PAVEMENT TESTING AND ANALYSIS OF HEAVY HAULS FOR SR 12

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16. Abstract

The pavement structure of SR 12 between Montesano and Elma, Washington was evaluated for the proposed heavy loads associated with construction of the Satsop power plant. Information used in evaluating SR 12 resulted from two sources which included field studies conducted by the Washington State Department of Transportation and development of various material strength parameters by the University of Washington. These data were used to model the pavement structure as a layered elastic system. By use of this analysis procedure, the stresses, strains and deflections were estimated for the expected range of loading conditions. The results indicate that the most probable amount of damage (fatigue and rutting) expected for the non-cement treated base structural sections is less than one to two percent of available pavement life for the "expected" loading condition. An increase in either or both the trailer wheel load and pavement temperature will act to produce greater losses in pavement life. It is estimated that the tensile stresses in the cement treated base may exceed the tensile strength of this material.

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TABLE OF CONTENTS

CHAPTER I. INTRODUCTION

The necessity to evaluate the pavement structure of SR 12 between Montesano and Elma, Washington, is due to the planned heavy loads associated with moving four steam generators and two reactor vessels to the Satsop nuclear power plant. It is understood that other possible haul routes are being considered but hauling these specific loads along the SR 12 route is of primary consideration. The planned operation would include off-loading each steam generator and reactor vessel at a barge slip to be constructed on the Satsop River near Montesano, hauling over an access road (to be constructed) which connects with SR 12 and then along SR 12 to Elma. At Elma the haul route will depart SR 12 and change to a county maintained road.

The pavement evaluation described in this report was made at the request of the Washington State Department of Transportation (WSDOT) and examines only those portions of the proposed hauls which include travel along SR 12 (a distance of about nine miles). The conclusions and recommendations resulting from this study will then be used by WSDOT in evaluating the issuance of a haul permit to Ebasco Services, the prime contractor at the Satsop nuclear power plant.

STUDY APPROACH

The available data and other information which were used in evaluating SR 12 for the planned heavy hauls resulted from two primary sources. The first source was field studies conducted by WSDOT and included the following:

1. Soil borings, samples of granular materials, and coring of asphalt concrete pavement (ACP) and cement treated base (CTB) pavement;
2. Benkelman Beam deflection survey; and
3. Plate bearing tests.

The soil borings, granular base and fill samples, ACP and CTB cores were obtained during January, 1979. The Benkelman Beam deflection survey preceded the soil borings and the data used in selecting soil boring locations. Plate bearing tests were performed during April, 1979, thus completing the WSDOT field studies. Additionally, Mr. Dick Stubstad of Dynatest Consulting, Inc., Ojai, California, used the Falling Weight Deflectometer (FWD) at selected stations along the SR 12 haul route to obtain deflection information and estimate modulus relationships for the pavement layers. This work was accomplished during May, 1979.

The second source of data was developed at the University of Washington (UW) Department of Civil Engineering pavement materials laboratory in Seattle. The overall goal of the laboratory program was to develop general strength parameters for the primary structural materials contained in the SR 12 cross sections and specifically to develop the required elastic parameters to enable modeling of the pavement structure as a layered elastic system. Modeling SR 12 as a layered elastic system allows the results of various loading configurations to be evaluated in terms of the resulting stresses, strains and deflections in each pavement layer of interest.

The kinds of data obtained in the UW laboratory study were primarily for the ACP and CTB cores and granular samples which were provided by WSDOT. The details of these tests will be described in greater detail in the following chapter and included the following:

1. ACP cores
   (a) Resilient modulus
   (b) Marshall stability and flow
   (c) Bulk and maximum specific gravity
2. CTB cores
   (a) Resilient modulus
   (b) Indirect tensile strength
3. Granular samples
   (a) Resilient modulus (repeated load triaxial and indirect tensile)
   (b) California Bearing Ratio (CBR)
   (c) Gradation analysis

As stated previously, some of the above tests were used to model the pavement sections which were treated as layered elastic systems. Other tests were performed at the request of others.

REPORT

The chapters which follow will describe the heavy haul pavement evaluation performed for SR 12 in this study. Chapter II will be used to describe the field and laboratory material investigation including test results. Chapter III will present the analysis which includes descriptions of the load configurations, pavement cross sections, pavement response predictions and use of these response predictions with appropriate failure criteria. The results of this analysis were used to estimate the pavement life which may be consumed by moving these heavy hauls along the proposed SR 12 route.

Pavement distress due to a slope stability failure along the proposed haul route was not considered part of the study and thus was not analysed for this report. Although, due to the highly variable subgrade soils along the proposed haul route, a slope stability failure should be treated as a possible failure mode due to the expected heavy hauls.

CHAPTER II. FIELD AND LABORATORY MATERIAL INVESTIGATION

FIELD EXPLORATION

The field exploration portion of this study was performed by WSDOT and included three kinds of data. First, a Benkelman Beam deflection survey was performed early in the study so that soil boring, disturbed granular samples and ACP and CTB coring locations could be selected. Criteria used for selecting these locations included high pavement surface deflections among other considerations. It is significant that essentially all Benkelman Beam recorded surface deflections were considered to be low (preferable condition). Also preceding the selection of the final boring and coring locations was a pavement condition survey which indicated that little surface distress was present. The only exception to this was near the Satsop River where a segmental overlay had been placed. This overlay area is not considered to be significant to the evaluation since it is understood that this portion of SR 12 will be bypassed for the heavy hauls in order to cross the river.

Following the Benkelman Beam Survey, soil borings, granular base and fill (loose) samples and ACP and CTB pavement cores were obtained at selected sites. Twenty-four soil borings were made by use of a hollow stem auger. The average depth for these borings was about 22 feet and spaced along the nine mile long SR 12 haul route. These borings were used to identify the soil types underlying this portion of SR 12 as well as obtain standard penetration blow counts and undisturbed and disturbed soil samples. Figure 1 shows the general soil types encountered as described by use of the soil borings and the locations where core samples were obtained.

Examination of Figure 1 reveals a soil profile which is quite variable with respect to the kinds and strengths of the various soil layers encountered. Delineation between the various soil layers has been shown where possible. Considering that the portion of SR 12 evaluated lies principally in the Satsop River Valley, the variation in soil layers should be expected since these layers are of fairly recent deposition. Even though this observed variation is not unexpected, it does make the task of evaluating SR 12 with respect to the planned heavy hauls more difficult.

The majority of the soil samples were retained by the WSDOT Materials Division for testing and some of the undisturbed soil samples were provided to UW. All of the ACP and CTB cores were delivered to UW for laboratory testing.

During April, 1979, WSDOT personnel obtained plate bearing test data at selected SR 12 locations. These locations (stations) were selected so that this kind of information would be obtained throughout the length of the proposed haul route and also at stations which appeared to be of a "critical" nature.

The plate bearing test included the use of both 12-in and 24-in diameter steel plates. The plates are seated on the pavement surface and carefully leveled with sand when necessary. A jacking type of load was applied to each plate and the corresponding pavement surface deflections were measured and recorded. The overall test procedure was conducted in accordance with ASTM Standard Method D1196 with the

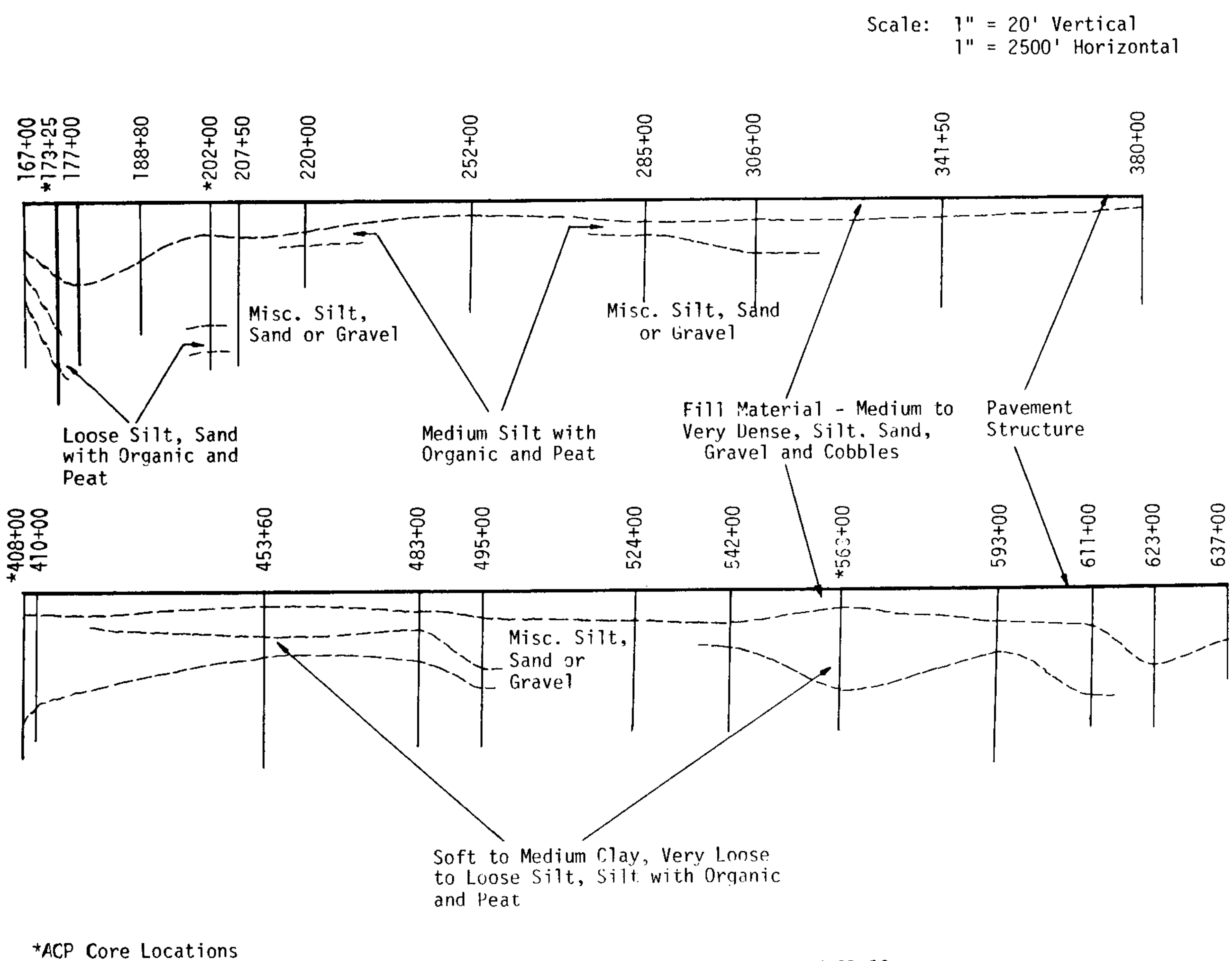

Figure 1. Generalized Soil Profile of Proposed SR 12 Haul Route.

exception of the plate sizes used. The detailed results from the plate bearing test will not be presented in this report but a brief summary of the results is listed in Table 1.

The information contained in Table 1 shows the maximum measured deflection for each of the two plates and the corresponding maximum load. The pavement temperature was also recorded and is listed in the table. The maximum deflection is the average of two dial gage readings which measured the deflection of each plate. Other deflection measurements were at one foot increments up to three feet from the plate. Thus, deflection basins were obtained for the plate bearing tests.

The maximum deflections shown in Table 1 range from a low of 0.005-in for the 24-in plate to a maximum of 0.100-in. The lower deflections listed occur at Stations 584+00 and 602+00. These stations contain 6-in thick CTB layers overlain by 3-in of ACP. Due to the high stiffness of the CTB layer, low deflections result when compared to the other locations (Stations 173+25 to 453+60). The principal structural layer for these other stations is 9-in of ACP. An overall indication of the variability of this data can be obtained by noting some of the significant differences which occur between the measured deflections at the same station for the outer wheelpaths (OWP) and the between wheelpath (BWP) test locations (a transverse distance of approximately three feet).

LABORATORY STUDY

Tests were performed on samples obtained from the field exploration phase of the study by the WSDOT Materials Division and the UW Department of Civil Engineering pavement materials laboratory. The laboratory

Table 1. Summary of Plate Bearing Tests - SR 12[1].

Station[2]	12" Dia. Plate			24" Dia. Plate		
	Max. Deflection (in.)	Max. Load (lbs.)	Pavement Temperature (°F)	Max. Deflection (in.)	Max. Load (lbs.)	Pavement Temperature (°F)
173+25 (OWP)[3]	0.033	19,600	62	0.048	27,400	63
173+25 (BWP)[4]	0.077	19,600	69	0.044	27,400	68
202+00 (OWP)	0.032	19,600	64	0.016	27,400	64
202+00 (BWP)	0.044	19,600	72	0.047	27,400	69
341+50 (OWP)	0.080	19,600	80	0.035	27,400	83
341+50 (BWP)	0.054	19,600	85	0.036	27,400	87
408+00 (OWP)	-	-	-	0.026	39,000	62
408+00 (BWP)	0.051	19,600	64	-	-	-
453+60 (OWP)	0.084	19,600	74	-	-	-
453+60 (BWP)	-	-	-	0.100	35,200	75
584+00 (OWP)	0.006	19,600	59	0.005	41,000	58
584+00 (BWP)	0.012	41,000	57	0.006	37,200	56
602+50 (OWP)	0.006	19,600	62	0.010	37,200	65
602+50 (BWP)	0.012	19,600	63	0.006	37,200	60

[1] Data provided by WSDOT

[2] All measurements made in eastbound outside lane

[3] Outside wheelpath

[4] Between wheelpaths

testing reported herein includes only the data and other information obtained from the UW study.

To enable the proper characterization of the materials contained in the various cross sections along the SR 12 haul route, several kinds of laboratory tests were performed. Some of these tests were necessary to model the elastic response of the pavement sections, other tests were made at the request of others.

In general, the tests performed by UW personnel can be categorized for three material groups: ACP cores, CTB cores and loose granular materials of which all groups were sampled by WSDOT. A few undisturbed subgrade soil samples were also provided to UW but these samples after initial examination were not incorporated into the laboratory testing program. These subgrade samples were not used due to the complex nature of the soil profile and the stress sensitivity of the resilient modulus for these materials. Thus, proper laboratory characterization of the subgrade materials would have required an extensive number of samples. Additionally, a visual examination of the available samples indicated that preparation for testing would be difficult and the results uncertain at best. Equally important was the fact that the required laboratory equipment was not available to properly test the size of subgrade samples available for resilient modulus determination; although, this equipment is available elsewhere.

The stations and specific samples selected for testing were felt to represent the more important sections of SR 12 which should be studied in detail. Some of the ACP cores provided by WSDOT were not included in the initial laboratory testing program so that these materials could be used at a later date if future, additional testing is required.

The sections which follow will be used to describe the kinds of tests and results for the three material groups studied.

ACP CORES

Figure 2 shows the testing sequence used for selected ACP cores with the two primary material characterization tests being resilient modulus and Marshall stability and flow. The following sections further describe the tests and results.

ACP Core Sample Preparation

The ACP cores as received were generally 9-in high with a 4-in diameter. These cores were then sawed and trimmed with a rotary laboratory asphalt concrete saw which yielded specimens ranging in height from about 1.5 to 2.6-in with a 4-in diameter. The location of saw cuts were made so that the original horizontal joints between lifts did not occur within any cut specimen. Following sawing, the thickness of each specimen was measured at three equally spaced points along the circumference with the average being recorded as the specimen thickness (Tables 2 and 4).

Bulk Specific Gravity

The bulk specific gravity for the ACP specimens was determined in accordance with ASTM Standard Method D2726 (Table 4). The bulk specific gravities are fairly consistent at each station and are not significantly different between stations. The exception to this is Station 563+00 which was ACP over a CTB layer.

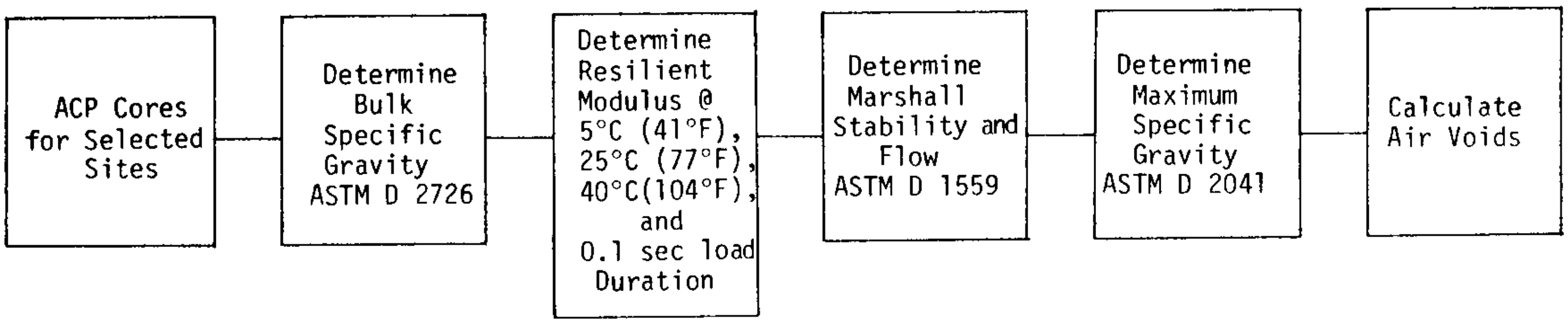

Figure 2. Test Sequence for ACP Cores from SR 12.

Resilient Modulus

The resilient modulus of the ACP specimens was determined in accordance with the proposed draft of an ASTM standard test method. The draft is entitled "Indirect Tensile Test Method for Resilient Modulus of Bituminous Mixtures" and is included as Appendix A.

Resilient modulus provides an estimate of the modulus of elasticity for asphalt concrete at a specific temperature. The temperature is quite important since the resilient modulus of these materials is a function of temperature.

A pneumatic pulse loading testing machine with a loading cycle of 0.5 Hz was used to determine resilient modulus for selected ACP specimens. The specimens were tested at temperatures of 5°C (41°F), 25°C (77°F) and 40°C (104°F); thus, the overall pavement response could be examined for a range of ACP temperatures. A dynamically applied vertical load of 100-lbs was applied through two steel loading strips each 0.75-in wide and covered with thin rubber membranes. The horizontal deformations resulting from the dynamic loads were measured using two Statham UC-3 transducers. The applied load and horizontal specimen deformations are used to calculate the resilient modulus as described in more detail in Appendix A. Additionally, vertical deformations of selected specimens were measured with a dial gage, thus providing data for calculation of Poisson's ratio. The results of the resilient modulus testing are shown in Tables 2 and 3 and Figures 3 through 6.

Table 2 lists individual specimen test results obtained from ACP cores at four stations. The specimen thickness, resilient modulus at the three test temperatures and Poisson's ratio are shown. The sample

Table 2. Summary of Resilient Modulus Results for SR 12 Cores.

Sample No.	Thickness (in.)	Resilient Modulus (psi)			Poisson Ratio
		5°C	25°C	40°C	
173-1-1	2.104	1,640,700	466,800	96,600	0.32
173-1-2	2.196	1,369,400	318,200	87,200	
173-1-3	2.361	1,357,700	435,000	94,100	
173-1-4	2.182	1,937,500	450,900	124,800	
173-3-1	2.185	1,854,200	458,800	108,300	
173-3-2	2.671	1,125,900	288,500	75,200	
173-3-3	2.340	1,285,100	424,000	72,800	
173-3-4	1.680	1,543,400	360,600	115,400	
202-1-1	1.978	1,520,300	437,000	120,400	0.31
202-1-2	2.420	1,242,700	335,300	63,800	
202-1-3	2.299	1,308,000	365,700	109,900	
202-1-4	2.622	1,271,400	269,400	63,600	
202-3-1	1.857	851,400	149,600	67,500	
202-3-2	2.251	828,000	80,700	47,800	
202-3-3	2.381	1,400,100	144,100	54,700	
202-3-4	2.036	1,628,100	327,500	80,300	
408-1-1	2.541	918,400	133,100	47,500	0.32
408-1-2	2.461	1,354,600	375,100	81,300	
408-1-3	1.574	1,597,600	439,200	123,100	
408-1-4	1.930	1,925,300	561,900	130,900	
408-3-1	2.087	1,597,400	290,300	57,000	
408-3-2	2.279	1,406,500	272,900	63,600	
408-3-3	2.651	1,760,500	409,000	104,800	
408-3-4	2.110	1,843,300	460,800	129,500	
563-1-1	2.412	1,612,500	425,100	91,500	0.32
563-1-2*	2.170	7,000,000	7,000,000	7,000,000	
563-1-3*	2.133	7,000,000	7,000,000	7,000,000	
563-2-1	2.472	1,348,600	343,000	67,800	
563-2-2*	2.071	7,000,000	7,000,000	7,000,000	
563-2-3*	2.110	7,000,000	7,000,000	7,000,000	

*CTB Cores

Table 3. Statistical Summary of Resilient Modulus Results for SR 12 Cores.

Station-Sample Location	Resilient Modulus (psi)								
	5°C			25°C			40°C		
	Mean	Std. Dev.	CV(%)	Mean	Std. Dev.	CV(%)	Mean	Std. Dev.	CV(%)
173-1	1,576,300	274,000	17.4	417,700	67,600	16.2	100,700	16,600	16.5
173-3	1,452,100	318,500	21.9	383,000	74,900	19.6	92,900	22,100	23.8
	1,514,200	282,900	18.7	400,300	68,600	17.1	96,800	18,600	19.2
202-1	1,335,600	126,000	9.4	351,800	69,500	19.8	89,400	30,000	33.6
202-3	1,176,900	400,400	34.0	175,500	106,100	60.5	62,600	14,400	23.0
	1,256,300	287,600	22.9	263,600	125,600	47.6	76,000	26,100	34.3
408-1	1,449,000	424,100	29.3	377,300	180,300	47.8	95,700	38,800	40.5
408-3	1,651,900	192,900	11.7	358,300	91,300	25.5	88,700	34,400	38.8
	1,550,500	323,700	20.9	367,800	132,700	36.1	92,200	34,200	37.1
563-1	1,612,500	-	-	425,100	-	-	91,500	-	-
563-2	1,348,600	-	-	343,000	-	-	67,800	-	-
	1,480,500	186,600	12.6	384,100	58,000	15.1	79,700	16,700	21.0

number used in this table and other tables which follow are composed of three sets of numbers. The first three numbers correspond to the station where the ACP core was sampled. The second number indicates the sampling location transversally across the lane, e.g. "1" indicates the cores were obtained in the outside wheelpath and "3" indicates the cores were obtained between wheelpaths. Lastly, the third number indicates the location of a sawed specimen with respect to depth in the original core, e.g., "1" indicates the specimen was at the pavement surface, "2", the specimen was below the surface specimen, etc.

The individual test values reported in Table 2 are informative but the statistical summary of this data provided in Table 3 provides a better review of the resilient modulus data. Table 3 contains the mean, standard deviation and coefficient of variation for the specimens tested. Caution is warranted in using these statistical values due to the small sample size but they still provide information otherwise unavailable. The station-sample location numbers use a similar identification scheme as described for Table 2 with the exception that specimen depth is not shown. The average resilient modulus at 25°C (77°F) indicates that the SR 12 ACP generally lies in an adequate stiffness range indicative of good quality materials and construction. The exception occurs at Station 202+00 in that the specimens obtained for the between wheelpath condition exhibit variable and somewhat low resilient modulus values. This is reflected in the coefficient of variation for this sampling location. As a rule-of-thumb, coefficient of variations of less than 15 percent for <u>laboratory</u> compacted and tested resilient modulus test specimens should be considered good to excellent with respect to material and testing variability [1]. Coefficient of variations for cored ACP samples higher than 15 percent are not unexpected but values which exceed 60 percent are considered to be quite high. The mean and standard deviations listed in Table 3 were used in Chapter III to set reasonable confidence levels for modeling the pavement response to various load conditions.

Figures 3 through 6 show resilient modulus mean and range plotted as a function of temperature for the four sections. These curves were used to estimate the resilient modulus for various temperatures which were required for the analysis reported in Chapter III. The ranges shown in these figures represent the highest and lowest resilient modulus values obtained.

Marshall Stability and Flow

Marshall stability and flow determinations were made for selected ACP specimens in accordance with ASTM Standard Method D1559. Specimen preparation prior to testing included immersion in a water bath for 12 hours at 60°C±1°C. The test was conducted using a standard loading rate of 2-in per minute until failure occurred. The resulting stability value was corrected for specimen height.

The results of the Marshall tests are reported in Table 4. Included in this table are the specimen bulk specific gravity, maximum specific gravity, percent air and Marshall stability and corresponding flow. The observed stabilities are considered to be representative of good quality ACP with the possible exception of the results obtained for the Station 202+00 specimens. Generally, a stability of 750-lbs is considered to be a minimum value for high traffic highways [3]. The flow values are considered to be unusually high for asphalt concrete materials approximately

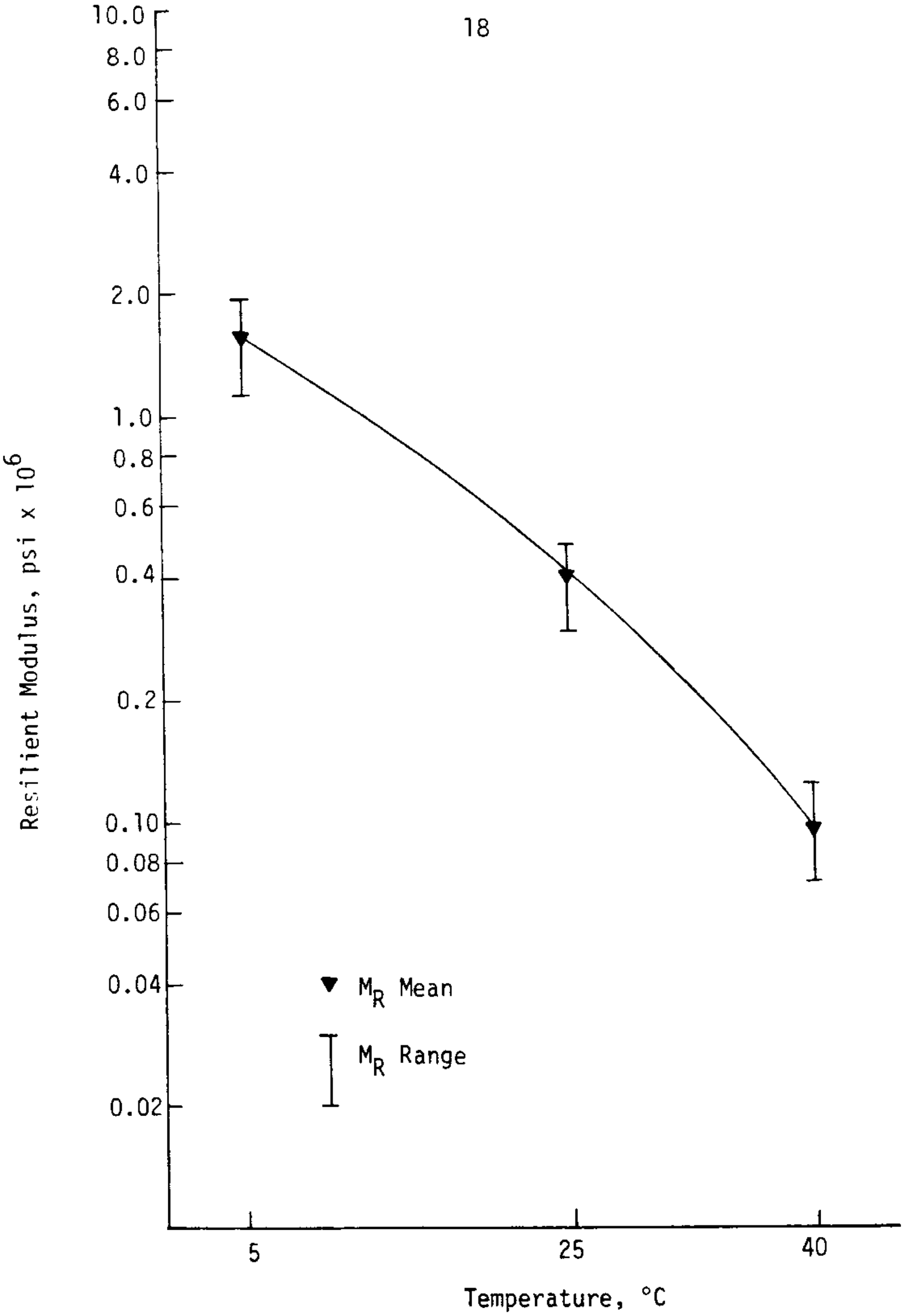

Figure 3. Resilient Modulus of SR 12 Cores as a Function of Temperature for Sta. 173+25.

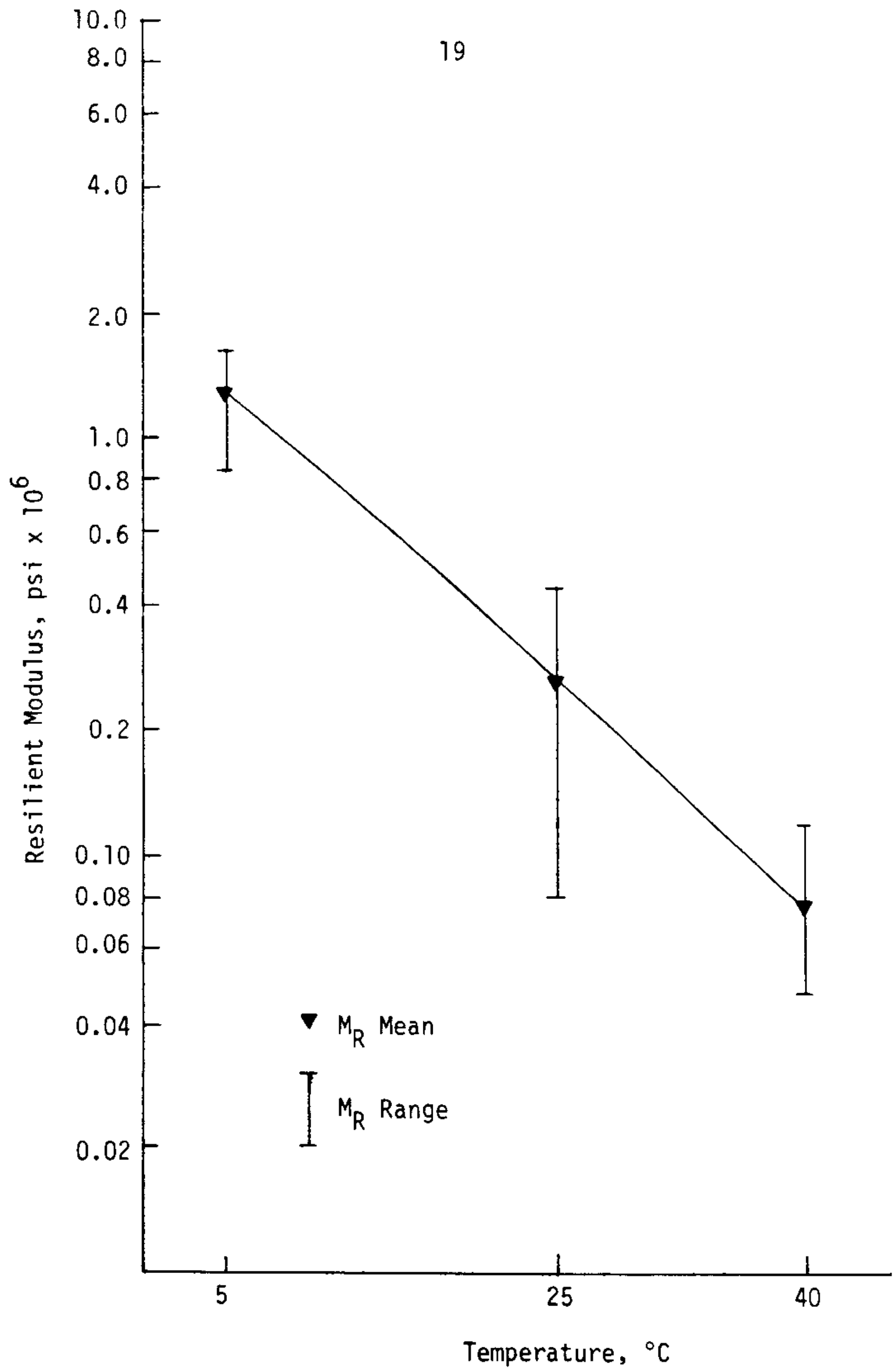

Figure 4. Resilient Modulus of SR 12 Cores as a Function of Temperature for Sta. 202+00.

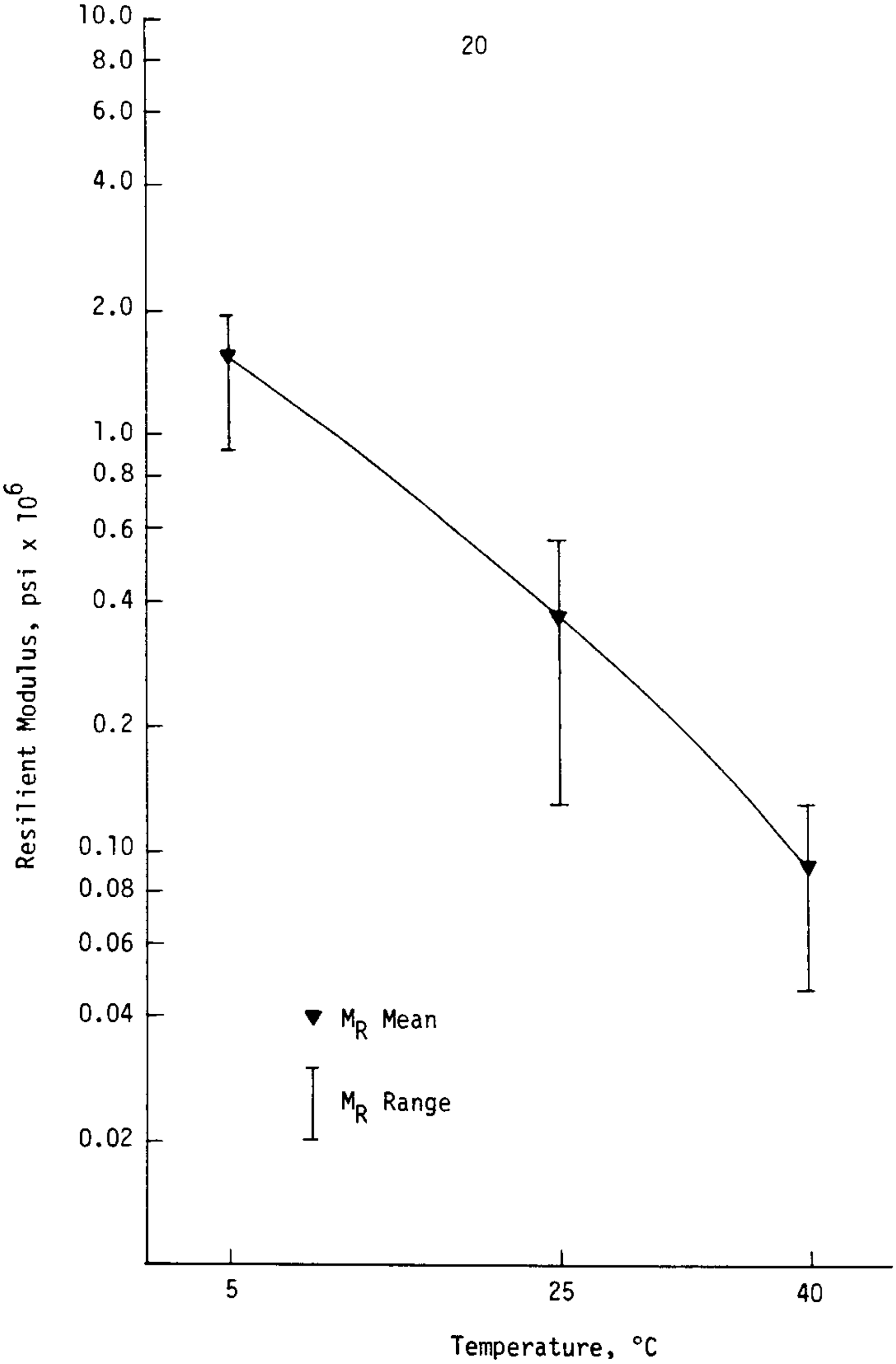

Figure 5. Resilient Modulus of SR 12 Cores as a Function of Temperature for Sta. 408+00.

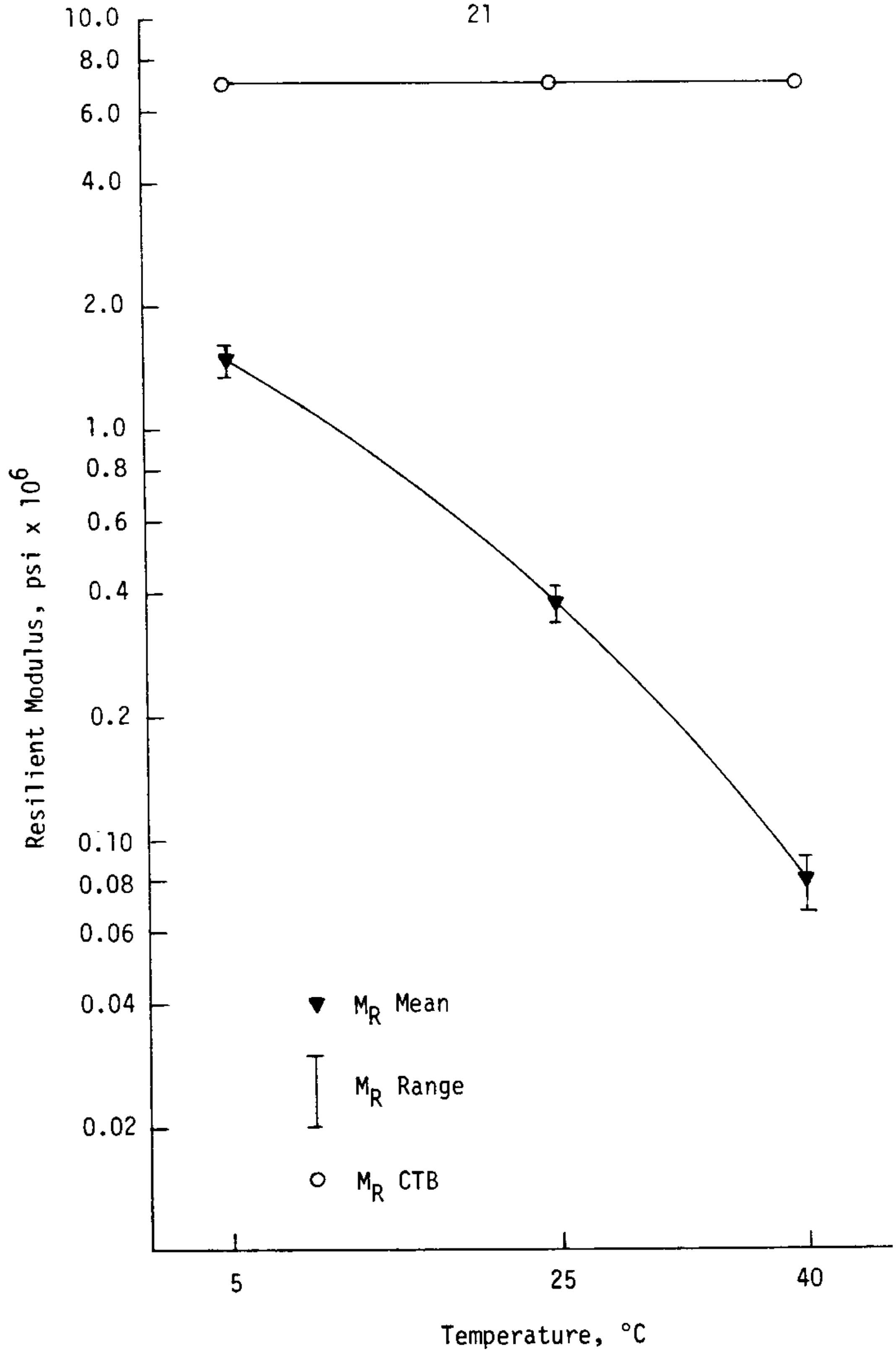

Figure 6. Resilient Modulus of SR 12 Cores as a Function of Temperature for Sta. 563+00.

Table 4. Summary of Marshall Test Results for SR 12 Cores.

Sample No.	Thickness (in.)	Bulk Specific Gravity	Maximum Specific Gravity	Air Void Content Percent Total Volume	Marshall Stability (lbs)	Marshall Flow (0.01 in)
173-1-1	2.104	2.47	2.59	4.9	1250	18
173-1-3	2.361	2.42		7.0	1025	20
173-3-2	2.671	2.45		5.7	1273	22
173-3-4	1.680	2.43		6.6	1102	16
202-1-1	1.978	2.43	2.64	8.6	1763	19
202-1-3	2.299	2.41		9.5	855	22
202-3-2	2.251	2.45		7.8	631	20
202-3-4	2.036	2.45		7.8	1144	21
408-1-1	2.541	2.51	2.60	3.6	1440	25
408-1-3	1.574	2.50		4.0	1675	16
408-3-2	2.279	2.51		3.6	1904	22
403-3-4	2.110	2.53		2.8	1716	18
563-2-1	2.472	2.34	2.43	3.8	1945	20

10 years old. An acceptable range for this type of data is normally considered to be a flow of 16 or less [3,4]. Most of the specimens are outside of this range but this fact is not considered to be of significance.

CTB CORES

Only two CTB core samples were obtained and both of these were from Station 563+00. These two core samples were sawed into four separate specimens in preparation for testing. The two tests conducted on these specimens were resilient modulus and indirect tensile strength.

Resilient Modulus

The resilient modulus determination was similar to that used for the ACP specimens with the results reported for Station 563+00 in Table 2. The resilient modulus values reported for the CTB specimens are considered to be unrealistically high. This occurred, most likely, due to the low dynamic load used in performing the test and the resulting difficulty in obtaining significant horizontal deformations. A visual examination of the CTB specimens reveal a superior appearing material with expected high stiffness. This is further verified by construction quality control information provided by WSDOT [2]. The information shows that approximately 5 percent by weight of cement was used to stabilize the gravel aggregate with resulting compressive strengths having a mean of 1800 psi and a standard deviation of 500 psi (based on 25 samples and 7 day strength). Currently, the compressive strength average is probably much higher since such materials tend to gain strength with time. For evaluation purposes, a resilient modulus of 3,000,000 psi was used which is more typical of a high quality CTB material.

Indirect Tensile Strength

This test provides a measure of the tensile strength for the CTB materials and is important in that large tensile stresses (strains) may occur at the bottom of the CTB layer. Thus, these stresses (strains) must be predicted and compared to allowable failure criteria (Chapter III). If predicted tensile stresses exceed the tensile strength, cracking of the CTB layer would be expected.

The test procedure is performed by loading a cylindrical CTB specimen with a compressive load which acts along the vertical diametrical plane as shown in Figures 7A and 7B [5]. The specimen fails due to the relatively uniform tensile stress which is developed along this vertical plane. The equation used to calculate the resulting indirect tensile strength is:

$$S_t = \frac{2P_{max}}{\pi t d}$$

where

S_t = Tensile strength (psi)

P_{max} = Applied load at failure (lb)

t = Specimen thickness (in)

d = Specimen diameter (in)

As stated previously, four specimens were available for testing, the results of which are shown in Table 5. One specimen was not tested and was retained for future work if necessary. Generally, the three

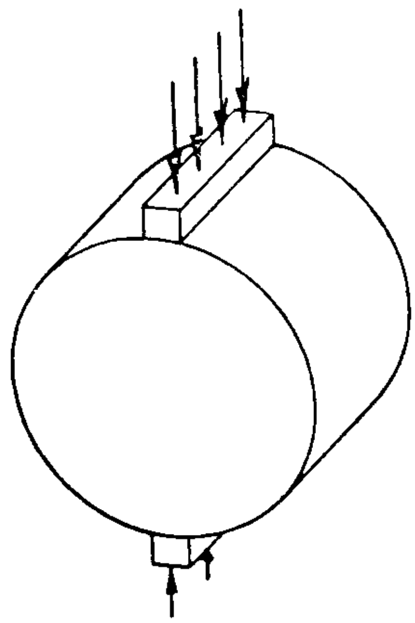

Figure 7A. Cylindrical Specimen with Compressive Load [5].

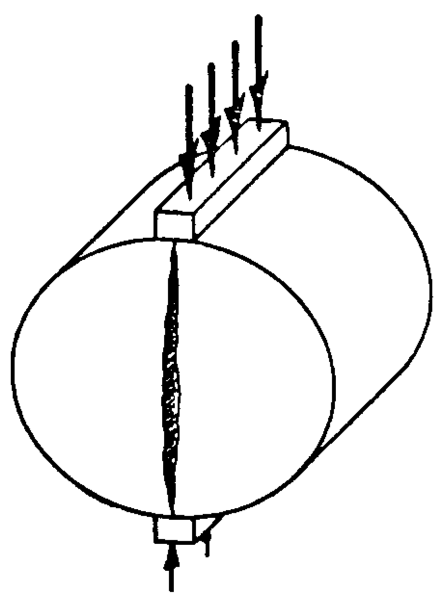

Figure 7B. Specimen Failing in Tension Under Compressive Load [5].

Table 5. Determination of Indirect Tensile Strength for Cement Treated Base - Station 563+00.

Sample Number	Tensile Strength (psi)
563-1-2	411
563-1-3	321
563-2-3	347

tensile strength values confirmed the fact that the CTB material was of good quality although the data represents only one station along the SR 12 proposed haul route.

GRANULAR MATERIALS

The granular, disturbed materials obtained by WSDOT were sampled from the shoulder area of SR 12 which underlies the relatively thin ACP shoulder surfacing. The samples were placed in canvas bags for delivery to the UW laboratory. Two kinds of granular samples were obtained. One type was crushed surfacing top course material and the other was the gravel fill material which underlies much of the SR 12 haul route. This gravel fill material contained some cobbles with sizes in excess of 2-in.

Figure 7C shows the laboratory sequence used to evaluate the granular materials. This sequence includes processing the samples for laboratory compaction and obtaining both resilient modulus and CBR data. Each of the major steps shown in the figure will be separately presented.

Sieve Analysis

First, a sieve analysis was performed on representative samples of the granular materials in accordance with ASTM Standard Methods C117 and C136. The results of these tests are shown in Figures 8 through 13. All samples tested (with the exception of Sample No. 4, Station 202+00) had less than six percent passing the No. 200 sieve. All of the gradations shown in these figures were similar except for Sample No. 7, Station 408+00, which was the one crushed top course

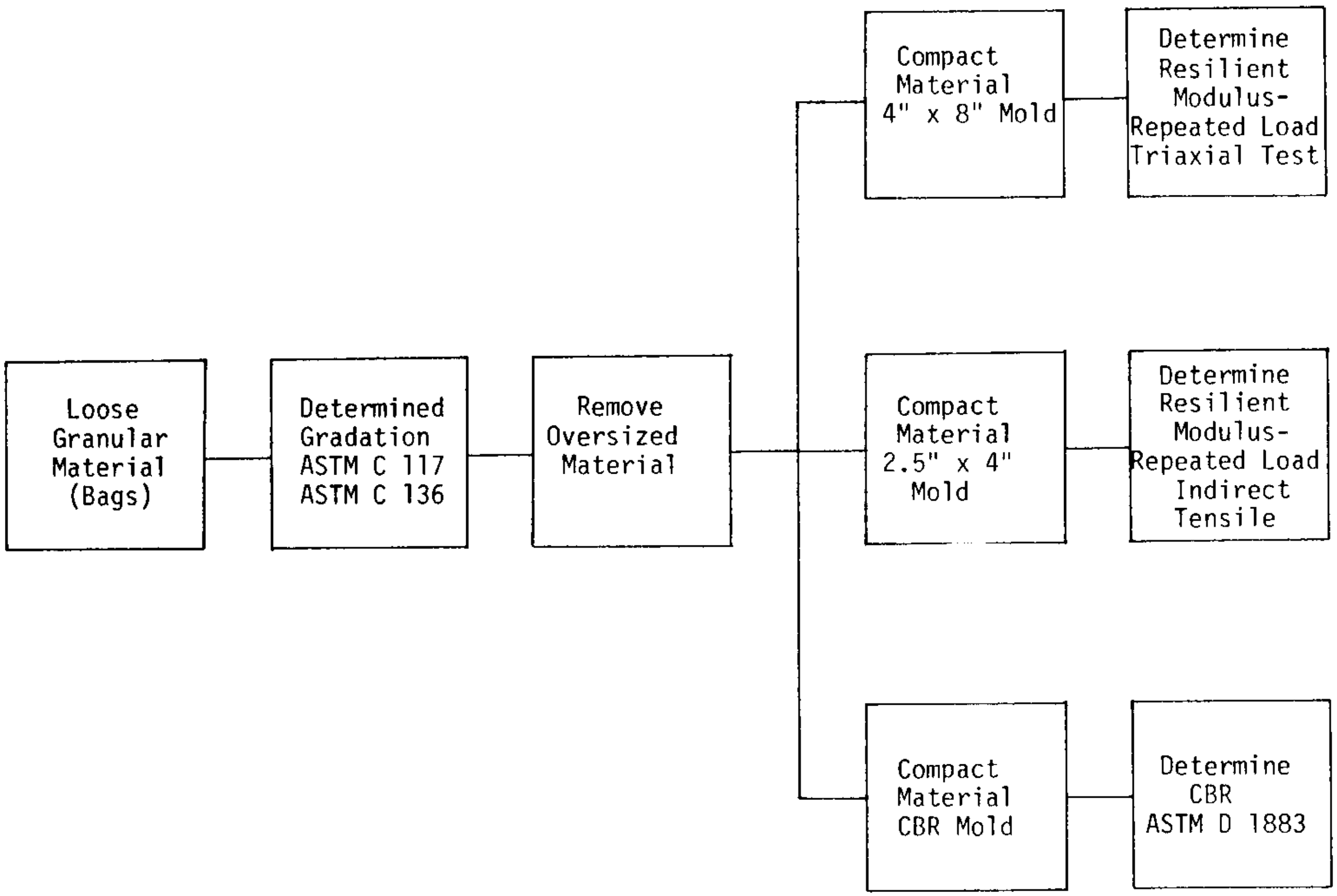

Figure 7C. Test Sequence for Granular (Loose) Samples from SR 12.

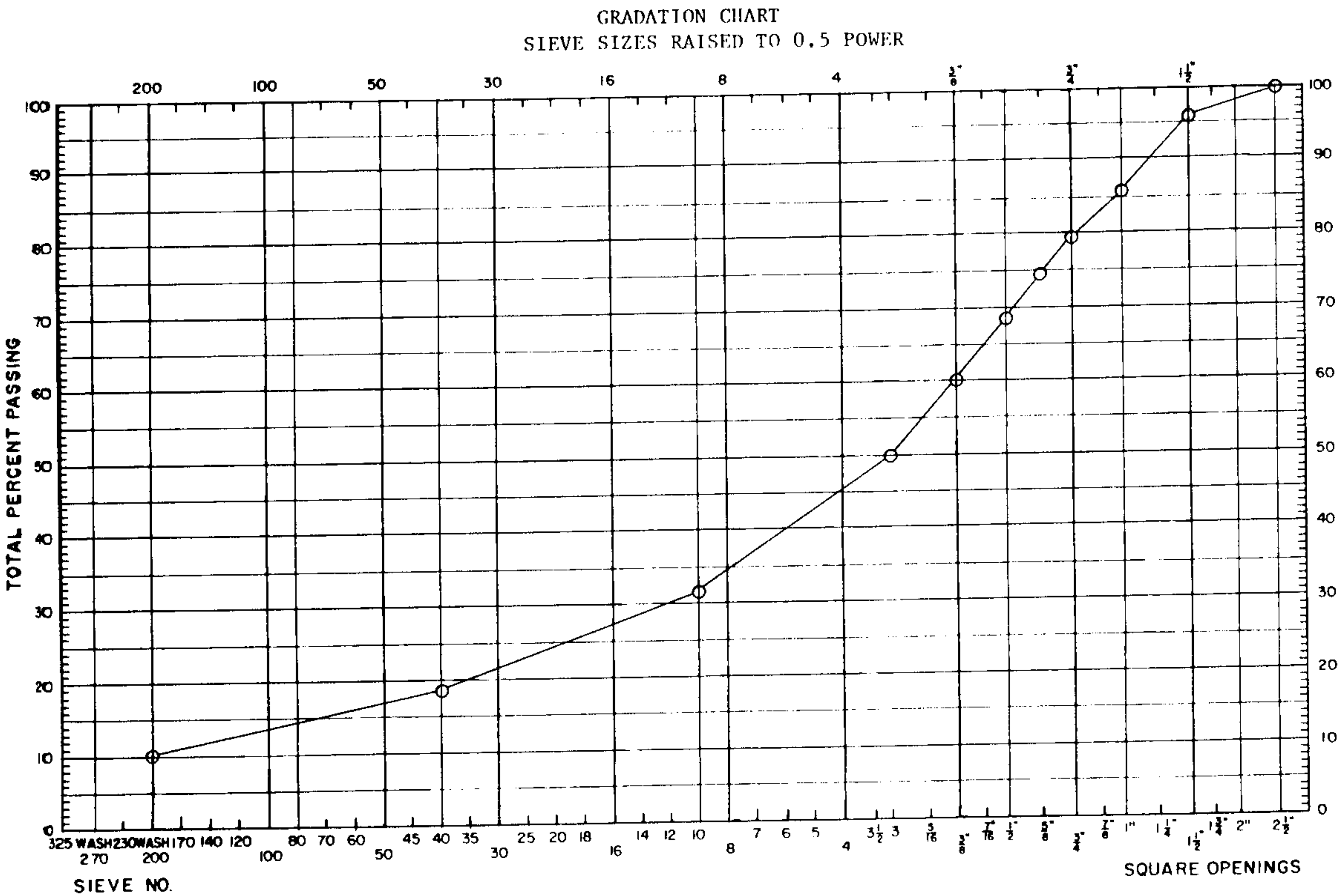

Figure 8. Gradation Curve of Granular Sample No.4, Sta. 202+00.

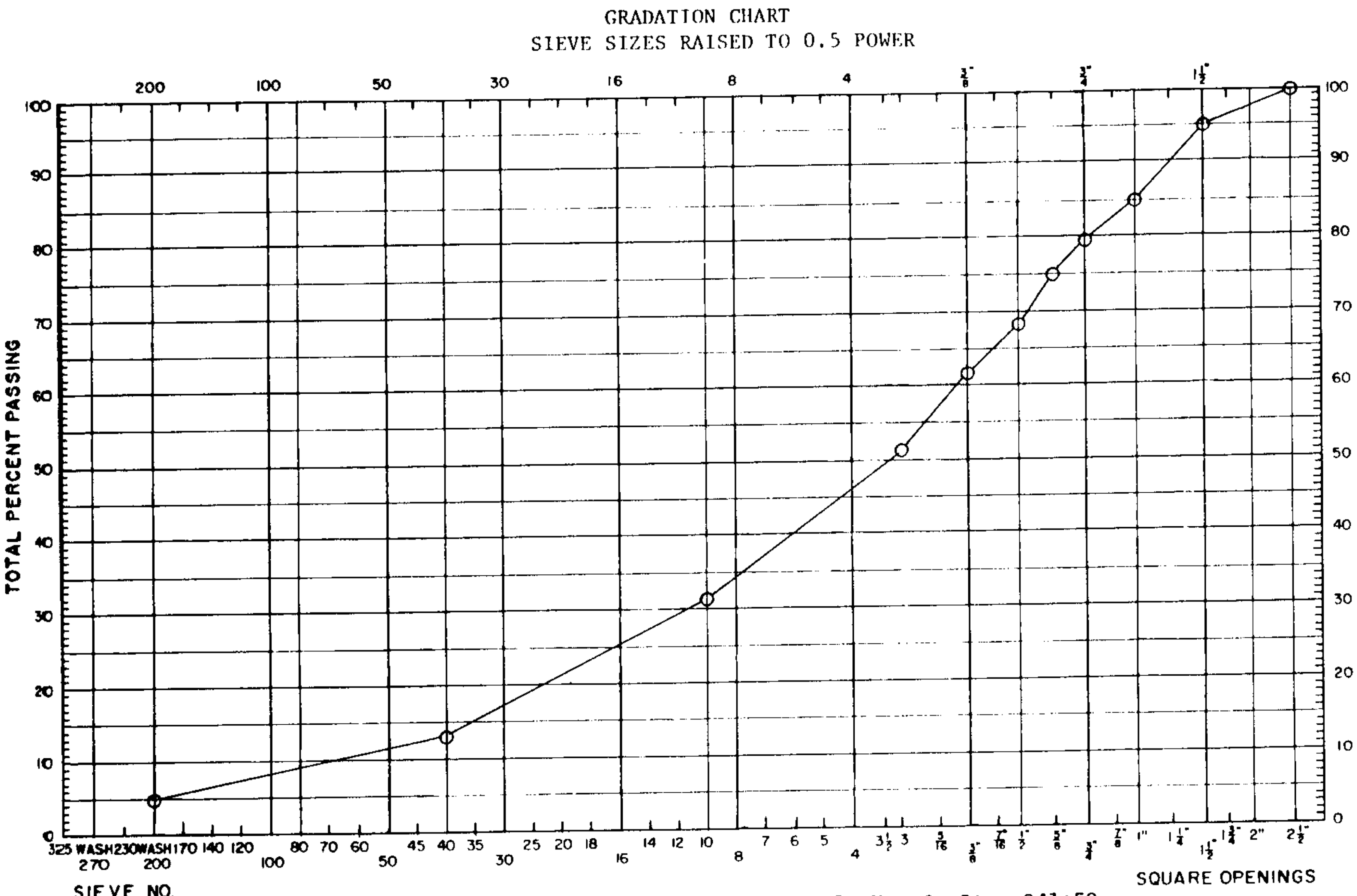

Figure 9. Gradation Curve of Granular Sample No. 6, Sta. 341+50.

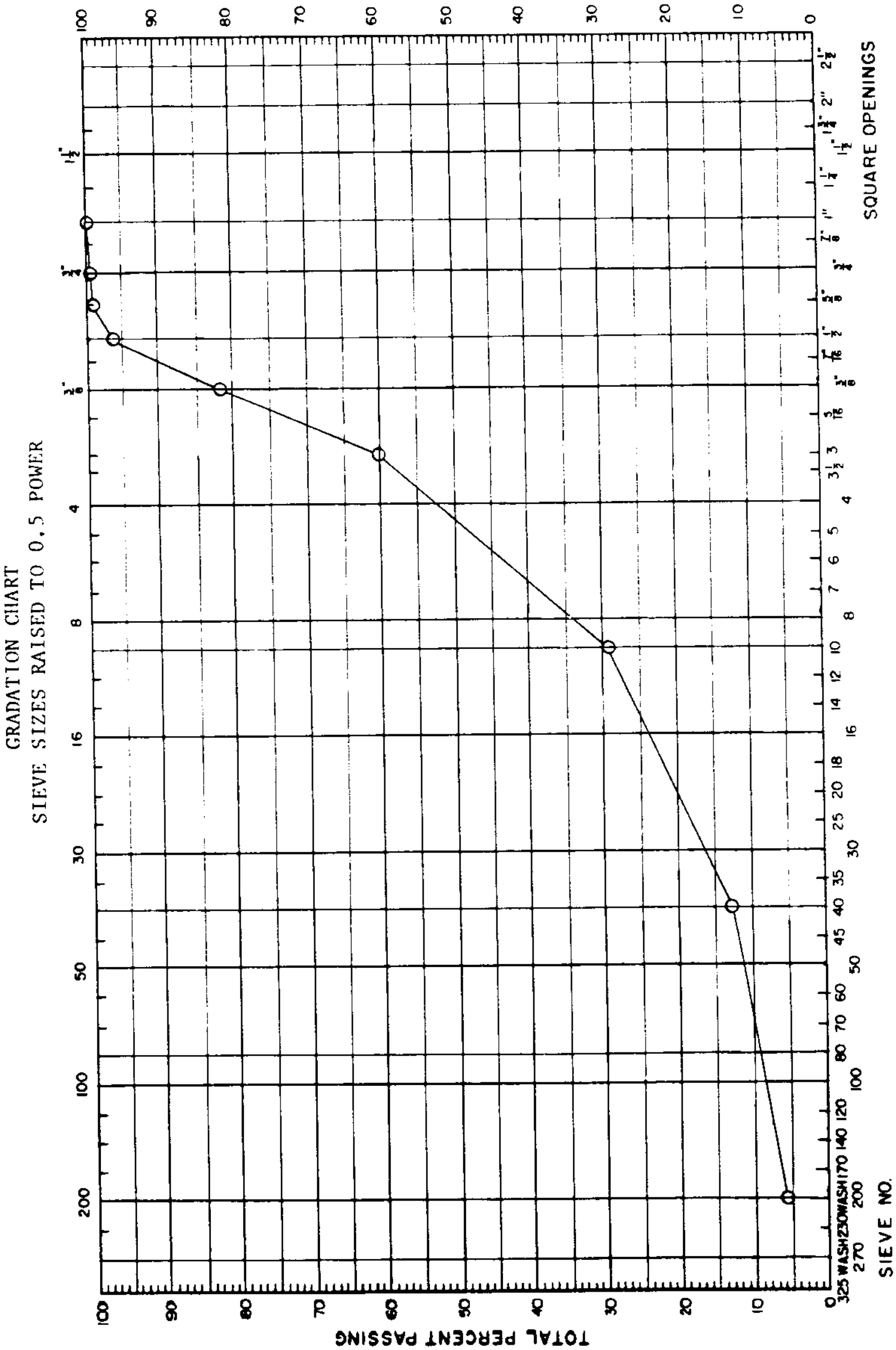

Figure 10. Gradation Curve of Granular Sample No. 7, Sta. 408+00.

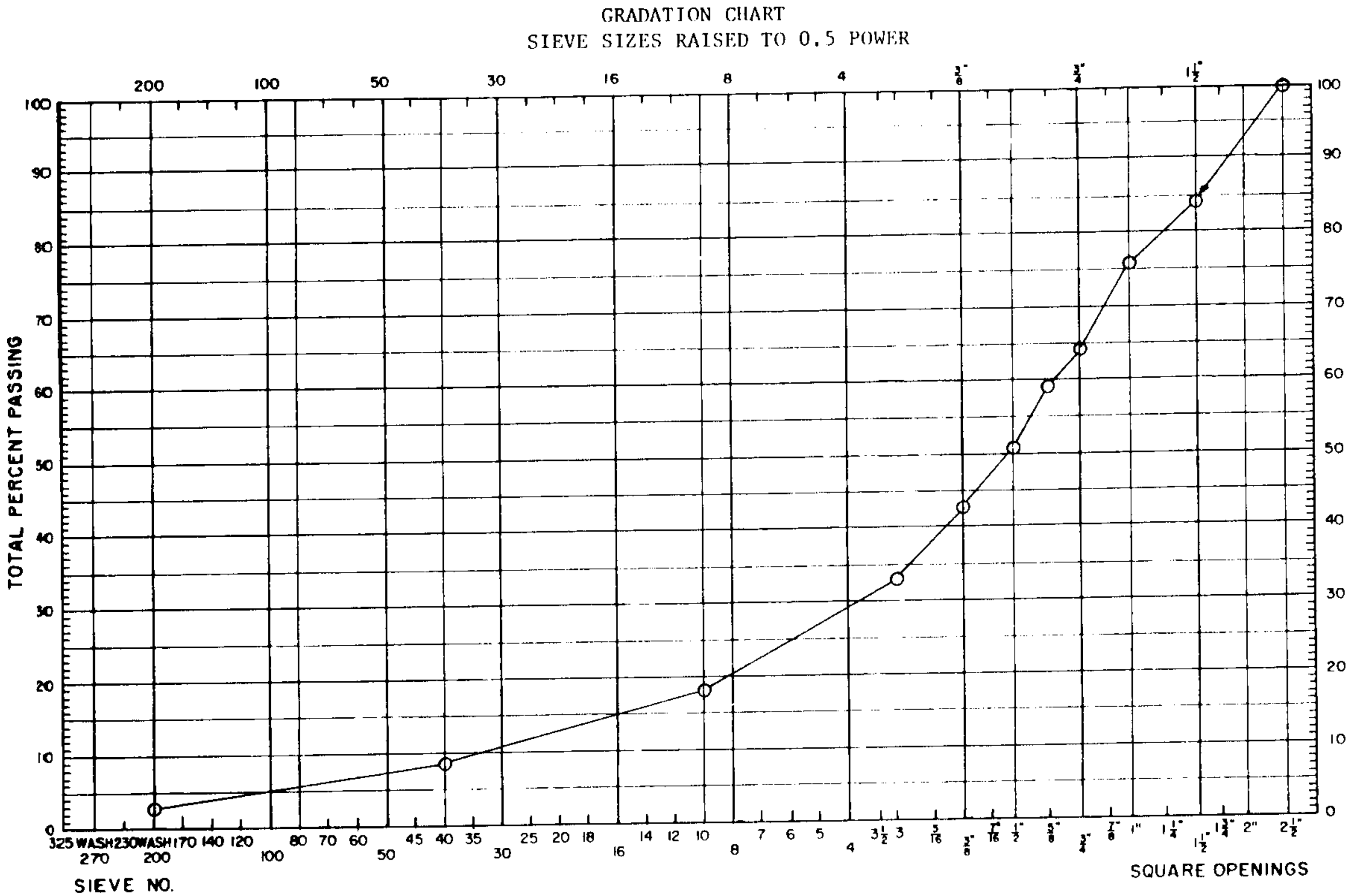

Figure 11. Gradation Curve of Granular Sample No. 9, Sta. 408+00.

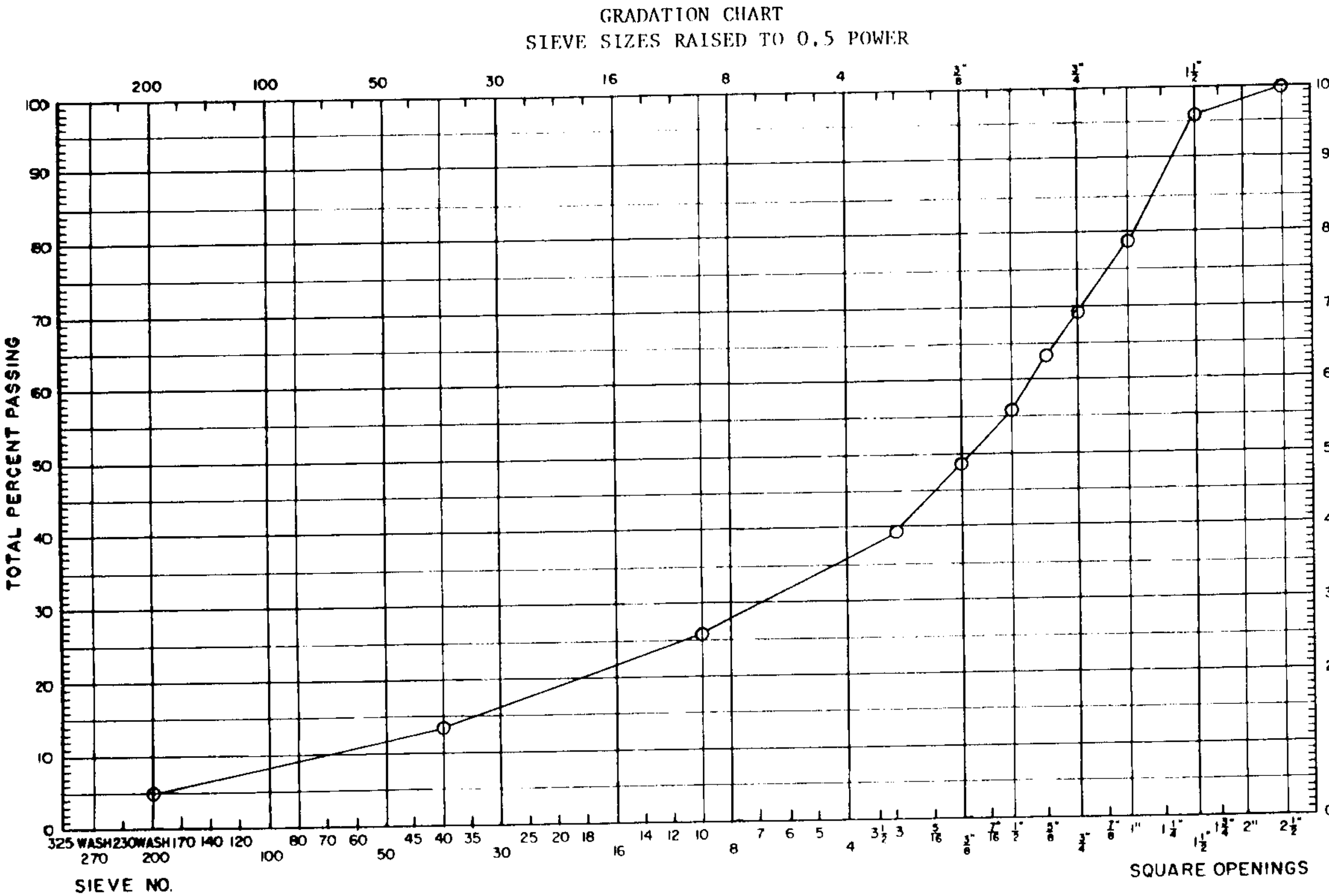

Figure 12. Gradation Curve of Granular Sample No. 11, Sta. 453+60.

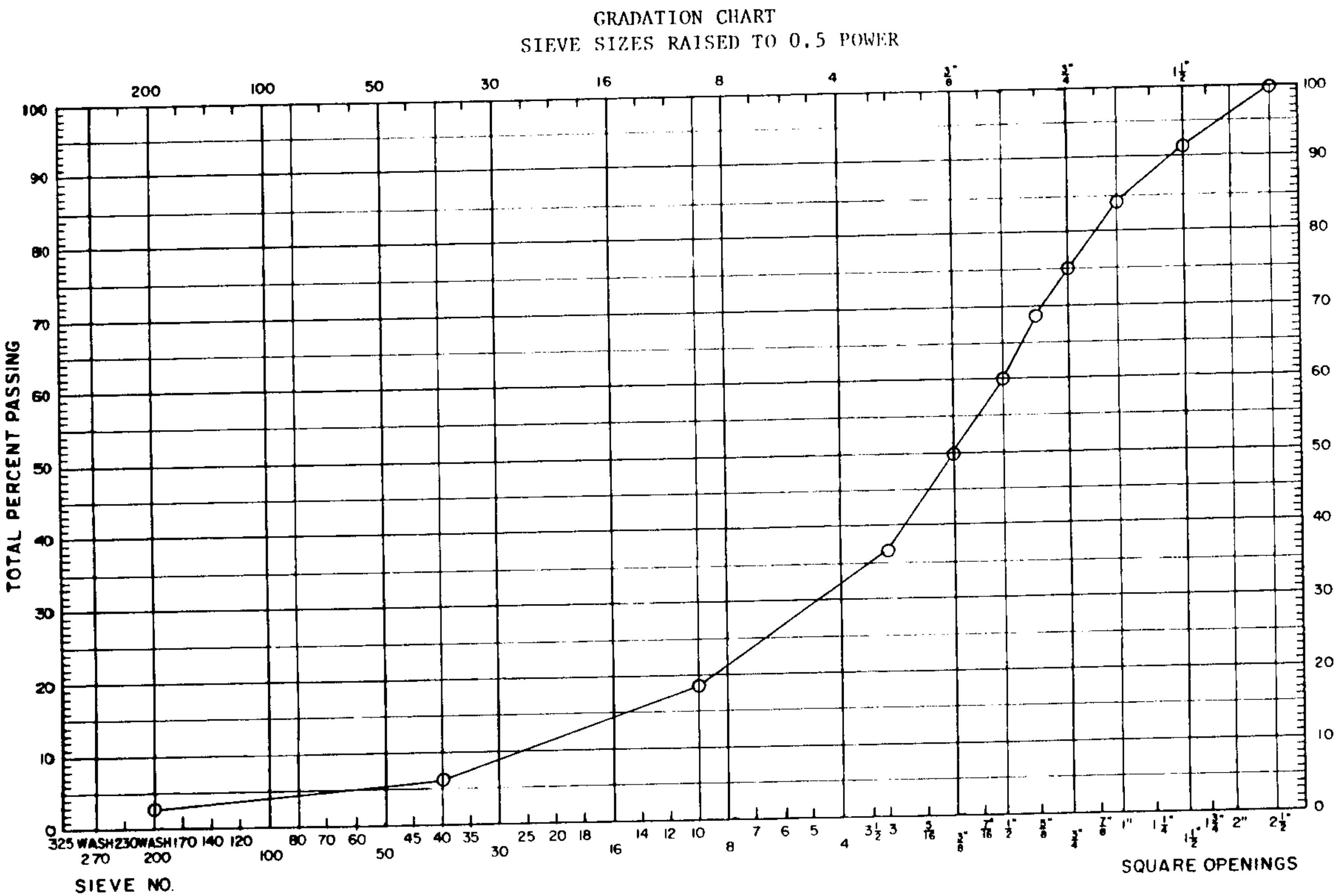

Figure 13. Gradation Curve of Granular Sample No. 12, Sta. 593+00.

sample examined. All other samples are of the gravel fill material.

California Bearing Ratio (CBR)

The CBR tests were conducted on selected granular samples in accordance with ASTM Standard Method D1883. Sample preparation was performed according to ASTM Standard Method D1557, Method D, which included the specified replacement of course material passing the 2-in sieve and retained on the 3/4-in. Prior to laboratory compaction, each sample was oven dried then mixed with the appropriate quantity of water to approximate the in situ moisture content. The CBR test was performed after the samples were soaked for 72 hours and drained for 15 minutes.

The test results are summarized in Table 6 with the load-penetration curves shown in Figures 14 through 19. Table 6 shows the wet density for these samples both for the in situ (field) and as molded (laboratory) states as well as the corresponding moisture contents. No swell for these materials were recorded during the soaking period and prior to determination of the CBR. The CBR values reported indicate that the samples which contained a larger percent passing the No. 40 sieve yielded slightly lower CBR values. However, the critical sieve size appears to be the No. 200. Testing of Sample No. 4, Station 202+00, resulted in the lowest CBR value of 24. A possible explanation for this deviation from the other CBR results is that this sample had approximately 10 percent passing the No. 200 sieve while the other samples had lower percents passing.

Caution is warranted in using these CBR results. The original gradations of these granular samples were modified in accordance with

Table 6. Summary of CBR Results for Granular Materials - SR 12.

Sample No.	Density (lb/ft^3)		Moisture Content (%)		Swell (%)	CBR
	In situ	As Molded	In situ	As Molded		
4 (Sta. 202+00)	146.3	151.2	7.9	7.8	0	24
6 (Sta. 341+50)	146.3	152.6	7.3	7.1	0	100+
7 (Sta. 408+00)	139.0	146.0	5.4	5.6	0	100+
9 (Sta. 408+00)	151.3	153.7	6.8	7.2	0	100+
11 (Sta. 453+60)	145.2	148.8	7.0	7.3	0	100+
12 (Sta. 593+00)	143.5	149.1	6.4	5.7	0	100+

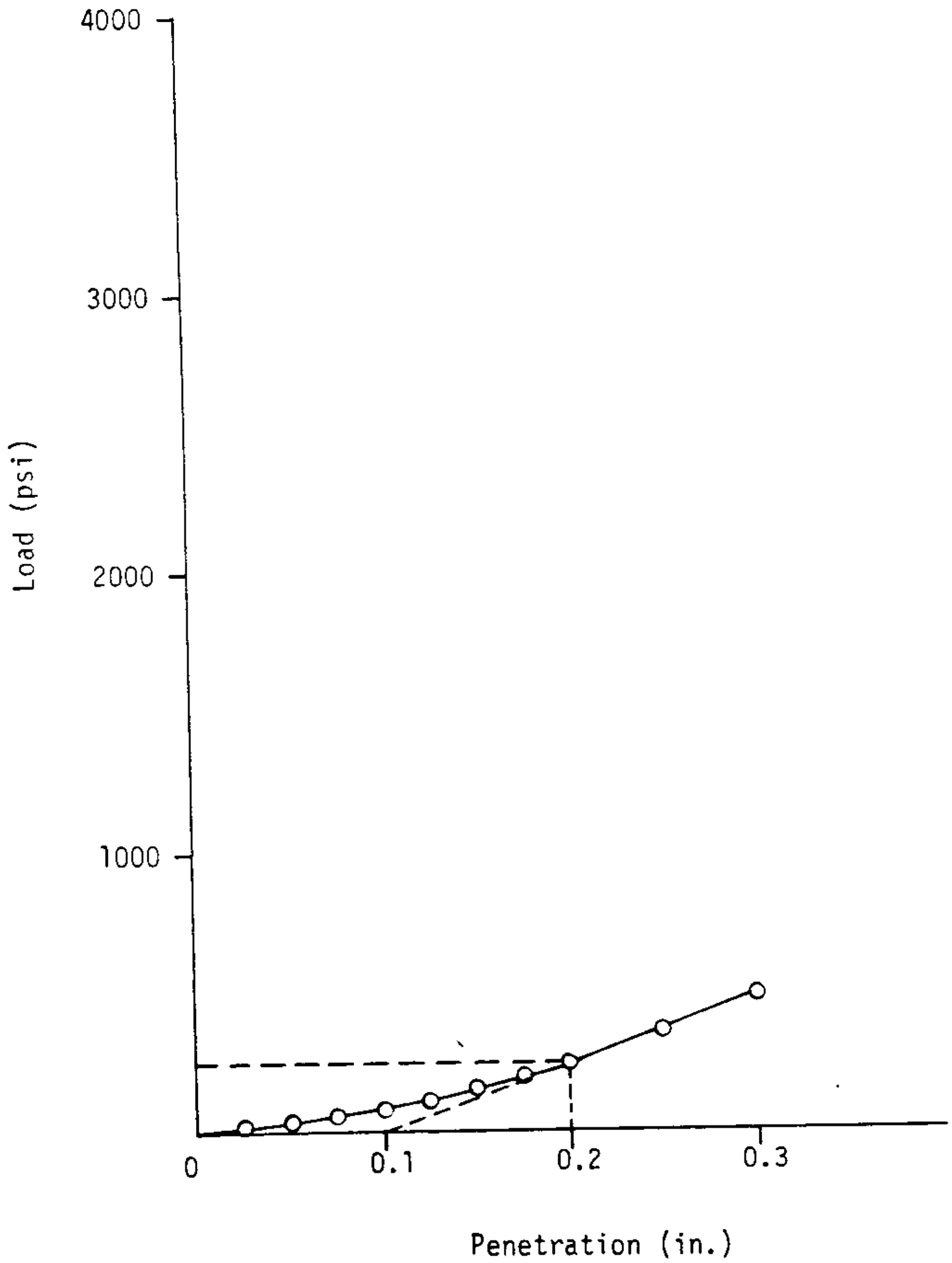

Figure 14. CBR Load vs. Penetration Curve for Sample No. 4, Sta. 202+00.

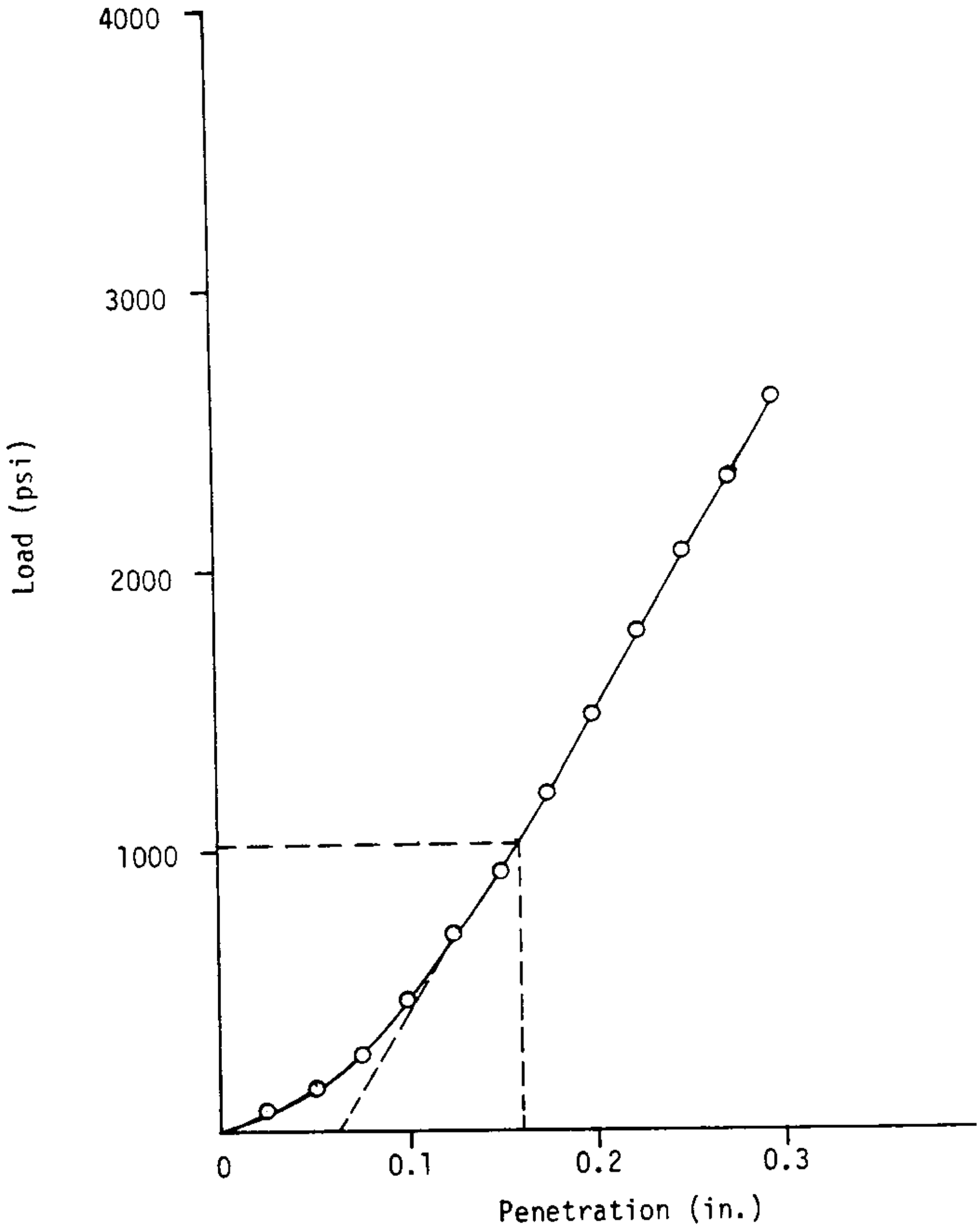

Figure 15. CBR Load vs. Penetration Curve for Sample No. 6, Sta. 341+50.

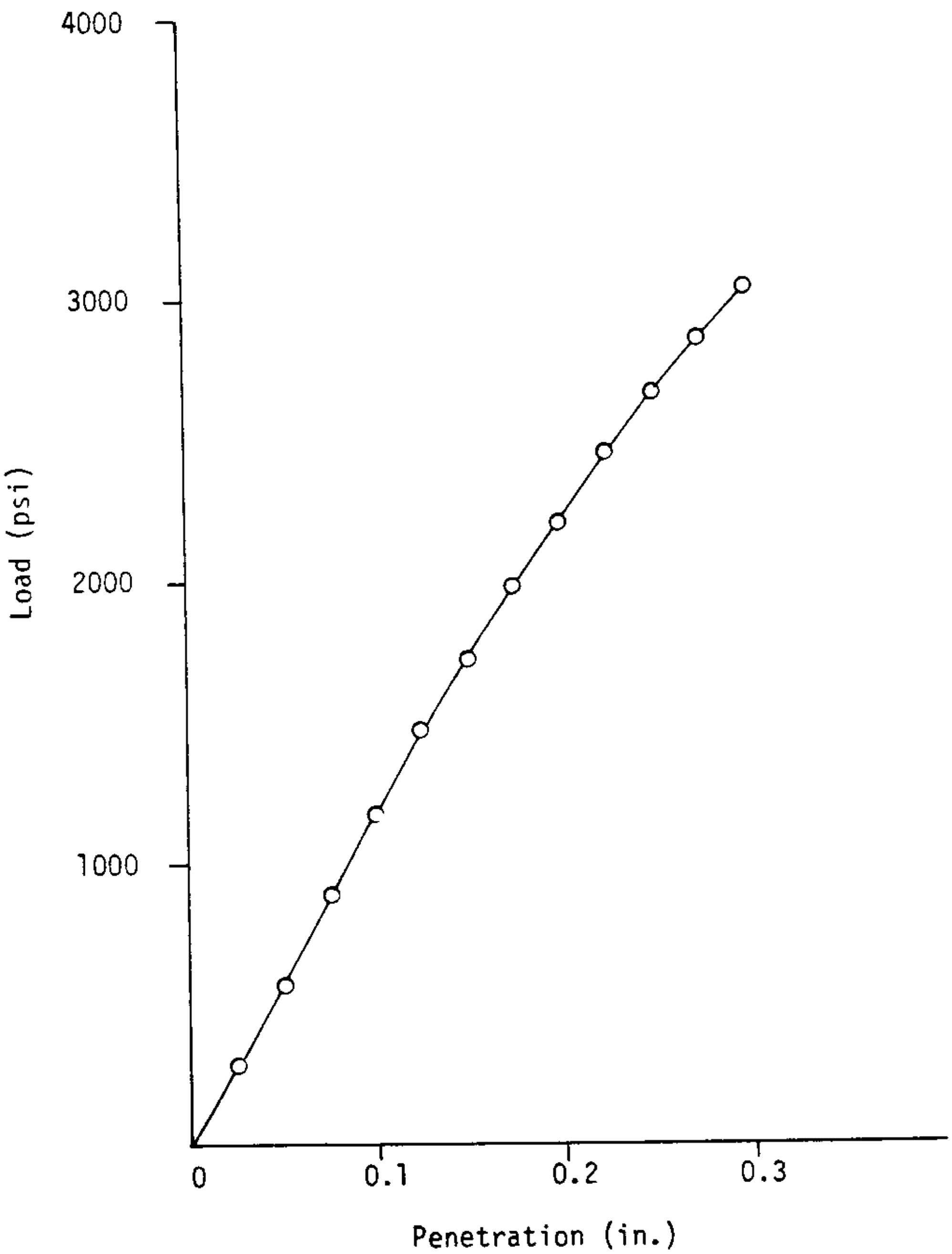

Figure 16. CBR Load vs. Penetration Curve for Sample No. 7, Sta. 408+00.

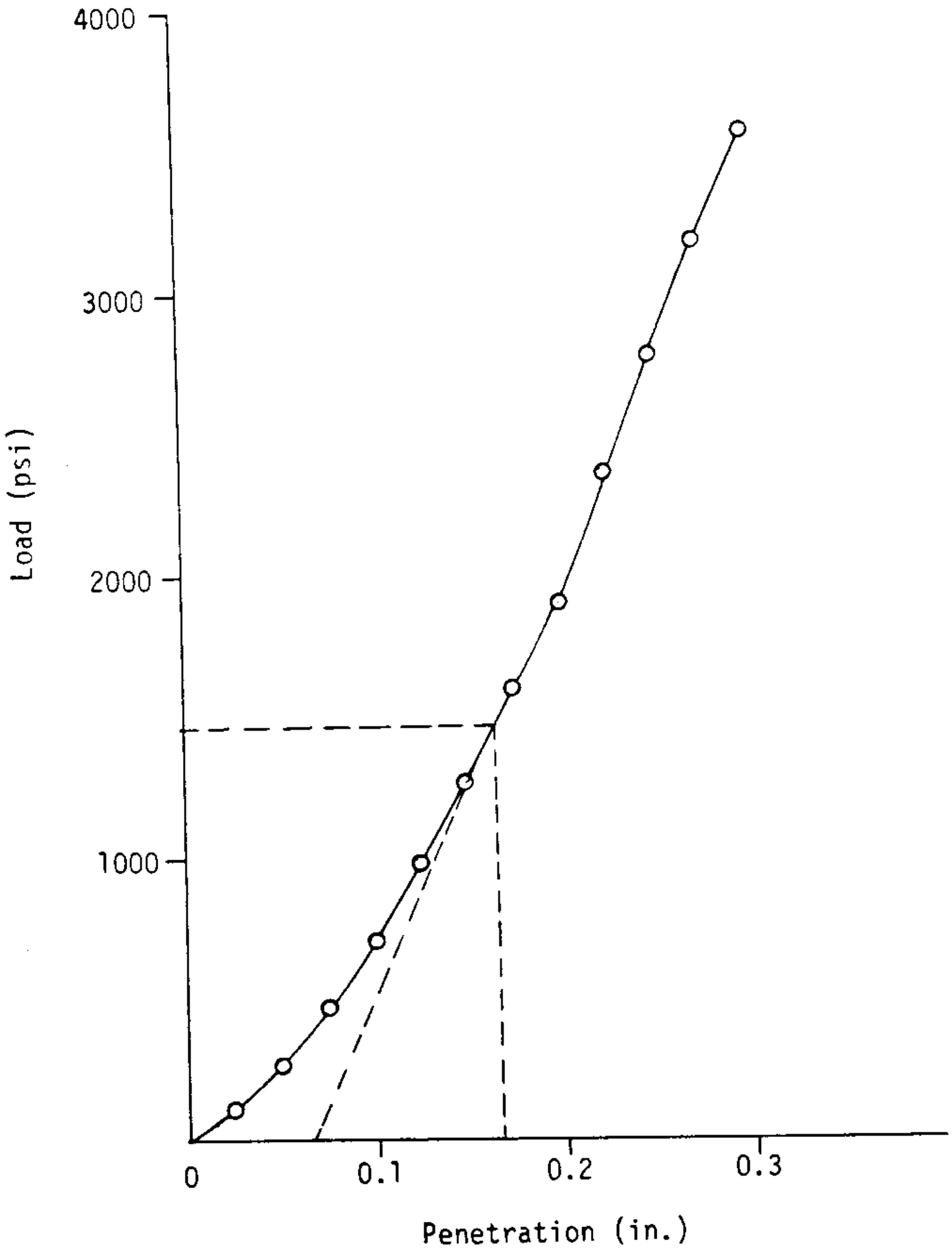

Figure 17. CBR Load vs. Penetration Curve for Sample No. 9, Sta. 408+00.

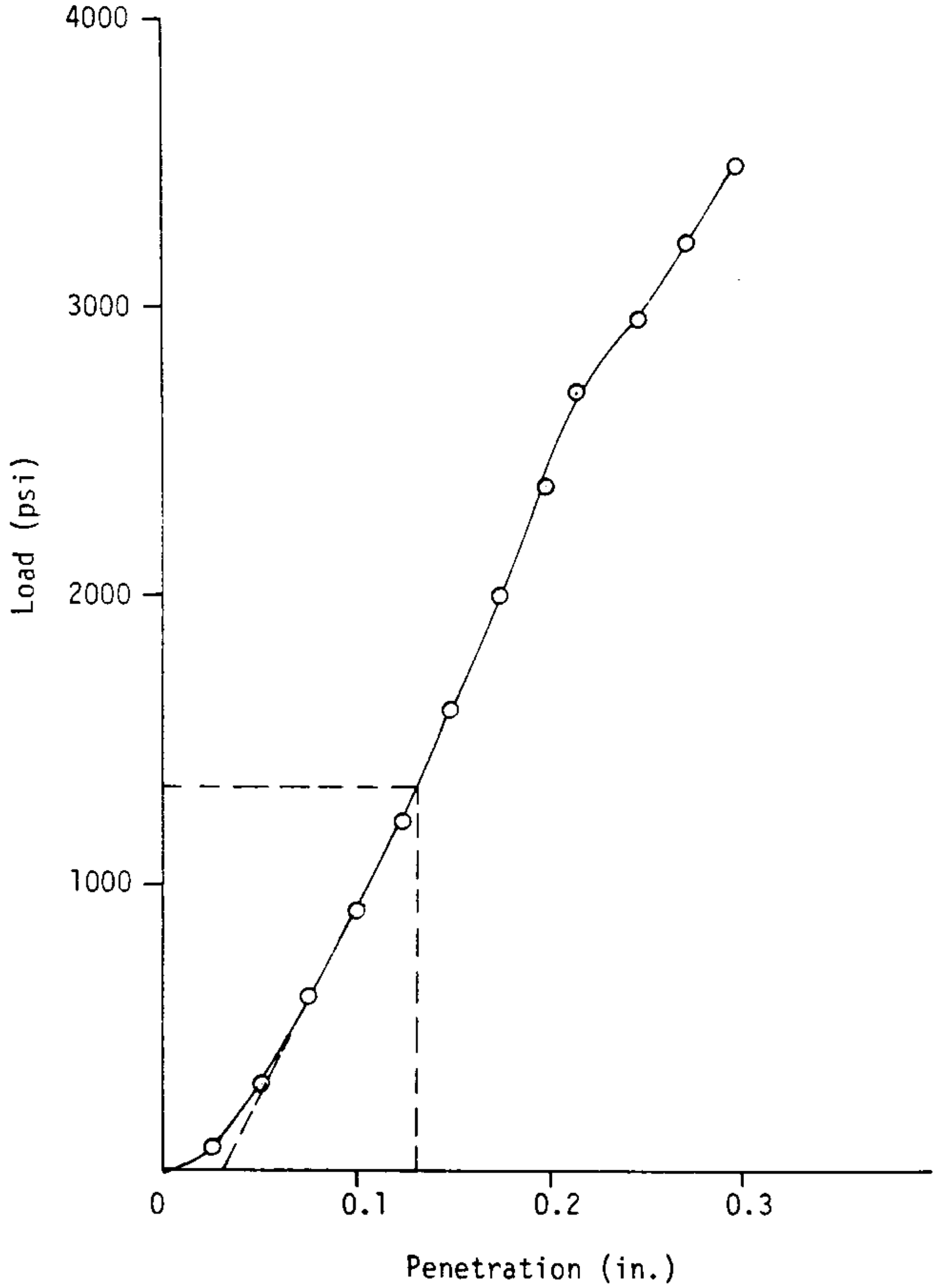

Figure 18. CBR Load vs. Penetration Curve for Sample No.11, Sta. 453+60.

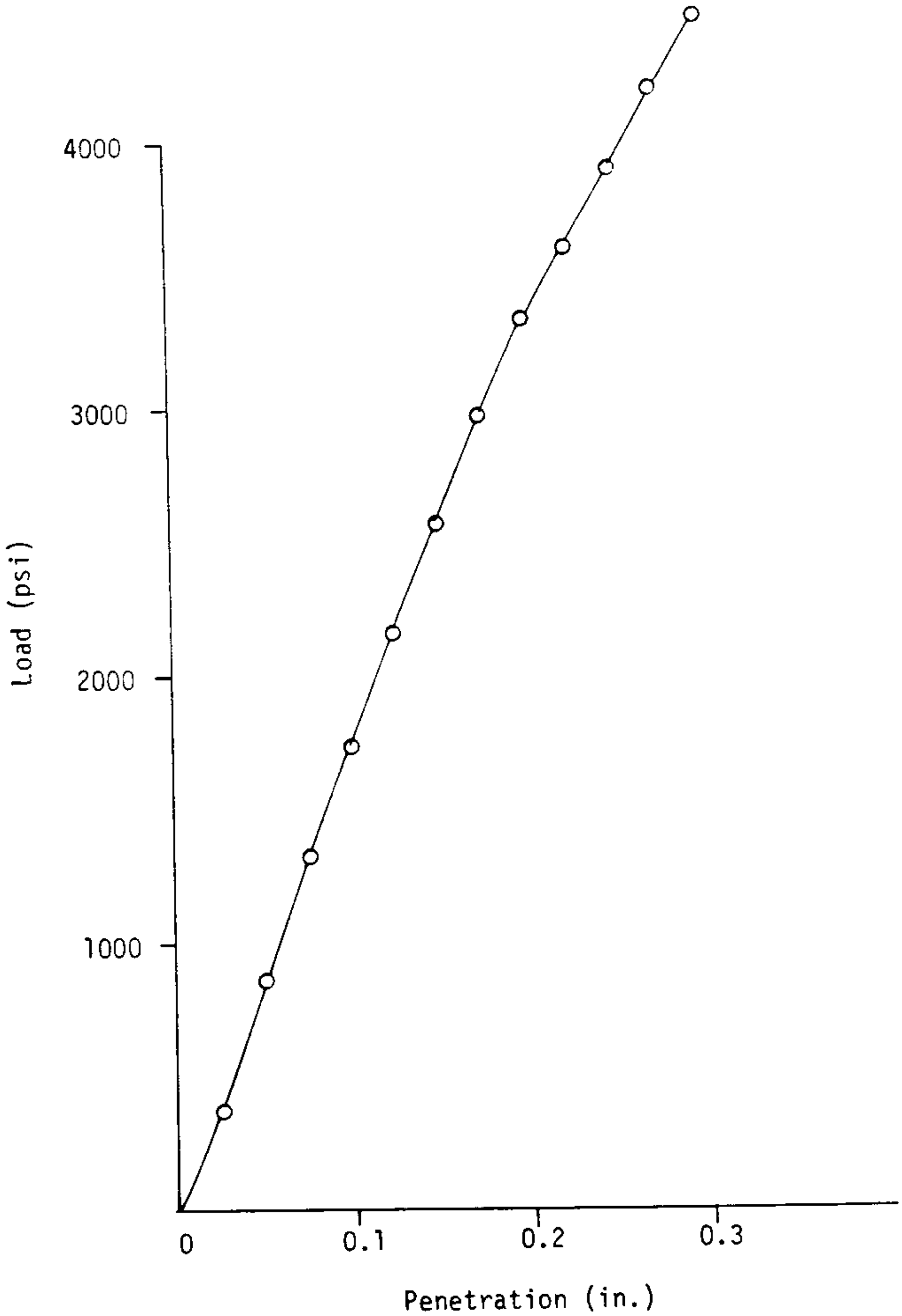

Figure 19. CBR Load vs. Penetration Curve for Sample No.12, Sta. 593+00.

standard procedures. What effect these changes have on the resulting CBR values is unknown. It is apparent that the materials are sensitive to changes in gradation based on the reported test results.

Resilient Modulus - Triaxial

Measurements of resilient response were made in a conventional triaxial cell on specimens nominally 4-in in diameter by 8-in high. The repeated axial compressive stresses were applied using a pneumatic loading system. Recoverable axial deformations for all specimens were measured using a mechanical system centered around a dial gage with 1×10^{-4}-in increments. The specimen was contained by a thin latex membrane and two end platens. The dial gage was attached to a circumferential ring which has three equally spaced and adjustable clamping pads. The ring was attached to the specimen's top end platen in the triaxial cell. The recoverable axial deformations were measured by extending the dial gage stem to the lower end platen.

The selected granular specimens were prepared in accordance with ASTM Standard Method, D1557, Method C, including specified replacement of the material passing the 2-in sieve and retained on the 3/4-in. The specimens were compacted at or near the reported in situ moisture content in a 4-in diameter by 8-in high split mold. After compaction the specimens were encapsulated in a thin latex membrane. Table 7 shows the in situ and as molded wet densities and moisture contents for these samples.

The confining pressure (σ_c) medium used in the triaxial cell was air. The following stress combinations of σ_c and deviator stress (σ_d) were used in sequence:

Table 7. Summary of Densities and Moisture Contents for Laboratory Compacted Granular Triaxial Samples.

Sample No.	Wet Density (lb/ft^3)		Moisture Content (%)	
	In Situ	As Molded	In Situ	As Molded
4 (Sta. 202+00)	154.6	157.9	7.9	6.0
6 (Sta. 341+50)	146.3	158.3	7.3	4.7
7 (Sta. 408+00)	139.0	155.0	5.4	7.0
9 (Sta. 408+00)	151.3	156.4	6.8	6.4
11 (Sta. 453+60)	145.2	150.0	7.0	6.4
12 (Sta. 593+00)	143.5	150.0	6.4	5.8

Level

1. $\sigma_c = 0$ $\sigma_d = 1.5$ psi
2. $\sigma_c = 1.0$ psi $\sigma_d = 3.0$ psi
3. $\sigma_c = 2.0$ psi $\sigma_d = 6.0$ psi
4. $\sigma_c = 2.5$ psi $\sigma_d = 7.5$ psi
5. $\sigma_c = 5.0$ psi $\sigma_d = 11.0$ psi
6. $\sigma_c = 5.0$ psi $\sigma_d = 15.0$ psi

At each stress combination the repeated load was applied to the specimens until a relatively consistent recoverable axial deformation was measured which usually occurred after 100 to 200 cycles. This recoverable axial deformation was used to calculate the resilient modulus (M_R) according to the following relation:

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

where σ_d = Deviator stress (psi)

ε = Recoverable axial strain (in/in)

Due to the variability of the resilient modulus values obtained with this testing procedure, all data obtained for the gravel fill samples were plotted in Figure 20. A regression fit of resilient modulus as a function of bulk stress (sum of principal stresses) results in the following relationship:

$$M_R = 5840\theta^{0.26}$$

where M_R = Resilient modulus (psi)

θ = Bulk stress (sum of principal stresses)

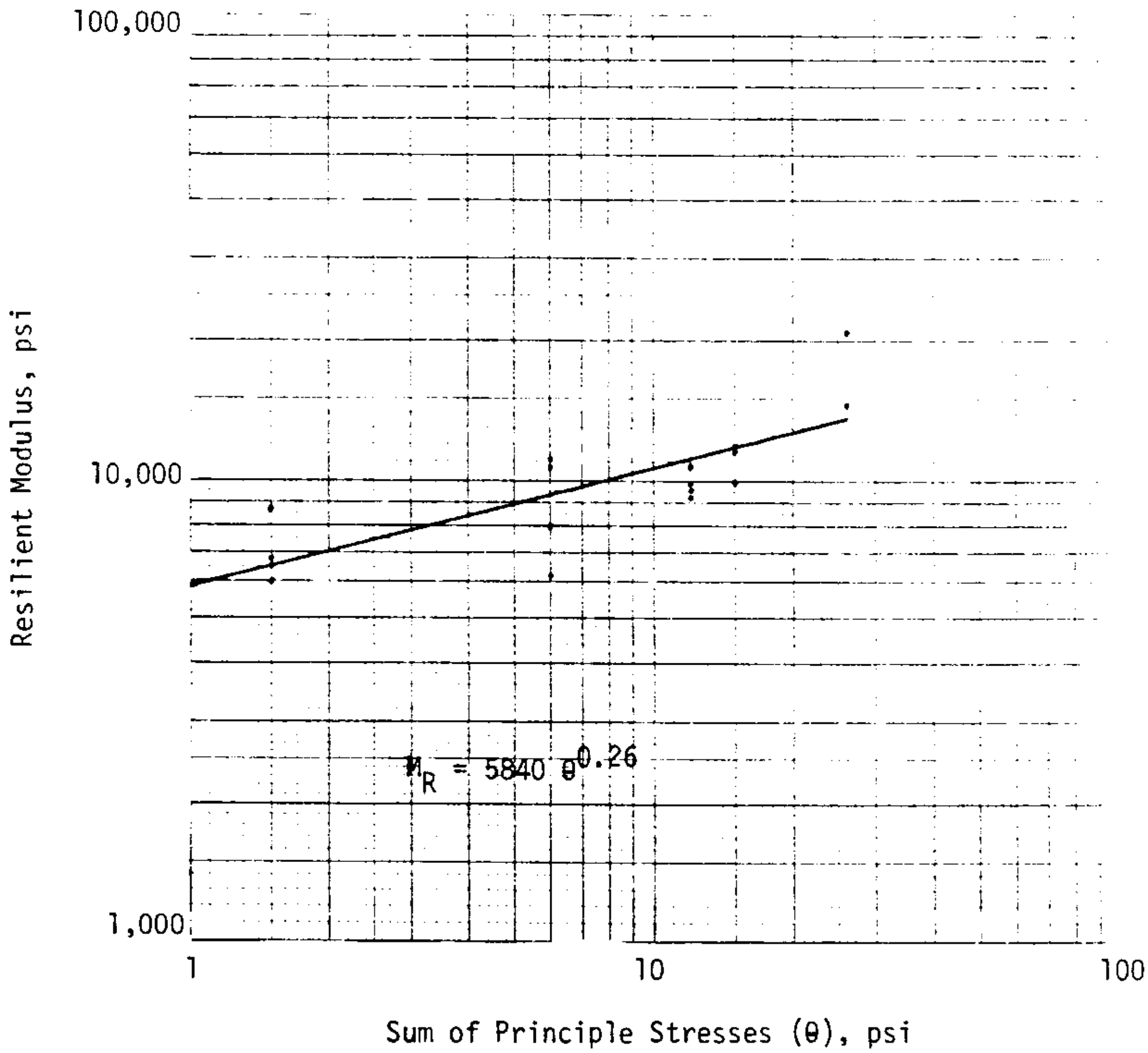

Figure 20. Resilient Modulus Relationship for Untreated Granular Fill Material - SR 12.

It is the opinion of the authors that the above relationship results in conservative estimates of resilient modulus. This conservatism is justified since the effect of modifying the sample gradations to allow conduct of the test is not known. Additionally, the relationship is based on the gravel fill material not the crushed top course material. The layer thicknesses of the crushed top course material are essentially insignificant when compared to the gravel fill layers. Thus, the gravel fill is of primary interest.

Resilient Modulus - Indirect Tensile

This test method was developed near the conclusion of the laboratory testing program and hence was not fully evaluated. Preliminary results indicated a rough agreement occurred between these resilient modulus test results and those obtained by use of the triaxial approach.

EVALUATION

Much of the data reported in this chapter will be used to model various cross sections for SR 12. Thus, by using elastic layered analysis techniques, the response of selected SR 12 pavement structures can be estimated for various load configurations.

CHAPTER III. EVALUATION

The overall goal of the field and laboratory material investigation was to obtain by testing,or to be able to otherwise estimate,the required elastic parameters which enable modeling of the SR 12 pavement structures as a layered elastic system. By using such an analysis procedure, the various and unusual loading configurations which the pavement structures may have to carry could be evaluated. This evaluation utilizes the calculated stresses, strains and deflections obtained from the layered elastic modeling and applies appropriate limiting values or failure criteria to them. Thus, estimates of potential pavement failure or reduction in pavement life can be made which can be attributed to the planned hauls.

The modeling of the response of the SR 12 pavement structures requires several steps and include the following:

1. Selection of the appropriate layered elastic computer program
2. Selection of the pavement structures (cross sections) to be evaluated and required material inputs
3. Determination of load configurations (dimensions and weights)
4. Selection of appropriate limiting values or failure criteria for the predicted pavement stresses, strains or deflections
5. Predicting stresses, strains and deflections by use of the layered elastic computer program (Step 1) and applying to these results the appropriate limiting value or failure criteria (Step 4).

Each of the above steps will be further discussed and presented in the sections which follow.

LAYERED ELASTIC COMPUTER PROGRAMS

Modeling a pavement structure as a layered elastic system is a complex process if calculations of pavement response are made manually. In fact, layered systems with greater than three layers are essentially impossible to treat by hand. Thus, various computer programs have been developed which can perform the necessary calculations to generate the required stresses, strains and deflections due to an imposed loading condition.

At least five such computer programs are available (some on a limited basis) which can be utilized. Detailed descriptions and examples of these programs were developed by Terrel [6] for WSDOT. These five computer programs are:

1. CHEVRON NLAYER (updated version of CHEV5L)
2. CHEV5L WITH ITERATION
3. SHELL BISAR
4. ELSYM5
5. PSAD2A

Computer programs 1, 3 and 4 are currently available at UW.

For the purposes of this study, both the CHEVRON NLAYER and BISAR programs were used. The Chevron Research Company is acknowledged for providing the new NLAYER program and the BISAR program was obtained from the Shell Oil Company, Houston, Texas. The detailed capabilities and differences between these two programs need not be detailed here as this information is available elsewhere [6]. However, it should be pointed out that the CHEVRON NLAYER program allows only one input load (wheel load) and up to 15 pavement layers for each cross section while BISAR provides for up to 10 wheel loads and 10 pavement layers. Thus, it is preferable to use BISAR when multiple wheel loads are being considered. An additional advantage of BISAR is that the output for stresses, strains and deflections is presented in both the cylindrical and cartesian coordinate systems. The CHEVRON NLAYER program utilizes only the cylindrical coordinate system thus making superposition of multiple wheel loads quite difficult due to the lengthy required hand calculations. The primary disadvantage of BISAR is that greater amounts of CPU time are required - hence, increased analysis costs.

SELECTION AND CHARACTERIZATION OF SR 12 PAVEMENT STRUCTURES

In order to utilize the capabilities of the two described programs, a number of pavement cross sections were identified for modeling. The cross sections at six stations were chosen primarily because they represent some of the more "critical" conditions encountered along the haul route. Conditions considered were depths to weak subgrade layers, thickness of weak layers and layer strengths. Additionally, these stations represent the major kinds of conditions which can vary significantly, e.g., asphalt base course as opposed to cement-treated base.

Figures 21 through 24 were used to represent the various pavement cross sections. Initially, cross sections at seven stations were chosen (as shown) but station 602+50 was subsequently not used since the cross section utilized at Station 584+00 was similar and slightly more critical. The information shown in these figures include the layer number, layer thickness and description, and elastic modulus value or relationship.

Station 173+25

Layer No.	Thickness	Material	Modulus (psi)
1	9.0"	Asphalt Concrete (Class B)	Table 3 & Figure 3
2	3.6"	Untreated Crushed Top Course	$5840\ \theta^{0.26}$
3	108.0"	Very Dense Gravel, Sand and Silt with Cobbles (Fill)	$5840\ \theta^{0.26}$
4	60.0"	Medium Gravelly Sand	6,000
5	120.0"	Loose Sandy Silt to Silty Sand with Organic	3,500
6	∞	Loose to Medium Sand and Gravel to Sandy Silt	4,000

Station 202+00

Layer No.	Thickness	Material	Modulus (psi)
1	10.0"	Asphalt Concrete (Class B)	Table 3 & Figure 4
2	3.6"	Untreated Crushed Top Course	$5840\ \theta^{0.26}$
3	48.0"	Very Dense Silt, Sand, Gravel and Cobbles (Fill)	$5840\ \theta^{0.26}$
4	108.0"	Medium to Very Dense Silt, Sand and Gravel	9,000
5	60.0"	Medium to Dense Sand and Gravel	7,000
6	48.0"	Loose Silt with Organic and Peat	2,000
7	∞	Soft Sandstone	25,000

Figure 21. Cross Sections for Stations 173+25 and 202+00.

Station 341+50

Layer No.	Thickness	Material	Modulus (psi)
1	9.0"	Asphalt Concrete (Class B)	Table 3 & Figure 5
2	3.6"	Untreated Crushed Top Course	$5840\ \theta^{0.26}$
3	24.0"	Very Dense Gravel and Sandy Silt (Fill)	$5840\ \theta^{0.26}$
4	12.0"	Dense Sand and Gravel	9,000
5	24.0"	Medium Sandy Silty Clay	3,500
6	12.0"	Dense Sand and Gravel	9.000
7	∞	Very Dense Silty Fine to Coarse Sand and Gravel with Sand Lenses	12,000

Station 408+00

Layer No.	Thickness	Material	Modulus (psi)
1	9.0"	Asphalt Concrete (Class B)	Table 3 & Figure 5
2	1.8"	Untreated Crushed Top Course	$5840\ \theta^{0.26}$
3	3.6"	Untreated Gravel Base (Class B)	$5840\ \theta^{0.26}$
4	24.0	Dense Gravelly Sandy Silt	7,000
5	204.0"	Loose Clayey Silt to Silty Clay	3,000
6	∞	Dense to Very Dense Sand and Gravel	12,000

Figure 22. Cross Sections for Stations 341+50 and 408+00.

Station 453+60

Layer No.	Thickness	Material	Modulus (psi)
1	9.0"	Asphalt Concrete (Class B)	Table 3 & Figure 5
2	1.8"	Untreated Crushed Top Course	$5840\ \theta^{0.26}$
3	12.0"	Dense Sandy Silty Gravel (Fill)	$5840\ \theta^{0.26}$
4	24.0"	Dense Gravelly Silt	7,000
5	36.0"	Loose to Medium Gravelly Silt with Organic	3,000
6	36.0"	Very Loose Clayey Silt	1,500
7	48.0"	Medium to Dense Sand and Gravel	7,000
8	∞	Loose to Medium Gravelly Sand	4,000

Station 584+00

Layer No.	Thickness	Material	Modulus (psi)
1	3.0"	Asphalt Concrete (Class B)	Table 3 & Figure 6
2	6.0"	Cement Treated Base	3,000,000
3	24.0"	Medium to Very Dense Sand and Gravel (Fill)	$5840\ \theta^{0.26}$
4	96.0"	Soft to Medium Clay, Very Loose to Medium Silt, Silt with Organic and Peat	2,000
5	∞	Medium to Very Dense Sand and Gravel	9,000

Figure 23. Cross Sections for Stations 453+60 and 584+00.

Station 602+50

Layer No.	Thickness	Material	Modulus (psi)
1	3.0"	Asphalt Concrete (Class B)	Table 3 & Figure 6
2	6.0"	Cement Treated Base	3,000,000
3	48.0"	Very Dense Sand and Gravel (Fill)	$5840\ \theta^{0.26}$
4	96.0"	Soft to Medium Clay, Very Loose to Medium Silt with Organic and Peat	2,000
5	∞	Medium to Very Dense Sand and Gravel	9,000

Figure 24. Cross Section for Station 602+50.

For the six primary cross sections analyzed, a minimum of five pavement layers (Station 584+00) and a maximum of eight (Station 453+60) were utilized. The thicknesses of the various pavement layers varied considerably with the exception of the ACP and CTB layers. Generally, the total ACP thickness was about 9 in for the non-CTB stations and about 3 in for the CTB stations. The CTB appeared to be uniformly 6-in thick as called for in the original pavement design. Approximately 23 percent of the proposed haul route along SR 12 contains CTB. The remaining layer thicknesses and descriptions contained in Figures 21 through 24 were developed from the WSDOT boring logs and blow counts. The layers as characterized represent the best estimate of the actual layers at each station.

The elastic parameters (resilient or elastic modulus and Poisson's ratio) developed for each pavement layer was a difficult task. This primarily occurred due to the complex and variable nature of the subgrade soils underlying each of the pavement sections. The reliability of the modulus values used decreases with increasing depth, i.e., the modulus values for the ACP and gravel fill materials are considered to be reasonable since these values and relationships were determined in the laboratory. The modulus values for the subgrade soils are certainly less reliable since they were not determined in the laboratory but instead estimated from correlations available from published sources.

The modulus for the ACP layer at each station was obtained from either Table 3 (resilient modulus statistical summary) or Figures 3 through 6. The figures were used if varying temperature conditions required. Table 3 values were used for modeling various load configurations at the "standard" ACP temperature of 25°C (77°F). This statistical summary provides the required data necessary to calculate a confidence interval for resilient modulus data. An 80 percent confidence interval was used for each of the stations, i.e., approximately 80 percent of the resilient modulus values at a specific station would be expected to be higher. The following values resulted for the six stations:

Station	Resilient Modulus (psi) (80% confidence interval)
173+25	366,000
202+00	201,000
341+50	301,000
408+00	301,000
453+60	301,000
584+00	355,000

For the gravel fill material which underlies all SR 12 stations (but of varying thickness), the resilient modulus relationship described in Chapter II was used. Granular materials such as these are stress dependent in that the resilient modulus changes with changes in the material stress state. Hence, resilient modulus is a function of bulk stress. The same relationship for both the gravel fill and crushed top course materials was used.

As previously stated, the moduli of the subgrade materials were primarily developed from published correlations. These correlations consist of expected ranges of modulus for typical material descriptions and blow counts (standard penetration test). Two such correlations are shown in Tables 8 and 9. Table 8 contains such information for sands and Table 9 for clays. Table 8 was used since the vast majority of the subgrade soils are sands, silts, gravels or combinations of the three.

In order to verify the modulus values used for each of the six cross sections, the plate bearing data obtained in the field and the BISAR computer program were used. At each station the maximum plate bearing load was entered into the computer program for both the 12-in and 24-in diameter plates. Each cross section was represented by the modulus values and thicknesses as previously described. Since the modulus of the ACP layer is dependent upon temperature, test locations were chosen for which reasonable estimates of the overall pavement temperatures could be made. The calculated results from the BISAR program for pavement surface deflection were then compared to the values obtained in the field from the plate bearing test. The results of these comparisons are shown in Table 10. In general, good agreement was achieved with a few notable exceptions. Thus, an independent confirmation of the modeled cross sections was achieved.

For the cases where major differences occur between calculated and measured pavement surface deflections in Table 10, brief comment is appropriate. For Station 408+00, the calculated (BISAR) surface deflection for the 24-in plate is 0.103-in with a measured deflection of 0.026-in. At the same station and for the 12-in plate, excellent agreement was achieved. Additionally, the adjacent station (453+60) has a somewhat similar and equally critical cross section. Interestingly,

Table 8. Relative Density of Sands by Standard Penetration Test and Corresponding Modulus of Elasticity [Refs. 7,9].

No. of Blows (N)	Relative Density	Modulus of Elasticity Range (psi)
0-4	Very Loose	-
4-10	Loose	1,500-3,500
10-30	Medium	-
30-50	Dense	7,000-12,000
Over 50	Very Dense	-

Table 9. Consistency of Clay by Standard Penetration Test and Corresponding Modulus of Elasticity [Refs. 7,8].

No. of Blows (N)	Consistency	Modulus of Elasticity Range (psi)
<2	Very Soft	50-400
2-4	Soft	250-600
4-8	Medium	600-1,200
8-15	Stiff	-
15-30	Very Stiff	-
>30	Hard	1,000-2,500

Table 10. Comparison of Measured and Calculated Plate Bearing Deflections for the 12 and 24-in Diameter Plates.

Station	12-in. Diameter Plate			24-in. Diameter Plate		
	Load (lbs.)	Measured Deflection (in.)	Calculated* Deflection (in.)	Load (lbs.)	Measured Deflection (in.)	Calculated* Deflection (in.)
173+25 (OWP)	19,600	0.033	0.039	27,400	0.048	0.051
202+00 (OWP)	19,600	0.032	0.034	27,400	0.016	0.0435
341+50 (OWP)						
341+50 (BWP)	19,600	0.067	0.046	27,400	0.035	0.057
408+00 (BWP)	19,600	0.051	0.055			
408+00 (OWP)				39,000	0.026	0.103
453+60 (OWP)	19,600	0.084	0.064			
453+60 (BWP)				35,200	0.100	0.106
584+00 (OWP)	19,600	0.006	0.034	41,000	0.005	0.069

* BISAR Computer Program

the calculated surface deflections for the 24-in plate at both of these stations are slightly greater than 0.100-in with the measured and calculated deflections at Station 453+60 being of excellent agreement. Thus, the observed differences between calculated and measured values for Station 408+00 are not considered significant. The remaining major differences between measured and calculated surface deflections occurs at Station 584+00 (a CTB section). The measured surface deflections are significantly lower than calculated. This is probably due to the fact that the subgrade soils are unusually variable near this station. Thus the cross section is probably modeled as containing weaker subgrade layers than in fact occur. It is also notable that a soil boring was not drilled at Station 584+00.

LOAD CONFIGURATIONS

Two types of heavy haul loads are considered in this analysis: (1) the two trailers used to carry the steam generators and nuclear reactor vessel and (2) the prime mover vehicle. Both the trailer and the prime mover have unique numbers of wheels and/or wheel loads.

TRAILER

The two trailers used to carry the reactor vessel and steam generators (each unit will be moved separately) is unique in that a total of 384 separate wheels will be utilized. Figure 25 is a plan view of one half of the overall trailer system and is composed of 12 axle sets of 16 wheels each (192 wheels total).

To input these loads into a layered elastic computer program, first the "critical location" had to be found as well as determination of the number of wheels which should be used. All 192 wheels could not

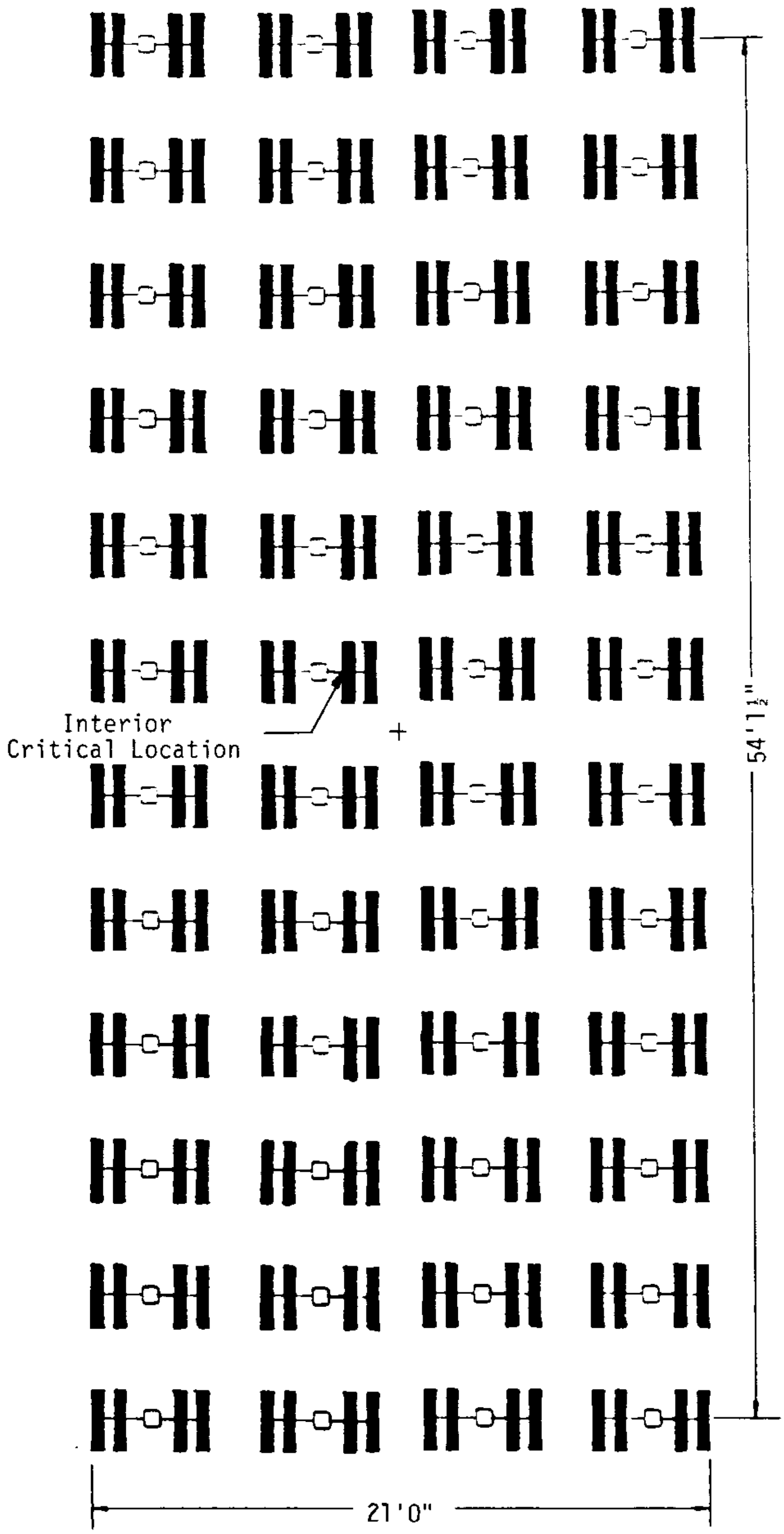

Figure 25. Plan View of One Trailer Unit (192 Wheels).

be inputted since the number and expense of the computer runs for each case studied would be prohibitive.

The "critical location" was determined by using the CHEVRON NLAYER program and this point is shown in Figure 25. The primary criteria for locating this point was horizontal tensile strain at the bottom of the ACP and vertical compressive sugrade strain at the top of the subgrade. Conceptually, these locations are shown in Figure 26 for an idealized three layer pavement.

The number of wheels which should be treated for the trailer case was 15 with these wheel locations clustered around the "critical point". This was determined with the CHEVRON NLAYER program. Contributions of other wheel loads further from the "critical point" than these 15 did not contribute significantly to the cumulative stress or strain condition. In fact, some wheels tend to reduce the net stress and strain at the critical point for some loading conditions.

The expected wheel load for each wheel on the trailers is 5675-lbs (steam generator move) and hence was the primary wheel load used in the analysis. Other wheel loads were examined which were higher. The expected total weight of a steam generator move including trailer weight is approximately 1089-tons (2,178,000-lbs).

PRIME MOVER

It is understood that two prime movers will be used for each haul - one to pull and one to push. These vehicles are heavily ballasted to increase tire contact friction with the pavement. The original information received indicated that a total prime mover load of 84-tons distributed on 10 wheels should be used. Later, WSDOT was informed that a total of

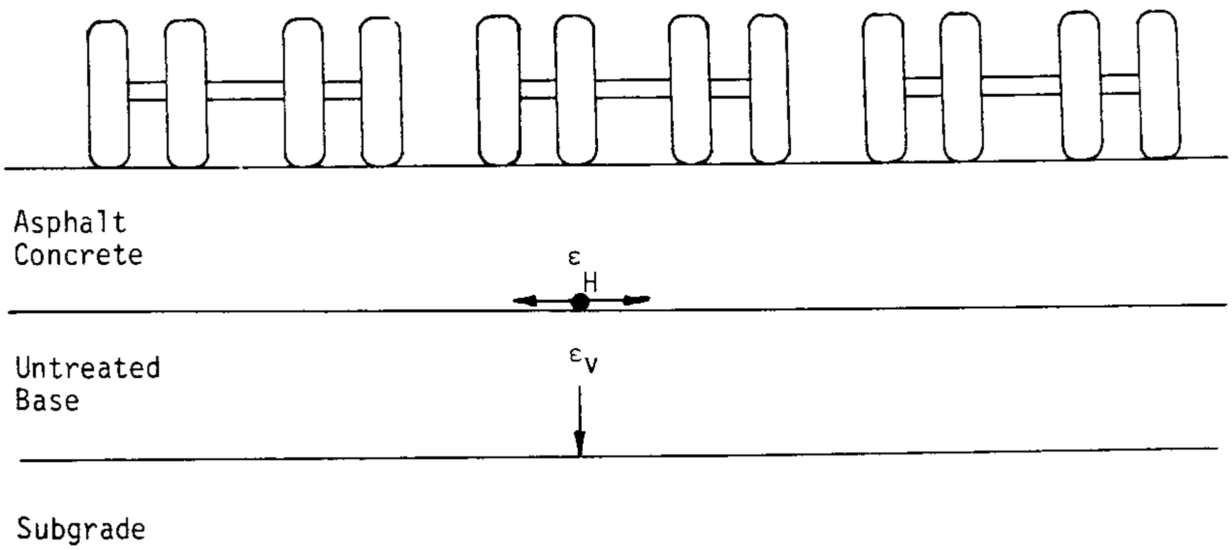

Figure 26. Location of Maximum Horizontal Tensile Strain and Vertical Compressive Strain for Multi-Wheel Configuration.

half that amount would be adequate for the SR 12 portion of the haul. Hence, two cases were studied for this vehicle: (1) wheel loads ranging from 16,000 to 17,000-lbs (84-tons total) and (2) wheel loads ranging from 8,000 to 8,500-lbs (42-tons total). A plan view of the prime mover wheel configuration is shown in Figure 27, which also indicates the "critical location".

LOAD CONFIGURATION SCENARIOS

Several different loading conditions were utilized in the analysis and included the following:

1. Trailer
   - (a) 5675-lb wheel load, ACP temperature of 77°F
   - (b) 5675-lb wheel load, ACP temperature of 90°F
   - (c) 7000-lb wheel load, ACP temperature of 77°F
   - (c) 8000-lb wheel load, ACP temperature of 77°F
2. Prime Mover
   - (a) 84-tons total
   - (b) 42-tons total
3. Standard Axle
   - (a) 18,000-lb dual-tired single axle, ACP temperature of 77°F

The reasons for scenarios 1 and 2 are obvious but possibly not for 3. By calculating the estimated stresses, strains and deflections for a "standard axle" the relative effects of the heavy hauls can be better evaluated.

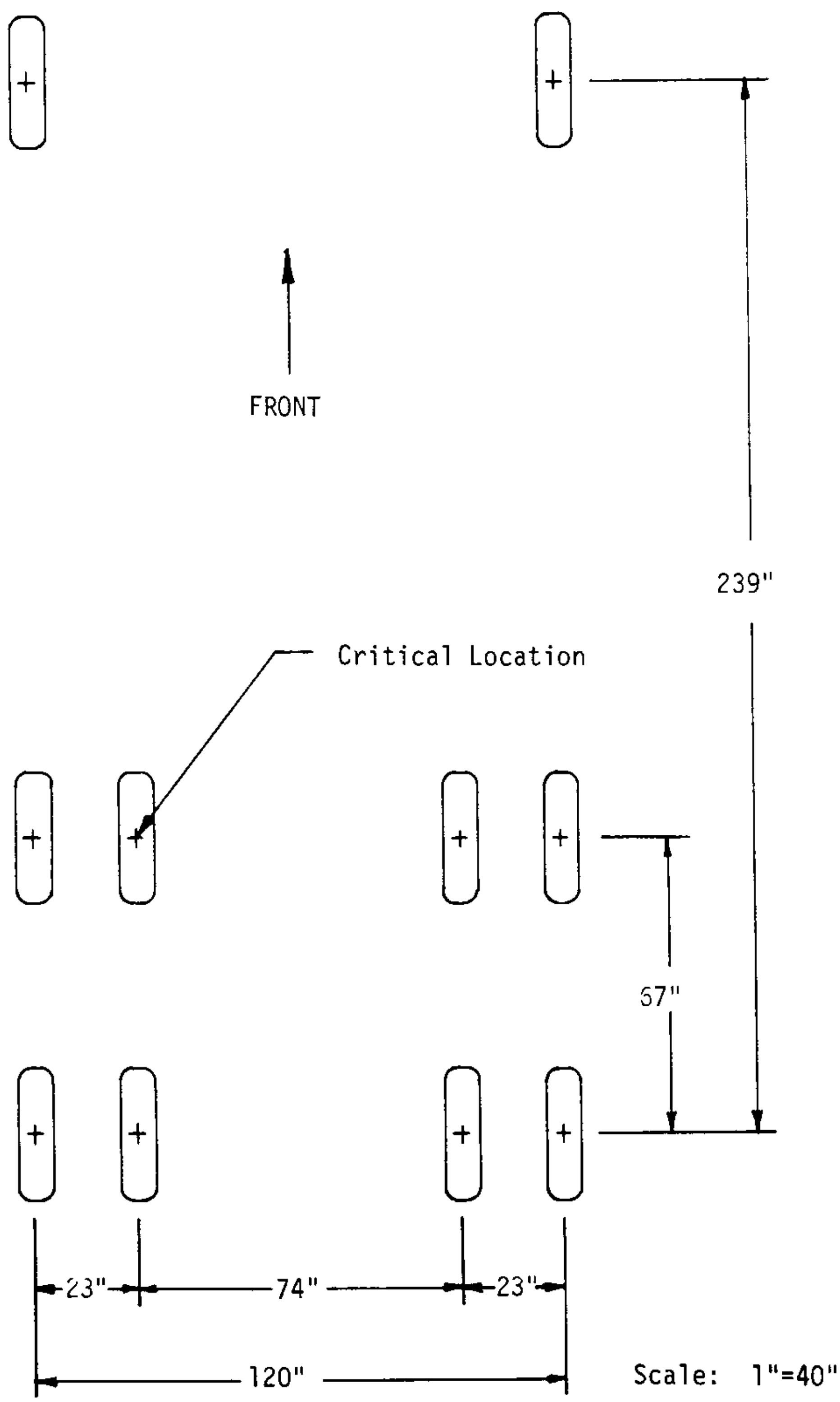

Figure 27. Plan View of Prime Mover Wheel Configuration.

LIMITING VALUES AND FAILURE CRITERIA

The results obtained from modeling the SR 12 pavement structures and determining the stresses, strains and deflections which result from various loading configurations would be of little value if some type of limiting criteria could not be utilized. Such criteria may (and does) include limiting tensile strains for the ACP, vertical compressive strains in the subgrade layers and tensile stresses and strains in the CTB. Various and often significantly different values and/or relationships have been previously developed and reported by others. The criteria selected for use in this study will be described in this section.

The limiting values and failure criteria used fall into the following groups:

1. Fatigue
   (a) Tensile strain at the bottom of the ACP layer
   (b) Tensile stress and strain at the bottom of the CTB layer
2. Rutting
   (a) Vertical compressive strain at the top of the subgrade layers
3. Strength
   (a) Tensile strength of CTB layer

Other criteria could have been used in the analysis but those stated above appeared to be the most significant. Each of these three criteria will be separately discussed in the sections which follow.

FATIGUE

Extensive work has been accomplished in various laboratories which have resulted in tensile stress or strain relationships as a function of the number of loads to failure for asphalt concrete materials. It is difficult to obtain a true consensus among engineers as to how well these laboratory derived relationships duplicate actual, in-service asphalt concrete pavements. Many research engineers believe that laboratory derived fatigue relationships underestimate fatigue life by more than an order of magnitude.

In an attempt to avoid some of the possible pitfalls associated with laboratory fatigue relationships, three fatigue relationships were obtained from available literature which were developed directly from AASHO Road Test data or laboratory results based on field-sawed specimens obtained from the Washington State University (WSU) test track at Pullman, Washington. All three of these relationships are shown in Figure 28.

The relationship which provides the most conservative estimates for limiting horizontal tensile strains contained in Figure 28 was developed by Austin Research Engineers for the Federal Highway Administration (FHWA) and is utilized in an overlay design procedure [9]. Data from 27 AASHO Road Test sections which included traffic repetitions and predicted tensile strains were utilized in the development of this relationship.

A much less conservative relationship was developed by Finn, et al. for the use in the PDMAP computer program which can be used to predict fatigue cracking and permanent deformation [10]. From AASHO Road Test data a shift-factor relationship was developed which predicts repetitions to failure for input values of predicted initial tensile strain and modulus.

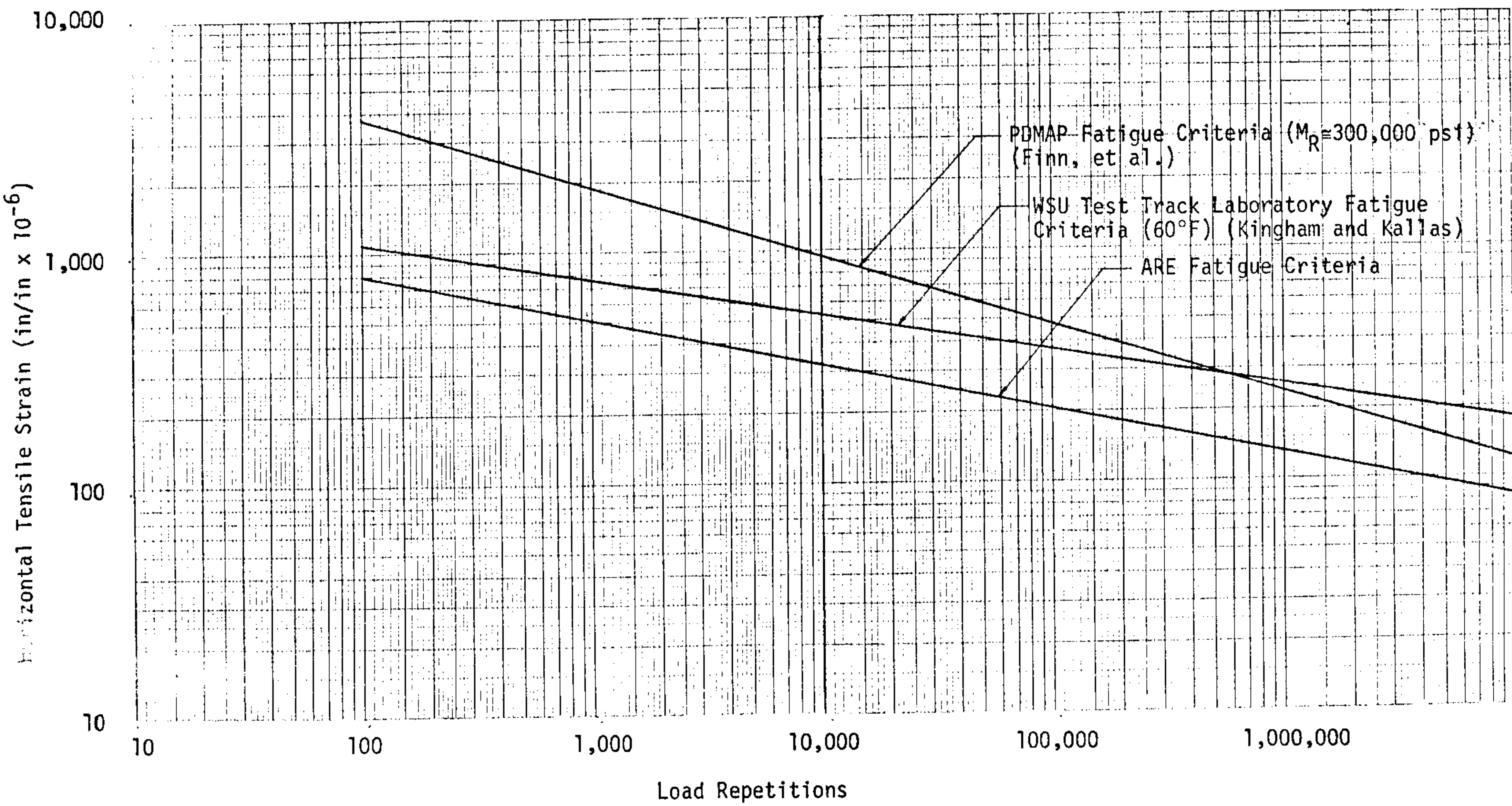

Figure 28. Fatigue Criteria for Horizontal Tensile Strain at the Bottom of the ACP Layer.

Finally, the laboratory fatigue relationship developed by Kingham and Kallas of the Asphalt Institue [11] tends to fall between the two previously described fatigue relationships. This relationship was obtained from testing flexural beams sawed from one of the experimental test rings constructed at WSU.

Fatigue is also an important consideration for CTB materials. Available information is somewhat sparse and further complicated by the wide range of CTB material properties reported in the literature. Criteria developed by Pretorius, Monismith, et al. [12] for CTB similar to that for SR 12 resulted in fatigue relationships as a function of strain and stress. Both of these relationships are shown in Figures 29A and 29B.

RUTTING

Various researchers have found that flexible pavement rutting can be reduced or prevented by limiting the vertical compressive strain in subgrade layers. By limiting this strain the cumulative permanent deformation is limited, hence reducing rutting. Three different estimates of rutting criteria are shown in Figure 30.

The first rutting criterion was presented by Dorman and Metcalf [13] in 1964 and was incorporated into the Shell flexible pavement design system. Later, Monismith, et al. [14] developed and presented criteria applicable to California pavements. Also shown in Figure 30 is the rutting criteria developed by Witczak [15] which was incorporated into the Asphalt Institute airfield pavement design procedure. The criteria developed by Monismith was subsequently used in the analysis since it was felt to best represent conditions in Washington.

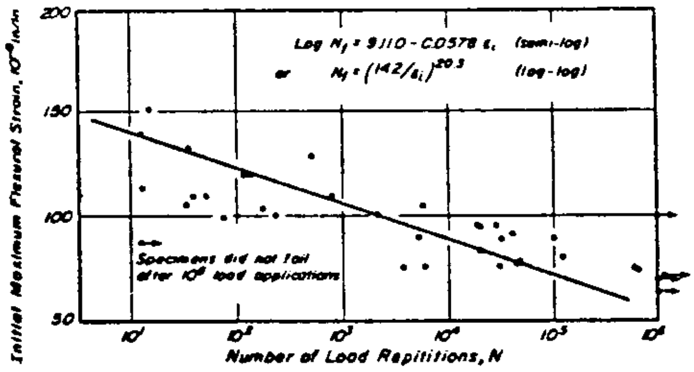

Figure 29A. Initial Strain Versus Repetitions to Fracture, Flexural Specimens [12].

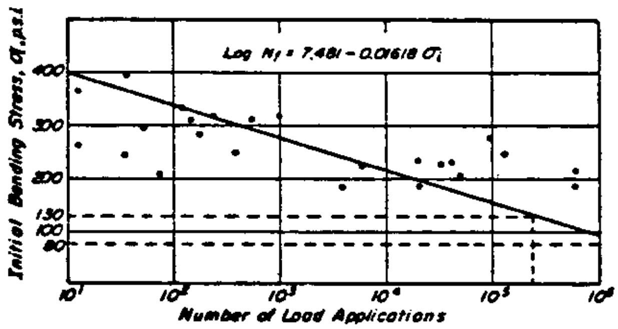

Figure 29B. Stress Versus Repetitions to Fracture, Flexural Specimens [12].

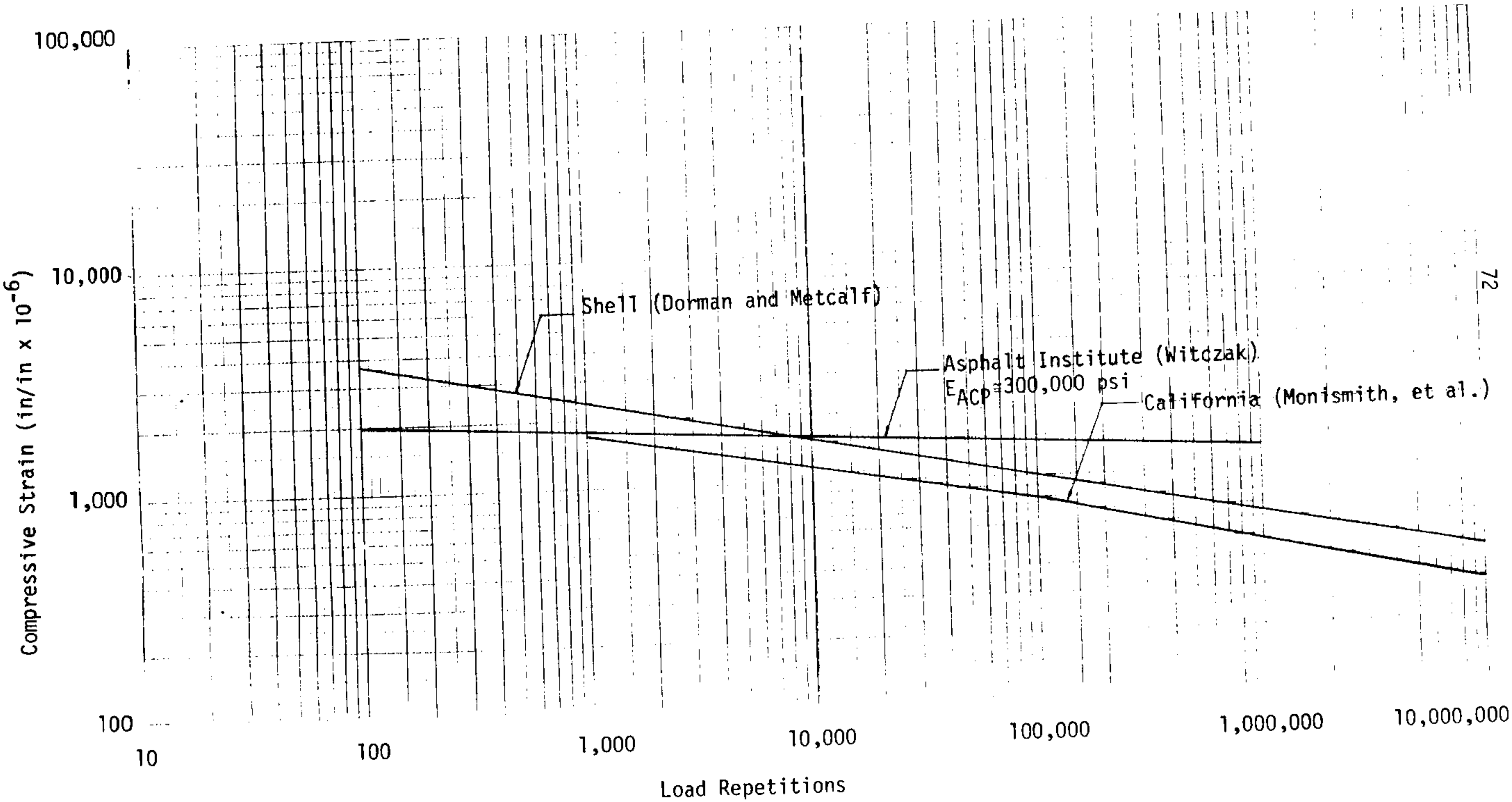

Figure 30. Criteria for Vertical Compressive Subgrade Strain to Minimize Rutting.

STRENGTH

A limiting value of strength was applied to the CTB layers evaluated for SR 12. Since CTB is a brittle material, excessive stresses induced by the heavy hauls could result in cracking and ultimately accelerated deterioration of the overall pavement structure. If calculated stresses exceed the CTB indirect tensile strenghts shown in Chapter II (Table 5), modifications to the applied loads or pavement structure might be appropriate.

PAVEMENT EVALUATION RESULTS

The preceding data and discussion presented in this report can now be used to estimate potential damage which may occur to the SR 12 pavement structures due to the planned heavy hauls. To accomplish this task, the modeled pavement cross sections and haul loads were inputted into the BISAR computer program thus providing estimates of the resulting stresses, strains and deflections. These data were then compared to appropriate failure criteria to estimate pavement damage.

The results obtained from the BISAR program are summarized in Tables 11 through 16 for the trailer load configuration, Tables 17 through 20 for the prime mover and Tables 21 and 22 for an 18,000-lb dual-tire single axle. Each set of tables will be separately discussed. The general format used in these tables to present the calculated results is similar. Only the vertical and maximum horizontal stresses and strains are presented for pavement layers of direct interest. Layer thickness, number and the deflection of the pavement surface are shown in the center of each table. Specific layer descriptions can be obtained from Figures 21 through 24.

Tables 11 through 13 contain the results obtained for the 5675-lb trailer wheel load configuration and ACP temperature of 77°F (typical spring or fall day). The calculated pavement surface deflections range from a low of 0.072-in (Station 202+00) to a high of 0.151-in (Station 453+60). The maximum tensile strain at the bottom of the ACP layer (Stations 173+25 to 453+60) is 341 x 10^{-6} in/in and occurs at Station 408+00. The maximum tensile stress for the CTB layer (Station 584+00) is 367 psi which is quite high for this type of material. The largest vertical compressive strains for the subgrade soils occur at Stations 408+00, 453+60 and 584+00, all of which slightly exceed 1000 x 10^{-6} in/in.

Table 14 shows the result of changing the ACP temperature to 90°F (typical of a moderate summer day) and using the same 5675-lb trailer wheel load configuration. This analysis was performed for Station 408+00 since it was felt to represent one of the more critical stations evaluated. Comparing the results shown in Table 14 to those in Table 12 for this station shows that changing the ACP temperature does not significantly affect the surface deflection. Subgrade vertical compressive strains are increased somewhat and horizontal tensile strain at the bottom of the ACP is increased significantly (from 341 to 443 x 10^{-6} in/in). This indicates that moving the trailers on SR 12 during a hot day (high ACP temperature) is probably undesirable. This will be more fully examined later in this section.

The results developed in Tables 15 and 16 were also for the trailer load configuration (Station 408+00) but the wheel loads were increased

Table 11. Calculated Stresses, Strains and Surface Deflection for Trailer with 5675-lb Wheel Load and ACP Temperature of 77°F - Stations 173+25 and 202+00.

Station 173+25

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.096"		Maximum Horizontal Strain	Vertical Strain
-100	-180	1	9.0"	-285	2
-9	129			257	-189
-9	-4	2	3.6"	257	-723
		3	108.0"		
-1	0	4	60.0"	116	-236
0	0	5	120.0"	82	-190
		6	∞		

Tension (+)
Compression (-)

Station 202+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.072"		Maximum Horizontal Strain	Vertical Strain
-100	-147	1	10.0"	-391	-90
-10	81			340	-252
-10	-4	2	3.6"	340	-846
		3	48.0"		
-4	-1	4	108.0"	159	-328
		5	60.0"		
-1	0	6	48.0"	63	-188
		7	∞		

Tension (+)
Compression (-)

Table 12. Calculated Stresses, Strains and Surface Deflection for Trailer with 5675-lb Wheel Load and ACP Temperature of 77°F - Stations 341+50 and 408+00.

Station 341+50

Stress (psi)		Layer Thickness, Number and Surface Deflection	Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.075"	Maximum Horizontal Strain	Vertical Strain
-100	-172	1 9.0"	-325	-14
-10	112		306	-224
-10	-4	2 3.6"	306	-802
		3 24.0"		
-5	-1	4 12.0"	238	-490
-4	-2	5 24.0"	295	-737
		6 12.0"		
		7 ∞		

Tension (+)
Compression (-)

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection	Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.136"	Maximum Horizontal Strain	Vertical Strain
-100	-194	1 9.0"	-373	34
-9	128		341	-257
-9	-3	2 1.8"	341	-815
		3 3.6"		
-7	-2	4 24.0"	334	-729
-4	-1	5 204.0"	460	-1017
		6 ∞		

Tension (+)
Compression (-)

Table 13. Calculated Stresses, Strains and Surface Deflection for Trailer with 5675-lb Wheel Load and ACP Temperature of 77°F - Stations 453+60 and 584+00.

Station 453+60

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.151"		Maximum Horizontal Strain	Vertical Strain
-100	-192	1	9.0"	-370	32
-10	123			329	-244
-10	-3	2	1.8"	329	-812
		3	12.0"		
-5	-1	4	24.0"	287	-663
-3	-1	5	36.0"	443	-929
-2	-1	6	36.0"	425	-1023
		7	48.0"		
		8	∞		

Tension (+)
Compression (-)

Station 584+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.122"		Maximum Horizontal Strain	Vertical Strain
-100	-129	1	3.0"	-220	-60
-89	-82			-84	-111
-89	-307	2	6.0"	-84	-1
-5	367			106	-37
-5	-2	3	24.0"	106	-398
-2	-1	4	96.0	322	-1021
		5	∞		

Tension (+)
Compression (-)

Table 14. Calculated Stresses, Strains and Surface Deflection for Trailer with 5675-lb Wheel Load and ACP Temperature of 90°F - Station 408+00.

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.142"		Maximum Horizontal Strain	Vertical Strain
-100	-175	1	9.0"	-498	1
-10	108			443	-330
-10	-3	2	1.8"	443	-979
		3	3.6"		
-8	-3	4	24.0"	410	-837
-4	-1	5	204.0"	479	-1067
		6	∞		

Tension (+)
Compression (-)

Table 15. Calculated Stresses, Strains and Surface Deflection for Trailer with 7000-lb Wheel Load and ACP Temperature of 77°F - Station 408+00.

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.167"		Maximum Horizontal Strain	Vertical Strain
-100	-219	1	9.0"	-438	78
-11	155			413	-310
-11	-3	2	1.8"	411	-988
		3	3.6"		
-9	-3	4	24.0"	409	-895
-5	-1	5	204.0"	566	-1258
		6	∞		

Tension (+)
Compression (-)

Table 16. Calculated Stresses, Strains and Surface Deflections for Trailer with 8000-lb Wheel Load and ACP Temperature of 77°F - Station 408+00.

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.192"		Maximum Horizontal Strain	Vertical Strain
-100	-237	1	9.0"	-485	113
-12	190			462	-348
-12	-4	2	1.8"	462	-1114
		3	3.6"		
-10	-3	4	24.0"	464	-1020
-6	-1	5	204.0"	647	-1430
		6	∞		

Tension (+)
Compression (-)

to 7000-lbs (Table 15) and 8000-lbs (Table 16). The stress and strains in all critical layers increased, although these increases were not significantly higher for the ACP layer than reported in Table 14 for the 5675-lb wheel load configuration with an ACP temperature of 90°F. These higher wheel loads did significantly increase the surface deflections.

The estimates of the pavement response to the 84-ton prime mover vehicle are reported in Tables 17 through 19 for the six stations evaluated. The observed stresses, strains and surface deflections for each station generally were significantly increased as compared to the trailer load configuration. This should not be unexpected since the prime mover wheel loads are estimated to be as high as 17,000-lbs for the 84-ton gross vehicle weight. Of particular concern is the high horizontal tensile stress estimated for the CTB layer at Station 584+00 (529 psi).

After obtaining the initial estimated pavement response results for the 84-ton prime mover, the load configuration was changed to 42-tons and evaluated for Stations 408+00 and 453+60 (two most critical stations). This information is presented in Table 20. These results indicate that the significant stresses and strains would be reduced to less than reported for the 5675-lb trailer load configuration - a desirable situation.

Tables 21 and 22 present the estimated pavement response of Stations 408+00 and 584+00 for a 18,000-lb dual-tire single axle load (the "standard" loading configuration). The horizontal tensile strain at the bottom of the ACP layer and vertical compressive strains in the subgrade soils are significantly lower than calculated for the previously discussed

Table 17. Calculated Stresses, Strains and Surface Deflection for 84-ton Prime Mover and ACP Temperature of 77°F - Stations 173+25 and 202+00.

Station 173+25

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.142"		Maximum Horizontal Strain	Vertical Strain
-100 -14	-247 193	1	9.0"	-411 407	117 -334
-14	-5	2	3.6"	407	-1180
		3	108.0"		
		4	60.0"		
		5	120.0"		
		6	∞		

Tension (+)
Compression (-)

Station 202+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.098"		Maximum Horizontal Strain	Vertical Strain
-100 -16	-190 139	1	10.0"	-536 543	48 -466
		2	3.6"		
		3	48.0"		
		4	108.0"		
		5	60.0"		
		6	48.0"		
		7	∞		

Tension (+)
Compression (-)

Table 18. Calculated Stresses, Strains and Surface Deflection for 84-ton Prime Mover and ACP Temperature of 77°F - Station 341+50 and 408+00.

Station 341+50

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.098"		Maximum Horizontal Strain	Vertical Strain
-100	-232	1	9.0"	-463	110
-15	187			482	-400
		2	3.6"		
		3	24.0"		
		4	12.0"		
		5	24.0"		
		6	12.0"		
		7	∞		

Tension (+)
Compression (-)

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.176"		Maximum Horizontal Strain	Vertical Strain
-100	-259	1	9.0"	-527	163
-14	209			532	-439
		2	1.8"		
		3	3.6"		
		4	24.0"		
		5	204.0"		
		6	∞		

Tension (+)
Compression (-)

Table 19. Calculated Stresses, Strains and Surface Deflection for 84-ton Prime Mover and ACP Temperature of 77°F - Stations 453+60 and 584+00.

Station 453+60

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.207"		Maximum Horizontal Stress	Vertical Strain
-100	-256	1	9.0"	-519	159
-14	200			510	-422
		2	1.8"		
		3	12.0"		
		4	24.0"		
		5	36.0"		
		6	36.0"		
		7	48.0"		
		8	∞		

Tension (+)
Compression (-)

Station 584+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.154"		Maximum Horizontal Strain	Vertical Strain
-100	-160	1	3.0"	-239	-17
-91	-101			-125	-88
		2	6.0"		
-7	529			155	-50
-7	-3	3	24.0"	155	-537
-4	-2	4	96.0"	402	-1130
		5	∞		

Tension (+)
Compression (-)

Table 20. Calculated Stresses, Strains and Surface Deflection for 42-ton Prime Mover and ACP Temperature of 77°F - Station 408+00 and 453+60.

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.089"		Maximum Horizontal Strain	Vertical Strain
-100	-173	1	9.0"	-320	12
-8	123			309	-259
-8	-1	2	1.8"	309	-750
		3	3.6"		
-5	-1	4	24.0"	283	-592
-2	-1	5	204.0"	251	-590
		6	∞		

Tension (+)
Compression (-)

Station 453+60

Stress (psi)		Layer Thickness Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.105"		Maximum Horizontal Strain	Vertical Strain
-100	-175	1	9.0"	-316	9
-8	118			298	-251
-8	-2	2	1.8"	298	-757
		3	12.0"		
-4	-1	4	24.0"	226	-469
-2	0	5	36.0"	227	-526
-1	-1	6	36.0"	204	-569
		7	48.0"		
		8	∞		

Tension (+)
Compression (-)

Table 21. Calculated Stresses, Strains and Surface Deflection for 18,000-lb Dual-Tire Single Axle and ACP Temperature of 77°F - Station 408+00.

Station 408+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.036"		Maximum Horizontal Strain	Vertical Strain
-70	-128	1	9.0"	-239	12
-5	88			225	-177
-5	-1	2	1.8"	225	-499
		3	3.6"		
-3	-1	4	24.0"	208	-406
-1	0	5	204.0"	177	-332
		6	∞		

Table 22. Calculated Stresses, Strains and Surface Deflection for 18,000-lb Dual-Tire Single Axle and ACP Temperature of 77°F- Station 584+00.

Station 584+00

Stress (psi)		Layer Thickness, Number and Surface Deflection		Strain (in/in x 10^{-6})	
Vertical Stress	Maximum Horizontal Stress	Pavement Surface Deflection = 0.028"		Maximum Horizontal Strain	Vertical Strain
-70	-75	1	3.0"	-92	-75
-63	-51			-48	-94
-63	-172	2	6.0"	-48	-6
-2	208			62	-18
-2	-1	3	24.0"	62	-172
-1	0	4	96.0"	134	-278
		5	∞		

loading configurations. The same holds true for the CTB layer at Station 584+00 in that the horizontal tensile stress is estimated to be 208 psi as opposed to 367 psi for the 5675-lb wheel load trailer configuration.

Tables 23 through 25 present the final summaries which provide estimates of pavement damage for the non-CTB pavement sections.

Tables 23 and 24 indicate the estimated allowable repetitions to failure for the various load configurations used in this analysis. Table 23 presents these estimates for the fatigue criteria and Table 24 for rutting, both of which are evaluated at Station 408+00. The maximum horizontal tensile strain at the bottom of the ACP layer (Table 23) and the maximum vertical compressive strain in the subgrade soils (Table 24) are also shown.

An examination of the estimated repetitions to failure in Table 23 shows the large differences between the three criteria listed. The criteria developed by Kingham and Kallas from WSU test track are felt to best represent the SR 12 ACP. The rutting criteria developed for California conditions (Table 24) are also felt to best represent the SR 12 subgrade soil conditions.

The maximum expected number of wheel load applications at any point along the proposed SR 12 haul route would be 144 repetitions for the two trailer units (six separate moves and 24 axles for each move) and 24 repetitions for the prime movers. By dividing the maximum expected repetitions by the allowable repetitions for a specific loading configuration, an approximate estimate of the pavement life consumed can be made. Such calculations were made and listed in Table 25. Since a

Table 23. Estimated Repetitions to Failure for Various Load Configurations and Fatigue Criteria - Station 408+00.

Load Configuration	Maximum Horizontal Strain (in/in x 10^{-6})	Estimated Repetitions to Failure		
		ARE Fatigue Criteria[1]	PDMAP Fatigue Criteria[2] (Finn et al.)	WSU Test Track[3] Laboratory Fatigue (Kingham and Kallas)
Trailer(each wheel):				
5675-lb (ACP 77°F)	341	7730	312,610	146,010
5675-lb (ACP 90°F)	443	2000	132,130	31,370
7000-lb (ACP 77°F)	413	2880	166,420	47,370
8000-lb (ACP 77°F)	462	1610	115,070	24,510
Prime Mover(total wt.)				
84-ton (ACP 77°F)	532	800	72,330	10,700
42-ton (ACP 77°F)	309	12,860	432,370	260,550
Standard Axle:				
18,000-lb (ACP 77°F)	225	66,130	1,228,210	1,681,080

[1]Equation: $W_{18} = 9.7255 \times 10^{-5}\left(\frac{1}{\varepsilon}\right)^{5.1627}$

[2]Equation: $\log N_f(\leq 10\%) = 15.947 - 3.291 \log\left(\frac{\varepsilon}{10^{-6}}\right) - 0.854 \log\left(\frac{E}{10^{-3}}\right)$

[3]Equation: $\log N_f = -17.2278 + 5.87687 \log\left(\frac{1}{\varepsilon}\right) + 0.033594(\mathrm{Temp})$

Table 24. Estimated Repetitions to "Failure" for Various Load Configurations and Rutting Criteria - Station 408+00.

Load Configuration	Maximum Vertical Subgrade Strain (in/in x 10^{-6})	Estimated Repetitions to Failure		
		Shell (Dorman and Metcalf)	California (Monismith et al.)	Asphalt Institute (Witczak)
Trailer(each wheel):				
5675-lb (ACP 77°F)	-1017	100,000	25,000	>10,000,000
5675-lb (ACP 90°F)	-1067	80,000	18,000	>10,000,000
7000-lb (ACP 77°F)	-1258	31,000	7,000	>10,000,000
8000-lb (ACP 77°F)	-1430	16,000	3,000	100,000
Prime Mover(total wt.):				
84-ton (ACP 77°F)	-	-	-	-
42-ton (ACP 77°F)	-592	1,700,000	400,000	>10,000,000
Standard Axle:				
18,000-lb (ACP 77°F)	-406	10,000,000	2,300,000	>10,000,000

Table 25. Pavement Life Used by Heavy Hauls (Station 408+00).

Load Configuration	Percent of Pavement Life Consumed					
	Most Probable		Optimistic		Pessimistic	
	Fatigue	Rutting	Fatigue	Rutting	Fatigue	Rutting
Trailer:						
5675-lb (ACP 77°F)	0.099	0.576	0.046	-	1.863	-
5675-lb (ACP 90°F)	0.459	0.800	0.109	-	7.200	-
7000-lb (ACP 77°F)	0.304	2.057	0.087	-	5.000	-
8000-lb (ACP 77°F)	0.588	4.800	0.125	-	8.944	-
Prime Mover:						
84-ton (ACP 77°F)	0.224	-	0.033	-	3.000	-
42-ton (ACP 77°F)	0.009	0.006	0.006	-	0.187	-
Standard Axle:						
18,000-lb(ACP 77°F)	-	-	-	-	-	-

wide range in the fatigue criteria was observed, pavement life reductions were calculated for all three. The "most probable" category was developed for the Kingham and Kallas criteria, "optimistic" for the Finn, et al. developed criteria, and "pessimistic" for the ARE developed criteria. Small percentage reductions in the pavement life are estimated for the load configurations considered for the "most probable" and "optimistic" categories. Somewhat larger values are reported for the "pessimistic" category, although the largest reduction occurs for the 8000-lb wheel load on the trailer. Even this value is less than 10 percent.

For the CTB SR 12 pavement structure, the expected tensile stresses at the bottom of the CTB layer (Table 13) may exceed the material strength (Table 5). If this occurs the CTB layer will probably crack. Such cracks would normally be expected to migrate to the pavement surface. Thus, over a period of time these cracks would allow extra moisture to enter the pavement structure which would probably accelerate pavement deterioration.

CHAPTER IV. CONCLUSIONS

The following conclusions appear to be warranted:

1. The subgrade soils along the proposed haul route are highly variable in composition and strength.
2. The existing SR 12 pavement structure was well constructed and is currently in good structural condition. The asphalt concrete and cement treated base materials are of good to high quality and strength.
3. The most probable amount of damage (fatigue and rutting) expected for the non-cement treated base pavement sections due to the 5675-lb trailer wheel loads and the 42-ton prime mover are small - less than one to two percent of available pavement life. By increasing the trailer wheel loads to 8000-lb, approximately five to ten percent of the available pavement life may be used. An increase in both the wheel loads (trailer and/or prime mover) and pavement temperature will act together to produce greater losses in pavement life. To illustrate this point, an increase of pavement temperature of only 13°F (77°F to 90°F) for the 5675-lb trailer wheel load indicates that the loss in pavement life can increase by a factor of one to almost four depending on the failure criterion used.
4. Based on limited data, tensile stresses in the CTB layer due to the 5675-lb trailer wheel loads may exceed tensile strength. The

possibility therefore exists that cracking of the layer may occur. Such cracking would accelerate pavement deterioration and ultimate failure.

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APPENDIX A

Draft No. 6
November, 1978
R.L. Terrel

INDIRECT TENSILE TEST METHOD

FOR

RESILIENT MODULUS OF BITUMINOUS MIXTURES

1. Scope

2. Applicable Documents

3. Summary of Method

4. Significance and Use

The values of the resilient modulus and resilient Poisson's ratio can be used for bituminous paving mixture design, as a supplement to standard values already used. The resilient properties can also be used in layered elastic analysis and thickness design of pavements. The test method may further be used in research investigations such as evaluation of materials performance with time (e.g. exposure tests) since the procedure is non-destructive.

5. Apparatus

6. Specimens

7. Procedures

8. Calculations

9. Report

Report the average resilient modulus at temperatures of 41, 77, and 104^{o} F (5, 25, and 40^{o} C) for each load and load frequency used in the test.

10. Precision

The precision of the method is being established.

INDIRECT TENSILE TEST METHOD

FOR

RESILIENT MODULUS OF BITUMINOUS MIXTURES

ASTM DESIGNATION ______

1. Scope

1.1 This method covers procedures for preparing and testing laboratory fabricated or field recovered cores of bituminous mixtures to determine resilient modulus values using the repeated-load indirect tensile test. The procedure described covers a range of temperatures, loads, loading frequencies, and load durations. The minimum recommended test series consists of testing at 41, 77*, and 104° F (5, 25*, and 40° C) at a loading frequency of 0.33 to 1.0 Hz for each temperature. This recommended series will result in 9 test values for one specimen which can be used to evaluate the overall resilient behavior of the mixture.

2. Applicable Documents

2.1 ASTM Standards:

D 1559 Resistance to Plastic Flow of Bituminous Mixture Using Marshall Apparatus

D 1561 Preparation of Test Specimens of Bituminous Mixtures by Means of Kneading Compactor

D 3515 Hot-Mixed, Hot Laid Asphalt Paving Mixture

D 3496 Method for Preparation of Bituminous Mixture Cylindrical Specimens

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*or ambient laboratory temperature as appropriate

D 3387 Test for Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM).

3. Summary of Method

3.1 The repeated-load indirect tensile test for resilient modulus is conducted by applying compressive loads with a haversine, square wave, or trapezoidal wave form. The loads act parallel to and along the vertical diametral plane of a cylindrical specimen of asphalt concrete (Fig. 1) at a given temperature and loading frequency. The resulting recoverable horizontal deformation of the specimen is measured and used to calculate the resilient modulus of elasticity with an assumed value of Poisson's ratio or with a calculated value using the measured recoverable vertical deformation.

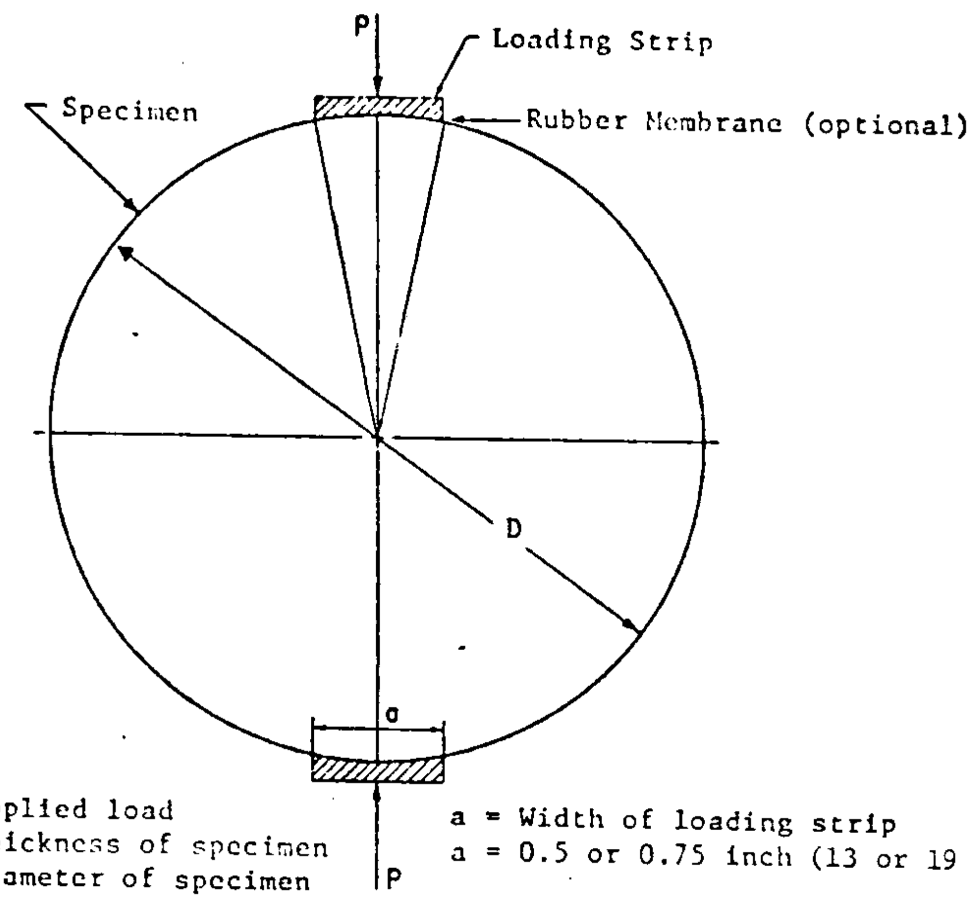
4. Significance and Use

4.1 The values of the resilient modulus and resilient Poisson's ratio can be used for bituminous paving mixture design, as a supplement to standard values already used. The resilient properties can also be used in layered elastic analysis and thickness design of pavements. The test method may further be used in research investigations such as evaluation of materials performance with time (e.g. exposure tests) since the procedure is non-destructive.

5. Apparatus

5.1 Testing machine - The testing machine should have the capability of applying a load pulse over a range of frequencies, load durations, and load levels.

Note 1 - An electro-hydraulic testing machine with a function generator capable of producing the prescribed wave form has been shown to be suitable for use in repeated-load indirect tensile testing; other commercially available or laboratory constructed testing machines such as those using pneumatic



P = Applied load
t = Thickness of specimen
D = Diameter of specimen

a = Width of loading strip
a = 0.5 or 0.75 inch (13 or 19 mm)

Fig 1. Indirect tensile test.

repeated loading can also be used. However, these machines may not have the load capability to handle larger specimens at the colder testing temperatures.

5.2 Temperature control system - The temperature control system should be capable of control over a temperature range. The temperature chamber should be large enough to hold an adequate number of specimens for a period of 24 hours prior to testing.

5.3 Measurement System - The measurement system should include a recorder or other measuring device for the horizontal and vertical deformations. If Poisson's ratio is to be assumed, then only horizontal deformations must be recorded. Loads should be measured and recorded or accurately calibrated prior to testing. The system should be capable of measuring deformations in the range of 0.00001 inches (0.00025 mm) of deformation. An alternate system could give deformation readout directly by suitable calibration of the loading and measurement components.

5.3.1 Recorder - The recorders should be independent of frequency for tests conducted up to 1.0 Hz.

5.3.2 Deformation Measurement - The values of vertical and horizontal deformation are measured by LVDT's or other suitable devices. The horizontal LVDT's should be at mid-height opposite each other on the specimens horizontal diameter. The sensitivity and type of measurement device should be selected to provide the deformation readout required in Section 4.3.

Note 2 - The Trans-TEX Model 350-000 LVTD and Statham UC-3 transducers have been found satisfactory for this purpose.

Note 3 - The gages should be wired to preclude the effects of eccentric loading so as to give the algebraic sum of the movement of each side of the specimen. Alternatively, each gage can be read independently and the results summed separately.

5.3.3 Load Measurement - Loads are measured with an electronic load cell capable of satisfying the specified requirements for load measurements in Section 5.3.

5.4 Loading Strip - A steel or aluminum curved-loading strip with radius equal to that of the test specimen is required to transfer the load from the testing machine to the specimen. The load strip shall be 0.5 or 0.75 inches (13 or 19 mm) wide for 4.0 or 6.0 inch (102 or 150 mm) diameter specimens, respectively; edges should be rounded in order to not cut the sample during testing. For specimens with rough textures, a thin hard rubber membrane attached to the loading strip has been found effective in reducing impact loading effects if vertical deformations are not monitored.

6. Specimens

6.1 Laboratory Molded Specimens - Prepare the laboratory molded specimens according to acceptable procedures such as ASTM Method D 1561. The specimens should have a height of at least 2 inches (50 mm) and a minimum diameter of 4 inches (102 mm), but not less than four times the maximum nominal size of the aggregate particles.

6.2 Pavement Cores - Core samples from an inservice pavement should have a minimum height of 1.5 to 2 inches (38 to 50 mm) and diameters of at least 4 inches (102 mm) but not less than four times the maximum nominal size of the aggregate particles. Cores should have relatively smooth parallel surfaces.

Note 4 - Laboratory molded specimens and pavement cores with diameters of 6 inches (150 mm) and heights of 3 inches (75 mm) or more have been used.

7. Procedures

7.1 Place test specimens in a controlled temperature cabinet and bring them to the specified test temperature. Unless temperature is monitored, and the actual temperature known, the specimens should remain in the cabinet at the specified test temperature for at least 24 hours prior to testing.

Note 5 - A dummy specimen with a thermocouple in the center can be used to determine when the desired test temperature is reached.

7.2 Place specimen into loading apparatus and position the steel or aluminum loading strips. Adjust and balance electronic measuring system as necessary.

7.3 Apply a preconditioning loading consisting of a repeated haversine, or other suitable waveform, loading to the specimen without impact for a minimum period sufficient to obtain uniform deformation readout. Depending upon the loading frequency, a minumum of 50 to 200 load repetitions is generally sufficient; however, the minimum for a given situation must be determined so that the resilient deformations are stable. A complete test will usually include measurements at three temperatures, e.g., 41 ± 2, 77 ± 2, and 104 ± 2^{o} F (5, 25, and 40^{0} C), at one or more loading frequencies, e.g., 0.33, 0.5, and 1.0 Hz, for each temperature. Recommended load range is from 10 to 50 percent of the tensile strength. Tensile strength can be determined from a destructive test on a specimen and the equation of Section 8.3.

Note 6 - Load duration is the more important variable and it is recommended that the duration be held to some minimum which can be recorded. The recommended range for load duration is 0.04 to

0.4 sec., with 0.1 sec. being representative of transient pavement loading. Recommended frequencies are 0.33 to 1.0 Hz. In lieu of tensile strength data, load ranges from 25 to 200 lbs.

7.4 Monitor the vertical and horizontal deformations during the test.

Note 7 - A typical load pulse-deformation trace is shown in Fig. 2, along with notations indicating the load-time terminology.

7.5 Each test should be completed within two minutes from the time specimens are removed from temperature control cabinet.

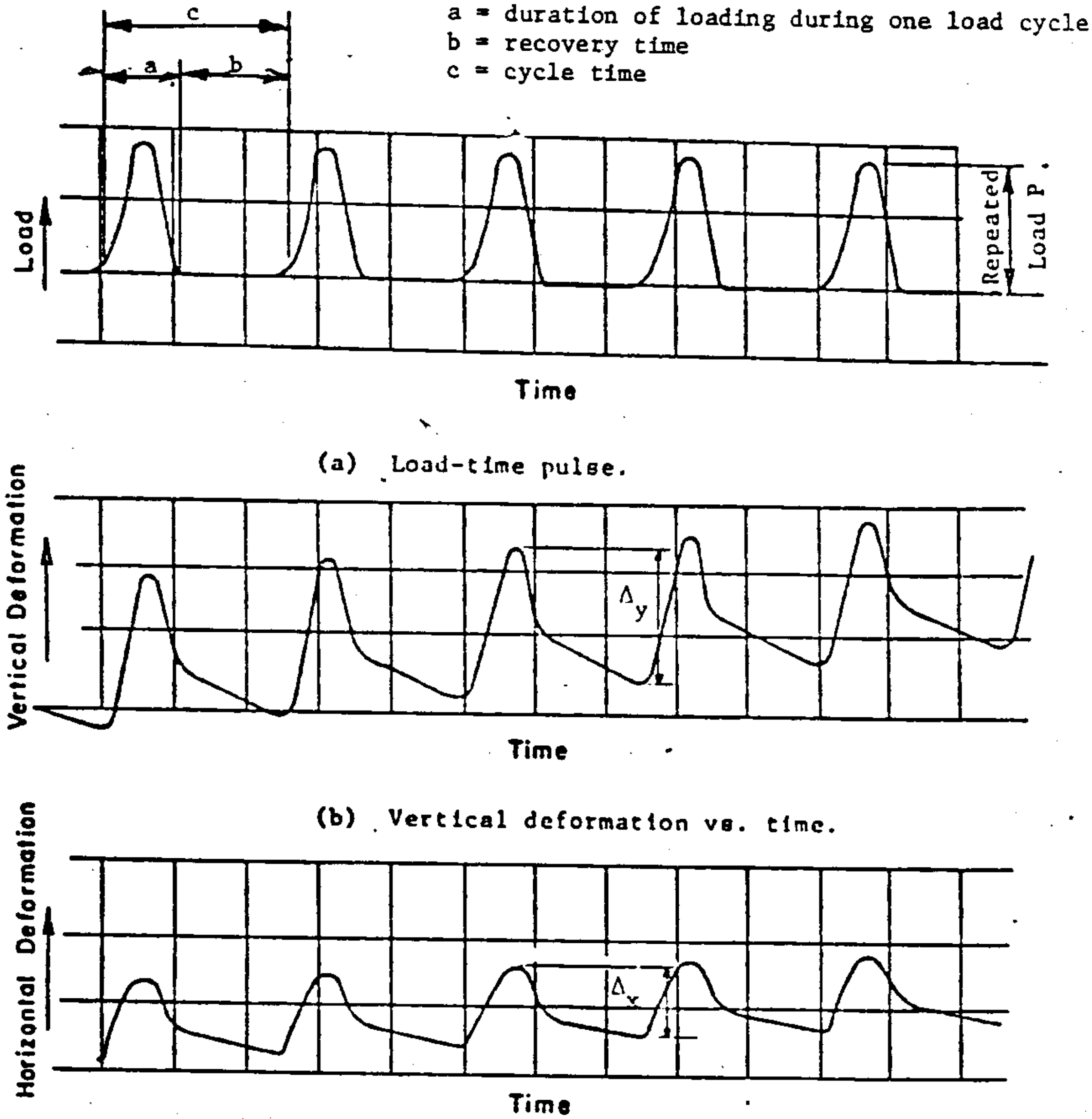
Note 8 - The two minute testing time limit is waived if loading is conducted within a temperature control cabinet meeting requirements in Section 5.2.

7.6 Each specimen should be tested more than once by rotating the specimen and loading through another diametral plane. Three laboratory fabricated specimens or three cores are recommended for a given test series with variables of temperature, load duration, and load. In order to reduce permanent damage to the specimen, testing should begin at the lowest temperature, shortest load duration, and smallest load. Subsequent testing on the same specimen should be for conditions producing progressively lower moduli. Bring specimens to specified temperature before each test.

Note 9 - If excessive total deformation, i.e., greater than 0.001 inch (0.0254 mm), occurs during a test, reduce the applied load, the test temperature, or both.

8. Calculations

8.1 Measure the average recoverable horizontal and vertical deformations over at least three loading cycles (see Fig. 2) after the repeated resilient deformation has become stable.



(a) Load-time pulse.

(b) Vertical deformation vs. time.

(c) Horizontal deformation vs. time.

Fig Typical load and deformation versus time relationships for repeated-load indirect tensile test.

8.2 Calculate the resilient modulus of elasticity E_R and Poisson's ratio ν using the following equations:

$$E_R = \frac{P(\nu + 0.27)}{t\Delta_x}, \text{ psi}$$

$$\nu = 3.59 \frac{\Delta_x}{\Delta_y} - 0.27$$

where

P = repeated load, lb.

ν = Poisson's ratio

t = thickness of specimen, in.

Δ_x = recoverable horizontal deformation, in.

Δ_y = recoverable vertical deformation, in.

Note 10 - Poisson's ratio can be calculated using the above equation for 4-inch and 6-inch diameter specimens with 0.5 inch or 0.75 inch wide loading strips, respectively, or the value can be assumed in which case vertical deformations are not required. A value of 0.35 for Poisson's ratio has been found to be reasonable for asphalt mixtures at 77° F (25° C).

8.3 The tensile strength S_T can be calculated using the following equation:

$$S_T = \frac{2P_{ult}}{\pi t D}$$

where

P_{ult} = the ultimate applied load required to fail specimen, lb.

t = thickness of specimen, in.

D = diameter of specimen, in.

9. Report

9.1 Report the average resilient modulus at temperatures of 41, 77, and 104^{o} F (5, 25, and 40^{o} C) for each load and load frequency used in the test.

10. Precision

10.1 The precision of the method is being established.