EVALUATION OF PAVEMENT REHABILITATION STRATEGIES ON ROUTE 165 AND PREDICTION PERFORMANCE

BY

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ABSTRACT

Many roads in Rhode Island are coming to their intended design life and are now considered in poor condition. Significant number of roads with severe deterioration is being rehabilitated through full depth reclamations (FDR) with various additives. FDR can rejuvenate subbase and pavement structures. Route 165 in Exeter, Rhode Island, was selected as a test road with four different treatments and a control. The road had severe pavement distresses such as alligator cracking, pot holes, shoving and raveling and was not a candidate for a resurfacing. The road had a FDR in 2013 which included a control section, three test sections with additives which consisted of calcium chloride, asphalt emulsion, and Portland cement, and a geo-grid section. Triaxial testing was performed on the subbase materials and subgrade soils before and after the FDR treatments to determine the resilient modulus. The results of the material testing were used to predict the performance of each of the test sections by using AASHTOWare Pavement ME Design software.

The 200 mm (8.0 in.) rehabilitated base/subbase layer was covered with 62.5 mm (2.5 in.) Hot Mix Asphalt (HMA) with Warm Mix Asphalt (WMA) additive base and 50 mm (2 in.) surface. The maximum sizes of base and surface aggregate were 19 mm (3/4 in.) and 12.5 mm (1/2 in.), respectively. Properties of HMA mixtures with WMA additives including dynamic modulus were determined as input parameters and for further analysis

Of the five test sections, it was predicted that the pavement with the FDR layer stabilized with Portland cement would perform the best overall. Next is calcium chloride followed by the control (no additive), geo-grid and asphalt emulsion.

A condition survey was conducted in 2015 winter, but there was no surface distresses found on any of the five test sections. A plan for long term performance evaluation has been developed, and an optimal strategy has been recommended, i.e., predicting performances before rehabilitating any broken roads.

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A research project entitled “Evaluation of Pavement Rehabilitation Strategies on Route 165 and Prediction of Performance” was submitted for funding to the Rhode Island Department of Transportation (RIDOT) by the Department of Civil & Environmental Engineering at the University of Rhode Island (URI) in April, 2003. A Research and Technology Development Cooperative Agreement was subsequently entered into by the RIDOT and the URI for conducting research involving the field performance of five tests sections on Route 165, Exeter, RI.

Introductory works, which preceded the start of the project, involved preparing a scope of work and project cost as well as sampling of Route 165 materials by another research grant from URI. Purchasing of a computer program from AASHTO was done in November, 2015. I am indebted to Messrs. Colin Franco and Michael Sock of Research and Technology Development Section, and Mr. Michael Byrne of Material Section of RIDOT.

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INTRODUCTION

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 et al. 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 material strategies such as full depth reclamations (FDR) and has been expanding their use.

RIDOT wants to build a better road, to have less physical maintenance and to control costs. The final product according to RIDOT is a high performance road. Reclaiming a roadway can fulfill RIDOTs wants by increasing the stiffness of the subbase and increase the pavement life.

Achieving higher performance at a 30% to 50% cost savings can be realized with a full reclamation according to pavement recycling systems. Reclaimed materials are retained and reused on site, consequently reducing trucking costs for new materials.

- Lifecycle cost savings: lower maintenance
- Environmental: recycles in-place material
- Uses value of existing pavement

- In-depth: eliminates cracking patterns
- Minimizes costly import/export
- Structural upgrade to base: R-Value, strength
- Shorter construction time, less disruption

(http://www.pavementrecycling.com).

In the 1980, RIDOT had a program to reclaim pavements throughout the state. In 2013, Route 165 was slated to re-reclaimed. An idea was formed to use four different strategies and a control (Figure 1.1). The objectives of this project are: to test the existing subbase materials before and after full depth reclamation, to predict the performance of different subbase strategies and to evaluate the short-term and long-term performance over time.

1.1 Objective of Study

Route 165 is a unique candidate for research because it is seven miles long without many intersections, the subgrade layer has a high water table and there is severe frost action in winter. The objectives of the present research project are as follows:

1. Predict the performance of five test sections on Route 165 which include the following subtasks:

   * Collect and/or determine environmental inputs including temperature, moisture and Freeze-Thaw etc.

   * Work with a URI research team to collect and/or determine basic properties and resilient modulus of the existing subbase materials of Route 165.

* Collect and/or determine properties of reclaimed base/subbase materials on the five test sections on Route 165.

* Collect and/or determine properties of two and a half inches of Class 19 warm mix asphalt (WMA) (19 mm Superpave) asphalt base (through working with RIDOT Material Section and Villanova University, Pennsylvania) on Route 165.

* Collect and/or determine properties of two inch Class 12.5 WMA (12.5 mm superpave) asphalt surface (through working with RIDOT Material section and Villanova University, Pennsylvania) on Route 165.

* Collect Weigh-In-Motion (WIM) data from the RIDOT Traffic Research Section for initial two-way average annual daily truck traffic (AADTT), percent trucks in design Lane, percent trucks in design lane, percent trucks in design directions, and operational speed on Route 165.

* Predict performance including rutting, fatigue cracking, thermal cracking and roughness in terms of International Roughness Index (IRI) using AASHTOWare Pavement ME Design (MED).

2. Perform a visual condition survey on Route 165.
3. Secure falling weight deflectometer (FWD) testing data from RIDOT on Route 165 after construction.

* Analyze data including deflection basin.

4. Collect profilometer test data from RIDOT to evaluate the smoothness after construction.

5. Evaluate distresses, if any, and determine which layer has weakness.

6. Evaluate the five different strategies and recommend the best one for future RIDOT rehabilitation projects from the Route 165 project. The five strategies are a control, calcium chloride, asphalt emulsion, Portland Cement, and geo-grid. RIDOT has used calcium chloride and cold recycled mix but not Portland cement no asphalt emulsion. Geo-grid has been used in the state for drainage problem areas.

7. Provide an optimal design and strategies for future RIDOT rehabilitation projects.

* Assist future testing and evaluation after the 2016 summer.

The outcome of this research project will provide a guideline for future maintenance and rehabilitation (M&R) projects.

1.2 Significance of Study

Chapter two discusses the construction methodologies and/or procedures performed on Route 165. It was important to keep the different test strategies as uniform as possible to have a comparison study. The special job specifications for the contractor were written to keep the depth of reclamation constant. Chapter three provides the results of the different subbase strategy tests such as sieve analysis, resilient modulus, and densities. Collection and compilation of the WIM station data on average annual daily truck traffic (AADTT), WMA mixes, material properties of the subbase and subgrade, and climate data. Performance comparisons and ranking between the different MED strategies. Chapter four consists of surface distresses and field condition surveys, forward calculations of FWD deflections, serviceability and roughness, comparative analysis between prediction and evaluation of the test strategies, best alternatives base on short term evaluations and optimal strategies. Chapter 5 provides the conclusion and recommendations of this research and guidelines for long-term evaluation and optimal rehabilitation design.

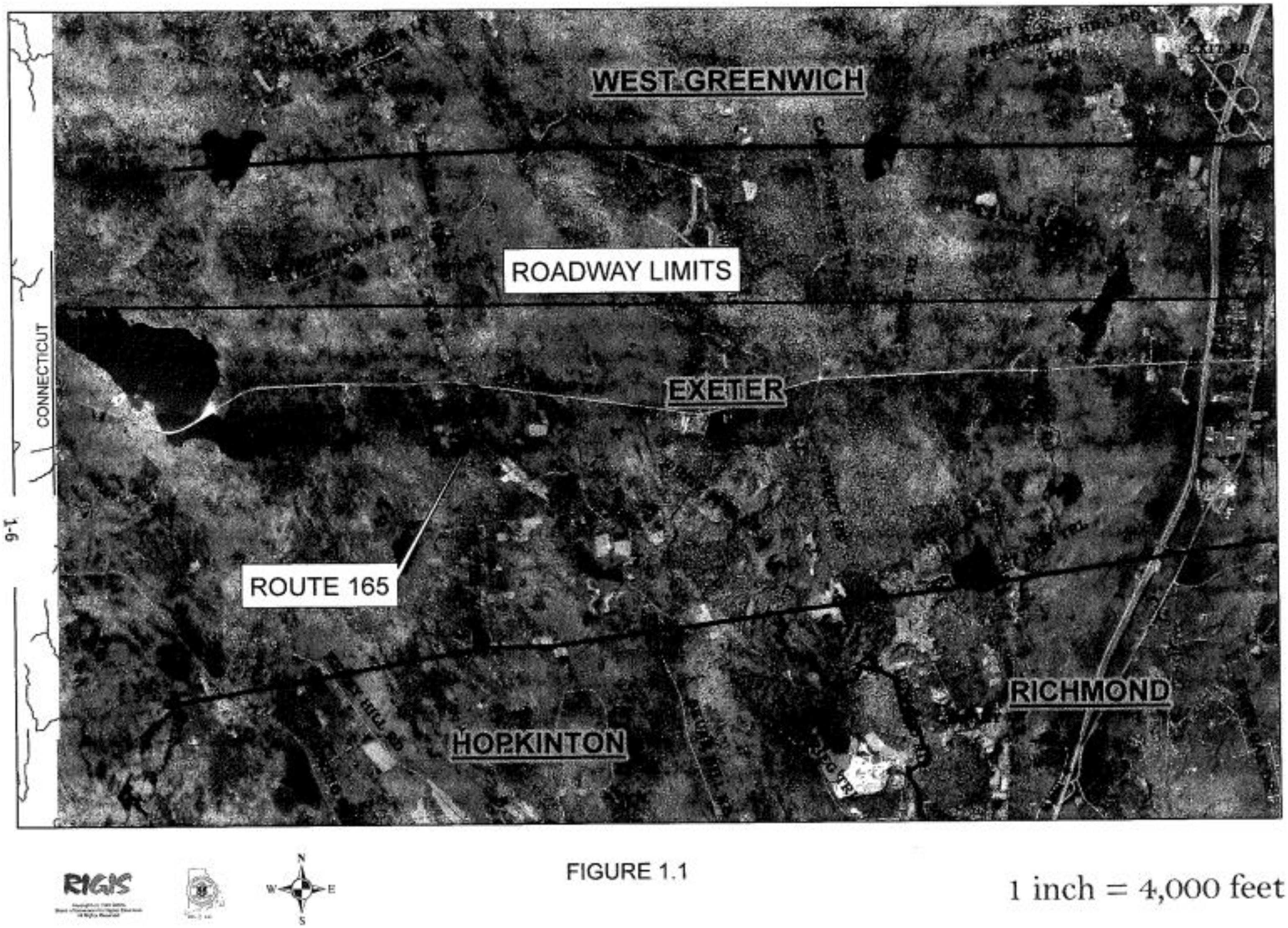


FIGURE 1.1

1 inch = 4,000 feet

Chapter 2

Rehabilitation of Rhode Island Route 165

2.1 History

Route 165 was last reconstructed in 1986. The roadway was reclaimed to a depth of 125 mm (5 in.) and 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 (Figure 2.1).

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 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 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.

The RIDOT, throughout the years, has performed both material and pavement testing on this roadway primarily through RIDOT maintenance programs. Skid tests measuring pavement smoothness were performed in 2003, 2006, and 2010 (see Appendix H). The results show that in 2003, Route 165 had skid numbers between 52 and 58. In 2010, the skid 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 rideability.

2.2 Pre-Construction Material Testing

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 tests. 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 (Table 2.2). 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 ME pavement program (Bradshaw et al. 2015).

2.3 Methodologies and/or Procedures

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 (see 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 2.2 through 2.6.

Equipment used consisted of: reclaimer, vibratory sheepsfoot rollers and motorized graders. Compaction was in accordance with AASHTO T180, Method D to a uniform density of no less than 95% of maximum and pavement operations took place during acceptable temperature ranges. However, a sudden downpour during Portland cement placement and washed all the material away. The Portland cement section was then regraded and new Portland cement applied.

Full depth reclamation (FDR) with calcium chloride 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).

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 $\pm$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).

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.

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.

The physical properties of each 2013 test sections were determined at URI and are shown in Tables 2.4 through 2.6. Physical properties of selected subgrade soils from Route 165 site sample locations are presented in Table 2.4 (Bradshaw et al. 2015). Physical properties of selected 1980s RAP blends from Route 165 site is presented in Table 2.5. (Bradshaw et al. 2015). Physical properties of selected FDR RAP blends of Route 165 site is presented in Table 2.6. (Bradshaw et al. 2015).

The rehabilitated eight inch base/subbase were covered with two and a half (2.5) inches of Class 19 WMA base and two inches of Class 12.5 WMA surface layer as shown in Figures 2.2 to 2.6. A summary of some of the volumetric data for the Class 12.5 WMA surface materials is shown in Tables 2.7 and 2.8.

Table 2. 1. Soil Survey for Route 165

		Flooding Frequency	High Water Table FT	High 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 Nuclear Guage Readings for Route 165

Sample ID	Station	Offset	Direction	Utility Pole	Date Sampled	Nuclear Gauge Readings			
						Dry Density	Wet Density	Water, lbs	Moisture %
1 CaCl	39+25 +/-	Center of Travel Lane	Westbound	Pole 304	11/27/2012	126.0	132.7	5.6	4.4
						122.3	129.2	5.3	4.3
2 PC	135+80 +/-	Center of Travel Lane	Eastbound	Pole 381	11/27/2012	125.8	129.8	3.9	3.1
						125.1	128.8	3.7	3.0
3 PC	162+70 +/-	Center of Travel Lane	Eastbound	Pole 400	11/29/2012	129.8	135.1	5.2	4.0
						129.4	134.5	5.1	3.9
4 PC	204+84 +/-	Center of Travel Lane	Westbound	Pole 428	11/29/2012	139.6	141.3	4.2	3.0
						135.7	139.9	4.2	3.1
5 Bit	327+85 +/-	Center of Travel Lane	Westbound	Pole 518	11/29/2012	144.5	149.8	5.2	3.6
						144.3	149.3	5.0	3.5
6 PC	260+10 +/-	Center of Travel Lane	Eastbound	Pole 468	11/29/2012	132.6	137.8	5.2	3.9
						131.6	136.8	5.1	3.9
7 CaCl	95+29 +/-	Center of Travel Lane	Westbound	Pole 347	12/4/2012	126.7	133.1	6.3	5.0
						127.5	133.4	6.3	4.9
8 Geo	285+00 +/-	Center of Travel Lane	Westbound	Pole 486	12/4/2012	130.4	135.7	5.2	4.0
						130.8	136.2	5.4	4.1
9 Bit	340+00 +/-	Center of Travel Lane	Eastbound	Pole 528	12/4/2012	125.0	131.0	6.0	4.8
						125.3	131.0	5.7	4.5
10 Geo	309+84 +/-	Center of Travel Lane	Eastbound	Pole 506	12/4/2012	129.7	134.6	4.9	3.8
						129.9	134.6	4.7	3.6
11 Recy	117+21 +/-	Center of Travel Lane	Eastbound	Pole 369	12/6/2012	131.0	136.7	5.7	4.3
						131.8	137.4	5.5	4.1
12 CaCl	62+54 +/-	Center of Travel Lane	Eastbound	Pole 321	12/6/2012	123.5	128.9	5.3	4.3
						129.6	134.8	5.2	4.0

(RIDOT Material Section 2013)

Table 2.3 Locations of Test Sections

Test Section	Station	Square yards	Elevations	.ocations	Utility Pole Locations	Segment Length in feet
Control Section	48+50 to 75+00	9,000	303	67+25	369	2,650
Calcium Chloride	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
Bituminous Stabilizer	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

Table 2.4 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

Bradshaw et al., 2015

Table 2.5 Summary of Physical Properties of Selected 1980s 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

Bradshaw et al., 2015

Table 2.6 Summary of Physical Properties of Selected FDR RAP/Virgin Aggregate Blends from Route

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

Bradshaw et al 2015

Notes: 1. The curing time is given in parenthesis.
2. The water content of the specimen after curing is given in parenthesis.
3. Maximum dry density and specimen dry densitity values have been corrected for oversize particles and represent field values.

Table 2.7 Cardi 12.5 mm Mix – RI 165 Lab Produced Mix Volumetrics

Cardi 12.5mm mix - RI 165
Lab Produced Mix Volumetrics

AC- Asphalt Content
AV- Air Void Content

Mixture Description	Sample No	Gmm	Gsb	Gmb	% Air Void
5.2AC-3.0AV	1			2.386	2.7
	2	2.452	2.630	2.373	3.2
	3			2.385	2.7
5.2AC-7.0AV	1			2.292	6.5
	2	2.452	2.630	2.289	6.6
	3			2.291	6.6
5.2AC-9.0AV	1			2.244	8.5
	2	2.452	2.630	2.240	8.6
	3			2.241	8.6
5.6AC-5.0AV	1			2.318	4.9
	2	2.438	2.630	2.325	4.6
	3			2.311	5.2
5.6AC-7.0AV	1			2.260	7.3
	2	2.438	2.630	2.267	7.0
	3			2.269	6.9
5.6AC-9.0AV	1			2.221	8.9
	2	2.438	2.630	2.212	9.3
	3			2.221	8.9
6.1AC-4.0AV	1			2.315	4.3
	2	2.420	2.630	2.314	4.4
	3			2.311	4.5
6.1AC-7.0AV	1			2.246	7.2
	2	2.420	2.630	2.239	7.5
	3			2.245	7.2
6.1AC-9.0AV	1			2.195	9.3
	2	2.420	2.630	2.191	9.5
	3			2.207	8.8

Table 2.8 Cardi 12.5 mm Mix – RI -165

Cardi 12.5mm mix - RI 165

Binder Grade PG 64-28V polymer modified with Evotherm, Warm Mix Asphalt

Binder Content	Low	5.2
	Optimum	5.6
	High	6.1
Effective Binder Content	Low	4.997
	Optimum	5.376
	High	5.879

Layer Properties, Route- RI 165

Old pavement structure 1.5"± bituminous surface course
1.5"± bituminous base course
12"± Gravel borrow

New pavement structure 2.0"±Modified class 12.5 HMA
2.5"± class 19.0HMA
8"± Reclamation

Gradation

Source		Cardi ½" Stone, % Passing	Cardi 3/8" Stone, % Passing	Cardi Stone Sand, %	PJ Keating Stone Sand, %	Overall Target Gradation
% of total mix		25	30.8	19.3	19.2	
% Passing	¾"	100	100	100	100	100.0
	½"	91.3	99.9	100	100	97.7
	3/8"	32.5	90.2	100	100	78.9
	#4	3.3	29.3	97.7	98.7	50.5
	#8	2	9.7	70.5	77.4	33.9
	#16	-	-	47.2	51.5	20.1
	#30	-	-	32.9	33.8	13.6
	#50	-	-	22.7	21.4	9.0
	#100	-	-	11	10.1	4.3
	#200	-	-	3.3	4.6	1.6

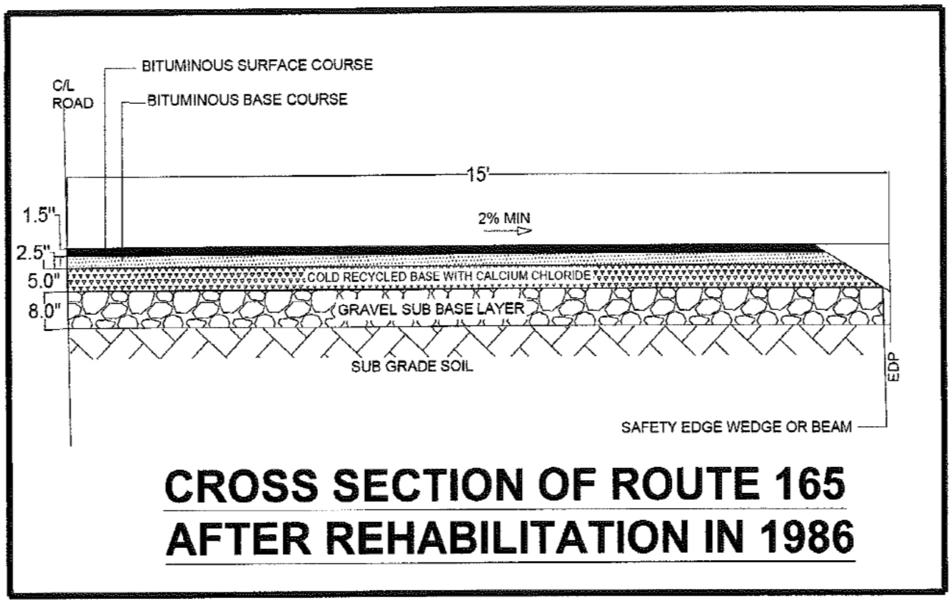


Figure 2.1 Cross Section of Route 165 after Rehabilitation in 1986

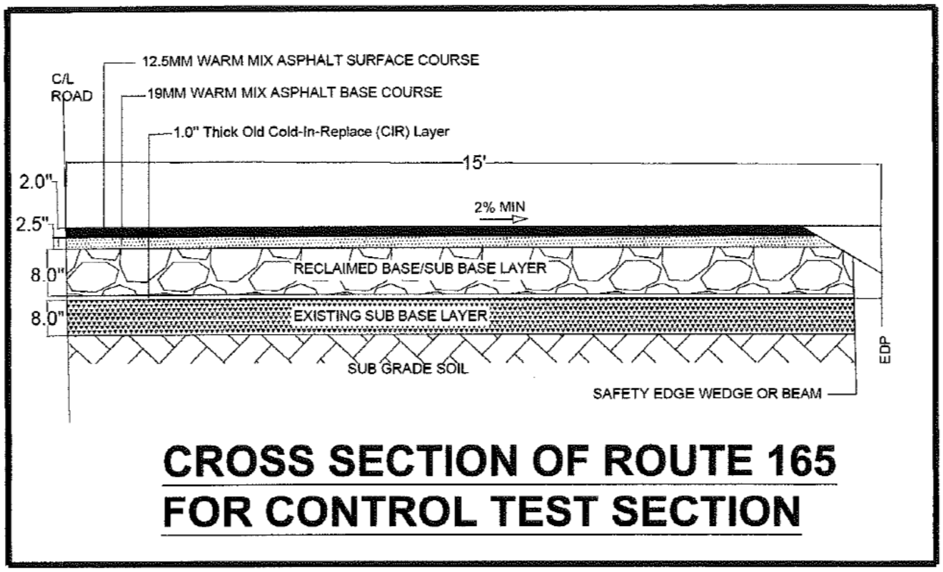


Figure 2.2 Cross Section of Route 165 with Control Test Section

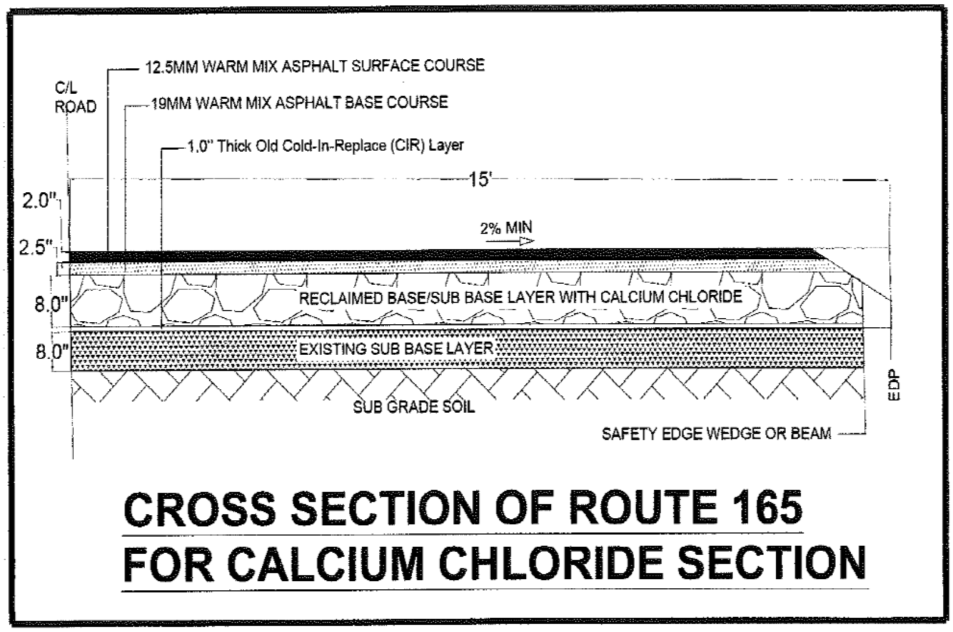


Figure 2.3 Cross Section of Route 165 with Calcium Chloride Section

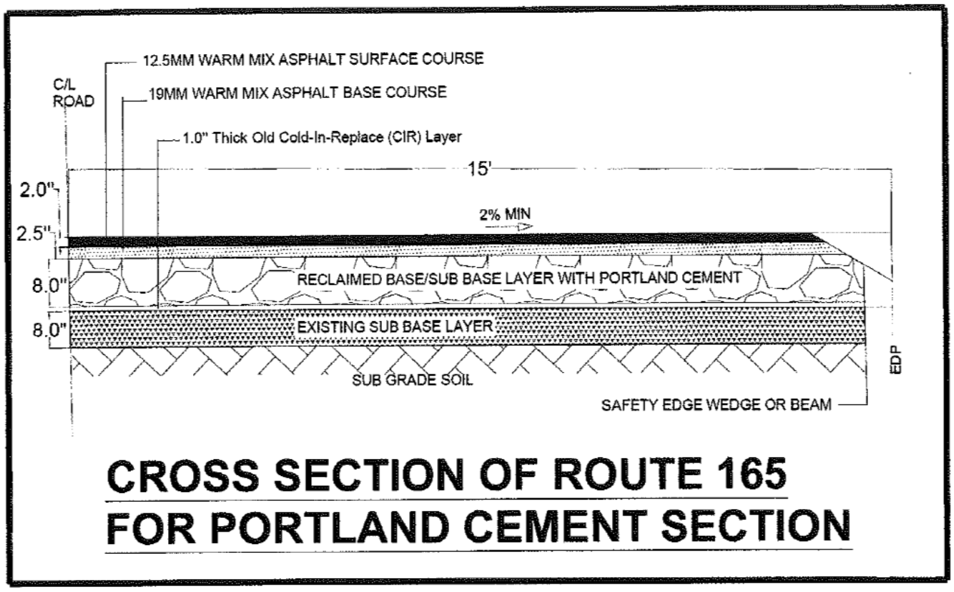


Figure 2.4 Cross Section of Route 165 with Portland cement Section

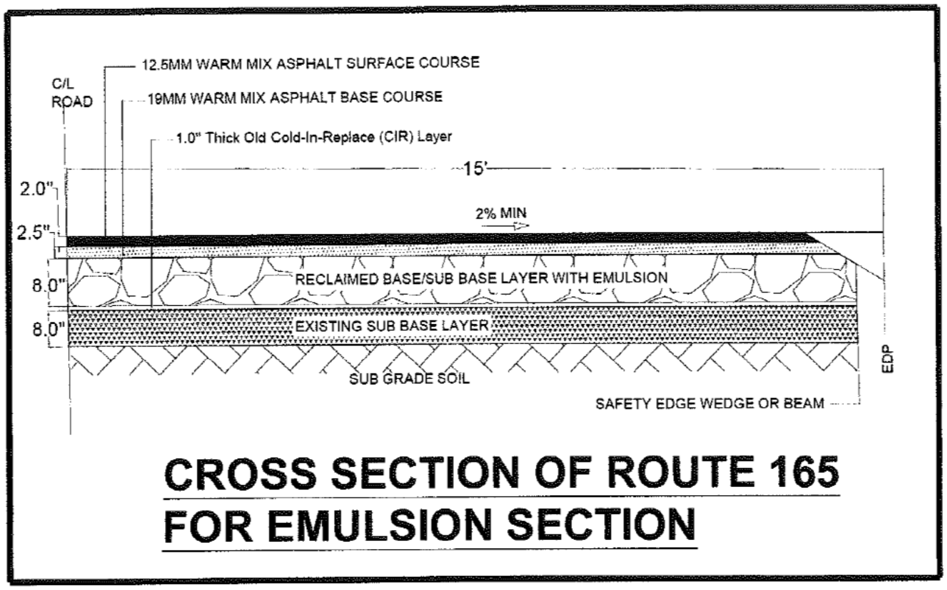


Figure 2.5 Cross Section of Route 165 with Emulsion Section

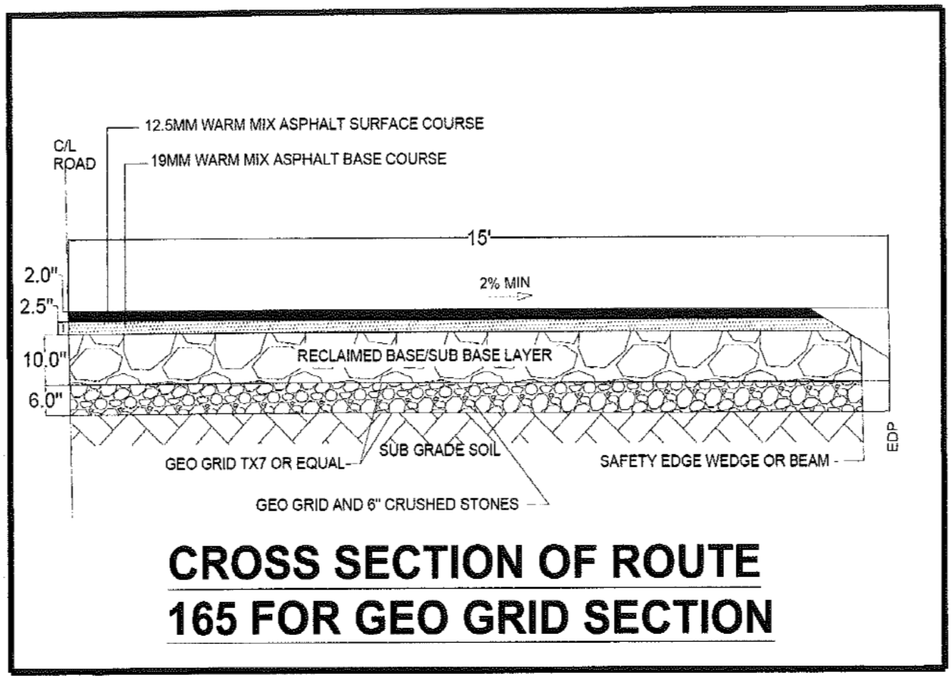


Figure 2.6 Cross Section of Route 165 with Geo Grid Section

Chapter 3

Performance Prediction of Rehabilitated Asphalt Pavement with AASHTOWare Pavement ME Design Software

Previously, Rhode Island used the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures to design the pavement thickness and subbase layers. The guide used graphs to calculate traffic equivalency values, freeze-thaw factors, and resilient modulus to find a design structural number.

The new updated method uses truck traffic, climate data, WMA mixture and subbase material properties for a mechanistic empirical design (Refer to Appendix A). There are three hierarchical levels in AASHTOWare Pavement ME Design (MED):

Level 1. Uses laboratory i.e. resilient modulus, HMA mixture properties that is project specific or a library of test materials results.

Level 2. Input values are estimated for correlations and regression equations

Level 3. Input parameters are estimated or global defaults are used. (AASHTO et al., 2015)

There are several WMA properties that are needed to be inputted into the Pavement ME program to perform a Level 1 design. The RIDOT performs fifteen AASHTO and ASTM testing on its materials and warm mix asphalt as shown in Table 3-1 for all its construction projects. These tests are the MED inputs that are needed to predict longitudinal cracking, alligator cracking, transverse cracking, rutting or other permanent deformation, IRI, and reflection cracking over a selected design life.

3.1 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 et al. 1993).

3.2 Resilient Modulus of Existing Subbase.

Before the Full Depth Reclamation (FDR), four inches 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 five inches 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 3.2.

The laboratory resilient moduli values varied from 17, 000 psi to 74,000 psi. Subsequent material testing of the ten samples for the percent RAP content using AASHTO T 308, are shown in Table 2.5. For the 1980 old recycled mixture, the subbase samples have an asphalt content between fifteen and forty percent with most of the samples having a pavement content of twenty-five percent.

A grain size distribution, AASHTO T 27, Figure 3.2 sieve analysis shows the samples as relatively uniform (Bradshaw, et al., 2015). The samples show that the contractor performed a good job of reclaiming.

3.3 Resilient Modulus of New Full Depth Reclamation (FDR) of New Base/Subbase

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 3.3 (Bradshaw et al., 2015).

Six samples for the percent asphalt (RAP) content, maximum dry unit weight, optimum moisture content and Dry unit weight shown in Table 3.4 and a grain size distribution, Figure 3.3, show the samples as relatively uniform (Bradshaw et al., 2015).

3.4 Determination of Physical Properties of Asphalt Base and Surface Layers

The Route 165 project used two and one half inches of Class 19 WMA for the base layer and two inches of Class 12.5 WMA for the surface layer. Mechanical properties of WMA including Dynamic modulus, ($E_{HMA,}$ $E^{*)}$ for the surface and base layers were acquired from Villanova University and Cardi Corporation's WMA testing, and are shown in Appendix B. An example of the physical Properties of Class 19 WMA base layer is shown in Table 3.5, and are used as input parameters for MED software.

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 3.6. Creep compliance is a test for thermal low temperature cracking.

3.5 Traffic Spectrum Obtained from Weigh-in-Motion (WIM) Data

All truck traffic data inputted into the MED software came from a WIM station on Route 165, and was obtained from RIDOT Research Section. Data included average annual daily traffic (AADT) which is broken down into vehicle classification, monthly adjustment factors, hourly adjustment factors, daily vehicle counts and percent trucks in design direction. The average annual daily truck traffic (AADTT) is calculated for Class 4 to Class 13 vehicles. FHWA Vehicle Classifications according to their class are shown in Figure 3.3. 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).

Monthly traffic counts from December 2014 to November 2015 are broken down into truck traffic counts and percent vehicles for each class are shown in Table 3.7. Monthly traffic reports and percent trucks in design direction are shown in Appendix C.

3.6 Climate Data

Climate is an important parameter in the MED and has to be downloaded from (hcd) files from the website www.me-design.com. The download from the website comprises of climate data for all 50 states of United States from 1997 to 2005. (AASHTO is in the process of updating their files to current climate data this spring). The closest active weather station to Route 165 is at TF Green Airport, Warwick. The downloaded climate data includes monthly temperature, precipitation, sunshine, air temperature, maximum frost and wind speed. Summaries of the climate data are shown on pages six and seven of the MED output reports. Shown in Appendix A.

MED uses the climate data for transverse cracks (non-load cracks), enhanced integrated climate model (EICM) calculates the WMA temperatures on an hourly basis and MED uses those hourly temperatures to estimate the WMA properties (creep compliance and indirect tensile strength) to calculate the tensile stress throughout the WMA surface (AASHTO et al., 2015).

3.7 Prediction of Performance with AASHTOWare Pavement ME Design Software

In the present study, the five test sections were run using the MED. There are six design parameters needed and have been explained in Sections 3.1 through 3.6.

- Resilient Modulus

  Lab values from URI

- <u>HMA properties for Class 12.5 WMA and Class 19 WMA</u>

  Creep compliance

  Effective binder content

  Air voids

  Asphalt Dynamic modulus

  Asphalt Binder

- <u>Traffic</u>

  AADTT

  Vehicle classification

  Percent trucks in Design lane

- <u>Climate</u>

  Green Airport, Warwick

- <u>Projected pavement design life</u>

  Estimated

- <u>Pavement layer make up</u>

  Ground penetrating radar

Performance distress prediction outputs will include:

- AC bottom-up fatigue cracking (% lane area)
- AC top-down fatigue cracking (ACTDFC) (ft/mile)
- AC thermal cracking (ft/mile)
- Permanent deformation - AC only (in)
- Permanent deformation - total pavement (in)
- Terminal IRI (in/mile)

Distress or performance indicators terms for asphalt pavements are important to recognize and accurately identify them in the field.

AC bottom-up fatigue cracking (alligator cracking) is caused by repeated wheel loads and on pavements has an alligator pattern. They originate from the bottom of an asphalt layer and travel up to the surface. Bottom up cracking show up as multiple short, longitudinal or transverse cracks in the wheel path.

AC top-down fatigue cracking (top down cracking) is also caused by repeated wheel loads but the cracking is parallel to the pavement centerline. Longitudinal cracks originate from the top and go down. Raveling and/or rack deterioration can occur along the edges of these cracks. These cracks do not look like alligator cracks.

AC thermal cracking are non-wheel load cracking due to low temperatures or thermal cycling.

Permanent deformation - AC only (reflection transverse cracking) is a non-wheel load cracking induced by reflection from transverse joint or crack in underlying pavement.

Permanent deformation is a surface depression in the wheel path resulting from plastic or permanent deformation in each layer. The rut depth is representative of the maximum vertical difference in elevation between the transverse profile on the HMA surface and a wire-line across the lane width. MED also computes the rut depths within the HMA, unbound aggregate layers and foundation. (AASHTO et al., 2015).

Terminal IRI's functional adequacy is quantified by pavement smoothness for both flexible and rigid pavements. Rough roads lead to user discomfort but higher vehicle costs.

From Table 3.3, shows five confining stresses of 21 kPa, 35kPa, 69 kPa, 103 kPa, and 138 kPa. Each confining stress has three deviator stresses and three Mr values. The mean Mr values for the four test sections for the 35 kPa confining stress are used in this study. The 35 kPA was selected because the 21 kPA appears too low. In the future when the pavement is measured in the field and downloaded from dTIMS for the five distresses, a determination can be made for which confining stress is the best fit. The MED can be rerun with a higher confining pressure and the predictions re-evaluated. This method is how the MED will be calibrated for future projects.

3.7.1 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 2.2, a one inch of old 1980 recycled blend with the old $CaCl_2$ subbase material was assumed left and is represented in the MED.

Table 3.8 shows the MED output from using the Mr from laboratory testing. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The target and predicted values are from the MED control URI generated report page 1. Shown in Appendix A.

3.7.2 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 2.3, a one inch of old recycled blend with $CaCl_2$ is assumed left and is represented in the MED.

Table 3.9 shows the MED output from using the Mr from laboratory tests. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking). The target and predicted values are from the MED Calcium chloride URI generated report page 1. Shown in Appendix A.

3.7.3 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 and is represented in the MED, Figure 2.4.

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. 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, sample 4a is used in this study.

The MED was run with the layer thicknesses and Mr as shown below in Table 3.11. There are no predicted output failures. The target and predicted values are from the MED Portland Cement URI generated report page 1. Shown in Appendix A.

3.7.4 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 2.5.

The MED was run with the layer thicknesses and Mr as shown below in Table 3.10. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking).

3.7.5 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, Figure 2.6. For this test, the control material for sample 7b Mr mean values for confining stress of 35 kPa were used from Table 3.3.

The MED was run with the layer thicknesses and Mr as shown below in Table 3.12. There is one predicted design output failure for AC top-down fatigue cracking (longitudinal cracking).

3.7.6 Summary

The comparison between the control and the other four test sections are shown below in Table 3.13. 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 year target of two thousand feet per mile. The Portland cement section is the only test section that did not have any predicted distresses for twenty years. The higher the AC top down cracking, the earlier the threshold distress is noted. The years to predicted threshold distress comes from the third table on page thirteen of each of the test sections MED reports. Shown in Appendix A.

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

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. All the test sections predict that there will not be any permanent deformation in the subbase or AC layer, or AC bottom up fatigue cracking (alligator cracking). The higher resilient moduli, the better the results for less distresses.

Table 3. 1 Material Testing for Route 165

		Laboratory Unbound Materials	
AASHTO	T 88	Particle Size Analysis of Soils	URI
	T 265	Laboratory Determination of Moisture Content of Soils	URI
	T 307	Determining of Resilient modulus of Soils and Aggregate Materials	URI
ASTM	D2487	Classification of Soils for Engineering Purposes	URI
		Asphalt Binder	
AASHTO	T 315	Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)	RIDOT Materials Section
	T 316	Determination of Asphalt Binder Using Rotational Viscometer	RIDOT Materials Section
		Hot Mix Asphalt	
AASHTO	T 27	Sieve Analysis of Fine and Course Aggregates	RIDOT Materials Section
	T 84	Specific Gravity and Absorption of Fine Aggregates	RIDOT Materials Section
	T 85	Specific Gravity and Absorption of Course Aggregates	RIDOT Materials Section
	T 166	Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface Dry	RIDOT Materials Section
	T 209	Theoretical Maximum Specific Gravity (GMM) and Density of Hot Mix Asphalt Paving Mixtures	RIDOT Materials Section
	T 308	Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method	RIDOT Materials Section
	T 312	Preparing and Determining the Density of Asphalt Mixture Specimens by Means of the Superpave Gyratory Compactor	RIDOT Materials Section
	T 342	Determining Dynamic Modulus of Hot Mix Asphalt	RIDOT Materials Section
		In Situ Deflection Measurements (FWD)	
		Material Specification	
	M 320	Performance-Graded Asphalt Binder	Villanova University
	M 323	Superpave Volumetric Mix Design	Villanova University
		Standard Practices and Terminology	
	R 37	Application of Ground Penetrating Road Radar (GPR) to Highways	RIDOT Material Section
	R 43	Quantifying Roughness of Pavement	Vendor

Table 3.2 Resilient Moduli of Sampled 1980s RAP/Virgin Blends from Route 165 Site

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

(Bradshaw et al., 2015)

Bradshaw et al 2015)

σ_3 = confining stress

'σ_d = deviator (cyclic) stress = $\sigma_1 - \sigma_3$

'θ = bulk stress = $(\sigma_1 + \sigma_2 + \sigma_3)$

τ_{oct} = octahedral shear stress = $2^{.5}/3 * \sigma_d$

Table 3.3. Summary of resilient moduli of selected FDR with and without additives.

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

(Bradshaw et al., 2015)

σ_3 = confining stress

'σ_d = deviator (cyclic) stress = $\sigma_1 - \sigma_3$

'θ = bulk stress = $(\sigma_1 + \sigma_2 + \sigma_3)$

τ_{oct} = octahedral shear stress = $2^{.5}/3 * \sigma_d$

Table 3.4 Summary of Basic Properties of the FDR Blends

Sample	RAP Content (%)	Max. Dry Unit Weight (kN/m^3)	Optimum Moisture Content (%)	Dry Unit Weight (kN/m^3)
FDR-NoTreat	63	21.2	3.9	20.4
FDR-NoTreat-R	63	21.2	3.9	20.4
FDR-CaCl2-24hr	71	21.2	3.4	20.6
FDR-Emuls-5D	57	20.5	4.0	19.8
FDR-PC-4hr	61	20.7	3.4	20.0
FDR-PC-7D	61	20.7	3.4	20.0

(Bradshaw et al., 2015)

Table 3.5 Physical Properties of Class 19 HMA Base Layer

Binder Grade	PG 64-28V polymer modified with Evotherm, Warm Mix Asphalt	
Binder Content	Low	5.9
	Optimum	6.3
	High	6.8
Effective Binder Content	Low	5.519
	Optimum	5.921
	High	6.423

(RIDOT Material Section 2014)

Table 3.6 Creep Compliance of both Mixtures with Respect to Creep Time

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)

Table 3.7 Monthly Traffic Counts from Dec 2014-Nov 2015

Vehicle Class	4	5	6	7	8	9	10	11	12	13	Total
Dec-14	28	1333	298	93	202	1625	13	1	12		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%

(RIDOT Research Section, Dec 2014 –Nov 2015))

Table 3.8 Control Section (Cold Recycled) Distress Prediction Summary Using a Twenty Year Design Life

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	8.28	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	2,800.40	Fail
AC thermal cracking (ft/mile)	1,000.00	84.34	Pass
Permanent deformation - total pavement (in)	0.75	0.52	Pass
AC only permanent deformation (in)	0.25	0.07	Pass
Terminal IRI (in/mile)	172.00	147.61	Pass

Table 3.9 Calcium chloride ($CaCl_2$) Section Prediction Summary Using a Twenty Year Design Life

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	5.63	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	2,585.29	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	0.07	Pass
Terminal IRI (in/mile)	172.00	147.20	Pass

Table 3.10 Portland Cement Section Distress Prediction Summary using a Twenty Year Design Life

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	2.19	Pass
AC top-down fatigue cracking (ft/mile)	2,000.00	1,782.67	Pass
AC thermal cracking (ft/mile)	1,000.00	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.49	Pass
Permanent deformation - AC only (in)	0.25	0.07	Pass
Terminal IRI (in/mile)	172.00	145.42	Pass

Table 3.11 Asphalt Emulsion Prediction Summary Using a Twenty Year Design Life			
Design Inputs			
Layer Type	**Thickness**	**Laboratory**	**Mr (psi)**
Flexible	2		
Flexible	2.5		
Cold recycled (FDR RAP)	3	Bradshaw et al., 2015	31,420
Cold recycled (FDR RAP)	5	Bradshaw et al., 2015	26,440
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	17.38	Pass
AC top-down fatigue cracking (ft/mile)	2000	3502.66	Fail
AC thermal cracking (ft/mile)	1000	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.54	Pass
Permanent deformation - AC only (in)	0.25	0.07	Pass
Terminal IRI (in/mile)	172	149.55	Pass

Table 3.12 Geo-Grid Prediction Summary Using a Twenty Year Design Life

Design Inputs

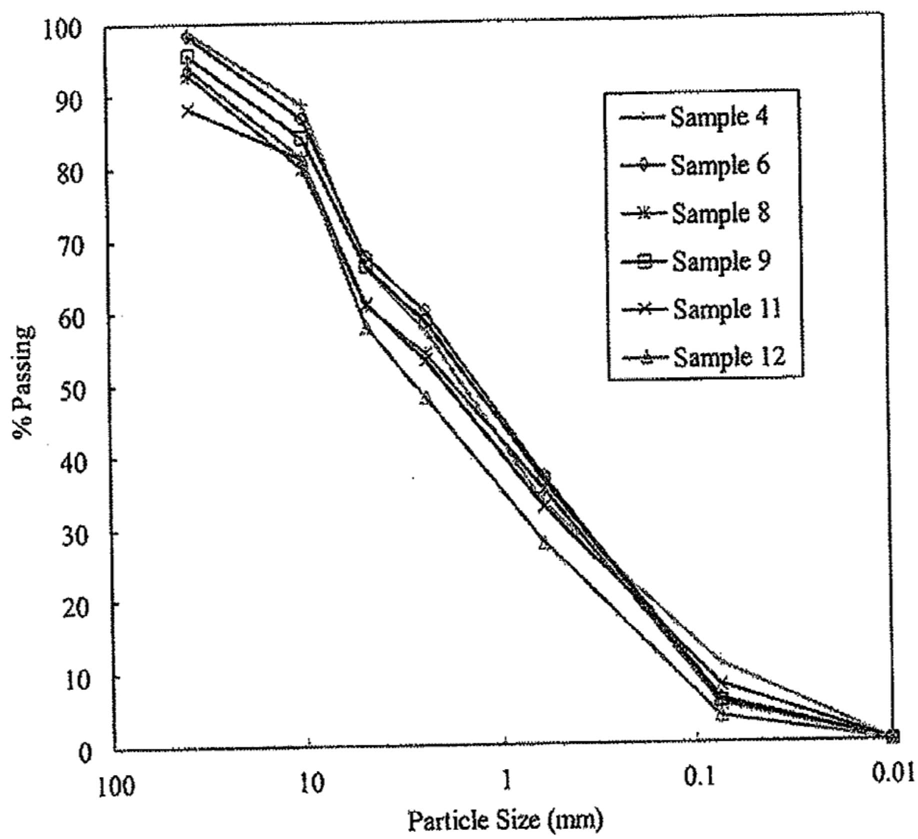
Layer Type	Thickness	Laboratory	Mr (psi)
Flexible	2		
Flexible	2.5		
Cold recycled (FDR RAP)	10	Bradshaw et al., 2015	30,040
Cold recycled (FDR RAP)	5	Bradshaw et al., 2015	13,620
Subgrade	Semi-infinite	Lee et al., 2003	9304

Design Outputs

Distress	Target	Predicted	Criterion Satisfied
AC bottom-up fatigue cracking (% lane area)	25	16.66	Pass
AC top-down fatigue cracking (ft/mile)	2000	3,367.31	Fail
AC thermal cracking (ft/mile)	1000	84.84	Pass
Permanent deformation - total pavement (in)	0.75	0.51	Pass
Permanent deformation - AC only (in)	0.25	0.07	Pass
Terminal IRI (in/mile)	172	148.35	Pass

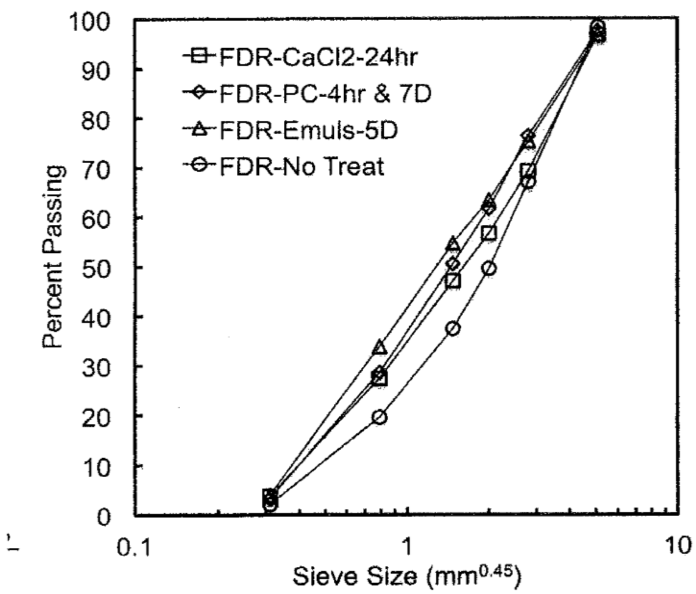
Table 3.13 Comparison of Performance Predictions						
Design Outputs		Control	$CaCl_2$	Asphalt Emulsion	Portland Cement	Geo-Grid
Distress	**Target**	**Predicted**	**Predicted**	**Predicted**	**Predicted**	**Predicted**
AC bottom-up fatigue cracking (% lane area)	25.00	8.28	5.63	17.38	2.85	16.66
AC top-down fatigue cracking (ACTDFC) (ft/mile)	2,000.00	2,800.40	2,585.29	3,502.66	1,782.67	3,367.31
AC thermal cracking (ft/mile)	1,000.00	84.34	84.34	84.84	84.34	84.34
Permanent deformation - AC only (in)	0.25	0.07	0.07	0.07	0.07	0.07
Permanent deformation - total pavement (in)	0.75	0.52	0.51	0.54	0.49	0.51
Terminal IRI (in/mile)	172.00	147.61	147.20	149.55	145.42	148.35
Years to predicted threshold distress ACTDFC (Years)		8	10	4.0	25	5

Figure 3.1 Grain Size Distribution for all 1980 RAP/Virgin Blend Samples



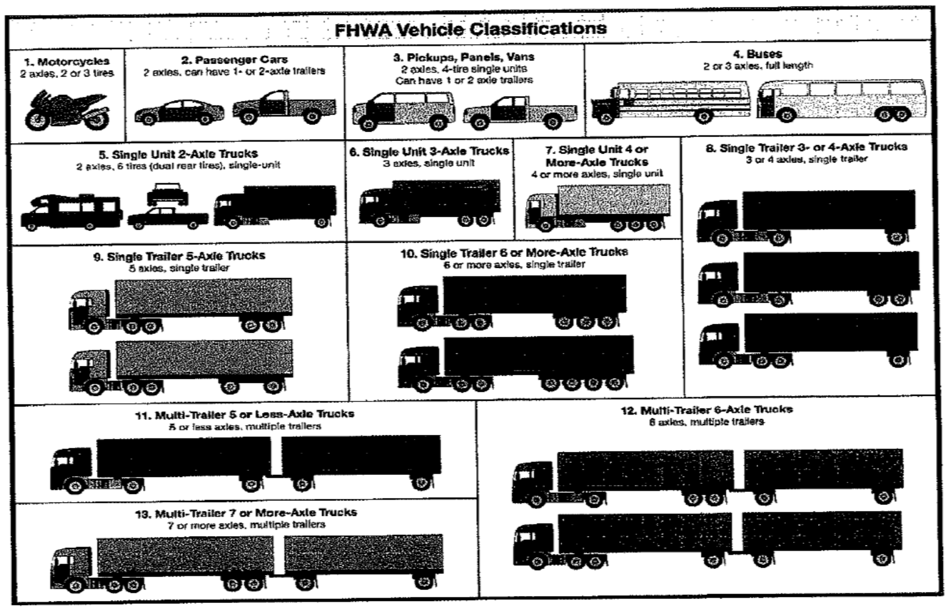
(Bradshaw et al., 2015)

Figure 3.2 Grain Size Distribution for all FWD Samples



(Bradshaw et al., 2015)

Figure 3.3 Vehicle Classification according to FHWA



Http://onlinemanuals.txdot.gov/txdotmanuals/tri/images/FHWA_Classification_Chart_Final.png

Chapter 4

Evaluation of Performance of Rehabilitated Asphalt Pavement

4.1 Surface Distresses and Field Condition Survey

On December 21, 2015, pavement windshield surveys were conducted on Route 165 by both URI Graduate students and Professor K. Wayne Lee and the results of these surveys are shown in Appendix D. 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, the Rhode Island Department of Transportation 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), respectively; and structured table outputs with column and rows.

(Lecture – Pavement Management System at RIDOT, 10/15/15). A copy of the Distress report is also in Appendix D. Below is an accounting of the distress found:

4.1.1 Permanent Deformation and Rutting

No deformation or rutting to report.

4.1.2 Fatigue Cracking

No fatigue cracking to report.

4.1.3 Thermal Cracking

No thermal cracking to report.

4.2 Serviceability and Roughness

4.2.1 International Roughness Index

MED predicts terminal International Roughness Index (IRI) or smoothness in inches per mile. Using parameters which are input into the program, the target, predicted, and reliability models are calculated and are shown on page one of the MED reports. The targeted value of 172 inch/miles is considered the upper limit found acceptable by AASHTOWare, IRI values under 95 inch/mile are ranked as acceptable. Table 4.1 shows both the results of the individual test pavement reports from MED and the final IRI which were done by the RIDOT Material Section for bonus/penalty performance. Page 12 of each of the MED reports (Refer to Appendix A) show that the starting IRI is between 70 and 95 inch/mile. The contractor's final IRI, shown in Appendix E, came in as low as 58 inch/mile to 88 inch/mile for the west bound lane, as available. All of the test sections passed and the MED predicts the IRI will not reach the upper 172 inch/mile limit for approximately twenty years. The contactor received a bonus for his IRI results.

Currently, RIDOT performs regular IRI tests on its roadways to monitor pavement performance over time. Based on pavement performance and distress type, Page 12 of the MED Analysis Output Charts show the IRI values increasing over time. Thus, based on these distresses and calculated MED 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 MED.

4.3 Structural Capacity and Deflection

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. Until now, an unexpected discovery was made on interpreting the FWD deflections through the Long Term Pavement Performance Hogg Method (Refer to Appendix F).

The results of the resilient moduli from both Figures 3.3 and the Hogg Method shown in Table 4.2 show promise but warrant further evaluation. It is assumed that the better Mr values from the Hogg Method are due to the calibrated sensors and computer upgrades.

The Hogg Method uses a Long Term Pavement Performance (LTPP) Excel spreadsheet to calculate moduli which can then be used to predict pavement performance. Deflection data, corresponding back-calculated moduli, or other deflection-based parameters strongly relate to pavement performance, and the premise of mechanistic-empirical design methods is to control stresses and strains as a response to traffic." (LTPP, 2006). The Hogg Method is based on the work of J. Boussinesq, 1885, whose equations estimate vertical soil stresses:

$$\theta_z = Q(3z^3)/2\pi\,(R^2 + z^2)^{5/2} \qquad 1$$

Q = point load

z = depth from ground surface to the place where θ_z is desired

R = horizontal distance from point load to the place where θ_z is desired.

There are cautions, however, with the use of the Hogg Method which state that the forward-calculated modulus data is not intended to replace back-calculated or any other form of modulus of elasticity measurements. The question becomes how realistic are the estimates for pavement evaluation and/or design? There are four approaches to evaluate and determine in situ stiffness, and distresses (LTPP, 2006) and they are (1) the triaxial tests, (2) AASHTOPavement ME design, (3) dTIMS and the (4) Hogg Method. The dTIMS results can be compared to the predictions of the MED which in turn can be used back calculate the Mr. The Mr values can then be verified by laboratory testing and the Hogg Method. Any previously reclaimed roadway which had been reclaimed can be de-constructed, if you will, to verify all parameters.

4.4 Analysis of Ground Penetrating Radar (GPR)

Ground penetrating Radar (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 WMA 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. (Pavement thickness is an input parameter within both the Hogg Method and MED). (Refer to Appendix G)

4.5 Comparative Analysis between Performance Prediction and Evaluation

After the MED reports for the five test sections were run, the MED output showed AC top down fatigue cracking (longitudinal cracking) as the only output distress in four of the five test sections (Table 4.3). The predicted distress was over the target or threshold value. The target value represents the amount of distress that would trigger some type of major rehabilitation activity (AASHTO, 2015). According to the MED report, using the asphalt emulsion and Portland cement test sections, for example, it would take five years to reach the two thousand feet per mile for AC top down cracking for the asphalt emulsion but twenty years for Portland cement. AC top down cracking is due to the pavement layer being too thin and a low Mr. In the test sections as shown in Table 4.4 to Table 4.7, extra pavement thickness, cost analysis, and years to failure, MED inputs were re-run to see what would cause the predicted AC top down cracking to go below the two thousand target threshold.

Thus, based on the aforementioned, we know that it will take a number of years before the threshold distress can be predicted for the five test sections i.e control, $CaCl_2$, asphalt emulsion, Portland cement, and geo-grid. We do know that year number eight is the "unofficial" time where pavements start showing distress and will most likely need to receive some form of maintenance treatment e.g crack sealing; chip seal. That said, the asphalt emulsion and geo-grid test sections are predicted to need treatment in four and five years in this study, with predicted AC top down cracking of 3,502 ft/mile and 3,367 ft/mile.

It was surprising to see how high the AC top down fatigue cracking number in the asphalt emulsion test section because the material has been successfully used as an FDR additive in many states.

4.5.1 Control Test Section

According to Table 4.4, the control section was predicted to reach a threshold distress in AC top down cracking in eight years. If the pavement thickness had been increased by one inch, all of the predicted values would have been below the target thresholds for twenty years. The down side to this would have been the approximately $526,000 in additional costs for 8,552 tons of asphalt pavement. The yearly cost for eight years is $332,676 and $159,368 for twenty years. The total cost with the extra asphalt pavement, the control test section ranks two out of five.

4.5.2 Calcium Chloride Section

The $CaCl_2$ section is predicted to reach a threshold of 2,585 linear feet of AC top down fatigue cracking (longitudinal cracking) in ten years see Table 4.5. If the pavement thickness had been increased by one inch, all of the predicted values would have been below the target thresholds for twenty years. The down side to this would have been the approximately $526,000 in additional costs for 8,552 tons of asphalt pavement. The yearly cost for ten years is $364,715 and $208,655 for twenty years. The total cost with the extra pavement, the calcium chloride section ranks four out of five.

4.5.3 Portland Cement Section

In Table 4.6, the Portland cement section, which had the highest Mr of any of the other sections, should not show any distress until roughly year thirty and it is in the terminal IRI (in/mile). In spite of this, the cement was not allowed to fully cure for seven days and thus micro-cracked after four hours. From Table 3.5, the Mr would have been 126,000 psi (mean of the Mr for the 35 kPa confining stress) if traffic had not been allowed on the travel lane. Both the width of the road and the traffic volumes prevented one lane from being closed for the seven days since a detour was not feasible. The cost for twenty-five years would be $158,206. The total cost with the extra pavement, the Portland cement section ranks first out of five.

4.5.4 Asphalt Emulsion Section

According to Table 4.7, the asphalt emulsion section was predicted to reach a threshold distress in AC top down cracking in four years. If the pavement thickness had been increased by one and a half inch, all of the predicted values would have been below the target thresholds. The down side to this would be the increased cost of 12,828 tons of asphalt needed to reach the additional thickness. The $788,000 is a high price to pay for fifteen additional service years. But the cost for four years would be $742,351 and $187,916 for twenty years. The total cost with the extra pavement, the calcium chloride section ranks three out of five.

4.5.5 Geogrid Section

According to Table 4.8, the geo-grid section was predicted to reach a threshold distress in AC top down cracking in five years. If the pavement thickness had been increased by one and a half inch, all of the predicted values would be below the target thresholds and the pavement would not see any predicted cracking for twenty years. The down side would be the increased cost of $788,000.00 for 12,828 tons of asphalt pavement. The up side, however, would be an increase in fifteen additional service years. The yearly cost for ten years is $1,326,543 and $357,933 for twenty years. The total cost with the extra pavement, the geo-grid section ranks five out of five.

4.6 Selection of Best Alternatives Based on Short-Term Evaluation

4.6.1 Forecasting Future Performance through Tie-ins with Pavement Structural Health Index (PSHI)

MED can fit within the states' preservation system by using performance indicators that dTIMS does not. For instance, dTIMS catalogues IRI, rutting, cracking and deformation through yearly field surveys, while MED uses AADTT, resilient modulus, pavement layer make-up, HMA properties, and climate to predict the same pavement distresses over time. DTIMS surveys the surface course and MED predicts the subsurface conditions. Thus, MED predictions can be adjusted accordingly.

4.6.2 Optimal Strategies for Rehabilitation

One finding of the MED, is determining the right combination of subbase Mr to pavement thickness. MED makes it very easy to run multiple models for worst and best case scenarios. Resurfacings and reclamations should have Mr values checked before construction to find if an additive would benefit the subbase stiffness. Monitoring pavement PSHI can catch a road before it deteriorates too far, but the subbase Mr should be evaluated before treatment is determined. For example, too many times RIDOT has milled two inches and put back two inches of pavement on a road only to have the pavement break up in a short amount of time. The MED reported output shows how longitudinal cracking can be a sign of too thin of a pavement thickness.

4.7 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 falling weight deflectometer (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.

The Materials Sections has equipment that performs tests that the Design Engineer needs to incorporate into their designs. A list of capabilities of this equipment should be shared.

RIDOT's Pavement Committee, currently made up of personnel from the Materials, Design, and Construction Sections, oversees all pavement designs on both reconstruction and resurfacing projects. Usually the only information available/discussed during the meetings include sieve analysis data, pavement cores, and traffic AADT. It is a mistake not to have subbase Mr and MED reports at that meeting.

Table 4.1 Comparison of Performance Predictions for IRI						
Design Outputs		Control	$CaCl_2$	Asphalt Emulsion	Portland Cement	Geo-grid
Distress	**Target**	**Predicted**	**Predicted**	**Predicted**	**Predicted**	**Predicted**
Terminal IRI (in/mile)	172.00	144.78	143.93	145.39	142.65	144.00
Average Final rideability Results Rte 165 - WB (in/mi) Left/right lane. Initial IRI = 63		58.4/46.9	44/40.43	65.8/88.9	64.3/52.9	63.7/56.7
Values from the RIDOT Material Section						

Table 4. 2 Comparison Between subbase URI Lab Results vs Hogg Method FWD Results

Material Type	Sample	Lab results* Range in psi Confining Stress 21 kPa to 138 kPa	Average lab Results psi	Hogg Model Range in psi	Average Hogg Method Mr - psi
Cold Recycled	7a	25,961 - 58,015	41,988	33,961-62,284	51,942
Cold Recycled	7b	34,809 - 83,831	59,320	33,961-62,284	51,942
CaCl	1	36,549 - 82,381	59,465	47,749-62,608	53,598
Cement 4 hour	4a	55,984 - 107,037	81,510	40,940-58,273	50,486
Cement 7 day	4b	76,579 - 275,281	175,930	40,940-58,273	50,486
Asphalt Emulsion	5	25,816 -53,808	39,812	36,596-47,076	41,959
Geo-Grid		Not Tested		23,164 - 40,997	34,414

* Bradshaw, Costa, Giampa, Genovesi, and Hernberg, 2015

Table 4.3 Comparison of Years to Predicted Threshold Distress						
Design Outputs		Control	CaCl2	Asphalt Emulsion	Portland Cement	Geo-Grid
Distress	**Target**	**Predicted**	**Predicted**	**Predicted**	**Predicted**	**Predicted**
AC bottom-up fatigue cracking (% lane area)	25.00	8.28	5.63	17.38	2.85	16.66
AC top-down fatigue cracking (ACTDFC) (ft/mile)	2,000.00	2,800.40	2,585.29	3,502.66	1,782.67	3,367.31
AC thermal cracking (ft/mile)	1,000.00	84.34	84.34	84.84	84.34	84.34
Permanent deformation - AC only (in)	0.25	0.07	0.07	0.07	0.07	0.07
Permanent deformation - total pavement (in)	0.75	0.52	0.51	0.54	0.49	0.51
Terminal IRI (in/mile)	172.00	147.61	147.20	149.55	145.42	148.35
Years to predicted threshold distress ACTDFC (Years)		8	10	4	25	5

Table 4.4 Control Section (Cold Recycled) - Pavement Performance

Design Outputs		2.5" Class 19 WMA Base	Additional 1" of Class 19 WMA Base (3.5")
Distress	Target	Predicted	Predicted
AC bottom-up fatigue cracking (% lane area)	25.00	8.28	2.01
AC top-down fatigue cracking (ft/mile)	2,000.00	2,800.40	1,840.61
AC thermal cracking (ft/mile)	1,000.00	84.34	84.34
Permanent deformation - total pavement (in)	0.75	0.52	0.47
AC only permanent deformation (in)	0.25	0.07	0.07
Terminal IRI (in/mile)	172.00	147.61	144.71
1" Additional tons of asphalt for 7 miles		0	4,276.00
Class 19 WMA @ $61.50/ton cost		0	$262,974.00
Years to predicted threshold distress ACTDFC (Years)		8	20
Additional years of Service		0	12

Cost Analysis per Year for the Control Section (Cold Recycled) (2.5")

Unit cost		Item	Quantity		Amount
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,238,303
$61.50	2.5"	Class 19 WMA	20,135	Ton	$1,238,303
$1.50	8"	Subbase	123,200	SY	$184,800
		Total			$2,661,405
		8 year Life cycle cost / year			$332,676

Cost Analysis per Year for the Control Section (Cold Recycled) (3.5")

Unit cost	Pavement Thickness	Item	Quantity	Unit	Amount
$92.50	2"	Class 12 WMA *	20,135	Ton	$1,238,303
$61.50	3.5"	Class 19 WMA	28,687	Ton	$1,764,251
$1.50	8"	Subbase	123,200	SY	$184,800
		Total			$3,187,353
		20 year Life cycle cost /year			$159,368

* WMA quantity had extra tons for berm and paved waterways.

Figure 4.5 Calcium Chloride Pavement Performance

Design Outputs		2.5" Class 19 WMA Base	Additional 1" of Class 19 WMA Base (3.5")
Distress	**Target**	**Predicted**	**Predicted**
AC bottom-up fatigue cracking (% lane area)	25.00	5.63	1.94
AC top-down fatigue cracking (ft/mile)	2,000.00	2,585.29	1,635.07
AC thermal cracking (ft/mile)	1,000.00	84.84	84.84
Permanent deformation - total pavement (in)	0.75	0.51	0.47
Permanent deformation - AC only (in)	0.25	0.07	0.07
Terminal IRI (in/mile)	172.00	147.20	144.47
Years to predicted threshold distress ACTDFC (years)		10.00	20.00
1 " Additional tons of asphalt for 7 miles		0.00	8,552.00
Class 19 WMA @ $61.50/ton		0	$525,948.00
Additional years of service		0	10

Cost Analysis per Year for Calcium Chloride (2.5")

Unit cost	**Pavement Thickness**	**Item**	**Quantity**	**Unit**	**Amount**
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,977,650.00
$61.50	2.5"	Class 19 WMA	20,135	Ton	$1,238,302.50
$3.50	8"	Subbase	123,200	SY	$431,200.00
		Total			$3,647,152.50
		10 year Life cycle cost / Year			**$364,715.25**

Cost Analysis per Year for Calcium Chloride (3.5")

Unit cost	**Pavement Thickness**	**Item**	**Quantity**	**Unit**	**Amount**
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,977,650.00
$61.50	3.5"	Class 19 WMA	28,687	Ton	$1,764,250.50
$3.50	8"	Subbase	123,200	SY	$431,200.00
		Total			$4,173,100.50
		20 year Life Cycle Cost /Year			**$208,655.03**

* WMA quantity had extra tons for berm and paved waterways.

Figure 4.6 Portland Cement Pavement Performance for 25 years

Design Outputs

Distress	Target	Predicted (2.5" Class 19 WMA Base)
AC bottom-up fatigue cracking (% lane area)	25.00	2.80
AC top-down fatigue cracking (ft/mile)	2,000.00	1,990.66
AC thermal cracking (ft/mile)	1,000.00	84.84
Permanent deformation - total pavement (in)	0.75	0.51
Permanent deformation - AC only (in)	0.25	0.08
Terminal IRI (in/mile)	172.00	160.78
Years to predicted threshold distress ACTDFC (years)		25.00
Additional tons of asphalt		0.00
Additional cost per ton $61.50		0
Additional years of service past 20 years		5

Cost Analys	Cost Analysis per Year for the Portland Cement (2.5")				
Unit cost	**Pavement Thickness**	**Item**	**Quantity**	**Unit**	**Amount**
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,977,650.00
$61.50	2.5"	Class 19 WMA	20,135	Ton	$1,238,302.50
$6.00	8"	Subbase	123,200	SY	$739,200.00
		Total			$3,955,152.50
	25 year Life cycle cost / Year				**$158,206.10**
* WMA quantity had extra tons for berm and paved waterways.					

Table 4.7 Asphalt Emulsion Pavement Performance

Design Outputs		2.5" Class 19 WMA Base	Additional 1.5" of Class 19 WMA Base (4")
Distress	**Target**	**Predicted**	**Predicted**
AC bottom-up fatigue cracking (% lane area)	25.00	17.38	1.95
AC top-down fatigue cracking (ft/mile)	2,000.00	3502.66	1,744.64
AC thermal cracking (ft/mile)	1,000.00	84.84	84.84
Permanent deformation - total pavement (in)	0.75	0.54	0.46
Permanent deformation - AC only (in)	0.25	0.07	0.07
Terminal IRI (in/mile)	172.00	149.55	144.38
Years to predicted threshold distress ACTDFC (years)		4.00	20.00
1.5 " Additional tons of asphalt for 7 miles		0.00	12,828.00
Class 19 WMA @ $61.50/ton cost		0	$788,922.00
Additional years of service		0	16

Cost Analysis per Year for the Asphalt Emulsion (2.5")

Unit cost	Pavement Thickness	Item	Quantity	Unit	Amount
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,238,302.50
$61.50	2.5"	Class 19 WMA	20,135	Ton	$1,238,302.50
$4.00	8"	Subbase	123,200	SY	$492,800.00
		Total			$2,969,405.00
		4 year Life cycle cost / Year			**$742,351.25**

Cost Analysis per Year for the Asphalt Emulsion (4")

Unit cost	Pavement Thickness	Item	Quantity	Unit	Amount
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,238,302.50
$61.50	4"	Class 19 WMA	32,963	Ton	$2,027,224.50
$4.00	8"	Subbase	123,200	SY	$492,800.00
		Total			$3,758,327.00
		20 year Life Cycle Cost /Year			**$187,916.35**

* WMA quantity had extra tons for berm and paved waterways.

Table 4.8 Geo-Grid Pavement Performance for 20 years

Design Outputs

		2.5" Class 19 WMA Base	Additional 1.5" of Class 19 WMA Base (4")
Distress	**Target**	**Predicted**	**Predicted**
AC bottom-up fatigue cracking (% lane area)	25.00	16.66	1.94
AC top-down fatigue cracking (ft/mile)	2,000.00	3,367.31	1,734.15
AC thermal cracking (ft/mile)	1,000.00	84.84	84.34
Permanent deformation - total pavement (in)	0.75	0.51	0.45
Permanent deformation - AC only (in)	0.25	0.07	0.07
Terminal IRI (in/mile)	172.00	148.35	143.69
Years to predicted threshold distress ACTDFC (years)		5.00	20.00
1.5 " Additional tons of asphalt 7 miles		0.00	12,828.00
Class 19 WMA @ $61.50/ton cost		0	$788,922.00
Additional years of service		0	15

Cost Analysis per Year for the Geo-Grid Section (2.5")

Unit cost	Pavement Thickness	Item	Quantity	Unit	Amount
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,238,302.50
$61.50	2.5"	Class 19 WMA	20,135	Ton	$1,238,302.50
$1.50	10"	Cold recycled (FDR RAP)	32,963	SY	$49,444.50
$15.00		Geo-grid Layer	246,400	SY	$3,696,000.00
$20.00	6"	Filter stone	20,533	CY	$410,666.67
		Total			$6,632,716.17
		5 year Life Cycle Cost / Year			$1,326,543.23

Cost Analysis per Year for the Geo-Grid Section (4")

Unit cost	Pavement Thickness	Item	Quantity	Unit	Amount
$92.50	2"	Class 12 WMA *	21,380	Ton	$1,238,302.50
$61.50	4"	Class 19 WMA	28,687	Ton	$1,764,250.50
$1.50	10"	Cold recycled (FDR RAP)	32,963	SY	$49,444.50
$15.00		Geo-grid Layer	246,400	SY	$3,696,000.00
$20.00	6"	Filter Stone	20,533	CY	$410,666.67
		Total			$7,158,664.17
		20 year Life Cycle Cost /Year			$357,933.21

* WMA quantity had extra tons for berm and paved waterways.

Table 4.9 Comparison of Years of Predicted Threshold Distress and Cost

Design Outputs		Control	CaCl2	Asphalt Emulsion	Portland Cement	Geo-Grid
Distress	**Target**	**Predicted**	**Predicted**	**Predicted**	**Predicted**	**Predicted**
Class 19 WMA as built		**2.5"**	**2.5"**	**2.5"**	**2.5"**	**2.5"**
AC bottom-up fatigue cracking (% lane area)	25.00	8.28	5.63	17.38	2.85	16.66
AC top-down fatigue cracking (ACTDFC) (ft/mile)	2,000.00	2,800.40	2,585.29	3,502.66	1,782.67	3,367.31
AC thermal cracking (ft/mile)	1,000.00	84.34	84.34	84.84	84.34	84.34
Permanent deformation - AC only (in)	0.25	0.07	0.07	0.07	0.07	0.07
Permanent deformation - total pavement (in)	0.75	0.52	0.51	0.54	0.49	0.51
Terminal IRI (in/mile)	172.00	147.61	147.20	149.55	145.42	148.35
Years to predicted threshold distress ACTDFC (Years)		8	10	4	25	5
Cost of pavement structure (Class 12.5, Class 19, subbase)		$2,661,405	$3,647,153	$2,969,405	$3,955,153	$6,632,716
Cost of pavement structure/ACTDFC per year for 7 miles		$332,676	$364,715	$742,351	$158,206	$1,326,543

Table 4.10 Cost Comparison for Thicker Class 19 Base Course						
Design Outputs		Control	CaCl2	Asphalt Emulsion	Portland Cement	Geo-Grid
Distress	**Target**	**Predicted**	**Predicted**	**Predicted**	**Predicted**	**Predicted**
Class 19 WMA revised thickness		**3.5"**	**3.5"**	**4"**	**2.5"**	**4"**
AC bottom-up fatigue cracking (% lane area)	25.00	2.01	1.94	1.95	2.85	1.95
AC top-down fatigue cracking (ACTDFC) (ft/mile)	2,000.00	1,840.61	1,635.07	1,744.64	1,782.67	1,744.64
AC thermal cracking (ft/mile)	1,000.00	84.34	84.84	84.84	84.34	84.84
Permanent deformation - AC only (in)	0.25	0.47	0.47	0.46	0.07	0.46
Permanent deformation - total pavement (in)	0.75	0.07	0.07	0.07	0.49	0.07
Terminal IRI (in/mile)	172.00	144.71	144.47	144.38	145.42	144.38
Years to predicted threshold distress ACTDFC (Years)		20	20	20	25	20
Cost of pavement structure (Class 12.5, Class 19, subbase)		$3,187,353	$4,173,101	$3,758,327	$3,955,153	$7,158,664
Cost of pavement structure/ACTDFC per year for 7 miles		$159,368	$208,655	$187,916	$158,206	$357,933

Chapter 5

Conclusions and Recommendations

5.1 Findings

Five test sections with FDR and various additives were established on Rt. 165 in Exeter, RI to study the performance of the road with the help of the Rhode Island Department of Transportation (RIDOT). This study included research, laboratory triaxial testing of subbase materials and subgrade soils, rehabilitation performance predictions by AASHTOWare Pavement ME Design software, and a condition survey to come up with long term performance monitoring and optimum pavement rehabilitation strategies.

There were several findings while investigating the performance of the five test sections.

Pavement Distress and Cost

Of the five test sections, the section with Portland cement performed the best overall in having the least amount of distresses and the longest predicted service life; next is calcium chloride, followed by cold recycled (control), geo-grid, and asphalt emulsion. Table 4.8 shows the comparison of years of predicted threshold distress and cost for AC top down fatigue cracking (ACTDFC) in feet per mile. The AC top-down fatigue cracking and predicted years were obtained from the AASHTOWare Pavement ME design software using the lab tested resilient moduli. According to the prediction models, in Table 4.8, the Portland cement section was the only section that doesn't fail in the ACTDFC. In addition, is predicted to last over twenty-five years. The other test sections are predicted to reach the ACTDFC between five and ten years. The costs of the pavement structure for all the sections were between 2.6 million to 6.6 million dollars. The asphalt emulsion and geo-grid sections are not cost effective since they are predicted to reach threshold distress in a very short amount of time.

<u>Permanent Deformations, AC Bottom Up, Fatigue Cracking and Thermal Cracking</u>

None of the test sections are predicted to have permanent deformation, AC bottom up fatigue, or thermal AC cracking for over a twenty year period as shown in Table 4.3. Terminal IRI is predicted to be sixteen percent below the target goal for twenty years. Other distresses such as permanent deformation in the asphalt layer and AC bottom up fatigue cracking are predicted not to be present for twenty years. As discussed, AC top down fatigue cracking fails in all of the test sections except for Portland cement.

<u>Portland Cement Section</u>

It appears that Portland cement is an excellent additive, but 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. Portland cement should be considered for future projects only where a detour is feasible. Route 165 has

150 heavy trucks per day, and would benefit greatly from a more durable pavement like Portland cement. However this roadway could not support a detour therefore used alternating traffic was used for construction.

Resilient Modulus

The various test sections had different lab calculated resilient moduli. The higher the Mr value used, the less AC top down fatigue cracking. When comparing Tables 3.2 and 4.3, the pavement layers and reclamation depths are held constant. The only real change is the resilient modulus and the only distress that failed is the AC top down fatigue cracking. Portland cement has the highest Mr values and the predicted cracking is not expected to exceed the threshold for 25 years.

Condition Survey

Condition surveys from the dTIMS database can be used to verify the MED predictions of the five test sections. For this project, a windshield survey was completed after construction in 2015 and an automated survey for download into dTIMS. RIDOT will track the pavement performance for years to determine the best performance section.

Pavement Performance

Table 4.9 compares the predicted threshold distress and cost over twenty years for thicker Class 19.0 WMA base courses comparing the five test sections. The control, calcium chloride, asphalt emulsion and geo-grid test sections would be able to pass all distress targets over twenty years by increasing the thickness of the Class 19 WMA by one inch to one and a half inches. Table 4.9 provides material costs and the total cost per year for all the test sections. Table 4.8

compares the cost/year of each test section on Route 165. The order from least expensive to most expensive section is Portland cement, calcium chloride, control, asphalt emulsion, and geo-grid. Compare that to Table 4.9 for costs associated with the ideal pavement thickness and the order changes slightly as follows: Portland cement, control, calcium chloride, asphalt emulsion and geo-grid.

5.2 Recommendations

There are several recommendations that will be presented to RIDOT on how to increase pavement performance. They are:

1. Perform triaxial tests and FWD testing on subbase material on future FDR projects in the planning stage of design. Any subbase material which has less than 25,000 psi of resilient modulus should be modified with an additive to increase its stiffness. The number of triaxial tests and FWDs can be determined by how distressed the pavement is. The poorer the pavement condition, the more testing should be done to determine the cause of failure.
2. MED and PHSI have great potential to predict and monitor future performance of FDR roads. Thus, RIDOT may consider using both tools for the design of new reclaimated pavement structures.
3. The Hogg Method should be investigated to see if its results can be used as MED input data. The Excel program can be modified and calibrated to lab results to better fit the state's subbase material. Triaxial tests can also be used to verify the Hogg Method results.
4. It is recommended that Portland cement should be used as an additive if there is heavy truck traffic and the pavement is wide enough to support detours or lane closures for at least the seven day recommended curing time without traffic loads.
5. Asphalt emulsion could be investigated further on another state road. Increasing the depth of the layer that contains emulsion from three inches to five or six inches could improve

performance, but it will increase the cost. We could analyze our predictions against the condition survey data from dTIMS to see how close the predictions were to the MED.

6. RIDOT leases the MED program from AASHTO on a yearly basis and should be renewed. Earlier versions of the MED were slow, but new software provides not only important prediction information but also runs in less than five minutes. The information that can be derived from MED can be highly useful if dTIMS, GPR, triaxial tests and/or FWDs are used in conjunction with the MED.
7. Collecting the data for the MED inputs took two weeks for all the test sections. All the information to do a Level 1 analysis is available but it is scattered throughout the RIDOT Departments. A library of pavement material, subbase resilient moduli, AADTT, cores, and GPR can improve collection time and prevent unnecessary extra testing.
8. If the RIDOT's falling weight deflectometer and ground penetrating radar are to be used to determine the pavement stiffness and pavement thickness, these machines need to be calibrated on a regular basis and receive scheduled maintenance.

5.3 Conclusions

Overall, the Portland cement test section has the highest predicted performance, but is difficult to construct due to curing time and traffic detours on Route 165. Calcium chloride and the control sections were least expensive, but needed to have a thick base course which would increase cost. Geo-grid is typically used in poor drainage areas with high water tables on highways and has a high cost. Asphalt emulsions can work if either the asphalt emulsion is mixed to a deeper depth or an additional inch and a half of base course is used.

It should be interesting to see in the coming years how well the MED predictions for the test sections compare with the future distresses.

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