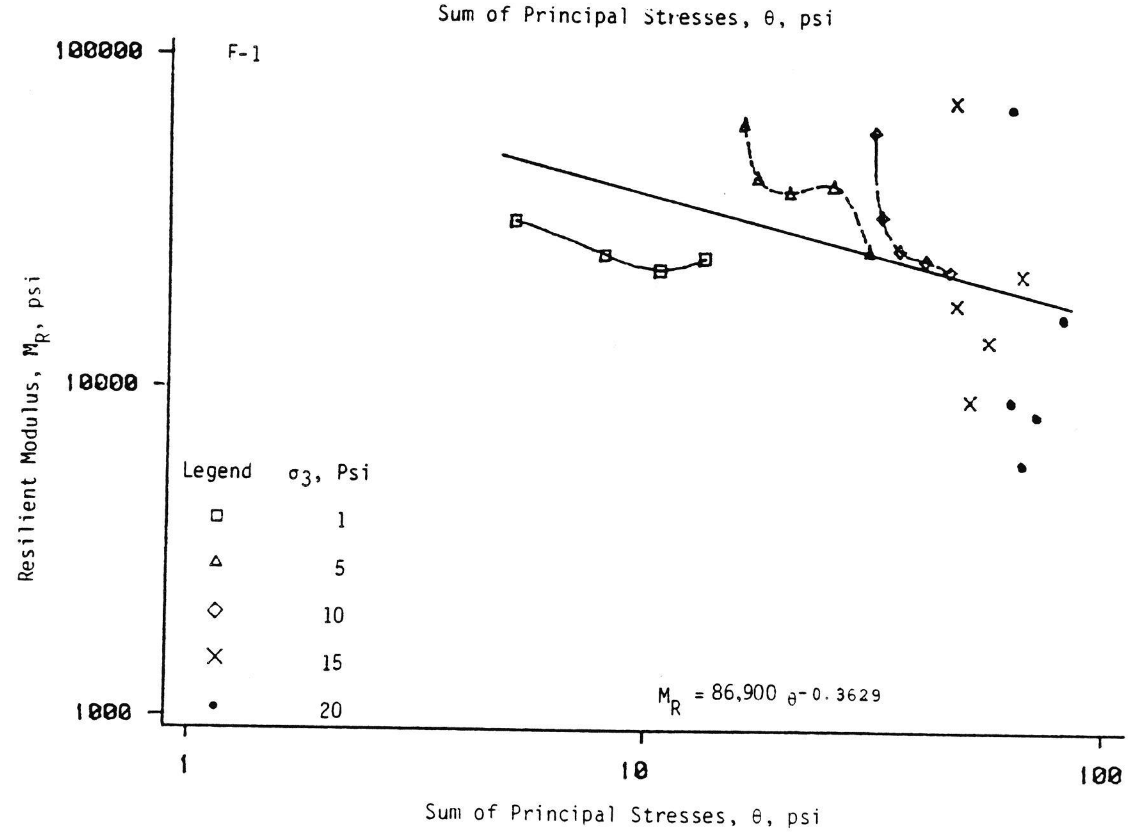

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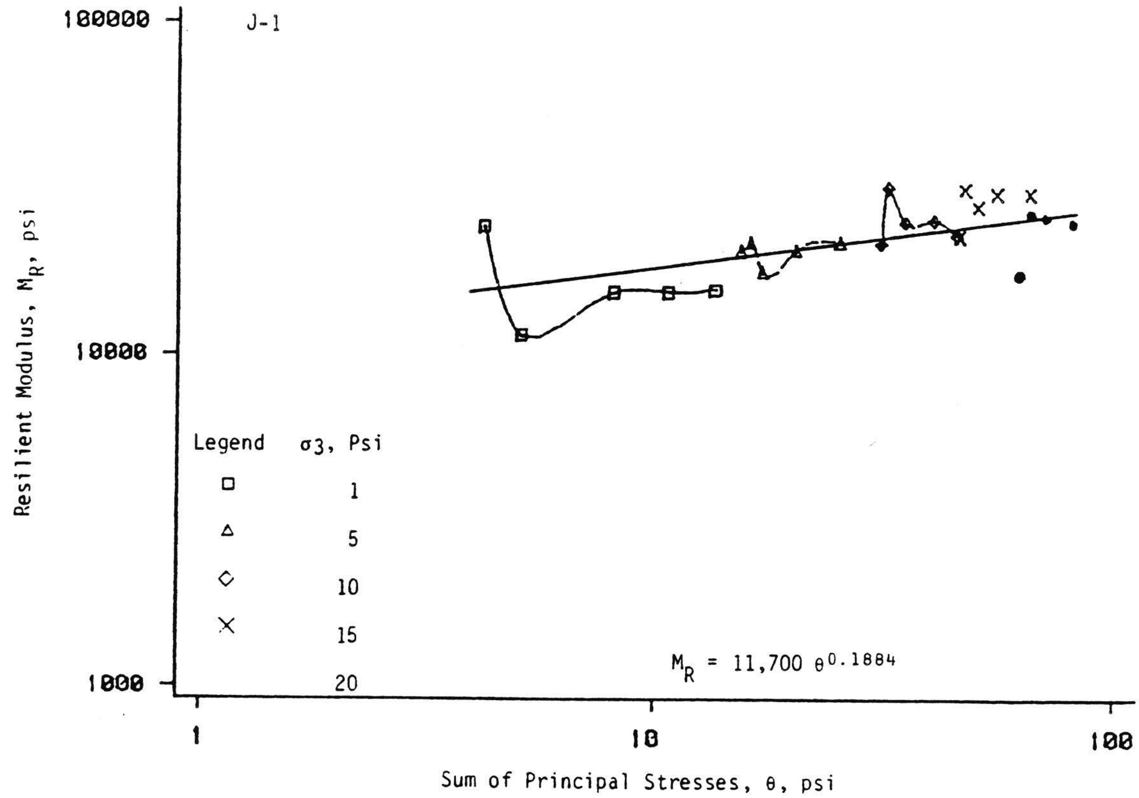
Resilient Modulus Test Results



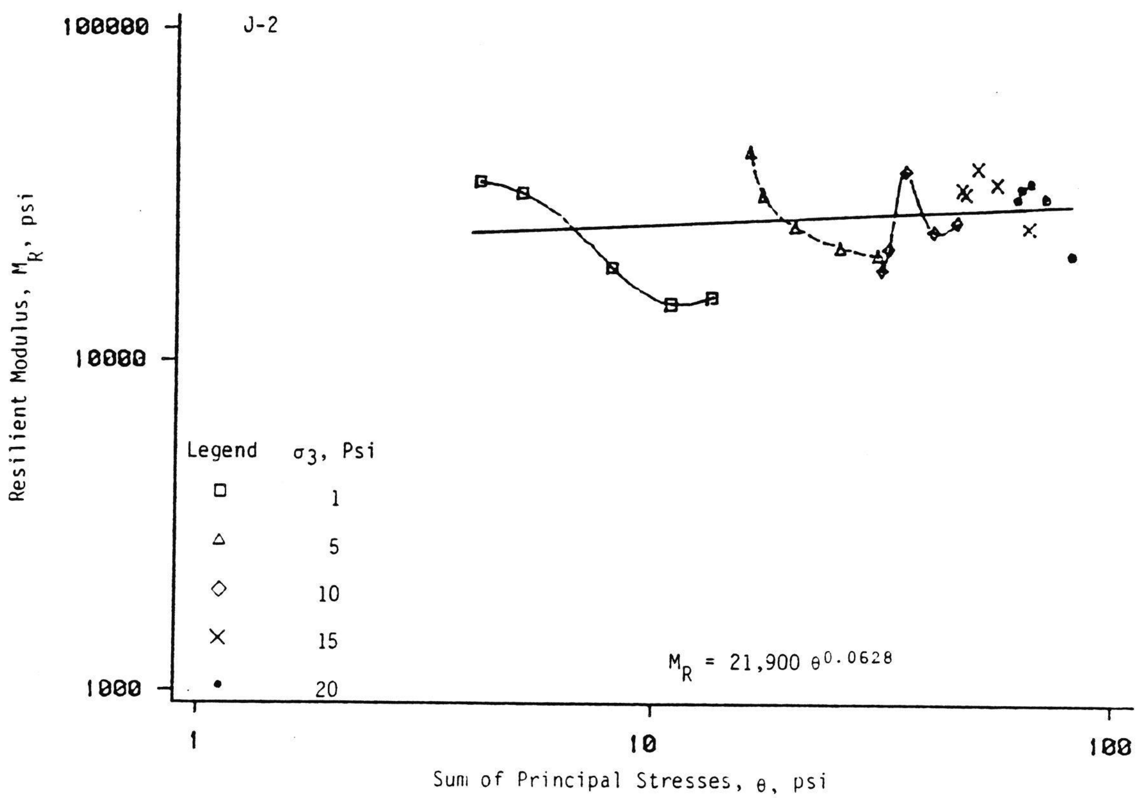
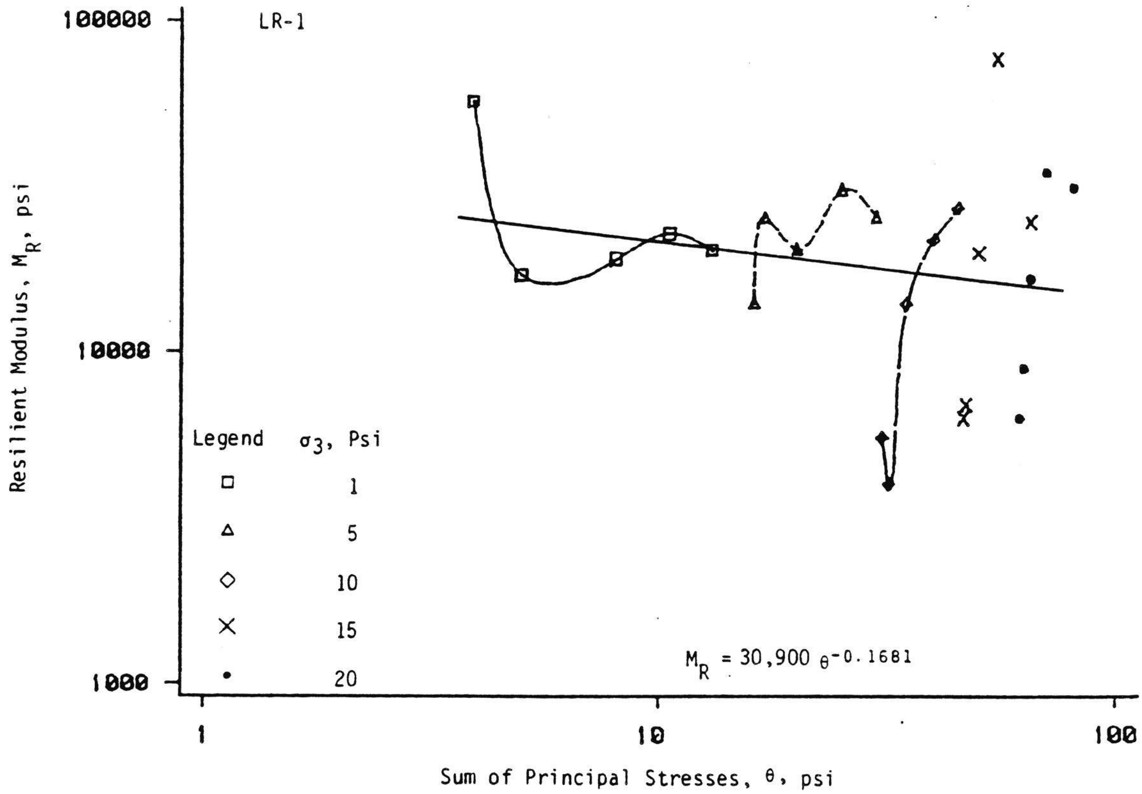


Figure A-3 g), h)
Resilient Modulus Test Results



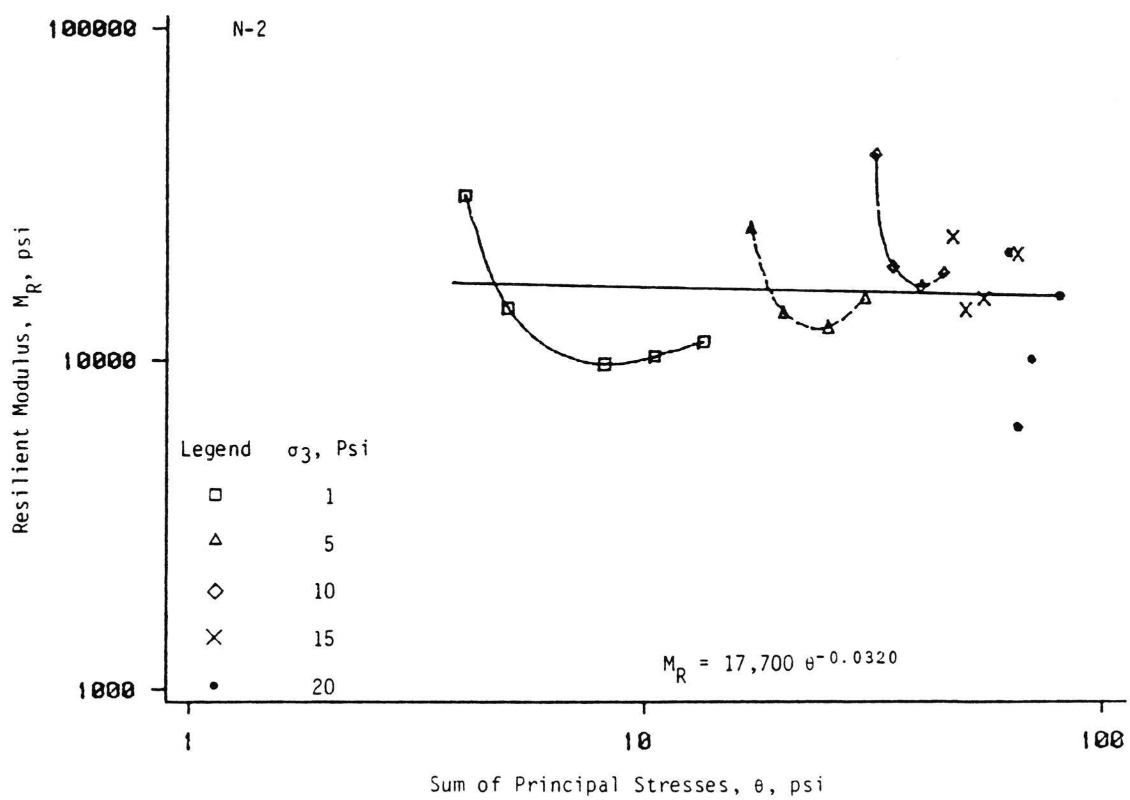
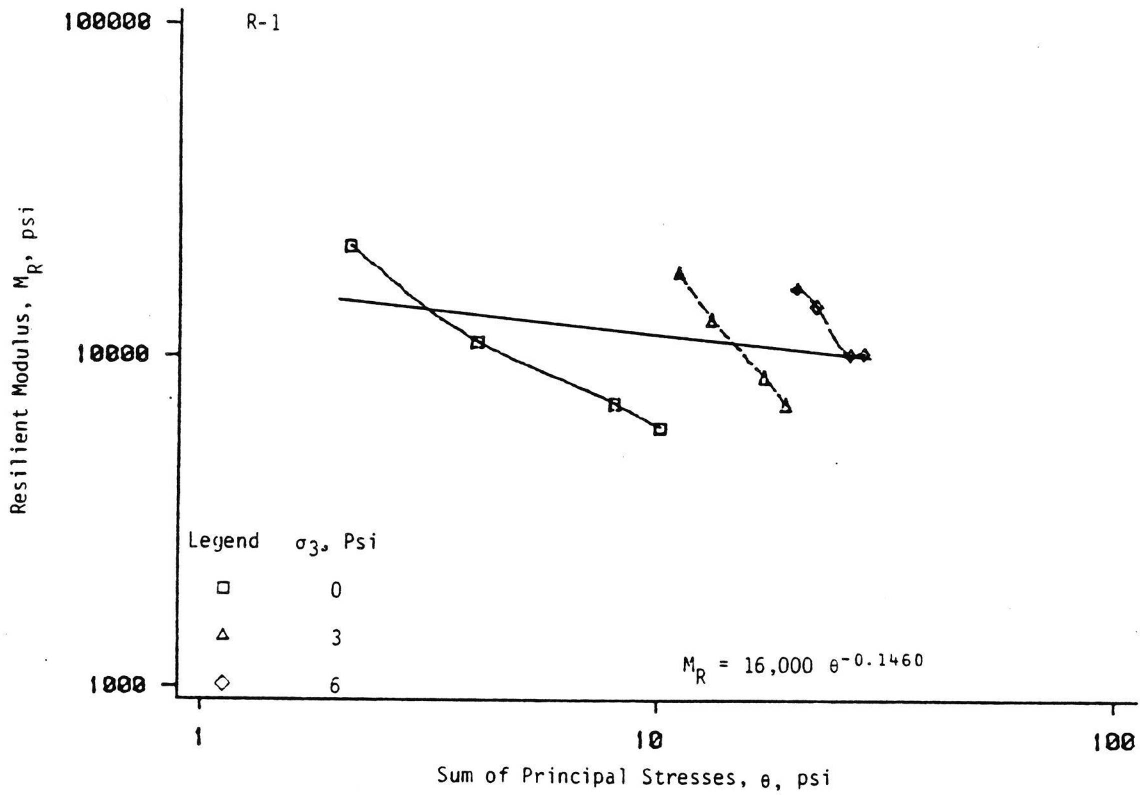


Figure A-3 i), j)
Resilient Modulus Test Results



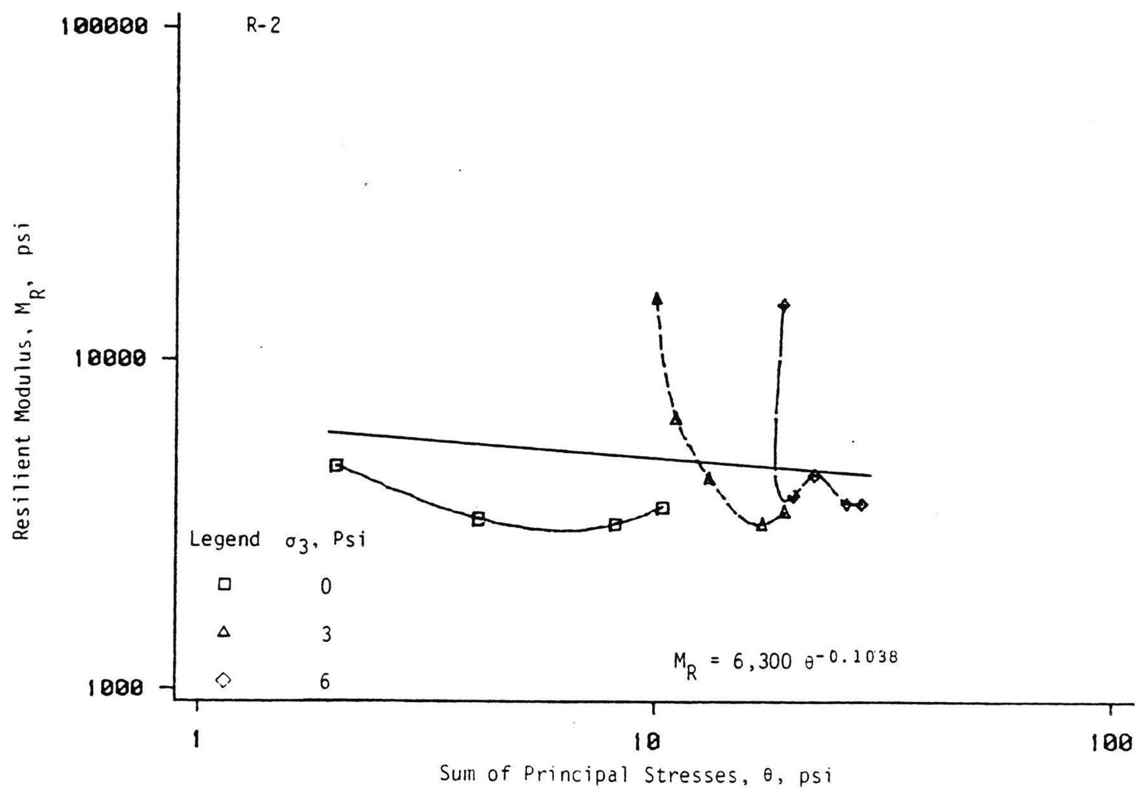
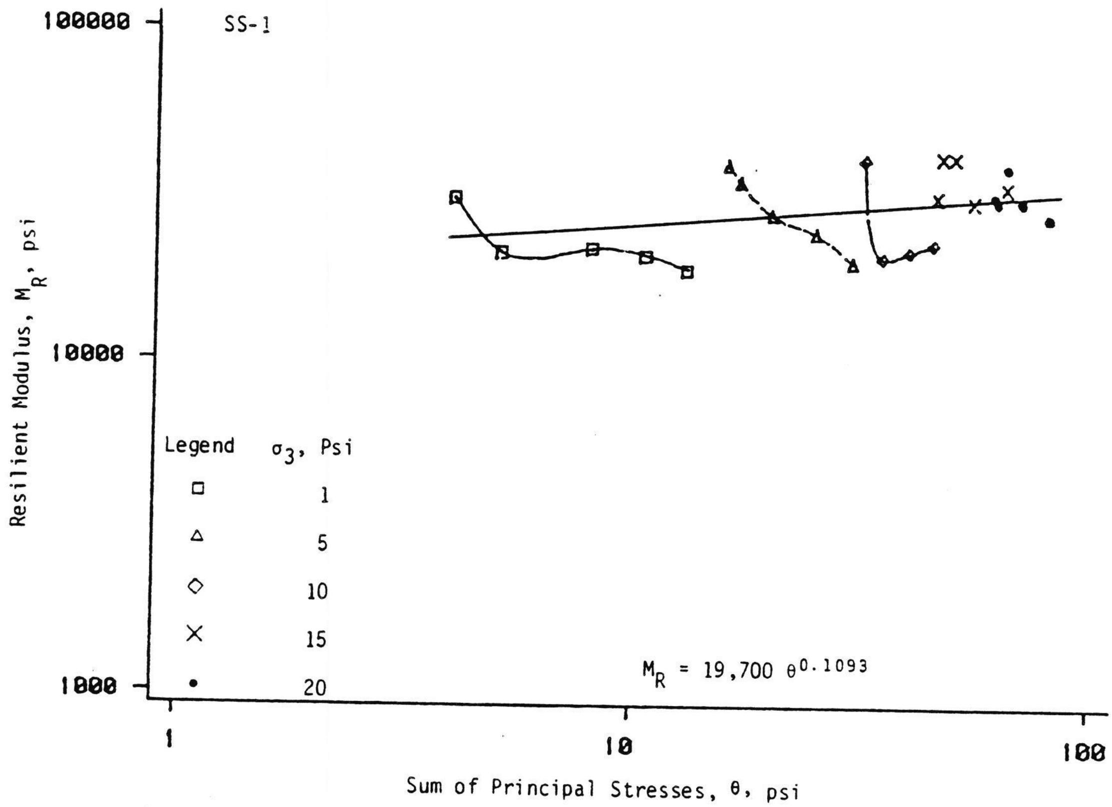


Figure A-3 k), l)
Resilient Modulus Test Results



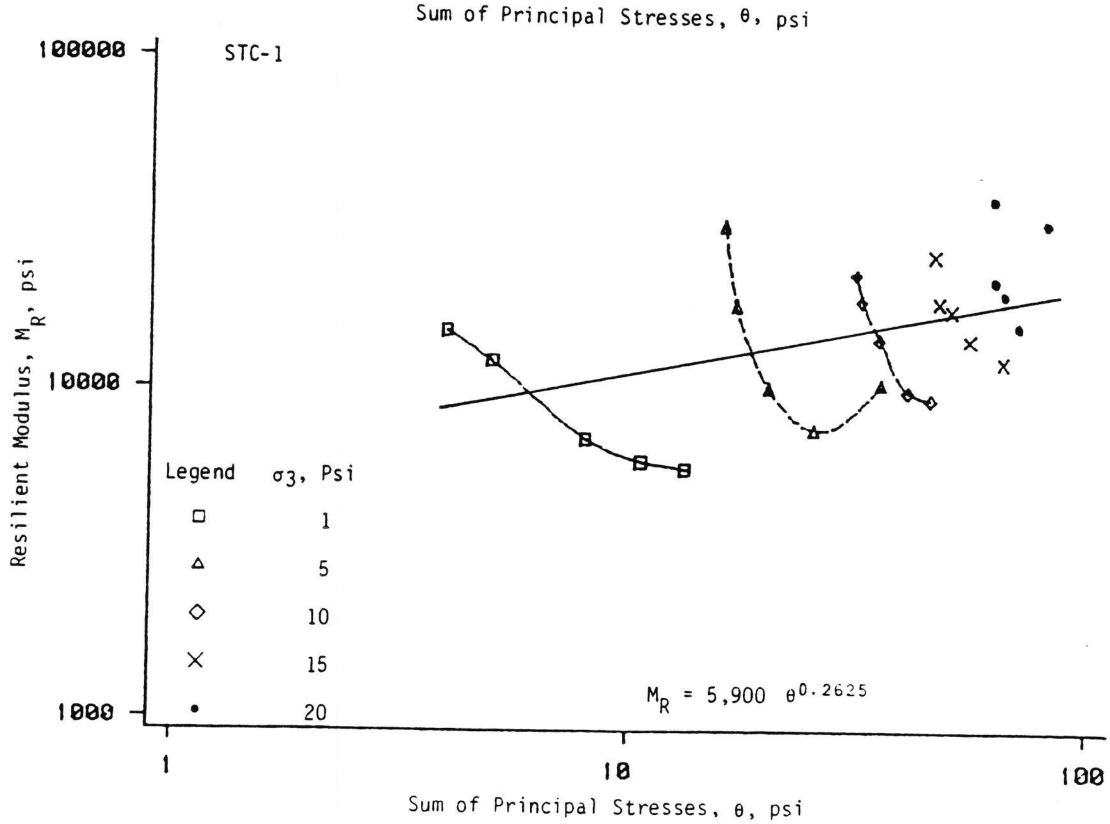


Figure A-3 m), n)
Resilient Modulus Test Results

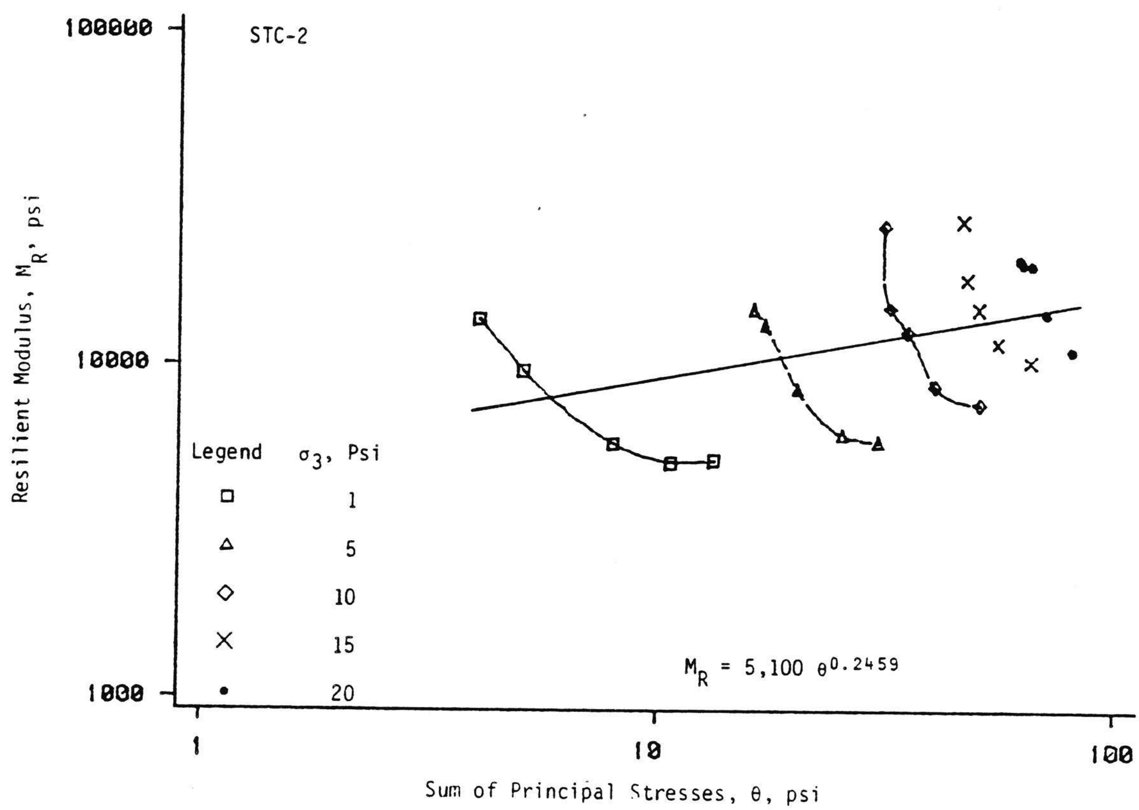
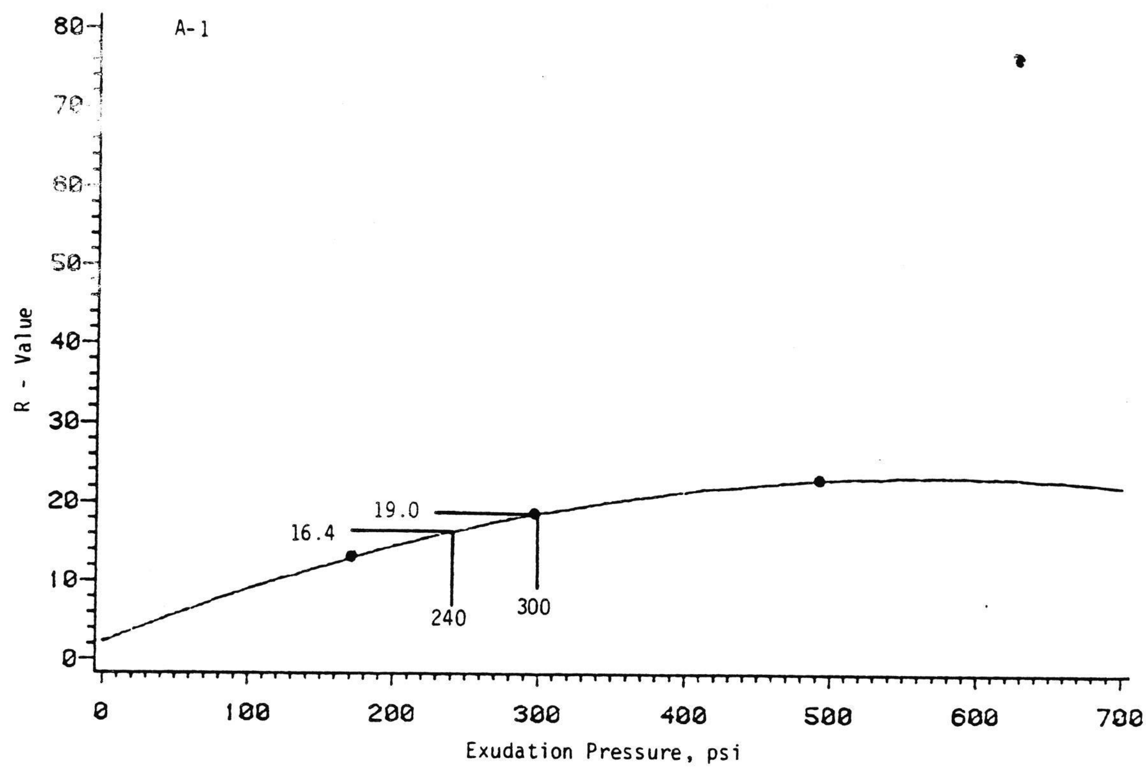


Figure A-3 o)
Resilient Modulus Test Results



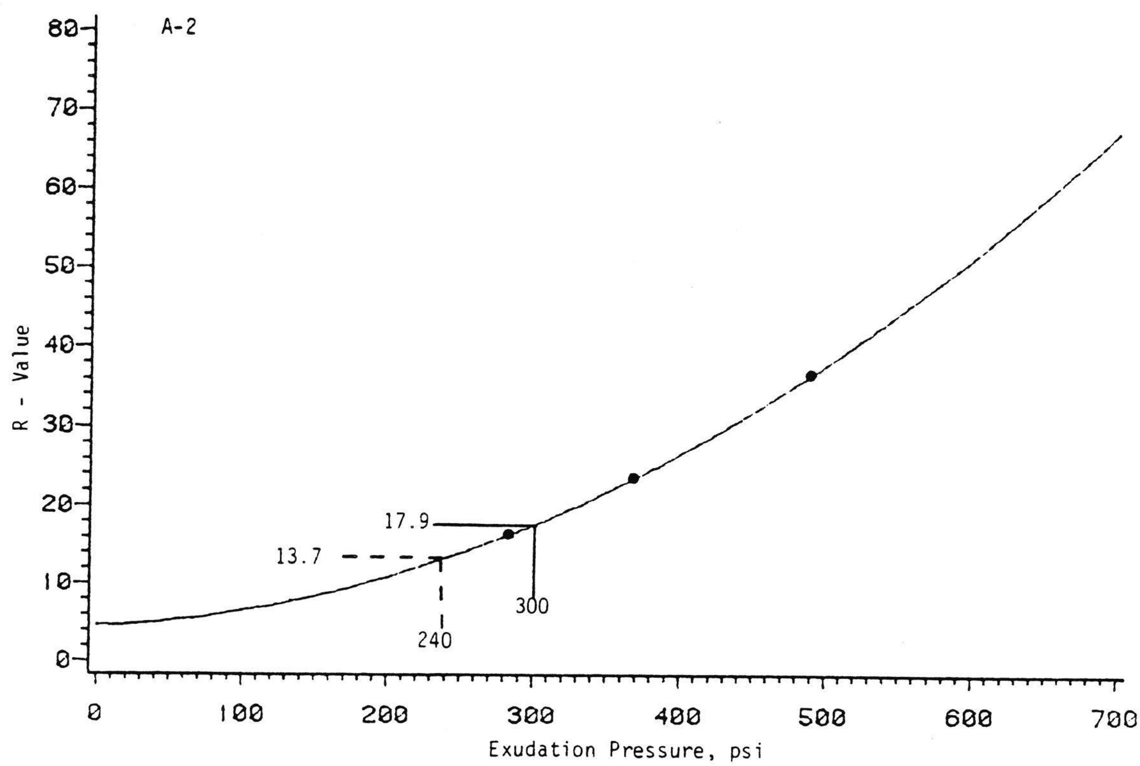
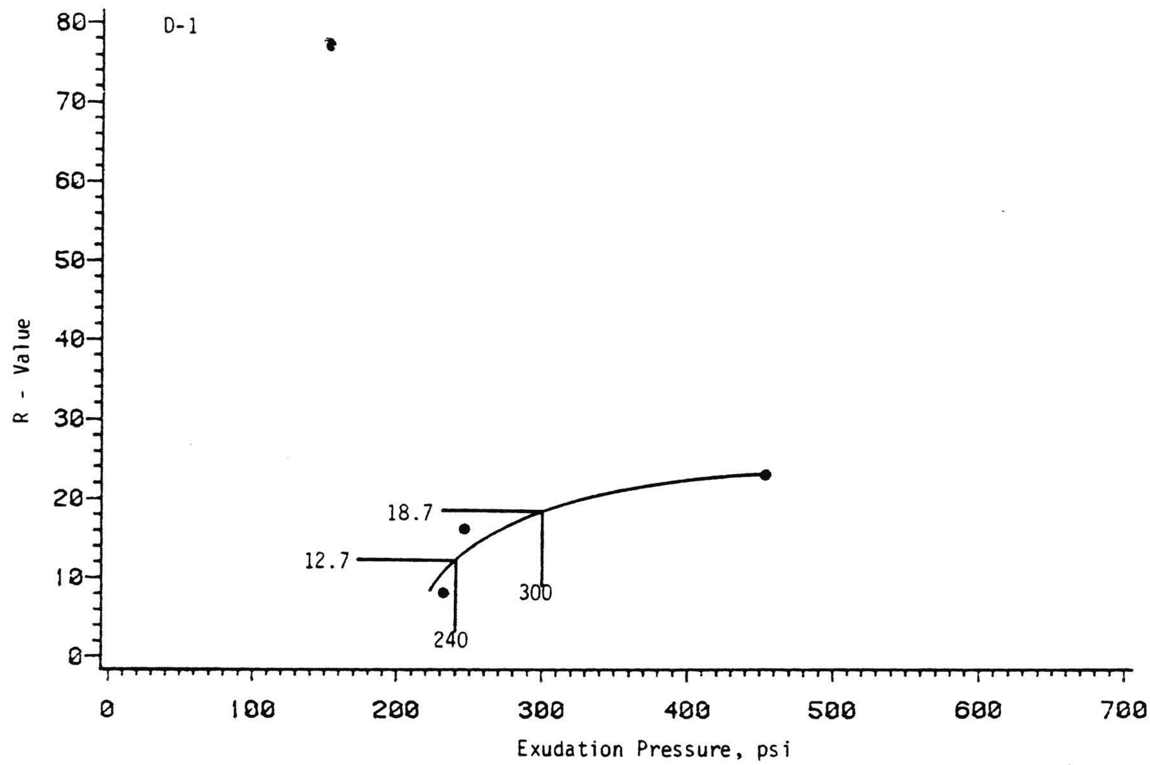


Figure A-4 a), b)
Hveem Stabilometer (R-value) Results



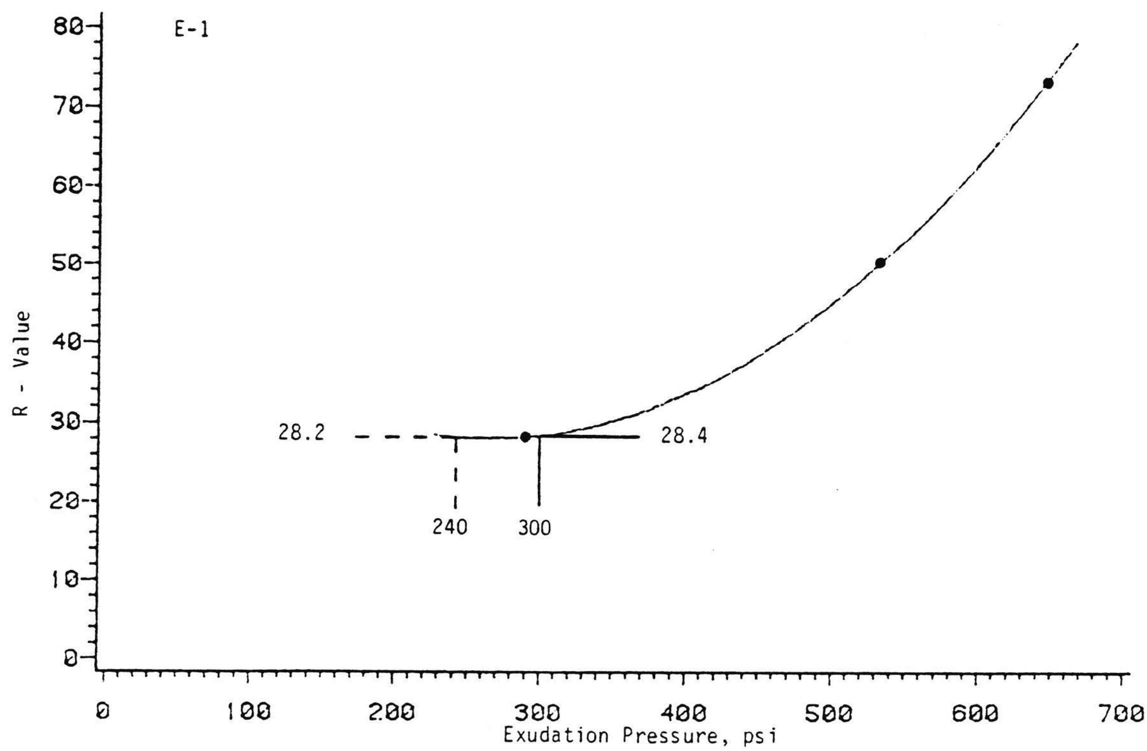


Figure A-4 c), d)
Hveem Stabilometer (R-value) Results

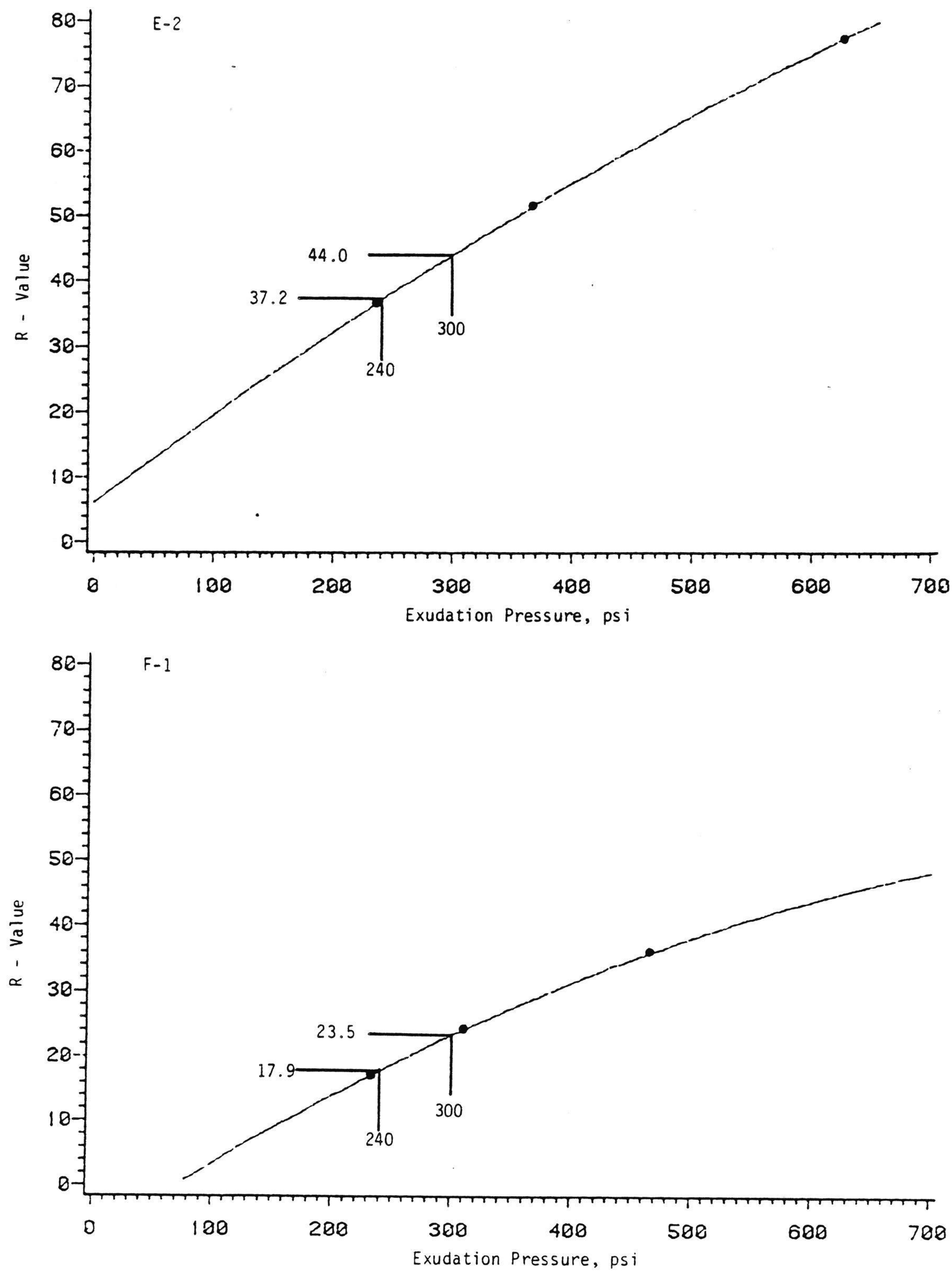
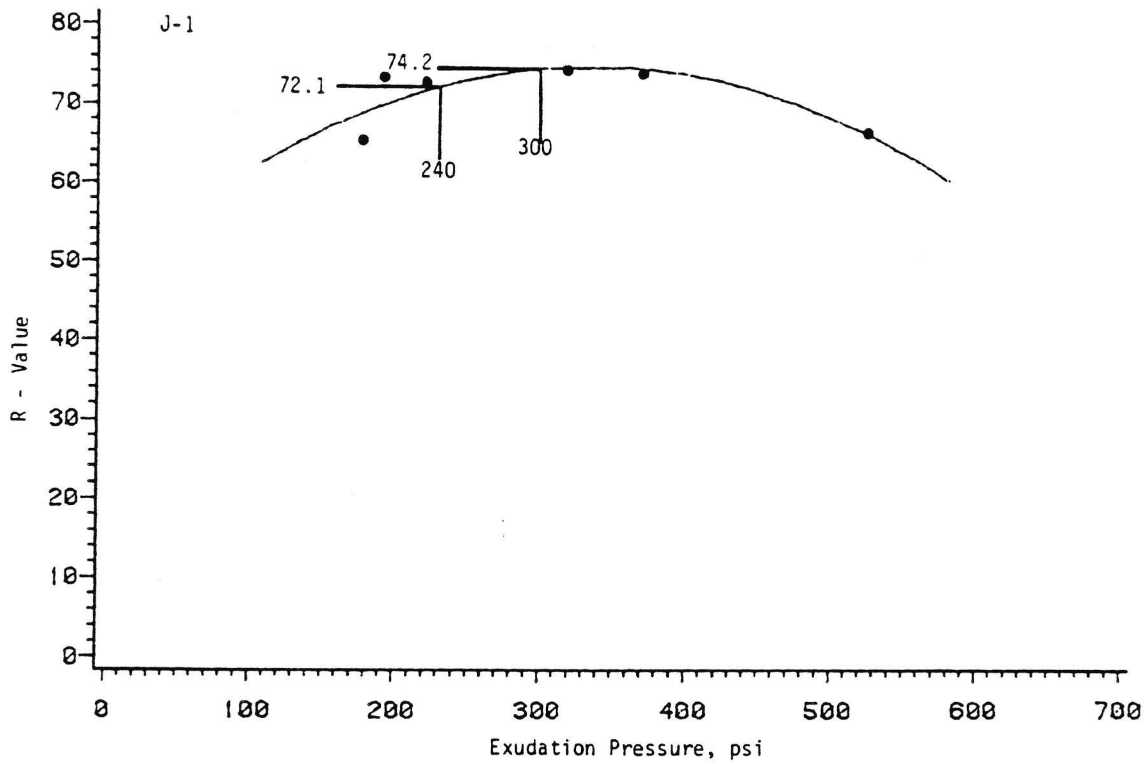


Figure A-4 e), f)
Hveem Stabilometer (R-value) Results



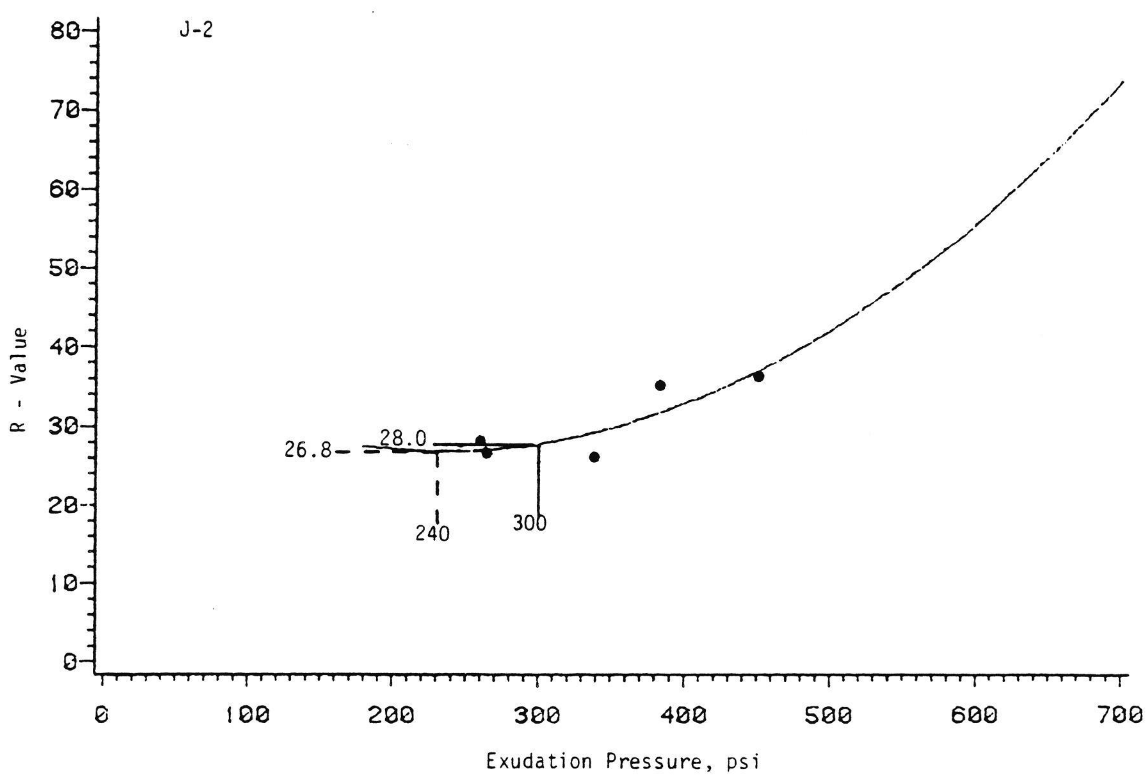
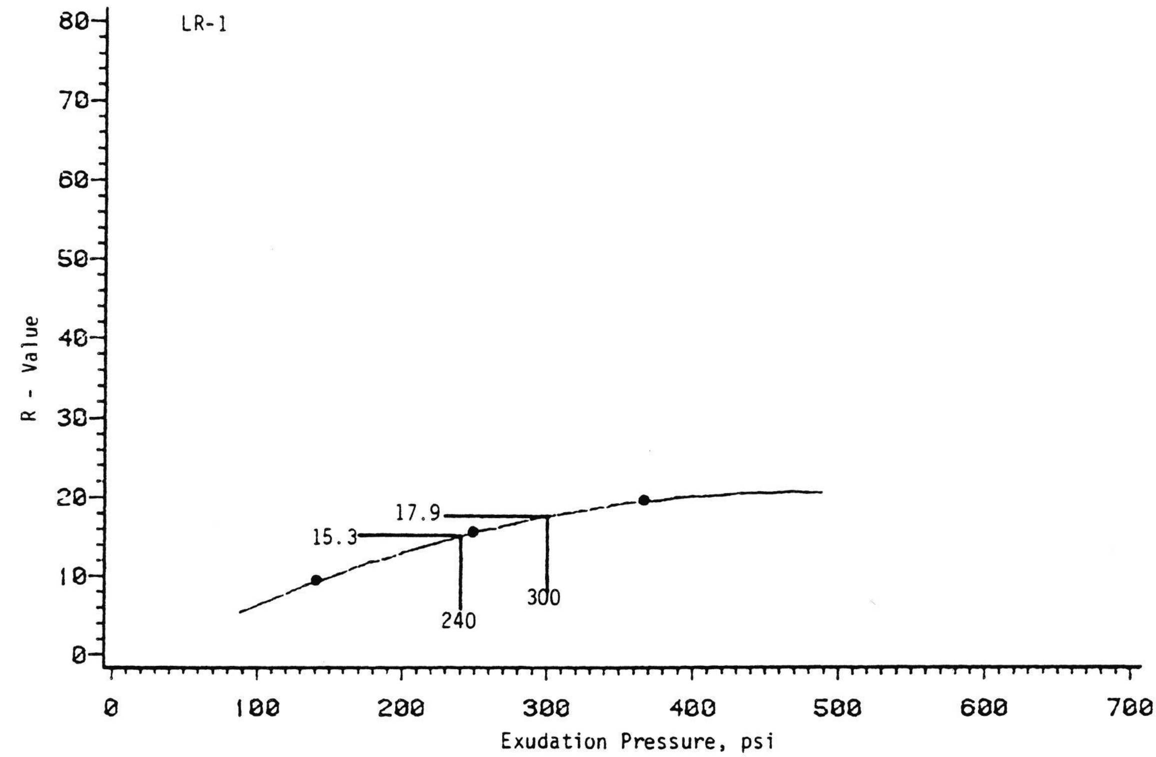


Figure A-4 g), h)
Hveem Stabilometer (R-value) Results



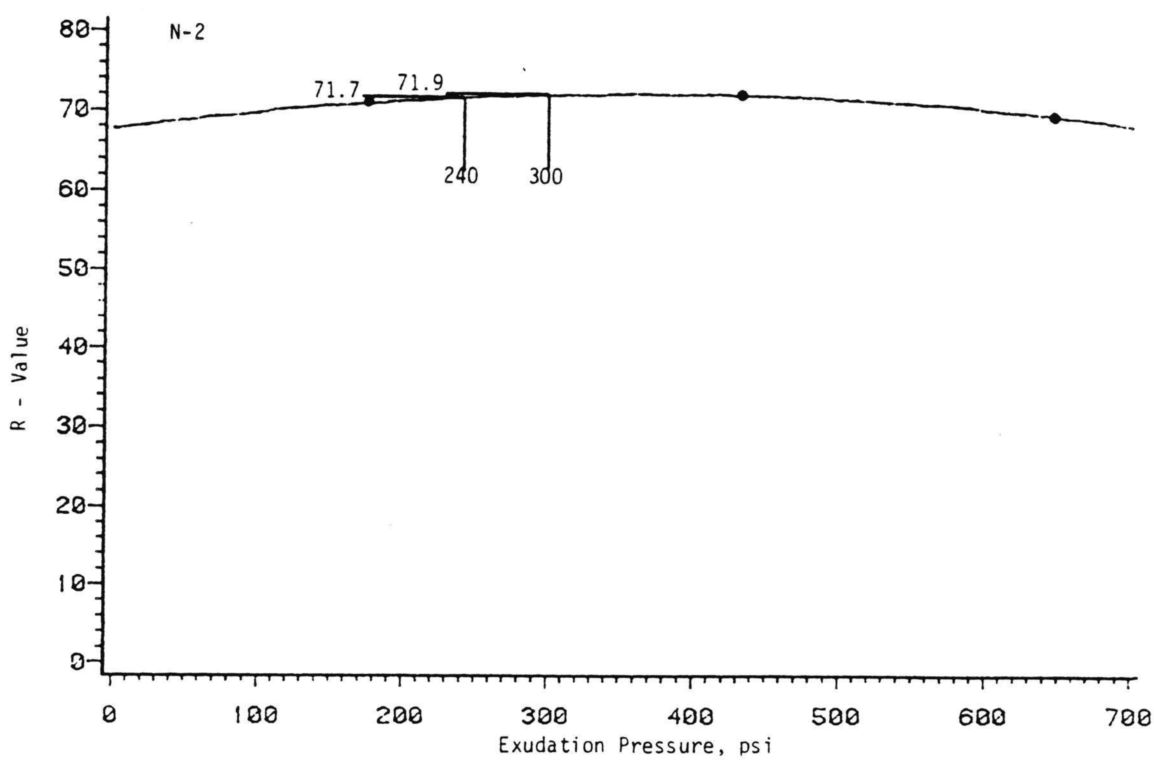
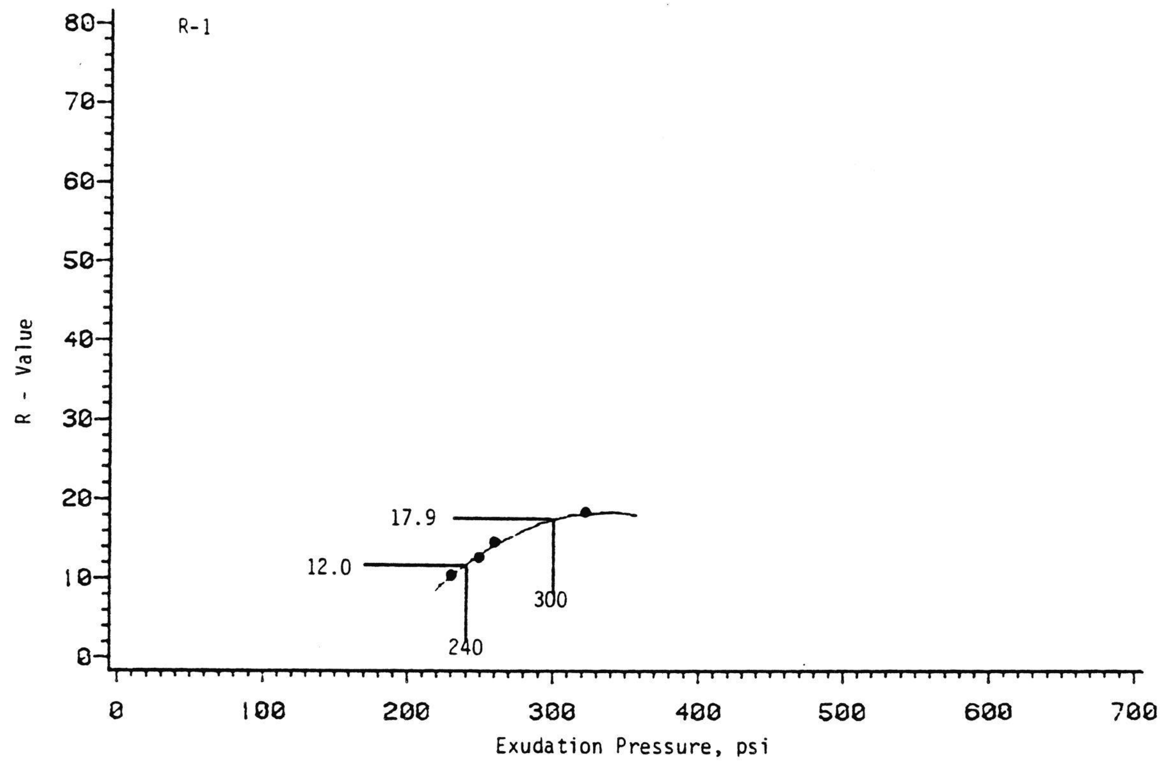


Figure A-4 i), j)
Hveem Stabilometer (R-value) Results



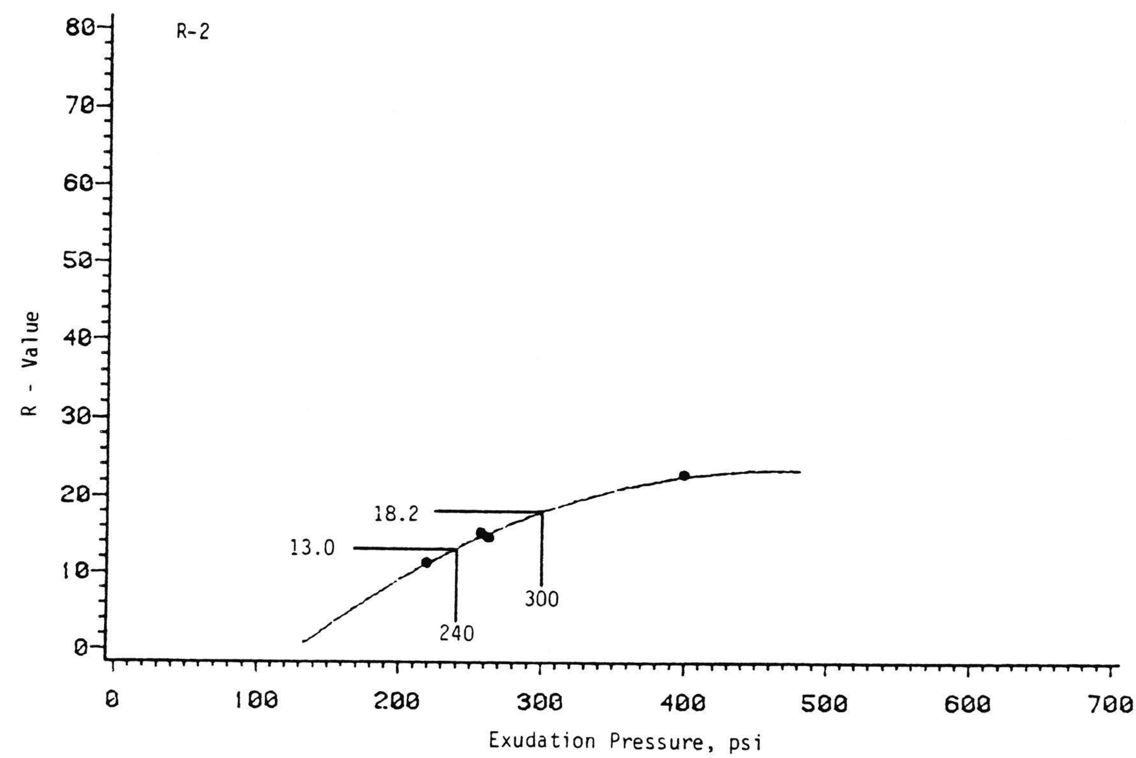
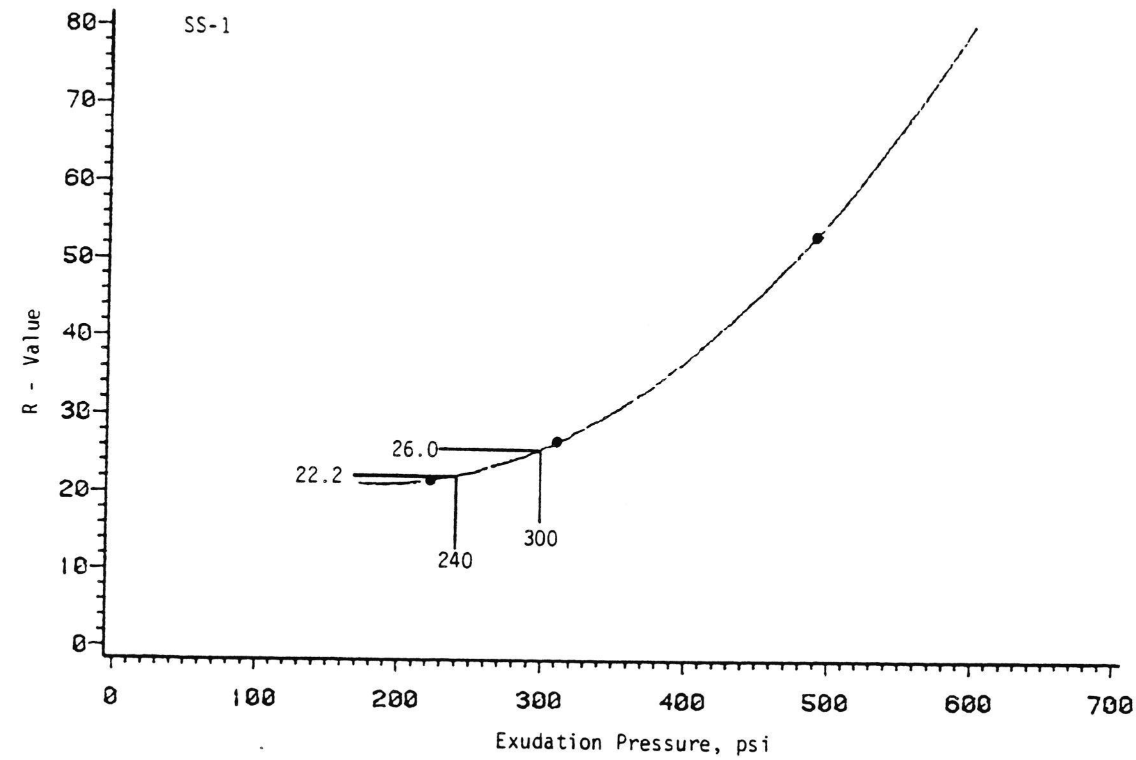


Figure A-4 k), l)
Hveem Stabilometer (R-value) Results



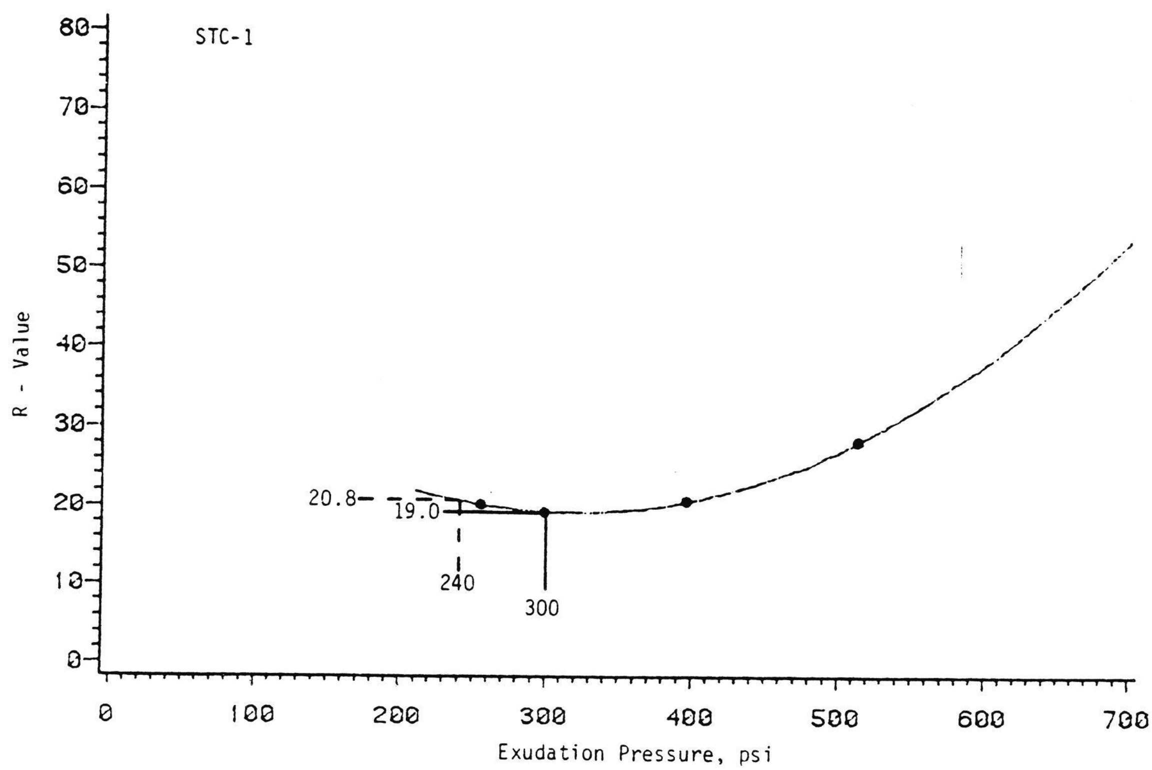


Figure A-4 m), n)
Hveem Stabilometer (R-value) Results

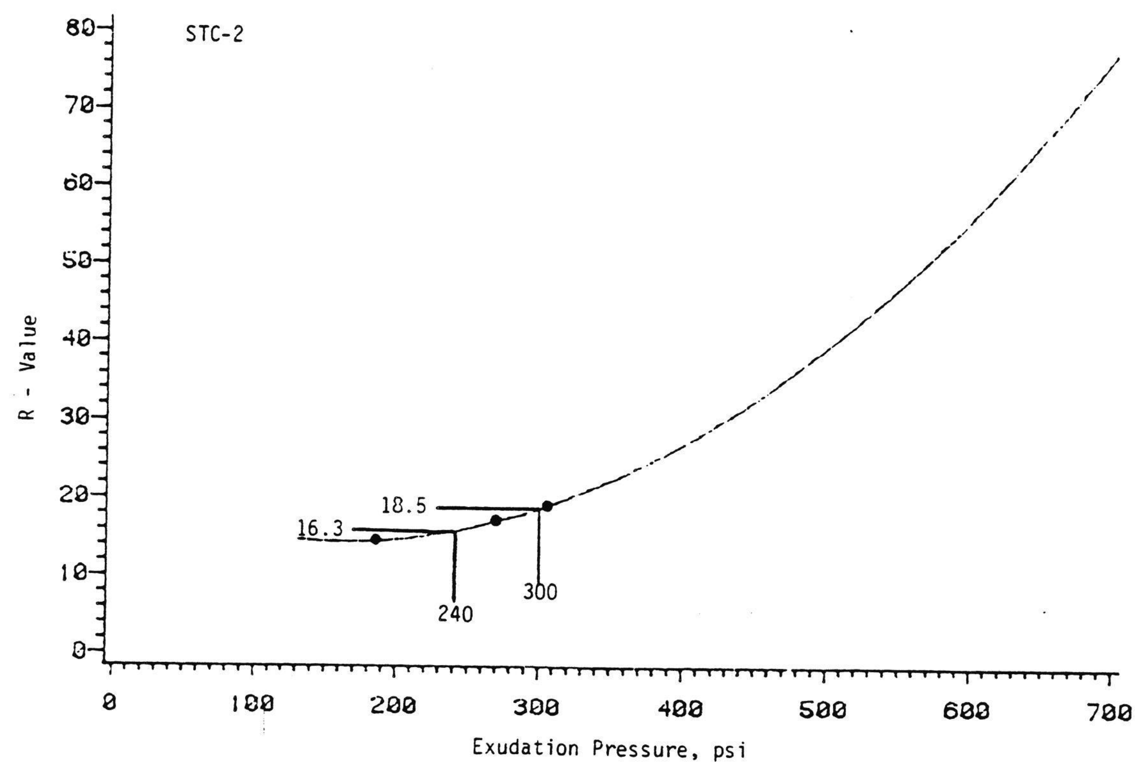


Figure A-4 o)
Hveem Stabilometer (R-value) Results

APPENDIX B

Appendix B includes the testing procedure and sample preparation of Resilient Modulus (M_R). Taken from the "Suggested Method Of Test For Resilient Modulus of Subgrade Soils". Prepared for Idaho Transportation Department, Division of Highways, Boise, Idaho; By Department of Civil Engineering, University of Idaho at Moscow, Idaho.

RESILIENT MODULUS OF SUBGRADE SOILS TEST PROCEDURE

RESILIENT MODULUS OF SUBGRADE SOILS

1. Scope

   1.1. This method covers the procedure for preparing and testing untreated soils for determination of dynamic elastic modulus under conditions that represent a reasonable simulation of the physical conditions and stress states of subgrade materials beneath flexible pavements subject to moving wheel loads.

2. Apparatus

   2.1. Resilient Modulus Mold and Collar (Figure B-1a), and compaction piston (Figure B-1b) capable of molding specimens of 6 $\pm$ 0.05 in. height by 2.75 $\pm$ 0.005 in. diameter.

   2.2. Compression Testing Machine with a minimum capacity of 10,000 lbs. with a loading rate of 0.05 in/min. and capable of maintaining load for up to 10 minutes.

   2.3. Material Test System 810, with a console equipped as follows: 5110 oscilloscope, 430 digital indicator, 410 digital function generator, 417 counter panel, 442 controller, 413 master control panel, 431 recorder. The console should be properly connected to a 22 kip load frame equipped with a 5.5 kip load cell.

   2.4. A compressor and pressure gauges and controls capable of applying air pressure of up to 20 $\pm$1 psi. Air hoses to connect compressor to gauges and gauges to chamber.

   2.5. Compression test chamber to accomodate 6 in. x 2.75 in. diameter specimen confined in a rubber membrane. Chamber must be equipped with a freely moving axial load piston with a 1 in. travel adapted to fit a 5.5 kip MTS load cell.

2.6. Porous stones, 2.75 in. diameter; rubber membrandes, 2.80 X 10 X 0.005 in.

2.7. Balance, 5000-g capacity, accurate 0.1gm.

2.8. Miscellaneous equipment, including tamping rod, knives, petroleum jelly, water content tares, rubber bands, triaxial membrane jacket, extrusion device, MTS plotter paper, etc.

3. Preparation of Soil Specimens

3.1. The test may be run for any desired density of a given soil sample. The volume of the soil specimen is 0.0206 ft^3. The desired density is attained by controlling the weight of the soil sample used in molding.

$$\text{density (pcf)} = \frac{\text{weight (lb)}}{0.0206 \text{ ft}^3}$$

Obtain a 1600-g sample of soil passing the 1/2" seive. Mix sample with water to desired water content. Perform a moisture content determination with a 200-g sample. Cover the remaining sample and allow to stand overnight for uniform moisture distribution.

3.2. The specimen is molded in three layers, each layer containing 1/3 of the weight necessary to obtain the desired density. Divide the sample into 3 equal 1/3 portions so that each portion is 1/3 of the weight necessary for the desired density. Cover two of the 1/3 portions and place the remaining 1/3 potion into the greased and assembled resilient modulus mold, collar, and base plate. Rod the sample 25 times. Adjust the collar on the compaction piston so that the top edge of the collar is aligned with the upper reference mark on the compaction piston. Grease the round edge of the teflon disk on the compaction piston and place the piston into the mold. Place the mold with soil and compaction piston on the compression testing machine. Apply load displacing the compaction piston until the top edge of the compaction piston collar is aligned with the top edge of the mold collar. Stop the displacement of the compaction piston, but maintain stationary load pressure for approximately 5 minutes. Release the load.

3.3. Mold the remaining two layers of the specimen as in 3.2. For each layer, add a 1/3 portion of the soil sample, tamp the surface 25 times, adjust the collar to the middle and lower reference marks, grease the edge of the teflon disc and compact.

Note 1 - When removing the sample after compacting a layer, rotate the compaction piston to shear any bonds between the soil and teflon disk. If vacuum between the soil and the compaction piston prevents removal of the compaction piston, the tamping rod may be used to remove the plug in the teflon disk by pushing down on the plug with the tamping rod while pulling up on the compaction piston.

Note 2 - Thoroughly scarify the top edge of the compacted layer with a knife so that a good bonding between the two layers can be achieved.

4. Sample Extrusion

4.1. The sample extrusion device must be capable of applying a concentric uniformly distributed load against the end of the sample in order to remove the sample without damage or deformation. Install a rubber membrane in the triaxial membrane jacket and apply a vacuum on the jacket. Remove the sample from the mold and place into the jacket.

Determine the weight of the compacted specimen to the nearest gram. Measure the height and diameter to the nearest 0.02 in. (0.5 mm).

5. Resilient Modulus Determination Using Material Test System 810

5.1. <u>System Set-Up</u>

The Material Test System 810 (MTS) may be used in the determination of the resilient modulus. The console properly connected to a 22 kip load frame with a 5.5 kip load cell is adequate. Provision must be made for the application of a 2.0 second load/release cycle such that application of the load takes 0.1 second of the 2.0 second load/release cycle.

5.2. <u>MTS Console Settings</u>

5.2.1. Digital Function Generator 410

A. Rate 1 = 1×10^{-4}

B. Rate 2 = 1×10^{-1}

C. STOP AT ZERO - In

D. HOLD AT BRKPT - In

E. RAMP - In

F. INVERT - In

G. BREAKPOINT: Percent dial - 100

Selector dial - NORMAL (LOCAL side)

H. CONTROL MODE dial: REMOTE = load/release cycle signal - on; LOCAL - load/release cycle signal - off.

5.2.2. Counter Panel 417

A. Counter input - oscillator

B. Count multiplier - X1

C. Counter reset - 000000

5.2.3. Controller 442

A. LOAD (range 4) - ±10%

B. STROKE (range 3) = ±20%

C. SET POINT - The set point dial should be adjusted so that the load cell on the load frame remians stationary while the hydraulic pump is running.

D. Meter - 0

5.3. <u>Pre-Test Adjustments</u>

5.3.1. LOAD (irrespective of load cell position) - Obtain zero readout on DIGITAL INDICATOR (ch 1) with zero dial on the A. C. CONDITIONER (stroke) located behind CONTROLLER panel. Switch HYDRAULIC PRESSURE control on the MASTER CONTROL PANEL to LOW (reset INTERLOCK RESET on CONTROLLER and RESET on MASTER CONTROL PANEL before turning the hydraulic pump on.)

5.3.2. Position load cell for light contact with mounted sample using SET POINT dial on CONTROLLER panel. DIGITAL INDICATOR (ch 1) should give a slight reading to indicate a slight load has been applied. Load cell movement is controlled with the SET POINT dial.

6. Performing the Resilient Modulus Test

6.1. There are two procedures for testing soils: one procedure for testing cohesive soils and the other procedure for testing granular soils. Cohesive soils are defined as having classifications A-2-6, A-2-7, A-6, and A-7 and granular soils are defined as having all other classifications according to AASHTO M145.

Each specimen must undergo sample conditioning prior to testing. Two hundred load/release cycles constitute one phase of conditioning/testing and load adjustments are allowed within the first 50 cycles of a phase for a given set of conditioning.

6.2. Cohesive Soils

6.2.1. Sample Conditioning. Sample condition of cohesive soils consists of 5 phases of load release cycles as follows:

σ_3, psi	σ_d, psi	P_d, lb*
6	1	6
6	2	12
6	4	24
6	8	48
6	10	60

* $P_d = \sigma_d$ X Area

The deviator load is set with the SPAN1 control on the CONTROLLER. It is suggested that the SPAN1 setting be recorded during conditioning and utilized as an initial setting for the same conditions during resilience resting. It is essential that the load return to zero (or near zero) during the release portion of the load/release cycle. Any zeroing of the load should be done between phases with either the SET POINT control or the zero dial on the A.C. CONDITIONER. DO NOT ATTEMPT TO ZERO THE LOAD WHILE THE SAMPLE IS UNDERGOING CYCLIC LOADING.

6.2.2. Resilience Testing. Resilience testing of cohesive soils consists of 15 phases of load/release cycles, as follows:

σ_3, psi	σ_d, psi	P_d, lb.
6	1	6
3	1	6
0	1	6
6	2	12
3	2	12
0	2	12
6	4	24
3	4	24
0	4	24
6	8	48
3	8	48
0	8	48
6	10	60
3	10	60
0	10	60

The SPAN1 setting for a given set of conditions during resilience testing should be about the same as for sample condition for the same σ_3 and P_d.

6.2.2.1. Begin the recorded resilient modulus test. Record the vertical recovered deformations using the 431 RECORDER on the MTS. Record the peak deviator load of the final 50 cycles of each phase. Record all data on a form for cohesive soils such as that shown in Figure B-2.

6.2.2.2. At the end of testing, disassemble the triaxial test chamber, remove the specimen from the rubber membrance and use the entire specimen for a water content determination.

6.3. Granular Soils

6.3.1. Sample Conditioning. Sample conditioning for granular soils consists of 6 phases of load/release cycles, as follows:

σ_3, psi	σ_d, psi	P_d, lb
5	5	30
5	10	60
10	10	60
10	20	120
15	15	90
15	20	120

The deviator load is set with the SPAN1 control on the CONTROLLER. It is suggested that the SPAN1 setting be recorded during conditioning and utilized as an initial setting for the same conditions during resilience testing. It is essential that the load return to zero (or near zero) during the release portion of the load/release cycle. Any zeroing of the load should be done between phases with either the SET POINT control or the zero dial on the A. C. CONDITIONER. DO NOT ATTEMPT TO ZERO THE LOAD WHILE THE SAMPLE IS UNDERGOING CYCLIC LOADING.

6.3.2. Resilience Testing. Resilience testing of granular soils consists of 25 phases of load/release cycles, as follows:

σ_3, psi	σ_d, psi	P_d, lb
20	1	6
20	2	12
20	5	30
20	10	60

20	20	120
15	1	6
15	2	12
15	5	30
15	10	60
15	20	120
10	1	6
10	2	12
10	5	30
10	10	60
10	15	90
5	1	6
5	2	12
5	5	30
5	10	60
5	15	90
1	1	6
1	2	12
1	5	30
1	7.5	45
1	10	60

The SPAN1 setting for a given set of conditions during resilience testing should be about the same as for sample conditioning for the same σ_3 and P_d.

6.3.2.1. Begin the recorded resilient modulus test. Record the vertical recovered deformations using the 431 RECORDER on the MTS. Record the peak deviator load of the final 50 cycles of each phase. Record all data on a form for granular soils such as that shown in Figure B-3.

6.3.2.2. At the end of testing, disassemble the triaxial test chamber, remove the specimen from the rubber membrane, and use the entire specimen for a water content determination.

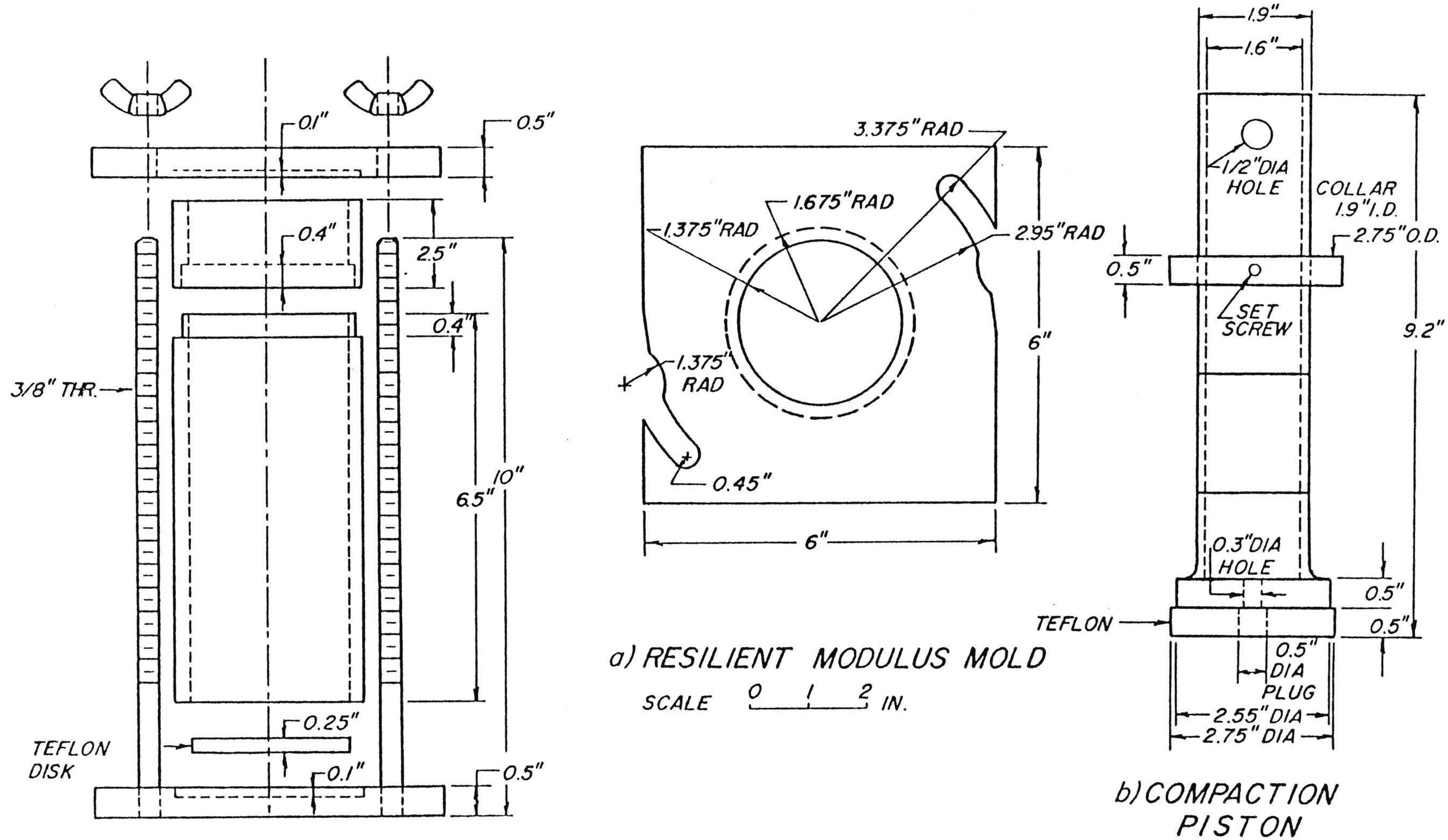


Figure B-1 Appratus for Static Compaction

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

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

*Not applicable for externally mounted LVDT's.

Figure B-2 Form for Resilient Modulus Tests on Cohesive Soils

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

Soil Specimen Measurements
Diameter
- Top ______
- Middle ______
- Bottom ______
- Average ______

Membrane Thickness ______
Net Diameter ______
Ht Specimen + Cap + Base ______
Ht Cap + Base ______
Initial Length, L_o ______
Inside Diameter of Mold ______

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

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

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

Constants
Vertical LVDT ______
Load Cell ______

Water Content After
Resilience Testing % ______
Comments ______

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

*Not applicable for externally mounted LVDT's.

Figure B-3 Form for Resilient Modulus Tests on Granular Soils