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Characterization of Nevada's Binders and Low Temperature Properties of Mixtures Using SHRP Tests.

Part II. Characterization of Nevada's Binders Using Superpave Technology.

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16. Abstract

In 1992, the Nevada Department of Transportation (NDOT) sponsored a research project at the university of Nevada to assess the applicability of the newly developed Superpave binder grading system under Nevada's conditions. The project comprised of three major tasks which consisted of evaluating the rheological properties of Nevada's binders and establishing their PG grades, evaluating the contribution of binders toward low temperature cracking of HMA mixtures and identify the best method to evaluate low temperature resistance of HMA mixtures, and developing a demerit system to be used with the Superpave binder grading system.

A total of fifty-five binders were evaluated and graded using the Superpave binder grading system. All of the binders were used on actual construction projects. A large percentage of the evaluated binders were polymer modified AC-20 referred to as "AC-20P". Such binders are very commonly used in northern Nevada due to the wide range of expected pavement temperatures. The performance grading of these binders showed that AC-20P binders can fall over a wide range of performance grades. The AC-20P's high temperature grades ranged from 52 to 64 while their low temperature grades ranged from -16 to -40. The Superpave recommended PG grades were determined for each project and checked against the PG grades of the binders used. It was concluded that the majority of the binders violate the Superpave recommended PG grade. On the other hand the field performance of these projects for the past six years did not indicate any potential problems. Therefore, it was concluded that the Superpave binder grading system must be modified prior to its full implementation in Nevada.

The low temperature cracking resistance of twenty-one mixtures was evaluated using the TSRST. The fracture temperatures of the mixtures were evaluated using samples manufactured from lab mixed-lab compacted (LMLC), field mixed-lab compacted (FMLC), and LMLC with PAV aged binders materials (PAV-LMLC). The objective of this experiment was to identify the most appropriate method of assigning a critical low temperature for the HMA mixtures. The data showed that the fracture temperature determined by the TSRST is not consistent. In some cases, the TSRST measured fracture temperatures on the LMLC mixtures are warmer than the PAV-LMLC mixtures. It was concluded that the critical low temperature determined by the Superpave Binder grading system using the BR device is conservative for the majority of the cases which makes it an acceptable alternative to mixture testing until a more consistent mixture evaluation system is developed.

The Superpave weather data base was used to identify the most appropriate PG regions throughout the state of Nevada. The new regions were developed in order to reduce the locations within the state where the 98 percent reliability is not met. This resulted in recommending six PG regions instead of the currently used five PG regions. Using the performance parameters of asphalt binders as identified by the Superpave system along with the relationships between these parameters and the performance of HMA mixtures established in the SHRP research, the research developed a demerit system to be used with the Superpave binder grading system.

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INTRODUCTION

Nationwide, the construction, rehabilitation, and maintenance of highways uses some 140 million barrels of asphalt a year. Specifications have become essential to the quality of asphalt binders because of variations between different refineries producing them. Asphalt binder is a residue from the distillation of crude oil, a by-product of gasoline, oil and petrochemical feed stock. Its qualities depend on a continuous and well-defined supply of crude oil, and they vary from source to source and refinery to refinery. Accordingly, state highway agencies have increasingly turned from empirical to performance-based specifications to obtain the right qualities.

AASHTO published specifications for penetration-graded asphalt binders in 1931. (1) The penetration-grading system was the first specification to measure binder consistency at an average pavement service temperature of 25°C. The system is still used by some highway agencies because it is simple and gives fast results concerning the consistency of the binder.

The next development was a viscosity-based grading system, which sought to replace the empirical penetration test with a more fundamental viscosity test. The test measured the consistency of asphalt at 60°C, which approximates the average pavement temperature on a hot summer day. Eventually, various viscosity tests were developed that suit different climates and applications of asphalt binders. The standard unit of measurement for viscosity became the poise.

By the early 1960's, the viscosity grading system had been adopted by the Federal Highway Administration (FHWA), the American Society of Testing and Materials (ASTM), AASHTO and many state highway departments (1). It is usually referred to as the AC grading system. It is now the system most widely used in the United States, including Nevada. It was the first step toward implementing rheological testing in asphalt specification.

During the development of the AC grading system, the chemical and physical properties of the asphalt after the mixing process became a subject of concern. The California Department of Highways and other state agencies experienced changes in the asphalt's properties after plant mixing. They recommended grading the asphalt after the lab aging so that all asphalt after mixing would behave about the same during construction. Therefore, another grading system based on the viscosity of the aged residue was developed. This is referred to as the AR grading system. (2) Currently, the AR grading system is most widely used in the western United States. NDOT used the AR grading system until 1986.

All three grading or specification systems (Penetration, AC, and AR) for asphalt binders are used across the United States, but today's current high traffic loading and advanced technology make an additional specification for asphalt binders both possible and desirable. That specification should be related to pavement performance. The goal is to specify materials for construction on the basis of their potential performance.

OBJECTIVES

In 1992, the Nevada Department of Transportation (NDOT) began a three-year research program to appraise the new binder specification system developed by the Strategic Highway Research Program (SHRP). In 1995, the same project was extended for an additional three-year period. The intention is to shift to the new system in the near future if it proves valid under Nevada conditions. NDOT has joined the Federal Highway Administration (FHWA) pool funds to purchase the laboratory equipment needed to implement SHRP's Superpave system. This report presents the results of the six years research efforts conducted by the Pavement/Materials Program of the University of Nevada to evaluate the binders.

The overall objective of this research effort was to evaluate the Superpave binder grading system for implementation under Nevada's conditions. The following three major tasks were completed:

Task 1:	Evaluate the rheological properties of selected NDOT asphalt binders using the Superpave binder grading system and identify their corresponding performance grades (PG).
Task2:	Evaluate the contribution of asphalt binders to the low temperature cracking of hot mixed asphalt (HMA) mixtures through the thermal stress restrained specimen test (TSRST).
Task 3:	Develop a demerit system to be used with the Superpave binder grading system in Nevada.

The above three tasks were completed over the six years period and the findings and recommendations are presented in this report.

TASK 1: EVALUATE RHEOLOGICAL PROPERTIES OF ASPHALT BINDERS AND IDENTIFY THEIR PG GRADES.

SHRP's asphalt research program aimed to develop a fundamental performance-based binder specification system. The intent is to assist highway agencies to build reliable, economical, and durable asphalt pavements. SHRP's research efforts produced a performance-based asphalt binder grading system referred to as "Superpave PG-grading system." The PG- grading system uses a distinct process to predict the performance characteristics of both neat and modified asphalt binders. The process includes:

a) Safety,

b) Rheological properties,

c) Aging, and

d) Environmental factors.

Figure 1 is a summary of the Superpave binder specification system. The following sections describe each process and the various testing equipment used.

Safety

The Standard Test Method for Flash and Fire Point by the Cleveland Open Cup (ASTM D 92) was adopted to determine the flash point of the binder. The flash point is the lowest temperature at which an open flame causes the vapor from the binder to ignite. The flash point temperature is obtained for safety regulations, and to determine if the binder has a high percentage of volatile and flammable materials.

Rheological Properties

Rheology is the science that studies the deformation and flow of material, whether in liquid, melt, or solid form, in terms of the materials' elastic and viscoelastic properties.(3) The science of rheology can be complex but rheology-testing itself need not be complicated. Some traditional methods used in determining the rheological characterization of asphalt binder include penetration measurement, determination of softening point temperature, and capillary viscosity measurements. Both the penetration and softening point tests are empirical. Therefore, such tests cannot be used to determine rheological behavior over a wide range of temperature. The capillary viscosity measurement, although a rational test, does not provide information on the time-dependency of the binder. To fully understand the behavior of asphalt binder, complete rheological information is preferred. It is therefore conclusive, since asphalt is a viscoelastic material, time and temperature effects are crucial in obtaining the rheological properties.

Both rational measurements and parameters are needed to obtain the rheological behavior of asphalt binder, which would serve as the basis of an effective performance-based binder specification. Basic rheological properties of asphalt binders include the following:

G': The storage modulus (elasticity) of the asphalt binder.

G'': The loss modulus (viscous loss) of the asphalt binder.

G^*: The complex modulus which is the amount of energy to deform the asphalt binder.

δ : The phase angle which is the difference between the phase of the sinusoidally varying input quantity and the phase of the output quantity which also varies sinusoidally at the same frequency. In binder testing, the input quantity represents the applied strain and the output quantity represents the resulting stress.

Figure 2 is a graphical representation of the relationship among the above variables. These material properties are used in the Superpave binder specification to evaluate the binder's resistance to tenderness, rutting, fatigue cracking, and thermal cracking.

In obtaining these rheological properties, sinusoidal shear strains (γ) are applied to the binder samples. At cold testing temperatures (below 34°C), the strains are kept constant at 1% and increased to 12% at higher test temperatures (above 46°C). Keeping the strain constant throughout a given test allows the sample to remain in the linear viscoelastic range. Although no material is perfectly linear under all conditions, linear viscoelastic characterization has been found in the past to best represent the rheological behavior of asphalt binders. These strain percentages (1% and 12%) were determined by other researchers using strain sweeps at a constant temperature. (3)

Since most rheological properties are time-dependent, it is essential to test the binder at a constant frequency, ω. The Superpave binder specification recommends a frequency of 10 rad/sec and strains varying between 1% and 12% to evaluate the rheological properties (G^*, G'', G', δ). If the stress-strain behavior of the binder is completely elastic, the resulting stress will be in phase with the applied strain, as illustrated in Figure 3. Otherwise, when the response is completely viscous, the stress response will be 90° out of phase with the applied strain. In general, the phase angle (δ) can very from 0° to 90° which indicates the amount by which the resulting stress is out of phase with the applied strain.

When dealing with high temperatures, the behavior of binder is dependent on its elastic properties. In measuring the elastic properties, the loss tangent (G''/G') has been found to be a good indicator. If the loss tangent is low at a given high temperature this indicates the presence of an elastic nature. This, in return, would help prevent deformation due to a low viscous flow.

Another important rheological property of a binder is its stiffness and the development of the master curve. Some of the current methods used to determine the stiffness modulus ($S(t)$) are the sliding-plate rheometer and the Van Der Poel nomograph. The Van Der Poel stiffness was determined by uniaxial tension-compression whereas the sliding plate applies a constant shear. When determining the stiffness, it should be clearly stated whether the value reported is for shear or extensional loading. With the stiffness data collected over a range of temperatures, the combined data can then be used to determine the master curve as illustrated in Figure 4. The master curve is obtained at a single reference temperature. Thus, for each stiffness curve determined at a particular test temperature, a horizonal shift factor is produced. The shift factor, one for each temperature, is an equivalency between time and temperature which is known as the time-temperature superposition. The resulting master curve provides a complete characterization of the linear stress-strain-time-temperature response of a typical binder. The time-dependency is reflected in the master curve, whereas the temperature is reflected by the shift factor.

Dynamic Shear Rheometer (DSR)

The DSR is one of four rheological testing equipments used in the Superpave performance-based binder grading system. The device used in this research was developed by Rheometrics and is referred to as the Rheometrics Asphalt Analyzer (RAA). The instrument can apply a precise oscillatory, steady, or step shearing strain to the test sample and precisely measure the sample's stress response. The responses obtained from the RAA for performance grading are G^*, G', G'' and δ.

One of the most important aspects of obtaining repeatable rheological data while conducting binder testing with the RAA is the sample and equipment preparation. The parallel plate configuration with the temperature sensor inside the upper plate was used in the test as shown in Figure 5. The size of the plates (i.e. 8 mm or 25 mm diameter) varies depending on the test temperature. For more details on sample preparation and testing, refer to the Superpave performance-based binder specifications (4).

The Bending Beam Rheometer (BBR)

The BBR is another device used in the performance-based specification. It is a "creep" test device operated by applying a constant load at the center of a simply supported asphalt binder beam for a selected period of time (Figure 6). During the loading time, the deflection at the center of the beam is continuously measured. The asphalt beam is 127 mm long, 12.7 mm wide and 6.3 mm thick, supported at both ends on metal supports that are 100 mm apart. The Cannon BBR was used in this research. The data generated from the test include the time history of load and deflection. The analysis of the data provides the stiffness (S(t)) values and the log slope (m) of the creep curve at selected loading times.

Direct Tension Device

The direct tension device measures the tensile failure properties of asphalt binders at low temperatures. The specimen is placed in an environmental chamber and subjected to a uniaxial tensile load. The specimen is 40mm x 6mm x 6mm (18mm gauge length) and is connected with plexiglass inserts on either end (Fig. 7).

During testing, the specimen is pulled and the deformation of the binder is measured by monitoring the elongation of the asphalt portion. At the same time the load is constantly monitored to keep the deformation rate constant. The maximum load and elongation are then used to calculate the stress and strain-to-failure of the material. The strain at failure is used to characterize the binder's resistance to low temperature cracking and thus to control initiation of cracking in pavements.

In the performance-based specifications, the direct tension is not used if the creep stiffness measured by the BBR is below 300,000 KPa. If the creep stiffness is between 300,000 and 600,000 KPA, the direct tension failure strain requirements can be used in lieu of the creep stiffness requirement, but the log slope of the creep curve must remain less than 0.30 (see Superpave specifications footnote).

Brookfield Viscometer

The rotational viscometer is used to determine the viscosity of the asphalt binder at high temperatures, for either blending, mixing, or field compaction operations. The viscometer used in this research is the Brookfield Digital Rheometer Model DV-III. The viscometer consists of a rotating spindle that can be used to measure the viscosity of asphalt binders in the range of 0.01 Pa*s (0.1 poise) to 200 Pa*s (2000 poise). These viscosities are measured in the typical temperature range of 100 to 260^{0}C (100 to 500^{0}F). The viscometer is operated by submerging the spindle in 10.5 ml of binder that is placed into a temperature-controlled thermosel. During the test the calibrated spindle is rotated by a motor with a specified rotational speed (20 rpm). Given the torque, the rotational speed of the spindle, and the geometry of the spindle and cup, the viscosity of the sample can be determined.

The Aging Process

To achieve a true performance-based specification, the binder tested in the laboratory must be treated like the one used in the field. To accomplish similitude between laboratory and field binders, two types of aging process are performed. For short- term aging, the rolling thin film oven test (RTFOT) is used to represent aging or hardening of the binder that may occur during the mixing and lay-down process. Nevada's experiences indicate that aging does not always occur in drum plants. To simulate long-term exposure in the field, the PAV was adopted into the Superpave specification. The value of G^* tested for several original, laboratory-aged, and field-aged binders validated the hypothesis that the PAV is closely related to field aging (3).

After short-term aging through the RTFOT, the asphalt binder is aged using the PAV for 20 hours under a constant pressure of 2.07 MPa and temperature between 90 and 110°C. The temperature of the test varies depending on the climate in which the asphalt will be used. The vessel can hold 10 thin trays with 50g ± 0.5g of binder per tray. The dimensions and levelness of the trays have to be maintained during the test to ensure the binder maintains a uniform film thickness while being aged.

Once the binder is aged through the PAV, its rheological properties are evaluated again with the RAA and the BBR. The temperature and pressure during a test are very critical and therefore must be constantly monitored and maintained with a tight tolerance (temperature ± 0.2°C and pressure ± 20 kPa).

Environmental Factors

To successfully acquire a performance-based binder specification, the SHRP research team categorized pavement temperatures into three binder performance groups: a) High, b) intermediate, and c) Low temperature. (3) The high temperature is based on the average seven days high pavement temperature in the summer months. The intermediate temperature is calculated empirically with respect to fatigue cracking, and the low pavement temperature is the expected low for the life of the pavement. To achieve specific environmental data for a given site, it is necessary to collect information from a local weather station. The atmospheric temperature is then converted to an expected pavement temperature. The conversion can be made using the Superpave temperature model or any other model that the agency may select.

Based on the expected high and low pavement temperatures, Superpave has selected the following ranges:

High Pavement Temperature (seven categories)

$<46^{o}$C ($<115^{o}$F)

$<52^{0}$C ($<126^{0}$F)

$<58^{0}$C ($<136^{0}$F)

$<64^{0}$C ($<147^{0}$F)

$<70^{0}$C ($<158^{0}$F)

$<76^{0}$C ($<169^{0}$F)

$<82^{0}$C ($<180^{0}$F)

Low Pavement Temperature (seven categories)

$>-10^{0}$C ($>14^{0}$F)

$>-16^{0}$C ($>3^{0}$F)

$>-22^{0}$C ($>-8^{0}$F)

$>-28^{0}$C ($>-18^{0}$F)

$>-34^{0}$C ($>-29^{0}$F)

$>-40^{0}$C ($>-40^{0}$F)

$>-46^{0}$C ($>-51^{0}$F)

It is anticipated that these temperature ranges would encompass all the temperature regimes which exist in U.S. and Canada.

Superpave Binder Grading System Criteria

The SHRP's primary objective in developing a performance-based specification for asphalt binder was to relate certain pavement failures with the binders' rheology at different temperatures and degree of aging (short- and long-term). Some of the most significant failures in pavements consist of tenderness (early rutting), rutting, fatigue cracking and thermal cracking. Therefore, SHRP considered these most common failures and developed test procedures for the binder grading specification. The following represents a discussion of the various failure modes as perceived by Superpave binder specification system.

Rutting

Pavement rutting is total plastic deformation at the surface. All layers in the system may contribute to total surface rutting. The discussion of rutting in this report will concentrate on the asphalt concrete layer. Although rutting in the asphalt concrete layer is primarily influenced by the aggregate interlock and mixture properties, the binder also influences rutting. In some cases polymerized binders show great resistance to rutting, and they are becoming more popular across the U.S. The occurrence of rutting is greatly influenced by high pavement temperatures. Based on the average seven day high pavement temperature in one year, SHRP selected seven testing temperatures (46^0, 52^0C, 58^0C, 64^0C, 70^0C, 76^0C, and 82^0C) for which the binders' rheological properties are to be determined.

While testing at these temperatures, a measurement of the non-recoverable deformation from the binder at a loading rate similar to traffic is established with respect to rutting resistance. To simulate the loading of a passing truck traveling at 80 km/h (50 mph), a sinusoidal loading is used at 10 rad/sec (1.6 Hz). The resulting $G^*/\sin\delta$ value is the specification criterion for rutting. The minimum acceptable value for the rutting criterion is 1.0 Kpa.

During the laydown process, a tender mix can also occur. In an attempt to prevent this, the Superpave specification adopted the same DSR test to be performed on the RTFO residue with a minimum value of 2.2 Kpa from $G^*/\sin\delta$.

Fatigue Cracking

The most difficult challenge in the Superpave binder specification is to assure satisfactory resistance to fatigue cracking. Since fatigue cracking generally occurs in the later life of the pavement, the PAV is used to simulate the long term aging of the binder. Based on evaluation of field data, SHRP adopted a specification criteria for fatigue cracking based on the dissipated energy, which is related to $G^*\sin\delta$.

In determining the fatigue behavior of asphalt binder, a loading time of 10 rad/sec (1.6 Hz) is applied to the PAV residue where $G^*\sin\delta$ is determined. For specification purposes, a minimum $G^*\sin\delta$ value of 5.0 MPa is acceptable.

Thermal Cracking

Thermal cracking is another serious failure in roadways and can result from a single thermal cycle where the temperature reaches a critical low. At low temperatures, the asphalt binder becomes brittle and loses the ability to absorb energy through viscous flow. As a result, the asphalt binder strain becomes intolerable and cracking occurs in the pavement. The temperature at which the pavement cracks is referred to as the limiting stiffness temperature. Since thermal cracking generally occurs in the later life of the pavement, the PAV is used to simulate the long term aging of the binder. At the limiting stiffness temperature the stiffness is obtained at a loading period of 60 seconds. In an effort to prevent thermal cracking, SHRP relates the limiting stiffness temperature at which a max stiffness of 300 MPa is obtained.

Another important factor in determining low temperature cracking is the time of loading that influences the magnitude of thermal shrinkage stresses. It was recognized from earlier SHRP contracts (A-002A and A-005) that thermal shrinkage is dependent on the time of loading. Because the time-dependency varies for different types of asphalt binders, especially polymerized, the shape of the master curve is a reliable means of determining the thermal shrinkage stress that develops during the cooling process. Therefore, SHRP adopted the slope (m) of the master curve to be implemented in the binder specification. The minimum criterion for the slope of the master curve is 0.30.

The tensile strength properties of the asphalt binder are also used to indicate its resistance to low temperature cracking. The tensile strain measured by the direct tension device is kept at a minimum of 1% in order to ensure that the binder has enough flexibility to resist cracking under the action of shrinkage stresses.

Laboratory Testing Program

Fifty-five binders were evaluated and fully discussed in this report. The majority of these binders were used on actual construction projects which are identified by a NDOT contrcat number. Also some binders have been graded using the AC-grading system while others were specified with PG-grades.

Prior to the start of the laboratory testing program, the Superpave binder specification system was thoroughly examined to identify the test parameters and the materials properties needed for the grading process. The following represents a summary of the identified properties.

* Rutting: original material with a minimum $G^*/\sin(\delta)$ value of 1.0 kPa measured at the maximum pavement temperature.

* Tenderness: RTFO residue with a minimum $G^*/\sin(\delta)$ value of 2.2 kPa measured at the maximum pavement temperature.

* Fatigue cracking: PAV residue with a maximum $G^*(\sin(\delta))$ value of 5,000 kPa measured at the intermediate temperature.

* Thermal cracking: PAV residue with a maximum stiffness value of 300,000 kPa at a minimum m-value of 0.30 measured at the minimum pavement temperature.

* Thermal cracking: PAV residue with a minimum failure strain of 1.0% at 1.0 mm/min at minimum pavement temperature.

It should be noted that the failure strain criterion is not used if the creep stiffness measured by the BBR is below 300,000 kPa. If the creep stiffness is between 300,000 and 600,000 kPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement, but the log slope of the creep curve must remain less than 0.30.

The process used in this research to test the asphalt binders under the Superpave grading system is summarized in Figure 8 and as fully described in the interim AASHTO test method. (5) It should be noted that this process may differ from one laboratory to another since the Superpave grading procedure has not been finalized yet.

As is the case with every grading system, the Superpave system can be used in two different ways: a) to check if a binder meets the requirements of a given grade, and b) to identify the appropriate grade for a given binder. Since the objective of this research was to identify the grades of the various Nevada binders, process b had to be followed.

Following process b requires more testing and more elaborate data analysis than process a. The initial objective of process b is to identify all the possible environments that a binder can fit. The final grade of the binder is then based on the widest environmental range. For example, if an AC-20P meets the requirements for PG52-10, PG52-16, and PG52-22, the final grade given to this binder would be PG52-22, since it represents the worst environmental conditions that this binder can withstand.

Data Analysis

The first step in grading a given binder consists of checking its flash point and viscosity against the specification limits. The SHRP's specification limits call for a minimum flash point of 230°C and a maximum viscosity of 3 Pa*s at 135°C. Checking the binders' data in Table 1 against these limits indicate that all binders satisfy the requirements of flash point and only one binder violates the viscosity criterion.

The second step in the evaluation process deals with the rheological properties of the original binder as it comes from the tank. The RAA DSR was used to evaluate the complex modulus and phase lag of all binders at 10 rad/sec frequency. At this stage, the class of the binder is unknown, therefore a temperature sweep must be conducted in order to identify the highest temperature at which the binder would reach the minimum specified value of $G^*/\sin(\delta)$.

Knowing that the $G^*/\sin(\delta)$ of an asphalt is inversely related to temperature, the testing proceeded at the lowest temperature (46°C) in the specification toward the highest temperature (82°C) (Figure 1). The relationships between $G^*/\sin(\delta)$ and temperature for all fifty-five binders were measured. Using these relationships, the temperatures at which the binders reach the minimum value of $G^*/\sin(\delta)$ of 1.00 kPa were identified

The third step in the grading process consists of evaluating the binders' properties after aging through the RTFO. As mentioned earlier, the RTFO is used to simulate the aging during the mixing and paving operations. The percentage of weight loss through RTFO aging should not exceed 1 percent. The percentage weight losses for all binders are summarized in Table 1. The data indicate that all binders would pass.

The rheological testing of the RTFO residues followed the same procedure as the one used for the virgin binders. The relationships between $G^*/\sin(\delta)$ and temperature for all binders after RTFO aging were measured. Using these relationships, the temperatures at which the binders reach the minimum value of $G^*/\sin(\delta)$ of 2.2 kPa were identified. At this point the high temperature grades of the binders can also be identified according to the specification in Figure 1. The high temperature grade assigned to the binder is the lower temperature between the two, e.g. lowest of temperature from the original binder and from the RTFO aged binder. The following is a summary of the high temperatures from the original and RTFO aged asphalt binders. Once the lower of the two temperatures is identified, a high temperature PG grade is assigned based on the available PG grades, e.g. staring at 46 and increasing by 6°C until a maximum of 82. The following high temperature grades were assigned to the tested binders.

Contract	AC-grade	Temperature (°C) at $G^*/\sin(d) = 1.0$ kPa	Temperature (°C) at $G^*/\sin(d) = 2.2$ kPa	High Temp Grade
N/A	AC-20	66.0	67.5	PG64
N/A	AC-20	66.3	68.3	PG64
2480	AC-20P	64.5	63.0	PG58
2491	AC-20P	61.5	57.8	PG52
2501	AC-30	68.7	69.8	PG64
2501	AC-20P	63.8	63.2	PG58
2530	AC-20P	63.4	63.6	PG58
2544	AC-20P	63.6	63.0	PG58
2545	AC-20P	67.5	58.4	PG58
2552	AC-20P	64.5	63.0	PG58
2558-93	AC-20P	62.1	58.7	PG58
2558-94	AC-20P	62.8	56.5	PG52
2567		68.5	68.9	PG64
2591	AC-20P	66.5	62.6	PG58
2594	AC-20P	63.6	63.3	PG58
2603	AC-20+TLA	74.4	72.6	PG70
2604	AC-40	70.0	68.6	PG64
2604	AC-30	70.1	70.0	PG70
2611	AC-20	66.5	66.7	PG64
2611	AC-20P	65.4	65.2	PG64
2615	AC-20P	64.7	62.5	PG58
2617	AC-20P	62.4	57.9	PG52
2622	AC-30P(1)	75.3	76.7	PG70
2622	AC-30P(2)	77.6	73.1	PG70
2704	AC-20P	65.1	60.8	PG58
2711	AC-20P	63.4	59.9	PG58
2726	AC-30P	76.2	71.4	PG70
2742	AC-20P	69.1	64.1	PG64
2751	AC-20P	63.8	62.0	PG58
Clark Co.	AC-30	67.5	69.2	PG64
2775		70.7	67.7	PG64
2775		67.2	66.5	PG64
N/A	AC-20P	64.0	64.9	PG64
2784		71.1	71.9	PG70
2784		70.6	71.0	PG70
2785		71.6	72.4	PG70
2825		76.5	76.1	PG76
2827		66.8	65.5	PG64
2827		67.0	68.3	PG64
2838		72.7	67.6	PG64

Contract	AC-grade	Temperature (°C) at $G^*/\sin(d)=1.0$ kPa	Temperature (°C) at $G^*/\sin(d)= 2.2$ kPa	High Temp Grade
2852		77.7	78.4	PG76
2874		65.6	66.3	PG64
2876		78.1	85.0	PG76
2880-I		65.9	65.8	PG64
2880-II	AC-20P	65.6	58.9	PG58
AC97-1159	AC-40	70.8	72.1	PG70
AC97-1162		67.1	67.8	PG64
AC97-1176		84.9	83.7	PG82
AC98-748	AC-20P	67.2	63.5	PG58
AC98-862	AC-20P	69.5	68.4	PG64
AC98-903		68.5	68.8	PG64
AC98-904		75.1	72.3	PG70
AC98-905		70.3	67.9	PG64
AC98-906		70.7	65.1	PG64
AC98-908		78.1	74.9	PG70

The fourth step in the grading process is the evaluation of the rheological properties of the binders after aging through the PAV. The PAV temperature is based on the high temperature grade of each binder. The specification requires the temperature at which $G^*\sin(\delta)$ reaches a maximum value of 5 MPa. Therefore, for each binder there will be a minimum temperature selected above which the maximum value of $G^*\sin(\delta)$ is not exceeded. After the PAV aging, all binders were tested at four temperatures. The relationships between temperature and $G^*\sin(\delta)$ were measured and the temperature at which $G^*\sin(\delta)=5.0$MPa is selected. As can seen from the Superpave grading diagram shown in Figure 1, this intermediate temperature does not become part of the binder grade but it identify the possible low temperature grades that a binder can be assigned.

The bending beam tests were then conducted on the PAV residues under two temperatures, namely -20 and -10°C. The two BBR points were used to draw the relationships between the stiffness (S(t)) and the slope (m) as a function of temperature. Using the bending beam results, the critical low temperatures of the binders are identified based on the maximum stiffness value of 300 MPa and a minimum slope value of 0.30. The BBR critical temperature is defined as the warmer of the two temperatures identified by the S(t) and m criteria. Using the the DSR and BBR results on the PAV aged asphalt binder, the following critical temperatures are identified. Once the high and low critical temperatures are identified, the full PG grade is assigned to the asphalt binder. It should be noted that the low temperature grades are also provided in -6°C increments between -10 And -46. The final grading of all fifty-five binders are as follows:

Contract	AC-grade	Temp.(°C) G*/sin(d)=5.0Mpa	BBR critical Temperature (°C)	PG Grade
N/A	AC-20	17.7	-11.7	PG64-16
N/A	AC-20	21.2	-13.0	PG64-22
2480	AC-20P	17.2	-18.9	PG58-28
2491	AC-20P	19.0	-18.2	PG52-16
2501	AC-30	22.7	-11.7	PG64-16
2501	AC-20P	16.9	-18.2	PG58-28
2530	AC-20P	15.9	-19.2	PG58-28
2544	AC-20P	13.2	-19.1	PG58-28
2545	AC-20P	19.5		PG58-22
2552	AC-20P	17.2	-19.9	PG58-28
2558-93	AC-20P	20.4	-14.2	PG58-22
2558-94	AC-20P	20.2	-12.6	PG52-16
2567		18.7	-18.4	PG64-28
2591	AC-20P	22.4	-17.5	PG58-16
2594	AC-20P	16.8	-18.4	PG58-28
2603	AC-20+TLA	30.5	-7.5	PG70-16
2604	AC-40	30.1	-5.3	PG64-10
2604	AC-30	23.0	-14.8	PG70-22
2611	AC-20	25.8	-12.5	PG64-22
2611	AC-20P	15.9	-19.9	PG64-28
2615	AC-20P	18.8	-19.0	PG58-28
2617	AC-20P	18.5	-14.0	PG52-22

Contract	AC-grade	Temp.(°C) G*/sin(d)=5.0Mpa	BBR critical Temperature (°C)	PG Grade
2622	AC-30P(1)	23.4	-12.2	PG70-22
2622	AC-30P(2)	23.3	-8.3	PG70-16
2704	AC-20P	16.5	-19.2	PG58-28
2711	AC-20P	20.1	-16.8	PG58-22
2726	AC-30P	18.2	-17.0	PG70-22
2742	AC-20P	13.3	-19.7	PG64-28
2751	AC-20P	20.8	-14.9	PG58-22
Clark Co.	AC-30	24.6	-10.5	PG64-16
2775		14.7	-22.4	PG64-28
2775		18.6	-18.4	PG64-28
N/A	AC-20P	17.7	-15.9	PG64-22
2784		26.7	-9.9	PG70-16
2784		25.5	-11.8	PG70-16
2785		25.4	-12.7	PG70-22
2825		15.1	-18.0	PG76-28
2827		9.3	-24.0	PG64-34
2827		11.0	-24.0	PG64-34
2838		8.2	-24.0	PG64-34
2852		28.0	-9.3	PG76-16
2874		1.6	-30.5	PG64-40
2876		6.2	-25.9	PG76-34
2880-I		21.0	-16.0	PG64-22
2880-II	AC-20P	18.7	-15.0	PG58-22
AC97-1159	AC-40	26.2	-12.4	PG70-22
AC97-1162		23.3	-14.0	PG64-22
AC97-1176		13.5	-20.0	PG82-28
AC98-748	AC-20P	11.8	-21.6	PG58-28
AC98-862	AC-20P	12.8	-23.4	PG64-28
AC98-903		16.9	-22.2	PG64-28
AC98-904		15.6	-21.3	PG70-28
AC98-905		15.3	-22.4	PG64-28
AC98-906		16.7	-21.6	PG64-28
AC98-908		16.2	-21.2	PG70-28

The Superpave grading system showed that the AC-30P, and AC-20+TLA binders would withstand the highest temperature. An AC-40 binder was graded at the high temperature similar to the AC-20 while another AC-40 was graded similar to the AC-30P. In the case of the low temperature, the grades of AC-30P and AC-30 binders ranged between -16 and -22^{0}C while the AC-40 binders had two extremes: one at -10 and the other at -22.

Among the binders with known AC-grades, the AC-20P binders were the most specified. A total of twenty-two AC-20P binders were tested. The following represents a breakdown of the AC-20P PG-grades.

High Temperature(C)					Low Temperature(C)				
52	58	64	70	76	-16	-22	-28	-34	-40
3	15	4	0	0	3	7	12	0	0

The majority of the AC-20P binders graded as 58-28 which is less than the expected level of performance for the polymer modified binders.

In the case of binders that do not have a corresponding AC-grades, the following distribution occurred.

High Temperature(C)					Low Temperature(C)				
52	58	64	70	76	-16	-22	-28	-34	-40
0	0	12	5	3	3	3	9	4	0

The PG64-28 was the most common grade among these binders which is well above the common PG-grade for the AC-20P binders. It should be noted that these binders were supplied in response to a specification which called for a specific grade. However, this data can lead to the observation that asphalt binders with wider PG-grades can be obtained when specified but not through the common grade of AC-20P. In other words, when asphalt suppliers are requested to provide a certain grade, they will meet it with properties to satisfy the minimum criteria regardless of whether it is the AC-20P of the PG-grade criteria. Therefore, the state highway agencies should be very specific about the required grade and not allow any room for misunderstanding.

Project Locations and Recommended Grades

In order to check the grading of the binders against the locations of the projects, the environmental conditions of the projects are needed. The Superpave model was used to identify the asphalt binder grades appropriate for the location of the projects. The Superpave data base contains environmental data for a total of 72 stations within the state of Nevada.

The pavement temperatures data, as generated by the Superpave model, are summarized in Table 2. The locations of the binders that do not have specific contract numbers were identified. The predicted high pavement temperatures seem to be higher than anticipated for these locations. Based on the predicted high and low pavement temperatures, the Superpave model recommends asphalt binder grades as shown in Table 3. The Superpave 98 percent reliability recommendations are based on the average pavement temperatures plus two standard deviations while the 50 percent reliability uses the average values.

The grading data (Table 3) indicate that if the Superpave recommendations are used, seven of the eighteen AC-20P's, all three AC-30's, the AC-20+TLA, the AC-40, and both AC-30P binders would meet the requirements under the 50 percent reliability. If the Superpave 98 percent reliability recommendation is used, none of the AC-20P's, the AC-20+TLA (2603), one of the AC-30's, and both AC-30P's would meet the requirements of the projects. In the case of the PG specified binders, all of these binders met the 50 percent reliability, and eight ou ten met the 98 percent reliability. The majority of the binders (31out of 55) met the low temperature requirements of the projects at the 50 percent reliability level and 19 out of 55 met the low temperature requirement of the projects at the 98 percent reliability. This further supports the initial observation that the high pavement temperature predicted by the Superpave model may be too conservative (i.e. too high).

Summary and Recommendations

The newly developed Superpave binder specification system was successfully used to grade fifty-five Nevada binders. All the rheological tests were conducted without any problems following the procedures recommended by Superpave.

The Superpave grading system clearly identified the AC-30P andAC-20+TLA binders as having different rheological properties from the other binders. They were identified to be applicable under warmer temperatures than the AC-20P's, while their low temperature characteristics were less desirable than the AC-20P's.

There were some discrepancies among the gradings of the various AC-20P binders. The Superpave grading system indicated that some AC-20P binders would be appropriate over a wider temperature range than others. One of the AC-20P binders was identified as being applicable under a very narrow range of temperatures (2491). However, this project is five years old and has not shown any signs of distress yet.

Based on Superpave PG grading, the AC-20P binders showed a disappointing trend with a lower range of performance indicating that the polymer modification may not be as effective as expected. However, based on the field performance of these projects, the AC-20P binders are providing the anticipated levels of performance. On the other hand when the binders were specified in term of the PG-grade, all of the supplied binders met the requirements. This indicates that binder suppliers would supply the minimum performance which satisfy the current specifications.

The environmental data needed for the recommendation of the specific binders were obtained from the Superpave model. In the majority of the cases, the Superpave recommendations did not coincide with the determined grades of the binders. The Superpave recommendations were too conservative on the high temperature grade.

It is highly recommended that the field performance of these projects should be monitored and field samples should be obtained to validate the applicability of the SHRP grading system for Nevada's conditions. Based on the findings of this research effort coupled with the field performance of the evaluated HMA mixtures, it is recommended that NDOT should modify the Superpave PG binder grading system prior to implementing it. Such modification would insure that the good performing binders such as the AC-20P's would not be replaced with less performing binders for the sake of meeting the Superpave recommended grades.

TASK 2. EVALUATE THE CONTRIBUTION OF BINDER TO LOW TEMPERATURE CRACKING OF HMA MIXTURES

The low temperature cracking of asphalt concrete pavements has been a serious concern to pavement/materials engineers for many years. It generally takes the form of transverse cracks which are nearly straight cracks across the pavement and perpendicular to the direction of traffic. This type of cracks are usually referred to as non-load associated cracking. Historically, the low temperature properties of asphalt concrete mixtures have not been included in any mix design procedure. In the past, it has been assumed that in regions where low in-service pavement temperatures are expected, the use of soft grade asphalt binders would help mitigate the low temperature cracking problem. This approach has worked in very special cases where the range of the in-service pavement temperature is narrow and the traffic level is low. In other cases where the range of the in-service pavement temperature is wide, such as the case in northern Nevada, the use of soft grade asphalt would develop rutting problems especially under the warm temperatures and heavy traffic loads. Therefore, as traffic volumes are expected to keep increasing, asphalt concrete mixtures should be designed to withstand the stresses generated by both the environment and the heavy loads under a wide range of in-service pavement temperatures.

The objective of this task was to evaluate the thermal cracking resistance of NDOT's HMA mixtures. Specifically, the research investigated the possibility of predicting the low temperature behavior of HMA mixtures using four different approaches:

a) Measure the rheological properties of the binder,

b) Measure the fracture temperature of lab mixed-lab compacted (LMLC) HMA mixtures manufactured with unaged binders,

c) Measure the fracture temperature of field mixed-lab compacted (FMLC) HMA mixtures, and

d) Measure the fracture temperature of lab mixed-lab compacted HMA mixtures manufactured with PAV aged binders (PAV-LMLC).

The goal of this investigation is to assess the differences among the various approaches to evaluate the critical low temperature for HMA mixtures. Based on this assessment, a recommendation will be made concerning the best approach to assign a low temperature grade for binders used in northern Nevada.

The theory of low temperature cracking has been identified with the volumetric change that occurs in the pavement as the temperature drops. If a pavement is unrestrained, the contraction associated with a drop in temperature will not cause any tensile stresses. However, in reality asphalt concrete pavements are restrained, and as their temperature drops, thermal stresses are generated. It is expected that both the binder and mixture control the magnitude of the generated low temperature stresses.

When dealing with thermal stresses in asphalt concrete, the major concern is the coefficient of thermal contraction of the binder. Due to the fact that the aggregate has a small thermal coefficient relative to binder, it is normally neglected. At extremely low temperatures (-20 to -40°C), asphalt binders become very brittle. This is referred to as the glassy state where all asphalt binders behave similarly. As the temperature is increased, the asphalt binder goes through a glassy transition zone and into a fluid state. The thermal coefficient of the binder varies greatly across these regions which in turn affects the generated thermal stresses.

Due to the nature of the asphalt binder, HMA mixtures have a viscoelatic behavior, therefore the material is expected to flow at warm temperatures (above glass transition) thus, dissipating the thermal stresses through stress relaxation. When the material is at cold temperatures (below glass transition) it becomes very stiff and brittle. As a result, the pavement is unable to dissipate the thermal stresses and a crack may develop. As mentioned earlier, thermally induced cracking in asphalt concrete pavements generally appear in the transverse direction of traffic. The spacing between the cracks depends on the age and width of the pavement section. For newly constructed pavements, cracks have been observed to develop at 100 to 300 feet of spacing. As the pavement ages, the binder oxidizes and becomes brittle. As a result, the crack spacing decreases to approximately 10 to 20 feet. If a transverse crack does not propagate the full width of the pavement, block cracking may occur. Another type of thermal cracking is the longitudinal crack that can develop along a pavement longitudinal lane joint since it represents a weak linear link in the pavement structure.

Over the years, there have been many attempts to determine the thermal stresses that cause cracking in HMA mixtures. One of the first successful models used mathematical equations based on infinite, completely restrained pseudo-elastic beam. The model determines the thermally induced stress while taking into account the average coefficient of contraction, initial and final temperatures, and the asphalt mix stiffness which is dependent upon time of loading and temperature.

Monismith used the stress equation that was developed by Humpherey and Martin in 1963 to predict temperature stresses in a viscoelastic slab which was assumed to be of infinite lateral extent and completely restrained.(6,7) Further investigations by Haas and Topper in 1969, showed that the stress predicted by Monismith were unrealistically high.(8) Haas and Topper also concluded that if Monismith's solution was modified to use a pseudo-elastic beam instead of a slab, the computed stresses would be slightly underestimated. The pseudo-elastic beam analysis was also supported by two road tests. Based on the road test data, the COLD model was developed to predict low temperature cracking in asphalt concrete pavements.(9)

Another way of determining thermal stresses in asphalt concrete pavements have been through indirect estimation of the mixtures strength at low temperatures. This is based on the binders properties such as; Penetration Index, softening point, and the stiffness-temperature relationship. The Van Der Poel nomograph is then used to predict the stiffness of the asphalt concrete mix. Using an assumed or measured coefficient of thermal contraction, the stress-temperature relationship is then obtained using the solution proposed by Hill and Brien.(10)

The direct measurement of the mixture's low temperature strength have been the most widely used. The use of laboratory tests would eliminate the need to measure or assume a coefficient of expansion of the mixture. One of the first to successfully use direct measurement for thermal stresses was Fabb in 1974.(11) This was accomplished by measuring the stress required to maintain a specimen at constant length under a constant rate of cooling (5, 10, 27°C/hr). Fabb concluded that the rate of cooling has little or no effect on the failure temperature. However, in 1980, Bloy recognized that different rates of cooling below 5°C/hr influenced the temperature at which cracking occurred in asphalt concrete, whereas rates of cooling above 5°C/hr had no influence.(12)

Factors Influencing Thermal Cracking

From a decade of extensive research in determining thermal stresses in asphalt concrete, it was found that the factors influencing low temperature cracking can be broadly categorized under: a) Material, b) Environmental, and c) Pavement Geometry. The following sections discuss each of these factors.

Material Factors

The material properties that can influence the thermal behavior of asphalt aggregate mixtures include:

1) Asphalt Cement: The stiffness or consistency of asphalt cement is the single most important factor that effects the degree of low temperature cracking in a HMA mix. At low temperatures, the asphalt binder becomes brittle and loses its ability to absorb energy through viscous flow. As a result, the binder strain becomes intolerable and cracking occurs in the pavement. The temperature at which the pavement cracks is referred to as the limiting stiffness temperature. In 1989, the Transportation Research Board published an extensive annotated bibliography on the temperature susceptibility of asphalt cement to low temperature cracking.(13)

2) Aggregate Type and Gradation: Aggregates that have high abrasion resistance, low freeze-thaw loss and low absorption are expected to have minimal effect on low temperature strength. On the other hand, high absorptive aggregate reduces low temperature strength because the aggregate has a tendency to absorb more and the asphalt binder available for bonding will be less than that of the non-absorptive aggregate. The gradation of the aggregate used in the mix have apparently shown little influence on low temperature cracking.

3) Asphalt Binder Content: The data generated in phase I of this research study indicated the for three out of six mixtures, the effect of the asphalt content was significant. (14) The tensile strength of these mixtures increased as their asphalt content increases up to an optimum value which coincided with the design optimum

asphalt content. For the other three mixtures, the effect of asphalt content was insignificant.

4) Air Voids Content: The effective air void content, compaction effort, and permeability of an asphalt concrete mix, are expected to influence the low temperature cracking of the mix. The data generated in phase 1 of this research study indicated that in general the low temperature tensile strength of an HMA mixture decreases as the air voids increases. (14) The same data also showed that the impact of air voids variations is insignificant when the actual air voids are between 6 and 8%.

Environmental Factors

The environmental factors that affect thermal behavior include temperature, rate of cooling, and pavement age. The discussion of each is included bellow:

1) Temperature: As the pavement cools, the thermal stress increase, thus creating a potential for thermal cracking. The pavement surface temperature is affected mainly by wind and ambient air temperature. When the temperature of the pavement is maintained below the glass transition temperature, thermal stresses exceeding the tensile strength of the mix maybe generated, and a thermal crack will occur on the surface.

2) Rate of Cooling: At higher rates of cooling the pavement has a greater tendency of thermal cracking. The highest variation in pavement temperature generally occur in the transition between night and day.

3) Pavement Age: Obviously, as the Pavement ages the chance of thermal cracking increases. Oxidation of asphalt cement is associated with the increase in stiffness. The presence of high air void content of the mix causes a faster rate of aging thus greater chance of cracking at a short service life. Also, with time in service, there is an increasing probability of occurrence of more extreme low temperatures as the pavement becomes older.

Pavement Geometry

The pavement structure geometry that affect thermal behavior include pavement width and thickness. The discussion of each is included below:

1) Pavement Width: Field data have shown that narrow pavements (24 feet) experience transverse cracks ranging approximately every 100 feet, whereas cracks on wide pavements (50-100 feet) can be spaced greater than 150 feet. As the pavement ages, alligator, block, and longitudinal cracks may develop where the difference in crack spacing may not be apparent.

2) Pavement Thickness: From the Ste. Anne Test Road, it was revealed that the thicker the asphalt concrete layer, the lower the incident of thermal cracking. In the road test, the thickness of the pavement varied from 4 to 10 inches.

Thermal Stress Restrained Specimen Test (TSRST)

As mentioned earlier, the objective of this phase of the study was to establish a procedure by which the critical low temperatures for pavements in northern Nevada can be evaluated. The thermal stress restrained specimen test (TSRST) was selected for this investigation because of its ability to provide a direct measurement of the fracture temperature of the mixture.

This test is based on a low temperature testing system originally developed by Arand.(15) The basic concept behind this test is that the asphalt concrete specimen is cooled down at a constant rate while being restrained from contracting. An electro-hydraulic system is used to maintain the specimen at a constant height. Figure 9 shows the testing set up for the TSRST.

As the HMA is cooled down and forced to maintain a constant height, tensile stresses will be generated throughout the length of the sample. It is expected that the thermally induced stresses will increase as the temperature of the specimen decreases until the specimen fractures. At the break point, the stress reaches its maximum value, which is referred to as the fractures stress, with a corresponding fracture temperature. Figure 10 shows a typical relationship between the stress and temperature for the TSRST. The slope of the thermally induced stress curve, dS/dT, increases until it reaches a maximum value. At colder temperatures, dS/dT becomes constant and the stress-temperature curve is linear. The transition temperature divides the curve into two parts, relaxation and non-relaxation. As the temperature approaches the transition zone, the asphalt cement becomes stiffer and the thermally induced stresses are not relaxed below this temperature. The slope tends to decrease again when the specimen is close to fracture. This may be due to the stiffness of the asphalt binder or the development of micro cracks.

Sample Preparation

The test specimen is 2"x2"x10" beam. The material is mixed using mechanical mixing and compacted using the kneading compactor. The original compacted beam is 3"x3"x15", therefore, the beam is cut on all four sides to the reduced size. All four sides are cut in order to create uniformity all around and to avoid localized failures. It is very critical that the beams are cut straight and free of local defects. Such defects may cause the specimen to fail at warmer temperatures and under lower stress levels.

Once the specimen is cut, it must be glued to the end platens with perfect alignment using epoxy compound. This is accomplished through a permanent laboratory set up exclusively developed for this test. Once the specimen is glued, it is left in the alignment stand at room temperature until the epoxy is cured, which is normally about twenty four hours.

Test Procedure

The SHRP A003A project has developed a complete laboratory procedure for the TSRST, while the following presents a concise summary of the test. (16) After the epoxy cured, the specimen with end platens is placed in an environmental room controlled at 5°C for twelve hours as a preconditioning period. At the end of the preconditioning period, the specimen is moved into the testing machine and the two LVDT's are connected to both sides (180 degrees) in order to control the change of length in the specimen. It should be noted here that the main objective of this test is to restrain the specimen from contraction as it cools down. Therefore, the LVDT's used on the specimen should have high resolution in order to tightly control the change in specimen length. In addition, the rods used to connect the LVDT's to platens should have very small coefficient of thermal expansion.

The environmental chamber on the testing machine can be cooled down by liquid nitrogen or a mechanical chilling device with a variable rate controlling unit. A resistance temperature device (RTD) is used to control the temperature inside the environmental chamber. Once the sample is in place, the LVDT's are zeroed, the environmental chamber is kept at 5°C for one hour after which the temperature is dropped at a constant rate of 10°C/hour. During this period, the load necessary to maintain the specimen at a constant length is measured.

Data Analysis

A total of twenty-one mixtures were selected for testing. All of the mixtures have dense gradations with maximum nominal size of 3/4" and percent passing #200 between 3 and 7 %. Basically there are no significant differences among all the gradations.

Fourteen out of the twenty-three projects use polymer modified binders (AC-20P), one project uses AC-30 binders, one project uses an AC-30P binder, one project uses an AC-20 binder, one project uses an AC-20+TLA binder, and three projects use PG graded binders. NDOT typically applies a 3/4" open graded friction course on top of dense graded asphalt concrete layer. The mixtures evaluated in this experiment all come from the dense graded layer since it represents the major structural layer of the section.

The laboratory experiment for this research project concentrated on evaluating the following processes:

a) Evaluate the critical low temperature of the HMA mixtures based on the rheological properties of the binders. This effort used the data generated in Task A.

b) Evaluate the critical low temperature of the HMA mixtures based on the TSRST fracture temperature of the LMLC mixtures fabricated with original binders.

c) Evaluate the critical low temperature of the HMA mixtures based on the TSRST fracture temperature of the FMLC mixtures.

d) Evaluate the critical low temperature of the HMA mixtures based on the TSRST fracture temperature of the PAV-LMLC mixtures.

The binders used in process d are the same ones used to evaluate the binders rheological properties in process a. In other words, the binders in processes a and d have been aged through the RTFO and PAV tests while the binder in process b has been aged through the Hveem mix design process and the binder in process c has been aged through the field mixing and paving operations. The most important comparison would be the one between processes a and d since the binder is normally selected based on process a while process d represents its actual performance as it ages in the field mixtures. Processes b and c should show colder fracture temperatures than a and d since the binder in these processes has not been subjected to long term aging. It should be noted that not all fifteen projects had the field mixed-lab compacted mixtures but the majority of projects have the data for processes a and d. The Superpave binder specification system was used to evaluate the critical low temperature of the binder. This system was fully described under Task A.

As described earlier, the TSRST was used to evaluate the fracture temperatures of all types of mixtures. Table 4 summarizes the critical low temperatures as evaluated from all four processes. Under perfect conditions of materials and testing procedures, the following observations should be possible:

- The critical low temperatures measured on the LMLC and FMLC mixtures should be very close. This observation was not met on the majority of the mixtures where the critical low temperatures of the two mixtures differ by up to 7°C. In five out of fourteen cases, the critical temperature of the two mixtures were within 2°C. Also, the data did not show any trend of LMLC mixtures being consistently more brittle than the FMLC mixtures or vise versa.

- The critical low temperatures measured on the LMLC mixtures should be colder than the ones measured on the PAV-LMLC mixtures. The data showed that this was true for thirteen cases. The data in Table 4 indicate that nine out of twenty mixtures provided critical low temperatures for LMLC and PAV-LMLC that are +/-2°C from each other. The data indicated that little over fifty percent of the mixtures supported this hypothesis while the other mixtures violate it.

- The critical low temperatures evaluated based on the Superpave binder specification system should be similar to the ones measured on the PAV- LMLC mixtures. The data in Table 4 showed that this observation was violated on almost all mixtures. The low temperature binder grade assigned by the Superpave binder specification system are significantly warmer than those that are measured on PAV- LMLC mxitures.

Summary and Recommendations

The objectives of this task were to evaluate the most applicable method for measuring the critical low temperature for HMA mixtures to be used in northern Nevada. These objectives were met by evaluating four different processes by which the critical low temperatures of HMA mixtures can be evaluated on twenty-one different mixtures from various locations throughout Nevada. Based on the analysis of this data, the following conclusions can be made:

- There is serious discrepancy among the data generated by the TSRST. The fracture temperatures measured by the TSRST on LMLC mixtures and LMLC mixtures made with PAV aged binders were very close to each other on almost fifty percent of the tested mixtures. This observation posses two serious questions: a) is the TSRST measuring the appropriate critical low temperature? and b) does the PAV provides the appropriate aging of the binder?. The other fifty percent of the tested mixtures showed that the LMLC mixtures have colder temperatures than the PAV- LMLC mixtures. It is interesting to note that all mixtures made with unmodified binders (total of three mixtures) showed colder temperatures for the LMLC mixtures than those of PAV- LMLC mixtures. On the other hand, the group of mixtures that showed equal temperatures for LMLC and PAV-LMLC mixtures were all made with polymer modified binders except for the AC-20+TLA binder. This may indicate that either the TSRST or the PAV binder aging process may have some limitations when used with polymer modified binders.

- For the majority of cases, the critical low temperatures identified by the Superpave binder specification system are warmer than the ones measured on all three types of mixtures (i.e. LMLC, FMLC, and PAV-LMLC). This was true for sixteen out of twenty-one mixtures. This indicates that the low temperature binder grade as identified by the Superpave binder specification system is the most conservative.

The only possible way for resolving any one of the discrepancies that arose from this research would be to monitor the field performance of these mixtures and correlate them back to the measured properties. In the meantime, it is recommended that NDOT starts using the Superpave binder specification system to assign critical low temperatures for HMA mixtures to be used in northern Nevada. Based on the data presented in this report, the low temperature grade identified by the Superpave binder specification system is conservative enough to insure that no low temperature cracking would be encountered at both the early and long term aging of the mixtures. The performance monitoring of the majority of these projects throughout the past six years has supported these recommendations.

TASK 3. DEVELOPMENT OF A DEMERIT SYSTEM

The overall objective of this task was to develop a demerit system which NDOT can use with the Superpave binder grading system. The characteristics of such a system would be to base the deduct values on the performance measures of asphalt binders. As indicated earlier, the Superpave binder grading system is based on evaluating the contribution of the asphalt binder toward rutting, fatigue, and low temperature cracking of the HMA mixtures. Therefore, any demerit system will have to be based on the fundamental properties of the asphalt binders which sought to influence the long term performance of HMA mixtures.

Prior to the development of the demerit, the NDOT current PG grading map was evaluated to ensure that the proposed regions are adequately covered with the proposed PG grades. This effort was needed in order to ensure that the demerit system will be based on accurate performance predictions according to the location of the projects. Therefore, Task 3 was divided into two subtasks:

Subtask 3.1: Update Nevada's Binder Grading Map

Subtask 3.2: Develop a Demerit System

Subtask 3.1: Update Nevada's Binder Grading Map

The objective of this subtask was to produce the most applicable binder grading map for the state of Nevada based on the Superpave 98% reliability level. The aim of the research was to draw a binder grading map where the majority of the locations would receive a reliability level of 98% or higher for both the high and low temperature grades.

The Superpave weather data base was used to obtain the pavement temperatures for the 86 weather stations located in the State of Nevada. (17) The state of Nevada currently has five PG regions, each region corresponds to a PG grade. Figure 11 shows the geographical boundaries of the current NDOT PG regions. By analyzing the current NDOT regions in conjunction with the weather conditions at the 86 weather stations, the following observations were made:

- 44.0% of the Superpave weather stations located in the 70-16 NDOT PG region have a reliability less than 98% for low temperature.
- 30.8% and 23.1% of the Superpave weather stations located in the 64-28 NDOT PG region have a reliability less than 98% for low and high temperature, respectively.
- 16% and 8% of the Superpave weather stations located in the 58-40 NDOT PG region have a reliability less than 98% for high and low temperature, respectively.

Reference 17 presents a detailed analysis of this data. The detailed analysis was left out of this report in an effort to keep the size of this report manageable. By looking at the above conditions, it was decided that the current NDOT PG regions should be redrawn in order to minimize the number of Superpave weather stations with reliabilities less than 98%. Figure 12 shows the Superpave weather stations that need to be reassigned to a different PG region. Figure 12 also shows the actual reliabilities for the stations that do not reach the 98% level. While redrawing the boundaries of the NDOT PG regions, extreme care was taken not to overestimate the reliability which would make it economically unfeasible.

Figure 13 shows the recommended NDOT PG regions. In summary, some of the existing regions were extended, one region was assigned a new grade and a new region was developed. Using the recommended PG regions in conjunction with the Superpave weather data for the 86 weather stations, the following observations were made:

- 11.1% of the Superpave weather stations located in the 64-28 new NDOT PG region have a reliability less than 98% for low and high temperatures.
- 10.0% of the Superpave weather stations located in the 70-22 new NDOT PG region have a reliability less than 98% for low temperature.
- Other violations in addition to the above listed ones are all below 5% which are very insignificant.

By comparing the percentages of the Superpave weather stations that have less than 98% reliabilities for the current and recommended NDOT PG regions, it can be seen that the recommended PG regions would significantly reduce the possibility of having a location within a PG region with lower reliability. Therefore, it is recommended that the new NDOT PG regions be implemented as shown in Figure 13. Reference 17 should be consulted for an in-depth presentation of the above analysis.

Subtask 3.2: Develop a Demerit System

Pay factors were determined using a life cycle analysis. A discount rate value of 4 percent and constant dollars were used in the in economic analysis. The analysis period was chosen to be 30 years for rutting and fatigue pay factors determination and 22 years for the low temperature cracking pay factors. The total present worth method was used to convert any present and future expenses to the same basis on today's dollars over the analysis period. The total present worth was then transformed into equivalent uniform annual costs. The next step transformed the equivalent uniform annual costs into total costs corresponding to the performance life of the pavement. The money lost is then computed as the difference between the total costs corresponding to the out of specification binders and the total costs corresponding to the binders meeting the Superpave criteria. The difference as percent of binder cost is finally calculated and subtracted from 100 to obtain the pay factor in percent. For an in-depth presentation of this analysis, reference 17 should be consulted.

The rutting factors were developed based on the relationship between the $G^*/\text{Sin}(\delta)$ of the RTFO aged binder and the number of cycles to produce 2% permanent shear strain in the repeated shear constant height (RSCH) test. Performance loss of the asphalt-aggregate is assimilated to the reduction in number of ESAL's to failure necessary to produce a specific rut depth. The limiting rut depth was chosen to be 0.22 in., which corresponds to 2% maximum permanent shear strain in the RSCH test. Table 5 summarizes the recommended rutting pay factors. The sensitivity analysis shows that the rutting pay factor calculation model is very stable. The HMA thickness is the input that has the most significant effect on the rutting pay factors for which a 10% increase in construction thickness will result in a 6.0% increase in pay factors.

The pay factors for fatigue were established using relationship between the $G^*Sin(\delta)$ of the PAV aged binder and the fatigue life of mixtures. The fatigue life of mixtures was defined as the number of cycles necessary to reduce the mixtures stiffness to a fifty percent level in the strain control flexural bending fatigue test. It was therefore possible to calculate the performance loss as the reduction in fatigue life versus $G^*\sin(\delta)$ values on PAV residue at 10 rad/s. The pay factors were then determined through a life cycle cost analysis over a 30 years analysis period. Table 6 summarizes the fatigue pay factors. A sensitivity analysis showed that the fatigue pay factors are very stable. (17)

The low temperature cracking pay factors were established considering a 22 years analysis period. The pay factors are based on a correlation between the cracking index at 7 years and the Superpave low temperature test results (stiffness and slope). The cracking Index was defined as the number of transverse cracks in a 500 feet section. The cracking index were determined at other years making an assumption about the cracking evolution with time. The maintenance costs were then calculated for any year by using a fixed crack filling cost and the cracking index estimated at the different years. A maximum amount of cracking corresponding to a cracking index of 33.3, was selected as the maximum amount of thermal cracking that would typically develop in a pavement. This amount of cracking will be reached after 7 years for a slope of 0.21 or a stiffness of 550 MPa. Consequently, the pay factors are low in this range of test results and are for instance 65 and 40%, respectively for m-value = 0.21 and stiffness = 550MPa. Table

7 summarizes the pay factors for low temperature cracking. The sensitivity analysis shows that the low temperature pay factor model is very stable for slope values greater than 0.22 and stiffness values smaller than 480 MPa.

It is recommended that NDOT implements all three types of pay factors on construction projects. The actual demerit imposed on the job should be the one resulting from the lowest pay factor.

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Table 1. Summary of flash point, viscosity, and RTFO weight loss for all binders.

Cont. Number	AC Type	Flash Pt. (^0C)	Brookfield Vis. @ 135^{0}C (cP)	RTFO Wt. Loss (%)
N/A	AC-20	293	.375	.14
N/A	AC-20	271	.400	.15
2480	AC-20P	257	.763	.33
2491	AC-20P	259	.363	.28
2501	AC-30	274	.463	.15
2501	AC-20P	266	.638	.13
2530	AC-20P	277	.650	.26
2544	AC-20P	319	.658	.05
2545	AC-20P	271	.319	.15
2552	AC-20P	279	.596	.26
2558-93	AC-20P	266	.313	.15
2558-94	AC-20P	266	.279	.21
2567		265	.462	.42
2591	AC-20P	280	.500	.10
2594	AC-20P	288	.725	.06
2603	AC-20+TLA	299	.566	.11
2604	AC-40	263	.371	.19
2604	AC-30	NA	.567	.28
2611	AC-20	NA	.350	.32
2611	AC-20P	NA	.750	.10
2615	AC-20P	307	.617	.20
2617	AC-20P	NA	.367	.21

Table 1. Summary of flash point, viscosity, and RTFO weight loss for all binders. (Continued)

Cont. Number	AC Type	Flash Pt. (^{0}C)	Brookfield Vis. @ 135^{0}C (cP)	RTFO Wt. Loss (%)
2622	AC-30P(1)	284	1.454	.08
2622	AC-30P(2)	299	2.567	.08
2704	AC-20P	277	.441	.59
2711	AC-20P	280	.262	.61
2726	AC-30P	321	1.051	.01
2751	AC-20P	254	.583	.36
Clark Co.	AC-30	305	.353	.19
2775		240	.825	.34
2775		243	.450	.32
NA	AC-20P	230	.363	.22
2784		296	.412	.24
2784		304	.375	.26
2785		276	.425	.13
2825		274	.950	.24
2827		266	.412	.27
2827		270	.438	.52
2838		280	1.225	.28
2852		304	.900	.12
2874		262	.771	.36
2876		306	1.329	.28
2880-I		340	.375	.14

Table 1. Summary of flash point, viscosity, and RTFO weight loss for all binders. (Continued)

Cont. Number	AC Type	Flash Pt. (^0C)	Brookfield Vis. @ 135^0C (cP)	RTFO Wt. Loss (%)
2880-II	AC-20P	282	.416	.39
97-1159	AC-40	286	.412	.26
97-1162		310	.287	.20
97-1176		298	3.308	.18
98-748	AC-20P	300	.604	.13
98-862	AC-20P	272	.604	.44
98-903		282	.400	.46
98-904		260	.870	.36
98-905		306	1.008	.53
98-906		278	1.075	.40
98-908		262	1.108	.32

Table 2. Minimum and maximum pavement temperatures as estimated by the Superpave data base.

Contract	Superpave (°C)			
	50 % Reliability		98 % Reliability	
	Min	Max	Min	Max
2480	-12	62	-17	68
2491	-24	57	-35	59
2501	-7	64	-12	66
2501	-7	64	-12	66
2530	-7	64	-11	68
2544	-27	56	-37	58
2545	-19	57	-27	61
2552	-11	65	-17	68
2558	-18	56	-27	59
2591	-24	57	-35	59
2594	-7	60	-12	62
2603	-7	64	-12	66
2604	-7	64	-12	66
2611	-26	54	-34	56
2615	-30	53	-39	57
2617	-15	59	-23	62
2622	-7	64	-12	66

Table 2. Minimum and maximum pavement temperatures as estimated by the Superpave data base (Continued).

Contract	Superpave (^0C)			
	50 % Reliability		98 % Reliability	
	Min	Max	Min	Max
2704	-30	53	-39	56
2711	-21	55	-30	58
2726	-12	62	-17	67
2751	-22	53	-28	57
Clark Co.	-12	62	-17	67
2775	-18	56	-27	58
2784	-6	62	-12	66
2785	-12	62	-17	67
2825	-18	56	-27	58
2827	-22	53	-30	56
2838	-21	57	-33	61
2852	-7	64	-12	66
2874	-27	53	-34	56
2876	-21	54	-29	57
2880	-19	57	-27	61

Table 3: Recommended binder grades based on Superpave data base.

Contract Number	AC Grade	PG Grade	Superpave 50% Reliability	Superpave 98% Reliability
2480	AC-20P	PG58-28	PG64-16	PG70-22
2491	AC-20P	PG52-16	PG58-28	PG64-40
2501	AC-30	PG64-16	PG64-10	PG70-16
2501	AC-20P	PG58-28	PG64-10	PG70-16
2530	AC-20P	PG58-28	PG64-10	PG70-16
2544	AC-20P	PG58-28	PG58-28	PG64-40
2545	AC-20P	PG58-22	PG58-22	PG64-28
2552	AC-20P	PG58-28	PG70-16	PG70-22
2558-93	AC-20P	PG58-22	PG58-22	PG64-28
2558-94	AC-20P	PG58-16	PG58-22	PG64-28
2591	AC-20P	PG58-16	PG58-28	PG64-40
2594	AC-20P	PG58-28	PG64-10	PG64-16
2603	AC-20+TLA	PG70-16	PG64-10	PG70-16
2604-1	AC-40	PG64-10	PG64-10	PG70-16
2604-2	AC-30	PG70-22	PG64-10	PG70-16
2611-1	AC-20	PG64-16	PG58-28	PG58-40
2611-2	AC-20P	PG64-28	PG58-28	PG58-40
2615	AC-20P	PG58-28	PG58-34	PG58-40
2617	AC-20P	PG52-22	PG64-16	PG64-28
2622-1	AC-30P	PG70-22	PG64-10	PG70-16
2622-2	AC-30P	PG70-16	PG64-10	PG70-16

Table 3: Recommended binder grades based on Superpave data base (Continued).

Contract Number	AC Grade	PG Grade	Superpave 50% Reliability	Superpave 98% Reliability
2704	AC-20P	PG58-28	PG58-34	PG58-40
2711	AC-20P	PG58-22	PG58-22	PG58-34
2726	AC-30P	PG70-22	PG64-16	PG70-22
2751	AC-20P	PG58-22	PG58-22	PG58-28
Clark Co.	AC-30	PG64-16	PG64-16	PG70-22
2775		PG64-28	PG58-22	PG64-28
2784		PG70-16	PG64-10	PG70-22
2785		PG70-22	PG64-16	PG70-22
2825		PG76-28	PG58-22	PG64-28
2827		PG64-34	PG58-22	PG58-34
2838		PG64-34	PG58-22	PG64-34
2852		PG76-16	PG64-10	PG70-16
2874		PG64-40	PG58-28	PG58-34
2876		PG76-34	PG58-22	PG58-34
2880		PG64-22	PG58-22	PG64-28
2880	AC-20P	PG58-22	PG58-22	PG64-28

Table 4. Low temperature cracking critical temperatures for various HMA mixtures.

Contract Number	AC Grade	Critical Temperature (C)			
		Binder	LMLC	FMLC	PAV-LMLC
2480	AC-20P	-30	-38		-29
2491	AC-20P	-28	-25	-28	-26
2501	AC-30	-22	-32		-26
2501	AC-20P	-28	-35		-37
2530	AC-20P	-29	-39	-33	-24
2545	AC-20P	-22	-33	-28	-27
2552	AC-20P	-30	-35	-39	-33
2558-93	AC-20P	-24	-21		-30
2558-94	AC-20P	-23	-32	-33	-33
2603	AC-20+TLA	-18	-15	-13	-16
2604-2	AC-30	-15	-26	-23	-24
2611	AC-20	-23	-25		-18
2611	AC-20P	-30	-33	-26	-35
2617	AC-20P	-24	-31		-31
2622	AC-30P	-22	-34	-29	-31
2704	AC-20P	-29	-35	-37	-30
2711	AC-20P	-27	-35	-35	-30
2742	AC-20P	-30	-37	-39	-36
2825		-28	-32	-35	-28
2838		-34	-45	-45	-39
2874		-41		-39	

Table 5. Rutting pay factors based on $G^*/\mathrm{Sin}(d)$ of the RTFOT aged binder.

Value of $G^*/\mathrm{Sin}(d)$	**Pay Factor (percent of contract price)**
2.00-2.20	100
1.60-1.99	80
< 1.60	65*

* Reject Status: the price adjustment applies if the HMA mixture is allowed to remain in place.

Table 6. Fatigue pay factors based on $G^*\mathrm{Sin}(d)$ of the PAV aged binder.

Value of $G^*\mathrm{Sin}(d)$	**Pay Factor (percent of contract price)**
5.0-5.1	100
5.1-5.2	95
5.2-5.4	90
5.4-5.6	80
5.6-6.0	70
> 6.0	55*

* Reject Status: the price adjustment applies if the HMA mixture is allowed to remain in place.

Table 7. Low temperature cracking pay factors based on S(t) of the PAV aged binder.

Value of S(t)	**Pay Factor (percent of contract price)**
300-330	100
330-390	95
390-450	90
450-510	85
510-570	40*
>495	0**

* Reject Status: the price adjustment applies if the HMA mixture is allowed to remain in place.
** Reject status: the asphalt binder should not be paid for if it is allowed to remain in place.

Table 8. Low temperature cracking pay factors based on (m) of the PAV aged binder.

Value of m	**Pay Factor (percent of contract price)**
0.28-0.30 and above	100
0.24-0.28	95
0.22-0.24	85
0.21-0.22	65*
<0.21	0**

* Reject Status: the price adjustment applies if the HMA mixture is allowed to remain in place.
** Reject status: the asphalt binder should not be paid for if it is allowed to remain in place.

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C[a]	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] G*/sinδ, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: G*/sinδ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: G*sinδ, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1: S, Maximum, 300 MPa, m - value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

[a] Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program, may be provided by the specifying agency, or by following the procedures as outlined in PPX.

[b] This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

[c] For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of G*/sinδ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry (AASHTO T201 or T202).

[d] The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 64- and above, except in desert climates, where it is 110°C.

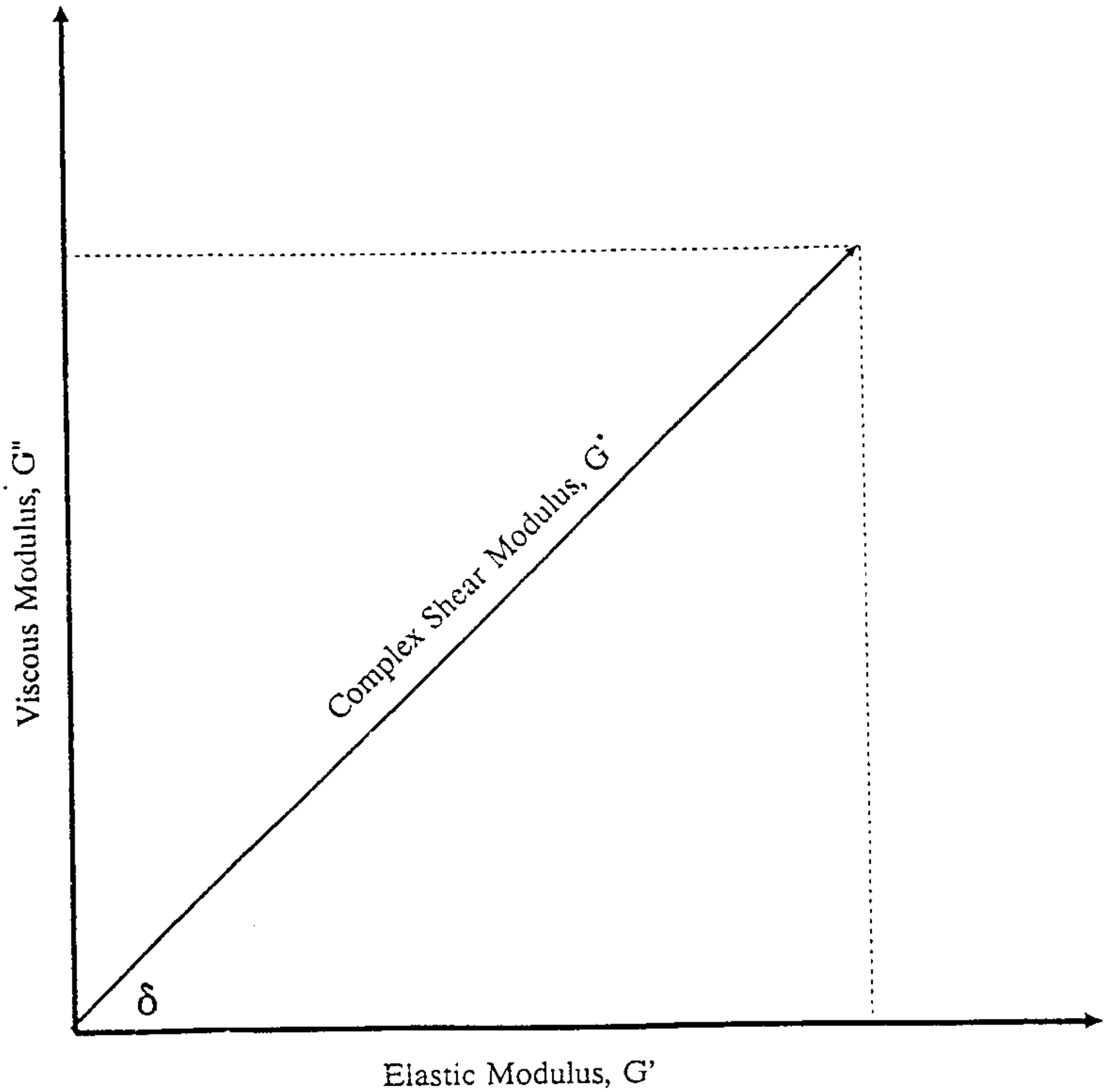
[e] Physical Hardening — TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and m-value are reported for information purposes only.

[f] If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

Figure 1. Superpave Binder Grading System.

PERFORMANCE GRADE	PG 70-						PG 76-					PG 82-				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average 7-day Maximum Pavement Design Temp, °C[b]	<70						<76					<82				
Minimum Pavement Design Temperature, °C[b]	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																
Flash Point Temp, T48: Minimum °C	230															
Viscosity, ASTM D4402:[d] Maximum, 3 Pa•s, Test Temp, °C	135															
Dynamic Shear, TP5:[c] G*/sinδ, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN (T179) RESIDUE																
Mass Loss, Maximum, percent	1.00															
Dynamic Shear, TP5: G*/sinδ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP1)																
PAV Aging Temperature, °C[d]	100(110)						100(110)					100(110)				
Dynamic Shear, TP5: G*sinδ, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening[e]	Report															
Creep Stiffness, TP1:[f] S, Maximum, 300.0 MPa, m - value, Minimum, 0.300 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24

Figure 1. Superpave Binder Grading System (Continued).



Storage (Elastic) Modulus	Loss (Viscous) Modulus	Loss Tangent
$G' = (G^*)\cos(\delta)$	$G'' = (G^*)\sin(\delta)$	$\tan(\delta) = G''/G'$

Figure 2. Relationship Among the Various Rheological Properties of Asphalt Binders

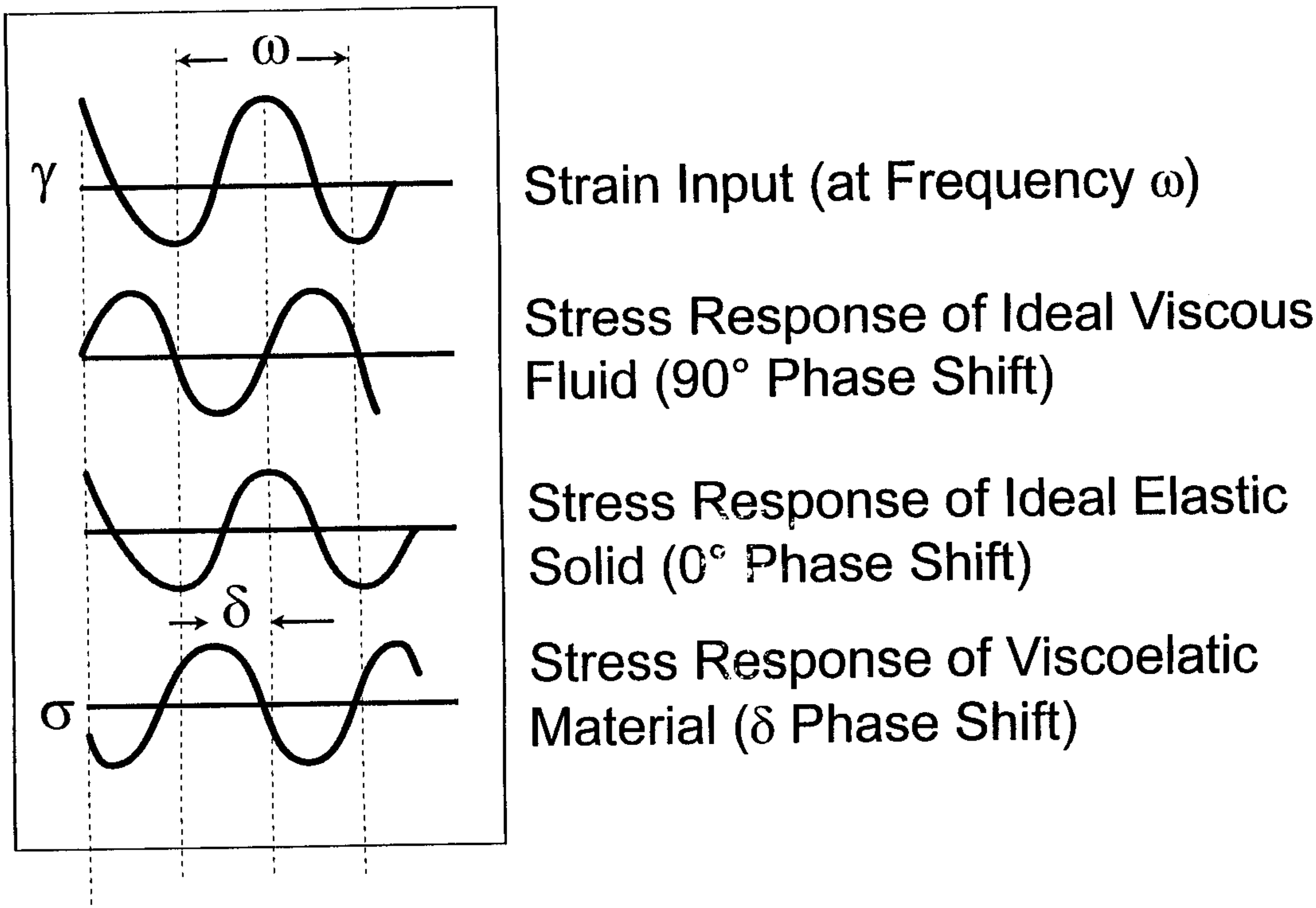


Figure 3. Relationship Among Stress-Strain-Phase Lag

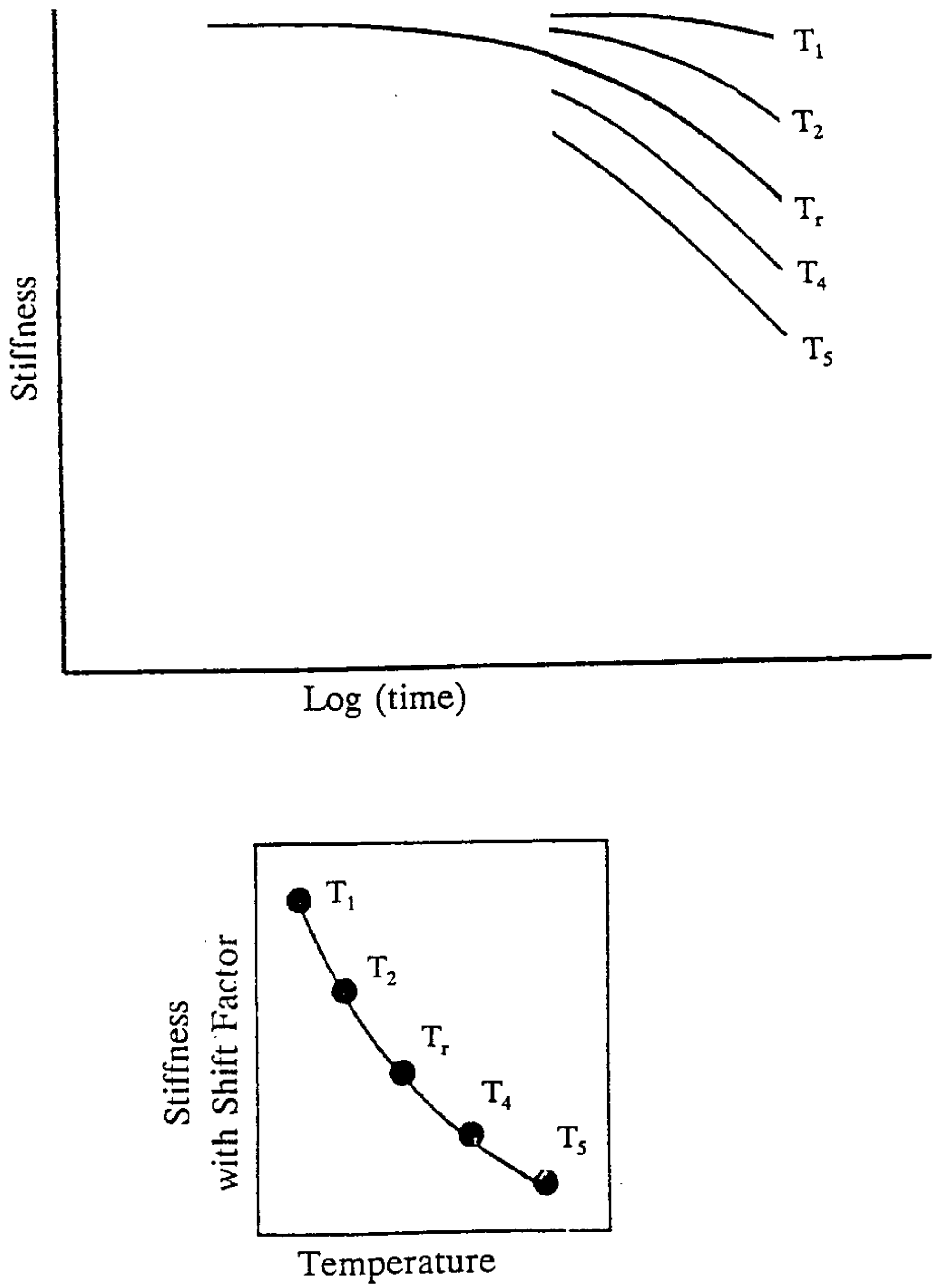


Figure 4. Master Curve for Asphalt Binders

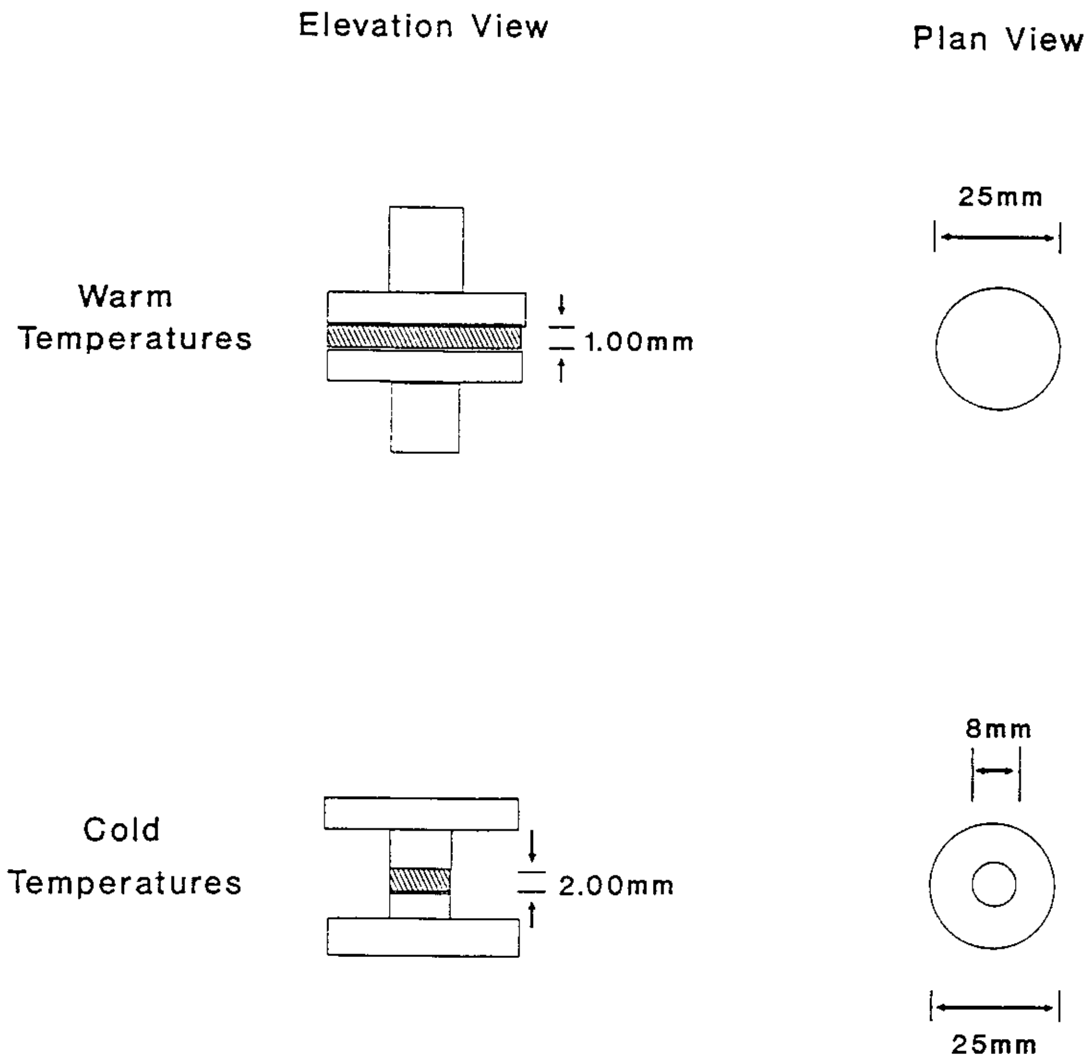


Figure 5. Parallel Plate Test Configurations

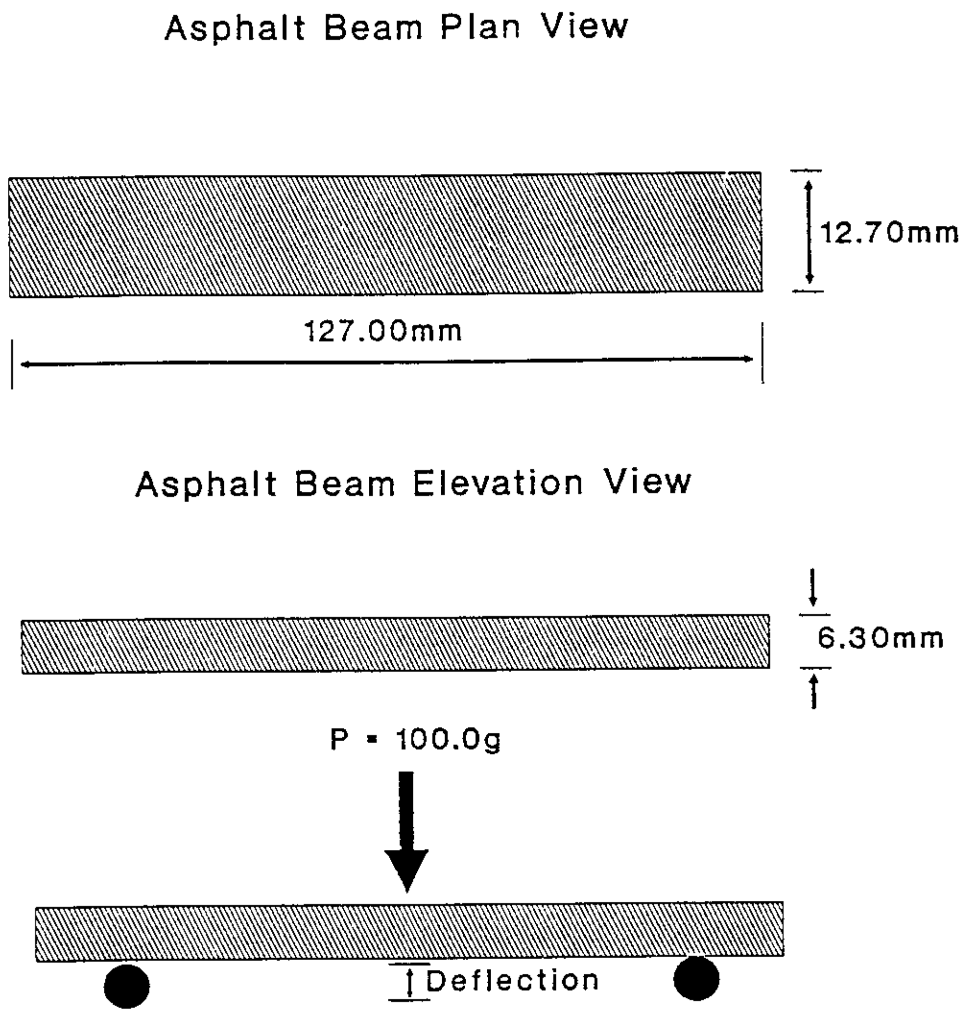


Figure 6. Bending Beam Test Configuration

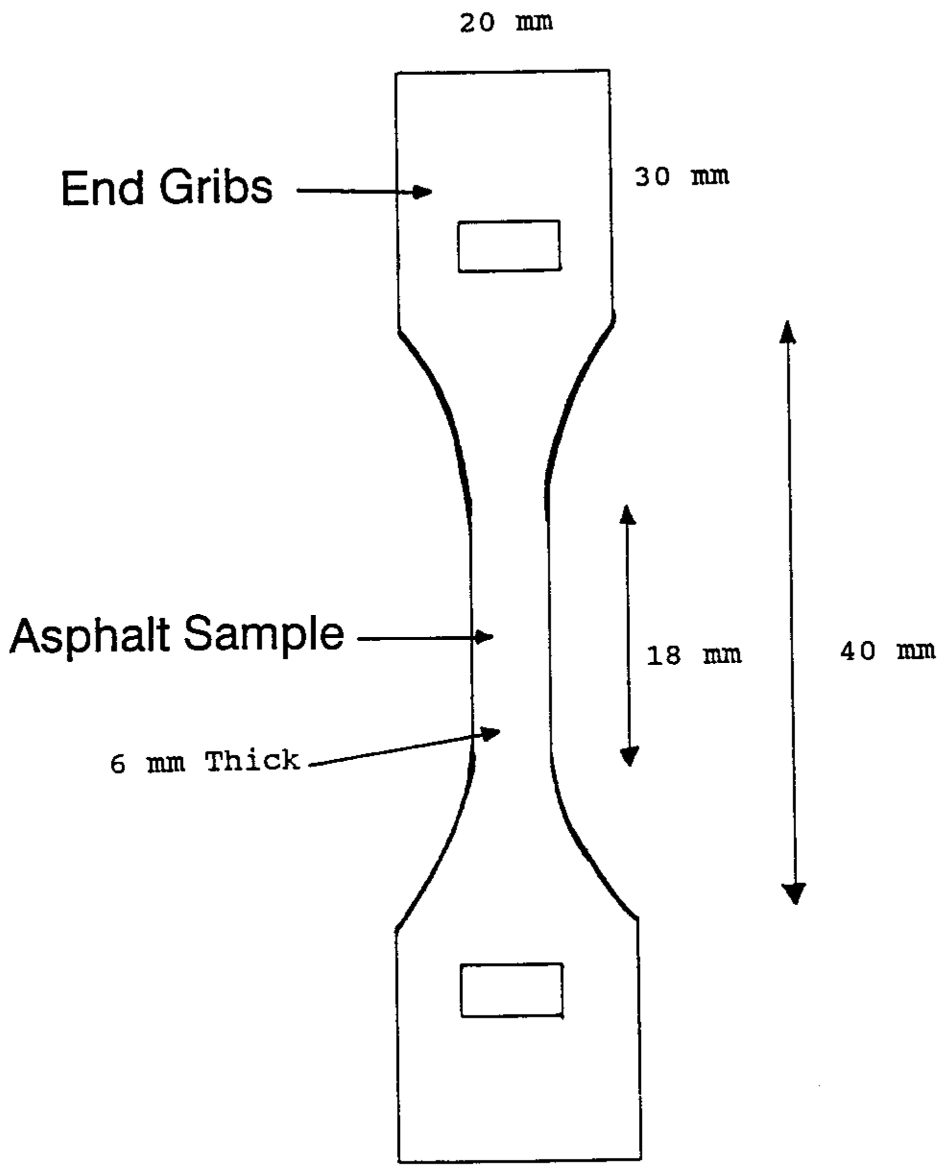


Figure 7. Schematic of the Direct Tension Test

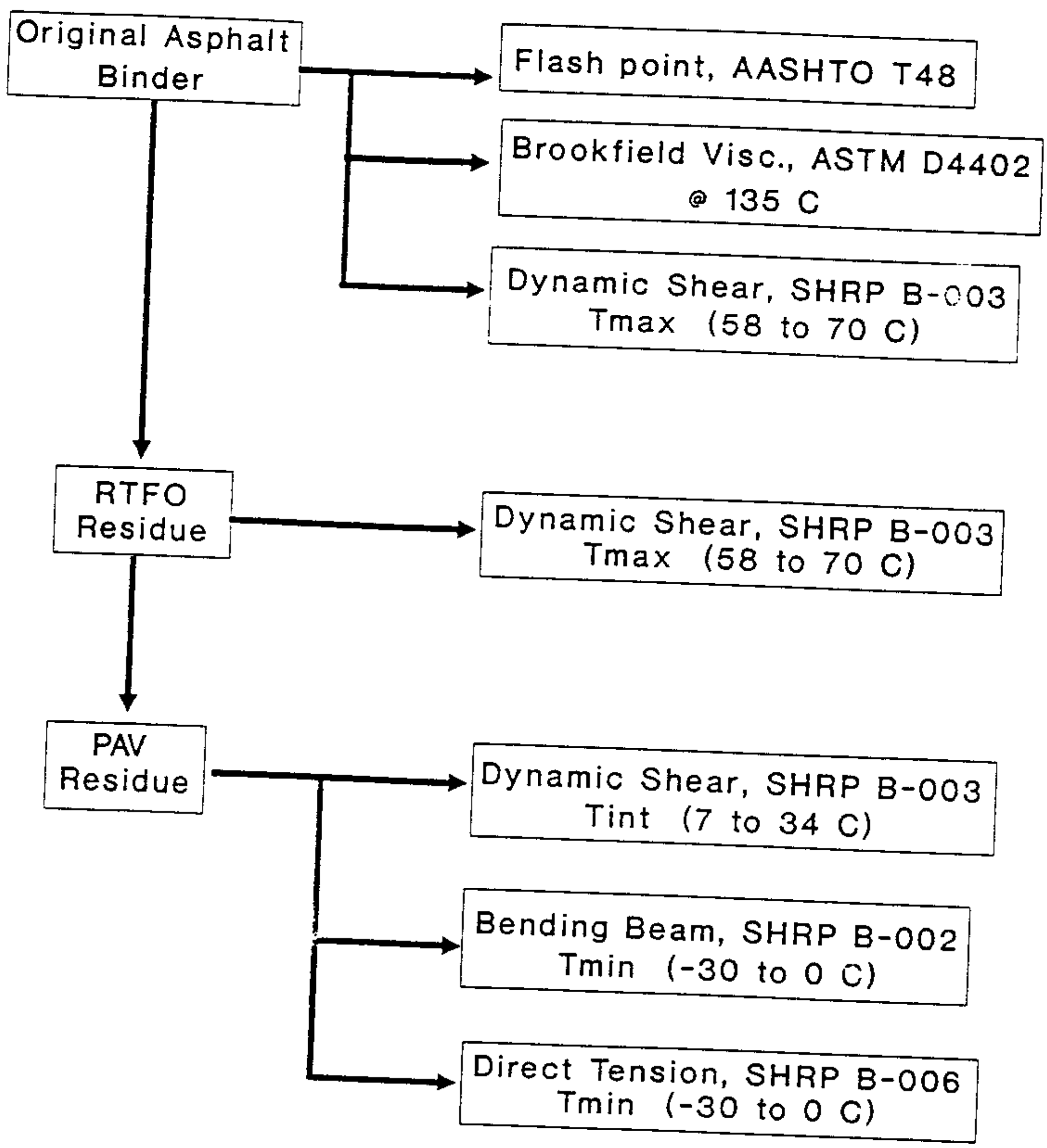


Figure 8. Flow Chart of the Laboratory Testing Program to Grade the Asphalt Binder

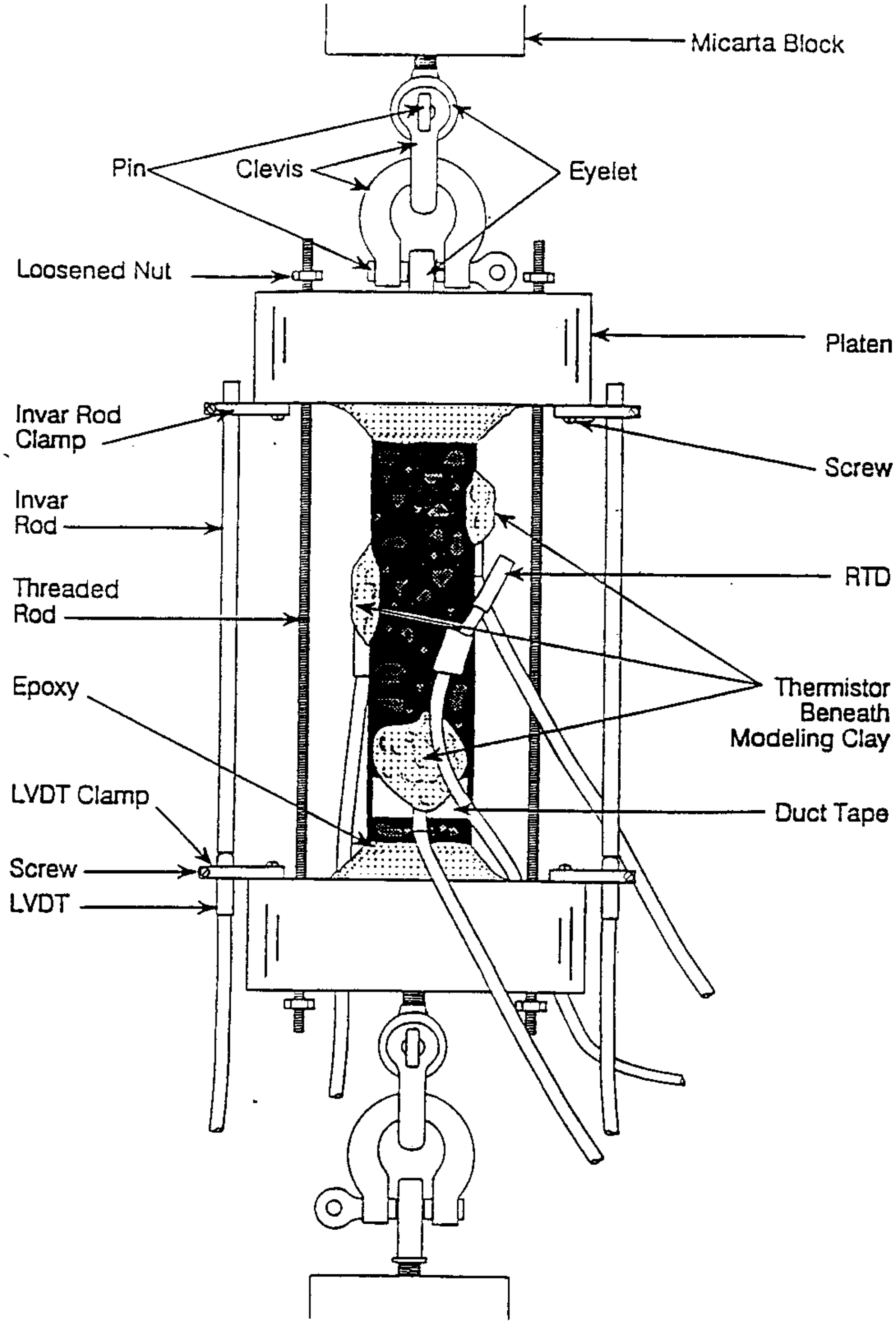


Figure 9. Schematics of the TSRST Testing Setup.

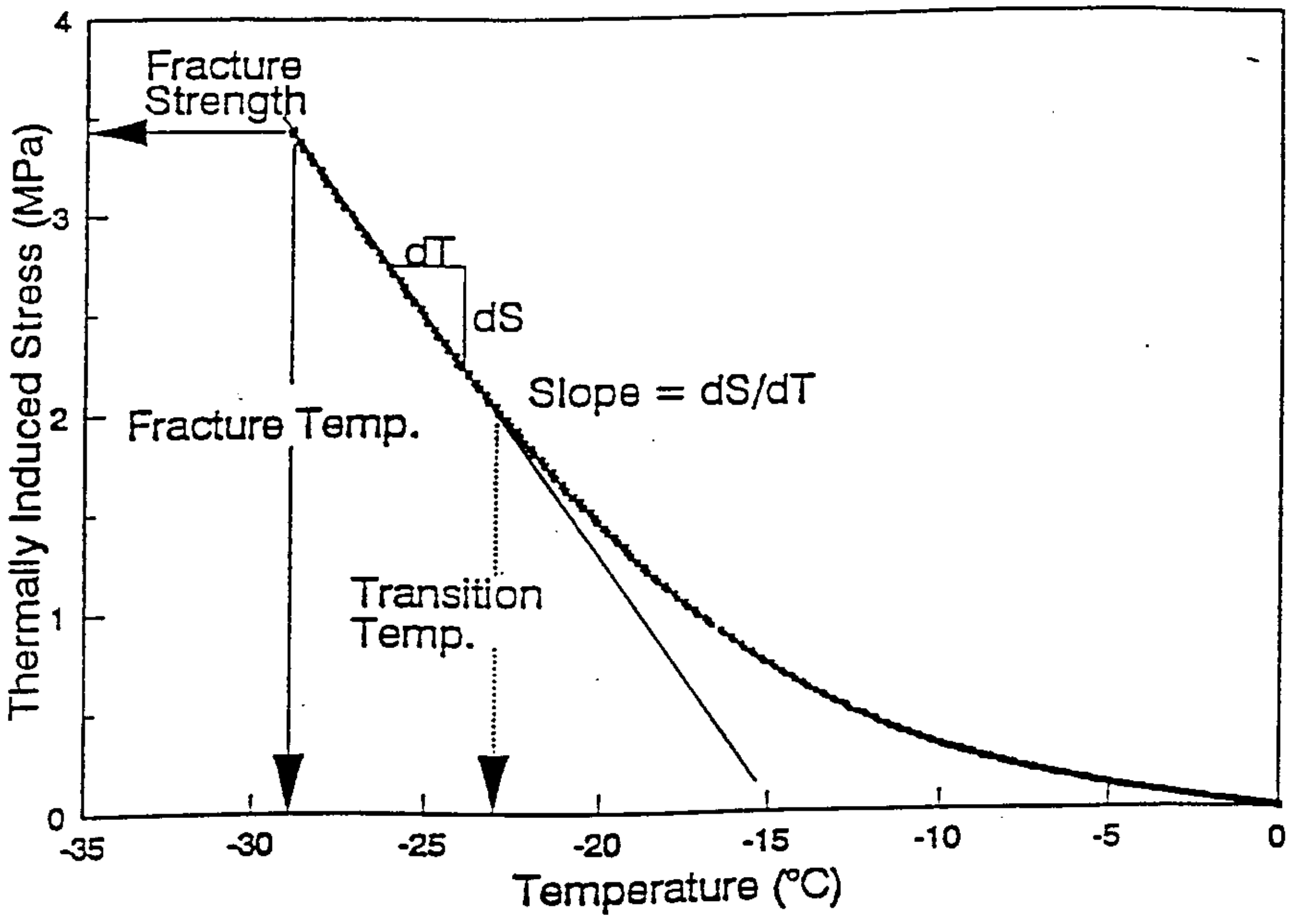


Figure 10. Typical Relationship between Thermal Stresses and Temperature.

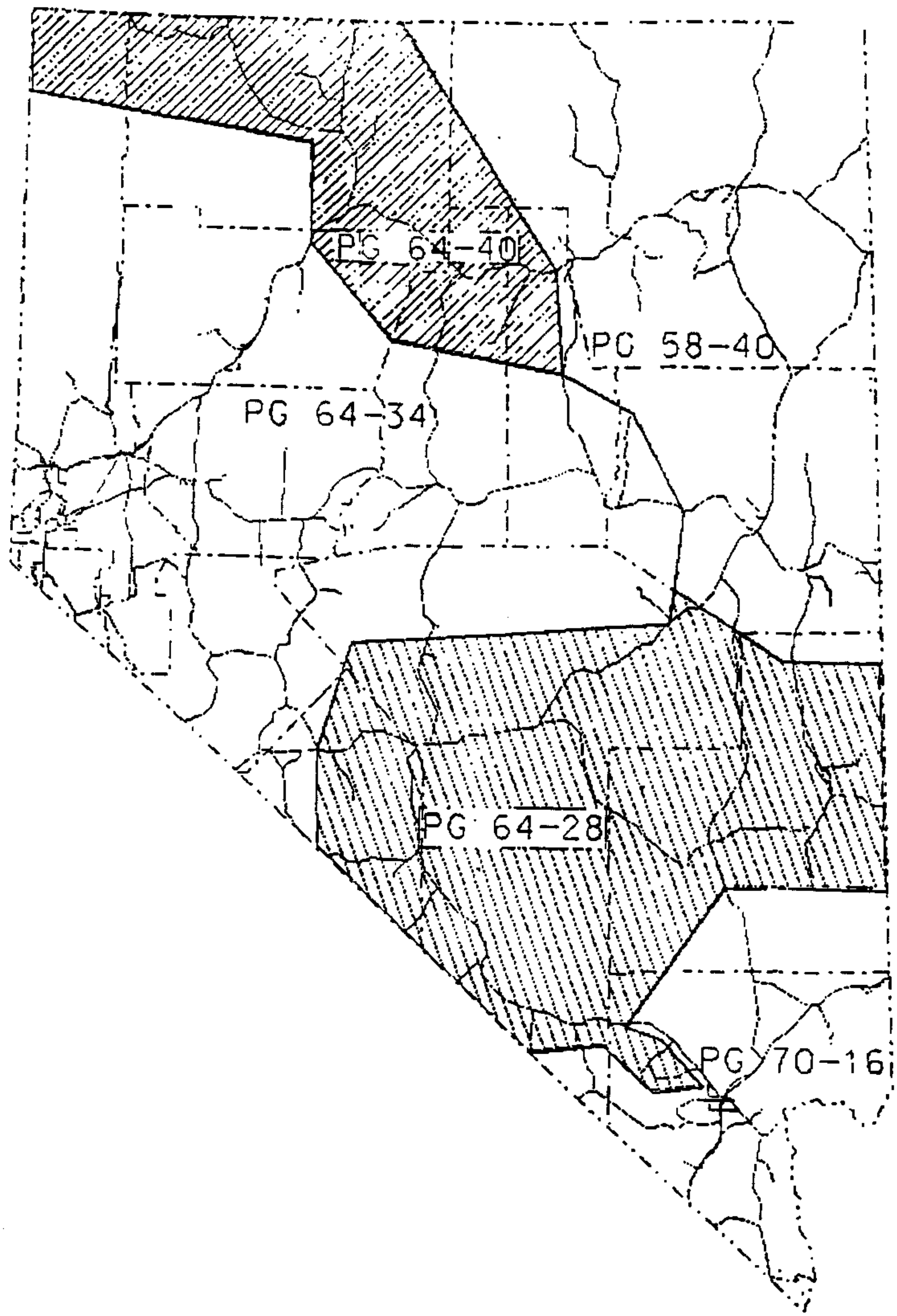


Figure 11. Existing Geographical Delineation of the Five NDOT PG regions.

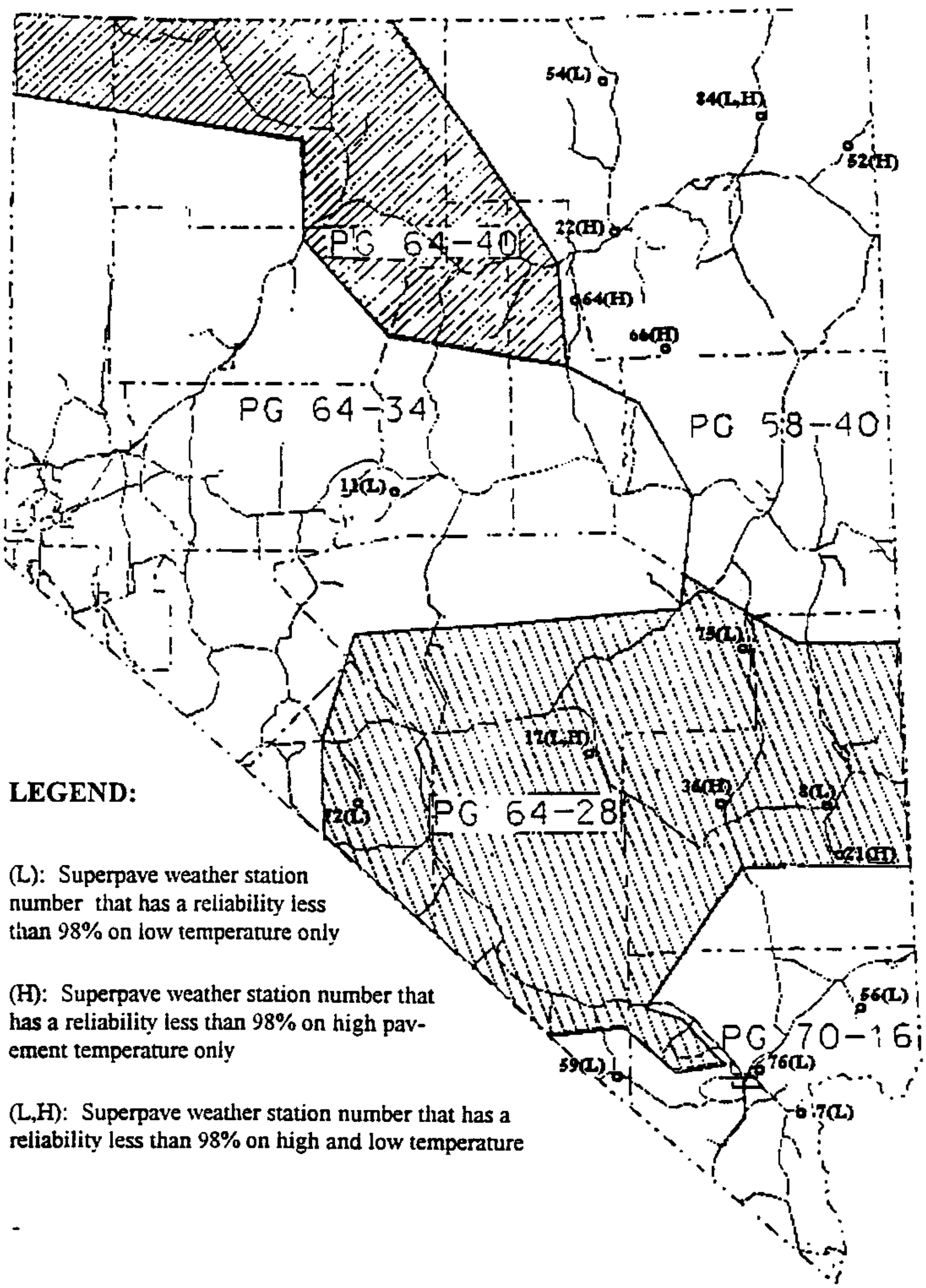


Figure 12. Existing NDOT PG regions and Superpave Weather Stations that do not Meet the 98% Reliability Criteria..

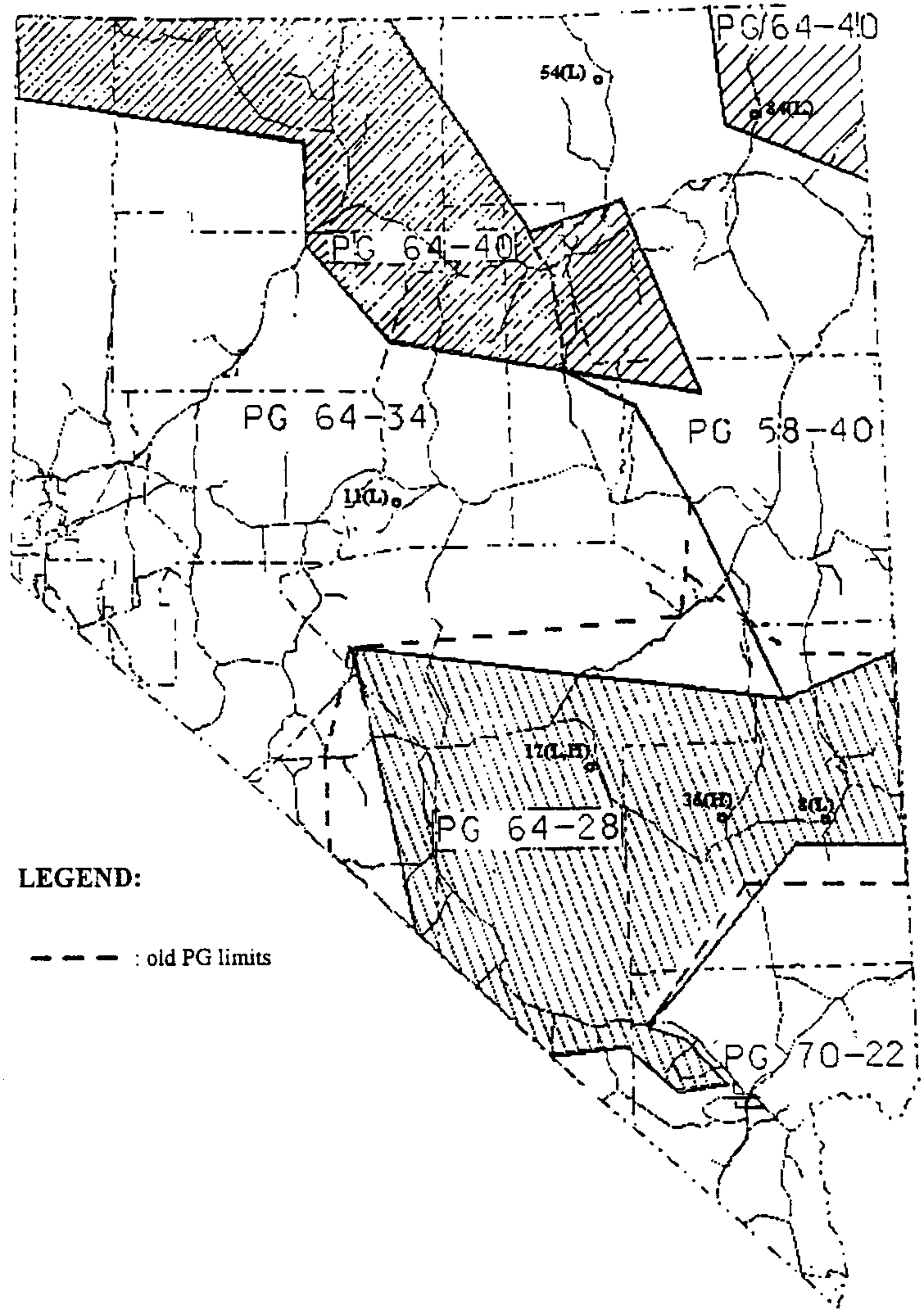


Figure 13. New NDOT PG Regions and Remaining Superpave Weather Stations that do not Meet the 98% Reliability Criteria..

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