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Development of Input Parameters to Predict the Performance of Rhabilitated Asphalt Pavements

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DEVELOPMENT OF INPUT PARAMETERS TO PREDICT THE PERFORMANCE OF RHABILITATED ASPHALT PAVEMENTS

BY

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Abstract

Rhode Island Department of Transportation (RIDOT) has been rehabilitating asphalt pavements, and looking for a sustainable strategy. A Rhode Island Highway, Route 165 in Exeter was rehabilitated in 2013 with five different strategies, i.e., control (without additive), calcium chloride, asphalt emulsion, Portland cement and Geo-grid. The base/subbase layer (between asphalt base and existing granular subbase) was prepared by Full Depth Reclamation (FDR) method. The goal of the project was to predict the performance of rehabilitated asphalt pavement with different base/subbase strategies and to select the best reclamation technique. The AASHTOWare Pavement ME Design (PavementME) software was selected to predict performance. It requires four different categories of inputs, i.e., general information, traffic input, climatic input, and material properties. Although RIDOT was able to prepare inputs of General information, Traffic, and Weather, there were no accurate material input parameters. Material properties required extensive testing which includes resilient modulus, dynamic modulus, creep compliance, thermal conductivity, poison's ratio and volumetric properties. Resilient moduli of subgrade soils, existing subbase materials, and reclaimed subbase/base materials with and without additives were determined at University of Rhode Island (URI). Properties of Hot Mix Asphalt (HMA) including dynamic modulus and creep compliance were determined in the present study. It was observed from the outputs that asphalt concrete top down fatigue cracking happened on all strategies whereas the one with Portland cement only passed for thermal cracking and permanent deformation in asphalt layer. It was also observed that the strategy with Portland cement performed the best whereas the one with asphalt emulsion did the worst, i.e., it would last for only 5

years. In summary, it was predicted with available input parameters that the reclaimed strategies in order of best performance were: Portland cement, calcium chloride, control, geo-grid and asphalt emulsion. It has been recommended that predicted performance will be evaluated with long-term field monitoring, and that results of the present study will be utilized for future rehabilitation projects.

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Chapter 1

Introduction

1.1. Pavement Rehabilitation

Pavement rehabilitation is a major activity for all highway agencies and has several consequences on agency resources and traffic disruptions because of extensive and extended lane closures. The traffic volumes on the primary highway system, especially in urban areas, have seen tremendous increases over the last 20 years, leading in many instances to earlier-than-expected failures of highway pavements. The aging of the Interstate highway system and other primary systems built during the 1950s and 1960s has resulted in the expenditure of a large portion of highway funds on pavement rehabilitation. Efforts continue to be made to develop techniques and procedures that will result in cost effective and longer-lasting pavement rehabilitation to serve the nation's highway system well into the 21st century. The process of pavement rehabilitation involves the following procedures:

1. Prioritization of pavements in need of rehabilitation, which incorporates monitoring activities to assess the functional and structural condition of pavements;

2. Development of feasible rehabilitation strategies;

3. Selection of the most cost-effective rehabilitation strategy given a set of constraints, which may include reduced service life, life-cycle costs, and budgetary constraints; and

4. Adequate prediction and measurement of performance of the rehabilitated pavements.

1.2. Pavement Evaluation

The state of the practice of pavement rehabilitation is good but can be better. In the last 10 years, significant improvements have been made in pavement evaluation techniques and in rehabilitation equipment and procedures. Considerable progress has been made in techniques to evaluate pavement condition. Equipment for measuring surface profiles and for assessing the structural capacity of pavements is widely used by highway agencies and other practicing pavement engineers. The common availability of the falling weight deflectometer (FWD) has resulted in a more objective assessment of the structural capacity of pavements and timely rehabilitation of under designed or overloaded pavements.

1.3. Asphalt Pavement Rehabilitation

Asphalt pavement rehabilitation typically involves milling and resurfacing of the existing asphalt pavement to mitigate the effects of rutting, cracking, potholes and other distresses. Resurfacing thickness may depend on the condition of the existing pavement, anticipated future truck traffic, and available funding. Under heavy truck traffic, the expected service life of the rehabilitated pavement is typically about 8 to 12 years.

It has been estimated that the amount of miles of truck traffic on our highways will be increasing and surpassing all other modes of freight shipments in the near future. Tractor trailers and heavy vehicles account for a majority of the damage done to highways (Lee and Peckham 1990). The States, especially Rhode Island, are having a hard time keeping up with and paying for maintenance and rehabilitation (M&R). This means there will be more wear done to our highways than ever before, and the States will have to do more M&R with less funding. To meet upcoming highway demand, the Rhode Island Department of Transportation (RIDOT) has been testing alternative subbase materials as reclamation strategies, and has been expanding their use. To meet up best rehabilitation strategy/technique for a pavement, it is necessary to predict its performance over a certain number of years.

1.4. Justification for and Significance of Study

Asphalt or flexible pavements typically designed for 20 years, and generally consist of four layers (namely subgrade soils, granular subbase, granular or asphalt base, and asphalt surface). With the passage of time the top surface of pavement deteriorates quickly due to mainly heavy truck traffic and adverse climate, if not, inadequately designed pavement structures, and end up having different kinds of asphalt distresses, e.g., rutting, fatigue cracking, thermal cracking, potholes, and roughness. To maintain and rehabilitate pavement there should be a solution or strategy to meet up the 20 years design life of pavement. RIDOT used different rehabilitation strategies in the past, such as use of Reclaimed Asphalt Pavement (RAP), Reclamation, Subbase stabilization, and Geo reinforcement etc. The use of RAP in cold recycling has been increased, and two categories were identified: partial depth recycling. e.g., Cold Central Plant Recycling (CCPR) and Cold In-Place Recycling (CIR) and Full Depth Reclamation (FDR). However, there is no effective guideline which strategy is the best in RI. In this research study FDR with different additives and/or stabilization strategies used on base/subbase of a Rhode Island (RI) Highway, i.e., Route 165 located in Exeter as shown in Figure 1.1 were focused.

Design of flexible pavement depends on different number of factors like properties of subgrade soils, traffic or loading, properties of materials (which includes aggregates and asphalt mixtures), climate or environment, and cost. Basically, all these engineering design factors except cost are the base for predicting the performance of pavement.

Since RIDOT established 5 test sections with RAP base/subbase materials with different treatments on RI Rt. 165, performance for different rehabilitation strategies can be predicted by using a mechanistic empirical approach, i.e., AASHTOWare Pavement ME Design (PavementME) software. With the help of this graduate research State of Rhode Island will be able to predict amount of distresses on pavement over a certain period as well as number of the years for which pavement will last under different rehabilitated strategies. However, there is no accurate material input parameters to use Pavement ME.

The results of this research study will also allow using proper input parameters and selecting the optimal rehabilitating strategy for future rehabilitation/reconstruction of projects and to develop guidelines for long-term pavement evaluation. Selection of optimal rehabilitation strategy will be done through prediction and evaluation, e.g., distress observations, deflection analysis, and roughness measurement etc.

1.5. Objectives

The objectives of this project were:

1. To collect existing data from the five test sections on Route 165 including resilient moduli of subbase materials before and after rehabilitation/reconstruction,
2. To determine accurate material input parameters,
3. To predict the performance of pavement structures with five different rehabilitation strategies using PavementME software in terms of rutting, cracking and roughness,
4. To select the best rehabilitation/reconstruction techniques for Rt. 165 test sections,

5. To come up with framework to select an optimal technique and/or strategy for future rehabilitation/reconstruction projects, and
6. To develop guidelines for long-term evaluation.

1.6. STRUCTURE OF THESIS

This research study consists of five Chapters. Chapter 1 starts with Introduction. Chapter 2 discusses about rehabilitation design of RI Route 165, Chapter 3 describes about determining accurate material input parameters, Chapter 4 provides the performance prediction of RI Route 165 by Pavement ME, Chapter 5 describes about evaluation methods for rehabilitated asphalt pavement and Chapter 6 provides conclusions and recommendations.

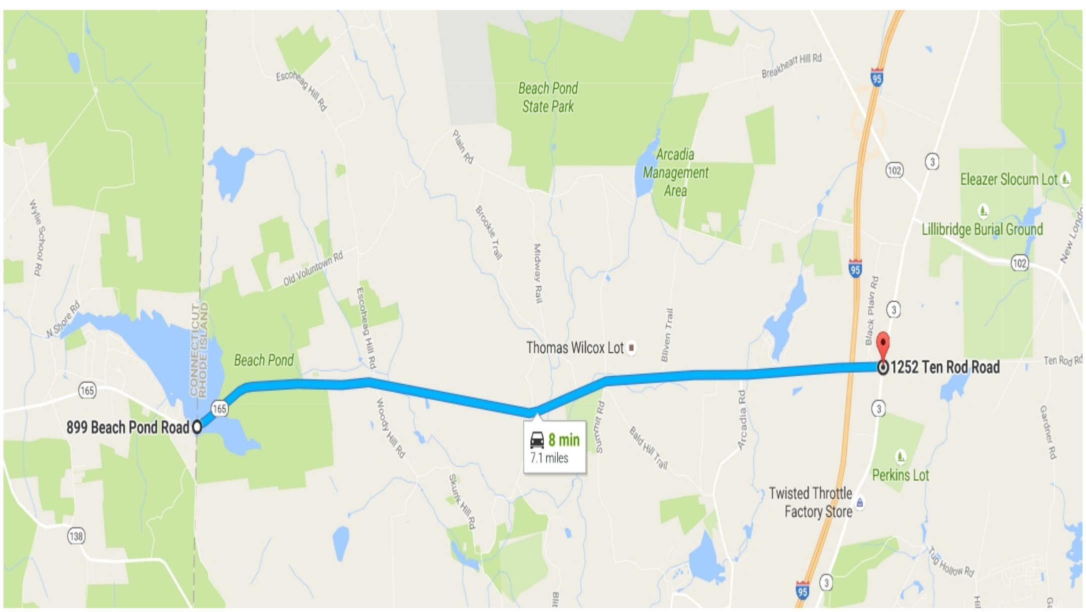


Figure 1.1 Location of Route 165 in Exeter Rhode Island

Chapter 2

Rehabilitation Design of RI Route 165

2.1. Pavement Evaluation for Rehabilitation Design

Rehabilitation design requires an evaluation of the existing pavement to provide key information. The first step in the pavement rehabilitation design process involves assessing the overall condition of the existing pavement and fully defining the existing pavement problems. Figure 2.1 shows the steps and activities for assessing condition of existing pavement for rehabilitation design. For rehabilitation design it is recommended that engineer prepares an evaluation plan that outlines all activities needed for investigating and determining the cause of the pavement defects. The field evaluation plan could consist of a detailed pavement condition survey, non-destructive testing, destructive sampling and testing, and traffic control, as a minimum. Field collection data and evaluation plan consists of following steps.

1. Collection of Historic Data

This step involves collecting of information such as location of the project, year constructed, year and type of major maintenance, pavement design features, material and soil properties, traffic climate conditions, and any available performance data.

2. Initial Field Survey

This step involves conducting a windshield and detailed distress survey of sampled areas within the project to assess the pavement condition. Data required includes distress information, drainage conditions, subjective smoothness, traffic control options, and safety considerations.

3. Initial data evaluation and the determination of additional data requirements:

This step requires determining critical levels of distress/smoothness and causes of distress and smoothness loss using information collected during the first field survey. This list will aid in assessing preliminary existing pavement condition and potential problems. Additional data needs can also be addressed during this step.

4. Second field survey:

This step involves conducting detailed measuring and testing such as coring and sampling, profile smoothness measurement, skid resistance measurement, deflection testing, drainage tests, and measuring vertical clearances.

5. Laboratory testing of samples:

This step involves conducting tests such as material strength, resilient modulus permeability, moisture content, composition, density and gradations using samples obtained from the second field survey.

6. Second data evaluation:

This step involves the determination of existing pavement condition and an overall problem definition. Condition will be assessed and the overall problem defined by assessing the structural, functional, and subsurface drainage adequacy of the existing pavement.

7. Final Field and office data composition:

This step involves a preparation of final evaluation report.

RI Route 165 rehabilitation design followed the above procedure and discussed in next sections.

2.2. Historical data collection of Route 165

Route 165 located in the State of Rhode Island's town Exeter. It also connects state of Rhode Island with state of Connecticut and runs parallel to Interstate 95 highway and carries a lot amount of heavy truck traffic. RI Route 165 was last reconstructed in 1986 and reclaimed to a depth of 125 mm (5 in.) mixed with calcium chloride. The pavement thickness, after resurfacing, was 37.5 mm (1-1/2 in.) of bituminous surface course and 62.5 mm (2-1/2 in.) of bituminous modified binder course over a 125 mm (5 in.) cold recycled base layer mixed with a ratio of 1:2 bituminous pavement/gravel and 200 mm (8 in.) of existing gravel subbase layer as shown in Figure 2.2.

2.3. Initial Field Survey of RI Route 165

A geotechnical engineering exploration and analysis was conducted at the request of RIDOT by V.A. Nacci and Associates, Consulting Soil, and Foundation Engineers on September 25, 1987. It may be noted that, Route 165 was originally built on soft deposits (swamp). Depending on the nature of the soft deposit, "construction" dealt with this in one of two ways: one was by removal of the unsuitable material and the other was by "floating the embankment on the soft soil, often with considerable settlement" (Nacci et al. 1987).

Eleven test borings were completed for the reconstruction, which found embankments consisting of sand, some gravel, silt, fibrous organic deposits (peat), and organic silt. Other test borings indicated that Route 165 was built on glacial till and stratified kame deposits. There were pockets in the granite bedrock near the surface, which contributed to a high water table. An exploration and analysis found an additional seven areas of swamp deposits.

Table 2.1 shows the Route 165's various soil types and properties, and American Association of State and Highway Transportation Officials (AASHTO) classifications of the soil ranges from A-1 to A-4 (USDOA 1981). Soils within Route 165 has a low shrink-swell potential but has a potential for frost action. Route 165 is not comprised of any clay 3 materials, therefore the plasticity index is zero. Areas of Route 165 that contain Adrian, Walpole, and Ridgebury have severe wetness, low strength, and severe frost action.

2.4. Initial Data Evaluation of RI Route 165

The RIDOT, throughout the years, has performed both material and pavement testing on Route 165 roadway primarily through RIDOT maintenance programs. One of the pavement testing include skid-resistance test which measure pavement smoothness and were performed in 2003, 2006, and 2010. The results obtained in 2003 showed that, Route 165 had skid-resistance number between 52 and 58 whereas in 2010, the skid-resistance number ranged from 50 to 56 while the 2006 data showed values between 43 to 49. Overall, Route 165 has shown a noticeable decrease in pavement smoothness and ride ability.

2.5. Second Field Survey and Laboratory testing of samples for RI Route 165

In 2012, the RIDOT Material Section, in conjunction with the URI Department of Civil and Environmental Engineering, performed testing on the unbound materials from five sample areas within Route 165. The areas where field samples were taken are in the general vicinity of the poorer numbers of skid-resistance values. Twelve field samples were taken between November 27, 2012 and December 6, 2012. Nuclear gauge readings were taken at the sample areas at the same time to measure in-situ dry density, wet density, water contents, and percent moisture. Stationing, utility pole numbers, and planned treatment areas were recorded to insure future samples were taken in the same locations. The 2012 samples were taken to URI for resilient modulus testing, and the results from RIDOT/URITC Project Number 000154 were used as parameters to run the AASHTOware Pavement ME Design (PavementME) program (Bradshaw et al. 2015). Physical properties of selected subgrade soils from Route 165 site sample locations are presented in Table 2.2 (Bradshaw et al. 2015). Physical properties of selected 1980s RAP blends from Route 165 site is presented in Table 2.3. Physical properties of selected FDR RAP blends of Route 165 site is presented in Table 2.4. (Bradshaw et al. 2015).

2.6. Second Data Evaluation and Final Field and Office Data Compilation of RI Rt. 165

After assessing the existing pavement condition of Route 165 it was selected for major rehabilitation. For that purpose a strategy was developed to rehabilitate Route 165 with five different sections having five different additives in its Base/Subbase layer that were (Control having no additive, Calcium Chloride, Portland cement, Asphalt Emulsion, and Geo-grid) and to predict the performance of all test sections in terms of distresses by using PavementME approach. Location description of all test sections are shown in Table 2.5. Methodology used to rehabilitate Route 165 with control and four other additives is discussed in next section.

2.7. Methodology and/or Procedure

A test road, i.e., Route 165 in Exeter was rehabilitated and used to predict and evaluate the performance of different strategies. Four test sections used the full depth eight-inch FDR base/subcase, and three of them were stabilized with calcium chloride, asphalt emulsion, and Portland cement. The fifth test section was reconstructed with geo-grid and six inches of filter stone sandwiched between the layers. The control section was reclaimed in a similar method as the rest of the reclaimed test sections and no additives was used. All four test and one control section were paved with two and a half inch thick Class 19 HMA base and two inches Class 12.5 HMA surface. As previously mentioned, Route 165 is approximately seven miles long consisting of seven hills and valleys. The reclaimed test sections were given at least one hill and valley. The geo-grid section has only a small section for this research project and each test section has a different segment length and area of construction as shown in Table 2.3.

Based on the RIDOT Job Specifications, each of the reclaimed test sections and the geo-grid section were designed to conform to the same material gradation with 95% to 100% passing a three inch sieve and 2% to 15% passing a number 200 sieve to achieve a comparable performance between the test sections. The contractor had to comply with not having any stone, rock, cobble, or asphalt material being more than four inches in width or length. Cross sections of each test section are shown in Figures 2.3 through 2.7.

Equipment used for rehabilitation consisted of Reclaimer, vibratory sheep foot rollers and motorized graders. Compaction was done in accordance with AASHTO T180, Method D to obtain a uniform density of no less than 95% of maximum. It was also make sure that pavement operations should took place only during acceptable temperature ranges.

2.7.1. Full Depth Reclamation with Calcium Chloride:

The first Full depth reclamation (FDR) consisted of using a calcium chloride ($CaCl_2$) solution. This procedure used AASHTO M 144 specifications for calcium chloride with a solution being at 35% +/- 1%, alkali chloride 2% maximum as NaCl, and magnesium at 0.1% maximum as MgCl. From the RIDOT's Specification 406.9901: A calcium pressure distributer was used to distribute the $CaCl_2$ solution at a rate of 0.1 to 2 gallons per square yard with a spray bar length of up to 20 feet. The distributor shall be equipped with a digital volumetric accumulator meter capable of measuring gallons applied and distance traveled. The volume and measuring device shall be equipped with a power unit for the pump so that the application is by pressure, not gravity. The spray nozzles and pressure system shall provide a sufficient and uniform fan–shaped spray of material throughout the entire length of the spray bar at all times while operating, and shall be adjustable laterally and vertically. The spray shall completely cover the roadway surface receiving the treatment (RIDOT SPC 406.9901).

2.7.2. Full Depth Reclamation with Portland cement

Based on RIDOT's Specification 406.9904, Portland cement was spread by distributing a measured amount of cement in front of the reclaimer. The spreader uniformly blended cement and existing materials to the specified percentage +/- three pounds /square yard (across the roadway. The Contractor was required to provide a method for verifying that the correct amount of cement was being applied. Additionally, the cement spreader was equipped with a tractor-trailer utilizing “a Drop behind system” which was pressure controlled. Each day the operator would calibrate the drop to make sure the correct application was being applied. The trailer was filled four to five times daily with bulk delivery trucks. Three pounds per square yard comes out to be four percent Portland cement mix.

2.7.3. Full Depth Reclamation with Asphalt Emulsion

Full depth reclamation with bituminous stabilizer consisted of using an asphalt emulsion of grade MS-2 or HFMS-2. This procedure used AASHTO M.03.03.4 144 specifications for asphalt emulsion. From the RIDOT’s Specification 406.9903: The asphalt emulsion distributor shall be capable of applying asphalt emulsion in measured quantities at any rate from 0.1 to 1.5 gallons per square yard of roadway surface, at any length of spray bar up to 12 feet. It shall be capable of maintaining the application rate to a tolerance of ± 0.03 gals/yd^2 regardless of change in grade, width or direction of the road. It shall be equipped with a thermometer for the emulsion and a digital volumetric accumulator meter capable of measuring gallons applied and distance traveled. The volume and measuring device shall be equipped with a power unit for the pump so that application is by pressure, not gravity. The spray nozzles and pressure system shall provide a sufficient and uniform fan-shaped spray of material throughout the entire length of the spray bar at all times while operating, and shall be adjustable both laterally and vertically. The spray shall completely and uniformly cover the roadway surface receiving the treatment (RIDOT SPC 406.9901).

2.7.4. Full Depth Reclamation with Geo-grid

A section of geo-grid mechanically stabilized layer was placed as another test section for a comparison. Distributors of the Tensar International Corporation Technologies were highly interested in demonstrating their product and made claims to its durability and strength. RIDOT decided to use geo-grid along with the reclaimed sections to have a complete test road. The Tensar product was used in an area of the road that has a high seasonal water table.

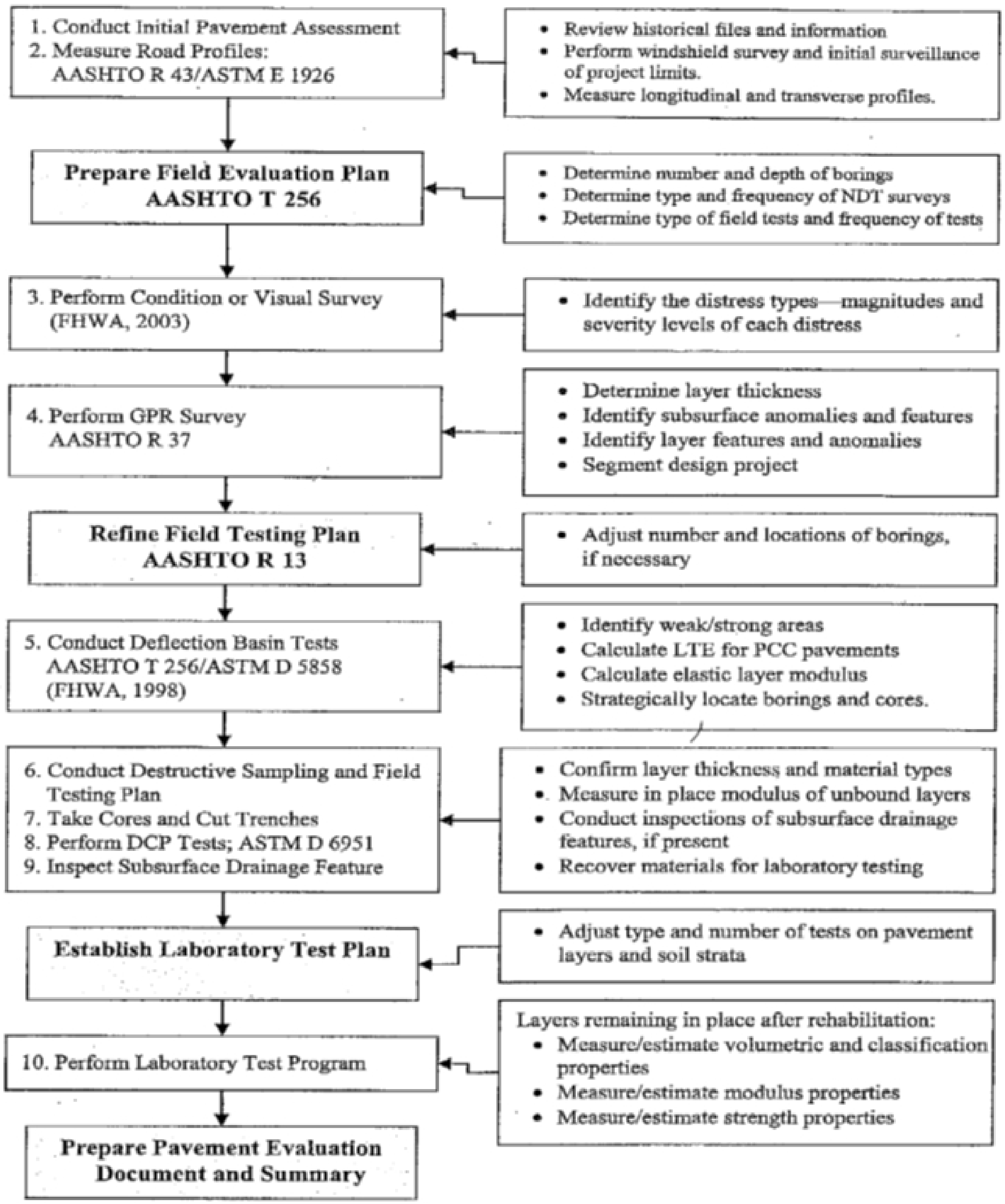


Figure 2.1: Steps and Activities for Assessing Condition of Existing Pavements for Rehabilitation Design

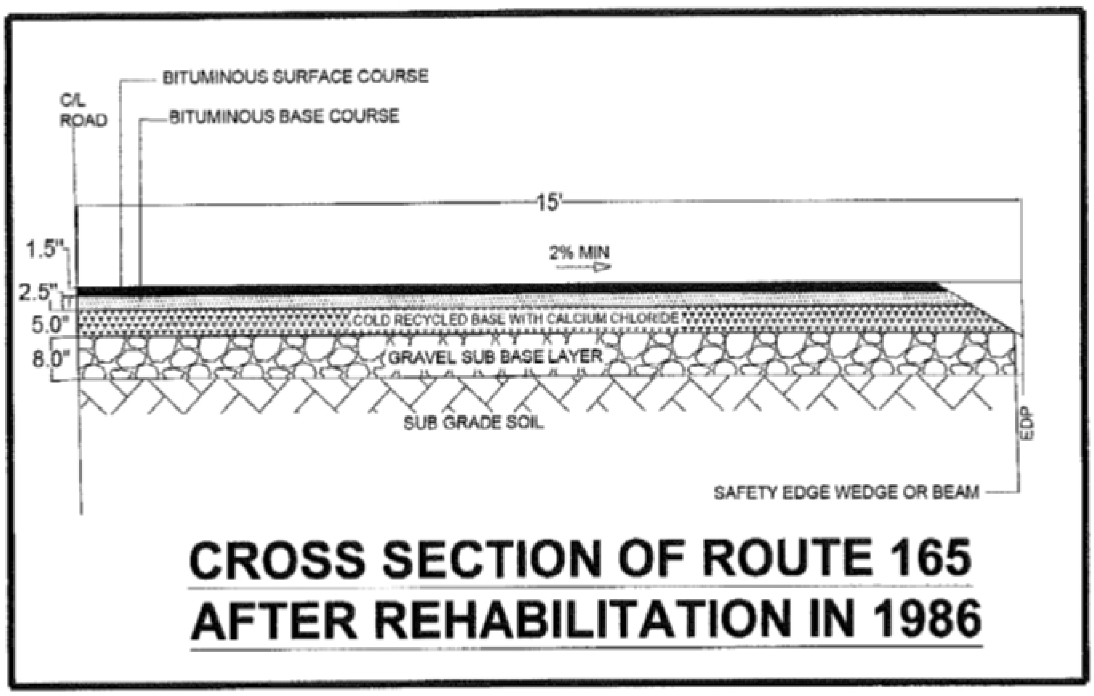


Figure 2.2: Cross section of Route 165 after Rehabilitation in 1986

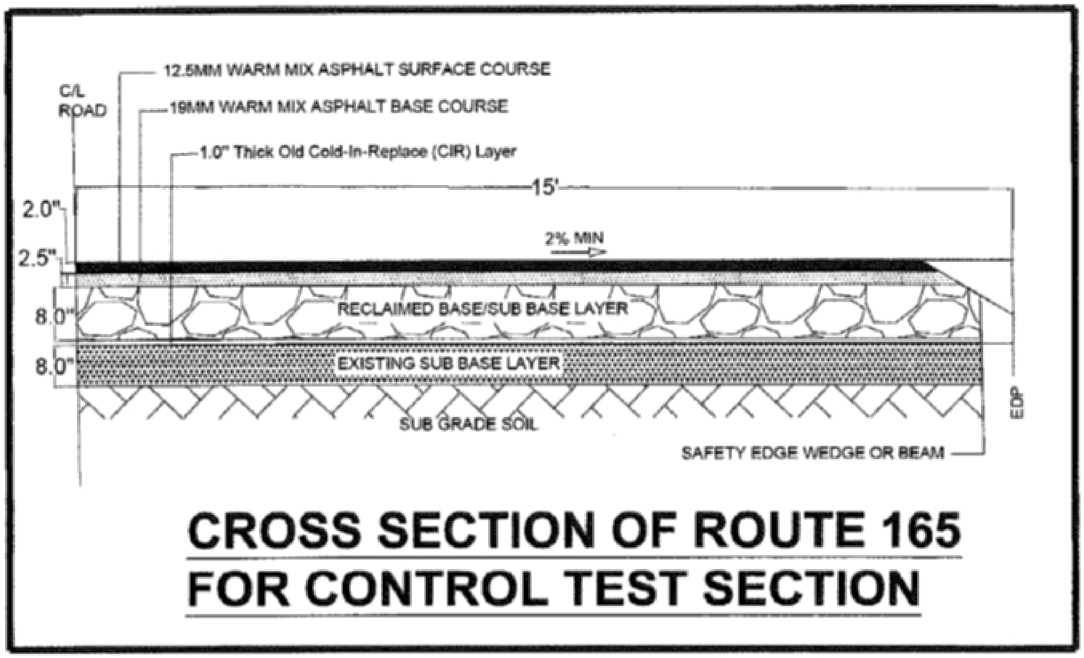


Figure 2.3: Cross section of Route 165 with Control Test Section

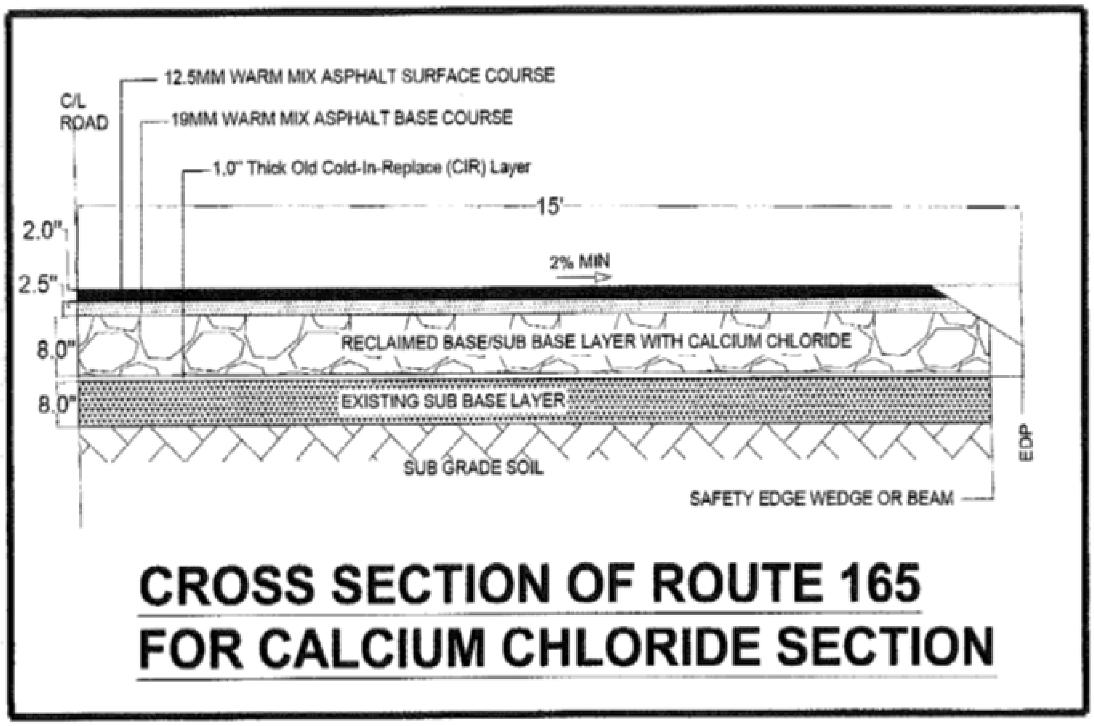


Figure 2.4: Cross section of Route 165 with Calcium Chloride Test Section

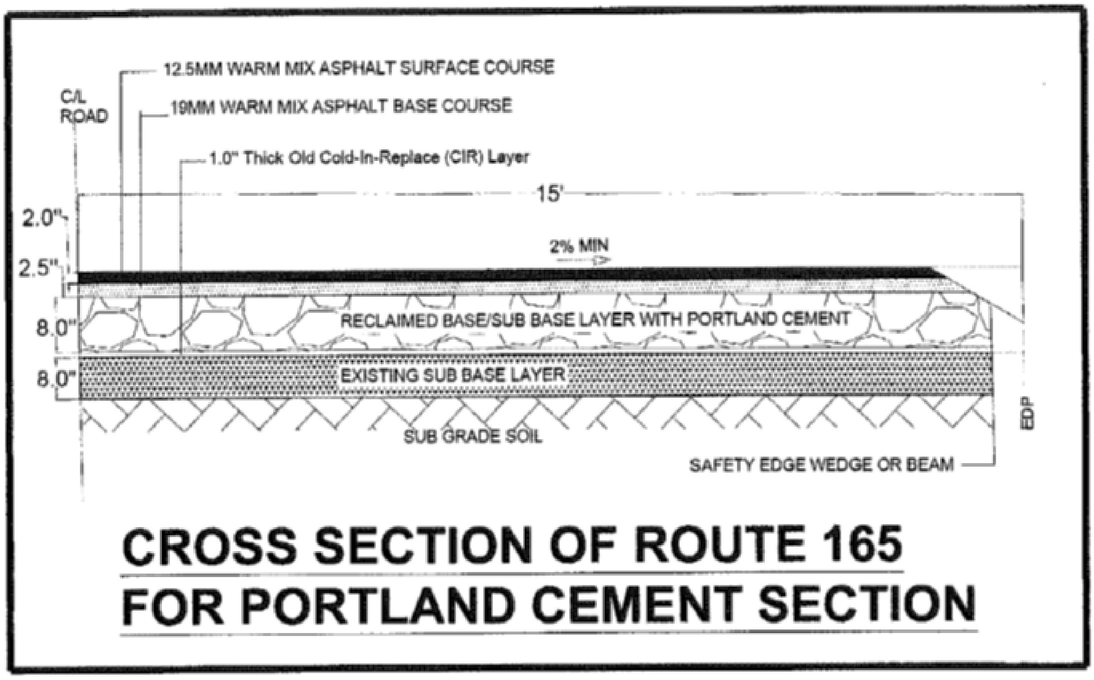


Figure 2.5: Cross Section of Route 165 with Portland cement Test Section

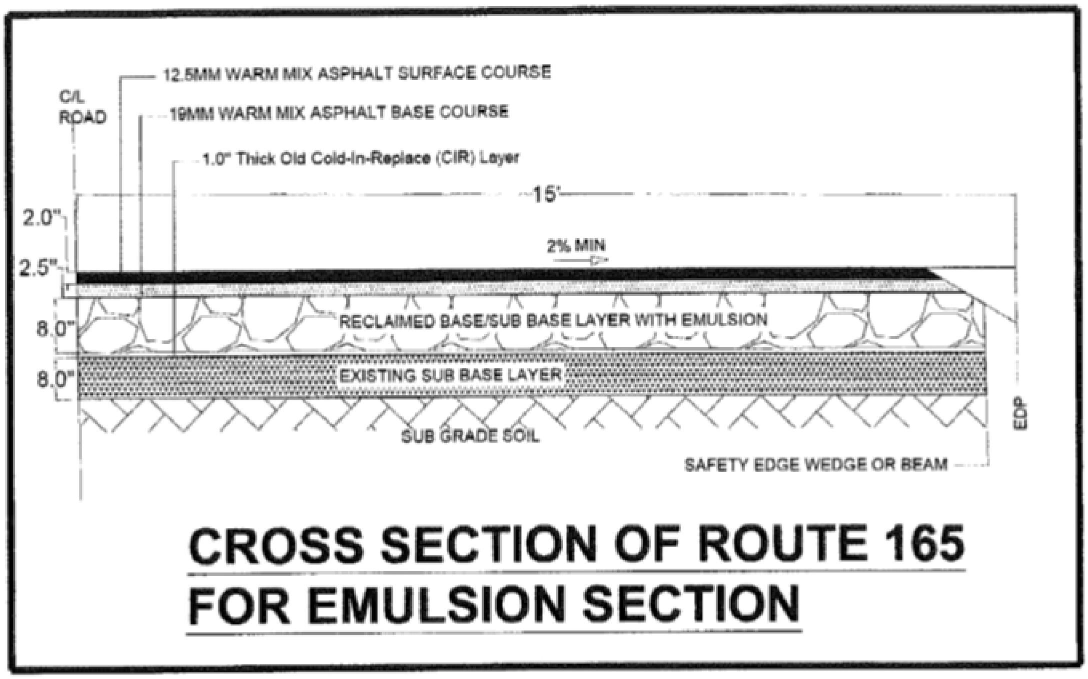


Figure 2.6: Cross Section of Route 165 with Asphalt Emulsion Test Section

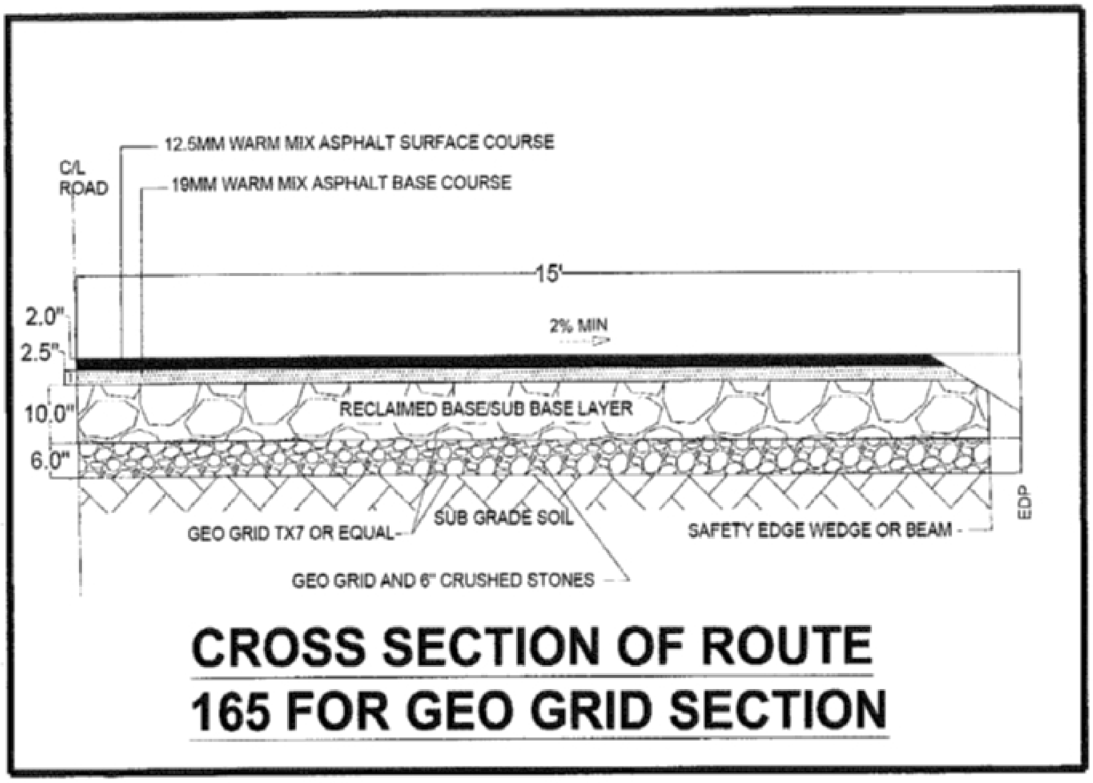


Figure 2.7: Cross Section of Route 165 with Geo-grid Test Section

Table 2.1: First Field Survey of Route 165 in 1981

		Flooding Frequency	High Water Table FT	Water Table Months	Potential Frost Action	Depth In	Permeability	Permeability Shrink-Swell Potential	AASHTO Classification	Liquid Limit	Plasticity Index	USDA Texture	Local Roads and Streets
Adrian	Aa	Frequent	0-1.0	Nov-May	High	20-60	6.0-20	Low	A-2,A-3, A-1		NP*	Sand, Fine, Loamy	Severe: wetness, floods, low strength
Bridgehampton	BnB*,BoC*	None	>6		High	8-41	.6-2.0	Low	A-2		NP	Gravelly sandy Loam, sandy Loam	Severe: Frost Action
Canton	CdA*,CdB*,CeC*,ChB*,ChC*	None	>6		Low	22-60	6.0-20	Low	A-2, A-4	0-<10	NP	Fine Gravelly Loam, Sandy Loam	Moderate: Slope
Carlisle	Co	Frequent	0-1.0	Sep-June	High	0-55	.2-6.0		PT			Sapric Material	Moderate: Low strength, wetness, floods
Enfield	EfB	None	>6		Moderate	25-60	>20	Low	A-1		NP	Very Gravelly sand, Gravelly Sand	Moderate: Frost action
Hinckley	HkA,HkC	None	>6		Low	17-60	>20	Low	A-1	<10	NP	Stratified Gravelly Loamy, Fine Sand to very Cobbly Course Sand	Slight, Moderate: Slope
Merrimac	MmA,MmB	None	>6		Low	25-60	6.0-20	Low	A-1		NP	Stratified Sand to very Gravelly Sand	Slight
Ridgebury	Rf	None	0-1.5	Nov-May	High	20-60	<.2	Low	A-2, A-4		NP	Sandy Loam, Gravelly Loam	Severe: Large stones, wetness
Scarboro	Sb	Rare	0-1.0	Nov-Jul	High	19-60	>6	Low	A-1, A-2		NP	Loamy Sand, Sand	Severe: Wetness
Sutton	StA, StB,SuB	None	1.5-3.5	Nov-Apr	Moderate	17-60	.6-6.0	Low	A-2, A-4		NP	Fine Sandy Loam, Gravelly Sandy Loam, Sandy Loam	Moderate: Frost action
Walpole	Wa	None	0-1.0	Nov-Apr	High	19-60	>6	Low	A-1, A-2, A-3		NP	Fine Sandy Loam, Sandy Loam, Gravelly Sandy Loam	Severe: Wetness, frost action

(Soil Survey of Rhode Island, United States Department of Agriculture, Soil Conservation Service in Cooperation with RI Agricultural Experiment Station, 1981)

Table 2.2: Summary of Physical Properties of Selected Subgrade Soils from Route 165 Site

Location of Sample Number	Max Dry Density kN/m^3	Max Dry Density (lbs/ft3)	Optimum Water Content (%)	Specimen Dry Density kN/m^3	Specimen Dry Density (lbs/ft^3)	Specimen Water Content (%)	Specimen Relative Water Content (%)
7 Control	21.1	134.3	5.2	19.8	126.1	5.3	94.0
11a Control	21.1	134.3	3.9	20	127.3	3.6	95.0
11b Control	21.1	134.3	3.9	20	127.3	3.7	95.0
12a $CaCl_2$	21.5	136.9	4.0	20.2	128.6	4.0	94.0
12b $CaCl_2$	21.5	136.9	4.0	20.2	128.6	3.5	94.0
2 Portland Cement	21.6	137.5	3.8	20.4	129.9	4.0	94.0

Table 2.3: Summary of Physical Properties of Selected 1980's RAP Blends from Route 165 Site

Location of Sample Number	Rap Content (%)	Max Dry Density kN/m^3	Max Dry Density (lbs/ft3)	Optimum Water Content (%)	Specimen Dry Density kN/m^3	Specimen Dry Density (lbs/ft^3)	Specimen Water Content (%)	Specimen Relative Water Content (%)
11 Control	20	21.3	135.6	4.7	20.6	131.1	3.3	97.0
12 $CaCl_2$	40	21	133.7	4.4	19.9	126.7	4.6	95.0
9 Asphalt Emulsion	26	20.9	133.1	5.4	20	127.3	5.0	96.0
4a Portland Cement	25	21	133.7	3.6	19	121.0	3.5	91.0
4b Portland Cement	25	21	133.7	3.6	20	127.3	3.4	95.0
6 Portland Cement	15	21	133.7	3.7	20.4	129.9	3.5	97.0
8a Geo-Grid	25	21.5	136.9	3.2	20.1	128.0	2.5	93.0
8b Geo-Grid	25	21.5	136.9	3.2	20.6	131.1	3.3	96.0
8c Geo-Grid	25	21.5	136.9	3.2	19.6	124.8	8.5	91.0
8d Geo-Grid	25	21.5	136.9	3.2	20.5	130.5	3.4	95.0

Table 2.4: Summary of Physical Properties of Selected FDR RAP/Virgin Aggregate Blends from Route 165 Site

Location of Sample Number	Max Dry Density kN/m^3	Max Dry Density (lbs/ft3)	Optimum Water Content (%)	Specimen Dry Density kN/m^3	Specimen Dry Density (lbs/ft^3)	Specimen Water Content (%)	Specimen Relative Water Content (%)
7A Control	21.2	135.0	3.9	20.4	129.9	3.6	96.0
7b Control	21.2	135.0	3.9	20.4	129.9	3.6	96.0
1 $CaCl_2$ (24 hours)	21.2	135.0	3.4	20.6	131.1	3.4 (3.3)	97.0
5 Asphalt Emulsion (5 Days)	20.5	130.5	4.0	19.8	126.1	4 (3.7)	97.0
4a Portland Cement (4 Hours)	20.7	131.8	3.4	20.0	127.3	3.3 (2.3)	97.0
4b Portland Cement (7 Days)	20.7	131.8	3.4	20.0	127.3	3.4 (.77)	97.0

Table 2.5: Location description of all Test sections of RI Route 165

Test Sections	Stationing	Area (Sq yd.)	Elevation (ft)	Locations	Utility Pole Locations	Length Segment (ft)
Control Section	48+50 to 75+00	9,000	303	67+25	369	2,650
Calcium Choride	0+00 to BK & 44+82 to 48+50	31,000	396	39+25	304	9,332
Portland cement	75+00 to 232+00	52,335	144	117+88	400	15,700
Asphalt emulsion	267+00 to 333+00	22,000	397	282+00	518	6,600
Geo-Grid	232+00 to 267+00	12,500	367	258+60	506	3,500

Chapter 3

Input Parameters for PavementME to Predict Performance

3.1. Background History of Pavement Design

AASHO road test conducted in Ottawa IL (1958-1960) was the initiative of modern mechanistic empirical approach. The information obtained from the AASHO Road Test was crucial in advancing knowledge of pavement structural design, pavement performance, load equivalencies, climate effects, and much more. The basic performance information resulted in the performance equations and nomographs used in the 1993 AASHTO Guide. 1993 AASHTO Guide has several old versions like 1961 interim guide and 1986 guide which includes material characterization. 1993 AASHTO guide is purely based on empirical performance equations and nomographs used to calculate the structural number of different layers whereas 2002 design guide is extensive and comprehensive which includes analysis and design of new, reconstructed and rehabilitated asphalt and concrete pavements, evaluates existing pavements, sub drainage design, recommendations for rehabilitation treatments and foundation improvements, recommendations for low volume road design, and life cycle cost analysis. AASHTOWare Mechanistic Empirical Pavement Design Guide (MEPDG) is purely based on 2002 design guide and used in this research study to evaluate and predict the performance of RI Route 165.

3.2. Introduction to AASHTOWare Pavement ME Design

The overall objective of AASHTOWare Pavement ME Design (PavementME) is to provide the highway community with a state of the practice tool for the design and analysis of new and rehabilitated pavement structures, based on mechanistic-empirical principles. This means that the design and analysis procedure calculates pavement responses (stresses, strains, and deflections) and uses those responses to compute incremental damage over time. This ME based procedure is shown in flowchart form in Figure 3.1. When analyzing a pavement design project using PavementME, whether new construction, overlay, or restoration, an iterative process that follows three basic steps is utilized:

1. Create a trial design for the project.
2. Run the PavementME to predict the key distresses and smoothness for the trial design.
3. Review the predicted performance of the triial design against performance criteria and modify trail design as needed to produce a feasible design that satisfies the performance criteria.

3.3. Significance and Use of PavementME

The PavementME provides a uniform and comprehensive set of procedures for the analysis and design of new and rehabilitated flexible and rigid pavements. PavementME design employs common design parameters for traffic, materials, subgrade, climate, and reliability for all pavement types, and is used to develop alternative designs using a variety of materials and construction procedures. The inputs generally used for PavementME are listed as bellow.

1. General Project Information
2. Design Criteria and Reliability level
3. Truck Traffic data
4. Climate data
5. Material Properties

The general approach for determining inputs for materials in PavementME is a hierarchical (level) system which includes following 3 levels.

1. Level 1 input involves comprehensive laboratory tests
2. Level 2 inputs are estimated through co-relations with other material properties that are commonly measured in laboratory or field.
3. Level 3 requires the designer to estimate the most appropriate design value of the material property based on experience with little or no testing.

In this research study level 1 input values were used which are described in later sections. The output from the PavementME at selected reliability level includes:

1. Permanent deflection-Total pavement (in.)
2. Asphalt Concrete bottom-up fatigue cracking (percent)
3. Asphalt Concrete thermal fracture (ft. /mi)
4. Asphalt Concrete top-down fatigue cracking (ft. /mi)
5. Permanent deformation Asphalt Concrete only (in.)
6. International Roughness Index (IRI) or smoothness

3.4. Input data of RI Route 165 for PavementME

3.4.1. General Project Information

General project information includes input values like design/analysis life and construction and traffic opening dates. Usually design life of a new or rehabilitated pavement is the time from initial construction until the pavement has structurally deteriorated to a specified pavement condition or the time when significant rehabilitation or reconstruction is needed. Design life for RI Route 165 was selected as 20 years which is shown in PavementME reports (Appendix). Construction and traffic opening dates for RI Route 165 was collected from RIDOT and shown in PavementME reports (Appendix).

3.4.2. Design Criteria and Reliability Level

Design performance and design reliability greatly affect deterioration of an adequately performing pavement. Performance criteria are used to ensure that a pavement design will perform satisfactorily over its design life. The designer select performance threshold distress values to judge the adequacy of a trail design. Designer also specifies the desired level of reliability for each distress type and smoothness. The level of design reliability could be based on the general consequences of reaching the terminal condition earlier than design life. For RI Route 165 design criteria and reliability level is shown in Table 3.1.

3.4.3. Truck Traffic Data

Truck traffic is a key data element for the structural design/analysis of pavement structures.

PavementME uses the full axle-load spectrum data for each axle type for both new pavement and rehabilitation design periods. Traffic volume, lane distribution, volume adjustment factors (i.e., class distribution, traffic growth factors, etc.) and weight data are used as inputs along with some miscellaneous data such as tire pressure. The axle-load spectra are obtained from processing weighing-in-motion (WIM) data which is a device usually embedded into a pavement used to calculate traffic flow. For RI Route 165 all truck traffic data from period of December 2014- November 2015 was collected from RIDOT Traffic section which includes data of average annual daily traffic (AADT) which further broken down into vehicle classification, monthly adjustment factors, hourly adjustment factors, daily vehicle counts and percent trucks in design direction. Average Annual Truck Traffic (AADTT) is calculated for Class 4 to Class 13 of FHWA vehicle classification chart which is shown in Figure 3.2. The AADTT from December, 2014 to November, 2015 is 150. The percent trucks in design direction was calculated at 51/49. Heavy trucks is cumulatively calculated as 295,762 truck vehicles over ten years and 627,848 truck vehicles in the highest design direction. These values are calculated by calculating the number of trucks per year and adding each year together for ten or twenty years with a 1.3% increase in truck traffic and multiplying that number by .51(the design direction). AADT data obtained from RIDOT for RI Route 165 is shown in Table 3.2.

3.4.4. Climate Data

Detailed climate data are required for predicting pavement distress with PavementME and include hourly temperature, precipitation, wind speed, relative humidity, and cloud cover. These data are used to predict the temperature and moisture content in each of the pavement layers as well as provide some of the inputs to the site factor parameter for the smoothness prediction models.

All of the climate data needed by PavementME are available from weather stations, generally located at airfields around the United States. PavementME has an extensive number of weather stations embedded in its software for ease of use and implementation. The longitude, latitude, elevation and number of months of available data are viewed by the user in selecting the weather stations to be used by the software to create a virtual weather station at the project location for the distress prediction.

For RI Route 165 project nearest weather station available in software i.e. Providence RI is selected which has latitude of 2.361 and longitude of -71.011 as shown in the PavementME reports (Appendix G).

3.4.5. Material Properties

The PavementME software requires that all material properties entered into the program for new layers represent the values that exist right after construction. The general approach for determining design inputs for materials in PavementME is a hierarchical (level) system as described earlier. In its simplest and most practical form, the hierarchical approach is based on the philosophy that the level of engineering effort exerted in the pavement design process for characterizing the paving materials and foundation should be consistent with the relative importance, size, and cost of the design project. For RI Route 165 project most of the input values used are Level 1, i.e., values obtained from comprehensive laboratory tests.

Fundamental properties are required for all HMA mixture types or layers to execute PavementME. Table 3.3 lists the HMA material properties that are required for the material types listed in Table 3.4, as well as identify the recommended test protocols and other sources for estimating these properties.

The input properties for all HMA material types are grouped into volumetric and engineering properties. The volumetric properties (Superpave Mix Design) include

1. Air Voids
2. Effective Asphalt Content by volume
3. Aggregate gradation
4. Mix density
5. Asphalt grade

For RI Route 165 project all volumetric properties were determined at RIDOT material section and discussed in later section. The engineering or mechanistic properties (discussed in later section) for HMA materials include

1. Resilient Modulus
2. Dynamic Modulus
3. Creep Compliance
4. Indirect Tensile Strength

3.4.5.1. Volumetric Properties

A. Super-Pave Mix Design

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials. The final product of the SHRP asphalt research program is a new system called Superpave, short for Superior Performing Asphalt Pavements. Superpave represents an improved system for specifying asphalt binders and mineral aggregates, developing asphalt mixture design, and analyzing and establishing pavement performance prediction. The system includes an asphalt binder specifications, HMA design and analysis system, and computer software that integrates the system components. The Superpave binder specification and mix design system include various test equipment, test methods, and criteria. A detailed Superpave mix design procedure and results for RI HMA is described in Appendix C.

The unique feature of the Superpave system is that it is a performance based specification system. The tests and analyses have direct relationship to field performance. The Superpave asphalt binder tests measure physical properties that can be directly related to field performance by engineering principles. The data obtained from RIDOT material section for RI Route 165 is shown in Table 3.5 and used in PavementME as an input.

B. Asphalt Grading

Asphalt binder grading system was developed in the mid-1990s and was purely based on performance grading. Performance graded (PG) binders are defined by a term such as PG 64-28. The first number 64, is the high temperature grade. This means that the binder possesses adequate physical properties up to at least 64^0C. This would correspond with the high pavement temperature in the climate in which the binder is expected to serve. Likewise, the second number (-22) is the "Low temperature grade" and means that the binder possesses down to at least -22^0C. Table 3.6 lists the new binder test equipment and purpose, however detailed procedure and lab reports for all those test with RI HMA material are described in Appendix B1-B6 (Asphalt Binder properties).

3.4.5.2.Engineering or Mechanistic Properties

A. Resilient Modulus

Resilient Modulus (M_r) is a fundamental material property used to characterize unbound pavement materials. It is a measure of material stiffness and provides a mean to analyze stiffness of materials under different conditions, such as moisture, density, and stress level. A detailed procedure of determining resilient modulus of RI subbase soil is described in Appendix A.

RI Route 165 as discussed in Chapter 2 consists of three unbound layers namely Subgrade soil, Existing Subbase soil, and new FDR Base/Subbase layer. Resilient modulus of all layers tested at University of Rhode Island laboratory in 2013 and discussed in further subheadings.

A.1. Resilient Modulus of Subgrade Soils

As discussed in Chapter 2, subbase and subgrade soil samples were collected during construction for testing. According to the results of a sieve analysis on the material, the subbase consisted of gravelly sand or A-1-b AASHTO classification which is consistent with the material shown for that location in the 1981 Soil Survey of Rhode Island; this soil was also found under a previous URI study (Lee et al. 2003). The URI study reported resilient modulus (Mr), which is deviator stress over recoverable strain, values for Rhode Island subgrade soils ranged from 7,506 psi to 9,304 psi (Lee et al. 2003) and an Idaho study for comparison shows the same types of gravel material ranged from 8,000 psi to 19,000 psi (Hardcastle 1993).

A.2. Resilient Modulus of Existing Subbase Materials

Before the Full Depth Reclamation (FDR), 100 mm (4 in.) of asphalt pavement were removed from the roadbed for ten test sections located throughout the length of the road. Approximately twelve inches of existing subbase layer including 125 mm (5 in.) of previously recycled material were collected. It should be noted, the collected samples were mix with seven inches of the existing gravel borrow and the five inches of previously reclaimed material was not tested separately. Resilient moduli of the ten subbase test sections were determined by using triaxial chamber apparatus according to AASHTO T 307-99 procedure. Resilient moduli values are presented in Table 3.7. The laboratory resilient moduli values varied from 17, 000 psi to 74,000 psi.

A.3. Resilient Modulus of New FDR Base/Subbase Materials

In construction, four inches of old asphalt surface and base layers were reclaimed into four inches of previously reclaimed subbase, and a new eight-inch homogeneous FDR base/subbase layer was formed. Samples were taken, before the new construction FDR base/subbase layer were mixed with the three different strategies, to URI for testing. Before triaxial testing, four samples were mixed with additives in the lab according to RIDOT specifications for Route 165. Out of the six samples two control FDR samples were tested without additives, one sample was mixed with $CaCl_2$, one sample was mixed with asphalt emulsion, and two samples were mixed with Portland cement. For the Portland cement samples, one was cured for 4 hours and the other 7 days before testing. The resilient moduli of FDR base/subbase layer were determined by using AASHTO T307-99, and results are shown in Table 3.8.

B. Dynamic Modulus of HMA

The dynamic modulus represents the stiffness of the asphalt material when tested in a compressive-type, repeated load test. The dynamic modulus is one of the key parameters used to evaluate both rutting and fatigue cracking distress predictions in the MEPDG. Dynamic modulus values measured over a range of temperatures and frequencies of loading can be shifted into a master curve for characterizing asphalt concrete for pavement thickness design and performance analysis. The values of dynamic modulus and phase angle can also be used as performance criteria for HMA design.

For Route 165 dynamic modulus test was conducted by University of Rhode Island and the test results are attached in Appendix E.

C. Creep Compliance and Indirect Tensile Strength (IDT) of HMA

Creep compliance is usually defined as the time dependent strain divided by applied stress. The values of creep compliance, tensile strength and Poisson's ratio determined with AASHTO T 322-03 method can be used in linear viscoelastic analysis to calculate the low temperature and fatigue cracking potential of asphalt concrete. A detailed procedure for calculating Creep Compliance and IDT is attached in Appendix D.

For Route 165 Creep compliance was acquired from a URI study, and used as an input parameter for the MED software (Lee et al., 2014). The creep compliance results are used according to the MEPDG for new pavement only. Those values are shown in Table 3.9.

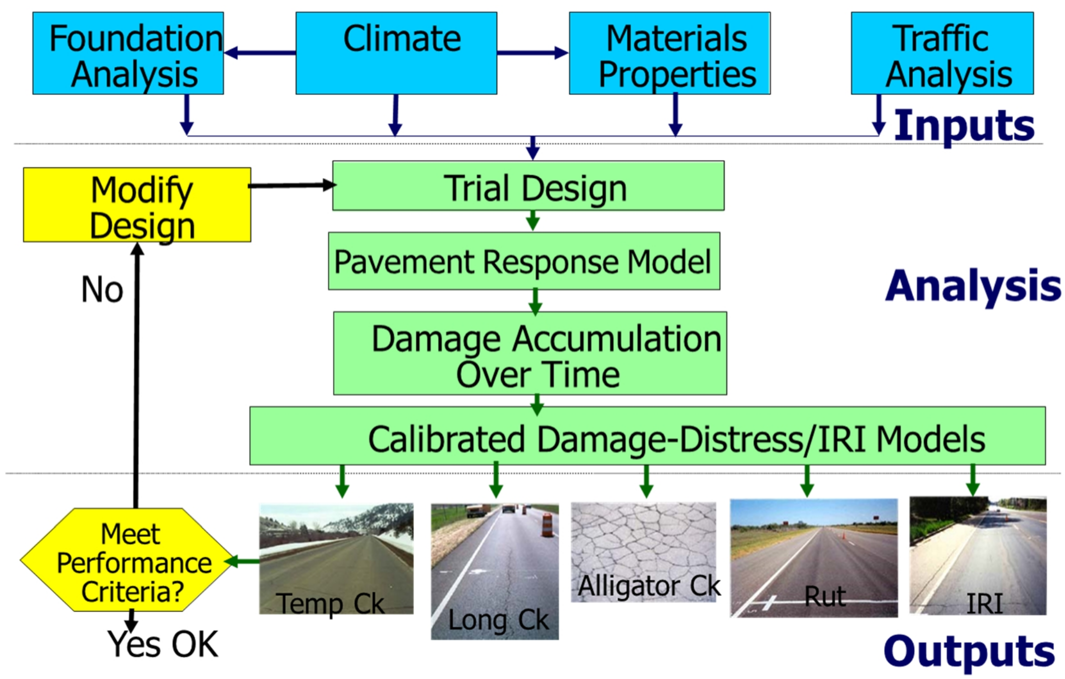


Figure 3.1 Flowchart of the analysis process of AASHTOWare Pavement ME Design

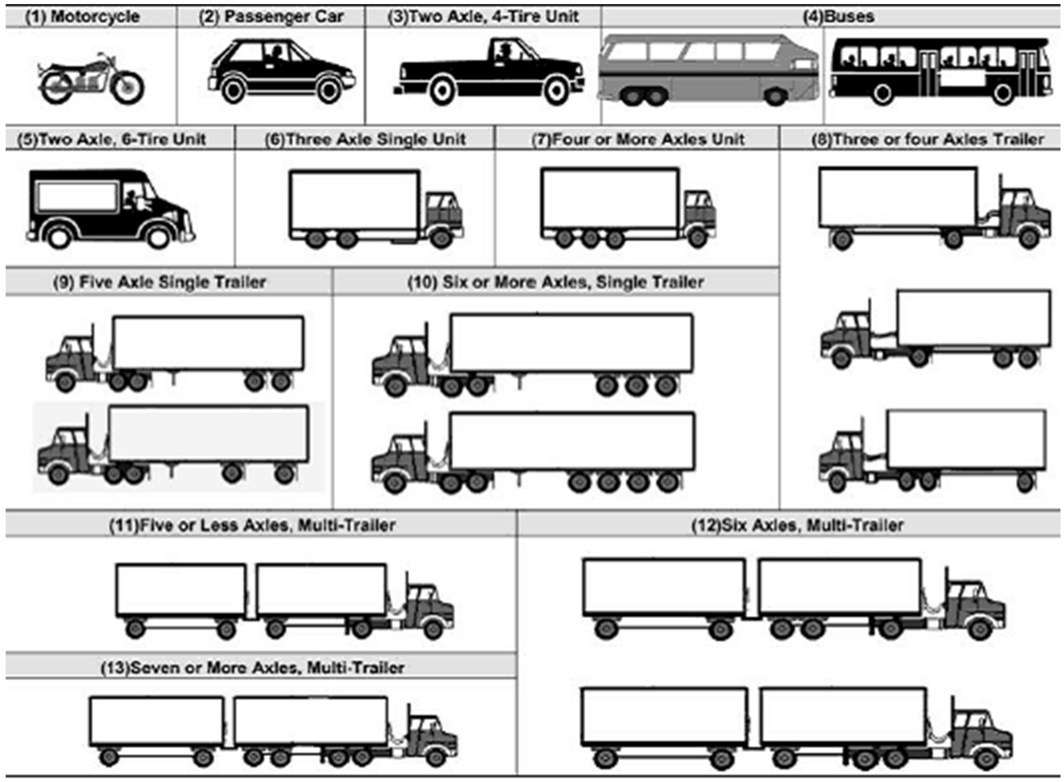


Figure 3.2 FHWA Vehicle Classification Chart

Table 3.1 Design Criteria or Threshold values and Reliability Level for RI Route 165

Distress Type	Performance Criteria	Reliability Level (%)
Permanent deflection-Total pavement (in.)	0.75	90
Asphalt Concrete bottom-up fatigue cracking (percent lane area)	25	90
Asphalt Concrete thermal fracture (ft. /mi)	1000	90
Asphalt Concrete top-down fatigue cracking (ft. /mi)	2000	90
Permanent deformation Asphalt Concrete only (in.)	0.25	90
Terminal IRI (in. /mi)	172	90

Table 3.2 Average Annual Daily Traffic of RI Route 165 from December 2014-November 2015

Vehicle Class	4	5	6	7	8	9	10	11	12	13	Total
14-Dec	28	1333	298	93	202	1625	13	1	12	0	3605
Jan	17	1168	298	84	139	1301	4	1	84	0	3096
Feb	19	1466	371	103	155	1495	2	2	38	0	3651
Mar	22	1389	229	22	245	1966	11	1	49	3	3937
Apr	38	1691	250	46	261	2136	13	0	30	1	4466
May	41	2116	335	140	261	2015	12	4	41	1	4966
June	64	2109	319	105	339	2180	25	5	65	1	5212
July	37	2432	387	106	407	2115	20	1	56	2	5563
Aug	43	2523	452	214	466	2064	16	6	45	1	5830
Sept	34	2408	285	63	353	2234	20	0	78	0	5475
Oct	24	1899	395	145	418	2144	7	2	64	0	5098
15-Nov	36	1296	296	119	281	1781	23	0	66	2	3900
Total	407	21835	3921	1247	3535	23065	176	34	640	24	54884
Vehicle Class Distribution	0.74%	39.81%	7.14%	2.27%	6.44%	42.05%	0.31%	0.05%	1.16%	0.03%	100%

Table 3.3: Asphalt Materials and test protocols for Measuring the Material property Inputs for New and Existing HMA layers.

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New HMA (new pavement and overlay mixtures), as built properties prior to opening to truck traffic	Dynamic modulus	X		AASHTO T 342
	Tensile strength	X		AASHTO T 322
	Creep Compliance	X		AASHTO T 322
	Poisson's ratio		X	National test protocol unavailable. Select AASHTOWare Pavement ME Design default relationship.
	Surface shortwave absorptivity		X	
	Thermal conductivity	X		ASTM E1952
	Heat capacity	X		ASTM D2766
	Coefficient of thermal contraction		X	National test protocol unavailable. Use AASHTOWare Pavement ME Design default values.
	Effective asphalt content by volume	X		AASHTO T 308
	Air voids	X		AASHTO T 166
	Aggregate specific gravity	X		AASHTO T 84 and T 85
	Gradation	X		AASHTO T 27
	Unit Weight	X		AASHTO T 166
	Voids filled with asphalt (VFA)	X		AASHTO T 209
Existing HMA mixtures, in-place properties at time of pavement evaluation	FWD backcalculated layer modulus	X		AASHTO T 256 and ASTM D5858
	Poisson's ratio		X	National test protocol unavailable. Use AASHTOWare Pavement ME Design default values.
	Unit Weight	X		AASHTO T 166 (cores)
	Asphalt content	X		AASHTO T 164 (cores)
	Gradation	X		AASHTO T 27 (cores or blocks)
	Air voids	X		AASHTO T 209 (cores)
	Asphalt recovery	X		AASHTO T 164/R 59/T 319 (cores)
Asphalt (new, overlay, and existing mixtures)	Asphalt Performance Grade (PG), OR	X		AASHTO T 315
	Asphalt binder complex shear modulus (G*) and phase angle (δ), OR	X		AASHTO T 49
	Penetration, OR	X		AASHTO T 53
	Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity, OR	X		AASHTO T 202 AASHTO T 201 AASHTO T 228
	Brookfield Viscosity	X		AASHTO T 316

Table 3.4: Major Material Types for AASHTOWare Pavement ME Design

Asphalt Materials	Non-Stabilized Granular Base/Subbase
• Stone Matrix Asphalt (SMA) • Hot Mix Asphalt (HMA) – Dense Graded – Open Graded Asphalt – Asphalt Stabilized Base Mixes – Sand Asphalt Mixtures • Cold Mix Asphalt – Central Plant Processed – Cold In-Place Recycling **PCC Materials** • Intact Slabs—PCC – High-Strength Mixes – Lean Concrete Mixes • Fractured Slabs – Crack/Seat – Break/Seat – Rubblized **Chemically Stabilized Materials** • Cement Stabilized Aggregate • Soil Cement • Lime Cement Fly Ash • Lime Fly Ash • Lime Stabilized Soils • Open-Graded Cement Stabilized Aggregate	• Granular Base/Subbase • Sandy Subbase • Cold Recycled Asphalt (used as aggregate) – RAP (includes millings) – Pulverized In-Place • Cold Recycled Asphalt Pavement; (HMA plus aggregate base/subbase) **Subgrade Soils** • Gravelly Soils (A-1; A-2) • Sandy Soils – Loose Sands (A-3) – Dense Sands (A-3) – Silty Sands (A-2-4; A-2-5) – Clayey Sands (A-2-6; A-2-7) • Silty Soils (A-4; A-5) • Clayey Soils, Low-Plasticity Clays (A-6) – Dry-Hard – Moist Stiff – Wet/Sat-Soft • Clayey Soils, High-Plasticity Clays (A-7) – Dry-Hard – Moist Stiff – Wet/Sat-Soft **Bedrock** • Solid, Massive, and Continuous • Highly Fractured, and Weathered

Table 3.5: Volumetric Properties of Base course material of RI Route 165

Items	Mix Design	Replicate A	Replicate B	Average	Remarks
1. Test Specimen ID		RI-DSN(Base)-R3	RI-DSN(Base)-R4		
2. Mix Design					
2.1 Compaction Level (%)	50 Gyrations				
2.2 Design Air Voids (%)	4				
2.3 Target Air Voids (%)	5				
2.4 Binder Type and Grade (PG)	PG 64-28				
2.5 Binder Content (%)	5.3				
2.6 Binder Specific Gravity (Gb)	1.033				
2.7 Aggregate Type	Dense				
2.8 Aggregate Bulk Sp. Gr. (Gsb)	2.743				
2.9 Design Rice Sp. Gr. (Gmm)	2.477				
3. Type of Aging		Short-term	Short-term		
4. Rice Sp. Gr. (Gmm)		2.529	2.529	2.529	
5. Average Height (mm)		151.5	150.8	151.2	147.5 ≤ h ≤ 152.5
6. Average Diameter (mm)		101.1	101.1	101.1	100 ≤ d ≤ 104
7. Standard Deviation of Diameter (mm)		0.15	0.08	0.11	σd ≤ 2.5
8. Bulk Sp. Gr. (Gmb)		2.399	2.402	2.401	
9. Air Voids (%)		5.1	5.0	5.1	
10. Asphalt Content (%)		5.3	5.3	5.3	
11. Effective Sp. Gr. of Agg. (Gse)		2.752	2.752	2.752	
12. Effective Binder Content by Volume (%)		12.0	12.1	12.0	
13. VMA (%)		17.2	17.1	17.1	
14. VFA (%)		70.1	70.6	70.3	
15. End Flatness					
15.1 Top (mm)		0.06	0.07	0.06	≤ 0.5
15.2 Bottom (mm)		0.05	0.06	0.06	≤ 0.5
16. Perpendicularity					
16.1 Top (mm)		0.13	0.45	0.29	≤ 1.0
16.2 Bottom (mm)		0.24	0.24	0.24	≤ 1.0

Table 3.6: Superpave Binder Test Equipment and its purpose

Equipment	Purpose
Rolling Thin Film Oven (RTFO) Pressure Aging Vessel	Simulate binder aging (hardening) characteristics
Dynamic Shear Rheometer (DSR)	Measure binder properties at high and intermediate temperatures
Rotational Viscometer (RV)	Measure binder properties at high temperatures
Bending Beam Rheometer (BBR) Direct Tension Tester (DTT)	Measure binder properties at low temperatures

Table 3.7: Resilient Moduli of Sampled 1980s RAP/Virgin Blends from Route 165 Site

Sample	Resilient Modulus Mpa (Ksi)														
4a (Cement)	124 (17.9)	141 (20.4)	154 (22.3)	170 (24.6)	191 (27.7)	201 (29.1)	251 (36.4)	272 (39.4)	268 (38.8)	270 (39.1)	285 (41.3)	317 (45.9)	324 (46.9)	338 (49.0)	369 (53.5)
4b (Cement)	123 (17.8)	141 (20.4)	161 (23.3)	178 (25.8)	207 (30.0)	225 (32.6)	266 (38.5)	305 (44.2)	316 (45.8)	290 (42.0)	318 (46.1)	368 (53.3)	355 (51.4)	379 (54.9)	424 (61.4)
6 (Cement)	120 (17.4)	132 (19.1)	145 (21.0)	163 (23.6)	182 (26.3)	193 (27.9)	244 (35.3)	269 (39.0)	275 (39.8)	281 (40.7)	293 (42.4)	325 (47.1)	331 (48.0)	344 (49.8)	373 (54.0)
8a (Geogrid)	121 (17.6)	137 (19.8)	153 (22.1)	176 (25.5)	198 (28.7)	212 (30.7)	270 (39.1)	301 (43.6)	313 (45.3)	317 (45.9)	336 (48.7)	381 (55.2)	387 (56.1)	407 (59.0)	452 (65.5)
8b (Geogrid)	139 (20.1)	154 (22.3)	168 (24.3)	188 (27.2)	209 (30.3)	221 (32.0)	272 (39.4)	299 (43.3)	303 (43.9)	295 (42.7)	316 (45.8)	352 (51.0)	350 (50.7)	368 (53.3)	404 (58.5)
8c (Geogrid)	148 (21.4)	168 (24.3)	189 (27.4)	214 (31.0)	243 (35.2)	265 (38.4)	342 (49.6)	388 (56.2)	405 (58.7)	362 (52.5)	393 (56.9)	458 (66.4)	414 (60.0)	443 (64.2)	513 (74.4)
8d (Geogrid)	158 (22.9)	175 (25.3)	194 (28.1)	226 (32.7)	252 (36.5)	267 (38.7)	335 (48.5)	368 (53.3)	374 (54.2)	373 (54.0)	392 (56.8)	440 (63.8)	433 (62.8)	454 (65.8)	502 (72.8)
9 (Emulsion)	123 (17.8)	135 (19.5)	149 (21.6)	167 (24.2)	184 (26.6)	196 (28.4)	242 (35.0)	263 (38.1)	269 (39.0)	272 (39.4)	284 (41.1)	312 (45.2)	318 (46.1)	329 (47.7)	354 (51.3)
11 (Control)	148 (21.4)	164 (23.7)	181 (26.2)	157 (22.7)	229 (33.2)	248 (35.9)	296 (42.9)	328 (47.5)	343 (49.7)	326 (47.2)	347 (50.3)	390 (56.5)	381 (55.2)	402 (58.3)	440 (63.8)
12 (CaCl)	147 (21.3)	162 (23.4)	177 (25.6)	196 (28.4)	220 (31.9)	236 (34.2)	300 (43.5)	329 (47.7)	337 (48.8)	340 (50.6)	360 (52.2)	396 (57.4)	406 (58.8)	424 (61.4)	455 (65.9)
σ_3 (kPa)	21	21	21	35	35	35	69	69	69	103	103	103	138	138	138
θ (kPa)	83	104	124	138	172	207	276	345	414	379	414	517	517	552	690
σ_d (kPa)	21	41	62	35	69	103	69	138	207	69	103	207	103	138	276
τ_{oct} (kPa)	10	20	29	16	32	49	32	65	97	32	49	97	49	65	130

Table 3.8: Resilient Moduli of selected FDR with or without additive of RI Route 165 Site

Sample	Resilient Modulus Mpa (Ksi)														
1 (CaCl)	252 (36.5)	236 (34.2)	247 (35.8)	291 (42.2)	305 (44.2)	313 (45.2)	408 (59.1)	419 (60.7)	412 (59.7)	462 (67.0)	472 (68.4)	496 (71.9)	541 (78.4)	555 (80.4)	568 (82.3)
4a (Cement)	386 (55.9)	390 (56.5)	407 (59.0)	457 (66.2)	489 (70.9)	507 (73.5)	613 (88.9)	627 (90.9)	585 (84.8)	633 (91.8)	659 (95.5)	684 (99.2)	727 (105.4)	749 (108.6)	738 (107.0)
4b (Cement)	528 (76.5)	646 (93.6)	718 (104.1)	743 (107.7)	874 (126.7)	1005 (145.7)	1102 (159.8)	1331 (193.0)	1541 (223.5)	1226 (177.8)	1376 (199.5)	1675 (242.9)	1456 (211.1)	1581 (229.3)	1898 (275.2)
5 (Emulsion)	178 (25.8)	177 (25.6)	187 (27.1)	208 (30.1)	218 (31.6)	224 (32.4)	285 (41.3)	295 (42.7)	291 (42.2)	321 (46.5)	328 (47.5)	343 (49.7)	352 (51.0)	364 (52.7)	371 (53.8)
7a (Control)	179 (25.9)	171 (24.8)	178 (25.8)	209 (30.3)	220 (31.9)	226 (32.7)	291 (42.2)	300 (43.5)	298 (43.2)	329 (47.7)	339 (49.1)	350 (50.7)	383 (55.5)	396 (57.4)	400 (58.0)
7b (Control)	240 (34.8)	232 (33.6)	244 (35.3)	285 (41.3)	303 (43.9)	315 (45.6)	411 (59.6)	430 (62.3)	428 (62.0)	456 (66.1)	475 (68.8)	505 (73.2)	541 (78.4)	561 (81.3)	578 (83.8)
σ_3 (kPa)	21	21	21	35	35	35	69	69	69	103	103	103	138	138	138
θ (kPa)	83	104	124	138	172	207	276	345	414	379	414	517	517	552	690
σ_d (kPa)	21	41	62	35	69	103	69	138	207	69	103	207	103	138	276
τ_{oct} (kPa)	10	20	29	16	32	49	32	65	97	32	49	97	49	65	130

Table 3.9: Creep Compliance of HMA base course material for RI Route 165 Site

Creep time t (s)	-20 °C HMA	-20°C CIR	-10 °C HMA	-10 °C CIR	0 °C HMA	0 °C CIR
0	8.97389E-08	9.41206E-07	4.13707E-07	8.54305E-07	5.21493E-07	2.55396E-06
1	9.63726E-08	9.69910E-07	4.57243E-07	9.22309E-07	6.20040E-07	2.73292E-06
2	1.02322E-07	1.00329E-06	4.91514E-07	9.63768E-07	7.00792E-07	2.88960E-06
5	1.18006E-07	1.07410E-06	5.55092E-07	1.04849E-06	8.78080E-07	3.13043E-06
10	1.36596E-07	1.15122E-06	6.26694E-07	1.13398E-06	1.08040E-06	3.39047E-06
20	1.66465E-07	1.25605E-06	7.29301E-07	1.24203E-06	1.35649E-06	3.73268E-06
50	1.99880E-07	1.46296E-06	9.41392E-07	1.44683E-06	1.94403E-06	4.35663E-06
100	2.45938E-07	1.70172E-06	1.19195E-06	1.67104E-06	2.56138E-06	5.00597E-06

(Lee et al., 2014)

Chapter 4

Performance Prediction of Rehabilitated Rt. 165 with PavementME Software

4.1. Outputs of PavementME Software for Rehabilitated RI Route 165

Output from PavementME Software generally consists of six types of distress types as follows:

1. Permanent Deformation-Total Pavement (in.)
2. AC bottom-up fatigue cracking (percent)
3. AC thermal fracture (ft./mil)
4. AC top-down fatigue cracking (ft./mil)
5. Permanent deformation-AC only (in.)
6. Terminal IRI (in./mil)

The above mentioned all distress types are compared with targeted value specified by standard and selected reliability level. If the predicted distresses and achieved reliability is within the target values it shows the criterion as pass otherwise fail. To achieve good results and longer life for pavement all distress type should pass the criterion.

Five sections of RI Route 165 were analyzed with PavementME by using all the above-mentioned inputs. Each section's performance prediction is discussed in further headings.

4.1.1. Prediction of Performance for Control Test Section (Cold Recycled)

The control test section on Route 165 was reclaimed to a depth of eight inches and did not receive any additives. After the FDR, as shown in Figure 2.3, 25 mm (I in.) of old 1980 recycled blend with the old $CaCl_2$ subbase material was assumed left over and is represented in the PavementME.

Table 4.1 shows the PavementME output from using the Mr from laboratory testing. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The report obtained from the PavementME of control section is attached in Appendix G.

4.1.2. Prediction of Performance for the Calcium Chloride Section

The calcium chloride section was full depth recycled and mixed with $CaCl_2$ to a depth of eight inches on Route 165. After the FDR, as shown in Figure 2.4, a one inch of old recycled blend with $CaCl_2$ is assumed left over and is represented in the PavementME.

Table 4.2 shows the PavementME output from using the Mr from laboratory tests. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The report obtained from the PavementME Calcium chloride's section is shown in Appendix G.

4.1.3. Prediction of Performance for Portland cement Section

The Portland cement section was full depth recycled to a depth of eight inches with the cement mixed throughout. A one inch of old recycled blend mixed with $CaCl_2$ is assumed left over and is represented in the PavementME, Figure 2.5.

There were two Portland cement samples tested for this project. Sample 4a was mixed with Portland cement (PC) and tested after four hours, while sample 4b was mixed with PC and tested after 7 days of curing. The Portland cement section on Route 165 was micro cracked after four hours and traffic was allowed on the newly compacted surface. Since micro cracking prevents the PC to gain any more stiffness, so, sample 4a is used as an input for Mr in this study.

The PavementME was run with the layer thicknesses and Mr as shown below in Table 4.3. There are no predicted output failures for cement's section. The report obtained from the PavementME Portland cement's section is shown in Appendix G.

4.1.4. Prediction of Performance for Asphalt Emulsion Section

The asphalt emulsion section was full depth recycled to a depth of eight inches with only the first three inches mixed with emulsion as shown in Figure 2.6.

The PavementME run with the layer thicknesses and Mr as shown below in Table 4.4. There is only one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The report obtained from the PavementME Asphalt emulsion's section is shown in Appendix G.

4.1.5. Prediction of Performance for Geo-Grid Section:

The Tensar geo-grid section used full depth recycled material to a depth of ten inches and did not receive any additives. To install the geo-grid, sixteen inches of subbase were removed from the road after FDR and stockpiled. The geo-grid was installed on top of the subgrade and six inches of filter stone were placed on the geo-grid. Another geo grid layer was placed over the filter stone and ten inches of FDR were placed and compacted as shown in Figure 2.7. For this test, the control material for sample 7b Mr mean values for confining stress of 35 kPa were used from Table 3.8.

The PavementME was run with the layer thicknesses and Mr as shown below in Table 4.5. There is only one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The report obtained from the PavementME Asphalt emulsion's section is shown in Appendix G.

4.2. Comparison of all Test Sections of RI Route 165

The comparison between the control and the other four test sections are shown in Table 4.6. The most prevalent distress, in the four test sections, is in the asphalt layer. AC top down fatigue cracking (longitudinal cracking) for the control, $CaCl_2$, asphalt emulsion and geo-grid predicted cracking will be greater than the estimated twenty years target value of 2,000 ft. /mile. The Portland cement section is the only test section that did not have any predicted distresses for twenty years. It was observed that the higher the AC top down cracking, the earlier the threshold distress is noted. Threshold distress is defined as the years to predicted distresses which is shown on the third page of PavementME report (Appendix G) of each section.

Having AC top down fatigue cracking (longitudinal cracking) also means the pavement layer of four and half inches in not thick enough for the actual truck traffic loading. Either the Class 12.5 HMA or the Class 19 HMA layer should have been thicker. Since the cost of the Class 19 HMA was thirty dollars less than the Class 12.5 HMA. So, by using the Class 19 HMA will save the overall cost of the future projects.

The test sections in order of best performance are: Portland cement, $CaCl_2$, control, geo-grid and asphalt emulsion with the smallest amount of cracking and highest predicted threshold distresses in years as shown in Graph 4.1. All the test sections predicted that there will not be any permanent deformation in the subbase or AC layer, or AC bottom up fatigue cracking (alligator cracking). Finally, it was concluded that higher the resilient moduli, better will be the results in terms of less distresses.

Table 4.1: Summary of AASHTOWare Pavement ME Design inputs and outputs for RI Route 165 Control test section

Design Inputs

Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2		
Flexible	2.5		
Cold recycled (FDR RAP)	8	Bradshaw et al., 2015	37,655
Non-stabilized (1980 Virgin RAP)	1	Bradshaw et al., 2015	30,650
Non-stabilized (Ex. gravel borrow)	8	Lee et al., 2003	13,620
Subgrade	Semi-infinite	Lee et al., 2003	9304

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25.00	4.60	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	2,548.73	Fail
AC thermal cracking (ft/mile)	1,000.00	84.34	Pass
Permanent deformation - total pavement (in)	0.75	0.51	Pass
AC only permanent deformation (in)	0.25	0.06	Pass
Terminal IRI (in/mile)	172.00	146.72	Pass

Table 4.2: Summary of AASHTOWare Pavement ME Design inputs and outputs for RI Route 165 Calcium Chloride test section

Design Inputs

Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2.00		
Flexible	2.50		
Cold recycled (FDR RAP)	8.00	Bradshaw et al., 2015	43,890
Non-stabilized (1980 Virgin RAP)	1.00	Bradshaw et al., 2015	31,510
Non-stabilized (Ex. gravel borrow)	8.00	Lee et al., 2003	13,620
Subgrade	Semi-infinite	Lee et al., 2003	9,304

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25.00	3.33	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	2354.80	Fail
AC thermal cracking (ft/mile)	1,000.00	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.51	Pass
Permanent deformation - AC only (in)	0.25	0.06	Pass
Terminal IRI (in/mile)	172.00	146.30	Pass

Table 4.3: Summary of AASHTOWare Pavement ME Design inputs and outputs for RI Route 165 Portland cement test section

Design Inputs

Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2.00		
Flexible	2.50		
Cold recycled (FDR RAP)	8.00	Bradshaw et al., 2015	70,240.00
Non-stabilized (1980 Virgin RAP)	1.00	Bradshaw et al., 2015	27,170.00
Non-stabilized (Ex. gravel borrow)	8.00	Lee et al., 2003	13,620.00
Subgrade	Semi-infinite	Lee et al., 2003	9,304.00

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25.00	2.01	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	1593.19	Pass
AC thermal cracking (ft/mile)	1,000.00	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.48	Pass
Permanent deformation - AC only (in)	0.25	0.06	Pass
Terminal IRI (in/mile)	172.00	144.84	Pass

Table 4.4: Summary of AASHTOWare Pavement ME Design inputs and outputs for RI Route 165 Asphalt Emulsion test section

Design Inputs

Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2.00		
Flexible	2.50		
Cold recycled (FDR RAP)	8.00	Bradshaw et al., 2015	31,420
Non-stabilized (1980 Virgin RAP)	1.00	Bradshaw et al., 2015	26,440
Non-stabilized (Ex. gravel borrow)	8.00	Lee et al., 2003	13,620
Subgrade	Semi-infinite	Lee et al., 2003	9,304

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25.00	9.54	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	3009.93	Fail
AC thermal cracking (ft/mile)	1,000.00	84.34	Pass
Permanent deformation - total pavement (in)	0.75	0.53	Pass
Permanent deformation - AC only (in)	0.25	0.06	Pass
Terminal IRI (in/mile)	172.00	148.06	Pass

Table 4.5: Summary of AASHTOWare Pavement ME Design inputs and outputs for RI Route 165 Geo grid test section

Design Inputs

Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2		
Flexible	2.5		
Cold recycled (FDR RAP)	10	Bradshaw et al., 2015	30,040
Cold recycled (FDR RAP)	5	Bradshaw et al., 2015	13,620
Subgrade	Semi-infinite	Lee et al., 2003	9304

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25	7.90	Pass
AC top-down fatigue cracking (ft/mile)	2000	2810.36	Fail
AC thermal cracking (ft/mile)	1000	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.52	Pass
Permanent deformation - AC only (in)	0.25	0.06	Pass
Terminal IRI (in/mile)	172	147.33	Pass

Table 4.6: Comparison of Performance Prediction of all Test Sections of RI Route 165

Design Outputs		Control	Calcium Chloride	Emulsion	Cemet	Geo-grid
Distress	Target	Predicted	Predicted	Predicted	Predicted	Predicted
Permanent deformation (in.)	0.75	0.51	0.51	0.53	0.48	0.52
AC bottom-up fatigue cracking (percent)	25	4.6	3.33	9.54	2.01	7.9
AC top-down fatigue cracking (ACTDFC) (ft/mile)	2000	2548.73	2354.8	3009.93	1593.19	2810.36
Permanent deformation-AC only (in.)	0.25	0.06	0.06	0.06	0.06	0.06
AC thermal cracking (ft/mi)	1000	84.34	84.3	84.3	84.3	84.3
Terminal IRI (in/mile)	172	146.72	146.3	148.06	144.84	147.33
Years to predict threshold distress ACTDFC (Years)	20	11	18	5	29	9

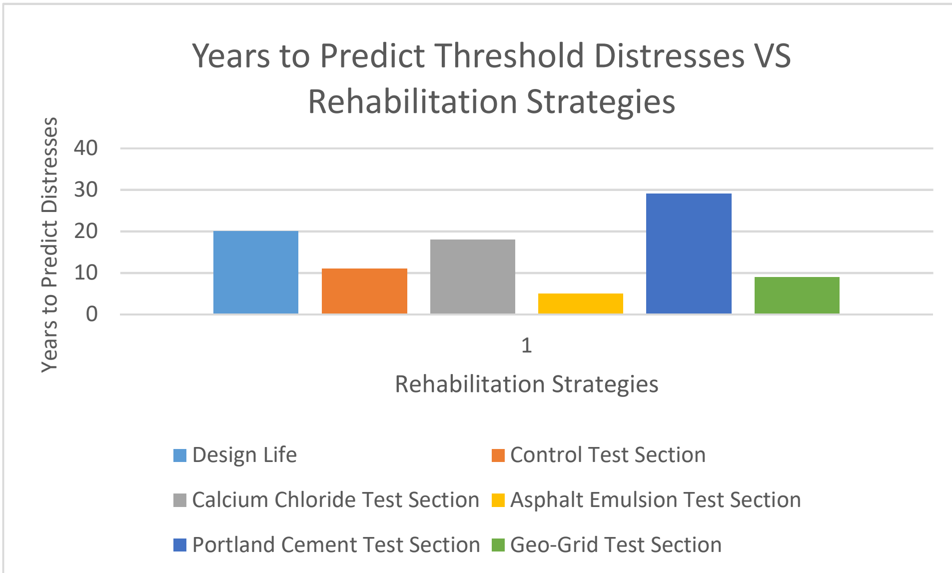


Figure 4.1: Comparison of all Test sections of RI Route 165 in terms of threshold distress period

Chapter 5

Evaluation of Performance for Rehabilitated Asphalt Pavement

5.1. Pavement Evaluation Methods

Pavement performance is a function of its relative ability to serve traffic over a period of time (HRB 1962). Originally, a pavement's relative ability to serve traffic was determined quite subjectively by visual inspection and experience. Typically, a system of objective measurements is used to quantify a pavement's condition and performance. These systems are used to aid in making the following types of decisions:

- Establish maintenance priorities. Condition data such as roughness, surface distress, and deflection are used to establish the projects most in need of maintenance and rehabilitation. Once identified, the projects in the poorest condition are more closely evaluated to determine repair strategies.

- Determine maintenance and rehabilitation strategies. Data from surface distress surveys are used to develop an action plan on a year-to-year basis; i.e., which strategy (patching, BSTs, overlays, recycling, etc.) is most appropriate for a given pavement condition.
- Predict pavement performance. Data, such as roughness, skid resistance, surface distress, or a combined rating, are projected into the future to assist in preparing long-range budgets or to estimate the condition of the pavements in a network given a fixed budget.

Pavement Evaluation generally consists of different steps which are mentioned below.

1. Initial Pavement Assessment
2. Condition or Visual Survey

3. Present Serviceability Index (PSI)
4. Ground Penetrating Radar Survey
5. Deflection Basins
6. Skid Resistance
7. International Roughness Index

5.2. Initial Pavement Assessment

The condition assessment needs to be begun with an assembly of historic data. Historic data can be obtained from a windshield pavement condition field survey of the entire project followed by a detailed survey of selected areas of the project, by reviewing construction files and results from previous borings and laboratory results, by considering previous distress and profile surveys and pavement management records to establish performance trends, and also by reviewing previous deflection survey.

RI Route 165 initial pavement assessment report is discussed in Chapter 2 subheading First Field Survey of RI Route 165.

5.3. Condition or Visual Survey

A key factor to determine the condition or strength of the existing pavement layers is the result from a detailed pavement condition index survey. Pavement visual surveys are performed to identify the types, locations, and severities of distress. The survey should be performed on the pavement, shoulder, and on any drainage feature along the project site. To conduct Condition survey, inspection sheet as shown in Figure 5.1 is used to determine the 19 different types of distresses, their severity, and their quantity. Most common distress types shown on the pavement are discussed below.

1. Alligator cracking (Bottom up cracking): Defined as a series of interconnected cracks (characteristically with an alligator pattern) that indicate at the bottom of the HMA layers. Alligator cracking as shown in Figure 5.2 is also one of the outputs of PavementME and calculated as a percent of total lane area.
2. Longitudinal Cracking (Top down Cracking): is a form of fatigue or wheel load related cracking that occurs within the wheel path and is defined as cracks predominantly parallel to the pavement centerline. Longitudinal cracks indicate at the surface of the HMA pavement and initially show up as short longitudinal cracks that become connected longitudinally with continued truck loadings. Raveling or crack deterioration may occur along the edges of these cracks but they do not form an alligator cracking pattern. Longitudinal cracking as shown in Figure 5.3 is also one of the outputs of PavementME and calculated as a total feet per mile, including both wheel paths.
3. Thermal Transverse Cracking: Defined as Non-wheel load-related cracking that is predominately perpendicular to the pavement centerline and caused by low temperature or thermal cycling. Thermal transverse cracking as shown in Figure 5.4 is also one of the outputs of PavementME and calculated in feet per 12-ft-wide lane.

On December 21, 2015, pavement windshield surveys were conducted on Route 165 by URI Graduate students and Professor K. Wayne Lee, and the results of these surveys are shown in Appendix F. Five pavement sections (10 feet wide x 100 feet in length) were selected near utility poles, previous FWD testing sites, and permanent land markers, for ease of identification. The pavement sections did not show any low, moderate or severe pavement distresses such as rutting or cracking but there were signs of minor raveling of the pavement. No major defects were expected since the pavement was recently placed in the summer of 2014. As a result, these December field surveys would become the base line for continuous monitoring of this road by URI students.

In addition to conducting the windshield surveys, RIDOT has on-going contracts with vendors whose responsibilities include measuring the IRI, rutting and cracking using vehicles equipped with computers, cameras, and lasers. Information is down-loaded into a Deighton's Total Infrastructure Management System (dTIMS) management database in the form of photographs, pavement defect data, and locations. DTIMS functions include three types of scoring: (1) a cross tab transformation to rate distresses and severity levels; (2) expression and formula transformations to place a deduct value from sample areas and calculate a pavement condition index (PCI) and/or Pavement Structural Health Index (PSHI), respectively; and structured table outputs with column and rows.

5.4. Present Serviceability Index

The present serviceability index (PSI) is based on the original AASHO Road Test PSR. Basically, the PSR was a ride quality rating that required a panel of observers to ride in an automobile over the pavement in question. PSI is based on the same 5-point rating system as PSR it goes beyond a simple assessment of ride quality. About one-half of the panel of raters found a PSR of 3.0 acceptable and a PSR of 2.5 unacceptable. Such information was useful in selecting a "terminal" (or failure) serviceability (PSI) design input for empirical structural design equations. Concept of PSI can be better understood by considering Figure 5.5.

PSI is highly subjective test to determine the performance of pavement so, for RI Route 165 PSI test was not conducted.

5.5. Ground Penetrating Radar (GPR) Survey

GPR is well established, high speed nondestructive technology used to estimate the thickness of different pavement and soil strata layers, and is frequently used to survey prior to use of destructive sampling. It is possible that GPR may be valuable in reducing the number of cores and borings required for a project, for example by segmenting the project based on similar subsurface features or anomalies identified with this technology prior to drilling the borings. Specifically, dielectric and thickness contours may be prepared along the project to locate areas with different structural features and material conditions. GPR data may be collected at highway speeds so that there is no interference with existing traffic flow. Figure 5.6 is showing GPR equipment on site.

GPR was performed on Route 165 in June 2015 after the final surface course was placed. The original, existing pavement thickness for each test section varied from 4.13 inches to 4.55 inches. The average core thickness for the Class 12.5 HMA pavement was 2.29 inches and the average pavement thickness for the 19mm pavement was 2.81 inches. The GPR was used to confirm that the roadway was being constructed according to RIDOT specifications. In areas where the pavement thickness varied between 4.13 and 4.21 inches, the west-bound geo-grid test, Portland cement, and recycled sections need to be monitored for possible premature cracking because of the relatively small pavement thickness.

5.6. Deflection Basin Test

The most widely used deflection testing device is the falling weight deflectometer (FWD) as shown in Figure 5.7. However, the use of seismic testing devices is increasing in popularity and does provide an estimate of the in-place modulus of pavement layers. Data from both of these Non-destructive testing technologies need to be calibrated to laboratory conditions in providing inputs to the PavementME. Deflection basin tests can be measured with different drop heights to evaluate the load-response characteristics of the pavement structure. Four drop heights are typically characterize the pavement structure into three distinct load-response categories; elastic, deflection softening, and deflection hardening. The spacing of the deflection tests will vary along a project. A closer spacing of testing points is suggested in pavements with history of fatigue cracking, In addition, deflection basin tests may be effective in cut and fill areas and in transition areas between cut and fill. Transition areas are where water can accumulate and weaken the underlying soils. The analysis of deflection basin data measured at different temperatures (morning and evening) may assist in determining the in-place properties of the HMA.

In an effort to predict roadway deflection, the RIDOT recently utilized its recently refurbished Kuab Falling Weight Deflectometer (FWD) on Route 165 which, incidentally, was one of the first roads it was used on. FWD results can be used to determine in situ resilient moduli of both the subgrade and subbase by back and forward calculations. RIDOT has been using FWD on their roads for years but, unfortunately, was neither able to internally interpret the results nor perform back calculations successfully. So, for this project resilient modulus values obtained from destructive testing was used as an input for PavementME.

5.7. Skid Resistance:

Skid resistance is the force developed when a tire that is prevented from rotating slides along the pavement surface (HRB 1972). Skid resistance is an important pavement evaluation parameter because:

- Inadequate skid resistance can lead to higher incidences of skid related accidents.
- Most agencies have an obligation to provide users with a roadway that is "reasonably" safe.
- Skid resistance measurements can be used to evaluate various types of materials and construction practices.

Skid resistance changes over time. Typically, it increases in the first two years following construction as the asphalt binder coating the top layer of aggregate is worn away by traffic, then decreases over the remaining pavement life as aggregates become more polished. Skid resistance is also typically higher in the fall and winter and lower in the spring and summer. This seasonal variation can skew skid resistance data if not properly compensated.

Some DOTs measure skid resistance using a locked-wheel skid tester as shown in Figure 5.8, which basically employs a test wheel that is locked up as it is rolling and skidded along the tested surface as a spray of water is applied in front (to simulate worst conditions). Data obtained are used to measure the tested surface's friction resistance.

On RI Route 165, skid resistance tests were performed in 2003, 2006, and 2010 and skid numbers obtained from test showed that there was significant increase of skid resistance after every passing year and overall ride ability and smoothness of pavement decreased. So, skid resistance test was become the base test to rehabilitate Route 165.

5.8. International Roughness Index (IRI)

Pavement roughness is an expression of irregularities in the pavement surface that adversely affect a vehicle's ride quality. Roughness is an important pavement characteristic because it affects not only ride quality but also vehicle operating costs, fuel consumption and maintenance costs. The World Bank found road roughness to be a primary factor in the analyses and trade-offs involving road quality vs. user cost.

The international roughness index (IRI), developed by the World Bank in the 1980s, is used to quantify roughness. IRI is based on the accumulated suspension of a vehicle (inches or mm) divided by the distance traveled by the vehicle during the measurement (miles or kilometers). The open-ended IRI scale is shown in Figure 5.9.

Roughness measurements can be made in a variety of ways including surveying instruments, portable inclinometers, profilographs, response type road roughness meters (RTRRMs) and profiling devices. The most common methods involve profilographs and profiling devices. For pavement condition surveys, some DOTs actually record the pavement's surface profile using laser equipment mounted in a specially equipped collection van as shown in Figure 5.10 and then convert this profile into a roughness measurement. In addition to collecting profile data, these vans also record rutting data.

IRI is one of the outputs of PavementME and calculated in in. /mile. For RI route 165, IRI tests were conducted in 2014 after paving road and found out the IRI values obtained from tests are much lower than the targeted value of IRI set by PavementME, i.e., 172 in. /mil as shown in Table 5.1.

Currently, RIDOT performs regular IRI tests on its roadways to monitor pavement performance over time. Based on pavement performance and distress type, PavementME Analysis Output Charts shows the IRI values increasing over time. Thus, based on these distresses and calculated PavementME IRI, it seems feasible that RIDOT would be able to compare their field generated values to the predicted charts in order to track performance of the PavementME.

5.9. Comparative Analysis of Performance Prediction of RI Route 165

PavementME output predicted AC top down fatigue cracking (longitudinal cracking) for almost all test section except Portland cement which passes for all distress types. ME Design reports also predicted the years to predict threshold distresses for all test section and it was observed that with current layer thickness only Portland cement can last for 20 years of design life. To evaluate the whole scenario generated with ME Design reports it was highly suggested that to find a way to overcome AC top down cracking. One of the causes of AC top down cracking is having thin top asphalt layer so, it was suggested to use additional 1 inch top Class 19mm base course asphalt layer and rerun all ME Design reports to check the outcomes. Finally it was observed that by increasing one inch of top asphalt layer AC top down cracking of all test sections came under the targeted value i.e. 2000 ft. /mil with same amount of years to predict threshold distresses. However, by additional one inch layer of asphalt it will increase the cost of the overall project but it is worth full to have additional one inch of asphalt layer now than to maintain or rehabilitate after four years of paving (in case of using Emulsion as an additive). The order of all test sections (lower to higher) in terms of cost analysis by paving with one inch additional asphalt layer is Portland cement, Control, Asphalt Emulsion, Calcium Chloride, and Geo-grid.

5.10. Guidelines for Long-Term Evaluation and Optimal Rehabilitation Design strategies

A material database consisting of resilient moduli, pavement core data and sieve analysis needs to be created for easy reference for design engineers. The RIDOT has years of collected data but unfortunately no "on-line" database. URI, on the other hand, has already done extensive resilient moduli testing with seasonal variations on subbase and subgrade materials and needs to incorporate these results into the state's database. The results of the testing should be included in one main database along with any new testing done (Lee et al. 2001).

LTPP currently has a Microsoft Excel Program that uses FWD deflections to predict resilient moduli of the asphalt layers, subbase and subgrade materials. The program, however, requires pavement and subbase thicknesses as input parameters which a GPR can provide. FWD testing is already being performed on state highways and this information should be appropriately documented and compiled into a database.

ASPHALT PAVEMENT INSPECTION SHEET

Zone ____________________ Pavement Type ____________________
Branch No. ____________________ Section Width ____________________
Branch ____________________ Section Length ____________________
Date ____________________ Section No. ____________________
Surveyed By ____________________ Sample Unit No. ____________________
Area of Sample ____________________

DISTRESS TYPES

1. Alligator Cracking
2. Bleeding
3. Block Cracking
4. Bumps and Sags *
5. Corrugation
6. Depression
7. Edge Cracking *
8. Jt. Refl Cracking *
9. Lane/Shldr Drop Off *
10. Long. & Trans. Crack
11. Patch and Utility Cut
12. Polished Aggregate
13. Potholes *
14. Railroad Cross
15. Rutting
16. Shoving
17. Slippage Crack
18. Swell
19. Weather/Ravel

Existing Distress Type, Quantity, and Severity						
Type →						
Quantity &						
Severity						
Total server L						
M						
H						

*All distresses are measured in square Feet except distances 4, 7, 8, 9, and 10 which are measured in linear feet. Distress # 13 is measured in number of potholes.

Drainage? (Y/N) ________
Type (CB/Dit/Other) ________
Condition (E/G/F/P) ________

Curbing? (Y/N) ________
Type (Conc/Gran/Bit) ________
Condition (E/G/F/P) ________
Avg. Curb Reveal ________

Figure 5.1: Inspection Sheet for Conducting Condition Survey

Figure 5.2: A View of Alligator Cracking on Pavement Surface

Figure 5.3: A View of Longitudinal Cracking on Pavement Surface

Figure 5.4: A view of Transverse Cracking on Pavement Surface

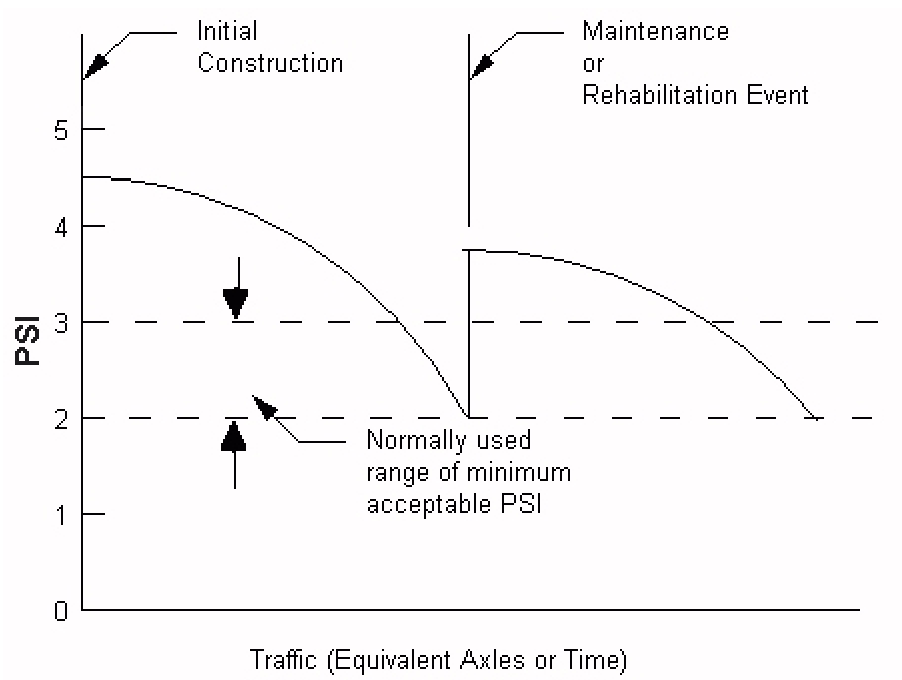


Figure 5.5: Concept of Pavement Performance Using Present Serviceability Index

Figure 5.6: Ground Penetrating Radar (GPR) Equipment

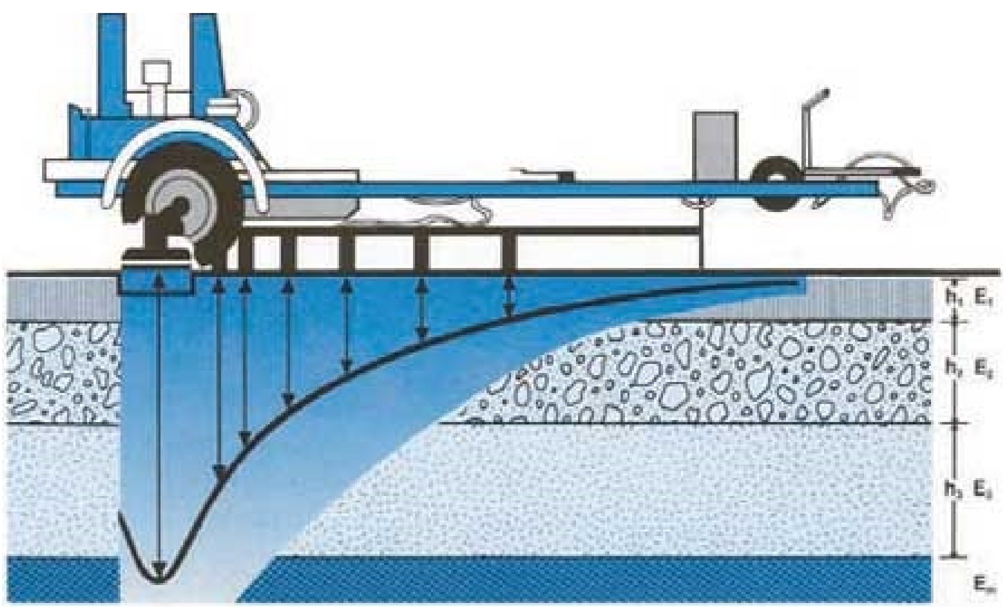


Figure 5.7: A View of Falling Weight Deflectometer (FWD) Mechanism

Figure 5.8: Lock Wheel Skid Tester Truck

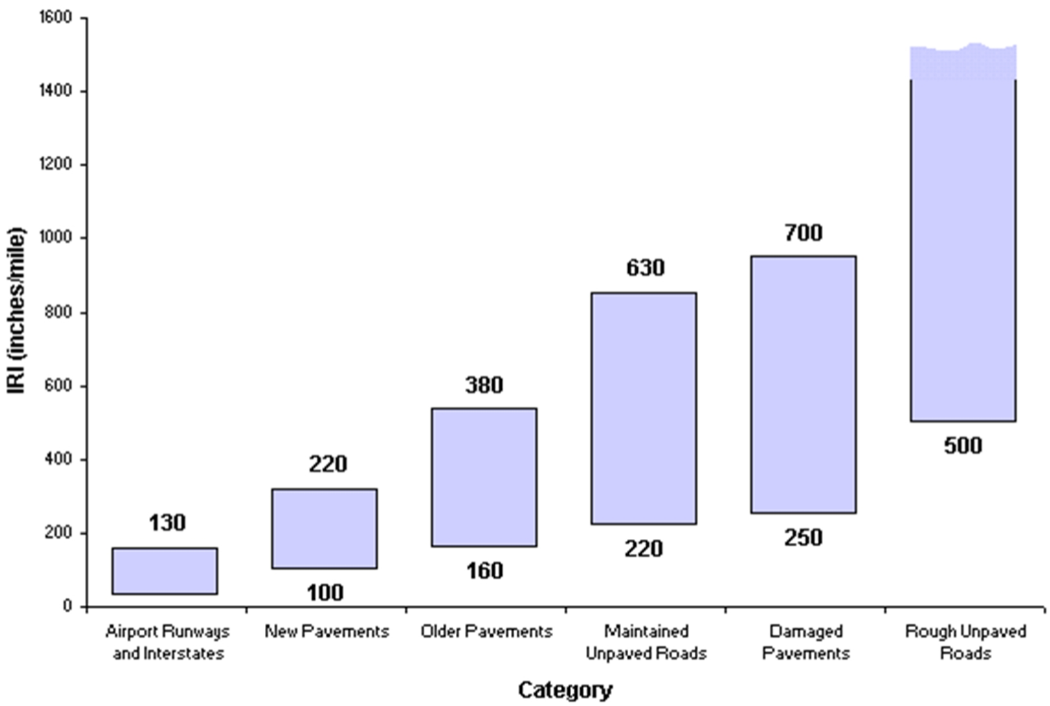


Figure 5.9: International Roughness Index (IRI) Roughness Scale

Table 5.1: Comparison of Performance Prediction for IRI of RI Route 165

Design Outputs		Control	$CaCl_2$	Emulsion	Portland Cement	Geo-grid
Distress	Target	Predicted	Predicted	Predicted	Predicted	Predicted
Terminal IRI (in./mil)	172.00	146.72	146.30	148.06	144.84	147.33
Av. Final Ride ability Results for WB (in. /mil) Left/Right lane.		58.4/46.9	44/40.43	65.8/88.9	64.3/52.9	63.7/56.7

Chapter 6

Conclusions and Recommendations

1. Recently RIDOT worked with URI, and established test sections successfully on RI Route 165 with Full Depth Reclamation (FDR) with five and additives and/or strategies, i.e., control, calcium chloride, asphalt emulsion, Portland cement, and geo-grid to find best practices.
2. Instron 8800 servo-hydraulic testing machine was successfully modified for determining dynamic modulus of HMA.
3. Instron 5582 machine was successfully used for determining creep compliance of HMA with modification, mainly software.
4. More accurate material input parameters including dynamic modulus and creep compliance of HMA were determined.
5. The performance of five different strategies was successfully predicted and evaluated using PavementME software. It required traffic, climate, and material properties data as an input and to predict the amount of distresses in terms of rutting, fatigue, thermal cracking, and roughness.
6. The outcomes obtained from this research study showed that all test sections observed AC top down (longitudinal cracking) except Portland cement section which passed for all distress type criteria. Pavement ME software also predicted the amount of years to show threshold distresses. The order in terms of performance (best to worst) for all test sections by PavementME software was Portland cement, calcium chloride, control, geo-grid, and asphalt emulsion.

7. It was observed that AC top down (longitudinal cracking) occurs due to thin top layer of asphalt. So, it was recommended that to provide additional one-inch asphalt layer i.e., 19 mm Class HMA to avoid AC top down cracking. PavementME software didn't show any distress type for all test sections with revised layer thicknesses.
8. It was also observed that higher the resilient modulus of pavement layer increases the stiffness of the material, and that the outcomes predicted by PavementME software didn't show any distresses. This could be confirmed because of the Portland cement section which has higher Mr value and didn't show any distress for almost 25 years.
9. It was recommended conducting condition survey to verify the PavementME software predictions. Furthermore, it was recommended that RIDOT will track all the performance data of Route 165 test sections.
10. Although it appeared that Portland cement is an excellent additive, the curing time can be a problem on narrow roads like Route 165 where detours are not possible. Detours drive up the costs for the project because of the additional traffic control and the delays to the traveling public. Thus, Portland cement could be considered for future projects where a detour is feasible.

Appendix A

Determining the Resilient Modulus of Subgrade soils and Aggregate Materials

Appendix A

Determining the Resilient Modulus of Subgrade Soils and Granular Materials

This method covers procedures for preparing and testing untreated subgrade soils and untreated subbase/base materials for determination of resilient modulus (M_r) under conditions representing a simulation of the physical conditions and stress states of materials beneath flexible pavements subjected to moving wheel loads.

Apparatus

The following apparatus were used in this test

1. Split Mold
2. Membranes
3. Vacuum Pump
4. Compacting Hammer and Machine
5. Triaxial cell
6. Pie tape and measuring ruler
7. Loading Device
8. Load and Specimen Response Measuring Equipment
9. Linear Variable Differential Transducers (LVDTs)
10. Signal excitation, conditioning and recording equipment
11. Triaxial Pressure Chamber shown in Figure A-1

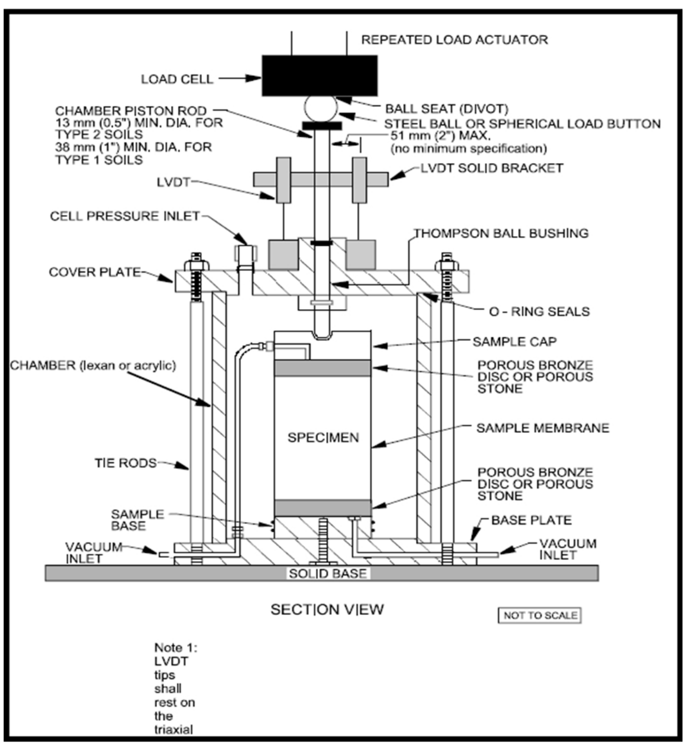


Figure A1: Triaxial Chamber for Resilient Modulus Testing

Procedure for determining Physical Properties:

1. Combined sample into 1 tray
2. Coned and quartered the sample (separated into four equal sections) and combined the diagonal piles
3. Setup tray and bowls to place materials passed ¾ in. sieve (-¾") and the ones which retained ¾ in. sieve (+¾").
4. Scalped out +¾" particles and place in bowl(s). Place remainder -¾"- particles into tray(s)
5. Recorded the weight of tray
6. Recorded the weight of the tray and material (-¾" and +¾")
   a. The difference of Step 5 and Step 6 was the weight of the wet sample
7. Air dried the wet sample (this should take ~1 week)

Resilient Modulus Material	Lab Testing
Moisture Content for -¾'' material = 1.44%	Moisture content for -¾'' material = 1.405%
Moisture content for +¾'' material = 0.65%	Moisture content for +¾'' material = 0.609%

8. Recorded the weight of the dry sample and determined the water content (-¾" and +¾")

Table A-1: Moisture Content of Sample

9. Performed grain size analysis on the entire sample
   a. Opening sizes in mm: 37.5, 10, 4.75, 2.38, 0.599, 0.075, and 0.01 (pan)
10. Performed specific gravity (G_s) test on +¾" particles only
11. Performed modified proctor test on -¾" particles only (AASHTO T180)
    a. When determining water content make sure to use air dry samples (DO NOT PLACE IN OVEN).
    b. From proctor curve: Maximum Dry Density, and Optimum Moisture Content (OMC)
12 Applied correction equations to convert OMC and Maximum Dry Density to field conditions.

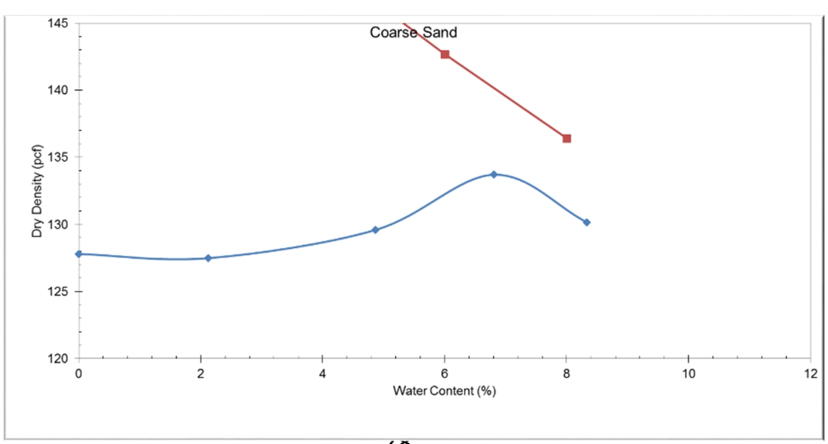


Figure A-2: Maximum Dry Density VS Water Content Curve

Sample Preparation:

1 Attached first membrane to base of triaxial cell using elastics
2 Attached Split Mold. Tightened screws on opposite direction with careful handling membrane, i.e., -not caught in split mold while tightening.
3 Stretched membrane over the top of the split mold and secured with elastics
4 Attached vacuum line with swage lock fittings to perimeter of split mold
5 Attached portable vacuum to the line so that membrane expands to the walls of the split mold
6 Placed filter paper at the bottom of split mold
7 Scalped out +1 ½'' particles from the testing material
8 Determined amount of material needed for sample preparation
9 Compacted resilient modulus sample in 6, 2-inch-thick, lifts
10 Added water to achieve OMC to sample material
11 Placed first lift of soil into split mold and compact using impact hammer
12 When finished released vacuum from split mold and applied a small vacuum to the base of the sample
13 Carefully folded membrane over the top and bottom of the sample
14 Measured and recorded the height and diameter of the sample
15 Assembled triaxial apparatus. Tightened rods the same amount
16 Removed vacuum from the base of the sample and transported to Instron testing machine.

Resilient Modulus Testing and Computer Operation:

Proportional Integral Derivative (PID) Settings:

1 Turned on Instron tower and computer tower
2 Turned on the water pump against the wall
3 From Instron Console (Control software for the load frame) on computer restored calibration
4 Placed triaxial cell with the sample in Instron and centered on the platen
5 Attached confining pressure line
6 Manually moved the Instron piston so that it is just above the triaxial cell piston
7 Applied seating load and confining pressure
8 Opened drainage valve at the base of the triaxial cell
9 Recorded the old and new Proportional Derivative Integral (PID) settings

Conditioning Phase:

1 Opened Wave matrix software (software where testing sequences are created)

2 Selected method (“conditioning Phase”)
3 Changed seating load and confining pressure according to AASHTO standard for conditioning phase
4 Set limits on Instron piston under Instron console
5 Attached LVDTs securely to brackets on triaxial cell
6 Finally started the test

Resilient Modulus Testing Phase

1 When conditioning phase was complete, clicked finish and continued project
2 Selected sequence type i.e., Subbase
3 Checked method tab, graphs, amplitude, rest period, etc.
4 Adjusted set point and confining pressure based on standard
5 Started the test and adjusted confining pressure when prompted throughout the test
6 Inserted data file into Mat Lab code and into a results summary file.

A typical stress vs strain relationship for each sequence for Rt. 165 materials is shown in Figure A-3.

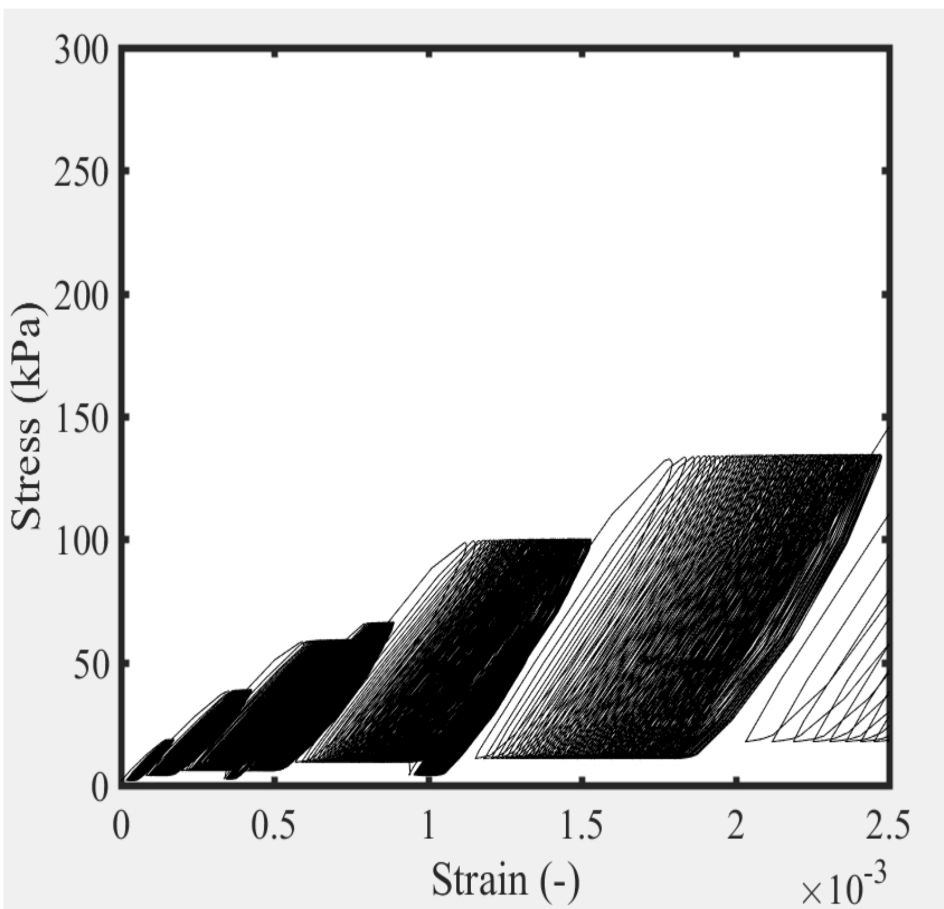


Figure A-3: Stress vs Strain Chart for Rt. 165 material

A typical test results for Rt. 165 FDR Base/Subbase materials are shown in Table A-2.

Table A-2. Resilient Modulus of Base/Subbase Material

Column #	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Parameter	Chamber Confining Pressure	Nominal Max. Axial Stress	Cycle No.	Actual Applied Max. Axial Load	Actual Applied Cycle Load	Actual Applied Contact Load	Actual Applied Max Axial Stress	Actual Applied Cycle Stress	Actual Applied Contact Stress	Recov Def. LVDT #1 Reading	Recov Def. LVDT #2 Reading	Average Recov Def LVDT 1 and 2	Resilient Strain	Resilient Modulus
Designation	S_3	S_{max}	C_1	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_1	H_2	H_{avg}	ϵ_r	M_r
Unit	kPa	kPa	__	kN	kN	kN	kPa	kPa	kPa	meter	meter	meter	m/m	MPa
Precision														
Sequence 1	20.7	20.7	100.00	0.34	0.30	0.03	18.51	16.61	1.90	4.09E-05		4.09E-05	1.34E-04	123.81
Sequence 2	20.7	41.4	100.00	0.71	0.64	0.07	38.95	34.96	3.99	8.45E-05		8.45E-05	2.77E-04	126.06
Sequence 3	20.7	62.1	100.00	1.08	0.97	0.11	59.27	53.23	6.03	1.25E-04		1.25E-04	4.10E-04	129.70
Sequence 4	34.5	34.5	100.00	0.59	0.54	0.05	32.31	29.78	2.53	6.26E-05		6.26E-05	2.05E-04	145.11
Sequence 5	34.5	68.9	100.00	1.21	1.10	0.11	66.30	60.37	5.93	1.21E-04		1.21E-04	3.97E-04	151.94
Sequence 6	34.5	103.4	100.00	1.83	1.66	0.17	100.18	90.78	9.40	1.80E-04		1.80E-04	5.91E-04	153.55
Sequence 7	68.9	68.9	100.00	1.22	1.14	0.07	66.48	62.48	4.00	1.07E-04		1.07E-04	3.51E-04	177.90
Sequence 8	68.9	137.9	100.00	2.46	2.26	0.20	134.59	123.66	10.93	2.02E-04		2.02E-04	6.64E-04	186.31
Sequence 9	68.9	206.8	100.00	3.70	3.38	0.33	202.46	184.63	17.83	2.97E-04		2.97E-04	9.75E-04	189.38
Sequence 10	103.4	68.9	100.00	1.22	1.18	0.04	66.63	64.56	2.07	9.74E-05		9.74E-05	3.19E-04	202.09
Sequence 11	103.4	103.4	100.00	1.84	1.74	0.10	100.82	95.29	5.52	1.39E-04		1.39E-04	4.55E-04	209.22
Sequence 12	103.4	206.8	100.00	3.72	3.43	0.29	203.18	187.32	15.86	2.61E-04		2.61E-04	8.58E-04	218.42
Sequence 13	137.9	103.4	100.00	1.85	1.78	0.07	100.98	97.39	3.59	1.30E-04		1.30E-04	4.25E-04	229.01
Sequence 14	137.9	137.9	100.00	2.47	2.34	0.13	135.27	128.21	7.05	1.65E-04		1.65E-04	5.40E-04	237.43
Sequence 15	137.9	275.8	100.00	4.97	4.59	0.38	271.79	250.94	20.85	3.11E-04		3.11E-04	1.02E-03	246.29
													ave	185.84

Analysis of Data Collection

By analyzing the above data, it was found that the resilient modulus of base/subbase material is 185.84 ksi (average value of all the test sequences) which shows the stiffness of the layer. Higher the resilient modulus value, stiffer will be the material

Conclusions

All resilient moduli used for performance prediction are included in Chapter 2. Those are for subgrade soils, subbase materials, and FDR base/subbase materials for five test sections in RI Rt. 165.

Appendix B

Properties of Asphalt Binder Used

Appendix B-1 Determination of Penetration of Bituminous Material

Purpose of Test

The purpose of this method was to determine the penetration of semi-solid and solid bituminous materials. Penetration can be best expressed as the consistency of a bituminous material expressed as the distance in tenth of a millimeter that a standard needle vertically penetrates a sample of the material under known conditions of time, loading and temperature.

Apparatus

The following apparatus used in this test

1. Penetration Apparatus that permits the needle holder to move vertically without measurable friction and is capable of measuring the depth of penetration to nearest 0.1mm.
2. Penetration needle
3. Sample Containers
4. Water Bath
5. Stop Watch
6. Thermometers
7. Cleaning Liquid

Test Procedure

1. First we prepare the sample for testing for that we put the sample (bitumen) in the oven and heat it in such a case that the temperature will not exceed to 90ºC the expected softening point of bitumen.
2. Then we pour the sample into two small containers such that when cooled to the temperature of test the depth of the sample is at least 10 mm greater than the depth to which needle is expected to penetrate.
3. Then we allowed the samples to get cool for some time so that their top surface become exactly flat.
4. For this test we should know the standard conditions of temperature (i.e. 25ºC or 77ºF), Load (i.e. 100g) and Time (i.e. 25 sec).
5. Then we examine the needle holder and tried to fix it so that the value on the scale attached to penetration apparatus goes to zero. Then we clean the needle by using cleaning liquid with the help of paper.
6. Then we placed the 50 g weight above the needle, making the total weight 100g.
7. Then we submerged the container into the water bath and paced the water bath on the stand of penetrometer.

8. After that we positioned the needle by slowly lowering it until its tip just makes contact with the surface of the sample. This can be accomplished by looking onto the sides of water bath.
9. Then we quickly released the needle by pressing the holder for a period of 5 seconds (time noted by using stop watch) and adjust the instrument to measure the distance penetrated in tenths of a mm.
10. We took at least three reading by doing the above procedure on a same container at different locations.
11. We cleaned the penetration needle every time we start our test to take reading with help of cleaning liquid.

Observations

Table B-1-1: Penetration rate in (mm) for Sample A

Test No.	Time (sec)	Load (g)	Temperature (°F)	Penetration Reading (mm)
1	5	100	77	107
2	5	100	77	93
3	5	100	77	85

Average value of penetration for sample A = $\frac{1 \quad +9 \; +8}{3}$ = 95 mm

Table B-1-2: Penetration rate in (mm) For Sample B

Test No.	Time (sec)	Load (g)	Temperature (°F)	Penetration Reading (mm)
1	5	100	77	80
2	5	100	77	106
3	5	100	77	143

Average value of penetration for sample B = $\frac{8 \; +1 \quad +1}{3}$ = 109.67 mm

Analysis of Data Collection

By analyzing the data collected by performing penetration test we can easily say that penetration rate is increasing after every time we performed test on a same sample as shown above in the tables of Sample A and B. This is because of the softening of asphalt with the passage of time and penetration needle go deeper and scale showed higher values of penetration.

Possible sources of error

There might be following possible errors conducting penetration test

1. Change in temperature during the test
2. Zero error in the scale measuring penetration rate
3. Error in measurement of time

Conclusion

By conducting Penetration test we can measure the consistency of all bituminous materials. Higher values of penetration indicate softer consistency. By performing this test we can easily differentiate the grades of all bituminous materials. Finally, we can say that penetration test is one of the best test to check one of the properties of bituminous materials i.e. Consistency. The penetration value of asphalt binder was 1,100.

Appendix B-2

Viscosity Determination of Asphalt using the Brookfield Thermosel Apparatus

Purpose of Test:

The purpose of this test is to measure the apparent viscosity, ratio of shear stress to shear rate for a Newtonian or non-Newtonian liquid, from 100 to 500 degrees F (38 to 260º C) using the Brookfield Thermosel apparatus.

Abbreviated Procedure:

Prepare the asphalt sample by heating in oven. Turn on the thermosel power and set the desired temperature. Once the equilibrium temperature is obtained with the appropriate spindle in the chamber, add the sample to the chamber. Approximately 8 to 10 mL of asphalt will be required, this mass must be calculated from the specific gravity. Reinsert the chamber into the viscometer and insert the spindle. Perform the test with viscometer at 20 rpm and observe at 20 rpm and observe the meter reading. Record three readings 60 sec apart at each temperature.

Observations:

The data collected from the viscometer can be seen below in table.

Test #	Asphalt	Weight (g)	Spindle #	Test Temperature (ºC)	Reading #	Time (min)	Viscosity (cP)
1	Unknown Sample	3.5	27	135	1 2 3	1 2 3	500 487 477
2	Unknown Sample	3.5	27	135	1 2 3	1 2 3	787 712 662
3	64-28	10.5	27	135	1 2 3	1 2 3	1325 1137 1012
4	64-28	10.5	27	135	1 2 3	1 2 3	2550 2000 1427

The viscometer directly reported the viscosity of the asphalt instead of reporting the % of the torque.

Analysis of Data

According to the procedure outlined in the ASTM specifications stated to plot viscosity VS. Temperature after the viscosity readings became consistent. Because these tests were all performed at a constant temperature of 135ºC Figure 1 through 4 below the viscosity vs. time for each of the tests.

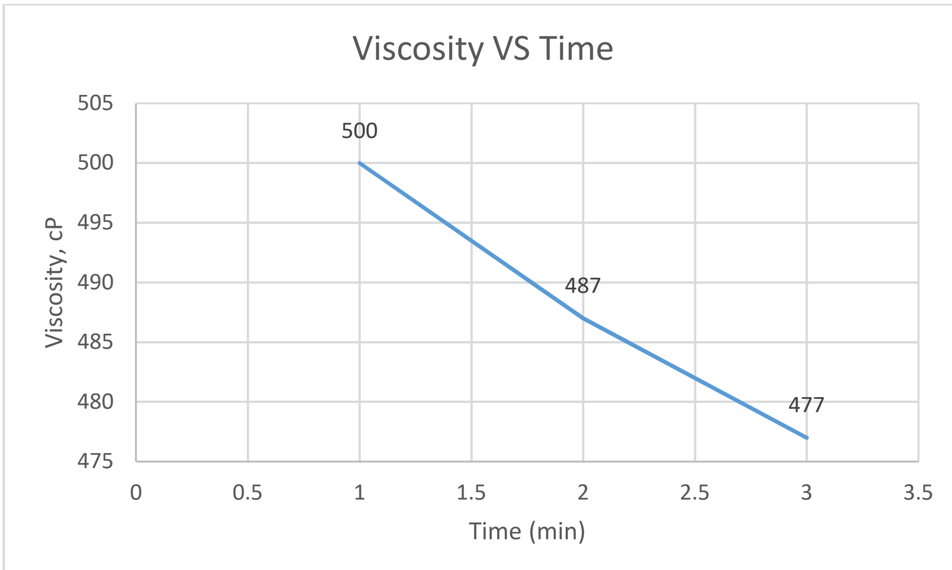


Figure B-2-1: Viscosity VS Time for reading 1

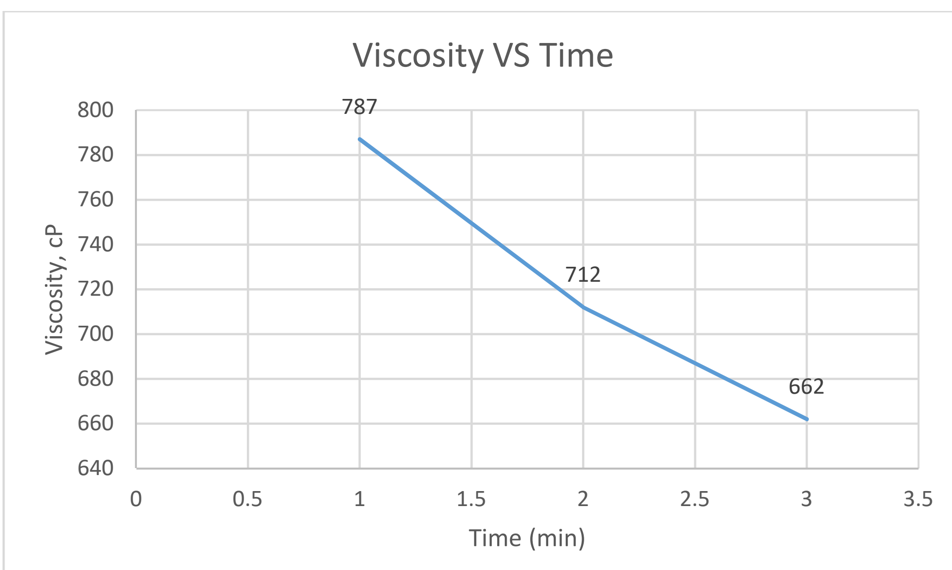


Figure B-2-2: Viscosity VS Time for reading 2

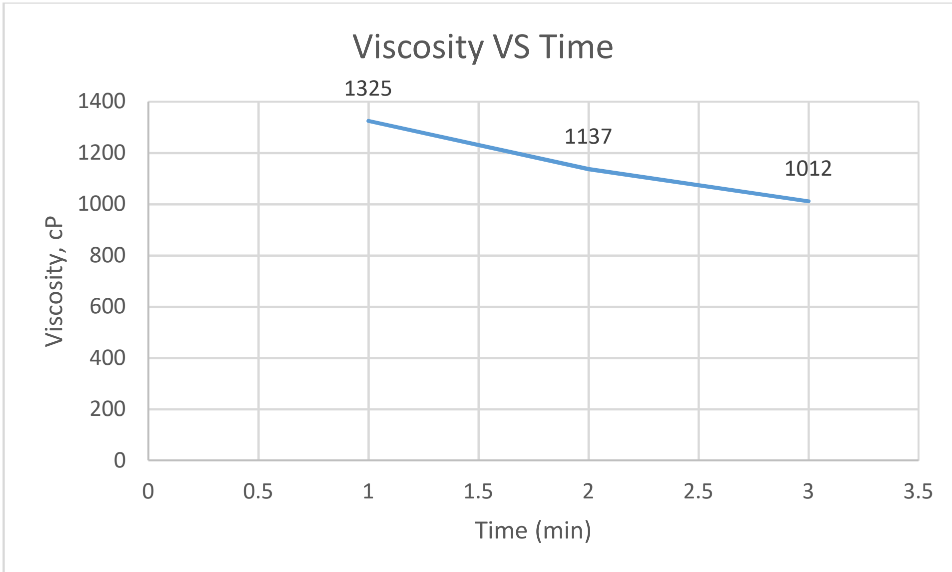


Figure B-2-3: Viscosity VS Time for reading 3

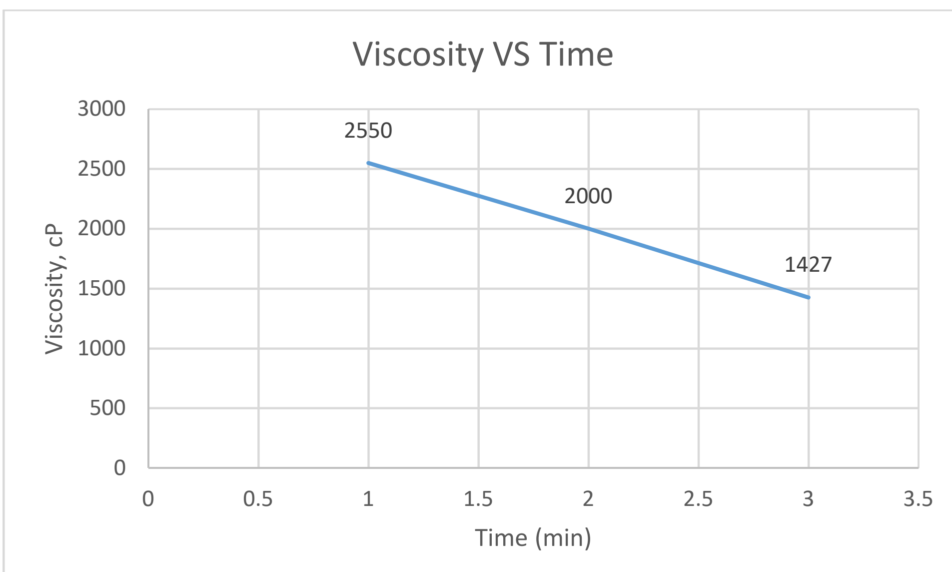


Figure B-2-4: Viscosity VS Time for reading 4

The unknown sample analyzed in test 1 and 2 has a final viscosity of 475 and 662 at 135ºC. Because this viscosity is near 500, I feel this asphalt has a viscosity grade of AC-5. This estimation of AC-5 could be verified with a penetration test.

Possible Source of Error

A possible error for this experiment was the fact that all of the tests were performed at a constant temperature of 135ºC instead of at various temperatures ranging from 38 to 260ºC (100-500ºF). This constant temperature made it impossible to plot the viscosity vs. changing temperature.

Conclusion

After analyzing the data gathered in this lab, I do not feel that the results were very accurate. The results for the same asphalt (both unknown sample and the PG 64-28) were inconsistent at the same temperature (135ºC). It was also impossible to plot viscosity vs. changing temperature due to the fact that all tests were performed at the same temperature. I was, however, able to plot the viscosity vs. time to show how viscosity decreases as the test is performed.

Appendix B-3

Determining the Rheology Properties of Asphalt Binder Using a Dynamic Shear Rheometer

Purpose of Test

This test method covers the determination of the dynamic shear modulus and phase angle of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate test geometry. This test is appropriate for unaged and aged asphalt binders both.

Apparatus

The following apparatus used in this test

1. Dynamic Shear Rheometer (DSR) Test system which consists of Parallel metal plates, an environmental chamber, a loading device and a control data acquisition system.
2. Metal plates made from stainless steel or aluminum with smooth ground surface. One set made up of 8.00 ± 0.02 mm in diameter and the other one of 25.00 ± 0.05mm in diameter.
3. Environmental Chamber used for controlling test temperature by heating or by cooling to maintain a constant specimen environment. The temperature in the chamber may be controlled by the circulation of fluid such as water and conditioned gasses like nitrogen etc.
4. Temperature Controller capable of maintaining specimen temperatures within $\pm 0.1^{o}$C.
5. Internal Temperature detector for the DSR used to control the temperature of specimen between two plates.
6. Loading Device apply a sinusoidal oscillatory load to the specimen at a frequency of 10.0 ± 0.1 rad/sec.
7. Control and data acquisition system which can provide a record of temperature, frequency, deflection angle and torque.
8. Specimen Mold
9. Specimen Trimmer
10. Wiping Material
11. Cleaning Solvents
12. Reference Thermometer
13. Optical Viewing Device
14. Electronic Thermometer

Figure B-3-1: Dynamic Shear Rheometer.

Test Procedure

1. This test is performed on unaged binders and binders that have been aged in a rolling thin film oven and pressure aging vessel.
2. At the beginning of the procedure, a 10 g sample is usually in a small container such as a “3” ounce tin. To prepare for testing heat the sample until it is sufficiently fluid to pour. The consistency should be less than 0.5 Pa-sec which is approximate consistency of motor oil. The sample should never be heated above 150ºC.
3. Turn on the rheometer air system by opening the supply regulator. The regulator is a valve affixed to the central laboratory air system and is normally located close to the rheometer. In many cases, the valve is part of a combination regulator/water filter system and is not a part of rheometer itself. It is important that the rheometer air system be on prior to manipulation of the rheometer to prevent damage to any components. Turn on personal computer system and temperature control system that circulates water.
4. Turn on rheometer and the attached computer. As we are using DSR by TA instruments so we used the Advanced Rheology Navigator software to run our test.
5. After initializing the software program do the calibration first with an operational thermometer and thermistor and check the calibrations with a testing material.
6. After doing calibration go to the main screen of the software then in scripts menu select utility scripts and then click on the zero gap option.
7. Once you click on the zero gap then software used you to attach 25 mm matching plates then select ok. Once you do that the upper head of the machine will start lowering its head to achieve 4500 microns value.
8. After that again from the script menu select original binder and enter your sample name and file name in the window then software will ask you about the range of temperature you want to run the test. In our case we put first temperature as 62ºC and the second temperature as 68ºC.
9. After that instrument will take some time to achieve the desired temperature and once again instrument will set zero gap.
10. In the mean while pour your sample into silicon made mold and give sample about 5 minutes to get cool and stiff.

11. After that software will ask you to load the sample simple remove the sample from mold and place it between two place and then select ok.
12. After doing that instrument will come down to achieve 1050 microns then software will ask you to trim the sample. Carefully by using trimming tools trim the sample from the sides of the plates and then select OK
13. Then instrument will go down further to 1000 microns and achieve desired temperature once again.
14. After that software will take 10 minutes to run the test at first given temperature and 10 more minutes to run the test for second given temperature.
15. After doing that software will tell you that test has been completed unscrew the plates and clean them and in the end it will print a 2 page report for you in which you can find the values of G*, Phase angle and G*/Phase angle.

Observations

After performing the above procedure we have following observations.

Table B-3-1: G*/sin delta of Asphalt Binder at 64ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Degrees)	G* (Pa)	G*/sin delta (kpa)
1	10.0	64.0	206.4	12.056	85.24	1717	1.723

Table B-3-2: G*/sin delta of Asphalt Binder at 70ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Degrees)	G* (Pa)	G*/sin delta (kpa)
1	10.0	70.0	98.14	11.972	86.66	819.7	0.821

Analysis of Data Collection

By analyzing the data collected by performing DSR test on unaged sample that our test passed on both temperatures provided by us that are 64 and 70ºC. According to PG Binder grading system G*/sin delta value for unaged sample should have the value minimum 1.00 KPa. In our case temperature at 64ºC pass the criteria for PG grading system.

Possible sources of error

There might be following possible errors conducting DSR test.

1. Calibration of the equipment

2. Not enough trimming of the sample after loading between plates
3. Not having enough good bulge as recommended by AASHTO specifications.

Conclusion

So in the whole we can say that this method is used to measure the complex shear modulus (G*) and Phase angle (sine delta) of asphalt binders using a dynamic shear rheometer and parallel plate test geometry. The test temperature from this method is related to temperature experienced by the pavement in the geographical area for which asphalt binder is intended to be used. The complex shear modulus is an indicator of the stiffness or resistance of asphalt binder to deformation under load. The complex shear modulus and the phase angle define the resistance to shear deformation of the asphalt binder in the linear viscoelastic region.

Appendix B-4

Determining the Rheology Properties of Asphalt Binder Using a Dynamic Shear Rheometer on Short term Aged Sample

Purpose of Test

This test method covers the determination of the dynamic shear modulus and phase angle of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate test geometry. This test is appropriate for unaged and aged asphalt binders both.

Apparatus

The following apparatus used in this test

1. Dynamic Shear Rheometer (DSR) Test system which consists of Parallel metal plates, an environmental chamber, a loading device and a control data acquisition system.
2. Metal plates made from stainless steel or aluminum with smooth ground surface. One set made up of 8.00 ± 0.02 mm in diameter and the other one of 25.00 ± 0.05mm in diameter.
3. Environmental Chamber used for controlling test temperature by heating or by cooling to maintain a constant specimen environment. The temperature in the chamber may be controlled by the circulation of fluid such as water and conditioned gasses like nitrogen etc.
4. Temperature Controller capable of maintaining specimen temperatures within $\pm0.1^{o}$C.
5. Internal Temperature detector for the DSR used to control the temperature of specimen between two plates.
6. Loading Device apply a sinusoidal oscillatory load to the specimen at a frequency of 10.0 ± 0.1 rad/sec.
7. Control and data acquisition system which can provide a record of temperature, frequency, deflection angle and torque.
8. Specimen Mold
9. Specimen Trimmer
10. Wiping Material
11. Cleaning Solvents
12. Reference Thermometer
13. Optical Viewing Device
14. Electronic Thermometer

Preparation of Short term Aged Sample using RTFO:-

1. For the preparation of aged sample we took three bottles of rolling thin film oven preheated at 163^{o}C in oven and pour 35gms of asphalt binder preheated in an oven.

2. Then we placed the bottles into the oven back which was preheated at 163°C. After that we set the air pressure of 4 Psi, closed the door of the oven, Turn on the rotational and air button on the oven.
3. We placed our sample in the oven for 85 minutes. After that we removed the bottles pour a little amount of sample into the 8mm silicon made mold for DSR testing on aged sample and remaining into the container for PAV testing on aged sample.

Test Procedure

1. This test is performed on binders that have been aged in a rolling thin film oven.
2. Turn on the rheometer air system by opening the supply regulator. The regulator is a valve affixed to the central laboratory air system and is normally located close to the rheometer. In many cases, the valve is part of a combination regulator/water filter system and is not a part of rheometer itself. It is important that the rheometer air system be on prior to manipulation of the rheometer to prevent damage to any components. Turn on personal computer system and temperature control system that circulates water.
3. Turn on rheometer and the attached computer. As we are using DSR by TA instruments so we used the Advanced Rheology Navigator software to run our test.
4. After initializing the software program do the calibration first with an operational thermometer and thermistor and check the calibrations with a testing material.
5. After doing calibration go to the main screen of the software then in scripts menu select utility scripts and then click on the zero gap option.
6. Once you click on the zero gap then software used you to attach 25 mm matching plates then select ok. Once you do that the upper head of the machine will start lowering its head to achieve 4500 microns value.
7. After that again from the script menu select original binder and enter your sample name and file name in the window then software will ask you about the range of temperature you want to run the test. In our case we put first temperature as 62ºC and the second temperature as 68ºC.
8. After that instrument will take some time to achieve the desired temperature and once again instrument will set zero gap.
9. In the mean while pour your sample into silicon made mold and give sample about 5 minutes to get cool and stiff.
10. After that software will ask you to load the sample simple remove the sample from mold and place it between two place and then select ok.
11. After doing that instrument will come down to achieve 1050 microns then software will ask you to trim the sample. Carefully by using trimming tools trim the sample from the sides of the plates and then select OK
12. Then instrument will go down further to 1000 microns and achieve desired temperature once again.
13. After that software will take 10 minutes to run the test at first given temperature and 10 more minutes to run the test for second given temperature.

14. After doing that software will tell you that test has been completed unscrew the plates and clean them and in the end it will print a 2 page report for you in which you can find the values of G*, Phase angle and G*/Phase angle.

Observation: -

After performing the above procedure we have following observations.

Table B-4-1: G*/sin delta value at 64ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Degrees)	G* (Pa)	G*/sin delta (kpa)
1	10.0	64.0	586.0	12.011	77.43	4902	5.023

Table B-4-2: G*/sin delta value at 70ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Degrees)	G* (Pa)	G*/sin delta (kpa)
1	10.0	70.0	293.8	12.095	79.93	2446	2.185

Analysis of Data Collection

By analyzing the data collected by performing DSR test on short term aged sample that our test passed on both temperatures provided by us that are 64 and 70ºC. According to PG asphalt binder grading system for sample under short term aging should have the value min 2.2 kPa. In our case temperature at 64ºC passed through this criteria. Finally we can say that 64ºC is the final maximum temperature for this sample.

Possible sources of error

There might be following possible errors conducting DSR test.

1. Calibration of the equipment
2. Not enough trimming of the sample after loading between plates
3. Not having enough good bulge as recommended by AASHTO specifications.

Conclusion

So in the whole we can say that this method is used to measure the complex shear modulus (G*) and Phase angle (sine delta) of asphalt binders using a dynamic shear rheometer and parallel plate test geometry. The test temperature from this method is related to temperature experienced by the pavement in the geographical area for which asphalt binder is intended to be used. The complex shear modulus is an indicator of the stiffness or resistance of asphalt binder to deformation under load. The complex shear modulus and the phase angle define the resistance to shear deformation of the asphalt binder in the linear viscoelastic region. In this report we predicted that 64^{o}C is the final maximum temperature of this sample.

TA

DSR Report

DSR Specifications

DSR Description: DHR2 5332-1153 UHP/Peltier

Bearing Type: Air/Magnetic

Temperature Control: UHP/Peltier

Files

C:\TA\Rheology\Results\sangkyu lee-0001o.rsl
10/7/2015 9:35:35 AM

C:\TA\Rheology\Results\sangkyu lee-0002o.rsl
10/7/2015 9:48:54 AM

Sample, Procedure, and Geometry Details

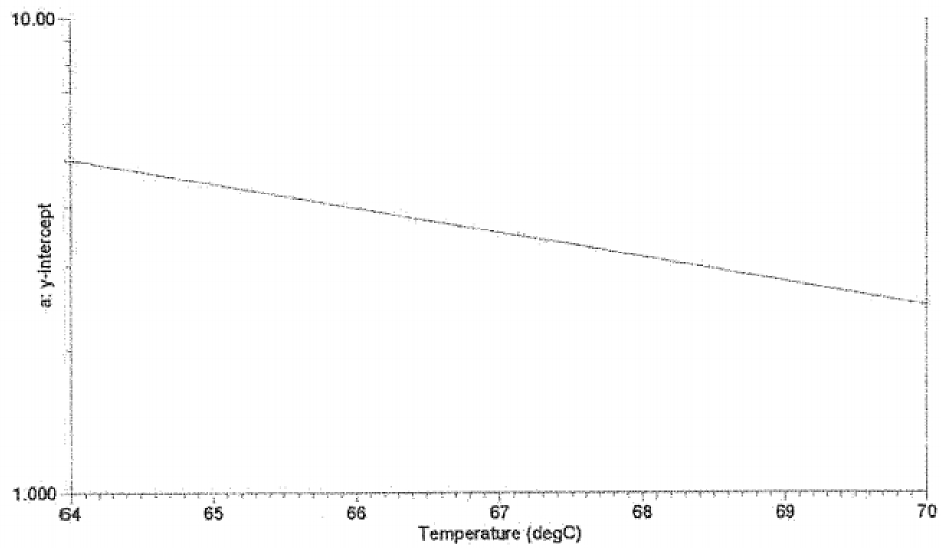
Sample name	sangkyu lee
Procedure name	Original Binder
Geometry name	25.0mm steel plate

Analysis Summary

	filename	QC Pass/Fail	a: y-intercept	Temperature
				degC
1	sangkyu lee-0001o.rsl	Pass	5.023	64
2	sangkyu lee-0002o.rsl	Pass	2.485	70

Fail Graph

Fail temperature = 77.76°C



Appendix B-5

Determining the Rheology Properties of Asphalt Binder Using a Dynamic Shear Rheometer on Pressurized Aging Vessel Sample

Purpose of Test

This test method covers the determination of the dynamic shear modulus and phase angle of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate test geometry. This test is appropriate for unaged and aged asphalt binders both.

Apparatus

The following apparatus used in this test

1. Dynamic Shear Rheometer (DSR) Test system which consists of Parallel metal plates, an environmental chamber, a loading device and a control data acquisition system.
2. Metal plates made from stainless steel or aluminum with smooth ground surface. One set made up of 8.00$\pm$0.02 mm in diameter and the other one of 25.00$\pm$0.05mm in diameter.
3. Environmental Chamber used for controlling test temperature by heating or by cooling to maintain a constant specimen environment. The temperature in the chamber may be controlled by the circulation of fluid such as water and conditioned gasses like nitrogen etc.
4. Temperature Controller capable of maintaining specimen temperatures within $\pm$0.1°C.
5. Internal Temperature detector for the DSR used to control the temperature of specimen between two plates.
6. Loading Device apply a sinusoidal oscillatory load to the specimen at a frequency of 10.0$\pm$0.1 rad/sec.
7. Control and data acquisition system which can provide a record of temperature, frequency, deflection angle and torque.
8. Specimen Mold
9. Specimen Trimmer
10. Wiping Material
11. Cleaning Solvents
12. Reference Thermometer
13. Optical Viewing Device
14. Electronic Thermometer

Preparation of Pressurized Aging Vessel (PAV) Sample:

1. Combine the hot residue from the RTFO into a single container, stir to blend, then transfer into TFOT pans for PAV conditioning.

2. Place the pan holder inside the pressure vessel. If an oven is used, place the pressure vessel inside the oven. If an integrated temperature control pressure vessel is used, turn on the heater. Select an aging temperature and preheat the pressure vessel to the aging pressure selected.
3. Place the TFOT pan on a balance and add 50 grams of asphalt binder to the pan. This will yield approximately a 3.2 mm thick film of asphalt binder.
4. If the vessel is preheated to other than the desired aging temperature, reset the temperature control on the heating device to the aging temperature.
5. Place the filled pans in the pan holder and then place the pan holder with filled pans inside the pressure vessel and close the pressure vessel.
6. Connect the temperature transducer line and the air pressure supply line to the loaded pressure vessel's external connections.
7. Wait until the temperature inside the pressure vessel is within 2ºC of the aging temperature, apply an air pressure of 2.1 MPa and then start timing the test.
8. Maintain the temperature and air pressure inside the pressure vessel for 20 hours.
9. At the end of the 20 hour test period slowly begin reducing the internal pressure of the PAV, using the air pressure bleed valve. Adjust the bleed valve to an opening that requires 9 minutes to equalize the internal and external pressures on the PAV thus avoiding excessive bubbling and foaming of asphalt binder.
10. If the temperature indicated by temperature recording device falls above or below the target aging temperature 0.5ºC for more than 10 minutes during the 20 hour aging period, declare the test invalid and discard the material.
11. Remove the pan holder and pans from PAV, and place in an oven set at 163ºC. Heat until sufficiently fluid to pour. Stir gently in the removal of air bubbles.
12. Pour a small amount of sample into rubber mold and allow sample to cool down for DSR testing.

Test Procedure

1. This test is performed on binders that have been aged in PAV.
2. Turn on the rheometer air system by opening the supply regulator. The regulator is a valve affixed to the central laboratory air system and is normally located close to the rheometer. In many cases, the valve is part of a combination regulator/water filter system and is not a part of rheometer itself. It is important that the rheometer air system be on prior to manipulation of the rheometer to prevent damage to any components. Turn on personal computer system and temperature control system that circulates water.
3. Turn on rheometer and the attached computer. As we are using DSR by TA instruments so we used the Advanced Rheology Navigator software to run our test.
4. After initializing the software program do the calibration first with an operational thermometer and thermistor and check the calibrations with a testing material.
5. After doing calibration go to the main screen of the software then in scripts menu select utility scripts and then click on the zero gap option.

6. Once you click on the zero gap then software used you to attach 25 mm matching plates then select ok. Once you do that the upper head of the machine will start lowering its head to achieve 4500 microns value.
7. After that again from the script menu select PAV residue and enter your sample name and file name in the window then software will ask you about the range of temperature you want to run the test. In our case we put first temperature as 25ºC and the second temperature as 22ºC.
8. After that instrument will take some time to achieve the desired temperature and once again instrument will set zero gap.
9. In the mean while pour your sample into silicon made mold and give sample about 5 minutes to get cool and stiff.
10. After that software will ask you to load the sample simple remove the sample from mold and place it between two place and then select ok.
11. After doing that instrument will come down to achieve 2050 microns then software will ask you to trim the sample. Carefully by using trimming tools trim the sample from the sides of the plates and then select OK
12. Then instrument will go down further to 2000 microns and achieve desired temperature once again.
13. After that software will take 10 minutes to run the test at first given temperature and 10 more minutes to run the test for second given temperature.
14. After doing that software will tell you that test has been completed unscrew the plates and clean them and in the end it will print a 2 page report for you in which you can find the values of G*, Phase angle and G*/Phase angle.

Observation:-

After performing the above procedure we have following observations.

Table B-5-1: G*/sin delta value at 25ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Deg)	G* (Pa)	G*/sin delta (kpa)
1	9.991	25.0	55920	0.9988	44.98	5.61E6	3970

Table B-5-2: G*/sin delta value at 22ºC

Sr. No	Angular Frequency (Rad/sec)	Temperature (ºC)	Osc. Stress (Pa)	% Strain	Delta (Deg)	G* (Pa)	G*/sin delta (kpa)
1	9.991	22.0	83840	1.016	42.19	8.28E6	5562

Analysis of Data Collection

By analyzing the data collected by performing DSR test on long term aged sample that our test passed on 25ºC temperature. According to PG grading system sample with long term aging should have G*/sin delta value less than 5000 kPa. In our case at 25ºC passed the criteria.

Possible sources of error

There might be following possible errors conducting DSR test.

1. Calibration of the equipment
2. Not enough trimming of the sample after loading between plates
3. Not having enough good bulge as recommended by AASHTO specifications.

Conclusion

So in the whole we can say that this method is used to measure the complex shear modulus (G*) and Phase angle (sine delta) of asphalt binders using a dynamic shear rheometer and parallel plate test geometry. The test temperature from this method is related to temperature experienced by the pavement in the geographical area for which asphalt binder is intended to be used. The complex shear modulus is an indicator of the stiffness or resistance of asphalt binder to deformation under load. The complex shear modulus and the phase angle define the resistance to shear deformation of the asphalt binder in the linear viscoelastic region.

DSR Report

DSR Specifications

DSR Description: TA Instruments AR500

Bearing Type: Air

Temperature Control: peltier

CA4-1

Sample, Procedure, and Geometry Details

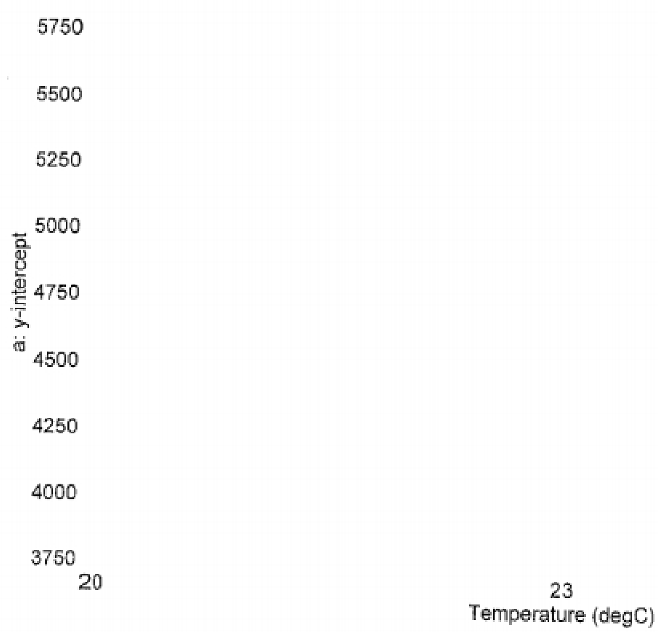
Sample name	ca~~44~~15
Procedure name	PAV Residue
Geometry name	8.0mm plate

Analysis Summary

	filename	QC Pass/Fail	a: y-intercept	Temperature
				degC
1	ca44-15-0001o.rsl	Pass	3970	25
2	ca44-15-0002o.rsl	Fail	5562	22

Fail Graph

Fail temperature = 22.95°C



Appendix B-6

Determining the Flexural Creep Stiffness of Asphalt Binder using Bending Beam Rheometer

Purpose of Test

This test method covers the determination of the flexural creep stiffness or compliance of asphalt binder by means of a bending beam rheometer. It is applicable to material having flexural stiffness value from 20 MPa to 1 GPa values in the range of (50 nPa^{-1} to 1 nPa^{-1}) and can be used with unaged material or with material aged using RTFOT or PAV test. This test apparatus is designed to test within the temperature range of -36ºC to 22ºC.

Apparatus

The following apparatus used in this test.

1. Bending Beam Rheometer Test System (Figure B-6-1)
2. Loading Frame
3. Loading System
4. Sample supports
5. Loading Shaft
6. Controlled Temperature fluid bath
7. Data Acquisition System
8. Thermometers
9. Test Beam molds

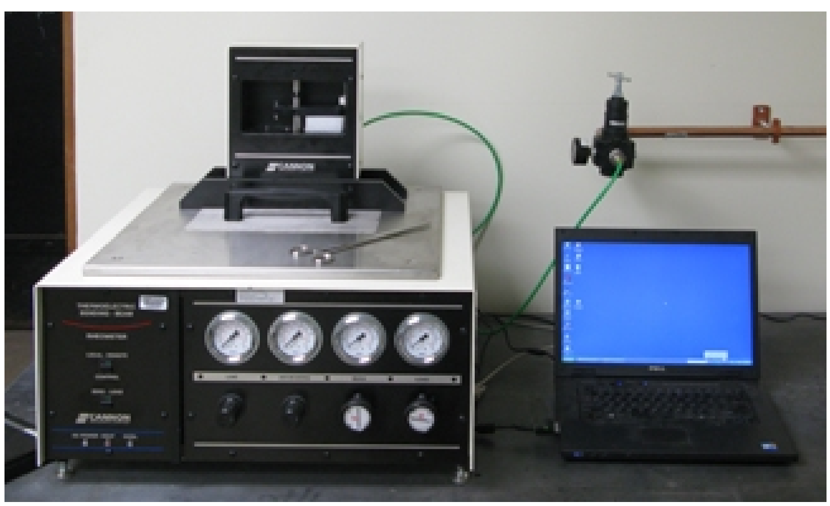


Figure B-6-1: Bending Beam Rheometer Test System

Preparation of Asphalt Binder Beam:-

Prepare the asphalt sample by heating in oven. Pour the sample in standard aluminum mold. Allow the sample to cool down at room temperature for 45 minutes then trim it from the top after then put in the refrigerator to cool down. Set the temperature of the water bath inside the BBR system for your test temperature in our case we set the temperature at -18ºC. Once asphalt in the mold gets hard remove from the mold and place the asphalt binder beam in the water bath for one hour to equalize temperature.

Test Procedure:-

First step to run BBR test is to do calibrations for Load, Deflection and Temperature. Use zero and load gauges on the BBR machine to lower or raise the shaft. Apply pressure of 40 psi to BBR machine. For load calibrations apply four different 100g weight on the machine on a thick beam and save the readings in the software. Similarly save the deflection calibration as instructed by software. After doing calibration run a confidence test on a thin beam by applying four different 100g weights and save the results in the software. Once it done then place the asphalt binder beam under the loading machine and run the test from the software and software will calculate the stiffness and deflection values of the beam in a given interval of time that is 240 seconds and Load.

Observations: Table B-6-1: Slope value (m) at Target temperature -18ºC

Time T (sec)	Load P (mN)	Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-values
8.0	979	0.144	548	548	0.000	0.216
15.0	981	0.166	476	475	-0.210	0.238
30.0	983	0.199	398	400	0.503	0.261
60.0	987	0.240	332	331	-0.301	0.285
120.0	990	0.296	270	269	-0.370	0.309
240.0	994	0.372	215	216	0.465	0.339

Table B-6-2: Slope value (m) at Target temperature -12ºC

Time T (sec)	Load P (mN)	Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-values
8.0	988	0.278	287	287	0.000	0.276
15.0	987	0.332	240	240	0.000	0.295
30.0	985	0.410	194	194	0.000	0.316
60.0	983	0.513	155	154	-0.645	0.338
120.0	981	0.652	121	121	0.000	0.389
240.0	981	0.842	93.9	93.9	0.000	0.380

Analysis of Data Collection

According to PG grading system m-value for asphalt binder sample should be minimum 0.3. By analyzing the data obtained from BBR test at two different temperature of -18ºC and -12ºC (as attached report shows) of same asphalt binder sample we came to know that the this binder is good for -26ºC which is slightly more than what we desire that is -28ºC so we have to add some modifiers to make it equal to PG 64-28 grading.

Possible sources of error

There might be following possible errors conducting BBR test.

1. Calibrating temperature to -18ºC and -12ºC inside water bath due to some external factors.
2. Calibrating load values due to the sensitivity of machine
3. No proper cleaning of shaft which have to stand on asphalt binder beam.

Conclusion

On the whole we can say that BBR test is good to use for testing temperature experienced by pavement in the geographical area for which asphalt binder is intended to use. The flexural creep stiffness or flexural creep compliance, determined from this test describe the low temperature, stress-strain time response of asphalt binder at the test temperature within the linear viscoelastic response range. Finally by looking into the chart given by FHWA as shown in Figure B-6-2 it was predicted that the sample used for testing to determine its PG grade is PG 64-28.

PG-BINDER GRADING SYSTEM

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-						PG 70-						PG 76-					PG 82-				
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Avg. 7-day max. Pavement design Temp., °C	<46			<52							<58					<64						<70						<76					<82				
Min. Pavement Design Temp., °C (a)	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																																					
Flash Point Temp., T48: Min., °C	230																																				
Viscosity , ASTM D 4402: Max. 3 Pa·s, Test Temp, °C (b) rotational	135																																				
Dynamic Shear TP5: G*/sin δ, Min., 1.00 kPa; Test Temp. @ 10 rad/s, °C (c)	46			52							58					64						70						76					82				
ROLLING THIN FILM OVEN RESIDUE (T240)																																					
Mass Loss, Max., Percent	1.00																																				
Dynamic Shear TP5: G*/sin δ, Min., 2.20 kPa; Test Temp. @ 10 rad/s, °C	46			52							58					64						70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP1)																																					
PAV Aging Temp., °C (d)	90			90							100					100						100(110)						100(110)					100(110)				
Dynamic Shear TP5: G*sin δ, Max., 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	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening (e)	BBR REPORT																																				
Creep Stiffness, TP1: Max., 300 MPa; m-value, Min., 0.300 Test Temp. @ 60 s, °C (f)	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP3: Failure strain, Min., 1.0%; Test Temp. @ 1.0 mm/min, °C (f)	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24

Figure B-6-2: PG Asphalt Binder Grading System Chart

BBR Report

Project :
Operator : pj
Specimen : 10-64-15
Test Time : 02:10:04 PM
Test Date : 11/03/2015
File Name : 15110303
BBR ID : 9728

Target Temp (°C) : -18.0
Min. Temp (°C) : -18.1
Max. Temp (°C) : -17.9
Temp Cal Date : 11/03/2015
Soak Time (min) : 60.0
Beam Width (mm) 12.70
Thickness (mm) : 6.35

Conf Test (GPa) : 217
Conf Date : 11/03/2015
Force Const (mN/bit) 0.15
Defl Const (µm/bit) : 0.151
Cmpl (µm/N) : 7.1
Cal Date : 11/03/2015
Software Version : BBR w 1.23

t Time (s)	P Force (mN)	d Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-value
8.0	979	0.144	548	548	0.000	[illegible]
15.0	981	0.166	476	475	-0.210	[illegible]
30.0	983	0.199	398	400	0.503	[illegible]
60.0	987	0.240	332	331	-0.301	0.[illegible]85
120.0	990	0.296	270	269	-0.370	[illegible]
240.0	994	0.372	215	216	0.465	[illegible]

A = 2.9 B = -0.145 C = -0.0395 R^2 = 0.999954

Force (t=0.0s) = 32 mN Deflection (t=0.0s) = 0.000 mm
Force (t=0.5s) = 954 mN Deflection (t=0.5s) = 0.083 mm

Max Force Deviation (t=0.5 - 5.0s) = -35, +0 mN
Max Force Deviation (t=5.0 - 240.0s) = -12, +6 mN

Average Force (t=0.5 - 240.0s) = 989 mN
Maximum Force (t=0.5 - 240.0s) = 995 mN
Minimum Force (t=0.5 - 240.0s) = 954 mN

Project :		Target Temp (°C) :	-12.0	Conf Test (GPa) :	211
Operator :	pjm	Min. Temp (°C) :	-12.0	Conf Date :	11/03/2015
Specimen :	10-64-15	Max. Temp (°C) :	-12.0	Force Const (mN/bit)	0.[illegible]
Test Time :	02:09:48 PM	Temp Cal Date :	11/03/2015	Defl Const (μm/bit) :	0.[illegible]
Test Date :	11/03/2015	Soak Time (min) :	60.0	Cmpl (μm/N) :	3.54
File Name :	15110301	Beam Width (mm)	12.70	Cal Date :	11/03/2015
BBR ID :	3024	Thickness (mm) :	6.35	Software Version :	BBR w 1.23

t Time (s)	P Force (mN)	d Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-value
8.0	988	0.278	287	287	0.000	0.[illegible]
15.0	987	0.332	240	240	0.000	0.[illegible]
30.0	985	0.410	194	194	0.000	0.[illegible]
60.0	983	0.513	155	154	-0.645	0.38
120.0	981	0.652	121	121	0.000	0.[illegible]
240.0	981	0.842	93.9	93.9	0.000	0.[illegible]

A = 2.68 B = -0.212 C = -0.0353 R^2 = 0.999999

Force (t=0.0s) = 40 mN Deflection (t=0.0s) = 0.000 mm
Force (t=0.5s) = 996 mN Deflection (t=0.5s) = 0.138 mm

Max Force Deviation (t=0.5 - 5.0s) = -0, +14 mN
Max Force Deviation (t=5.0 - 240.0s) = -2, +7 mN

Average Force (t=0.5 - 240.0s) = 982 mN
Maximum Force (t=0.5 - 240.0s) = 996 mN
Minimum Force (t=0.5 - 240.0s) = 980 mN

Appendix C

Super Pave Volumetric Design for Hot-Mix Asphalt

Appendix C
Super Pave Volumetric Design for Hot-Mix Asphalt

Purpose of test

The objective of the lab was to use optimum gradation provided to determine optimum binder content from samples compacted using the SuperPAVE Gyratory Compactor.

Apparatus

Gyratory Compactor, Computer, Mixer, Oven, Mixing Tools, Containers, Scales, Vacuum Device, Vibratory Device & Gloves.

Test procedure

1. Sample Preparation and Compaction:
    a. Weigh out 4600 grams of aggregate for each of the eight specimens being made. Four gradations will be used, with two samples of each gradation. Prepare loose mix as per Marshall Design Method, using 4.5%, 5.0%, 5.5% and 6.0% asphalt content (AC) by weight for a set of two specimens.
    b. Marshall Design method involves curing loose aggregate mix in oven at 135^{o}C for approximately 24 hours. Heat the Asphalt binder for an hour to reach its liquid state and weigh the % to be added to the heated aggregate mix. Mix them in the mixer
    c. Leave the mix in the oven for 4 hours to ensure aging of the sample.
    d. Also at the same time, leave the gyratory mold and base plates in the oven to be heated up.
    e. Turn on the Super pave Gyratory Compactor and the computer associated with it. Pull up the Excel program with the Samples in the columns
    f. After time is due, remove the 6" diameter SGC mold and fill the mold with the asphalt mix. Place two paper platers on the base and top plate.
    g. Set the SGC pressure to 600kPa, the angle of gyration to 1.25^{o} and speed of gyration is standardized at 30 rpm.
    h. Place the mold with the asphalt mix inside and hit Enter on SGC screen to start the compactor. At the same time, select Specimen 1 in Excel sheet and click "Launch Comm. Module". The data starts generating in the column.
    i. After achieved maximum gyrations of 205, extract the compacted mold and place it between piers to extract the sample.
    j. Make a note of the %AC used for the sample on the paper circular disc and repeat the process for each sample at its varying asphalt content percentage.
    k. Calculate the optimum trial blend. The end result is a creation of 4 sets of 2 samples with optimum trial blend, as per Marshall Mix Design with 4600 grams of aggregate. % AC will equal the estimated binder content minus 0.5%, as is, plus 0.5%, and plus 1.0%.
    l. Clean the work area and calculate optimum asphalt content.

2. Specific Gravity of Coarse Aggregate:
    a. Weigh out 2000 grams sample of coarse aggregate blend in question and submerge overnight for nearly 15 hours.
    b. Determine submerged mass.
    c. Dry sample with a towel such that no film remains on the surface of the aggregate. Determine the mas and is called as Saturated Surface Dry mass.
    d. Oven-dry the sample overnight at 110ºC and determine the mass.
3. Specific Gravity of Fine Aggregate:
    a. Weigh out 1000 grams of the fine aggregate blend in question.
    b. Determine mass of pycnometer filled with water at 23ºC
    c. Determine mass of pycnometer filled with the aggregate and water to the calibration line.
    d. Allow surface of sample to dry using a blow dry until the sample just fails the cone and determine the SSD mass.
    e. Combine Gsb of Fine and Coarse values
4. Specific Gravity of Compacted Mix
    a. Determine the compacted sample weight in air after being retrieved from the SGC mold.
    b. Submerge the sample for 15 to 30 minutes after the sample reaches room temperature. Determine the submerged mass.
    c. Remove the sample from the water and dry the surface using a cloth so that no water film stays on the surface. Determine the mass again.
    d. Multiply by a correction factor as needed for the temperature.
5. Specific gravity of Loose Mix
    a. Retrieve 1000 grams of HMA sample prior to its compaction. This requires careful planning of preparing a HMA over 5100 grams for every sample and separating 500 grams from each.
    b. Cure the sample for 2 hours at 230ºF and let it cool to 25ºC.
    c. Weigh the container to be used
    d. Fill the container with water and make a note of the weight
    e. Add the sample to the water in the container.
    f. Place the container with the sample on the vibratory device. Fasten the lid and start the vibratory device making sure the vacuum is attached.
    g. Let it run for 15 minutes so that the air between the HMA mixtures is removed.
    h. Weight the container with the water and sample.

Data collected:

Densification Data for Blend 1, 4.5% Asphalt Content								Densification Data for Blend 1, 4.5% Asphalt Content						
Specimen 1								Specimen 2						
Wm=mass of dry specimen for 4.5 % A.C = 4705.5 gms								Wm=mass of dry specimen for 4.5 % A.C = 4710.4 gms						
Bulk Specific Gravity of stones=Gsb= 2.55								Bulk Specific Gravity of stones=Gsb= 2.55						
Aggregate Content = Ps= 95.5%								Aggregate Content = Ps= 95.5%						
Specific Gravity of mixture measured= Gmm (meas)= 2.58								Specific Gravity of mixture measured= Gmm (meas)= 2.58						
Bulk Specific Gravity of mixture = Gmb (meas) = 2.39								Bulk Specific Gravity of mixture = Gmb (meas) = 2.33						
%Gmm@N(ini) = 92.64														
%Gmm@N(des) =Average %Gmm = 91.5														
Va=% Air Voids= 100- %Gmm@N(des)= 100-91.5= 8.5														
VMA %= 100-(%Gmm@N(des)*Gmb (meas)*Ps)/Gsb = 100-(91.5*2.58*0.955)/2.55 = 11.59														
VFA %= 100*(VMA-Va)/VMA= 100*(11.59-8.5)/11.59 = 26.66														
Gyrations	Ht, mm	V_{mx}	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$		Gyrations	Ht, mm	Vmx	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$
5	123.6	2183.1	2.155	1.109	2.390	92.64		5	124.6	2200.748	2.14	1.09	2.33	90.31
9=N(ini)	121.1	2138.9	2.200	1.086	2.390	92.64		9	122.0	2154.825	2.19	1.07	2.33	90.31
10	120.6	2130.1	2.209	1.082	2.390	92.64		10	121.5	2145.994	2.19	1.06	2.33	90.31
15	118.9	2100.1	2.241	1.067	2.390	92.64		15	119.8	2115.968	2.23	1.05	2.33	90.31
20	117.8	2080.6	2.262	1.057	2.390	92.64		20	118.6	2094.773	2.25	1.04	2.33	90.31
30	116.3	2054.1	2.291	1.043	2.390	92.64		30	117.2	2070.045	2.28	1.02	2.33	90.31
40	115.4	2038.3	2.309	1.035	2.390	92.64		40	116.2	2052.383	2.30	1.02	2.33	90.31
50	114.7	2025.9	2.323	1.029	2.390	92.64		50	115.5	2040.019	2.31	1.01	2.33	90.31
60	114.2	2017.1	2.333	1.024	2.390	92.64		60	114.9	2029.421	2.32	1.00	2.33	90.31
70	113.8	2010.0	2.341	1.021	2.390	92.64		70	114.5	2022.356	2.33	1.00	2.33	90.31
80	113.4	2002.9	2.349	1.017	2.390	92.64		80	114.1	2015.291	2.34	1.00	2.33	90.31
90	113.2	1999.4	2.353	1.016	2.390	92.64		90	113.9	2011.759	2.34	1.00	2.33	90.31
100	112.9	1994.1	2.360	1.013	2.390	92.64		100	113.6	2006.46	2.35	0.99	2.33	90.31
125	112.4	1985.3	2.370	1.008	2.390	92.64		125	113.1	1997.629	2.36	0.99	2.33	90.31
126=N(des)	112.4	1985.3	2.370	1.008	2.390	92.64		126	113.1	1997.629	2.36	0.99	2.33	90.31
149	112.1	1980.0	2.377	1.006	2.390	92.64		149	112.7	1990.564	2.37	0.98	2.33	90.31
150	112.1	1980.0	2.377	1.006	2.390	92.64		150	112.7	1990.564	2.37	0.98	2.33	90.31
200	111.6	1971.1	2.387	1.001	2.390	92.64		200	112.2	1981.733	2.38	0.98	2.33	90.31
204=N(max)	111.6	1971.1	2.387	1.001	2.390	92.64		204	112.1	1979.966	2.38	0.98	2.33	90.31
		AVERAGE % Gmm =(0.93+0.90)/2 = 0.915= 91.5%												

Table C-1: Densification Data for 4.5 % Asphalt Content

Table C-2: Densification Data for 5.0 % Asphalt Content

Densification Data for Blend 2, 5.0% Asphalt Content	Densification Data for Blend 2, 5.0% Asphalt Content
Specimen 1	Specimen 2
Wm=mass of dry specimen for 5.0 % A.C = 4715.4 gms	Wm=mass of dry specimen for 5.0 % A.C = 4701.3 gms
Bulk Specific Gravity of stones=Gsb= 2.55	Bulk Specific Gravity of stones=Gsb= 2.55
Aggregate Content = Ps= 95%	Aggregate Content = Ps= 95%
Specific Gravity of mixture measured= Gmm (meas)= 2.489	Specific Gravity of mixture measured= Gmm (meas)= 2.489
Bulk Specific Gravity of mixture = Gmb (meas) = 2.371	Bulk Specific Gravity of mixture = Gmb (meas) = 2.369

%Gmm@N(ini) = 95.26

%Gmm@N(des) =Average %Gmm = 95.22

Va=% Air Voids= 100- %Gmm@N(des)= 100-95.22=4.78

VMA %= 100-(%Gmm@N(des)*Gmb (meas)*Ps)/Gsb = 100-(95.22*2.489*0.95)/2.55 =11.71

VFA %= 100*(VMA-Va)/VMA= 100*(11.71-4.78)/11.71 = 59.18

Gyrations	Ht, mm	Vmx	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$		Gyrations	Ht, mm	Vmx	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$
5	125.7	2220.176	2.12	1.12	2.371	95.26		5	124.7	2202.5	2.13	1.11	2.369	95.18
9	123.2	2176.02	2.17	1.09	2.371	95.26		9	122.2	2158.4	2.18	1.09	2.369	95.18
10	122.7	2167.189	2.18	1.09	2.371	95.26		10	121.8	2151.3	2.19	1.08	2.369	95.18
15	121.0	2137.163	2.21	1.07	2.371	95.26		15	120.2	2123.0	2.21	1.07	2.369	95.18
20	119.9	2117.734	2.23	1.06	2.371	95.26		20	119.1	2103.6	2.23	1.06	2.369	95.18
30	118.4	2091.24	2.25	1.05	2.371	95.26		30	117.6	2077.1	2.26	1.05	2.369	95.18
40	117.5	2075.344	2.27	1.04	2.371	95.26		40	116.7	2061.2	2.28	1.04	2.369	95.18
50	116.8	2062.98	2.29	1.04	2.371	95.26		50	116.1	2050.6	2.29	1.03	2.369	95.18
60	116.3	2054.149	2.30	1.03	2.371	95.26		60	115.6	2041.8	2.30	1.03	2.369	95.18
70	115.8	2045.318	2.31	1.03	2.371	95.26		70	115.1	2033.0	2.31	1.02	2.369	95.18
80	115.5	2040.019	2.31	1.03	2.371	95.26		80	114.8	2027.7	2.32	1.02	2.369	95.18
90	115.2	2034.72	2.32	1.02	2.371	95.26		90	114.6	2024.1	2.32	1.02	2.369	95.18
100	115.0	2031.188	2.32	1.02	2.371	95.26		100	114.3	2018.8	2.33	1.02	2.369	95.18
125	114.5	2022.356	2.33	1.02	2.371	95.26		125	113.9	2011.8	2.34	1.01	2.369	95.18
126	114.4	2020.59	2.33	1.02	2.371	95.26		126	113.9	2011.8	2.34	1.01	2.369	95.18
149	114.1	2015.291	2.34	1.01	2.371	95.26		149	113.5	2004.7	2.35	1.01	2.369	95.18
150	114.1	2015.291	2.34	1.01	2.371	95.26		150	113.5	2004.7	2.35	1.01	2.369	95.18
200	113.6	2006.46	2.35	1.01	2.371	95.26		200	113.0	1995.9	2.36	1.01	2.369	95.18
204	113.5	2004.694	2.35	1.01	2.371	95.26		204	113.0	1995.9	2.36	1.01	2.369	95.18

AVERAGE % Gmm =(95.26+95.18)/2 = 95.22%

Table C-3: Densification Data for 5.5 % Asphalt Content

Densification Data for Blend 1, 4.5% Asphalt Content								Densification Data for Blend 1, 4.5% Asphalt Content						
Specimen 1								Specimen 2						
Wm=mass of dry specimen for 5.5 % A.C = 4748.1 gms								Wm=mass of dry specimen for 5.5 % A.C = 4992.0 gms						
Wm=mass of dry specimen for 4.5 % A.C = 4705.5 gms								Wm=mass of dry specimen for 4.5 % A.C = 4710.4 gms						
Bulk Specific Gravity of stones=Gsb= 2.55								Bulk Specific Gravity of stones=Gsb= 2.55						
Aggregate Content = Ps= 94.5%								Aggregate Content = Ps= 94.5%						
Specific Gravity of mixture measured= Gmm (meas)= 2.528								Specific Gravity of mixture measured= Gmm (meas)= 2.528						
Bulk Specific Gravity of mixture = Gmb (meas) = 2.44								Bulk Specific Gravity of mixture = Gmb (meas) = 2.42						
%Gmm@N(ini) = 96.52														
%Gmm@N(des) =Average %Gmm = 96.12														
Va=% Air Voids= 100- %Gmm@N(des)= 100-96.12= 3.88														
VMA %= 100-(%Gmm@N(des)*Gmb (meas)*Ps)/Gsb = 100-(96.12*2.528*0.945)/2.55 =9.96														
VFA %= 100*(VMA-Va)/VMA= 100*(9.96-3.88)/9.96 =61.04														
Specimen 1								Specimen 2						
Gyrations	Ht, mm	V_{mx}	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$		Gyrations	Ht, mm	V_{mx}	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$
5	123.1	2174.3	2.18	1.12	2.44000	96.52		5	128.7	2273.2	2.20	1.10	2.42	95.73
9	120.6	2130.1	2.23	1.09	2.44000	96.52		9	126.1	2227.2	2.24	1.08	2.42	95.73
10	120.2	2123.0	2.24	1.09	2.44000	96.52		10	125.7	2220.2	2.25	1.08	2.42	95.73
15	118.4	2091.2	2.27	1.07	2.44000	96.52		15	124.0	2190.2	2.28	1.06	2.42	95.73
20	117.3	2071.8	2.29	1.06	2.44000	96.52		20	122.8	2169.0	2.30	1.05	2.42	95.73
30	115.8	2045.3	2.32	1.05	2.44000	96.52		30	121.3	2142.5	2.33	1.04	2.42	95.73
40	114.8	2027.7	2.34	1.04	2.44000	96.52		40	120.4	2126.6	2.35	1.03	2.42	95.73
49	114.2	2017.1	2.35	1.04	2.44000	96.52		49	119.7	2114.2	2.36	1.02	2.42	95.73
50	114.1	2015.3	2.36	1.04	2.44000	96.52		50	119.7	2114.2	2.36	1.02	2.42	95.73
60	113.6	2006.5	2.37	1.03	2.44000	96.52		60	119.1	2103.6	2.37	1.02	2.42	95.73
70	113.1	1997.6	2.38	1.03	2.44000	96.52		70	118.7	2096.5	2.38	1.02	2.42	95.73
80	112.8	1992.3	2.38	1.02	2.44000	96.52		80	118.4	2091.2	2.39	1.01	2.42	95.73
90	112.5	1987.0	2.39	1.02	2.44000	96.52		90	118.1	2085.9	2.39	1.01	2.42	95.73
100	112.3	1983.5	2.39	1.02	2.44000	96.52		100	117.8	2080.6	2.40	1.01	2.42	95.73
125	111.8	1974.7	2.40	1.01	2.44000	96.52		125	117.4	2073.6	2.41	1.01	2.42	95.73
126	111.8	1974.7	2.40	1.01	2.44000	96.52		126	117.4	2073.6	2.41	1.01	2.42	95.73
150	111.4	1967.6	2.41	1.01	2.44000	96.52		150	117.0	2066.5	2.42	1.00	2.42	95.73
200	111.0	1960.5	2.42	1.01	2.44000	96.52		200	116.6	2059.4	2.42	1.00	2.42	95.73
204	110.9	1958.8	2.42	1.01	2.44000	96.52		204	116.6	2059.4	2.42	1.00	2.42	95.73
				AVERAGE % Gmm =(96.52+95.73)/2 =96.12%										

Table C-4: Densification Data for 6.0 % Asphalt Content

Densification Data for Blend 2, 5.0% Asphalt Content	Densification Data for Blend 2, 5.0% Asphalt Content
Specimen 1	Specimen 2
Wm=mass of dry specimen for 5.0 % A.C = 4715.4 gms	Wm=mass of dry specimen for 5.0 % A.C = 4701.3 gms
Bulk Specific Gravity of stones=Gsb= 2.55	Bulk Specific Gravity of stones=Gsb= 2.55
Aggregate Content = Ps= 94%	Aggregate Content = Ps= 94%
Specific Gravity of mixture measured= Gmm (meas)= 2.49	Specific Gravity of mixture measured= Gmm (meas)= 2.49
Bulk Specific Gravity of mixture = Gmb (meas) = 2.41	Bulk Specific Gravity of mixture = Gmb (meas) = 2.415
%Gmm@N(ini) = 96.79	
%Gmm@N(des) =Average %Gmm = 96.89	
Va=% Air Voids= 100- %Gmm@N(des)= 100-96.89=3.11	
VMA %= 100-(%Gmm@N(des)*Gmb (meas)*Ps)/Gsb = 100-(96.89*2.49*0.94)/2.55 =11.07	
VFA %= 100*(VMA-Va)/VMA= 100*(11.07-3.11)/11.07 = 71.90	

Specimen 1								Specimen 2						
Wm=mass of dry specimen for 6.0 % A.C = 4704.2 gms								Wm=mass of dry specimen for 6.0 % A.C = 4704.8 gms						
Gyrations	Ht, mm	V_{mx}	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$		Gyrations	Ht, mm	V_{mx}	Gmb (est)	"C"	G_{mb} (corr)	$\%G_{mm}$
5	122.8	2169.0	2.17	1.11	2.41	96.79		5	123.2	2176.0	2.16	1.12	2.415	96.99
9	120.3	2124.8	2.21	1.09	2.41	96.79		9	120.7	2131.9	2.21	1.09	2.415	96.99
10	119.9	2117.7	2.22	1.08	2.41	96.79		10	120.3	2124.8	2.21	1.09	2.415	96.99
15	118.2	2087.7	2.25	1.07	2.41	96.79		15	118.6	2094.8	2.25	1.08	2.415	96.99
20	117.1	2068.3	2.27	1.06	2.41	96.79		20	117.4	2073.6	2.27	1.06	2.415	96.99
30	115.6	2041.8	2.30	1.05	2.41	96.79		30	116.0	2048.9	2.30	1.05	2.415	96.99
40	114.7	2025.9	2.32	1.04	2.41	96.79		40	115.0	2031.2	2.32	1.04	2.415	96.99
49	114.1	2015.3	2.33	1.03	2.41	96.79		49	114.4	2020.6	2.33	1.04	2.415	96.99
50	114.0	2013.5	2.34	1.03	2.41	96.79		50	114.3	2018.8	2.33	1.04	2.415	96.99
60	113.5	2004.7	2.35	1.03	2.41	96.79		60	113.8	2010.0	2.34	1.03	2.415	96.99
70	113.1	1997.6	2.35	1.02	2.41	96.79		70	113.4	2002.9	2.35	1.03	2.415	96.99
80	112.8	1992.3	2.36	1.02	2.41	96.79		80	113.0	1995.9	2.36	1.02	2.415	96.99
90	112.5	1987.0	2.37	1.02	2.41	96.79		90	112.8	1992.3	2.36	1.02	2.415	96.99
100	112.3	1983.5	2.37	1.02	2.41	96.79		100	112.5	1987.0	2.37	1.02	2.415	96.99
125	111.8	1974.7	2.38	1.01	2.41	96.79		125	112.1	1980.0	2.38	1.02	2.415	96.99
126	111.8	1974.7	2.38	1.01	2.41	96.79		126	112.1	1980.0	2.38	1.02	2.415	96.99
150	111.5	1969.4	2.39	1.01	2.41	96.79		150	111.7	1972.9	2.38	1.01	2.415	96.99
200	111.0	1960.5	2.40	1.00	2.41	96.79		200	111.2	1964.1	2.40	1.01	2.415	96.99
204	111.0	1960.5	2.40	1.00	2.41	96.78714859		204	111.2	1964.1	2.40	1.01	2.415	96.99
AVERAGE % Gmm =(96.79+96.99)/2 =96.89%														

Data analysis

Results and graphs of mix properties versus asphalt content is shown in Figures C1 to C3. All calculations were performed in accordance to AASHTO specifications.

Results shown from the graphs tell us that optimum binder content will be 5.4 % as it passed all the checks for % Air voids i.e. 4.0, % VMA i.e. 10.5 which is greater than 10% for 37.5 mm aggregate size materials and % VFA i.e. 61% which is greater than 60-75 % range of Design VFA.

Table C-5: Mix Volumetric Properties at N (des)

MIX Volumetric Properties at N(des)			
% AC	%Air Voids	%VMA	%VFA
4.5	8.5	11.59	26.66
5	4.78	11.71	59.18
5.5	3.88	9.96	61.04
6	3.11	11.07	71.9

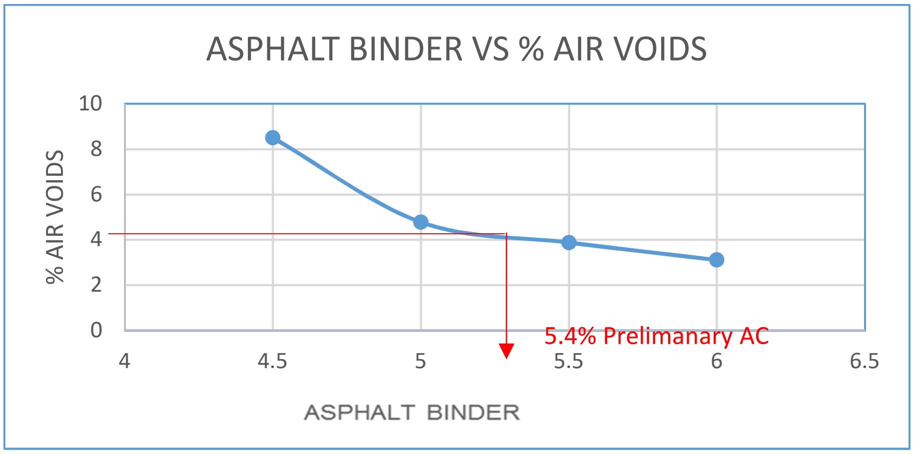


Figure C-1: Asphalt Binder VS Air Voids

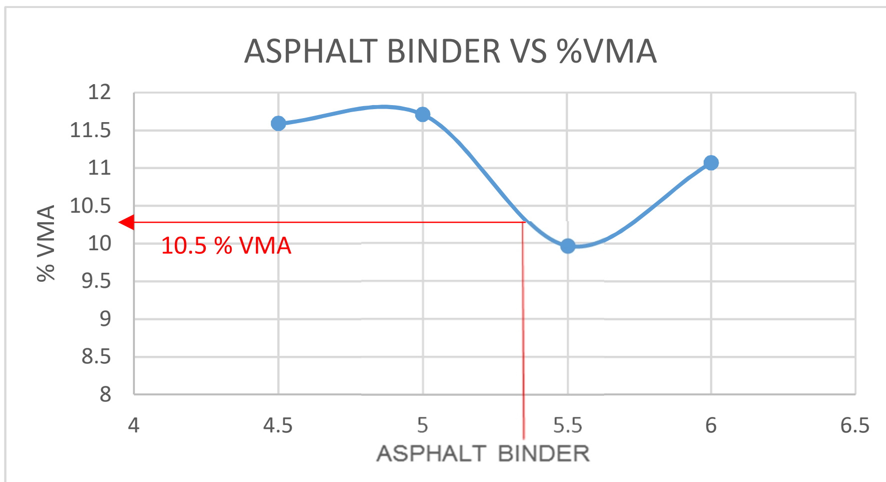


Figure C-2: Asphalt Binder VS %VMA

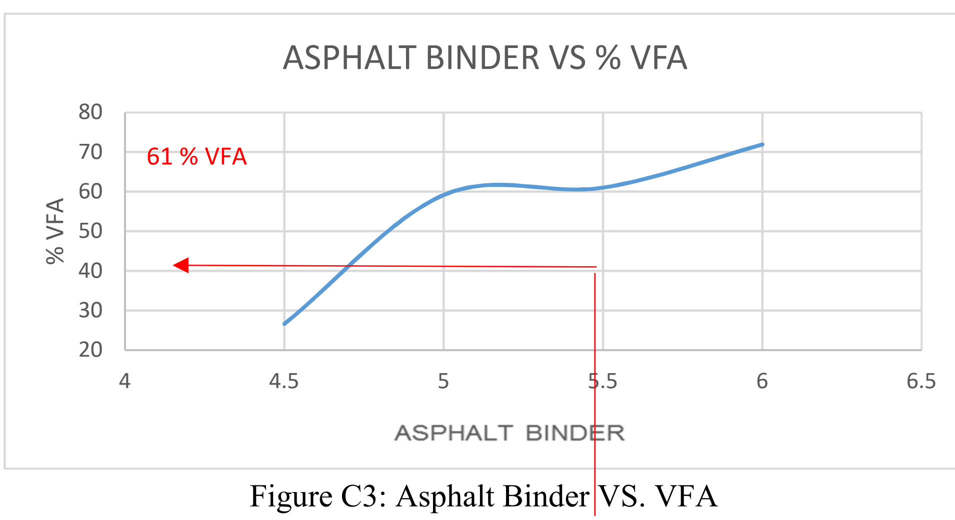


Figure C3: Asphalt Binder VS. VFA

Table C-6: Design Mixture Properties at 5.4% Binder Content

Design Mixture Properties at 5.4% Binder Content			
Mix Property	Result	Criteria	Checks
Air Voids %	4	4	OK
VMA %	10.5	10 Min	OK
VFA %	61	60-75	OK

Possible sources of error

Given the large scope of this lab, the inexperience of the students performing the lab, and the multiple attempts taken and mixes prepared to arrive at proper numbers, it is to be expected that our results will suffer from at least small error. Multiple attempts at achieving reasonable numbers means that values such as Gsb, Gmb and Gmm that were used as if they were from one sample may actually have represented samples with slight differences. With different equipment and possibly by different procedures than those used in the URI asphalt labs, we might achieve different results. Error derived from these circumstances is expected to be small.

Conclusion

In conclusion, using the optimum trial gradation provided in class we were able to determine optimum asphalt content though which might have certain scope of error.

Appendix D

Determination of Creep Compliance Test of Hot Mix Asphalt (HMA)

Appendix D

Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using Indirect Tensile Test Device

Introduction

Federal Highway Administration (FHWA) and Transportation Research Board (TRB) have been promoting new approach to design pavement structures, i.e., Mechanistic Empirical Pavement Design Guide (MEPDG). With the Rhode Island Department of Transportation (RIDOT) beginning to fully implement the newer AASHTOWare Pavement ME Design (Pavement ME) to design new and rehabilitated pavement structures, there is a need to determine material input parameters, e.g., creep compliance and dynamic modulus of RI Hot Mix Asphalt (HMA) and other asphaltic mixtures of flexible pavements. The American Association of Highway and Transportation Officials (AASHTO) provided a test method T322-03 to determine the creep compliance and strength of HMA which is needed as input to the Pavement ME.

Creep Compliance is defined as time-dependent strain per unit stress while indirect tensile (IDT) strength is the capacity to withstand the indirect tensile load which is induced from the compression load along the diameter of circular specimens. Both properties are determined using the IDT test; i.e., a cylindrically shaped specimen is loaded in compression across its diameter thus indirectly causing tension in opposite directions perpendicular to and beginning at the line of loading. As HMA is considered a visco-elastic material, creep compliance and tensile strength are not only dependent on the HMA mix constituent properties, constituent proportions, and compacted mix properties (e.g., % air voids), but also highly temperature dependent. Additionally, creep compliance is dependent on the load/unload duration and tensile strength is dependent on loading rate.

RIDOT contracted with the University of New Hampshire and Villanova University to determine creep compliance and tensile strength of asphaltic mixes used in RI Route 165, those values were not adequate to run Pavement ME by a research team of the University of Rhode Island (URI). Thus, URI team decided to perform the test to generate appropriate input parameters for Pavement ME and to calculate the thermal (low-temperature) cracking distress.

Specimen Preparation

Hot Mix Asphalt specimens having 6 in. diameter and 4.3 in. (110 mm) of height were prepared in Superpave gyratory compactor (SGC) by using Rhode Island Class I specification and P.J. Keating material. Four different asphalt binder ratio was used to prepare the specimens i.e. (4.5%, 5.0%, 5.5%, and 6.0%) to calculate the optimum binder content (See Appendix C). After determining optimum binder content cylindrical specimens were sawed to suitable heights of 38 to 50 mm (1.5 to 2 in.) from their original heights i.e. 110 mm (4.3 in.) thus making two specimens from one cylindrical specimen. Sawing was accomplished by using wet sawing, an appropriate time frame of at least 24 hours was necessary to allow the specimens to dry before testing.

The sawing requires a constant water flow to cool the blade and the sample while cutting. This will also help to reduce the noise and binds most the sawed chips. However, constant water flow also allows water to pervade the specimen. Since testing is conducted at temperature far below the freezing point of water, the specimen's performance is highly susceptible to any water content.

The cylindrical samples have a height of about 110 mm (4.3 in.), thus two specimens with the required height could be produced. Due to the quality of the saw, a high level of accuracy could be maintained.

In total, eight specimens for each asphalt binder content were produced with the thickness in the range of 41 to 44 mm. These met the requirement of the procedure of T 322, which are 38 to 50 mm (AASHTO 2011).

It was observed that the different behavior of the materials could be seen even during specimen preparation. The fine materials of the P.J. keating were less strongly integrated into the material and therefore chipping was increased during sawing. In order to still obtain usable specimens, care had to be taken to saw the specimens fast enough to minimize wobbling of the blade and at the same time slow enough not to rip out particles instead of cutting through them.

Creep Compliance Testing:

Creep compliance test as per AASHTO T322-07 was conducted using Instron 5582 machine connected with Blue Hill 2 software. Blue Hill 2 software-controlled testing systems can perform variety of tension (pull), compression (push), flex (bend), cyclic, creep, and relaxation applications. In this case, creep compression relaxation method was used. A method according to AASHTO T322-07 specifications was created by using method tab on the home screen of Blue Hill 2 software. Geometric dimensions of specimen were inputted into the method tab. As creep compliance test is a non-destructive test, per AASHTO procedure a static load of fixed magnitude without impact to the specimen for at least 1,000 seconds must be applied and that load should produce a horizontal deformation of 0.00125 mm to 0.0190 mm for 150 mm (6 in.) diameter specimens. By using numerous trials hold criteria of compressive load in this case was selected as 1000 N with a rate of 30 mm/min. 1000 seconds of time was selected to run the complete analysis for each specimen.

Creep Compliance test generally consists of following five major steps.

- Step 1- Balance strain gauges and load cell
- Step 2- Lower loading ram and apply load < 1.0 kN (Noise)
- Step 3- Load> 1.0 KN and Increase load to desired level
- Step 4- Keep load constant for test duration of 1,000 seconds
- Step 5- Remove load and end the test

To follow the above steps strain gauges i.e., Linear Variable Differential Transducers (LVDTs) must be calibrated before using.

Calibration of LVDTs:

There were four LVDTs configured with Lucas Conditioners which further attached to the computer with versa channel. A mechanical setup was created with the help of Mr. Jim Brynes from Mechanical Engineering Department of URI as shown in Figure D-1. A voltmeter was connected to the versa channel to read the amount of voltage. Each LVDT should read ±10V within a range of 0.01 in. By using Lucas Conditioner manual and coarse fine buttons on the conditioners voltage was increased to desired level i.e. ±10 V. Zero voltage point was determined by screwing the screw gauge attached with the digital dial meter. Then by using the mechanical setup, it was made sure that with the increment of +0.01 in. LVDT should read exactly +10V and if the increment is in opposite direction form 0V i.e. -0.01 in. LVDT should read -10V.

Once correct voltage was obtained on both directions channels 1 to 4 were calibrated on Blue Hill 2 software by clicking on transducer setup menu by giving calibration point and extensometer range point which already set up by the screw gauge attached with dial meter.

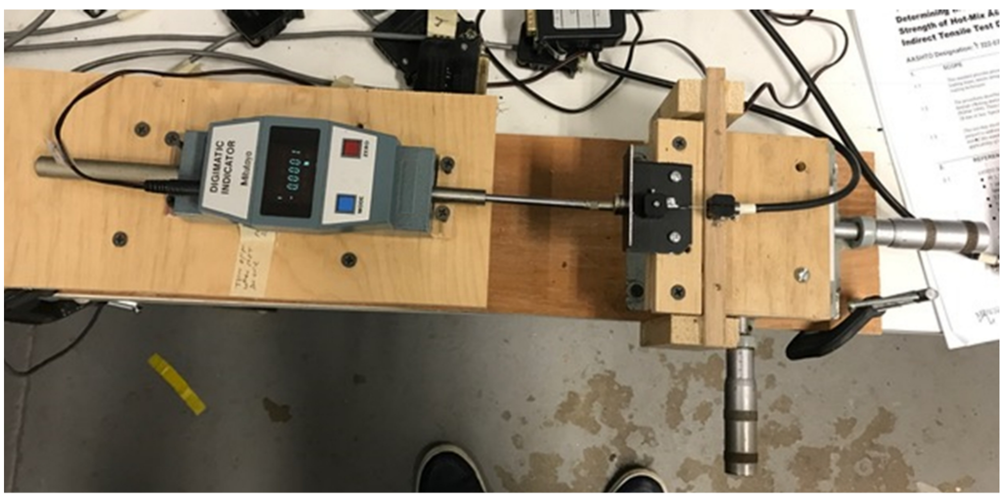


Figure D-1: A Mechanical Setup Created to Calibrate LVDTs

In the next step after calibration brass buttons on the cylindrical specimens were glued by using template to mount the LVDTs on both faces of the specimen as shown in Figure D-2.

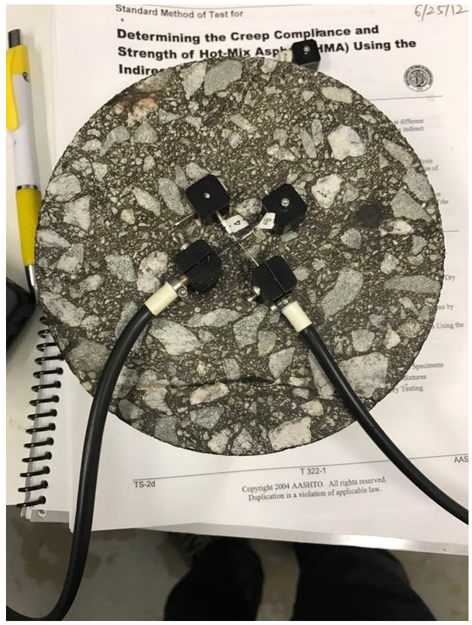


Figure D-2: LVDTs mounted on the face of the Specimen

After plugging the LVDTs, specimen was placed in the chamber of Instron 5582 machine carefully and the cables (related to computer) were connected with the all four channels mounted on both faces of the specimen.

Loading ram was lowered until it slightly above to the specimen as shown in Figure D-3.

Figure D-3: Specimen inside the Loading Frame Chamber of Instron Machine

For creep compliance test specimens, must be tested at three different temperatures i.e., 0^0C, -10^0C, and -20^0C. Temperature controller unit attached with Instron machine was responsible for maintaining temperature inside the chamber. Specimens were kept in the chamber for approximately 2 to 3 hours before testing so that specimens could attain the desired temperature (AASHTO 2011).

After maintaining temperature test was conducted by using Blue Hill 2 software for about 1000 seconds. A constant load was maintained throughout the test i.e. 1,000 N as shown in the Figure D-4.

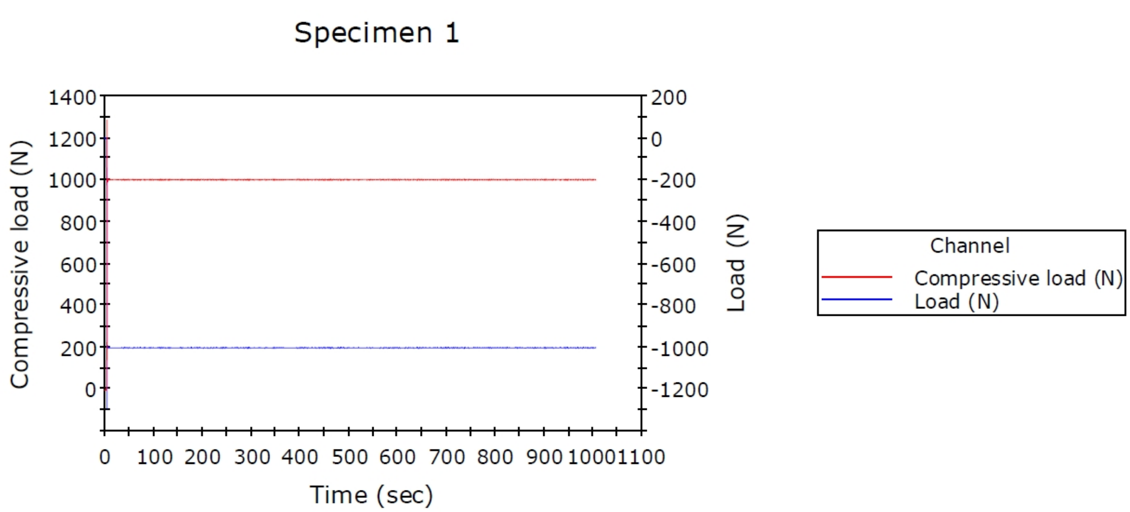


Figure D-4: Constant Static Load Applied to the Specimen

Table D-1: Specimen 1 Data at -20^{0}C to calculate Creep Compliance

Time	Extension	Load	Displacement (Channel 1)	Displacement (Channel 2)	Displacement (Channel 3)	Displacement (Channel 4)	Compressive extension
(sec)	(mm)	(N)	(mm)	(mm)	(mm)	(mm)	(mm)
0	-8.75432	-0.13177	0	0	0.00000	0.00000	8.75432
1	-9.25422	-0.20989	-0.001564	-0.000607	0.00096	0.00066	9.25422
2	-9.7543	-0.19475	-0.00951	-0.001206	0.00173	0.00091	9.7543
5	-11.2543	-17.639	-0.010467	-0.002383	0.00501	0.00511	11.2543
10	-11.4545	-999.882	-0.011482	-0.003442	0.00616	0.00811	11.45447
20	-11.4572	-999.513	-0.013621	-0.004548	0.00801	0.00904	11.45716
50	-11.4614	-999.837	-0.014419	-0.005312	0.00935	0.01027	11.46141
100	-11.4659	-999.782	-0.015486	-0.006398	0.01007	0.01212	11.46589
200	-11.473	-1,000.16	-0.016321	-0.007892	0.01089	0.01251	11.47295
400	-11.4824	-1,000.12	-0.017549	-0.008657	0.01123	0.01283	11.4824
600	-11.4893	-999.696	-0.018956	-0.009389	0.01205	0.01328	11.48933
800	-11.495	-1,000.35	-0.019674	-0.010420	0.01286	0.01452	11.49495
1,000.00	-11.4994	-999.264	-0.020521	-0.01068	0.01307	0.01508	11.49943

Figure D-5 also shows displacement (mm) for all four LVDTs vs testing time (sec).

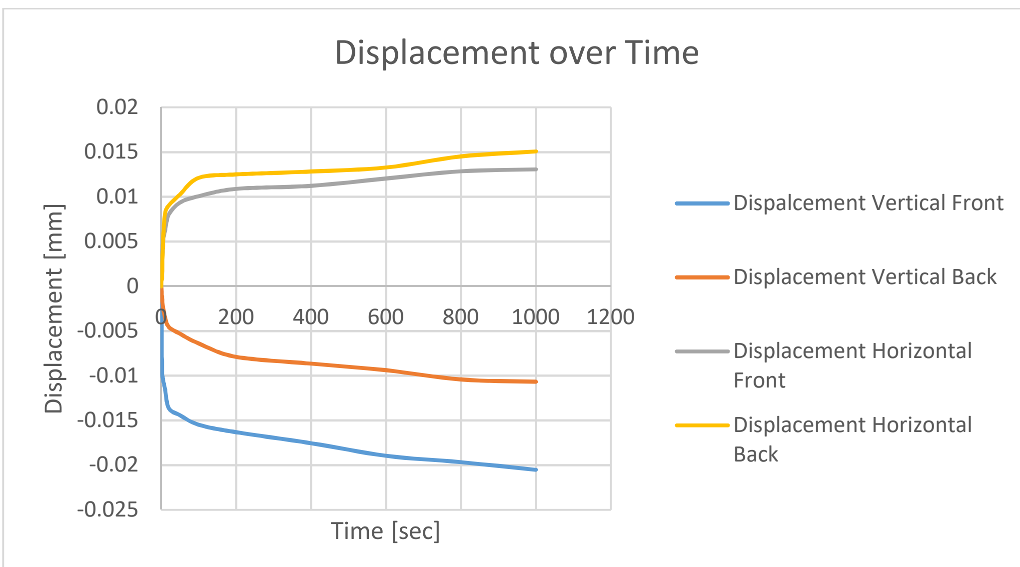


Figure D-5: Displacement in mm for Specimen 1 VS Testing Time in sec at -20ºC

The next step was the control of deflections in order to see if faulty data were recorded. Figure D-5 depicts the displacements of the 4 strain gauges attached to both faces of specimen 1 over the 1,000 seconds' test time, which includes the creep period as well as load adjustment and removal.

It can be observed that both horizontal deformations are approximately equal. This is a good result as it proves consistency within the specimen and probability of getting close to the true value is increased by calculating the average. The vertical displacements deviate a little more. This can be due to multiple reasons, starting from inhomogeneous material, influence of large or interlocked pieces of aggregates, or aggregate gaps on the path from the loading piston to the center of the specimen that are filled with binder or air voids. Also, sometimes the fault can lie within the strain gauges, although this is rather rare.

Subsequently the horizontal and vertical deformations of all specimens at the analyzed temperature were averaged and normalized in order to compare them. This can be accomplished by using equations from (AASHTO T 322-03)

$$\Delta X_{n,i,t} = \Delta X_{i,t} \cdot \frac{b_n}{b_{avg}} \cdot \frac{D_n}{D_{avg}} \cdot \frac{P_{avg}}{P_n}$$

$$\Delta Y_{n,i,t} = \Delta Y_{i,t} \cdot \frac{b_n}{b_{avg}} \cdot \frac{D_n}{D_{avg}} \cdot \frac{P_{avg}}{P_n}$$

where

$\Delta X_{n,i,t}$	normalized horizontal deformation of specimen n for face i at time t [mm]
$\Delta Y_{n,i,t}$	normalized vertical deformation of specimen n for face i at time t [mm]
$\Delta X_{i,t}$	measured horizontal deformation of specimen n for face i at time t [mm]
$\Delta Y_{i,t}$	measured vertical deformation of specimen n for face i at time t [mm]
b_n, D_n, P_n	thickness, diameter, creep load of specimen n
b_{avg}, D_{avg}, P_{avg}	average thickness, diameter, creep load of all replicate specimens at this temperature

Since all specimens have a diameter of 150 mm, the second fraction is 1. In the test method, ΔX and ΔY are treated as arrays. In this study, this is achieved by calculating single values in a table in the spreadsheet software. After executing these equations, normalized deformations are obtained that enable the user to directly compare the deflections of all three specimens to one another. Figure D-6 shows the normalized deflections of HMA specimen at -20^{0}C.

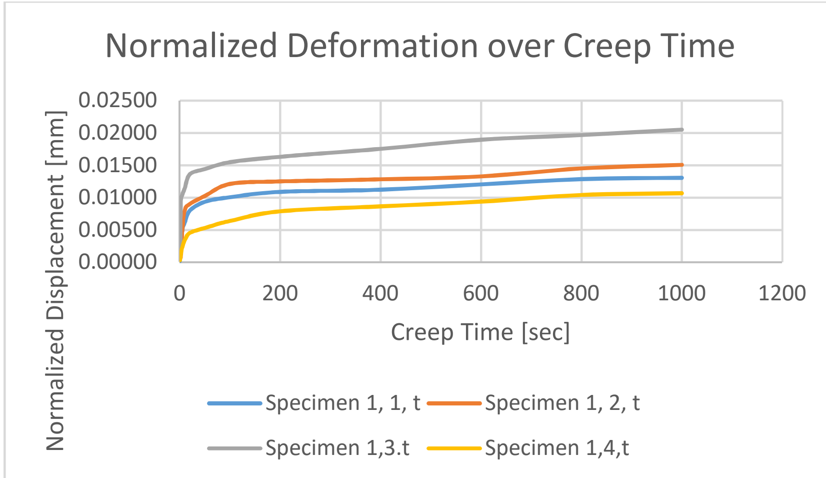


Figure D-6: Normalized Horizontal Deformation of Specimen 1 at -20^{0}C

The average horizontal and vertical deformations for every face are needed in order to determine the ratio of the horizontal to vertical deformations X/Y, Poisson's ratio, and a coefficient, C_{cmpl}, these are needed for the calculation of creep compliance. The average deformations occur after half the total creep time and are obtained using equations shown below.

$$\Delta X_{a,i} = \Delta X_{n,i,t_{mid}}$$

where

$\Delta X_{a,i}$	average horizontal deformation for face i
$\Delta X_{n,i,t}$	normalized horizontal deformation at a time corresponding to half the total creep test time for face i, here t = 500 s

The vertical deformations were obtained by applying the same calculations to the ΔY values. Then, the trimmed mean of the deflections ΔX_t and ΔY_t needed to be obtained. For this, the six $\Delta X_{a,I}$ and $\Delta Y_{a,I}$ values were ranked numerically and the highest and lowest values were disregarded. The average of the middle four values was determined according to Equation shown below.

$$\Delta X_t = \frac{\sum_{j=2}^{5} \Delta X_{r,j}}{4}$$

where

ΔX_t	trimmed mean of horizontal deformations
$\Delta X_{r,j}$	$\Delta X_{a,i}$ values in ascending order

The ratio of the horizontal to vertical deformation X/Y was computed according to Equation shown below.

$$\frac{X}{Y} = \frac{\Delta X_t}{\Delta Y_t}$$

Consequently, C_{cmpl} was determined by using following equations.

$$C_{cmpl} = 0.6354 \cdot \left(\frac{X}{Y}\right)^{-1} - 0.332$$

The above equation must be true if

$$\left[0.704 - 0.213\left(\frac{b_{avg}}{D_{avg}}\right)\right] \le C_{cmpl} \le \left[1.566 - 0.195\left(\frac{b_{avg}}{D_{avg}}\right)\right]$$

$$\nu = -0.10 + 1.480\left(\frac{X}{Y}\right)^2 - 0.778\left(\frac{b_{avg}}{D_{avg}}\right)^2\left(\frac{X}{Y}\right)^2$$

It may be noted that Poisson's ratio v should always be between 0.05 and 0.50. These calculations were carried out in a spread sheet program, i.e., (Microsoft Excel). These were performed for all temperatures. Table D-2 shows the results for HMA at -20⁰C.

Table D-2: Calculations prior to Creep Compliance at -20⁰C

-20°C			
ΔXa,1	0.011623	ΔYa,1	0.018259
ΔXa,2	0.01305	ΔYa,2	0.008989
ΔXt	0.01233	ΔYt	0.013624
$\frac{X}{Y}$=0.90			
0.644 <0.90 <1.511		**Check OK**	

Based on the trimmed mean of the deflections (deflection arrays) $\Delta X_{tm,t}$ with respect to variable time t following the same numerical ranking for the average deformations in Equation shown above, the creep compliance D(t) can finally be computed using Equation shown below.

$$D(t) = \frac{\Delta X_{tm,t} \cdot D_{avg} \cdot b_{avg}}{P_{avg} \cdot GL} \cdot C_{cmpl}$$

where

$D(t)$	creep compliance [1/TPa]
GL	gauge length (0.038 for 150 mm specimen)

The above formula allows the computation of the creep compliance for any time recorded in the present study every half-second. The AASHTOWare pavement ME Desgn requires the creep compliance only at certain time points. For greater precision of the requested data points $\Delta X_{tm,t}$ shown in Table D-3 below, Also, the used version of the program only allows the input of the results in US customary units, while the test method consistently uses SI units. Therefore, the creep compliance is firstly calculated in [1/GPa] (SI unit) since it results from the calculations above. Technically, the obtained unit was [1/TPa], but dividing by 10^3, the unit [1/GPa] was obtained. Then the conversion factor of $(145000)^{-1}$ was applied to obtain the customary unit of [/psi].

Table D-3: Creep Compliance of HMA at -20^{0}C for AASHTOWare ME Design

Creep Time t [s]	$\Delta X_{tm,t}$ [mm]	D(t) [1/Gpa]	D(t) [1/psi]
0	0.000305	7.82325E-05	5.40E-10
1	0.00081	0.000207765	1.43E-09
2	0.00132	0.00033858	2.34E-09
5	0.00506	0.00129789	8.96E-09
10	0.007135	0.001830128	1.26E-08
20	0.008525	0.002186663	1.51E-08
50	0.00981	0.002516265	1.74E-08
100	0.01109	0.002844585	1.96E-08

Similarly, all above calculations were performed for -10^{0}C and 0^{0}C and the results are shown below.

Table D-4 : Specimen 1 data at -10^{0}C from Software to calculate Creep Compliance

Time	Extension	Load	Displacement (Channel 1)	Displacement (Channel 2)	Displacement (Channel 3)	Displacement (Channel 4)	Compressive extension
(sec)	(mm)	(N)	(mm)	(mm)	(mm)	(mm)	(mm)
0	-7.89981	0.25056	0.00	0.00	0.00	0.00	7.89981
1	-8.39989	0.62981	-0.001463	-0.000543	0.000743	0.00054	8.39989
2	-8.89973	0.73177	-0.007530	-0.000987	0.001235	0.00090	8.89973
5	-10.39991	-0.24143	-0.009989	-0.001865	0.004856	0.00325	10.39991
10	-11.46553	-1,000.47	-0.011325	-0.002834	0.006232	0.00687	11.46553
20	-11.46846	-1,000.24	-0.012897	-0.004032	0.007862	0.00898	11.46846
50	-11.47163	-1,000.05	-0.013765	-0.005105	0.008789	0.00932	11.47163
100	-11.47385	-1,000.51	-0.015326	-0.006021	0.009657	0.01089	11.47385
200	-11.47456	-999.49334	-0.016281	-0.007532	0.010679	0.01172	11.47456
400	-11.47612	-999.42582	-0.016998	-0.008567	0.011130	0.01263	11.47612
600	-11.47594	-1,000.45	-0.017854	-0.008964	0.011960	0.01303	11.47594
800	-11.476	-1,000.97	-0.019582	-0.009986	0.012326	0.01368	11.476
1,000.00	-11.47767	-999.65734	-0.021251	-0.010456	0.012996	0.01495	11.47767

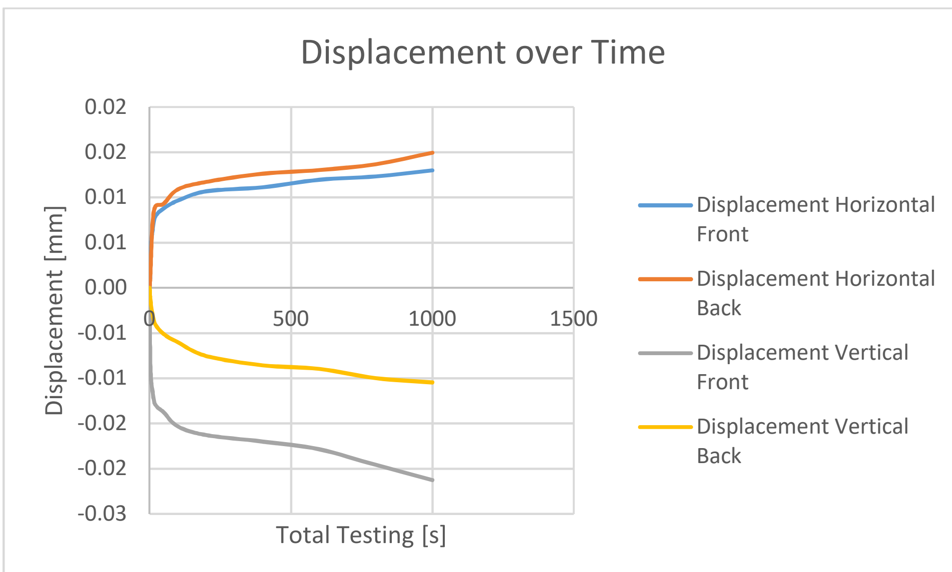


Figure D-7: Displacement in mm for Specimen 1 VS Testing Time in sec at -10ºC

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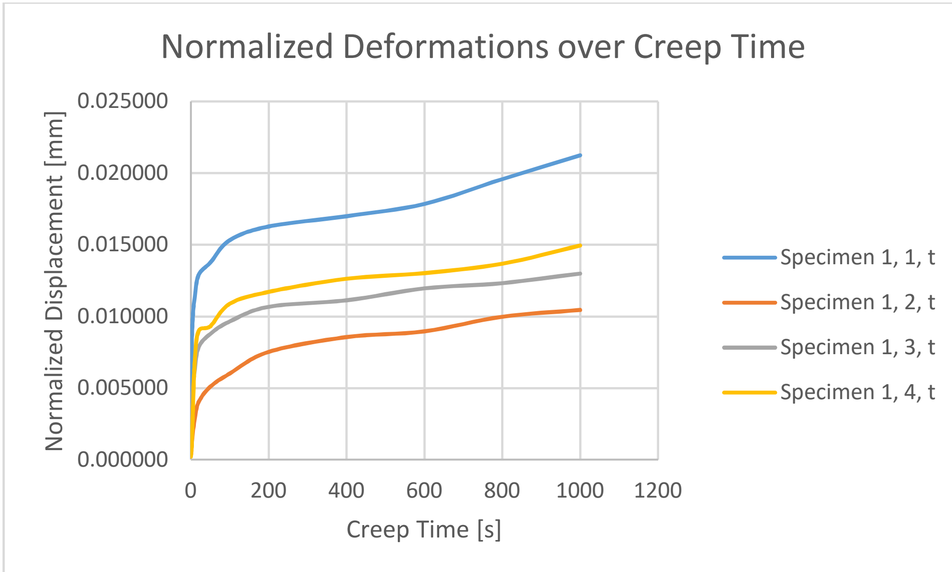


Figure D-8: Normalized Horizontal Deformation of Specimen 1 at -10^{0}C

Table D-5: Calculations prior to Creep Compliance at -10^{0}C

-10^{0}C			
ΔXa,1	0.01134	ΔYa,1	0.01732
ΔXa,2	0.01305	ΔYa,2	0.00876
ΔXt	0.01219	ΔYt	0.01304
$\frac{\Delta}{Y}$=0.93			
0.644 <0.93 <1.511		**Check OK**	

Table D-6: Creep Compliance of HMA at -10^{0}C for AASHTOWare ME Design

Creep Time t [s]	$\Delta X_{tm,t}$ [mm]	D(t) [1/Gpa]	D(t) [1/psi]
0	0.000417	0.000110526	7.63E-10
1	0.000641	0.000169897	1.17E-09
2	0.00106	0.000280953	1.94E-09
5	0.00405	0.001073453	7.41E-09
10	0.00655	0.001736078	1.20E-08
20	0.008421	0.002231986	1.54E-08
50	0.00905	0.002398703	1.66E-08
100	0.01027	0.002722064	1.88E-08

Table D-7: Specimen 1 data at 0^0C from Software to calculate Creep Compliance

Time	Extension	Load	Displacement (Channel 1)	Displacement (Channel 2)	Displacement (Channel 3)	Displacement (Channel 4)	Compressive extension
(sec)	(mm)	(N)	(mm)	(mm)	(mm)	(mm)	(mm)
0	-9.34833	0.74532	0.00	0.00	0.00	0.00	9.34833
1	-9.84817	-0.17254	-0.001387	-0.000503	0.000652	0.000430	9.84817
2	-10.34843	-0.69469	-0.006920	-0.000856	0.001154	0.000880	10.34843
5	-11.20946	-1,000.53	-0.008954	-0.001790	0.004526	0.003050	11.20946
10	-11.23206	-1,000.41	-0.010321	-0.002544	0.006023	0.006230	11.23206
20	-11.2534	-1,000.48	-0.012354	-0.003876	0.007523	0.008740	11.2534
50	-11.28658	-1,000.24	-0.013254	-0.005050	0.008432	0.009020	11.28658
100	-11.3172	-999.60361	-0.014765	-0.005968	0.009387	0.010320	11.3172
200	-11.35319	-1,000.10	-0.015978	-0.007354	0.009980	0.011230	11.35319
400	-11.39534	-1,000.48	-0.016640	-0.008356	0.010780	0.012210	11.39534
600	-11.42207	-999.85749	-0.017231	-0.008643	0.011560	0.012980	11.42207
800	-11.44216	-999.48617	-0.019156	-0.009756	0.012090	0.013320	11.44216
1,000.00	-11.45824	-1,000.10	-0.020989	-0.010110	0.012789	0.014450	11.45824

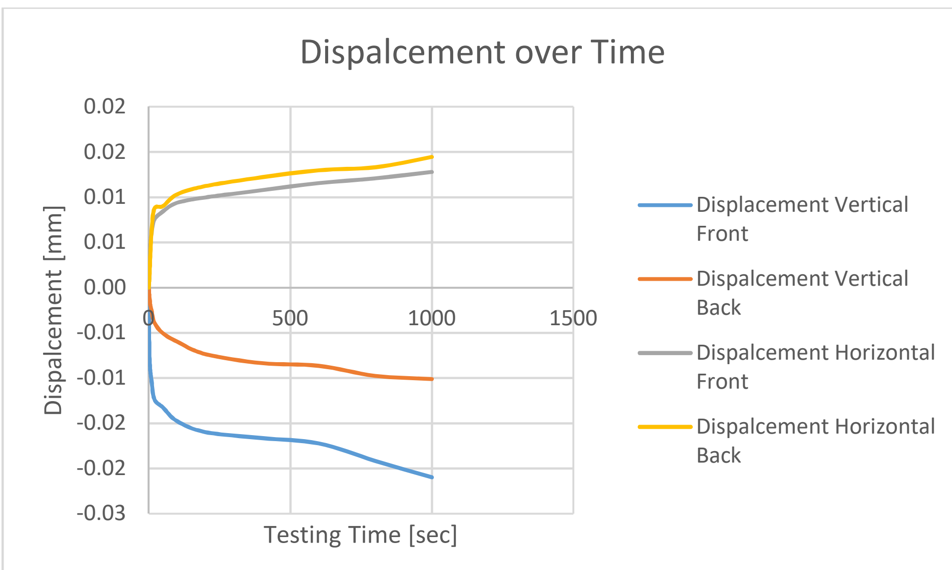


Figure D-9: Displacement in mm for Specimen 1 VS Testing Time in sec at 0^oC

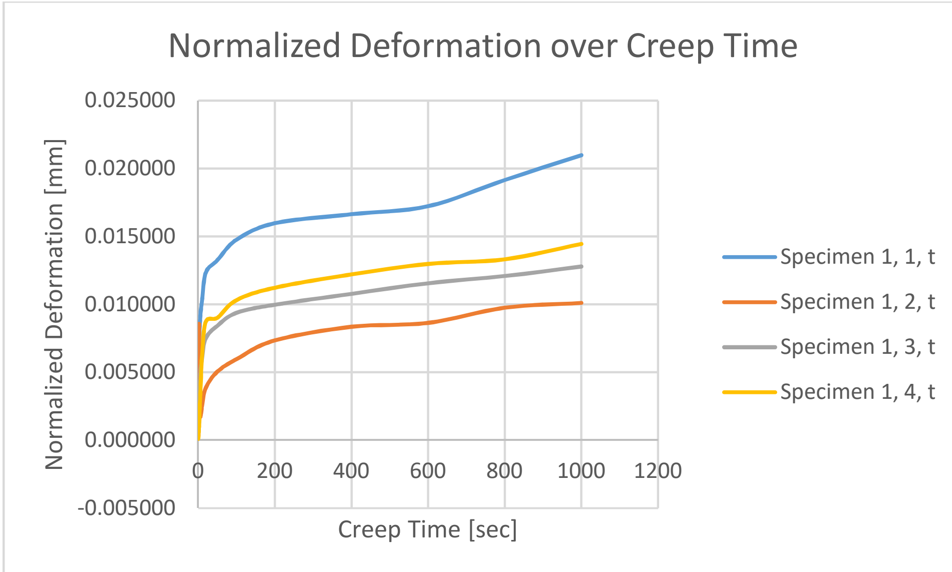


Figure D-10: Normalized Horizontal Deformation of Specimen 1 at 0^0C

Table D-8: Calculations prior to Creep Compliance at 0^0C

0^0C			
ΔXa,1	0.01189	ΔYa,1	0.01654
ΔXa,2	0.01367	ΔYa,2	0.00865
ΔXt	0.01278	ΔYt	0.01259
$\frac{\Delta}{Y}$=1.015			
0.644 <1.015 <1.511		**Check OK**	

Table D-9: Creep Compliance of HMA at 0^0C for AASHTOWare ME Design

Creep Time	$\Delta X_{tm,t}$ [mm]	D(t) [1/Gpa]	D(t) [1/psi]
0	0.000327	0.00093195	6.43E-09
1	0.000541	0.000154185	1.06E-09
2	0.001017	0.000289845	2.00E-09
5	0.003788	0.00107958	7.45E-09
10	0.006126	0.00174591	1.20E-08
20	0.0081315	0.002317478	1.60E-08
50	0.008726	0.00248691	1.72E-08
100	0.009853	0.002808105	1.94E-08

Appendix E

Determination of Dynamic Modulus Hot Mix Asphalt (HMA)

Appendix E

Determining Dynamic Modulus of Hot Mix Asphalt (HMA)

Introduction

The dynamic (complex) modulus of a visco-elastic test is a response developed under sinusoidal loading conditions. It is a true complex number as it contains both a real and imaginary component of the modulus and is normally identified by E* (or G*). In visco-elastic theory, the absolute value of the complex modulus |E*| is the Dynamic Modulus. In the general literature, however, the term, "Dynamic Modulus", is often used to denote any type of modulus that has been determined under "non-static" load conditions.

For linear visco-elastic materials such as HMA mixtures, the stress-strain relationship under a continuous sinusoidal loading is defined by its complex dynamic modulus (E*). This is a complex number that relates stress to strain for linear visco-elastic materials subjected to continuously applied sinusoidal loading in the frequency domain. The complex modulus is defined as the ratio of the angular load frequency, ω, $\delta = \delta o \sin(\omega t)$ and the amplitude of the sinusoidal strain $\varepsilon = \varepsilon o \sin(\omega t - ø)$, at the same time and frequency, that results in a steady state response.

$$E^* = \delta/\varepsilon = \delta_o e\ i\omega\ t/\varepsilon_o e^{\ i(\omega\ t - ø)} = \delta_o \sin\omega t/\varepsilon_o \sin(\omega t - ø)$$

Where, δ_o = peak (maximum) stress,

ε_o = peak (maximum) strain,

ø = phase angle, degrees,

ω = angular velocity,

t = time, seconds, and

i = imaginary component of the complex modulus.

Mathematically, the dynamic modulus is defined as the absolute value of the complex modulus, or:

$$|E^*| = \delta_o/\varepsilon_o$$

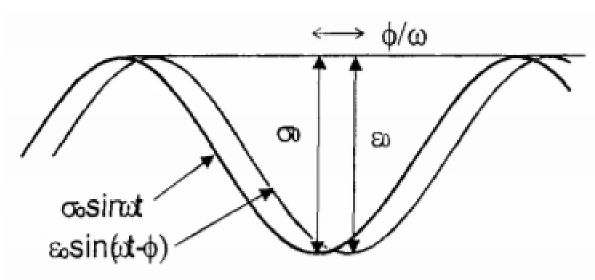


Figure E-1: Dynamic Modulus Test Curve

For a pure elastic material, ø = 0, and it is observed that the complex modulus (E*) is equal to the absolute value, or dynamic modulus. For pure viscous materials, ø = 90º. The dynamic modulus testing of asphaltic materials is normally conducted using a uniaxially applied sinusoidal stress pattern as shown in Figure E1. The primary output variables of the test are the dynamic modulus |E*|, and the phase angle (ø), which is a direct indicator of the elastic-viscous properties of the mix or binder material.

Specimen Preparation

Samples for dynamic modulus testing were prepared by mixing the aggregates with PG 64-28 graded asphalt cement in the present study. Test samples were prepared in accordance with the requirements of AASHTO T 342-1.

Sample Requirements

The AASHTO T 342-11 requirements for dynamic modulus test samples are provided in Table 1. Dynamic modulus testing requires a 150 mm high by 100 mm diameter sample, of a target air void content, be cored from 175 mm high by 150 mm diameter sample. There is no simple conversion factor for compaction of a 175 mm high, 150 mm diameter SGC compacted sample to a cored dynamic modulus (E*) sample with a given target air void content. The two samples will not have the same VTM due to a density gradient present in SGC compacted samples. A trial and error procedure is required to determine the density or void content of the larger sample required to produce a cored and sawed test sample of the intended void content.

Recommended target air void contents for HMA samples are 4-7%. For this project, the HMA test samples were compacted to a void content of 4.5 ± 1 % VTM. After several trials, it was determined that a 175 mm high by 150 mm diameter sample compacted to 6.0 ± 1% VTM would yield a dynamic modulus test sample of the target 4.5 ± 1% void content.

Table E-1. Criteria for Acceptance of Dynamic Modulus Test Specimens

Criterion Items	Requirements
Size	Average diameter between 100 mm and 104 mm Average height between 147.5 mm and 152.5 mm
Gyratory Specimens	Prepare 175 mm high specimens to required air void content (AASHTO T 312)
Coring	Core the nominal 100 mm diameter test specimens from the center of the gyratory specimen. Check the test specimen is cylindrical with sides that are smooth parallel and free from steps, ridges and grooves
Diameter	The standard deviation should not be greater than 2.5 mm
End Preparation	The specimen ends shall have a cut surface waviness height within a tolerance of ± 0.05 mm across diameter The specimen end shall not depart from perpendicular to the axis of the specimen by more than 1 degree
Air Void Content	The test specimen should be within ± 1.0 percent of the target air voids
Replicates	For two LVDT's, two replicates with a estimated limit of accuracy of 13.1 percent
Sample Storage	Wrap specimens in polyethylene and store in environmentally protected storage between 5 and 26.7° C (40 and 80° F) and be stored no more than two weeks prior to testing

Batching:

A 5,700 to 6,300 gram batch of aggregate, batched to the desired gradation, was required to produce a 175 mm high by 150 mm diameter test specimen with 6.0 ± 1% VTM. When the compacted sample was cored to 100 mm diameter and sawed to the required sample height of 150 mm, the required target void content of 4.5 ± 1% VTM was obtained.

Mixing:

All samples were mixed in a bucket mixer. The asphalt cement was stirred occasionally to prevent localized overheating while being heated to the mixing temperature of 325o F. The aggregates were heated for a minimum of four hours at the mixing temperature of 325o F. Approximately one hour before mixing, the compaction molds, spoons and spatulas were placed in the oven and brought to the mixing temperature. For mixing, the aggregates were placed in the bucket mixer and the desired amount of asphalt cement added. The mixture was mixed until well coated, approximately two minutes.

Compaction:

After mixing, the mixture was placed in a large flat pan and placed in an oven set at the compaction temperature 148ºC (300ºF) for two hours in accordance with AASHTO R 30.

The samples were compacted in a 150 mm diameter mold to a height of 175 mm using a Pine SGC. To produce the required 175 mm high by 150 mm diameter sample with a void content of 6.0 ± 1 %, 5,700 to 6,300 grams of aggregate were required. Twenty to twenty-five gyrations were typically required to reach a height of 175 mm. A compacted HMA specimen is shown in Figure E2.

Figure E-2: Compacted HMA specimen for Dynamic Modulus Testing

Coring and Sawing:

After compaction, the samples were extruded from the compaction molds, labeled and allowed to cool to room temperature. Next, the compacted samples were cored and sawed to obtain a 150 mm tall by 100 mm diameter test sample with 4.5 ± 1 % air voids. The samples were cored using a diamond studded core barrel to obtain the required diameter of 100 mm (figure E3). The cored samples were then sawed to obtain the required 150 mm height (figure E4). The cored and sawed samples were washed to eliminate all loose debris. After cleaning, the samples were tested for bulk specific gravity in accordance with AASHTO T 166.

Figure E-3 Sample being cored to required test diameter.

Figure E-4 Sample being sawed to obtain parallel faces.

Setting up Testing Method:

Specimens were tested for dynamic modulus per AASHTO TP 62-03. The procedure is briefly explained in Figure E5. The test parameters are provided in Table E2.

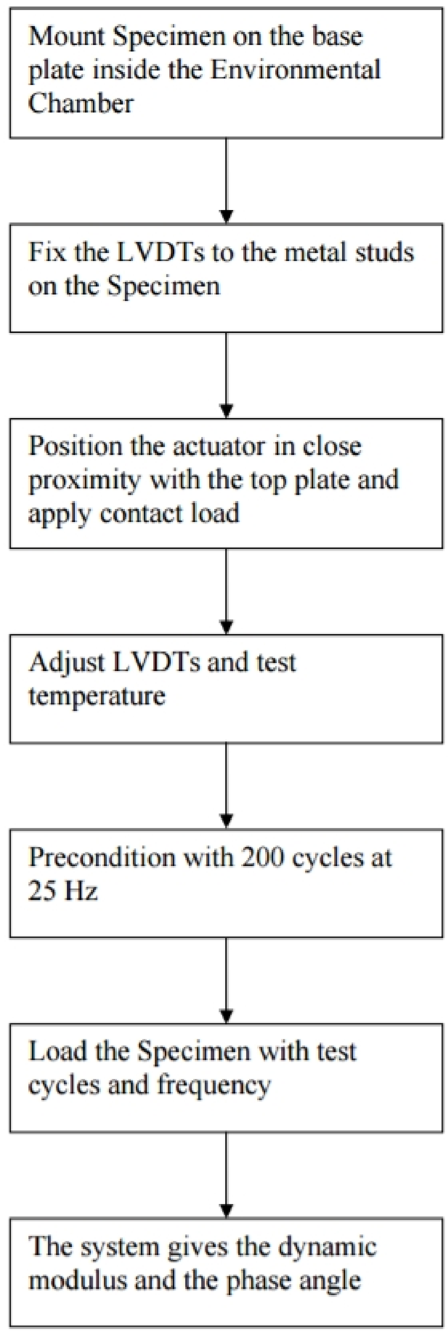


Figure E-5: Test procedures for dynamic modulus of HMA sample.

Table E-2. Test Parameters for Dynamic Modulus Test

Specimen Temperature, ºC (ºF)	Time from Room Temperature, h 25ºC (77ºF)	Time from Previous Test Temperature, h
-10 (14)	Overnight	Overnight
4 (40)	Overnight	4 hours or overnight
21 (70)	1	3
37 (100)	2	2
54 (130)	3	1

Temperature, ºC (ºF)	Range, kPa	Range, psi
-10 (14)	1400 to 2800	200 to 400
4 (40)	700 to 1400	100 to 200
21 (70)	350 to 700	50 to 100
37 (100)	140 to 250	20 to 50
54 (130)	35 to 70	5 to 10

Frequency, Hz	Number of Cycles
25	200
10	200
5	100
1	20
0.5	15
0.1	15

Figure E6 shows the dynamic modulus testing machine i.e. Instron 8800 fast track, which comprises of 5000N capacity of load cell connected with 2 LVDTs having ±0.25 mm range and Instron console and wave matrix software. Instron console is required to setup calibrations of LVDTs and load cell and to set up the limits during the test whereas instron wave matrix software is required to create test sequences according to AASHTO procedure.

Figure E-6: Dynamic Modulus Testing Machine

A sequence was created in wave matrix software with the help of Instron senior service support engineer as shown in Figure E7. All sequences were created in stress controlled mode. A ramp duration of 3 seconds was provided before start of every testing sequence. Hold duration of 2 minutes was also provided between every frequency change in testing sequences per AASHTO procedure requirements. Amplitude was selected according to the range provided by the AASHTO standard to obtain axial strains between 50 and 150 microstrain.

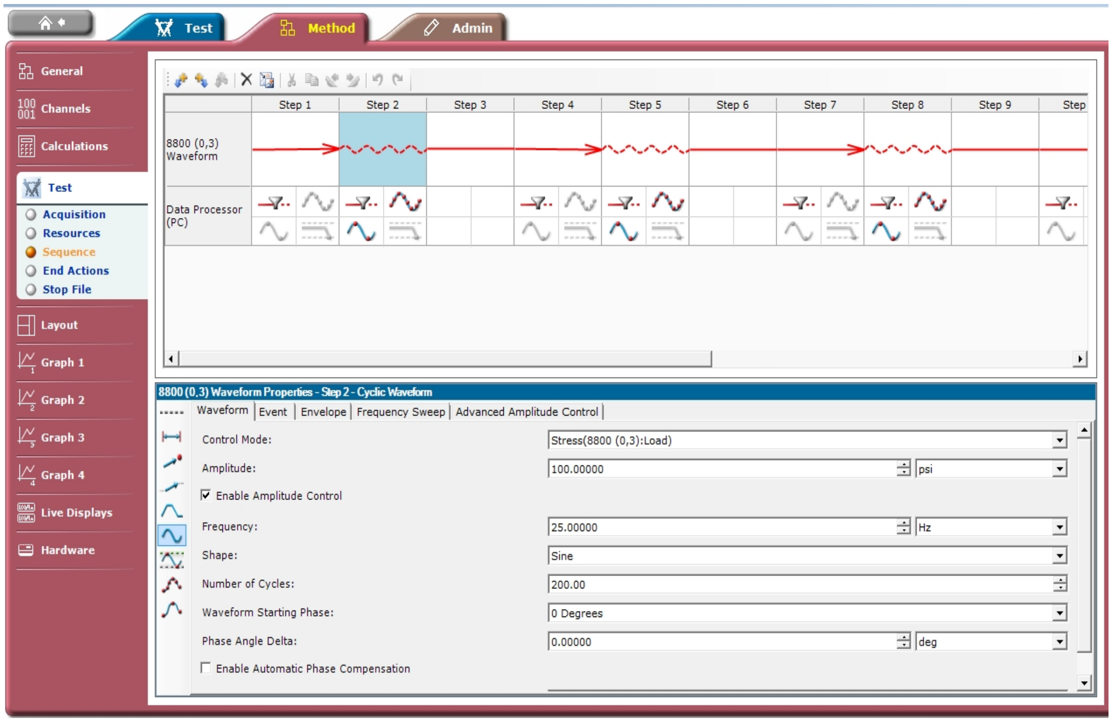


Figure E-7: Test Sequences on Instron Wave Matrix Software

A total of 16 steps were created which includes 5 ramp duration intervals, 5 two-minute hold duration intervals, and 6 actual testing sequences with different frequencies, amplitudes, and number of cycles.

Testing:

A HMA specimen was conducted on above mentioned temperatures. A jig was designed to fix LVDTs on two sides of the specimen as shown in Figure 9. It was made sure to restore the calibrations of load, strain 1, and strain 2 on Instron Console. Load and strains were balanced and proper limits were set before conducting the test. Piston was pushed downward so that it came into the contact with the specimen. Test was started by using the created testing sequence.

Figure E-8: Specimen along with a Jig to fix LVDTs

Analysis of Raw Data:

Instron Wave matrix software is very useful in terms of analyzing data and doing calculations from the raw data. In calculation option from the method tab Dynamic Mechanical Analysis (DMA) calculation was selected as shown in Figure E9. Correlation method was selected to analyze the raw data. Parameters for calculating dynamic modulus includes specimen geometry which was selected as Axial- Cylindrical. 4 in. diameter and 6 in. height was inputted into the software as shown in Figure E9 to perform DMA calculations.

After running successful testing sequence on a specimen instron wave matrix created a raw data on a excel spread sheet and also calculate the dynamic modulus values by performing DMA calculations as shown in Table E3.

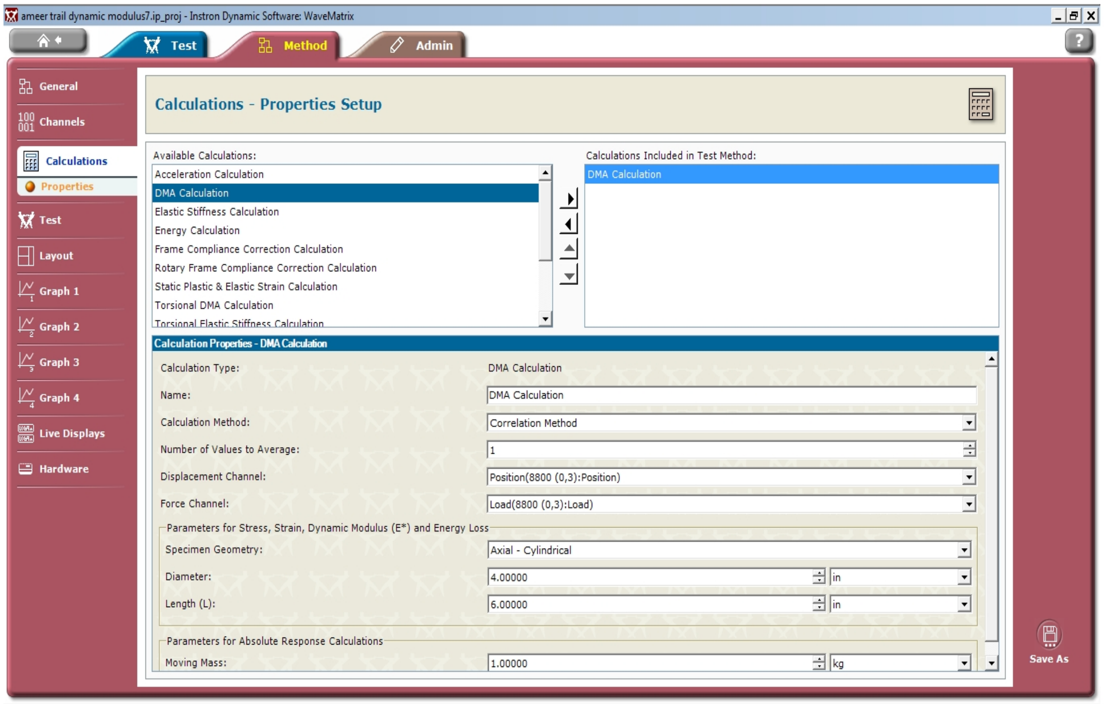


Figure E-9: DMA calculation Screen to calculate Dynamic Modulus Value

Table E-3: Dynamic Modulus of HMA Specimen.

T (ºF)	0.1 Hz	0.5 Hz	1 Hz	5 Hz	10 Hz	25 Hz
14	1591378.3	1932420.5	2067904.4	2348196.0	2453128.6	2577269.3
40	602057.2	896143.6	1012970.1	1372388.2	1501016.3	1684233.2
70	126021.9	213337.4	257818.4	430836.2	522374.9	668378.0
100	47425.64	63186.44	73229.86	118821.5	149934.2	206914.4
130	35511.67	38375.26	40593.26	49927.52	56707.92	69585.82

Appendix F

Condition Survey of Route 165

Appendix F

Condition Survey of Route 165

Asphalt Pavement Inspection Sheet for Control Test Section

ASPHALT PAVEMENT INSPECTION SHEET

BRANCH Rte. 165 SECTION Control

DATE 12/21/15 SAMPLE UNIT Control

SURVEYED BY ______ AREA OF SAMPLE ______

Distress Types

1. Alligator Cracking
2. Bleeding
3. Block Cracking
*4. Bumps and Sags
5. Corrugation
6. Depression
*7. Edge Cracking
*8. Jt Reflection Cracking
*9. Lane/Shldr Drop Off
*10. Long & Trans Cracking
11. Patching & Util Cut Patching
12. Polished Aggregate
*13. Potholes
14. Railroad Crossing
15. Rutting
16. Shoving
17. Slippage Cracking
18. Swell
19. Weathering and Raveling

SKETCH: 357 west bnd lane 7122.5

EXISTING DISTRESS TYPE, QUANTITY & SEVERITY

TYPE →					
QUANTITY & SEVERITY	Minor Raveling				
TOTAL SEVERITY L	L				
M					
H					

PCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
q=	TOTAL DEDUCT VALUE		
CORRECTED DEDUCT VALUE (CDV)			

PCI = 100 − CDV =

RATING =

* All Distresses Are Measured In Square Feet Except Distresses 4,7,8,9 and 10 Which Are Measured In Linear Ft; Distress 13 Is Measured In Number of Potholes.

Asphalt Pavement Inspection Sheet for Calcium Chloride Test Section

ASPHALT PAVEMENT INSPECTION SHEET

BRANCH Rte 165 SECTION CACL

DATE ______ SAMPLE UNIT ______

SURVEYED BY ______ AREA OF SAMPLE ______

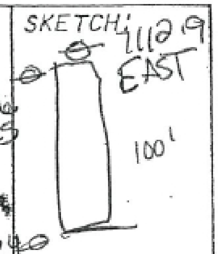
Distress Types

1. Alligator Cracking
2. Bleeding
3. Block Cracking
*4. Bumps and Sags
5. Corrugation
6. Depression
*7. Edge Cracking
*8. Jt Reflection Cracking
*9. Lane/Shldr Drop Off
*10. Long & Trans Cracking
11. Patching & Util Cut Patching
12. Polished Aggregate
*13. Potholes
14. Railroad Crossing
15. Rutting
16. Shoving
17. Slippage Cracking
18. Swell
19. Weathering and Raveling

Pole 325

Pole 324

SKETCH:



EXISTING DISTRESS TYPE, QUANTITY & SEVERITY

TYPE →						
QUANTITY & SEVERITY	~~Minor Ravel~~ Nothing					
TOTAL SEVERITY L						
M						
H						

PCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
q=	TOTAL DEDUCT VALUE		
CORRECTED DEDUCT VALUE (CDV)			

PCI = 100 − CDV =

RATING =

* All Distresses Are Measured In Square Feet Except Distresses 4,7,8,9 and 10 Which Are Measured In Linear Ft; Distress 13 Is Measured In Number of Potholes.

Asphalt Pavement Inspection Sheet for Portland cement Test Section

ASPHALT PAVEMENT INSPECTION SHEET

BRANCH Rte 165 SECTION Cement

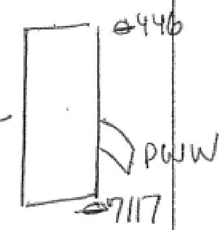
DATE ______ SAMPLE UNIT Pole 446

SURVEYED BY ______ AREA OF SAMPLE ______

Distress Types

1. Alligator Cracking
2. Bleeding
3. Block Cracking
4. * Bumps and Sags
5. Corrugation
6. Depression
7. * Edge Cracking
8. * Jt Reflection Cracking
9. * Lane/Shldr Drop Off
10. * Long & Trans Cracking
11. Patching & Util Cut Patching
12. Polished Aggregate
13. * Potholes
14. Railroad Crossing
15. Rutting
16. Shoving
17. Slippage Cracking
18. Swell
19. Weathering and Raveling

SKETCH:



EXISTING DISTRESS TYPE, QUANTITY & SEVERITY

TYPE						
QUANTITY & SEVERITY	Minor Raveling					
TOTAL SEVERITY L						
TOTAL SEVERITY M						
TOTAL SEVERITY H						

PCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
q=	TOTAL DEDUCT VALUE		
CORRECTED DEDUCT VALUE (CDV)			

PCI = 100 − CDV =

RATING =

* All Distresses Are Measured In Square Feet Except Distresses 4,7,8,9 and 10 Which Are Measured In Linear Ft; Distress 13 Is Measured In Number of Potholes.

Asphalt Pavement Inspection Sheet for Asphalt Emulsion Test Section

ASPHALT PAVEMENT INSPECTION SHEET

Rte 165

BRANCH Asphalt Emulsion SECTION Pole 542 West

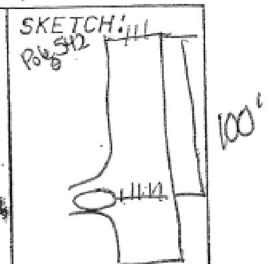
DATE Dec. 21, 2015 SAMPLE UNIT ~~100~~' Emulsion

SURVEYED BY ____ AREA OF SAMPLE 100

Distress Types

1. Alligator Cracking	*10. Long & Trans Cracking
2. Bleeding	11. Patching & Util Cut Patching
3. Block Cracking	12. Polished Aggregate
*4. Bumps and Sags	*13. Potholes
5. Corrugation	14. Railroad Crossing
6. Depression	15. Rutting
*7. Edge Cracking	16. Shoving
*8. Jt Reflection Cracking	17. Slippage Cracking
*9. Lane/Shldr Drop Off	18. Swell
	19. Weathering and Raveling

SKETCH:



EXISTING DISTRESS TYPE, QUANTITY & SEVERITY

TYPE						
QUANTITY & SEVERITY	minor Raveling					
TOTAL SEVERITY L	Low					
TOTAL SEVERITY M						
TOTAL SEVERITY H						

PCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
q=	TOTAL DEDUCT VALUE		
CORRECTED DEDUCT VALUE (CDV)			

PCI = 100 − CDV =

RATING =

* All Distresses Are Measured In Square Feet Except Distresses 4,7,8,9 and 10 Which Are Measured In Linear Ft; Distress 13 Is Measured In Number of Potholes.

Asphalt Pavement Inspection Sheet for Geo-grid Test Section

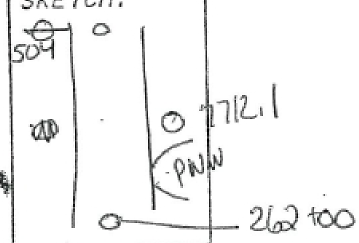
ASPHALT PAVEMENT INSPECTION SHEET

BRANCH Rte 165 SECTION Geo Grid
DATE 12/21 SAMPLE UNIT 7712.1
SURVEYED BY ________ AREA OF SAMPLE ________

Distress Types

1. Alligator Cracking
2. Bleeding
3. Block Cracking
*4. Bumps and Sags
5. Corrugation
6. Depression
*7. Edge Cracking
*8. Jt Reflection Cracking
*9. Lane/Shldr Drop Off
*10. Long & Trans Cracking
11. Patching & Util Cut Patching
12. Polished Aggregate
*13. Potholes
14. Railroad Crossing
15. Rutting
16. Shoving
17. Slippage Cracking
18. Swell
19. Weathering and Raveling

SKETCH:



EXISTING DISTRESS TYPE, QUANTITY & SEVERITY

TYPE	Raveling					
QUANTITY & SEVERITY						
	Minor					
TOTAL SEVERITY L	L					
TOTAL SEVERITY M						
TOTAL SEVERITY H						

PCI CALCULATION

DISTRESS TYPE	DENSITY	SEVERITY	DEDUCT VALUE
q=	TOTAL DEDUCT VALUE		
CORRECTED DEDUCT VALUE (CDV)			

PCI = 100 − CDV =

RATING =

* All Distresses Are Measured In Square Feet Except Distresses 4,7,8,9 and 10 Which Are Measured In Linear Ft; Distress 13 Is Measured In Number of Potholes.

Appendix G

Summary of AASHTOWare ME Design Reports for Route 165 Test sections

Appendix G

Summary of AASHTOWare ME Design Reports for Route 165 Test sections

Route 165 Control Test Section

File Name: E:\Route 165 Control Test Section.dgpx

Design Inputs

Design Life: 20 years
Design Type: FLEXIBLE

Base construction: April, 2013
Pavement construction: July, 2014
Traffic opening: August, 2014

Climate Data Sources (Lat/Lon): 42.361, -71.011

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.5
NonStabilized	Cold recycled asphalt - RAP (includes millings)	8.0
NonStabilized	A-1-a	1.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-1-b	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	5.6
Air voids (%)	5.1

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	150
2024 (10 years)	295,373
2034 (20 years)	668,818

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	146.72	90.00	98.31	Pass
Permanent deformation - total pavement (in)	0.75	0.51	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	4.60	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	84.34	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	2548.73	90.00	83.64	Fail
Permanent deformation - AC only (in)	0.25	0.06	90.00	100.00	Pass

Route 165 Calcium Chloride Test Section

File Name: E:\Route 165 Calcium Chloride Test Section.dgpx

Design Inputs

Design Life: 20 years
Design Type: FLEXIBLE

Base construction: April, 2013
Pavement construction: July, 2014
Traffic opening: August, 2014

Climate Data Sources (Lat/Lon): 42.361, -71.011

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.5
NonStabilized	Cold recycled asphalt - RAP (includes millings)	8.0
NonStabilized	Cold recycled asphalt - RAP (includes millings)	1.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-1-b	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	5.6
Air voids (%)	5.1

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	150
2024 (10 years)	295,373
2034 (20 years)	668,818

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	146.30	90.00	98.38	Pass
Permanent deformation - total pavement (in)	0.75	0.51	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	3.33	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	84.34	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	2354.80	90.00	85.88	Fail
Permanent deformation - AC only (in)	0.25	0.06	90.00	100.00	Pass

Route 165 Portland Cement Test Section

File Name: E:\Route 165 Portland Cement Test Section.dgpx

Design Inputs

Design Life: 20 years
Design Type: FLEXIBLE

Base construction: April, 2013
Pavement construction: July, 2014
Traffic opening: August, 2014

Climate Data Sources (Lat/Lon): 42.361, -71.011

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.5
NonStabilized	Cold recycled asphalt - RAP pulverized in place	8.0
NonStabilized	Cold recycled asphalt - RAP (includes millings)	1.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-1-b	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	5.6
Air voids (%)	5.1

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	150
2024 (10 years)	295,373
2034 (20 years)	668,818

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	144.84	90.00	98.59	Pass
Permanent deformation - total pavement (in)	0.75	0.48	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	2.01	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	84.34	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	1593.19	90.00	94.70	Pass
Permanent deformation - AC only (in)	0.25	0.06	90.00	100.00	Pass

Route 165 Asphalt Emulsion Test Section

File Name: E:\Route 165 Asphalt Emulsion Test Section.dgpx

Design Inputs

Design Life:	20 years	Base construction:	April, 2013	Climate Data Sources (Lat/Lon)	42.361, -71.011
Design Type:	FLEXIBLE	Pavement construction:	July, 2014		
		Traffic opening:	August, 2014		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.5
NonStabilized	Cold recycled asphalt - RAP (includes millings)	3.0
NonStabilized	Cold recycled asphalt - RAP (includes millings)	5.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-1-b	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	5.6
Air voids (%)	5.1

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	150
2024 (10 years)	295,373
2034 (20 years)	668,818

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	148.06	90.00	98.08	Pass
Permanent deformation - total pavement (in)	0.75	0.53	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	9.54	90.00	99.99	Pass
AC thermal cracking (ft/mile)	1000.00	84.34	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	3009.93	90.00	78.22	Fail
Permanent deformation - AC only (in)	0.25	0.06	90.00	100.00	Pass

Route 165 Geo-Grid Test Section

File Name: E:\Route 165 Geo-Grid Test Section.dgpx

Design Inputs

Design Life: 20 years
Design Type: FLEXIBLE

Base construction: April, 2013
Pavement construction: July, 2014
Traffic opening: August, 2014

Climate Data Sources (Lat/Lon): 42.361, -71.011

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.5
NonStabilized	Cold recycled asphalt - RAP pulverized in place	10.0
NonStabilized	Crushed stone	6.0
Subgrade	A-1-b	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	5.6
Air voids (%)	5.1

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	150
2024 (10 years)	295,373
2034 (20 years)	668,818

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	172.00	147.33	90.00	98.21	Pass
Permanent deformation - total pavement (in)	0.75	0.52	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	7.90	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	84.34	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	2810.36	90.00	80.61	Fail
Permanent deformation - AC only (in)	0.25	0.06	90.00	100.00	Pass

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