Mobil Oil Corporation

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NEW YORK, NEW YORK 10017

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August 8, 1974

Mr. A.W. Lane, Materials Engineer
Vermont, Department of Highways
Montpelier, Vermont 05602

Dear Mr. Lane:

I am enclosing two copies of Mobil's research studies relative to the work on I-91. This study was partially designed to investigate the potential use of newer analytical laboratory testing devices.

We want to acknowledge the considerable amount of help and data you, Russ H. Snow, Frank E. Aldrich, and others in Vermont have given us to make this study possible. After you have had a chance to review this, we would be pleased to discuss the results and other matters at your convenience.

No further work is planned here at this time.

Sincerely,

Ralph G. Adams

Ralph G. Adams

RGA/af
Attachments

Mobil

ASPHALT TRANSVERSE CRACKING STUDY -
VERMONT I 91 TEST ROAD

Mobil Research and Development Corp.
Research Department, Paulsboro, N.J.
July 29, 1974
C. A. Pagen
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MOBIL RESEARCH AND DEVELOPMENT CORPORATION
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PRODUCTS RESEARCH AND TECHNICAL SERVICE DIVISION
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Aviation Lubes and Special Products Group

Project 328-00

ASPHALT TRANSVERSE CRACKING STUDY - VERMONT I 91 TEST ROAD

July 29, 1974

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ASPHALT TRANSVERSE CRACKING STUDY -
VERMONT I 91 TEST ROAD

INTRODUCTION

Transverse cracking of asphalt concrete roads is a serious and costly problem in the Northern United States, Canada, and in comparable cold climates throughout the world. As a result, symposia (1), (2)* have been held to consider possible solutions to this problem; and technical papers related to this problem have been presented at recent meetings. (3), (4) Several states, in which this is a critical problem, are carrying out the recommendations of these symposia in their road construction and maintenance programs. In 1970, the state of Vermont initiated such a program involving most of their construction and maintenance work during that year. Their studies included an asphaltic concrete overlay of Vermont I 91, a 12 mile, four lane road, just north of the Massachusetts border. In this project, the use of a softer than normal asphalt (120/150 penetration versus 85/100 penetration) to reduce transverse cracking was evaluated. For this study, about 80,000 tons of hot asphaltic concrete mix were applied to the road as a two-inch overlay in two one-inch thick lifts. To rehabilitate the badly cracked road, approximately 35,000 bbl. of Mobil's E. Providence asphalt were used. Because an economical solution to the transverse cracking problem in the Northeastern United States would enhance future Mobil asphalt sales, NAD Marketing consequently requested MRDC to follow this Vermont project closely to assist wherever possible in this critical evaluation of Mobil asphalts.

A major goal of this exploratory study was to develop technology to evaluate and relate changes in asphalts and asphaltic concrete mixes with road durability for use in future research programs.

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*Underlined numbers in parentheses refer to the list of references.

SUMMARY

1. The original Vermont I 91 road was designed for a 20 year (1958-1978) service life. The highway was built on a good foundation soil and with an overall thickness design adequate for the predicted traffic loads. The original road has a granular base and subbase 30 inches thick and a three-inch asphaltic concrete surface. Its premature failure in 1970 appeared to be due to thermal effects resulting in transverse cracking which was not considered in the thickness design procedures.

2. The selection of a two-inch asphalt concrete overlay to remedy the cracked condition of the Vermont I 91 on this 12 mile strip was based on AASHTO 1972 standards. It did not meet the Asphalt Institute recommendations for asphalt concrete overlays. In the Asphalt Institute publication MS-17, a four-inch overlay would have been designated as the minimum thickness for a road carrying the traffic of Vermont I 91. The selection of a two-inch overlay for this road resulted in a severe test for the overlay materials.

3. Samples of: a) asphalt from the refinery, b) asphalt-aggregate hot-mixes, aggregates and asphalts from the contractor's mixing plant, and c) road core samples obtained from the road after construction, at 4, 8 and 12 months by the Vermont Highway Department were analyzed and tested by MRDC and Vermont Highway Department Laboratory. ASTM, Asphalt Institute or Mobil standard methods of testing were used for all experiments in the research program.

4. The refinery asphalt met all the Vermont Highway Department penetration grading specifications in use during 1970. The average penetration was about 140 at 77°F meeting the 120-150 penetration specified instead of the 85-100 penetration grade usually used for overlay and construction purposes in Vermont.

5. MRDC tests on the road cores and the hot-mix samples obtained from the Vermont laboratory and construction contractor's plant showed that the asphalt and aggregate were of high quality and properly proportioned to meet ASTM and AI mix design criteria. The mix design of the asphalt overlay met all the requirements of

the Marshall ASTM method for high quality asphalt concrete. Tests on the laboratory compacted hot-mix samples produced Marshall test specimens with high Marshall stability, adequate flow and low air voids meeting the Asphalt Institute criteria.

6. Road core samples were obtained by Vermont for ASTM Marshall and rheological tests by MRDC during the first year of road service. Major conclusions of laboratory tests on road core samples are:

- Asphalt content within specifications
- Asphalt physical and chemical properties in agreement with tests on residue from ASTM thin film oven test
- Aggregate content within range
- Aggregate properties in agreement with specifications
- All the road cores had low Marshall stability and high flow values when judged against the general Asphalt Institute mix designs. However, there are no required Marshall test standards for road core samples. This test on road cores did show, however, the additional compaction due to traffic.

Summarizing the above results, it would seem that the proper materials and asphalt paving hot-mixes had been used by Vermont. But a stronger pavement overlay may might have been constructed by proper compaction of a single two-inch lift rather than two one-inch lifts.

7. Physical and chemical tests were made on the original asphalt, asphalt recovered from the hot-mix plant blends and asphalt recovered from the road core samples taken over a year's road service. The most rapid rate of change in physical and chemical properties took place during the short time of asphalt batch plant mixing, transportation and construction operations. During the year's road service no significant changes had taken place in the asphalt binder.

8. Chemical composition tests on the original asphalt and the asphalt recovered from hot-mixes generally followed any major changes in the asphalt physical properties.

9. Gradient elution chromatography, nuclear magnetic resonance and infrared absorption were the most informative chemical tests in evaluating fundamental changes in the properties of the asphalt. These tests confirm that the majority of the asphalt aging takes place due to oxidation during the plant mixing and construction operations. These results may be of value in future asphalt research programs within the Joint Asphalt Research Project CR-42.

Chemical changes in the binder take place very slowly during road service. Of interest were the thin film oven test results which checked the chemical and physical changes occurring during the hot mixing and road construction operations. Road cores and hot-mixes with asphalt contents above or equal to the design optimum generally had significantly less age hardening. Mixes with low asphalt contents and thinner asphalt films - generally exhibited considerably more age hardening and subsequent increases in the asphalt mixture stiffness. The road cores with lower air voids also seemed to have less age hardening and better durability.

10. After eight months of service, from September 1970 to April 1971, the 12-mile section of overlay developed a transverse cracking pattern over certain portions of the road. At that point the cracking did not impair the riding characteristics or overall appearance of the road. The performance appeared to be very good and completely acceptable to Vermont. During the next three years of service (1971-1974), there was no significant change in the degree or location of transverse cracking in this overlay.

11. An area analysis of the transverse cracking pattern
? ? showed that the cracking which occurred was directly related to road area rather than to asphalt, aggregate, hot-mix or road core void content, voids in the mineral aggregate, density, composition, etc. While the composition of the overlay was relatively uniform, in contrast the transverse cracking frequently was not uniform. Many of the 0.1 mile road test sections had no cracks, whereas some sections had cracks at 22 foot intervals (240 cracks per mile).

12. Comparison of road areas indicated the following significant differences in cracking frequency:

- Passing lane (high cracking frequency) vs. travel lane (low cracking frequency)
- Northern section (high cracking frequency) vs. southern section (low cracking frequency)

The good performance of the travel lane is due to the beneficial compaction of traffic. The better performance of the southern section may be due to many factors such as: construction methods, climatic, drainage, and soil conditions or differences in the old road condition before the overlay.

13. The predictive formulas of Anderson and Shields, McLeod, or Hajak and Haas were applied to the data on the asphalt extracted from the overlay road cores. The formulas predict that no transverse cracking should be experienced in the overlay in that service period. Published freezing index contour maps of the U.S. were used to determine typical temperature data for the calculations. Since cracking occurred, it is concluded that the transverse cracking in the two-inch overlay is mainly reflective cracking over the larger cracks in the old road. This conclusion was confirmed by cutting cores directly over the overlay transverse cracks through the old road laying underneath and observing the transverse cracks in the original pavement.

14. It is recommended that the same mix design be used in future overlays, but that thicker compaction lifts and greater thickness be used in future overlays. It is also recommended that these thicker overlays be placed and subjected to increased amounts of compaction until the cores show higher initial Marshall stability values. The use of nuclear devices to evaluate asphalt contents and pavement compaction should be initiated. cost?

15. It is believed that the use of Full-Depth construction for new roads in place of the relatively thin asphalt pavement surfaces with thick granular bases now used in Vermont would reduce transverse cracking in this northern area. Recent technical ??

publications have indicated that increasing the thickness of the asphaltic concrete layer will significantly reduce induced thermal stress and subsequent cracking frequency in the asphalt layers when all other variables are the same.[3],[4] Full-Depth field test sections are therefore proposed to evaluate these technical advances.

16. The uniform properties of the asphalts, hot-mixes and road cores from representative locations in the I 91 highway overlay contrast sharply with the random transverse cracking interval (TCI) data. This excludes any correlations between the asphalt and the asphalt mixture stiffness moduli, other properties studied and transverse cracking interval data unless other overlays throughout Vermont with different asphalts are included in the research program. This is beyond the limited scope of the present exploratory study.

17. Asphalt pavement cores were tested at periodic intervals during the initial one-year road service period at The Ohio State University using materials science test procedures under the direction of MRDC. These exploratory tests evaluated the strength properties of the road cores in constant-load creep compression and sinusoidal dynamic tests over a wide range of experimental temperatures using special MTS universal testing equipment. Marshall and unconfined compression tests were also performed on the road cores at MRDC. The test results indicated:

- Marshall stability increased with age due to traffic compaction. Marshall flow, voids in the mineral aggregate, and air voids decreased with service life.
- The fundamental mechanical behavior of road cores can be defined by the linear viscoelastic and time-temperature superposition concepts. These concepts can be applied to the creep and dynamic data to investigate the influence of experimental loading time and temperature on stiffness, rheological strength properties and road performance.
- Repeatability of experimental data obtained from core test specimens was excellent. Mechanical properties of representative road cores were uniform.

- The creep moduli, dynamic stiffness, unconfined compressive strength and moduli of elasticity increased with road service life.
- This new technology can be used directly in future pavement design, asphalt mixture aging, asphalt additive and transverse cracking studies. Foamed asphalt stabilized mixes and Mobilplast binder mixes can be evaluated using this technology.

18. For the conditions and materials studied, the use of fundamental stiffness properties of the asphalt and dynamic stiffness of the asphaltic concrete mixtures show considerable promise for future field and laboratory asphalt pavement research as well as pavement design studies. The exploratory study has clearly indicated the importance of evaluating the fundamental properties of not only aggregates and asphalts, but also the combination of these components - the paving mix used in actual road service conditions. Future basic studies should utilize this type of research approach and evaluate the dynamic stiffness of the paving materials which can be related to transverse cracking interval data, road durability, layer thickness equivalency values, road design criteria and performance data.

19. Two approaches to determine the stiffness of bituminous materials were used in this investigation:

- Direct testing in creep and dynamic experiments over a wide range of temperatures and loading times
- Indirect estimation using the van der Poel and McLeod stiffness modulus nomographs

The indirect methods for roughly estimating the stiffness moduli of asphaltic mixes are quick and easy to use; however, there are many limitations to their use and application. The most serious objection is that large errors in estimating stiffness are possible using the indirect methods, as compared to the use of direct testing methods.

The direct testing phase of the study has established that the mechanical properties of road cores can be defined by the linear viscoelastic and time-temperature superposition concepts. As a result, the road designer is now able to establish the stiffness moduli of asphalt paving materials for a wide range of loading times and temperatures desired by use of the master stiffness moduli - time functions and the temperature shift factor - temperature plots. The assumption that time and temperature are interchangeable has been experimentally verified in the rheological tests. In other words, by conducting creep tests on road cores at only a few temperatures, the designer is able to determine the stiffness moduli over large loading time and temperature ranges. Although beyond the scope of this limited exploratory program, the final step would be to directly correlate the stiffness moduli of road cores from test roads with the transverse cracking frequency of the pavements.

The new technology outlined in this report will allow the stiffness of road cores obtained from selected test roads to be directly evaluated and correlated with transverse cracking frequency. The study should evaluate the test roads for about a four year testing period. Establishing the fundamental strength properties of bituminous mixes by direct testing will supply the road design engineer with a valuable engineering research tool to investigate the low temperature shrinkage cracking of bituminous pavements. The subgrade type and properties can also greatly influence low temperature shrinkage cracking and should be incorporated into the research program.

The resilient modulus test device recently installed at Paulsboro is a simple, reliable, cheap and nondestructive test apparatus which can be utilized to directly evaluate the stiffness moduli of road cores over extensive loading time and temperature ranges.

20. Although not directly related to this exploratory study, the development of economical techniques to retard or eliminate the reflection of shrinkage cracks through asphalt overlays warrants considerable attention because of the thousands of miles of cracked roads in North America which are overlaid or soon will require such upgrading. Today our existing technology and

materials are unable to economically prevent the rapid reflection of cracks through overlays, and this has limited many transverse cracking research studies.

DETAILED REPORT

A. Design and Performance of Original I 91 Roadway

The original road was constructed in 1958-1960 with a design life of 20 years. A summary of the thickness design details which were obtained from the Vermont State Highway Department is shown in Table 1. A standard 85/100 penetration grade asphalt was used in its construction. The road has 30 inches of granular base and subbase with a three inch asphaltic concrete surfacing. The location of the Vermont I 91 project is shown in Figure 1. A conversion chart to change road stations to distance in miles from Massachusetts border is shown in Figure 2 for reference. The thickness of the various structural layers were as follows:

Original I 91 Structural Courses	Thickness (inches)	
Bituminous Concrete	3	
Crushed Stone Base	3	
Crushed Rock Subbase	21	
Sand Cushion	6	
Total	33	inches

Using the Asphalt Institute thickness equivalency conversion factors,[5],[6] it was shown that a 33-inch gravel based road is structurally equal to about an 11-inch Full-Depth asphalt concrete road constructed directly on the subgrade soil. The Asphalt Institute thickness design formulas also indicate that a six-inch Full-Depth asphalt concrete road layed directly on the soil would have been structurally adequate for the 1958-1978 traffic estimate using the 1958 data and calculations. It appears that the Vermont design calculations have been more conservative than the present Asphalt Institute conversion factors and have included other significant environmental design factors such as climate, fatigue, shrinkage fracture, permanent deformation, etc. The location of the I 91 section involved has an approximate freezing index of 1000 degree days[7],[8] and a frost penetration depth of 52 inches. Since the original road is 33 inches thick, the extra depth may have been designed to approach the frost penetration depth with the granular material to eliminate the critical temperatures and free water necessary for frost heave. It should be noted that, although the total thickness appeared to be adequate, the asphalt concrete pavement layer of only three inches is below the minimum of four inches for a medium traffic road and five inches for a heavy traffic road as recommended by the Asphalt Institute publications in 1970. The Vermont Highway Department does not have the road performance record or inspection details during the 1958-1970 period and consequently no area analysis of the transverse cracking pattern could be made on the old road. This reference material for the transverse cracking pattern would be of value in quantitatively measuring the effectiveness of the overlay in repressing transverse cracking. Identifying reflection cracking areas in the overlay would also be possible with these data. It was consequently assumed that the entire 12-mile section had sufficient transverse cracking in critical sections to require an overlay.

B. Thickness Design of 1970 Overlay

The design of a strengthening asphalt concrete overlay calls for the calculation of a thickness that will strengthen the pavement structures sufficiently to accommodate the estimated traffic for the design period. An asphalt concrete overlay bonds to the underlying road pavement and becomes an integral part of the total pavement structure. The added road thickness provides sufficient stress distribution properties to prevent critical tensile and compressive stresses from being reached[(9),(10)]. Inspection of Table 2 indicates that a two-inch asphalt concrete overlay was selected because it upgraded the total road strength to a value approximately equal to the original road. Using the Asphalt Institute conversion factors,[(5)] the structural layers of the old road plus the two-inch asphalt overlay was equivalent to 11 inches of Full-Depth asphalt (the value calculated for the 1958 road design). The Asphalt Institute calculations call for a total of 7.5 inches of Full-Depth asphalt for the increased traffic as compared to the original requirement of six inches. The structural design of 11 inches should have provided sufficient structural strength to reduce cracking problems in the road.

Although the total road effective thickness was adequate, it should be noted that the asphalt surface layer consists of three inches total - one inch equivalent from the old road surfacing and two inches from the new overlay. The Asphalt Institute recommends a minimum of four inches for asphalt pavement layers over untreated granular bases [5], [6].

Recent technical publications have shown that by increasing the thickness of the asphaltic concrete paving layer the thermally induced stresses and subsequent shrinkage fracture in roads is greatly reduced [11], [12]. Research programs have used theoretical viscoelastic calculations, direct laboratory testing of asphaltic mixes and full-scale test roads to document these conclusions. The Asphalt Institute has also recently published technical information [5], [6] which indicates that roads with granular bases are susceptible to water and thermal effects; and consequently, the AI no longer favors this type of construction.

C. Design of 1970 Overlay Paving Mixture

Design of the asphalt concrete overlay was carried out by Vermont using the Marshall method of mix design (ASTM Designation D 1559). The standard ASTM, AASHTO or Mobil methods of test were used throughout the experimental program. The selection and proportion of the aggregates met the quality standards of the Asphalt Institute as shown by the lab tests performed on the hot-mix samples and lab compacted Marshall specimens reported in Table 3 and Figure 4. The Marshall stability tests performed on the laboratory compacted hot-mix samples shows that the 93 percent aggregate and 7 percent asphalt mixes had following average properties: 1905 lb. stability and 0.13 inches flow. These data are plotted in Figure 3 with Marshall test data obtained from tests on the road cores.

An analysis was conducted by MRDC of the air voids, voids in the mineral aggregate (VMA) and extracted asphalt content of the undisturbed road core samples obtained by Vermont and also of laboratory compacted Marshall samples prepared from the hot-mix samples obtained from the construction contractor's plant by Vermont. The results of the analysis are presented in Tables 3 and 4 and Figure 5 and indicate the probable service performance of the road and degree of mix compaction during construction. The road performance and mix compaction will be reviewed in later sections.

D. Construction of the 1970 Overlay

The overlay was constructed during the period from July 6 to August 28, 1970 during a very favorable weather period. Prior to the overlay application, the major transverse cracks in the old road pavement were filled with a polymer-asphalt material, coated with a sand-asphalt emulsion surface treatment to seal the fine cracks and then leveled with a thin hot mix leveling application where necessary. The overlay was applied in two one-inch thick lifts as indicated by the construction schedule shown in Table 5.

E. Composition and Properties of Road Cores

As part of their final acceptance procedure, Vermont removed core samples at 22 locations in the northbound lanes and 26 locations in the southbound lanes immediately after construction. Cores of both the top one-inch course and the base one-inch course of the asphalt overlay were evaluated separately and tested to determine specific gravity, asphalt content, and aggregate content and gradation. Vermont reported that the values were within the variations permitted by their highways specifications. This preliminary analysis by the Vermont highway engineers was made for construction evaluation and to distinguish between different sections of the road from the standpoint of uniformity of overlay compaction. It was noticed and recorded that the top courses were generally not as well compacted as the base courses indicating a lower degree of consolidation in the top courses.

At the same time a similar set of core samples at 15 locations in the northbound travel lane and 15 locations in the southbound travel lane was also taken by Vermont for testing by MRDC. An initial set of cores was taken in September 1970. A second set of samples was taken in December 1970, a third set in April 1971 and a fourth set in August 1971. The road cores were sealed in polyethylene bags filled with an inert gas (carbon dioxide) and shipped to Paulsboro via the Albany Marketing District. For each period of sampling eight cores were selected as being representative of the entire road and separate Marshall analyses were made on these field compacted cores. The results of the Marshall stability and flow, material specific gravity determinations, air voids, voids in the mineral aggregate, extracted asphalt content and sieve analysis are shown in Tables 3 and 4 and Figures 3, 4 and 5. An interpretation of the results using Asphalt Institute criteria [5] indicates that: (1) the pavement overlay was not compacted to a density of at least 97 percent of the laboratory compacted Marshall specimens prepared in accordance with ASTM Designation D 1559 from asphalt hot-mix samples obtained at the contractor's plant; (2) the Marshall stability values and flow values on the original cores did not approach the laboratory asphaltic concrete mix design criteria recommended by the Asphalt Institute; all cores had high flow and low stability values at 140°F; (3) the VMA and air void values were high indicating that sufficient initial construction compaction had not taken place at the time of sampling for the original cores to obtain the design stability; it was also noted that the Marshall stability load increased steadily with time probably due to the compaction under traffic.

At the end of a 12-month road core sampling period the overlay was practically within the specified limits of the Asphalt Institute regarding the Marshall laboratory mix design criteria. It should be noted that there are no required standards for Marshall stability and flow values for field compacted road cores. The Marshall tests were conducted on the road cores to evaluate the changes in the mechanical properties of the pavement with time. The state of compaction cannot be uniquely characterized by a single parameter; the simplest, that of the density of the road core, is affected by the density of the constituent materials. More useful parameters are those which take into account the strength of the materials in pavement service conditions or in representing the state of compaction, the effect of density of aggregates and binder. These are the mixture fundamental strength properties evaluated in this research program or density. Density can be expressed as a percentage of the theoretical maximum density or void content. It was noted that the initial VMA and air void values of the new overlay were high and decreased due to traffic compaction; this would also confirm that sufficient compaction had not taken place at the time the original cores were taken after construction. After 12 months service, the voids in the mineral aggregate and the air voids had decreased to a marginal pass on the Asphalt Institute specifications. The somewhat high Marshall flow values do indicate that the asphalt road overlay is more flexible and plastic than the conventional asphalt mixes usually employed. Figure 3 shows the increase in Marshall stability values of the road cores with time after exposure to traffic and weather on the road. Figure 5 indicates the rate of decrease in air voids in the overlay during the 12-month period.

F. Physical Properties of Original and Recovered Asphalts

In Table 6 the physical test data are given on the original 120/150 penetration asphalt made from Panuco crude at the E. Providence refinery for the I 91 road study. Comparison of the results with the Vermont specifications for 1971 show that this asphalt meets the specifications being used in the New York and New England area at the present time. Under these viscosity specifications, the asphalt would be classed as an AC-10 paving asphalt with good low-temperature ductility after the thin film oven test. Other supplementary quality tests such as the penetration index, vispen, viscosity-temperature susceptibility, and Kinnaird's characterization factor indicate that the original asphalt was of excellent quality.

Table 6 and Figures 6 and 7 show the important changes in the key physical properties of the asphalts during hot mixing, road construction operations and roadway service. The penetration values in Figure 6 show good agreement between the residue from the thin film oven test (D 1754) and the asphalts from the road cores recovered using the modified Abson method (ASTM 1856).

Since the degree of hardening during the standard thin film oven test has been shown to be about equal to the hardening in asphalt that occurs during carefully controlled hot-mix plants, the agreement shown in Figure 6 clearly indicates that the mixing, handling, spreading and compaction of the hot mix was done with adequate control. The penetration values of the asphalts recovered from the road cores showed no major change between the 1970 cores and the 1971 cores. After the initial rapid change from an average penetration of about 140 in the original asphalt to 82 after the hot mix operation and to 64 in the newly constructed road, the exposure of the asphalt in the road to oxidation, evaporation or other hardening effects appeared to be very greatly reduced. The changes that occur in the asphalt during exposure in the road are of a relatively slow nature so that they must usually be measured in terms of years instead of days or months. Traffic compaction would also reduce the rate of age hardening.

In Table 6 the viscosities of the asphalts recovered from the hot mix samples and from the road cores increased from 835 poises at 140°F in the original asphalt to approximately 1850 poises in the hot mix and 2600 poises in the 1970 road cores and 2800 poises in the 1971 road cores. The viscosity of the residue from the thin film oven test was 3247 poises at 140°F. The viscosity data plotted in Figure 7 did not correlate as well as the penetration data, but they do confirm the trends and the conclusions obtained from the penetration data, that physical changes in the asphalts occur rapidly during the hot mix plant and spreading operations and the rate of these changes are greatly reduced during the service life of the road. The viscosity changes also indicate that the hot mixing, rolling and spreading operations were normal and carefully controlled. The penetration index values of the asphalts recovered from the hot mix, the road cores and the thin film oven test were practically constant indicating that the type of asphalt is not changed during any of these exposure periods. This is also supported by the uniformity of Kinnaird's blowing factor for all the samples tested.

G. Chemical Properties of Original and Recovered Asphalts

1. Gradient Elution Chromatography

The most informative analyses of the chemical nature of asphalts has been obtained by gradient elution chromatography, GEC, using Mobil Method M-1012 (13) developed by W. R. Middleton. This test separates an asphalt into seven components that are chemically different and also retain their identity in material balances during blending and physical separations. The seven GEC hydrocarbon components are listed below with the range of GEC values for typical good paving asphalts and the original asphalt used in the Vermont overlay. The GEC composition range of typical good paving asphalts was established by evaluating many good paving asphalts produced from Mobil's worldwide sources that have been used successfully in pavements throughout the world.

GEC Components	GEC Analysis, % Wt. — Typical GEC Composition Range for a Good Paving Asphalt	GEC Analysis, % Wt. — Original Vermont Asphalt
Saturates	7-12	12
Mono and Dinuclear Aromatics	8-11	11
Polynuclear Aromatics	16-17	17
Soft Resins	24-26	24
Hard Resins	13-15	13
Eluted Asphaltenes	13-16	14
Non-Eluted Asphaltenes	9-14	9
Total	100	100

The good agreement in the M-1012 component analysis of the asphalt used in the Vermont overlay and the GEC criteria for good paving asphalts indicated that the asphalt prepared at E. Providence was of good quality in terms of the original distribution of its fundamental chemical components.

The effects of hot mixing, spreading, compaction and road exposure on the seven GEC component distribution of the asphalt are shown in Table 7 and also in Figures 8 and 9. The results are also in agreement with the changes occurring in the asphalt exposed to the thin film oven test. The eluted and non-eluted asphaltenes increased during construction at the expense of the soft resins and to a lesser extent the other components. It can be seen that the major changes in component distribution took place during the mixing and construction of the road while the asphalt was hot and exposed in thin films on the aggregate particles. Practically no change occurred in the asphalt GEC component distribution from the 1970 and 1971 road cores. This is demonstrated graphically in Figures 9 and 10.

2. Infrared Absorption

During the infrared absorption studies, the compositional changes in the asphalt were monitored by measuring the following:

- Infrared absorbance index (I_A) - ratio of the average absorbance at 12.4 and 13.4 microns due to aromatic components divided by the absorbance at 13.85 microns due to aliphatic components.
- Infrared transmittance index (I_T) - ratio of the transmittance near 3.4 microns due to aliphatic C-H stretching divided by the transmittance near 3.3 microns which corresponds to aromatic C-H stretching vibrations.
- Aromatic Ring Condensation Index (I_R) - ratio of the mean aromatic absorbance near 12.35 and 13.41 microns to the absorbance at 13.85 of the paraffinic methylene chain.
- Carbonyl index ($I_{c=o}$) - the absorbance in the c=o region (5.75 $\rightarrow$ 5.93 microns) divided by infrared cell length.

Basically, I_A, I_T and I_R are measurements of the relative amounts of aromatic and aliphatic hydrocarbon types present in the carbon disulfide soluble portion of the asphalts. If the relative aromatic or aliphatic content of the asphalt changed then this would be reflected in changing values of I_A, I_T and I_R. The carbonyl index ($I_{c=o}$) is a measure of the extent of aging (oxidation) of the asphalt in the road surface.

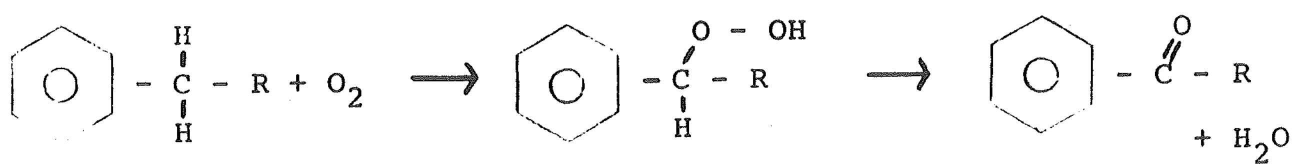
As can be seen from the data in Table 7, the Dell Indices (I_A and I_T) and the aromatic ring condensation index (I_R) changed little during the hot mixing, spreading and compaction operations. However, this was not the case with the carbonyl index ($I_{c=o}$). The absorption value of $I_{c=o}$, listed in Table 7 and plotted in

Figure 11 show that some limited oxidation of the asphalt took place during the hot mixing, spreading and compaction operations. Oxidation of the asphalt continued during the period of road service but, as can be seen from the data in Table 7, this occurred at a greatly reduced rate.

In the infrared absorption studies, the values for the original Vermont asphalt are typical for conventional good paving asphalts. The absorption value in the $I_{C=O}$ region is customarily used in oxidation studies of asphalts. Using the $I_{C=O}$ infrared absorption value, as a criterion for asphalt oxidation, it can be seen that the oxidation obtained in the thin film oven test is in agreement with the oxidation taking place during the hot mixing, spreading and compaction operations. The chemical properties of the asphalt were evaluated using standard MRDC research test procedures.

3. Nuclear Magnetic Resonance

The nuclear magnetic resonance test shows the distribution of hydrogen atoms in the asphalt in terms of the position of the hydrogen atoms in relation to the carbon atom in the aromatic ring structure. The hydrogens consequently can be classed as attached to a carbon atom attached directly to a carbon atom in the ring called an alpha hydrogen, or in the beta and gamma positions, each one is a carbon atom further removed from the ring carbon. The NMR values given in Table 7 show that the original asphalt has a normal distribution of hydrogen atoms for a typical paving asphalt. The distribution of hydrogen atoms in the asphalts recovered from both the hot mix samples and the road cores showed a lowering of the percentage of hydrogen atoms on the alpha carbons which results in a relative gain in the other positions. This is due to oxidation. The preferred position of oxygen (air) attack is the hydrogen atoms on carbon atoms adjacent (alpha) to aromatic rings. In the normal asphalt air-blowing process at high temperatures, the asphalt is dehydrogenated without addition of oxygen. At lower temperatures such as in road laying a more normal oxidation process takes place. Hydrogen atoms are removed and replaced by oxygen. The NMR data and infrared data definitely tie-in with the expected oxidation mechanism which can be represented by these chemical equations:



The loss of hydrogen from the alpha carbon was rapid during the period of hot mixing, spreading and rolling and matched the loss occurring during the thin film oven test. Subsequent exposure in the road did not seem to cause major changes in the alpha hydrogen values. The relation of the loss of hydrogen from the alpha positions with the aging in the hot mix and road exposures is plotted in Figure 12.

4. Gel Permeation Chromatography

The gel permeation chromatography data, which is contained in Table 7, is reported in terms of average molecular size Ai. The three listed values of Ai are: An (number average), Aw (weight average) and Az (z average). These values are the result of calculating the average molecular size distribution of the asphalt sample from the same G.P.C. data, but using different data resolution techniques. The An value gives equal statistical weight to all fractions in the asphalt sample while both the Aw and Az calculated values are statistically weighted to the larger molecules. Manipulation of data in the fashion enables one to not only determine if changes in the asphalt are taking place, but also to determine which portions of the asphalt sample are undergoing the major changes. It must be kept in mind when analyzing the G.P.C. data that the Ai values are relative determinations of molecular size distribution and do not have a simple relationship which is easily converted to actual molecular weight or carbon number distribution.

The gel permeation chromatography data have not shown a consistent trend in this testing program. However, a continued evaluation of this technique will be made in an attempt to refine it and make the results more meaningful. The differences between the original asphalt and the recovered asphalts are negligible showing that there is very little change in components during exposure in the road. It is believed that the chemical composition of the asphalt does change during the mixing and laying of the road and with time during the service life of the roadway and that these chemical changes are revealed by the gradient elution method but not the gel permeation chromatography.

By inspection of the data from comparable cores it was noted that in general, the asphalts recovered from the mixes and road cores with higher asphalt contents experienced less oxidation and adverse changes in their chemical composition and physical properties during the construction operations. Low asphalt content and effective asphalt content mixes had more age hardening as measured by their chemical composition and specification properties. These results were confirmed by measured changes in the stiffness moduli of the asphalt mixes. A clear relationship was also noted between percent air voids in the road cores and increases in paving mixture stiffness. Road cores with high air voids had greater age hardening and higher dynamic stiffness moduli for given conditions. This relationship was confirmed by the asphalt chemical and physical test data.

H. Road Performance of the Vermont I 91 Overlay

Vermont has furnished MRDC with periodic inspection data on the transverse crack interval (TCI in feet) from the I 91 roadway overlay taken late during the winters of 1970-1971, 1971-1972, 1972-1973 and 1973-1974. The TCI is the average distance in feet between major transverse shrinkage cracks (those cracks which extend across at least 75 percent of a 12 foot traffic lane). They reported that during the period from September 1970 to February 1974 the general appearance and riding qualities of the road were excellent and that they were pleased with the performance of the overlay. The overlay, however, did not completely eliminate transverse cracking. For this report, the Vermont TCI data were converted to transverse cracking frequency, TCF, (cracks per mile) for analyses. The transverse cracking frequency data are summarized in Table 8 and typical TCF data for February 1973 are plotted in Figure 13. Figure 14 is a picture showing typical transverse cracks in a section of the I 91 highway taken during February 1971.

Table 8 shows the location and degree of transverse cracking as TCF (cracks per mile) that occurred during 1971-74. In February 1971 after several months of winter exposure, it can be noted that the overlay at this time, had about 2,880 transverse cracks in the 48 lane miles of road for an average of about 60 cracks per lane mile. Subsequent inspection on February 28, 1973 showed a very slight increase to 3,450 total or 72 cracks/mile. These data clearly indicate that the early transverse cracking developed in the overlay and is a form of reflection cracking due to underlying transverse cracks. Normally it would be several years before transverse cracking occurred to this extent. In February 1974 the Vermont and Asphalt Institute highway engineers reported that the transverse cracking frequency has not changed significantly from the February 1973 TCF values as shown in Table 8. The general location of the transverse cracking and riding qualities of the road have also not changed significantly during this period.

It can also be seen in Figure 13 that some locations were essentially free of transverse cracks and other sections had excessive transverse cracking - as high as 240 cracks per mile. The area analysis of the transverse cracking data given in Table 9 and Figure 15 show that there is no significant difference in TCF for the northbound and the southbound lanes of travel (65 vs. 73). However, the travel lanes had a significantly lower cracking frequency than the passing lanes (42 vs. 96 TCF) due to traffic compaction. The TCF area analysis also showed that the section of the road from 0 to 6 miles above the Massachusetts border had significantly fewer cracks than the area 6 to 12 miles above the Massachusetts border 36 vs. 102 TCF). The variation in cracking performance of the overlay is in sharp contrast to the uniformity of the composition and strength of the road cores

"Predicting Low-Temperature Cracking Frequency of Asphalt Concrete Pavements."(16) They proposed the use of a mathematical model, derived empirically from road performance studies in Ontario, to predict the cracking frequency of asphalt concrete roads from the following variables:

s = Stiffness modulus of original asphalt according to McLeod's method in $kg/cm^2 \times 10$ at temperature m, load time 20,000 seconds

I = Cracking Index as defined by Ontario - the number of full plus one half of the half transverse cracks per 500 foot section

t = Thickness of asphalt concrete, inches

a = Age of asphalt concrete, years

d = Subgrade type; clay = 2, loam = 3, sand = 5

m = Winter design temperature, °C

The numerical form of their predictive equation is:

$$10^{I} = 2.497 \times 10^{30} \times s^{(6.7966 - 0.874t + 1.3388a)} \times (7.054 \times 10^{-3})^{d}$$

$$\times (3.193 \times 10^{-13})^{m} \times d^{0.60265s}$$

This equation has been developed into a nomograph in Figure 23 for simplicity. This nomograph was applied to the Vermont I 91 data. Figures 19, 21, 22, 24 and 25 were used in converting the known variables into the form required by the nomograph in Figure 23.

The use of the nomograph in Figure 23 indicates that the Vermont I 91, two-inch overlay of asphalt concrete, if placed directly on an adequately stable base would not fail by transverse cracking for more than 15 years using the assumptions and correlations developed. This is in contrast to the transverse cracking behavior of the actual road, which reached a cracking frequency in eight months equal to that predicted by the equation for about 15 years. A summary of the data and calculations are presented in Table 11. The cracking in the overlay at eight months must be due to reflection cracks from the old road as indicated by the field core photographs in Figure 16.

J. Rheological, Stability and Durability Properties of Vermont I 91 Field Cores

Paving mixtures are designed not only to provide structural stability under traffic loads and riding surfaces of high quality for safe traffic operation, but also to retain these properties during the service life of the pavements, i.e., these mixtures are designed with respect to stability as well as durability under in-service environmental conditions. According to the stability design criteria utilized, the paving mixtures are selected to resist the applied traffic loads. The durability design criterion, however, is concerned with performance of paving mixtures under repeated loads occurring during the service life and under the adverse environmental factors to which pavements are exposed. Then to ensure selection of a mixture with the highest performance characteristics, consideration must be given to the mixture stability, temperature susceptibility, age hardening and softening characteristics and shrinkage crack resistance properties.

In this study, the strength and rheological characteristics of asphaltic concrete field cores were investigated. Vermont I 91 asphaltic pavement cores designated as Batch B-1, B-2, and B-3 were subjected to creep compression, unconfined compressive strength and axial sinusoidal loading conditions, and the results are presented graphically as well as in tabulated form. Road core Batch B-1, B-2 and B-3 were obtained during September 1970, December 1970, and August 1971, respectively, by Vermont highway engineers. The cores under study represent initial construction, three month old and one year old test specimens. The objectives of the above rheological experiments, limitations and conclusions reached are presented. Complete details of the experimental procedures and equipment used can be found in the following technical reference publications of the authors[(17)],[(18)]. Because Paulsboro does not have the expensive and sophisticated MTS universal testing equipment to carry out such laboratory testing, the actual tests were conducted at The Ohio State University under the direction of MRDC.

K. Summary of Rheological; Durability and Stability Study

The plan of testing carried out in this investigation consisted of the following phases:

1. Sample Preparations

All pavement cores obtained by Vermont were sawed at MRDC to a uniform height of two inches so that L/D ratio (ratio of height to core diameter) was constant for all specimens. The diameter of all cores was 3.75 inches. Only those specimens having parallel top and bottom surfaces were selected for testing. Prior to testing, the density of all the pavement cores was determined using geometric and specific gravity methods and the average average values are presented in Table 12.

2. Constant-Load Creep Experiments

Selected pavement cores were subjected to uniaxial compression creep experiments at temperatures of 41°, 77° and 104°F. Test specimens were subjected to a mechanical conditioning test in which loads were applied and removed while creep response was recorded. The load/unload cycle, after which there is no significant difference in the creep response, was selected for experimentation. The data indicated that the third or fourth loading cycle could be selected for experimental characterization with no further mechanical conditioning.

To check the linear viscoelastic response of the creep experimental data, creep tests were conducted at three different stress levels, and the assumption of linear viscoelastic response of the field cores was verified. The strain-time relations of all field cores experiments were studied to check the repeatability of all test methods and specimens. The effect of temperature on the creep response was evaluated by conducting creep tests at three different test temperatures of 41°, 77° and 104°F. Also, the temperature-dependence function, a_T, and master creep curves were prepared from the experimental data.

3. Determination of Strength Characteristics of Field Cores at Three Temperatures

The unconfined compressive strength of field cores at 41°, 77° and 104°F was obtained. The Young's modulus of elasticity was also determined for these specimens.

4. Determination of the Response of Specimens Under Dynamic Loading Simulating Traffic Loading Conditions

The creep data were transformed into dynamic data by use of an available analytical method. A limited number of dynamic tests were also carried out using the MTS universal testing machine. Viscoelastic model parameters describing the viscoelastic response of field cores were also determined for the road cores.

L. Discussion of Rheological Test Results

1. Creep Experiments

In the compression creep experiments, each specimen is subjected to a constant compressive load and the resulting axial deformation or strain, ε_{zz}, is recorded.

The results of creep experiments are often reported in terms of axial strain-time relations (ε_{zz}-t), or in terms strain per unit-stress time relation, ($\frac{\varepsilon_{zz}}{\sigma}$ -t) which is referred to as creep compliance, $\sigma(t)$ - time relation. Analogous to time-independent materials, the results can also be presented in terms of creep modulus E_c-time relation. In such a case, creep modulus is determined as the inverse of the creep compliance; i.e., $E_c(t) = \frac{1}{\sigma(t)}$. In this report, data are presented in both E_c-t and $\sigma(t)$ - t relationships.

A creep curve of a time-dependent material such as asphaltic concrete may consist of three distinct parts; namely, (1) an initial or instantaneous elastic deformation, (2) a delayed elastic deformation, and (3) a viscous deformation as shown in Figure 26A. The instantaneous elastic deformation part has been primarily associated with the nature of aggregate skeleton; whereas the delayed elastic and viscous deformation are affected by rheological characteristics of asphaltic binder as well as the properties of the aggregate system. Since asphaltic concrete mixtures as other viscoelastic materials are temperature dependent, the relative magnitude of the components of creep deformation is also affected by the testing temperature.

In this investigation, the test specimens were first subjected to a mechanical conditioning test in which loads were applied and removed while the changes in the creep response were noted. The load/unload cycle after which there is no significant difference in the creep deformation was selected for the analysis of results.

In Figure 26B, the result of a typical mechanical conditioning test is shown for the Vermont road cores. It is noted that after the fourth loading cycle there is no significant change in the creep response. Furthermore, to ensure validity of the assumption of linear viscoelastic response for the three mixtures, linearity tests were performed.

The typical results presented in Figure 27 shows that for the range of loads applied, the creep deformation at a given time is a linear function of applied load. Therefore, for the range of compressive stresses used in this investigation, the asphaltic field cores can be treated as linear-viscoelastic material and the mechanical properties of the road cores studied can be used in pavement design investigations.

In Figures 28 through 33, the results of creep experiments are presented. In Figures 28 through 30, the creep compliance J(t) for the three batches of field cores at temperatures of 41°, 77° and 104°F are presented. In Figures 31 through 33, the creep modulus-time relation for these specimens at different test temperatures are shown. As noted previously, the creep modulus is an inverse of creep compliance and it decreases with an increase in loading time. The creep modulus measured at any time also increases with a decrease in testing temperature.

In Figures 34, 35 and 36, the repeatability of experimental data are presented. In these figures for the purpose of clarity, the repeatability data are only presented for typical experimental data at one temperature. The repeatability of experimental data is shown as a scatter band including the average and the upper and lower limits of observation. Despite the inherent variability of all field cores, the creep modulus data are shown to be quite reproducible indicating uniform test specimens. The greatest variability in the creep modulus data was found at 41°F temperature; whereas, the least variability is noted for specimens tested at 77°F.

In Figure 37, the long-term deformation characteristics of asphaltic field core batches B-1 and B-2 are presented. In this experiment field cores were subjected to 21 psi uniaxial compressive stress at the temperature of 77°F. After the initial preconditioning period (three load/unload cycles), these specimens were stressed and creep strains were recorded for a period of 6000 seconds. The shape of the creep-strain-time relation is identical with the data presented previously for short term creep tests. It is interesting to note that these mixtures do not exhibit substantial steady-state deformation, and the creep compliance tends to approach to a constant and limiting value probably caused by interlock of the aggregates.

From the results of creep experiments presented graphically in the previous figures and also by Tables 13 through 18 in Appendix A, the following conclusions can be reached:

(a) There is a noticeable difference in long term creep deformation of field cores batches B-1, B-2 and B-3. The field core batch B-1 exhibits the highest creep compliance (lowest creep modulus), whereas mixture batch B-3 attains the lowest creep compliance (highest creep modulus). The mixture batch B-2 exhibits an intermediate response. In brief, there are changes in long term creep deformation of these specimens which are attributed to differences in the asphaltic mixture stiffness and asphalt stiffness and viscosity of these specimens due to traffic compaction and aging.

(b) Because of inherent limitation of creep experiments, it is difficult to accurately establish differences between instantaneous elastic deformation of these specimens. For this reason, direct dynamic tests were performed at various loading frequencies, and the instantaneous elastic deformations were determined using the dynamic modulus or stiffness at high frequencies using the MTS equipment. In Figures 38 and 39, the dynamic modulus-frequency relation for batches B-1, B-2 and B-3 are presented. It is interesting to note that mixture batches B-2 and B-3 exhibit similar response; whereas batch B-1 shows higher dynamic response corresponding to a greater instantaneous elastic response at higher frequencies. It can be postulated that this response may be due to the elastic nature of the aggregate structure in the paving mix. After excessive compaction by traffic and exposure to weather in the road the particle structure is broken and the response is essentially controlled by the stiffness of the mixture and asphalt as well as the compacted aggregate structure. At low frequencies, corresponding to long creep loading times, the dynamic response will approach to that of creep modulus response.

To investigate the effect of temperature on the rheological characteristics of field cores, the creep modulus data at various temperatures were subjected to time-temperature superposition principle[17]. In Figures 40 through 43 the master curves which resulted from superposition of the short term creep modulus data are presented. The temperature shift factor, a_T, which describes the temperature dependency of asphaltic mixtures is shown in Figure 44[18].

2. Strength Evaluation

To evaluate the strength-deformation characteristics of pavement field cores, unconfined compressive strength tests were conducted at 41°, 77° and 104°F using the standard ASTM test procedures (ASTM D 1074). The unconfined compressive strength ($\sigma'c$) data and the Young's modulus of elasticity (E) of the field core data are presented in Table 19 and graphically in Figures 45 and 46. The results indicate that the E and $\sigma'c$ do not greatly change with age although the specimen batch B-3 generally has the greatest $\sigma'c$ strength; whereas batch B-1 generally has the lowest unconfined compressive strength. The field core specimen batch B-2 exhibits intermediate strength values. This observation is in agreement with the results of creep deformation data presented previously.

The results of Young's modulus of elasticity data indicate, in general, that mixture B-3 has the greatest modulus; whereas mixture B-1 exhibits the lowest. The Young's modulus of mixture B-2 appears to be intermediate between those of batch B-1 and B-3.

3. Dynamic Response

As noted previously in this study, the dynamic response of paving mixtures under cyclic loading was determined using direct dynamic testing as well as operational mathematics transformation techniques[17]. In Figures 38 and 39 and also Tables 20 and 21, the dynamic modulus of paving mixtures determined by direct MTS testing at temperatures of 41°F and 77°F are presented. For the range of frequencies at which these tests were conducted, the asphaltic mixtures behave as linear elastic materials. Therefore, the magnitude of the dynamic modulus primarily reflects the instantaneous elastic response characteristics of these mixtures. The dynamic modulus determined at lower frequencies, however, not only reflects the elastic characteristics but also characterizes the damping and viscous nature of the asphaltic materials.

In this study, the creep compliance data were subjected to an operational mathematical transformation technique in which response in time domain was transformed to a frequency domain response. The creep data were first approximated by an equation of the form:

$$J(t) = Jo + J\infty - \sum_{i=1}^{n} c_i \, e^{-\alpha_i t}$$

where: $J_o = \frac{1}{E_o}$ initial compliance

E_o = Instantaneous Elastic Response (Spring Modulus)

$J\infty$ = Final Compliance

$J\infty = \sum_{i=1}^{n} \frac{1}{E_i}$ summation of $(\frac{1}{E_i})$ of all Kelvin elements

where c_i and α_i are constants describing the viscoelastic behavior of the mixtures.

These constants were determined by the procedure developed by Brisbane[19] used in this study. Using a well-known transformation technique, the dynamic modulus and phase angle can easily be calculated[17]. In Figures 47, 48 and 49, and in Tables 22 through 30, the dynamic modulus-frequency temperature data are presented for essentially the same mix tested at three different ages. Frequency and dynamic modulus values tabulated in Tables 22 and 30 have units of CPS and psi, respectively. The values of the coefficients (alpha$_i$ and c_i) shown in the tables are the visco-elastic generalized Voigt model parameters describing the mechanical properties of the materials[(17),(19)]. The creep compliance values, c_i have units of $\frac{1}{psi}$ and the retardation times have units of seconds and are equal to $\frac{1}{\alpha i}$ in the tables. In the lower intermediate frequency range, it is noted that batch B-3 has a higher modulus than batches B-1 and B-2. At higher frequencies, however, where only instantaneous deformation takes place comparable to fast traffic loads, a reverse trend is clearly observed. A typical Voigt mechanical model is shown in Figure 50 to illustrate the utility of the data in Tables 22-30. The values of the coefficients from Table 22 have been used to construct a typical model which defines the response of the asphaltic concrete batch B-1 to applied stresses at 41°F. These models can be directly used in pavement design calculations and materials evaluations. This new technology on the aging and subsequent changes in the rheological properties of highway pavements evaluated by tests on road cores is not available in the literature.

4. <u>Summary of Results from Exploratory Rheological Tests</u>

(a) Field core specimens designated as batch B-3 have the greatest stiffness modulus and lowest creep deformation.

(b) Field core specimens designated as batch B-1 have the lowest stiffness and highest creep deformation.

(c) Field core specimens designated as batch B-2 exhibit an intermediate behavior. The response and stiffness moduli of these mixtures are very similar to that of batch B-3.

(d) Rheological testing procedures can be used to evaluate the fundamental time and temperature dependent stiffness moduli of asphalt paving mixes.

(e) The mechanical behavior of dense asphaltic road cores can be defined using the linear viscoelastic theory. Direct experimental verification was obtained by creep and dynamic tests using MTS testing equipment.

(f) The time-temperature superposition concept can be applied to pavement road cores as a good engineering approximation for future pavement design studies. The number of experiments required to evaluate the stiffness properties of the materials over a range of loading times and temperatures can be greatly reduced. Laboratory tests have provided direct verification of this application. The new technology developed in this study will provide the pavement design engineer with a powerful research method to evaluate the low temperature shrinkage fracture of bituminous pavements. The stiffness moduli of road cores can now be directly evaluated and correlated with transverse cracking frequency instead of being roughly approximated using indirect stiffness modulus nomographs.

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Table 1

VERMONT I 91 ROAD STUDY
THICKNESS DESIGN OF ORIGINAL ROAD
1958 CONSTRUCTION

Road Design Estimates:

Average Daily Traffic ADT (1958)	2200
Average Daily Traffic ADT (1978)	6640
Daily Heavy Vehicles DHV (1978)	1000
Trucks, %	7.5
Traffic Growth Rate, %	5.0
Average Subgrade California Bearing Ratio	12.8

Thickness Design From Vermont Design Curves:

Bituminous Concrete, Item 361 B, inches	3.0
Tack Coat on Crushed Stone 0.4 gal./sq. yd.	
Crushed Stone Base, Item 211, inches	3.0
Crushed Rock Subbase, Item 204, inches	21.0
Sand Cushion, inches	6.0
Total Thickness, inches	33.0

Conversion of Vermont Thickness Design to Full-Depth Pavement Equivalent:

	Thickness Inches	Factor	T_A Equivalent Inches
Bituminous Concrete	3	1.0	3.0
Crushed Stone Base	3	0.4	1.2
Crushed Rock Subbase	21	0.3	6.2
Sand Cushion	6	0.1	0.6
			11.0

Thickness Design in Full-Depth Construction From Asphalt Institute Tables:

Initial Daily Traffic IDT (1958)	2200
Daily Heavy Trucks	80
Gross Weight of Trucks, lb.	35,000
Initial Traffic No. I.T.N.	50
Design Period, yr.	20
Growth Rate, %	5
I.T.N. Factor	1.67
Design Traffic No.	83
Average Subgrade California Bearing Ratio	12.8
Required Total Thickness Full-Depth Pavement	6.0 inches

Table 2

VERMONT I 91 ROAD STUDY THICKNESS
DESIGN OF PAVEMENT OVERLAY
1970 CONSTRUCTION

Road Overlay Design Estimates:

Average Daily Traffic ADT 1970	8700
Average Daily Traffic ADT 1990	14,100
Average Daily Load (18 KIP Equiv.)	853.7
Design Hour Directional Distribution, % S	69
% Trucks in Design Hour	3
Design Hour to ADT, %	18
Percent of Trucks of ADT, %	9
Traffic Growth Rate, %	5

Thickness Design From Vermont Design Curves:

Asphalt Concrete Overlay, inches	2
Old Road Structural Sections:	
Bituminous Concrete, inches	3
Crushed Stone, inches	3
Crushed Rock, inches	21
Sand Cushion, inches	6
Total, inches	35

Conversion of Vermont Thickness Design to Full-Depth Equivalent:

	Thickness Inches	Factor	T_A Equivalent Inches
Bituminous Concrete Overlay	2	1.00	2.0
Old Bituminous Concrete	3	0.33	1.0
Crushed Stone Base	3	0.40	1.2
Crushed Rock Subbase	21	0.30	6.2
Sand Cushion	6	0.10	.6
			11.0

Thickness Design in Full-Depth Construction
From Asphalt Institute Tables:

Initial Daily Traffic IDT (1970)	8700
Daily Heavy Trucks	376
Gross Weight of Trucks, lb.	40,000
Initial Traffic No. I.T.N.	270
Design Period, yr.	20
Growth Rate, %	5
I.T.N. Adjustment Factor	1.67
Design Traffic No.	627
Average Subgrade California Bearing Ratio	12.8

Required Total Thickness, Full-Depth T_A 7.5 inches

Table 3

VERMONT I 91 ROAD STUDY-COMPOSITION AND PROPERTIES OF HOT-MIX SAMPLES

Sample Identification	Hot Mix Corp. July 17, 1970	Hot Mix Corp. July 27, 1970	Hot Mix Corp. July 28, 1970	Vermont Highway Laboratory Batch 87	Vermont Highway Laboratory Batch 77	Vermont Highway Laboratory Batch 78	AI and ASTM Specifications
Aggregate Grading, Wt. %							
Pass 1/2"	100.0	100.0	100.0	100.0	100.0	100.0	100
" 3/8"	99.3	99.8	100.0	100.0	100.0	99.7	90-100
" #4	70.1	78.7	74.2	70.7	76.5	73.0	60-80
" #8	47.8	53.6	47.6	46.8	52.8	50.4	35-65
" #16	38.2	41.2	36.0	36.4	40.6	40.8	
" #30	26.8	26.2	23.2	25.1	28.2	28.8	
" #50	13.5	12.1	12.2	12.9	14.6	15.6	6-25
" #100	6.3	6.0	5.8	6.8	7.4	7.7	
" #200	2.8	2.3	2.6	2.8	2.7	7.7	2-10
Gradation Modulus	605	620	602	602	623	619	
Marshall Data for Laboratory Compacted Test Specimens - Heavy Traffic Category (75 Blows)							
Aggregate, Wt. %, Ps*	93.15	92.99	93.51	93.11	92.85	93.39	
Asphalt, Wt. %, Pb	6.85	7.01	6.49	6.89	7.15	6.61	5-10
Aggregate, Gsb	2.614	2.614	2.614	2.614	2.614	2.614	
Asphalt Sp. Gr., Gb	1.028	1.028	1.028	1.028	1.028	1.028	
Compacted Mix Sp. Gr., Gmb	2.360	2.340	2.350	2.370	2.360	2.380	
Voidless Mix Sp. Gr., Gmm	2.451	2.451	2.451	2.451	2.451	2.451	
Voids in Mineral Agg., %, VMA	15.90	16.76	15.94	15.58	16.17	14.97	16+
Air Voids, %, Pa	3.71	4.52	4.12	3.30	3.71	2.89	3-5
Marshall Stability, lb.	2007	1875	1792	1825	1968	1960	750+
Marshall Flow, 0.01 inch	13.7	12.8	12.7	13.2	13.7	14.5	8-16

*"Mix Design Methods for Asphalt Concrete," The Asphalt Institute, MS-2, 1974.

Table 4

VERMONT I 91 ROAD STUDY
PROPERTIES OF ROAD CORES

Location (Stations)	North Bound Lane 1/50 - 4/70				North Bound Lane 7/05 - 8/05				South Bound Lane 1/25 - 4/50				South Bound Lane 8/75 - 11/15				A.I. Specifications-Heavy Traffic Category
Date Cored (Month/yr.)	9/70	12/70	4/71	8/71	9/70	12/70	4/71	8/71	9/70	12/70	4/71	8/71	9/70	12/70	4/71	8/71	
Extraction																	
Asphalt, wt. %, Pb*	7.20	7.10	[illegible]	7.19	7.13	7.30	6.94	7.41	7.45	7.3	8.02	7.34	7.24	7.2	7.29	7.10	
Aggregate, wt. %, Ps	92.80	92.90	92.96	92.81	92.87	92.70	93.06	92.59	92.55	92.7	91.98	92.66	92.76	92.8	92.71	92.90	
Aggregate Sieve Analysis																	
Pass #8, wt. %	51.7	48.9	52.3	47.7	53.2	53.8	55.3	51.1	53.4	52.0	53.7	52.8	49.7	52.7	51.9	56.3	
Pass #200, wt. %	3.34	3.9	3.5	4.1	3.62	3.3	3.7	3.5	3.74	3.8	4.1	3.5	3.56	3.3	3.6	3.9	
Marshall Data for Road Cores																	
Voids in Mineral Agg., %, VMA	21.78	17.06	18.08	16.44	18.53	19.02	18.70	16.99	19.86	18.67	20.00	18.70	20.39	17.87	17.95	17.42	16+
Air Voids, %, Pa	9.98	4.82	6.06	4.00	6.44	6.87	6.87	4.40	7.71	6.46	7.24	6.45	8.55	5.65	5.63	5.20	3-5
Marshall Load, lb.	216	643	384	700	330	294	-	805	200	389	526	545	230	475	919	779	750+
Marshall Flow, 0.01 inch	23	26	19	25	21	22	-	26	25	21	25	25	21	22	22	23	8-16
Specific Gravity of Core, Gmb	2.20	2.33	2.30	2.35	2.29	2.28	2.28	2.34	2.26	2.29	2.27	2.29	2.24	2.31	2.31	2.32	

*"Mix Design Methods for Asphalt Concrete," The Asphalt Insitute, MS-2, 1974.

Table 5

VERMONT I 91 ROAD STUDY
CONSTRUCTION SCHEDULE OF OVERLAY

Schedule of Overlay

Date	Item	Stations
July 6-7, 1970	1" Base Course NBL Station	400 - 620
July 8-9	1" Base Course SBL Station	400 - 620
July 10-14	1" Top Course NBL Station	400 - 620
July 15-17	1" Top Course NBL Station	400 - 620
July 27-30	1" Base Course NBL Station	0 - 400
August 3-7	1" Top Course NBL Station	0 - 400
August 17-20	1" Base Course SBL Station	0 - 400
August 24-28	1" Top Course SBL Station	0 - 400

See Figure 2
To convert station to miles above
Massachusetts border (station = 100 ft.)

Special Overlay Construction Details

1. All cracks were filled in with a polymer-asphalt joint sealer prior to overlay construction.

2. A hot mix leveling course was applied where needed to smooth out the road.

3. A sand-asphalt emulsion was applied to all lanes to seal the fine cracks and to break the bond between the overlay and existing pavement in an attempt to reduce reflection surface cracking.

4. A one-inch hot mix base course was applied and opened to traffic.

5. A second one-inch hot mix surface course was applied.

Table 6

VERMONT I 91 ROAD STUDY
PHYSICAL TESTS ON ASPHALTS

Sample Identification	Original Refinery Sample	Residue From Thin Film Oven Test 1/8"	Recovered From Hot Mix: Hot Mix Corporation	Recovered From Hot Mix: Vermont Laboratory	Recovered From Road Cores September 1970 Avg.	Recovered From Road Cores September 1970 Range	Recovered From Road Cores December 1970 Avg.	Recovered From Road Cores December 1970 Range	Recovered From Road Cores April 1971 Avg.	Recovered From Road Cores April 1971 Range	Recovered From Road Cores August 1971 Avg.	Recovered From Road Cores August 1971 Range
	1,000,000 gallons											
	Panuco Asphalt											
ASTM Standard Tests												
Softening Point, R&B, °F	110	123	122	120	129	123/131	126	125/126	128	127/128	128	125/129
°C	43.3	50.6	50.0	48.9	53.9	50.6/55.0	52.2	51.7/52.2	53.3	52.8/53.3	53.3	51.7/53.9
Specific Gravity, 77/77	1.0282											
Flash Point, COC °F	535											
Penetration 77/100/5	140	64	81	83	61	56/64	65	64/65	64	63/64	64	59/74
59/100/5	47		31	30								
Viscosity, Poises at 77°F	6.3 X 10^5											
140°F	835	3247	1826	1888	2877	2540/3098	2351	2037/2782	2500	2121/2903	3212	2250/3750
158°F	273											
275°F	2.67											
Ductility 77°F, 5 cm/m	140+	140+										
59°F, 5 cm/m	140+	100										
Penetration Index	-.04	-.04	0	-.02	0	-	+.02	-	+.03	-	+.03	-
Viscosity-Temperature Slope	-3.62											
Blowing Factor (Kinnaird)	22	18	25	24	23	-	22	-	23	-	23	-

Table 7

VERMONT I 91 ROAD STUDY
CHEMICAL TESTS ON ASPHALTS

Sample Identification	Original Refinery Sample	Residue From Thin Film Oven Test 1/8"	Recovered From Hot Mix Corp. and Vermont State Highway Lab.	Recovered From Road Cores September 1970	Recovered From Road Cores December 1970	Recovered From Road Cores April 1971	Recovered From Road Cores August 1971
Chemical Tests							
Mobil Gradient Elution Chromotography							
1. Saturates	11.8	11.2	12.6	12.7	10.9	10.9	10.8
2. Mono & Dinuclear Aromatics	10.6	7.9	10.9	11.3	8.1	8.2	7.8
3. Polynuclear Aromatics	17.2	16.1	15.1	12.1	15.1	15.6	15.1
4. Soft Resins	23.9	20.0	19.2	17.7	18.6	18.6	18.3
5. Hard Resins	13.2	11.4	13.1	10.3	11.8	11.1	11.6
6. Eluted Asphaltenes	14.0	16.8	14.9	19.7	20.9	19.8	20.8
7. Non-Eluted Asphaltenes	9.3	16.6	14.2	16.2	14.6	15.8	15.6
Infrared Absorption							
1. Infrared Absorbance Index, I_A	0.03	0.021	0.031	0.032	0.032	0.030	0.025
2. Infrared Transmittance Index, I_T	0.47	0.44	0.44	0.50	0.51	0.49	0.51
3. Aromatic Ring Condensation Index, I_R	1.14	1.09	1.13	1.14	1.11	1.10	1.18
4. Carbonyl Index, $I_{c=o}$	0	0.66	0.69	0.77	0.66	0.93	1.00
Nuclear Magnetic Resonance							
1. Aromatic Hydrogen, %	5.2	5.9	6.9	7.5	5.6	6.0	5.8
2. Alpha Hydrogen, %	21.6	15.5	15.9	16.6	14.3	15.0	17.1
3. Beta Hydrogen, %	57.5	61.5	57.2	58.9	59.5	57.7	58.7
4. Gamma Hydrogen, %	15.7	17.1	20.0	17.0	20.6	21.3	18.4
Gel Permeation Chromatography							
1. Number Average Molecular Size, A_N	39	41	43	42	39	43	40
2. Weight Average Molecular Size, A_W	89	99	101	96	93	94	91
3. Z Average Molecular Size, A_Z	213	260	229	256	251	227	229
Carbon, %	83.8			83.3			
Hydrogen, %	10.0			10.0			
Sulfur, %	5.0			--			
Oxygen, %	0.5			0.5			
Nickel, ppm	66						
Vanadium, ppm	350						
Wax LCPC, %	0.40						

Table 8

VERMONT I 91 ROAD STUDY
TRANSVERSE CRACK FREQUENCY (CRACKS PER MILE)*

	PASSING LANE						TRAVEL LANE					
Date Inspected	2/4/71	3/18/71	4/8/71	3/25/72	2/28/73	2/28/74	2/4/71	3/18/71	4/8/71	3/25/72	2/28/73	2/28/74
Location of Test Sections												
Southbound Lane												
Miles from Mass. Border												
1. 0.25 to 0.35	50	50	70	50	60	80	0	0	0	0	0	0
2. 1.20 to 1.30	40	50	50	40	40	60	0	0	0	0	0	0
3. 1.95 to 2.05	80	80	70	70	91	101	20	20	20	40	40	50
4. 2.60 to 2.70	30	50	60	60	60	60	0	0	0	0	0	0
5. 3.60 to 3.70	0	0	0	0	0	0	0	0	0	0	0	0
6. 4.45 to 4.55	0	0	0	20	20	20	0	0	0	0	0	0
7. 5.60 to 5.70	203	189	189	203	182	203	70	70	70	90	90	100
8. 6.45 to 6.55	120	129	139	150	151	170	0	0	0	0	0	0
9. 7.90 to 8.05	100	110	110	110	110	110	30	70	70	90	90	110
10. 9.45 to 9.55	151	160	160	170	170	170	30	40	40	40	40	50
11. 11.10 to 11.20	155	220	220	210	240	188	203	230	230	190	278	150
Average TCF	90	100	103	98	102	106	35	42	42	41	49	42
Northbound Lane												
Miles from Mass. Border												
12. 0.60 to 0.70	30	40	40	50	40	89	20	30	30	30	50	80
13. 1.50 to 1.60	0	0	0	0	0	0	0	0	0	0	0	0
14. 1.95 to 2.05	40	70	70	70	70	100	70	70	70	70	60	70
15. 2.35 to 2.45	20	40	50	60	70	89	0	0	0	0	0	0
16. 3.25 to 3.35	90	100	110	110	100	100	50	60	60	40	60	70
17. 4.65 to 4.75	90	80	80	110	120	138	80	80	80	90	80	100
18. 5.85 to 5.95	120	129	129	140	129	150	50	60	60	50	66	80
19. 7.00 to 7.10	70	80	80	100	100	110	20	20	20	20	20	30
20. 8.00 to 8.10	139	139	140	140	139	150	100	110	110	100	100	120
21. 9.40 to 9.50	50	50	50	70	70	80	0	0	0	0	0	0
22. 10.15 to 10.25	80	80	80	70	80	110	30	30	30	30	30	40
23. 10.95 to 11.05	211	240	240	230	230	230	30	30	40	40	50	60
Average TCF	78	87	88	96	96	112	38	41	42	39	43	54
TCF Grand Average for all Lanes	60	68	69	69	72	78						

* Based on inspection of the twenty-three 0.1 mile test sections of road.

Table 9

VERMONT I 91 ROAD STUDY
AREA ANALYSIS OF TRANSVERSE CRACK FREQUENCY
APRIL 1971 INSPECTION

Locations	Average Transverse Cracking Frequency Cracks Per Mile	Difference From Total Road Average
All Passing Lanes	96	+27
All Travel Lanes	42	-27
0-6 Miles Above Mass. Border	36	-33
6-12 Miles Above Mass. Border	102	+33
Northbound Lanes	65	-4
Southbound Lanes	73	+4
Road Average	69	0

Table 10

VERMONT I 91 ROAD STUDY
ANDERSON-SHIELDS TRANSVERSE CRACKING FORMULA

Asphalt Sample Identification	Penetration 77/100/5	Softening Point R+B, °F	Viscosity 140°F Poises	Minimum Value A-S Viscosity Poises
1. Vermont I 91 Original 70-93374	140	110	835	670
2. TFOT Resid	64	123	3247	1800
3. RTOF Resid	60	126	3204	2000
4. Hot Mix, Vermont Highway Lab.	80	122	1800	1350
5. Sept. 1970 Road Cores	61	130	2877	1900
6. Dec. 1970 " "	65	126	2351	1500
7. April 1971 " "	64	128	2500	1800
8. August 1971 " "	64	128	3212	1800

Minimum Value (Anderson-Shields)

Log Viscosity = 5.6 - (1.28 Log Penetration at 77/100/5)
at 140°F

Table 11

VERMONT I 91 ROAD STUDY
PREDICTION OF TRANSVERSE CRACKING
USING HAJEK AND HAAS MODEL

1.	Freezing Index Vermont I 91 Location, Degree Days	1000	Figure 19
2.	Winter Design Temperature, °C	-23	Figure 24
3.	Original Asphalt		
	a. Penetration 77/100/5 (25°C)	140	
	b. Viscosity at 275°F, cs	267	
	c. Penetration Index	-0.5	Figure 25
	d. Difference in °C Between Base and Penetration Temperature	17	Figure 22
	e. Base Temperature, °C, [Penetration Temperature (25°C) + Base Temperature (17°C)]	42	Figure 22
	f. Degree Difference Between Base and Design Temperature, °C (42° + 23°)	65	
	g. Stiffness Modulus of Asphalt Cement, 20,000 sec, -23°C		Figure 21
	Kg/cm^2	20.4	
	psi	288	
4.	Thickness of Asphalt Concrete Overlay, inches	2	
5.	Predicted Cracking Index From Figure 23 in Cracks per 500 feet		Figure 23

Age of Overlay in Years	0	0.7	15
Over Clay	0	0	7
Over Loam	0	0	5
Over Sand	0	0	2

6. Approximate Actual Cracking Index of I 91 From Table 8 in Cracks per 500 feet (Average for all Lanes)

Age of Overlay in Years Over Cracked Road	0	0.7	1.7	2.7	3.7
TFC Grand Average ÷ 10	0	7	7	7	8

Table 12

Average Density of Field Core Specimens

Vermont Highway Core Batch No.	Date Cores Removed From I 91	Average Density, lb./cu.ft.
B-1	Septmeber 1970	140.4
B-2	December 1970	143.0
---	April 1971	143.3
B-3	August 1971	144.8

Figure 1

LOCATION OF VERMONT I 91 TEST ROAD

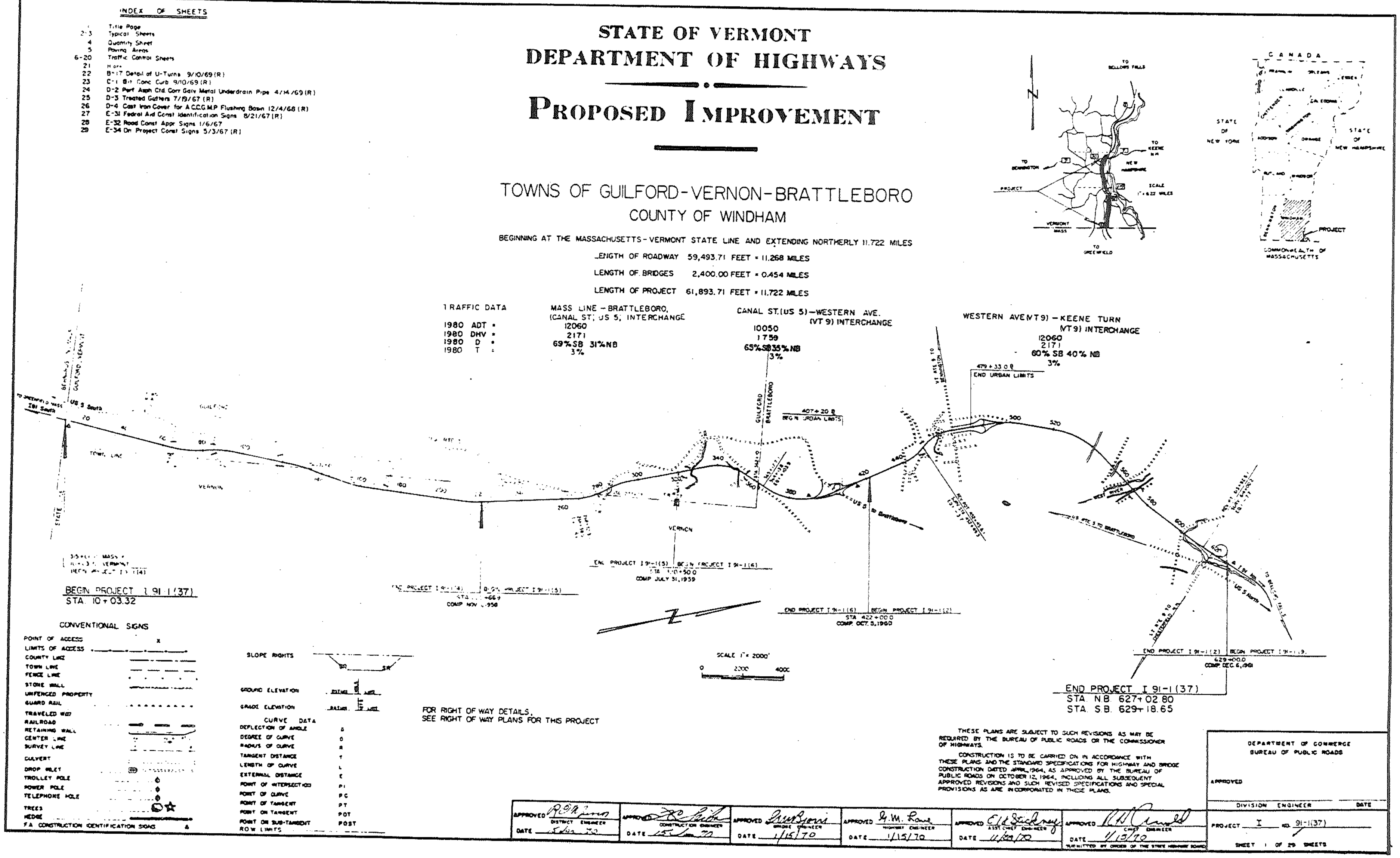


Figure 2

VERMONT I 91 ROAD STUDY -
CONVERSION CHART
STATIONS TO DISTANCE FROM MASSACHUSETTS BORDER

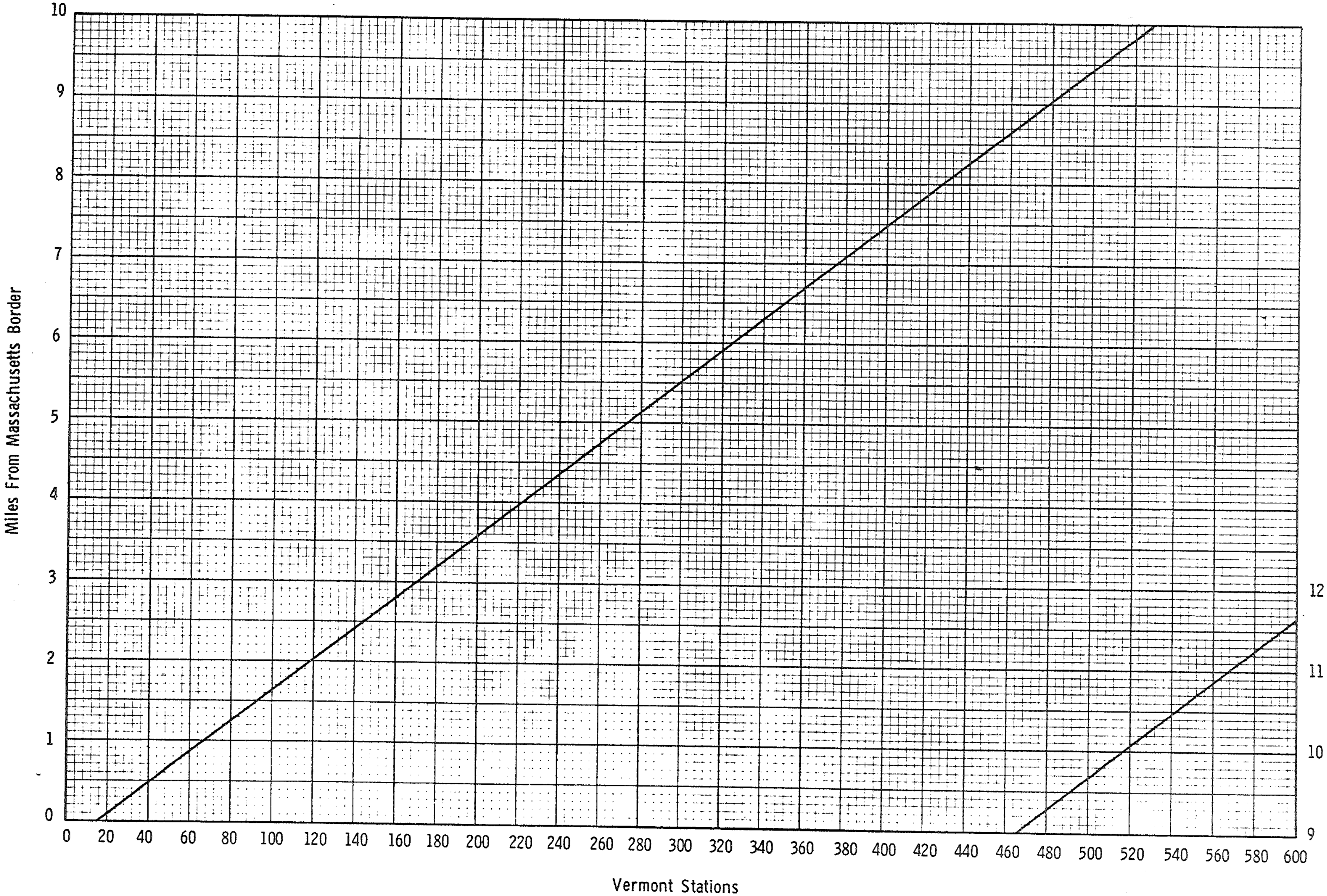


Figure 3

VERMONT I 91 ROAD STUDY – MARSHALL STABILITY AND FLOW

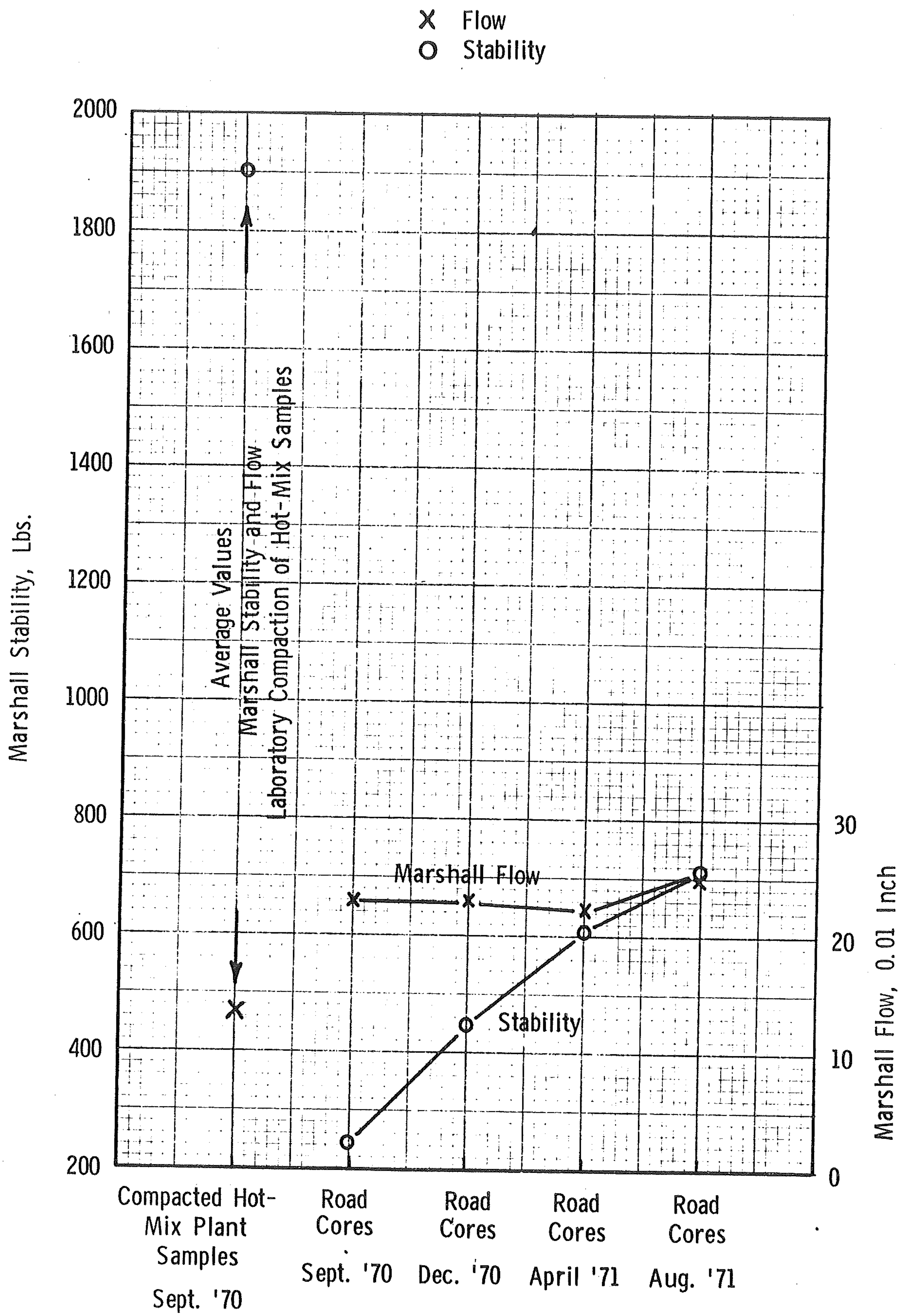
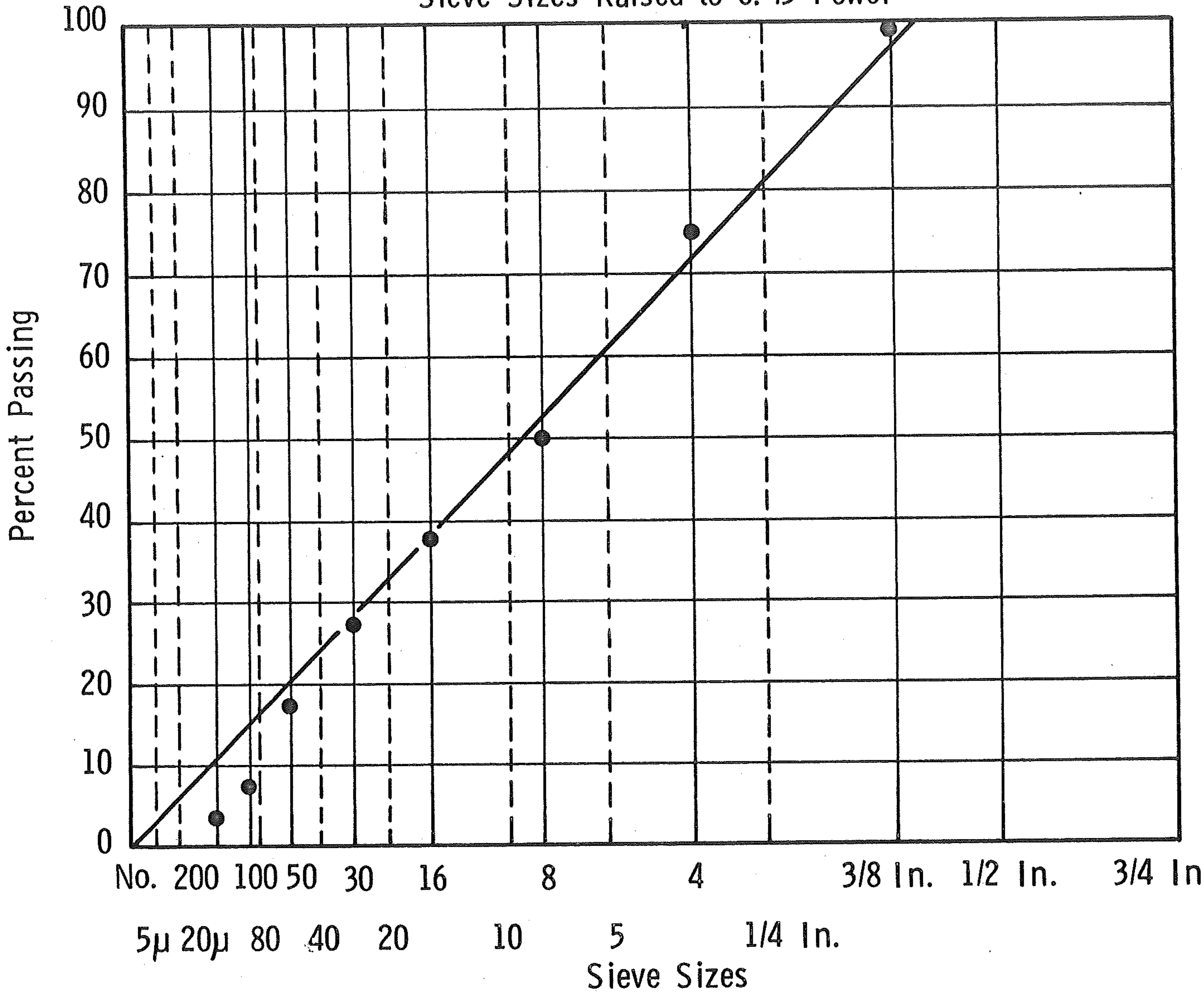


Figure 4

VERMONT I 91 ROAD STUDY - DRY SIEVE ANALYSIS C-136
ON AGGREGATE EXTRACTED FROM HOT-MIX

Average of Six Mixes

Gradation Chart
Sieve Sizes Raised to 0.45 Power



Federal Highway Administration 0.45 Power Gradation Chart

Figure 5

VERMONT I 91 ROAD STUDY - AIR VOIDS

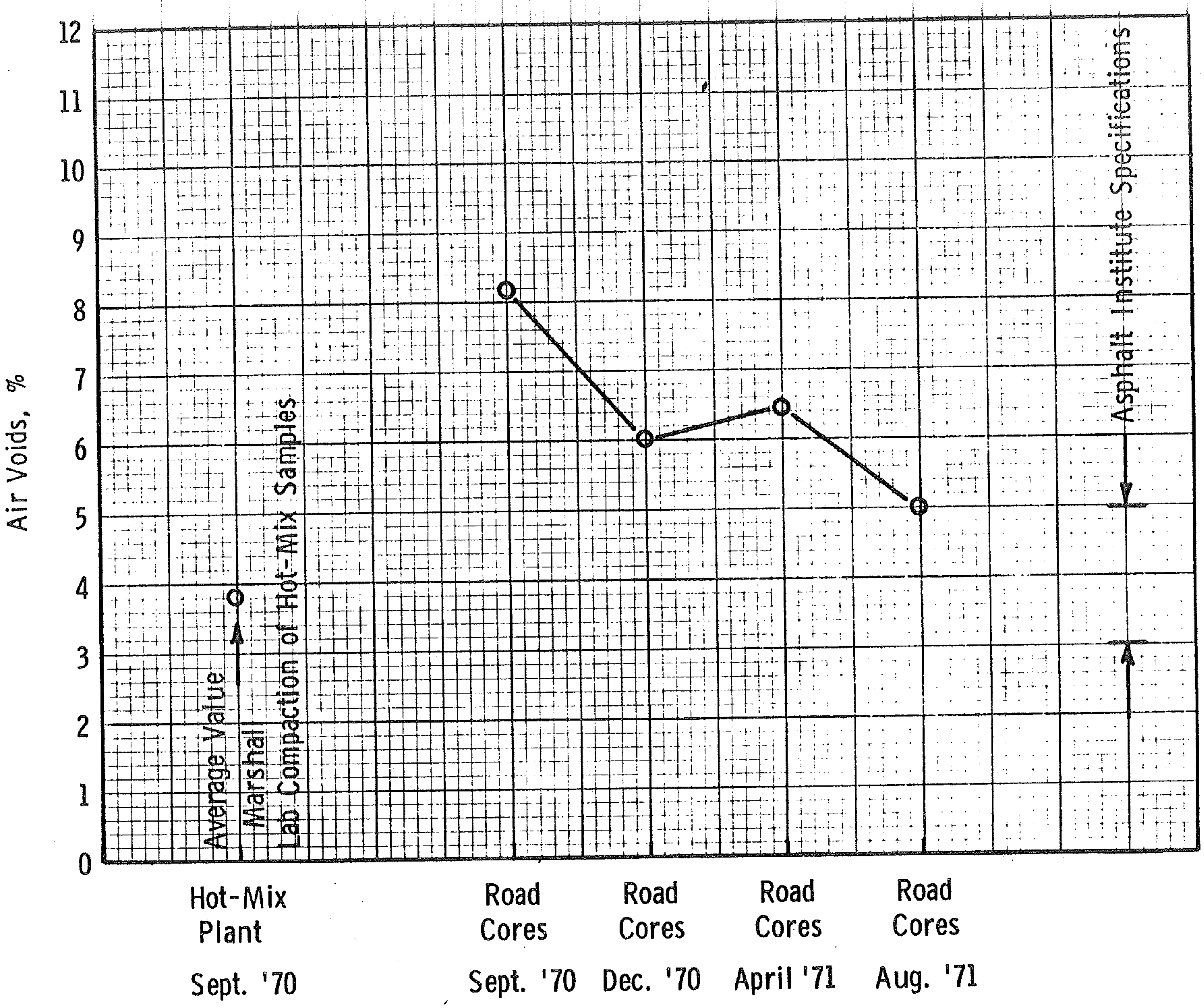


Figure 6

VERMONT I 91 ROAD STUDY - ASPHALT PENETRATIONS

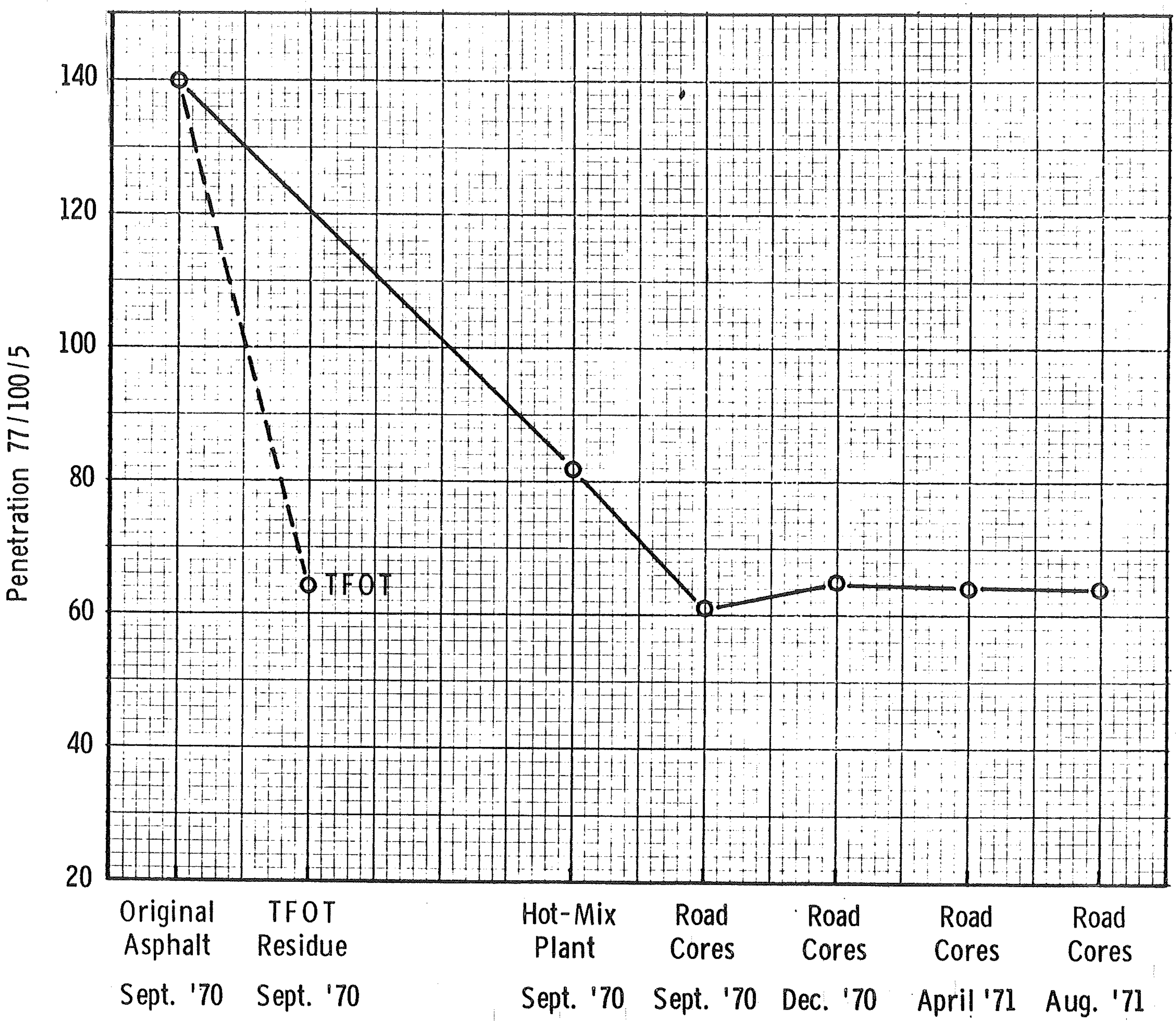


Figure 7

VERMONT I 91 ROAD STUDY - ASPHALT VISCOSITIES

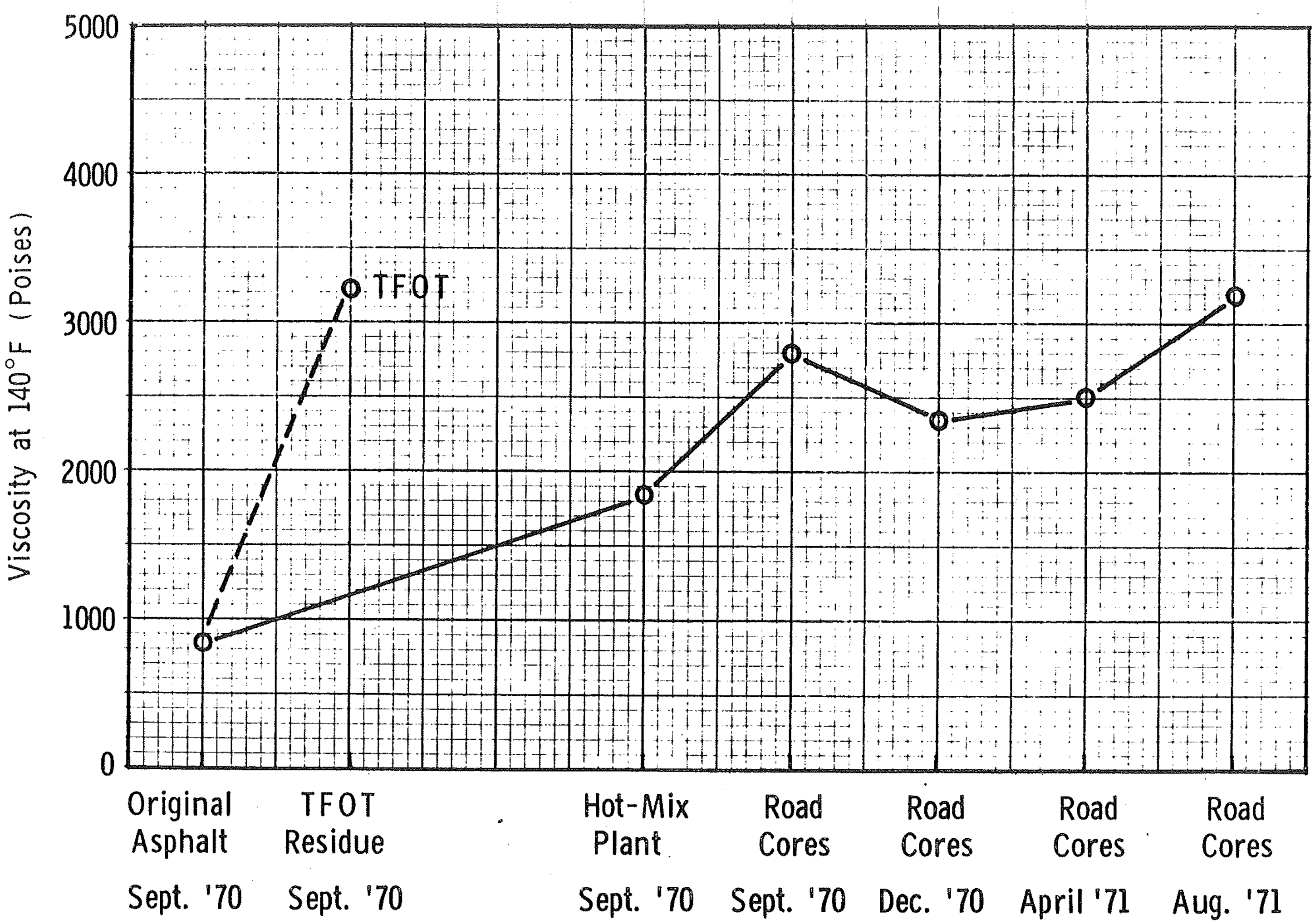


Figure 8

VERMONT I 91 ROAD STUDY - COMPOSITION OF ASPHALT
BY GRADIENT ELUTION CHROMOTOGRAPHY
MOBIL METHOD 1012

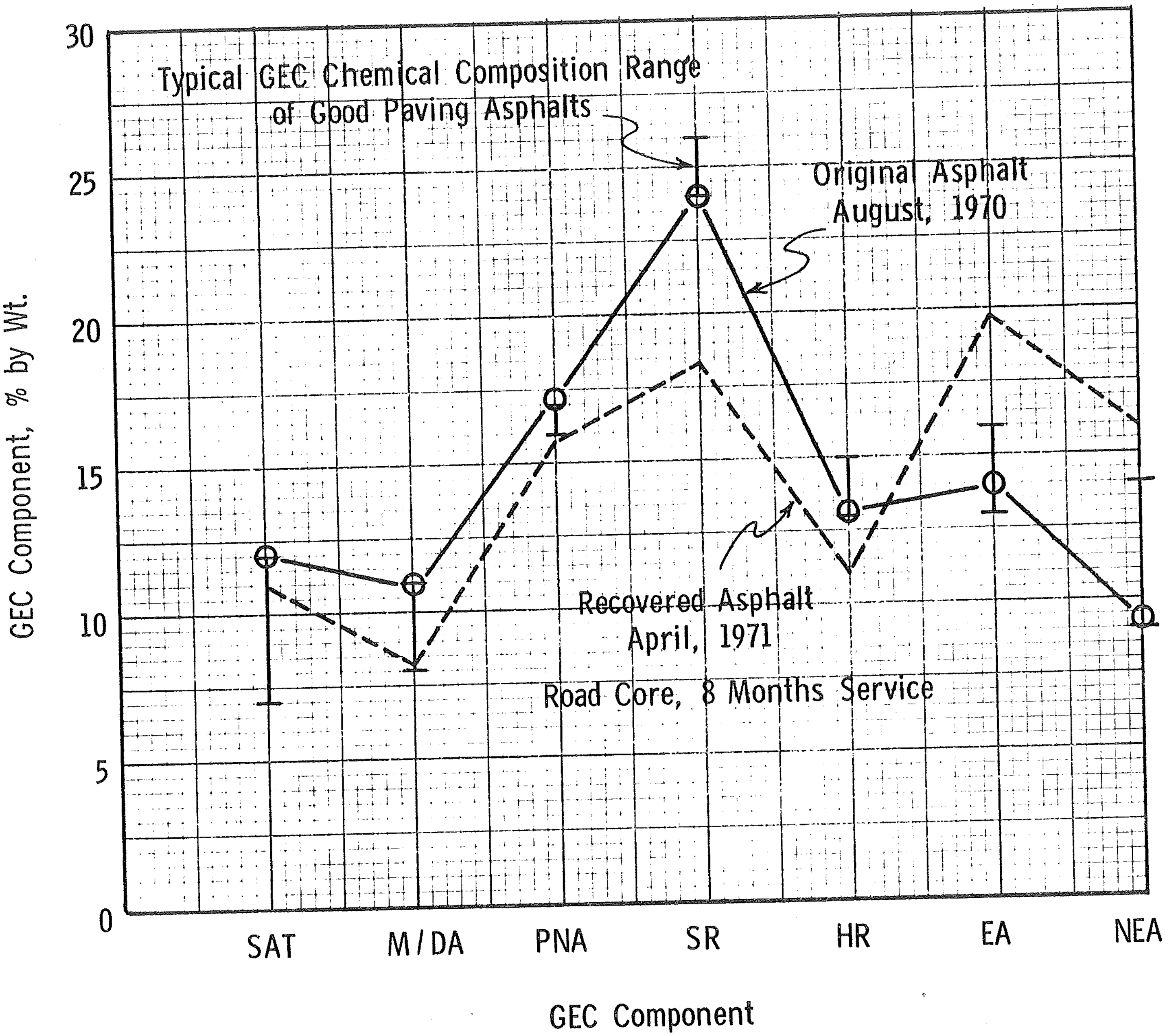


Figure 9

VERMONT I 91 ROAD STUDY -
CHANGES IN CHEMICAL COMPOSITION
EVALUATED BY GRADIENT ELUTION CHROMOTOGRAPHY
MOBIL METHOD 1012

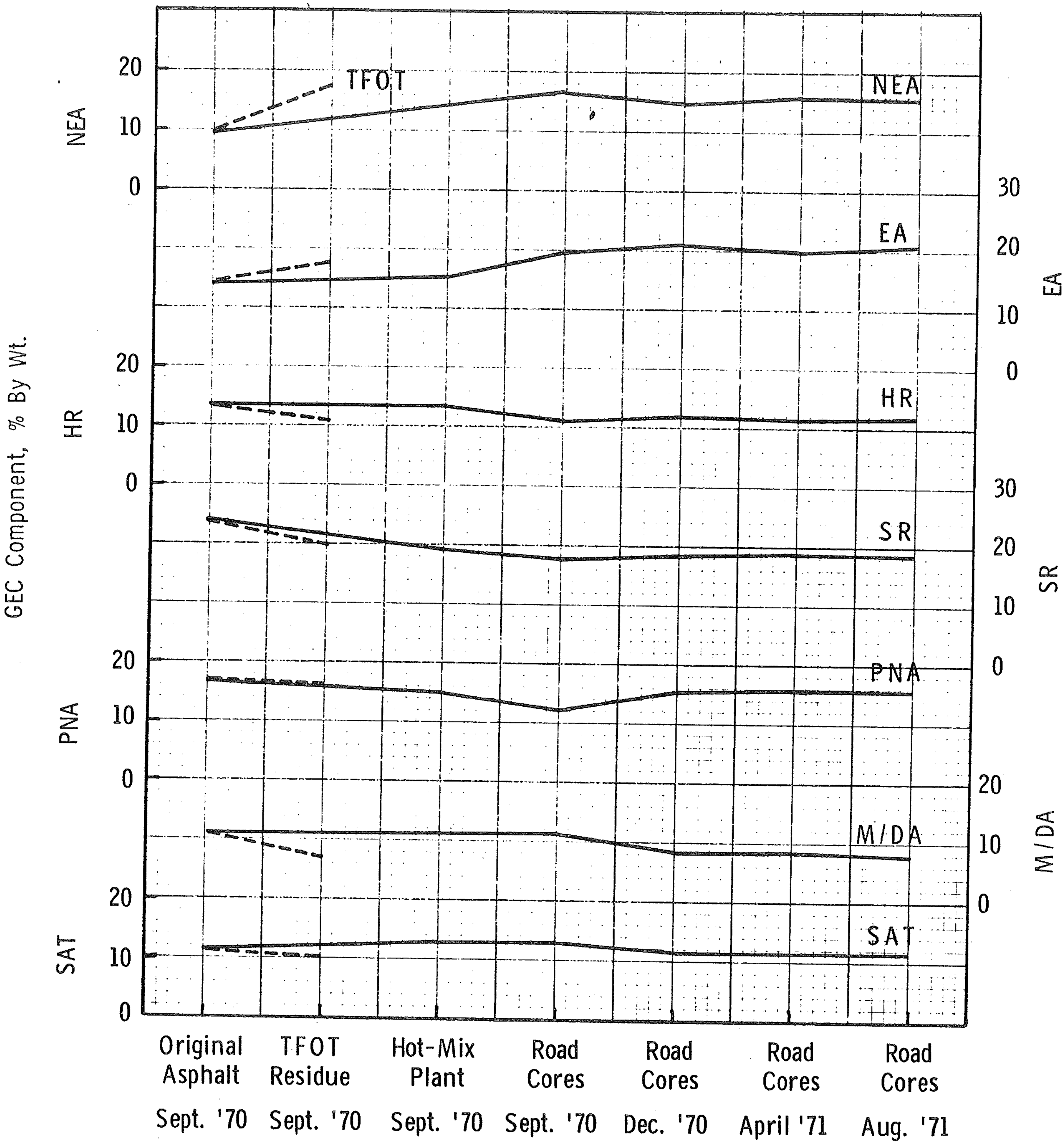


Figure 10

VERMONT I 91 ROAD STUDY –
CHANGES IN ASPHALTENE CONTENT BY
GRADIENT ELUTION CHROMATOGRAPHY
MOBIL METHOD 1012

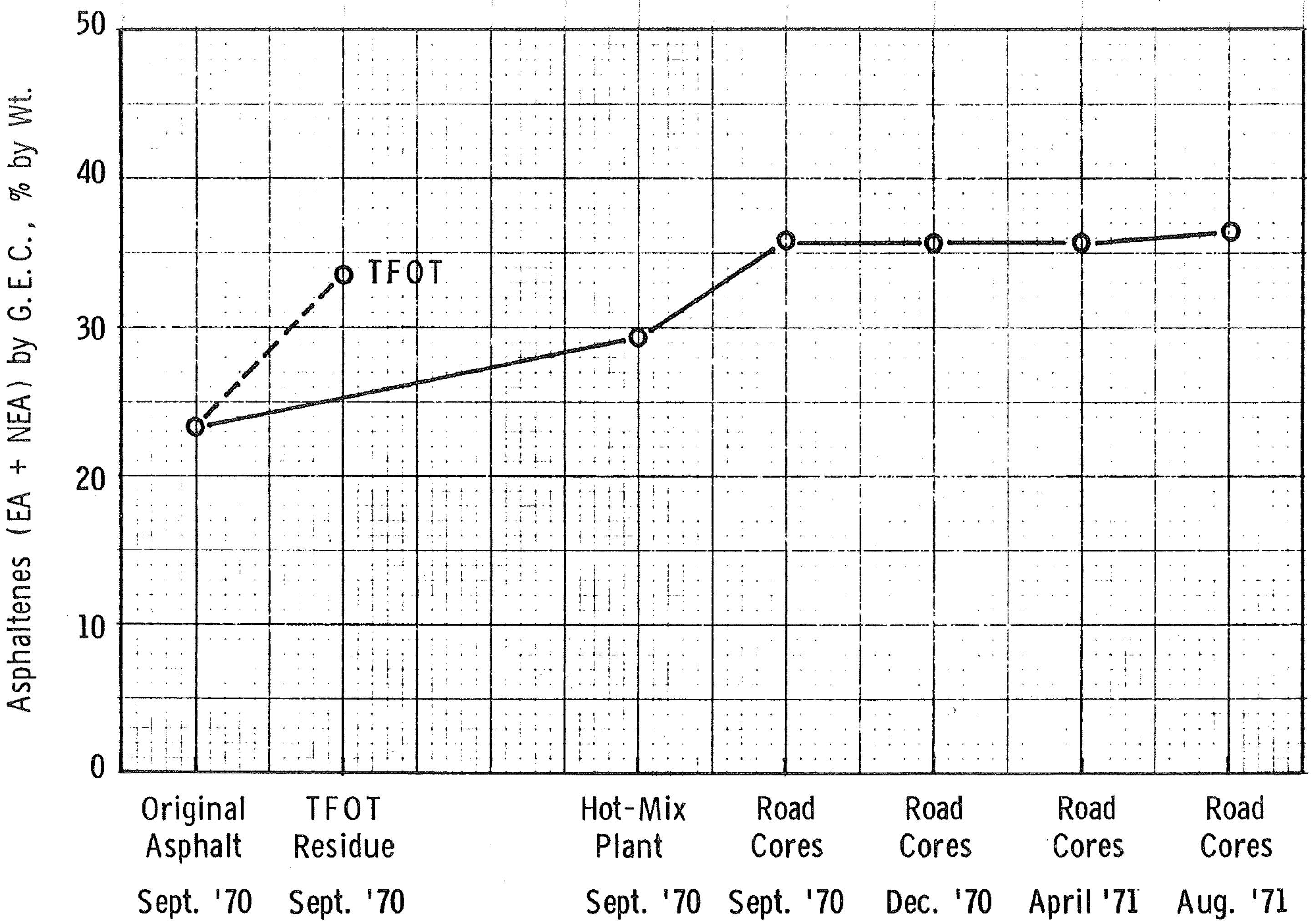


Figure 11

VERMONT I 91 ROAD STUDY -
CHANGES IN $I_{C=O}$ ABSORPTION BY INFRA-RED

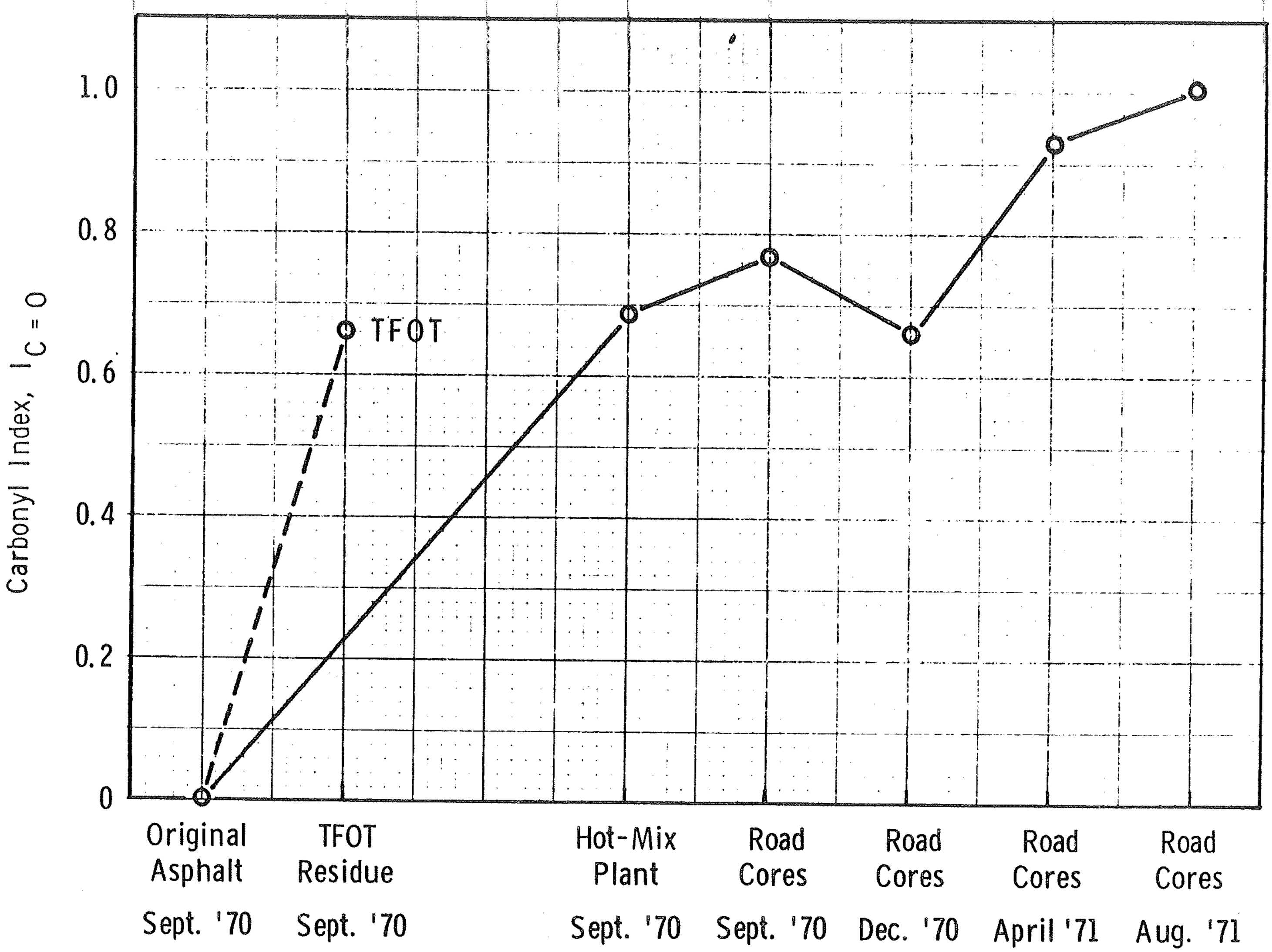


Figure 12

VERMONT I 91 ROAD STUDY -
CHANGES IN HYDROGEN ON ALPHA CARBON BY N.M.R.

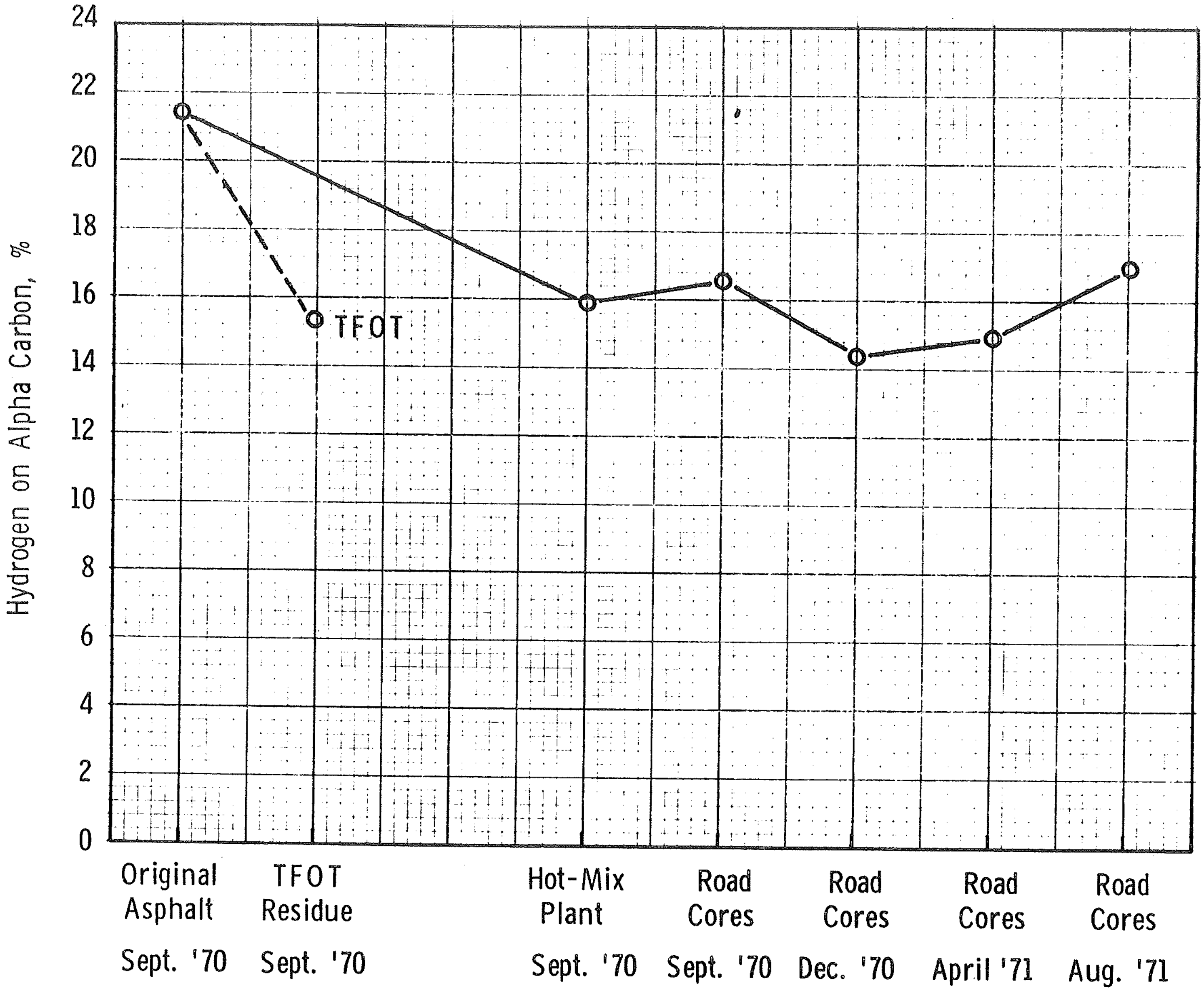


Figure 13

VERMONT I 91 ROAD STUDY - TRANSVERSE CRACKING FREQUENCY FEBRUARY, 1973

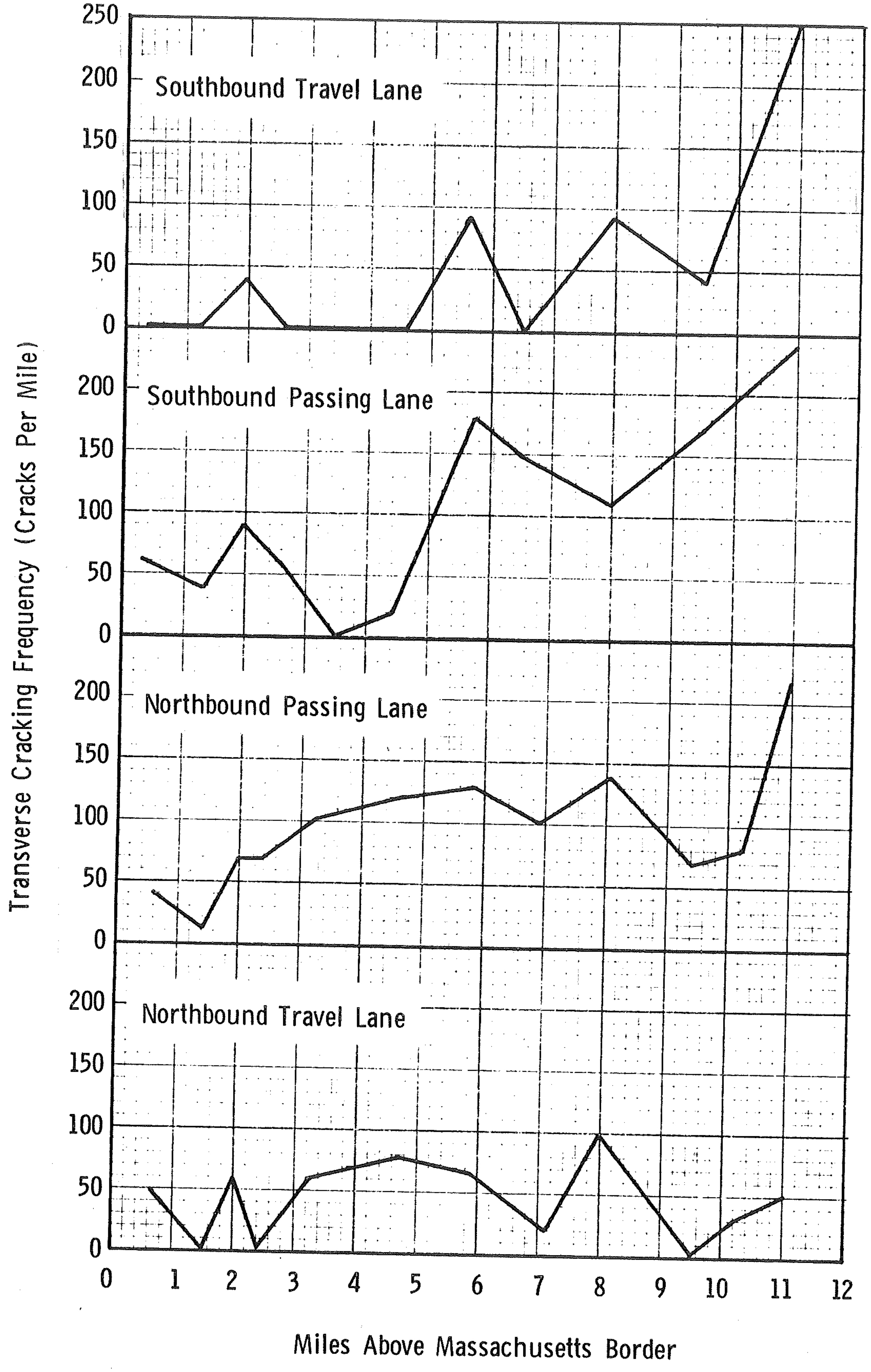


Figure 14

VERMONT I 91 ROAD STUDY
PICTURE OF ROAD SURFACE SHOWING TYPICAL
TRANSVERSE CRACKING PATTERN

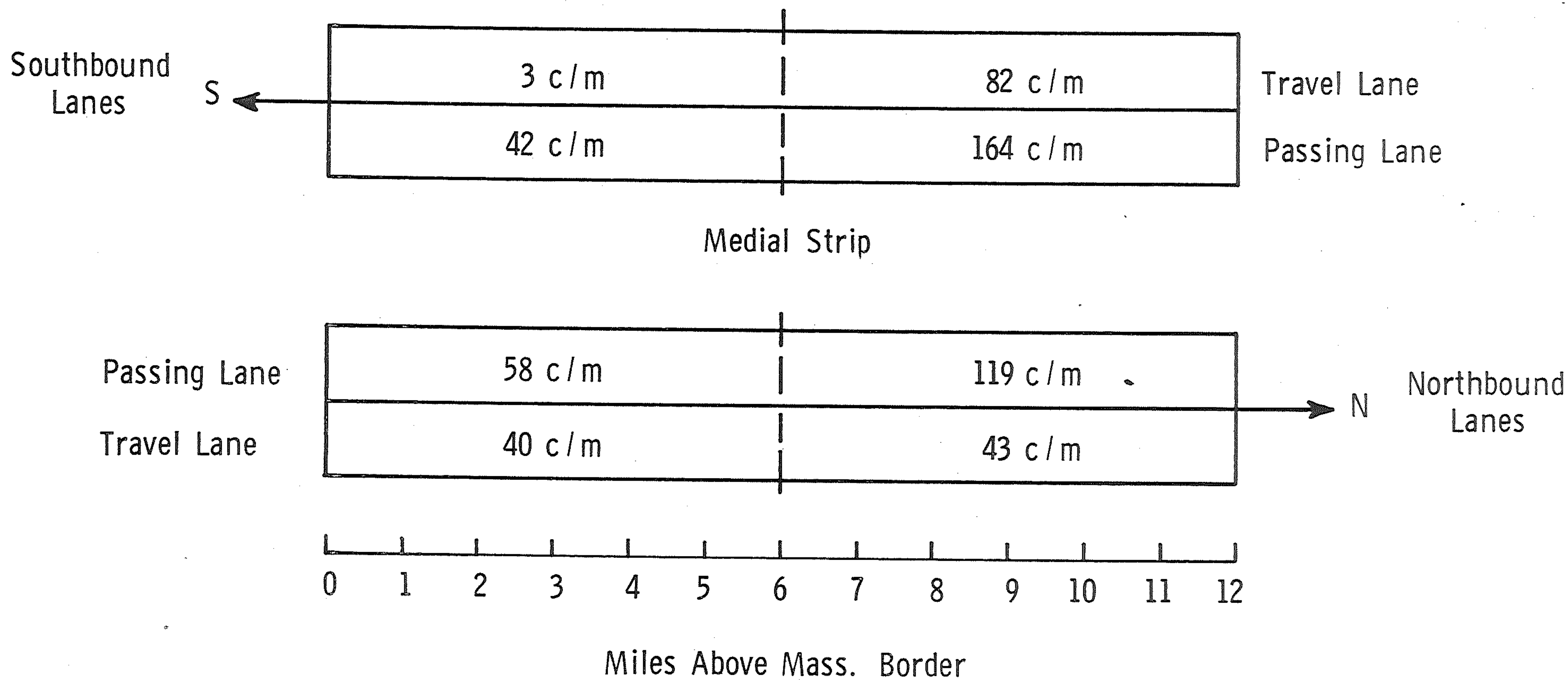
February 4, 1971 5°F ±

Guilford-Vernon-Brattleboro I 91-1 (37)

Picture taken in area of delineator 10/85 SB

Figure 15

VERMONT I 91 ROAD STUDY –
AREA ANALYSIS OF TRANSVERSE CRACKING FREQUENCY
APRIL 1971 INSPECTION BY VERMONT ENGINEERS



Total Road Average 69 c/m

Average 0-6 Miles 36 c/m

Average 6-12 Miles 102 c/m

Figure 16

VERMONT I 91 ROAD STUDY
PICTURE OF CORES SHOWING
TYPICAL REFLECTION CRACKING

GUILFORD-VERNON
BRATTLEBORO
I 91-1 (37)
REFLECTIVE
CRACKING

NB 0/55 SB 9/50

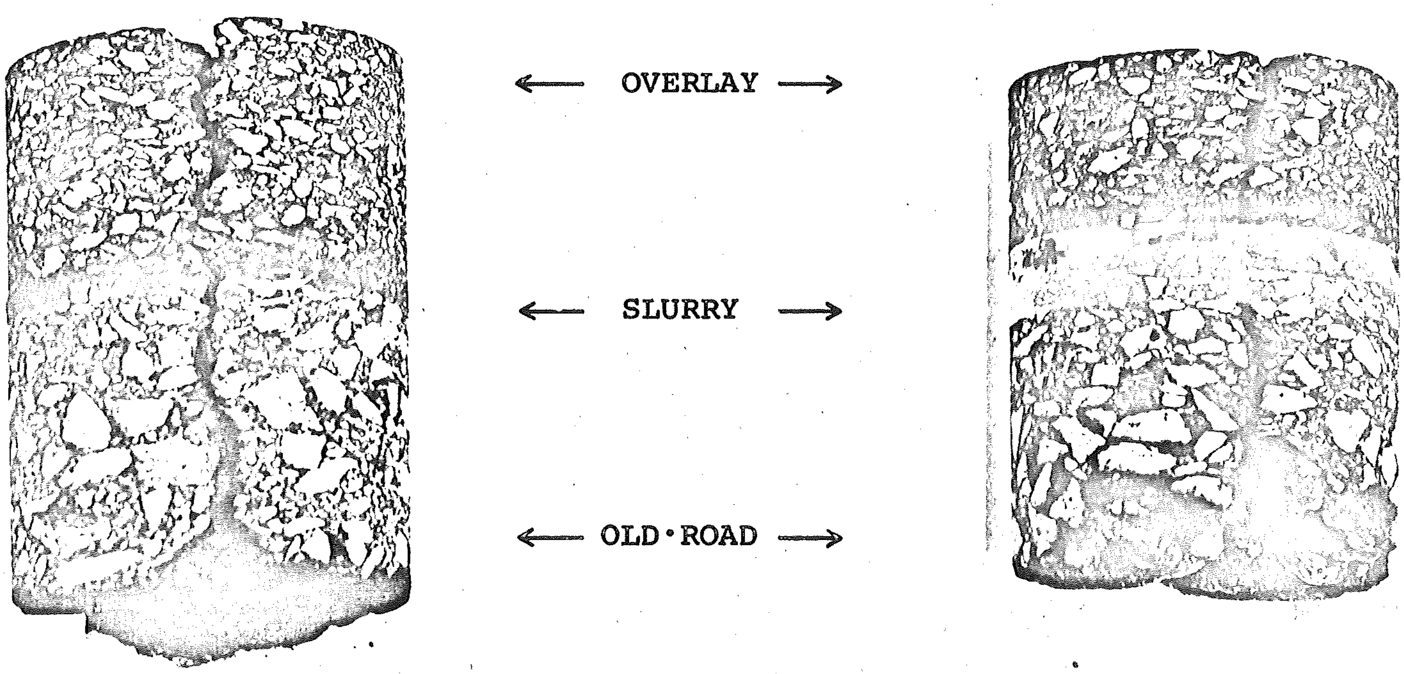


Figure 17

VERMONT I 91 ROAD STUDY – ANDERSON AND SHIELDS TRANSVERSE CRACKING FORMULA

(Data Plotted From Table 10)

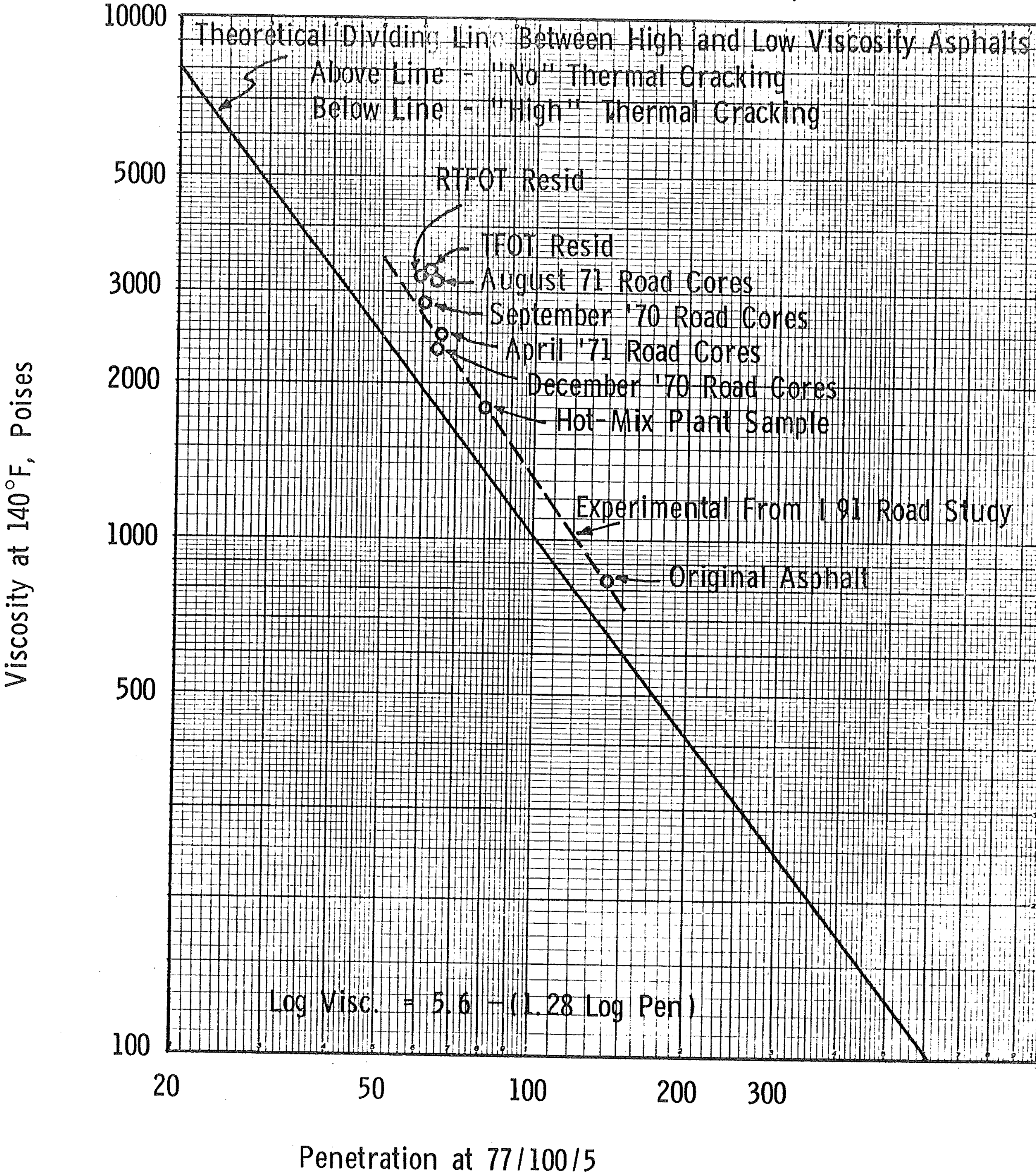


Figure 18

VERMONT I 91 ROAD STUDY -
CALCULATED MODULUS OF STIFFNESS OF ASPHALT
CONCRETE VERSUS TRANSVERSE CRACKS PER MILE

8-Yr Pavements - Canada Per McLeod

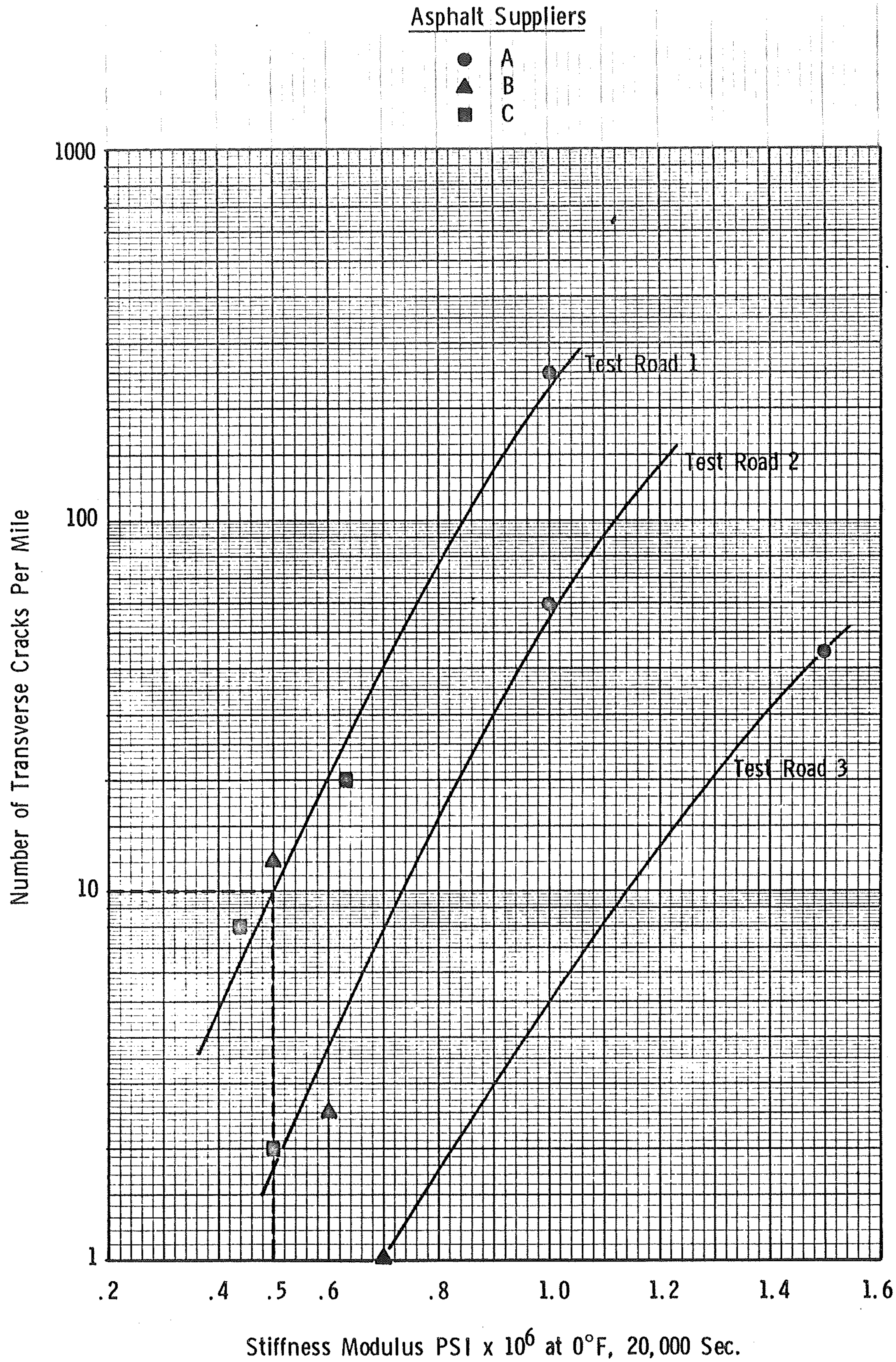


Figure 19

FREEZING INDEX MAP OF THE UNITED STATES

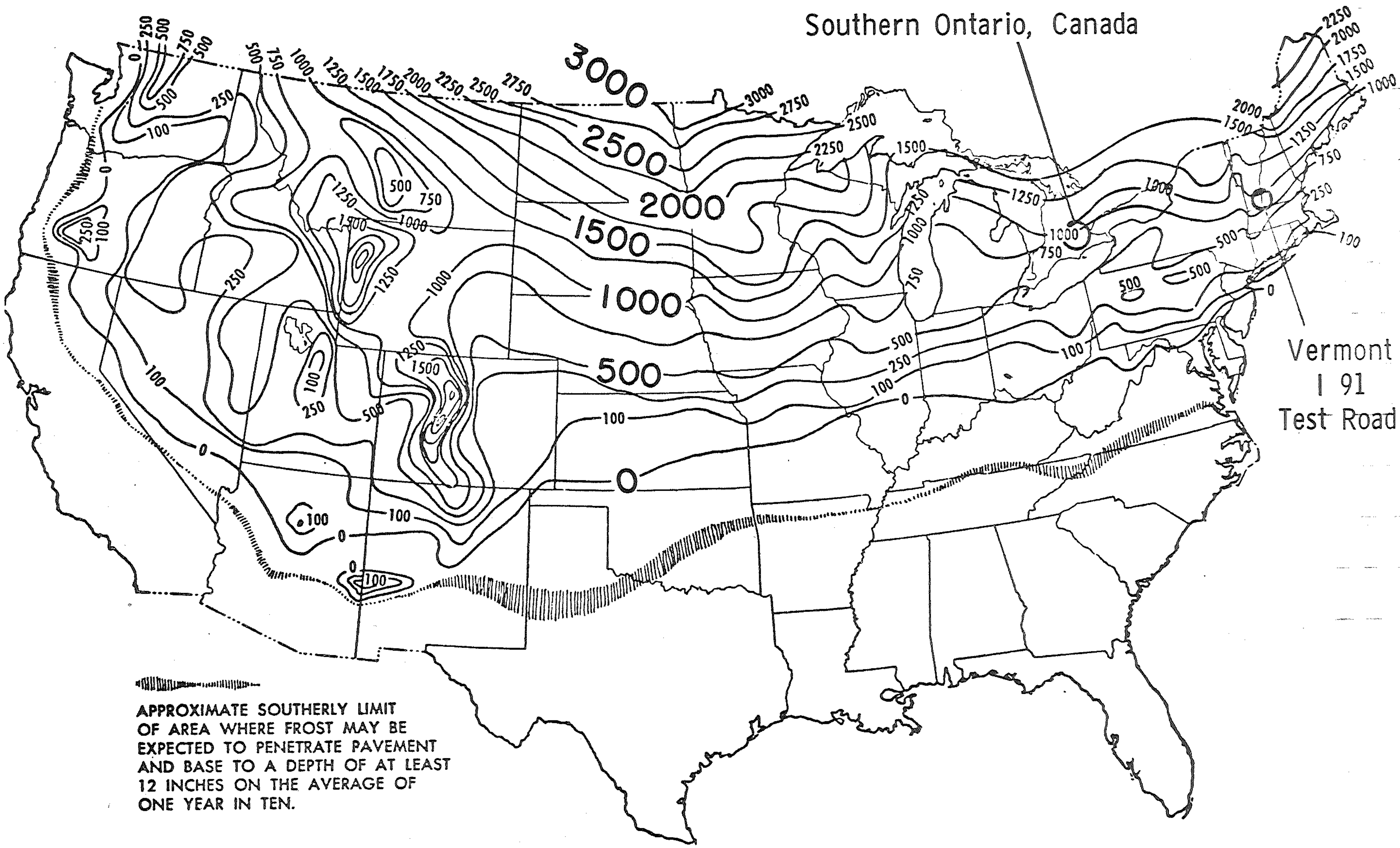


Figure 20

VERMONT I 91 ROAD STUDY - RELATIONSHIPS BETWEEN MODULI OF STIFFNESS OF ASPHALT CEMENTS AND OF PAVING MIXTURES CONTAINING THE SAME ASPHALT CEMENTS (Based on Heukelom and Klomp)

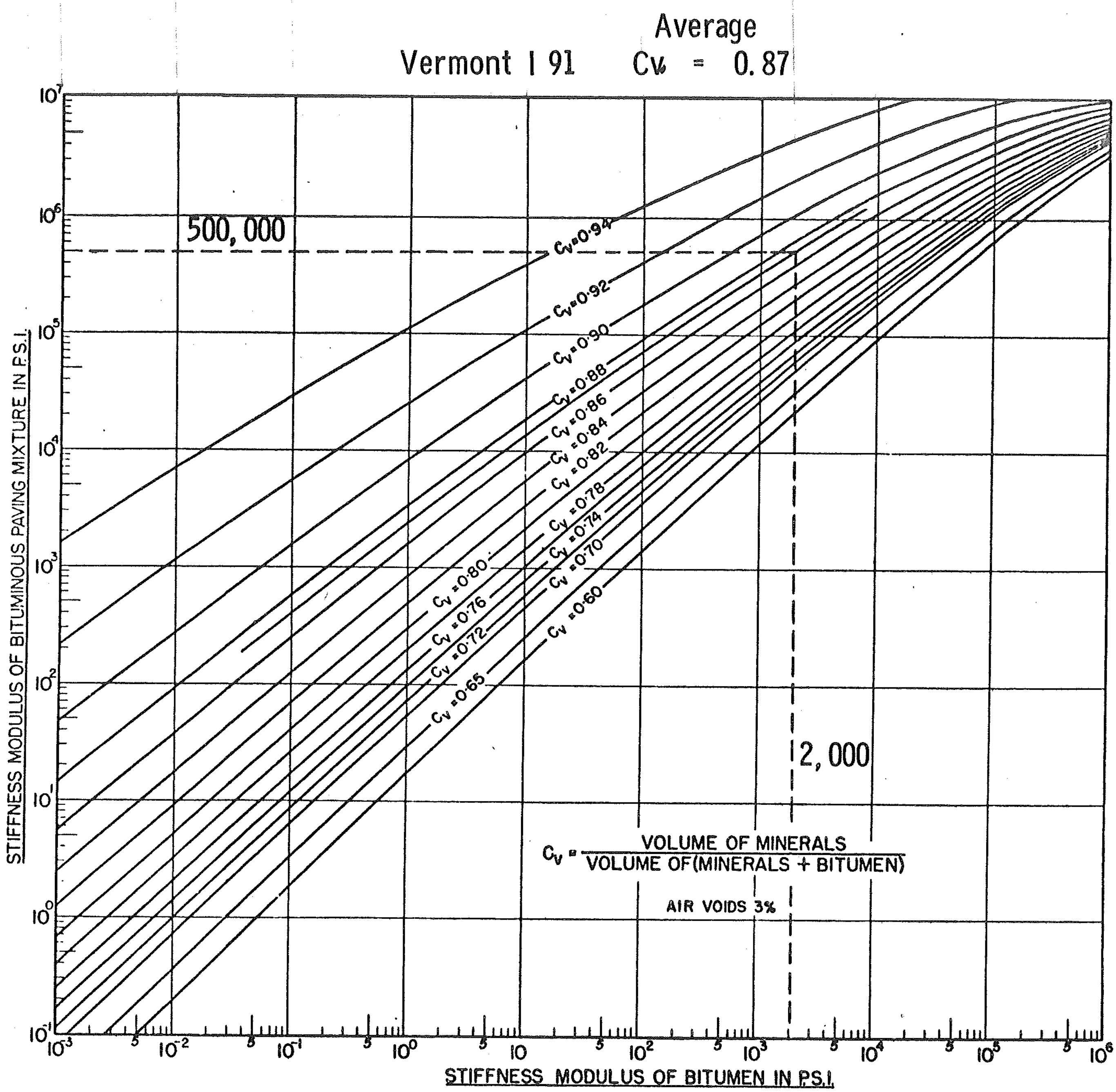
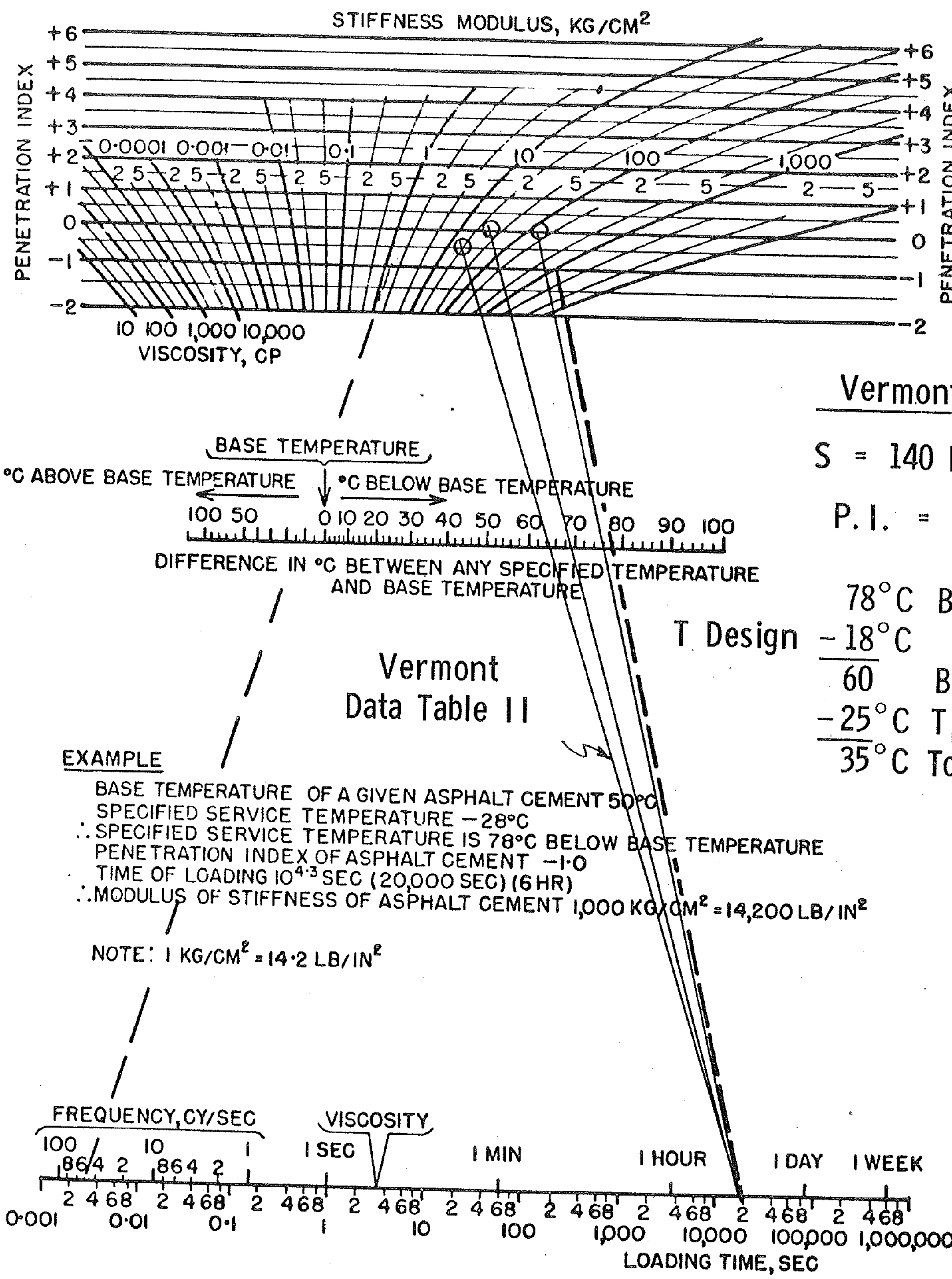


Figure 21

SUGGESTED MODIFICATION OF HEUKELOM'S AND KLOMP'S VERSION OF VAN DER POEL'S NOMOGRAPH FOR DETERMINING MODULUS OF STIFFNESS OF ASPHALT CEMENTS (AFTER McLEOD)



Vermont Data

S = 140 Kg/cm²

P.I. = 0.0

78°C Below Base Temp.

T Design −18°C

60 Base $T_{R\&B}$

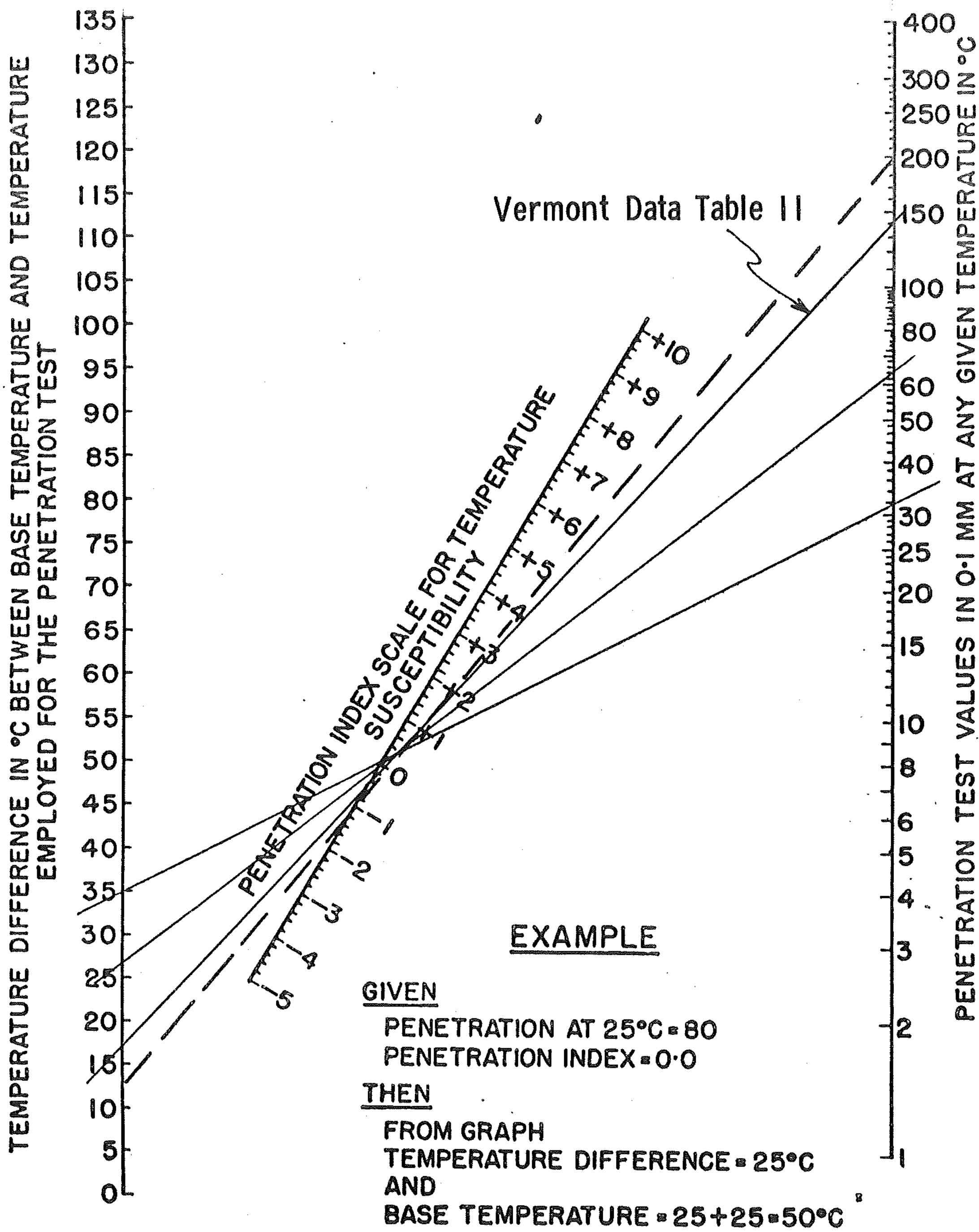
−25°C T_{Pen}

35°C To Figure 22

Figure 22

SUGGESTED MODIFICATION
OF HEUKELOM'S VERSION OF PFEIFFER'S AND VAN DOORMAL'S
NOMOGRAPH FOR RELATIONSHIP BETWEEN PENETRATION,
PENETRATION INDEX AND BASE TEMPERATURE (AFTER McLEOD)

$T_{Dif} = T_{R\&B} - T_{Pen}\,°C$



Vermont Data

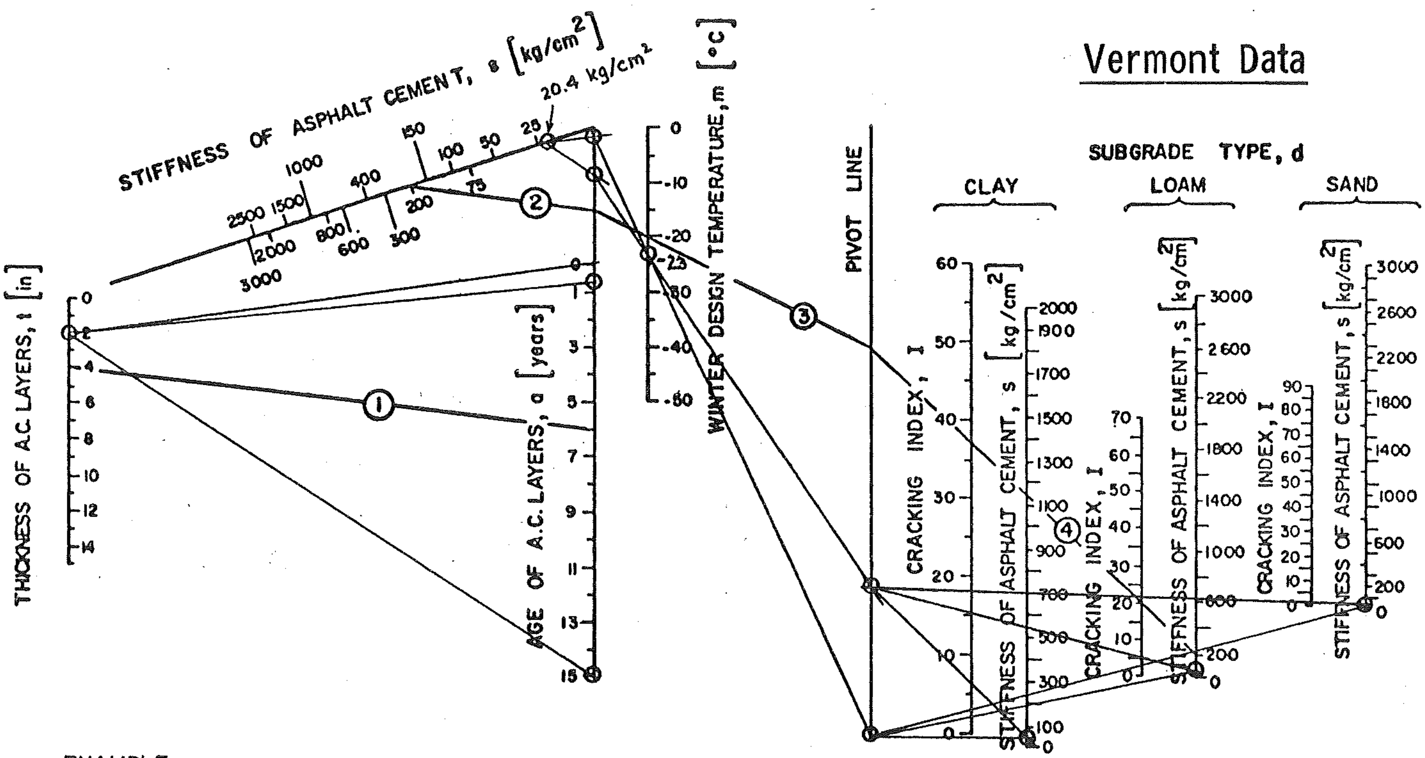
P.I. = 0.0

27° T_{Dif}
+ 25° T_{Pen}
52°
+ 18° T_{Design}
70°C To Figure 21

Figure 23

VERMONT I 91 ROAD STUDY – NOMOGRAPH FOR PREDICTING LOW-TEMPERATURE CRACKING FREQUENCY OF ASPHALT PAVEMENTS

MODEL : $10^{I} = 2.4970 \times 10^{30} \cdot s^{(6.79660 - 0.87403t + 1.33884a)} \cdot (7.0539 \times 10^{-3})^{d} \cdot (3.1928 \times 10^{-13})^{m} \cdot d^{0.60263s}$



EXAMPLE:

thickness = 4 inches
age = 6 years
stiffness of original asphalt cement = 200 kg/cm²*
winter design temperature = -20°C
subgrade type = loam
*for temp. = m and time = 20000 sec.

NOTE:

A) lines ① and ② are parallels
B) in step ④ select scales for the appropriate subgrade type

RESULT: cracking index = 20 at 6 years

Figure 24

VERMONT I 91 ROAD STUDY – RELATIONSHIP BETWEEN FREEZING INDEX AND WINTER DESIGN TEMPERATURE

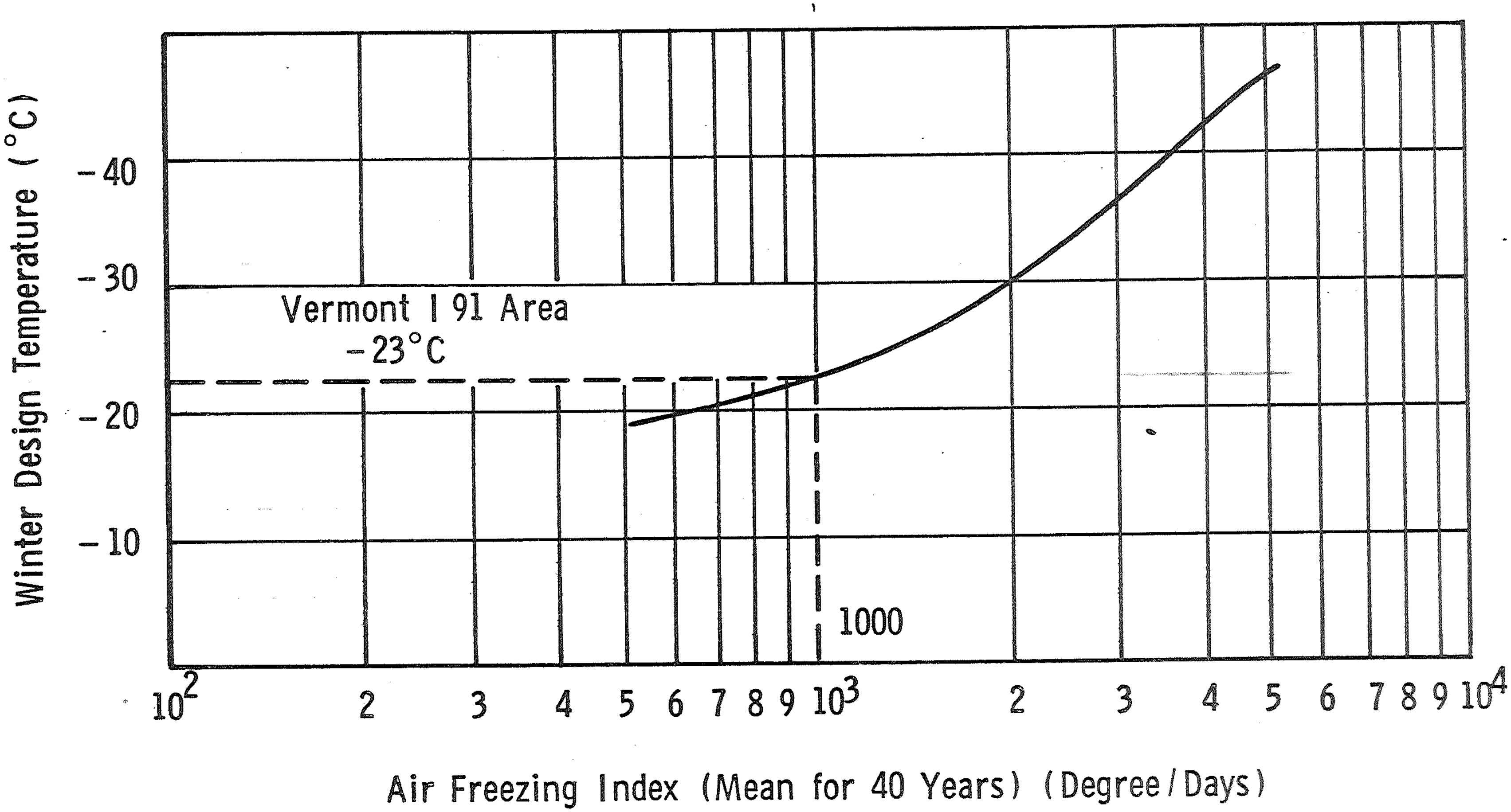


Figure 25

VERMONT I 91 ROAD STUDY -
SUGGESTED MODIFICATION OF
McLEOD'S GRAPH FOR ESTIMATION OF P.I.

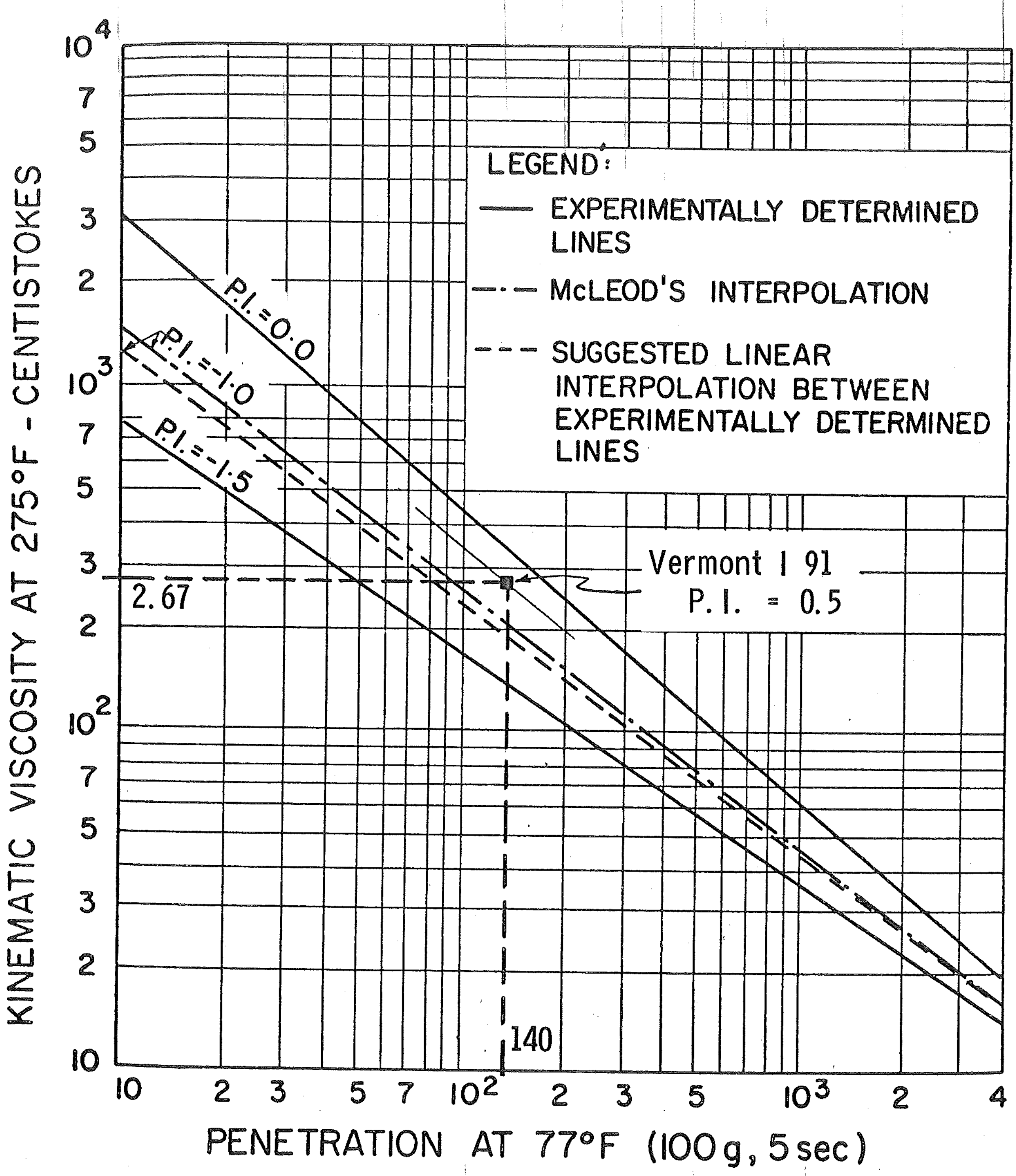


Figure 26A

AXIAL CREEP STRESS AND STRAIN VS. TIME RELATIONS FOR VISCOELASTIC BURGERS MODEL

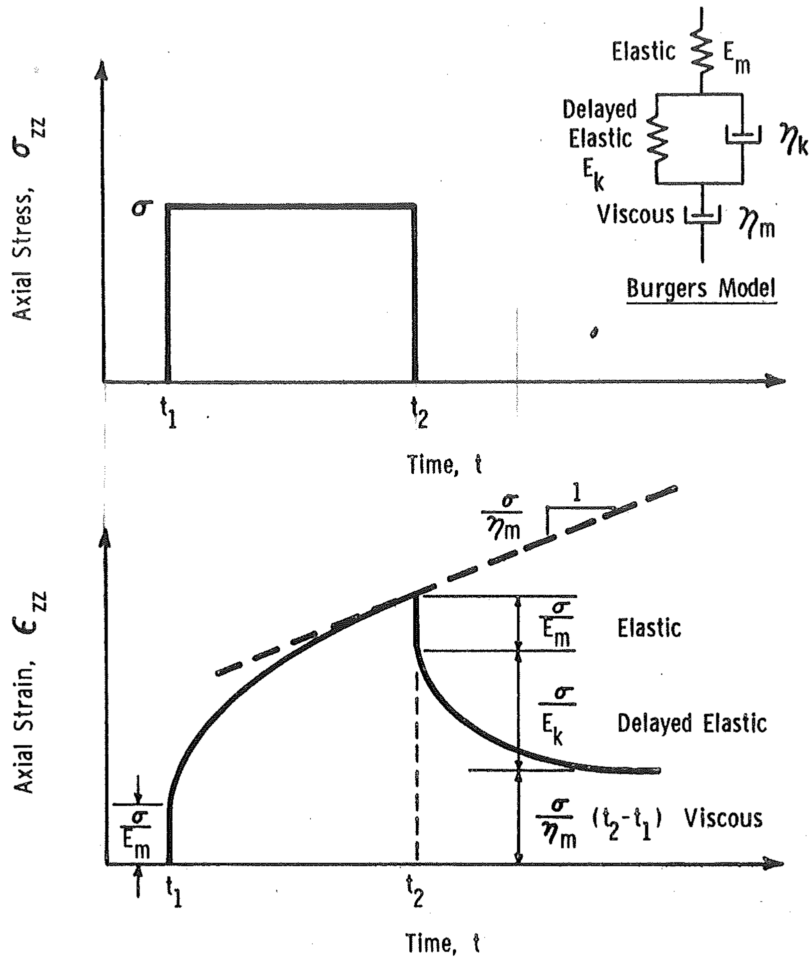


Figure 26B

TYPICAL MECHANICAL CONDITIONING--AXIAL CREEP STRAIN UNDER REPEATED LOADING FOR ASPHALTIC CONCRETE FIELD CORE

(BATCH B-3 AT 77°F)

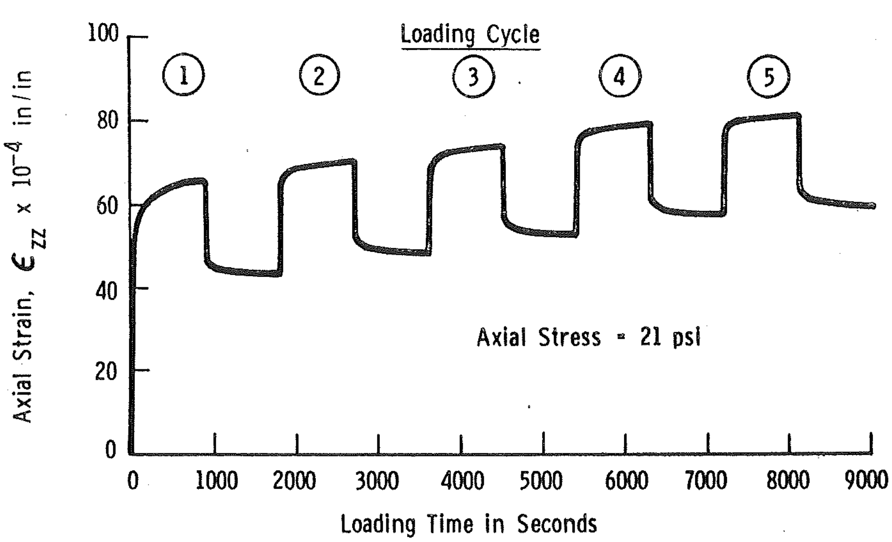


Figure 27

Typical Linearity Test at 77°F: Axial Strain Versus Load

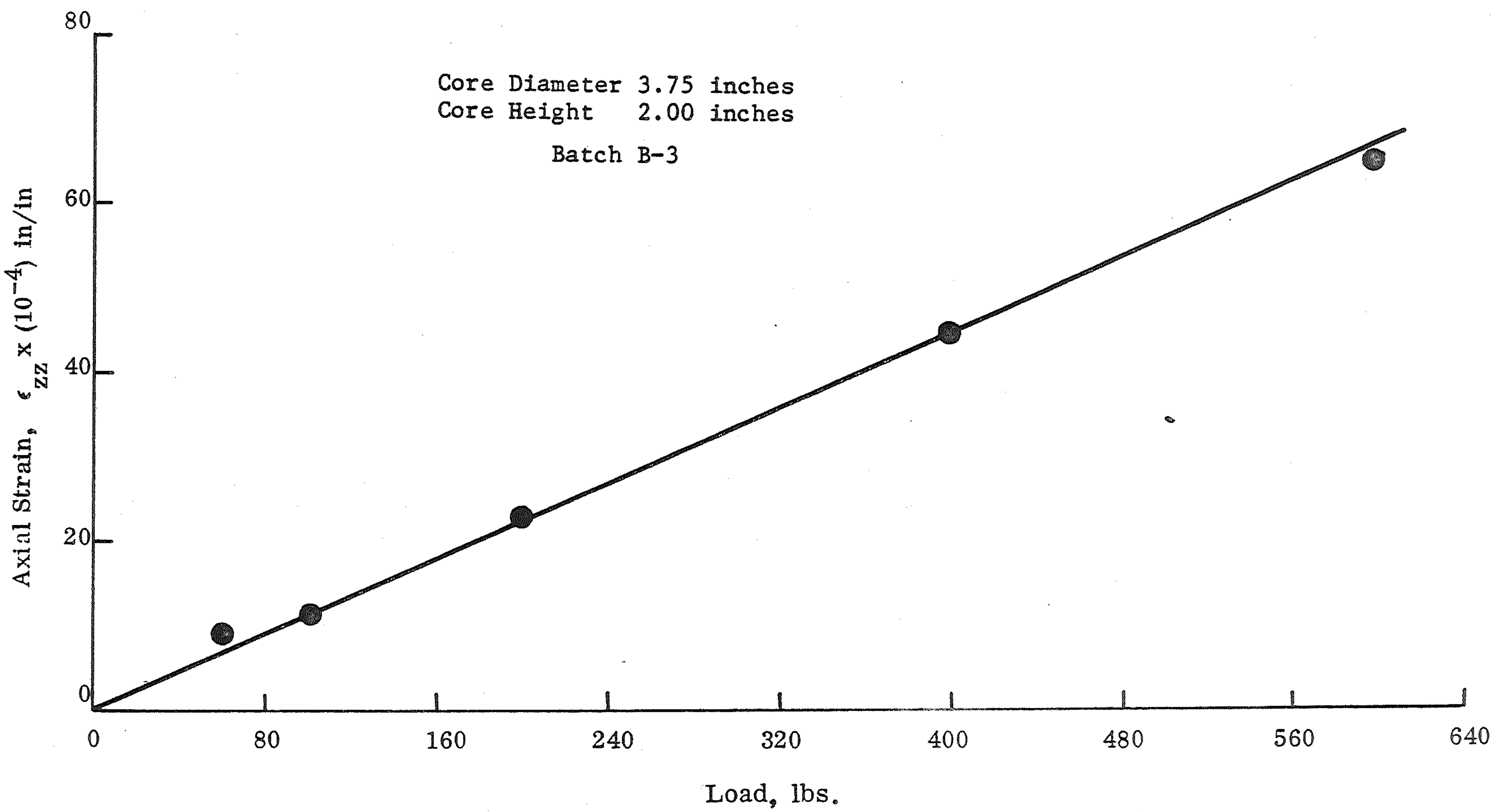


Figure 28

Creep Compliance Versus Loading Time in Seconds for Asphaltic Concrete Field Cores at 77°F

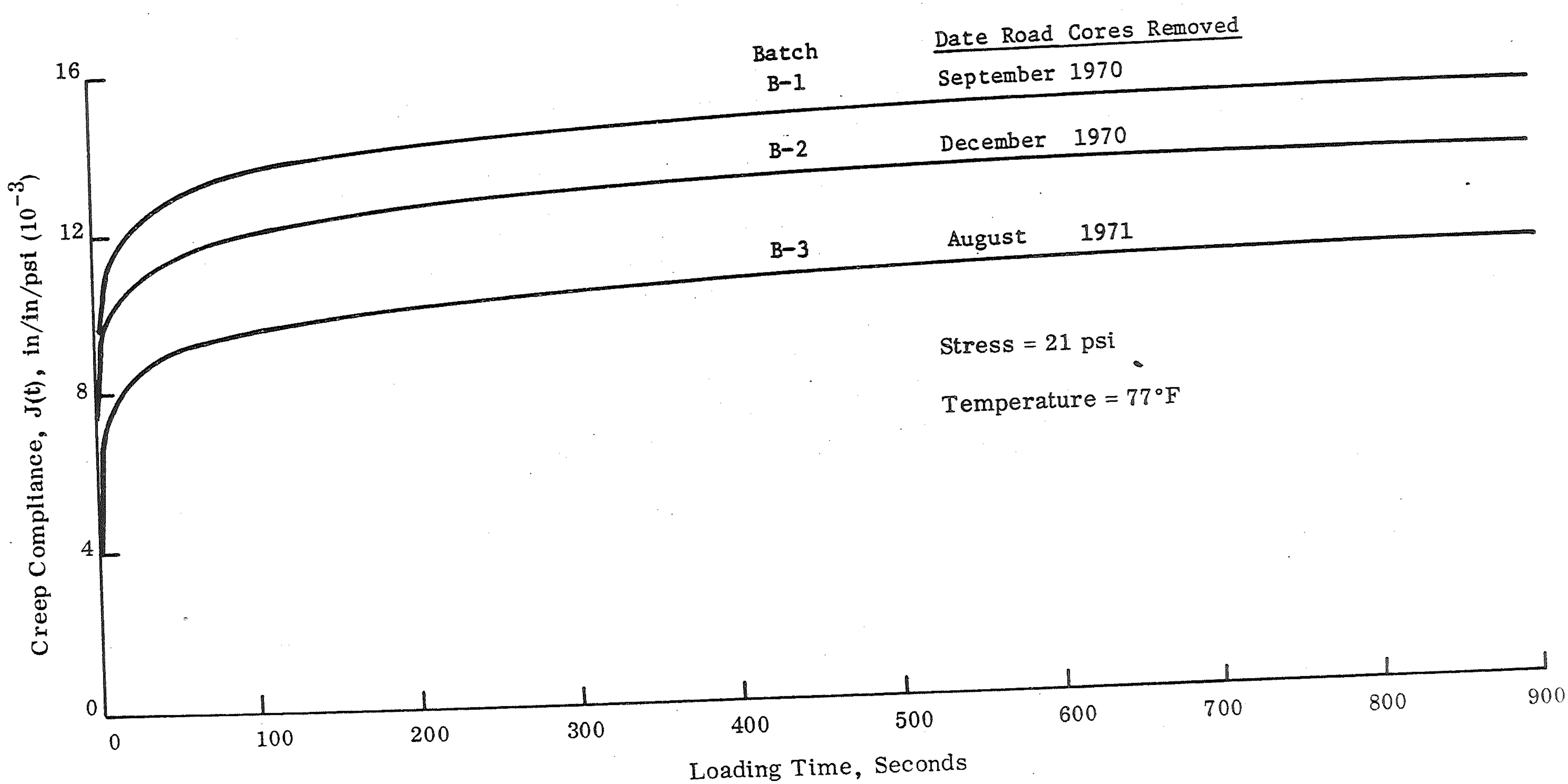


Figure 29

Creep Compliance Versus Loading Time in Seconds for Asphaltic Concrete Field Cores at 104°F

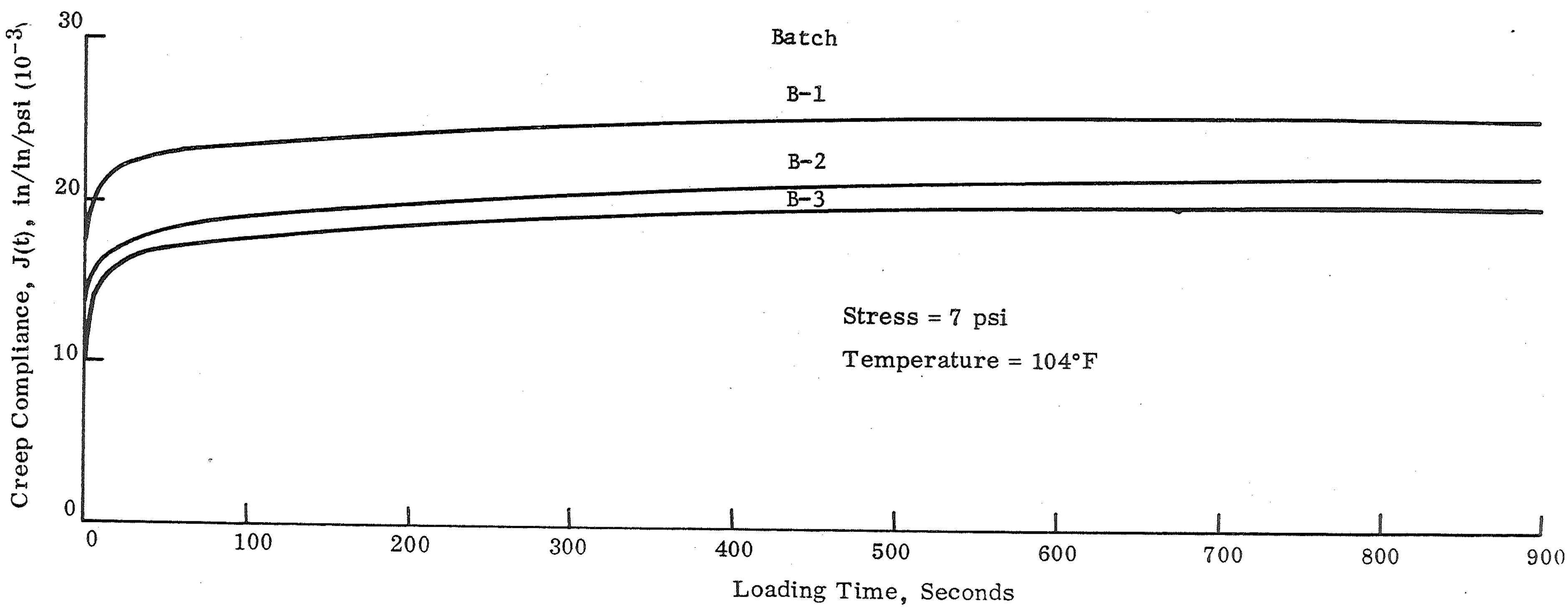


Figure 30

Creep Compliance Versus Loading Time for Asphaltic Concrete Field Cores at 41°F

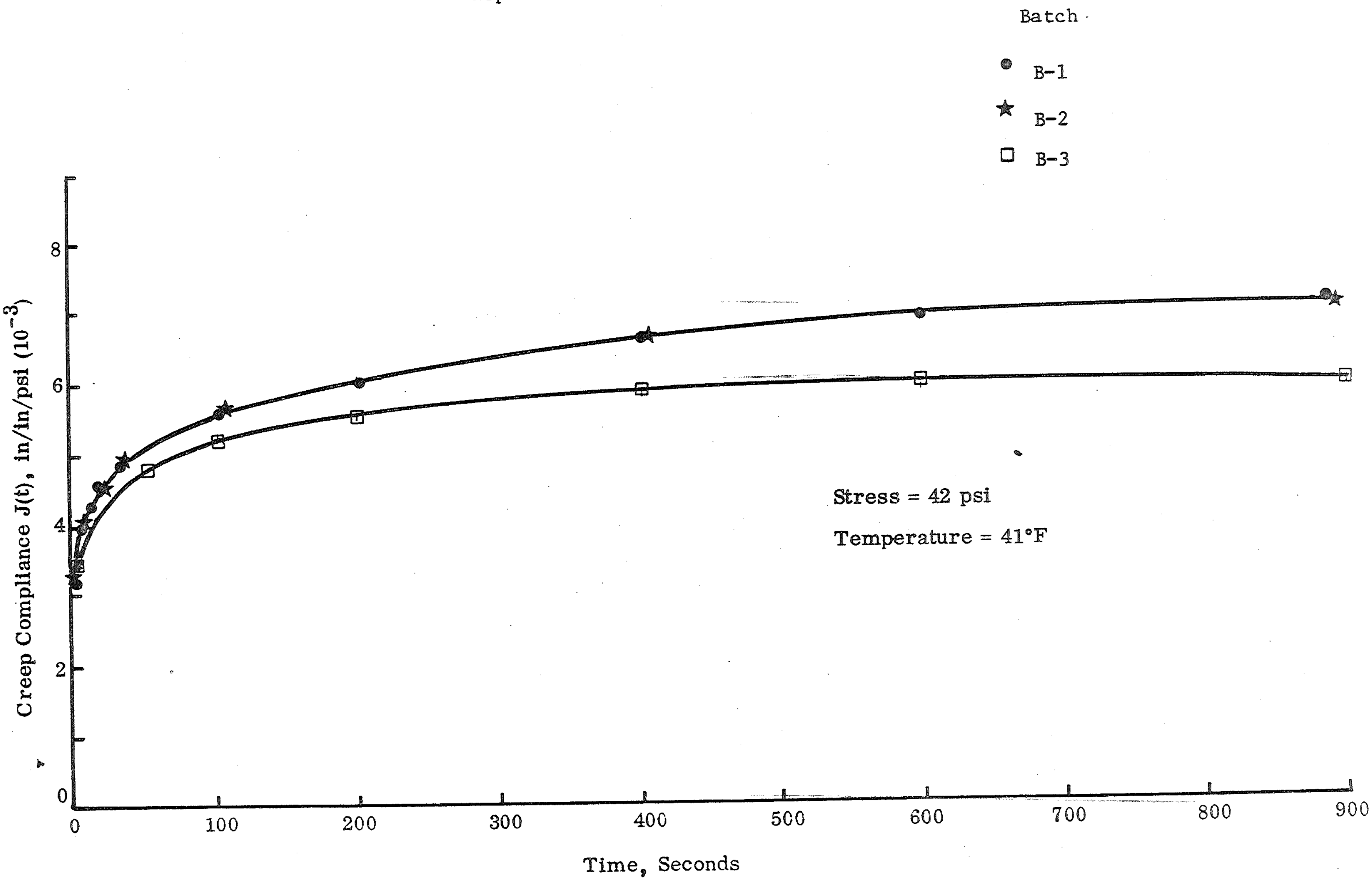


Figure 31

Creep Modulus Versus Time of Asphaltic Concrete Field Cores (Batch B-1) at Different Temperatures

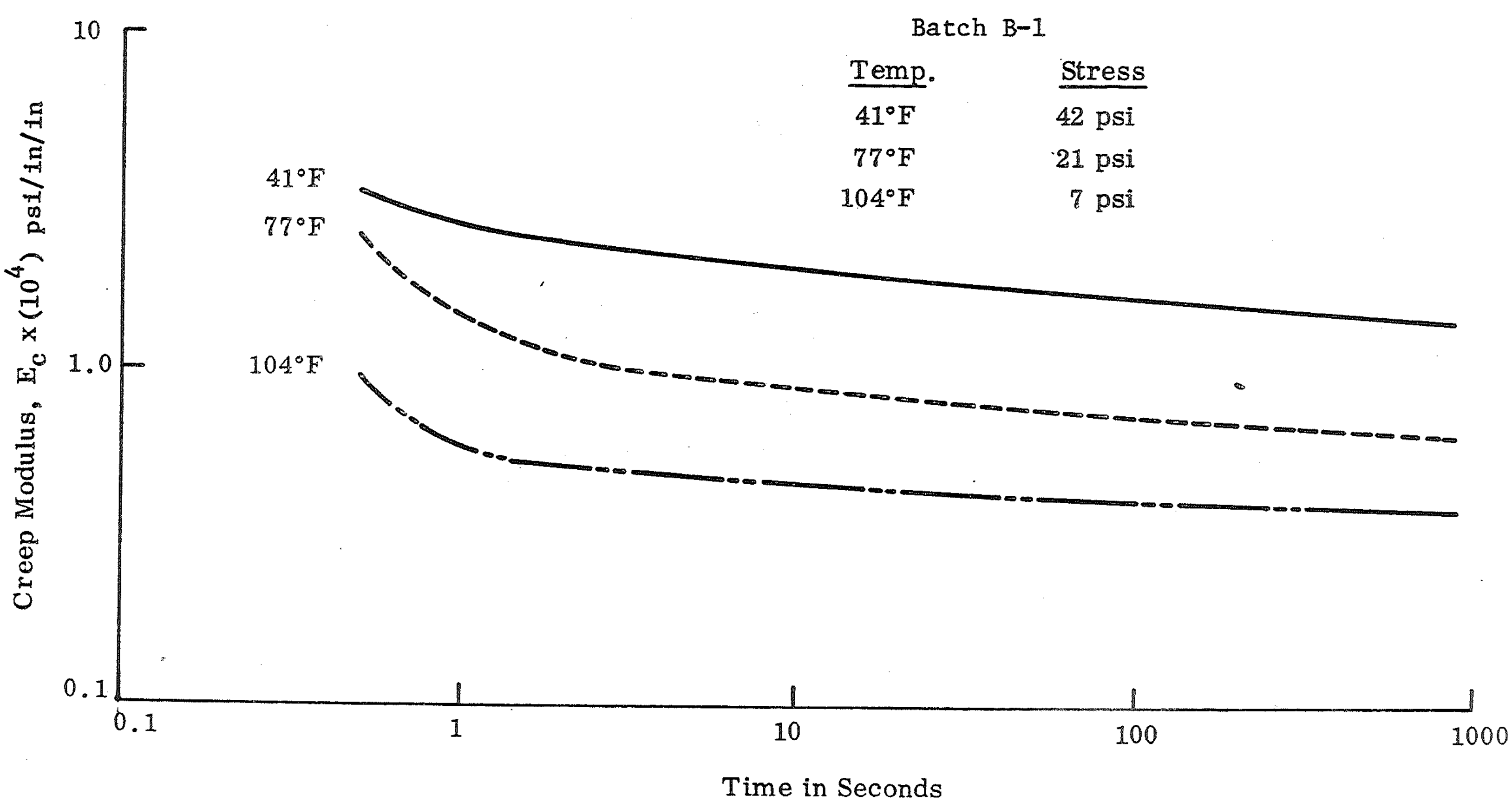


Figure 32

Creep Modulus Versus Time for Asphaltic Concrete Field Cores (Batch B-2) at Different Temperatures

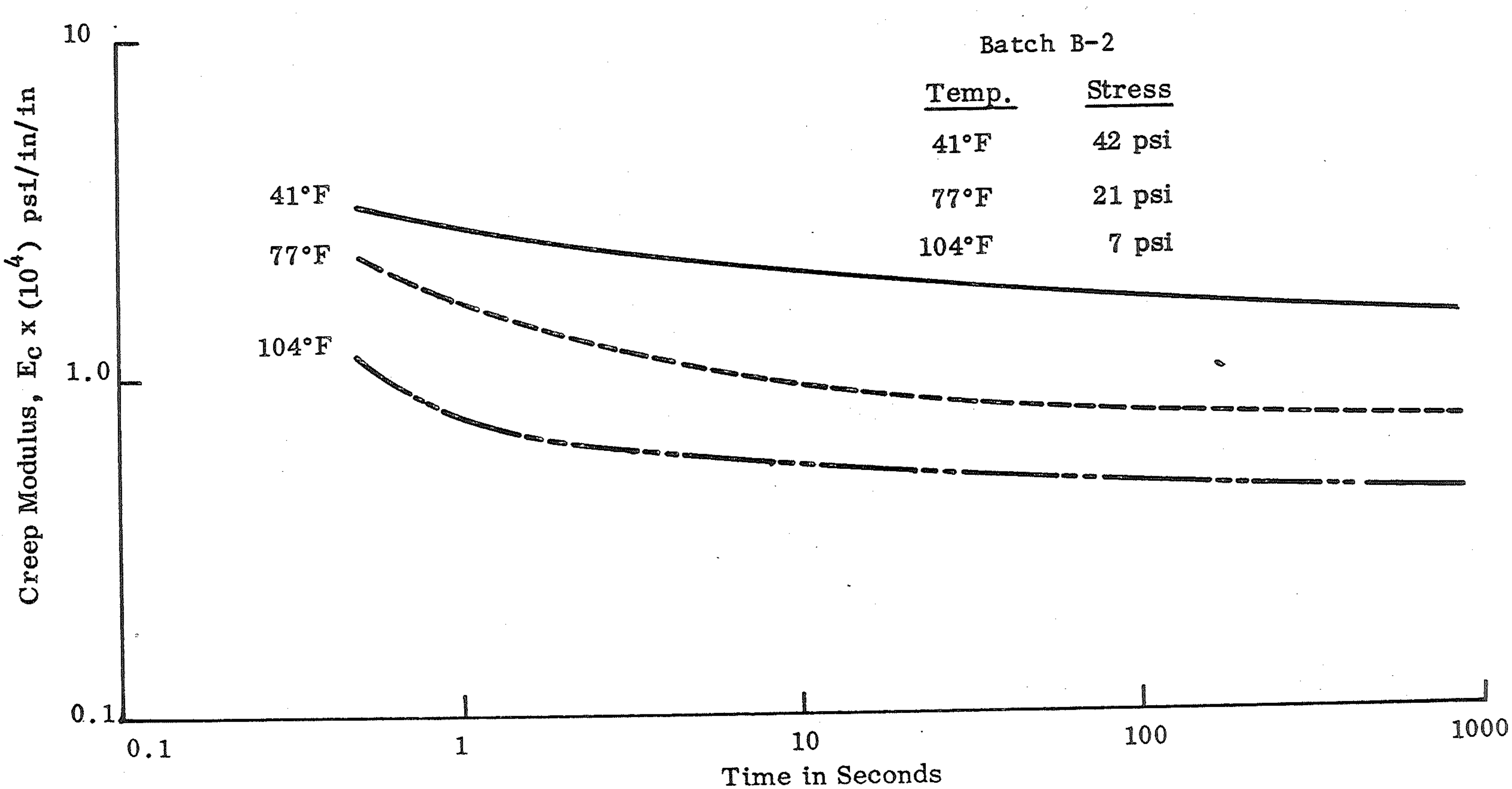


Figure 33

Creep Modulus Versus Time for Asphaltic Concrete Field Cores (Batch B-3) at Different Temperatures

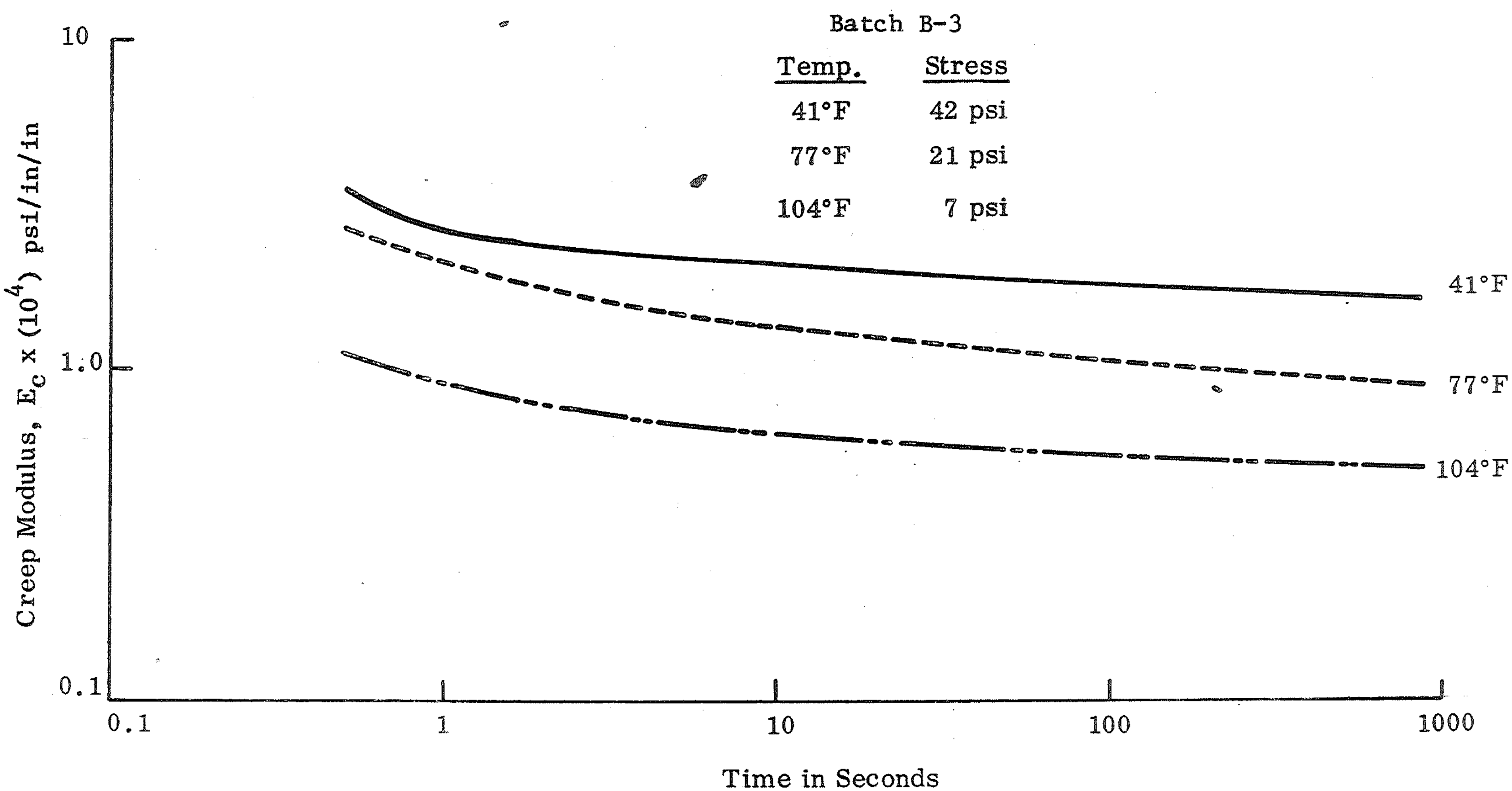


Figure 34

Creep Modulus Versus Time of Asphaltic Concrete Field Cores (Batch B-1) at Different Temperatures

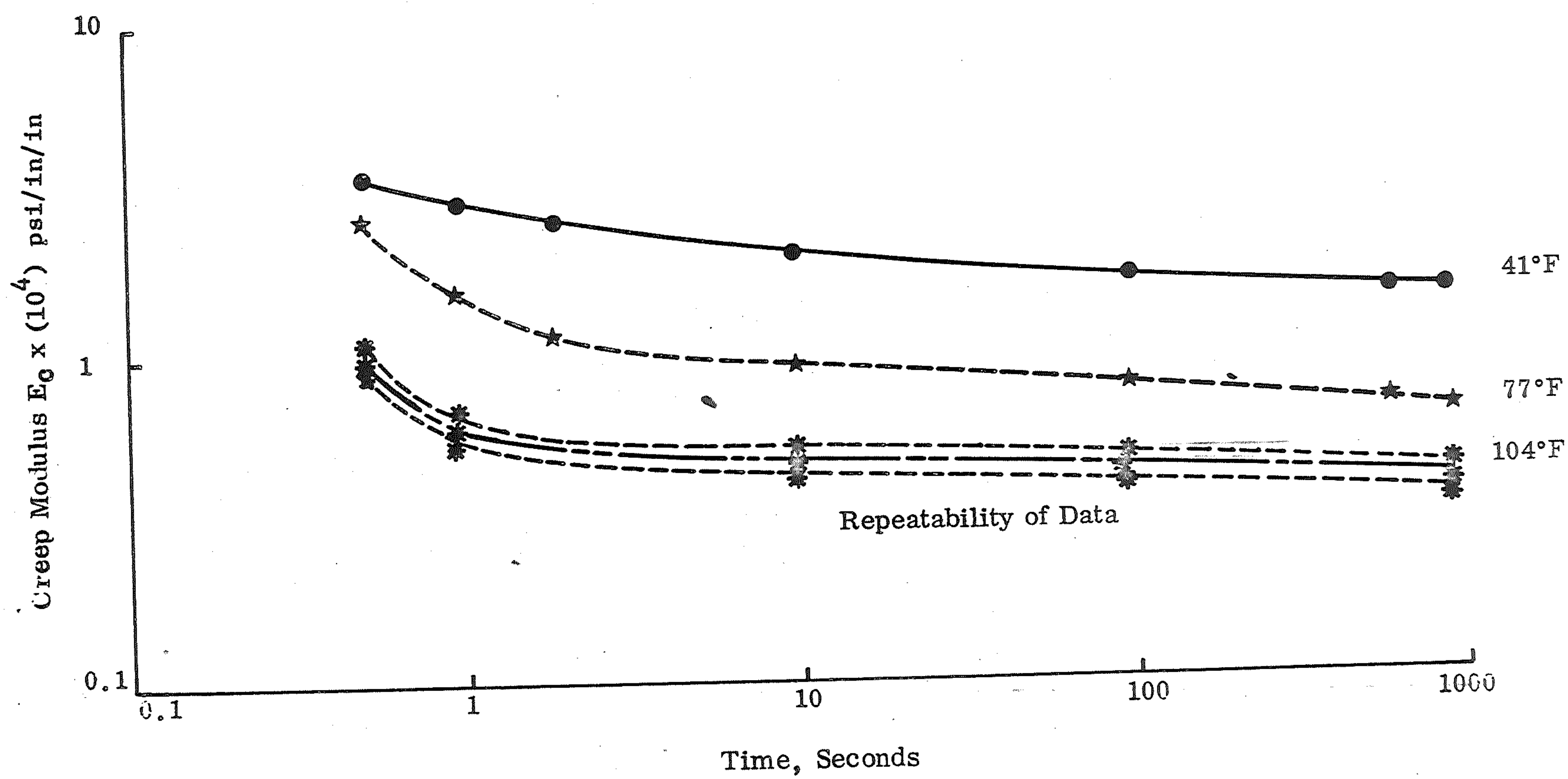


Figure 35

Creep Modulus Versus Time for Asphaltic Concrete Field Cores (Batch B-2) at Different Temperatures

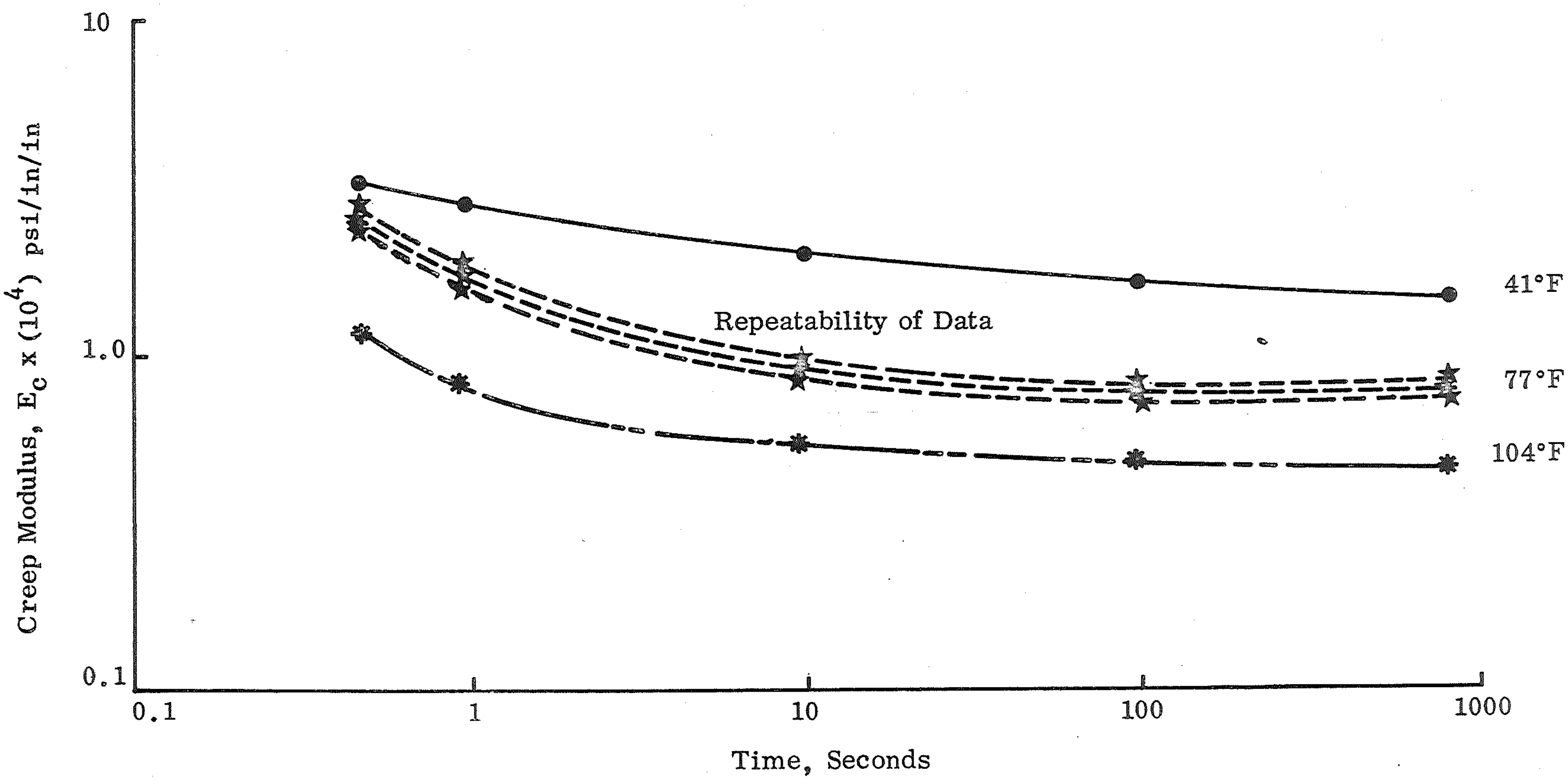
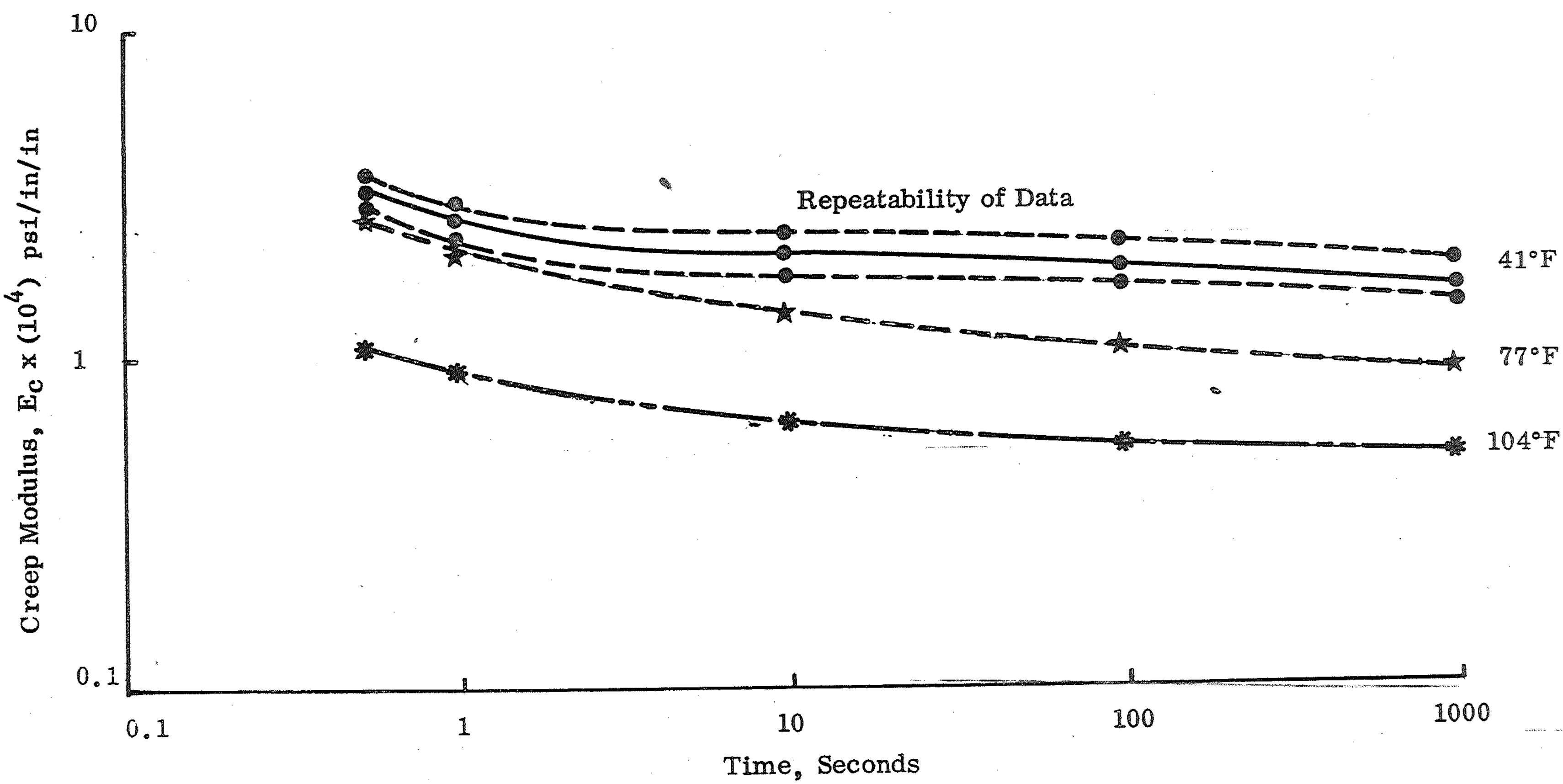


Figure 36

Creep Modulus Versus Time for Asphaltic Concrete Field Cores (Batch B-3) at Different Temperatures



84

Figure 37

Long-Term Creep Deformation of Field Cores

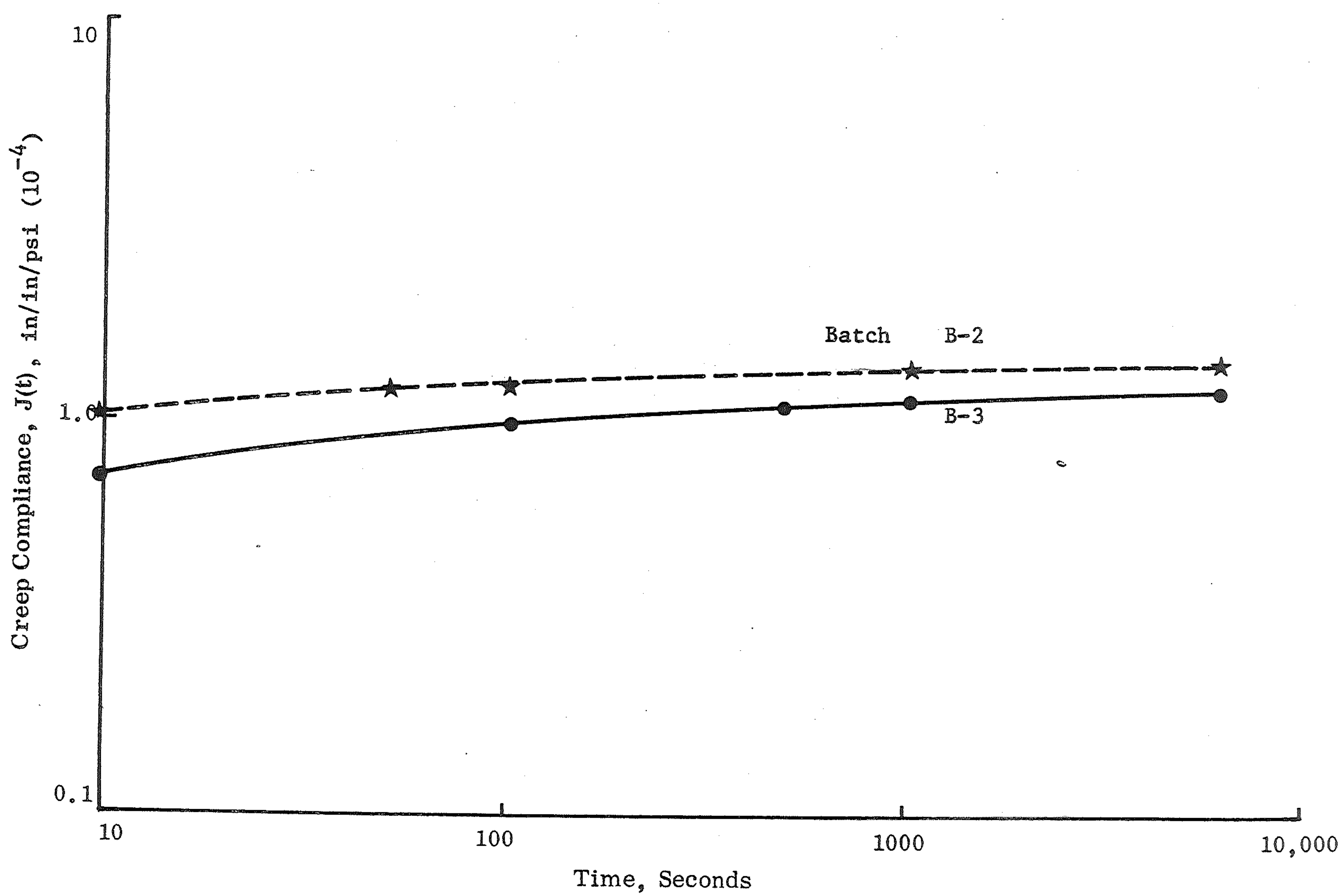


Figure 38

Complex Modulus Versus Frequency for Asphaltic Concrete Field Cores at 41°F

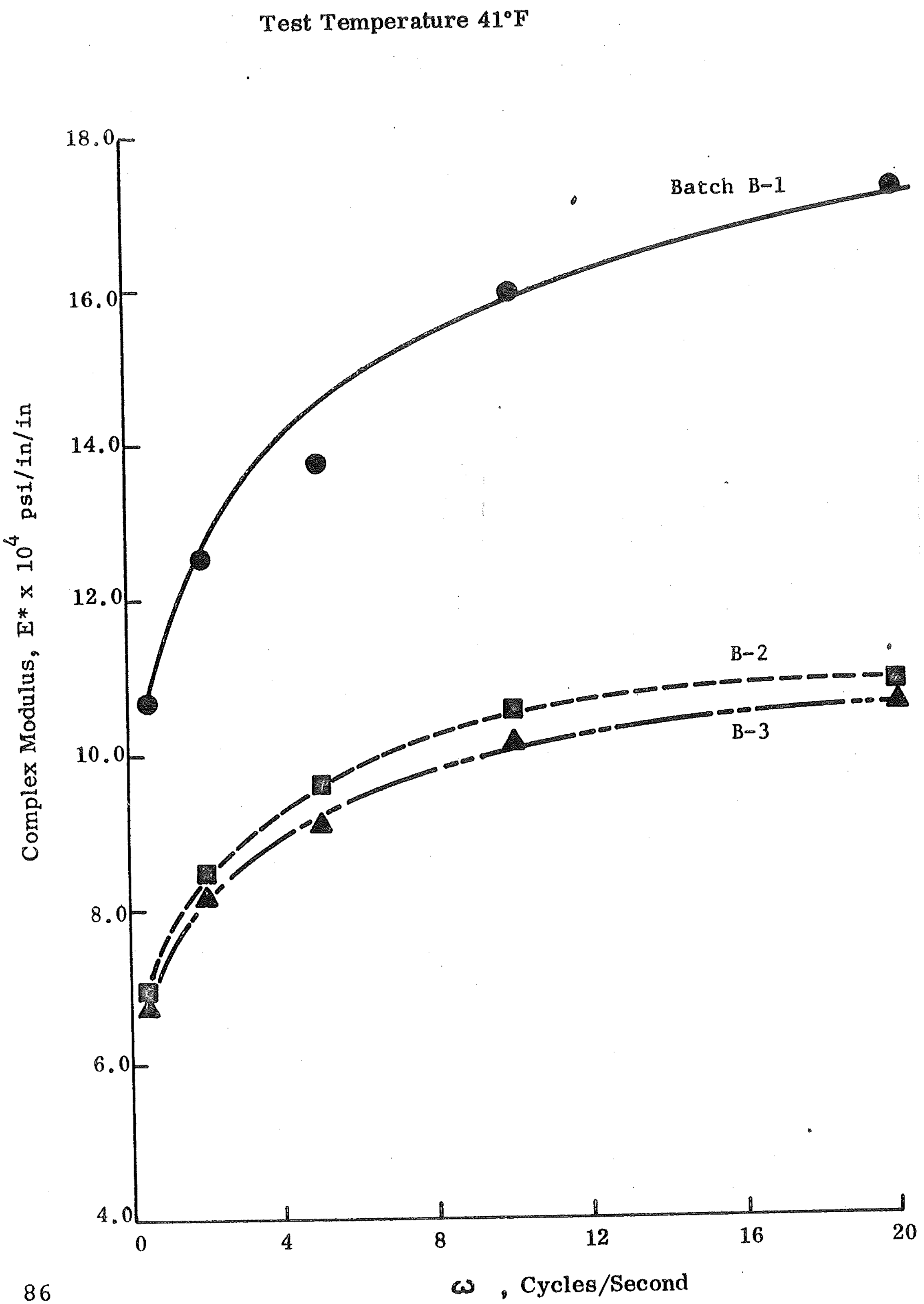


Figure 39

Complex Modulus Versus Frequency for Asphaltic Concrete Field Cores at 77°F

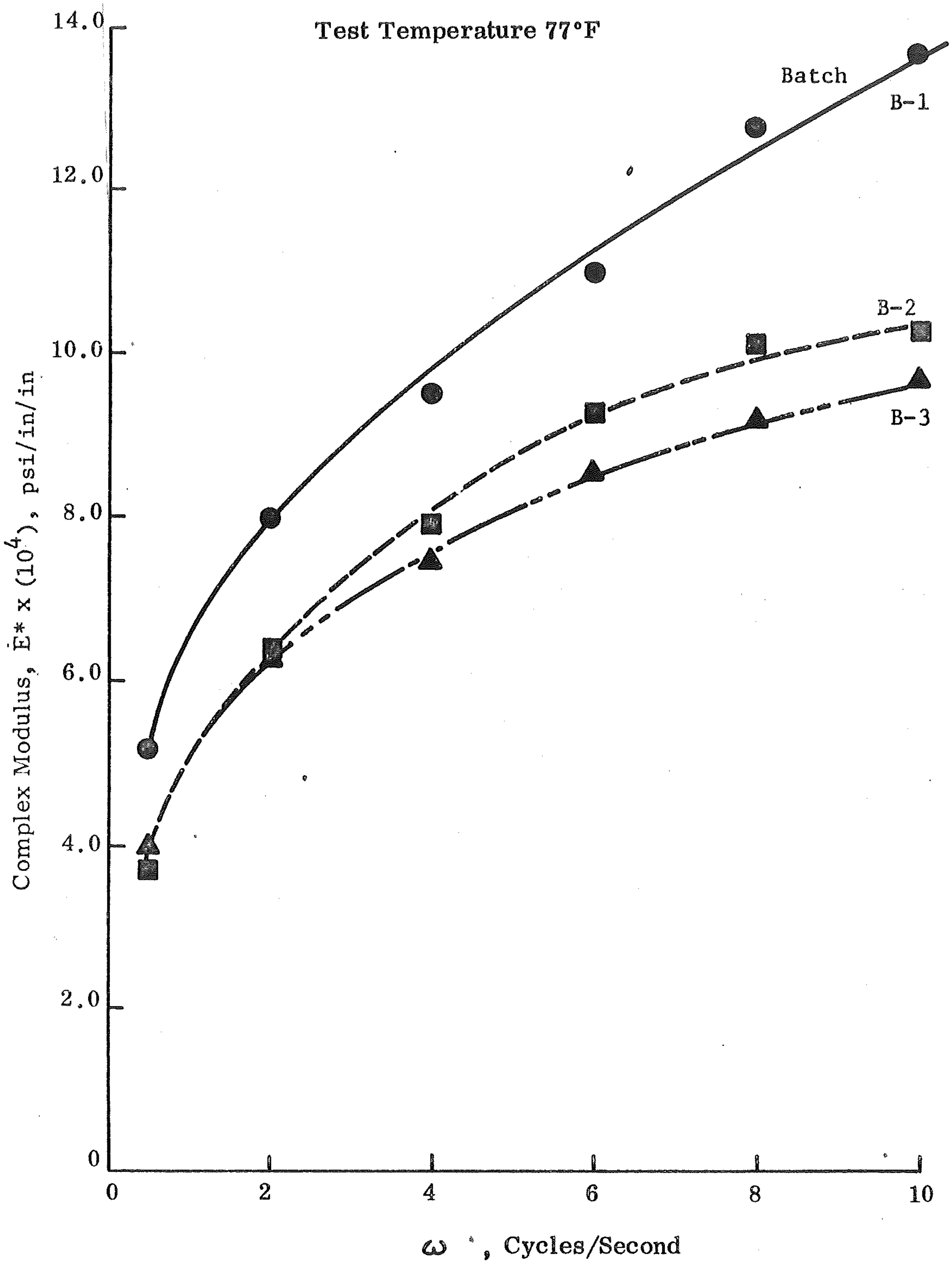


Figure 40

Composite Master Creep Modulus Curve
For Asphaltic Concrete Field Cores Obtained by Time-Temperature Equivalence Concept

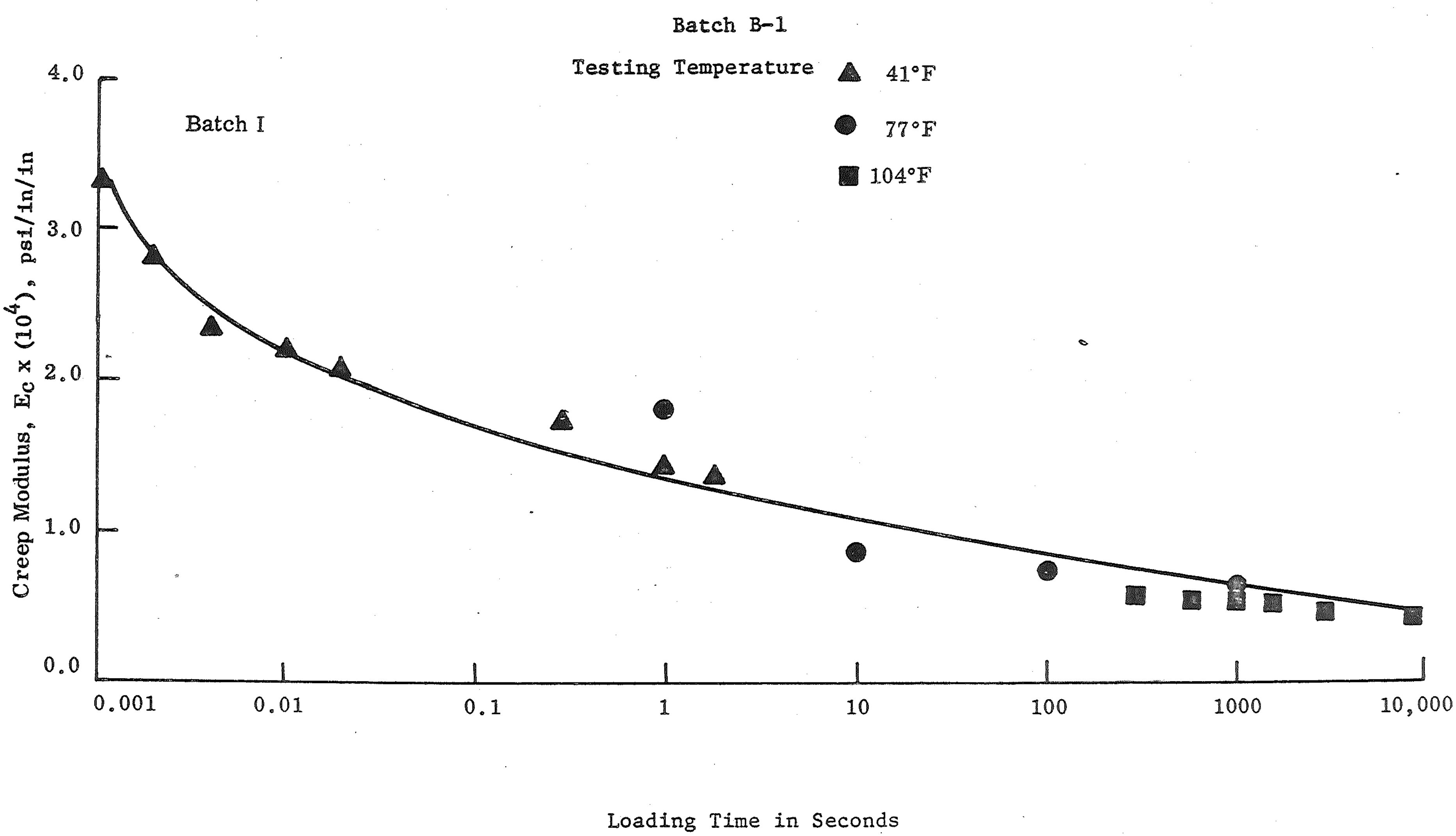


Figure 41

Composite Master Creep Modulus Curve
For Asphaltic Concrete Field Cores Obtained by Time-
Temperature Equivalence Concept

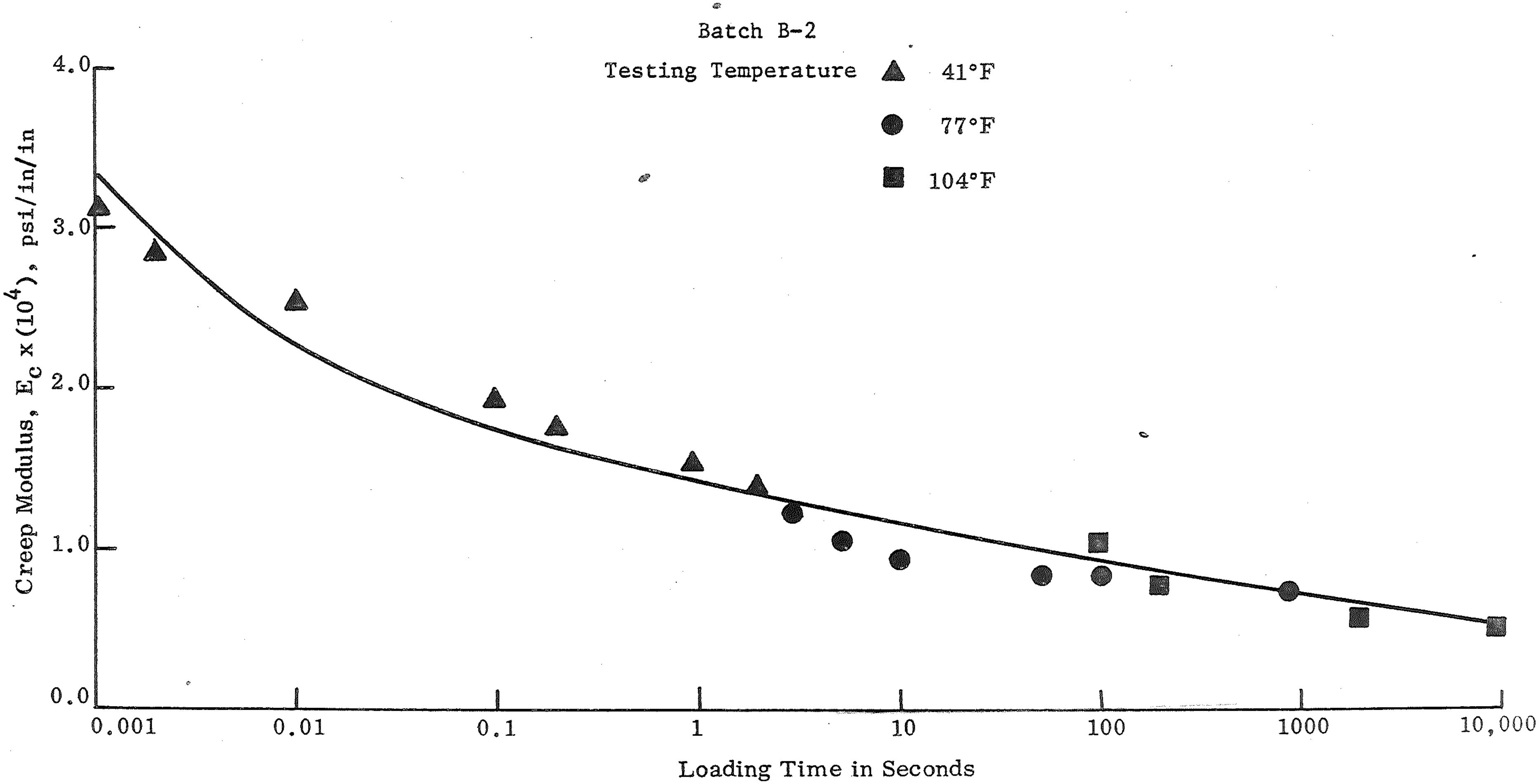


Figure 42

Composite Master Creep Modulus Curve
For Asphaltic Concrete Field Cores Obtained by Time-
Temperature Equivalence Concept

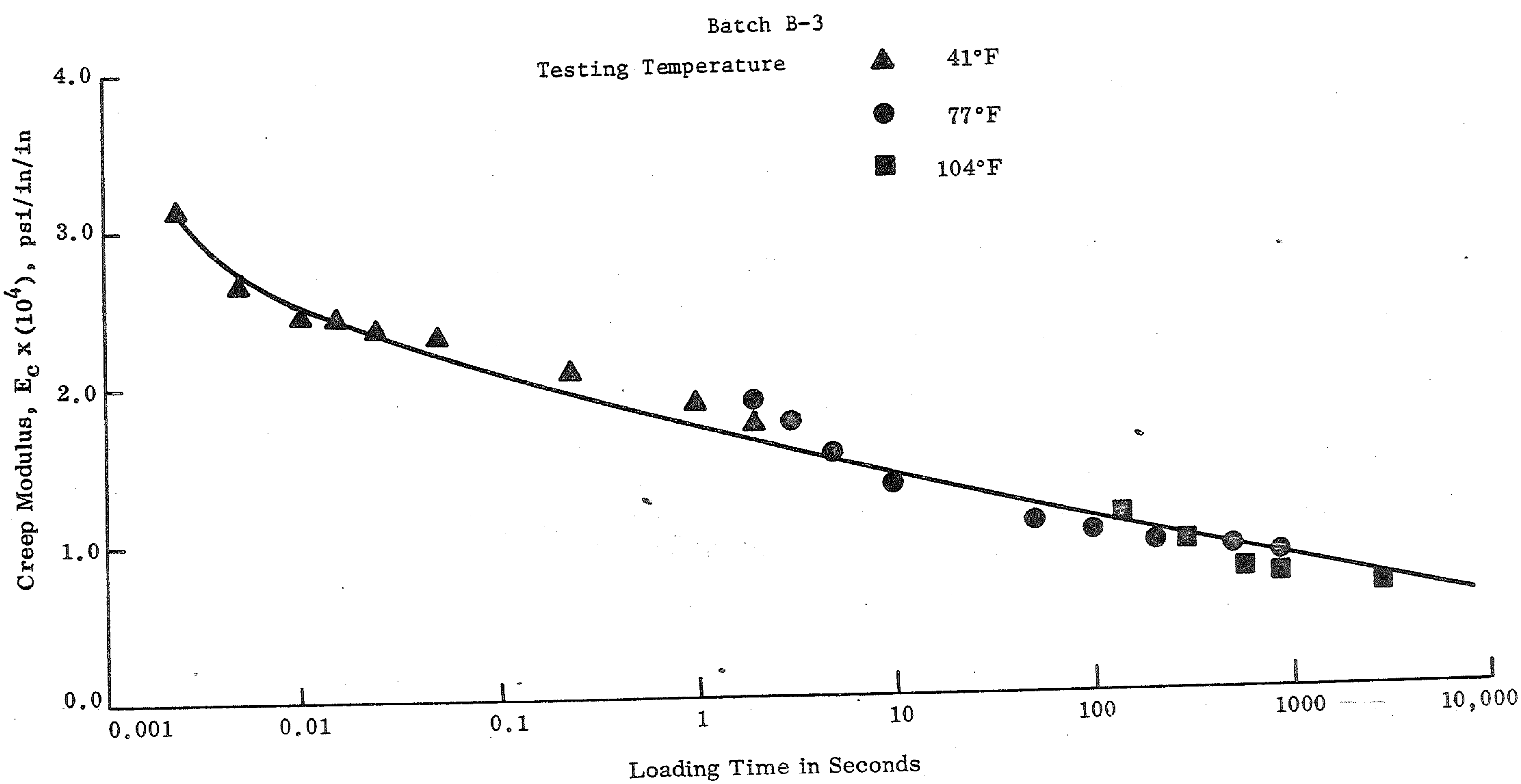


Figure 43

Composite Master Creep Modulus Curve for Three Batches of Asphaltic Concrete Field Cores Obtained by Time-Temperature Equivalence Concept

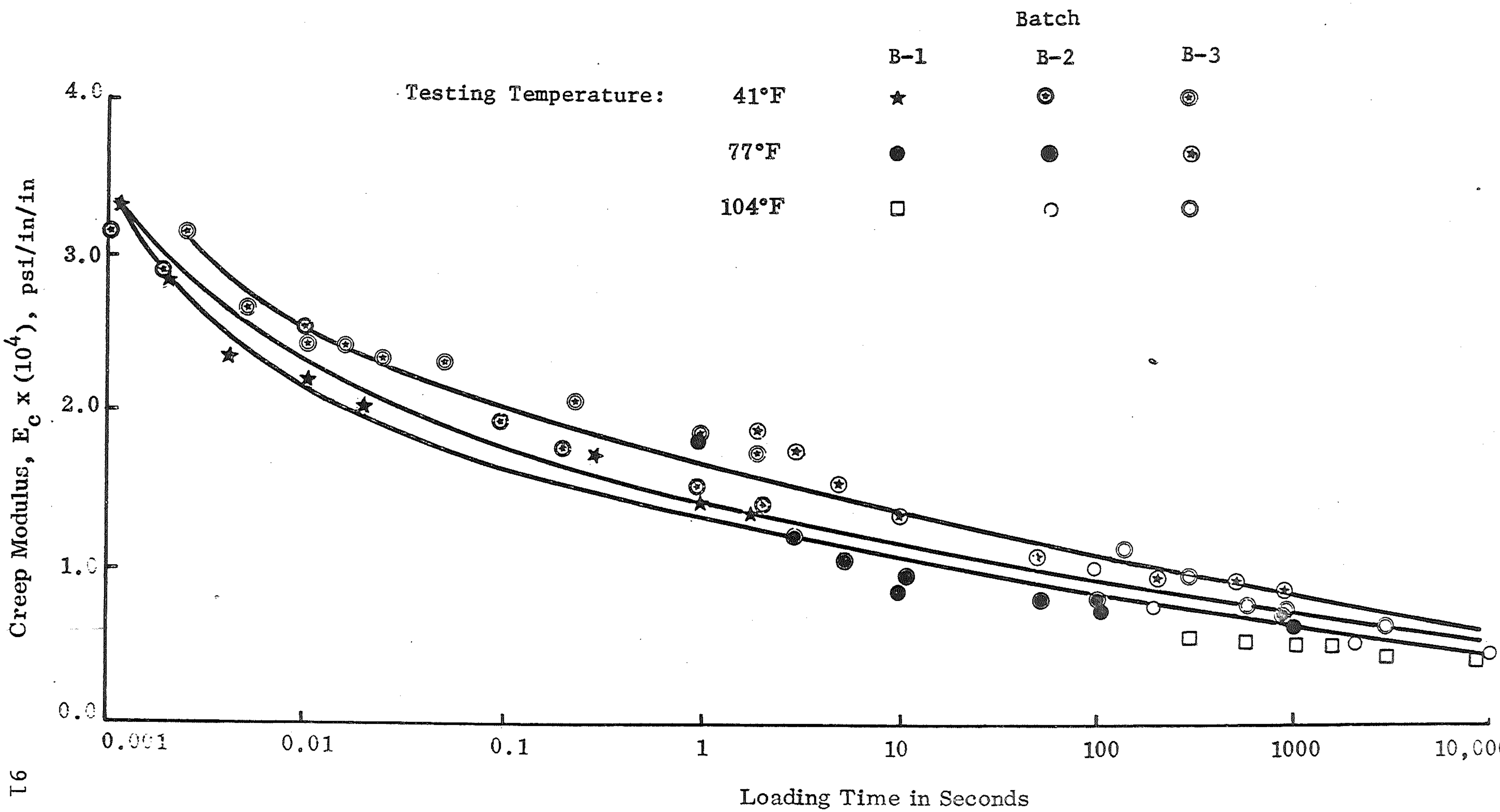


Figure 44

Log a_T Versus Temperature for Asphaltic Concrete Field Cores

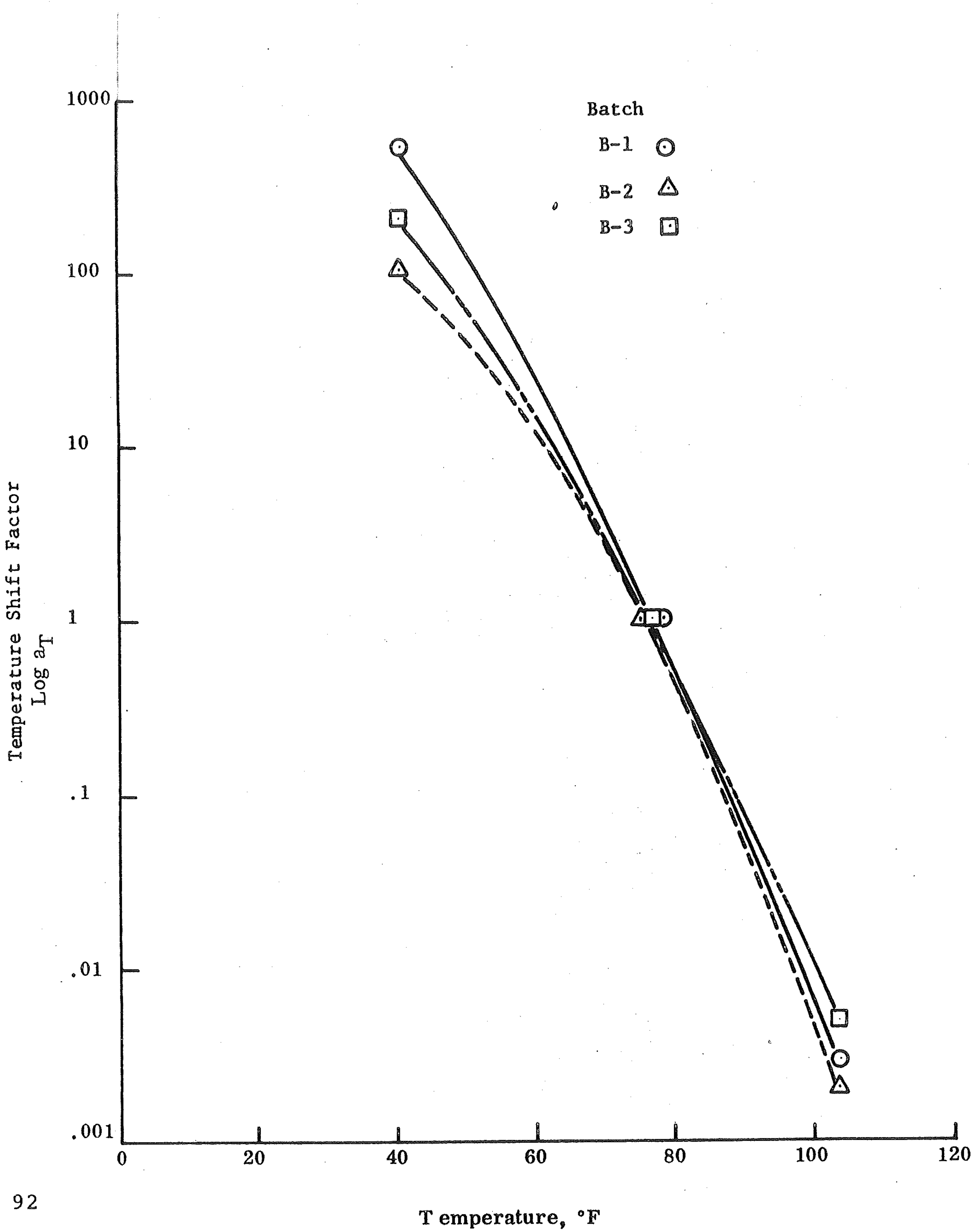


Figure 45

Variation of Modulus of Elasticity with the Age of Field Cores

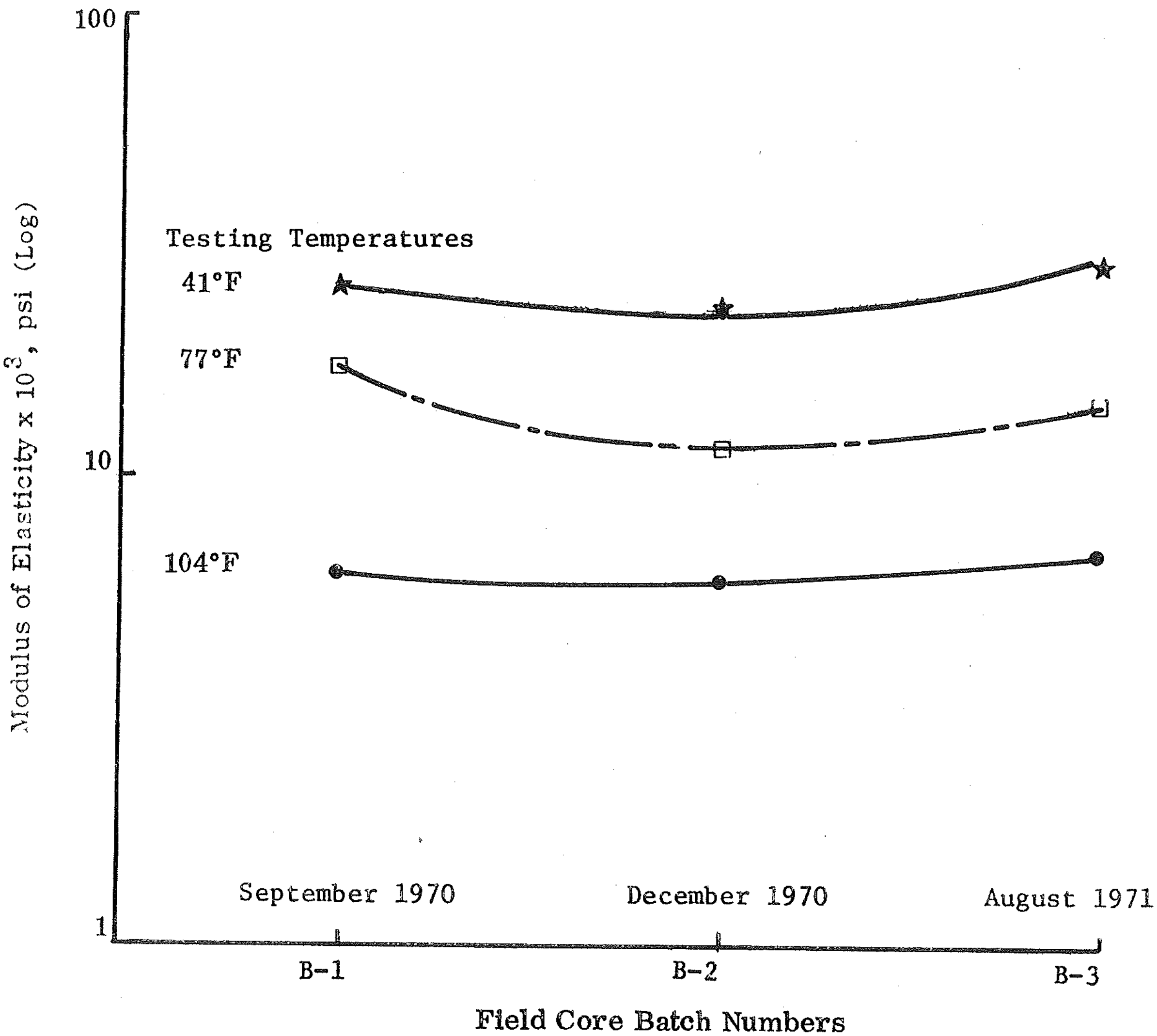


Figure 46

Variation of Unconfined Compressive Strength with the Age of Field Cores

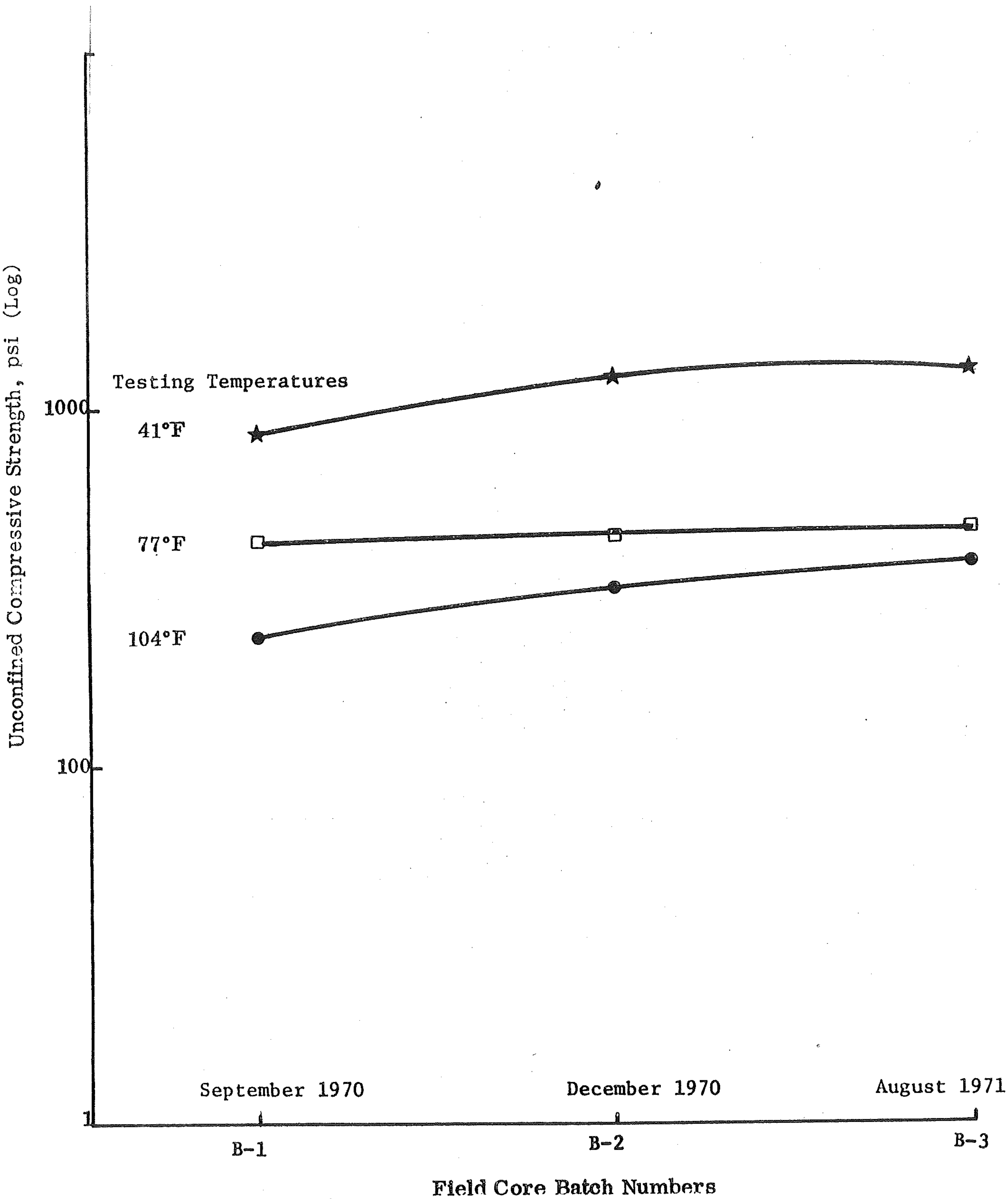


Figure 47

Dynamic Modulus Versus Frequency for Asphaltic Concrete Field Cores at 41°F

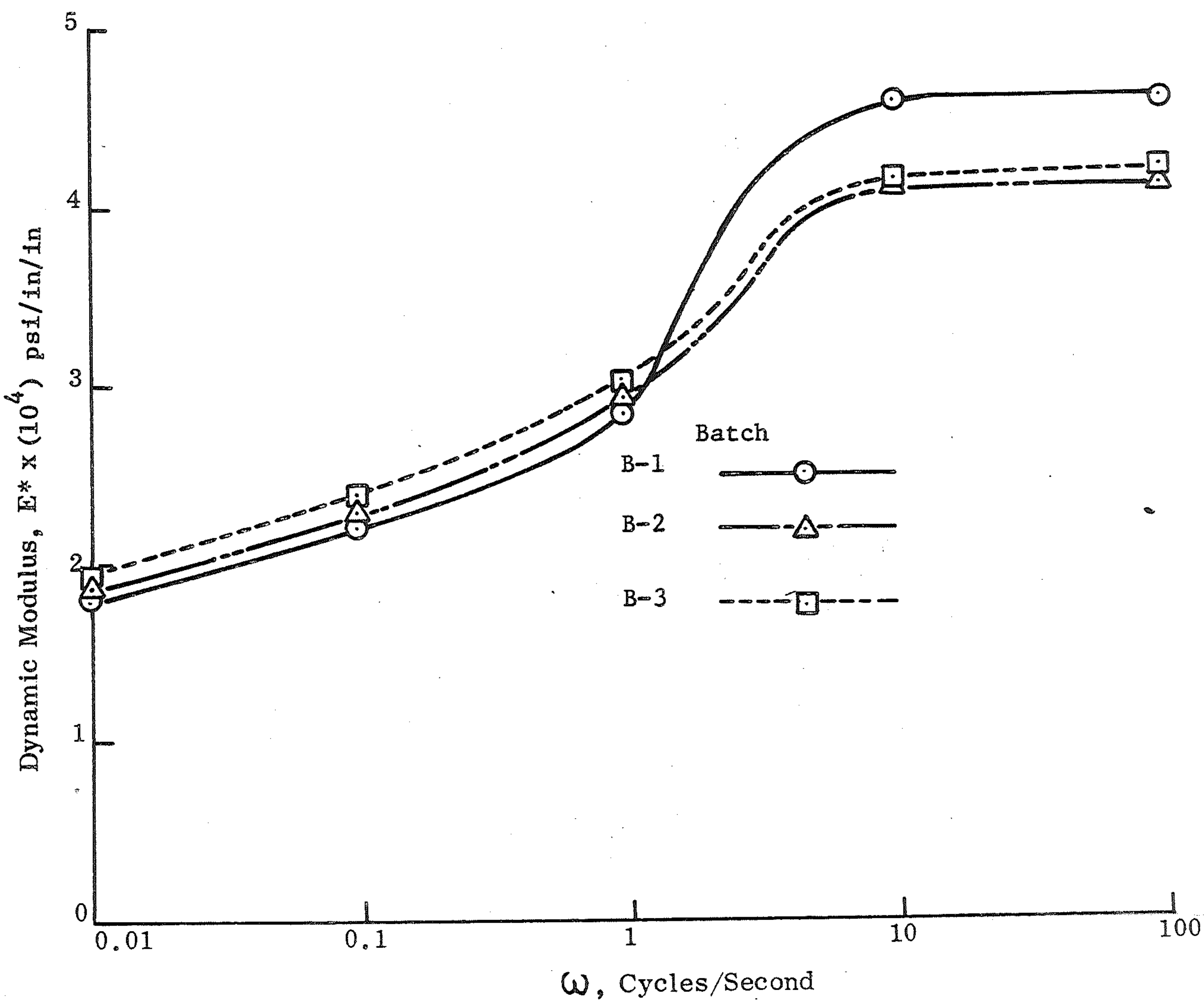


Figure 48

Dynamic Modulus Versus Frequency
for Asphaltic Concrete Field Cores at 77°F

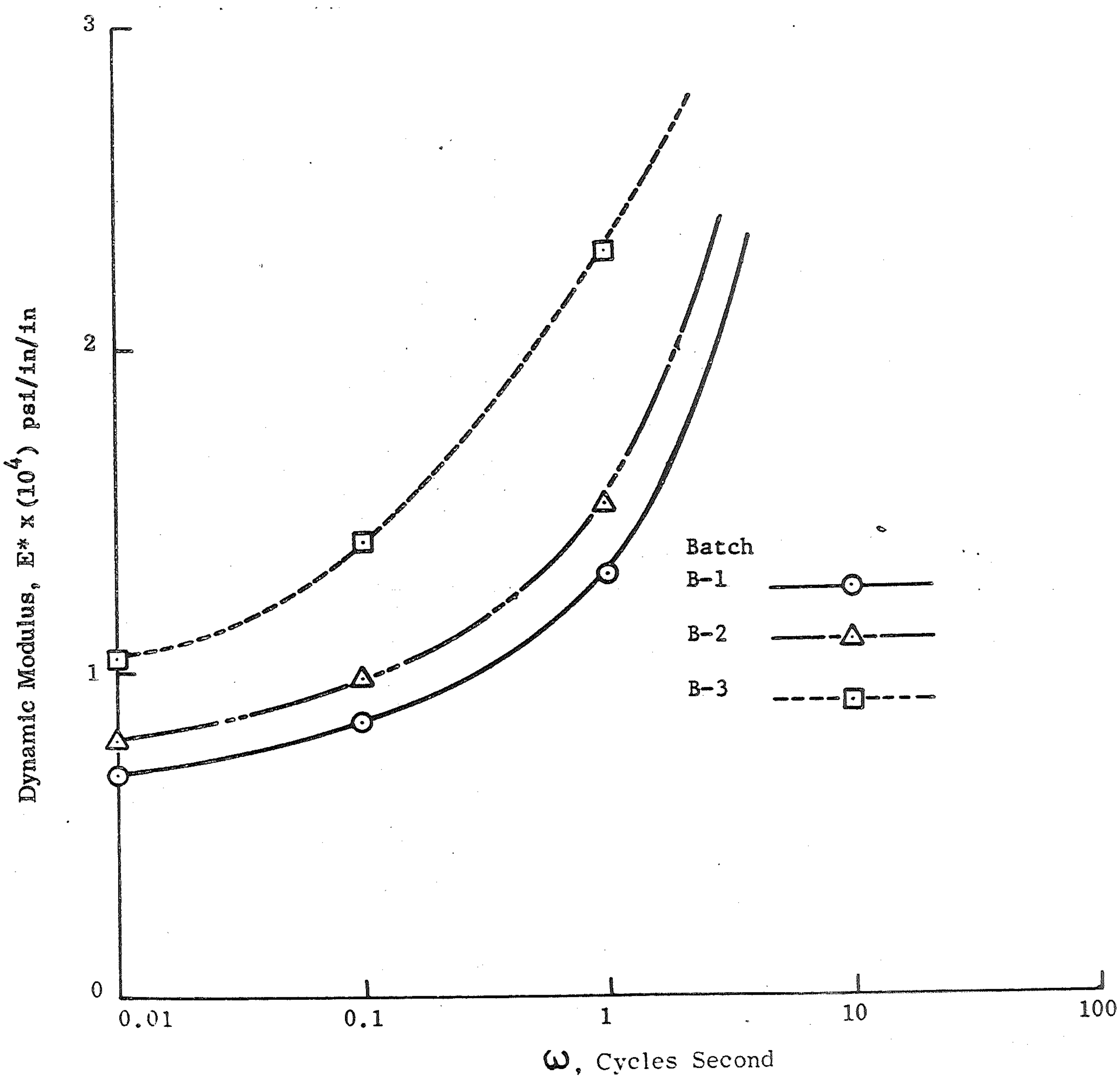


Figure 49

Dynamic Modulus Versus Frequency
for Asphaltic Concrete Field Cores at 104°F

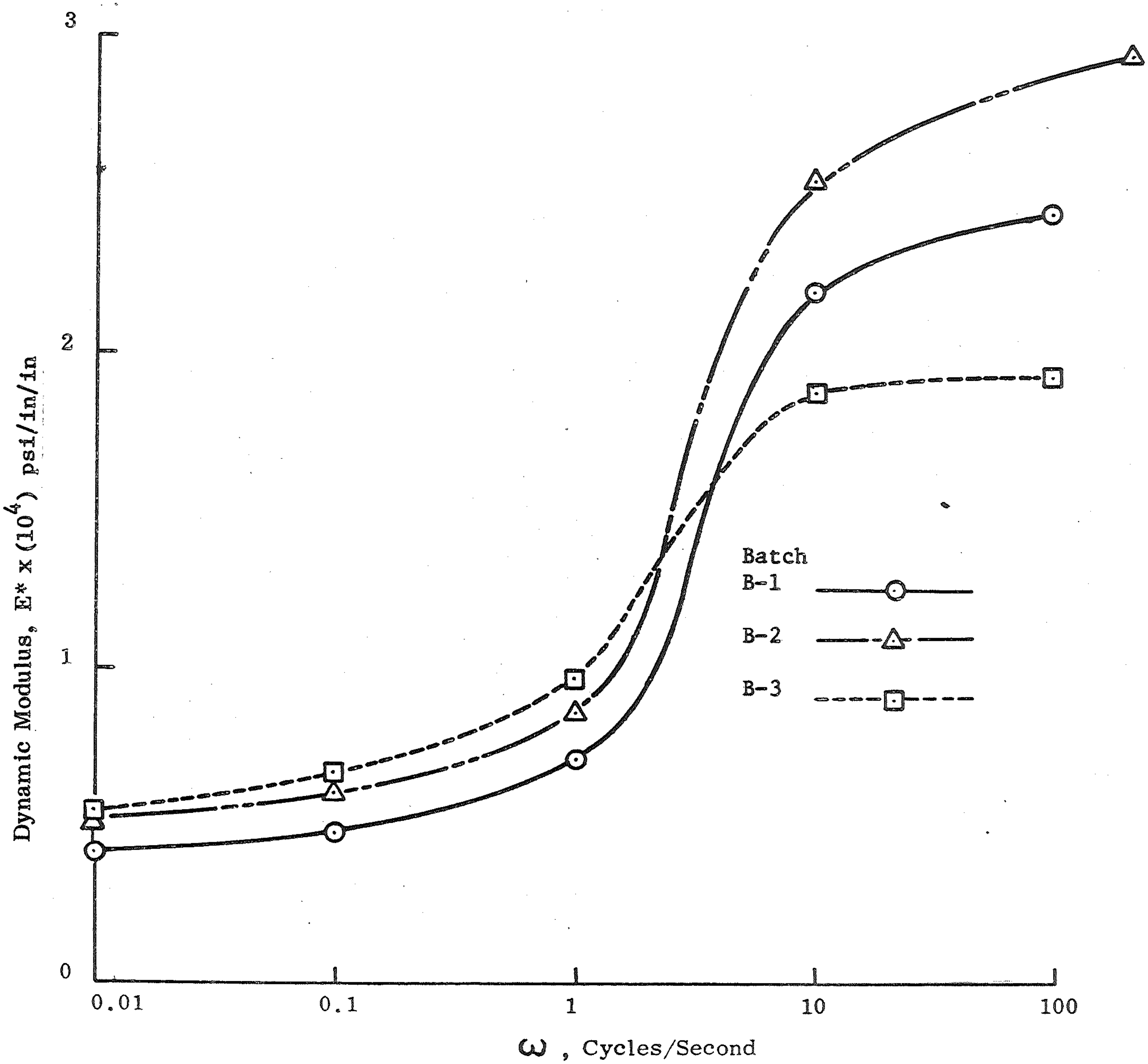
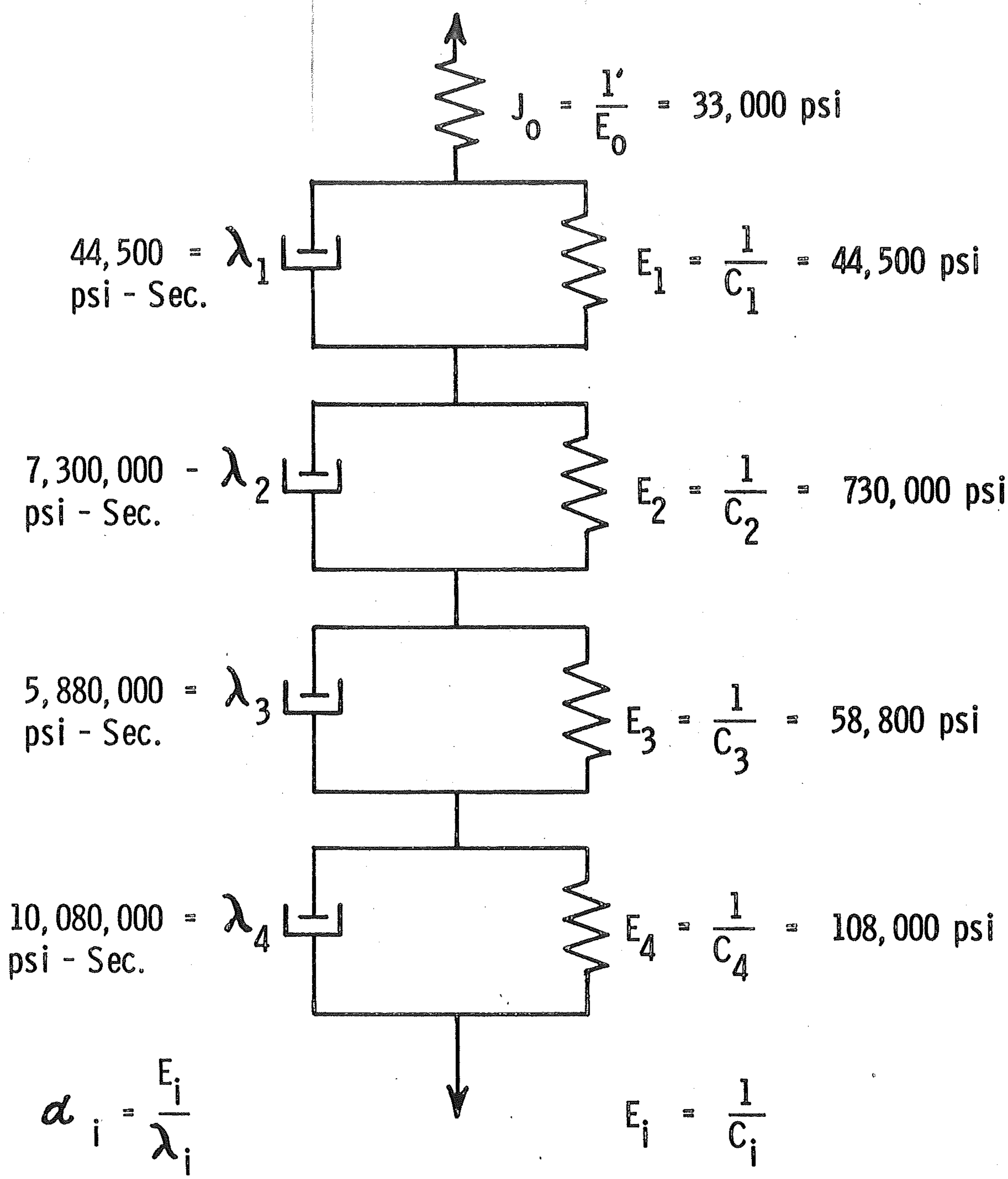


Figure 50

TYPICAL VOIGT MODEL FOR ASPHALTIC CONCRETE BATCH B-1 AT 41°F

Model Coefficients From Table 22



APPENDIX

Vermont I 91 Road Study

Rheological Properties of Field Cores

TABLE 13

CREEP COMPLIANCE OF ASPHALTIC CONCRETE FIELD CORES AT TEMP = 41°F

TIME (SEC)	CREEP COMPLIANCE J_t X 10^{-5} 1/psi		
	Road Core Batch Number		
	1	2	3
0.5	3.03	3.12	3.03
1	3.57	3.51	3.75
2	4.24	4.00	4.02
3	4.29	4.22	4.05
5	4.52	4.29	4.18
10	4.71	4.69	4.35
15			
30	5.0		
50		5.03	4.85
100	5.62	5.71	5.18
200	5.95		5.46
400	6.41	6.41	5.75
600	6.76		5.88
900	7.14	7.09	6.17

TABLE 14

CREEP COMPLIANCE OF ASPHALTIC CONCRETE
FIELD CORES AT TEMP = 77°F

TIME (SEC)	CREEP COMPLIANCE J_t X 10^{-5} 1/psi		
	1	2	3
0.5	4.17	4.08	3.81
1	5.56	5.74	4.51
2	9.52	8.13	5.18
3	10.75	8.19	5.68
5	11.43	9.71	6.54
10	11.90	10.75	7.63
50	13.33	11.76	9.09
100	13.81	12.19	9.61
200	14.18	12.66	10.2
400	14.77	12.98	10.64
600	15.01	13.33	10.99
900	15.34	13.69	11.36

TABLE 15

CREEP COMPLIANCE OF ASPHALTIC CONCRETE
FIELD CORES AT TEMP = 104°F

TIME (SEC)	CREEP COMPLIANCE J_t X 10^{-5} 1/psi		
	1	2	3
0.5	10.00	8.44	8.55
1	17.86	12.69	10.52
2	19.30	15.27	13.16
3	19.61	16.02	14.28
5	20.00	16.53	14.56
10	21.00	17.15	15.72
15			
30	22.32		
50		18.31	17.15
100	23.58	19.27	17.73
200	24.33	20.04	18.59
400	25.13	20.83	19.45
600	25.44	21.14	19.61
900	25.71	21.83	19.88

TABLE 16

CREEP MODULUS OF ASPHALTIC CONCRETE FIELD CORES AT TEMP = 41°F

TIME (SEC)	CREEP MODULUS E_C X 10^4 psi		
	1	2	3
0.5	3.3	3.20	3.3
1	2.8	2.85	2.67
2	2.36	2.50	2.49
3	2.33	2.37	2.47
5	2.21	2.33	2.39
10	2.12	2.13	2.30
30	2.00		
50		1.99	2.06
100	1.78	1.75	1.93
200	1.68		1.83
400	1.56	1.56	1.74
600	1.48		1.70
900	1.40	1.41	1.62

TABLE 17

CREEP MODULUS OF ASPHALTIC CONCRETE FIELD CORES AT TEMP = 77°F

TIME (SEC)	CREEP MODULUS $E_C \times 10^4$psi		
	1	2	3
0.5	2.40	2.45	2.620
1	1.80	1.74	2.215
2	1.05	1.23	1.93
3	.93	1.22	1.76
5	.875	1.03	1.53
10	.84	.93	1.31
50	.75	.85	1.10
100	.724	.82	1.04
200	.705	.79	.98
400	.677	.77	.94
600	.666	.75	.91
900	.652	.73	.88

TABLE 18

CREEP MODULUS OF ASPHALTIC CONCRETE FIELD CORES AT TEMP ≜ 104°F

TIME (SEC)	CREEP MODULUS $E_C \times 10^4$ psi		
	1	2	3
0.5	1.00	1.184	1.17
1	.56	.788	.95
2	.518	.655	.76
3	.510	.624	.70
5	.50	.605	.686
10	.476	.583	.636
30	.448		
50		.546	.583
100	.424	.519	.564
200	.411	.499	.538
400	.398	.480	.514
600	.393	.473	.510
900	.389	.458	.503

Table 19

Unconfined Compressive Strength Test Data and Elastic Moduli of Field Cores at Different Temperatures (ASTM D 1074)

Temperature °F	Average Unconfined Compressive Strength, psi			Average Young's Modulus of Elasticity, psi		
	Batch B-1	B-2	B-3	B-1	B-2	B-3
41	850	1,116	1,214	25,800	23,850	28,100
77	430	430	453	17,400	11,950	14,500
104	234	319	357	6,360	6,210	8,780

TABLE 20

Temperature T = 77°F

Dynamic Modulus Data Obtained by using MTS Testing System

Frequency, 'ω' Cycles/Second	$E^* \times 10^4$, psi		
	B-1	B-2	B-3
0.5	5.16	3.66	4.04
2.0	7.99	6.26	6.35
4.0	9.46	7.89	7.41
6.0	10.98	9.20	8.50
8.0	12.77	10.05	9.12
10.0	13.61	10.20	9.61

TABLE 21

Temperature T = 41°F

Dynamic Modulus Data Obtained by using MTS Testing System

Frequency 'ω'	$E^* \times 10^4$, psi		
Cycles/Second	B-1	B-2	B-3
0.5	10.66	6.94	6.67
2.0	12.51	8.44	8.18
5.0	13.78	9.56	9.16
10.0	15.95	10.54	10.18
20.0	17.24	10.82	10.67

TABLE 22

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad\qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.2248990E-04
ALPHA(2)	=0.10000	C(2) =	0.1372129E-05
ALPHA(3)	=0.01000	C(3) =	0.1701058E-04
ALPHA(4)	=0.00100	C(4) =	0.9232860E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.1494419E+05
0.00400	0.1641671E+05
0.00700	0.1737749E+05
0.01000	0.1830827E+05
0.03000	0.2119586E+05
0.06000	0.2200904E+05
0.10000	0.2238940E+05
0.20000	0.2292549E+05
0.40000	0.2400636E+05
0.60000	0.2545404E+05
1.00000	0.2894664E+05
2.00000	0.3653316E+05
4.00000	0.4301570E+05
10.00000	0.4622029E+05
40.00000	0.4691270E+05
100.00000	0.4695279E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-1 at 41°F

TABLE 23

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad\qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.1784963E-04
ALPHA(2)	=0.10000	C(2) =	-0.2140095E-06
ALPHA(3)	=0.01000	C(3) =	0.2077654E-04
ALPHA(4)	=0.00100	C(4) =	0.8408470E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.1497329E+05
0.00400	0.1644165E+05
0.00700	0.1757161E+05
0.01000	0.1872211E+05
0.03000	0.2254404E+05
0.06000	0.2352477E+05
0.10000	0.2379588E+05
0.20000	0.2410316E+05
0.40000	0.2498574E+05
0.60000	0.2624873E+05
1.00000	0.2920482E+05
2.00000	0.3498861E+05
4.00000	0.3923738E+05
10.00000	0.4111544E+05
40.00000	0.4150292E+05
100.00000	0.4152514E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-2 at 41°F

TABLE 24

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad Ei = \frac{1}{Ci}$$

ALPHA(1) =1.00000	C(1) =	0.1606489E-04
ALPHA(2) =0.10000	C(2) =	0.3404813E-05
ALPHA(3) =0.01000	C(3) =	0.1271317E-04
ALPHA(4) =0.00100	C(4) =	0.5770496E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.1699578E+05
0.00400	0.1823733E+05
0.00700	0.1910780E+05
0.01000	0.1994116E+05
0.03000	0.2245672E+05
0.06000	0.2333592E+05
0.10000	0.2395980E+05
0.20000	0.2485536E+05
0.40000	0.2604962E+05
0.60000	0.2738910E+05
1.00000	0.3036692E+05
2.00000	0.3599830E+05
4.00000	0.4000066E+05
10.00000	0.4173269E+05
40.00000	0.4208714E+05
100.00000	0.4210743E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-3 at 41°F

TABLE 25

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad\qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.1075961E-03
ALPHA(2)	=0.10000	C(2) =	0.1843435E-04
ALPHA(3)	=0.01000	C(3) =	0.2176211E-04
ALPHA(4)	=0.00100	C(4) =	0.7254403E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.6680980E+04
0.00400	0.6948586E+04
0.00700	0.7159668E+04
0.01000	0.7357336E+04
0.03000	0.7945699E+04
0.06000	0.8258594E+04
0.10000	0.8588609E+04
0.20000	0.9131328E+04
0.40000	0.9869543E+04
0.60000	0.1074361E+05
1.00000	0.1306544E+05
2.00000	0.2067711E+05
4.00000	0.3808589E+05
10.00000	0.9195087E+05
40.00000	0.3161853E+06
100.00000	0.5076927E+06

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-1 at 77°F

TABLE 26

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{E_i}{\lambda_i} \qquad E_i = \frac{1}{C_i}$$

ALPHA(1)	=1.00000	C(1) =	0.7851129E-04
ALPHA(2)	=0.10000	C(2) =	0.2273740E-04
ALPHA(3)	=0.01000	C(3) =	0.1837459E-04
ALPHA(4)	=0.00100	C(4) =	0.8573391E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.7542383E+04
0.00400	0.7897551E+04
0.00700	0.8135074E+04
0.01000	0.8353719E+04
0.03000	0.9025922E+04
0.06000	0.9463531E+04
0.10000	0.9977687E+04
0.20000	0.1083630E+05
0.40000	0.1184675E+05
0.60000	0.1292139E+05
1.00000	0.1569277E+05
2.00000	0.2451858E+05
4.00000	0.4294370E+05
10.00000	0.8051787E+05
40.00000	0.1113176E+06
100.00000	0.1143021E+06

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-2 at 77°F

TABLE 27

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.2735989E-04
ALPHA(2)	=0.10000	C(2) =	0.2793035E-04
ALPHA(3)	=0.01000	C(3) =	0.2372164E-04
ALPHA(4)	=0.00100	C(4) =	0.3738240E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.9155895E+04
0.00400	0.9737078E+04
0.00700	0.1018586E+05
0.01000	0.1062275E+05
0.03000	0.1208661E+05
0.06000	0.1308388E+05
0.10000	0.1432533E+05
0.20000	0.1654518E+05
0.40000	0.1861186E+05
0.60000	0.2010924E+05
1.00000	0.2316623E+05
2.00000	0.2960821E+05
4.00000	0.3520665E+05
10.00000	0.3802761E+05
40.00000	0.3864250E+05
100.00000	0.3867818E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-3 at 77°F

TABLE 28

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.1466168E-03
ALPHA(2)	=0.10000	C(2) =	0.2753777E-04
ALPHA(3)	=0.01000	C(3) =	0.3539761E-04
ALPHA(4)	=0.00100	C(4) =	0.6759999E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.3945215E+04
0.00400	0.4060161E+04
0.00700	0.4174086E+04
0.01000	0.4282410E+04
0.03000	0.4602238E+04
0.06000	0.4766437E+04
0.10000	0.4935992E+04
0.20000	0.5213844E+04
0.40000	0.5602594E+04
0.60000	0.6066727E+04
1.00000	0.7270598E+04
2.00000	0.1081512E+05
4.00000	0.1646727E+05
10.00000	0.2233683E+05
40.00000	0.2436033E+05
100.00000	0.2449184E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-1 at 104°F

TABLE 29

VALUES OF COEFFICIENTS

$$\alpha_i = \frac{Ei}{\lambda i} \qquad Ei = \frac{1}{Ci}$$

ALPHA(1)	=1.00000	C(1) =	0.1263357E-03
ALPHA(2)	=0.10000	C(2) =	0.7574199E-05
ALPHA(3)	=0.01000	C(3) =	0.3657361E-04
ALPHA(4)	=0.00100	C(4) =	0.1368442E-04

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.4731172E+04
0.00400	0.4973848E+04
0.00700	0.5157129E+04
0.01000	0.5329621E+04
0.03000	0.5819078E+04
0.06000	0.5972043E+04
0.10000	0.6075562E+04
0.20000	0.6253637E+04
0.40000	0.6631621E+04
0.60000	0.7159750E+04
1.00000	0.8569668E+04
2.00000	0.1275984E+05
4.00000	0.1950481E+05
10.00000	0.2661869E+05
40.00000	0.2910451E+05
100.00000	0.2926674E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-2 at 104°F

TABLE 30

VALUES OF COEFFICIENTS

$\alpha_i = \frac{E_i}{\lambda_i}$ $\qquad\qquad$ $E_i = \frac{1}{C_i}$

ALPHA(1)	=1.00000	C(1) =	0.8403283E-04
ALPHA(2)	=0.10000	C(2) =	0.2184864E-04
ALPHA(3)	=0.01000	C(3) =	0.3596010E-04
ALPHA(4)	=0.00100	C(4) =	0.5770616E-05

FREQUENCY= cps	DYNAMIC MODULUS= psi
0.00100	0.5110574E+04
0.00400	0.5291949E+04
0.00700	0.5485816E+04
0.01000	0.5675203E+04
0.03000	0.6250527E+04
0.06000	0.6521176E+04
0.10000	0.6781832E+04
0.20000	0.7192168E+04
0.40000	0.7706789E+04
0.60000	0.8276262E+04
1.00000	0.9661801E+04
2.00000	0.1307691E+05
4.00000	0.1673938E+05
10.00000	0.1896668E+05
40.00000	0.1949847E+05
100.00000	0.1952989E+05

Viscoelastic Parameters and Dynamic Modulus Obtained from Creep Data for Batch B-3 at 104°F