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DEVELOPMENT OF A SUSTAINABLE TRANSPORTATION INFRASTRUCTURE MANAGEMENT SYSTEM FOR A TYPICAL COLLEGE CAMPUS

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DEVELOPMENT OF A SUSTAINABLE TRANSPORTATION INFRASTRUCTURE MANAGEMENT SYSTEM FOR A TYPICAL COLLEGE CAMPUS

BY

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ABSTRACT

An attempt was made to develop a sustainable Transportation Infrastructure Management System (TIMS) for a typical college campus and/or any low volume road network. First, a sustainable Pavement Management System (PMS) was developed using different Maintenance and Rehabilitation (M&R) techniques. MicroPAVER^{TM} PMS software was utilized to analyze the present condition and to predict the future condition of pavement for the University of Rhode Island (URI) Kingston campus. Economic analysis was performed using the default rates of MicroPAVER^{TM} for different M&R policies and compared with the ones used in one of typical municipalities, i.e., The City of Cranston, Rhode Island (RI).

Secondly, model input parameters were formulated for the American Association of State Highway and Transportation Officials (AASHTO) Mechanistic-Empirical Pavement Design Guide (MEPDG) to use in RI, because having a strong initial pavement is essential for sustainable PMS. Superpave specifications were used to generate material parameters to design the Upper College Road as a model pavement structure. Then energy efficient and environment friendly M&R strategies were explored to introduce into the developed PMS, i.e., Cold In-Place Recycling (CIR) and Warm Mix Asphalt (WMA). A series of tests were conducted to predict their performances, e.g., thermal cracking, fatigue cracking and permanent deformation or rutting. Finally, a Geographic Information System (GIS) was used as a tool for automated drafting and data storage. Maps generated with different color coding will assist decision makers quicker and better interpretation.

The results of the present study can be used for any pavement networks particularly with low traffic volumes, e.g., cities and towns, and also have a reasonable potential to be used by any other larger agencies, e.g., State Department of Transportation (DOT).

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CHAPTER 1. INTRODUCTION

It is essential to have good roadways for the movement of goods, for travelling to and from work, for services, for social and recreational purposes, and for many other activities necessary to the functioning of our complex society. Thus, maintaining and rehabilitating pavements in an orderly and systematic fashion has been around for many years. However, all public works officials are familiar with the problem of crumbling streets, e.g., cracked pavements, and potholes, etc. In the present time of rising cost, increased use, and reduced resources it has become necessary to optimise the use of resources and to place more emphasis on system maintenance and rehabilitation (M&R) rather than expansion.

A process of assisting the engineers and decision makers in finding the cost effective optimum strategies for providing, evaluating, and maintaining the pavements in a serviceable condition is called Pavement Management System (PMS) (Hass et al. 1994). It includes the distress data collection, processing, analyzing and reporting construction and maintenance history. This data can then be utilized in identifying the current and future deficiencies and needs, developing the M&R program, priority programming within available budget, and providing the feedback on the performance of designs, materials, and M&R techniques and levels. Transportation agencies are adopting PMS for a variety of reasons: to develop a physical inventory, to prioritize M&R needs and to justify budget increase, etc. A PMS improves the efficiency of decision making, provides feedback on the consequences of decisions, facilitates the coordination of the activities within the agencies, and ensures the consistency of decisions made at different management levels within the same organization.

To promote a sustainable transportation infrastructure management system (TIMS) it is desirable to develop a model for low-volume roads, e.g., URI Kingston campus roadway network. Fortunately a research team has been studying and analysing road network of URI Kingston campus for its distresses using PMS over 20 years hopefully to provide recommendations to the Facility Office efficient M&R policies. It also developed prediction modeling in order to have optimum resource utilization and future forecasting.

1.1 Problem Statement

The problem of maintaining roads and/or transportation infrastructure in an orderly and systematic fashion has been around for many years. In the present time of rising cost, increased system utilization, and reduced resources, it becomes more urgently necessary to provide the best service to the users.

One of the greatest challenges cities, towns and colleges face is maintaining, preserving and restoring its paved streets, roads and parking lots. Even though street/road maintenance of transportation is one of the few areas where cities receive regional and state monies, needs continue to exceed available funding. Most public agencies face financial constraints and must make choices about how to spend their limited transportation maintenance budget.

PMS includes the distress data collection, processing, analyzing and reporting the data on pavement sections and their construction and maintenance history. This data can then be utilized in identifying the current and future deficiencies and need. And also developing the M&R program, it needs priority programming of work within available budget, and providing the feedback on the performance of designs, material, rehabilitation technique and maintenance levels. To promote a sustainable transportation infrastructure management system (TIMS) there is a definite need to develop a model PMS, particularly for municipal-maintained and/or low-volume roads first.

1.2 Objectives of the Study

Fortunately, a research team has been studying and analyzing road network of the University of Rhode Island (URI) Kingston campus for its distresses using Micro PMS, in which cost required to efficiently maintain and rehabilitate pavement network for future 10 years can be evaluated. The M&R policies and prediction modeling can be also developed in order to have optimum resource utilization and future forecasting, respectively. Thus, the present study set up the following objectives utilizing the URI Kingston campus roadway network:

- Development of a sustainable pavement management system for low-volume roads
- Identification and evaluation of M&R alternatives for the sustainable pavement
- Cost optimization on different M&R alternatives of PMS.
- Integration of PMS with Geographic Information System (GIS).

The outcome of this study can be used for larger pavement network such as towns, cities and even states as a sustainable PMS, which will become a component of comprehensive TIMS.

1.3 Research Approach

This research was carried out using the condition data of the different pavement sections of URI Kingston campus, which was selected as pilot network for previous studies (Lee and Bowen 1991). The condition of the pavements in the URI network was studied on the basis of distresses. The distresses in the road pavements were identified and quantified based on the standard criteria and guidelines of PMS. In order to implement a PMS, first the network was divided into the four more hierarchical components, i.e., Zone, Branch, Section, and Sample Units as suggested by MicroPaver software (APWA). The last component in hierarchy, i.e., sample unit was used to collect the distress data in every representative section. MicroPaver converts this data into Pavement Condition Index (PCI), for every section of the pavement network. This PCI was used to analyze the current pavement condition, and predicted future conditions using MicroPaver model with past 20 years data.

In order to provide any M&R strategy for each section, MicroPaver's four different M&R policies were used first: Stop-Gap Preventative, Local Preventative, Global Preventative and Major M&R policies. These were also compared with the ones generated using the policies of City of Cranston, Rhode Island (RI).

Four different M&R policies have different works and their corresponding rates for the implementations. These works can be also modified in terms of M&R needs. For example, the M&R can be done with the Reclaimed Asphalt Pavement (RAP), Cold In-Place Recycling (CIR) or warm mix asphalt (WMA) etc. which are economical and environmental friendly as compared to conventional Hot Mix Asphalt (HMA). But before introducing them, these new techniques and materials were studied for their performance, e.g., thermal and fatigue cracking and permanent deformation (rutting) resistance. For thermal cracking resistance, performance prediction was done using Thermal Cracking (TC) model of Mechanistic Empirical Pavement Design Guide (MEPDG). For rutting resistance, Asphalt Pavement Analyzer (APA) was used.

1.4 Scope and Limitations

Deteriorations of pavements have been investigated for a number of decades and very closely after the advent of computer. The rate of deteriorations and effect of M&R polices were then came into light to provide optimized and best serviceable pavements to the users. In the recent era a spatial technology, i.e., Geographical Information System (GIS) is being used to provide more visualized and reliable tools.

The scope of this study was to provide a model PMS for low volume roads utilizing the URI Kingston campus network and the best M&R policies, which can be used for similar and/or larger networks like towns, cities or states.

In order to develop a sustainable PMS, there should be a very extensive study on various factors on which the road pavement depends. These include performance parameters in terms of strength of subgrade soils, traffic loading, material properties, and climate. Some parameters require more testing over the years. Having a sustainable PMS is a continuing process, which has to improve with subsequent studies. Since the URI Transportation Research Canter (TRC) laboratory has the ability to perform some testing, e.g., SuperPave binder and mixture tests etc. the present study was carried out with the available resources for the best achievable results.

Results of literature review relevant to this study are summarized in Chapter 2. Chapter 3 deals with fundamentals of MicroPAVER PMS, and Chapter 4 presents implementation of MicroPAVER PMS in the URI Kingston campus. Chapter 5 discusses about pavement design and materials, and M&R alternatives are described in Chapter 6. Chapter 7 contains results of GIS application. Conclusions and recommendations based on findings of this study are presented in Chapter 8.

CHAPTER 2.LITERATURE REVIEW

Significant numbers of research have been conducted on pavement Management in last few decades. A large number of pavement management systems (PMS) have been developed and implemented in the last two decades ranging from very simple to very sophisticated systems (AASHTO 2001) (Lee and Bowen 1991). Agencies have tried to manage their system from making a prioritization table in the spreadsheet to sophisticated software aided with spatial data in order to provide the best PMS in optimizing resources. Yet, it seems that there is no universally accepted sustainable PMS for low-volume roads such as college campus. A comprehensive literature review was performed to obtain further information about the PMS, its implementation by various agencies, its pros and cons, types and cause of distresses etc. for the present study.

2.1 Levels of Pavement Management System

Conventionally, the decision-making process of Pavement Management System (PMS) has been divided into network and project levels. However, recent PMS literature introduces an innovative PMS model with three decision levels, differentiating the network level into program and project selection levels:

1) Program level: programming involves planning and allocating budgets for network optimization;

2) Project selection level: project selection ranks candidate projects within the constraints of the available budget;

3) Project level: the project level is concerned with detailed design decisions for implementing individual projects chosen at the project selection level. In other words, it

involves assisting designers in constructing, maintaining, or rehabilitating a section of roadway by preventative maintenance, resurfacing or reconstruction and other treatment options for the selected project.

2.2 Pavement Condition Monitoring

The most important attribute of a PMS is that it assists agencies to attain the best possible pavement at the least amount of money. To bring into play the attribute, pavement condition monitoring is the most important activity in a PMS. During the first 75 % of a pavement life, it performs well (Figure 2.1). After that, the pavement deteriorates so rapidly. If the roads and streets are properly maintained in excellent/good condition, the total annual investment for the maintenance is four to five times less than if the pavement is allowed to cycle through to the poor/failed condition, and then rehabilitated. By monitoring pavement deterioration with well- developed condition rating schemes, agencies can identify this critical condition or optimum maintenance period, and schedule appropriate pavement investments to maintain the network at a high service level at the lowest possible cost (Johnson 1983).

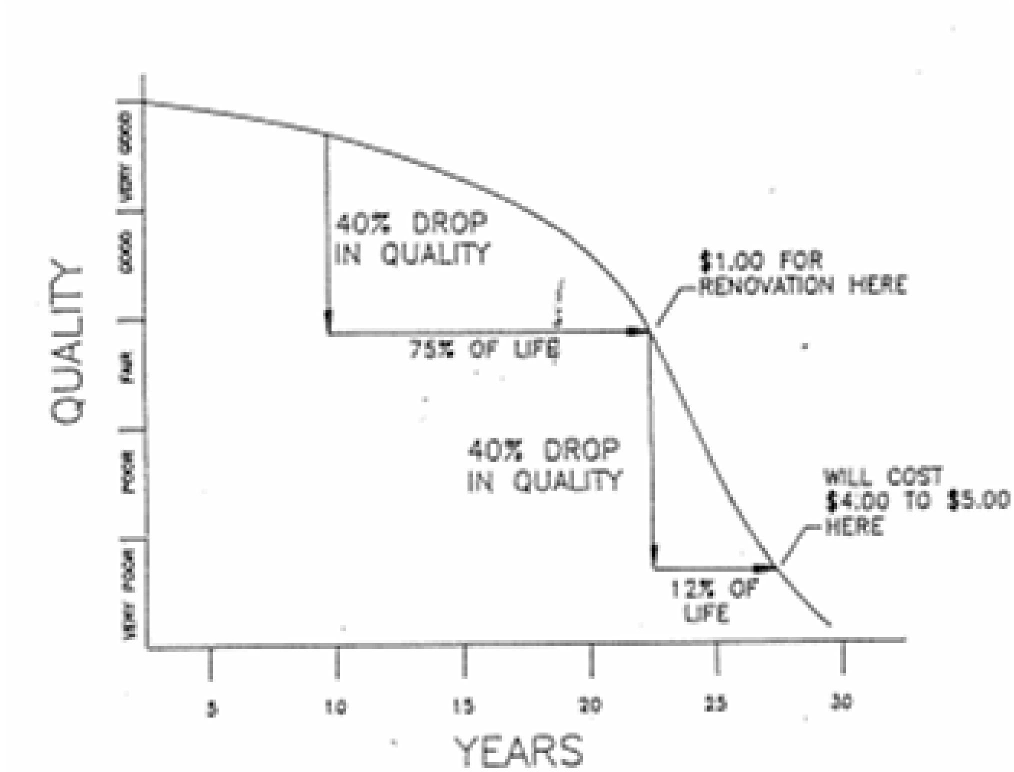

Figure 2.1 Typical Pavement Condition Life Cycles (Monismith et al. 1988)

2.3 PMS at The University of Rhode Island

In September of 1986, a report by the Rhode Island League of Cities and Towns (RILOCAT) revealed that Rhode Island ranked at the bottom of all states on state highway aid to cities and towns and near the bottom on per capita highway spending (RILOCAT 1986). Unlike most other states, Rhode Island has not used its fuel tax revenue directly for road improvement projects. Almost all major work was financed through federal highway aid matched by bond revenues, obligating the Rhode Island Department of Transportation (RIDOT) to costly annual appropriations.

Recognizing the problem of pavement maintenance, then Governor DiPrete proposed a three year, $8 million Pavement Management program in September 1987 (DiPrete 1987). In July of 1988, the Governor's program also supported a research team from the URI Department of Civil Engineering to implement an appropriate PMS at the municipal level. The initial research program was jointly coordinated with the Governor's Office, the RIDOT, the RI Department of Administration (DOA), and the RILOCAT. The objective of this project was to evaluate available PMSs, to identify the most appropriate PMS for municipally maintained roads in Rhode Island, and to implement the selected system by providing training and support (Lee and Bowen 1992). Thus, the URI roadway network has been used as a standard one, and used for training students and municipal engineers.

2.4 PMS in Rhode Island Cities and Towns

2.4.1 PMS in Town of South Kingstown

The Town of South Kingstown (TSK) is approximately 62.3 square miles located in mostly rural Washington County of Rhode Island. Approximately 400 miles of roads are located within TSK's border of which it is responsible for maintaining 125 miles. It may be noted that the average mileage of RI municipalities is 130 miles. The TSK is administered by a Town manager and five Town Council members. The Public Works Director reports to these elected officials. The TSK Department of Public Works has three divisions: Engineering, Highway, and Maintenance. The Engineering division is responsible for the implementation of PMS, and is headed by a Professional Engineer and has one non-registered engineer, two engineering technicians, one construction inspector and one clerical person.

TSK's physical inventory of town maintained roads was developed with the aid of an in-house computer file called the "Street Inventory". This annually updated computer file contained all roads within TSK's border in alphabetical order by street name. It also lists start limits, right-of way length and width, ownership status, maintenance status, and other reference material. The construction and major maintenance files are in a similar type of a format as the Street Inventory.

The pavement management network was divided into eight zones using permanent or physical obstructions (such as natural/semi natural barrier or state /major roads). Brach numbers were assigned to the town maintained roads in alphabetical order within each zone. Pavement sections were based on structural composition or surface type, with section lengths of 100 feet data was collected east to west and south to north. The pavement condition survey was conducted over a one year period.

MicroPaver's PCI reports are used to analyze the condition of TSK's pavement network. The annual maintenance and repair (M&R) requirements consist of activities for prevention and safety. Preventative M&R consist of localized or global maintenance activities that slow down the rate of deterioration to preserve the pavement investment, which includes crack sealing and various patching techniques. Global preventative maintenance includes various methods of surface sealing for asphalt concrete pavements. Safety M&R involves pothole patching and lane shoulder drop-off leveling. The TSK's current M&R policy uses patching for localized preventative maintenance. For global preventative maintenance, the following surface treatments are used: sand chip seal, stone seals, road oiling, asphalt concrete overlays, and reconstruction. Each year TSK issues a proposal to prospective contractors in search of a lowest bid from their Road Improvement Program (RIP)

2.4.2 PMS in City of Cranston

City of Cranston is approximately 30 square miles located in Providence County, South of Providence, Rhode Island. The City is responsible for maintaining some 11,384 streets totalling 426 miles. With the population of 80,000 Cranston is the second largest city in State. However, it ranks first for the mileage of streets maintenance (O'Rourke 1993). Cranston's Public Works Department (PWD) is headed by a director, and his assistant director. PWD has six divisions including the Engineering and Highway Maintenance. The Engineering Division is responsible for design and preparation plans, specifications and supervision of the construction of any projects when undertaken by the city. The Highway Maintenance is responsible for all general city work including street sweeping, snow plowing, tree trimming, catch basin cleaning, minor curb repair, etc. The City of Cranston agreed to work with the University of Rhode Island (URI) in setting up a pavement management pilot study in its first political ward.

The first political ward, located in the eastern most part of Cranston, serves as the pilot area. The area contains 191 streets with 38.9 miles of road. Ward one is subdivided into seven voting districts. While the physical inventory of the streets contains all roads in ward one, only District 2 was visually surveyed for distress data. Investigations of physical inventory data began with collecting the length of 191 roads as branches. Sections of the branches were created so that where two roads cross one another one road (typically major road) has a continuous section through the intersection. Inspection data were collected in variable directions beginning at the busier branch limits and proceeding to the less busy branch limit. The section category may be defined as either rural or urban. The pavement rank is in accordance with Highway Functional Classification

System for the State of Rhode Island. After examining the MicroPAVER's PCI report, the average PCI of the city was 79, or very good.

The M&R policies in Cranston, when required, performed with one of the three actions; pothole patching, surface overlay, or reconstruction. Each year, the city puts out bid for the City-wide paving contracts that perform all overlay and reconstruction projects. The proposed work generally includes resurfacing, construction/reconstruction, patching, sidewalk repair, and ancillary work (O'Rourke 1993).

2.5 Performance Parameter for Asphalt Concrete Pavements

Asphalt concrete constitutes three components: course and fine aggregates, asphalt binder and air. In process of production, on-site construction, and in due course of time, the properties of aggregate and asphalt binder can affect the serviceability of the pavements. The performance of the asphalt concrete pavement can also be affected by traffic loading and climatic factors. These all can lead to development of distresses in the pavement surface. Researchers identified aggregate gradation, binder content, and air void contents as basic properties, which influence the behaviour of asphalt concrete in service. In order to analyze the pavement performance, the distresses has been classified into two categories: load related distresses e.g., permanent deformation and fatigue cracking; and non- load related distresses e.g., thermal cracking.

2.5.1 Load related distresses

2.5.1.1 Permanent Deformation

Permanent deformation or rutting is a consequence of plastic deformation of pavement layers due to repeated traffic load, with combination of volumetric reduction and shear strain in asphalt concrete at high temperatures. The main cause of rutting is inappropriate compaction at the time of construction. It is recognized to be the major distress mechanism in flexible pavements as a result of increase in tire pressure and axle loads.

Rutting susceptibility of the asphalt concrete can be evaluated with the Asphalt Pavement Analyzer (APA). APA is a multifunctional Loaded Wheel Tester (LWT) used for evaluating permanent deformation (rutting). Testing time for a complete permanent deformation evaluation is 2 hours and 15 minutes which is taken by 8,000 cycles of ram movement. The testing time for fatigue cracking evaluation is dependent upon the fatigue behaviour of the mix being evaluated.

Permanent deformation (rutting) susceptibility of mixes is assessed by placing beam or cylindrical samples under repetitive wheel loads and measuring the amount of permanent deformation under the wheel path. The APA features an automated data acquisition system, which obtains rutting measurements and displays these measurements in a numeric and/or graphical format. Five measurements can be taken during a single pass. The limiting value of typical rut depth in Rhode Island is 14 mm.

2.5.1.2 Fatigue Cracking

Often called alligator cracks, these are the series of interconnected cracks developed at the surface of asphalt concrete pavement caused by fatigue failure of surface layer or base layer under repeated loading. The main cause of is distress are inadequate thickness and improper mix design and poor quality of construction. These cracks increase roughness to the surface, allows infiltration of water, and leads to more deterioration like potholes. The fatigue cracking susceptibility of asphalt concrete can be evaluated by APA and Simple Performance Test (SPT) etc.

2.5.2 Non- Load related distresses

2.5.2.1 Thermal Cracking

Thermal Cracking or sometimes called low temperature cracking is one non- load related distress. These are transverse that cracks often run perpendicular to the centre-line of the road and approximately equally spaced. These types of cracks which are due to diurnal temperature change occur when the temperature at the surface drops sufficiently to produce a thermally induced shrinkage stress in the HMA layer that exceeds the tensile strength of the asphalt mixture. These cracks usually initiate at the top and propagate downwards through the mixture. These cracks can be controlled to large extent using the proper performance Grade (PG) grade asphalt binder specified for the region.

In order to predict the performance of asphalt concrete, the SuperPave Indirect Tension Tests can be used with a thermal cracking (TC) model to predict the performance of a given pavement in terms of crack spacing and service life.

CHAPTER 3. MICROPAVER FOR LOW-VOLUME ROADS

Based on AASHTO definition, low volume roads or streets might be equated to a local road or street, which serves primarily for land access. From an administrative perspective, a low volume road is any road or street administered by a local road or street agency. On a financial basis, a low volume road is any road or street that has been classified as eligible or ineligible for funds from certain pots or money (TRB, 1975). Until recently the maintenance of low volume roads was neglected in United States (US). But now they begin realizing the fact of its importancee, as it needs a large amount of money for its restoration. This pilot project dealt with low volume roads, i.e., URI Kingston campus network.

3.1 Network Components

The network is divided into zones, branches, sections and sample units. This study is only for the pavement of road and streets. Therefore, all of branches are identified as roadway. The roadways functionally classified as Primary, Secondary and Tertiary roads for URI network. URI network distribution is shown in Figure 3.1.

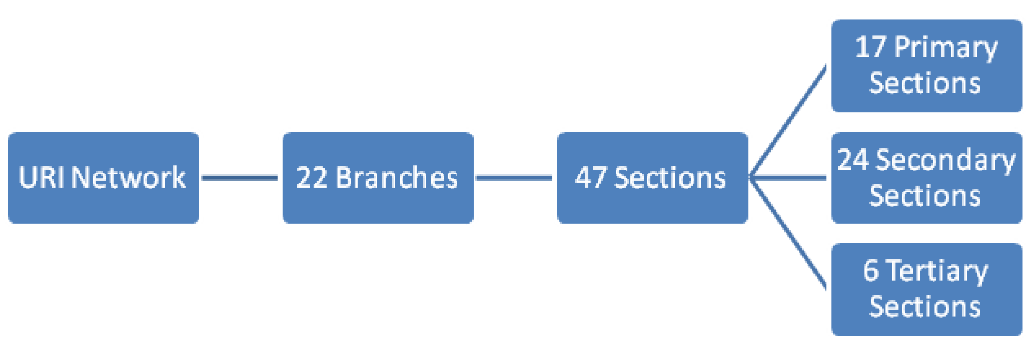

Figure 3.1: URI Roadway Network

3.2 Pavement Definition and Family

Families are generally established according to the branch use, surface type, zone and pavement rank. However, since there was only one branch use and surface type, both were not used to establish families in the URI network. Thus, families for URI network were established based on pavement rank, i.e., Primary, Secondary and Tertiary road.

3.3 PCI Concept

The primary component of the MicroPAVER PMS is the pavement condition survey and rating procedures that are used to derive the Pavement Condition Index (PCI) (Figure 3.2). The PCI ranges from zero to 100, with 100 being excellent. A truly accurate reflection of pavement condition incorporates several complex factors. Direct measurement of all these condition indicators invariably requires expensive equipment and highly trained personnel. The PCI method, however, accomplishes indirect measurement by evaluating visible pavement distress. Accordingly, the degree of pavement deterioration can be assessed as a function of (1) types, (2) severity, and (3) density of distress.

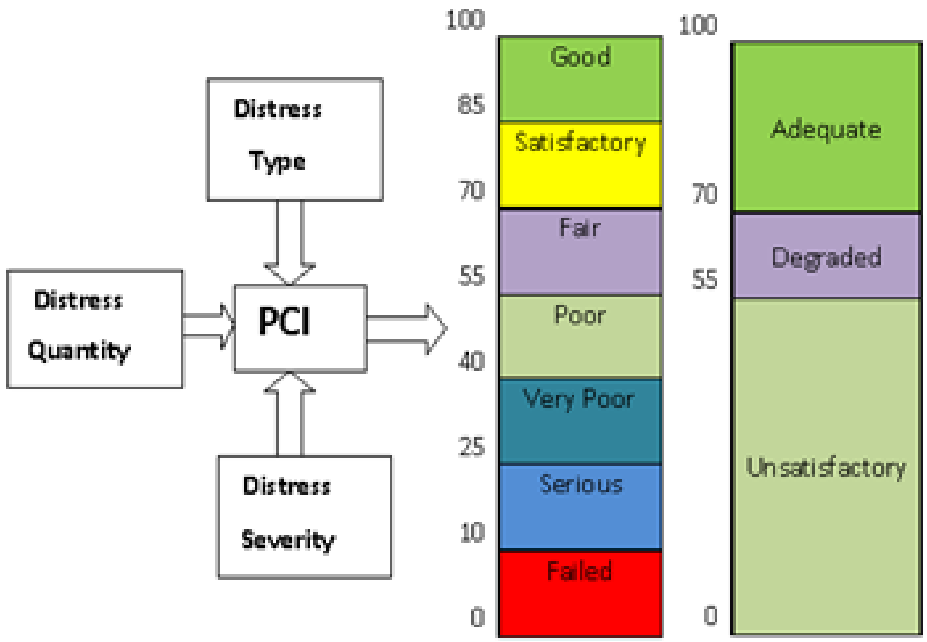

Figure: 3.2: PCI rating

3.4 Critical PCI

Before an agency starts to analyze future M&R needs and cost, critical PCI must be established. The critical PCI is defined as the PCI value below which the pavement shows a significant increase in both the rate of deterioration and preventive, maintenance cost (Shahin and Walther 1990).Therefore, critical PCI is a turning point from which an agency should consider major M&R strategies instead of preventive strategies. However, the critical PCI can be adjusted to reflect the behaviour of sections of the pavement based on the experience. Because future PCI prediction and M&R plan is performed based on the family curve analysis, critical PCI is established from each family. Critical PCI from a family can be found through the following steps (Shahin 1994):

1. Visually select the critical PCI range based on the shape of the family deterioration curve,
2. Apply the preventive M&R strategies to pavement sections in the family, and estimate the cost for each individual distress, then sum up the estimated cost section by section.
3. Plot the cost of the preventive M&R activity per unit area versus PCI for each section, and
4. Select the critical PCI based on the results from steps 1 through 3.

3.5 Prioritization Methods

When funds are limited, all pavements that are now or projected to be below certain condition, at which major M&R strategies is needed, should be prioritize for selecting a time for M&R activities. The simplest method would be "Worst First" method. In this method, all pavements that require major M&R are sorted according to their condition: the worst condition pavement will be the top of the list, and the best pavement will be the last one. The sorted pavement will be ranked in ascending order, and the worst condition pavement receives the first priority.

3.6 Prediction Model

For future PCI prediction, "Constrained Least Squares Estimation Method" and "Backtracking Method" can be employed. (Kon 1998). Constrained least squares estimation method has been employed to predict future PCI value within the maximum age available in the database. The first step is to establish pavement families. The next step is to plot inspected past PCI values versus corresponding pavement ages. When the data for a pavement family have been identified, datum points are filtered to remove points in error. An error may occur in the phases of data collection, data coding and/or data input. To avoid the error involved in further analysis, upper and lower boundaries are introduced. If a datum is above or below the boundaries, the datum is recognized an error and removed (Figure 3.3). (Kon 1998)

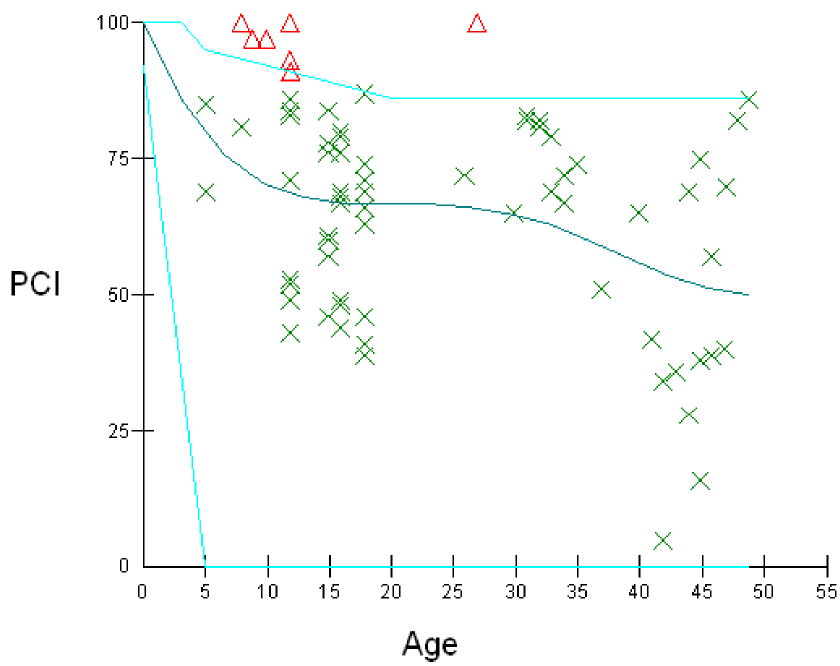

Figure 3.3: Sample from the filtered Procedure

The filtered data are then examined to identify datum points that are statically outliers. In the outlier procedure, it is assumed that the errors between the predicted and observed PCIs are normally distributed. A confidence interval is set by the agency. Data beyond this interval are identified as outliers (Figure 3.3). The remaining data points are then fitted with a fourth degree polynomial by using the constrained least-squares procedure. This curve is constrained because it is not allowed to have positive slope because of PCI's inability to increase with age. The prediction equation for a family represents the average behaviour of all section of that family. The PCI prediction at the section level is performed by assuming that the deterioration of the entire pavement in a family is similar and is a function of only their present condition, regardless of the ages. Then, a section prediction curve is developed through the section's latest PCI vs. age point, parallel to its family curve. (Kon 1998)

Finally, prediction of the condition beyond the maximum age available in the database is performed by using backtracking method. The slope of the line between the last data and the data point corresponding to an age 3 years before the last data is determined. This slope is used to extend the curve beyond the last data point. (Kon 1998)

3.7 Maintenance & Rehabilitation Strategies

Maintenance and Rehabilitation (M&R) strategies are combination of pavement distress and corresponding maintenance technique to reduce safety hazard, to deter further progress of distress or to bring pavement as constructed condition. Maintenance techniques differ according to type, severity and density of the distress. For example, a localized low temperature cracking that has narrow opening will be maintained by crack sealing and severe alligator cracking spreading upon entire pavement will be repaired through reconstruction. A public work agency must select an appropriate maintenance technique corresponding to each type, severity and density distress.

In addition, cost of maintenance technique is a factor to determine an M&R strategy. For example, reconstruction recovers all types of distress; however, it is not always cost effective. Fog sealing is very economical maintenance technique, but its effect may last for a few years. A public work agency should consider the cost effectiveness when they determine M&R strategies.

Usually, PMS activities are performed for section by section, and M&R cost is estimated for an entire section because it is defined as the smallest management unit in PMS. On a pavement section, there is not only one distress but also different type and severity. Therefore, when an agency performs any M&R activity, a set of M&R strategies should be formulated. A set M&R strategies and their unit cost is refers as M&R policy. There are seven types of M&R policies in the MicroPAVER: A localized stopgap, a localized preventive, three global preventives and two major policies. M&R policies are assigned to each pavement section based on the section's PCI with respect to the critical PCI as shown in Figure 3.4.

The primary objective of the localized preventative M&R policy is to slow down the rate of condition deterioration. These strategies include crack sealing and various patching techniques. This M&R policy is applied to pavement above the critical PCI. Usually, application of a localized preventive M&R policy is to prevent section with PCIs below the critical PCI is not cost effective.

Localized stopgap M&R policy is defined as a set of the localized distress M&R strategies needed to keep the pavement in a safe and operational condition. A stopgap M&R policy is different from preventive M&R policy because it will usually include only high severity level distresses that could be a safety hazard. Stopgap M&R policy should only be applied to the pavement with PCI below the critical PCI.

Global preventive M&R policy is defined as a set of strategies that are applied to the entire pavement section with the primary objective of slowing the rate of condition deterioration. These strategies include chip sealing, rejuvenation and fog sealing. Global preventive M&R policy is applied to pavement above the critical PCI is often not cost effective. Three types of global preventative M&R policies are assigned to the pavement section based on the distress types (Figure 3.4) Micro PAVER recommends chip sealing for pavement with skid causing distresses such a bleeding. Rejuvenation is recommended for pavement with climate–related distress such as weathering. Fog sealing is recommended for pavement with other distresses except load–related types.

Major M&R policy is applied to the entire pavement section to correct and improve existing structural or functional requirement. Major M&R policy is divided into two types: (1) major M&R policy applied to pavement section with condition above the critical PCI and (2) major M&R policy applied to pavement section with condition below the critical PCI (Shahin 1994)

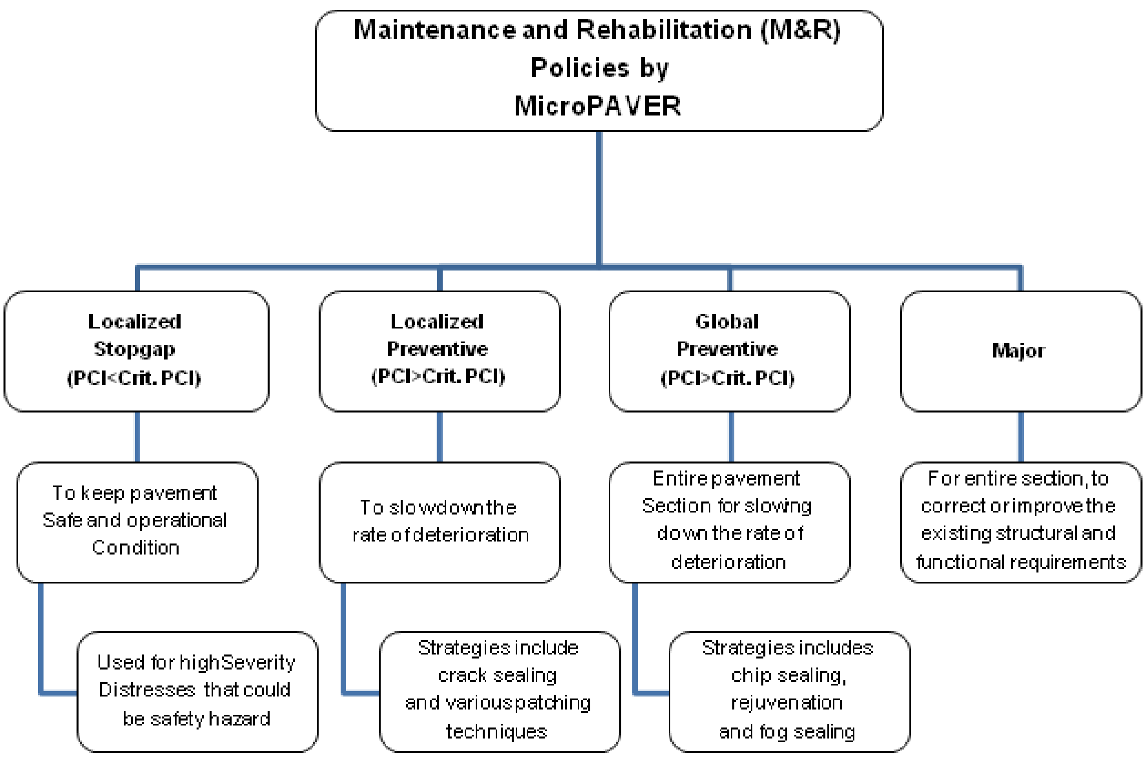

Figure. 3.4: Different M&R Policies and their Implementations

3.8 Economic Analysis

Economic analysis can be done in various phases like in current condition of the pavement, how much will be the expenditure to retain the pavement for next one year without any further deterioration. Future expenditure required to keep the pavement in best condition can be customized according to need or available budget. For example the pavements can be fixed to any PCI value below 100, if there is not a substantial budgeting allocation.

CHAPTER 4. IMPLEMENTATION OF MICROPAVER PMS ON THE URI KINGSTON CAMPUS

The University of Rhode Island (URI) is a medium-size state university with its main campus located in the rural village of Kingston in southern Rhode Island. The campus itself is representative of the suburban area. There is route 138 going through the campus from east to west, which divides the main campus from other URI facilities. The roadway network generally consist of two-lane asphalt concrete surfaces roadways, and on-street parking and sidewalks are present on some streets. The roadways have been functionally classified as primary (circulator), secondary (access) or tertiary (service) roads.

4.1 Data Collection

Different survey crews have performed the pavement condition surveys for more than last 20 years as a part of class projects. The initial data collection was begun in the summer of 1988, and it was part of training process for the town of South Kingstown personnel (Lee and Bowen, 1991). From 1989, students in the Highway Engineering course have performed the survey. Condition assessments with nineteen types of distress are performed annually by students in the crew of two to three in this course as a group project. There was an effort to analyze 10 years of data from 1988 to 1997 (Kon 1998). Thus, thirteen year data from 2000 to 2012 was used in this study.

The URI network is divided into 22 branches and 47 sections. One to five sections are established for each branch according to their structural composition or surface conditions. This study is only for the pavement of roads and streets. Therefore, all of branches are identified as roadway. Families are generally established according to the branch use, surface type, zone and pavement rank. However, branch use, surface type and zone are not used to establish families in the URI network because there was only one branch use. Accordingly, families for the URI network are established based on pavement rank: Circulator, Access and Service roads.

4.2 Analysis

The Micro PAVER PMS analysis was performed in three phases for network level (Figure 4.1). Therefore, most works in this study were related to the network level PMS analysis, and for project level PMS, several techniques are just presented in the last section for demonstration purpose.

In phase one, current pavement condition and corresponding M&R cost were analyzed. Arithmetic as well as area weighted average PCI value was obtained in prioritized list for each section. Corresponding M&R cost for URI network and resulted pavement condition were determined by applying appropriate type of M&R policies. In the phase two, the long-term pavement deterioration trend, the future condition and critical PCI were analyzed. Long- term pavement deterioration trend was obtained through MicroPAVER 6.1.2 software in the form of a regression curve. The first step in developing the trend curve was to group the pavements. Groupings were selected based on pavement rank, such as circulator, access and service roads. After grouping, previous available data of PCIs and pavement age were plotted, and the fourth degree polynomial regression curve was developed for each group. The future pavement conditions were determined from long- term pavement deterioration trend curve. An individual pavement prediction curve was drawn through the current PCI/Age point for the pavement being inspected, parallel to the trend curve for the group of the pavement. Then, the predicted PCI value in the future was determined.

A few tentative critical PCI were established based on the M&R costs, the long-term deterioration trend and safety considerations. The long-term deterioration trend curve shows as a certain PCI value below which it decreases rapidly with age. Routine M&R cost for each pavement and its PCI value were plotted to find the point of rapid increase of routine cost versus PCI. Combining those PCI points and considering safety matters, the tentative critical PCIs were established.

In phase three, economic analysis was performed according to the tentative critical PCIs, M&R policies, future pavement condition prediction, budget constraints and current inflation rate. Minimum PCIs for inspection and future inspection schedule were established in this phase.

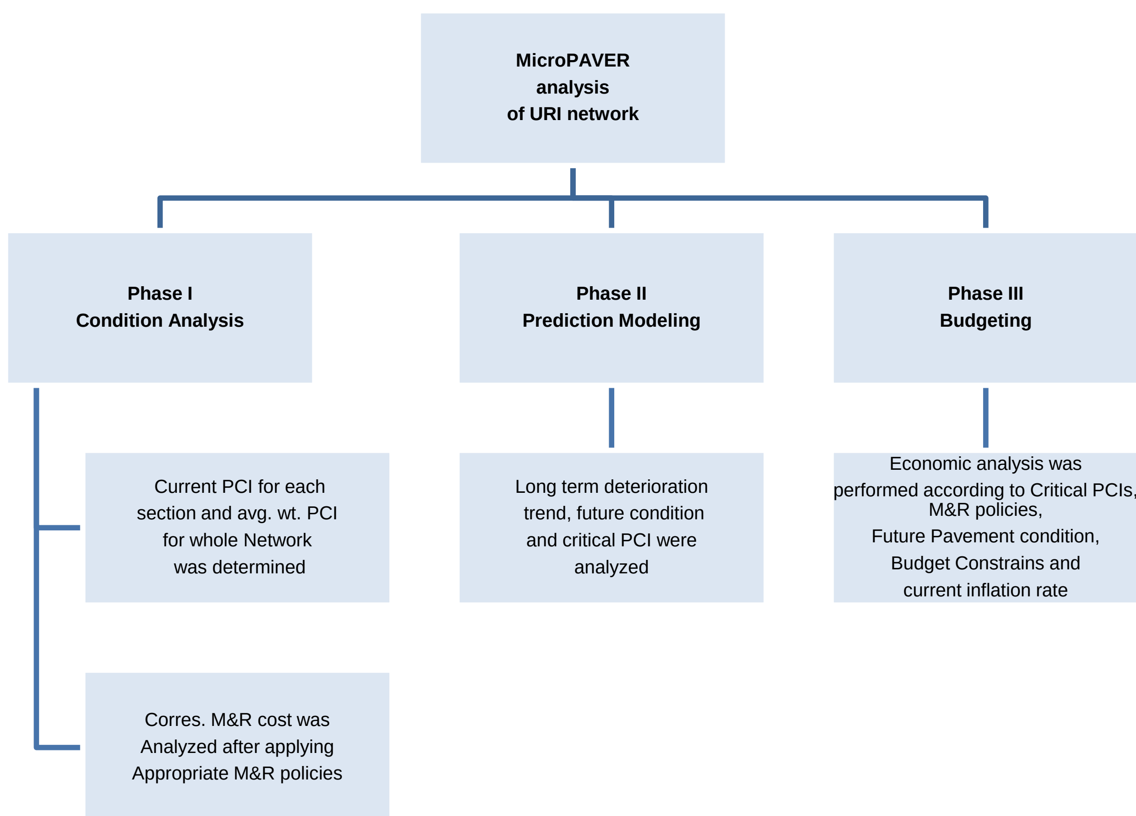

Figure 4.1: Phases of MicroPaver PMS analysis

According to the best fit critical pavement condition, the minimum PCI for inspection was established. It is necessary to inspect and established appropriate M&R plan for a pavement before it reaches critical PCI, which is a turning point of M&R policy. Minimum PCI for the inspection was established by considering deterioration rate and delay for decision making. Finally, condition inspection schedule was developed based on the minimum PCI for inspection.

Current pavement condition and M&R cost

It was found that the total area of the URI network is 765,358 square feet. All the pavements in this study are used for roadways. The current average PCI for the whole network was calculated to be 56 and the weighted average condition was 60 which are faircondition. And the wt. avg. condition for circulator, access and service roads are 62, 57 and 67, respectively (Figure 4.2). The pavement condition ranking at the last inspection was analyzed. There were 9% of failed, 4% of serious, 13% of very poor, 17% of poor, 17% of fair, 19% of satisfactory, and 21% of good pavement area (Figure 4.3). Fifty seven percent of pavement area was in fair or better rankings, and 43% was poor or worse rankings.

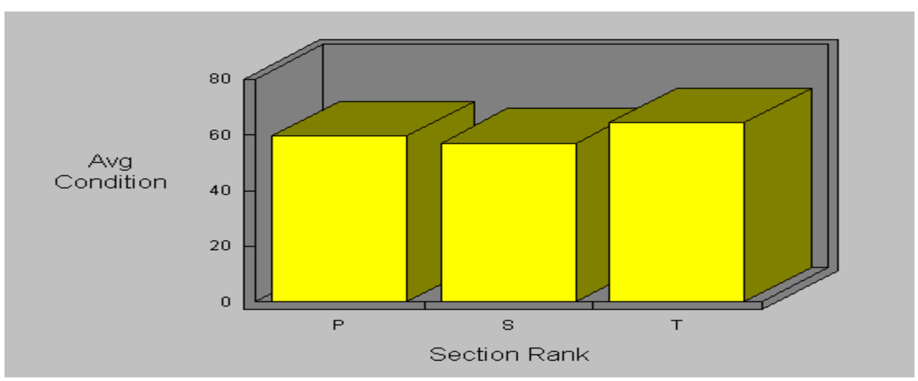

Figure 4.2: Average Condition and Section rank

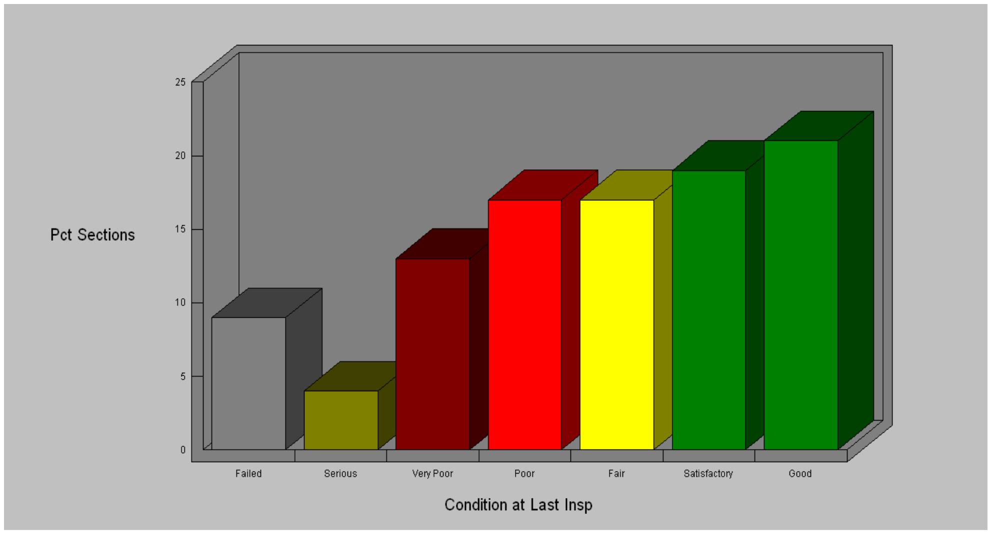

FIGURE 4.3: Average condition and condition at last inspection

Localized distress maintenance cost

Cost of preventative measure required to bring the pavement to the safe condition was estimated by applying the localized preventative and localized stopgap M&R policies and were compared. In this study, the default tables of MicroPAVER 6.1.2 were employed. For localized preventive M&R policy, the cost was determined to be $1,216,000 and the resulted average PCI would increase from 62 to 77 (Figure 4.4). If localized stopgap M&R policy is applied the total cost would be $277,637. The localized stopgap M&R is only applied for high severity level distress and low and medium level of distresses would be left without any treatment (Figure 4.5).

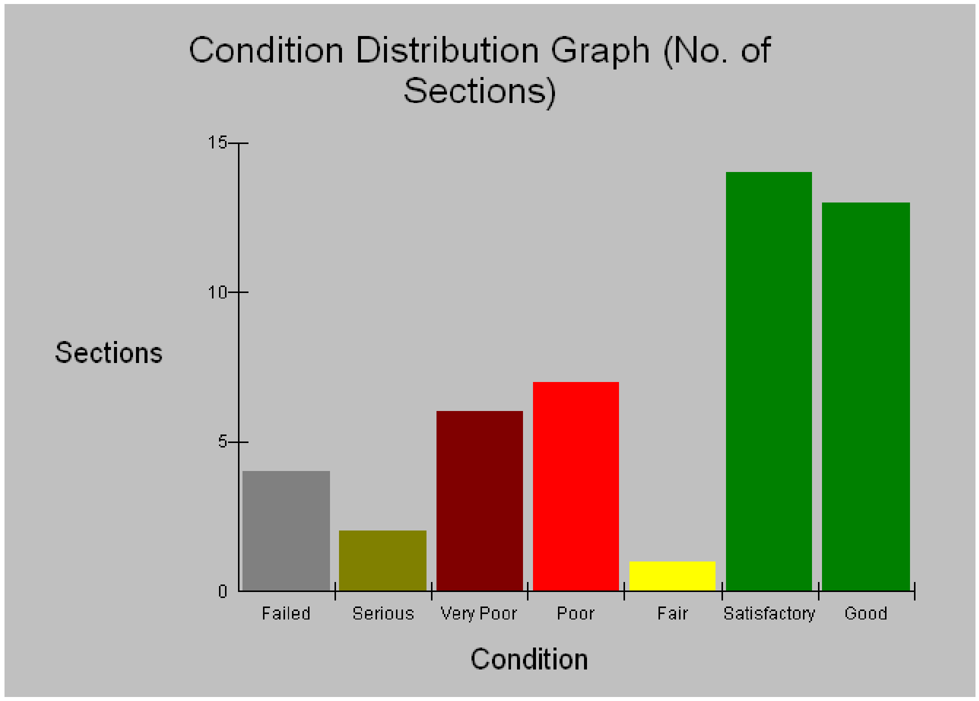

FIGURE 4.4: Consequence of Localized Preventative Policy

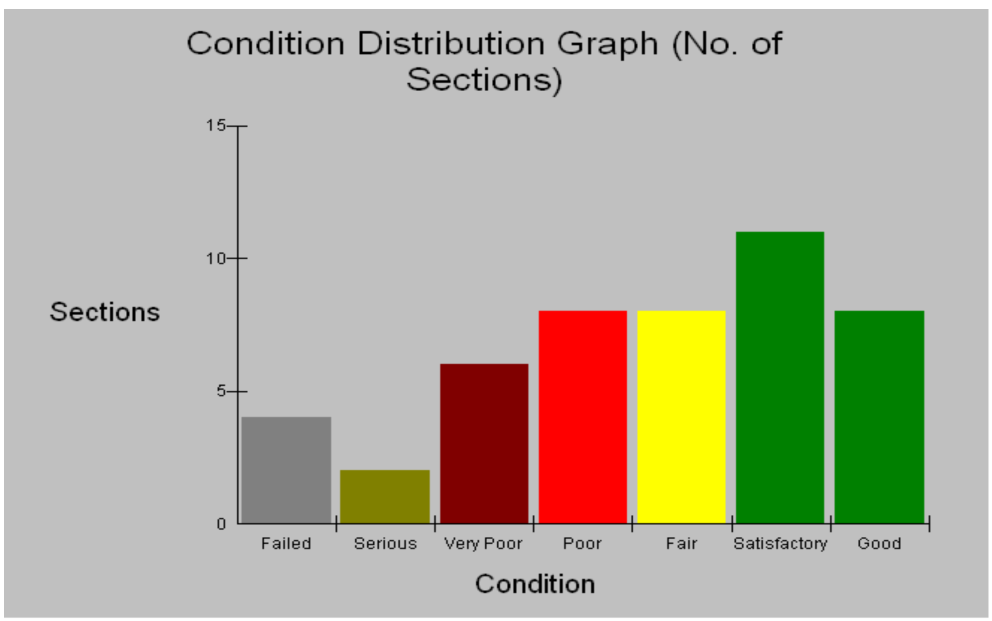

FIGURE 4.5: Consequence of Localized Stop-gap Policy

4.3 Prediction Modelling

In order to predict the deterioration trend of any family of pavement prediction modeling was carried out. The deterioration curve between the PCI and age of the selected families was drawn by the MicroPAVER 6.1.2 individually for circulatory, access and service sections. The datum points were filtered to remove points in error by introducing upper and lower boundaries The resulted family curves are shown in Figure 4.6. In this study upper boundaries introduced by Kon (1998) were used again as follows, with confidence level of 95% to identify datum points.

- 100 PCI at the age of 3,
- 95 PCI at the age of 5,
- 92 PCI at the age of 10, and
- 86 PCI at the age of 20
- The lower boundaries were also as follows:
- 92 PCI at the age of 0,
- 72 PCI at the age of 1,
- 36 PCI at the age of three,
- 0 PCI at the age of 10

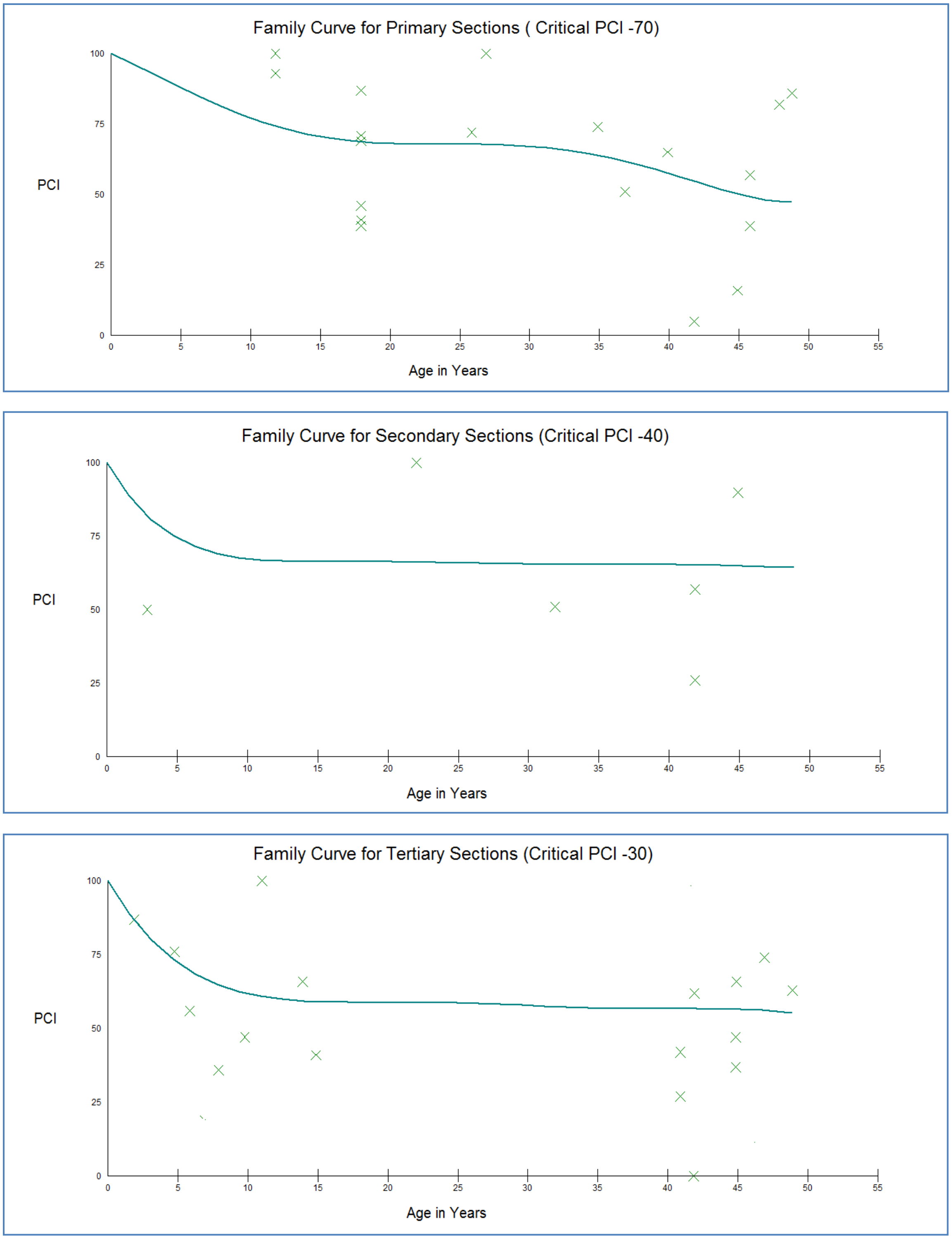

FIGURE 4.6: Family deterioration curves for Circulatory (Primary), Access (Circulatory) and Service (Tertiary) sections

4.4 Budget Analysis

M & R study was carried out on the network as whole. However, it can be done on individual sections as well as different families depending on the need. Major M&R policy was also compared with the policy and the rates used for a typical New England city, i.e., Cranston, RI. (Lee et al. 1992). The M&R strategies used for the city is shown in Table 4.1. For cost comparison of the four different M&R policies the rates of different works were taken as default from the MicroPAVER 6.1.2. Details of M&R policies rates and expenditure can be found from Appendix C. Table 4.2 shows expenditure summary of the URI network using Cranston data.

Table 4.1 M&R policies for Cranston Project Report

PCI	General M&R Strategy	Est.
0-10	Recycle and Build base with a 3"	$ 1.95/SF
11-40	Mill to grade with a 2" overlay CM-	$ 1.65
41-55	Crack Sealing	$ 0.25/SF
56-85	Routine Maintenance	…
86-	None	…

Table 4.2 Expenditure Summary of the URI network using Cranston Data

Category	Total	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Budget	$3,000,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Work Planner	$1,281,516.71	$103,965.00	$101,698.00	$99,652.00	$293,545.89	$293,655.46	$0.00	$287,302.36	$0.00	$0.00	
Total Expenditure	$1,281,516.71	$103,965.00	$101,698.00	$99,652.00	$293,545.89	$293,655.46	$0.00	$287,302.36	$0.00	$0.00	

4.5 Results and Discussions

Budget analysis was performed for the next ten years using URI work rates and critical PCI plan of the MicroPAVER 6.1.2. The budget constraint used was $300,000 per year and the cost of the M&R policy for the whole URI network was $1,281,516 using URI rates. For comparison of cost with different M&R policies, the default values from MicroPAVER 6.1.2 were used for the whole network. Expenditure summary is shown in Table 4.3.

Table 4.3 Expenditure Summary of the URI network using default MicroPaver Policies for different M&R strategies.

	Major M&R Policy using Default values from the MicroPaver 6.1.2										
Category	Total	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Budget	$3,000,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Work Planner	$2,420,049.34	$298,164.03	$294,097.34	$292,510.14	$293,545.89	$293,655.46	$274,712.36	$287,302.36	$294,963.35	$91,098.40	
Total Expenditure	**$2,420,049.34**	$298,164.03	$294,097.34	$292,510.14	$293,545.89	$293,655.46	$274,712.36	$287,302.36	$294,963.35	$91,098.40	
	Localised Stop-Gap M&R Policy using Default values from the MicroPaver 6.1.2										
Category	Total	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Budget	$3,000,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Work Planner	$277,637.22	$18,166.05	$20,293.24	$22,031.30	$23,886.07	$25,974.70	$28,195.29	$30,720.32	$33,279.21	$36,055.75	$39,035.30
Total Expenditure	**$277,637.22**	$18,166.05	$20,293.24	$22,031.30	$23,886.07	$25,974.70	$28,195.29	$30,720.32	$33,279.21	$36,055.75	$39,035.30
	Localised Preventative M&R Policy using Default values from the MicroPaver 6.1.2										
Category	Total	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Budget	$3,000,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Total Expenditure	**$1,215,943.66**	$95,696.11	$88,035.80	$96,624.36	$106,383.38	$115,400.24	$125,970.18	$136,411.56	$140,939.83	$150,466.65	$160,015.55
	Global Preventative M&R Policy using Default values from the MicroPaver 6.1.2										
Category	Total	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Budget	$3,000,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00	$300,000.00
Planned Project(s)											
Work Planner	$23,667.95	$10,397.64					$13,270.31				
Total Expenditure	**$23,667.95**	$10,397.64					$13,270.31				

CHAPTER 5. PAVEMENT DESIGN AND MATERIALS

Pavement management deals and focuses on the optimization of budget and meeting user's expectations of the pavements. This requires detailed evaluation of the pavement performance and its behavior throughout its life-cycle. Deterioration process of the pavement depends on the characteristics of its materials including subgrade soils on which pavement rests. The structural or thickness design of the asphalt pavement uses resilient modulus and elastic moduli of different layers of the pavement. Therefore in order to evaluate the performance of pavements, it is imperative to determine the properties of materials. This Chapter describes a model pavement design and material characterization for Upper College Road at the University of Rhode Island (URI) in Kingston, RI.

5.1 Resilient Modulus of Subgrade Soils

Effective soil Resilient Modulus is a typical parameter for pavement design in most states including Rhode Island. It represents the strength of the soil in order to bear the traffic load, and the modulus can be determined by performing the testing of AASHTO T 307-99 (AASHTO 2005). The testing procedure and results of soil resilient modulus are given in Appendix B. The optimum moisture content (OMC) of URI soil specimen was determined as 9.5%, and the modulus testing specimen was fabricated at OMC-2% or 7.5% based on previous study experience (Kovacs et al. 1991). The resilient modulus of URI campus soil was determined as 2,696 psi, which did not consider seasonal variation. To consider seasonal variation, this study decided to use the value predicted by Lee et al. (1994), i.e., 6,900psi. The prediction equation used was as follows:

$$\mathbf{\log M_R = -1.0476 + 0.5730\log\theta - 0.0371(w/c) + 0.016\gamma_d - 0.0031T - 0.0002T^*(w/c)} \quad (5\text{-}1)$$

where M_R = resilient modulus,
θ = bulk stress,
w/c = water content
γ_d = dry density, and
T = temperature.

5.2 Mineral Aggregates

Mineral aggregate constitute major portions of asphalt concrete which makes it necessary to evaluate properties prior to manufacturing Hot Mix Asphalt (HMA). In this study mineral aggregates were obtained from the Cardi Corporation, Warwick, and P. J. Keating , Cranston, Rhode Island. Various tests were performed, and properties are summarized below.

5.2.1 Gradation Test or Sieve Analysis

According to procedures of AASHTO T27 and ASTM C136, gradation tests were performed for the particle size distribution, in order to obtain maximum density of aggregate. In this project Rhode Island Class I-1 gradation was used to prepare HMA samples as shown in Appendix D.

5.2.2 Particle Shape and Uncompacted Void Content of Fine Aggregates

The strength of HMA is influenced by particle shape, surface texture, and gradation.

This test is used to determine the angularity of fine aggregates in order to provide the maximum interlocking of the aggregate in asphalt concrete. This test is for Superpave Level 1 HMA design procedure which was developed through the Strategic Highway Research Program (SHRP). Test procedure for the angularity of fine aggregates is given by AASHTO TP33 and/or ASTM C1252, and details can be found from Appendix D.

5.2.3 Resistance to Degradation of Small-Size Coarse Aggregates

The resistance to degradation of small-size coarse aggregates can be determined by Abrasion and Impact in the Los Angeles Machine. This test was performed in accordance with AASHTO Designation T 96-02 and/or ASTM Designation C131-01 to measure impact, abrasion, and grinding due to wheel loads. The test procedure and results are in Appendix D.

5.2.4 Specific Gravity of Coarse and Fine Aggregates

Specific gravities were determined in accordance with ASTM C127/AASHTO T85 Specific Gravity and Absorption of Coarse Aggregate and ASTM C128 /AASHTO T84 Specific Gravity and Absorption of Fine Aggregates. Test procedures and results are given in Appendix D.

5.3 Asphalt Binders

Bituminous materials are used extensively for pavement construction, primarily because of their excellent binding or cementing power. A great majority of asphalts used are the residue of crude oils. A wide variety of refinery processes may be used to produce asphalt of different consistency and other desirable properties. A large number of different laboratory tests are performed on bituminous materials for the purpose of checking compliance with the specifications that are being used. The US Congress enacted the Strategic Highway Research Program (SHRP) in 1987, and the asphalt research program developed a system called Superior Performing Asphalt Pavements (Superpave). The Superpave system combines performance-based, asphalt materials characterization with design environmental conditions in order control permanent deformation, fatigue cracking and thermal cracking in asphalt pavements. The Superpave uses a new system for testing, specifying, and selecting asphalt binders. In this study Superpave tests were performed to confirm the performance grading (PG) of asphalt used for production of HMA.

5.3.1 Rotational Viscometer Tests

Rotational Viscometer (RV) test was used to measure the viscosity of asphalt binder used at high temperature for its workability in asphalt concrete mix and performed in accordance with the test procedure of AASHTO T 316 and/or ASTM D 4402. The RV tests were done at 135°C (275°F).

The RV test measured the torque required to maintain a constant speed of a cylindrical spindle while submerged in the asphalt binder at constant temperature. This torque is converted into dynamic viscosity and displayed. For unaged PG 64-28 sample the dynamic viscosity was 0.24 Pa.s, which was within the ranges of 0.2 to 2.00 Pa.s.

5.3.2 Dynamic Shear Rheometer (DSR) Test

This test procedure is given by AASHTO T316 and ASTM D 7175. This test is used to find the visco-elastic properties of asphalt binder in terms of Dynamic Shear Modulus (G*) and phase angle (δ). This test is used for Superpave PG asphalt binder grading system, and performed on virgin, Rolling Thin Film Oven (RTFO) aged, and Pressure Aging Vessel (PAV) aged asphalt binders. The test procedures and results are given in Appendix D.

5.3.3 Bending Beam Rheometer Test

The AASHTO T 313-04 Bending Beam Rheometer (BBR) test provides a measure of flexural creep stiffness and creep compliance of asphalt binder at low temperature. These properties determined describe the low temperature stress-strain time response of the binder at test temperature within linear visco-elastic response range. The parameters give an indication of asphalt‘s ability to resists low temperature cracking. This test is done on PAV aged asphalt.

Three tests were performed on a sample of PG 64-28. Each sample was prepared in accordance with the procedure of AASHTO T 313. After the sample was removed from the mould it was immediately placed in the testing bath to condition it to low temperature. The test beam was then placed on the test support and the test was initiated. After entering the specimen identification information, load, temperature, and time the specimen was placed in the bath. The load was manually applied to ensure the contact with loading head. The load was increased and held eventually decreased and held again, giving the beam time to recover. The results were then recorded.

All three tests were performed on PAV aged asphalt, the stiffness values varied from 31 to 43 MPa which falls under the maximum range of 300 MPa. The m-values from the slope of the log stiffness and log time closely matched that were recorded from the computer output, at around 0.400. These values fall above minimum of 0.300 of Superpave Specifications low temperature requirement. These stiffness and m-values confirmed that the asphalt tested was PG 64-28. The detailed testing results are given in Appendix D.

5.3.4 Asphalt Binder Cracking Device (ABCD) Test

Low temperature thermal shrinkage cracking is one of the major failure modes in asphalt pavement, together with rutting, fatigue cracking, and moisture damage. This cracking is caused in the asphalt pavement when the thermal tensile stress within the asphalt pavement that results from a temperature drop exceeds its strength at that temperature. Thermal cracks typically appear as transverse cracks (cracks perpendicular to the direction of traffic) at regular intervals in road pavements.

Asphalt Binder Cracking Device (ABCD) was used to provide low temperature characteristics of asphalt binders and to validate the ABCD tests by comparing its results with low temperature field performance data.

The ABCD uses the differential coefficients of thermal expansion/contraction of asphalt binders and metallic ring, to directly determine the cracking temperature of asphalt binders. It consists of metal rings of 2 in. outer dia. and ½ in. height made up of metal alloy (usually, Ni-Fe alloy) of low coefficient of thermal expansion/contraction relative to asphalt. Asphalt binder is then placed around these rings on the outer periphery in uniform thickness. The assembly of ring and asphalt was cooled down to very low temperature in a cooling chamber. Due to different rate of thermal contraction of the two material viz. metal ring and the binder, tensile stresses develop between the contact of binder and metal which leads cracking in the asphalt binder annular ring (this occurs at temperature around -35 to -45^{o} C depending on the asphalt grade). The thermal strain is detected by the strain gauge placed in inner side of the metal rings and used to calculate fracture stress of the asphalt binder. When the specimen cracks, the accumulated thermal stress is relieved and shown as a sudden jump in the strain reading. The cracking temperature of the asphalt binder is directly determined as the temperature where the sudden jump of measured strain occurs, through data acquisition system on computer output.

From the result it is clear that the asphalt with lower stiffness is cracking at lower temperature than the asphalt of higher stiffness. The cracking temperature is also satisfying with the performance grade of the asphalt binder. Since asphalt used for the lower and medium stiffness is PG 70-22 and PG 76-22, respectively, the cracking temperature is very far ahead of them nearly -35^{o}C with respect to -22^{o}C of minimum allowable temperature. The result also indicates that the asphalt of medium stiffness, e.g. MM13, MM06 is showing the cracking at low temperature than the low stiffness asphalt specimen, which should be due to polymer concentration. The two types of specimen “LL” and “MM” are modified with polymer SBS; this is also effecting it significantly. The third type of asphalt specimen is of higher stiffness that is cracking at the temperature of -12^{o}C to -13^{o}C.

The feedback from these test results will be useful to determine the type of asphalt binder with or without polymer modified, to be used in their respective climatic conditions in order to consider low temperature cracking. With the rotation table and other specified procedure, still there is no way to get rid of the air bubble formation in the ring. While pouring the binder specimen in the moulds the temperature of the specimen drops, which may affect the results. Detailed procedure and results are given in Appendix D.

5.4 Superpave Asphalt Mix-Design

After determining appropriate mineral aggregates and asphalt binder, the design aggregate structure should be selected by creating trial blends. This is accomplished by mathematically combining the gradations of the individual aggregate into single gradation. Then gradation control is based on four control sieves: the maximum sieve, the nominal maximum sieve, the 2.36-mm sieve, and the 0.075-mm sieve.

The remaining steps of the Superpave mix-design procedures were as follows:

Blending asphalt with the trial blends

Short-term oven aging the mixtures

Compacting the specimens with the Superpave Gyratory Compactor (SGC)

Analyzing the volumetrics of the trial blends

Selecting the “best” blend based on the design aggregate structure

Compacting samples of the design aggregate structure at several asphalt contents to determine the optimum binder content (OBC).

5.5 Pavement Performance

After preparing HMA specimens with OBC, various laboratory tests were conducted to evaluate in-service performance of the flexible pavement. Some of tests conducted are described below.

5.5.1 Rutting or Permanent Deformation

Rutting or permanent deformation is one of the key performance parameters, which is depression along the wheel path caused by tire pressure and can also be due to layer compaction at the time of laying the pavement. There are various tests to evaluate the rutting in the pavement as listed below.

1. Asphalt Pavement Analyzer (APA)
2. Hamburg Wheel Tracking Device (HWTD)
3. French Pavement Rut Tester (FPRT), and
4. PURwheel

Of all these test procedures APA is the most common and was used for this study. The test procedure is given by AASHTO Designation: TP 63-03. The limiting value for rutting in Rhode Island is 14 mm for asphalt concrete wearing course. The resulted rutting was under the specifications. Test results and analysis are given in Appendix D.

5.5.2 Fatigue Cracking

This is also one of the key performance parameter for HMA. This performance largely depends on the property of asphalt binder, for which the DSR tests have been performed on Pressure Ageing Vessel (PAV) aged asphalt binder in sections 5.3.2, and results and analysis are summarized in Appendix D.

5.5.3 Thermal or Low Temperature Cracking Test

This key performance parameter is influenced by the tensile strength and stiffness of HMA at low temperature. These properties of asphalt binder at low temperature are evaluated by the Bending Beam Rheometer (BBR) and discussed in section 5.3.3 and Appendix D7.

Low temperature cracking resistance in HMA is evaluated by performing Indirect Tensile Test to find the tensile strength and creep compliance at different temperatures as -20°C, -10°C and 0°C. The testing procedure is given by AASHTO T322-03.

CHAPTER 6. MAINTENANCE AND REHABILITATION ALTERNATIVES FOR SUSTAINABLE PAVEMENTS

After prioritizing sections for maintenance and rehabilitation (M&R), next step is selecting M&R strategies at project level PMS. This chapter reviews current URI M&R practices and the suitability of MicroPAVER strategies to the low volume roads, e.g., URI Kingston campus network. Then, adoption of new strategies, i.e., Cold In-Place Recycling (CIR) and Warm Mix Asphalt (WMA) has been examined.

6.1 URI M&R Practice

Office of Facility and Operations at the University of Rhode Island (URI) typically establishes yearly street maintenance budgets that emphasizes M&R on a worst-case first basis, or in response to complaints and University priorities. This approach works satisfactorily, as long as sufficient funding is available. However, while funding sources dries up and maintenance budgets decreases or stay constant, the need for improvements increases due to greater traffic volumes, aging of pavement and inflated material costs.

Instead of providing preventive maintenance at an early stage, streets are left until much more expensive reconstruction is needed. Unfortunately, the short span of extra service years, during the delay of maintenance, is purchased at a very high price in terms of increased upgrade costs. To orderly prioritize streets for maintenance at the earlier, cost-effective time, it appears that a sustainable PMS is needed through utilizing MicroPAVER and new strategies.

6.2 MicroPAVER M&R Practice

Before applying any M&R strategy, the present and future conditionshave to be assessed for the pavement sections. USA-CERL has developed family curve technique as method of predicting pavement condition, and determining the consequences of M&R budgets in MicroPaver PMS. The family curve represents the condition of sections with same characteristics. Predicting pavement condition at the section level assumes that the deterioration of all sections in a family is similar and is a function only of their present condition regardless of age. M&R planning uses critical Pavement Condition Index (PCI) concept (Figure 3.4). The point when the rate of deterioration and the cost of applying preventative localized maintenance increase significantly, is defined as the critical PCI. (Shahin 1994).

The MicroPAVERprocedure involved determining the critical PCI, assigning M&R alternatives to the pavement sections, and assigning M&R priorities to pavement sections. The work plan report uses the family curves and budget constraints to optimally assign M&R. The deterioration rates of the families helped determine optimal M&R scheduling while the budget limits demanded optimal M&R scheduling. A comparison of the resulting work plans outlines the best scenario.

Pavement condition prediction is based on development of family curves which was done in following four steps:

(1). Grouping pavements with similar material characteristics and traffic conditions,

(2). Identifying and omitting the condition data that are obviously in error,

(3). Identifying other statistically outlier data, and

(4). Fitting the remaining data with a constrained polynomial curve.

Family curves were based on pavement use, pavement rank, surface type, traffic loading, and other factors.

The work plan was developed through considering both life and cost of the pavement for selecting best M&R strategy. It was based on critical PCI and consisted of following steps:

Identify the critical PC1 for each pavement family.

Assign appropriate M&R type to each pavement section for each year in the analysis period.

Rank M&R requirements on the basis of budget limitations.

Calculate M&R cost, future PCI, and backlog of M&R for each budget scenario.

Critical PCI for the families was identified by following steps:

(1) Visually select the critical PC1 range on the basis of the shape of the family deterioration curve.

(2) Select a localized preventive distress maintenance policy to be used in the analysis of budget scenarios.

(3) Apply the selected preventive distress maintenance policy to pavement sections in the family. This can be done using the MicroPAVER network maintenance report.

Plot the cost of localized preventive maintenance per unit area versus PCI for each of the sections.

(5) Select the critical PC1 from the results of Steps 1 and 4.

After establishing the critical PCIs to each of the families, M&R strategies were assigned according to the default policies of MicroPaver PMS (Figure 6.1). There are four M&R policies viz, Localized Stopgap (Safety), Localized Preventive, Global Preventive and

Major M&R as explained in section 3.4. The calculations and results of different M&R cost requirement are given in appendix C.

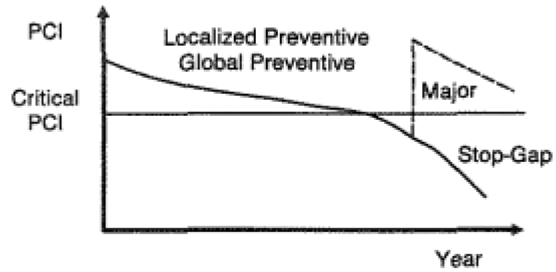

Figure 6.1: M&R types

6.3 Cold In-Place Recycled Asphalt Concrete

A sustainable approach of rehabilitating roadways is to recycle the cracked and broken pavement and reuse it (Maag and Fager 1996). The asphalt paving industry has had great success with recycling asphalt pavements (RAPs) and other recycled materials such as shingles, glass, and ground tire rubber. Recycling of asphalt pavements dates back to 1915, but it did not become a common practice until the early 1970s, when asphalt binder prices skyrocketed as a result of the Arab oil embargo. Asphalt paving technologists reacted to this situation by developing recycling methods to reduce the demand on asphalt binder and, thereby, reduce the costs of asphalt paving mixtures. Many practices that were initially developed during that period are still in use today and have become part of routine operations for pavement construction and rehabilitation. (West 2010). CIR technology leads to various economic and environmental benefits over conventional HMA production method. Use of RAP also reduces the landfill space, which is of great concern. To make asphalt concrete in plants, RAP can be mixed safely from 12% to 15%. However, goal of National Asphalt Pavement Association (NAPA) is to increase it to 25% (West 2010).

These enormous benefits associated with RAP can be considered to develop an effective sustainable M&R strategies for any PMS. In this study these were considered and studied for adoption to URI network. Thermal and fatigue cracking resistance of CIR was evaluated in the present study.

The in-place recycling can save tremendous amounts of time, money, materials and trucking (Epps 1990; Kandhal and Koehler 1987). In the process of CIR, existing pavement materials, particularly wearing and binder course are scraped out and milled together with recycling agents like emulsion and new material without the application of heat and paved again on site. The method can be used to eliminate a variety of distresses such as rutting, cracks, and irregularities while maintaining the original profile and with a minimum traffic disruption. The process can be carried out by using a single machine for milling, mixing, and laydown, or by a train of specialized machines for different steps including milling, crushing, screening of the RAP, and mixing as shown in Figure 6.2.

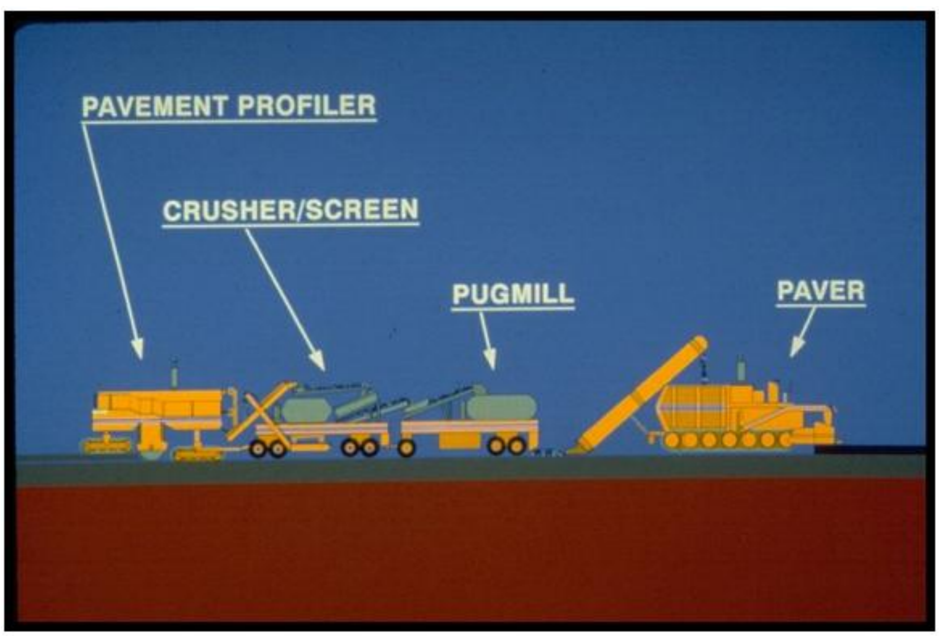

Figure 6.2: Schematic of recycling train

However, experience has shown that some formerly conducted projects with CIR did not work out as desired, which might be due to the inappropriate mix-design (Kearney 1997). Variations in every step of the rebuilding process could also lead to poor reliability which would be necessary to ensure the expected pavement performance.

Most practices are based on existing mixdesign methods for hotmix asphalt (HMA) (Cross and Ramaya 1996). The Oregon Method, for example, uses the Oregon State Highway Division test method 126 to determine the total liquid content, i.e., the sum of water and emulsion content (Rogge 1992). A formula that considers base emulsion content, gradation, residual asphalt content and viscosity gives the amount of emulsion to be added, the remainder is to be filled with water. Observations in the field can justify adjustments. This method seems to lead to good results in some extent, however does not supply comfort for highly reliable performance.

The Asphalt Institute (AI) Method is based on their emulsified asphalt method. The Centrifuge Kerosene Equivalent test is used to determine the optimum binder content (OBC) to produce samples (AI 1979). Strength, modulus and retained strength after moisture conditioning are determined, and the AI recommends the use of the heaviest asphalt that is feasible. Although more methods in addition to the ones mentioned above do exist, none of them offers the desired standard. For that reason, an attempt was made to develop a rational and standardized mix design procedure for CIR using the Superpave gyratory compactor (SGC) (Lee et al. 2002).

Evaluation of the modified Marshall Mix design method in accordance with the AASHTO Task Force No. 38 has suggested that this method is not the future for CIR mix-designs. Expanding use of the Superpave system deems it virtually necessary to provide a mix design for CIR similar to that for HMA with modifications for the nature of cold mixes. Therefore, a volumetric mix design using SGC has been developed for use with CIR materials. The mix design was successfully developed primarily for partial-depth CIR, using emulsion as the recycling additive.

6.3.1 Sample Preparation

RAP was acquired from a construction site of Rhode Island Route 3, and CSS-1h emulsion was used. For the determination of the ‘Optimum Emulsion Content (OEC)’, the mix-design was carried out by first keeping the water content constant and varying the emulsion content. The unit weight of 130 pcf was used, and 116 gyrations were adequate for field density reproduction. Emulsion contents varied from 0.5% (of total mix mass) to 2.0% with increments of 0.5%, while the water content stayed constant at 3.0%. The optimum emulsion content was determined to be 0.7%.

With the emulsion content optimized, the water contents were varied between 2.0 to 3.5%. After performing the same steps as for the OEC determination in the previous step, the Optimum Water Content (OWC) was determined as 3.0%.Two different curing time 6 hrs and 24 hrs in order to simulate early and long-term strength in pavement were used. Also two different curing times were used in order to perform the tests, therefore total of 64 Superpave cylindrical samples were fabricated with two for each combinations.

6.3.2 Performance Tests

An attempt was made to predict the performance of the CIR mixtures prepared with OEC and OWC through the new volumetric mix-deign. The distress modes that were investigated for the performance analysis were rutting, fatigue cracking, and low temperature cracking. The distress modes of rutting and fatigue cracking were investigated using the computer program VESYS some years ago (Steen 2001). Thus, the resistance characteristics against thermal cracking were mainly studied in the present project using indirect tensile (IDT) tester (Kennedy and Anagnos 1993)and Superpave thermal cracking model, TCMODEL (Butler and Roque 1994) (Mueller & Lee, 2012).

6.3.3 Test Results and Analysis

The CIR specimens were tested using the IDT tester in accordance with the AASHTO T 322 procedure at temeperatures of -20, -10, 0^{o}C (-4, 14 and 32^{o}F) (AASHTO 2011). The horizontal and vertical deformation at the analysis temperature were averaged and normalized in order to compare them. Creep compliance *D(t)* was obtained for each testing temperature separately, and it was observed that the CIR mixtures exhibit higher compliance than HMA ones as show in Figure 6.3 and 6.4. CIR mixtures were found to be generally weaker than HMA, exhibit rather ductile behavior, as shown in Figures 6.5 and 6.6. Thus, it has been recommended to use CIR mixture as base materials rather than surface layer.

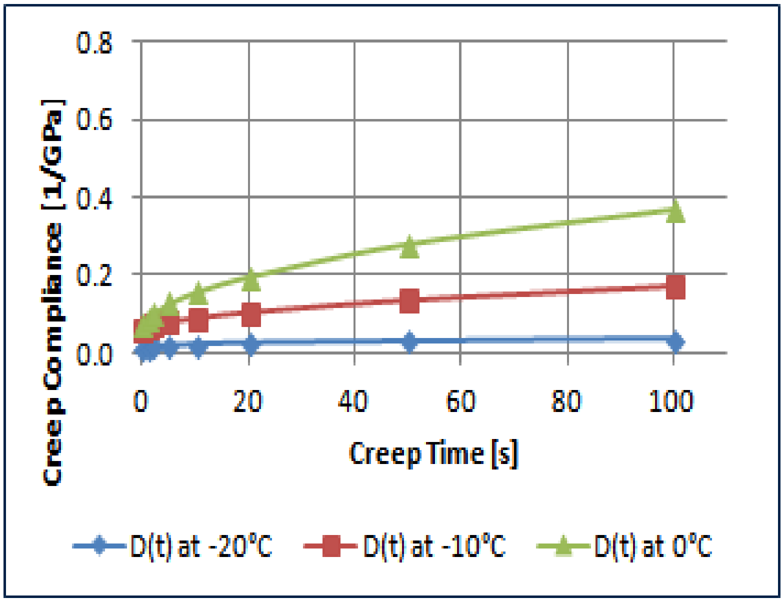

Figure 6.3: Creep Compliance of HMA Mixture

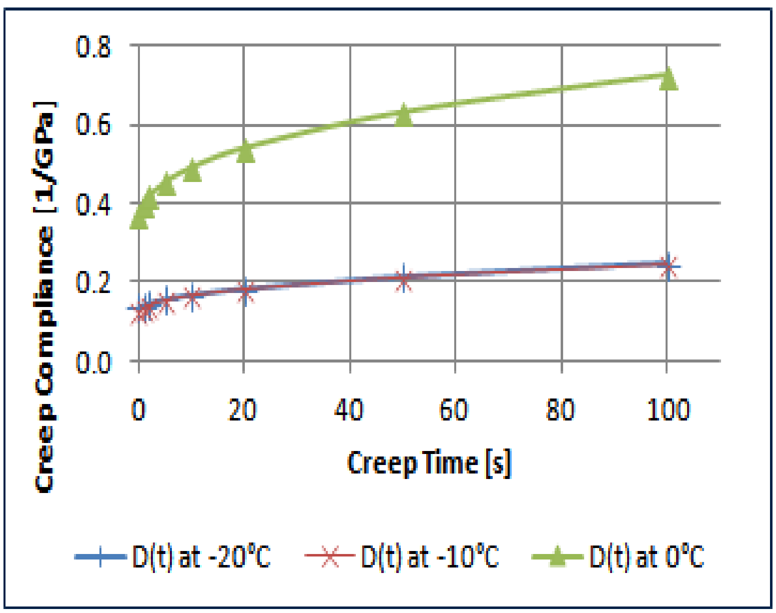

Figure 6.4: Creep Compliance of CIR Mixture

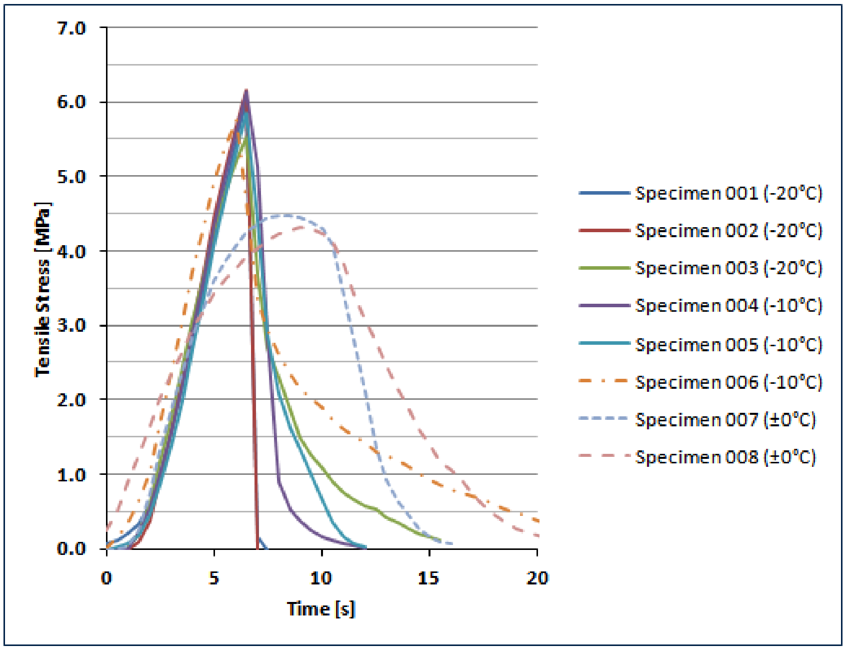

Figure 6.5: Tensile Stress for HMA Mixture

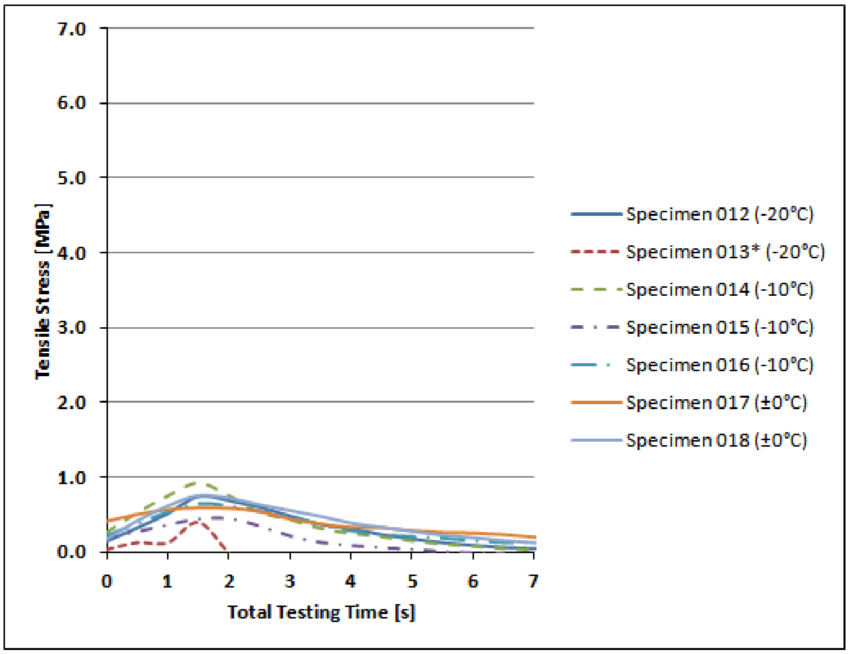

Figure 6.6: Tensile Stress for CIR Mixture

Creep compliance and tensile strength results obtained by the IDT testing were incorporated into Thermal Cracking (TC) Model of Mechanistic-Empirical Pavement Design Guide (MEPDG) software. Since the climate plays a very important role for thermal cracking, the climate files were created based on the history that was known for weather stations in the vicinity of the project site. Then, other parameters, such as subgrade soil properties, layer materials and properties, and traffic, were inputed MEPDG software. The detailed input parameters can be found from Appendix D10. The results of the MEPDG TC model prediction indicated that there will be no thermal cracking for both HMA and CIR mixtures for the simulated pavement structures. The detailed results can be found more from Appendix D11. It may be noted that thermal cracking is not load, but temperature-related distress. Furthermore, creep compliance results showed that CIR mixture allows higher deflections, but ductile behaviour reduces risk of (sudden) cracking. (Lee et al. 2013).

6.4 Warm Mix Asphalt Concrete

Warm mix Asphalt (WMA) is the generic term for a variety of technologies that allow producers of Hot Mix Asphalt (HMA) pavement material to lower temperatures at which the material is mixed and placed on the road (FHWA 2013)

Apart from sustainability, this technology can lead to various beneficial effects like

- Reduce paving costs.
- Extend the paving season.
- Improve asphalt compaction.
- Allow asphalt mix to be hauled longer distances.

- Improve working conditions by reducing exposure to fuel emissions, fumes, and odours.

WMA concrete productions method uses 30°F to 120°F lower temperatures than conventional HMA. Due to this it has been seen that fuel consumption can be reduced to 20 percent. At the time of paving, due to less temperature difference between mix and outside ambient temperature, that makes it feasible to work at night and cold weather conditions depending upon the additive used. WMA also produces less emissions, thus is relatively more environment friendly as compared with HMA (Figure 6.6).

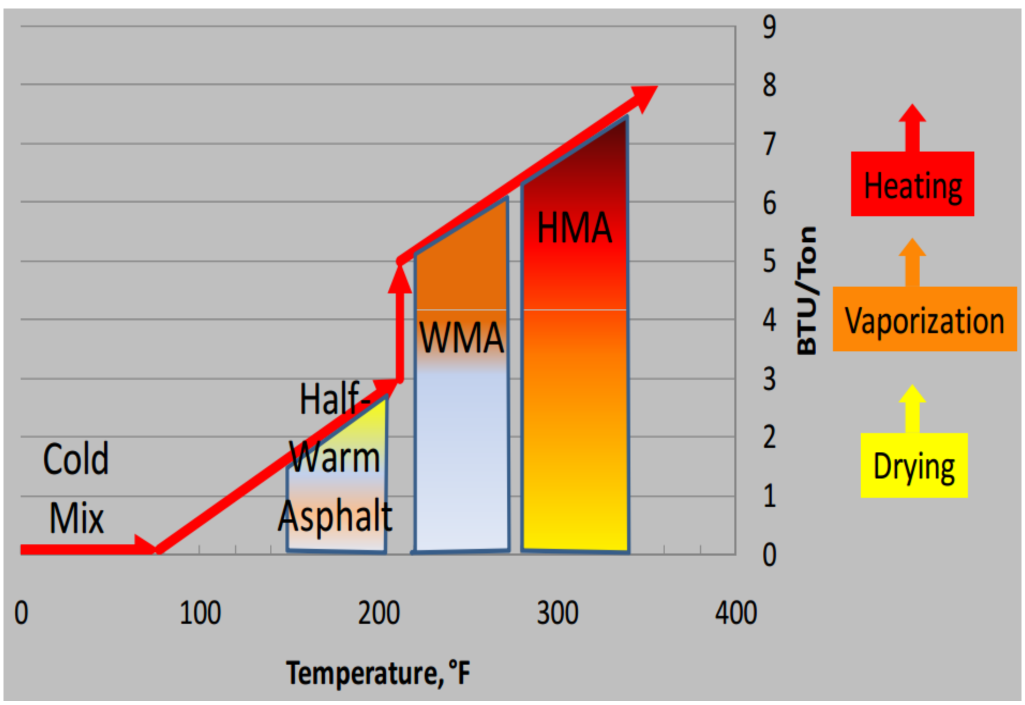

Figure 6.7: Fuel savings vs. mix technology (Source; *NAPA)*

The process of production of WMA involves key additives which are water based, organic, and chemical or hybrid. These additives are added to the mixture to reduce the viscosity by producing foam and thus increasing its workability for compaction at lower temperature. Some of the additives which are in use with their trade names are; **Aspha-Min®, WAM-Foam®, Sasobit®, Evotherm™, Advera® WMA, Asphaltan B®, LeadCAT®.**There are three technologies that have been developed and used in European countries to produce WMA: (FHWA 2013)

1. The addition of a synthetic zeolite called Aspha-Min® during mixing at the plant to create a foaming effect in the binder.
2. A two-component binder system called WAM-Foam® (Warm Asphalt Mix Foam), which introduces a soft binder and hard foamed binder at different stages during plant production.
3. The use of organic additives such as Sasobit®, a Fischer-Tropsch paraffin wax and Asphaltan B®, a low molecular weight esterified wax.

The Aspha-Min and Sasobit products have been used in the United States. Additional technologies have been developed and used in the United States to produce WMA: ((FHWA 2013).

1. Plant production with an asphalt emulsion product called Evotherm™, which uses a chemical additive technology and a "dispersed asphalt technology" delivery system.
2. The addition of a synthetic zeolite called Advera® WMA during mixing at the plant to create a foaming effect in the binder.

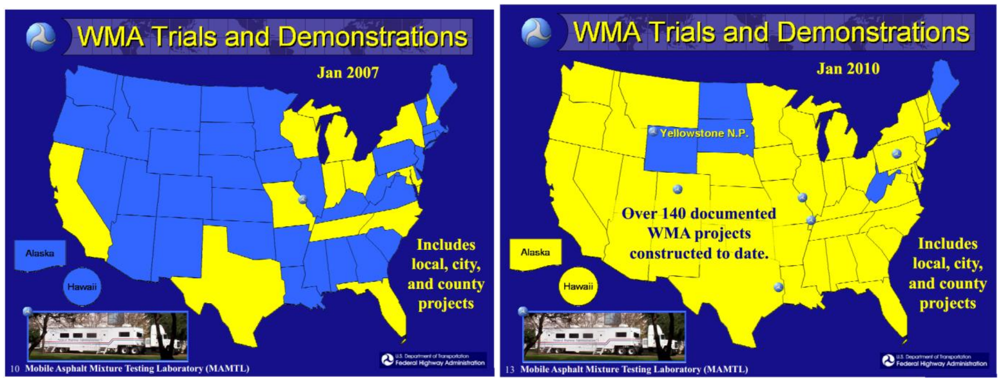

Figure 6.8 WMA Technology Advancement in USA (source NAPA 2013)

Warm Mix Asphalt (WMA) promises various benefits, but probably the most significant is the possibility to reduce the carbon footprint of asphalt, thus supporting the demands of Kyoto protocol for lowering greenhouse gas emissions in the atmosphere. Table 6.1 shows process variables for life cycle inventory (LCI) calculation (Zaumanis 2010).

Table 6.1 Process variables for Life Cycle Inventory (LCI) calculation (Zaumanis 2010)

Production process	Process Variations		
	WMA additive	Energy usage of Asphalt Plant	Compaction effort
HMA, Reference	0	100%	100%
WMA, Production process -20%	3%	80%	100%
WMA, Production process -50%	3%	50%	100%
WMA, Compaction -20%	3%	100%	80%
WMA, Compaction -20%, Production -20%	3%	80%	80%
HMA with 20% RAP	0%	115%	115%
WMA with 40% RAP	3%	100%	100%

The findings from different research discuss the potential problem areas of WMA compared to conventional HMA. The results show varying performance for WMA using different additive products. However, it was found that the performance of WMA products varies depending on circumstances. Therefore, careful examination should be performed with the local materials and in the given climatic conditions to examine the characteristics of a particular WMA product before implementation in local asphalt industry. The assessment also involves selection of the right test methods that would give appropriate evaluation compared to conventional HMA. Based on literature findings, some changes in the methods or conditions for evaluation of WMA are proposed. However most of them are only theoretical.

Performance of WAM pavements was studied in RI to decrease the fuel emissions which are endangering our environment faster than ever (Martinez-Perez and Lee 2012). Not only can energy be saved, but also the cost can be reduced since the necessity to heat up the aggregates and binder in order for the mixture components to bind together, is no longer as high as in HMA. This study evaluated WMA pavement performance, and no significant discrepancies were observed.Tivin and Lee (2013) also studied asphalt binders with crumb rubber (CR) for green highways. CR is promising as an additive in WMA, and reduces the temperature needed to work with asphalt. This would create fewer emissions, reduce energy consumption needed for production, and reduce effects on the health and safety of workers. However there is little information about the long term performance of using it. Thus, the purpose of the present study was to look at the elastic properties of CR modified asphalt binders and to evaluate the performance of these binders with the addition of warm mix additives using two approved AASHTO methods. Elastic Recovery (ER) testing results showed that binders with the additive did not perform as well as binders without it. On the other hand, the Multiple Stress Creep Recovery (MSCR) testing results indicated that there is little difference between binders with or without the given additive. Although these tests show two different conclusive results, WMAs may not comparable to HMAs but over time they may perform equally. More testing would be needed to further support these initial findings.

However, WMA concrete can be the replacement of conventional HMA in future. After extensive investigation and research a special mix design method is required for WMA. Although NCHRP Project 9-43 is proposing a recommended Appendix to AASHTO R 35 to properly prepare and evaluate WMA mixtures for Superpave Volumetric Mix Designs, it is anticipated that these recommendations may be revised prior to their adoption by AASHTO. At this level WMA is shown to be economical and easy for production, applicable at relatively low ambient temperature, which can be used in night too. These characteristics can be considered ideal for implementing it as one of the M&Roption like Stop-Gap M&R policy (especially for pothole repair) before any major M&R.

CHAPTER 7. GEOGRAPHIC INFORMATION SYSTEM APPLICATION

A geographic information system (GIS) integrates hardware, software, and data for capturing, managing, analyzing, and displaying all forms of geographically referenced information. GIS allows us to view, understand, question, interpret, and visualize data in many ways that reveal relationships, patterns, and trends in the form of maps, globes, reports, and charts. A GIS helps us answer questions and solve problems by looking at our data in a way that is quickly understood and easily shared. GIS technology can be integrated into any enterprise information system framework.

The data can be in the form of vector, raster, image or attribute. Geo-referenced spatial data is referenced to the spatial location in two or three dimensional space in the form of point line and polygon in vector data set and in form of pixels in raster data set. This data can be linked to the attributes like length, width, construction history or some other information. In order to have adjacency, connectedness and containment of the data, topology is maintained with some geographically referenced position. GIS can also used as a tool for automated drafting and regeneration of graphics and plots of information maintained in database (Adeyinka 1992). Each element in a GIS has descriptive characteristics contained in an attribute file that may be used for organization, query, and display of large amount of spatial information.

GIS is becoming popular in many infrastructural planning management systems, due to its accuracy and tremendous data handling capabilities (Adeyinka and Lee 1992). GIS plays an increasing role in development of new Pavement Management System (PMS), in order to produce visual and graphically aided support for easy and time saving interpretations. This leads to high improvement in decision making for pavement section identification for distresses, maintenance and rehabilitation strategy and project scheduling. GIS can perform geographic queries in straightforward, intuitive fashion rather than being limited to textural queries (Jain and Nanda 2003).

Since the GIS, with their spatial analysis capabilities, match the geographical nature of the road network, they are considered to be the most appropriate tools to enhance pavement management operations, with features such as graphical display of pavement condition. After the advent of advance digital technologies like in Global Positioning System (GPS), GIS is increasingly used by public for road navigation system, which is increasing the responsibility of authorities to use the GIS technology in various important sector of infrastructure development. PMS can be one of the most important sectors to get the benefit (Adeyinka and Lee 1993). In most of the organization and authorities, there is a growing trend toward integrating PMS data into the GIS. With the technologies advances of such in computer hardware and software, this integration is becoming more realistic. Advantages of such integration includes flexible database editing and ability to visualize display the results of database queries, statistics ad charting, pavement management analyses on a map of the highway network, view network condition through dynamic colour coding of the highway sections, and access sectional data through the graphical map interface (Parida 2005).

7.1 Integration of GIS with PMS

ArcGIS version 10.1 developed by ESRI was used as GIS tool for integration of PMS database with GIS. Main components of Arc GIS suit are ArcMap and ArcCatalog. ArcMap is used to manipulate data set to create maps, symbolize, and analyze the features, using the data within a dataset. ArcCatalog is used to store and retrieve the data to be used in ArcMap or other components of ArcGIS.

Base map for town map of South Kingstown, i.e., a polygon database was obtained from USGS/RIGIS dataset. The base map for the road network was obtained from Rhode Island Geographical Information System (RIGIS) which is available online in TIGER line format system (Figure 7.1). After importing the database into the software, study area was separated out using clip applications of ArcGIS, to get focused URI road network.

The reference system used was Rhode Island State Plane coordinate System, which is being used in the state of Rhode Island as a reference system. The data for streets and roads in URI network was also collected manually in the form of X and Y coordinates, using a handheld GPS device. This manual data was then mapped in the ArcMAP of ArcGIS software, and was compared with one obtained from RIGIS. There were some discrepancies in topology of manual data and RIGIS data, due to local attractions and relatively inprecise device. Therefore, RIGIS street map was used as a vector base data for further integration of URI PMS data. The RIGIS database contains very large number of attributes which are not required for PMS, so they were sorted and deleted.

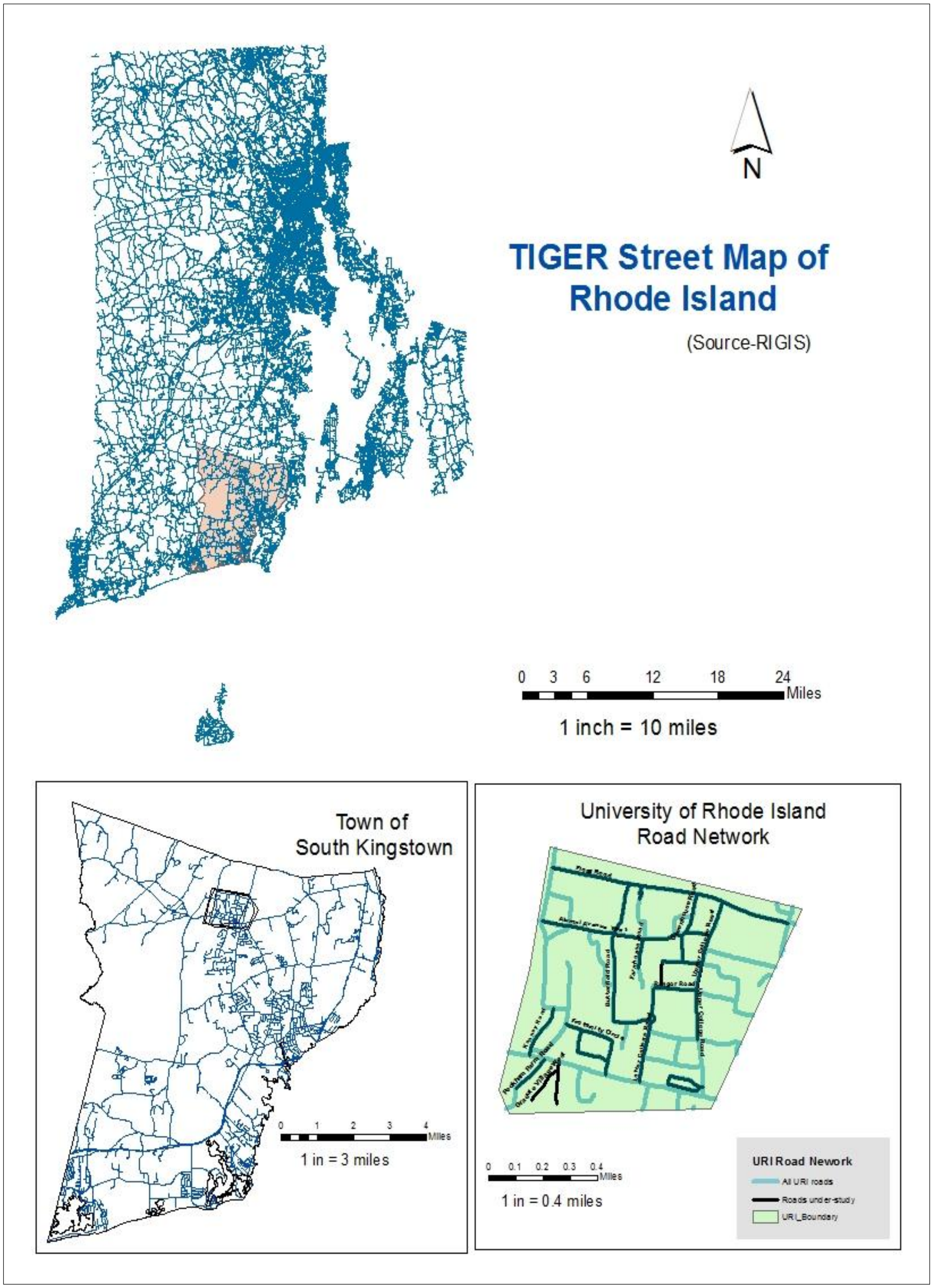

Figure 7.1: TIGER Street Map of Rhode Island

Next step was to obtain PMS database from MicroPaver PMS software. The attribute table of MicroPaver database contains network database based on sections (Figure 7.2). The PMS table attributes are construction history, inspection records, and pavement conditions for different years, etc. (Figure 7.3). These tables were exported to a spreadsheet. These tabular data were all set to be join to the spatial data (centerlines), but the unique identifier did not exist for the spatial data. This required individual coding of each line with its correct ID number. To do this, an item was added in the line attribute file called object_ID. This item attribute was matched to the spatial data which can be identified by the ArcMap.

This tabular information of spatial Data and MicroPaver was joined together in ArcMAP using JOIN through unique identifier "Object_ID". Now this data was available for further manipulation, query and display.

A summary of existing condition and the queried results were mapped using ArcMap and displayed in graphical representation in from of color coded PMS maps. These maps can be provided to the URI Office of Facilities and Operations as a reference of the condition of the University's roadway network. These maps also can be used to present to elected officials for securing budgets for maintenance and repair.

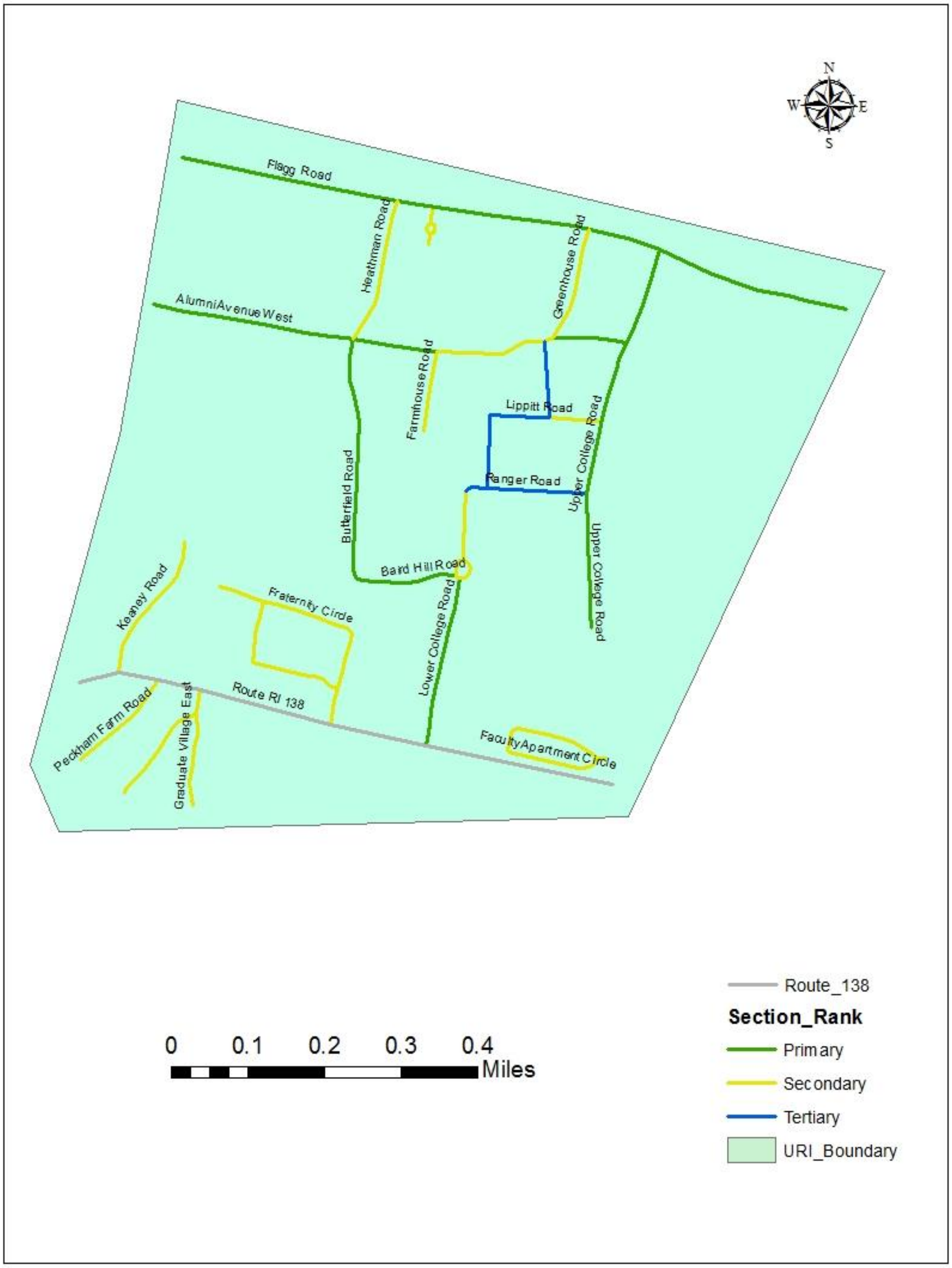

Figure 7.2: Pavement Section Rank of URI Kingston Campus Roadway Network

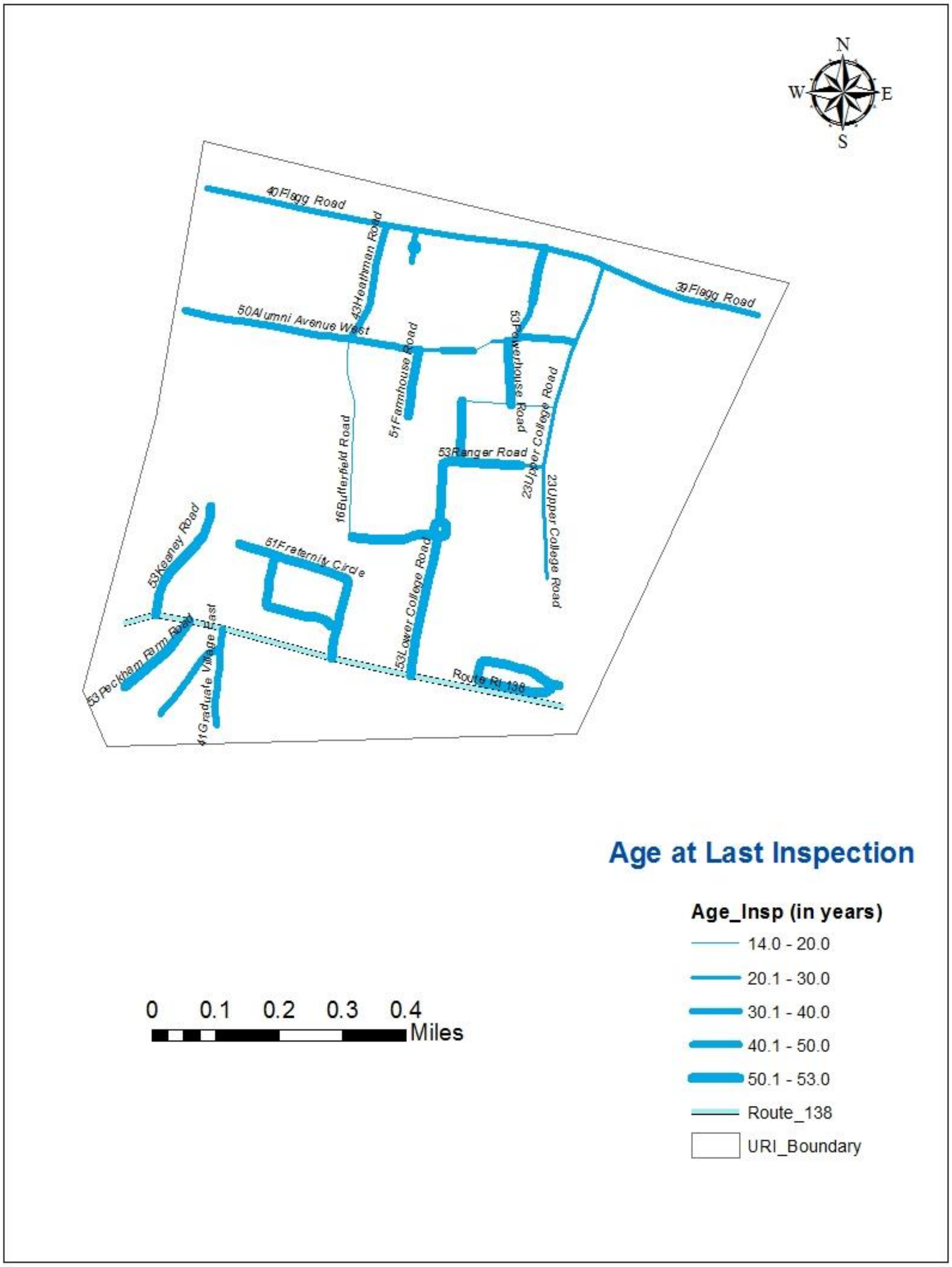

Figure 7.3: Age of Pavement Section at Last Inspection

7.2 GIS Mapping for URI Campus

URI research team provided the pavement section condition of Kingston campus roadway for the years of 1986 and 1990 as shown in Figures 7.4 and 7.5 (Adeynika 1992).

This study started updating the GIS map for the URI Kingston campus roadway network with 1995 pavement condition data, and the result of the PMS and GIS integration is shown in Figure 7.6. The updating efforts were continued with pavement condition data of years 2000, 2005, 2010 and 2013, and results of the PMS and GIS integration are shown in Figures 7.7 through 7.10. The tabular information for the integration is given in Appendix E. For the future study, deflection values using the Falling Weight Deflectometer (FWD) was also integrated with GIS map as shown in Figure 7.11.

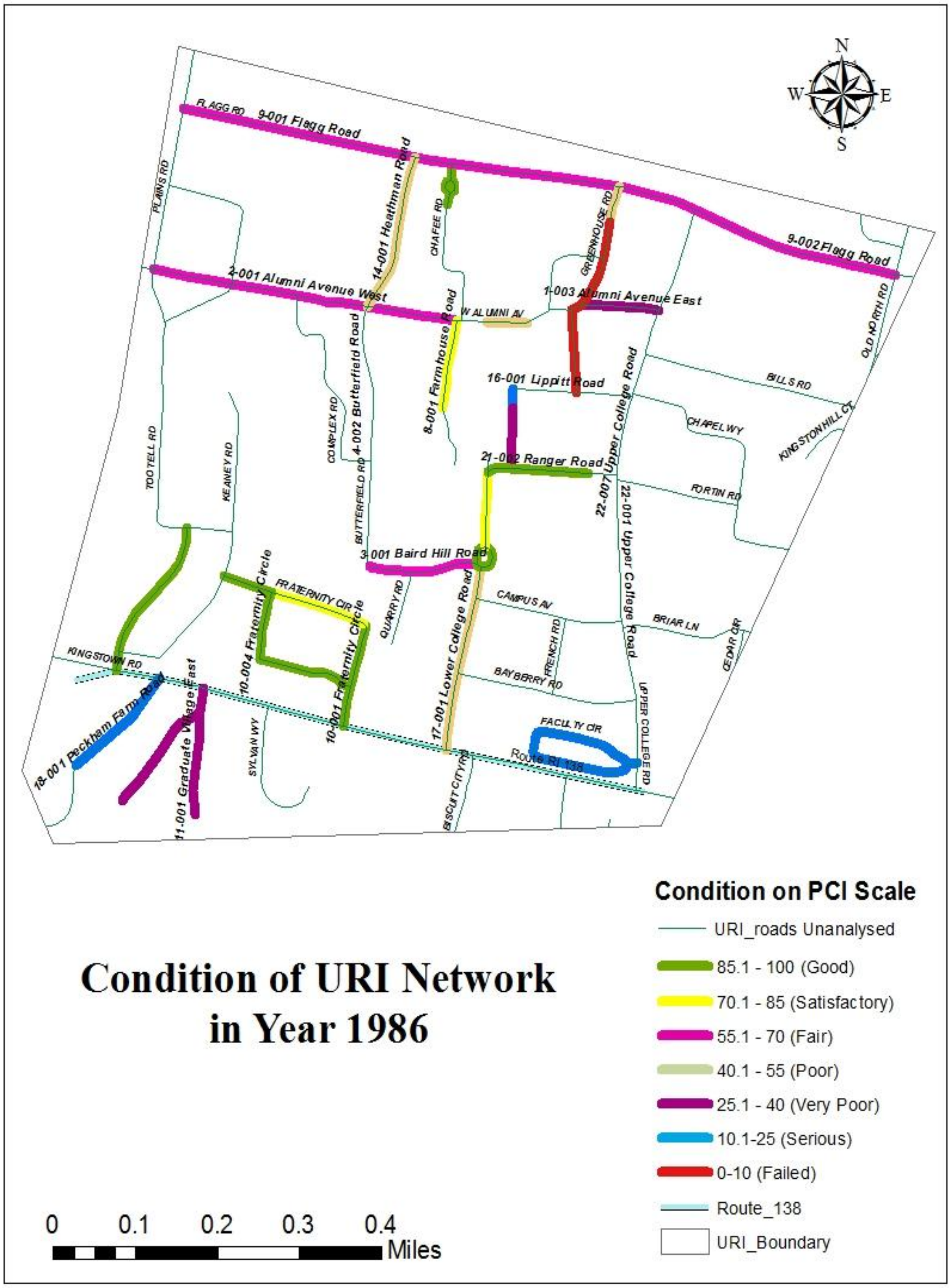

Figure 7.4: Condition of URI Network in Year 1986

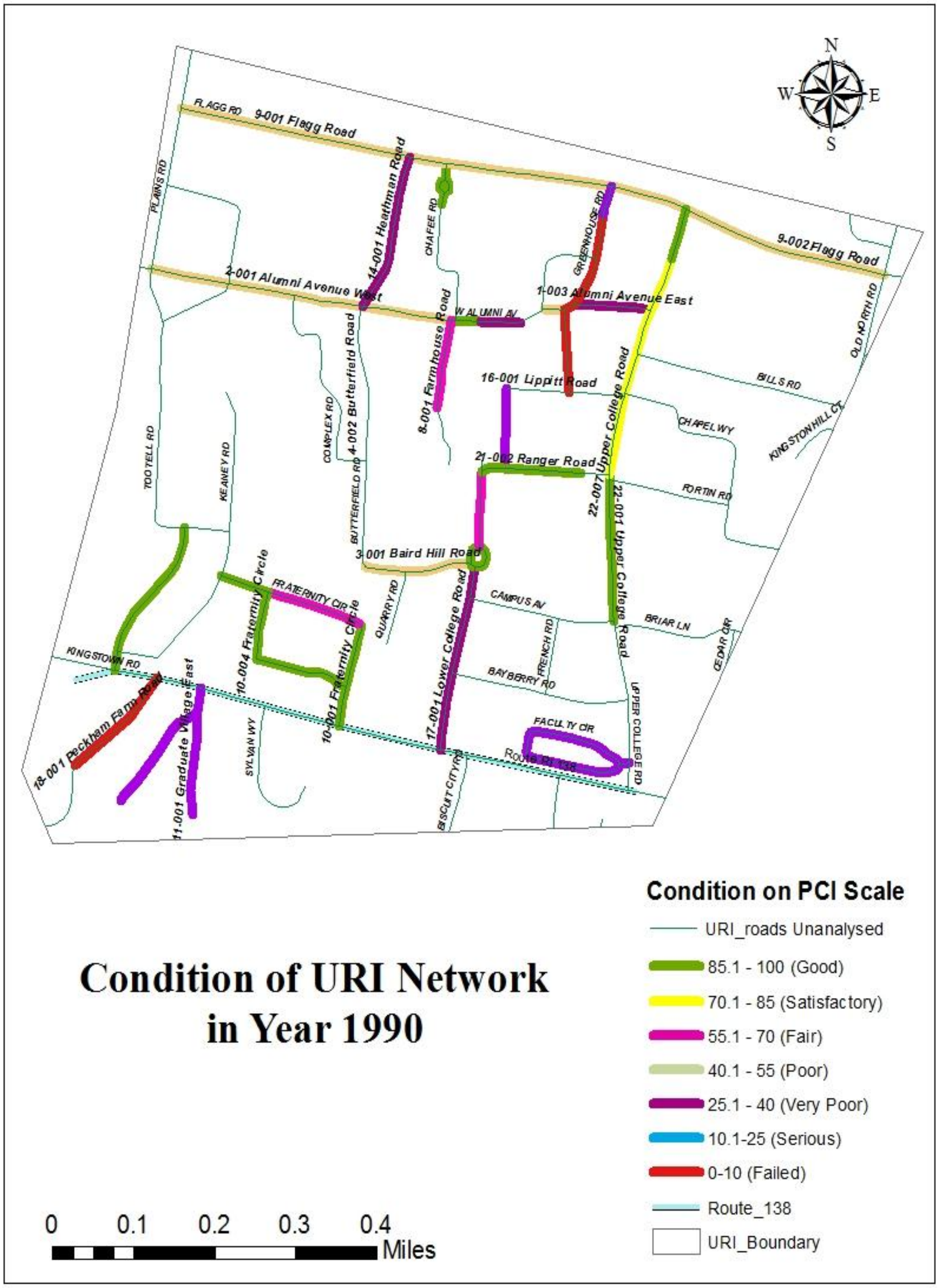

Figure 7.5: Condition of URI Kingston Campus Roadway Network in Year 1990

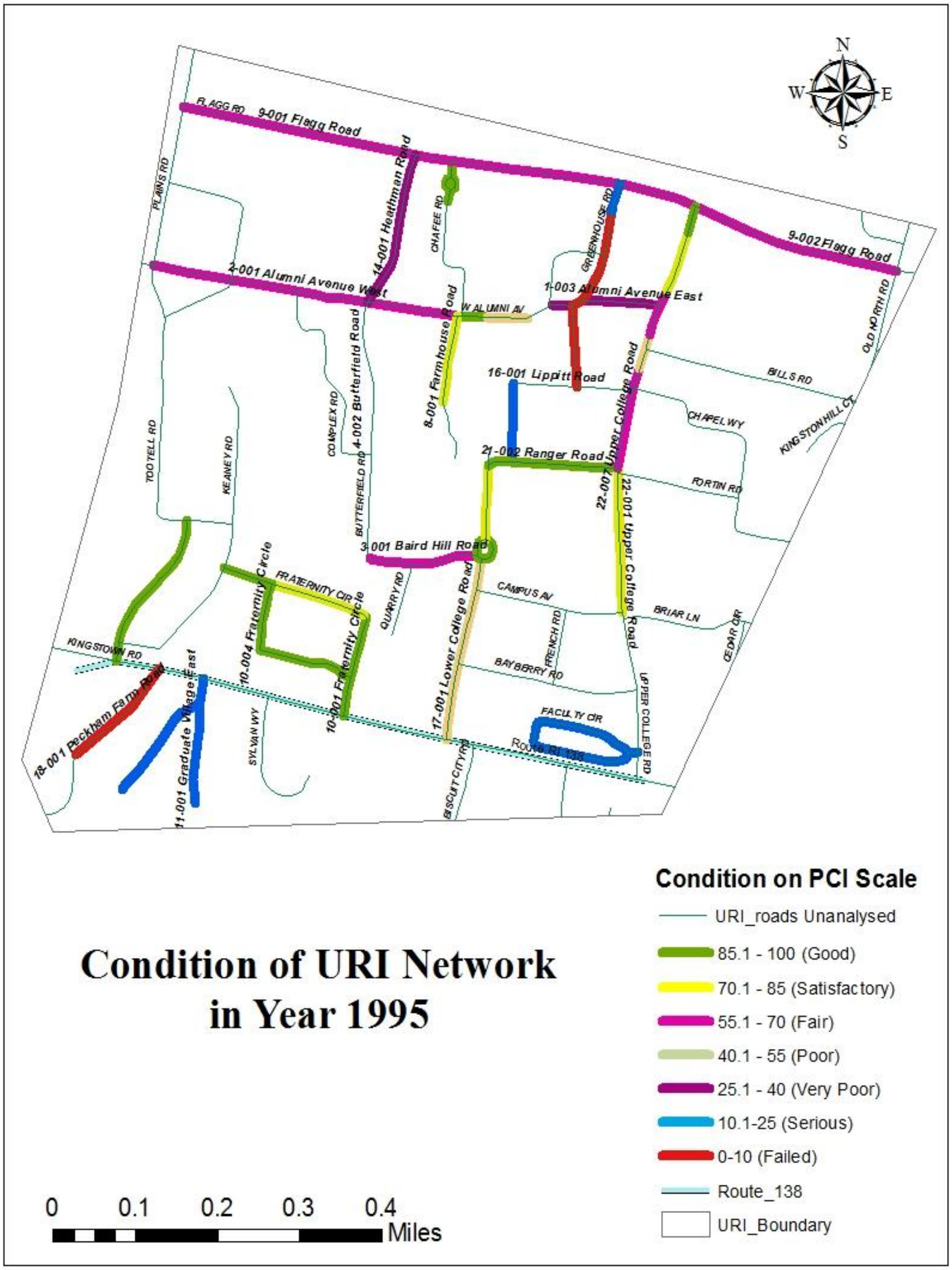

Figure 7.6: Condition of URI Kingston Campus Roadway Network in Year 1995

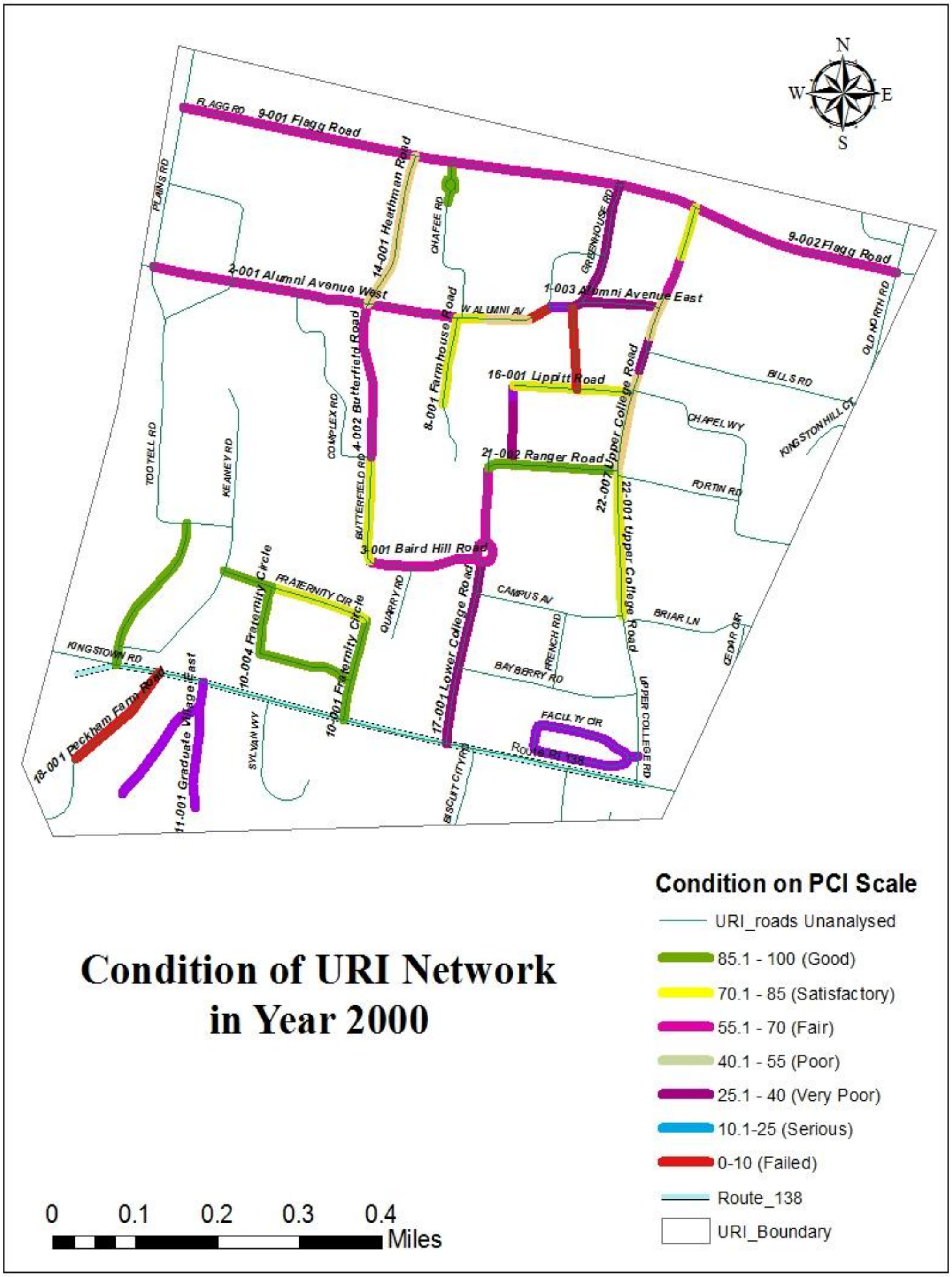

Figure 7.7: Condition of URI Kingston Campus Roadway Network in Year 2000

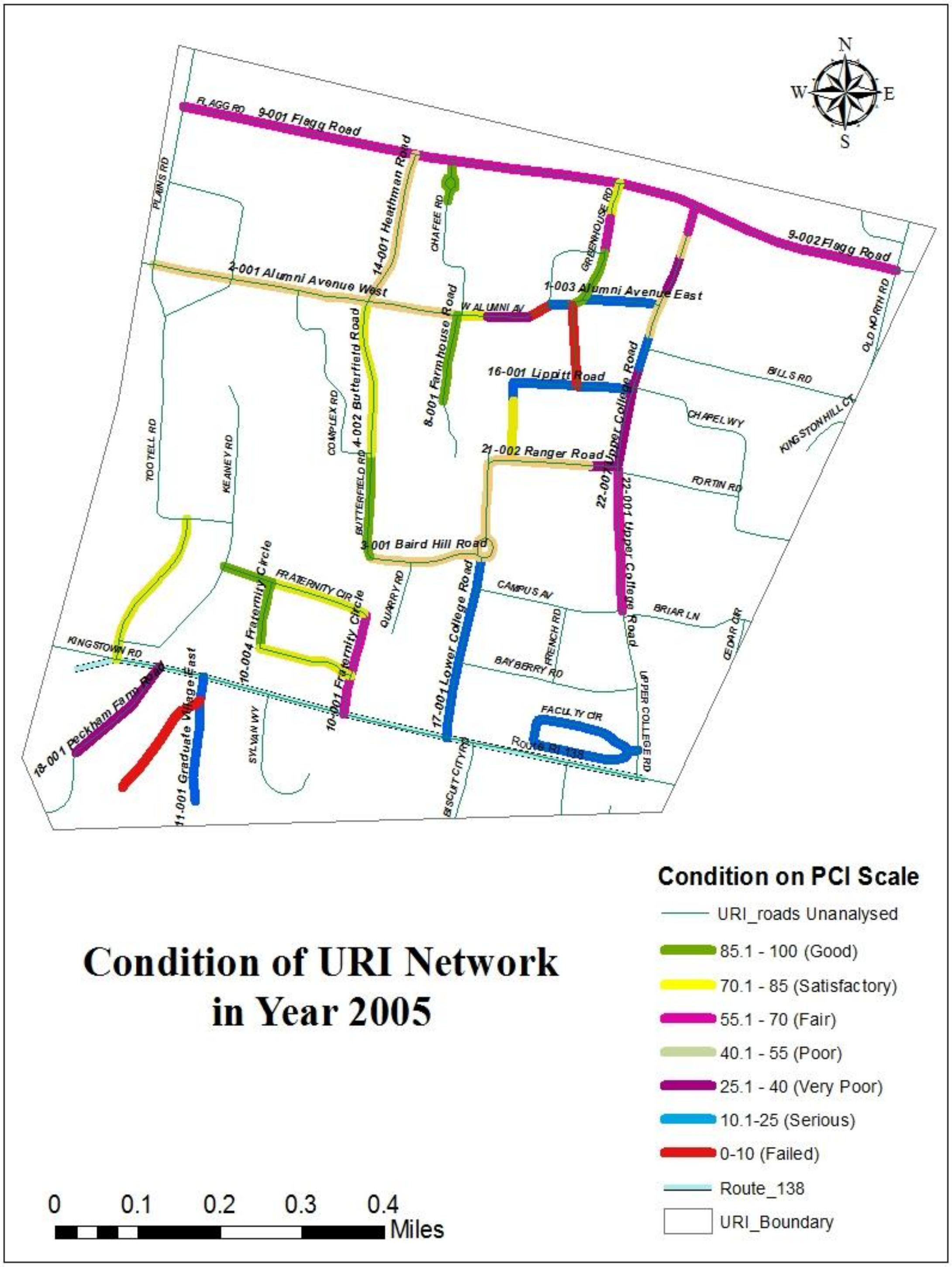

Figure 7.8: Condition of URI Kingston Campus Roadway Network in Year 2005

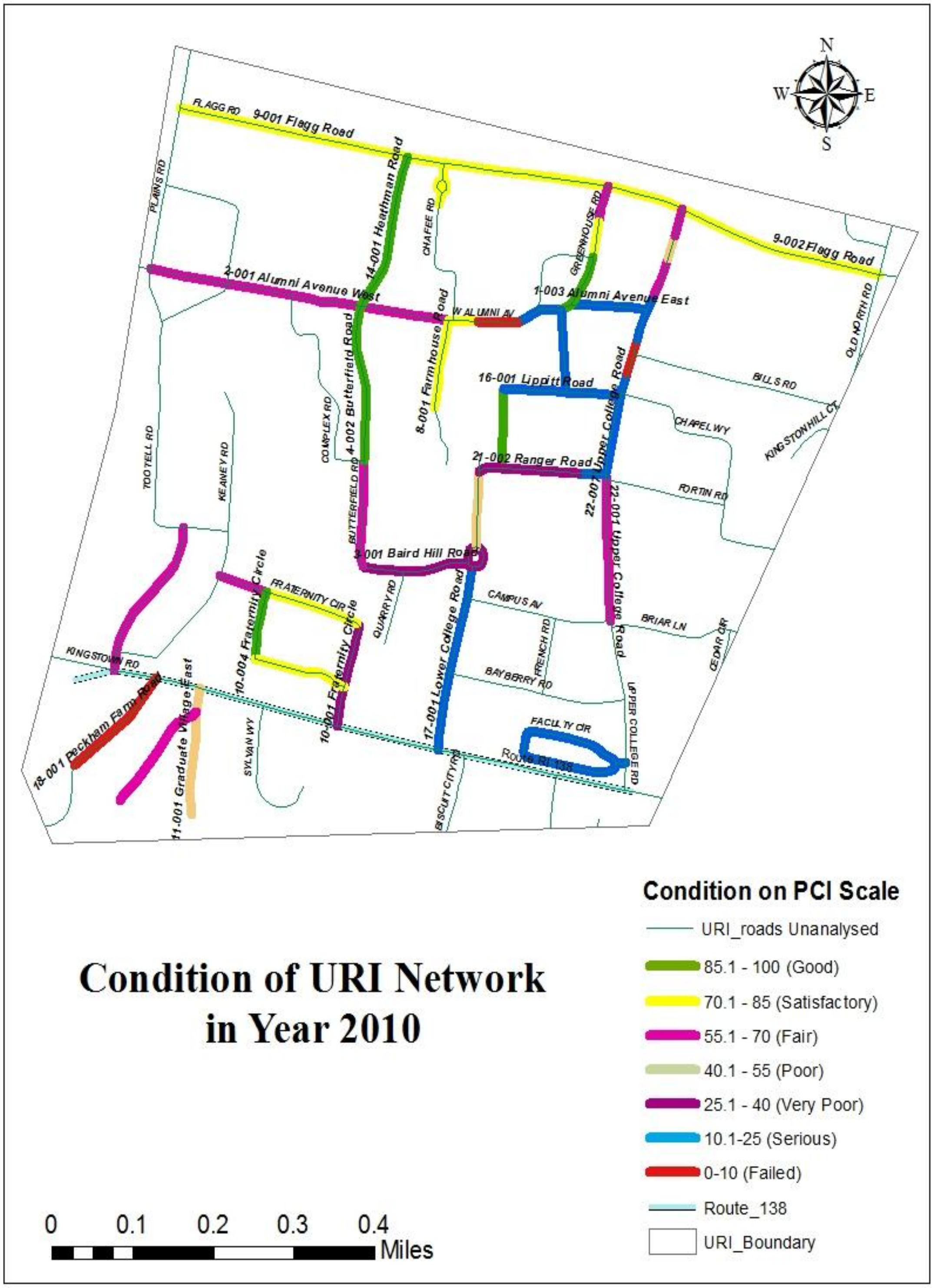

Figure 7.9: Condition of URI Kingston Campus Roadway Network in Year 2010

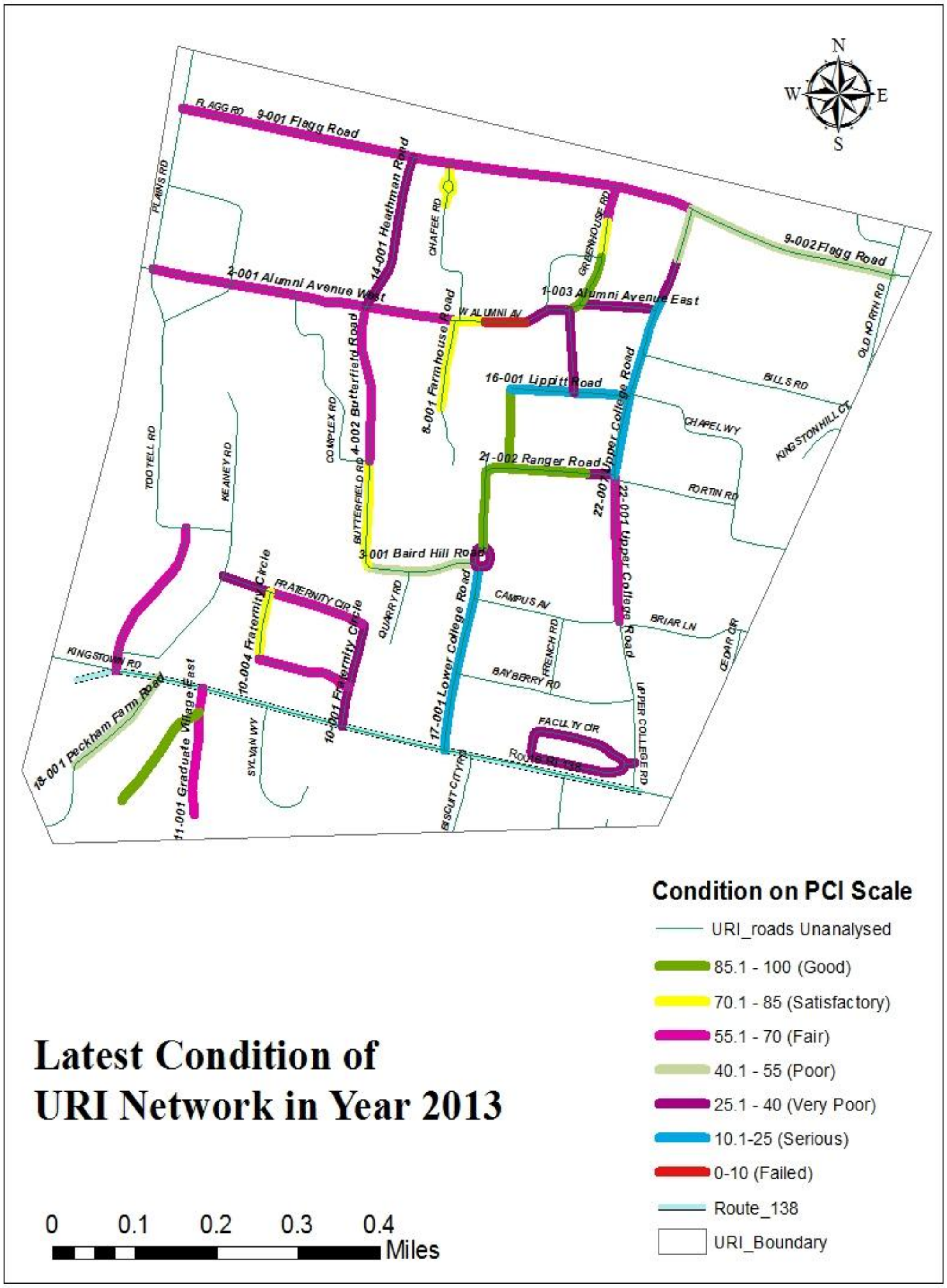

Figure 7.10: Condition of URI Kingston Campus Roadway Network in Year 2013

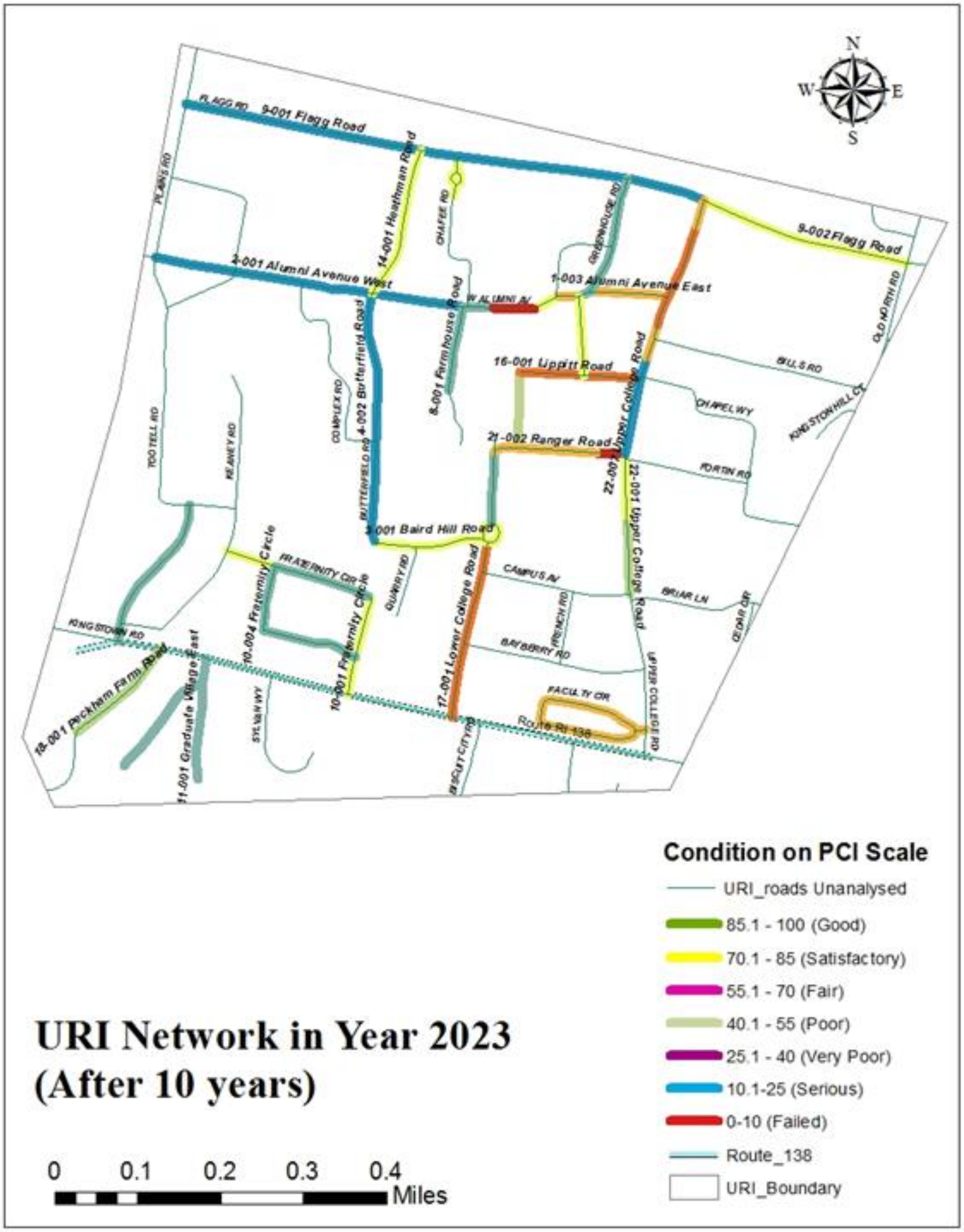

Figure 7.11: Condition of URI Kingston Campus Roadway Network in Year 2023

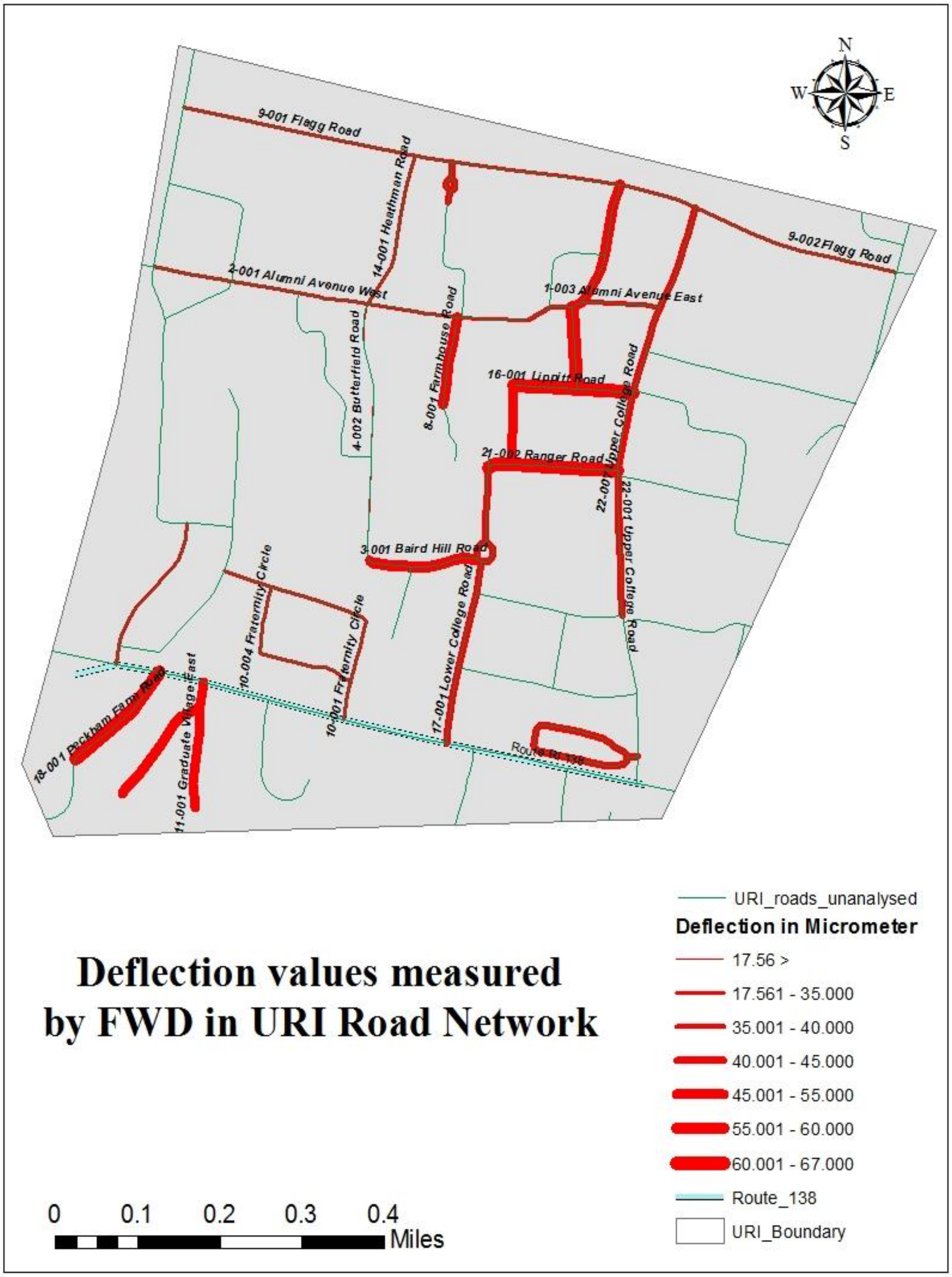

Figure 7.12: Deflection Values Measured by FWD for URI Campus Roadway Network

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations based on results of this study are summarized below.

8.1 Conclusions

1. Asustainable PMS for the URI Kingston Campus was developed using past 13 years distress data with MicroPAVER 6.1.2 computer program.
2. Prioritization of the sections for M&R need was carried out according to PCI values on the basis of 'Worst First' method option.
3. In order to evaluate the future condition of the pavement sections, the prediction model curves were generated between PCI and age for three family group: circulatory (primary), access (secondary) and service (tertiary) in order to consider their unique characteristics and to efficiently apply M&R policies.
4. The deterioration trends followed typical path, i.e., more rapid rate after three-fourth of the total life. It was observed that circulatory sections have least rate of deterioration followed by access and service sections. Critical PCIs of the different policies were selected as 70, 40 and 30 for circulatory (primary), access (secondary) and service (tertiary) sections, respectively.
5. Budget analysis was performed with the above critical PCI and default values of the MicroPAVER 6.1.2 policies as well as existing New England rates for comparison. Costs required for 10 years from current year were evaluated. Annual budget selected was $300,000 with inflation rates of 5 %. Costs of Major, Stop-

gap, Localized Preventative and Global Preventative M&R policy were $2.4M, $0.3M, $1.3M, and $0.03 million, respectively. Cost of Major M&R policies with the rates of Cranston, RI was $ 1.28 M, which was lower than the one with MicroPAVER. It might be due to relatively older rate used in New England.

6. M&R strategies of URI and New England cities and towns were reviewed, and new approaches were explored, e.g., Cold-In-Place Recycling (CIR) and Warm Mix Asphalt (WIM). In this study, CIR and WIM were further examined for their in-service performance, e.g., thermal cracking resistance. Based on limited study, they appear to be promising as alternative M&R policies to provide sustainable PMS in URI and other low-volume roadway networks.

   GIS was successfully integrated with MicroPAVER PMS for URI Kingston campus roadway network to provide fast visual outputs of the conditions and to give better understanding of current and future conditions of pavements for reliable decision making.

8.2 Recommendations

1. It has been recommended that the URI Office of Facilities and Operations should consider adopting the developed MicroPAVER PMS.
2. The Highway Engineering classes should continue to perform pavement condition surveys on the URI Kingston campus roadway network. In order to find more consistent results, work ratesand M&R policies should be regularly updated.

3. The PMS could incorporate other performance monitoring parameters e.g., International Roughness Index (IRI) for evaluating smoothness, and structural capacity in terms of deflection, particularly for high volume roadways.
4. More research needs to be done on recycled pavement materials as M&R alternatives in order to be more reliable. In addition, pavement section families could consider traffic study data like traffic volume, capacity and loading.
5. In order to have visual interpretation of the results and analysis, GIS applications have to be introduced on larger scale. GIS can be incorporated with PMS data collection method using Linear Referencing System (LRS) being used in Rhode Island Department of Transportation.
6. MicroPaver 6.2 incorporated GIS should be improved to have more ability of graphical representation of network conditions, for further analysis and monitoring. GIS maps can be reproduced for implementation of M&R policies in any future year considered, using predicted conditions data of MicroPaver 6.2.
7. MicroPaver PMS should create space in its database for more inventories including sidewalks, catch basins, manholes etc.
8. It is hoped that this effort will assist other campus and agencies which have similar characteristics to URI Kingston campus pavement network, and agencies which has larger scale network for the development of the more comprehensive PMS.

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APPENDIX A

DESCRIPTION OF MICROPAVER^{TM} SOFTWARE

APPENDIX A DESCRIPTION OF MicroPAVER™ SOFTWARE

MicroPAVER is one of the menu-driven microcomputer based computer programs, which incorporates an objective and repeatable visual distress survey methodology. Accordingly, the URI research team developed a comprehensive Pavement Management System (PMS) for the URI Kingston campus roadway using the MicroPAVER (Lee et al. 1988). This study used MicroPAVER (Version 6.1.2), the most recent upgrade of the software.

MicroPAVER is the microcomputer version of the PAVER mainframe program, which was originally developed by a government agency, the U.S. Army Construction Engineering Laboratory (CERL). Currently, the American Public Works Association (APWA) provides continuous support and periodic updates are finished to its users. The MicroPAVER PMS provides the user with practical decision making procedure for identifying cost effective M&R strategies for roads and streets. It provides the user with report generation capabilities for critical information. The MicroPAVER PMS provides its users with many important capabilities. These includes pavement network definition, data storage and retrieval, pavement condition rating, project prioritization, inspection scheduling, determination of present and future network condition, determination of M&R needs, performance of economic analysis and budget planning (Shahin and Kohn 1982).

The following is a brief overview of the MicroPAVER™ components and capabilities.

INVENTORY

MicroPAVERis based on hierarchical inventory system, which is composed of network, branched, sections and the sample units. The information can also be feed by user defined field in which used can define their own fields to meet their management requirements. Virtual inventory allows user to create virtual copies of the existing inventory. It can automatically calculate and updates pavement surface based on the work history information

INSPECTION

Pavement condition in MicroPAVER is accessed using Pavement Condition Index, which ranges from 0 to 100, 0 being failed condition and 100 being good on the scale. As shown in figure A1

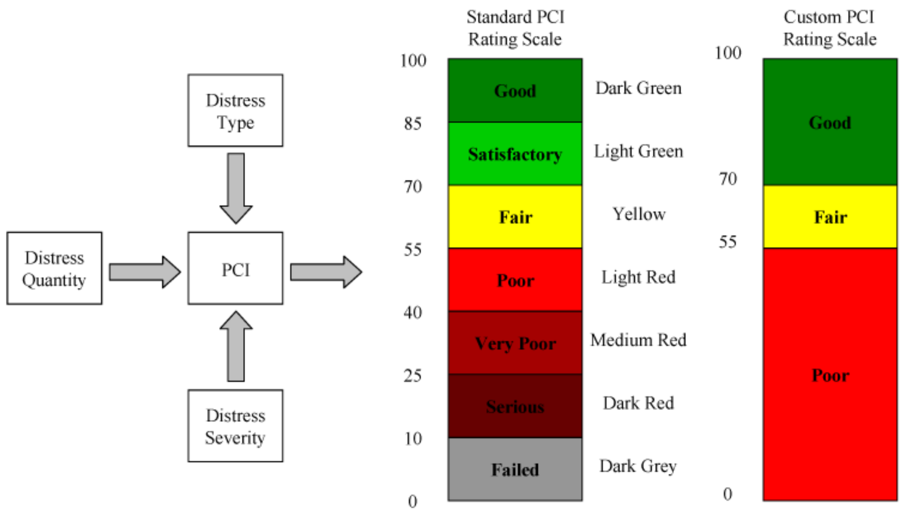


Figure A1 PCI Index

PREDICTION MODELLING

Prediction Modeling conducted by MicroPAVER helps identify and group pavements of similar construction that are subjected to similar traffic, weather, and other factors affecting pavement performance. The pavement condition historical data are used to build a model that can accurately predict the future performance of a group of pavements with similar attributes, As shown in figure A2 (MicroPAVER 6.1 Manual)

The Prediction Modeling function in MicroPAVER™ helps identify and group pavements of similar construction that are subjected to similar traffic, weather, and other factors affecting pavement performance. The pavement condition historical data are used to build a model that can accurately predict the future performance of a group of pavements with similar attributes, Figure 3.

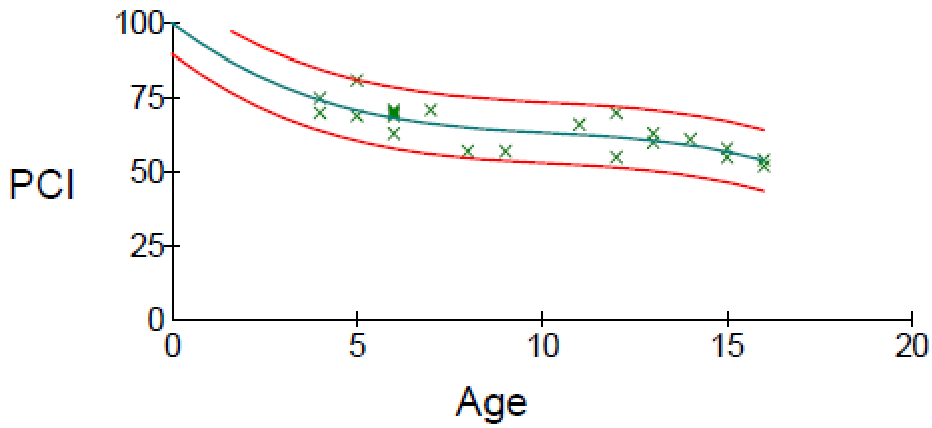


Figure A2 Pavement Prediction Model

CONDITION ANALYSIS

This allows user to view the current and future condition of the pavement based on the interpolated values of previously inspected database. These conditions can be graphically represented as map. Various maps are presented in chapter 7

WORK PLANNING

MicorPAVER is used for planning, scheduling, budgeting and analyzing alternative pavement maintenance and repair (M&R) activities. Group of sections are assigned to M&R families which uses different pavement cost tables, or receive similar type of M&R work. Work planer these families along with inspection data, maintenance policies, maintenance costs, and predictions of future conditions to recommend M&R activities at the section level. This can be done by using diffent options of MicroPAVER^{TM}, like localized preventative, Global or Major M&R policy.

The MicroPAVER™ work plan provides two ways to analyze budgets scenarios. The first way determines the consequence of a selected budget on pavement condition and the resulting backlog of Major M&R (unfunded). In addition to a single budget scenario, MicroPAVER™ 6.1 now offers a new budget split feature. The budget split feature allows the user to split a budget based on different M&R work types. This can aid a user that has a set budget for Global work and a different set budget for Major work. The second way determines the budget requirements to meet specific management objectives, i.e. backlog elimination or PCI goal. This enables managers to develop a variety of funding scenarios to support their decisions. (MicroPAVER^{TM} 6.1 Manual)

PROJECT PLANNIG

This tool allows used to develop projects based on user- specified required work and MicroPAVER recommended work, which helps user in planning projects and upon completion of the project, it automatically update the data.

SELECTION TOOL

This tool is used ot select the required section, or branch in the network

GIS INTERFACE AND ASSIGNMENT TOOLS

MicroPAVER^{TM} has internal mapping to view GIS maps. it can also produce shape files of the reports. Can be used for visual inspection of condition analysis, work plans etc.

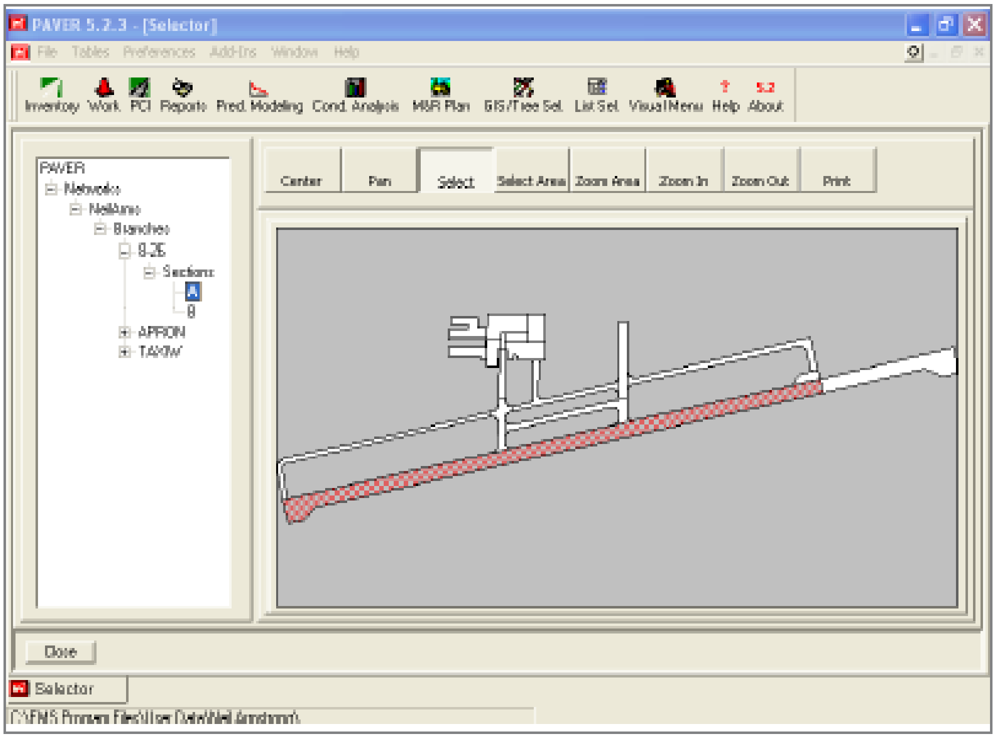


Figure A3 GIS interface window for tree selector in MicroPAVER™

APPENDIX B

SOIL PARAMETERS FOR PAVEMENT DESIGN

APPENDIX B. RESILIENT MODULUS OF URI SUBGRADE SOILS

Subgrade soils have a major impact on the design, construction, structural response, and performance of a pavement. A series of fundamental tests were carried out, results are shown in Table B.1 and Figure B12. All pavement structural design procedures require a subgrade soil input, e.g., California Bearing Ratio (CBR), soil support number, resilient modulus (M_R), k, and R values etc. Without an adequate "working platform" critical pavement construction details may not be accomplished within acceptable tolerances. Frequently, such construction deficiencies are undetected because they are hidden in the finished pavement. A large percentage of the surface deformation of a pavement is originated from the subgrade. Adequate subgrade characterization requires consideration of the fluctuating of subgrade soil properties as function of space (various location with depth in the subgrade soil properties and longitudinal location along the project) and time (seasons of the year and climatic variation).

The recent emphasis in subgrade and granular material evaluation has been with repeated load testing. Resilient modulus and permanent deformation can be quantified based on appropriate repeated load testing data. However, the permanent strain accumulated per load cycle is very small as compared to the total strain in a well design pavement system.

In the 1993 AASHTO Guide and Mechanistic-Empirical Pavement Design Guide (MEPDG), resilient moduli are used to characterize subgrade soils and to assign layer coefficient to granular subbase and base layers. However State Highway Agencies (SHAs) are experiencing considerable difficulties in establishing the appropriate resilient modulus inputs to design pavement structures.

B.1 Repeated Load Testing

Test procedures for repeated load tests have been proposed by several agencies. AASHTO had adopted three procedures T292-91, and T-294-94 and T-307-99 (2003). In these procedures triaxial test conditions (generally constant confining pressure) are used for granular materials. Cohesive soils can be tested in unconfined compression or under triaxial conditions.

Pneumatic and electrohydraullic repeated load equipment has been successfully utilized to apply pulse load in the form of haversine curve. Specimen deformations are typically measure using Linear Variable Displacement Transducer (LVDT). Fig. B.1illustrates the response of a soil to a repeated load pulse.

The intent of many laboratory studies is to simulate the field conditions. For the M_R test the results should duplicate the dynamic loading that the pavement experience and confining pressure the soil undergoes. The M_R is influence by the dynamic load (or deviator stress), and by the density, freeze-thaw cycles and method of compaction. The M_R is therefore the ratio of the stress due the dynamic load to the recoverable axial strain.

$$M_R = \sigma_d/\varepsilon_R \qquad \text{B-1}$$

Where

σ_d =P/A = Stress due to dynamic load , deviator stress,

P = applied axial load

A = Cross sectional area

ε_R = Δ/L_g = Recoverable or resilient axial strain.

Δ = axial deformation, and

L_g = gauge length

B.2 Modulus of Granular Materials

Granular materials "stiffen" (or increased resilient modulus) as the stress state increases. Repeated load on granular soils and materials has demonstrated the highly significant effect on the M_R test results. Many factors that influence the resilient modulus have been identified, such as the number of stress applications, the stress intensity, the age at initial loading, the method of compaction, and moisture content. The M_R for granular material is calculated by

$$M_R = K_1 (\theta)^{k2} \qquad \text{B-2}$$

Where,

K_1& K_2 = experimentally derived factors, and

θ= Bulk stress

$\theta = \sigma_1 + \sigma_2 + \sigma_3 (= \sigma_1 + 2\sigma_3$ in triaxial test)

A typical plot of M_R verses stress state is shown in Figure B1.

B.3 Determination of Resilient Modulus of subgrade Soils

AASHTO MEPDG recommends using the procedure of AASHO T-307-99 for determination of soil resilient modulus. In this procedure, a cylindrical specimen of 6 inches dia. and 12 inch height is prepared and placed in a confining cylindrical cell which allows varying pressure to simulate field conditions. A suitable loading system which can produce repeated load pulse in the form of haversine curve is used with 0.1 sec loading time and 0.9 sec resting time. The deformation in the specimen can be read through LVDTs. To prepare specimens, the soil was obtained from one of the URI construction sites. The soil is granular and larger size than 1 ½ were removed, and Optimum Moisture Content (OMC) was determined using ASTM D698 (AASHTO T99) procedures. The soil specimen was prepared at OMC -2%, or 7.45% moisture content using the procedure of AASHO T-307-99.

Servo-Hydraulic system was used for the testing. It consists of hydraulic component to apply dynamic loading, air pressure to apply confining pressure, and computer component with wave-matrix software to process and read the result. LVDTs were used to record the deformations. The loading sequences used for the test and results are given in Table B.2 and Table B.3, respectively. Test results are also shown in Figure B.2.

Table B1 Proctor test

Water Content of in -situ soil = 7.45%

OMC = 9.45 %

Volume of Mould = 1/30 ft^3

Sample No.	Wt. of Compacted soil in mold (excluding wt. of Mould) (lb)	Water Content (%)		Density of Soil (lbs/ft^3), γ	Density of Soil (lbs/ft^3), γ_d [$= \gamma/(1+w)$]
1	4.41	7.45	in-situ	132.300	123.127
2	4.5	9.45	(+2%)	135.000	123.344
3	4.54	11.45	(+4%)	136.200	122.207
4	4.49	13.45	(+6%)	134.700	118.731

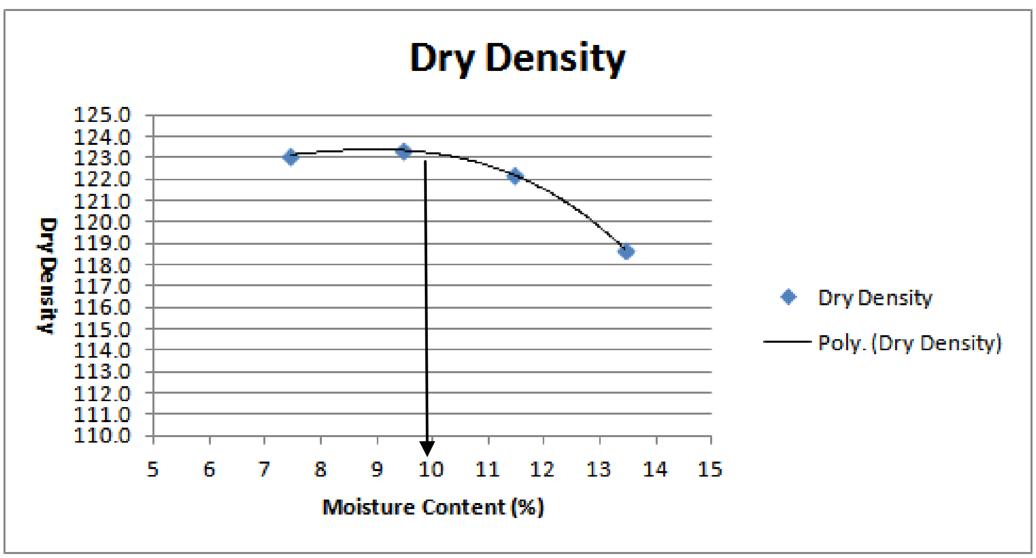


Figure B1 Calculation of OMC of Subgrade Soil

Table B2- Testing Sequence for Subgrade Soil													
Sequence/Step No.	**Confining Pressure (S_3)**		**Max. Axial Stress (S_{max}) (6" dia. Sample)**			**Cyclic Stress, S_{cyclic} (6" dia. Sample)**			**Constant Stress 0.1 S_{max} (6" dia. Sample)**			**No. of Load Applications**
	kPa	**psi**	**kPa**	**psi**	**Load (Lb)**	**kPa**	**psi**	**Load (Lb)**	**kPa**	**psi**	**Load (Lb)**	
0 (Condit..)	41.4	6	27.6	4	113.14	24.8	3.6	**101.83**	2.8	0.4	11.31	500-1000
1	41.4	6	13.8	2	56.57	12.4	1.8	**50.91**	1.4	0.2	5.66	100
2	41.4	6	27.6	4	113.14	24.8	3.6	**101.83**	2.8	0.4	11.31	100
3	41.4	6	41.4	6	169.72	37.3	5.4	**152.74**	4.1	0.6	16.97	100
4	41.4	6	55.2	8	226.29	49.7	7.2	**203.66**	5.5	0.8	22.63	100
5	41.4	6	68.9	10	282.86	62	9	**254.57**	6.9	1	28.29	100
6	27.6	4	13.8	2	56.57	12.4	1.8	**50.91**	1.4	0.2	5.66	100
7	27.6	4	27.6	4	113.14	24.8	3.6	**101.83**	2.8	0.4	11.31	100
8	27.6	4	41.4	6	169.72	37.3	5.4	**152.74**	4.1	0.6	16.97	100
9	27.6	4	55.2	8	226.29	49.7	7.2	**203.66**	5.5	0.8	22.63	100
10	27.6	4	68.9	10	282.86	62	9	**254.57**	6.9	1	28.29	100
11	13.8	2	13.8	2	56.57	12.4	1.8	**50.91**	1.4	0.2	5.66	100
12	13.8	2	27.6	4	113.14	24.8	3.6	**101.83**	2.8	0.4	11.31	100
13	13.8	2	41.4	6	169.72	37.3	5.4	**152.74**	4.1	0.6	16.97	100
14	13.8	2	55.2	8	226.29	49.7	7.2	**203.66**	5.5	0.8	22.63	100
15	13.8	2	68.9	10	282.86	62	9	**254.57**	6.9	1	28.29	100

Table B 3 Calculation of Resilient Modulus of Subgrade Soil

1. SAMPLE NUMBER :-	1	
2. MATERIAL TYPE :-	Granular Subgrade Soil	
3. TEST DATE :-	3/5/2014	
4. RESILIENT MODULUS TESTING		
5. Sample Height	305 mm	
6. Sample Diameter	152 mm	

Calculation of Soil Resilient Modulus

COLUMN #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	14
PARAMETER	Chamber Confining Pressure	Nominal Maximum Axial Stress	Cycle No.	Actual Applied Max. Load	Actual Applied Cyclic Load	Actual Applied Contact Load	Actual Applied Max. Stress	Actual Applied Cyclic Stress	Actual Applied Contact Stress	Recov Def. LVDT #1 Reading	Recov Def. LVDT #2 Reading	Average Recov Def. LVDT 1&2 Reading	Resilient Strain	Resilient Modulus	Resilient Modulus
DESIGNATION	S3	S_{max}	c_1	P_{max}	P_{cyclic}	$P_{contact}$	S_{max}	S_{cyclic}	$S_{contact}$	H_1	H_2	H_{avg}	ε_r	M_r	M_r
UNIT	kPa	kPa	——	N	N	N	kPa	kPa	kPa	mm	mm	mm	mm/mm	MPa	psi
PRECISION	——	——	——	——	——	——	——	——	——	——	——	——	——	——	——
SEQUENCE 1	41.4	13.8	96		-0.2595			-14.30314961		-0.061569202			-0.0002	70.85459108	10276.59
			97		-0.2591			-14.28009474		-0.061456815			-0.0002	70.86974642	10278.79
			98		-0.259			-14.27281814		-0.061228157			-0.0002	71.09816374	10311.91
			99		-0.2596			-14.30579154		-0.061319338			-0.0002	71.15645017	10320.37
			100		-0.2591			-14.27975361		-0.061472838			-0.0002	70.84958159	10275.86
	COLUMN AVERAGE							**-14.28832153**						**70.9657066**	**10292.7**
	STANDARD DEVIATON													**0.149137551**	**21.63057**
SEQUENCE 2	41.4	27.6	96		-0.5163			-28.45143066		-0.15423467			-0.00051	56.26287754	8160.238
			97		-0.5164			-28.45888305		-0.15431048			-0.00051	56.2499665	8158.366
			98		-0.5158			-28.42581487		-0.15455582			-0.00051	56.09541934	8135.951
			99		-0.5163			-28.45184893		-0.15456072			-0.00051	56.14501488	8143.144
			100		-0.5162			-28.44992949		-0.1541661			-0.00051	56.28493225	8163.437
	COLUMN AVERAGE							**-28.4475814**						**56.2076421**	**8152.227**
	STANDARD DEVIATON													**0.082662252**	**11.98914**
SEQUENCE 3	41.4	41.4	96		-0.7684			-42.34838742		-0.40201945			-0.00132	32.12844096	4659.835

			97		-0.7681			-42.3279629		-0.40268511			-0.00132	32.05986108	4649.889
			98		-0.7676			-42.2999394		-0.40318956			-0.00132	31.99855055	4640.996
			99		-0.768			-42.32532208		-0.40459359			-0.00133	31.90664299	4627.666
			100		-0.7683			-42.33817902		-0.40586166			-0.00133	31.81661604	4614.609
	COLUMN AVERAGE							**-42.32795816**						**31.98202232**	**4638.599**
	STANDARD DEVIATON													0.123300196	**17.88318**
SEQUENCE 4	41.4	55.2	96		-1.0212			-56.27811974		-0.83333578			-0.00273	20.59773135	2987.448
			97		-1.0221			-56.32828		-0.83601363			-0.00274	20.55005419	2980.533
			98		-1.0221			-56.32481915		-0.83806066			-0.00275	20.49859952	2973.07
			99		-1.0219			-56.3171149		-0.84009232			-0.00275	20.44622911	2965.474
			100		-1.0213			-56.28230803		-0.84221629			-0.00276	20.38206118	2956.167
	COLUMN AVERAGE							**-56.30612836**						**20.49493507**	**2972.538**
	STANDARD DEVIATON													**0.084751706**	**12.29219**
SEQUENCE 5	41.4	68.9	96		-1.2738			-70.19772963		-1.4353831			-0.00471	14.91609281	2163.396
			97		-1.2737			-70.19227383		-1.4396383			-0.00472	14.87084882	2156.834
			98		-1.2734			-70.1735588		-1.442594			-0.00473	14.83642344	2151.841
			99		-1.2721			-70.10423161		-1.4462087			-0.00474	14.78471997	2144.342
			100		-1.2737			-70.18972779		-1.4505286			-0.00476	14.75866589	2140.563
	COLUMN AVERAGE							**-70.17150433**						**14.83335019**	**2151.395**
	STANDARD DEVIATON													**0.063648598**	**9.231446**
SEQUENCE 6	27.6	13.8	96		-0.2384			-13.13569133		-1.160015			-0.0038	3.453736249	500.922
			97		-0.2383			-13.13249611		-1.1596303			-0.0038	3.454041613	500.9663
			98		-0.2372			-13.07306761		-1.1588018			-0.0038	3.44086937	499.0558
			99		-0.2378			-13.1024512		-1.159918			-0.0038	3.445284595	499.6962
			100		-0.2379			-13.10919986		-1.1599645			-0.0038	3.446920967	499.9335
	COLUMN AVERAGE							**-13.11058122**						**3.448170559**	**500.1147**
	STANDARD DEVIATON													**0.005671063**	**0.822518**
SEQUENCE 7	27.6	27.6	96		-0.4996			-27.53210474		-1.3136074			-0.00431	6.39254312	927.1598
			97		-0.4996			-27.53282061		-1.3137688			-0.00431	6.391923972	927.07
			98		-0.4992			-27.51016858		-1.313765			-0.00431	6.386683629	926.3099
			99		-0.4994			-27.52381855		-1.3138585			-0.00431	6.389397836	926.7036
			100		-0.4994			-27.52360307		-1.3143462			-0.00431	6.38697699	926.3525
	COLUMN AVERAGE							**-27.52450311**						**6.389505109**	**926.7191**
	STANDARD DEVIATON													**0.002713118**	**0.393504**
SEQUENCE 8	27.6	41.4	96		-0.7533			-41.5128686		-1.5759008			-0.00517	8.034404781	1165.292
			97		-0.7535			-41.5223457		-1.5769051			-0.00517	8.031120858	1164.815
			98		-0.7527			-41.47828105		-1.5779593			-0.00517	8.017238291	1162.802
			99		-0.7528			-41.4869216		-1.5788202			-0.00518	8.014535847	1162.41
			100		-0.753			-41.49857221		-1.5802321			-0.00518	8.009623728	1161.697

	COLUMN AVERAGE							**-41.49979783**						**8.021384701**	**1163.403**
	STANDARD DEVIATON													**0.010802028**	**1.566701**
SEQUENCE 9	27.6	55.2	96		-1.0069			-55.48925018		-2.0772775			-0.00681	8.147308824	1181.667
			97		-1.0069			-55.49016499		-2.0800918			-0.00682	8.136419903	1180.088
			98		-1.0061			-55.44392298		-2.0832016			-0.00683	8.117503611	1177.344
			99		-1.0062			-55.4480231		-2.0859505			-0.00684	8.107405734	1175.879
			100		-1.0057			-55.42519142		-2.0890229			-0.00685	8.092148431	1173.667
	COLUMN AVERAGE							**-55.45931053**						**8.120157301**	**1177.729**
	STANDARD DEVIATON													**0.02211232**	**3.20712**
SEQUENCE 10	27.6	68.9	96		-1.2557			-69.20161145		-2.7928795			-0.00916	7.557251036	1096.086
			97		-1.2557			-69.19875129		-2.7974176			-0.00917	7.544679473	1094.263
			98		-1.2561			-69.22277884		-2.8025623			-0.00919	7.533444501	1092.633
			99		-1.2555			-69.18804912		-2.8070128			-0.0092	7.517726667	1090.354
			100		-1.256			-69.21707506		-2.8120475			-0.00922	7.50741511	1088.858
	COLUMN AVERAGE							**-69.20565315**						**7.532103357**	**1092.439**
	STANDARD DEVIATON													**0.02005344**	**2.908505**
SEQUENCE 11	13.8	13.8	96		-0.2221			-12.24008601		-2.5198843			-0.00826	1.481507001	214.8744
			97		-0.2219			-12.23067558		-2.5192486			-0.00826	1.480741541	214.7633
			98		-0.2235			-12.31549558		-2.5187457			-0.00826	1.491308214	216.2959
			99		-0.2226			-12.26679902		-2.5196733			-0.00826	1.484864605	215.3613
			100		-0.2228			-12.27674675		-2.5192482			-0.00826	1.486319514	215.5724
	COLUMN AVERAGE							**-12.26596059**						**1.484948175**	**215.3735**
	STANDARD DEVIATON													**0.004238959**	**0.614809**
SEQUENCE 12	13.8	27.6	96		-0.4841			-26.68007745		-2.6599168			-0.00872	3.059277502	443.7106
			97		-0.4837			-26.6583215		-2.6599682			-0.00872	3.056723783	443.3402
			98		-0.4848			-26.71871992		-2.6604738			-0.00872	3.063067028	444.2602
			99		-0.4842			-26.68274914		-2.6602984			-0.00872	3.059144977	443.6914
			100		-0.4848			-26.71749815		-2.6598256			-0.00872	3.063673399	444.3481
	COLUMN AVERAGE							**-26.69147324**						**3.060377338**	**443.8701**
	STANDARD DEVIATON													**0.002922981**	**0.423942**
SEQUENCE 13	13.8	41.4	96		-0.7378			-40.6570972		-2.8919474			-0.00948	4.287911546	621.9088
			97		-0.738			-40.66900957		-2.892425			-0.00948	4.288459656	621.9883
			98		-0.7375			-40.64501068		-2.8937172			-0.00949	4.284015127	621.3437
			99		-0.7368			-40.60472872		-2.8946316			-0.00949	4.278417419	620.5318
			100		-0.7374			-40.63826148		-2.8959009			-0.00949	4.280073862	620.7721
	COLUMN AVERAGE							**-40.64282153**						**4.283775522**	**621.3089**
	STANDARD DEVIATON													**0.004514332**	**0.654748**
SEQUENCE 14	13.8	55.2	96		-0.9914			-54.63519984		-3.3474844			-0.01098	4.977987635	721.9959
			97		-0.991			-54.61178566		-3.3503512			-0.01098	4.971596597	721.0689

102

			98		-0.9905			-54.58821276		-3.3532902			-0.01099	4.965095145	720.126
			99		-0.9912			-54.62142423		-3.3562843			-0.011	4.963683914	719.9213
			100		-0.9909			-54.60833142		-3.3592014			-0.01101	4.958184729	719.1237
	COLUMN AVERAGE							**-54.61299078**						**4.967309604**	**720.4472**
	STANDARD DEVIATON													**0.007643812**	**1.108641**
SEQUENCE 15	13.8	68.9	96		-1.2395			-68.31029968		-4.0722555			-0.01335	5.116241209	742.0479
			97		-1.2407			-68.37431435		-4.0770205			-0.01337	5.115050532	741.8752
			98		-1.2398			-68.32616006		-4.0820009			-0.01338	5.105211716	740.4482
			99		-1.2401			-68.34184961		-4.0871002			-0.0134	5.100012995	739.6942
			100		-1.2403			-68.35146063		-4.0918651			-0.01342	5.094790513	738.9367
	COLUMN AVERAGE							**-68.34081687**						**5.106261393**	**740.6004**
	STANDARD DEVIATON													**0.009335029**	**1.353931**
										Average Value of First thirteen Sequences				**18.60**	**Mpa**
														2697.61	**psi**

Soil Resilient Modulus Calculations

Sequence	Mr (psi)	STDDEV	Deviator stress (Kpa)	Deviator stress (psi)	Confining stress(psi)	Bulk stress (θ)	Mr (psi)
1	10292.7	21.63057	-14.28832153	2.072	6.000	20.072	10292.70286
2	8152.227	11.98914	-28.4475814	4.126	6.000	22.126	8152.227133
3	4638.599	17.88318	-42.32795816	6.139	6.000	24.139	4638.598959
4	2972.538	12.29219	-56.30612836	8.167	6.000	26.167	2972.538244
5	2151.395	9.231446	-70.17150433	10.178	6.000	28.178	2151.394994
6	500.1147	0.822518	-13.11058122	1.902	4.000	13.902	500.1147271
7	926.7191	0.393504	-27.52450311	3.992	4.000	15.992	926.7191252
8	1163.403	1.566701	-41.49979783	6.019	4.000	18.019	1163.403188
9	1177.729	3.20712	-55.45931053	8.044	4.000	20.044	1177.728939
10	1092.439	2.908505	-69.20565315	10.037	4.000	22.037	1092.438947
11	215.3735	0.614809	-12.26596059	1.779	2.000	7.779	215.3734679

12	443.8701	0.423942	-26.69147324	3.871	2.000	9.871	443.8700902
13	621.3089	0.654748	-40.64282153	5.895	2.000	11.895	621.308949
14	720.4472	1.108641	-54.61299078	7.921	2.000	13.921	720.4471601
15	740.6004	1.353931	-68.34081687	9.912	2.000	15.912	740.600408

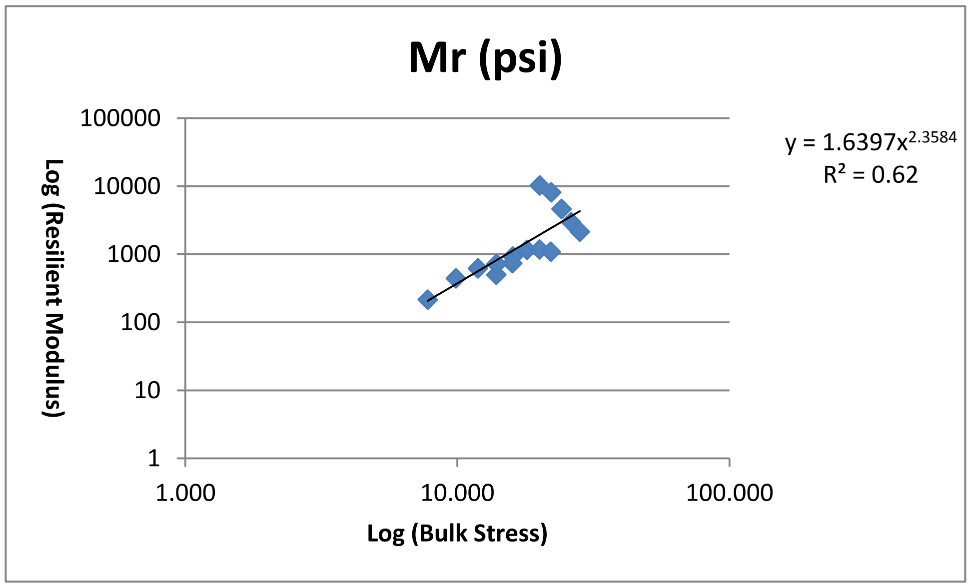


Figure B 2 Log M_R vs Log (Bulk Stress)

APPENDIX C

MAINTENANCE POLICIES AND BUDGET ANALYSIS

Prioritization Table

Network ID	Branch ID	Report_Branch.Name	Section ID	From	To	Last Construction Date	Last Inspection Date	Sec. Rank	Surface	PCI
URI_00-12	002	Alumni Avenue West	003	Rodman Hall	Alumni Avenue East	5/5/1968	4/4/2012	S	AC	2
URI_00-12	020	Quarry Road	001	Baird Hill Road	Quarry Road Parking Lot	5/1/1960	4/10/2012	S	AC	10
URI_00-12	016	Lippit Road	002	Powerhouse Road	Upper College Road	6/1/1999	4/4/2012	S	AAC	18
URI_00-12	022	Upper College Road	007	Lippit Road	University Club	6/1/1990	4/10/2012	P	AC	20
URI_00-12	022	Upper College Road	009	University Club	Bills Road	6/1/1990	4/10/2012	P	AC	22
URI_00-12	016	Lippit Road	001	Davis Road	Powerhouse Road	6/1/1999	4/11/2012	T	AAC	23
URI_00-12	017	Lower College Road	001	Route 138	Memorial Union Bus Circle	4/1/1960	4/10/2012	P	AC	28
URI_00-12	022	Upper College Road	011	Bills Road	Alumni Ave East	6/1/1990	4/10/2012	P	AC	29
URI_00-12	007	Faculty Apartment Circle	001	Upper College Road	Upper College Road	6/6/1960	4/9/2012	S	AC	37
URI_00-12	021	Ranger Road	001	Upper College Road	Edwards Hall	3/1/1991	4/10/2012	T	AAC	39
URI_00-12	001	Alumni Avenue East	002	Section 002	Section 003	6/6/1987	4/10/2012	S	AAC	42
URI_00-12	001	Alumni Avenue East	001	Alumni Avenue West	Section 002	6/4/1997	4/10/2012	S	AC	43
URI_00-12	010	Fraternity Circle	001	Route 138	Alpha Epsilon Pi Building	6/1/1962	4/10/2012	S	AC	43
URI_00-12	014	Heathman Road	001	Alumni Avenue West	Flagg Road	6/1/1970	4/10/2012	S	AC	44
URI_00-12	017	Lower College Road	002	Memorial Union Bus Circle	Memorial Union Bus Circle	6/6/1960	4/10/2012	S	AC	44
URI_00-12	010	Fraternity Circle	003	Chi Omega Building	Keaney Road	3/1/1962	4/10/2012	S	AC	44
URI_00-12	005	Chaffee Road	001	Chaffee Parking Lot	Flagg Road	5/1/1972	4/4/2012	S	AC	46
URI_00-12	001	Alumni Avenue East	003	Section 002	Upper College Road	6/9/1963	4/9/2012	P	AC	46
URI_00-12	022	Upper College Road	013	Alumni Ave East	Fine Arts North	6/1/1990	4/10/2012	P	AC	46
URI_00-12	019	Powerhouse Road	001	Lippet Road	Alumni Avenue East	6/1/1960	4/20/2012	T	AAC	46
URI_00-12	022	Upper College Road	005	Fortin Road	Lippit Road	6/1/1990	4/10/2012	P	AC	47
URI_00-12	003	Baird Hill Road	001	Butterfield Road	Lower College road	6/12/1960	4/10/2012	P	AC	52

URI_00-12	018	Peckham Farm Road	001	Route 138	Dead End	6/1/1960	4/10/2012	S	AC	56
URI_00-12	022	Upper College Road	017	Greenhouse	Flagg Road	6/1/1990	4/10/2012	P	AC	57
URI_00-12	009	Flagg Road	002	Upper College Road	Old North Road	1/1/1974	4/10/2012	P	AC	58
URI_00-12	022	Upper College Road	015	Fine Arts North	Greenhouse	3/1/1990	4/10/2012	P	AC	63
URI_00-12	022	Upper College Road	001	Campus Avenue	Independance Hall	6/1/1990	4/10/2012	P	AC	65
URI_00-12	015	Keaney Road	001	Route 138	Mackal Field House	6/1/1960	4/10/2012	S	AC	66
URI_00-12	011	Graduate Village East	001	Grad. Village East Parking Lot	Route 138	6/1/1972	4/10/2012	S	AC	68
URI_00-12	013	Greenhouse Road	003	Section 002	Flagg Road	2/1/1980	4/4/2012	S	AC	68
URI_00-12	010	Fraternity Circle	002	Alpha Epsilon Pi Building	Chi Omega Building	5/1/1962	4/10/2012	S	AC	68
URI_00-12	010	Fraternity Circle	005	Sigma Kappa Building	Sigma Pi Building	8/1/1962	4/10/2012	S	AC	68
URI_00-12	009	Flagg Road	001	Plains Road	Upper College Road	3/1/1973	4/11/2012	P	AC	72
URI_00-12	002	Alumni Avenue West	001	Plains Road	Farmhouse Road	6/30/1963	4/4/2012	P	AC	74
URI_00-12	004	Butterfield Road	002	Elephant Walk	Heathman Road	6/10/1997	4/10/2012	P	AC	74
URI_00-12	022	Upper College Road	003	Independance Hall	Fortin Road	6/1/1990	4/10/2012	P	AC	75
URI_00-12	002	Alumni Avenue West	002	Farmhouse Road	Rodman Hall	8/21/1990	4/9/2012	S	AAC	82
URI_00-12	004	Butterfield Road	001	Baird Hill Road	Elephant Walk	6/11/1997	4/9/2012	P	AC	83
URI_00-12	008	Farmhouse Road	001	Alumni Avenue West	Farmhouse Road Parking Lot	6/9/1962	4/10/2012	S	AC	83
URI_00-12	013	Greenhouse Road	002	Section 002	Section 003	3/1/1960	4/4/2012	S	AC	83
URI_00-12	010	Fraternity Circle	004	Chi Omega Building	Sigma Kappa Building	7/1/1962	4/10/2012	S	AC	84
URI_00-12	013	Greenhouse Road	001	Alumni Avenue East	Section 002	7/1/1960	4/4/2012	S	AC	89
URI_00-12	021	Ranger Road	002	Edwards Hall	Lower College Road	6/1/1960	4/10/2012	T	AAC	89
URI_00-12	006	Davis Road	001	Lippitt Road	Section 002	6/5/1960	4/10/2012	T	AAC	92
URI_00-12	017	Lower College Road	003	Memorial Union Bus Circle	Davis Road	5/6/1960	4/10/2012	S	AC	92
URI_00-12	012	Graduate Village West	001	Graduate West Parking Lot	Route 138	6/2/1972	4/10/2012	S	AC	94
URI_00-12	006	Davis Road	002	Section 001	Ranger Road	6/5/1960	4/10/2012	T	AAC	94

Table C1Rates of M&R Policies and Expenditure Summary

Rates of M&R policies and expenditure summary have been documented below.

Default Rates for Major M&R Policies			
Code	Name	Amount	Work Unit
CM-OL-2	2 in Cold Mill & Overlay	$1.65	SqFt
OL_2	2 in overlay	$1.20	SqFt
OL_4	4 in overlay	$2.00	SqFt
OL_6	6 in overlay	$3.00	SqFt
AR-CO	AC Surface Recycling - Cold	$0.75	SqFt
AR-HO	AC Surface Recycling - Hot	$1.00	SqFt
BR-SE	Break & Seat & Overlay	$5.00	SqFt
CR-AC	Complete Reconstruction - AC	$6.50	SqFt
CR-PC	Complete Reconstruction - PCC	$10.00	SqFt
CRCK	Crack Sealing	$0.25	SqFt
NC-AC	New Construction - AC	$6.50	SqFt
NC-PC	New Construction - PCC	$9.00	SqFt
OL-AF	Overlay - AC Fabric	$2.40	SqFt
OL-AS	Overlay - AC Structural	$2.00	SqFt
OL-AT	Overlay - AC Thin	$1.20	SqFt
OL-PF	Overlay - PCC Fully Bonded	$5.00	SqFt
OL-PP	Overlay - PCC Partially Bonded	$5.00	SqFt
OL-PU	Overlay - PCC Unbonded	$5.00	SqFt
SU-AC	Surface Course - AC	$1.25	SqFt
SU-DB	Surface Course - Double Bitum.	$0.50	SqFt
SU-PC	Surface Course - PCC	$4.00	SqFt
SU-PF	Surface Course - Porous Friction	$0.80	SqFt
SR-AC	Surface Reconstruction - AC	$3.50	SqFt
SR-PC	Surface Reconstruction - PCC	$7.50	SqFt

Table C2

Defaullt rates for Global M&R Policies			
Code	Name	Amount	Work Unit
NONE	No Global M & R	$0.00	SqFt
OL-AT	Overlay - AC Thin (Global)	$0.90	SqFt
SS-CT	Surface Seal - Coal Tar	$0.10	SqFt
SS-FS	Surface Seal - Fog Seal	$0.10	SqFt
SS-RE	Surface Seal - Rejuvenating	$0.20	SqFt
ST-CS	Surface Treatment - Cape Seal	$0.30	SqFt
ST-MS	Surface Treatment - Micro Surface	$0.35	SqFt
ST-ST	Surface Treatment - Sand Tar	$0.25	SqFt
ST-SB	Surface Treatment - Single Bitum.	$0.25	SqFt
ST-SS	Surface Treatment - Slurry Seal	$0.35	SqFt

Table C3

	Default Localized Preventive M&R		
Code	Name	Amount	Work Unit
CS-AC	Crack Sealing - AC	$1.00	Ft
CS-PC	Crack Sealing - PCC	$1.50	Ft
GR-PP	Grinding (Localized)	$4.00	Ft
JS-SI	Joint Seal - Silicon	$2.75	Ft
JS-LC	Joint Seal (Localized)	$1.50	Ft
NONE	No Localized M & R	$0.00	SqFt
PA-AD	Patching - AC Deep	$7.00	SqFt
PA-AL	Patching - AC Leveling	$1.20	SqFt
PA-AS	Patching - AC Shallow	$4.50	SqFt
PA-PF	Patching - PCC Full Depth	$25.00	SqFt
PA-PP	Patching - PCC Partial Depth	$7.00	SqFt
SH-LE	Shoulder leveling	$1.20	Ft
SL-PC	Slab Replacement - PCC	$15.00	SqFt
UN-PC	Undersealing - PCC	$1.75	Ft

Table C4

	Default Localised Stopgap M&R		
Code	Name	Amount	Work Unit
CS-AC	Crack Sealing - AC	$1.00	Ft
CS-PC	Crack Sealing - PCC	$1.50	Ft
GR-PP	Grinding (Localized)	$4.00	Ft
JS-SI	Joint Seal - Silicon	$2.75	Ft
JS-LC	Joint Seal (Localized)	$1.50	Ft
NONE	No Localized M & R	$0.00	SqFt
PA-AD	Patching - AC Deep	$7.00	SqFt
PA-AL	Patching - AC Leveling	$1.20	SqFt
PA-AS	Patching - AC Shallow	$4.50	SqFt
PA-PF	Patching - PCC Full Depth	$25.00	SqFt
PA-PP	Patching - PCC Partial Depth	$7.00	SqFt
SH-LE	Shoulder leveling	$1.20	Ft
SL-PC	Slab Replacement - PCC	$15.00	SqFt
UN-PC	Undersealing - PCC	$1.75	Ft

Table C5

Expenditure for Major M&R Policy								
Year	NetworkID	BranchID	SectionID	True Area	Area	PCI	Cost	Delay
3/11/201	URI_00-	2	1	43,662.00	SqF	71.6	$72,567.00	8.37%
3/11/201	URI_00-	2	2	3,938.00	SqF	74.9	$5,811.00	14.82
3/11/201	URI_00-	6	1	2,542.00	SqF	80.7	$3,050.00	23.62
3/11/201	URI_00-	9	2	52,800.00	SqF	55.0	$173,871.00	9.91%
3/11/201	URI_00-	22	1	12,610.00	SqF	63.1	$31,084.00	10.10
3/11/201	URI_00-	22	15	4,316.00	SqF	60.6	$11,781.00	10.40
3/11/201	URI_00-	1	2	2,592.00	SqF	41.7	$13,046.00	5.24%
3/11/201	URI_00-	4	1	18,976.00	SqF	77.1	$27,077.00	12.18
3/11/201	URI_00-	9	1	133,880.0	SqF	69.4	$253,974.00	8.42%
3/11/201	URI_00-	1	1	4,375.00	SqF	42.6	$22,595.00	5.27%
3/11/201	URI_00-	10	1	14,775.00	SqF	42.6	$76,306.00	5.27%
3/11/201	URI_00-	10	3	6,424.00	SqF	43.6	$32,327.00	5.28%
3/11/201	URI_00-	13	3	6,324.00	SqF	66.4	$14,793.00	5.47%
3/11/201	URI_00-	14	1	23,790.00	SqF	43.6	$119,717.00	5.28%
3/11/201	URI_00-	17	2	5,320.00	SqF	43.6	$26,772.00	5.28%
3/11/201	URI_00-	5	1	6,732.00	SqF	45.5	$33,794.00	5.29%
3/11/201	URI_00-	8	1	11,550.00	SqF	69.4	$24,199.00	11.70
3/11/201	URI_00-	10	2	15,136.00	SqF	66.3	$37,341.00	5.16%
3/11/201	URI_00-	10	5	13,200.00	SqF	66.3	$32,565.00	5.16%
3/11/201	URI_00-	11	1	19,250.00	SqF	66.3	$47,491.00	5.16%
3/11/201	URI_00-	13	2	7,781.00	SqF	69.4	$16,321.00	11.63
3/11/201	URI_00-	15	1	21,781.00	SqF	65.5	$55,827.00	5.30%
3/11/201	URI_00-	18	1	12,240.00	SqF	55.5	$46,008.00	5.29%
3/11/201	URI_00-	10	4	8,492.00	SqF	68.5	$19,657.00	9.81%
3/11/201	URI_00-	13	1	7,750.00	SqF	69.5	$16,920.00	12.00
3/11/201	URI_00-	19	1	9,600.00	SqF	43.7	$53,135.00	6.08%
3/11/201	URI_00-	21	1	4,862.00	SqF	36.7	$32,465.00	6.12%
3/11/201	URI_00-	21	2	18,200.00	SqF	65.6	$48,795.00	14.20
3/11/201	URI_00-	22	5	11,700.00	SqF	43.4	$65,271.00	6.65%
3/11/201	URI_00-	22	9	4,420.00	SqF	18.4	$34,921.00	5.00%
3/11/201	URI_00-	22	17	4,680.00	SqF	48.7	$22,490.00	7.61%
3/11/201	URI_00-	1	3	17,238.00	SqF	41.8	$105,198.00	6.58%
3/11/201	URI_00-	3	1	23,100.00	SqF	46.3	$125,087.00	6.78%
3/11/201	URI_00-	22	13	7,280.00	SqF	41.8	$44,427.00	6.58%
3/11/202	URI_00-	2	3	4,978.00	SqF	1.23	$43,361.00	5.00%
3/11/202	URI_00-	16	2	10,819.00	SqF	17.2	$94,240.00	5.00%
3/11/202	URI_00-	20	1	7,410.00	SqF	9.23	$64,546.00	5.00%
3/11/202	URI_00-	22	7	3,380.00	SqF	15.2	$29,442.00	5.00%

3/11/202	URI_00-	22	11	6,396.00	SqF	24.2	$55,713.00	5.00%
3/11/202	URI_00-	17	1	32,250.00	SqF	22.6	$294,963.00	5.00%
3/11/202	URI_00-	16	1	9,486.00	SqF	19.1	$91,098.00	5.00%
						Total	$2,420,046.0	

Table C6

M&R Cost requirement for Global M&R									
Year	NetworkID	Branch ID	Section ID	True Area	Area Units	Work Description	PCI Before	PCI After	Cost
3/11/2014	URI_00-12	4	2	29,568.00	SqFt	(2) Surface Seal - Fog Seal	71.62	74.12	$4,435.00
3/11/2014	URI_00-12	12	1	19,521.00	SqFt	(2) Surface Seal - Fog Seal	82.77	94.59	$2,928.00
3/11/2014	URI_00-12	17	3	10,608.00	SqFt	(2) Surface Seal - Fog Seal	81.45	92.56	$1,591.00
3/11/2014	URI_00-12	22	3	9,620.00	SqFt	(2) Surface Seal - Fog Seal	72.38	75.13	$1,443.00
3/11/2019	URI_00-12	4	2	29,568.00	SqFt	(2) Surface Seal - Fog Seal	69.2	70.64	$5,661.00
3/11/2019	URI_00-12	12	1	19,521.00	SqFt	(2) Surface Seal - Fog Seal	72.67	78.54	$3,737.00
3/11/2019	URI_00-12	17	3	10,608.00	SqFt	(2) Surface Seal - Fog Seal	72.05	77.5	$2,031.00
3/11/2019	URI_00-12	22	3	9,620.00	SqFt	(2) Surface Seal - Fog Seal	69.62	71.28	$1,842.00
							Total		$23,668.00

APPENDIX D

PROPERTIES AND PARAMETERS TO DESIGN THE UPPER COLLEGE ROAD AS A MODEL PAVEMENT STRUCTURE

APPENDIX D1. AGGREGASTE GRADATION

This appendix presents the job mix formula (JMF) used for the Superpave mix design of dense graded asphalt mixtures which meet the RIDOT Class I-1 mixture master grading range.

Table D1 Job Mix Formula and Combined Gradation (Dense Graded Asphalt Concrete)

Sieve Size	Components								Total		RIDOT Specs.
	1/2"		3/8"		Screen		Fine Sand				
	[21%]		[21%]		[40%]		[18%]				
	% Passing	% Retained	% Passing	% Retained	% Passing	% Retained	% Passing	% Retained	% Passing	% Retained	% Passing
3/4"	21.0	0.0	21.0	0.0	40.0	0.0	18.0	0.0	100.0	0.0	100
1/2"	18.3	2.7	21.0	0.0	40.0	0.0	18.0	0.0	97.3	2.7	80-100
3/8"	3.4	14.9	19.7	1.3	40.0	0.0	18.0	0.0	81.1	16.2	70-90
#4		3.4	4.2	15.5	39.2	0.8	18.0	0.0	61.4	19.7	50-70
#8				4.2	27.6	11.6	17.8	0.2	45.4	16.0	35-50
#30					10.4	17.2	16.6	1.2	27.0	18.4	18-29
#50					6.0	4.4	13.7	2.9	19.7	7.3	13-23
#100					3.2	2.8	6.5	7.2	9.7	10.0	
#200					1.6	1.6	1.4	5.1	3.0	6.7	3 to 8
Pan						1.6		1.4		3.0	

APPENDIX D2. UNCOMPACTED VOID CONTENT OF FINE AGGREGATES

The purpose of AASHTO Designations T 304-96 (2000) is to determine the uncompacted void content of a given sample of fine aggregates. This is useful in selecting proportions of components used in a variety of mixtures.

A 100mL cylinder was filled with fine aggregate by allowing the sample to flow through a funnel from a fixed height into the measure. The aggregate was leveled off at the top and then weighed. This process was repeated two more times. The uncompacted void content was calculated as the difference between the volume of the cylindrical measure and the absolute volume of the fine aggregates collected therein. A specific gravity of 2.305 was found.

This experiment required a user to fill a container with fine aggregate, and then allow the aggregate to flow through a funnel into a cylinder of known weight and volume. Once full, the top was leveled and the sample was weighed. The process was repeated 3 times to ensure reliable data. Table D2 shows the test results and analysis.

Table D2Mass, Volume, and Void Content

	Total Mass (g)	Mass of Aggregate (g)	Volume of Cylinder	Uncompacted Voids (%)
	352.5	142.5	100	38.0
	353.5	143.5	100	37.6
	355.5	145.5	100	36.7

The uncompacted void of the sample was calculated using the eq. D1.

$$U = \frac{V - {}^{W}/_{G}}{V} \times 100$$

D-1

where:

V = volume of cylinder (mL) (100ml),

F = net mass of aggregate (g)

G = bulk dry specific gravity of fine aggregate (2.305)

U = uncompacted voids (%)

The mean uncompacted void of the sample was calculated using Eq. D-2.

$$U_{MEAN} = \frac{U_1 + U_2 + U_3}{3} \qquad \text{D-2}$$

The mean compacted void of the sample was 12.8%.

The uncompacted voids (of trial one) was calculated as follows:

$$U = \frac{100 - {}^{142.5}/_{2.305}}{100} \times 10 = 38.0\ \%$$

The mean uncompacted void content of the sample was calculated as follows:

$$U_{MEAN} = \frac{38.0 + 37.6 + 36.7}{3} = \mathbf{37.4}\ \%$$

APPENDIX D3. RESISTANCE TO DEGRADATION OF SMALL-SIZE COARSE AGGREGATE BY THE LOS ANGELES MACHINE

The purpose of AASHTO Designations: T 96-02 and/or ASTM Designations: C 131-01 was to determine the percentage of mass of a given sample that was lost due to abrasion and impact. This test is an indicator of the relative quality or competence of various sources of aggregate having similar mineral qualities.

The test sample was placed with six charges in the Los Angeles machine, and was rotated at a speed of 30 to 33 rpm for 500 revolutions. After the prescribed number of revolutions, the material was discharged from the machine, and sieved through the #12 sieve. The loss (difference between the original mass and the final mass of the test sample) was calculated as a percentage of the original mass of the test sample. The test results are shown ing Table D3.

Table D3 Initial and Final Weights of Test Sample

Initial	Final
Size: - #4 to + #8 Weight: 5000g	Size: smaller than #8 sieve Weight: 1170 g
	Size: #8 to #100 sieve Weight: 3752g
	Lost in transition Weight: 78g

The loss (as a percentage of the original mass) was calculated using the following formula,

$$Loss = \frac{M_i - M_f}{M_i} x100 \qquad Eq\ D3$$

The loss (as a percentage of mass) was calculated as follows:.

$$\frac{5000g - 3752g}{5000g} x100 = 35.0\%$$

Thus, the loss for this aggregate was found to be 35.0%.

APPENDIX D 4. SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATES

The purpose of AASHTO Designation: T 85-91 (2000) and/or ASTM Designation: C 127-88 (1993) was to determine the specific gravity and absorption of coarse aggregates by determining bulk specific gravity, saturated-surface-dry bulk specific gravity, and apparent specific gravity of coarse aggregates retaining the #4 sieve which had been soaked in water for 15 hours.

The first step of the test procedure was to remove the test sample from the soaking container and to dry the individual aggregate. Care was taken using a dry cloth to wipe larger particles individually. It was optional to use a stream of air in unison with wiping to dry the surface of the aggregate. This process was performed with care but also done quickly as to avoid the possibility of water evaporation from the saturated aggregate. The aggregate was then weighed on a scale to determine the mass of the sample in the surface-dry condition.

After the surface-dry condition mass was recorded, the sample was immediately placed in a sample container and lowered into a tank of water to determine the sample's mass in water at 23.0 ± 1.7^{o}C. Care was taken to make sure that all trapped air in the sample was removed before the mass was recorded by shaking the sample while it was submerged in the water. The final step consisted of removing the sample from the tank of water. Test results are shown in Table D4.

Table D4 Test Results

Sample	A	B	C
1	2987.5g	3004.5g	1934.0g
2	2984.0g	3000.5g	1934.0g

Note: A = Mass of oven-dry test sample in air, g;
B = Mass of saturated-surface-dry test sample in air, g;
C = Mass of saturated test sample in water, g.

Bulk Specific Gravity was computed as follows:

$$BulkSpecificGravity = \frac{A}{(B-C)} = \frac{2987.5g}{(3004.5g - 1934.0g)}$$

$$= \mathbf{2.791}$$

Apparent Specific Gravity was calculated as follows:

$$ApparentSpecificGravity = \frac{A}{(A-C)} = \frac{2987.5g}{(2987.5g - 1934.0g)}$$

$$= \mathbf{2.836}$$

Average Specific Gravity Values was calculated as follows:

$$G = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2} + \cdots \frac{P_n}{100G_n}}$$

= **2.7945**

APPENDIX D 5. SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATES

The purpose of AASHTO Designation: T 84-00 and/or ASTM Designation: C 127-97 was to determine the specific gravity and absorption of fine aggregate by determining bulk specific gravity, saturated-surface-dry bulk specific gravity and apparent specific gravity.

The first step of the test procedure was to remove the test sample from the soaking container and to dry the individual aggregate. Care was taken using a dry cloth to wipe larger particles individually. It was optional to use a stream of air in unison with wiping to dry the surface of the aggregate. This process was performed with care but also done quickly as to avoid the possibility of water evaporation from the saturated aggregate. The aggregate was then weighed on a scale to determine the mass of the sample in the surface-dry condition.

After the surface-dry condition mass was recorded, the sample was immediately placed in a sample container and lowered into a tank of water to determine the sample's mass in water at 23.0 ± 1.7^{o}C. Care was taken to make sure that all trapped air in the sample was removed before the mass was recorded by shaking the sample while it was submerged in the water. Test results are shown in Table D5.

Table D5 Test Results

Sample	A	B	C	S	Bulk Specific Gravity	App. Specific Gravity
1	485.1g	711.00g	1000.50g	500.0g	2.305	2.48
2	483.5 g	712.00g	1001.50 g	500.0g	2.3	2.492
Average value					**2.303**	**2.486**

Note: A = oven dried

B = just water and pycnometer

C = specimen and water

D = saturated - surface dry specimen mass

The bulk SG and apparent SG were computed from test result as follows:

$$BulkSpecificGravity = \frac{A}{(B+S-C)} = \frac{485.1g}{(711g+500-1000.5g)}$$

$$= \mathbf{2.305}$$

$$ApparentSpecificGravity = \frac{A}{(B+A-C)}$$

$$= \frac{485.1g}{(711g+485.1g-1000.5g)} = \mathbf{2.480}$$

APPENDIX D 6. DYNAMIC SHEAR RHEOMETER TEST

AASHTO Designation T316 and/or ASTM Designation D 7175 Dynamic Shear Rheometer (DSR) test was used to characterize the viscous and elastic behavior of asphalt binder at different temperature range used in Superpave Performance Grading (PG) asphalt binder specification. This test was performed for three types of asphalt; (a) Virgin asphalt, (b) Rolling Thin Film Oven (RTFO) aged asphalt, and (c) Pressure Aging Vessel (PAV) aged asphalt. The purpose of the test was to determine two values, the Dynamic Shear Modulus (G*) and the phase angle (δ). The experiment was performed on a sample of PG 64-28 asphalt.

Rolling Thin Film Oven Aged Asphalt

Rolling Thin-Film Oven (RTFO) simulates short-term aging by heating a moving film of asphalt binder in an oven for 85 minutes at 163° C (325° F). The effects of heat and air are determined from changes incurred in physical properties measured before and after the oven treatment by other test procedures. Then this binder is tested for its viscous and elastic behavior in Dynamic Shear Rheometer (DSR). This characterization is used in the Superpave PG asphalt binder specification. This test is one of the tests which are required for the Superpave system of asphalt concrete design for rutting consideration.

The moving film is created by placing the asphalt binder sample in a small jar then placing the jar in a circular metal carriage that rotates within the oven. The RTFO test is generally considered superior to the previous thin film oven test (TFOT) because:

It uses a rolling action that:

Allows continuous exposure of fresh asphalt binder to heat and air flow

Allows asphalt binder modifiers, if used, to remain dispersed in the sample

Prevents the formation of a surface skin on the sample, which may inhibit aging

RTFO test to simulate short-term asphalt binder aging.

Approximate test time 3 hours from sample preparation to scraping of final bottle. The procedures are as follows:

Heat a sample of asphalt binder until it is fluid to pour. Stir sample to ensure homogeneity and remove air bubbles.

Pour 1.23 oz (35 g) of asphalt binder into each bottle. Immediately after pouring each bottle, turn the bottles on their side without rotating or twisting and place them on a cooling rack.

Allow all bottles to cool 60 to 180 minutes.

Place the bottles in the RTFO oven carousel, close the door, and rotate carousel at 15 RPM for 85 minutes. During this time, maintain the oven temperature at 325°F (163°C) and the airflow into the bottles at 244 in/min (4000 ml/min).

Remove the bottles one at a time from the carousel. Residue from the remaining bottles should be transferred to a single container. Remove residue from each bottle by first pouring as much material as possible, then scraping the sides of the bottle to remove any remaining residue. There is no standard scraping utensil but at least 90 percent of the asphalt binder should be removed from the bottle. RTFO residue should be tested within 72 hours of aging.

Pressure Aging Vessel, Aged Asphalt

The Pressure Aging Vessel (PAV) provides simulated long term aged asphalt binder for physical property testing. Asphalt binder is exposed to heat and pressure to simulate in-service aging over a 7 to 10 year period. This test is one of the tests which are required for the Superpave system of asphalt concrete design for fatigue cracking consideration.

The basic PAV procedure takes RTFO aged asphalt binder samples, places them in stainless steel pans and then ages them for 20 hours in a heated vessel pressurized to 305 psi (2.10 MPa or 20.7 atmospheres). Samples are then stored for use in physical property tests. Approximate test time was 3 hours from sample preparation to scraping of final bottle. RTFO aged asphalt binder is placed in an unpressurized PAV preheated to the test temperature. When the PAV nears the test temperature it is pressurized to 300 psi (2.07 MPa). After 20 hours of treatment the samples are removed, degassed and stored for future testing.

Dynamic Shear Rheometer (DSR) Test

DSR test uses a thin asphalt binder sample sandwiched between two circular plates. The lower plate is fixed while the upper plate oscillates back and forth across the sample at 10 rad/sec (1.59 Hz) to create a shearing action. The test is largely software controlled. The detailed procedures are as follows:

Heat again the asphalt binder from which the test specimens are to be selected until the binder is sufficiently fluid to pour the test specimens.

1. Select the testing temperature according to the asphalt binder grade or testing schedule. Heat the DSR to the test temperature. This preheats the upper and lower plates which allow the specimen to adhere to them.
2. Place the asphalt binder sample between the test plates.
3. Move the test plates together until the gap between them equals the test gap plus 0.002 inches
4. Trim the specimen around the edge of the test plates using a heated trimming tool.
5. Move the test plates together to the desired testing gap. This creates a slight bulge in the asphalt binder specimen's perimeter.
6. Bring the specimen to the test temperature. Start the test only after the specimen has been at the desired temperature for at least 10 minutes.
7. The DSR software determines a target torque at which to rotate the upper plate based on the material being tested (e.g., unaged binder, RTFO residue or PAV residue). This torque is chosen to ensure that measurements are within the specimen's region of linear behavior.
8. The DSR conditions the specimen for 10 cycles at a frequency of 10 rad/sec (1.59 Hz).
9. The DSR takes test measurements over the next 10 cycles and then the software reduces the data to produce a value for complex modulus (G*) and phase angle (δ).

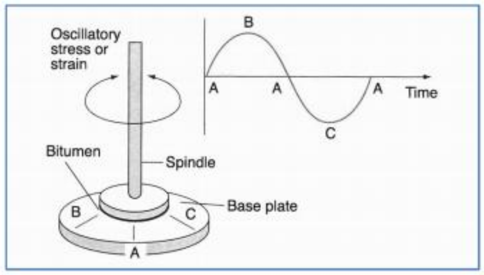


Figure D1 DSR mode of testing

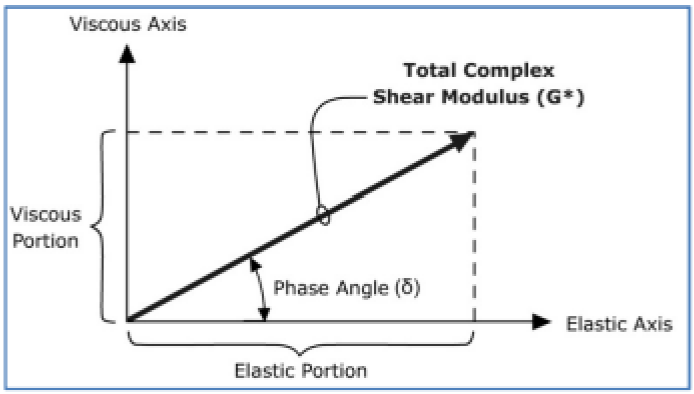


Figure D2 Relationship between shear modulus (G*) and Phase angle (δ)

The test results and analysis are shown below.

Table D 6 Test Results of Dynamic Shear Rheometer

	PG Grade	Temp. (°C)	G* (kPa)	δ (°)	G*/sinδ (kPa)	Strain Amplitude (%)	Permissible G/sinδ(kPa)
Virgin Asphalt							
Sample 1	64-28	64	7.13	78.2	**7.3**	10.02	
Sample 2	64-28	64	7.55	77.9	**7.72**	9.97	≥ 1.00 (passed)
			Average		**7.5**		
RTFO Aged Asphalt							
Sample 1	64-28	64	1.66	69	**1.78***	10.01	
Sample 2	64-28	64	1.14	68.4	**12.26**	9.91	≥ 2.20
			Average		12.26		
Pressure Aged Vessel Asphalt							
	PG Grade	**Ambt. Temp. (°C)**	**G* (kPa)**	**δ (°)**	**G*sinδ (kPa)**	**Strain Amplitude (%)**	**Permissible G*sinδ(kPa)**
Sample 1	64-28	25±1	556.93	31.3	**289.05**	1.04	
Sample 2	64-28	25±1	550.06	30.2	**280**	1.04	≤ 5000 (Passed)
			Average		**284.525**		

Parameters Measured were: (1) Complex modulus (G*) and Phase angle (δ). The complex modulus (G*) can range from about 0.07 to 0.87 psi (500 to 6000 Pa), while the phase angle (δ) can ranges from about 50 to 90°. A δ of 90° is essentially complete viscous behavior. Polymer-modified asphalt binders generally exhibit a higher G* and a lower δ. This means they are, in general, a bit stiffer and more elastic than unmodified asphalt cements.

Next step was to determine G*/sinδ for Virgin and RTFO aged Asphalt, and G*sinδ for PAV aged asphalt. PG asphalt sample passed all the tests except one sample of RTFO aged asphalt, which was discarded.

APPENDIXD7 BENDING BEAM RHEOMETER TEST

Bending Beam Rheometer (BBR) test was performed in accordance with AASHTO DesignationT 313-04 procedure, the results are shown below.

Table D 7(I) BBR test

Tests	Test 1	Test 2	Test 3
Temperature, °C	-10.4	-10.2	-9.9
Stiffness, MPa at 60 Sec	43.9153	31.858	31.5818
m-value	0.410758	0.391503	0.389543

Table D 7(II) BBR test

	Test 1	Test 2	Test 3
t, time (sec)	**Measured Stiffness (Mpa), S**	**Measured Stiffness (Mpa), S**	**Measured Stiffness (Mpa), S**
8	89.7039	63.9965	63.3657
15	73.5804	52.8525	52.16
30	57.9242	41.8127	41.1152
60	**43.9153**	**31.858**	**31.5818**
120	32.3468	23.8865	23.6795
240	23.6632	17.8998	17.7475

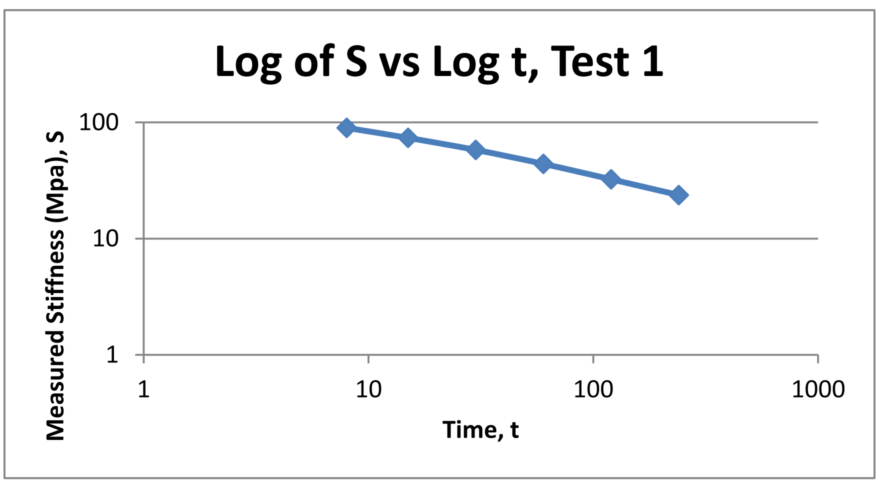


Figure D3 Test Results of Bending Beam Rheometer

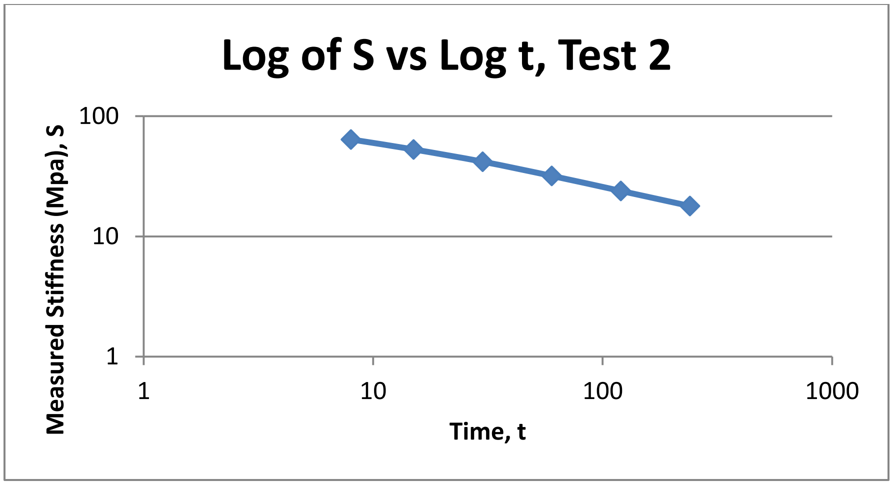


Figure D 4 Test Results of Bending Beam Rheometer

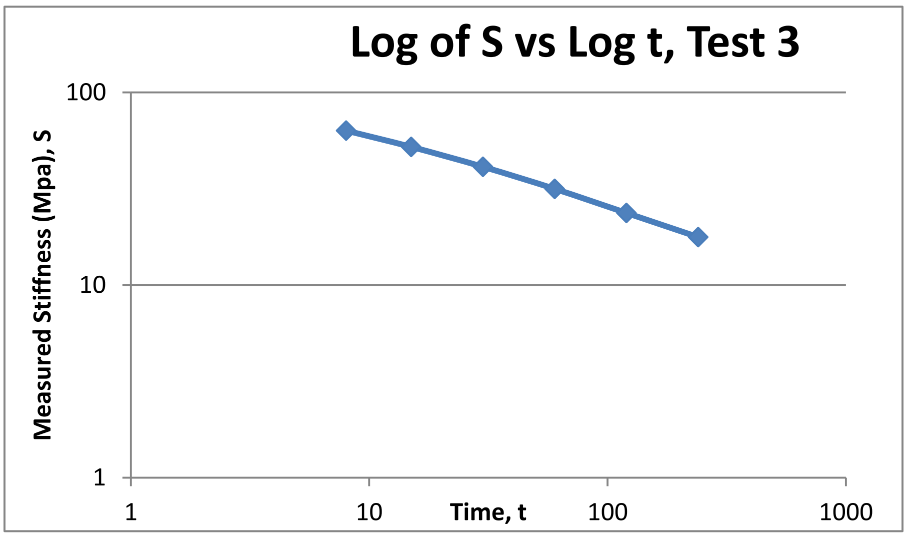


Figure D 5 Test Results of Bending Beam Rheometer

Table D 7(I) BBR Test Results

Material of Concern	Value	Results	Specification	HMA Distress
PAV Residue	Creep Stiffness at 60 sec (MPa)	43.9153	≤300 MPa (43.5 psi)	Low Temperature Cracking
		31.858		
		31.5818		
PAV Residue	m- Values	0.410758		Low Temperature Cracking
		0.391503	≥ 0.300	
		0.389543		

APPENDIX D 8 ASPHALT BINDER CRACKING DEVICE (ABCD) TEST

The Asphalt Binder Cracking Device (ABCD) test to evaluate the thermal cracking temperature was performed on the 9 samples sent from the EZasphalt Technology, LLC (University of Ohio). This effort was an Inter Laboratory Study (ILS) conducted by FHWA through EZasphalt Technology, LLC. Nine samples of asphalt were categorized into three groups on the basis of the stiffness e.g., low, medium and high. All the samples were unaged, but low and medium stiffness asphalt sample were SBS polymer modified.

The cracking potential of the asphalt was measured by ABCD to simulate the climatic conditions. When the asphalt is reduced to low temperature inside the silicon mould and invar metal ring, then due to different rate of thermal contraction of two materials (asphalt and invar metal) thermal stresses develop at the narrowest plain of fracture of the asphalt ring which leads to the cracking at that section. And the cracking at that temperature is recorded. This temperature is then compared to the low temperature performance grade of the asphalt. The evaluation of cracking temperature is essential, in order to ensure the rutting, fatigue cracking and even frosting of asphalt when used in the flexible pavement in the colder climate.

Table D8 ABCD Test Samples

Binder	Type
LL 21 /70-22	S-B-S Modified
LL 24 /70-22	S-B-S Modified
LL 27 /70-22	S-B-S Modified
MM 06/76-22	S-B-S Modified
MM 13/76-22	S-B-S Modified
MM 24/76-22	S-B-S Modified

HA-10	--
HD-132	--
HD140	--

The cracking temperature of the asphalt binder depends on factors such as cooling rate, and it has been determined by FHWA team that the cracking temperature increases with the increase in the rate of cooling of asphalt binder. This is due to that, at higher cooling rate, the rate of thermal stress accumulation is faster than the rate of stress relaxation, leading to rapid stress development and early warm fractures (Kim 2008). Physical hardening of the asphalt occurs when it is kept for extended period of time at low temperature, and its modulus increases with time (Kim 2008). In order to investigate this, various test on the unaged and RTFO/PAV aged asphalt samples were performed by the research team (Kim 2008).

EXPERIMENTAL PROCEDURE

Sample in the form of annular ring of binder were prepared in the silicon moulds. Asphalt was heated at specified temperature of 150^{o}C for low and medium stiffness polymer modified asphalt and 160^{o}C for high stiffness asphalt. Then this liquid asphalt was poured in the space between metal rings and lubricated silicon mould using special rotating base in order to get rid of any air bubble formation and for easiness in pouring (Figures D6 to D9). The excess amount of asphalt was then trimmed of using heated spatula. In this way four samples were prepared using same asphalt for one test.

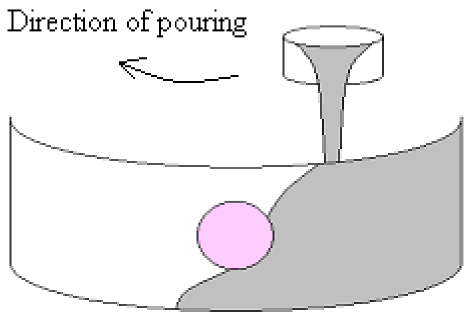


Figure D6

Figure D7

Figure D8

Figure D9

Cooling was done in the special ABCD chamber (Figure D10). The specimens within the mould were paced in the cooling chamber and stain gauge was then placed on each specimen (Figure D9). The cooling was done as per the specified cooling rate of 20^{o}C per hour. In the manner that in first 30 min. the temperature of the specimen get reduced to 0^{o}C from ambient temperature and then in next three hours it reduces to -60^{o}C. This all is programmed by the computer software. Throughout the process of cooling the computer program recorded the temperature and the strain (micro strain) in the asphalt ring, in the graphical form between the stain and temperature (Figures D11 and D12).

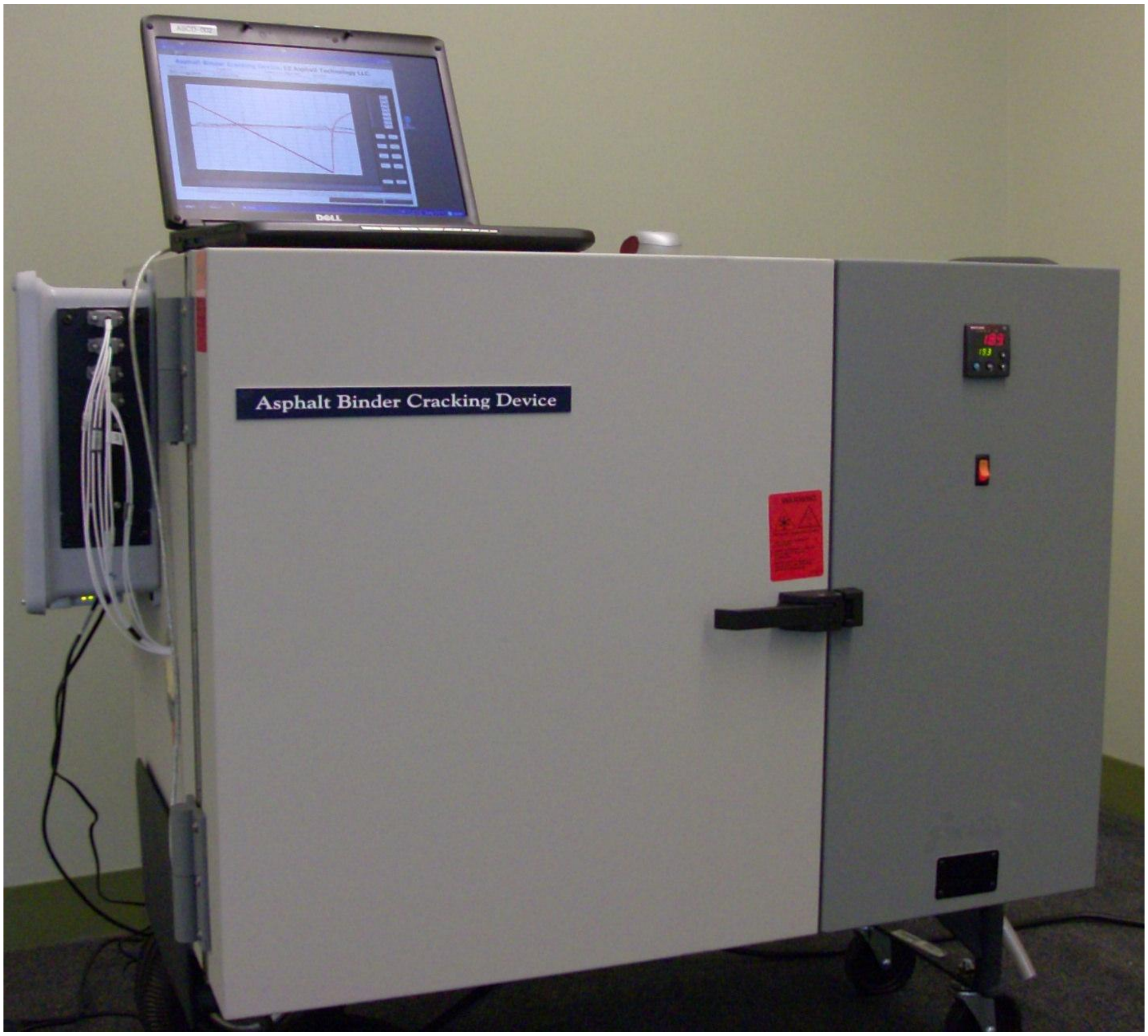


Figure D10 ABCD Cooling Chamber

When the asphalt rings cracks then the stain at that particular temperature jumps to a high value. This is recorded as the cracking temperature of the asphalt.

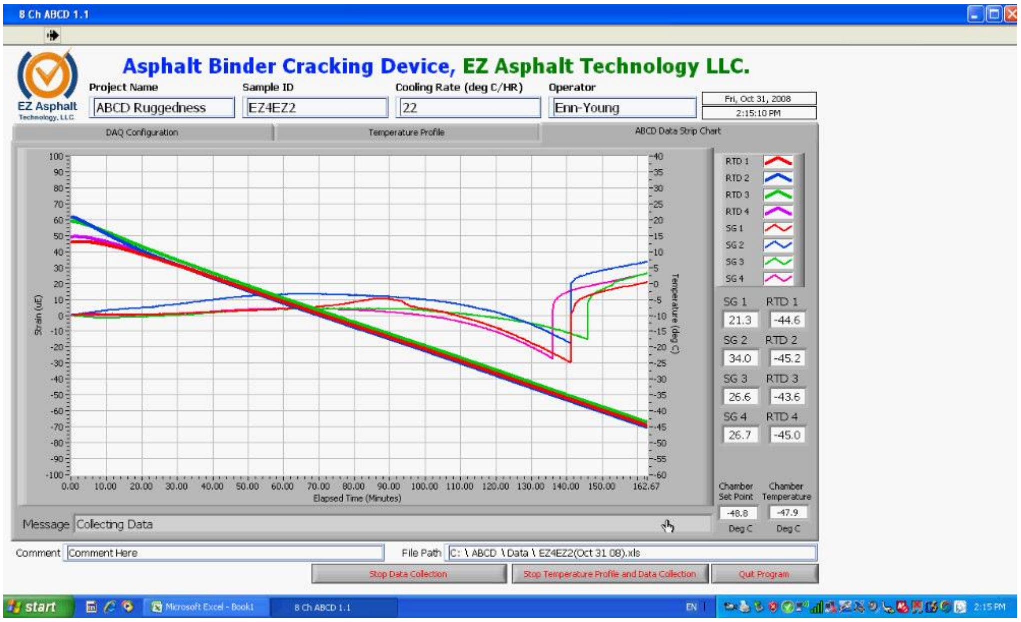


Figure D11 Graphical representations of stain and temperature with time of cooling

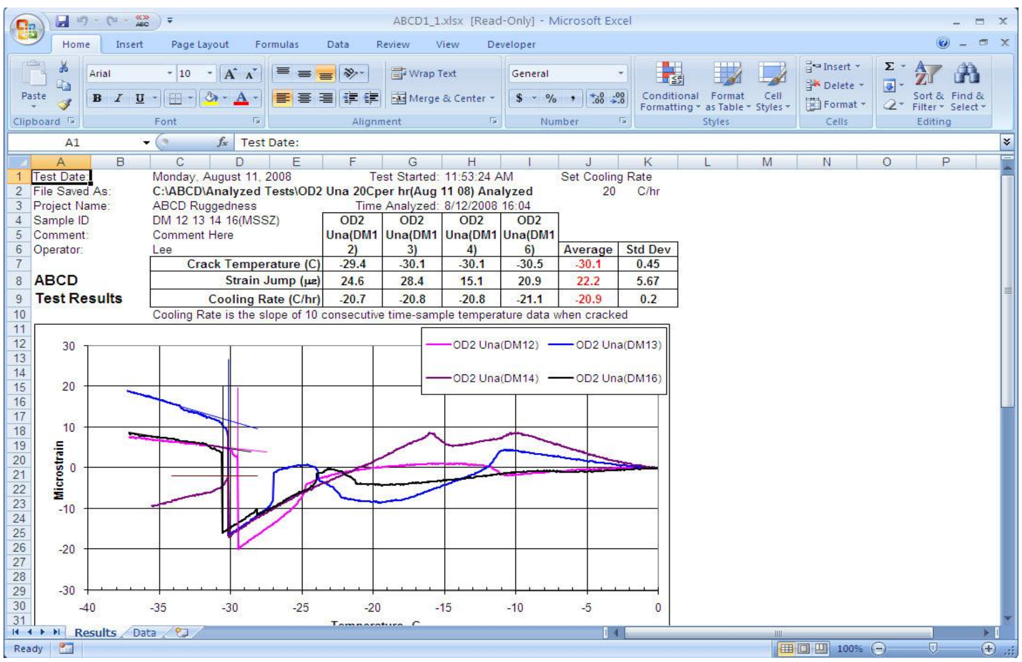


Figure D12 Cracking Temperature and Strain Jumps

After performing the ruggedness tests on the 9 given asphalt samples the following result was obtained as shown in Table D9, Figure D10 and subsequent result report forms.

Table D9 Cracking temperature of different asphalt sample

Binder	Sample 1	Sample 2	Sample 3	Sample 4	Average
LL 21 /70-22	**-29.5**	**-38.3**	**-37.8**	**-37.2**	**-35.7**
LL 24 /70-22	**-35.6**	**-35.4**	**-34.2**	**-**	**-35.1**
LL 27 /70-22	**-31.2**	**-30.0**	**-33.6**	**-**	**-31.6**
MM 06/76-22	**-39.3**	**-38.4**	**-39.5**	**-36.8**	**-38.5**
MM 13/76-22	**-34.9**	**-37.4**	**-38.6**	**-35.5**	**-36.6**
MM 24/76-22	**-35.4**	**-34.8**	**-31.4**	**-32.8**	**-33.6**
HA-10		**-11.8**		**-12.8**	**-12.3**
HD-132	**-14.8**	**-14.2**	**-9.7**	**-14.0**	**-13.2**
HD140	**-14.1**	**-11.8**	**-13.1**	**-9.4**	**-12.1**

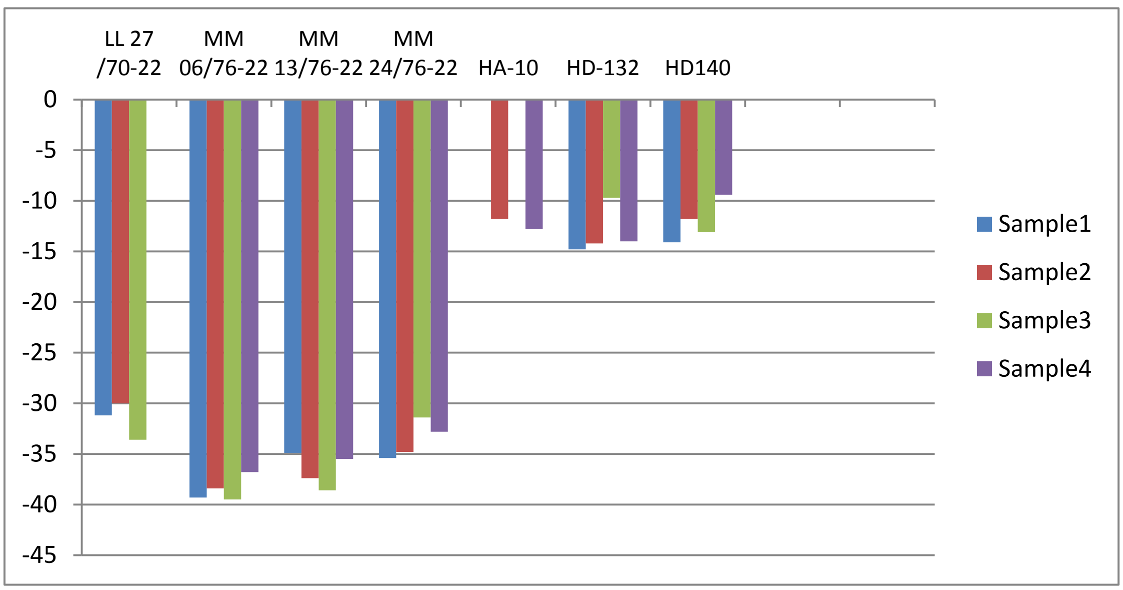


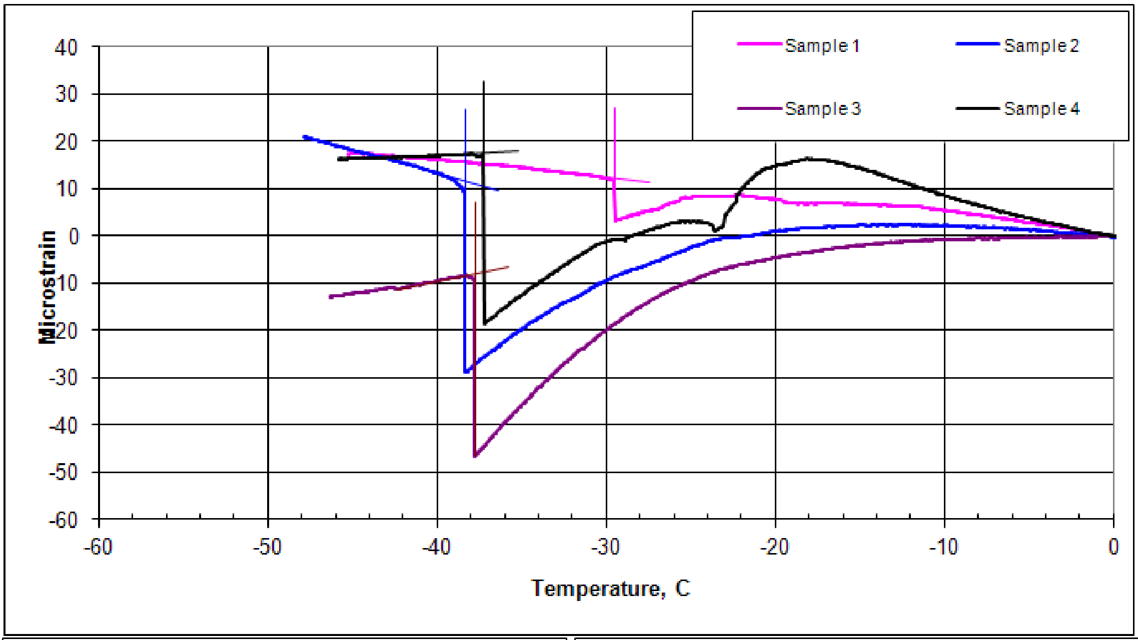
Figure D13 Relation between cracking temperature and the type of binder

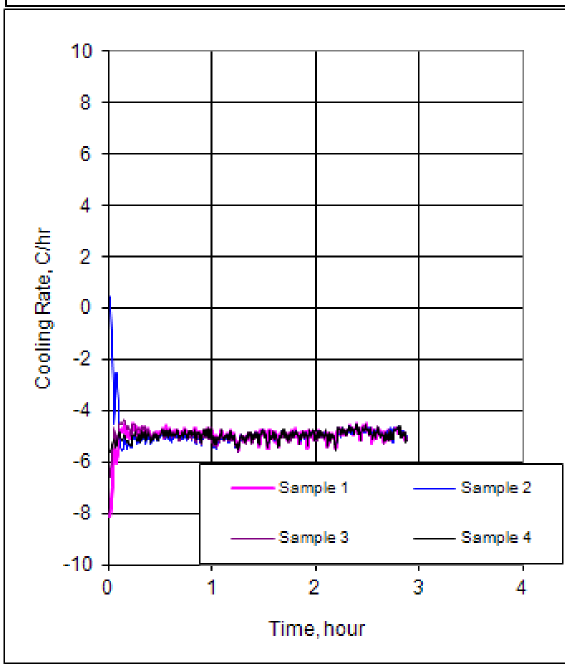
Test Date:	Monday, 10 May, 2010	Test Started: 8:58:02 PM	Set Cooling Rate
File Saved As:	**C:\ABCD\Analyzed Tests\LL 21 76-22(1) Analyzed**		20 C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 5/11/2010 2:20	
Sample ID			
Comment:	Comment Here		
Operator:			

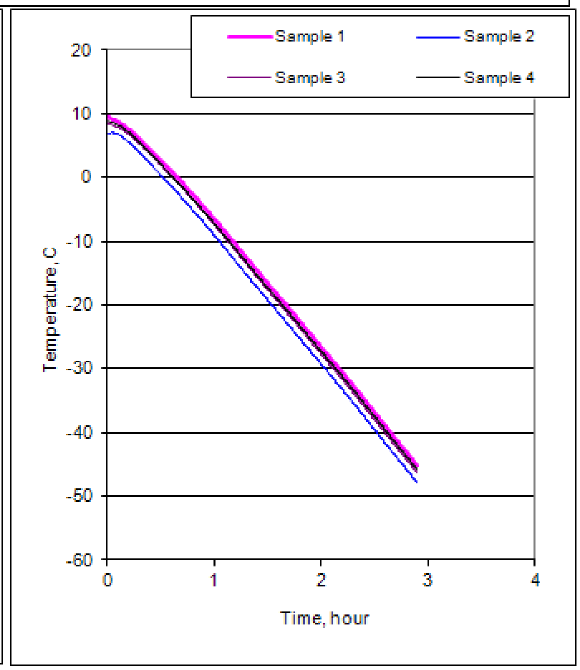
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-29.5	-38.3	-37.8	-37.2	-35.7	4.17
Strain Jump (με)	8.9	40.6	38.6	36.1	31.1	14.89
Cooling Rate (C/hr)	-5.0	#N/A	#N/A	#N/A	#N/A	#N/A

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





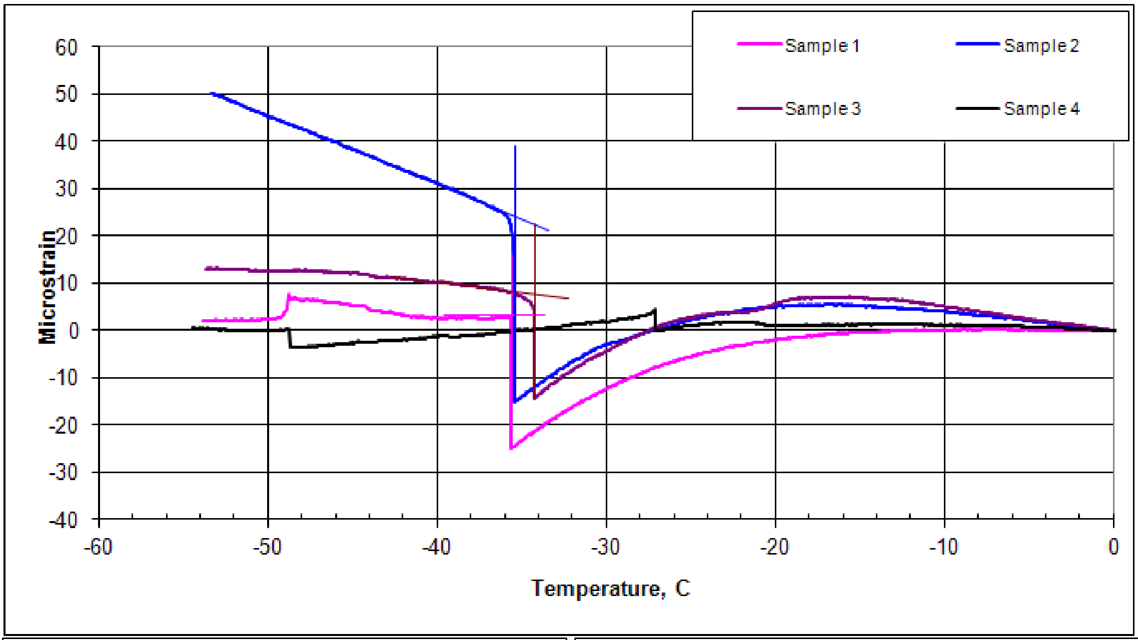


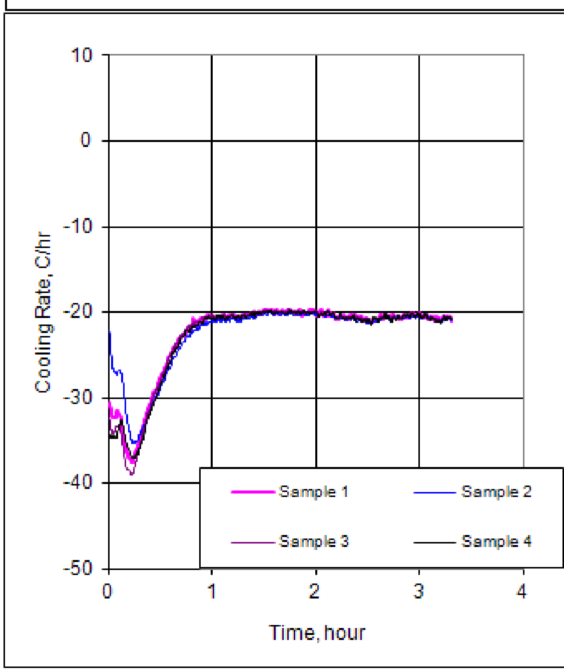
Test Date:	Thursday, April 22, 2010	Test Started: 11:55:05 AM			Set Cooling Rate		
File Saved As:	**C:\ABCD\Analyzed Tests\LL-24 70-22 Analyzed**				20 C/hr		
Project Name:	ABCD Ruggedness	Time Analyzed: 4/22/2010 16:27					
Sample ID							
Comment:	Comment Here						
Operator:							

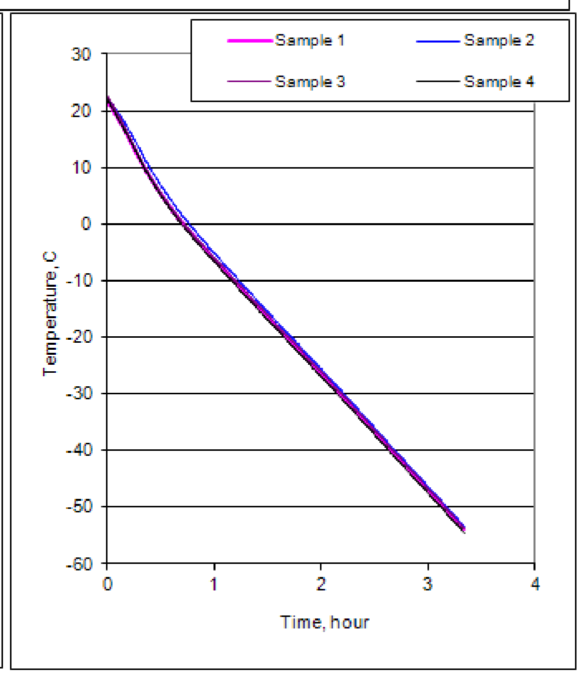
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-35.6	-35.4	-34.2		-35.1	0.72
Strain Jump (με)	28.2	39.3	22.0		29.8	8.79
Cooling Rate (C/hr)	-20.7	-20.9	-20.7	#N/A	#N/A	#N/A

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked







Test Date: Wednesday, 12 May, 2010 Test Started: 2:35:26 PM Set Cooling Rate

File Saved As: **C:\ABCD\Analyzed Tests\LL27(2) 76-22 Analyzed** 20 C/hr

Project Name: ABCD Ruggedness Time Analyzed: 5/12/2010 18:50

Sample ID

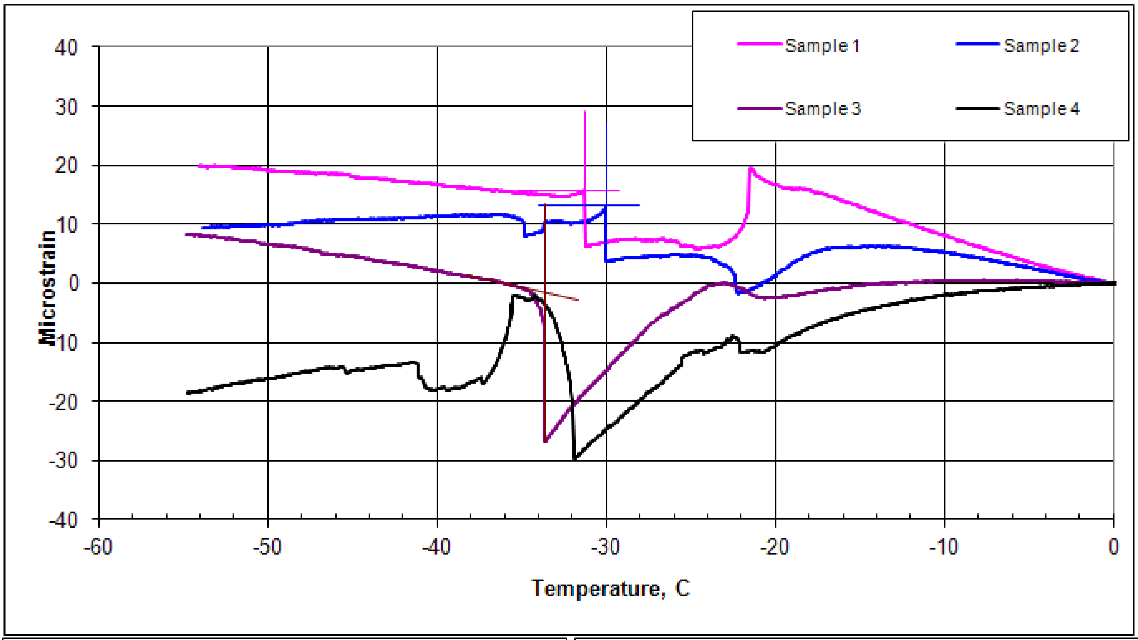
Comment: Comment Here

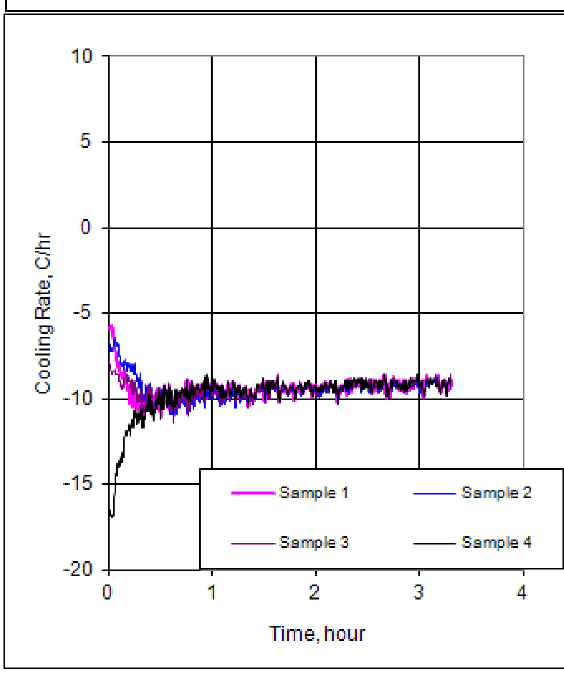
Operator:

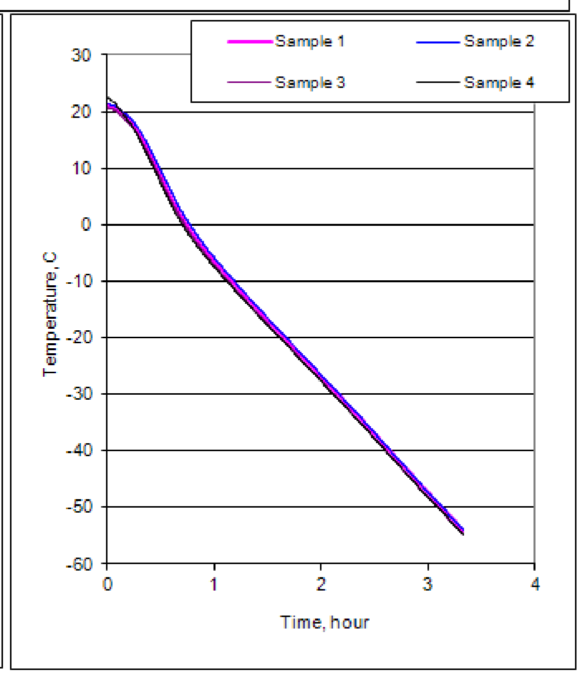
ABCD
Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-31.2	-30.0	-33.6		-31.6	1.84
Strain Jump (με)	9.6	9.4	25.3		14.8	9.11
Cooling Rate (C/hr)	#N/A	#N/A	#N/A	-9.6	#N/A	#N/A

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





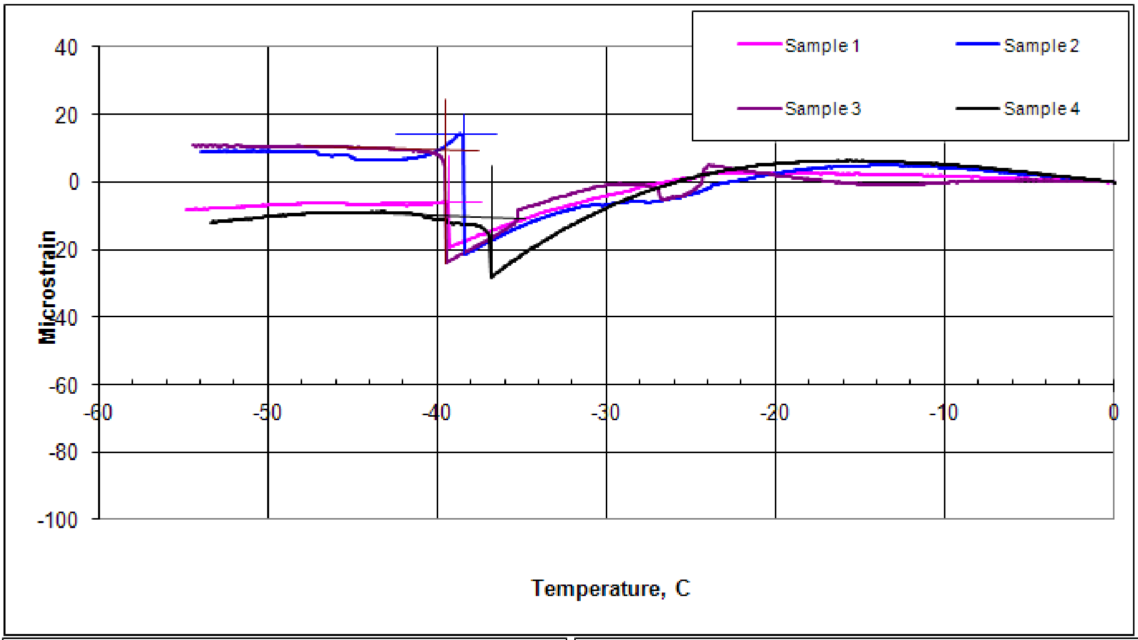


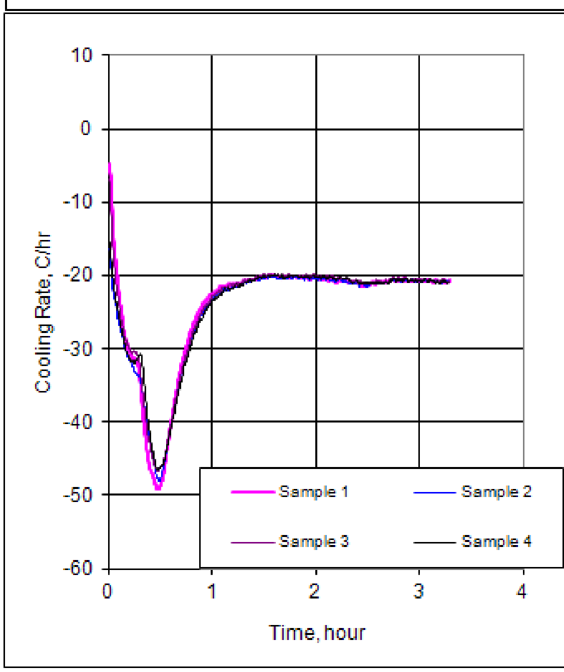
Test Date:	Monday, April 19, 2010	Test Started: 9:14:20 PM	Set Cooling Rate
File Saved As:	**C:\ABCD\Analyzed Tests\mm-06 76-22 Analyzed**		20 C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 4/20/2010 10:29	
Sample ID	MM-06 76-22		
Comment:	Comment Here		
Operator:	Ajay Singh		

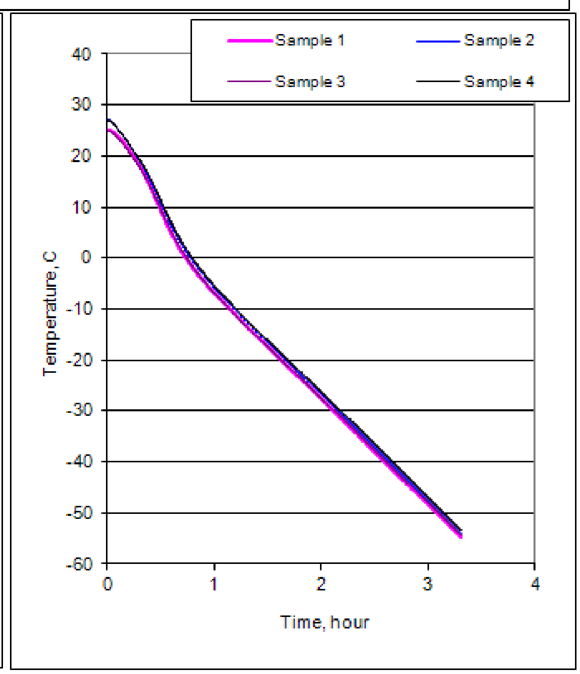
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-39.3	-38.4	-39.5	-36.8	-38.5	1.23
Strain Jump (με)	13.5	36.0	33.4	17.7	25.2	11.20
Cooling Rate (C/hr)	-20.8	-20.8	-20.7	-20.8	-20.8	0.1

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





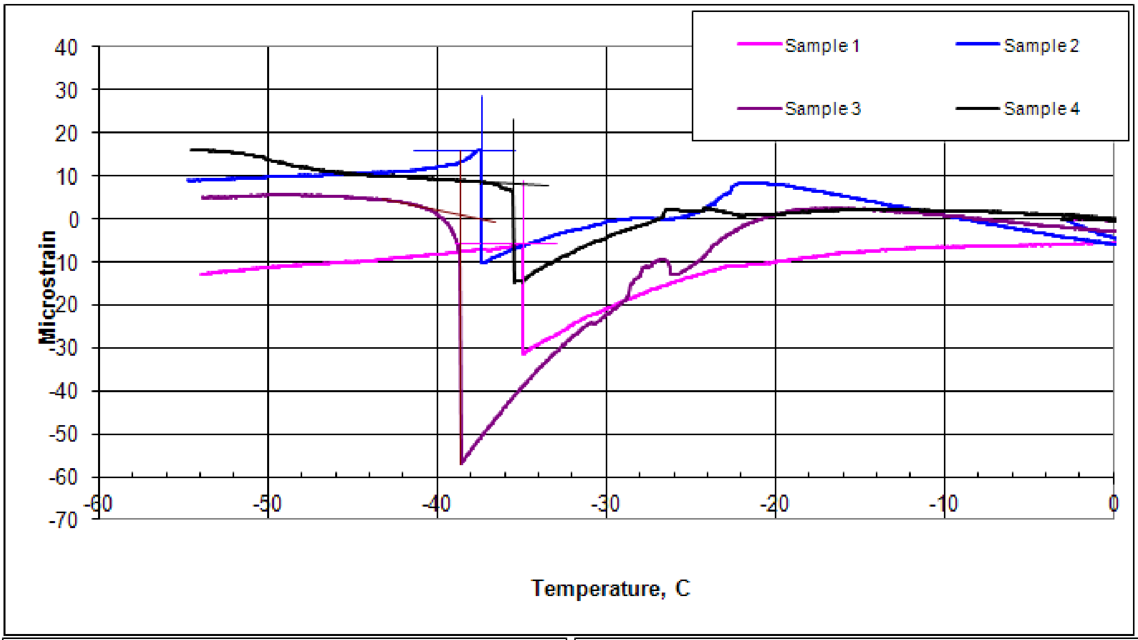


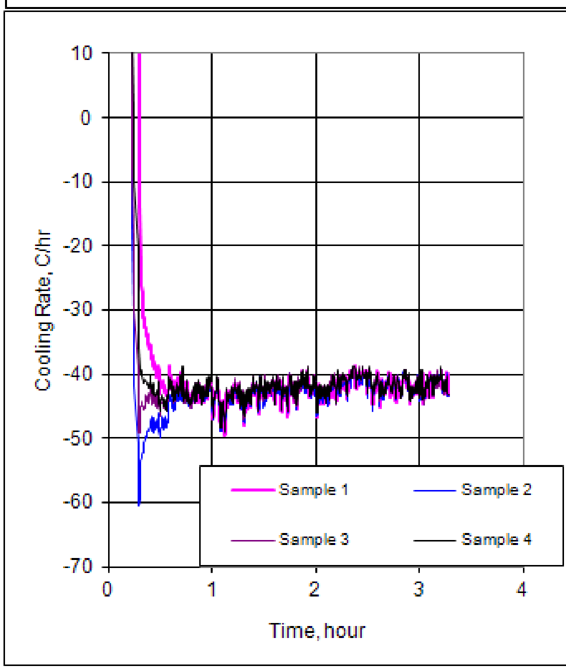
Test Date:	Tuesday, 11 May, 2010		Test Started: 3:56:31 PM			Set Cooling Rate	
File Saved As:	C:\ABCD\Analyzed Tests\mm 13 76-22 Analyzed					20	C/hr
Project Name:	ABCD Ruggedness		Time Analyzed: 5/11/2010 20:12				
Sample ID	MM13 76-22						
Comment:	Comment Here						
Operator:	Ajay Singh						

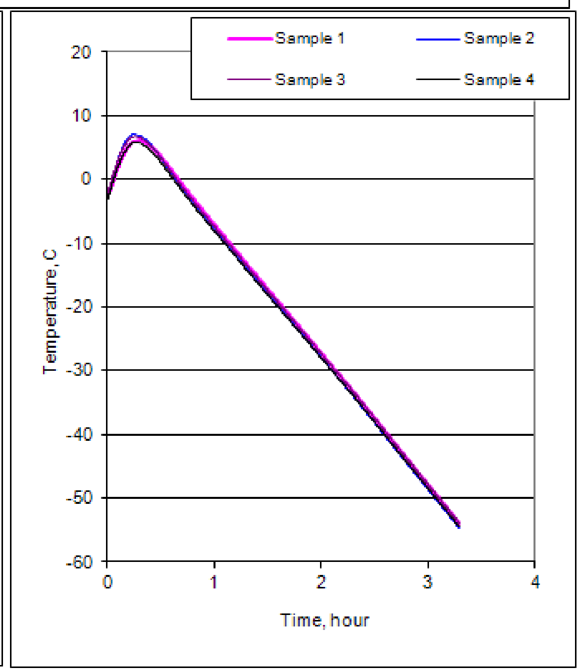
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-34.9	-37.4	-38.6	-35.5	-36.6	1.70
Strain Jump (με)	25.4	26.4	57.9	23.0	33.2	16.54
Cooling Rate (C/hr)	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





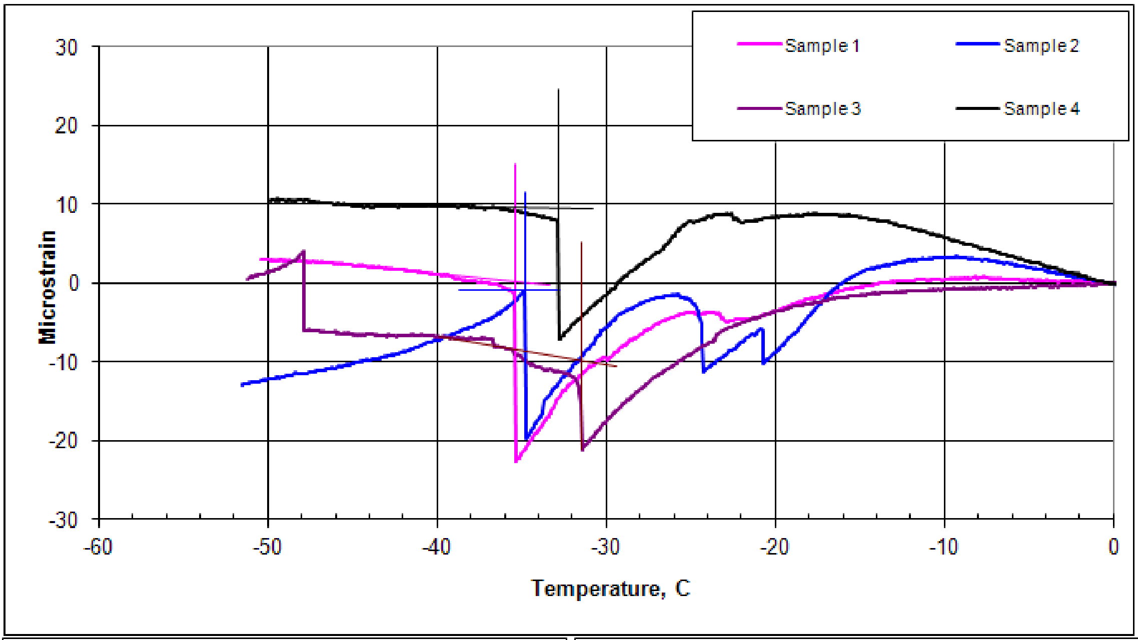


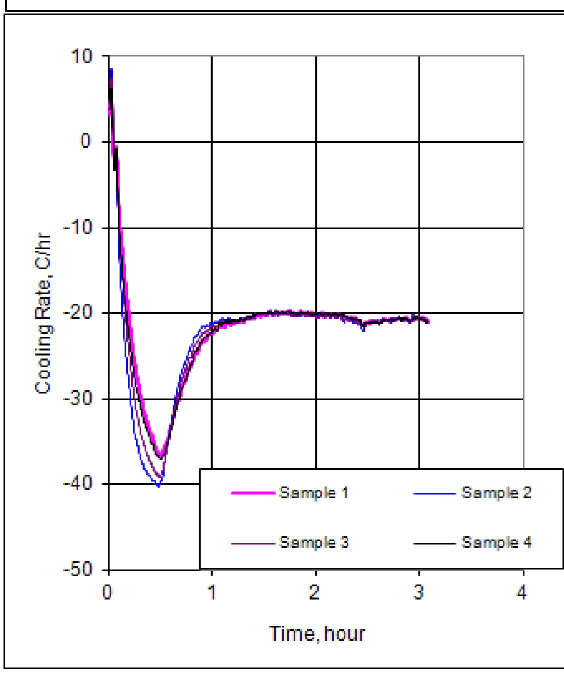
Test Date:	Monday, 10 May, 2010	Test Started: 3:43:41 PM				Set Cooling Rate	
File Saved As:	**C:\ABCD\Analyzed Tests\MM24 76-22 Analyzed**					20	C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 5/10/2010 19:39					
Sample ID	MM24 76-22						
Comment:	Comment Here						
Operator:	Ajay Singh						

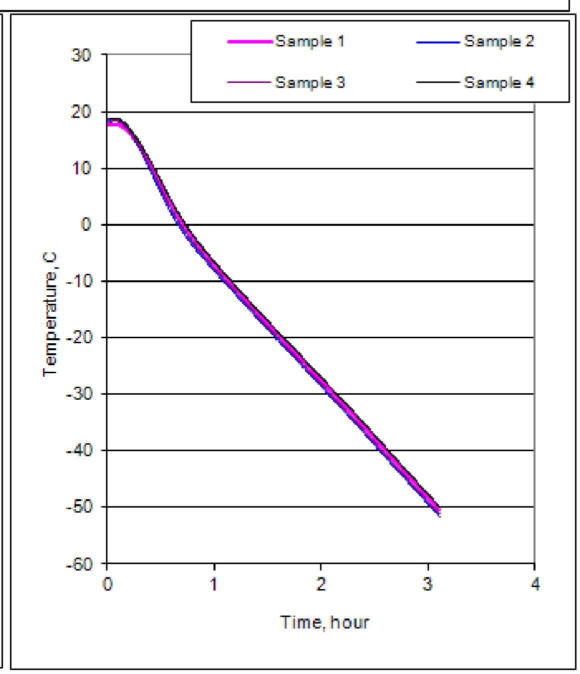
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-35.4	-34.8	-31.4	-32.8	-33.6	1.80
Strain Jump (με)	23.0	18.7	11.3	16.7	17.4	4.83
Cooling Rate (C/hr)	-20.5	-20.7	-20.5	-20.4	-20.5	0.1

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





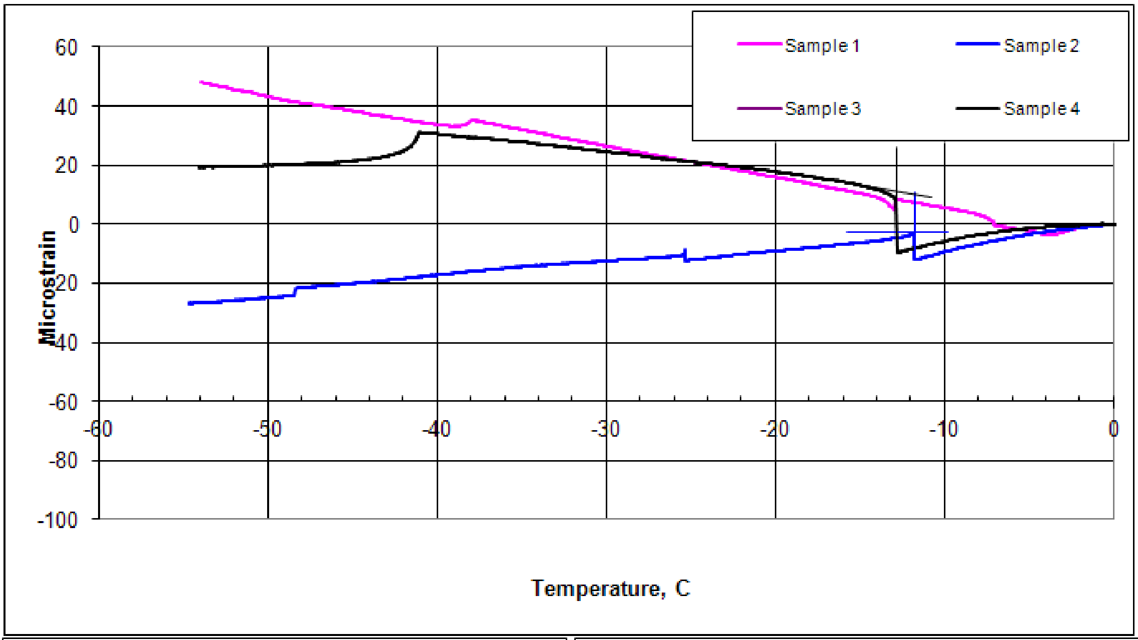


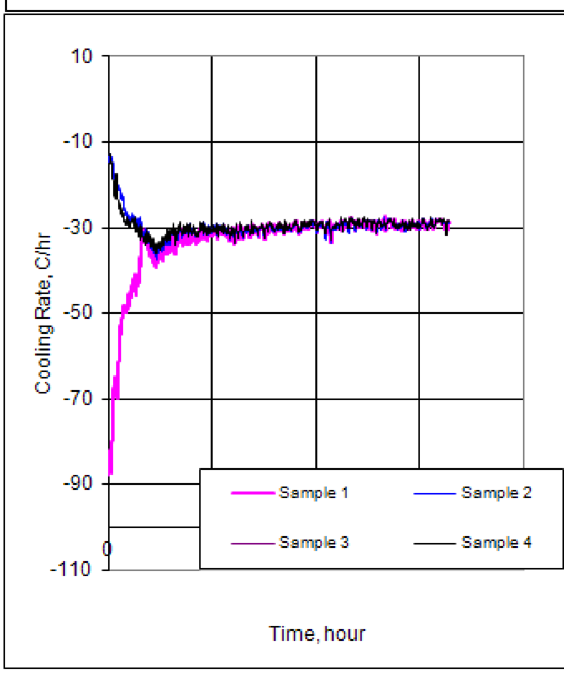
Test Date:	Wednesday, 12 May, 2010	Test Started: 7:28:49 PM	Set Cooling Rate
File Saved As:	**C:\ABCD\Analyzed Tests\HA 10 Analyzed**		20 C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 5/13/2010 0:28	
Sample ID	HA 10		
Comment:	Comment Here		
Operator:	Ajay Singh		

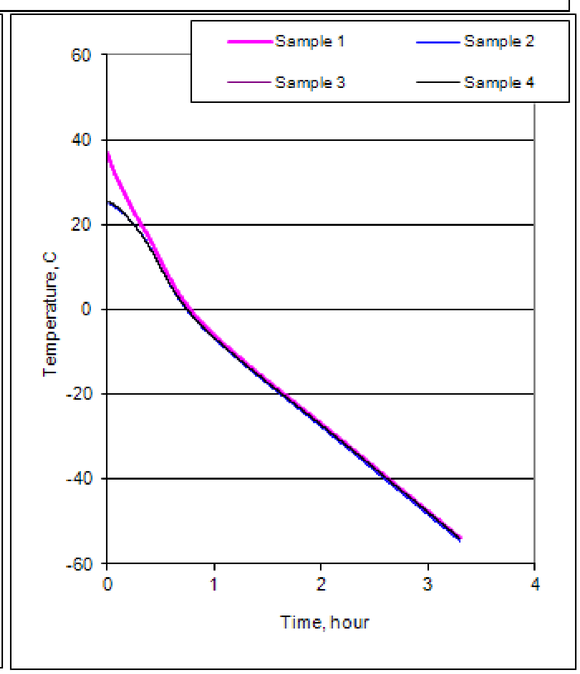
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)		-11.8		-12.8	-12.3	0.74
Strain Jump (με)		9.3		20.7	15.0	8.10
Cooling Rate (C/hr)	#N/A	-32.8	#N/A	-29.0	#N/A	#N/A

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





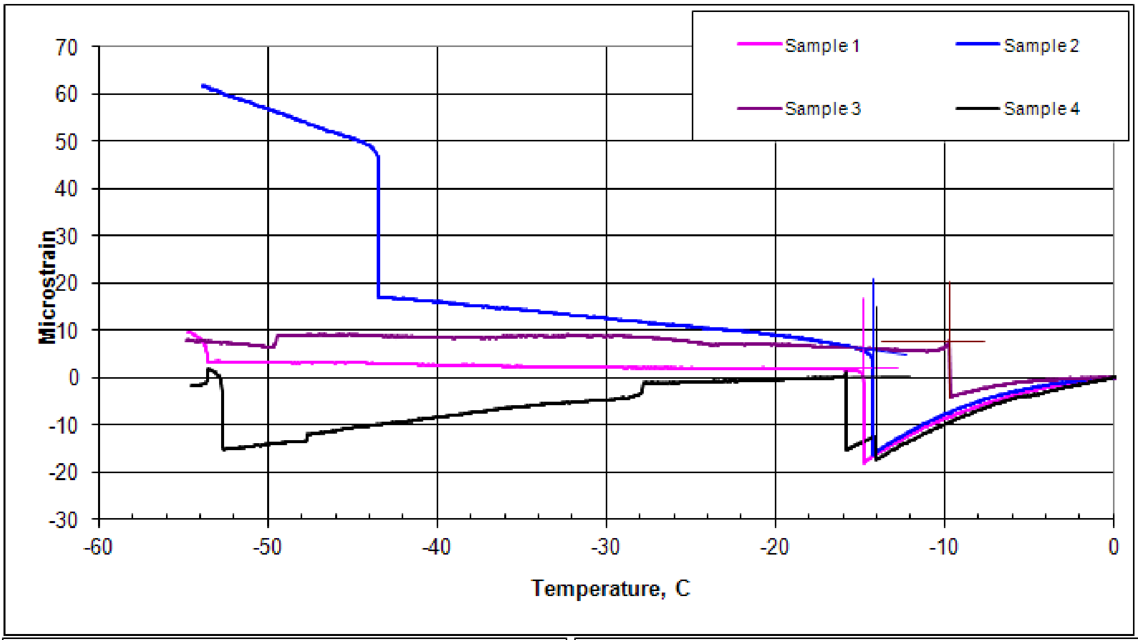


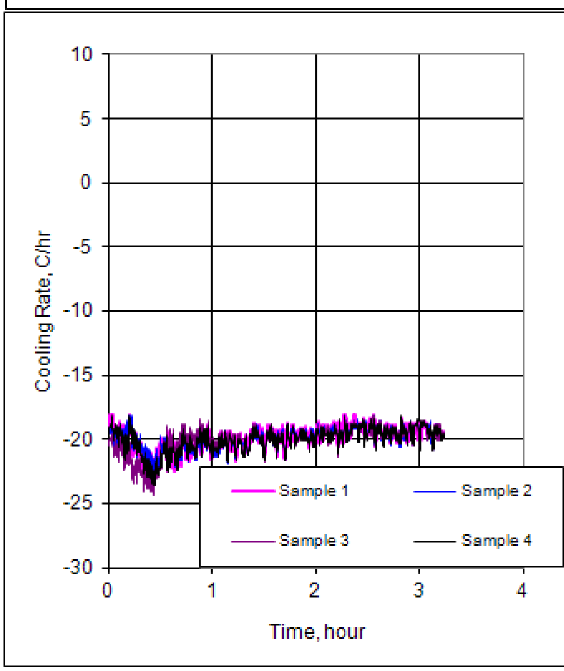
Test Date:	Thursday, 13 May, 2010	Test Started: 11:57:39 AM	Set Cooling Rate
File Saved As:	**C:\ABCD\Analyzed Tests\HD132 Analyzed**		20 C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 5/13/2010 18:09	
Sample ID			
Comment:	Comment Here		
Operator:			

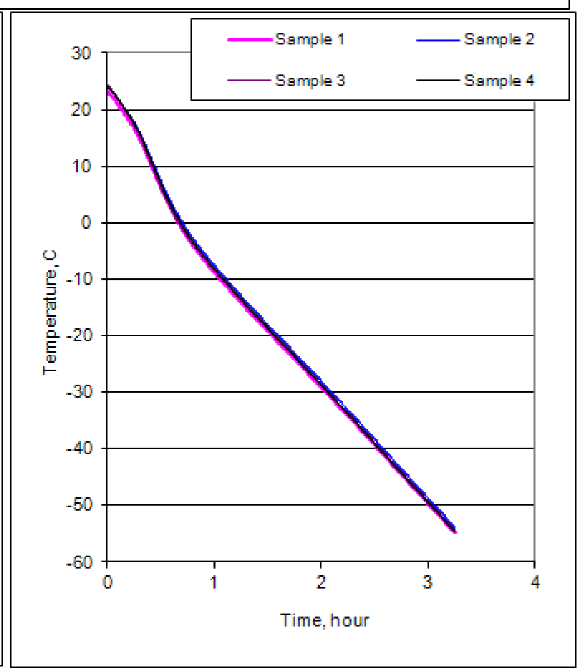
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-14.8	-14.2	-9.7	-14.0	-13.2	2.36
Strain Jump (με)	19.9	22.2	11.9	17.5	17.9	4.42
Cooling Rate (C/hr)	-19.6	-20.2	-20.7	-20.1	-20.1	0.5

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked





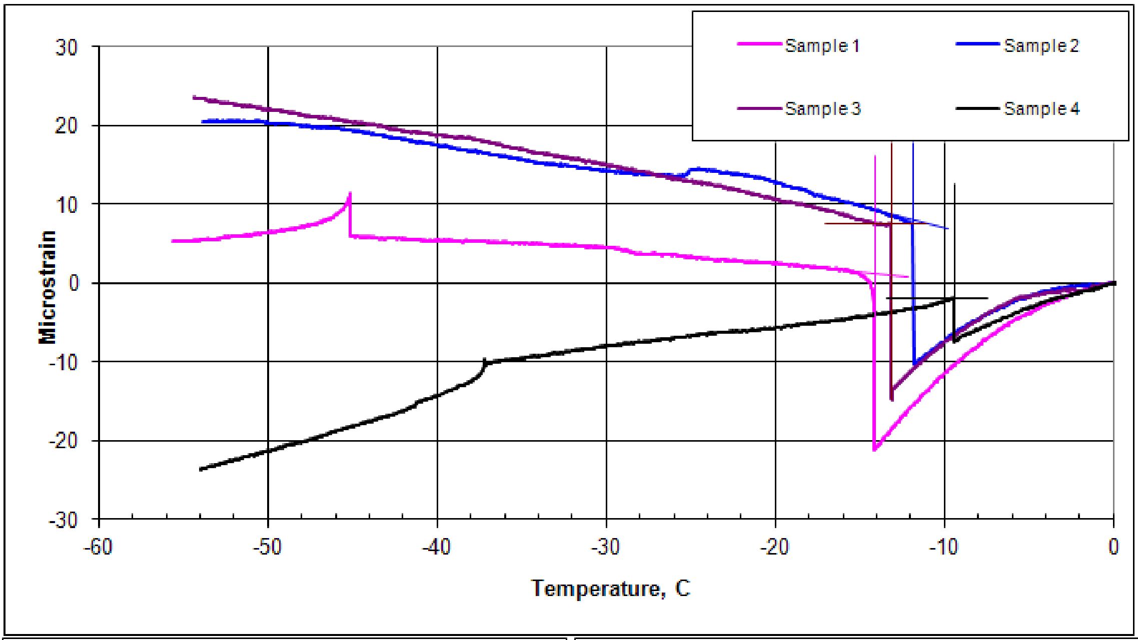


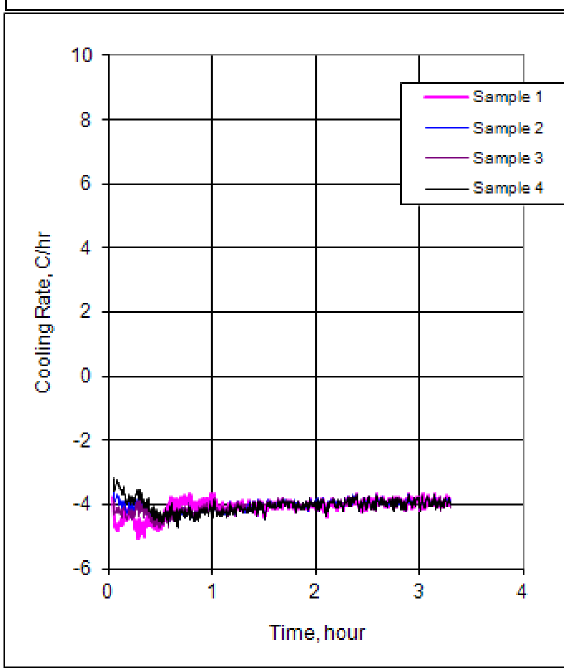
Test Date:	Thursday, 13 May, 2010	Test Started: 8:14:55 PM	Set Cooling Rate
File Saved As:	**C:\ABCD\Analyzed Tests\HD140(1) Analyzed**		20 C/hr
Project Name:	ABCD Ruggedness	Time Analyzed: 5/14/2010 1:01	
Sample ID	HD140		
Comment:	Comment Here		
Operator:	Ajay Singh		

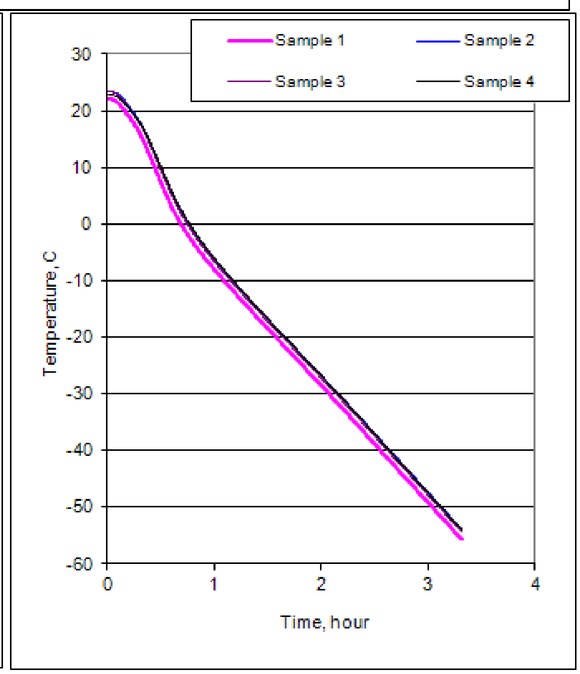
ABCD Test Results

	Sample 1	Sample 2	Sample 3	Sample 4	Average	Std Dev
Crack Temperature (C)	-14.1	-11.8	-13.1	-9.4	-12.1	2.02
Strain Jump (με)	22.3	18.3	22.4	5.6	17.2	7.96
Cooling Rate (C/hr)	-4.1	-4.0	-4.0	-4.3	-4.1	0.1

Cooling Rate is the slope of 10 consecutive time-sample temperature data when cracked







APPENDIX D9 TEST TO EVALUATE RUTTING REISTANCE

The purpose of AASHTO Designation**:** T P 63-03 is to measure detraction and to determine rutting susceptibility of asphalt mixtures using the asphalt pavement analyzer (APA). In this experiment percent air voids and bulk specific gravity have to be determined, and the rutting depth of the specimens would be measured.

The apparatus used for this experiment was the asphalt pavement analyzer (APA), a thermostatically controlled device designed to test the rutting susceptibility of asphalt mixtures by applying repetitive linear loads to compacted test specimens through three pressurized hoses via wheels. Before conducting this experiment the specimens have to be prepared, typically six cylindrical specimens.

Once specimens have been made they are placed for testing chamber and conditioned as desired under the pressurized rubber tube rack. The test consists of 8,000 times of back and forth passing of the rack. A raw data chart is generated during the course of the testing indicating the number of completed cycles, rut depths and cabin temperature for each of the three sets of specimens.Set the hose pressure gap. Stabilize the testing chamber temperature. Secure the preheated, molded specimens in the APA. Apply 25 cycles to seat the specimens before initial measurements, open chamber doors, and place rut depth measurement template over the specimen. Zero the digital measuring gauge. Secure sample by holding it in the tray. Select the pre set counter to 8,000 cycles. Start test, and when the APA reaches 8,000 cycles it will stop, and the load wheels will automatically retract.Test results are shown in Table D3 and Figure D9. The average rut depth at all location combined left, center and right was of about 5.3 mm.

TABLE D10 APA Test Results

Left Sample ID	001			Bulk S Gravity	2.407				% Air Void	5.3
	Temperature		Depth Gauge Reading							
STROKE COUNT	**F**	**C**	**1**	**2**	**3**	**4**	**5**	**Man Average Depth**	**APA Average**	**Percent Change**
0	**32**	0						0		
25	**149**	65	0.621915	-0.053691	0	3.0379887	4.4652615		2.0178687	
4000	**149**	65	7.207952	6.2012553	0	9.1631823	8.5904827		7.7907182	74.10%
7976	**149**	65	10.99313	9.8566847	0	12.138532	10.4786		10.866738	28.31%
8000	**32**	0						0		

Avg. 6.8917748

Center Sample ID				Bulk S Gravity					% Air Void	
	Temperature		Depth Gauge Reading							
STROKE COUNT	**F**	**C**	**1**	**2**	**3**	**4**	**5**	**Man Average Depth**	**APA Average**	**Percent Change**
0	32							0		
25	149	65	0.536349	-0.349133	0	-4.9030471	-5.0346045		-2.437609	
4000	149	65	4.179481	3.8860054	0	0.6426077	0.5009298		2.3022559	205.88%
7976	149	65	6.137663	5.995986	0	2.7171679	2.383214		4.3085077	46.56%
8000	32							0		

Avg. **3.3053818**

Right Sample ID			Bulk S Gravity					% Air Void		
	Temperature		Depth Gauge Reading							
STROKE COUNT	**F**	**C**	**1**	**2**	**3**	**4**	**5**	**Man Average Depth**	**APA Average**	**Percent Change**
0	32							0		
25	149	65	0.816583	-0.775295	0	5.0784111	5.3903646		2.6275159	
4000	149	65	0	0	0	0	0		0	0.00%
1842	149	65	4.35358	4.2343035	0	13.381866	12.510233		8.6199956	100.00%
8000	32							0		

Avg. **5.6237558 (Failed)**

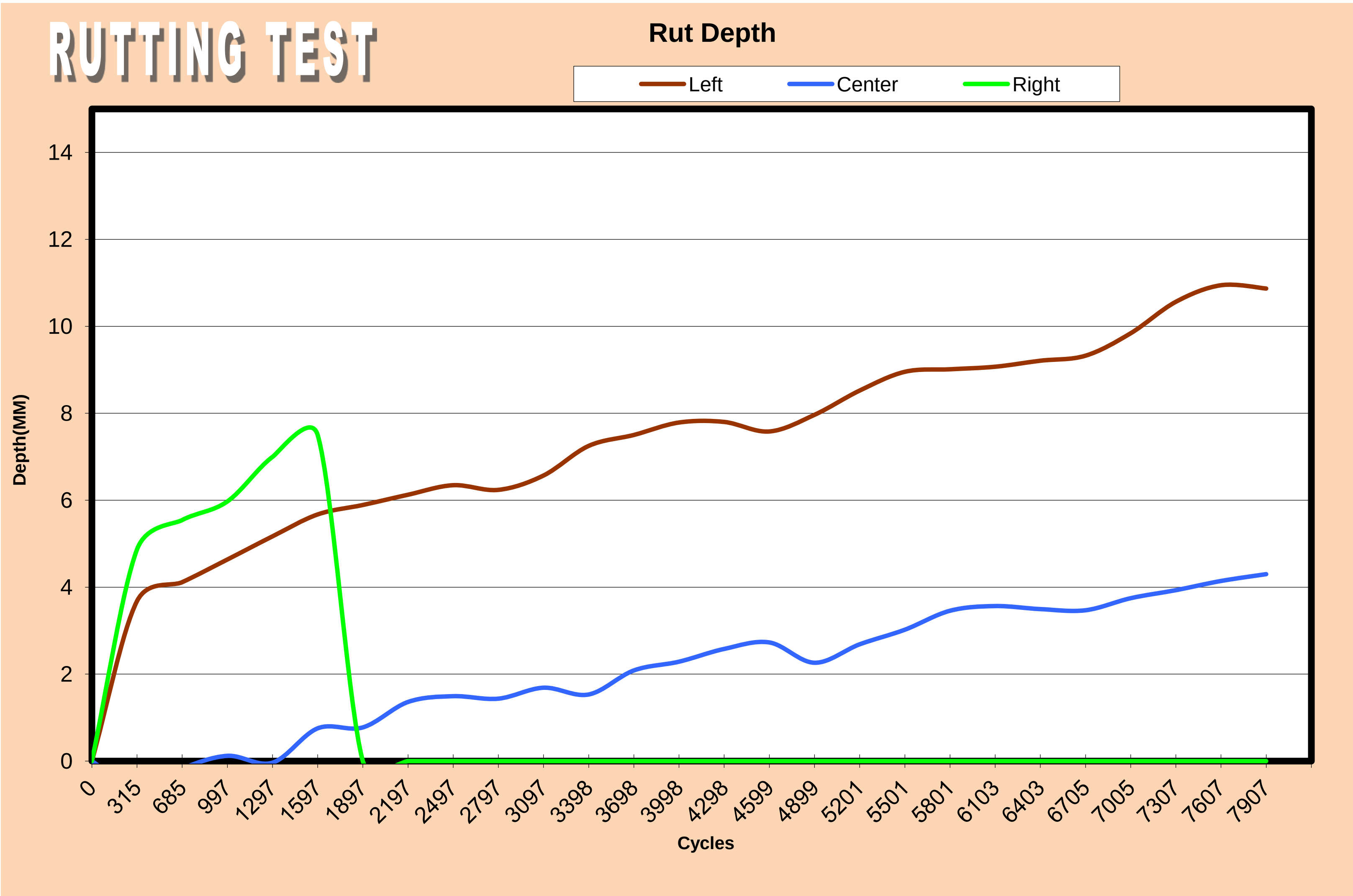


Figure D14 APA Test Results

APPENDIX D10 INPUT PARAMETERS FOR MEPDG SOFTWARE TO DESIGN MODEL PAVEMENT STRUCTURE FOR UPPER COLLEGE ROAD

Mechanistic- Empirical Pavement Design Guide (MEPDG) requires several input parameters, such as project information, traffic, climate, and structural inputs as can be seen in Figure D 10.1.

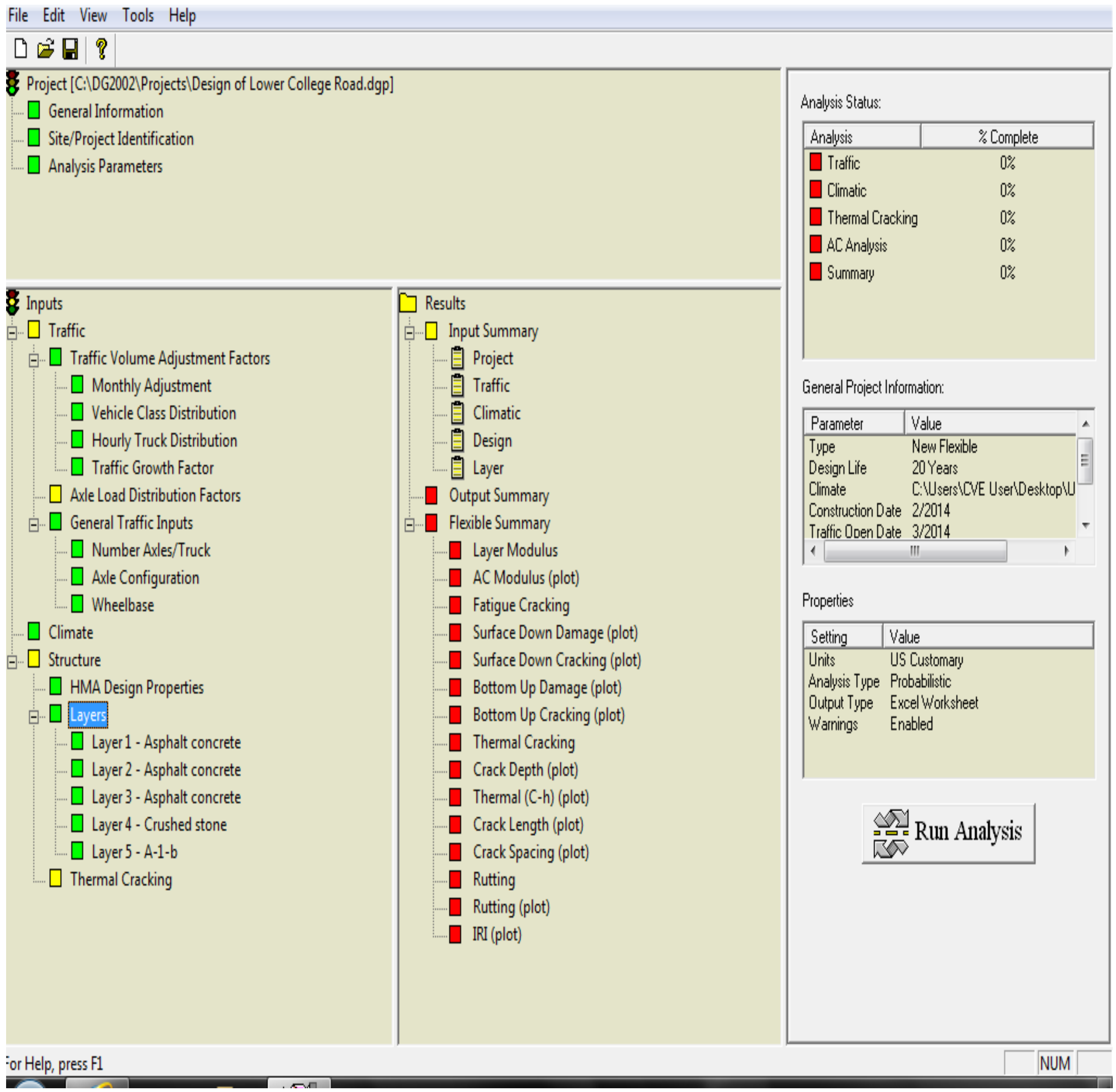


Figure D10.1 Input Parameters for MEPDG

Project information includes general information, site/project identification, and analysis parameters including design criteria.

Traffic inputs require traffic volume adjustment factors, axle load distribution factors and general traffic inputs. Traffic volume adjustment factors used for the present study and monthly adjustment factorswere taken from MEPDG as default values, vehicle class distribution (default values), and the growth factor of 1.25% was used from the reference of Rt 2 design data. Default values were used for load distribution factors, general traffic were default values for number axle/truck, axle configuration and wheel base.

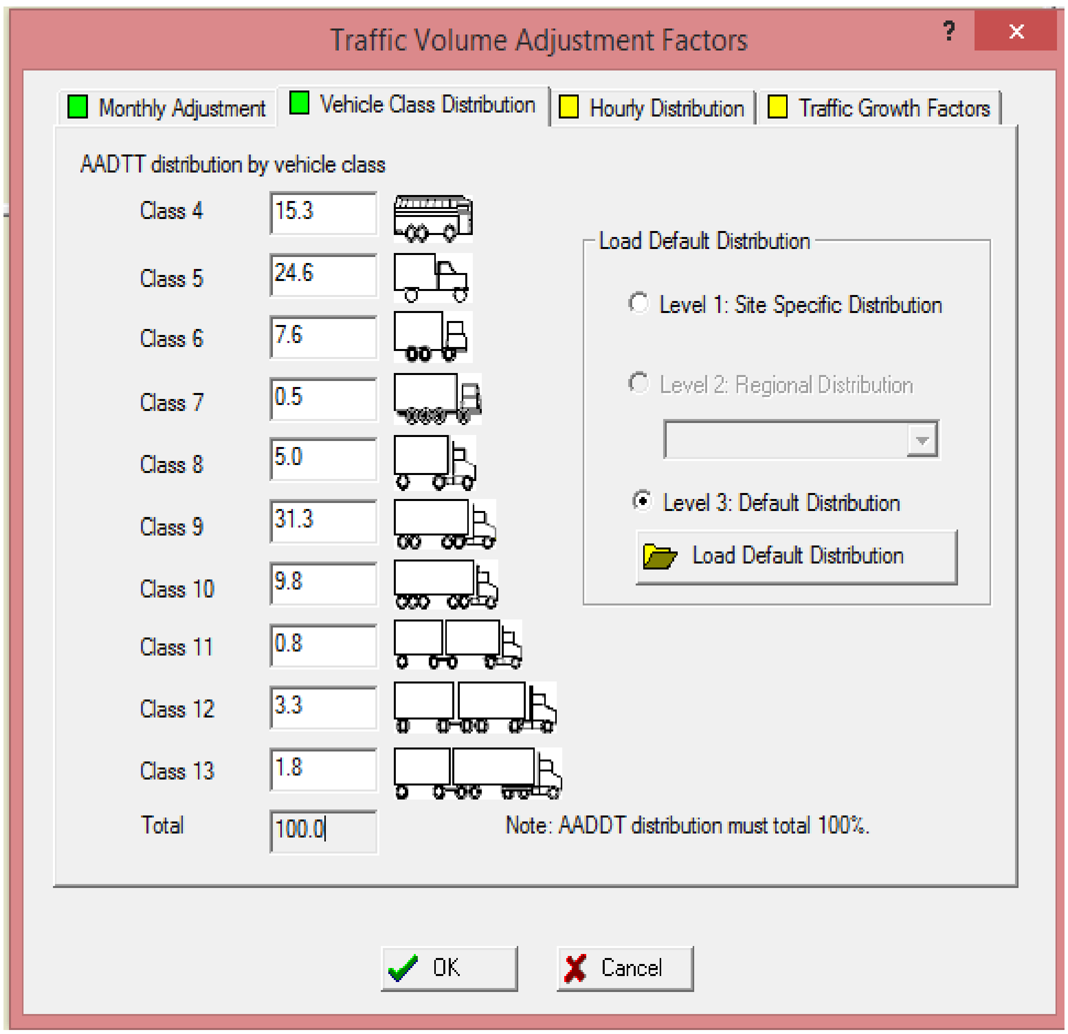


Figure D10.2 Vehicle Class Distribution for MEPDG

Environment/Climate inputs were generated for a specific weather station i.e., Providence, RI. The average depth of water table was taken as 10 feet (W.R.T Figure D 10.5 and Table D 10.2)

For structural inputs, 5 layers system for Upper College Road was used as shown in Figure D10.3.

Default values for HMA design properties were used.Asphalt Binder used was PG 64-28 with superpave grading system. Gradation used for layer 1 was class I-1, and gradation for binder, base and sub base can be found in Table D10.1.

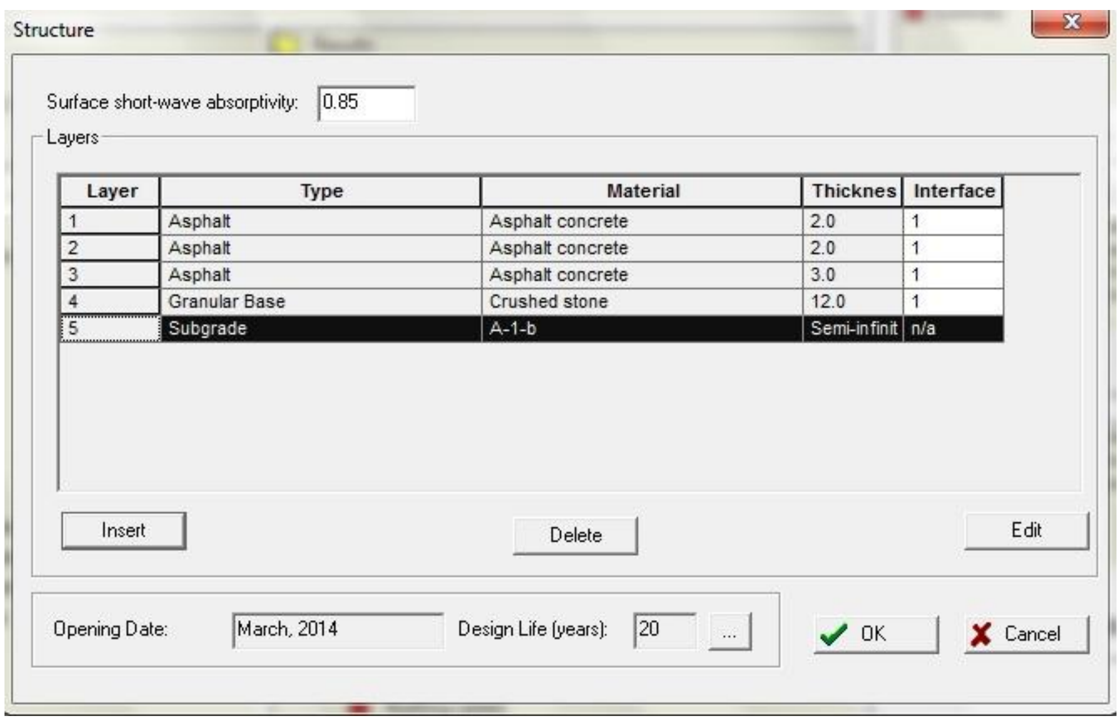


Figure D10.3Pavement Structure for Upper College Road used for MEPDG

Table D10.1 : Gradation for Base coarse, Binder coarse and Surface coarse

				SURFACE COURSES		FRICTION COURSES	
SIEVE SIZE	BASE COURSE	BINDER COURSE	BRIDGE BINDER	CLASS I-1	CLASS I-2 or SIDEWALK	DENSE	RAMP
GRADATION: PERCENT PASSING BY WEIGHT							
1¼"	100						
1"		100				100	
3/4"	70-100	70-100		100		90-100	100
½"			100	80-95	100	70-90	95-100
3/8"	46-74	46-74	70-100	70-90	95-100	45-75	70-100
#4			25-45	50-70	55-75	20-40	25-45
#8	22-52	22-52	20-35	35-50	40-55	8-18	20-35
#30	10-34	10-34		18-29	20-30		8-15
#50	6-26	6-26	8-17	13-23	10-20	4-12	5-12
#200	3-8	3-8	2-6	3-8	3-8	2-6	2-6
Asphalt % By Weight	4.0 - 6.5	4.0 - 6.5	5.0 - 7.0	5.5 - 7.0	6.0 - 7.5	4.5 - 5.5	5.0 - 7.0
Marshall Stability lbs.(min.)	1600	1600	750	1000	1000	750	750
% Voids	3 - 8	3 - 8	3 - 8	3 - 5	3 - 5	8 min.	5 min.
VFA	60 - 75	60 - 75	-	65 - 85	65 - 85	-	-
Flow (0.01in)	8 - 16	8 - 16	8 - 16	8 - 16	8 - 16	-	8 - 16
Mixing Temp.°F	300	300	260	300	300	260	260
Compaction #blows at ea. end.	75	75	50	50	50	50	50

The resilient modulus determined for URI, UCR in 2003 was 7,982 psi. But the minimum value used in MEPDG was 16000 psi. Similarly, the elastic modulus of granular subbase was determined to be 13,620 psi for Rt. 2 (having the similar as of UCR) but in MEPDG minimum value of 20000 psi was used (Lee et al., 2003), table D 10.2.

Input parameter for thermal cracking requires creep compliance at -4, 14 and 32^{o} F and average tensile strength at 14^{o} F**.** Those inputs can be seen in Figure D10.5

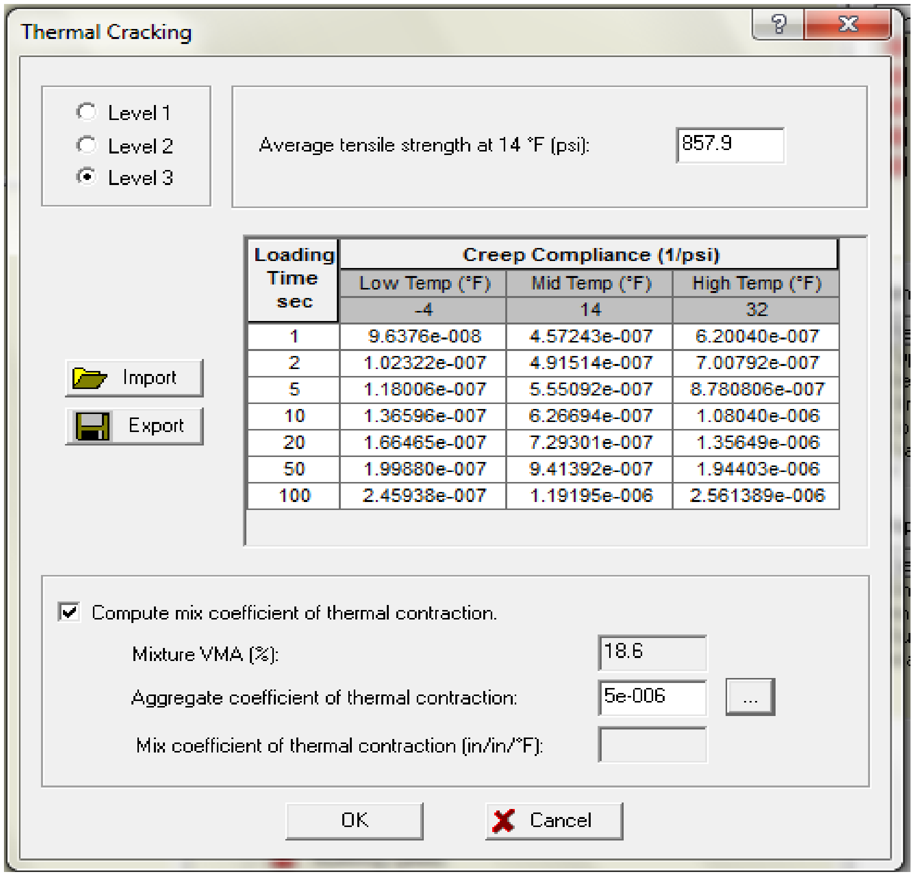


Figure D10.4Thermal Cracking Model of MEPDG

Table D10.2 Properties of Subgrade Soil and Subbase Material (Lee et al. 2003)

Effective Resilient Modulus of Each site						**Layer Coefficent for Subbase Material**		
Site	Soil Type	Bulk Stress psi	Dry Density psi	Average Relative Damage	Efefctive M_{R}, psi	Specimen ID	E_{SB} (psi)	Layer Coefficient a_3
Rt. 2S	A-1-b	10.02	131.1	0.07	9,304	SB-1	<u>13,620</u>	0.1
Rt. 146N	A-1-b	10.79	133.1	0.19	8,782	SB-2	13,185	0.1
UCR	A-1-b	7.52	132	0.1	<u>7,982</u>	SB-3	NM	xx
RWW	A-1-b	8.32	121.5	0.09	8,733	SB-4	NM	xx
Rt. 107	A-1-b	8.08	134.7	0.12	7,506	SB-5	12,069	0.09
Jamestown	A-1-b	8.78	127.5	0.08	8,795	SB-6	NM	
Charles Street	A-1-b	7.83	122.4	0.14	7,711	SB-7	18,539	0.13
Rt. 146S	A-1-b	10.79	133.1	0.19	8,782	SB-8	12,143	0.09
			Average		**8,402**	**Average**	**13,911**	**0.1**
			S.D.		**669**			

Note: 1. Bull Stress is at ADSS

2. Effective M_R using the prediction equation

Note: 1. a3 =0.227 log (E_{SB})-0.839

2. NM stands for no material

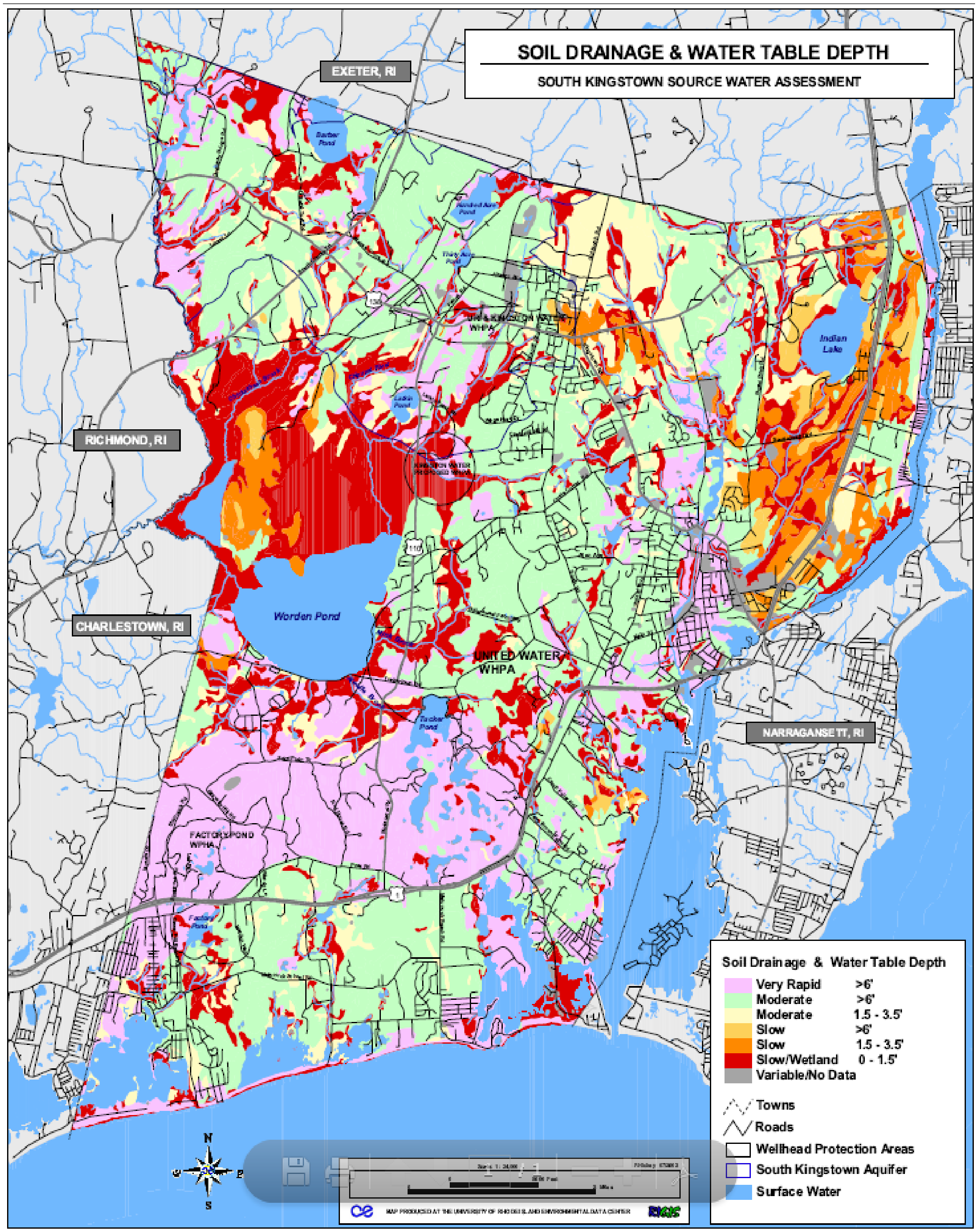


Figure D 10.5 Water Depth estimation for Upper College Road

Table D 10.3 Water Depth estimation for Upper College Road

TABLE 3: SUMMARY OF GROUND-WATER LEVELS December 2007 PROVISIONAL
(NOTE: Wells with * also available in real-time at top of Ground-Water Data page; OWc, monthly measured value used in high ground-water level estimation report, USGS Open-File Report 80-1205.)

WELL	L T I O T P H O O	START YEAR OF RECORD	NET CHANGE IN MONTH (FEET)	NET CHANGE IN ONE YEAR (FEET)	DEPARTURE FROM MONTHLY MEDIAN (FEET)	WATER LEVEL BELOW LAND-SURFACE DATUM (OWc) (FEET)	DAY
			RHODE ISLAND				
BURRILLVILLE 187	TS	1968	+ 0.09	- 2.50	- 2.45	17.50	21
BURRILLVILLE 395	UT	1992	-----	-----	-----	-----	
BURRILLVILLE 396	VT	1992	-----	-----	-----	-----	
BURRILLVILLE 397	HT	1992	-----	-----	-----	-----	
BURRILLVILLE 398	HT	1992	-----	-----	-----	-----	
CHARLESTOWN 18	FS	1946	- 0.04	- 4.35	- 2.77	21.15	21
CHARLESTOWN 586	VT	1992	-----	-----	-----	-----	
CHARLESTOWN 587	ST	1992	-----	-----	-----	-----	
COVENTRY 342	VS	1991	+ 0.22	- 2.24	- 2.65	10.60 <	21
COVENTRY 411	SS	1961	- 0.03	- 2.31	- 1.37	22.89	21
COVENTRY 466	VT	1992	-----	-----	-----	-----	
CRANSTON CITY 439	ST	1992	-----	-----	-----	-----	
CUMBERLAND 265	SS	1946	+ 0.90	- 1.34	- 1.45	13.03	21
EXETER 6	VS	1948	+ 0.05	- 1.64	- 1.14	6.82	21
EXETER 158	ST	1991	-----	-----	-----	DRY	21
EXETER 238	FT	1991	+ 0.06	- 0.26	- 0.50	12.16	21
EXETER 278	HT	1991	-----	-----	-----	DRY	21
EXETER 475	VS	1981	- 0.01	- 3.19	- 1.94	16.78 <<	21
EXETER 554	SS	1988	+ 0.02	- 2.22	- 2.12	11.84 <	21
FOSTER 40	HT	1991	+ 1.46	- 2.00	- 3.03	6.65	21
FOSTER 290	HT	1992	-----	-----	-----	-----	
HOPKINTON 67	ST	1991	- 0.03	- 7.05	- 5.67	21.91 <	21
LINCOLN 84	VS	1946	+ 0.03	- 0.93	- 1.00	6.01	21
LITTLE COMPTON 142	ST	1992	-----	-----	-----	-----	
NEW SHOREHAM 258	UT	1991	- 0.23	- 2.70	- 2.10	14.08 <	22
NORTH KINGSTOWN 255	VS	1954	+ 0.11	- 2.84	- 2.05	10.37	21
NORTH SMITHFIELD 21	TS	1947	+ 0.17	- 2.94	- 2.78	10.30 <	21
PORTSMOUTH 551	HT	1992	-----	-----	-----	-----	
PROVIDENCE 48	TS	1944	+ 0.02	- 0.86	+ 1.72	4.61	17
RICHMOND 417	VS	1976	+ 0.06	- 1.42	- 1.18	7.82 <	21
RICHMOND 600*	TS	1977	- 0.07	- 2.55	- 1.70	35.89 <	21
RICHMOND 785	FS	1989	- 0.37	- 3.94	- 2.28	26.54 <	21
SOUTH KINGSTOWN 6	VS	1955	+ 0.29	- 2.73	- 1.52	13.98	21
SOUTH KINGSTOWN 1198	FS	1988	+ 0.23	- 3.51	- 2.85	11.01 <	21
WARWICK 59	ST	1991	+ 1.58	- 12.41	- 12.47	17.40 <	17
WESTERLY 522	FS	1969	+ 0.49	- 1.45	- 1.61	13.52	21
WEST GREENWICH 181	US	1969	+ 0.25	- 1.17	- 1.33	16.48	21
WEST GREENWICH 206	ST	1991	+ 0.39	- 1.30	- 1.43	5.42	21

\>> SET NEW HIGH OR EQUALED HIGHEST RECORDED WATER LEVEL FOR PERIOD OF RECORD
\> SET NEW HIGH OR EQUALED HIGHEST RECORDED WATER LEVEL FOR END OF SEPTEMBER
<< SET NEW LOW OR EQUALED LOWEST RECORDED WATER LEVEL FOR PERIOD OF RECORD
< SET NEW LOW OR EQUALED LOWEST RECORDED WATER LEVEL FOR END OF SEPTEMBER
------ - DATA NOT AVAILABLE
TOPOGRAPHIC (TOPO) SETTING: F=FLAT, G=FLOOD PLAIN, H=HILLTOP, S=HILLSIDE, T=TERRACE, U=UNDULATING, V=VALLEY, W=UPLAND DRAW, LITHOLOGY (LITHO): G=GRAVEL, R=ROCK, S=SAND, T=TILL

The NOAA National Weather Service (NWS) Drought Severity Index for the period ending December 29, 2007 shows normal conditions for the region (Table 4). The Crop Moisture Index for the same time period shows wet conditions (Table 5).

www.wrb.ri.gov

INPUT PARMETERS FOR MEPDG

Project: Upper College Road

General Information

Design Life	20 years	Description: *Design of Upper College Road with Thermal Cracking resistance performance prediction.*
Base/Subgrade construction:	August, 2015	
Pavement construction:	October, 2015	
Traffic open:	November, 2015	
Type of design	Flexible	

Analysis Parameters

Performance Criteria	Limit	Reliability
Initial IRI (in/mi)	63	
Terminal IRI (in/mi)	172	90
AC Surface Down Cracking (Long. Cracking) (ft/mile):	2000	90
AC Bottom Up Cracking (Alligator Cracking) (%):	25	90
AC Thermal Fracture (Transverse Cracking) (ft/mi):	1000	90
Chemically Stabilized Layer (Fatigue Fracture)	25	90
Permanent Deformation (AC Only) (in):	0.25	90
Permanent Deformation (Total Pavement) (in):	0.75	90
Reflective cracking (%):	100	

Location:	Upper College Road
Project ID:	P-007
Section ID:	
Date:	4/8/2014
Station/milepost format:	Feet: 00 + 00
Station/milepost begin:	
Station/milepost end:	
Traffic direction:	North bound

Default Input Level

Default input level — Level 3, Default and historical agency values.

Traffic

Initial two-way AADTT:	100
Number of lanes in design direction:	1
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	100
Operational speed (mph):	45

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Level 3, Default MAF)

Month	Vehicle Class									
	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	15.3%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	1.8%

Hourly truck traffic distribution
by period beginning:

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Traffic Growth Factor

Vehicle Class	Growth Rate	Growth Function
Class 4	1.3%	Compound
Class 5	1.3%	Compound
Class 6	1.3%	Compound
Class 7	1.3%	Compound
Class 8	1.3%	Compound
Class 9	1.3%	Compound
Class 10	1.3%	Compound
Class 11	1.3%	Compound
Class 12	1.3%	Compound
Class 13	1.3%	Compound

Traffic -- Axle Load Distribution Factors

Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking):	18
Traffic wander standard deviation (in):	10
Design lane width (ft):	12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Axle Configuration

Average axle width (edge-to-edge) outside dimensions,ft):	8.5
Dual tire spacing (in):	12

Axle Configuration

Tire Pressure (psi) :	120

Average Axle Spacing

Tandem axle(psi):	51.6
Tridem axle(psi):	49.2
Quad axle(psi):	49.2

Climate

icm file:	C:\Users\CVE User\Desktop\UCR_URI.icm
Latitude (degrees.minutes)	41.4 3
Longitude (degrees.minutes)	- 71.2 6
Elevation (ft)	53
Depth of water table (ft)	10

Structure--Design Features

HMA E* Predictive Model:	NCHRP 1-37A viscosity based model.
HMA Rutting Model coefficients:	NCHRP 1-37A coefficients
Endurance Limit (microstrain):	100

Structure--Layers

Layer 1 -- Asphalt concrete

Material type:	Asphalt concrete
Layer thickness (in):	2

General Properties

General

Reference temperature (F°):	70

Volumetric Properties as Built

Effective binder content (%):	11.6
Air voids (%):	7
Total unit weight (pcf):	150

Poisson's ratio:	0.35 (user entered)

Thermal Properties
Thermal conductivity asphalt (BTU/hr-ft-F°): 0.67
Heat capacity asphalt (BTU/lb-F°): 0.23

Asphalt Mix

Cumulative % Retained 3/4 inch sieve: 0
Cumulative % Retained 3/8 inch sieve: 23
Cumulative % Retained #4 sieve: 40
% Passing #200 sieve: 6

Asphalt Binder

Option: Superpave binder grading
A 10.3120 (correlated)
VTS: -3.4400 (correlated)

High temp. °C	Low temperature, °C						
	-10	-16	-22	-28	-34	-40	-46
46							
52							
58							
64							
70							
76							
82							

Thermal Cracking Properties

Average Tensile Strength at 14ºF: 857.9
Mixture VMA (%) 19.5
Aggreagate coeff. thermal contraction (in./in.) 0.000005
Mix coeff. thermal contraction (in./in./ºF): 0.000013

Load Time (sec)	Low Temp. -4ºF (1/psi)	Mid. Temp. 14ºF (1/psi)	High Temp. 32ºF (1/psi)
	1E-07	5E-07	6E-07
	1E-07	5E-07	7E-07
	1E-07	6E-07	9E-07
	1E-07	6E-07	1E-06
	2E-07	7E-07	1E-06
	2E-07	9E-07	2E-06
	2E-07	1E-06	3E-06

Layer 2 -- Asphalt concrete

Material type: Asphalt concrete
Layer thickness (in): 2

General Properties

General
Reference temperature (F°): 70

Volumetric Properties as Built
Effective binder content (%): 11.6
Air voids (%): 7
Total unit weight (pcf): 150

Poisson's ratio: 0.35 (user entered)

Thermal Properties
Thermal conductivity asphalt (BTU/hr-ft-F°): 0.67
Heat capacity asphalt (BTU/lb-F°): 0.23

Asphalt Mix

Cumulative % Retained 3/4 inch sieve: 0
Cumulative % Retained 3/8 inch sieve: 23
Cumulative % Retained #4 sieve: 40
% Passing #200 sieve: 6

Asphalt Binder

Option: Superpave binder grading
A 10.3120 (correlated)
VTS: -3.4400 (correlated)

High temp. °C	Low temperature, °C						
	-10	-16	-22	-28	-34	-40	-46
46							
52							
58							
64							
70							
76							
82							

Layer 3 -- Asphalt concrete

Material type: Asphalt concrete
Layer thickness (in): 3

General Properties

General
Reference temperature (F°): 70

Volumetric Properties as Built

Effective binder content (%):	11.6
Air voids (%):	7
Total unit weight (pcf):	150
Poisson's ratio:	0.35 (user entered)

Thermal Properties

Thermal conductivity asphalt (BTU/hr-ft-F°):	0.67
Heat capacity asphalt (BTU/lb-F°):	0.23

Asphalt Mix

Cumulative % Retained 3/4 inch sieve:	0
Cumulative % Retained 3/8 inch sieve:	23
Cumulative % Retained #4 sieve:	40
% Passing #200 sieve:	6

Asphalt Binder

Option:	Superpave binder grading
A	10.3120 (correlated)
VTS:	-3.4400 (correlated)

High temp. °C	Low temperature, °C						
	-10	-16	-22	-28	-34	-40	-46
46							
52							
58							
64							
70							
76							
82							

Layer 4 -- Crushed stone

Unbound Material:	Crushed stone
Thickness(in):	12

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	20000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	6
Compacted Layer	No
Passing #200 sieve (%):	8.7
Passing #40	20
Passing #4 sieve (%):	44.7
D10(mm)	0.1035
D20(mm)	0.425
D30(mm)	1.306
D60(mm)	10.82
D90(mm)	46.19

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	
#50	
#40	20
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8"	57.2
1/2"	63.1
3/4"	72.7
1"	78.8
1 1/2"	85.8
2"	91.6
2 1/2"	
3"	
3 1/2"	97.6
4"	97.6

Calculated/Derived Parameters

Maximum dry unit weight (pcf):	127.2 (derived)
Specific gravity of solids, Gs:	2.70 (derived)
Saturated hydraulic conductivity (ft/hr):	0.05054 (derived)
Optimum gravimetric water content (%):	7.4 (derived)
Calculated degree of saturation (%):	61.2 (calculated)

Soil water characteristic curve parameters:	Default values

Parameters	Value
a	7.2555
b	1.3328
c	0.8242
Hr.	117.4

Layer 5 -- A-1-b

Unbound Material:	A-1-b
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	16000

ICM Inputs

Gradation and Plasticity Index

Plasticity Index, PI:	1
Liquid Limit (LL)	11
Compacted Layer	No
Passing #200 sieve (%):	13.4
Passing #40	37.6
Passing #4 sieve (%):	74.2
D10(mm)	0.01398
D20(mm)	0.1895
D30(mm)	0.3103
D60(mm)	1.582
D90(mm)	17.77

Sieve	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	13.4
#100	
#80	20.8
#60	
#50	
#40	37.6
#30	
#20	
#16	
#10	64
#8	
#4	74.2

3/8"	82.3
1/2"	85.8
3/4"	90.8
1"	93.6
1 1/2"	96.7
2"	98.4
2 1/2"	
3"	
3 1/2"	99.4
4"	99.4

Calculated/Derived Parameters

Maximum dry unit weight (pcf): 123.7 (derived)

Specific gravity of solids, Gs: 2.70 (derived)

Saturated hydraulic conductivity (ft/hr): 0.002303 (derived)

Optimum gravimetric water content (%): 9.1 (derived)

Calculated degree of saturation (%): 68.1 (calculated)

Soil water characteristic curve parameters: Default values

Parameters	Value
a	5.8206
b	0.4621
c	3.8497
Hr.	126.8

Distress Model Calibration Settings - Flexible

AC Fatigue — Level 3: NCHRP 1-37A coefficients (nationally calibrated values)

k1: 0.007
6

k2: 3.949
2

k3: 1.281

AC Rutting — Level 3: NCHRP 1-37A coefficients (nationally calibrated values)

k1: -3.354

k2: 1.560
6

k3: 0.479
1

Standard Deviation Total Rutting (RUT): 0.24*POWER(RUT,0.8026)+0.001

Thermal Fracture — Level 3: NCHRP 1-37A coefficients (nationally calibrated values)

k1	1.5
Std. Dev. (THERMAL):	0.1468 * THERMAL + 65.027

CSM Fatigue	Level 3: NCHRP 1-37A coefficients (nationally calibrated values)
k1	1
k2	1

Subgrade Rutting	Level 3: NCHRP 1-37A coefficients (nationally calibrated values)
Granular:	
k1	2.03
Fine-grain:	
k1	1.35

AC Cracking

AC Top Down Cracking

C1 (top)	7
C2 (top)	3.5
C3 (top)	0
C4 (top)	1000
Standard Deviation (TOP)	200 + 2300/(1+exp(1.072-2.1654*log(TOP+0.0001)))

AC Bottom Up Cracking

C1 (bottom)	1
C2 (bottom)	1
C3 (bottom)	0
C4 (bottom)	6000
Standard Deviation (TOP)	1.13+13/(1+exp(7.57-15.5*log(BOTTOM+0.0001)))

CSM Cracking

C1 (CSM)	1
C2 (CSM)	1
C3 (CSM)	0
C4 (CSM)	1000
Standard Deviation (CSM)	CTB*1

IRI

IRI HMA Pavements New

C1(HMA)	40
C2(HMA)	0.4
C3(HMA)	0.008
C4(HMA)	0.015

IRI HMA/PCC Pavements

C1(HMA/PCC)	40.8
C2(HMA/PCC)	0.575
	0.001
C3(HMA/PCC)	4
	0.008
C4(HMA/PCC)	3

OUTPUT RESULT OF MEPDG

Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	172	90	102.6	99.19	Pass
AC Surface Down Cracking (Long. Cracking) (ft/mile):	2000	90	19.2	97.73	Pass
AC Bottom Up Cracking (Alligator Cracking) (%):	25	90	0	99.999	Pass
AC Thermal Fracture (Transverse Cracking) (ft/mi):	1000	90	1	99.999	Pass
Chemically Stabilized Layer (Fatigue Fracture)	25	90			N/A
Permanent Deformation (AC Only) (in):	0.25	90	0.05	99.999	Pass
Permanent Deformation (Total Pavement) (in):	0.75	90	0.28	99.999	Pass

APPENDIX D11COLD IN-PLACE RECYCLING MIXTURES AS SUSTAINABLE PAVEMENT MATERIALS

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Abstract

The high cost and environmental impact of traditional pavement maintenance and rehabilitation (M&R) has led to an increase in the use of Cold In-Place Recycling (CIR) as an effective alternative to other rehabilitation strategies. However, there was not a universally accepted or standard mix-design for CIR.

In order to offer a new and more sustainable alternative to reconstruction of asphalt pavements, cold in-place recycling (CIR) method has been recommended. The resistance characteristics against low-temperature cracking were investigated to examine performance of CIR mixtures.

Therefore, an attempt was made to develop a rational mix-design method for use with the CIR of asphalt pavement with the assistance from the Federal Highway Administration (FHWA). Thus, the Superpave mix design procedure for the hot mix asphalt (HMA) was modified for the nature of cold mixes through laboratory evaluation and field verification. Consequently, a new volumetric mix-design utilizing the Superpave gyratory compactor

(SGC) was developed for use with CIR materials. It was primarily developed for partial-depth CIR, using emulsion as the recycling additive. The new mix-design procedure was evaluated using materials from five geographically varied locations in North America: Connecticut, Kansas, Ontario, Arizona and New Mexico. It requires that specimens be prepared at densities similar to those found in the field. A test section had been established with CIR mixtures in Arizona using the new mix design procedure in October 2000, and is performing well with no visible cracking or distresses.

The performance of CIR mixtures prepared and constructed in accordance with the new mix-design has been further evaluated in the laboratory as well as in the field. Creep compliance and strength of the mixtures have been determined at 0^{o}C (32^{o}F), -10^{o}C (14^{o}F), and -20^{o}C (-4^{o}F) using the Superpave Indirect Tensile Tester (IDT) in an effort to evaluate the resistance against low-temperature cracking. The rational mix-design has been disseminated through the pavement recycling community and further research is currently on going to verify and/or improve the mix-design.

INTRODUCTION

A sustainable way of rebuilding roadways is the use material that is in-place, refurbish it and use it again. The use of cold in-place recycling can save the user tremendous amounts of time, money, material and trucking. However, experience has shown that some formerly conducted projects did not work out as desired, which is expected to be due to a lack of a standard mix-design method. Variations in every step of the rebuilding process lead to a lack of reliability which would be necessary to ensure the expected pavement performance.

Current practices are represented in existing mix-design methods. The Oregon Method, e.g., uses the Oregon State Highway Division test method 126 to determine the total liquid content, i.e. the sum of water and emulsion content. A formula that considers base emulsion content, gradation, residual asphalt content and viscosity gives the amount of emulsion to be added, the remainder is to be filled with water. Observations in the field can justify adjustments. This method seems to lead to considerable results, however does not supply a desired standardized procedure.

The Asphalt Institute Method is according to their emulsified asphalt method. The Centrifuge Kerosene Equivalent test is used to determine the optimum binder content to produce samples. Strength, modulus and retained strength after moisture conditioning are determined, and the AI recommends the use of the heaviest asphalt that is feasible. Although more methods such as the ones mentioned above do exist, none of them offers the desired standard. For that reason, this study aimed at an approach towards a standardized way of performing cold in-place recycling (CIR) using the SUPERPAVE method.

WORK PLAN

Scientific Approach

After forming an Expert Task Group consisting of representatives from research and industrial parties as well as state and federal agencies, the work plan, shown in **Table** , offered a structured approach toward the objective. Based on the evaluation of present material and current practice (modified Marshall Mix-Design), analysis thereof could lead to the development of a new, performance-based Mix-Design Method.

Table D11: ETG Work Plan

Phase	Task
I	Identify Sensitivities
II	Procure and test RAP + Emulsion
III	Evaluation of Modified Marshall Mix-Design
IV	Development of Performance Based Mix-Design
V	Limited Field Evaluation

The identified sensitivities comprise the water sensitivity as well as the distresses due to rutting, fatigue and low temperatures. Procuring reclaimed asphalt pavement (RAP) material also meant to ensure a representative function of this material for different climatic boundary conditions in North America, which is why material from Kansas, Connecticut, Ontario, Arizona and New Mexico was obtained. The third phase meant to evaluate the Modified Marshall mix-design method which is recommended by the AASHTO Task Force No. 38. It was found out that the modified Marshall mix-design lead to major disadvantages. The coarse RAP does not seem suitable for proper compaction with the Marshall hammer, leading to a too high percentage of air voids. Furthermore, produced specimens weighed too little, specifications about the procedure such as when to add the components, durations of curing periods and how to determine the optimum emulsion content (OEC) are missing. Based on that, a new mix-design method is to be developed in an approach to improve weaknesses detected in the previous phase.

Finally, a field evaluation is supposed to give an objective estimate about the quality of the new method and its performance in the field. The test section mentioned earlier may be of young age, yet tendencies give the user a first estimate about the achieved quality.

Material Properties

To characterize the used material, usual methods of determining the RAP were used. Determination of gradation by sieve analysis of both the RAP as well as extracted aggregate, and asphalt content determination by asphalt extraction method (AASTHO T164-93) or asphalt ignition oven gave comparable characteristics of the RAP material from the different origins. Table D12 shows the gradation of the RAP material obtained from Kansas.

Table D12: Kansas RAP Gradation

Sieve Size [mm]	% Passing
31.8	100
25	100
19.1	90.4
12.5	76.1
9.5	65.5
4.75	42.6
2.36	23.3
1.18	15.8
0.6	8.7
0.3	3.5
0.15	1.5
0.075	0.4

In terms of the emulsion type, it was regarded to as most suitable to use the emulsion usually used with the asphalt in question. e.g., the Kansas sample was accompanied by CSS-1h emulsion supplied by Koch Materials Co. Of course, all emulsion had to meet ASTM specifications.

EXPERIMENTAL PROCEDURE

Overview

The first step in developing a new mix-design was to perform a pilot volumetric mix-design on the RAP materials obtained from Kansas, Ontario, and Connecticut using Superpave Gyratory Compactor (SGC) and to see how the different RAP material react to the compaction by SGC. The modified Marshal mix- design procedure was also used for the pilot modified Superpave mix-design with some adjustments. Optimum Emulsion Content (OEC) and Optimum Water Content (OWC) for different RAP materials were determined and compared with the number of gyration necessary for the required air void. Various pilot trials were performed in order to consider the effect of certain important variable on CIR mix-design.

Development of New mix-design

It was found that the unit weight is the important factor to consider for new CIR pavements. Other variables under study include emulsion content (EC), total liquid content (TLC), Curing time, curing temperature. The emulsion content in increment of 0.5% from 0.5% to 2.0% of total mix by weight was used considering 1.2 % OEM determined from the pilot study. This EC also covers most emulsion content that would be found in the field. Review of the literature and results from the survey show that two level TLC 3.5% and 4.0%. TLC was used as a parameter of water content due to its high use as a parameter for mix-designs. Two different curing time 6 hrs and 24 hrs in order to simulate early strength and long term strength in pavement were used. Also two different curing temperatures were used in order to perform the tests, therefore total of 64 samples were fabricated two for each combinations.

Table D13: Specimen Preparation Plan

<table>
<tr><td rowspan="5"></td><td colspan="8">Curing Temperature</td></tr>
<tr><td colspan="4">25°C(77°F)</td><td colspan="4">60°C(140°F)</td></tr>
<tr><td colspan="8">Curing Time (Hours)</td></tr>
<tr><td colspan="2">24</td><td colspan="2">6</td><td colspan="2">24</td><td colspan="2">6</td></tr>
<tr><td colspan="8">Total Liquid Content (%)</td></tr>
<tr><td>Emulsion Content (%)</td><td>3.5</td><td>4.0</td><td>3.5</td><td>4.0</td><td>3.5</td><td>4.0</td><td>3.5</td><td>4.0</td></tr>
<tr><td>0.5</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td></tr>
<tr><td>1.0</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td></tr>
<tr><td>1.5</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td></tr>
<tr><td>2.0</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td><td>2</td></tr>
</table>

In order to investigate the effects of the above parameters on CIR mixtures, it was imperative that the densities of the laboratory specimens simulate fields densities. To achieve this density, one or more of the parameters of SGC needed to be changed from the HMA specifications. SGC collects the height data of the specimen for each gyration during the compaction process, this information along with the mass of the mix, can be used to estimate the specific gravity of the specimen after every. This accomplishes by measuring the bulk specific gravity of the compacted specimen and comparing tit to the last estimated specific gravity after the last gyration with a correction factor. Using these procedure 37 gyrations were required to achieve the density of 130 pcf as of Connecticut material. Thus, 37 gyrations were applied to compact the specimen for the experimental program in order to develop the new mix-design.

TEST RESULTS AND DATA ANALYSIS

The bulk specific gravity of each specimen was measured twice. The first measurement took place two hrs after the end of the curing period in order to bring the specimens to the room temperature from 60°C (77°F). The second measurement was performed one week after the compaction to allow all water to leave the specimen. The test results are show in the table below.

Table D14: Unit weight (pcf) for experimental program using Connecticut RAP w/ HFMS- 2 emulsion (*2 Hours after Curing*)

	Curing Temperature							
	25°C(77°F)				**60°C(140°F)**			
	Curing Time (Hours)							
	24		**6**		**24**		**6**	
	Total Liquid Content (%)							
Emulsion Content (%)	**3.5**	**4.0**	**3.5**	**4.0**	**3.5**	**4.0**	**3.5**	**4.0**
0.5	132.6	131.9	132.5	130.3	129.9	129.6	132.5	131.3
1.0	129.0	131.6	132.4	133.2	129.8	129.5	131.6	131.2
1.5	131.0	131.8	135.1	135.2	134.4	131.4	130.3	130.4
2.0	131.0	130.6	132.2	131.4	133.6	133.5	132.5	131.4

Table D15: Unit weight (pcf) for experimental program using Connecticut RAP w/ HFMS- 2 emulsion (*1 week after curing*)

	Curing Temperature							
	25°C(77°F)				**60°C(140°F)**			
	Curing Time (Hours)							
	24		**6**		**24**		**6**	
	Total Liquid Content (%)							
Emulsion Content (%)	**3.5**	**4.0**	**3.5**	**4.0**	**3.5**	**4.0**	**3.5**	**4.0**
0.5	132.0	130.9	130.2	132.8	130.2	130.1	131.3	130.6
1.0	128.5	130.3	131.1	131.1	129.7	129.7	131.0	131.0
1.5	130.6	130.8	133.6	133.1	134.6	131.6	129.9	130.0
2.0	130.4	129.8	131.7	131.0	133.8	133.8	132.3	131.0

With the use of the Minitab statistical software and analysis of variance (ANOVA) was performed on this data to investigate the effect of the four variables (Emulsion content, Total Liquid Content, Cure Time, and Curing Temperature). The results indicated that all four variables are essential to the preparations of CIR mixtures and need to be taken into consideration in the new mix-design methods.

The values for the two unit weight measurements i.e., 2 hours and 1 week after curing were analyzed to determine if there is a difference between them. The null hypothesis is that the mean of the unit weight values two hours after the curing are equal to the mean of the unit weight values one week after curing. The results show that there is the largest differences between the two measurements occur for the specimens that were cured for 6 hrs and the specimen that were cured at 25°C (77°F). The obvious reason for this is that the short time and cooler temperature does not allow all the mixing water to leave the specimen. One week would allow most, if not all, of the water to leave the specimen. The 24 hours curing time and the 60°C (140°F) curing temperature would more easily allow the water to remove from the specimen, thus resulting in less difference between values.

Based on the above analysis, specimen preparations have been formulated for the new mix design method for CIR, which are as follows:

The specimens are cured for 24 hours at 60°C (140°F) after compaction.

A minimum of four emulsion contents are used.

The number of gyrations used to compact the specimens should be adjusted to achieve densities similar to those found in the field.

Applications and performance prediction of new mix-design method

Application of new mix-design

Once the new mix- design procedure was develop, the next step was to apply the mix –design. In order to be representative of the different types of the materials that are being used for CIR, the mix-design should be performed using material from the various locations throughout North America. Therefore, the new mix-design was applied using materials from five geographically varied locations, i.e., Connecticut, Kansas, Ontario, Arizona and New Mexico. RAP was received from roads that were under construction or were soon undergo construction using CIR pavement rehabilitation techniques. The application of the new mix design method took place with milled samples.

First step necessary to perform the new mix-design on the obtained materials was to determine comparative effort of the Superpave Gyratory Compactor (SGC) to simulate field density for each material. Field density was measured for each CIR project and the unit weights obtained is listed in the table below. The numbers of gyrations to be applied to achieve the field unit weight were determined from the height data collected from the SGC.

Table D 16: Field Unit Weights and Gyrations required simulating Field Density

RAP Material	Field Unit Weight (pcf)	Number of Gyrations to Simulate Field Density
Connecticut	130.0	37
Kansas	130.0	33
Ontario	140.0	90
Arizona	127.5	48
New Mexico	131.5	97

PERFORMANCE PREDICTION

An attempt was made to predict the performance of the CIR mixtures prepared using the new volumetric mix-deign. The distress modes that were investigated for the performance analysis were rutting, fatigue cracking, and low temperature cracking. The distress modes of rutting and fatigue cracking were investigated using the computer program VESYS.

Prediction of the future pavement performance for low temperature cracking was attempted on the material obtained from the different regions using Superpave thermal cracking model TCMODEL using indirect tensile tester (IDT).

Creep compliance *D(t)* obtained for each testing temperature separately and it was found that the CIR mixtures exhibit larger deformations than HMA and the difference between the the defeormations at two different temperature of -20 and -10°C was very small as show in figure D15 and D16.

CIR mixtures were found to be generally weaker than HMA, exhibit rather ductile behavior, have higher deviations, as shown in the graphs below.in figure D17 and D 18.

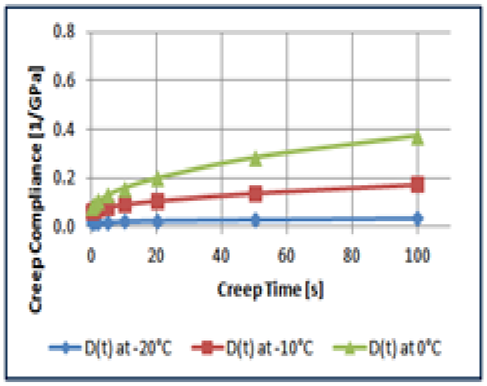
Creep Compliance and Tensile Strength results obtained by the IDT testing were incorporated into Thermal Cracking (TC) Model of Mechanistic-Empirical Pavement Design Guide (MEPDG) software, and the interpretations of the results obtained are:

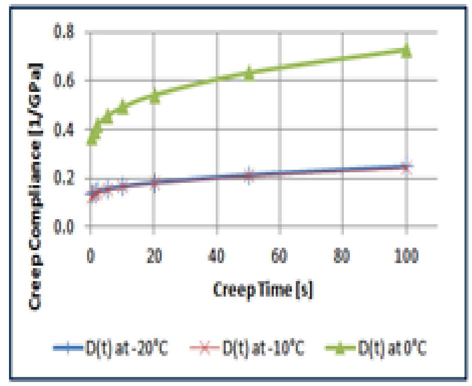
-- No thermal cracking predicted for both HMA and CIR mixtures because of the fact that,

-- Not load-, but temperature-related

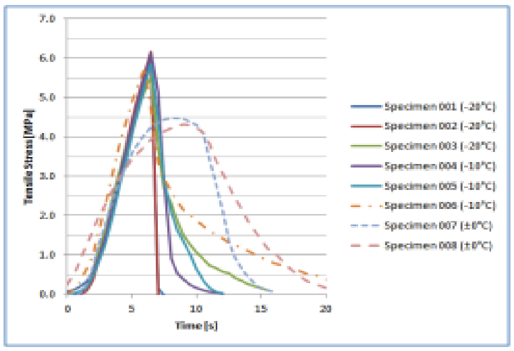
-- Creep compliance results show that mixture allows higher deflections

--Ductile behavior reduces risk of (sudden) cracking.





FigureD15 : Creep Compliance of HMA Mixture   Figure D 16: Creep Compliance of CIR Mixture



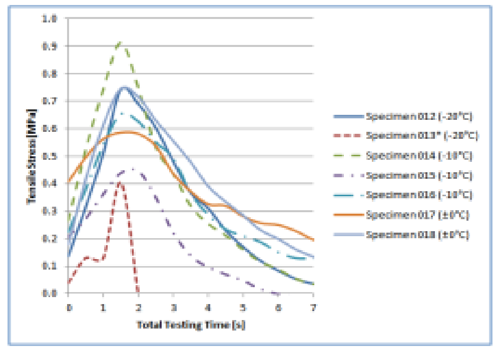


Figure D 17: Tensile Stress over Time for HMA Mix.   Figure D 18: Tensile Stress over Time for CIR Mix.

FIELD VERIFICATION

A test section using the new mix-design was constructed on Route94 in Gila Indian Reservation in Arizona on October 2, 2000. Thus test section is in a desert climate. The road is two lane highways with moderate vehicular traffic. The mix-design for the test section was performed using RAP milling that was taken from the test site. The OEC was determined to be 2.5% and OWC was determined to be 2.0%. Observations taken in March 2002, i.e., about 18 months after construction, which included two winters, showed no distress in the roadway.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The modified Marshall mix-design method cannot be the desired method for CIR constructions in the future. Too many weaknesses require a new standard method.

The determination of the optimum emulsion content (OEC) and optimum water content (OWC) are a vital part of mix-design. Its results are shown to the reader in following Table

Table D 17: OEC and OWC of all origins

	OEC	OWC
Connecticut	1.5%	2.9%
Kansas	1.4%	2.5%
Ontario	1.3%	2.2%
Arizona	2.6%	1.8%
New Mexico	1.1%	1.8%

Instead, a test section in Arizona with the new mix-design was established. Although the obtained results from that test section are not very old by pavement standards, but first observations indicate a satisfactory performance.

Using TC modelling of MEPDG software the conclusions were made as:

- Both HMA and CIR mixtures performed well in terms of low-temperature cracking.
- HMA does not crack due to high tensile strength
- CIR avoided distresses through higher creep compliance

Recommendations

On the basis of the achieved results, further research is highly desirable. This may contain further analysis of IDT test results with a finished version of TCMODEL. But other distresses such as fatigue and rutting will reveal vital behaviour characteristics of the pavement constructed with the developed method. Also, ongoing monitoring of the test section mentioned above may provide a thorough field validation of the developed method. Also, more test sections could strengthen the field validation by representing a wider range of environmental boundary conditions.

Another aspect that may post an interesting focus for future research will be other additives instead of emulsions. Fly ash, cement, or lime slurry may be suitable alternatives.

APPENDIX E

INTEGRATION OF PMS WITH GIS FOR THE URI KINGSTON CAMPUS ROADWAY NETWORK

APPENDIX E INTEGRATION OF PMS WITH GIS FOR THE URI KINGSTON CAMPUS ROAD NETWORK

The tabular information is given below.

Table E1 PMS Data obtained from MicroPaver PMS to be used to integrate with Spatial Data															
Object_ID	Network ID	Branch ID	Section ID	Y_1986	Y_1990	Y_1995	Y_2000	Y_2005	Y_2010	Y_2013	Surface	Width	Age_Insp	Last Insp. Date	*Deflec. values*
URI_00-12_1_1	URI_00-12	1	1	N/A	N/A	N/A	19	14	44	43	AC	25	16	4/10/2013	*39.23*
URI_00-12_1_2	URI_00-12	1	2	N/A	83	59	37	39	41	42	AAC	27	26	4/10/2013	*39.23*
URI_00-12_1_3	URI_00-12	1	3	69	63	57	44	27	36	46	AC	34	50	4/9/2013	*39.23*
URI_00-12_2_1	URI_00-12	2	1	83	81	77	73	70	69	73	AC	23	50	4/4/2013	*34*
URI_00-12_2_2	URI_00-12	2	2	N/A	100	95	91	92	75	79	AAC	22	23	4/9/2013	*34*
URI_00-12_2_3	URI_00-12	2	3	79	74	69	63	57	7	2	AC	19	45	4/4/2013	*34*
URI_00-12_3_1	URI_00-12	3	1	85	83	80	77	73	52	51	AC	33	53	4/10/2013	*59*
URI_00-12_4_1	URI_00-12	4	1	N/A	N/A	N/A	89	98	69	81	AC	32	16	4/9/2013	*17.56*
URI_00-12_4_2	URI_00-12	4	2	N/A	N/A	N/A	78	87	85	73	AC	32	16	4/10/2013	*17.56*
URI_00-12_5_1	URI_00-12	5	1	100	100	100	100	97	80	76	AC	22	40	4/20/2011	*40.43*
URI_00-12_6_1	URI_00-12	6	1	53	45	36	28	33	83	87	AAC	31	53	4/10/2013	*53.34*
URI_00-12_6_2	URI_00-12	6	2	62	57	50	42	93	97	89	AAC	31	53	4/10/2013	*53.34*
URI_00-12_7_1	URI_00-12	7	1	53	46	37	29	33	44	37	AC	20	53	4/9/2013	*43*
URI_00-12_8_1	URI_00-12	8	1	90	88	86	84	99	80	80	AC	22	51	4/10/2013	*59.45*
URI_00-12_9_1	URI_00-12	9	1	87	83	78	76	80	75	71	AC	40	40	4/11/2013	*29.25*
URI_00-12_9_2	URI_00-12	9	2	86	82	77	71	81	77	57	AC	40	39	4/10/2013	*29.25*
URI_00-12_10_1	URI_00-12	10	1	100	100	100	100	78	53	43	AC	25	51	4/10/2013	*31.57*
URI_00-12_10_2	URI_00-12	10	2	92	91	90	88	92	78	67	AC	22	51	4/10/2013	*31.57*
URI_00-12_10_3	URI_00-12	10	3	100	100	100	100	100	64	44	AC	22	51	4/10/2013	*31.57*
URI_00-12_10_4	URI_00-12	10	4	100	100	100	100	100	83	81	AC	22	51	4/10/2013	*31.57*
URI_00-12_10_5	URI_00-12	10	5	100	100	100	100	94	76	67	AC	22	51	4/10/2013	*31.57*
URI_00-12_11_1	URI_00-12	11	1	65	55	43	31	41	57	67	AC	25	41	4/10/2013	*45.23*
URI_00-12_12_1	URI_00-12	12	1	68	60	49	38	16	68	89	AC	27	41	4/10/2013	*54.23*
URI_00-12_13_3	URI_00-12	13	3	74	58	39	49	90	68	67	AC	31	53	4/4/2013	*57.61*
URI_00-12_13_2	URI_00-12	13	2	45	37	27	45	84	81	80	AC	31	53	4/4/2013	*57.61*
URI_00-12_13_1	URI_00-12	13	1	42	33	22	42	96	85	85	AC	31	33	4/4/2013	*57.61*
URI_00-12_14_1	URI_00-12	14	1	75	68	61	53	67	82	44	AC	26	43	4/10/2013	*34*
URI_00-12_15_1	URI_00-12	15	1	100	100	100	100	89	67	66	AC	23	53	4/10/2013	*30.81*
URI_00-12_16_1	URI_00-12	16	1	N/A	N/A	N/A	89	30	35	23	AAC	31	14	4/11/2013	*67*
URI_00-12_16_2	URI_00-12	16	2	N/A	N/A	N/A	92	37	41	18	AAC	31	14	4/4/2013	*67*

Object_ID	Network ID	Branch ID	Section ID	Y_1986	Y_1990	Y_1995	Y_2000	Y_2005	Y_2010	Y_2013	Surface	Width	Age_In sp	Last Insp. Date	*Deflec. values*
URI_00-12_17_1	URI_00-12	17	1	77	73	69	49	39	40	28	AC	30	53	4/10/2013	*40.76*
URI_00-12_17_2	URI_00-12	17	2	99	99	99	74	73	52	44	AC	20	53	4/10/2013	*40.76*
URI_00-12_17_3	URI_00-12	17	3	90	89	87	71	71	56	88	AC	24	53	4/10/2013	*40.76*
URI_00-12_18_1	URI_00-12	18	1	49	41	31	22	61	26	56	AC	16	53	4/10/2013	*60.34*
URI_00-12_19_1	URI_00-12	19	1	37	27	15	3	20	43	46	AAC	20	53	4/20/2013	*60.34*
URI_00-12_21_1	URI_00-12	21	1	N/A	N/A	100	100	48	34	39	AAC	26	22	4/10/2013	*65.65*
URI_00-12_21_2	URI_00-12	21	2	100	100	100	100	65	51	85	AAC	26	53	4/10/2013	*65.65*
URI_00-12_22_1	URI_00-12	22	1	N/A	99	92	85	81	66	64	AC	26	23	4/10/2013	*40.65*
URI_00-12_22_3	URI_00-12	22	3	N/A	99	92	86	79	64	74	AC	26	23	4/10/2013	*40.65*
URI_00-12_22_7	URI_00-12	22	7	N/A	98	77	57	52	37	20	AC	26	23	4/10/2013	*40.65*
URI_00-12_22_9	URI_00-12	22	9	N/A	97	73	49	45	22	22	AC	26	23	4/10/2013	*40.65*
12_22_11	URI_00-12	22	11	N/A	98	76	55	65	44	29	AC	26	23	4/10/2013	*40.65*
12_22_13	URI_00-12	22	13	N/A	99	86	74	57	67	46	AC	26	23	4/10/2013	*40.65*
12_22_15	URI_00-12	22	15	N/A	99	93	88	71	57	62	AC	26	23	4/10/2013	*40.65*
12_22_17	URI_00-12	22	17	N/A	100	96	92	79	66	56	AC	26	23	4/10/2013	*40.65*

Table E2 Spatial Data Obtained from the Shape File of RIGIS (after Clipping into URI boundary)

OBJECTID	SHAPE_Length	Branch	Branch_ID	Section_ID	Section_Rank	Object_ID
3	175.1704732	Alumni Avenue East	1	1	S	URI_00-12_1_1
4	130.0823346	Alumni Avenue East	1	2	S	URI_00-12_1_2
6	500.8249045	Alumni Avenue East	1	3	P	URI_00-12_1_3
10	1984.213874	Alumni Avenue West	2	1	P	URI_00-12_2_1
11	195.1967689	Alumni Avenue west	2	2	S	URI_00-12_2_2
12	260.9219492	Alumni Avenue West	2	3	S	URI_00-12_2_3
13	695.3434213	Baird Hill Road	3	1	P	URI_00-12_3_1
14	642.1866694	Butterfield Road	4	1	P	URI_00-12_4_1
15	950.7504468	Butterfield Road	4	2	P	URI_00-12_4_2
16	483.1548228	Chafee Road	5	1	S	URI_00-12_5_1
17	114.3527575	Davis Road	6	1	T	URI_00-12_6_1
18	371.3648535	Davis Road	6	2	T	URI_00-12_6_2
19	1486.695139	Faculty Apartment Circle	7	1	S	URI_00-12_7_1
20	536.154736	Farmhouse Road	8	1	S	URI_00-12_8_1
21	3360.419208	Flagg Road	9	1	P	URI_00-12_9_1
22	1354.754557	Flagg Road	9	2	P	URI_00-12_9_2
23	641.8324738	Fraternity Circle	10	1	S	URI_00-12_10_1
24	633.119136	Fraternity Circle	10	2	S	URI_00-12_10_2
25	317.5598724	Fraternity Circle	10	3	S	URI_00-12_10_3
26	414.7796687	Fraternity Circle	10	4	S	URI_00-12_10_4
27	619.527908	Fraternity Circle	10	5	S	URI_00-12_10_5
28	780.7149947	Graduate Village East	11	1	S	URI_00-12_11_1
29	746.3630875	Gradute Village West	12	1	S	URI_00-12_12_1
30	220.9687669	Greenhouse Road	13	3	S	URI_00-12_13_3
31	219.9305773	Greenhouse Road	13	2	S	URI_00-12_13_2
32	400.4816998	Greenhouse Road	13	1	S	URI_00-12_13_1
33	975.3060812	Heathman Road	14	1	S	URI_00-12_14_1
34	1011.255002	Keaney Road	15	1	S	URI_00-12_15_1
35	408.4740681	Lippitt Road	16	1	T	URI_00-12_16_1
36	361.1220251	Lippitt Road	16	2	S	URI_00-12_16_2
37	1139.404386	Lower College Road	17	1	P	URI_00-12_17_1
38	366.0934471	Lower College Road	17	2	S	URI_00-12_17_2

39	460.4315027	Lower College Road	17	3	S	URI_00-12_17_3
40	758.5007698	Peckham Farm Road	18	1	S	URI_00-12_18_1
41	502.1648618	Powerhouse Road	19	1	T	URI_00-12_19_1
42	188.3895356	Ranger Road	21	1	T	URI_00-12_21_1
43	654.6450935	Ranger Road	21	2	T	URI_00-12_21_2
44	598.9568814	Upper College Road	22	1	P	URI_00-12_22_1
46	394.8661814	Upper College Road	22	3	P	URI_00-12_22_3
51	634.6578851	Upper College Road	22	7	P	URI_00-12_22_7
53	214.3768846	Upper College Road	22	9	P	URI_00-12_22_9
55	294.6420833	Upper College Road	22	11	P	URI_00-12_22_11
57	245.2951767	Upper College Road	22	13	P	URI_00-12_22_13
59	143.117188	Upper College Road	22	15	P	URI_00-12_22_15
60	167.5366304	Upper College Road	22	17	P	URI_00-12_22_17

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