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16. Abstract

This report presents the results of research intended to define the influence of various properties of the asphalt concrete on pavement cracking due to low temperatures experienced on several projects in West Texas. Six pavement sections were selected in District No. 6 (Odessa) exhibiting various degrees of pavement cracking. Field samples of the pavement surface were obtained from each of the sections for detailed laboratory testing. Age of the pavements at the time of sampling ranged from four to five years.

Cracking due to thermal shrinkage can be predicted by methods that have been used in the colder climates of Canada and elsewhere. The computer based prediction model for low temperature cracking referred to as COLD (Computation of Low-Temperature Damage) was used to analyze data from this project. Two sections were compared, one which exhibited cracking, with the other having no cracking. Thermally induced stresses that exceeded the tensile strength were obtained for the one section, but were far less than the expected strength for the other section exhibiting no cracking. Use of the COLD program to predict cracking by thermal shrinkage yields results which are compatible with observed field performance of these two pavements in West Texas.

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Asphalt Concrete Factors Related to

Pavement Cracking in West Texas

by

K. O. Anderson and J. A. Epps

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Study No. 2-9-80-287

Sponsored by the

State Department of Highways and Public Transportation
in cooperation with the
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SYNOPSIS

This report presents the results of research intended to define the influence of various properties of the asphalt concrete on pavement cracking due to low temperatures experienced on several projects in West Texas. Six pavement sections were selected in District No. 6 (Odessa) exhibiting various degrees of pavement cracking. Field samples of the pavement surface were obtained from each of the sections for detailed laboratory testing. Age of the pavements at the time of sampling ranged from four to five years.

Conventional mix design tests such as Hveem and Marshall Stability and Flow, resilient modulus tests, thermal expansion, direct tension and indirect tension were performed at various loading rates and temperatures on the asphalt concrete cores taken from the test sections. The resilient modulus tests were conducted at 0.1 sec loading at 22 C (72 F), 0.6 C (33 F) and -23 C (10 F). The linear thermal expansion tests were over a temperature range of -18 C (0 F) to 21 C (70 F). The direct and indirect tension tests were conducted at temperatures of 24 C (75 F), 0.6 C (33 F) and -23 C (-9 F) and loading rates of 51, 5.1 and 0.51 mm/min (2.0, 0.2 and 0.02 in/min).

Tests on the recovered asphalt cement included penetration at 25 C (77 F) and 4 C (39.2 F), kinematic viscosity at 135 C (285 F), absolute viscosity at 25 C using the sliding plate microviscometer, and ring and ball softening point. Aggregate was tested for resistance to abrasion and soundness. A modification of ASTM C-88 Test Method to determine the resistance of aggregate to disintegration by a saturated solution of magnesium sulfate after four cycles of wetting and drying was used in a further investigation on aggregates used in 23 sections having various degrees of pavement cracking.

Field condition surveys were performed to quantify performance of these pavement sections. A recent inspection of these sections at ages of 13 to 15 years from initial construction, together with a review of maintenance and overlays required, confirms the earlier performance observations and trends.

A number of possible mechanisms to explain the formation of cracks in the asphalt concrete pavements subjected to the environmental conditions of West Texas were examined. Minimum air temperatures of -17 C (2 F) were experienced between construction and the initial condition survey. Corresponding rates of temperature drop were in the order of 3 C/hr (5F/hr).

Tensile properties of the asphalt concrete have been shown to be related to the extent of cracking observed in the pavement sections. Those with low direct tension failure stress at -12 C (10 F) became badly cracked early in their pavement life, while the highest strength pavement remains uncracked after 13 years of service.

Predicted cracking temperatures based on a limiting stiffness of 1×10^9 N/m^2 (1.45×10^5 psi) at a half-hour loading time were all below the minimum expected pavement temperature, although the most badly cracked section was within 4 C. The two sections that had not cracked had the lowest predicted cracking temperatures.

Cracking due to thermal shrinkage can be predicted by methods that have been used in the colder climates of Canada and elsewhere. The computer based prediction model for low temperature cracking referred to as COLD (Computation of Low-Temperature Damage) was used to analyze data from this project. Two sections were compared, one which exhibited cracking, with the other having no cracking. Thermally induced stresses that exceeded the tensile strength were obtained for the one section, but were far less than the expected strength for the other section exhibiting no cracking. Use of the COLD program to predict cracking by thermal shrinkage yields results which are compatible with observed field performance of these two pavements in West Texas.

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The original Research Study No. 2-8-73-18 enabled the gathering and partial reduction of field and laboratory data. The current Research Study No. 2-9-80-287 has provided the opportunity to review this previous work and for further analysis and development to this present stage.

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ASPHALT CONCRETE FACTORS RELATED TO PAVEMENT CRACKING IN WEST TEXAS

INTRODUCTION

Deterioration of asphalt pavements arising from non-traffic load associated cracking has been of great concern to agencies responsible for pavements in the colder climates of northern United States, Canada and the arid portions of the western United States. A great deal of effort since the mid-1960's has been expended in attempts to understand and to develop methods to control or minimize the problem. (1).

Several mechanisms can be responsible for this type of pavement cracking. It is therefore difficult to state definitely the causes and which factors should be given most consideration in the design and construction of asphalt pavements subjected to this type of environmental deterioration. Many individual papers have been written, providing a wealth of information on particular situations. With the purpose to synthesize this vast experience, comprehensive reviews have been published by various groups to provide state-of-the-art reports for general use. Examples of such are an early summary of Canadian experience (2), followed by a report by The Asphalt Institute (3) and later NCHRP studies (4, 5). The most recent report was published by The Asphalt Institute in December of 1981 (6).

Application of principles derived from these comprehensive studies and reports can be made in attempting to determine specific causes and solutions to pavement cracking in a particular area, such as in West Texas. This paper will present the results of research intended to define the influence of various properties of the asphalt concrete on pavement cracking experienced on several projects in West Texas and will supplement earlier reports (7, 8).

Objectives

This part of the study was intended to be concerned primarily with the role of the asphalt concrete and the contributions to cracking made by the components, namely, the asphalt cement binder, and the aggregate. Results of laboratory and field tests to identify physical properties of the asphalt concrete and components were to be correlated with field performance in order to more adequately define the pavement cracking problem for this area of the United States.

PREVIOUS STUDIES IN WEST TEXAS

Surveys in this area have indicated that pavement cracking is an extensive form of distress over several thousands of miles of primary highways. It has been estimated that over twenty million dollars are spent annually on maintenance in Texas as a result of this form of deterioration (8). Despite the lack of extremely low temperatures as experienced in northern United States and Canada, this form of distress can be related to the environment and materials utilized in this area.

The experience reported by McLeod (9) that low-temperature cracking is commonly found only in areas with a design freezing index over 250, tended to discount shrinkage of the asphalt concrete as the cause of pavement cracking. Search for and examination of other possible mechanisms was made.

The role of freeze-thaw activity of the base course as an important mechanism in pavement cracking has been discussed in detail earlier (7, 8). Briefly it has been shown that freezing coefficients for base course materials were sufficient to produce tensile stresses exceeding base tensile strengths, but stresses in the asphalt concrete were of the same order as typical mixture tensile strengths.

OVERVIEW OF PRESENT STUDY

Meetings with Texas State Department of Highways and Public Transportation (SDHPT) personnel established that transverse and longitudinally cracked pavements existed in at least eight Districts in Texas, namely Districts No.'s 3 (Wichita Falls), 4 (Amarillo), 5 (Lubbock), 6 (Odessa), 7 (San Angelo), 8 (Abilene), 24 (El Paso), and 25 (Childress). A review of literature indicated that the properties of the asphalt concrete could be responsible for the initiation of cracking under low temperatures (9-12).

The original purpose of this research was to define the influence of the following properties of the asphalt concrete on pavement cracking:

1. Tensile strength,
2. Coefficient of thermal expansion,
3. Thermal conductivity,
4. Flexure fatigue,
5. Resilient modulus,
6. Stability,
7. Properties of the asphalt cement.

These asphalt cement and mixture properties were to be utilized as a basis for the development of a method to predict the occurrence of cracking of asphalt concrete pavements subjected to both traffic and environmental loads.

This limited laboratory and field study has been used to investigate the use of presently available low temperature crack prediction models for West Texas conditions.

Test Sections

One district in West Texas No. 6 (Odessa), was chosen to enable the selection of pavement sections exhibiting various degrees of pavement cracking. Initially 24 sections were identified, from which six were selected for detailed study. These are all located in the Midland-Odessa-Pecos-Ft. Stockton area of West Texas, as shown in Figure 1.

Construction records and discussions with state personnel enabled typical structural cross-sections to be sketched for each pavement section, as shown in Figure 2.

The Interstate projects had varying thickness of granular base and had a surface treatment to carry traffic for 2 or 3 years before applications of the hot-mix asphalt concrete surface (HMAC) consisting of 50 mm (2 in) of SDHPT Type C followed with 30 mm (1.25 in) of Type D HMAC. All surfaces were constructed with an AC-20 grade of asphalt cement, with the exception of Interstate Highway 10 which used AC-10 grade. All asphalt cements came from the same refinery.

I-10 was overlaid with 30 mm (1.25 in) of Type HMAC in 1974 which contained an AC-10 asphalt cement.

Table 1 provides further details concerning each of the test sections, together with comments of District 6 and SDHPT personnel concerning the general observations about the quality of the aggregate used in the HMAC surface.

Table 2 shows the available information for the base courses. Since this study was to concentrate on the asphalt concrete surface, the characteristics of the base course material are not clearly defined and are given here only for general information.

Procedures

Laboratory Tests. Field samples of the pavement surface were obtained from each of the six highway test sections.

At each site, six slabs, (two slabs at each of three locations) 600- by 300-mm (24- by 12-in) were obtained and transported to the laboratory. These slabs were cut into sample sizes appropriate for the particular laboratory test to be performed. Tests on the asphalt concrete were:

1) Marshall Stability and Flow, (ASTM D1559),
2) Hveem Stability, (ASTM D1560)
3) Direct tension at various loading rates and temperatures,
4) Indirect tension at various loading rates and temperatures,
5) Resilient modulus tests at various temperatures,
6) Thermal expansion over a range of temperatures,
7) Density and air void analysis (ASTM D2041 and D2726),
8) Asphalt extraction (ASTM D2172).

Tests on the recovered asphalt were performed to determine:

1) Penetration at 25 C (77 F) and 4 C (39.2 F), (ASTM D5),
2) Kinematic viscosity at 135 C (275 F), (ASTM D2170),
3) Absolute viscosity at 25 C (77 F),
4) Ring and ball softening point, (ASTM D36).

Prior to construction the aggregates used in the HMAC were obtained and tested to determine:

1) Resistance to abrasion (ASTM C131)
2) Soundness or resistance to magnesium sulfate (ASTM C88 modified)

Field Tests. For each section of highway, serviceability indexes were determined from Mays Meter runs. A pavement condition rating form was used to conduct and record comprehensive visual surveys. In addition detailed crack maps were drawn. Dynaflect measurements were also taken throughout the sections.

LABORATORY TEST RESULTS

Standard American Society for Testing and Materials (ASTM) test procedures were followed whenever possible. Specialized tests were conducted according to what were considered to be the most practical and appropriate for the Texas A&M Bituminous Laboratory at the time.

A. ASPHALT CONCRETE

Conventional Mix Design Tests

Results of conventional mixture property tests performed on cored samples such as asphalt content (ASTM D2172), maximum theoretical specific gravity, (ASTM D2041), air voids and both Hveem (ASTM D1560) and Marshall stabilities (ASTM D1559) are shown in Table 3.

A cursory examination of these data identifies section I-10 as being low in asphalt content with a corresponding high amount of air voids. This could possibly contribute to the high Hveem stability values.

The low air voids in section FM 1053 would suggest problems with stability or pavement flushing. This will be considered in a later section when the condition survey is presented and discussed. The generally high Marshall stability values, despite the low Hveem Stability values with the exceptions of I-20 and I-10, reflects the influence of the consistency of the asphalt cement on Marshall Stability values. This casts doubt on the usefulness of Marshall Stability tests conducted on cores taken from aged pavements.

Resilient Modulus Tests

Results of resilient modulus tests conducted on 100 mm (4.0-in) diameter cores at 0.1 sec loading, at 22 C (72 F), 0.6 C (33 F), and -23 C (-10 F) using the Schmidt apparatus (13) are shown in Table 4.

The exceptionally high value for the resilient modulus at 22 C (72 F) for the I-10 section supports the trends shown by the high Hveem and Marshall stability values. Experience has shown this test to be particularly responsive to the consistency of the asphalt cement binder.

The resilient modulus values at the two lower test temperatures do not show the same large differences between sections. This may imply that at this fast loading rate and low temperatures the behavior of the asphalt cement binder is more elastic rather than viscous thereby reducing the differences in mixture properties.

Direct Tension Tests

Specimens approximately 40- by 40- by 60-mm (1.5- by 1.5- by 2.3-in) in length were cut from the test slabs and tested at three different temperatures and strain rates. It was intended to have triplicate tests for any particular condition, therefore 27 specimens were required for each highway section. Due to various difficulties in fabrication and testing the actual number of tests reported ranged from 10 to 21 for each individual test section.

Individual specimens were fitted with end caps attached with epoxy, and loaded by means of an Instron testing machine at constant rate of deformation in a manner similar to that described elsewhere (14). Deformation (or strain) rates of 51, 5.1 and 0.51 mm/min (2.0, 0.2 and 0.02 in/min) at temperatures of 24 C (75 F), 0.6 C (33 F) and -23 C (-9 F) were used for this sequence of tests.

The original test data was transferred to punch cards for computer reduction. Average values for the triplicate specimens are used. The following sections will present a summary of these data.

Influence of Strain Rate. Figures 3 and 4 show the influence of strain rate on the failure stress and E-modulus at failure for the highest and lowest temperatures tested. Insufficient data was available for the 0.6 C (33 F) temperature. The term E-modulus has been used in this report to be the tensile stress at failure divided by the corresponding tensile strain. This is the same as the secant modulus at failure, or the term "stiffness modulus" as used by some other investigators.

At the lowest temperature -23 C (-9 F), the behavior is essentially independent of strain rate, however at the highest temperature, 22 C (72 F), both failure stress and E-modulus are rate dependent.

Since the application of this study involves thermally induced stresses and hence slow rates of deformation, the presentation and analyses of data to follow will be based on the slowest rate of strain, namely 0.51 mm/min (0.02 in/min). This generally produced failure in the order of 40-100 seconds, which is however many times faster than the time of failure that could be expected in the field from thermally induced stresses. The near independence on strain rate at the lowest temperature could reduce this discrepancy between times used in laboratory testing and anticipated field loading conditions.

Failure Stress and E-modulus at 0.51 mm/min (0.02 in/min). For each of the six pavement sections the failure stress and E-modulus values are summarized in Table 5. In order to display trends this data is also presented in Figures 5 and 6.

Examination of Figure 5 leads to a general description of two high strength pavements and four relatively low strength pavements, over the range of testing temperatures.

Figure 6 shows that each pavement has slightly different E-modulus values at each temperature over the range of testing temperatures. Very large differences are evident at the temperature of 24 C (75 F).

Indirect Tension Tests

Four inch diameter cores were cut from the project test slabs. Thickness of the slabs ranged from 33 to 53 mm (1.3 to 2.1-in) in thickness. Duplicate samples were tested at three different temperatures and strain rates. They had previously been tested for resilient modulus at 22 C (72 F).

The indirect tension test procedure was similar to that used at the University of Texas at Austin (15, 16). By measuring the applied loads and horizontal and vertical deformations continuously to failure, it was possible to calculate the tensile strength and modulus of elasticity of the specimen. The values reported are those calculated for failure conditions and are the average results of two individual tests.

The load was monitored with a load cell and the vertical deformation rates were controlled at 51, 5.1 and 0.51 mm/min (2.0, 0.2 and 0.02 in/min). Horizontal deformation of the specimen were measured using two cantilevered arms with strain gages attached.

Temperatures of 23 C (73 F), 0.6 C (33 F) and -23 C (-9 F) were used in this series of tests. While temperatures and vertical deformation rates were similar to the direct tension tests, the actual tensile deformation rates were different due to the basic test configuration and biaxial state of stress.

The failure stress and E-modulus values were calculated using a preprogrammed hand calculator. The following sections will present a summary of these data.

Influence of Strain Rate. Figure 7 shows the influence of strain rate on failure stress for the two extremes of temperature, namely 23 C (73 F) and -23 C (-9 F). The intermediate temperature tests were omitted for clarity.

At the lowest temperature, the behavior is only slightly dependent on strain rate, while at the highest temperature the failure stress is quite rate dependent. Comparison of this graph with Figure 3, shows similar trends. Again the presentation and analysis of data to follow will be based on the slowest rate of vertical deformation, namely 0.51 mm/min (0.02 in/min).

Failure Stress and E-modulus values. The failure stress and E-modulus values at 0.51 mm/min (0.02 in/min) are summarized in Table 6. No data was available at this rate for the US 285 (195) section.

Figure 8 presents the failure stress values over the range of testing temperatures. Comparison of this plot with Figure 5 shows that significantly lower failure stress values were obtained with the indirect tension test at the lower temperatures.

Figure 9 shows a comparison of failure stresses obtained by indirect tension versus those by direct tension testing for the three temperatures and rates of deformation. Approximately one-half of the values fall between lines representing 0.8 to 1.2 of the line of equality.

Figure 10 shows the E-modulus values plotted against temperature. Comparison with the corresponding Figure 6 shows the apparently low E-modulus values determined by the indirect tension test at the lowest temperature of -23 C (-9 F).

Figure 11 gives a comparison of E-modulus values determined by indirect tension tests to those by the direct tension method. The trend towards lower ratios at higher E-modulus values is apparent.

Reasons for this obvious deficiency in the indirect tension test method and analysis are not readily apparent. Difficulties encountered with measuring the horizontal deformations with the cantilever gages may be part of the explanation. The current method of test uses two LVDT gages attached across the horizontal diameter of the specimen. This should be investigated more fully in future test programs involving indirect and direct tension testing.

Thermal Expansion Tests

It was intended that two samples for each pavement section be tested in order to determine coefficients of linear thermal expansion over a range of temperature from -18 C (0 F) to 21 C (70 F).

LVDT's were attached to opposite sides of a specimen approximately 50 mm (2-in.) in length, with expansion and temperature readings taken as the sample was allowed to warm up. Instrumentation difficulties were experienced and not all testing was completed. Available results are presented in Table 7.

The results appear to be reasonable and are in the range of values compiled by Finn (17) reporting on the work of others, with the possible exception of FM 1053. This value of 6.30×10^{-5}/C (35.0×10^{-6}/F) seems quite high, although this could be partially explained by the low air voids of this specimen and larger volume concentration of asphalt. Increased asphalt content has been reported to increase the linear coefficient of expansion (17). Monismith has reported linear expansion values of 1.2 to 1.4×10^{-5}/F (2.2 to 2.5×10^{-5}/C) for 85-100 penetration asphalt cement and a dense graded granitic aggregate over a range from -10 to 70 F. The thermal coefficient for contraction used by Gaw,et al. (6, 18) to represent asphalt concrete at the Ste. Anne Test Road was 1.58 x 10^{-5}/C.

B. RECOVERED ASPHALT CEMENTS

Several samples of asphalt cement were recovered from the slabs taken from each of the six test sites and subjected to conventional physical tests. The results of these tests were given in Table 8. Individual results are presented, rather than averages, in view of the variability in some of the results.

Consistency

Despite being constructed with the softest grade of asphalt cement, namely AC-10 rather than AC-20 for the other sections, I-10 appears to be the hardest. This can be attributed not only to being one or two years older, but more importantly having relatively low asphalt content and high air voids.

The standard penetration values range generally from a low of 15 to a high of 55, after rejecting the one low value of 5 as an outlier.

Asphalt cement recovered from SH 18 appears to be softer than all the other asphalts tested. This is evident in penetration at 25 C (77 F), viscosity at 25 C (77 F) and lower ring-and-ball softening point.

Tests for viscosity at 60 C (140 F) were attempted but were reported as being too hard to test with the standard viscometers available.

Tests for viscosity at 25 C (77 F) were conducted using the sliding plate microviscometer.

Temperature Susceptibility

Figure 12 shows the results for each recovered sample tested for penetration at 25 C (77 F) and kinematic viscosity at 135 C (275 F) plotted and joined by a straight line on the Bitumen Test Data chart (BTDC) (19). The softening point values were also plotted corresponding to a penetration value of 800. They did not fall on the line as expected, but were in the order of 5 to 7 F higher. Since there were no viscosity values reported at 60 C (140 F) nor penetration values with the standard weight at another temperature any possible discontinuities in this line could not be established. This was evident for each of the other test sections plotted so there may be something in error in conducting the softening point test. A possible reason for this could be the presence of "waxy" constituents as reported by Kopvillem and Heukelom for some asphalts (20).

Figure 13 shows the results plotted for each of the test sections. Only slight variations in the slope are evident, indicating apparent Penetration Index (PI) values in the range of +0.5 to -0.5, determined graphically by the method described by Heukelom (19), assuming a straight line typical of Type S (normal) bitumens. The actual PI's could be significantly different, however if the asphalt had the characteristics of a Type W (waxy) as Type B (blown) bitumen. As mentioned previously, this cannot be established with the data available.

Based on the above discussion it could be inferred that the temperature susceptibilities of the recovered asphalt cements are normal and do not differ greatly from each other.

Another indication of temperature susceptibility is the Penetration Ratio (21). All values were well above specification limits normally used for original asphalts and generally higher than other reported values for recovered asphalt cements (11).

Penetration Index values could be calculated according to the original method of Pfeiffer and Van Doormaal as described by Van der Poel (22), however this could likely lead to erroneous results because of incorrect softening point values. Individual results were plotted on the chart developed by

McLeod (23), and generally were in the vicinity of PI = 0.0 or above.

The test values of kinematic viscosity at 135 C (275 F) and penetration at 25 C (77 F) were plotted on a chart used by McLeod to develop his Pen-Vis Numbers (PVN's) from data at these two temperatures. Resulting PVN's were in the range of -0.2 to -0.5 with two values approaching -1.0.

Summary

It appears that the original asphalt cements used have hardened to varying degrees to the range of 15 to 50 penetration at 25 C (77 F).

Indicators of temperature susceptibility are inconclusive, however use of PI's or PVN's in the order of -0.2 to -0.5 would seem to be reasonable, with -1.0 being the lowest value to be expected.

C. AGGREGATE

Aggregates obtained from sources used on the initially identified sections were tested to determine resistance to abrasion and soundness. Results of these tests are given in Table 9.

Abrasion and Soundness Tests

The results of the standard abrasion tests did not show any large differences betwen the various test sections. The aggregate used on US 285 (195) was slightly more resistant to abrasion than the others.

An indication of the soundness of the aggregate used was determined using a modification of ASTM C-88 Test Method. This test determines the resistance of the aggregate to disintegration by a saturated solution of magnesium sulfate after 4 cycles of wetting and drying. Details of this test procedure are given by McCall who reported on an earlier study (24) of various aggregates used in hot mix asphaltic concrete. That investigation was undertaken following early deterioration and cracking experienced on I-10 in Pecos and Reeves County. A maximum value of 25 percent material after 4 cycles was recommended and has been used in District 6 as a specification requirement on Type "C" and "D" hot mix aggregates since 1971.

In view of the potential of this soundness test to indicate aggregates with unsuitable characteristics, a further investigation was undertaken at the Texas Transportation Institute on aggregates used on 23 sections originally identified as having various degrees of pavement cracking. Field surveys were taken to document details of pavement condition.

Field Comparison

Figures 14, 15 and 16 show various parameters related to pavement condition plotted against values of percent loss in the soundness test. Examination of these graphs suggests that high levels of Serviceability Index and Condition Rating was experienced with varying percentages losses in the soundness test. Similar difficulties are evident when attempting to relate length of cracking with percent loss in the soundness test.

It can be observed from these data that by itself, the percent loss in the soundness test is not a reliable indicator of future pavement condition, at least within the ages of pavements examined.

FIELD TEST RESULTS

Condition Surveys

Six test sections were selected for detailed study from the 24 sections considered initially. The selection was based on the amount of observed field cracking, Mays Ridemeter results, visual maintenance evaluations, scaled and drawn cracking patterns and Dynaflect tests. Representative data taken at the six test sites are presented in Table 10.

It can be noted that only two sections exhibited no cracking, namely SH 18 and US 285 (186). FM 1053 was reported as having some cracking but a later inspection showed this to be away from the test slab sampling location and hence has been considered as having no cracking.

A follow-up visual maintenance examination performed approximately two years later showed that I-20 and I-10 had been overlaid. I-20 showed transverse cracking from 1-4 per station (30 m or 100 ft) of slight severity. No distress was reported for I-10.

FM 1053 exhibited from 5-9 transverse cracks per station of slight severity. US 285 (195) was reported as having 5-9 transverse cracks per station of moderate severity.

Both SH 18 and US 285 (186) still were uncracked. Slight flushing was noted over 16-30 percent of the area on US 285. One section of SH 18 had slight flushing in excess of 30 percent of the area.

Dynaflect readings taken in the spring of 1974 are also reported in Table 10. The values given are mean $\overline{x}$ and standard deviation for the readings taken from Sensor 1.

Spring 1982 Inspection

In April of 1982 an inspection of the six test projects was made to determine pavement conditions some 13 to 15 years after initial construction. No detailed measurements were taken, however an overall assessment of the present pavement condition was made.

The section of I-20 had been overlaid in 1980. This new overlay was part of an experimental section which contained geotextiles and various thicknesses of asphalt concrete. Some of the pavement was not overlaid and has been used for comparison purposes with the experimental sections.

The older overlay was cracked into blocky patterns with about 7 or 8 transverse cracks per station. There was also some longitudinal cracking evident.

The new overlay had some fine cracks extending into the outside lane from the shoulder, with several transverse reflection cracks.

These most recent observations verify the earlier surveys which reported the large amount of cracking for the original asphalt concrete surface on the I-20 test section.

The test section on I-10 west of Ft. Stockton in the vicinity of the Reeves-Pecos County line now forms the westbound lanes of the divided highway. The overlay pavement is now very badly cracked into blocky patterns in the order of 2 to 3 m spacing (5 to 10 ft). The cracks have been sealed, giving the impression of an exceptionally distressed pavement. This also confirms the earlier reports of the tendency for this pavement to crack.

The section of FM 1053 north of Ft. Stockton has been seal coated once or twice since initial construction. It now has extensive cracking throughout the length of the project. Crack spacing varies from 0.7 to 3 m (2 to 10 ft) in a blocky type pattern, being somewhat variable along the project. Although earlier reports were that cracking was away from the vicinity of the test slab, it is evident that there is extensive cracking now although in a very irregular areal extent.

The section of SH 18 from North of Ft. Stockton to Grandfalls still is free from transverse cracking. There is a small amount of longitudinal cracking along the centerline with the occasional partial lane width crack perpendicular to and near the centerline. The most obvious surface condition is the severe flushing throughout most of the project. There appears to be more flushing in the north bound lanes, which in some sections, was estimated to comprise 50 to 80 percent of the area. One section on the southbound lane near a wayside aggregate production quarry showed severe flushing in nearly 100 percent of the travelled lane.

US 285 from Ft. Stockton to I-20 near Pecos had the two test sections with the Reeves and Pecos County line as the dividing line. The section near Ft. Stockton in the Pecos County, designated US 285 (186) was still largely uncracked. There was some centerline cracking with irregular cracks extending about 0.6 m (2 ft) from and perpendicular to the centerline. A chip seal had been placed over the test section, however there was no evidence of transverse cracks in the original surface. There is some slight flushing in the wheel paths.

The section in the Reeves County, US 285 (195) had been overlaid and chip sealed since original construction. It now has extensive transverse cracking in excess of 10 per station, or 2 to 3 m (5 to 10 ft) spacing. This could very likely be cracking in the overlay reflecting from the original surface which was noted as being cracked in earlier surveys.

In summary it can be stated that test sections on I-20, I-10 and US 285 (195) have experienced extensive amounts of transverse and blocky type cracking and have been overlaid at least once since original construction.

Test sections on SH 18 and US 285 (186) still have not cracked significantly after 13 years of service. Flushing is most extensive on SH 18. Some slight flushing in the wheel paths on parts of US 285 (186) was noted.

ANALYSIS

As discussed earlier in this report there are a number of possible mechanisms to explain the formation of cracks in asphalt concrete pavements that can be related to materials and environmental conditions. In the view of Ad Hoc Committee of the Asphalt Institute (6) two of the several mechanisms suggested in the literature can be described in the following manner or crack types.

"a) Stresses induced by pavement thermal shrinkage result in surface cracking that propagates through the asphalt concrete layer. The cracking may be initiated by sudden thermal shock or low frequency cycling at low temperatures.

b) Stresses in the non-asphalt treated base layer can cause transverse cracks which ultimately reflect through to the surface."

This second type (b) is based on the freeze-thaw activity reported for base course materials in West Texas (7, 8).

These two mechanisms will be used with the data collected in order to help explain the vastly differing pavement cracking histories of the six test sections in West Texas.

Climate

A summary of minimum air temperature with corresponding temperature drops recorded at weather stations (25) nearby to the test sections in West Texas, for the period of 1967-1976, is given in Table 11. This period was selected to span the years from construction of the pavement to the last date of condition survey.

Examination of this Table indicates that minimum air temperatures in the order of -17 C to -16 C (2 F to 4 F) were experienced during the winter of 1971 and 1972. Corresponding air temperature drops ranged from 15 C to as high as 33 C (28 F to 60 F), considering the maximum of the previous day to the minimum for the day of record. Since the cooling could take place over a period of 12 hours or less, this would represent a very rapid rate of cooling. Maximum air temperature drops of 70 F in 12 hours have been noted in the West Texas area.

Pavement surface temperatures corresponding to those ambient temperatures may be estimated in a number of ways. One method, based on Ste. Anne Test Road recorded temperatures, (26) related minimum ambient to surface temperatures.

For granular base courses on clay subgrade, and 100 mm (4 in) HMAC, the surface temperature $T_s = 6.02 + 0.87\ T_{ma}$ where T_{ma} is the minimum air temperature in degrees F. For 2 F this would correspond to T_s = -13 C (8 F). Considering a maximum pavement surface temperature of at least 21 C (70 F) for the air temperature of 16 C (60 F), this would correspond to a rate of temperature drop of approximately 3 C/hr (5 F/hr).

Rates of temperature change for the Ste. Anne Test Road were approximately 1 C/hr (2 F/hr) for the 12 hours immediately prior to reaching a minimum surface temperature (26).

From the above discussion it would appear reasonable to assume minimum air temperature in the order of 2 F to 4 F with corresponding pavement surface temperatures of -13 C to -12 C (8 F to 10 F) as being representative of low temperatures in the vicinity of the test sections in District 6. Cooling rates are somewhat greater than that experienced at the Ste. Anne Test Road in Manitoba, Canada.

Cracking Relative to Tensile Strength and Induced Stresses due to Base Course Freezing

Calculations have shown that induced stresses in the asphalt concrete surface due to base course freezing can be in excess of 4140 kPa (600 psi), depending upon the freeze coefficient of the particular base material (7). While insufficient data is available to estimate this parameter for the base courses of the test sections, a comparison of asphalt concrete tensile strengths at low temperatures expected in the pavement may be useful.

Using the direct tension data presented in Figure 5, values for failures stresses at a temperature of -12 C (10 F) have been interpolated. This temperature was selected to be representative of the lowest pavement surface temperatures as discussed previously. Table 12 shows this data arranged in the order of increasing failure stress together with general comments on the extent of pavement cracking.

It is readily evident that the two sections having the lowest failure stress became badly cracked early in their pavement life, while the highest strength pavement still has not cracked.

While this may be an overly simplistic approach it does seem to indicate that the tensile strength of the asphalt concrete does have an important bearing on the extent of cracking in these test sections.

If base course shrinkage is indeed the operative mechanism causing the pavement to crack, relative tensile strengths of the asphalt concrete surface at all sub-freezing temperatures down to the minimum expected are of interest and concern.

Predicting Pavement Cracking by Critical Stiffness

One of the simplest means of predicting cracking is to estimate the temperature at which the asphalt reaches a certain critical or "limiting" stiffness. Different investigators have adopted various values of stiffness and loading times based on their particular experiences. A recent analysis of available data has led the Ad Hoc Committee of The Asphalt Institute (6) to consider the temperature at which the asphalt stiffness reaches the limiting stiffness of 1.45 x 10^5 psi (1 x 10^9 N/m^2) at a half-hour loading time as the predicted cracking temperature.

With the test data presented in Table 8 for the recovered asphalt cements and the Van der Poel Nomograph for determining the stiffness modulus of asphalts, predicted cracking temperatures were calculated and are presented in Table 13.

Although these predicted cracking temperatures are all below the minimum expected pavement temperature of -13 C to -12 C (8 F to 10 F) the most badly cracked section I-10 comes within 4 C (7 F). The two sections still not cracked, namely SH 18 and US 285 (186), have the lowest predicted cracking temperatures.

While this simple method does not yield precise and definitive results, it does demonstrate that the asphalt cements recovered from pavements in West Texas are at least approaching what other investigators have considered as "critical stiffness" at temperatures that can be expected in the field. More rapid-cooling rates experienced in West Texas may tend to increase the predicted cracking temperatures determined by this method by justifying a shorter loading time.

Predicting Cracking by Thermal Shrinkage

Several methods are available to attempt to predict the incidence of cracking of the type described previously as "due to stresses induced by pavement thermal shrinkage" (6). As the temperature drops, tensile stresses are induced which may exceed the tensile strength of the material and cracking takes place.

Hills and Brien (27) described a procedure for determining predicted cracking temperatures by estimating the thermal stress buildup and comparing this with the tensile strength of the asphalt concrete at a series of temperature intervals. Benson (28) used this technique and found that where transverse cracking occurred and sufficient data were available for computations, minimum temperatures approached or surpassed estimated crack formation temperatures. He concluded that thermally induced stresses are responsible for transverse cracking of some pavements in West and Central Texas and also that this type of cracking is directly related to hardness of the pavement binder.

Comparisons of field cracking temperatures with various mathematical models for thermally induced stresses in Alberta and Manitoba lends support to this approach (29). The computer based prediction model for low-temperature cracking developed by Christison was modified so as to arrive at the damage prediction model referred to as COLD, (Computation of Low-Temperature Damage) under the National Cooperative Highway Research Program Project 1-10B by Woodward-Clyde Consultants, San Francisco (30).

Two of the test sections were used to provide data for input to the COLD program for possible verification or calibration. The section on I-20 was selected as one exhibiting cracking with the section on US 285 (186) as having no cracking. Since solar radiation data were available from the weather station at Midland, this nearby station was used (31).

A very important input to this program is the stiffness modulus of the asphalt concrete. This was estimated for appropriate temperatures by using the nomographs of Van der Poel and Heukelom and Klomp as presented by McLeod (32). The time of loading of 7200 seconds was used, since this was found to be appropriate for conditions in Manitoba. Table 14 gives these values for the two sections.

Direct tensile strength values obtained at 0.51 mm/min (0.02 in/min), as presented in Table 5 were used, with interpolated strengths for intermediate temperatures.

Other specific data input requirements were estimated and compiled in accordance with operational instructions for the COLD program.

The program has been developed to compute increments of induced stress from 0 C (32 F) at which temperature the stress was assumed to be zero. For the selected test sections this resulted in erroneous compressive stresses to be calculated for days when the pavement temperature was above this initial assumed value. The program was therefore modified to begin stress increments from a temperature of 16 C (60 F), which was considered reasonable for this particular temperature regime. This adjustment resulted in peak tensile stresses being about 520 kPa (75 psi) greater for section I-20, which was sufficient to change the prediction of cracking. The adjustment in stresses for section US 285 (186) was only in the order of 20 kPa (3 psi) due to the much lower stiffness modulus values at these temperatures.

A partial summary of COLD program outputs for critical low temperature periods is given in Table 15. Examination of Table 15 indicates that the expected thermally induced stresses exceed the expected strength for I-20, but are far less than the expected strength for section US 285 (186). This is compatible with the observed field performance of these two pavement sections.

It appears that the COLD program does produce predictions of low temperature cracking that are verified by two sections of pavement in West Texas. Time does not permit similar calculations for remaining test sections in this report, however the validity of this approach appears to have been demonstrated.

CONCLUSIONS AND RECOMMENDATIONS

Previous studies have documented that environmental deterioration of pavements by transverse and longitudinal cracking is experienced in West Texas and is similar to forms of crack patterns observed in colder climates of Canada and many areas of the western United States. On the basis of an extensive laboratory investigation to determine the properties of asphalt concrete sampled from six pavements in District No. 6 (Odessa) of West Texas together with field observations, the following conclusions may be drawn.

Conclusions

1. Conventional mix design tests conducted on cores taken from the six test pavements are of limited usefulness. Marshall Stability values are generally high, despite low Hveem Stability values, and reflect the influence of consistency of the aged asphalt cement.

2. Resilient modulus tests conducted at 22 C (72 F) and 0.1 sec loading, similarly show high resilient modulus values responsive to the consistency of the asphalt cement. Tests at lower temperatures of 0.6 C (33 F) and -23 C (-10 F) do not show large differences and may imply that the response is more elastic rather than viscous.

3. Direct tension tests conducted over the range of testing temperatures from 24 C (75 F) to -23 C (-9 F) have shown failure stress and E-modulus, a term similar to "stiffness modulus", essentially independent of strain rate at the lowest temperature tested.

4. Direct tension tests at a loading rate of 0.51 mm/min (0.02 in/min) indicate failure stresses to be relatively high for two pavements, SH 18 and FM 1053, and lower for the remaining four pavements, particularly over the middle of the range of testing temperatures. Large differences in E-modulus are evident at the temperature of 24 C (75 F).

5. Indirect tension tests conducted at a loading rate of 0.51 mm/min (.02 in/min) indicate significantly lower failure stress values at the lower temperatures when compared to those obtained by direct tension. For all three temperatures and loading rates, approximately one-half of the ratios of indirect to direct failure stresses were between 0.8 and 1.2.

6. E-modulus values determined by the method of indirect testing and instrumentation used were different than those obtained by the direct tension method. The difference was greater at higher E-modulus values.

7. Thermal expansion tests produced reasonable coefficients of linear thermal expansion despite instrumentation difficulties with the test.

8. Recovered asphalt cements obtained from the six test sites had standard penetration values ranging from 15 to 55. Section I-10 is the hardest with SH 18 being the softest.

9. The aggregate soundness determined by the resistance of the aggregate to disintegration by a saturated solution of magnesium sulfate after four cycles of wetting and drying, is not a reliable indicator of pavement condition, at least within the ages of the pavements examined.

10. Field condition surveys taken at a pavement age of 5 to 7 years at locations representative of the six test sites, indicated that two sections exhibited no cracking, namely SH 18 and US 285 (186). Sections I-10 and I-20 had experienced extensive cracking.

11. A recent inspection has indicated that sections on I-20, I-10 and US 285 (195) have experienced extensive amounts of transverse and blocky type cracking and have all been overlaid at least once. Test sections on SH 18 and US 285 (186) still have not cracked significantly after 13 years of service. Flushing is extensive on SH 18.

12. Analysis of weather station records nearby to the test sections show that the minimum air temperatures experienced betwen the time of construction and condition survey occurred in the winters of 1971 and 1972 and were in the order of -17 C to -16 C (2 F to 4 F). Rates of temperature drop could be as high as 3 C/hr (5 F/hr).

13. Pavement sections having the lowest direct tension failure stress at -12 C (10 F) became badly cracked early in their pavement life, while the highest strength pavement still has not cracked.

14. Predicting pavement cracking by assuming a critical stiffness of 1.45×10^5 psi (1×10^9 N/m^2) gives cracking temperatures that are all below the minimum expected temperature of the pavement. The most badly cracked section I-10 comes fairly close, while the two sections still not cracked have the lowest predicted cracking temperatures.

15. Use of the COLD program to predict cracking by thermal shrinkage indicates that the expected thermally induced stresses exceed the expected strength for section I-20, but are far less than the expected strength for section US 285 (186). This is compatible with the observed field performance of these two pavement sections.

Recommendation

Since it has been shown that the properties of the asphalt concrete have an important influence in the low temperature cracking problem and that sufficiently low temperatures occur in Texas to develop cracking by thermal shrinkage, the following recommendations are made:

1. Efforts should be continued in determining the tensile properties of asphalt concretes using a range of current asphalt cements and aggregates likely to be used in regions of Texas subjected to low temperatures. This should include direct tension testing in order to confirm or modify the currently used method of indirect tension testing and instrumentation.

2. Further use of the COLD program as a predictive method for low temperature cracking should be made. This could be useful in applying the information expected from the present Project 2-9-80-287.

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Table 1. Test Sections in West Texas.

Pavement Section	County	Control Section	Length Miles	Location	Construction Dates	Aggregate Source (HMAC)	General Observations (1973)
I-20	Midland	5-14	14.7	Ector Co. to SH 349	3/68 12/68	Hoard Pit	Poor aggregate
I-10 (US 290)	Pecos & Reeves			6 mi. west of Reeves-Pecos County line to 6 mi. east of line	Prime 1968 6/66 or 6/67	Willbanks Pit	Bad, lot of maintenance. Dissolved aggregate. Many problems.
FM 1053	Pecos	866-5	29.7	Ft. Stockton to Imperial	11/68 - 9/69	Phipps Pit	Good aggregate.
SH 18	Pecos	292-485	24	5 mi. N of Ft. Stockton to Grand Falls	5/69 to 7/69	Strain Pit	Good aggregate.
US 285 (186)	Pecos	139-8	10.9	W. of Ft. Stockton to County line.	4/69 to 5/69	Clayton William Pit	Fair aggregate.
US 285 (195)	Reeves	139-5 139-6	25.1	I-20 to County line	5/68 to 7/68	Hoban Pit	Good aggregate.

Table 2

Base Course Material

Highway Section	Year Con-struction	Liquid Limit %	Plasticity Index	Percent Passing # 40 Sieve (425 μm)
IH 20	1966	30-32	8-9	35-25 Top course
IH 10	1964	26-28	10	25-28
FM 1053	1968	24-27	6-8	
SH 18	1955		-	
US 285 (186)	1967	21-23	6-8	25-30
US 285 (195)	1965	23.5	6	32

Table 3

Conventional Mixture Properties

	Section					
Property	I-20	I-10	FM 1053	SH 18	US 285 (186)	US 285 (195)
Asphalt Content, percent by wt. of aggregate	7.3	5.9	7.1	7.3	7.7	7.6
Max. theoretical specific gravity	-	2.400	2.339	2.373	2.349	2.285
Air Voids, percent	4.3	9.3	0.6	2.8	2.6	2.2
Hveem Stability	36	57	23	24	23	17
Marshall Stability, lbs.	4650	6220	2450	2170	3390	3060
Marshall Flow, 0.01 in.	14	14.7	17.3	15.3	20	16.3

Table 4

Resilient Modulus Values
at 0.1 sec loading time

	Section					
Property	I-20	I-10	FM 1053	SH 18	US 285 (186)	US 285 (195)
Resilient Modulus x 10^3 psi at 72°F (22°C)	920	1480	348	392	592	616
at 33°F (0.6°C)	2100	2050	1940	2260	1560	1790
at -10°F (-23°C)	3300	2980	3130	2090	2200	2340

Table 5. Summary of Direct Tension Test Data at 0.02 in/min (0.51 mm/min).

Pavement Section	Direct Tension Failure Stress (psi)			E Modulus (psi) @ Failure x 10^3		
	75°F (24°C)	33°F (0.6°C)	-9°F (-23°C)	75°F (24°C)	33°F (0.6°C)	-9°F (-23°C)
I-20	107	258	310	70.6	341	1230
I-10 US 290	104	-	400	328	-	2220
FM1053	59	405	436	20	317	821
SH 18	28	468	444	2.4	980	2880
US 285 (186)	54	242	473	9.9	193	2550
US 285 (195)	86	-	422	24.4	-	1890

Table 6. Summary of Indirect Tension Test Data at 0.02 in/min (0.51 mm/min).

Pavement Section	Indirect Tension Failure Stress - psi			E-modulus @ Failure x 10^3 psi		
	73°F (22°C)	33°F (0.6°C)	-9°F (-23°C)	73°F (22°C)	33°F (0.6°C)	-9°F (-23°C)
I-20	77	223	262	39.6	183	151
I-10	131	248	195	144	265	261
FM 1053	54	195	389	17.8	189	239
SH18	38	324	317	8.0	195	225
US 285 (186)	40	305	364	7.3	166	186
US 285 (195)	Incomplete Data					

Table 7

Thermal Expansion Test Data

0°F to 70°F

(-18°C to 21°C)

	Highway Section					
	I-20	I-10	1053	SH 18	US 285 (186)	US 285 (195)
Thermal Coefficient of Linear Expansion per °F x 10^{-6}	19.8	13.8	35.0	-	16.5	-
Per °C x 10^{-5}	3.56	2.48	6.30	-	2.97	-

Table 8. Summary of Physical Properties of Recovered Asphalt Cement.

	Highway Section					
Property	I-20	I-10	FM 1053	SH 18	US 285 (186)	US 285 (195)
Penetration at 77°F (25°C), dmm	18 22 15	15 15 14	21 21	31 55	25 5	32 20
Penetration at 39.2°F (4°C), dmm	15 7 10	9 5 9	10 12	19 22	10 0	12 11
Penetration Ratio*	83 32 67	60 33 64	48 57	61 40	40 -	38 55
Viscosity at 77°F (25°C), x 10^6 poises	29.0 21.0 38.0	23.0 32.0 47.6	14.8 21.8	8.4 6.4	15.6	18.0 22.0
Viscosity at 140°F (60°C) poises	**	**	**	**	**	**
Viscosity at 275°F (135°C), poises	10.5 7.9 11.2	10.8 12.1 11.7	11.5 12.0	7.7 3.6	5.9	7.5 10.9
Softening Point °F (°C) R & B	157(69) 155(68) 161(72)	161(72) 164(73) 167(75)	160(71) 153(67)	130(54) 126(52)	140(60) 185(85)	151(66) 156(69)

*$\frac{\text{Penetration}}{\text{Penetration}}$ at $\frac{39.2\ °F}{77°F}$ X 100, Loading $\frac{200g,\ 60\ sec,\ 39°F}{100g,\ 5\ sec,\ 77°F}$

**Too hard to test

Table 9

Aggregate Properties

	Highway Section					
	I-20	I-10	1053	SH 18	US 285 (186)	US 285 (195)
Resistance to Abrasion % Loss L A Abrasion	27.2	28.4	28.2	31.4	31.4	18.7
Soundness $MgSO_4$ - 4th cycle % Loss	46.5	77.2	4.2	27.3	27.3	5.8

Table 10

Field Condition Evaluation Data
Summer 1974

	Highway Section					
	I-20	I-10	FM 1053	SH 18	US 285 (186)	US 285 (195)
Pavement Rating PRS	34	75	82	90	92	87
Serviceability Index	2.4	4.0	4.4	4.2	4.5	4.4
Length of Cracking ft/100 ft (1974)	75	60	63*	0	0	91
Length of Cracking (1976)	180	120	0	0	0	141
Dynaflect $\bar{x}$ σ	0.293 0.071	0.085 -	0.329 0.051	0.176 0.084	0.209 0.075	0.157 -

*Later inspection in 1976 showed this cracking to be away from the test slab.

Table 11

Climatological Data in the

Vicinity of Test Sections in West Texas

Summary of Minimum Temperatures and Corresponding Temperature Drops

Weather Station	Midland		Pecos		Ft. Stockton	
Elevation-ft	2857		2610		3000	
Projects	I-20		US 285 (195) I-10		SH 18 FM 1053 US 285 (196)	
Year \ Temperature °F	Min °F	Max ΔT	Min °F	Max ΔT	Min °F	Max ΔT
1967	7	34	6	59	12	57
1968	15	36	14	59	15	45
1969	18	50	17	30	19	17
1970	11	28	10	34	13	29
1971	2	28	9	25	6	26
1972	3	41	3	45	4	60
1973	11	32	7	41	10	69
1974	14	*	*	*	*	*
1975	14	*	*	*	*	*
1976	12	*	*	*	*	*

*Not summarized due to relatively high minimum temperatures recorded at Midland.

Table 12

Direct Tensile Failure Stress
at 10°F (-12°C)

Pavement Section	Failure Stress psi at 0.02 in/min	Pavement Condition
I-20	285	Badly cracked. Now overlaid twice.
I-10	330	Very badly cracked. Overlaid early and cracked.
US 285 (195)	345	Cracking of moderate severity. Now over-laid and cracked.
US 285 (186)	365	No cracking.
FM 1053	420	Some early cracking but away from test slabs. Now has ex-tensive blocky type cracking.
SH 18	455	No cracking. Moderate to severe flushing.

Table 13

Predicted Cracking Temperature for a

Limiting Stiffness of 1 x 10^9 N/m^2 (1.45 x 10^5 psi)

Pavement Section	Softening Point R&B °F	°C	Predicted Cracking Temperature** °F	°C
I-20	158	70	-4	-20
I-10	164	73	+1.4	-17
FM 1053	156	69	-5.8	-21
SH 18	128	53	-34.6	-37
US 285 (186)	140*	60*	-22	-30
US 285 (195)	154	68	-7.6	-22

*One test only.

**Temperature difference from R&B softening point for P. I. = -1.0 = 90°C from Van der Poel nomograph at 1/2 hour loading time.

Table 14

Summary of Recovered Asphalt Cement Properties and Stiffness Modulus of Mix Estimated by Nomograph

Characteristic		Highway Section I-20	US 285(186)
Penetration @ 77°F, (25°C), dmm		18	25
Softening Point, R&B °F (°C)		158(70)	140(60)
Penetration Index		-1.0	-1.0
Asphalt Content percent by wt. of aggregate		7.3	7.7
Air Voids - percent		4.3	2.6
Stiffness Modulus psi @ 7200 sec.			
Temperature	75°F (24°C)	3,000	300
	50°F (10°C)	70,000	7,000
	33°F (0.6°C)	500,000	48,000
	10°F (-12.2°C)	1,800,000	500,000
	-9°F (-23°C)	2,000,000	1,800,000

1 psi = 6.89 kPa

Table 15

Summary of COLD Program Outputs
for Critical Low Temperature Periods

Date		January 5, 1971	
Day		12	
Time Period	133	134	135
Air Temperature, °F	2	6.7	11.3
Section I-20			
Pavement Temperature, °F	17.2	17.0	18.6
Tensile Strength, psi	275.8	276.1	274.2
Induced Stress, psi	343.5*	349.4*	311.5*
Section US 285(186)			
Pavement Temperature, °F	18.4	17.8	19.5
Tensile Strength, psi	319.2	322.5	313.8
Induced Stress, psi	34.1	36.1	30.6

* Indicates that expected thermally induced stress has exceeded expected strength.

1 psi = 6.89 kPa

temp C = (temp F-32)/1.8

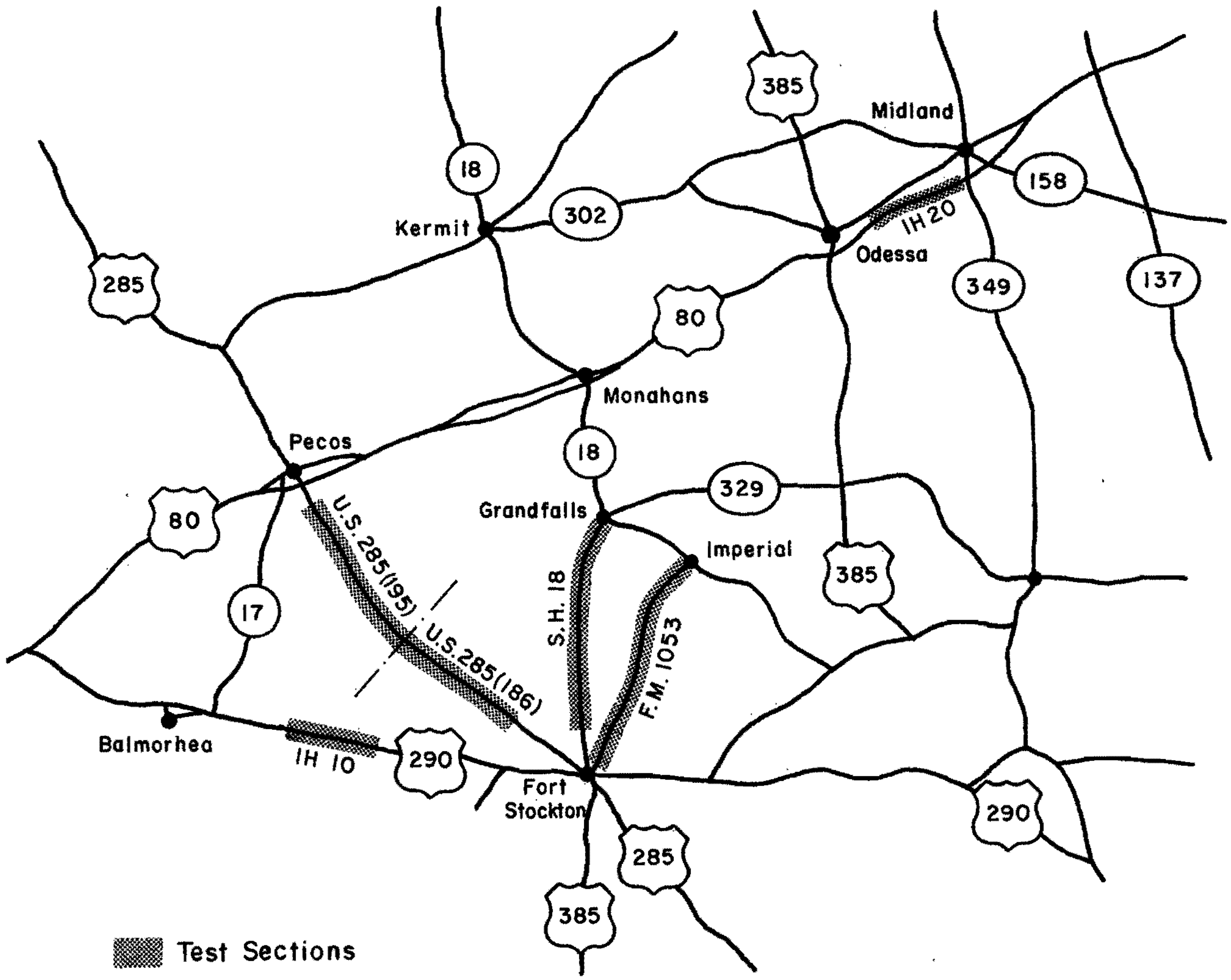


Figure 1. Location of Test Sections in West Texas.

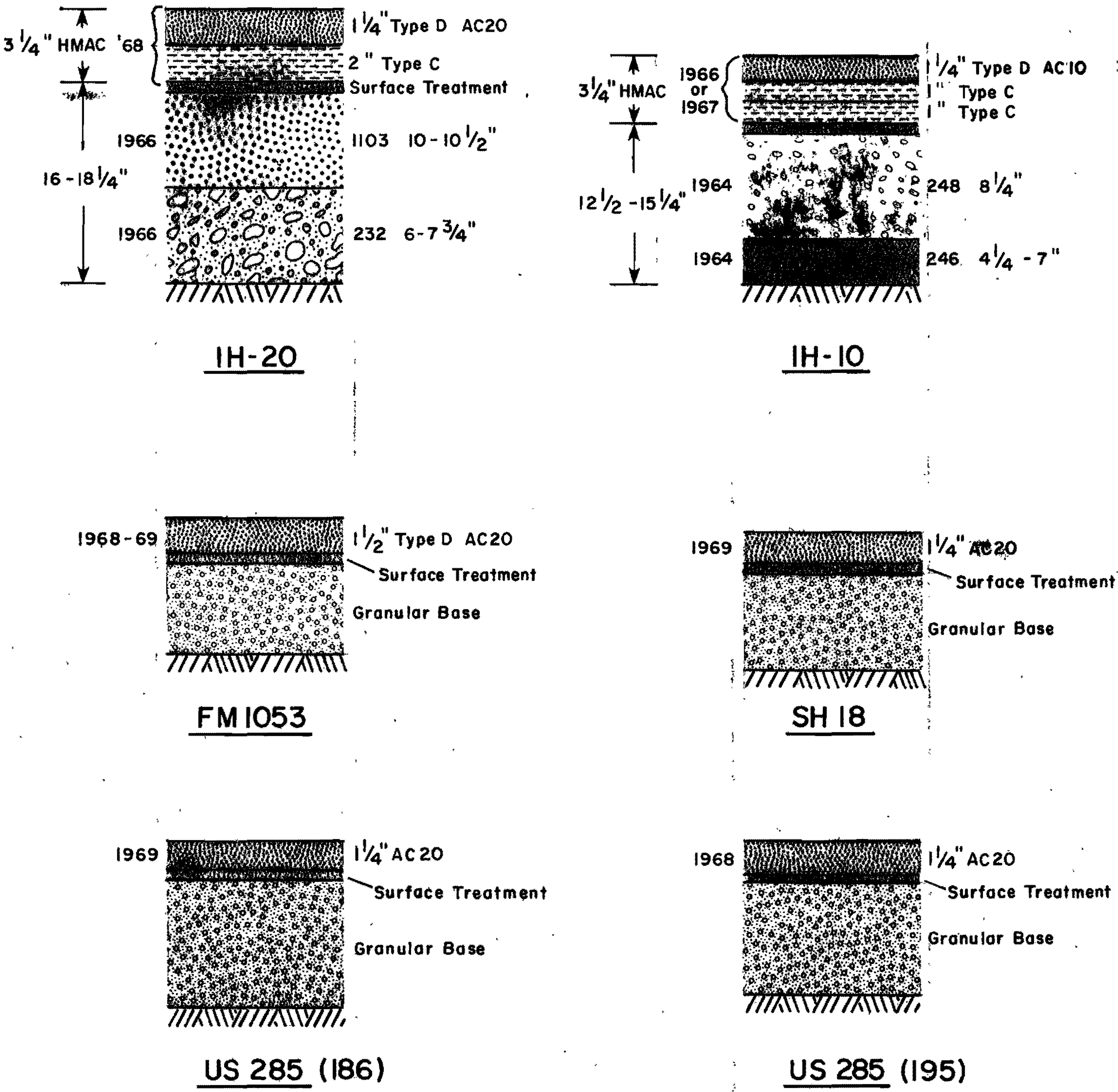


Figure 2. Typical Structural Cross-sections of Test Pavements.

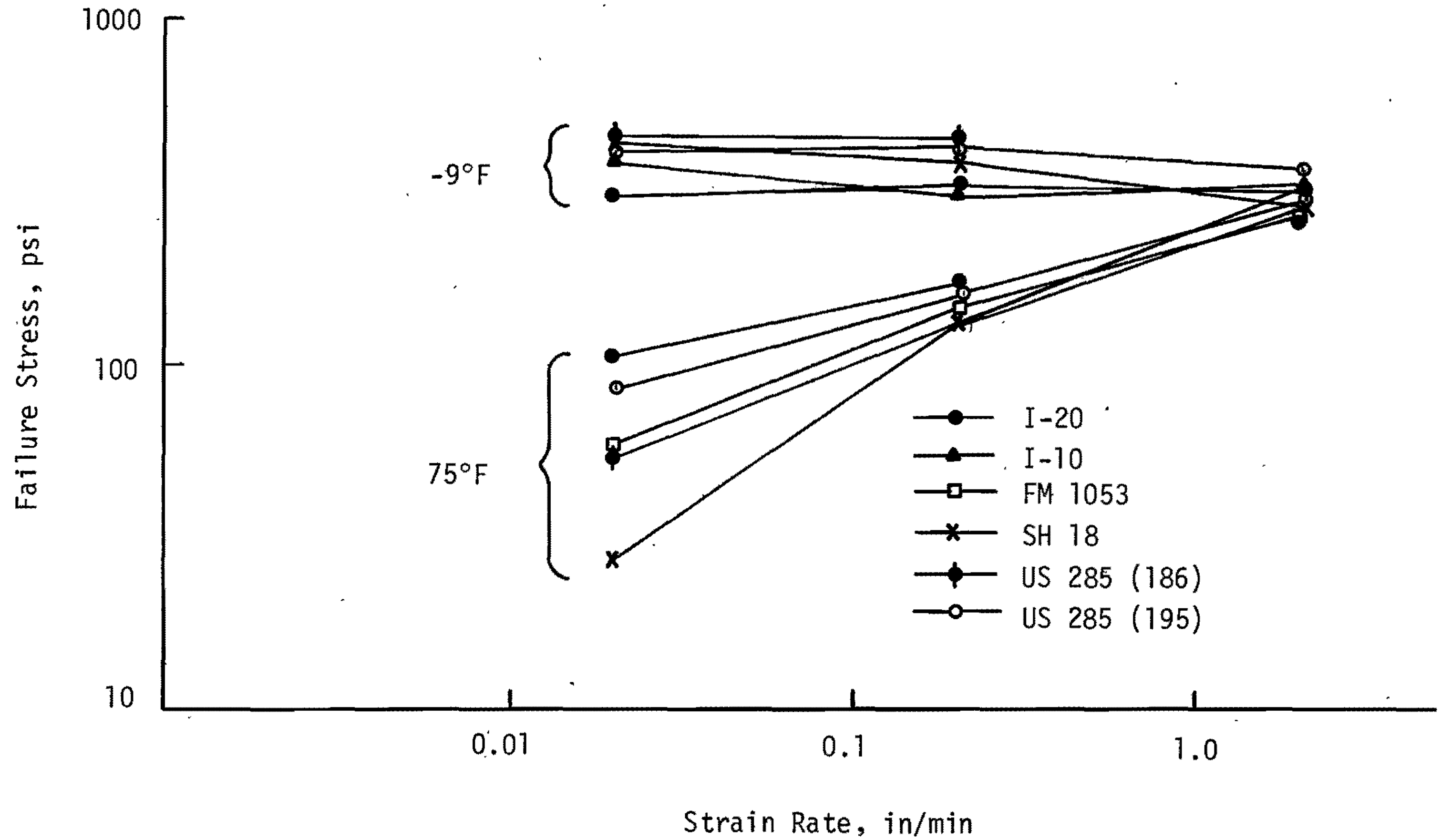


Figure 3. Influence of Strain Rate and Temperature on Failure Stress, Direct Tension

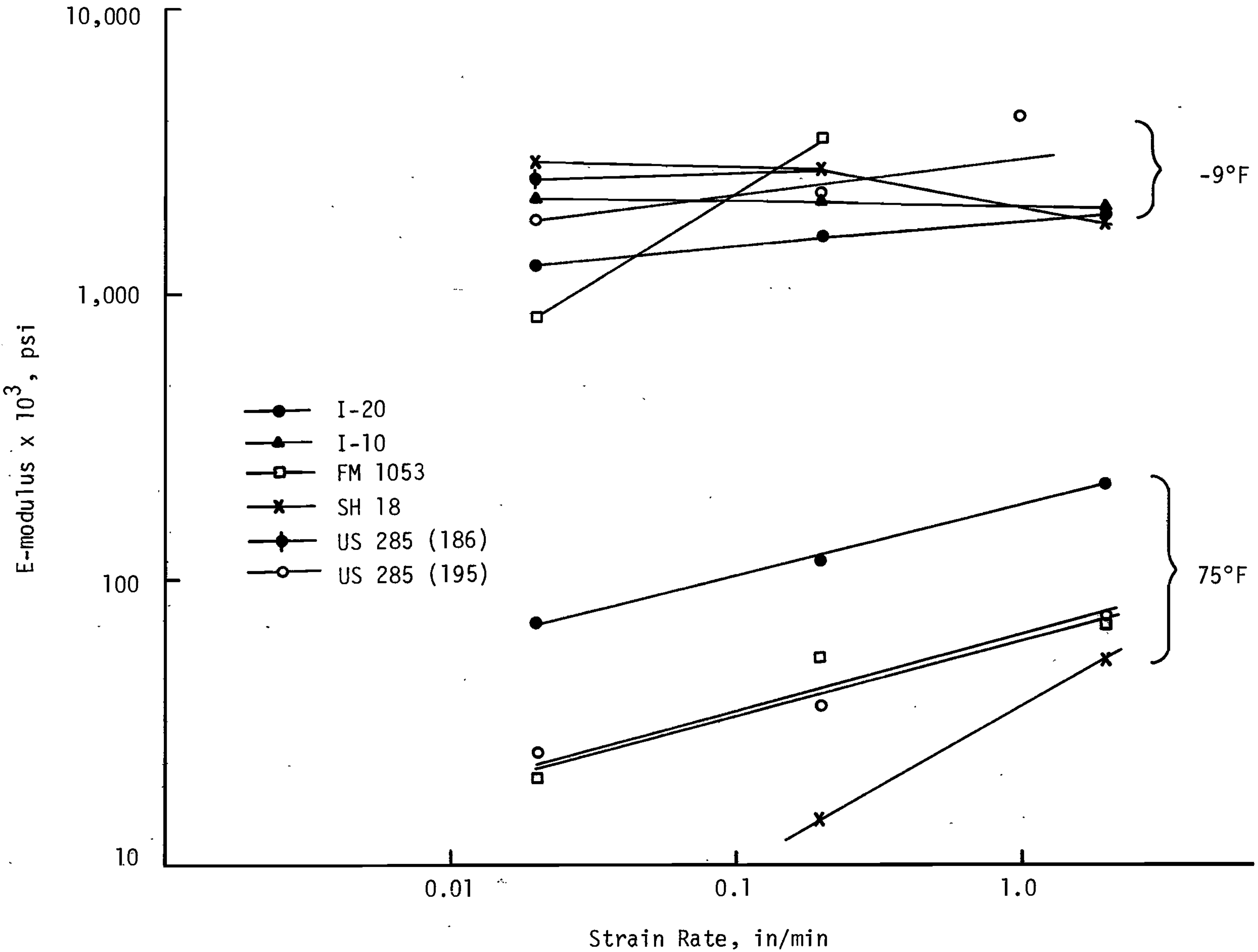


Figure 4. Influence of Strain Rate and Temperature on Failure Stress, Direct Tension.

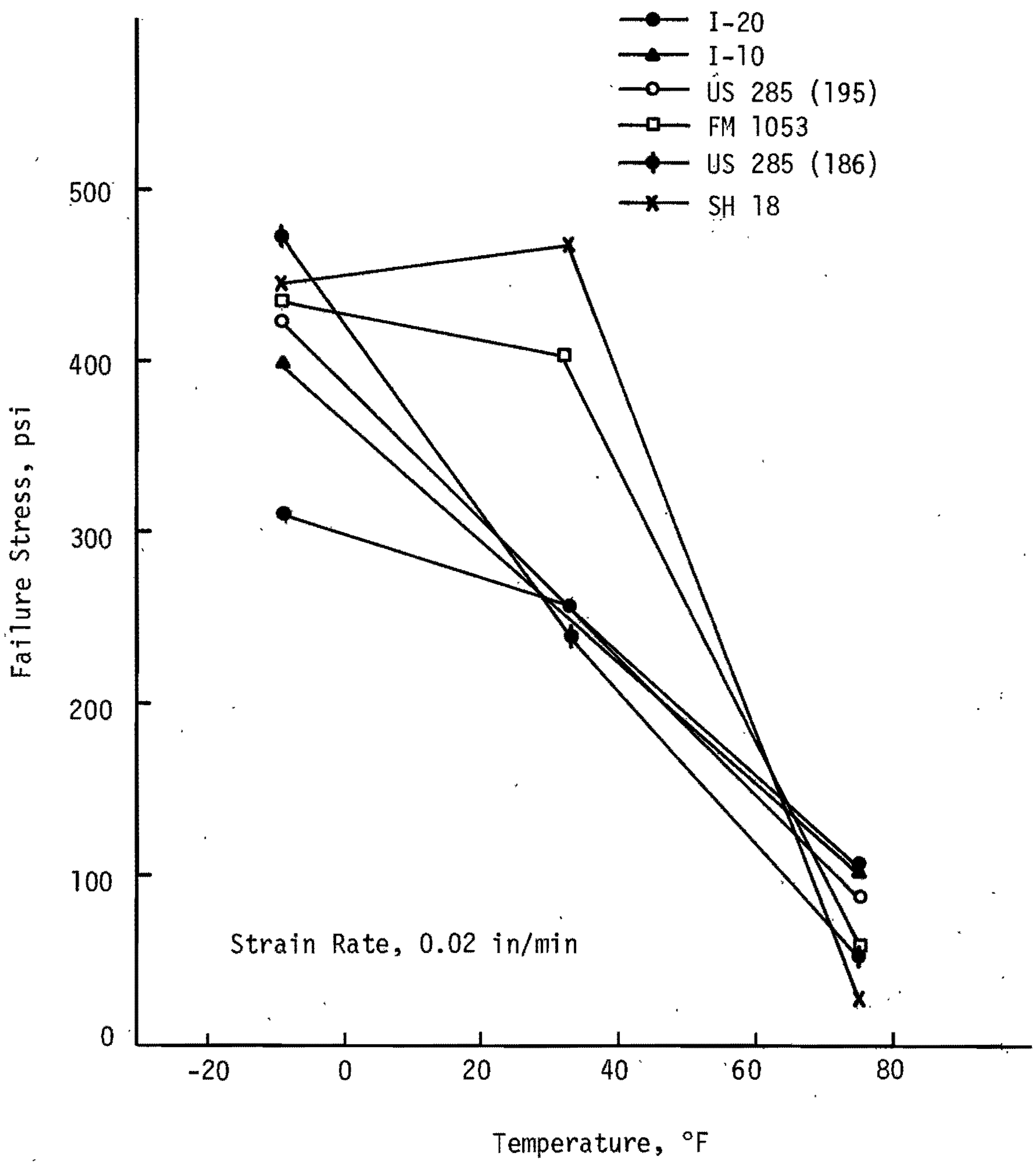


Figure 5. Failure Stress vs Temperature, Direct Tension

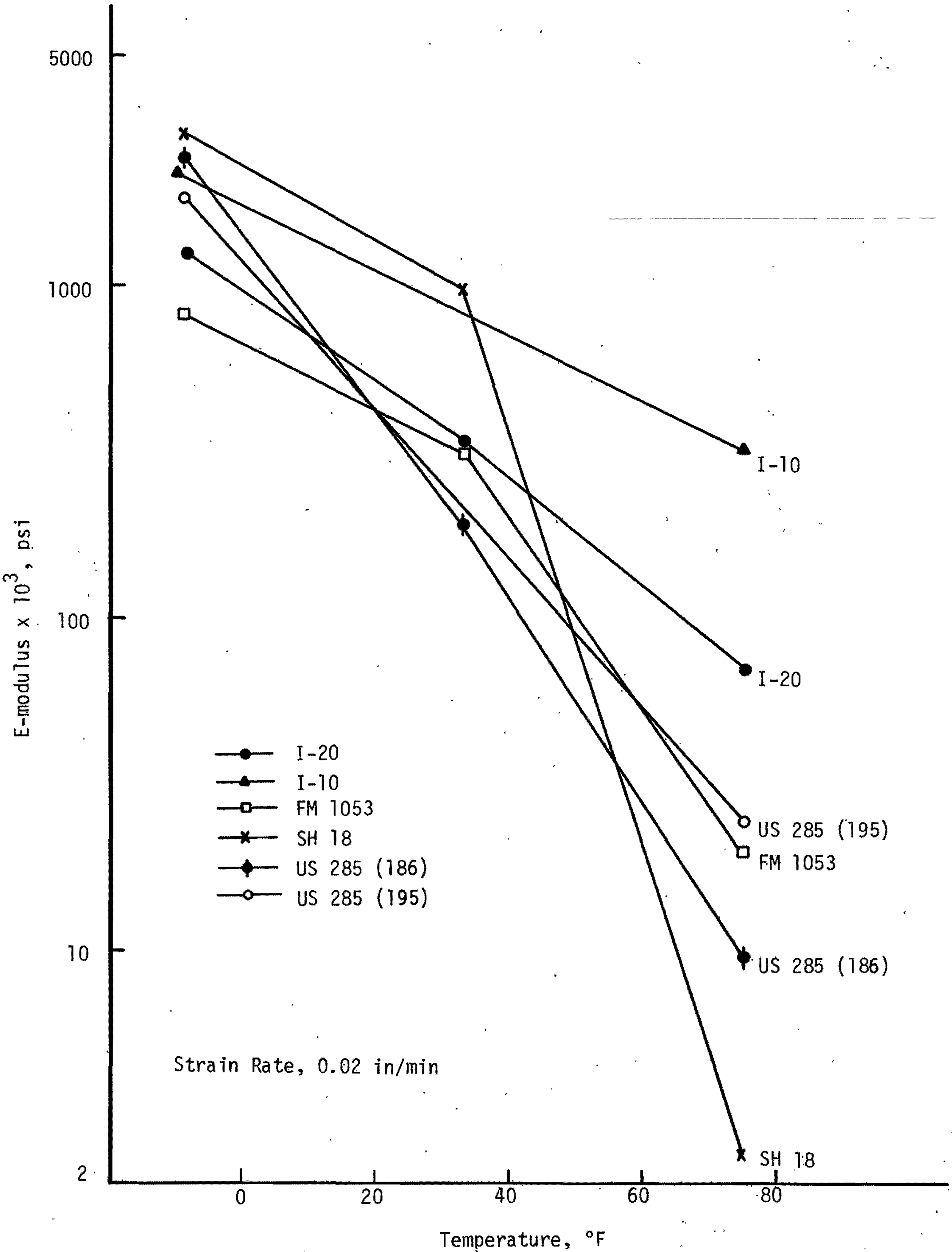


Figure 6. E-modulus vs Temperature, Direct Tension

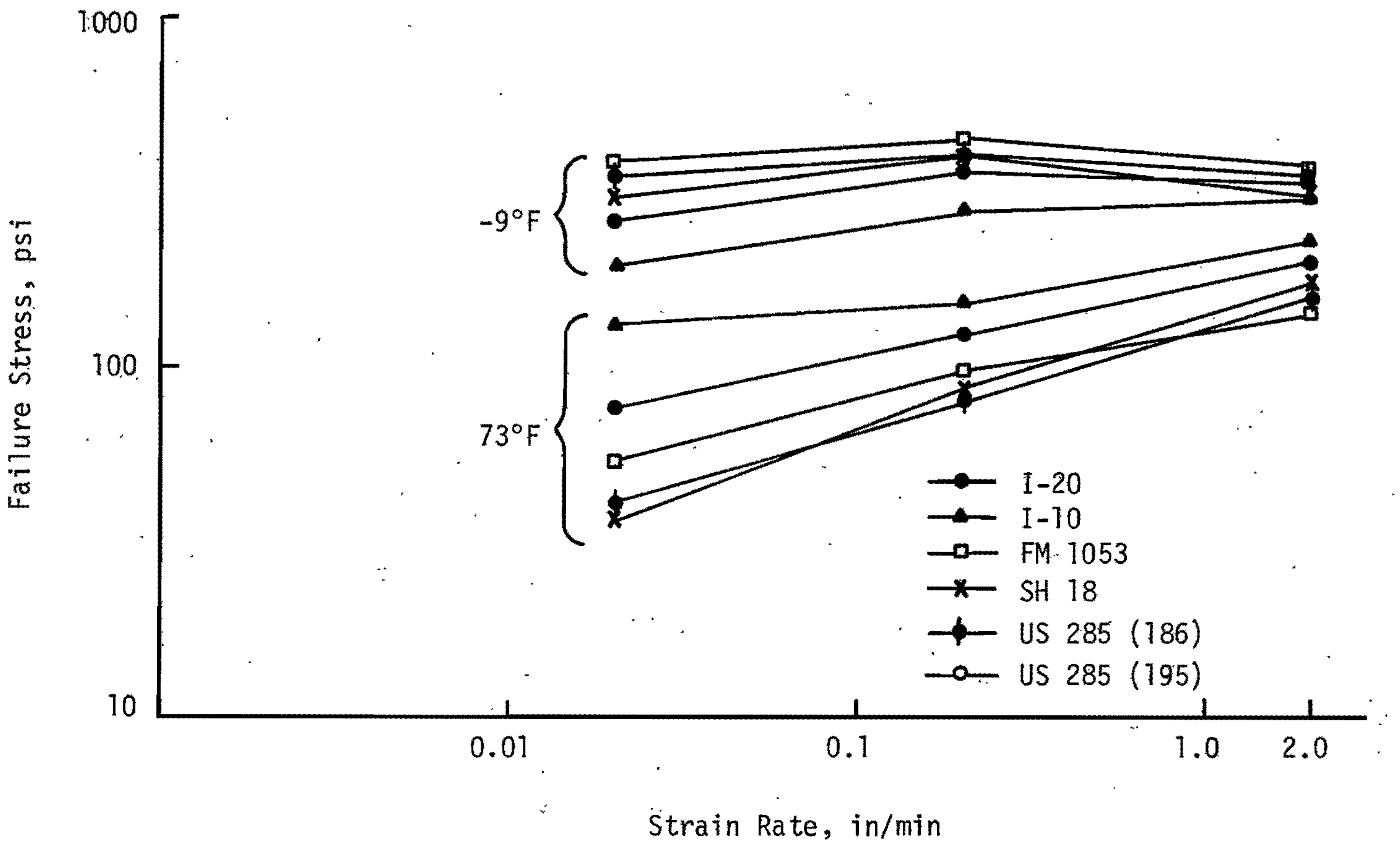


Figure 7. Influence of Strain Rate and Temperature on Failure Stress, Indirect Tension.

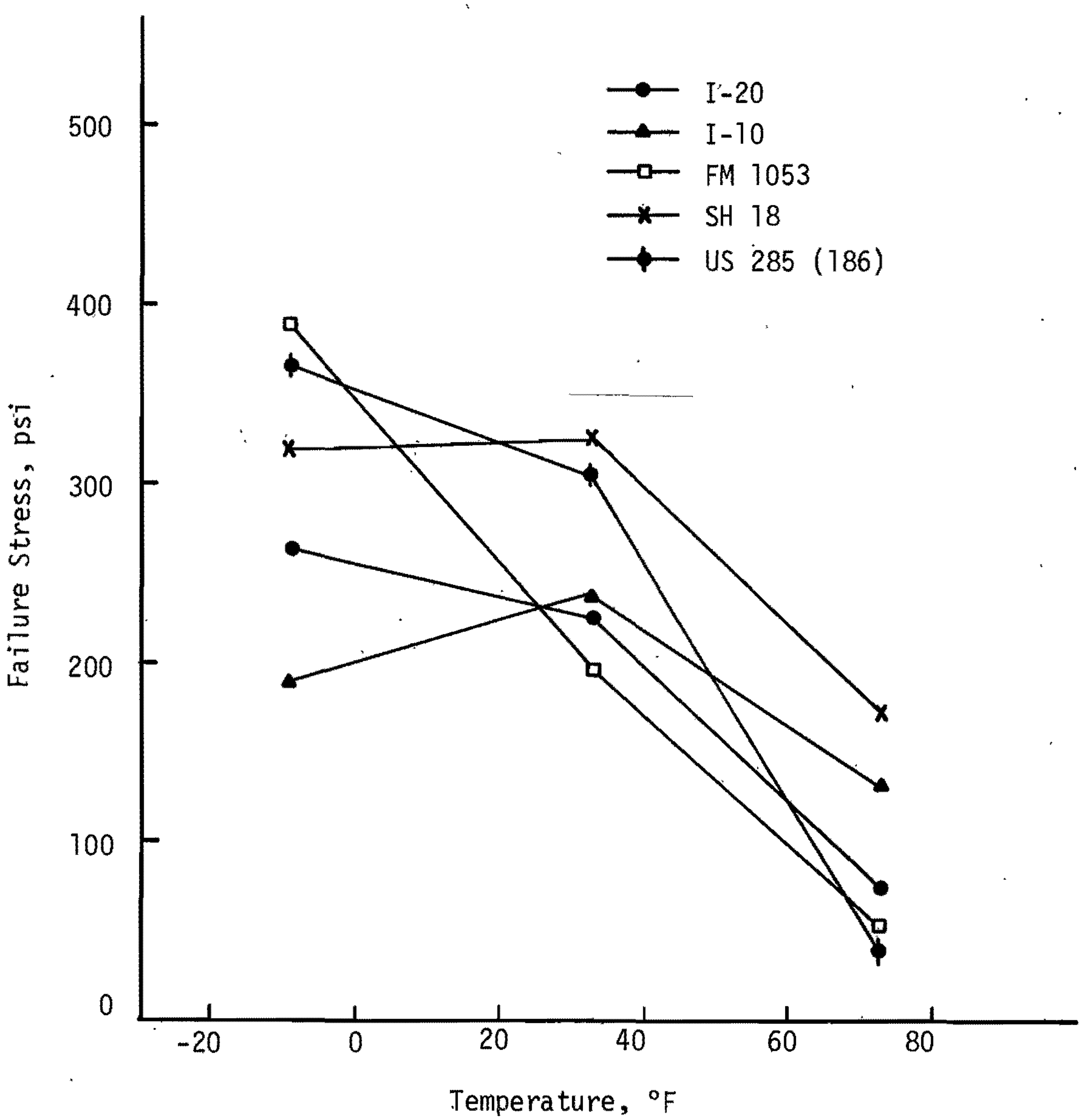


Figure 8. Failure Stress vs Temperature, Indirect Tension.

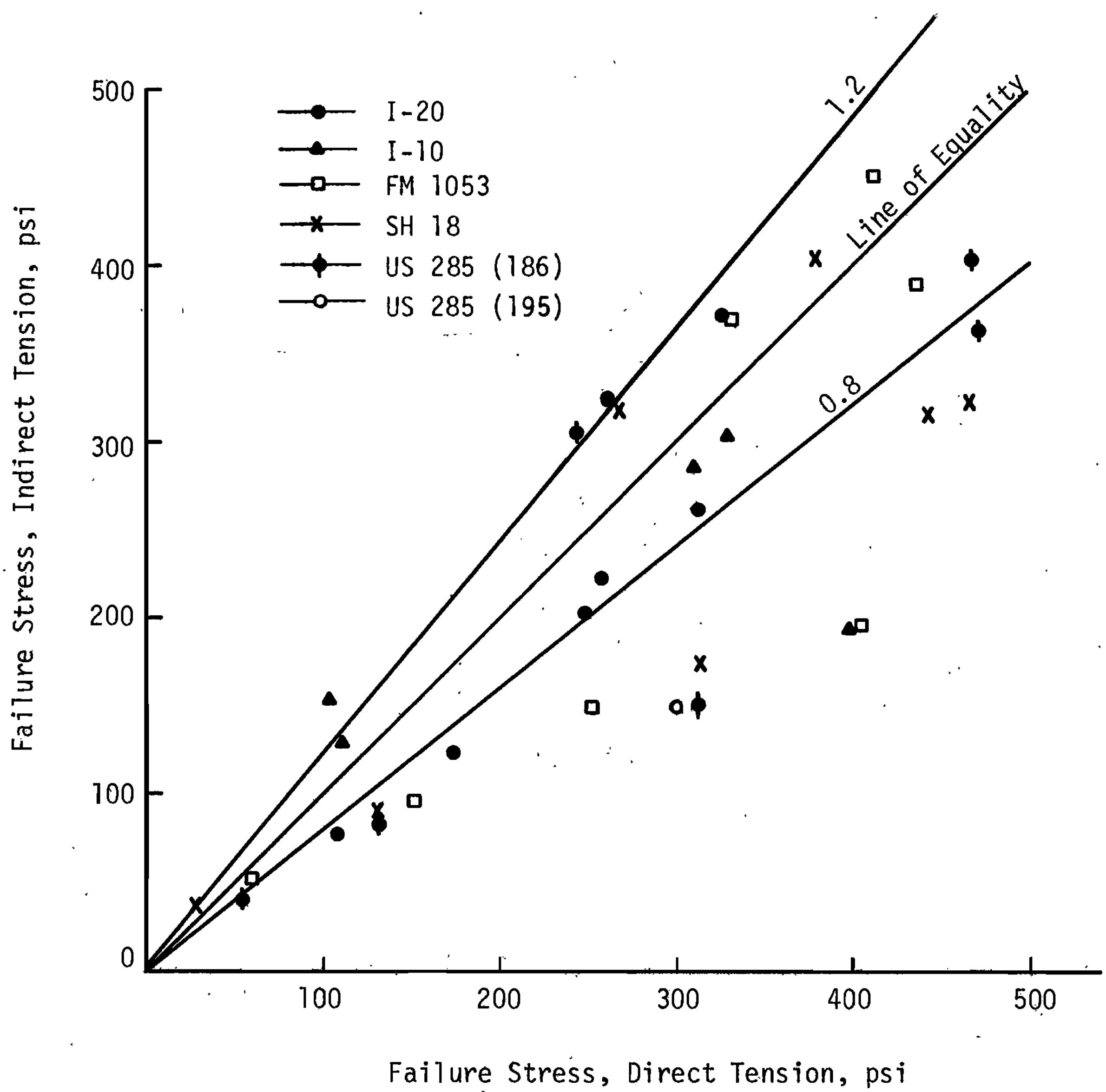


Figure 9. Comparison of Failure Stress from Indirect vs Direct Tension Tests.

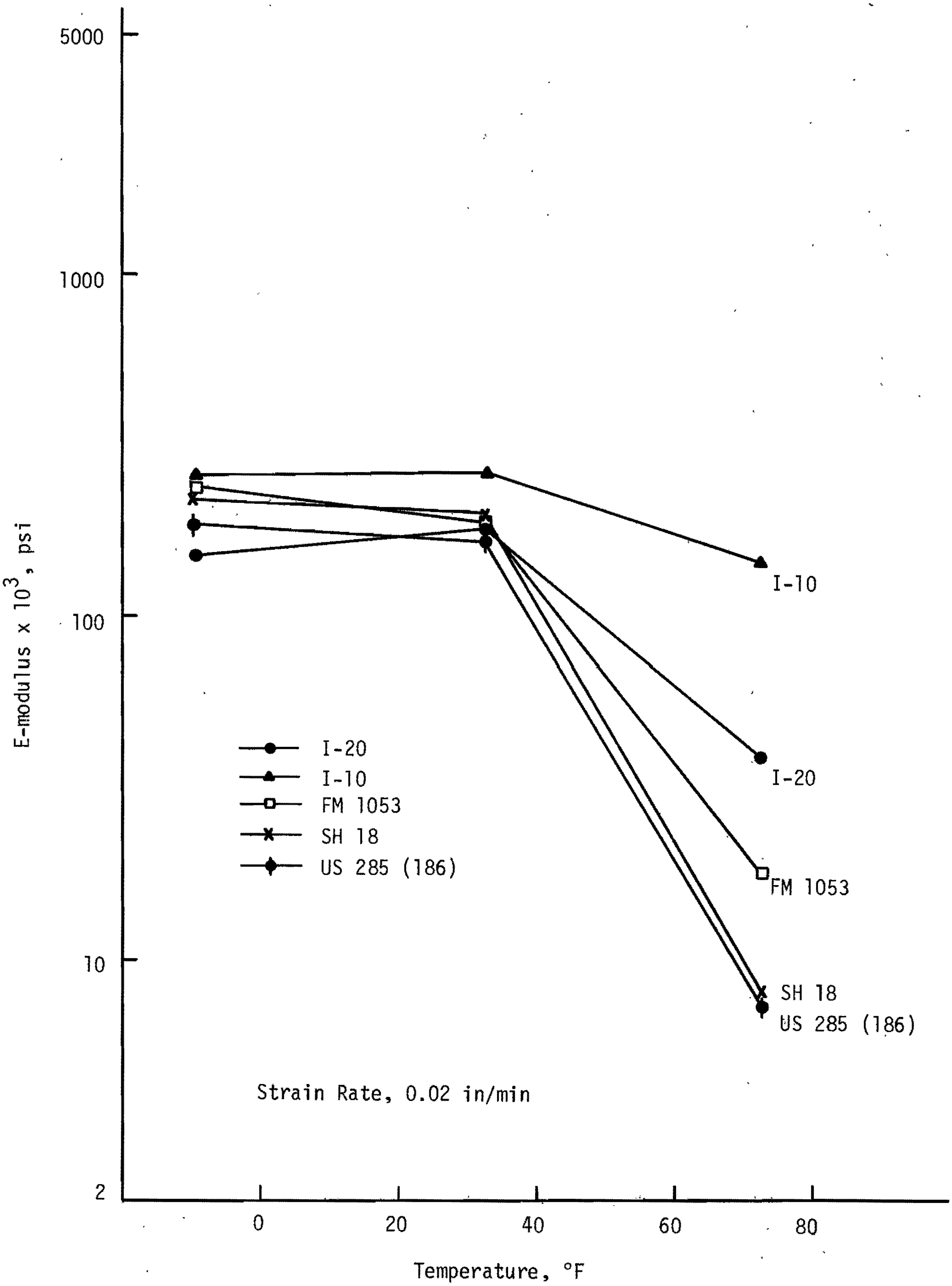


Figure 10. E-modulus vs Temperature, Indirect Tension

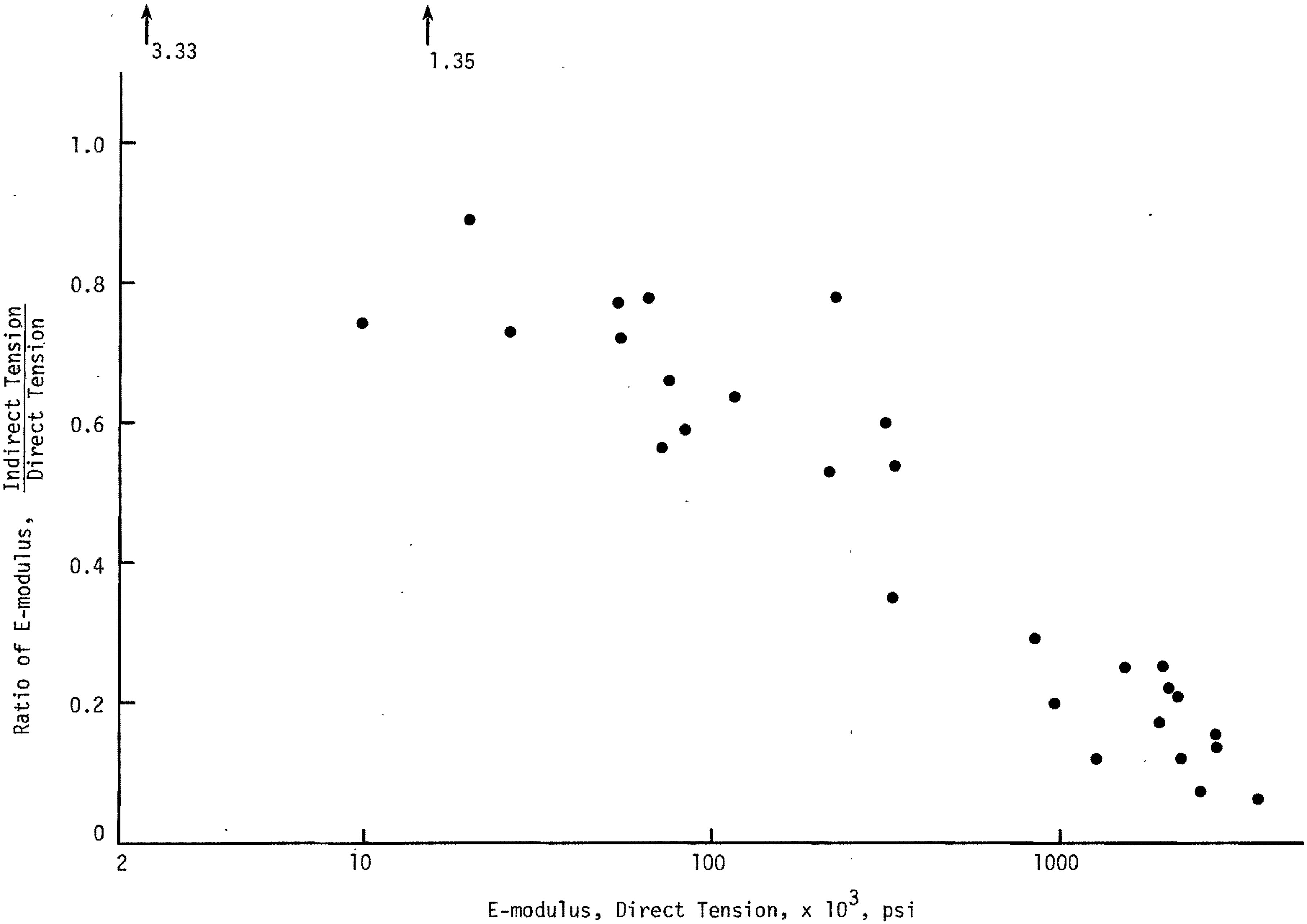


Figure 11. Comparison of E-modulus values determined by Indirect and Direct Tension Tests.

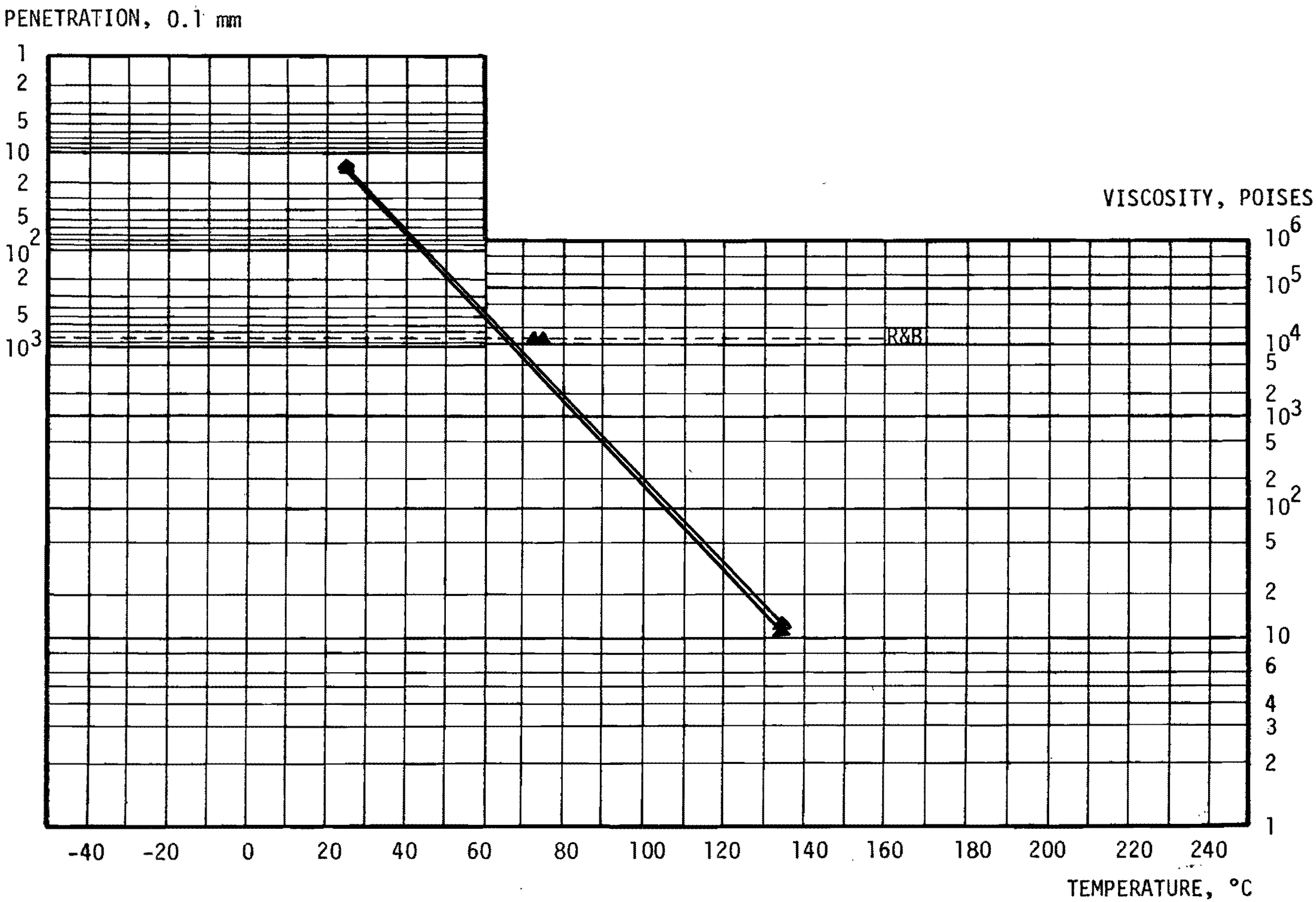


Figure 12. Recovered asphalt cement properties from Section I-10 plotted on the Bitumen Test Data Chart.

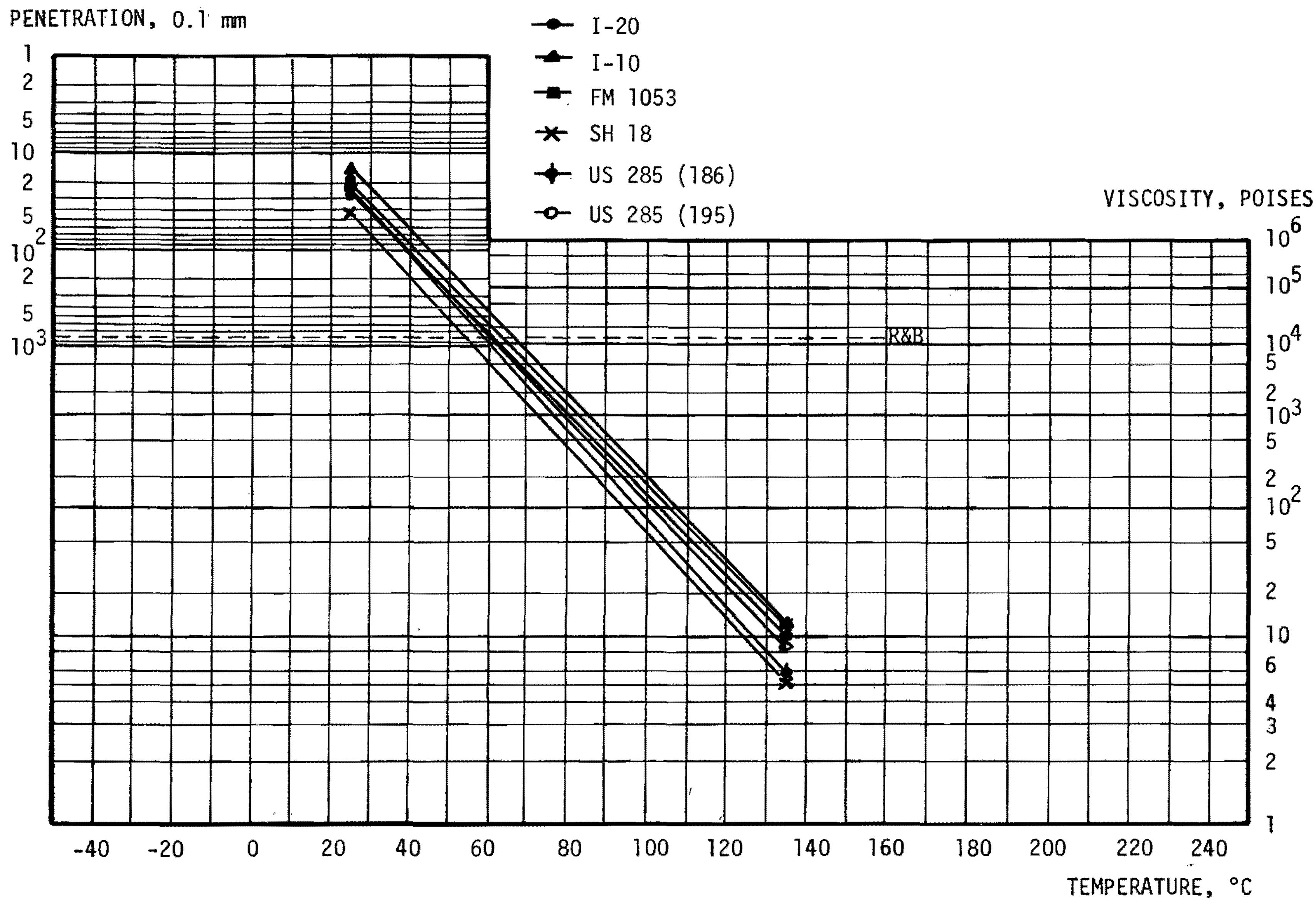


Figure 13. Recovered asphalt cement properties from all sections plotted on the Bitumen Test Data Chart.

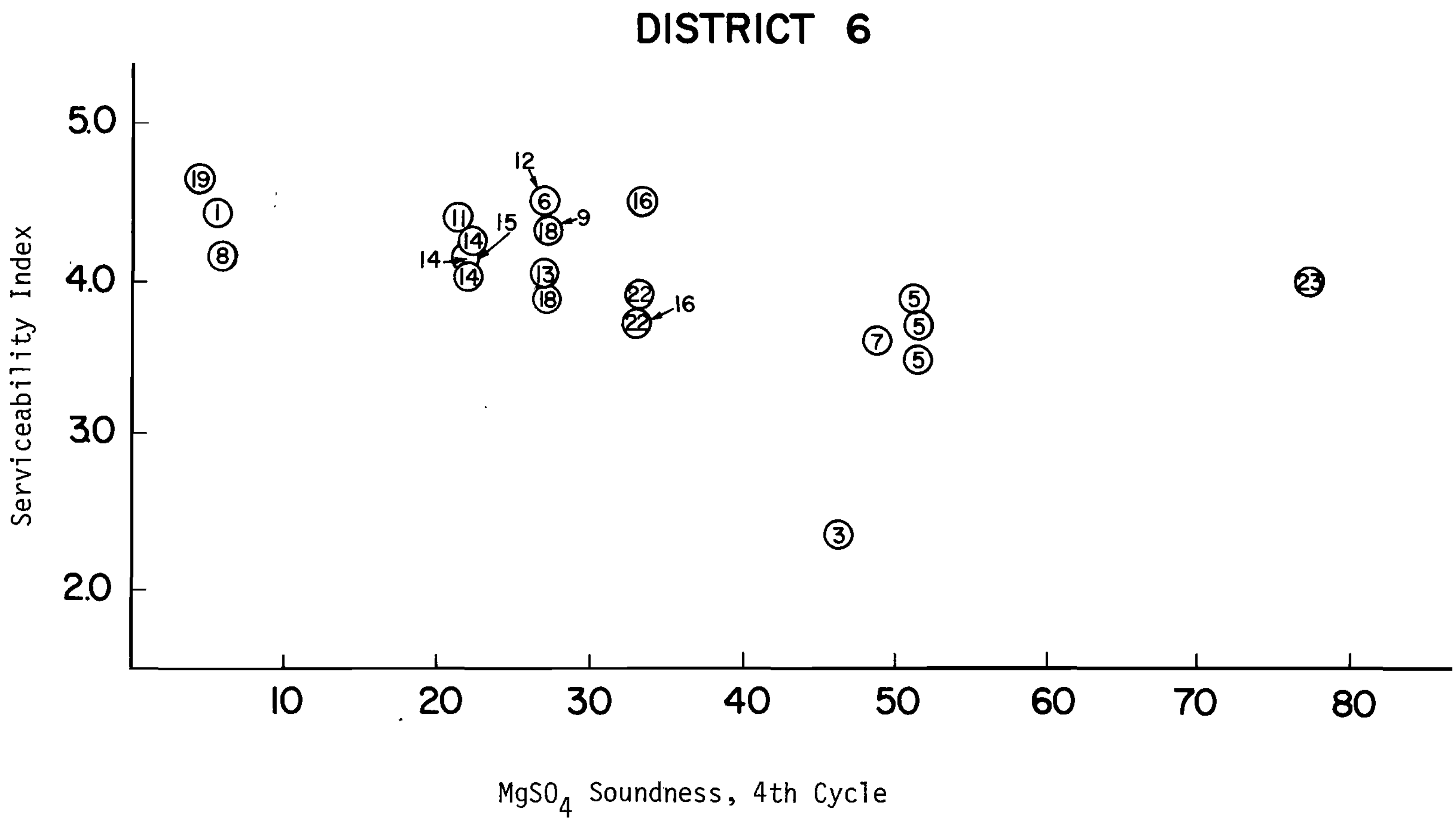


Figure 14. Serviceability Index vs Percent Loss in Soundness Test.

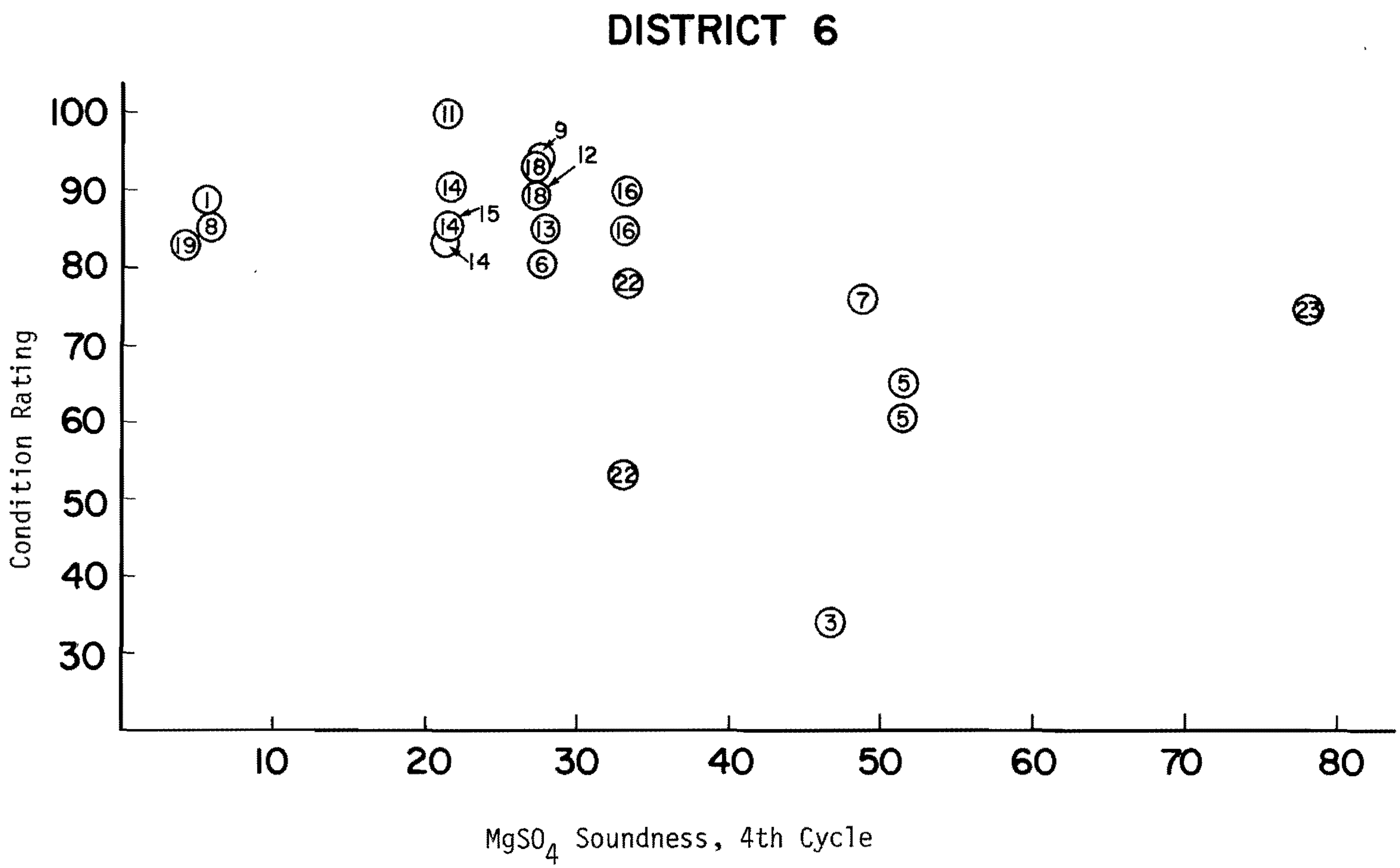


Figure 15. Condition Rating vs Percent Loss in Soundness Test.

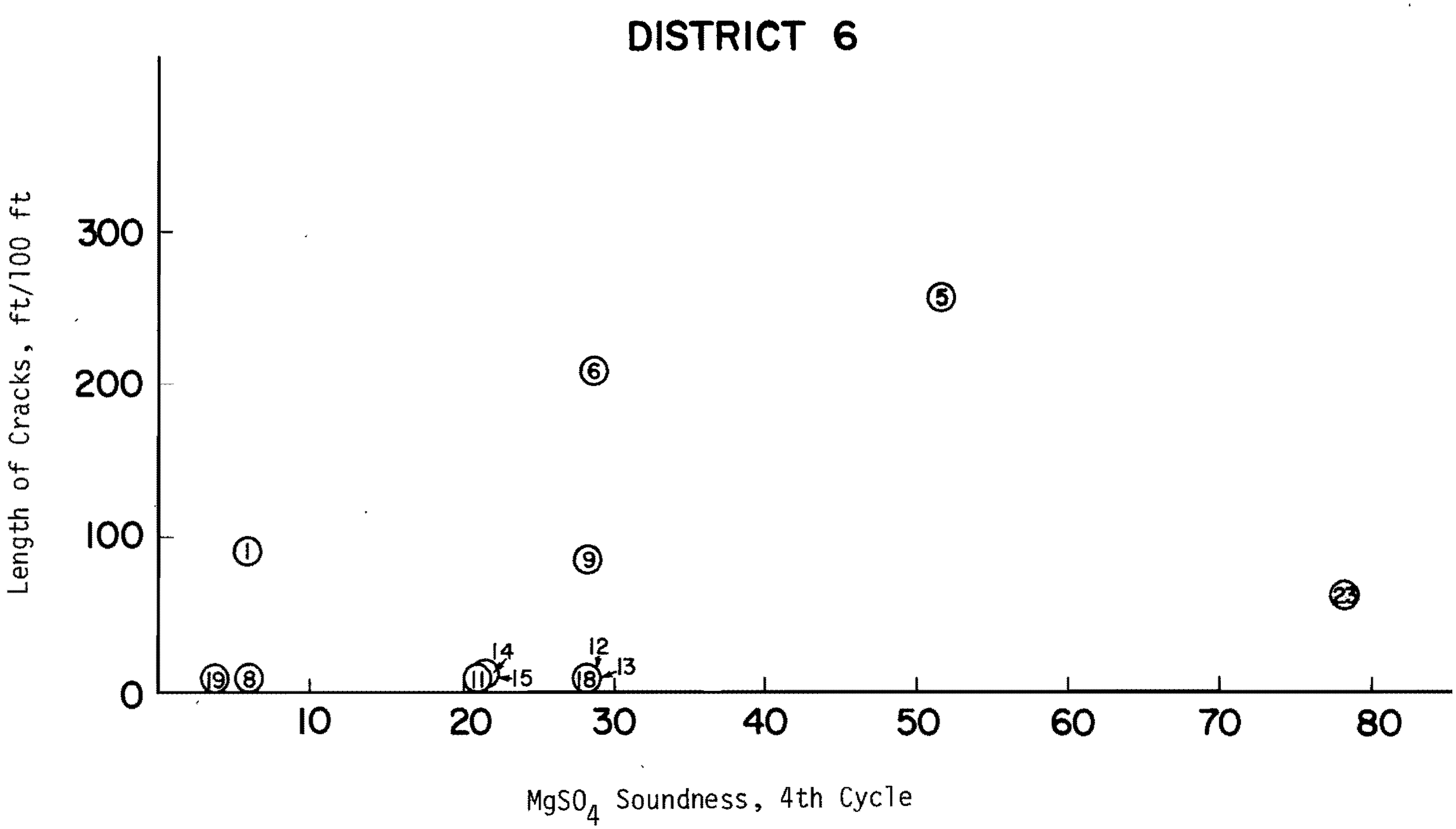


Figure 16. Length of Cracks vs Percent Loss in Soundness Test.

APPENDIX A - COLD Program Printout

************************************ PROBABILISTIC PREDICTION OF LOW TEMPERATURE CRACKING ************************************

PROJECT DESCRIPTION---- PROJECT 2287-COLD PROGRAM FOR WEST TEXAS DATA-SECTION I-20 CRACKED.

INPUT DATA FOR TEMPERATURE-STRESS PROGRAM

OPTION DETERMINING METHOD OF DATA INPUT = 0
OPTION ON PRINTING INPUT DATA = 0
NUMBER OF DAYS IN TEMPERATURE REGIME = 15
NUMBER OF LAYERS IN PAVEMENT = 3
NUMBER OF AIR TEMPS SPECIFIED PER DAY = 25
NUMBER OF RADIATION READINGS PER DAY = 25

TIME INCREMENT AT WHICH TEMPS TO BE CALC = 0.125
DEPTH INCREMENT AT WHICH TEMPS TO BE CALC.= 2.000
TEMP AT WHICH FREEZING STARTS IN SUBGRADE = 29.000
TEMP AT WHICH GIVEN PERCENT OF WATER IN
SUBGRADE IS FROZEN = 24.000

ABSORPTIVITY OF THE SURFACE = 0.900
EMISSIVITY OF THE SURFACE = 0.950
CONVECTION COEFFICIENT = 2.700

CONDUCTIVITY (UNFROZEN)	CONDUCTIVITY (FROZEN)	HT.CAPACITY (UNFROZEN)	HT.CAPACITY (FROZEN)	DRY DENSITY	MOISTURE CONTENT	THICKNESS
0.840	0.840	0.220	0.220	140.100	1.000	3.000
0.750	1.100	0.400	0.300	115.000	10.000	15.000
0.650	1.250	0.540	0.370	110.000	15.000	54.000

PERCENT OF WATER FROZEN AT TEMP F = 80.00 90.00

INITIAL TEMPERATURE GRADIENT

60.0	59.0	59.0	59.0	58.0	58.0	58.0	57.0
57.0	57.0	56.0	56.0	56.0	55.0	55.0	55.0
54.0	54.0	54.0	53.0	53.0	53.0	52.0	52.0
52.0	51.0	51.0	51.0	51.0	51.0	50.0	50.0
50.0	50.0	50.0	50.0	50.0			

1ST EXTREME TEMP AT 2 HRS
2ND EXTREME TEMP AT 14 HRS
SUNRISE TIME 8 HRS
SUNSET TIME 18 HRS

DATE	FIRST EXTREME TEMP (DEG F)	SECOND EXTREME TEMP (DEG F)	SOLAR RADIATION (LANGLEYS)
DEC. 25,1970	30.0	52.0	331.0
DEC. 26,1970	24.0	64.0	316.0
DEC 27,1970	34.0	75.0	281.0
DEC. 28,1970	43.0	74.0	185.0
DEC. 29,1970	45.0	61.0	86.0
DEC. 30,1970	37.0	65.0	338.0
DEC. 31,1970	26.0	62.0	254.0
JAN. 1,1971	35.0	70.0	316.0
JAN 2,1971	35.0	64.0	125.0
JAN. 3,1971	22.0	55.0	270.0
JAN. 4,1971	13.0	33.0	321.0
JAN. 5,1971	2.0	30.0	328.0
JAN. 6,1971	16.0	32.0	352.0
JAN. 7,1971	14.0	36.0	336.0
JAN. 8,1971	13.0	64.0	359.0

LAST TEMPERATURE GRADIENT

39.83	42.78	44.58	45.03	44.74	44.24	43.69	43.84
44.11	44.75	46.59	46.48	47.35	48.14	48.84	49.45
49.97	50.40	50.74	51.02	51.22	51.37	51.46	51.50
51.50	51.47	51.41	51.32	51.21	51.09	50.98	50.81
50.65	50.50	50.33	50.17	50.00			

CODE DENOTING CONDITION = 1.00

NUMBER OF VALUES FROM STIFFNESS
MODULUS/TEMPERATURE RELATIONSHIP = 5

NUMBER OF VALUES FROM THE TENSILE
STRENGTH/TEMPERATURE RELATIONSHIP = 5

COEFFICIENT OF THERMAL EXPANSION = 0.000020

COEFFICIENT OF VARIATION OF MODULUS = 0.25

COEFFICIENT OF VARIATION OF STRENGTH= 0.25

STIFFNESS MODULUS (PSI)	TEMPERATURES COMPATIBLE WITH STIFFNESS (DEG F)
3000.0	75.0
70000.0	50.0
500000.0	33.0
1800000.0	10.0
2000000.0	-9.0

STRENGTH VALUES TEMPERATURES

FROM HIGHEST TEMPERATURES (PSI)	COMPATIBLE WITH STRENGTH VALUES (DEG F)
107.0	76.0
195.0	50.0
258.0	33.0
284.0	10.0
310.0	-9.0

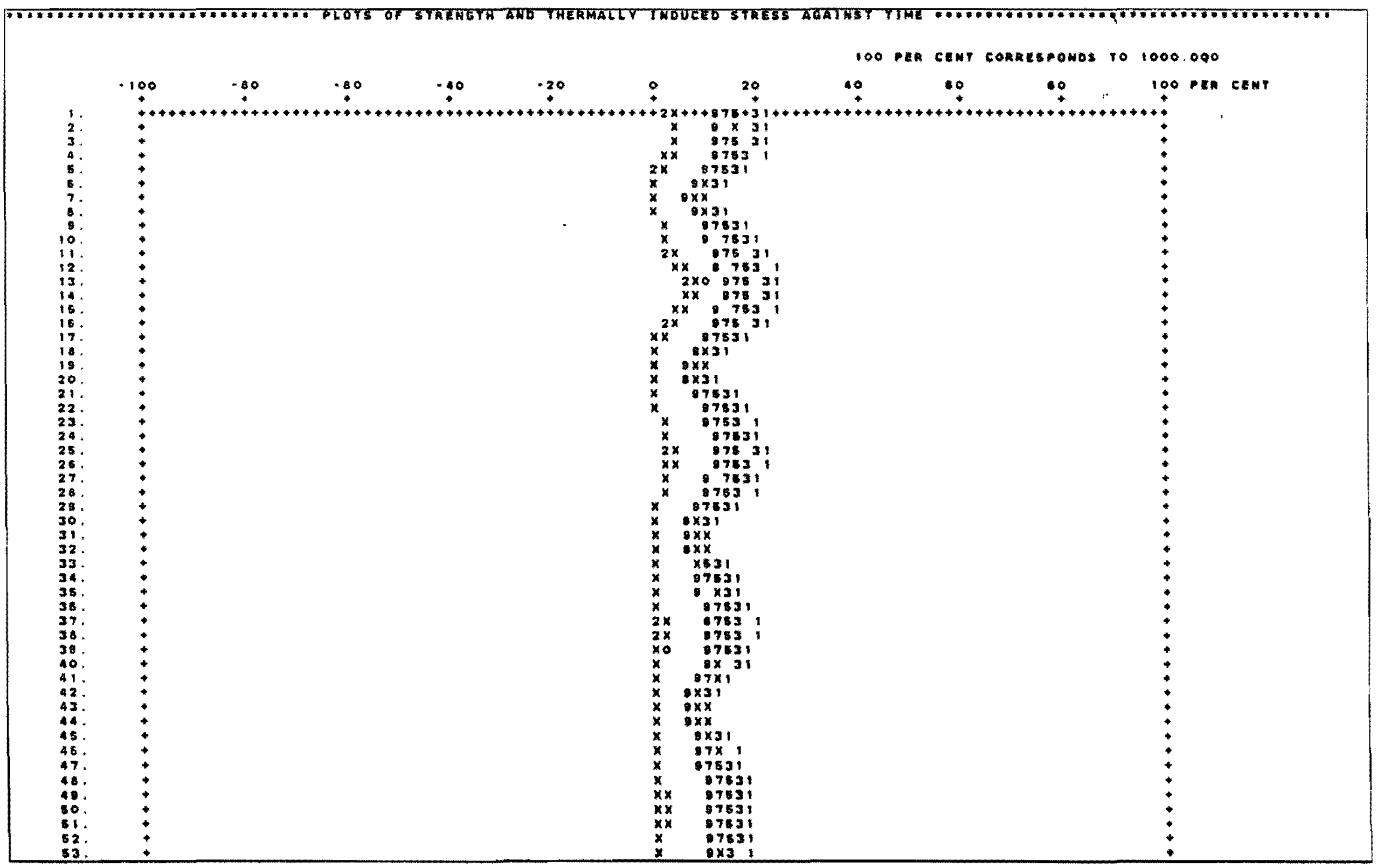
******************************* SUMMARY TABLE OF PREDICTIONS OF LOW TEMPERATURE CRACKING ***

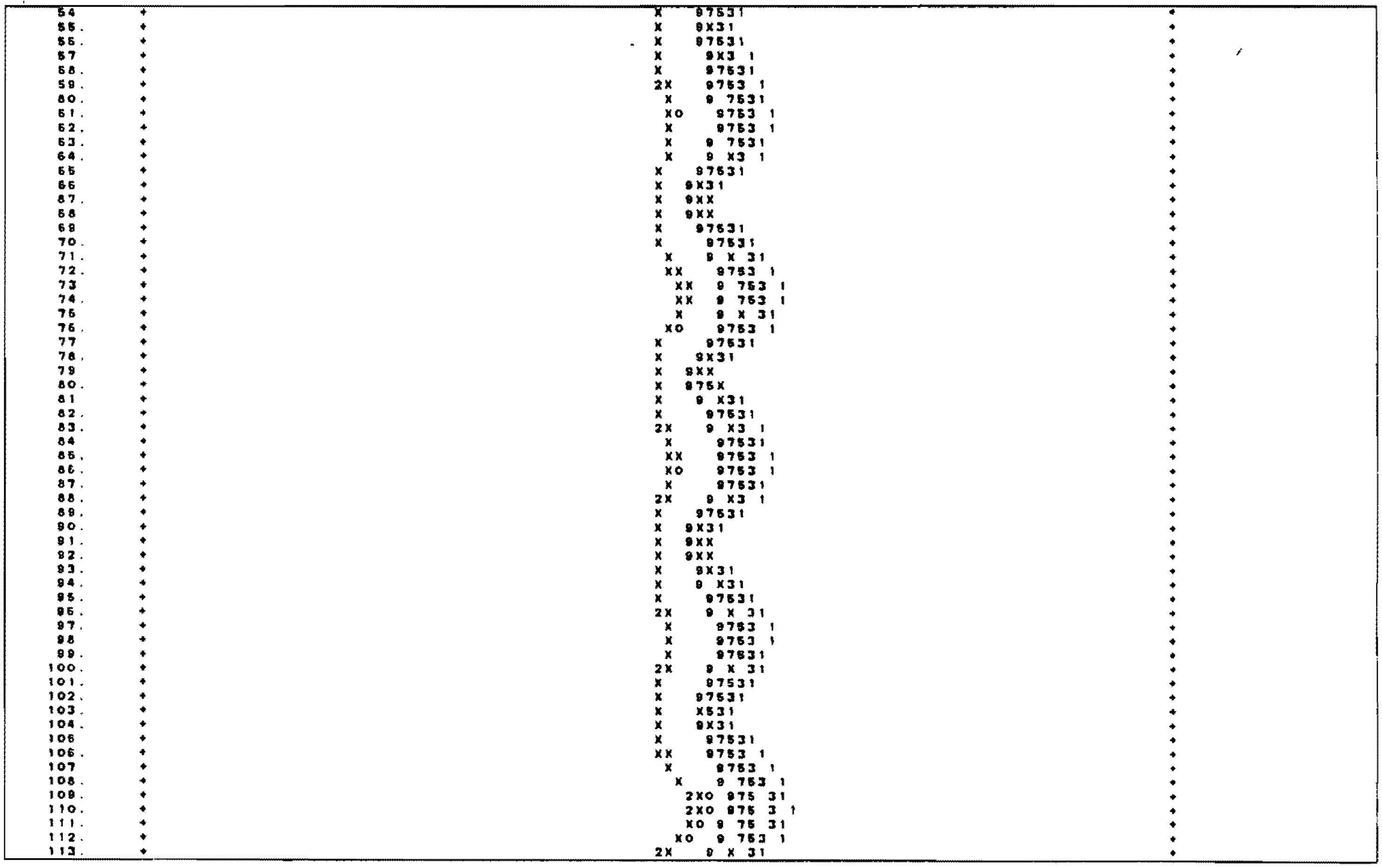
NOTE- * INDICATES THAT EXPECTED THERMALLY INDUCED STRESS HAS EXCEEDED EXPECTED STRENGTH

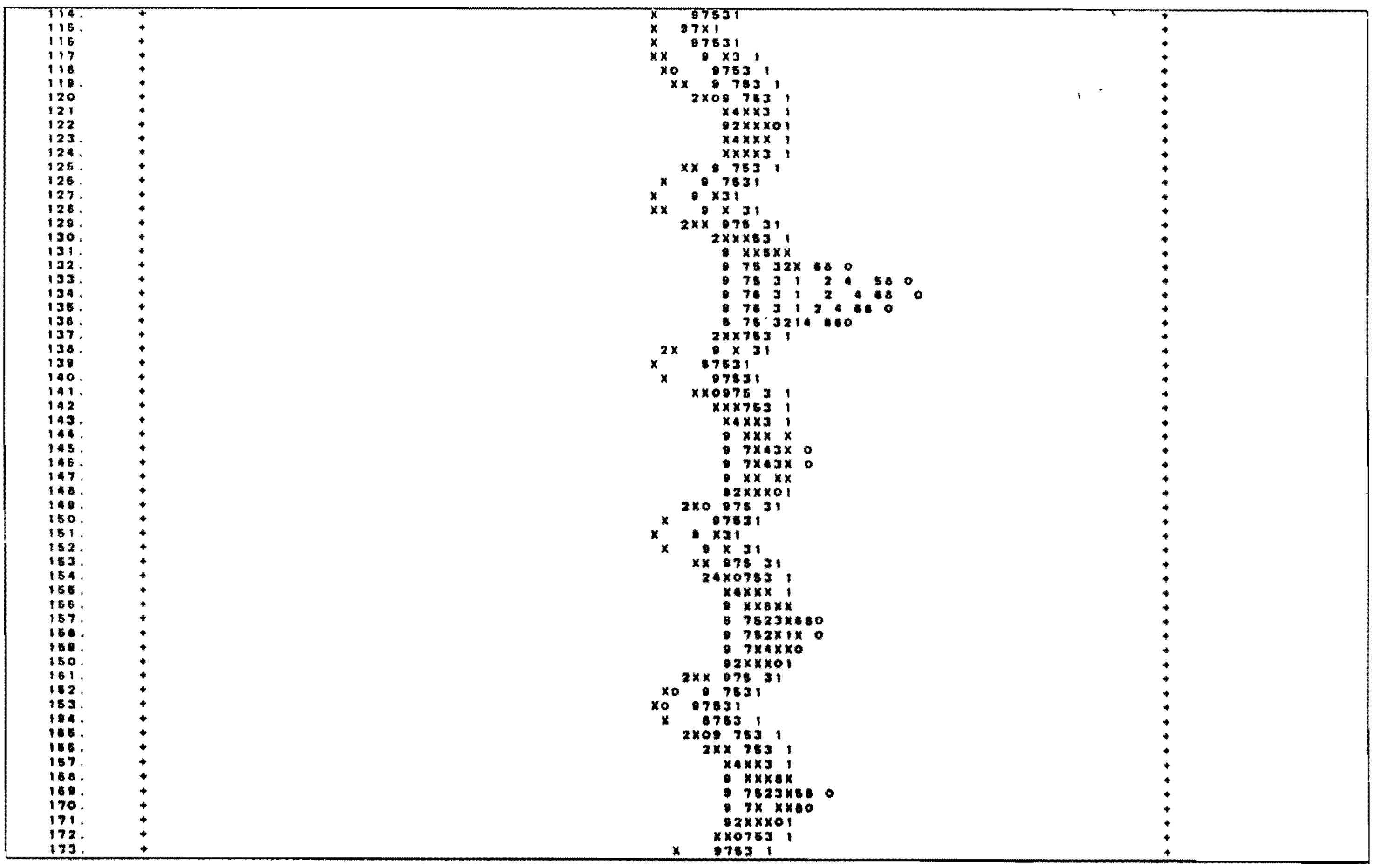
				-EXPECTED VALUES-		75PRCNT RELIABILITY		90PRCNT RELIABILITY		95PRCNT RELIABILITY		99PRCNT RELIABILITY	
DAY	TIME PERIOD	AIR TEMP	PVMNT TEMP.	STRENGTH	INDUCED STRESS	STRENGTH	INDUCED STRESS	STRENGTH	INDUCED STRESS	STRENGTH	INDUCED STRESS	STRENGTH,	INDUCED STRESS
0	0	32.0	60.0	0.0	0.0								
1													
	1	30.0	43.2	220.2	29.0	190.4	33.0	163.8	36.5	147.7	38.6	117.6	42.6
	2	33.7	41.9	224.9	33.3	194.6	37.4	167.4	41.8	150.9	44.3	120.1	48.8
	3	37.3	42.7	222.1	30.7	192.2	34.9	165.3	38.6	149.1	40.9	118.6	45.1
	4	41.0	44.2	216.7	26.3	187.4	29.9	161.2	33.0	146.4	35.0	115.7	38.6
	5	44.7	53.0	164.4	9.9	159.[illegible]	11.3	137.2	12.5	123.8	13.2	98.5	14.5
	6	48.3	63.6	147.2	3.5	127.3	4.0	109.6	4.4	98.8	4.7	78.6	5 2
	7	52.0	74.9	107.3	1.8	92.8	2.0	78.8	2.2	72.0	2.3	57.3	2.8
	8	47.3	66.1	138.2	2.8	119.5	3.2	102.8	3.5	92.7	3.8	73.8	4.1
	9	42.7	52.4	185.4	11.1	161.3	12.6	138.7	14.0	125.1	14.8	99.8	16.3
	10	38.0	47.2	206.3	18 8	177.8	21.3	152.7	23.6	137.7	25.0	109.5	27.6
	11	33.3	43.2	220.3	28.9	190.9	32.9	163.9	36.4	147.8	38.5	117.6	42.4
	12	28.7	39.4	234.4	43.9	202.8	49.8	174.4	55.2	157.3	58.4	125.2	64.4
2													
	1	24.0	35.6	248.3	66.6	214.8	75.6	184.7	83.7	166.6	88.5	132.6	97.7
	2	30.7	36.4	245.2	60.8	212.1	69.0	182.4	76.4	164.5	80.8	130.9	89.2
	3	37.3	39.5	234.1	43.7	202.6	49.6	174.2	54.8	157.1	58.0	125.0	64.0
	4	44.0	43.2	220.3	28.1	190.5	33.0	163.9	36.6	147.8	38.7	117.6	42.7
	5	50.7	54.0	180.9	7.9	156.5	9.0	134.8	9.9	121.4	10.5	96.5	11.6
	6	57.3	66.5	136.9	1.6	118.4	1.8	101.9	2.0	91.9	2.1	73.1	2.3
	7	64.0	79.8	107.0	0.0	92.6	0.0	79.8	0.0	71.8	0.0	57.1	0.0
	8	59.0	72.3	116.6	0.5	100.8	0.6	86.7	0.7	78.2	0.7	62.2	0.8
	9	54.0	59.5	161.7	3.8	139.9	4.3	120.3	4.8	108.5	5.0	86.4	5.5
	10	49.0	54.5	179.1	6.8	154.9	7.7	133.2	8.5	120.1	9.0	95.6	9.9
	11	44.0	50.6	193.0	10.9	166.9	12.4	143.6	13.7	129.5	14.5	103.1	16.0
	12	39.0	46.8	207.0	17.2	179.0	19.5	154.0	21.7	138.9	22.9	110.5	25.3
3													
	1	34.0	43.0	221.0	27.0	191.1	30.7	164.4	33.9	148.3	35.9	118.0	39.6
	2	40.6	44.0	217.3	24.1	188.0	27.3	161.7	30.3	145.8	32.0	116.1	35.3
	3	47.7	47.2	205.3	16.4	177.6	18.6	152.6	20.6	137.8	21.8	109.5	24.1
	4	54.5	51.2	190.8	10.2	165.0	11.6	142.0	12.8	128.0	13.5	101.9	15.0
	5	61.3	61.5	154.6	2.3	133.7	2.6	115.0	2.9	103.7	3.1	82.5	3.4
	5	68.2	73.3	113.0	-0.0	97.7	-0.1	84.0	-0.1	75.8	-0.1	60.3	-0.1
	7	75.0	85.8	107.0	-0.9	92.6	-1.0	79.6	-1.1	71.8	-1.2	57.1	-1.3
	8	68.7	79.1	107.0	-0.5	92.6	-0.6	79.6	-0.6	71.8	-0.8	57.1	-0.7
	9	64.3	67.4	133.8	0.8	115.7	0.9	99.5	1.0	89.8	1.0	71.4	1.1
	10	59.0	62.8	150.8	1.8	130.4	2.1	112.2	2.3	101.2	2.5	80.6	2.7
	11	53.7	58.5	165.0	3.4	142.8	3.9	122.8	4.3	110.7	4.5	88.1	5.0
	12	48.3	54.6	178.9	5.9	154.8	6.7	133.1	7.4	120.1	7.8	95.5	8.6
4													
	1	43.0	50.5	192.8	10.0	166.8	11.3	143.5	12.5	129.4	13.3	103.0	14.5
	2	46.2	50.9	191.8	9.6	165.9	10.9	142.7	12.1	128.7	12.8	102.4	14.1
	3	53.3	53.2	183.9	7.2	159.1	8.1	136.8	9.0	123.4	9.5	98.2	10.5
	4	58.5	56.0	173.7	4.9	150.3	5.6	129.2	6.1	116.6	6.5	92.8	7.1
	5	63.7	63.2	148.6	1.6	128.8	1.8	110.6	2.0	99.7	2.1	79.4	2.3
	6	68.6	71.3	118.9	0.1	103.7	0.1	89.2	0.2	80.4	0.2	64.0	0.2

	7	74.0	80.0	107.0	-0.5	92.6	-0.6	79.6	-0.7	71.8	-0.7	57.1	-0.8
	8	69.2	75.3	107.0	-0.3	92.6	-0.3	79.6	-0.3	71.8	-0.3	57.1	-0.4
	9	64.3	66.8	136.0	0.7	117.6	0.8	101.2	0.9	91.2	0.9	72.6	1.0
	10	59.5	62.7	150.2	1.6	129.9	1.8	111.7	2.0	100.8	2.1	80.2	2.4
	11	54.7	59.2	162.7	2.6	140.8	3.3	121.1	3.8	109.2	3.9	88.9	4.3
	12	49.8	55.6	175.2	4.9	151.5	5.6	130.3	6.1	117.8	6.5	93.6	7.2
5													
	1	45.0	52.0	187.8	8.1	162.5	9.1	139.7	10.1	128.0	10.7	100.3	11.8
	2	47.7	51.6	189.8	8.7	164.2	9.9	141.2	10.9	127.4	11.6	101.4	12.7
	3	50.3	52.3	186.9	7.8	161.7	8.8	139.1	9.8	125.4	10.4	99.8	11.4
	4	53.0	53.6	182.5	6.6	157.9	7.5	135.8	8.3	122.5	8.7	97.5	9.6
	5	55.7	56.8	170.9	4.1	147.9	4.7	127.2	5.2	114.7	5.5	91.3	6.1
	6	58.3	60.7	157.4	2.3	136.2	2.6	117.1	2.9	105.6	3.1	84.1	3.4
	7	61.0	64.6	142.9	1.1	123.8	1.3	106.3	1.4	95.9	1.6	76.3	1.6
	8	57.0	61.9	153.3	1.9	132.5	2.1	114.0	2.4	102.9	2.5	81.9	2.8
	9	53.0	56.8	171.1	4.2	148.0	4.7	127.3	5.3	114.8	5.6	91.4	6.1
	10	49.0	53.7	181.9	5.5	157.4	7.3	135.4	8.1	122.1	8.5	97.2	9.5
	11	45.0	50.8	192.3	9.5	166.3	10.9	143.0	12.0	129.0	12.7	102.7	14.1
	12	41.0	47.8	203.1	14.1	175.7	16.0	151.1	17.7	136.3	18.8	108.4	20.7
6													
	1	37.0	44.8	214.2	20.7	185.3	23.4	159.4	25.9	143.7	27.4	114.4	30.3
	2	41.7	45.3	212.5	19.5	183.8	22.1	158.1	24.5	142.8	25.9	113.5	28.6
	3	46.3	47.4	204.7	15.0	177.1	17.0	152.3	18.8	137.4	19.9	109.3	22.0
	4	51.0	50.0	195.0	10.7	168.7	12.1	145.1	13.4	130.9	14.2	104.2	15.7
	5	55.7	60.0	159.9	1.7	138.3	2.0	119.0	2.2	107.3	2.3	85.4	2.5
	6	60.3	71.5	118.9	-1.1	102.9	-1.3	88.5	-1.4	79.8	-1.5	63.5	-1.6
	7	65.0	84.0	107.0	-2.1	92.6	-2.3	79.6	-2.6	71.8	-2.7	57.1	-3.0
	8	58.5	74.8	107.7	-1.5	93.2	-1.7	80.1	-1.9	72.3	-2.0	57.5	-2.2
	9	52.0	60.1	159.3	1.8	137.8	2.0	118.5	2.3	106.9	2.4	85.1	2.6
	10	45.5	53.9	181.2	6.7	156.7	6.4	134.8	7.1	121.8	7.5	96.8	8.3
	11	39.0	48.7	199.7	12.0	172.7	13.7	148.5	16.1	134.0	18.0	106.6	17.6
	12	32.5	43.7	218.2	23.3	188.7	26.4	162.3	29.2	146.4	30.9	116.5	34.1
7													
	1	26.0	38.8	238.5	43.1	204.6	49.0	176.0	54.2	158.7	57.3	126.3	63.2
	2	32.0	38.8	236.5	43.0	204.5	48.8	176.0	54.0	158.7	57.2	126.3	63.1
	3	38.0	41.2	227.7	32.3	196.9	36.7	169.4	40.6	152.8	43.0	121.6	47.4
	4	44.0	44.3	216.0	22.0	186.9	25.0	160.7	27.8	145.0	29.3	115.4	32.3
	5	50.0	53.3	183.6	5.8	158.7	6.6	136.5	7.3	123.1	7.8	98.0	8.6
	6	56.0	63.8	147.0	-0.3	127.2	-0.3	109.4	-0.3	98.7	-0.3	78.5	-0.4
	7	62.0	74.7	108.2	-2.0	93.6	-2.3	80.5	-2.6	72.8	-2.8	57.8	-2.9
	8	57.5	68.6	129.6	-1.4	112.1	-1.6	96.4	-1.7	87.0	-1.8	69.2	-2.0
	9	53.0	58.0	166.8	2.0	144.3	2.3	124.1	2.6	112.0	2.7	89.1	3.0
	10	48.5	53.7	181.9	5.0	157.3	5.6	136.3	6.3	122.1	6.6	97.1	7.3
	11	44.0	50.2	194.3	8.9	168.1	10.1	144.6	11.2	130.4	11.8	103.8	13.1
	12	39.5	46.8	206.9	14.7	179.0	16.7	153.9	18.4	138.8	19.6	110.5	21.5
8													
	1	35.0	43.4	219.5	23.3	189.9	26.4	163.3	29.2	147.3	30.9	117.2	34.1
	2	40.8	44.1	216.7	21.1	187.5	23.9	161.3	26.5	145.4	28.0	115.7	30.9
	3	46.7	45.8	206.7	14.5	178.8	16.6	153.8	18.4	138.7	19.4	110.4	21.4
	4	52.5	50.2	194.3	9.0	168.1	10.2	144.6	11.3	130.4	11.9	103.8	13.1
	5	58.3	60.6	157.9	0.0	136.5	0.0	117.4	0.0	105.9	0.0	84.3	0.0
	6	64.2	72.5	115.7	-2.7	100.1	-3.1	86.1	-3.4	77.8	-3.6	61.8	-3.9
	7	70.0	85.2	107.0	-3.5	92.6	-4.1	79.6	-4.5	71.8	-4.8	57.1	-5.3
	8	64.2	77.2	107.0	-3.1	92.6	-3.5	79.6	-3.9	71.8	-4.1	57.1	-4.6
	9	58.3	63.8	146.4	-1.1	126.6	-1.2	108.9	-1.3	98.2	-1.4	78.2	-1.6
	10	52.5	58.3	165.8	1.0	143.4	1.1	123.4	1.2	111.2	1.3	88.5	1.4
	11	46.7	53.7	181.9	4.1	157.3	4.6	135.3	5.1	122.0	5.4	97.1	6.0
	12	40.8	49.3	197.5	9.3	170.9	10.8	147.0	11.7	132.5	12.4	105.5	13.7
9													
	1	35.0	44.9	213.8	18.2	185.0	20.6	159.1	22.9	143.5	24.2	114.2	26.7
	2	39.8	44.8	214.3	18.5	185.4	21.0	159.4	23.2	143.8	24.6	114.4	27.1
	3	44.7	46.7	207.3	14.2	179.3	16.1	154.2	17.8	139.1	18.8	110.7	20.8
	4	49.5	49.2	197.9	9.6	171.2	10.9	147.3	12.1	132.8	12.8	105.7	14.1
	5	54.3	54.7	178.3	3.3	154.3	3.7	132.7	4.1	119.7	4.4	95.2	4.8
	6	59.2	61.1	156.1	-0.2	135.0	-0.3	116.1	-0.3	104.7	-0.3	83.3	-0.4
	7	64.0	67.8	132.3	-1.9	114.4	-2.2	98.4	-2.4	88.8	-2.5	70.7	-2.8
	8	57.0	63.4	147.8	-1.0	127.9	-1.2	110.0	-1.3	99.2	-1.4	78.9	-1.5
	9	50.0	55.5	175.5	2.7	151.8	3.1	130.6	3.4	117.8	3.6	93.7	4.0
	10	43.0	50.4	193.6	7.9	167.5	9.0	144.0	10.0	129.9	10.5	103.4	11.8
	11	36.0	45.4	211.9	17.1	183.3	19.4	157.7	21.5	142.2	22.7	113.2	26.1
	12	29.0	40.4	230.6	33.8	199.5	38.4	171.6	42.5	154.8	45.0	123.2	49.6
10													
	1	22.0	35.2	249.7	64.6	216.0	73.3	186.8	81.1	167.8	85.8	133.3	94.7
	2	27.5	35.0	250.6	66.4	218.8	75.4	186.4	83.5	168.1	88.3	133.8	97.4
	3	33.0	37.1	242.9	51.8	210.1	58.7	180.7	65.0	163.0	68.8	129.7	75.9
	4	38.5	39.9	232.5	35.7	201.1	41.6	173.0	46.1	156.0	48.8	124.2	53.8
	5	44.0	48.8	199.4	9.4	172.5	10.7	148.4	11.9	133.8	12.6	106.5	13.8
	6	49.5	59.2	162.5	-1.2	140.8	-1.4	120.9	-1.5	109.1	-1.6	86.8	-1.7
	7	55.0	70.3	123.4	-4.2	106.8	-4.6	91.8	-5.3	82.8	-5.6	65.9	-6.2
	8	48.0	62.7	150.4	-2.7	130.1	-3.1	111.9	-3.4	100.9	-3.6	80.3	-4.0
	9	41.0	50.0	195.2	8.0	168.8	9.1	145.2	10.0	131.0	10.6	104.2	11.7
	10	34.0	43.9	217.8	20.9	188.4	23.8	162.0	26.3	146.1	27.8	116.3	30.7
	11	27.0	38.5	237.5	42.6	205.6	48.3	176.7	53.5	159.4	56.6	126.8	62.4
	12	20.0	33.3	257.0	81.6	222.3	92.6	191.2	102.7	172.4	108.7	137.2	119.9
11													
	1	13.0	28.0	263.6	142.0	228.0	161.2	196.1	178.3	176.9	188.7	140.8	206.2
	2	16.3	28.8	265.0	158.7	229.2	180.1	197.2	199.3	177.8	210.8	141.5	232.7
	3	19.7	27.5	264.3	149.3	228.6	169.4	196.6	187.5	177.3	198.4	141.1	218.8
	4	23.0	28.8	262.6	132.4	227.3	150.3	195.5	166.3	176.3	176.0	140.3	194.1
	5	26.3	37.2	242.5	63.1	209.8	80.3	180.4	88.7	162.7	70.6	129.5	77.9
	6	29.7	47.3	206.0	12.3	177.3	14.0	152.5	15.4	137.5	16.3	109.5	18.0
	7	33.0	58.2	166.1	-0.8	143.7	-0.9	123.6	-1.0	111.5	-1.1	88.7	-1.2
	8	27.8	49.3	197.5	8.1	170.8	9.2	146.9	10.2	132.5	10.8	105.6	11.9
	9	22.7	35.5	248.6	70.0	215.1	79.4	185.0	87.9	166.8	93.0	132.8	102.6
	10	17.5	30.0	261.4	123.5	226.1	140.1	194.5	155.1	175.4	164.1	139.6	181.0
	11	12.3	25.6	268.4	183.1	230.5	207.6	198.2	230.0	178.8	243.4	142.3	268.4
	12	7.2	21.4	271.2	254.7	234.5	289.1	201.7	319.9	181.9	338.5	144.8	373.4
12													
	1	2.0	17.2	275.8	343.5*	238.5	389.9	205.2	431.4	185.1	456.5	147.3	503.6
	2	5.7	17.0	276.1	349.4*	238.8	396.5	205.4	438.8	185.2	464.3	147.4	512.2
	3	11.3	18.8	274.2	311.5*	237.2	353.6	204.0	391.3	184.0	414.0	146.4	456.7
	4	16.0	20.9	271.7	284.6	235.0	300.4	202.2	332.4	182.3	351.7	145.1	386.0
	5	20.7	30.4	280.9	115.5	225.7	131.1	194.1	145.0	175.1	153.5	139.3	169.3
	6	25.3	41.8	225.6	29.7	195.0	33.7	167.7	37.3	151.3	39.5	120.4	43.6
	7	30.0	53.8	181.2	2.6	156.8	3.0	134.8	3.3	121.6	3.5	96.8	3.8
	8	27.7	45.5	208.0	13.5	179.9	15.3	154.7	17.0	139.6	18.0	111.1	19.8
	9	25.3	34.4	252.9	76.0	218.8	86.5	188.2	96.0	169.7	103.5	135.1	114.3
	10	23.0	30.8	260.5	113.8	225.3	128.9	193.8	142.8	174.8	150.9	139.1	166.6
	11	20.7	28.4	263.2	142.2	227.7	161.4	195.8	178.6	176.5	189.0	140.5	208.5
	12	18.3	28.3	265.5	170.6	229.7	193.8	197.8	214.3	178.2	225.7	141.8	250.1
13													

1	16.0	24.4	267.8	200.8	231.6	227.9	199.2	252.2	179.7	266.4	143 0	294.3
2	18.7	24.4	267.7	199.8	231.6	226.7	199.2	250.9	179.6	265.5	142.9	292.9
3	21.3	25.6	266.5	183.6	230.5	208.3	198.3	230.5	178.8	243.9	142.3	269 1
4	24.0	26.8	265.0	163.6	229.2	186.7	197.1	205.5	177.8	217.5	141.5	239.9
5	26.7	35.8	247.5	87.7	214.1	78.8	184.1	85.0	166.0	90.0	132.1	99.2
6	29.3	46.8	207.6	17.9	179.6	20.3	154.4	22.5	139.3	23.8	110 9	26.2
7	32.0	56.1	166.4	3.1	144.0	3.6	123.6	3.9	111.7	4.1	88.9	4.5
8	29.0	49.4	197.4	11.8	170.8	13.5	146.9	14.9	132.5	15.8	105.4	17.4
9	26.0	35.8	247.5	70.8	214.1	80.4	184.2	88.9	166.1	94.1	132.2	103.8
10	23.0	31.5	259.7	109.6	224.6	124.3	193.2	137.8	174.2	145.6	138.7	160.6
11	20.0	28.6	263.0	144.5	227.5	164.0	195.7	181.5	176.6	192.1	140.4	211.9
12	17.0	25.9	266.0	160.8	230.1	208.2	197.9	227.1	178.6	240.3	142.0	265.0
14												
1	14.0	23.4	268.8	220.7	232.5	250.6	200.0	277.2	180.4	293.3	143.5	323.6
2	17.7	23.6	268.6	217.5	232.3	246.9	199.8	273.2	180.2	289.1	143.4	318.9
3	21.3	25.1	266.6	193.5	230.9	219.6	198.6	243.0	179.1	257.2	142.6	283.7
4	25.0	27.0	264.8	165.2	229.0	187.6	197.0	207.6	177.6	219.6	141.4	242.2
5	28.7	35.3	245.7	66.8	212.6	78.1	182.8	86.4	164.8	91.4	131.2	100.9
6	32.3	47.3	204.9	20.9	177.2	23.7	152.4	28.3	137.5	27.8	109.4	30.7
7	36.0	59.1	162.9	7.1	140.9	8.1	121.2	9.0	109.3	9.5	87.0	10.5
8	32.2	50.7	192.5	14.4	166.5	16.3	143.3	18.1	129.2	19.1	102.8	21.1
9	28.3	37.4	241.8	63.0	209.1	71.5	179.9	79.1	162.2	83.7	129.1	92.4
10	24.5	32.8	258.3	100.3	223.4	113.9	192.2	126.0	173.3	133.4	137.9	147.1
11	20.7	29.3	282.1	138.6	228.8	167.3	195.0	174.0	175.9	184.2	140.0	203.1
12	16.8	26.2	285.7	180.8	229.8	206.2	197.7	227.0	178.3	240.2	141.9	265.0
15												
1	13.0	23.1	269.2	229.5	232.8	260.6	200.2	288.3	180.6	305 1	143.7	336.5
2	19.8	24.3	267.8	209.3	231.6	237.6	199.2	262.9	179.7	278.2	143.0	306 9
3	26.7	27.6	264.1	160.9	221.4	182.6	198.5	202.1	177.2	213.8	141.0	235 8
4	33.5	31.6	259.6	112.7	22+.6	128.0	193.2	141.6	174.2	149.8	138.6	165.3
5	40.3	43.5	219.0	30.3	189.4	34.4	162.9	38.1	146.9	40.3	116.9	44.5
6	47.2	57.4	169.0	6.1	146.2	6.9	126.7	7.6	113.4	8.1	90.2	8.9
7	54.0	72.1	117.3	1.4	101.4	1.8	87.2	1.8	78.7	1.9	62.6	2.1
8	50.0	64.1	145.2	2.7	125.8	3.0	108.0	3.4	97.4	3.6	77.5	3.9
9	46.0	50.5	193.2	13.2	167.1	15.0	143.7	16.8	129.6	17.5	103.2	19 3
10	42.0	46.0	209.7	21 1	181.4	24.0	158.1	26.5	140.7	28 1	112.0	30.9
11	38.0	42.7	222.0	30.2	192.1	34.2	165.2	37.9	149.0	40.1	118 6	44.2
12	34.0	39.6	233.4	42.3	201.9	48.0	173.7	53.1	166.6	56.2	124 7	62.0







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     174.      +                                                 X    9X31                                           +
     175.      +                                                 X  9X31                                             +
     176.      +                                                 X   9X31                                            +
     177.      +                                                  X   9753 1                                         +
     178       +                                                  XO   97531                                         +
     179.      +                                                   X   975 31                                        +
     180.      ++++++++++++++++++++++++++++++++++++++++++++++++++++XX++9+753+1++++++++++++++++++++++++++++++++++++++++++++
               +         +         +         +         +         +         +         +         +         +         +

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     ******  TIME PERIOD

CURVE 1   -   EXPECTED TENSILE STRENGTH IN PSI
CURVE 2   -   EXPECTED THERMAL STRESS IN PSI
CURVE 3   -   TENSILE STRENGTH WITH 75 PERCENT RELIABILITY
CURVE 4   -   THERMAL STRESS WITH 75 PERCENT RELIABILITY
CURVE 5   -   TENSILE STRENGTH WITH 90 PERCENT RELIABILITY
CURVE 6   -   THERMAL STRESS WITH 90 PERCENT RELIABILITY
CURVE 7   -   TENSILE STRENGTH WITH 95 PERCENT RELIABILITY
CURVE 8   -   THERMAL STRESS WITH 95 PERCENT RELIABILITY
CURVE 9   -   TENSILE STRENGTH WITH 99 PERCENT RELIABILITY
CURVE10   -   THERMAL STRESS WITH 99 PERCENT RELIABILITY
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