TRANSPORTATION DEPARTMENT

ASPHALT QUALITY EVALUATION

RESEARCH PROJECT 62

RESEARCH SECTION

ACKNOWLEDGEMENTS

This report was prepared under the supervision of James W. Hill, Research Supervisor of the Idaho Transportation Department.

This project was initiated by L. F. Erickson, former Materials and Research Engineer. His help and suggestions are appreciated, as are those of Gene Wortham, former Testing Engineer.

Asphalt tests were performed by Ellis Clark and Gail Derr.

Aggregate for tensile split testing was prepared by Gene Brooks, Jim Gore, Ben Hartman, and Jim Wright.

Tensile split specimens were prepared by Bill Clark, Floyd Crichton, and Dick Howell.

Tensile split and resilient modulus testing were done by Bill Clark, Floyd Crichton, Ben Hartman, Larry Hippler and Dave Turner.

Pat Galindo and Kathy Whitehead typed the report.

Asphalt testing at 39°F, preparing large quantities of aged asphalt, and determining the resilient modulus of asphaltic concrete are not standard practices in the Idaho Division of Highways. Several of the technicians mentioned above made valuable suggestions about modifying standard equipment for use in this program of non-routine testing. Their suggestions and the cooperation of all participants in this project are appreciated.

CONTENTS

INTRODUCTION AND WORK OUTLINE

Introduction

Asphalt grade and source, aggregate source, environmental temperature, moisture, and asphalt aging are among the factors which influence the physical properties of asphaltic concrete (AC) pavement. Large variations can exist in these factors, causing corresponding variations in the properties of AC pavement in service. The project described in this report was undertaken to document and compare property variations associated with variations in most factors mentioned above. Some nonroutine test procedures and conditions were included, which provided information not given by the Division's routine testing operations.

Outline of Testing Program

The testing program can be divided conveniently into two phases: asphalt testing and asphaltic concrete testing. In both phases, parallel testing was done at two different temperatures to indicate the effect of environmental temperature. Similarly, parallel testing was done using fresh asphalt and artificially aged asphalt.

In the asphalt testing phase, two to four penetration grades from each of six refineries were subjected to the following tests: penetration, AASHTO T 49, 77°F (100 g, 5 sec.) and 39°F (200 g, 60 sec.); viscosity AASHTO T 201, 275°F; viscosity, AASHTO T 202, 140°F; ductility, AASHTO T 51, 77°F (5 cm/min.); softening point, AASHTO T 53. The foregoing tests were done on fresh samples and also on aged samples. The following tests were made only on fresh asphalt: loss on heating, AASHTO T 179; flash point, AASHTO T 48; solubility, AASHTO T 44; specific gravity, AASHTO T 228; spot test, AASHTO T 102; presence of anti-strip additive, Idaho T 99.

In the asphaltic concrete testing phase only 120-150 grade asphalt from each of the six refineries was used. Two series of specimens were made, one with fresh asphalt and the other using aged asphalt. Each of these two groups contained three subgroups, each employing aggregate from a different Idaho source. Parallel testing was done at nominal temperatures of 77°F and 39°F. Under each combination of the foregoing conditions, one set of three specimens was tested dry and a second set was tested after soaking in water under partial vacuum. The test used was tensile splitting, Idaho T11. During the tensile split tests, measurements were made to permit estimation of resilient modulus, using the procedure described by Lottman (7)*.

*Numbers in parenthesis denote sources of additional information, identified in the reference list under the same numbers.

CONCLUSIONS

Considerable differences in temperature susceptibility exist among asphalts which have been supplied for road construction in Idaho.

Laboratory test results indicate low temperature cracking of AC pavements could be a problem in some parts of the State. Some field data is available to support the existence of such a problem.

Methods are available for incorporating low temperature asphalt properties into asphalt concrete pavement design and for providing some control over low temperature asphalt properties by appropriate restrictions in the purchase specifications.

Within the range of asphalt sources commonly used in Idaho, asphalt source does not exert a statistically significant influence on instantaneous resilient modulus of asphalt concrete at 39°F or 77°F. Nonetheless, resilient modulus could be changed by a factor of 2 or more by a change in asphalt source.

Aggregate source can have a strong effect on instantaneous resilient modulus of asphalt concrete at 77°F or 39°F. Results of this project suggest the problem aggregates may be those which are moisture susceptible also.

Asphalt source does not exert a statistically significant effect on moisture susceptibility of asphalt concrete at 39°F or 77°F, at least within the range of sources commonly used in Idaho.

Moisture can cause large decreases in both tensile strength and instantaneous resilient modulus of asphalt concrete made with moisture susceptible aggregate.

In its present form the Lottman displacement transducer used in the tensile split test has some operational disadvantages, particularly when specimen temperature differs from ambient temperature.

RECOMMENDATIONS

Low temperature asphalt properties should be incorporated into the Department's asphalt cement specifications. As a minimum, Table 2 of AASHTO M 226 should be adopted in preference to Table 1 or 3 of that specification. More desirable, however, would be the use of M 226 in conjunction with direct consideration of low temperature behavior. A suggested procedure for asphalt cement selection is presented on the following two pages. AASHTO M 226 is used as the basic specification, with the addition of a penetration requirement for thin film oven residue at 39°F. The suggested procedure is based on laboratory and field research by other organizations and has not been field tested in Idaho. A field trial is recommended.

When resilient modulus testing of asphalt concrete is performed for pavement design, test specimens should be made with aggregate from the aggregate source which will be used for construction. The asphalt used in such specimens may need to be only of the proposed grade. However, further evaluation of the effect of asphalt source should be made to determine the practical importance of the observed resilient modulus variations associated with changes in asphalt source.

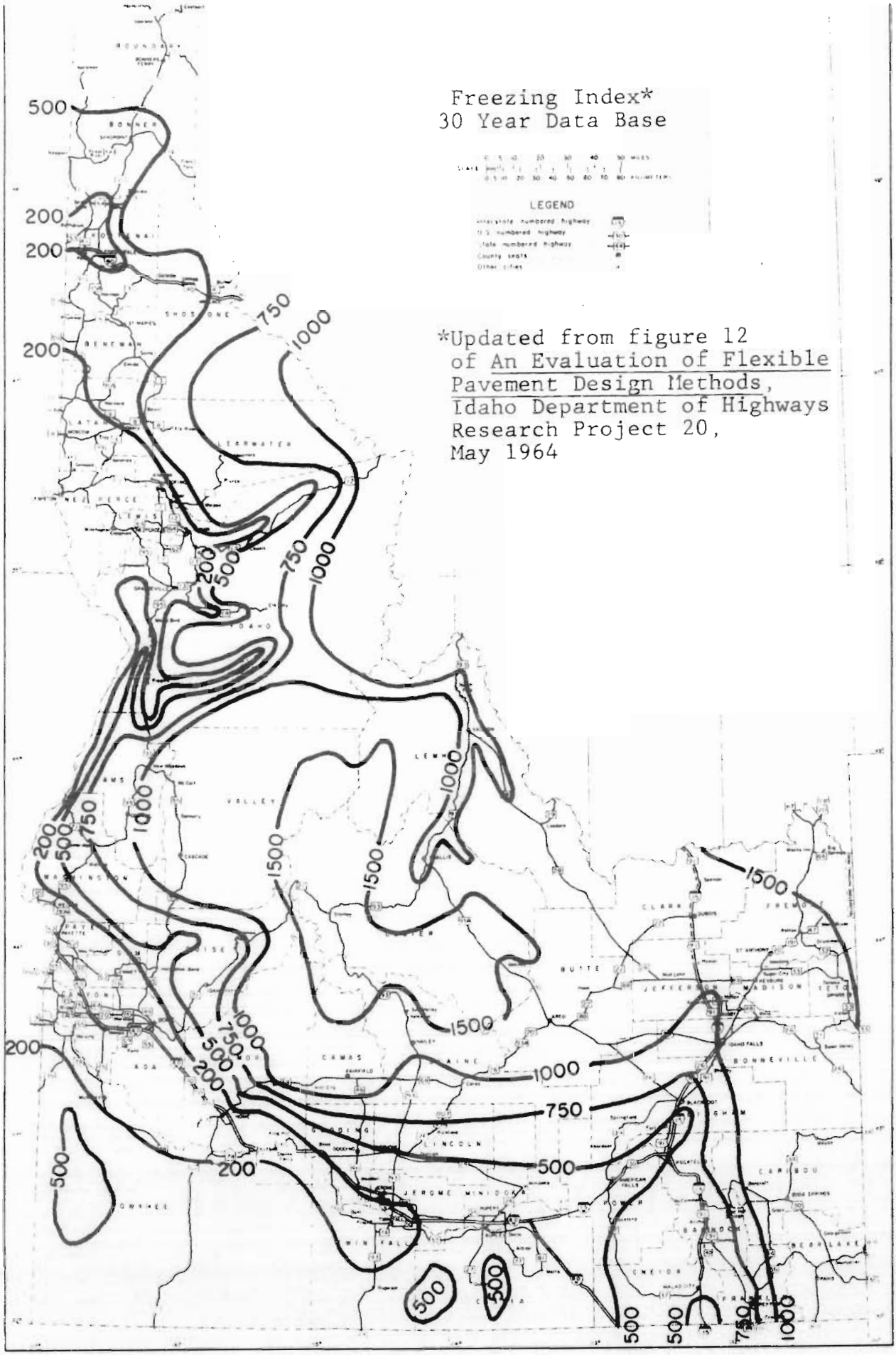
Improvements should be made to the resilient modulus test equipment used in this project if further resilient modulus testing of asphaltic concrete specimens is undertaken. One desirable change would be acquisition of an improved power supply for the transducer. Another would be a change to nondestructive, impulse-type loading, especially if tests are to be done at temperatures other than ambient.

Table for Selecting Asphalt Cement Grade
to Minimize Low Temperature Pavement Cracking
(See Appendix E for an outline of the method used to develop this table.)

	AASHTO M 226 Table 1		AASHTO M 226 Table 2		AASHTO M 226 Table 3	
Freezing Index	Grade	*Min. Pen., 39°F	Grade	*Min. Pen., 39°F	Grade	*Min. Pen., 39°F
200	AC-10	13	AC-20	14	AR-40	14
	AC-5	14	AC-10	14	AR-20	15
500	AC-10	16	AC-20	16	AR-40	16
	AC-5	17	AC-10	17	AR-20	17
750	AC-5	18	AC-20	17	AR-20	19
	AC-2.5	20	AC-10	18	AR-10	20
1000	AC-5	20	AC-20	18	AR-20	21
	AC-2.5	22	AC-10	20	AR-10	22
1500	AC-2.5	28	AC-5 AC-2.5	28 29	AR-10	28
above 1500	AC-2.5	29	AC-5 AC-2.5	29 30	AR-10	29

Grades listed for any freezing index can also be used at a lower value of freezing index if summer stability is adequate. Generally, however, the hardest usable grade should be selected to minimize the chance of instability in hot weather.

*Minimum allowable penetration (AASHTO T 49, 200 g, 60 sec.) of residue from the thin film oven test (AASHTO T 179).


Freezing Index*
30 Year Data Base
LEGEND
*Updated from figure 12
of An Evaluation of Flexible
Pavement Design Methods,
Idaho Department of Highways
Research Project 20,
May 1964
500
200
750
1000
1500
BONNER
SHOSHONE
BENEWAH
LATAH
CLEARWATER
NEZ PERCE
LEWIS
IDAHO
ADAMS
VALLEY
WASHINGTON
GEM
PAYETTE
BOISE
ELMORE
BUTTE
CLARK
FREMONT
JEFFERSON
MADISON
TETON
BONNEVILLE
CAMAS
BLAINE
GOODING
LINCOLN
ADA
OWYHEE
JEROME
MINIDOKA
TWIN FALLS
CASSIA
POWER
BANNOCK
CARIBOU
BEAR LAKE
ONEIDA
FRANKLIN

REVIEW OF RECENT LITERATURE

Preliminary Remarks

A large body of literature on asphalt properties and performance has been built up. Much of the older literature is referenced, summarized, and evaluated in various reports published in the past several years. With few exceptions, this review includes only reports published in 1970 or later. Appropriate references to older literature can be found in the reports mentioned here, and in other asphalt literature not included in this review. The remarks in this section are purposely somewhat brief. The discussion sections contain more detailed information from several of the literature references.

Asphalt Composition

Asphalt contains many different organic compounds. In terms of general description, these compounds range from colorless liquids to dark solids, with many gradations between. In highway research literature, these compounds are often divided into five groups according to a chemical separation process devised by F. S. Rostler and his colleagues. The groups are called: asphaltenes, nitrogen bases, first acidaffins, second acidaffins, and paraffins. Other separation schemes giving different numbers of groups are sometimes used (2).

Asphalt's physical properties are strongly related to the proportioning of the various groups of compounds mentioned above (2, 3, 9, 10, 12, 14). Asphalt or asphalt component groups from widely different crude oil sources can be successfully blended as a means of modifying asphalt properties (3). In addition, very high molecular weight polymers such as rubber or synthetic polymers can be mixed into an asphalt as a second method of modifying the physical properties (4, 19). Air blowing is another way to change asphalt properties by changing the chemical composition (27).

Three recent studies (10, 12, 14) of field performance and/or artificial aging provide evidence that increasing the second acidaffins fraction may decrease the asphalt aging rate and increase pavement durability. Furthermore, high paraffin content increases temperature sensitivity of asphalt, which can reduce pavement durability at winter temperatures (10, 23).

Asphalt Physical Properties

For design purposes, information about the physical behavior of asphalt is of direct concern. Many different procedures are available to measure asphalt properties and to evaluate the property changes associated with temperature changes and aging. Different authors recommend different combinations of tests for adequate characterization of asphalt (6, 8).

A property of fundamental concern is viscosity. It is conveniently measured above room temperature by capillary tube viscometers. At lower temperatures where asphalt is less fluid, viscosity can be determined by sliding plate, rotating plate, or cone and plate viscometers. The empirical penetration test is essentially a low temperature viscosity test also, and penetration data for many asphalts can be correlated reasonably well with viscosity (8, 15). The correlation is less precise, however, for asphalts having significant shear susceptibility (20). Shear susceptibility is a measure of the viscosity change associated with a change in shearing rate.

Ductility at temperatures below normal room temperature and shear susceptibility have been mentioned as indicators of asphalt serviceability, particularly in connection with cold weather pavement cracking (9, 10, 12, 14, 19). Shear susceptibility can be measured with sliding or rotating viscometers. A relationship between ductility and shear susceptibility has been observed in laboratory testing (15). Penetration results may also reflect shear susceptibility (20).

Viscosity and penetration measurements can be used to describe the changes in asphalt behavior caused by temperature changes and aging. Temperature susceptibility can be evaluated by comparing viscosities or penetrations at two different temperatures (3, 6, 13, 23). Aging effects can be evaluated by comparing viscosities before and after oven aging (10, 14).

Resilient Modulus and Thermal Cracking

Asphalt is not a strictly elastic material. Nonetheless, within limits asphaltic concrete (AC) pavement does react elastically under the repeated transient wheel loadings typical of highway conditions. In the past few years, several methods have been suggested for measuring a resilient modulus (M_R) to characterize this behavior (5, 7, 11). M_R is the same type of property as the familiar Young's Modulus E, and has the same units.

In the past, M_R has had little practical application in pavement design due to the somewhat empirical nature of most standard pavement design procedures. Design methods are being developed which make extensive use of elastic theory and stress analysis (11). As these procedures come into wider use, M_R will necessarily be used more, since it is a fundamental design property in such methods.

The numerical value of M_R is a function of many factors, including duration of applied load, pavement temperature, asphalt content, asphalt viscosity, asphalt aging, and air voids (5, 11, 13). M_R generally increases as viscosity increases (11). It follows that asphalt aging and decreasing temperature should both lead to increases in M_R, because both conditions increase viscosity. The expected trends of increased M_R with aging or reduced temperature have been verified experimentally (11, 13, 16).

Laboratory fatigue testing shows an increase in pavement fatigue life at a given loading as temperature falls (11, 16). Tensile strength and M_R initially increase as temperature decreases. As M_R continues to increase, tensile strength increases to a maximum, then decreases (16). Also, as M_R increases, the asphalt becomes more brittle, which may in practice negate any beneficial effect on fatigue life. Under brittle conditions a slight unforseen overstress could fracture the asphalt, causing pavement cracking. Another feature of asphalt behavior is that as M_R increases and the asphalt becomes more brittle, less work is needed to fracture the pavement (1, 16).

Because high values of M_R lead to brittle behavior with greatly increased risk of pavement cracking, a number of technologists recommend the use of paving asphalts which will not develop undesirably high M_R at low winter temperatures (8, 13, 18, 22, 25). This would require a reasonably accurate means of measuring or estimating the M_R versus temperature and M_R versus aging relationships of a given asphalt. Procedures are available for estimating asphalt M_R using standard asphalt tests, and for predicting a minimum satisfactory service temperature (8, 13, 18, 22, 25). Such methods are based on combinations of theory, laboratory testing, and evaluation of field performance.

It should be emphasized that methods for predicting low temperature shrinkage cracking tendency often employ M_R of the asphalt itself, whereas design methods for load carrying capacity require M_R of the total paving mixture. As mentioned previously, M_R of a paving mix is influenced by a number of factors other than asphalt physical properties. In order to give proper consideration to all variables, the preferred method of estimating M_R for load capacity calculations is by physical tests on samples of paving mix, simulating field conditions as closely as possible. On the other hand, reasonable prediction of minimum satisfactory service temperature may be possible on the basis of asphalt properties alone. Brittleness of the asphalt may be the dominant factor with regard to shrinkage cracking at low temperatures. Formulas are available (24, 25) for estimating mix M_R based on asphalt M_R and mix composition, and have been used by some in preference to asphalt M_R alone for predicting low temperature cracking.

ASPHALT TESTING

Preliminary Remarks

Penetration, viscosity, and ductility tests were made on samples of fresh asphalt and on asphalt samples aged in the thin film oven (AASHTO T 179).

Penetration and ductility tests were done both at 77°F and 39°F. No suitable mechanical refrigeration unit was on hand to cool the water baths required for these tests. In some instances, ice was used to cool the baths, and in other cases the cooling was done with a mixture of dry ice and methanol.

Shear susceptibility is a property which is mentioned frequently in recent asphalt research literature. Due to the limited resources available for this study, shear susceptibility was not measured. Penetration and ductility test results can be influenced by shear susceptibility, but the influences may be hard to analyze without direct testing of this factor. Shear susceptibility will be discussed only briefly and in general terms because of the lack of specific data.

The chart on which penetration and viscosity results are plotted was developed by Shell Laboratories (8) and modified by Chevron Research Company (13). This chart has several useful features. First, it combines penetration and viscosity data on a single graph illustrating the properties of asphalts throughout a wide temperature range. In addition, the shape of the line defined by the test data gives a reliable indication of asphalt type. Many paving asphalts give straight lines on the chart. This behavior is called type S, and is illustrated in Figure A1.

A second characteristic behavior defined by using this chart is termed type W. If enough test points are available, the test results for type W asphalt plot as two roughly parallel straight line segments connected by a curved transition. Test points at only four temperatures cannot provide enough information for complete plotting of a type W curve. Nonetheless, data at 39°F, 77°F, 140°F and 275°F is often enough to define the two straight line portions reasonably well. The 77°F is sometimes within the transition zone, which can cause slight inaccuracy unless a nonstandard testing procedure is used. This consists of lowering the asphalt temperature below the transition range, then warming to 77°F for testing (8, 25). The modified procedure was not used in this project. When the two straight line portions of a type W asphalt plot are not parallel, their intersection forms a broad arrowhead pointing toward the lower left of the chart. See Figure A1 for an illustration of type W behavior.

A third characteristic set of properties is called type B and is associated with air-blown asphalts. Materials of this type often yield two straight line segments forming a broad arrowhead

pointing toward the upper right of the chart as seen in the upper curve of Figure A1. None of the asphalts tested in this project appear to be of this type.

Another feature of this chart is that the slope of a test data line is a measure of the temperature susceptibility of the asphalt. This slope can be used to find Penetration Index (PI) from the nomograph at the top of the chart. PI in turn can be used in another nomograph developed by Shell (8) to predict whether or not contraction cracking of the pavement is likely to occur at a given winter temperature. This will be discussed more fully in a later section. A type B or W asphalt may have a significantly different PI at winter temperatures than during summertime, as indicated by the slope changes mentioned in the proceeding paragraphs.

Test Results

All asphalt test results are given in Tables A1 through A6.

Penetration and viscosity results are presented graphically in Figures A2 through A7. Examples of both type W and type S behavior are seen.

Discussion - Asphalt Properties at Winter Temperatures

Low temperature asphalt performance has been a topic of some concern in Canada and the northern U.S. in recent years. Type W asphalts are of particular interest in this regard. The modified Shell test data chart mentioned earlier provides a graphical interpretation of the type W problem. As described previously, if adequate test data is available, test points for type W asphalt may form two nearly parallel straight line segments connected by a curved transition when plotted on the Shell test data chart. The transition range lies between winter and summer temperatures. The low temperature segment is always offset in the direction of stiffer or more viscous behavior. Also, type W asphalt data may plot as two intersecting straight line segments with the low temperature segment steeper than the high temperature portion. This contrasts with a type S paving asphalt, whose properties plot as a single straight line on the test data chart. Therefore, if a type W and a type S asphalt have the same properties at high temperatures, the type W materiall will always be stiffer at winter temperatures. In effect, it will behave as a stiffer grade of type S asphalt when the temperature drops below the transition range. This may be of little concern to pavement designers in areas where only type S asphalt is supplied and where winter temperatures remain high enough so that temperature-associated brittleness is not a problem. As the test data for this project show, however, some type W asphalt is supplied in Idaho. Furthermore, very cold winters occur regularly in parts of the State.

In cold climates, the low temperature properties of both type S and type W asphalts require consideration to minimize thermal cracking. Many asphalt specifications and pavement design proce-

dures consider mixing properties and warm summer stability, with no direct consideration for minimizing the possibility of brittle behavior at winter temperatures. Highway agencies in some Canadian Provinces and northern States have incorporated lower temperature requirements into their paving asphalt specifications, attempting to define the required properties in such a way that satisfactory performance is attained throughout the expected range of service temperature. In Utah and Washington, this has resulted in minimum requirements on ductility. Utah specifies a minimum ductility at 39°F, while Washington uses a temperature of 45°F. Reference 10 describes a Utah field test of four asphalts in which the two with higher ductility at 39°F exhibited considerably less transverse cracking than the two asphalts with poorer 39°F ductility. The province of British Columbia has adopted a low temperature specification based on penetration (18). Penetration limits are specified at 50°F and 39°F in addition to the usual temperature of 77°F. This specification was developed by observations of field cracking, together with computations of asphalt stiffness at expected exposure temperatures. R. J. Schmidt of Chevron Research Company proposed a similar method for estimating low temperature performance, using penetration at 39°F and 77°F (13). Based on laboratory testing and field observations of pavement cracking in Washington, he concluded the correlation between 39°F penetration and limiting stiffness is better than the correlation between 45°F ductility and limiting stiffness.

Both penetration and ductility tests were made at 39°F in our project, providing data for comparison between the two. First, Table A7 compares the variability of the two tests. Sample size for each variability determination is only two, so the results must be considered only as indicators rather than definitive measures of variability. A distinct trend is evident as asphalt grade goes from hard to soft. For the harder grades, ductility results are generally less variable than penetration. For the two softer grades, however, all ductility results show greater variability than the corresponding penetration results. In some cases, the ductility test variability appears undesirably large for these soft grades. Because softer grades are generally used to reduce cold weather cracking, and because the penetration test at 39°F apparently has better repeatability than the ductility test for these softer grades, penetration testing would seem preferable for a cold weather performance specification.

Despite the comments in the previous paragraph, ductility testing might seem necessary because it appears to measure an important property not evaluated by the penetration test. As Figure A8 shows, however, our testing indicates a notable degree of correlation between ductility and penetration at 39°F. To a considerable extent, then, penetration and ductility testing at 39°F appear to evaluate the same fundamental property or combination of properties. A qualitative explanation for such an association can be developed by considering the property of shear susceptibility mentioned in the literature review. Shear susceptibility and viscosity are both involved in the ductility test (12, 13). This can be visualized by first noting that viscosity itself is defined

in terms of shearing. Next, the ductility test obviously involves considerable internal shearing as the material is drawn out. Additionally, the shear rate probably varies during the test, incorporating shear susceptibility into the results. As mentioned in the literature review, the penetration test is also strongly related to viscosity. Additionally, the shear rate varies during the test because of the shape of the needle (20, 21). The fact that viscosity and shear susceptibility affect both penetration and ductility testing probably accounts at least partly for the observed correlation. Because of this correlation, the use of a minimum penetration in a low temperature specification would also tend to exclude asphalts of low ductility.

Another point of comparison between penetration and ductility testing at low temperature is the expense involved in performing the tests. If cost were to be the deciding factor in choosing between the two tests, the penetration test would appear preferable. No specific cost information was assembled for this comparison, but IDH asphalt testing technician Ellis Clark has estimated that the ductility test requires up to 50% more time than penetration testing, including sample preparation and cleanup. Moreover, the cost of equipment needed for routine testing at 39°F is likely to be considerably less for penetration than for ductility. Low temperature ductility test equipment is relatively expensive because in addition to the need for a refrigerated water bath, the mechanical design of the test device must be modified considerably to compensate for the increased tensile strength of asphalt at low temperature. In contrast, the only equipment expense needed for routine penetration testing at 39°F would be the cost of a refrigerated water bath for conditioning the asphalt samples, and the cost of a 60 second timer.

There is a second approach to predicting low temperature asphalt stiffness which does not require physical testing at temperatures below 77°F. Only the 77°F penetration and the viscosity at 140°F or 275°F are used. A measure of temperature susceptibility is obtained from these results, then a modified Shell stiffness nomograph is used to predict stiffness at low temperatures. This method (25) was developed by McLeod. The Shell test data charts discussed previously can be used to visualize why this method may be effective. For a type S asphalt, of course, test data at any two points will completely define the temperature susceptibility because of the straight line behavior. Low temperature behavior in such a case can be predicted accurately by linear extrapolation on the data chart. Although type W asphalt cannot be completely characterized by two points, a line defined by data at 77°F and 140°F or 275°F may be influenced by type W behavior because 77°F is sometimes in the transition range. In such a case, the line will be steeper than the true high temperature branch, thus indicating greater temperature susceptibility and possibly brittle properties at low temperature. This is the same effect which would result from a completely defined type W curve, so the error introduced by the two-point approximation tends to be self-compensating if 77°F is not above the transition range. A possible objection to this method is that the two-point McLeod

line may not be as steep as the true low temperature branch of the type W curve, thus underestimating the effect on low temperature behavior.

Either the Schmidt or McLeod procedure might be used to establish specification limits to minimize thermal cracking. Under the Schmidt method the temperature susceptibility of asphalt is restricted by specifying a minimum penetration at 39°F in addition to the usual limits at higher temperatures. This directly affects the temperature susceptibility of the low temperature branch of any type W asphalt performance curve. Under the McLeod method, on the other hand, temperature susceptibility is specified between 77°F and some higher temperature, using a penetration index. Either method could be used in conjunction with the specifications for various AASHTO grades of paving asphalt to estimate low temperature behavior under various limitations on 39°F penetration or penetration index. The Schmidt method was selected for use in this project largely because it makes direct use of 39°F test data, whereas the McLeod method extrapolates to this temperature range based on measurements at 77°F and above.

Figure A9 shows some low temperature behavior differences allowable under existing specifications. Three AASHTO specifications requiring nearly the same residue viscosity (140°F) are plotted, and a line of greatest allowable temperature susceptibility for type S asphalt is drawn in each case. To estimate residue properties, the lines for the two AC-10 grades have been shifted parallel to the steepest lines allowed by the specifications for unaged material. It is seen that Tables 1 and 3 allow noticeably greater temperature susceptibility (steeper lines) than Table 2. The Schmidt procedure was used to predict the temperature at which cold weather cracking might be expected in each case. This limiting temperature was computed to be +6°F for Tables 1 and 3 and -7°F for Table 2, assuming type S asphalt. Type W asphalts conforming to M 226 might have higher cracking temperatures. To insure type S behavior in the foregoing example, minimum residue penetration at 39°F would have to be specified as 13 for Tables 1 and 3 and 19 for Table 2, in addition to the existing specification limits. Similar treatment of each grade in AASHTO M 226 shows the same effect: Table 2 provides better low temperature type S properties than Tables 1 or 3, but the effect of type W behavior is not well controlled by the existing specification.

As an indicator of the potential for low temperature pavement cracking in Idaho, the Schmidt method was used to predict the minimum acceptable service temperatures for the 85-100 and 120-150 asphalts tested in this project. The computed minimums for 85-100 ranged from +6 to -14°F while the range for 120-150 was -5 to -23°F, based on properties of thin film oven residue. There are several areas within the State where winter temperatures often fall to these values or lower. In at least one of these areas, extensive transverse pavement cracking has been documented on a highway section where 60-70 grade asphalt cement was used in the pavement (26).

The lowest temperature expected during the design life is not always used as the critical temperature for estimating cracking tendency. Haas (25) mentions the use of a design temperature "at or below which only 1 percent of the hourly air temperatures in January occur for the severest winter during a 10 year period". He presents a graph developed by Hajek relating this temperature to freezing index. Freezing index data were compiled for Idaho under IDH research project 20 and can be used to estimate winter design temperatures on the basis of the Hajek relationship.

Appendix E outlines the procedure used in this project to relate low-temperature penetration, design temperature and freezing index. The effects of long-term asphalt aging are not considered in the procedure. Accelerated simulation of in-service aging was not possible within the limited scope of the project. In addition, no field data is available on long-term changes in field penetration at 39°F for asphalts in Idaho pavements, so no compensating factor could be derived on the basis of existing pavements. The procedure does, however, include an approximation of aging associated with construction. This results from specifying penetration on asphalt which has undergone thin film oven conditioning.

Economic Considerations

When a specification change is proposed, its economic effects should be carefully considered. If low temperature penetration requirements were added to existing specifications, some asphalt suppliers might have to resort to blending, air blowing or other methods to produce the specified material. This might increase asphalt prices, but the amount of increase is impossible to predict. The intent of such a specification change would be to reduce cold weather transverse cracking and the resultant problems of crack filling, subsidence at the cracks, and early repair or replacement of the affected pavement. The monetary benefit associated with such improvements could possibly be estimated by reviewing IDH accounting records and interviewing maintenance foremen. Without information on likely asphalt cost increases, however, an accurate estimate of the overall economic effect cannot be made. The recommendation for a modified asphalt specification has, therefore, been made on the basis of technical considerations only, since insufficient information is available to assess the overall economic effect.

ASPHALTIC CONCRETE (AC) TESTING

Preliminary Remarks

Tensile split tests were carried out on specimens made with fresh 120-150 asphalt and on a separate series of specimens made with aged 120-150 asphalt. Aging was done by heating the asphalt at 275°F in an insulated kettle until the penetration at 77°F was the same as the penetration of residue from the thin film oven test. Continuous mechanical stirring was used to promote temperature uniformity and to introduce some air into the material. This method of aging was chosen to produce the necessary large quantity of aged material in a relatively short time. It was assumed that by matching the 77°F penetration with that of thin film oven residue, a material approximating thin film residue was produced. Thus the aging comparisons may not be precisely the same as if thin film residue had been used. It is assumed this has no significant effect on the qualitative validity of the comparisons.

During the tensile split tests, horizontal displacement of each specimen was measured to permit calculation of M_R. A bridge-type strain gage transducer based on Lottman's design (7) was built for these measurements. Output of the transducer was fed into a strip chart recorder. Photographs of the test equipment appear in Appendix C. The method described by Lottman (7) was used to estimate M_R for instantaneous loading.

The effect of aggregate type was investigated by performing three separate test series, each using aggregate from a different Idaho source. The same 1/2 inch gradation was used for all specimens, but asphalt content was determined separately for each aggregate source using the Hveem mix design method. This resulted in slight differences in air voids also. The same asphalt content was used for aged asphalt as for fresh. The use of slightly different asphalt content for each aggregate type might seem to cloud the results of comparisons between pits, since changes in asphalt content and voids may cause changes in tensile strength and M_R. Even though the mixes incorporate variables other than aggregate source, the between pit comparisons are believed valid in terms of field application. The reason for this is that the mix made with each aggregate incorporates the combination of variables which would be specified for that aggregate on a construction project.

AC specimens to be tested in the water-soaked condition at 77°F were brought to temperature in a laboratory-type controlled temperature water bath. Specimens to be tested at 39°F, either dry or soaked, were brought to temperature in a kitchen refrigerator set to maintain a nominal temperature of 39°F as verified with an accurate thermometer. Specimens to be tested dry at a nominal 77°F were temperature conditioned in the open room, where the temperature varied over a range of several degrees in the mid 70's.

All tensile split testing was done in the open room. The numerical values obtained from this testing are subject to slightly greater uncertainty than they would have been if more stringent temperature control had been exercised. Nonetheless, it is believed this does not seriously affect the validity of comparisons among the test results.

Test Results

Table B1 contains information about the three aggregate sources and the three mix designs.

All test results are presented in Tables B2 through B5.

Discussion of Test Results

Table B6 summarizes M_R values for dry specimens tested at room temperature. This is the only group of specimens for which a nearly complete set of M_R test results was obtained, for reasons which are discussed in Appendix E. Inspection of the row and column averages reveals wider variations among aggregate sources than among asphalt suppliers. Two-way analysis of variance was performed on each of the data groups of Table B6 to investigate the effect more thoroughly. Phillips was omitted from the analysis because of the missing data point for Ad-95.

Tables B7 and B8 summarize the analyses for unaged and aged asphalt respectively. In each case, the asphalt brand is found to have no statistically significant effect on M_R. Aggregate source, on the other hand, is found to exert a statistically significant influence on M_R in both data sets. Under the specific conditions of this project, then, M_R is influenced considerably more by aggregate source than by asphalt source. In attempting to generalize the results of these two analyses, however, it should be kept in mind that although the group of asphalt brands includes most of those in common use within Idaho, the aggregate pit group includes only a very small percentage of the aggregate sources within the State. Also, at least one of the aggregate sources (Bg-77) was chosen for testing because of known performance problems, and it consistently produced mixes with much lower M_R than did the other two sources. Because of the small sample size and non-random selection procedure, the aggregate source effect measured here may reflect a bias which limits its generalization. On the basis of these results, it is apparent that M_R can be influenced very strongly by aggregate source. Nonetheless, not enough testing was done to rule out the possibility that this influence is characteristic of a relatively small number of sources.

Analysis of variance procedures test the statistical possibility that several experimental results came from the same population. The mathematical meaning must be kept in mind when applying the results described in the foregoing paragraph. The statement that asphalt brand has no statistically significant effect

on M_R does not necessarily mean no real differences exist among the suppliers. It merely means that the observed properties could, with high probability, have come from the same population. Even so, some within-population variation must be expected. In fact, Table B6 shows that M_R may vary by a factor of 2 or more as asphalt source is changed. This is much less than the maximum variation resulting from changes in aggregate source, but might nonetheless require consideration in the design process, depending on how strongly it affects the design.

In reviewing the test results for M_R and tensile strength, a possible correlation between the two was observed. These test data are shown graphically in Fibure B1. The plotted data include values for aged and unaged asphalt, dry and soaked samples, room temperature and 39°F, all six asphalt brands and all three aggregate sources. Obviously considerable interrelation exists between M_R and tensile strength, at least within the range of variables investigated here. Previous research by others (16) has shown divergence between tensile strength and modulus at temperatures below about 40°F, so a graph such as Figure B1 would probably show a less distinct relationship if test values at lower temperatures were included.

One of the important considerations in AC pavement design and performance is water susceptibility. This may be measured in various ways, but in this work it was evaluated by using tensile strength ratio (TSR). TSR is the ratio of tensile strength of a dry specimen to the tensile strength of an identical specimen saturated with water. Table B9 contains the TSR results for this project. Visual inspection indicates asphalt source has little effect on TSR, but aggregate source can have considerable effect. In order to check the statistical validity of these observations, two-way analysis of variance was performed on each of two of the data groups of Table B9. The aged room temperature group and the aged 39°F group were chosen for this statistical testing. In each case, asphalt content was found to be a statistically insignificant factor and aggregate was found to be statistically significant with a high degree of confidence. Aggregate influence is, of course, already well known from previous experience and research.

Specimens made with aged asphalt exhibited substantially higher M_R than corresponding specimens made with fresh binder. This is to be expected as a result of asphalt hardening which accompanies the aging process. The comparisons illustrate that the aging process can materially increase pavement stiffness. As mentioned earlier, increased asphalt stiffness is accompanied by a tendency toward brittle behavior, so asphalt pavements would generally be epxected to become less resistant to thermally induced cracking as they age in service. No correlation was available to relate the laboratory aging effect to an equivalent in terms of field service.

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RESEARCH PROJECT NO. 62

Table A1

AMERICAN

ASPHALT GRADE	60 - 70 *				85 - 100				120 - 150				200 - 300 *			
CONDITION TESTED	ORIGINAL		RESIDUE		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C					89	90	47	47	138	138	59	60				
Penetration, 4° C					23	21	13	14	31	30	19	20				
Viscosity, 135° C, Cs					233	233	358	356	172	172	271	273				
Viscosity, 60° C, P					809	801	2293	2287	450	458	1561	1564				
Ductility, 25° C					150+	150+	150+	150+	147	150	150+	150+				
Ductility, 4° C					10.5	10.0	1.5	1.5	14.0	17.5	4.7	4.0				
Softening Point, °F					111.5	112.5	122.0	122.0	108	108	120.5	120.0				
L.O.H. Test, 5 hr., %					.46	.47		-	.57	.64						
Flashpoint, C.O.C., °F					506	510			485	480						
Solubility, %					99.48	99.40			99.35	99.40						
Specific Gravity 25°C					1.015				1.0212							
Spot Test, 35% Z					Neg.				Neg.							
Anti-strip Test					Neg.				Neg.							

REMARKS

* American didn't furnish these grades.

° Residue from loss on heating test (AASHTO T 179)

RESEARCH PROJECT NO. 62

Table A2
CHEVRON

ASPHALT GRADE	60 - 70				85 - 100				120 - 150				200 - 300			
CONDITION TESTED	ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C	58	57	36	37	88	88	55	55	149	146	77	77	300	301	159	162
Penetration, 4° C	25	25	12	12	39	39	21	22	47	47	30	30	89	89	64	59
Viscosity, 135° C, Cs	512	507	1091	1081	347	351	562	569	235	240	366	364	172	172	246	243
Viscosity, 60° C, P	3638	3504	19860	21532	1602	1573	5022	5242	729	740	2047	2011	282	284	765	758
Ductility, 25° C	150+	150+	47	67	150+	150+	150+	150+	114	122	150+	150+	To fluid to Test		80	75
Ductility, 4° C	8.0	7.5	0.5	0.5	13.0	15.0	3.5	3.5	150+	150+	8.5	9.0	150+	150+	38	61
Softening Point, °F	124	124	139	141	116	116	126	126.5	108	110	118.5	119.5	99	98.5	107.5	106.5
L.O.H. Test, 5 hr., %	.498	.516			.459	.449			.380	.392			.288	.279		
Flashpoint, C.O.C., °F	505	510			515	510			525	520			560	565		
Solubility, %	99.80	99.82			99.78	99.85			99.82	99.81			99.73	99.84		
Specific Gravity 25°C	1.028				1.023				1.019				1.011			
Spot Test, 35% Z	Neg.				Neg.				Neg.				Neg.			
Anti-strip Test	Neg.				Neg.				Neg.				Neg.			

REMARKS ° Residue from loss on heating test (AASHTO T 179)

RESEARCH PROJECT NO. 62

Table A3

EXXON

ASPHALT GRADE	60 - 70				85 - 100				120 - 150				200 - 300			
CONDITION TESTED	ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE°	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C	61	62	39	39	80	81	55	55	134	136	75	74	237	241	124	123
Penetration, 4° C	23	21	16	17	35	37	22	22	43	43	30	29	60	59	40	41
Viscosity, 135° C, Cs	350	350	499	501	277	276	356	399	211	213	292	296	148	148	202	207
Viscosity, 60° C, P	2293	2328	6347	6358	1610	1555	4149	3959	708	698	1713	1695	329	331	670	676
Ductility, 25° C	150+	150+	150+	150+	150+	150+	150+	150+	120	126	150+	150+	--	--	150+	150+
Ductility, 4° C	9.5	8.5	6.5	6.5	11	11	4.0	4.0	78	96	6.5	7.0	150+	150+	59.0	63.5
Softening Point, °F	116.5	117.5	127.0	127.0	116	116	125	125.5	107.5	109.5	119.0	118.0	95.5	96.0	104.0	102.5
L.O.H. Test, 5 hr., %	.01	.01			+.104	+.107			+.043	+.053			+.05	+.06		
Flashpoint, C.O.C., °F	595	600			615	610			605	605			565	563		
Solubility, %	99.91	99.90			99.89	99.88			99.87	99.84			99.76	99.73		
Specific Gravity 25°C	1.030				1.025				1.022				1.018			
Spot Test, 35% Z	Neg.				Neg.				Neg.				Neg.			
Anti-strip Test	Neg.				Neg.				Neg.				Neg.			

REMARKS + Weight Gain

° Residue from loss on heating test (AASHTO T 179)

RESEARCH PROJECT NO. 62

Table A4

FARMERS

ASPHALT GRADE	60 - 70				85 - 100				120 - 150				200 - 300			
CONDITION TESTED	ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE°	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C	64	63	42	43	92	92	58	58	116	117	67	70	235	234	116	116
Penetration, 4° C	22	20	14	15	35	34	20	21	40	42	27	26	52	51	36	37
Viscosity, 135° C, Cs	337	336	458	479	247	248	349	350	224	223	302	303	155	155	209	210
Viscosity, 60° C, P	1928	1923	4855	4876	1029	1041	2653	2501	810	800	1847	1768	455	450	735	715
Ductility, 25° C	150+	150+	150+	150+	150+	150+	150+	150+	142	150+	150+	150+	--	--	120	128
Ductility, 4° C	9.0	9.0	6.5	6.5	14.5	11.0	8.0	8.0	34	42	6.0	6.5	150+	150+	49.0	24.5
Softening Point, °F	117.5	117.0	125.5	126.0	114	114	123	123.5	109	109	119.0	119.5	95.5	96.0	106.0	105.0
L.O.H. Test, 5 hr., %	+.0048	+.0136			+.0078	+.012			.009	.010			.02	.03		
Flashpoint, C.O.C., °F	590	585			625	615			605	605			580	575		
Solubility, %	99.86	99.86			99.46	99.58			99.72	99.72			99.70	99.70		
Specific Gravity 25°C	1.027				1.019				1.019				1.015			
Spot Test, 35% Z	Neg.				Neg.				Neg.				Neg.			
Anti-strip Test	Neg.				Neg.				Neg.				Neg.			

REMARKS

+ Weight Gain

° Residue from loss on heating test (AASHTO T 179)

RESEARCH PROJECT NO. 62

Table A5

HUSKY

ASPHALT GRADE	60 - 70				85 - 100				120 - 150				200 - 300			
CONDITION TESTED	ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE°		ORIGINAL		RESIDUE°	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C	64	64	40	40	91	92	58	58	125	125	75	75	234	230	127	127
Penetration, 4° C	22	20	16	16	25	28	18	19	35	34	29	31	64	64	39	40
Viscosity, 135° C, Cs	422	428	614	618	339	339	473	472	277	277	390	395	177	177	249	252
Viscosity, 60° C, P	2527	2511	5987	5902	1563	1560	3301	3212	1001	1043	2284	2246	398	398	856	839
Ductility, 25° C	150+	150+	150+	150+	150+	150+	150+	150+	129	142	150+	150+	--	--	140	142
Ductility, 4° C	11.0	10.5	6.0	6.5	33.0	34.5	9.0	9.0	150+	150+	15.0	13.5	150+	150+	139.0	133.0
Softening Point, °F	116.5	116.5	125.5	126.5	108.5	108.5	122.0	120.5	106.0	104.5	116.5	115.0	95.5	95.0	101.0	102.0
L.O.H. Test, 5 hr., %	.15	.15			.13	.13			0.22	0.22			.31	.30		
Flashpoint, C.O.C., °F	590	592			585	590			565	562			510	515		
Solubility, %	99.87	99.88			99.85	99.85			99.88	99.86			99.82	99.87		
Specific Gravity 25°C	1.025				1.029				1.0266				1.023			
Spot Test, 35% Z	Neg.				Neg.				Neg				Neg.			
Anti-strip Test	Neg.				Neg				Neg				Neg.			

REMARKS ° Residue from loss on heating test (AASHTO T 179)

RESEARCH PROJECT NO. 62

Table A6

PHILLIPS

ASPHALT GRADE	60 - 70				85 - 100				120 - 150				200 - 300			
CONDITION TESTED	ORIGINAL		RESIDUE °		ORIGINAL		RESIDUE°		ORIGINAL		RESIDUE°		ORIGINAL		RESIDUE °	
SAMPLE NUMBER	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2
Penetration, 25° C	58	58	35	37	83	83	50	50	126	126	70	69	232	232	109	113
Penetration, 4° C	18	18	13	12	33	33	20	18	38	39	27	29	59	59	38	42
Viscosity, 135° C, Cs	345	347	533	529	276	272	421	420	224	224	324	324	170	171	257	254
Viscosity, 60° C, P	1441	1438	5197	5301	909	918	2651	2779	604	599	1586	1654	315	316	986	985
Ductility, 25° C	150+	150+	150+	150+	144	150+	150+	150+	114	115	150+	150+	To fluid to test		117	120
Ductility, 4° C	6.5	6.5	1.0	1.0	9.5	9.5	3.0	3.0	23	25	4.5	5.0	95	150	7.5	6.5
Softening Point, °F	121	122	132.5	133	118	118	126.5	127.5	113.5	114	121.5	121.5	106.5	107.5	116.5	116.5
L.O.H. Test, 5 hr., %	.404	.404			.430	.433			.493	.490			.575	.580		
Flashpoint, C.O.C., °F	505	505			500	500			490	500			495	500		
Solubility, %	99.63	99.56			99.77	99.82			99.56	99.56			99.60	99.65		
Specific Gravity 25°C	1.021				1.016				1.014				1.011			
Spot Test, 35% Z	Neg.				Neg.				Neg.				Neg.			
Anti-strip Test	Neg.				Neg.				Neg.				Neg.			

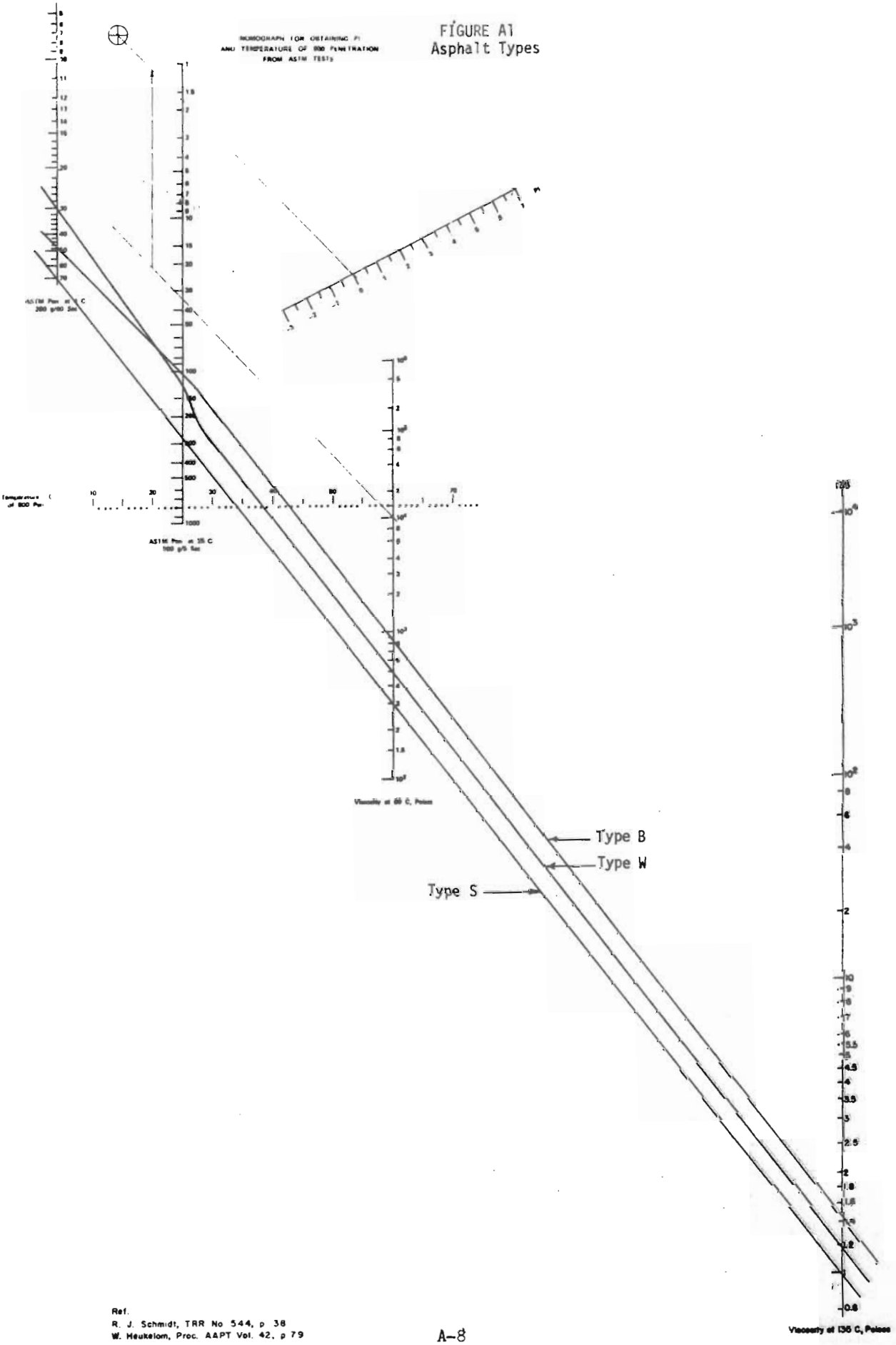
REMARKS °Residue from loss on heating test (AASHTO T 179)

Table A 7

Testing Variability, Penetration vs. Ductility, 39°F
Thin Film Oven Residue (AASHTO T 179)

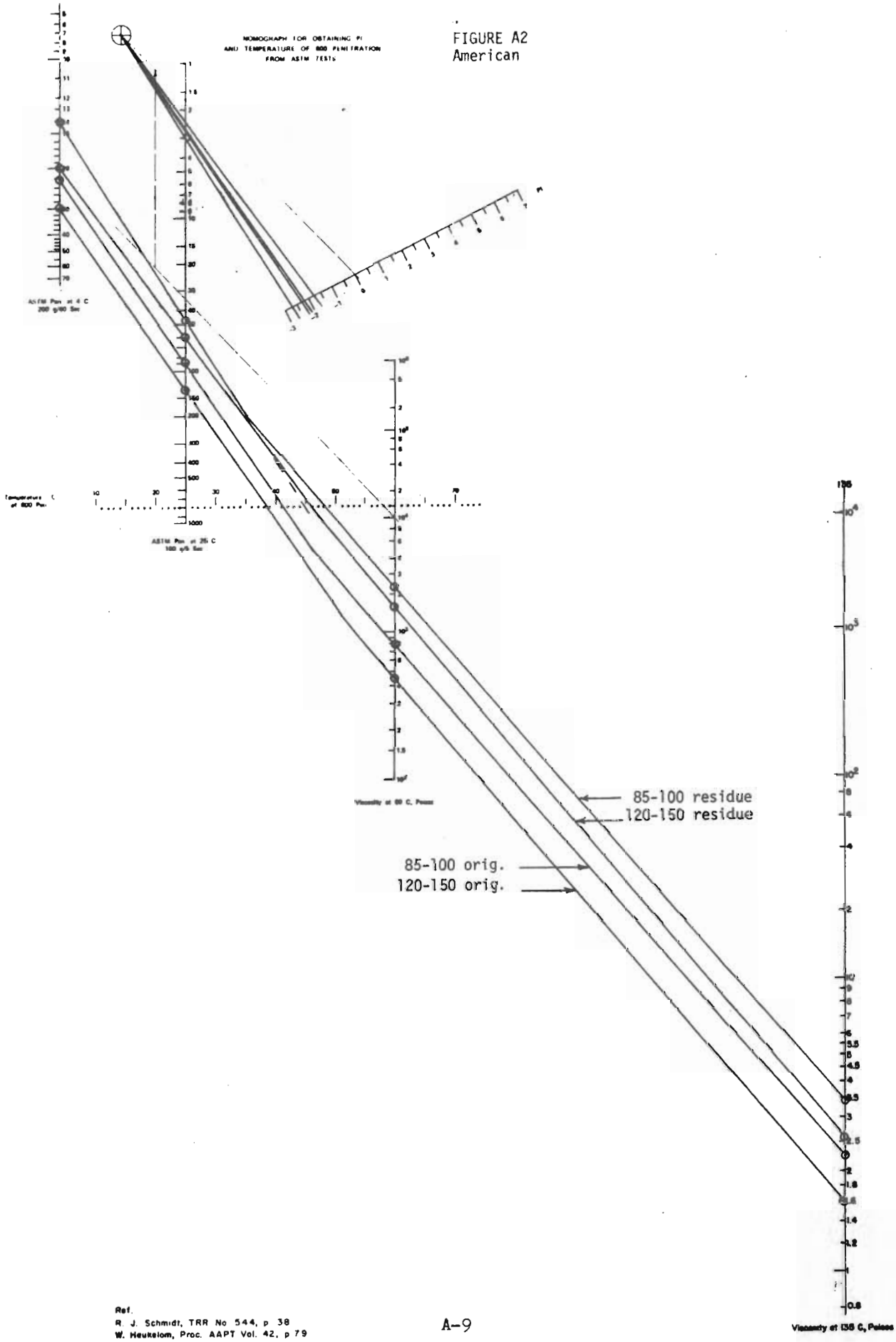
Grade	Supplier	(A) Avg. Pen.	(B) Pen. Range	(C) Avg. Duct.	(D) Duct. Range	(B)/(A) Pen. Variability	(D)/(C) Duct. Variability
60-70	American	--	--	--	--	--	--
	Chevron	12	0	0.5	0.0	.00	.00
	Exxon	16.5	1	6.5	0.0	.06	.00
	Farmers	14.5	1	6.5	0.0	.07	.00
	Husky	16	0	6.2	0.5	.00	.08
	Phillips	12.5	1	1.0	0.0	.08	.00
85-100	American	13.5	1	1.5	0.0	.07	.00
	Chevron	21.5	1	3.5	0.0	.05	.00
	Exxon	22	0	4.0	0.0	.00	.00
	Farmers	20.5	1	8.0	0.0	.05	.00
	Husky	18.5	1	9.0	0.0	.05	.00
	Phillips	19	2	3.0	0.0	.15	.00
120-150	American	19.5	1	4.4	0.7	.05	.16
	Chevron	30	0	8.8	0.5	.00	.06
	Exxon	29.5	1	6.8	0.5	.03	.07
	Farmers	26.5	1	6.2	0.5	.04	.08
	Husky	30	2	14.2	1.5	.07	.11
	Phillips	28	2	4.8	0.5	.07	.10
200-300	American	--	--	--	--	--	--
	Chevron	61.5	5	49.5	23.0	.08	.46
	Exxon	40.5	1	61.2	4.5	.02	.07
	Farmers	36.5	1	36.8	24.5	.03	.67
	Husky	39.5	1	136.0	6.0	.03	.04
	Phillips	40	4	7.0	1.0	.10	.14

FIGURE A1
Asphalt Types



Ref.
R. J. Schmidt, TRR No 544, p 38
W. Heukelom, Proc. AAPT Vol. 42, p 79

FIGURE A2
American



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R. J. Schmidt, TRR No 544, p 38
W. Heukelom, Proc. AAPT Vol. 42, p 79

FIGURE A3
Chevron

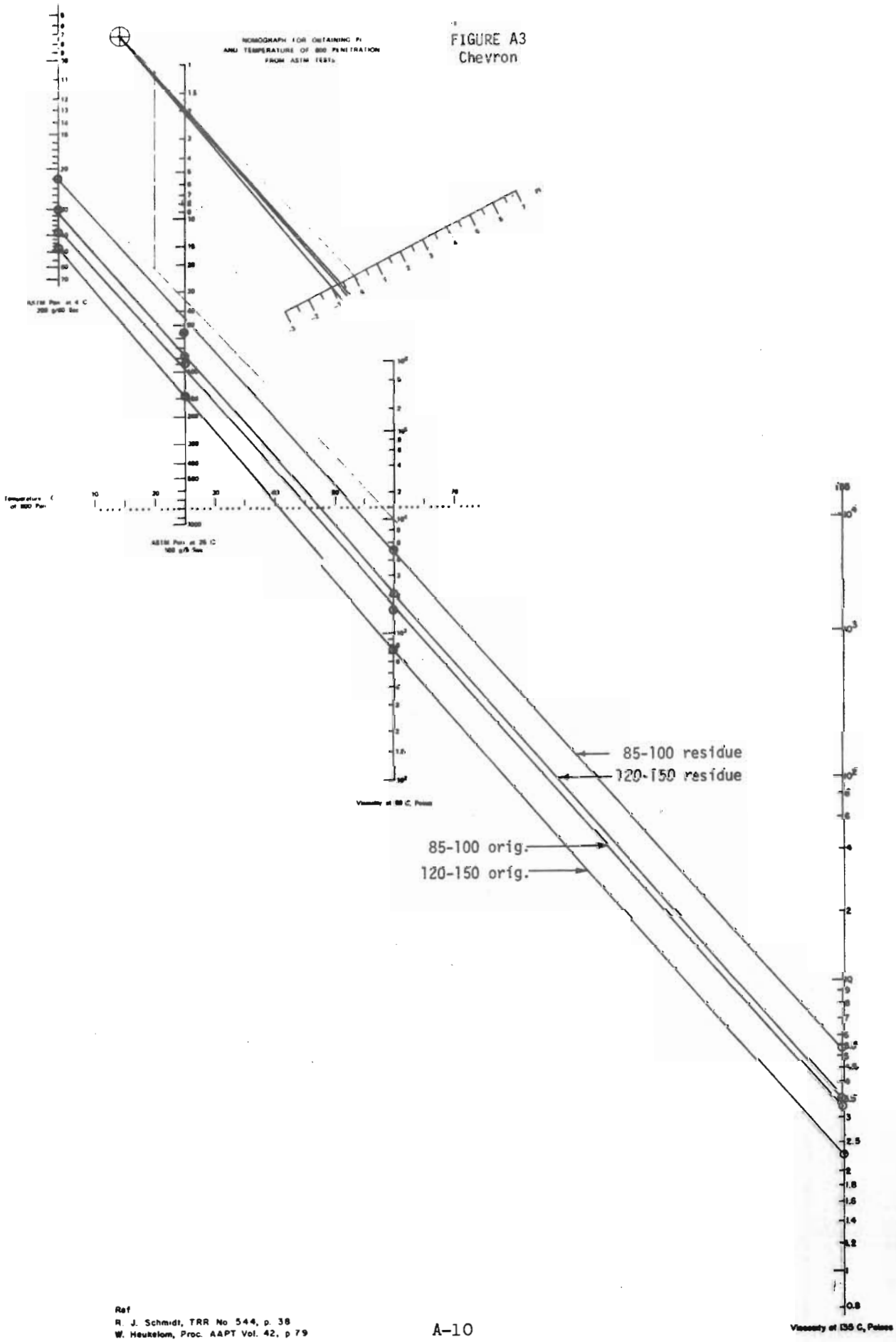

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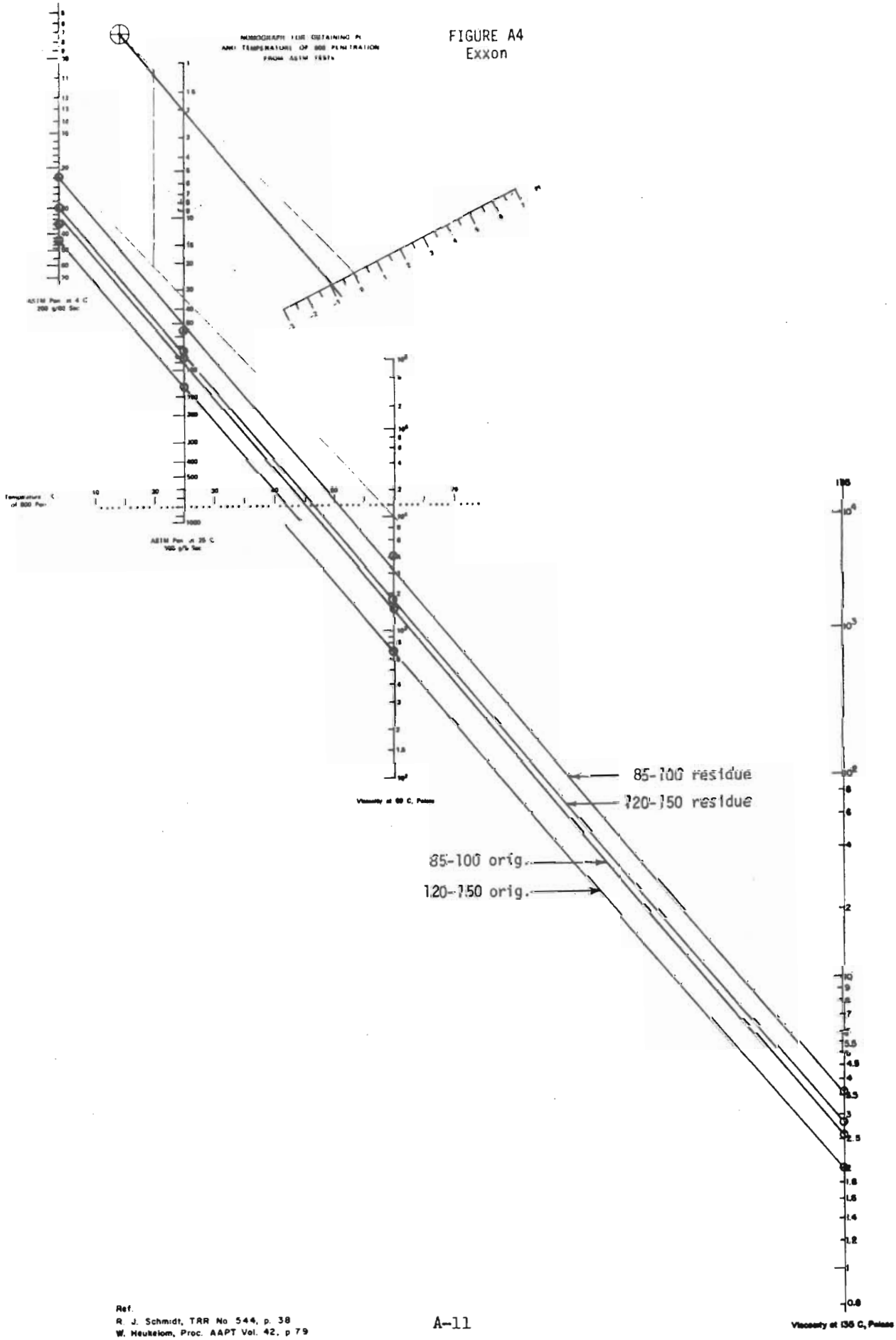
FIGURE A4
Exxon



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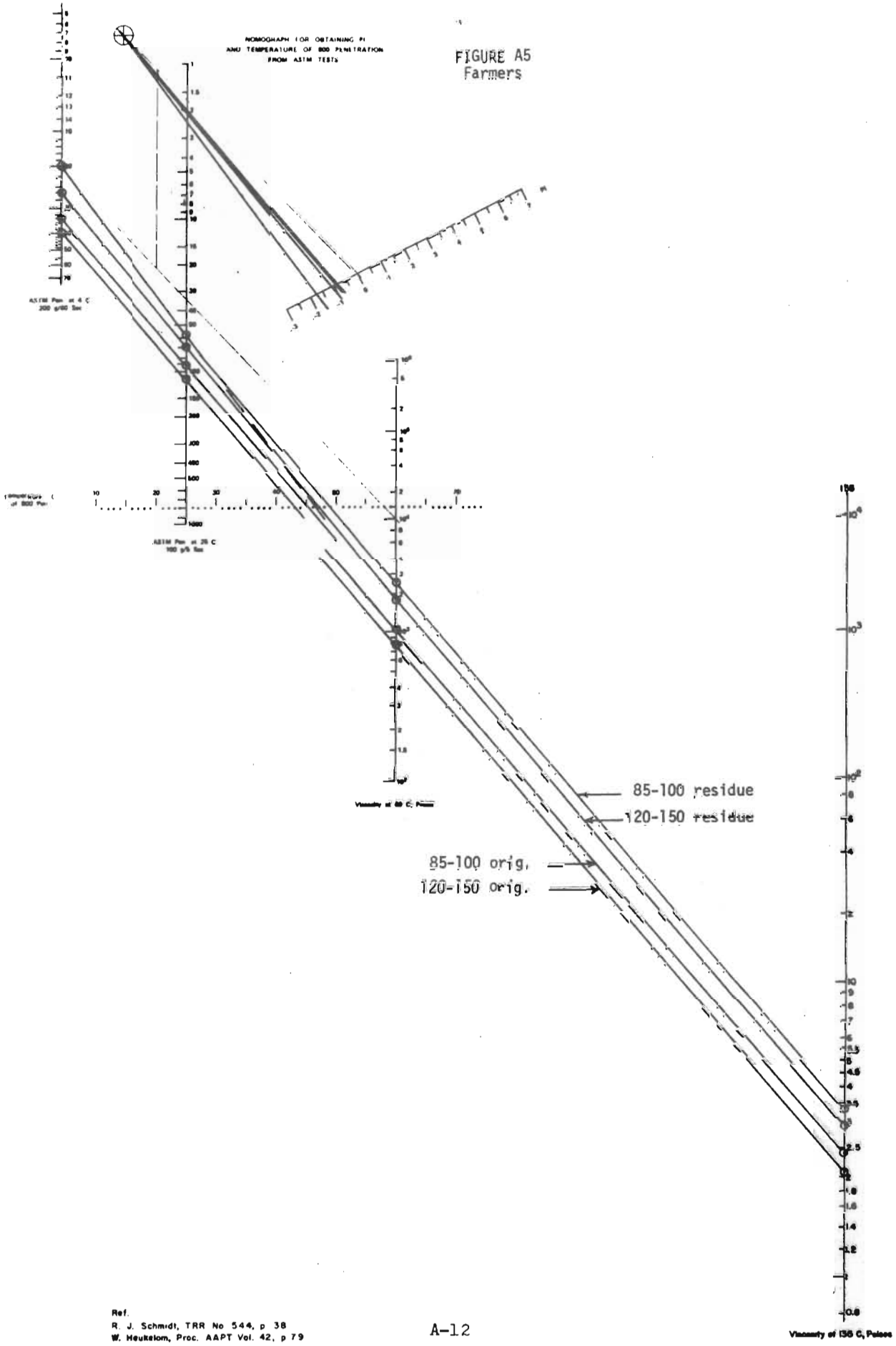

FIGURE A5
Farmers

Ref.
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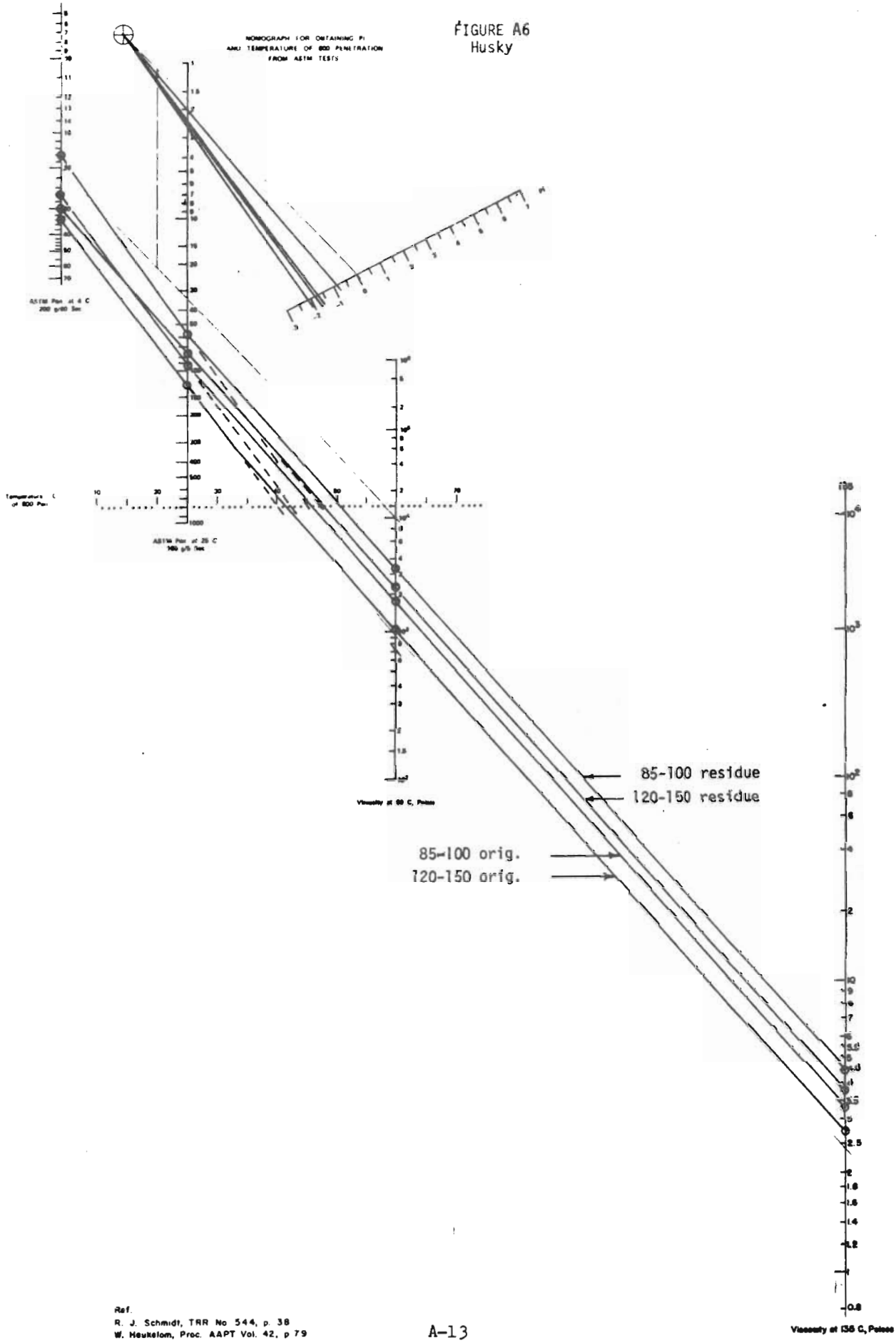
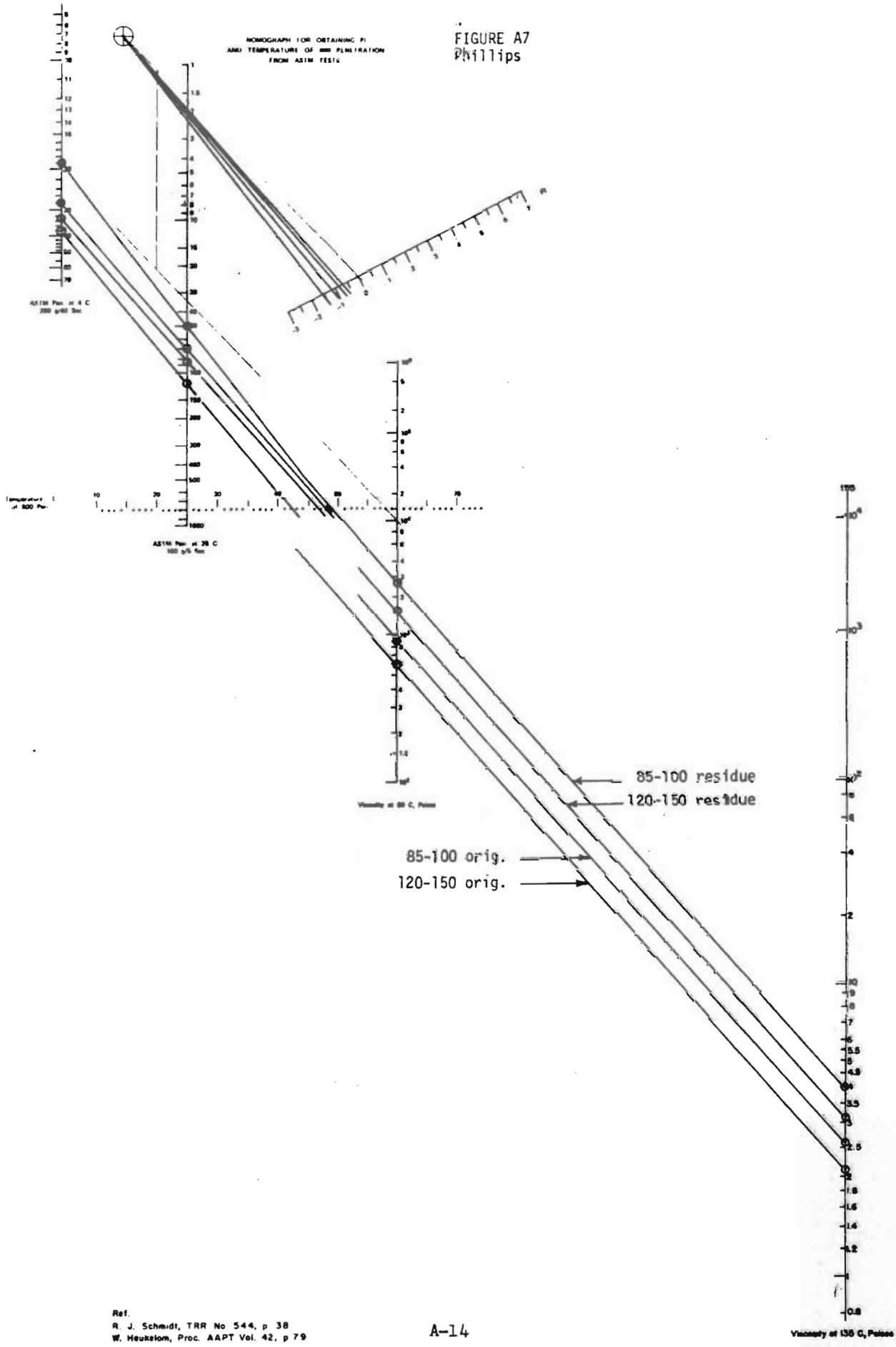
FIGURE A6
Husky



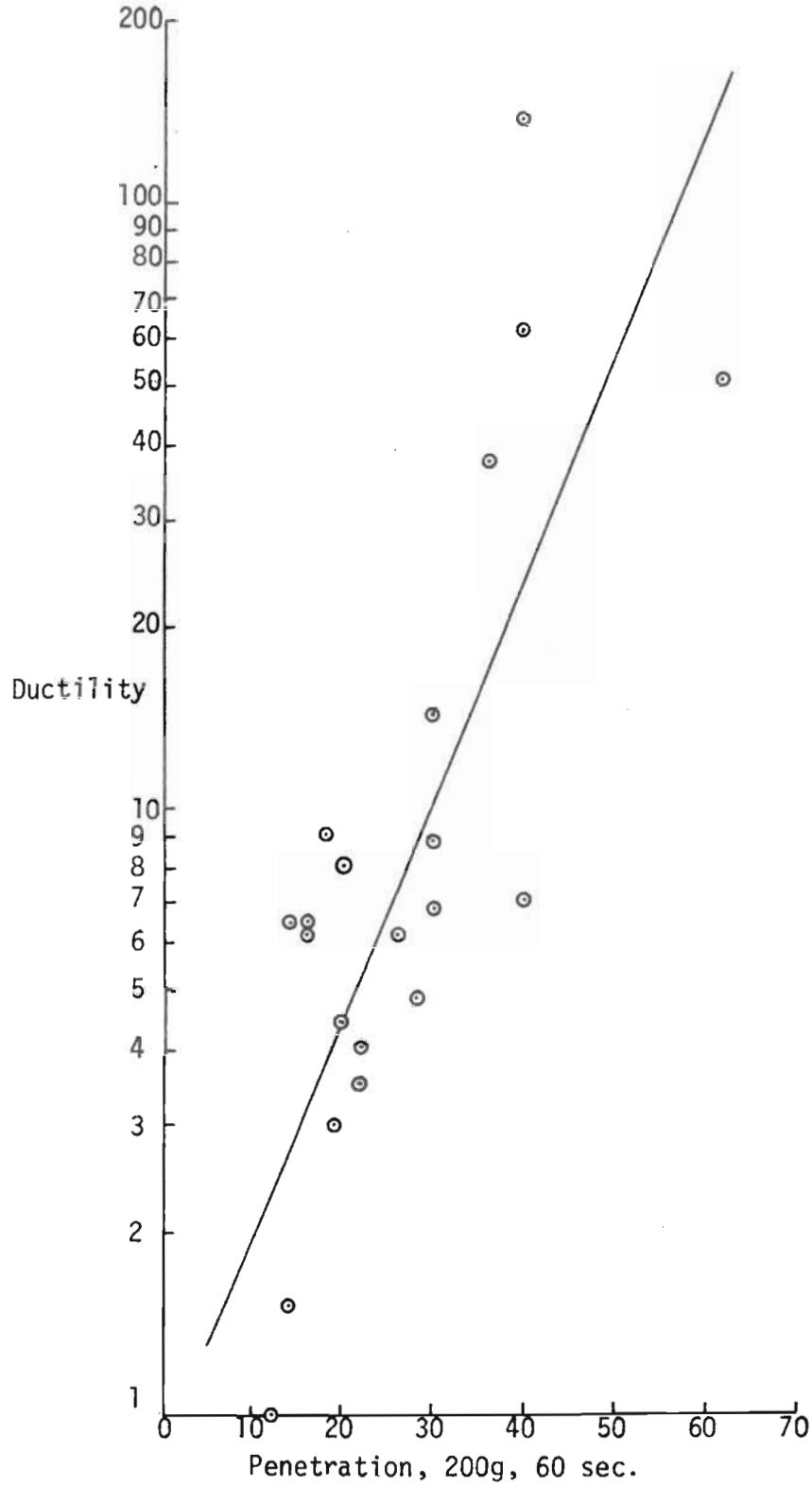
Ref.
R. J. Schmidt, TRR No 544, p. 38
W. Heukelom, Proc. AAPT Vol. 42, p 79

FIGURE A7
Phillips



Ref.
R. J. Schmidt, TRR No 544, p 38
W. Heukelom, Proc. AAPT Vol. 42, p 79

FIGURE A8
Penetration and Ductility
39°F



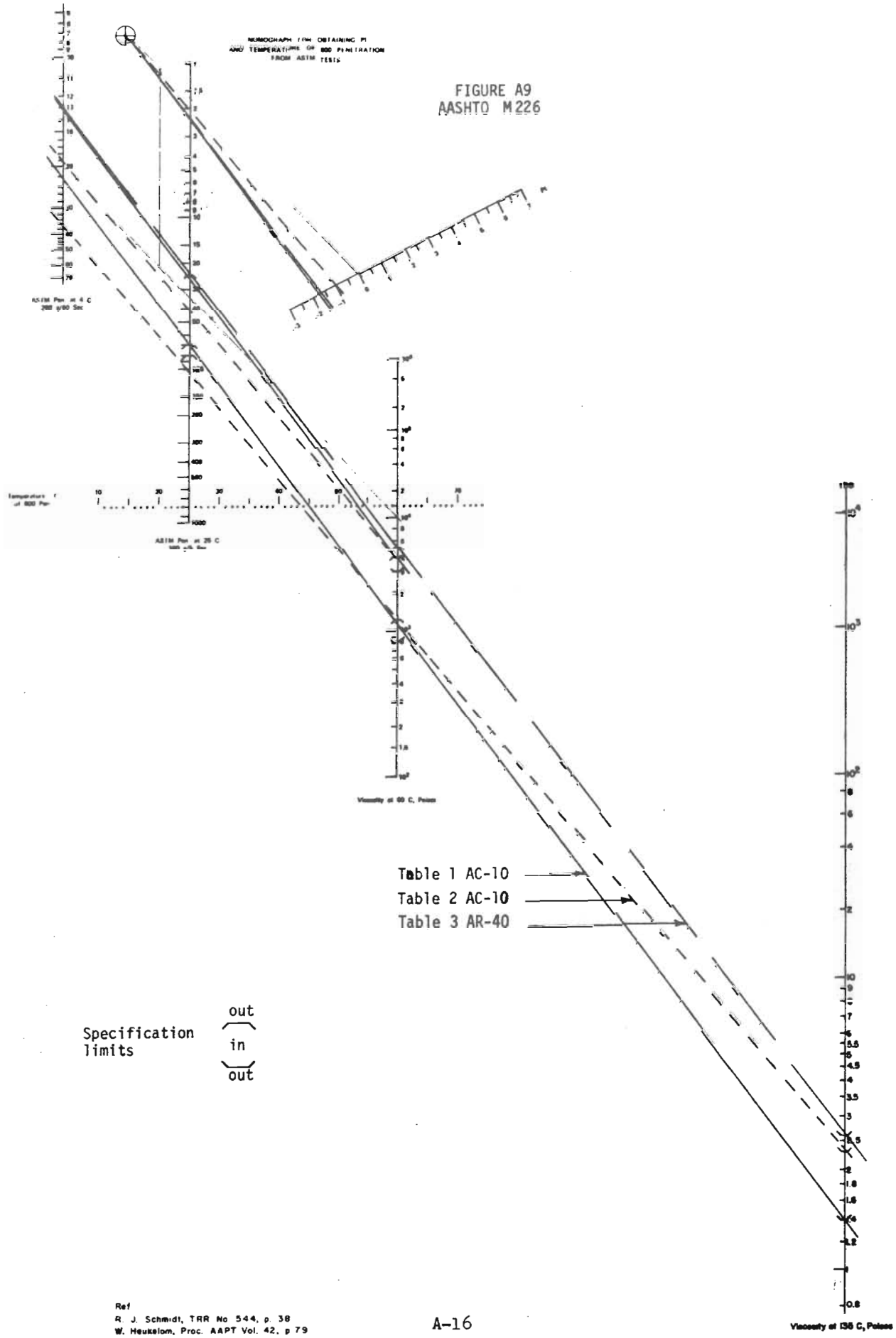


FIGURE A9
AASHTO M226

Ref
R. J. Schmidt, TRR No 544, p. 38
W. Heukelom, Proc. AAPT Vol. 42, p 79

Table B 1

Pit Data

Aggregate Source	Ad 95	Bg 77	Bk 142
Location	Boise	Shelley	Portneuf
Description	River Terrace	Riverside Deposit	Combination Riv. Terrace & Quarry
Mineral	basalt, diorite, granite, grano-diorite, quartz granite porphyry rholite porphyry	quartzite, limestone, quartz calcareous quartzite	quartz, quartzite small amounts of limestone and sandstone
Apparent Spec. Grav.	2.63	2.64	2.66

Mix Data

Aggregate Source	Ad 95	Bg 77	Bk 142
% Asphalt	6.7	5.8	5.1
% Air Voids	5	3	3
Hveem Stability	30	32	34

Table B 2

Tensile Strength in psi and M_R (zero time) in kpsi of AC Mixes
Unaged 120-150 Asphalt; Tests at Room Temperature

	Aggregate Source		Ad 95			Bn 77			Bk 142		
		Test No.	1	2	3	1	2	3	1	2	3
American	Dry	Tens. Str.	68	67	63	17	18	17	40	42	41
		M_R	60	70	75	8.3	8.7	8.3	28	28	30
	Wet	Tens. Str.	42	49	50	(1)	(1)	(1)	41	39	42
		M_R	30	33	38	(1)	(1)	(1)	44	34	39
Chevron	Dry	Tens. Str.	54	55	50	31	32	28	30	27	27
		M_R	38	44	41	17	17	19	38	22	15
	Wet	Tens. Str.	51	54	54	(3)	(3)	(3)	(4)	(4)	(4)
		M_R	34	42	(2)	(3)	(3)	(3)	(4)	(4)	(4)
Exxon	Dry	Tens. Str.	51	54	55	26	26	28	24	24	29
		M_r	46	46	46	(4)	19	9.8	14	14	18
	Wet	Tens. Str.	78	57	54	2	2	1	28	28	27
		M_R	(2)	(2)	(2)	(3)	(3)	(3)	17	14	9
Farmers	Dry	Tens. Str.	54	52	52	12	12	10	31	34	38
		M_R	41	48	40	7.6	6.1	4.8	30	47	37
	Wet	Tens. Str.	49	57	47	(1)	(1)	(1)	27	31	37
		M_R	36	62	30	(1)	(1)	(1)	(4)	(4)	(4)
Husky	Dry	Tens. Str.	60	62	63	12	15	14	32	30	28
		M_R	54	59	65	5.8	7.7	9.2	33	22	18
	Wet	Tens. Str.	51	54	53	(1)	(1)	(1)	35	33	32
		M_R	31	36	36	(1)	(1)	(1)	30	23	25
Phillips	Dry	Tens. Str.	58	54	58	29	26	28	43	36	39
		M_R	(4)	(4)	(4)	20	14	24	52	23	28
	Wet	Tens. Str.	34	36	42	(1)	(1)	(1)	46	42	44
		M_R	(4)	(4)	(4)	(1)	(1)	(1)	(2)	48	76

Notes: (1) sample came apart in water bath; (2) extreme scatter; (3) sample extremely weak; (4) equipment problem

Table B 3

Tensile Strength in psi and M_R (zero time) in kpsi of AC Mixes
Aged 120-150 Asphalt; Tests at Room Temperature

	Aggregate Source					Bg 77			Bk 142		
	Test No.		1	2	3	1	2	3	1	2	3
American	Dry	Tens. Str.	96	96	98	37	37	30	62	63	64
		M_R	140	180	200	21	21	19	72	66	66
	Wet	Tens. Str.	67	64	68	(1)	(1)	(1)	54	55	52
		M_R	69	76	110	(1)	(1)	(1)	69	81	89
Chevron	Dry	Tens. Str.	67	72	81	47	43	46	54	58	54
		M_R	65	67	81	24	26	32	64	62	57
	Wet	Tens. Str.	64	68	69	(3)	(3)	(3)	57	54	61
		M_R	69	77	76	(3)	(3)	(3)	(4)	(4)	(4)
Exxon	Dry	Tens. Str.	68	69	71	39	43	50	38	36	43
		M_R	42	(4)	65	16	40	27	32	24	42
	Wet	Tens. Str.	69	73	67	16	17	18	33	46	38
		M_R	69	97	62	6.7	12	(4)	25	46	30
Farmers	Dry	Tens. Str.	75	75	77	28	30	31	49	47	48
		M_R	73	77	86	17	22	19	58	44	58
	Wet	Tens. Str.	68	72	71	6	8	8	47	46	51
		M_R	58	120	94	2.7	5.8	5.7	(4)	(4)	(4)
Husky	Dry	Tens. Str.	91	83	76	22	23	23	49	47	54
		M_R	87	120	88	11	12	16	43	37	55
	Wet	Tens. Str.	74	76	69	(1)	(1)	(1)	49	52	60
		M_R	85	-10	85	(1)	(1)	(1)	60	50	97
Phillips	Dry	Tens. Str.	85	78	80	60	63	58	53	50	51
		M_R	(4)	(4)	(4)	60	78	55	62	57	54
	Wet	Tens. Str.	60	68	61	(1)	(1)	(1)	49	47	55
		M_R	(4)	(4)	(4)	(1)	(1)	(1)	62	52	83

Notes: (1) sample came apart in water bath; (2) extreme scatter; (3) sample extremely weak

Table B 4

Tensile Strength in psi and M_R (zero time) in kpsi of AC Mixes
Unaged 120-150 Asphalt; Test Temperature 39°F

	Aggregate Source		Ad 95			Bg 77			Bk 142		
		Test No.	1	2	3	1	2	3	1	2	3
American	Dry	Tens. Str.	205	187	181	145	136	131	179	190	181
		M_R	760	1200	(2)	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	127	179	141	68	67	48	122	108	103
		M_R	1400	1200	1400	(4)	(4)	(4)	(4)	(4)	(4)
Chevron	Dry	Tens. Str.	219	270	261	178	161	106	167	169	167
		M_R	(4)	(4)	(4)	(2)	660	260	(4)	(4)	(4)
	Wet	Tens. Str.	166	203	167	7	4	6	115	118	109
		M_R	(4)	(4)	(4)	(3)	(3)	(3)	(4)	(4)	(4)
Exxon	Dry	Tens. Str.	189	220	181	133	84	77	182	163	165
		M_R	(4)	(4)	(4)	(4)	520	310	(4)	(4)	(4)
	Wet	Tens. Str.	142	166	178	33	61	24	118	110	105
		M_R	(4)	(4)	(4)	(2)	(2)	(2)	(4)	(4)	(4)
Farmers	Dry	Tens. Str.	225	230	227	156	125	116	182	177	173
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	176	210	216	70	53	13	131	123	111
		M_R	(4)	(4)	(4)	(2)	(2)	(3)	(4)	(4)	(4)
Husky	Dry	Tens. Str.	314	266	289	133	116	100	123	112	95
		M_R	1000	720	940	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	210	191	195	(1)	(1)	(1)	104	100	112
		M_R	540	540	740	(1)	(1)	(1)	(4)	(4)	(4)
Phillips	Dry	Tens. Str.	279	223	248	107	88	79	175	169	124
		M_R	(4)	(4)	(4)	(2)	290	150	(2)	(2)	(2)
	Wet	Tens. Str.	126	163	149	(1)	(1)	(1)	116	134	120
		M_R	(4)	(4)	(4)	(1)	(1)	(1)	(4)	(4)	(4)

Notes: (1) sample came apart in water bath; (2) extreme scatter; (3) sample extremely weak; (4) equipment problem

Table B 5

Tensile Strength in psi and M_R (zero time) in kpsi of AC Mixes
Unaged 120-150 Asphalt; Test Temperature 39°F

	Aggregate Source		Ad 95			Bg 77			Bk 142		
		Test No.	1	2	3	1	2	3	1	2	3
American	Dry	Tens. Str.	187	264	222	161	133	121	206	199	185
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	192	182	159	83	70	40	168	150	136
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
Chevron	Dry	Tens. Str.	315	239	226	205	181	191	255	238	229
		M_R	(4)	(4)	(4)	(2)	1300	1100	(4)	(4)	(4)
	Wet	Tens. Str.	170	212	182	14	6	14	181	180	176
		M_R	(4)	(4)	(4)	(3)	(3)	(3)	(4)	(4)	(4)
Exxon	Dry	Tens. Str.	291	226	227	147	127	102	234	228	228
		M_R	(4)	(4)	(4)	(2)	450	178	(4)	(4)	(4)
	Wet	Tens. Str.	150	156	188	54	53	54	134	107	155
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
Farmers	Dry	Tens. Str.	318	322	292	228	229	184	214	168	227
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	245	261	261	48	74	84	136	131	158
		M_R	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)	(4)
Husky	Dry	Tens. Str.	330	330	315	119	175	150	147	135	119
		M_R	3400	3400	1200	(4)	(4)	(4)	(4)	(4)	(4)
	Wet	Tens. Str.	159	128	227	22	25	9	155	139	128
		M_R	900	740	540	(4)	(4)	(4)	(4)	(4)	(4)
Phillips	Dry	Tens. Str.	277	215	250	133	110	114	163	141	123
		M_R	(4)	(4)	(4)	(2)	235	470	(4)	(4)	(4)
	Wet	Tens. Str.	180	160	177	(1)	(1)	(1)	145	138	130
		M_R	(4)	(4)	(4)	(1)	(1)	(1)	(4)	(4)	(4)

Notes: (1) sample came apart in water bath; (2) extreme scatter; (3) sample extremely weak; (4) equipment problem

Table B 6

M_R (zero time) of Dry Specimens at Room Temperature

	Unaged Asphalt			
	Ad 95	Bg 77	Bk 142	Row Means
American	68,000	8,400	29,000	35,000
Chevron	41,000	18,000	25,000	28,000
Exxon	46,000	14,000	15,000	25,000
Farmers	43,000	6,200	38,000	29,000
Husky	59,000	7,600	24,000	30,000
Phillips	No Test	19,000	34,000	26,000
Column Means	51,000	12,000	28,000	29,000

	Aged Asphalt			
	Ad 95	Bg 77	Bk 142	Row Means
American	170,000	20,000	68,000	86,000
Chevron	71,000	27,000	61,000	53,000
Exxon	54,000	28,000	33,000	38,000
Farmers	79,000	19,000	53,000	50,000
Husky	98,000	13,000	45,000	52,000
Phillips	No Test	64,000	58,000	61,000
Column Means	94,000	28,000	53,000	57,000

Each M_R is the average of the three separate determinations listed in Table B2 or B3 for the corresponding conditions of testing.

Table B 7

Analysis of Variance
M_R (zero time) of Dry Specimens
Unaged Asphalt, Room Temperature

M_R

	Ad 95	Bg 77	Bk 142	Row Sums
American	68,000	8,400	29,000	105,400
Chevron	41,000	18,000	25,000	84,000
Exxon	46,000	14,000	15,000	75,000
Farmers	43,000	6,200	38,000	87,200
Husky	59,000	7,600	24,00	90,600
Column Sums	257,000	54,200	131,000	442,200

M_R^2

	Ad 95	Bg 77	Bk 142	Row Sums
American	46.2×10^8	0.7×10^8	8.4×10^8	55.3×10^8
Chevron	16.8×10^8	3.2×10^8	6.2×10^8	26.2×10^8
Exxon	21.2×10^8	2.0×10^8	2.2×10^8	25.4×10^8
Farmers	18.5×10^8	0.4×10^8	14.4×10^8	33.3×10^8
Husky	34.8×10^8	0.6×10^8	5.8×10^8	41.2×10^8
Column Sums	137.5×10^8	6.9×10^8	37.0×10^8	181.4×10^8

Table B 7
(Continued)

$$S_T = 181.4\times10^8 - \frac{44.2^2\times10^8}{15} = (181-130)10^8 = 51\times10^8$$

$$S_{Asph} = \frac{(10.5^2+7.5^2+8.7^2+9.12}{3} - 130)10^8 = (132-130)10^8 = 2\times10^8$$

$$S_{Agg} = \frac{(27.7^2+5.4+13.0^2}{5} - 130)10^8 = (172-130)10^8 = 42\times10^8$$

$$S_E = [51-(2+42)]10^8 = 7\times10^8$$

Summary

Source of Variation	Sums of Squares	Degree of Freedom	Mean Square	F Ratio
Asphalt	2×10^8	4	0.5×10^8	$\frac{0.5}{0.9} = 0.6$
Aggregate	42×10^8	2	21×10^8	$\frac{21}{0.9} = 23$
Unknown	7×10^8	8	$.9\times10^8$	
Total	51×10^8	14		

F 4.8;α = .0051 = 8.8 but 0.6 < 8.8 so asphalt source has no significant effect at 0.5% level.

F 2.8;α = .005 = 11.0 but 23 > 11 so aggregate source does have significant effect at 0.5% level.

Table B 8

Analysis of Variance
M_R (zero time) of Dry Specimens
Aged Asphalt, Room Temperature

M_R

	Ad 95	Bg 77	Bk 142	Row Sums
American	170,000	20,000	68,000	258,000
Chevron	71,000	27,000	61,000	159,000
Exxon	54,000	28,000	33,000	115,000
Farmers	79,000	19,000	53,000	151,000
Husky	98,000	13,000	45,000	156,000
Column Sums	472,000	107,000	260,000	839,000

M_R^2

	Ad 95	Bg 77	Bk 142	Row Sums
American	289.0×10^8	4.0×10^8	46.2×10^8	339.2×10^8
Chevron	50.4×10^8	7.3×10^8	37.2×10^8	94.9×10^8
Exxon	29.2×10^8	7.8×10^8	10.9×10^8	47.9×10^8
Farmers	62.4×10^8	3.6×10^8	28.1×10^8	94.1×10^8
Husky	96.0×10^8	1.7×10^8	20.2×10^8	117.9×10^8
Column Sums	527.0×10^8	24.4×10^8	142.6×10^8	694.0×10^8

$$S_T = (694.0 - \frac{83.9^2}{15})10^8 = (694-469)10^8 = 225 \times 10^8$$

$$S_{Asph} = \frac{(25.8^2+15.9^2+11.5^2+15.6^2}{3} - 469)10^8 = (507-469)10^8 = 38 \times 10^8$$

$$S_{Agg} = (\frac{47.2^2+10.7^2+26.0^2}{5} - 469)10^8 = (604-469)10^8 = 135 \times 10^8$$

$$S_e = [225-(38+135)] \times 10^8 = 52 \times 10^8$$

Table B 8
(Continued)

Summary

Source of Variation	Sums of Squares	Degrees of Freedom	Mean Squares	F Ratio
Asphalt	38×10^8	4	9.5×10^8	$\frac{9.5}{6.5} = 1.5$
Aggregate	135×10^8	2	68×10^8	$\frac{68}{6.5} \doteq 10.5$
Unknown	52×10^8	8	6.5×10^8	
Total	225×10^8	14		

$F_{4,8;\alpha=.01} = 7.0$ but $1.5 < 7.0$ so asphalt source has no significant effect at 1% level

$F\ 2.8; \alpha = .01 = 8.5$ but $10.5 > 8.5$ so aggregate source does have significant effect at 1% level.

Table B9
Tensile Strength Ratio (TSR)
TS wet/TS dry

Room Temp.

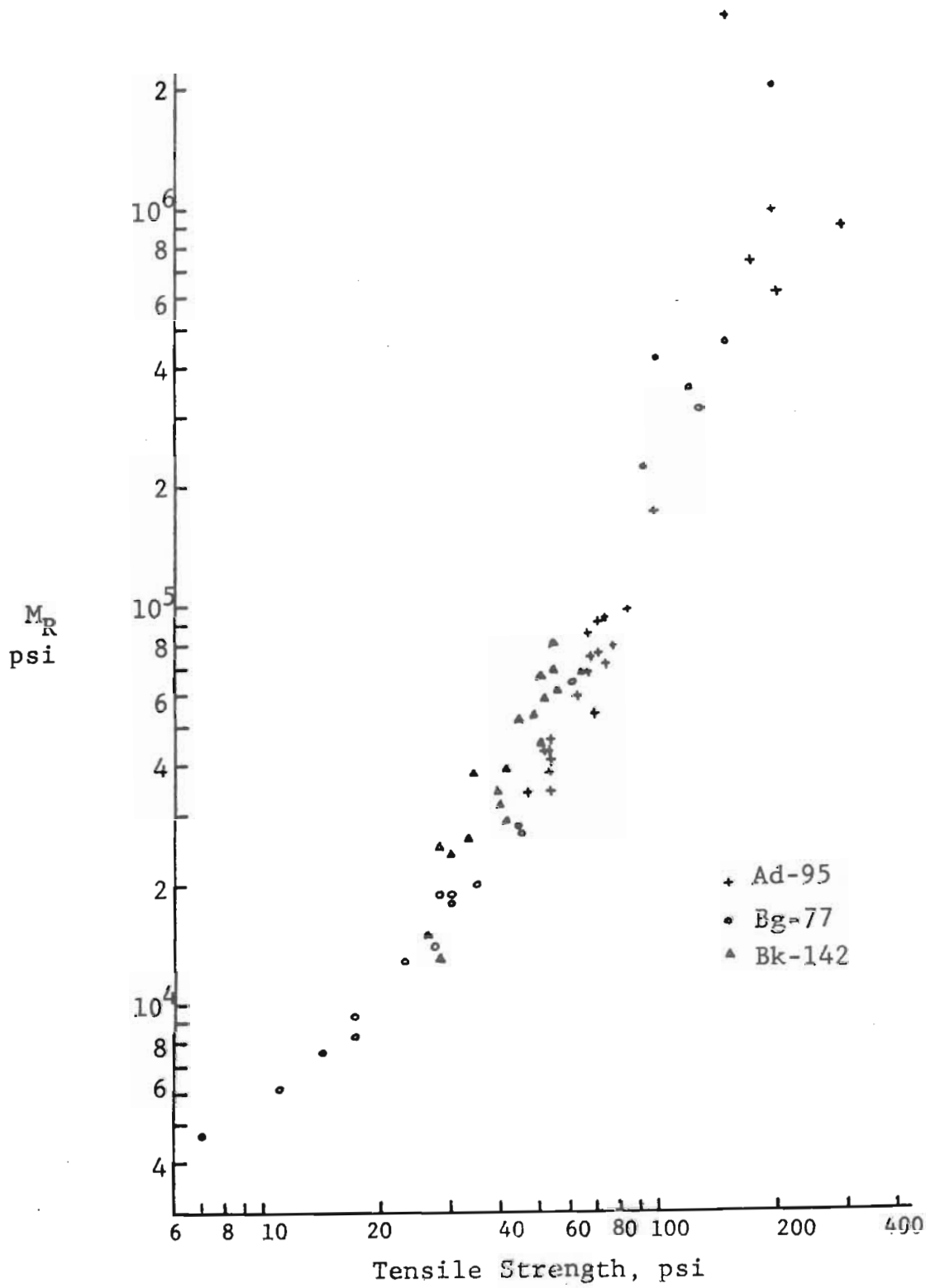
	Unaged				Aged			
	Ad-95	Bg-77	Bk-142	row means	Ad-95	Bg-77	Bk-142	row means
American	0.71	0.00	1.00	0.57	0.68	0.00	0.86	0.51
Chevron	1.00	0.00	1.14	0.71	0.92	0.00	1.04	0.65
Exxon	1.19	0.07	1.08	0.78	1.01	0.39	1.00	0.80
Farmers	0.96	0.00	0.94	0.63	0.92	0.23	1.00	0.72
Husky	0.85	0.00	1.10	0.65	0.88	0.00	1.08	0.65
Phillips	0.65	0.00	1.13	0.59	0.78	0.00	0.98	0.59
column means	0.89	0.01	1.06		0.86	0.10	0.99	

39°F

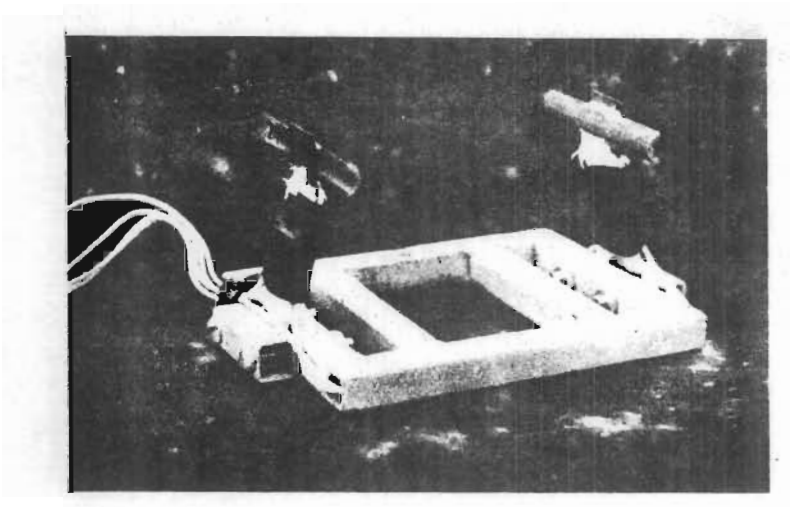
	Unaged				Aged			
	Ad-95	Bg-77	Bk-142	row means	Ad-95	Bg-77	Bk-142	row means
American	0.79	0.44	0.61	0.61	0.82	0.46	0.75	0.68
Chevron	0.72	0.04	0.65	0.47	0.73	0.06	0.75	0.51
Exxon	0.80	0.39	0.65	0.61	0.64	0.42	0.57	0.54
Farmers	0.87	0.35	0.67	0.63	0.84	0.33	0.70	0.62
Husky	0.69	0.00	1.00	0.56	0.53	0.13	1.08	0.58
Phillips	0.60	0.00	0.75	0.45	0.68	0.00	1.00	0.56
column means	0.74	0.20	0.72		0.71	0.23	0.81	

Notes: 1. Specimens which disintegrated in the water bath were assigned a tensile strength of zero.
2. Values are based on average results of Tables B2 through B5.

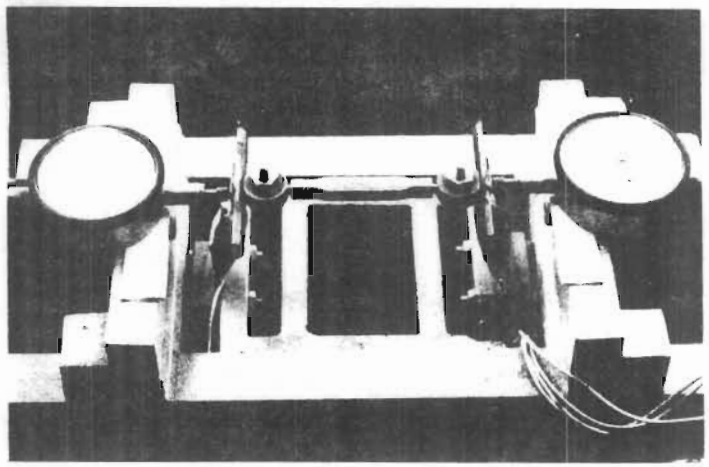
Figure B1



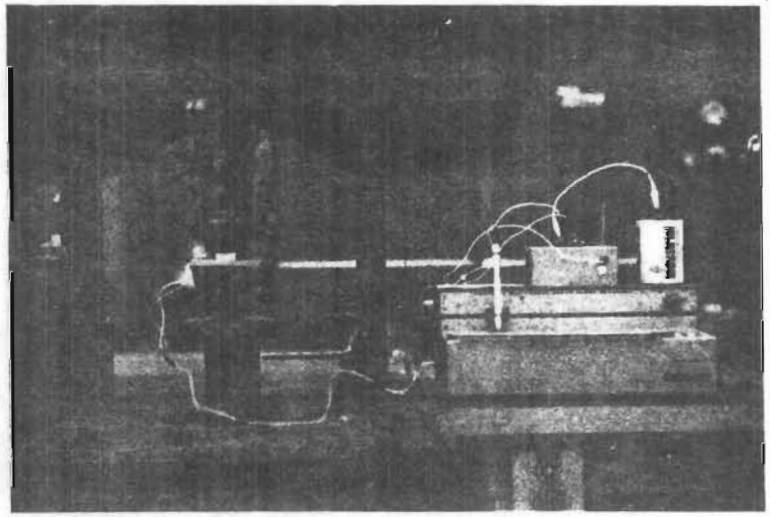
Includes All Testing Conditions



Displacement Transducer



Transducer in calibration jig.



Transducer and strip chart recorder ready for test.

APPENDIX D

Discussion of Resilient Modulus Testing Problems

Various problems occurred during M_R testing, resulting in the loss of considerable data. Some of these losses were associated with the strip chart recorder. On a few occasions the recorder malfunctioned during tests, invalidating the data. This was not a major problem but it does emphasize one drawback to destructive testing. Data lost due to equipment problems during a test can only be obtained by fabricating a new test specimen. This represents a loss in time and money in addition to the expense of correcting the equipment trouble.

The major problem in M_R testing occurred at the 39°F test temperature. Testing was done in the open room with only the specimen itself chilled to 39°F. After the displacement transducer was placed on the specimen, its electrical output drifted for several minutes even though no load was applied to the specimen. Moreover, this drift was in the direction opposite to the signal produced by loading the specimen. Several tests were run before the consequences of the drift were realized. The effect was to delay each indicated value of displacement beyond the true time it occurred. This made the specimens appear much sitffer than they actually were, and the test results had to be discarded.

Two possible factors in this signal drift are the electrical strain gages and the spring steel legs on which they are mounted. The photographs in Appendix C illustrate the transducer configuration. Each spring leg has one strain gage mounted on the side facing the specimen and one on the side facing outward toward the room. The gages facing the specimen are nearer the cool specimen and probably were cooled more than the outer gages, thus changing their resistance. In addition, the spring legs themselves were exposed to the cool specimen on one side and room temperature on the other; and the temperature differential probably caused them to flex slightly. This of course would affect the strain gages also.

The problem described above could be solved by testing in a temperature controlled chamber, resulting in somewhat increased complexity and cost. A different approach was tried. Before each test series, the transducer was placed over a can of ice water to simulate a chilled AC specimen. Electrical drift was monitored until stability was indicated, then the transducer was placed over the 39°F specimen and testing was begun. Between tests, the transducer was replaced on the can of ice water to maintain its thermal stability.

Unfortunately, the thermal stabilizing technique described

just above was omitted during some later test work. Even in cases where it was used, however, a problem was sometimes evident. The signal drift is fairly rapid when the device is first placed on the cold can, but slows down considerably when nearing thermal equilibrium. It can be difficult to judge when the device is sufficiently stabilized to permit testing. Even a small drift rate may cause a problem when it acts over the one minute or longer period needed to split the specimen.

In addition to the problems mentioned above, our experience in this project revealed some other less critical aspects of the M_R test procedure where improvements could be made. First, dry cells were used to supply power for the strain gages. This power source is adequate for short term use, but if M_R testing becomes standard procedure, an electronic power supply would be preferable. This would eliminate nuisances caused by the relatively short useful life and voltage drift of dry cells.

The numerical estimation of zero time M_R took longer than might be desired. One to two hours were required to read the data from the strip charts, make the computations, and plot the results for each group of three tests. This is an inherent feature of performing the M_R test concurrently with tensile splitting.

Some of the problems and undesirable features of the M_R test equipment and procedure used here could be reduced or eliminated by employing modified methods and test devices. As an example, Schmidt (5) describes a means of measuring M_R which appears preferable in several ways. First, it is nondestructive so multiple tests can be run on a single specimen under various test conditions. A second advantage of the nondestructive method is that the specimen is not lost if an equipment problem occurs during testing. As soon as the problem is located and corrected, testing can resume on the same specimen.

The time required for data reduction would be much less under the Schmidt method that was needed in this project. Our method required computation of several values of M_R at various times during the tensile split test, followed by graphical extrapolation back to zero time. In contrast, the Schmidt system employs a pulsed test load of only 0.1 second duration so the M_R value calculated from this single point is representative of short term loading and can be used directly in many cases.

The Schmidt method of M_R determination also seems likely to offer a solution to the thermal drift problem mentioned earlier. One reason is that his displacement transducer has a different configuration than the one used in our project. The electrical elements of Schmidt's transducer would not be subjected to the temperature differential mentioned earlier because in his design, the electrical elements are located well away from the specimen and are exposed to room temperature on all sides. His device

APPENDIX E

Outline of Method Used to Derive Asphalt Selection Table Incorporating Residue Penetration at 39°F

1. On the penetration-viscosity nomograph, plot M 226 specification limits for a given asphalt grade.

2. Draw the steepest straight line allowed by the specification limits. This represents the most temperature susceptible type S asphalt allowed by the specification.

3. Draw a straight line through the upper limit of 60°C residue viscosity (thin film oven test) parallel to the line drawn in Step 2. Record the 4°C penetration value of this line. This approximates the lowest residue pen. to be expected from a type S asphalt meeting the specification.

4. Draw additional straight lines from the upper limit of 60°C residue viscosity to 4°C penetration values near the 4°C pen. value found in Step 3. At least one line should be drawn to a 4°C pen lower than the value found in Step 3 to show the effect of moderate type W behavior.

5. For each of the lines drawn in Step 4, record the PI and the temperature of 800 pen.

6. Use each PI to find a temperature adjustment from Table D1. This table is derived from the Van Der Poel stiffness nomograph using a loading time of 10^4 seconds and a limiting asphalt stiffness of 1.38×10^8 Pa as recommended by Schmidt (13).

7. Add each temperature adjustment from Step 6 to its corresponding temperature of 800 pen. from Step 5, then convert to the Fahrenheit scale.

8. Add 10°F to each temperature derived in Step 7, to correct approximately for the correlation offset noted by Schmidt (13).

9. Using Table D2, find the freezing index corresponding to each temperature from Step 8. This table is derived from a chart recommended by Haas (25) for relating design temperature to freezing index.

   Step 9 gives a maximum freezing index corresponding to each 4°C penetration value from Steps 3 and 4. An asphalt with a given 4°C penetration should not be used in a location having a higher freezing index than the corresponding freezing index from Step 9.

10. Repeat Steps 1 - 9 for other asphalt grades of interest.

11. Arrange the results of Steps 1 - 10 in a convenient form for selecting asphalt grade and 4°C residue pen. suitable for use with a given freezing index.

TABLE D1

Temperature Adjustment (°C) From Van Der Poel Nomograph, for 10^4 sec. Loading Time and 1.38×10^8 Pa Critical Stress

PI	Temp. Adj.
-0.4	-85
-0.4	-84
-0.6	-84
-0.7	-83
-0.8	-82
-0.9	-80
-1.0	-79
-1.1	-79
-1.2	-78
-1.3	-77
-1.4	-76
-1.5	-75
-1.6	-74
-1.7	-73
-1.8	-72
-1.9	-71
-2.0	-70
-2.1	-69
-2.2	-68
-2.3	-67
-2.4	-67
-2.5	-66
-2.6	-66
-2.7	-65

TABLE D2

Design Temperature Corresponding to Various Values of Freezing Index*

Freezing Index	Design Temp., °F
100	+10
200	+5
300	+1
400	0
500	-1
600	-3
700	-4
750	-5
800	-6
900	-7
1000	-8
1200	-11
1400	-13
1500	-15
1600	-17

*This table is derived from Fig. 9 of reference 25.

does employ mechanical followers in contact with the specimen and a metal mounting frame clamped to the specimen, so these elements might be subject to slight distortions caused by thermal effects from a cold specimen. Even so, any slight signal drift would be much less serious in the Schmidt method because the displacement is measured over a period of only a tenth of a second. A slight signal drift would have very little effect over this time period, in contrast to the major effect which can occur over the one minute or greater loading period of the tensile splitting test.

One apparent drawback to the Schmidt procedure is equipment cost. A special loading frame, electrical load cell, transducer and probably a special strip chart recorder are required. As part of the Department's Research Project 76, very similar equipment will be purchased for use in measuring soil resiliency. The transducer is the only item not common to both soil and pavement resiliency testing. Therefore, the only extra equipment cost for applying the Schmidt technique to AC paving specimens would be the price of the transducer and calibrator, provided the manufacturer is willing to furnish them separately from the other items of the asphalt concrete test set. As an alternative, the Lottman-type transducer might be modified and tested under pulsed loading. A suggested modification would consist of a clamp fitting against the flat faces of the test specimen. This would prevent the transducer from slipping under the influence of cyclical loading.