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SELECTION OF ASPHALT RECYCLING METHODS AND RECYCLED ASPHALT MIXTURE PROPERTIES

VOLUME I

By

Nicholas James Cerullo

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ABSTRACT

SELECTION OF ASPHALT RECYCLING METHODS AND RECYCLED ASPHALT MIXTURE PROPERTIES

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The socioeconomic growth of a country depends on the health of its transportation network. To maintain the highway portion of the network, the needs for fixing pavements has to compete with other demands on the limited public funds. Asphalt pavement recycling is one way to stretch existing resources and if adopted properly, it can precipitate in substantial savings in cost, energy and natural resources.

In this study, methodologies and procedures for the adoption of asphalt pavement recycling in Pakistan were developed based on field and laboratory investigations. Twelve asphalt concrete (AC) pavement test sites were selected and each test site was subjected to the detailed field investigation stated below. In addition, one source of virgin binder, two sources of recovered material, and two sources of virgin aggregates were identified and sampled. The samples were subjected to the lab investigation stated below.

The field investigation consisted of: distress surveys, material sampling, longitudinal and transverse profiles, and nondestructive deflection testing (NDT). The laboratory investigation consisted of characterizing the properties of virgin, blended and recovered materials. Based on the field and laboratory investigations, guidelines for hot in-place (HIR), cold in-place (CIR) and in-plant (IPR) recycling were developed. The guidelines were implemented to determine the recommended recycling methods for each of the twelve test sites.

TO MY PARENTS

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The extensive highway systems of modern civilizations originate in the antedate of recorded history. The first hot mix asphalt (HMA) surfaced road was built in the Mesopotamia around 3500 B.C. Ancient civilizations came to realize the importance of roads and transportation systems (Baladi & Synder 1992). The Egyptians built great roads to aid in the construction of the pyramids, and Babylon paved its streets using the hot asphalt binder that was oozing out of the oil field. Later, Egypt was connected to Babylon and Rome via a stone surfaced road constructed in three layers. Remnants of this road can be found today throughout Europe, Syria, Israel, and the rest of the Middle East countries.

Since the beginning of the 20th century, modern civilizations have witnessed significant increases in the public demands for paved roads and refined oil products. Hence, the use of HMA in constructing roads skyrocketed. Unfortunately, as is the case with any natural or manufactured products, highway systems cannot be made to last forever. They deteriorate and disintegrate at an accelerated rate unless they are properly and continually maintained, rehabilitated, redesigned and reconstructed. A constant flow of money is required on an annual basis to keep highway pavements operating in safe and acceptable conditions. This sum of money, however small, is not available to the highway agencies. Lack of funds, coupled with dwindling natural resources and high public demand to repair the highway systems, makes the agencies job a difficult one. This difficulty is further compounded by several other factors including (Baladi & Synder 1992):

- The number of available alternatives to repair the highway systems and their associated costs.
- The number of miles of the different classes of the highway pavements that are in need of repair.
- The increasing traffic volume and load.

This difficulty and the multi dimensions of the problem forced the highway agencies to re-examine their road building policies to arrive at cost effective and environmentally friendly solutions for rehabilitating the rapidly deteriorating transportation infrastructures. HMA recycling is one of the alternatives that is evolving in the rehabilitation and reconstruction of flexible pavements.

Recycled asphalt pavement (RAP) is a widely accepted rehabilitation technique that has many benefits. For the highway agencies, these benefits include saving on costs and dwindling natural resources. Research on the material characteristics of HMA pavement has yielded a better understanding of its behavior throughout its life and its salvage values. In some developing countries however, such as in Pakistan, the characteristics and salvage value of existing HMA pavements are misunderstood. Consequently, deteriorated HMA pavements were removed and piled next to the roads as waste materials. Recycling of HMA did not exist in Pakistan. Recently, the highway authorities in Pakistan became aware of the value of these piled asphalt materials and have started recycling them.

To properly design and recycle existing HMA pavements in a cost-effective manner, two major steps need to be taken:

1. The pavement must be subjected to functional and structural evaluation to determine the causes of distress and the integrity of the lower pavement layers and roadbed soils. It should be noted that, in general, pavements deteriorate due to traffic, environmental and material factors (AASHTO 1993).
2. The engineering and physical properties of the existing and the virgin HMA materials must be evaluated. The results would determine the appropriate recycling technique to be used and the percent recycled materials to be included in the new HMA mix (Epps et al. 1978, Kandhal & Millick 1997).

Recycling technologies have evolved over time. Starting as early as the 1900's, cold recycling was attempted on roadways in the U.S. In the 1930's the first case of hot in-place recycling was recorded (ARRA 2001). In the mid 1970's, advances in in-plant and in-place recycling equipment and construction occurred (Terrel et al. 1997). The combination of advances in the equipment, construction and research has made it possible that RAP exhibits the same, if not better, qualities than virgin HMA pavement.

1.2 PROBLEM STATEMENT

In order to continue the socioeconomic growth in Pakistan there needs to be a solution to the deteriorating transportation infrastructure networks caused by increases in traffic. Recycling asphalt pavements is one of several cost-effective solutions to rehabilitate and/or reconstruct asphalt pavements. Currently, Pakistan has very little knowledge on selecting the proper technique for asphalt pavement recycling and on characterizing the recycled and virgin asphalt materials. The research addresses those issues by evaluating several existing asphalt pavements in Pakistan and testing recycled asphalt mixtures.

1.3 OBJECTIVES

The main objectives of the research are as follows:

- Establish criteria for evaluating the structural and functional conditions of pavements.
- Determine the appropriate asphalt recycling method based on the pavement structure evaluation.
- Determine the characteristics of recovered, virgin and recycled HMA material and analyze what will affect the properties of the recycled asphalt mix.
- Determine proper HMA mixture design involving recovered material.
- Establish guidelines for recycling asphalt pavements.

1.4 RESEARCH PLAN

Selecting the appropriate recycling method and correctly designing RAP must be based on extensive pavement evaluation and material testing. In order to achieve the above objectives test sites need to be selected. Since this research is intended to be used for Pakistan, the tests sections need to represent the variability of pavement distress, traffic and climate within Pakistan. Material tests were conducted in order to determine how the aged asphalt binder has an effect on the RAP performance. To accomplish the objectives of this thesis a comprehensive research plan was designed. The research plan includes the following four tasks:

Task – 1: Literature Review

An extensive literature review was conducted on HMA and RAP pavements. The first part of the review focuses on four recycling methods, hot in-place, cold in-place, hot in-plant and cold in-plant recycling. In the second part of the review, characteristics of

HMA materials are discussed. Differences in traditional (Marshall) and new (Superpave) asphalt mix designs are also ascertained. Also a review was done on the performance of RAPs from past experiences.

Task – 2: Field and Laboratory Investigation

The first step in determining if the pavement is feasible for recycling is to evaluate the condition of the pavement in question. Prior to research done in this thesis, twelve asphalt concrete (AC) test sites were selected by a USA and Pakistan research team. In order to evaluate the twelve AC test sites, the Pakistan research team also conducted:

- Distress surveys
- Coring/borings
- Longitudinal/transvers profile measurements
- Falling Weight Deflectometer (FWD) tests

The distress and transverse/longitudinal profile information for the twelve test sites was organized so that pavement condition of the tests sites could be determined. In order to determine the structural capacity of the existing pavements the nondestructive deflection data needed to be organized so that Task 3 can be carried out.

Following the pavement evaluation, laboratory testing was conducted by a Pakistan research team to determine the properties of virgin and recycled asphalt material. Virgin and recycled asphalt binder was tested using conventional and performance grading testing procedures. Virgin and recycled mixtures were created using Marshall and Superpave mix design procedures. The laboratory data was then organized so that Task 3 can be conducted for this thesis.

Task – 3: Analyzing Distress, Deflection and Material Characterization Data

In this task the data that was collected in the field and laboratory investigation was analyzed. Distress mapping from the Pakistan research team was used to determine the overall condition of each site. The transverse profiles were measured to ascertain if there was presence of a lower layer weakness. Deflection data was used to determine the structural capacity of the pavement layers and to check the variability of the pavement's condition along the site. The combination of the functional and structural analysis of the pavement led to determining what recycling method is best for the site, if the case for recycling is made.

When the case for recycling is made, a variety of material characteristics need to be addressed to properly design the mix of the RAP. The laboratory results from Task 2 was analyzed in such a way that appropriate blending charts of recovered and virgin materials leads to a suitable mix designs for in-plant recycling.

Task – 4: Guidelines for Recycling Asphalt Pavements

The accomplishments in Task 3 led to the objectives in Task 4. In this task, guidelines were established for recycling asphalt pavements. The guidelines addressed the proper selection of asphalt recycling methods and recycled asphalt mixture design.

1.5 THESIS LAYOUT

The thesis layout first begins with the introduction chapter. In this chapter, a background to asphalt pavements and recycling asphalt pavements was given. Also the objectives and tasks of the thesis were described. Following the introduction is the literature review chapter. The literature review describes various recycling methods, properties of recycled asphalt materials and the results of recycled asphalt pavements from past researchers. The third chapter is the field and laboratory investigation. This chapter presents the data for various pavement condition tests of the twelve asphalt concrete (AC) test sites. Also presented in this chapter three are the properties of the recycled asphalt materials. The fourth chapter is the data analysis. In this chapter the pavement's condition of the twelve AC test sites was determined. Also the recycled asphalt material properties were analyzed in order to determine critical temperatures of the blended binders and master curves of the recycled asphalt mixtures. Also at the conclusion of this chapter, guidelines were established for selecting the appropriate recycling method. Using the guidelines, candidate recycling methods were selected for the twelve AC test sites. Finally, the fifth chapter is the summary, conclusions and recommendations.

CHAPTER 2

LITERATURE REVIEW

This literature review was conducted by several people and is divided into six sections as follows:

- Section 2.1 cold planing
- Section 2.2 asphalt pavement recycling methods
- Section 2.3 pavement evaluation for the selection of asphalt pavement recycling
- Section 2.4 material characterization
- Section 2.5 hot mix asphalt mixture design containing RAP
- Section 2.6 performance of recycled asphalt pavement

An extensive report conducted by Miss Jaya Adhikari was used as a reference for Sections 2.2 and 2.3. A literature review by Dr. S.W. Haider was also used as a reference for Sections 2.5 and 2.6.

2.1 COLD PLANING (MILLING)

Cold planing (CP) is the controlled removal of an existing pavement to a desired depth, longitudinal profile, and cross-slope, using special equipment (milling machine). The resulting textured surface can be immediately used as a driving surface. Asphalt pavements can be milled to a minimal depth necessary for surface improvements or up to a depth of 10 inches (Valentine Surfacing 2009). CP could also be used to roughen the pavements to restore low friction numbers and eliminate slipperiness. Furthermore, the surface can be further treated with one of the other asphalt recycling methods. Once it is cleaned and tack coated, it can be overlaid with HMA or recycled asphalt mix.

The modern cold planer or milling machine has a large diameter rotary cutting drum which is equipped with specially designed replaceable tungsten carbide cutting "teeth" or "tools" that remove or "mill" the existing pavement. Most milling machines are equipped with automatic grade control systems to mill to the specified elevations and grades. The RAP generated during the CP operation can be removed from the site by loading onto haul trucks. The RAP is then recycled using either cold or hot recycling process. Also, this RAP could be reused as base aggregate for roadway construction and widening, ditch linings, pavement repairs or as a dust free surfacing of gravel roads (ARRA 2001). The uses of CP technique and its advantages include:

- Remove wheel ruts, deteriorated pavement surfaces, and/or oxidized asphalt mixes
- Rectify longitudinal profile and cross-slope
- Restore drainage
- Remove the total asphalt layer for further recycling on roadway reconstruction or shoulder widening projects
- Remove of existing cracks in asphalt layer or seal coats prior to HMA overlays
- Improve pavement surface friction
- Prepare the pavement surface prior to an additional form of asphalt recycling
- Conserve energy
- Increase project efficiency and reuse the existing materials

- Reduce user costs through higher productivity and less disruption to traffic

2.2 ASPHALT PAVEMENT RECYCLING METHODS

Asphalt pavements can be recycled using two general methods, hot and cold recycling. Furthermore, hot or cold recycling can be divided into hot or cold in-plant recycling, and hot or cold in-place recycling.

2.2.1 Hot In-Plant Recycling

In this method, the recovered material is transported to a plant, where it is mixed with required quantity of virgin aggregates and asphalt binder and/or recycling agents. The mixing is performed at a high temperature, thus named hot in-plant recycling. RAP can be either obtained by milling or crushing the existing pavement. "Successfully-completed projects across the U.S.A. have shown both cost effectiveness and quality of pavement produced by this time-tested process. Reclaimed aggregates and asphaltic cements are broken down to their original state and reused to produce high quality asphaltic concrete, the performance of which has been proven to equal that of conventional mixes" (ARRA 2008).

The method can be used to correct the gradation and mix properties of the deteriorated pavements. It can be also used to correct or maintain the horizontal or vertical geometries of the pavement. Hot in-plant recycling advantages include:

- It can be done repeatedly with the same materials
- It needs very little modifications in the existing HMA equipments and plants
- It eliminates disposal and pollution problems

There are two basic construction methods for hot in-plant recycling, batch plant and drum plant. There are many methods available for recycling in a batch plant. The most common and widely used method of recycling pavement in batch plant is Maplewood Method (see Figure 2.1). In this method, virgin aggregates are superheated to a desired temperature which is governed by the moisture level of RAP, amount of RAP used and the final mix temperature required. RAP, which is stored separately in bins, is inserted in the weight hopper by a conveyor system. Superheated aggregates are also inserted in the weight hopper; hence the method is also called "weight bucket" batch facility technique. "With the "weight bucket" method of recycling in a batch facility, cold, wet RAP is added to the weigh hopper where the batch controler weighs RAP as an additional material. The RAP is mixed with superheated virgin materials, and conductive heat transfer occurs in the weighbox and the pugmill." (NAPA 2007)

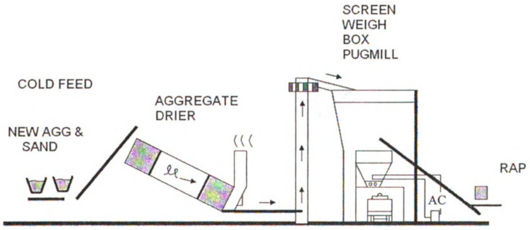


Figure 2.1 Maplewood method of batch plant hot recycling

Some modification has to be made in the HMA batch plant to facilitate recycling. A better exhaust system has to be maintained in the aggregate drier unit, as significant amount of aggregates are superheated. The asphalt binder feeder needs to have a system to introduce recycling agent in the pugmill or weight bucket. It is also important to maintain proper exhaust in the pugmill as moist RAP and dry heated aggregates produce a significant amount of steam. Other modifications may also be necessary. Other batch plant methods are available including: Bucket Elevated recycling, Pugmill recycling, RAP into dryer with bucket elevated and RAP dryer recycling.

Center Entry method (Middle Drum Entry method) is the most commonly used method to recycle pavement in a drum plant. A parallel flow drum plant, shown in Figure 2.2, is used in the method.

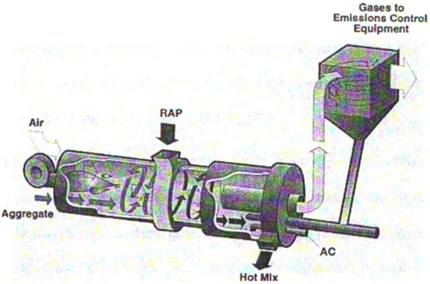


Figure 2.2 RAP in a parallel flow drum

As a first step "Aggregates are dried and heated convectively with the aggregate getting fed in the end with the burner, and the exhaust gases travelling through the dryer in the same direction as the aggregate-hence the term parallel flow". (NAPA 2007). It is heated to about 500°F. RAP is inserted in the center of the drum where it is mixed with superheated aggregates. It is heated by conduction from aggregates and a veil of aggregate prevents direct contact of flame and RAP. However emission of hydrocarbons from asphalt binder in the gas stream is a major problem in this method. It depends on the amount of RAP, amount of fines in the RAP, moisture content of aggregates and the exposure time to direct heat. Thus several modifications are made to the drum plant to control emissions.

"In practice, conventional batch plants (with suitable modifications) can handle up to 15–20% RAP. Above this level, preheating of the RAP may be required. Whereas drum mix plants can handle up to 70% RAP" (Widyatmoko 2006). HMA pavers and compactors can be used to lay and compact the hot recycled mix from the plants. "No special techniques or equipment are required for lay down or compaction when using recycled mixes. However recycled mixes are frequently placed at slightly cooler temperatures than virgin mixes in an effort to reduce the negative effect of superheated temperatures on the facility equipment" (NAPA 2007).

2.2.2 Hot In-Place Recycling

This is an on site, in-place recycling method. In this method, the existing pavement is heated and softened and then milled or scarified to a desired depth. Since it is an in-place method, the hauling expenses is reduced and thus results in considerable construction cost savings. Any pavement with surface distresses and a stable base can be a candidate for hot in-place recycling. This method cannot be used to correct base or subbase problems and distresses due to structural inadequacy. This method generally recycles the asphalt surface to depths of 3/4 to 2 inches. It can be used for the following purposes:

- Eliminate surface cracks

- Correct rutting problem
- Rejuvenate aged asphalt
- Modify asphalt content and aggregate gradation
- Improve drainage and pavement geometry
- Correct other surface distresses like stripping, raveling, shoves and holes.
- Improve surface friction

Hot in-place recycling can be performed in a single pass or multiple pass. In single pass operation, the virgin material and RAP are mixed and placed as a single layer. In multiple pass operation, the RAP is re-compacted and then a new wearing course is laid. The construction methods and additives are detailed in the following paragraphs. According to Asphalt Recycling and Reclaiming Association (ARRA), for hot in-place recycling, the pavement should have temperature equal or greater than 50°F, and it should be free from standing water. The pavement should also have a good base and a roadway width of at least 20 feet. Hot in-place constitutes the following basic steps:

1. Heat application of about 350°F to 225°F for softening the pavement
2. Mechanical removal or scarification of the softened pavement
3. Mixing of RAP with recycling agents and/or virgin material
4. Paving of the recycled mix

The main method is sub divided into three sub methods, namely, surface recycling, repaving and remixing. Virgin aggregate is added only in repaving and remixing. The construction of these three methods varies slightly as detailed below.

Surface recycling is generally used to remove cracks and other irregularities from the pavement surface. Generally the depth of this operation is ¾ to 1 inches. This method involves a single or double pass. If the new overlay is constructed in a separate operation, it is called double pass, otherwise single pass. After compacting the recycled layer, a seal coat, micro surfacing or thin overlay can be laid if required. Figure 2.3 to 2.5 shows different construction units, a preheating unit, a heating and recycling unit and a roller.

The basic construction steps for the process are given below:

1. Preheating the pavement
2. Heating the pavement
3. Scarification or milling
4. Collection of RAP and mixing it with predetermined quantity of recycling agent
5. Placement of recycled mix and compaction

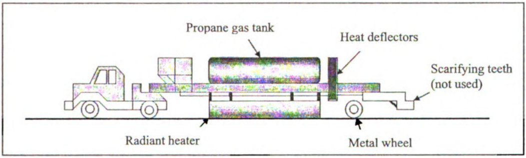


Figure 2.3 Preheating unit

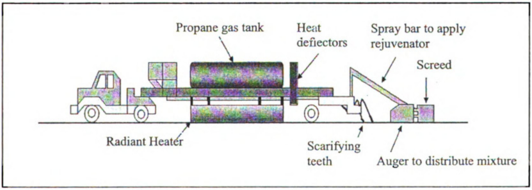


Figure 2.4 Heating and recycling unit

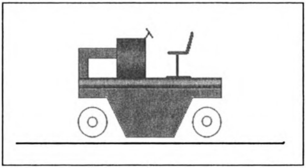


Figure 2.5 Rubber-tired rollers

The preheating unit, starts heating the pavement. This is followed by heating and recycling unit, which heats the pavement to about 350°F to 225°F. Generally two sets of heater are used for the process. This unit also scarifies the heated pavement with the help of multiple loaded spring mounted scarifiers. Rejuvenators are generally added during the process; however, new aggregates are not added.

After scarification, aged and oxidized binder can be restored by adding rejuvenating agents during the mixing process. Sometimes they are also added before scarification, in such cases the scarifiers, scarifies as well as mixes the additives uniformly. In rare cases, additives are added like fog coat after compaction of the recycled pavement. The rate of the application of the rejuvenating agents generally ranges from 0 to .6 gallons/sq. yard.

A free floating screed attached to heating and recycling unit is used to place and level the recycled mix as shown in Figure 2.4. Modified HMA pavers can be also used for the same purpose. The recycled mix is finally compacted with traditional compactors or rollers. This completes the construction process of surface recycling, and traffic can resume as soon as the pavement cools down.

The repaving method is very similar to surface recycling. The only difference is that in repaving, new HMA overlay is laid simultaneously with surface recycling. This method can be used to cure minor rutting, raveling and/or cracking. The construction steps are the same as surface recycling; the only addition is the placement of a new HMA layer after surface recycling. The pavement is heated to about 370°F and the temperature of the pavement at the time of overlaying is about 200°F. The construction can be done in single or multiple passes.

In single pass method, the last equipment which is heating unit, also places new HMA overlay over the recycled mix. The new HMA is conveyed by standard tandem axle haul truck. Both layers are then compacted simultaneously. In case of multiple passes, a separate unit paves the overlay. The paving of new HMA is done according to HMA construction procedure. Then as a final step both layers are compacted. Special mix, like polymer modified mixes can be also used instead of HMA overlay. The method can successfully construct recycled layer and overlay layers, each of 1-2 inch thick. Repaving can be used when conventional HMA is not feasible or when a very thin HMA or specialty mix is required. HMA wearing course of 0.5 inch can be also achieved by the method.

The final method, remixing, is used when addition of virgin aggregate and/or new HMA is needed. Remixing can improve asphalt binder content, aggregate gradation and other mix properties. The remixed layer can function as a wearing course as well. This method includes the following construction steps:

1. Preheating the pavement
2. Heating the pavement

3. Scarifying or milling
4. Collecting the RAP and mixing it with predetermined quantity of recycling agent, virgin aggregates and/or binder in the pugmill
5. Placing the recycled mix and compaction

The process is sequentially depicted in Figure 2.6.

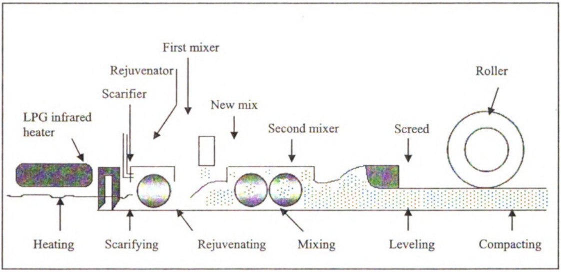


Figure 2.6 Concept of remixing method

The basic construction units are the same as surface recycling. The pavement is heated to a temperature of about 185°F to 219°F. This method can be performed in single or multiple passes. In single pass, the heated pavement is scarified to a depth of about 1 to 2 inches, whereas in multiple passes, the pavement is heated and scarified in layers. Generally in multiple passes, scarification is done in two to four layers of depths of about 1.5 to 3 inches.

The scarified material is collected. Recycling agents and virgin aggregates and/or binder is added to RAP in predetermined quality. It is mixed in a pugmill or drum mixing plant. All the additives are added before the materials enters the pugmill, and the recycling agent is added first, to allow good mixing time. "The rate of mixing recycling agent can vary from 0 to 0.5 gallon/sq. yard and the admix addition is limited to 30% by weight of the recycling mix. Often, 15-25% fresh asphalt mix is added to account for ruts, but additions in the range of 0-50% asphalt material have been reported," (Robert 2006) Twin shaft pugmill or rotary drum mixers are suggested by ARRA to obtain a good uniform mix. The mixture from the pugmill or rotary drum is then laid as a homogeneous mix with either free floating screed or separate asphalt paver. The temperature of pavement at the time of paving is about 110°F to 130°F, and of the mix is about 230°F to 256°F, this forms a thermal bond between both layers. Also the width of heating bed is extended beyond scarification width, so that while paving, a thermal bond can be developed for better stability of longitudinal construction joint.

Previously direct flame was used to heat the pavement; however, recently infrared heating and indirect radiant heating is more commonly used. "The mechanism of infrared heater can be explained as infrared heat is created by heating metal plates until they reach sufficiently high temperatures. The heat from the metal plates is then radiated down onto the surface of the pavement," (Karlsson & Isacsson 2006). Air heaters are also being used. Martec recycling corporation uses air heating in their equipment. A recycling project was carried out on Mississippi interstate using this type of equipment and it was concluded that "although construction problems were encountered, it has been shown that this equipment can adequately heat an existing HMA pavement to permit hot in place recycling to a depth of 50mm (2 inch)" (Crawley 1999).

The problem with HIR has been heat and smoke emissions. The gaseous hydrocarbon and particulate emissions may disturb the surrounding air quality. However new equipments are available that solve the problem. For example Pyropaver 300E HIR system has built in emission control system. Equipments from other manufacturers are also available with similar features, for example AR2000 Super Recycler of Martec recycling corporation, EnviroPaver R1015 of Patterson. These equipments use after burners, to burn the hydrocarbons and particulars emitted and thus solve the emission problem. Quality control in construction is also very important for the success of the recycling project. "Quality control for HIR construction should be similar to that used for conventional hot mix asphalt construction" (Button et al. 1990).

2.2.3 Cold In-Plant Recycling

Cold in-plant asphalt recycling is an in-plant method to recycle pavement without the application of heat. It is performed in a cold central plant hence the named called cold in-plant recycling (CIPR). Cold in-plant is used when the projects require high rates of production and/or close control of the mix design. The method may also be used when cold in-place recycling is not feasible and/or good qualities of RAP stockpiles are available. The method is used to correct distresses in the surface and/or base layers such as ride quality and reflective cracking. However it is commonly used in case of base layer deficiencies. The method can also be used for pavement widening or to lower the pavement profile. Generally a fresh surface course is constructed after recycling. The basic construction steps are as following:

1. Removal of existing pavement
2. Crushing and stockpiling
3. Mixing
4. Laying, aeration and compaction

5. Capping with a virgin asphalt layer

Existing pavement can be removed by ripping and then crushed in-plant. The steps can be ripping, breaking and pulverizing in-place or cold milling followed by crushing and sizing in the plant and stockpiling for future use.

In-plant usually consists of only a mixer and certain auxiliary equipments for feeding the asphalt, water, reclaimed asphalt pavement, and aggregate to the mixer. This makes it, easy to transport and erect. Conventional drum plant or batch plant or continuous (stabilization) plant may be used for mixing; however, the use of continuous plant is most common. Figure 2.7 shows a continuous plant. In this plant "A positive displacement asphalt metering pump controls the automatic feeder, which in turn, measure and govern the flow of material." (NCAT 1997). Sometimes screens are placed between the bins and the pugmill to stop the entry of oversized material. They may be also equipped with storage silo or bin for storing recycled mix and thus facilitating continuous operation.

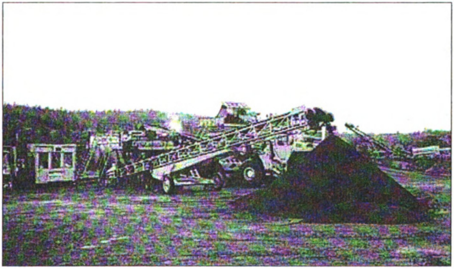


Figure 2.7 Continuous plant

The recycled mix can be stored for future use in bins or stockpiles, or it can be directly taken to the site for laying. Mix can be laid by conventional asphalt paver if aeration is not required. If aeration is required to reduce the fluid content of the mix, then motor grader may be used to grade as well as aerate the mix. The methods for compaction, curing and surface application are the same as in CIR. Generally dense graded recycled mix is compacted to a thickness of 3" or less, and open graded mix to about 4".

The mix design process for CIPR is the same as CIR. The components of the mix are RAP, virgin aggregate (optional), water (optional) and additives. The additives commonly used are asphalt emulsions, foamed asphalt, hydrated lime, fly ash, cement or a combination of these additives. "When the recycled cold mix is to be stockpiled for later use, an emulsion with cutters, such as diesel fuel, may be used. Rejuvenators may also be used to enhance the mix workability. When the recycled cold mix is to be placed immediately a quicker setting emulsion may be used." (ARRA 2008). Some commercial recycling agents by Martin asphalt company specially for CCRP are as following: AES-300 High float, anionic asphalt emulsion, MS-2, MS-1 anionic medium setting asphalt emulsions, CMS-1 cationic medium setting asphalt emulsion, CSS-1 cationic slow setting asphalt emulsion, ETR-1 cationic emulsified rejuvenator oil, SCM-1 and SCM-2 high performance cutback cold mix asphalts. Other commercial products are also available and can be chosen on per need basis.

2.2.4 Cold In-Place Recycling

Cold in-place recycling (CIR) is an in-place method of asphalt pavement recycling. In this method the existing deteriorated pavement is milled or crushed, and then recycled without the application of heat. The recycled pavement is laid down as a base course, and capped with a virgin asphalt layer. The method could be used to:

- Correct wheel ruts
- Correct pot holes, raveling, bleeding and other irregularities
- Eliminate problem of shoving and polished aggregates
- Eliminate longitudinal, transverse and reflective cracks

"Cold in-place asphalt recycling has developed from the early 1980s in Oregon and New Mexico," (Murphy & Emery 1996). Basically CIR can be applied to a pavement of good structural capacity. The method can be used to correct any surface distress. The method is cost effective since no heating is required, which results in considerable fuel savings. "Typically cost savings vary from 25% to 33% over other equivalent alternatives," (ARRA 2008).

CIR was initially divided as full and partial depths recycling. The depth of recycling is generally 3 to 5 inches. The basic construction steps are as following:

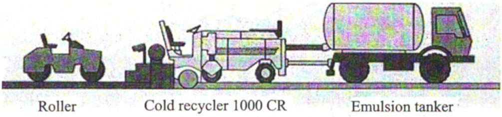
1. Pulverization of the existing pavement to a particular depth
2. Sizing of the RAP
3. Addition of recycling agents and/or new materials
4. Mixing
5. Placement of the recycled mix
6. Compaction
7. Curing
8. Capping with a surface virgin asphalt course

CIR should be done on a pavement that has a good subgrade. If the subgrade is weak, cold milling becomes difficult and the machine may get stuck. If a pavement section selected as a candidate for CIR based on detailed pavement evaluation, the method can be performed by two types of construction units, namely, single pass equipment/single machine/single unit train and single pass equipment train/multi unit train. "The heart of these machines is a large milling drum fitted with tungsten-tipped steel teeth that makes it possible to recycle pavements that include thick asphalt layers," (Lewis & Collings 1999).

In the single unit train, one piece of equipment is used to pulverize and mill the pavement and also to mix the additives. Some single units also pave the recycled mix, whereas some equipments deposit the recycled mix in the windrow. The recycled mix is then paved by a traditional asphalt paver. Figure 2.8 shows a single unit train which uses one unit to mill, recycle and pave. "Stabilizing agents can be injected directly into the mixing chambers while the milling operation is being carried out," (Lewis & Collings 1999).

The advantages of single unit operation include: its suitability for urban projects and areas with short turning radius, high production rate and its simple operation. But the method has the limitation of shallow depth of treatment.

The single pass equipment train/multi unit train has different units or equipments for different tasks. The equipment train basically consists of cold milling machine, crushing and screening unit, mixer and paver, which is followed by the compactor. The units are shown in Figure 2.9 and 2.10. The milling machine pulverizes the pavement to precise depth and width. The screening unit then screens the RAP and limits the maximum aggregate size, which is generally 1.25 inch (ARRA 2001). The oversized material is send to a crusher and re-screened. The screened RAP is then mixed in pugmill with additives. The amount of additives is controlled by computers (microprocessor control), and thus different additives can be added simultaneously and mixed thoroughly in the pugmill. The pugmill ensures good coating and a uniform recycled mix.



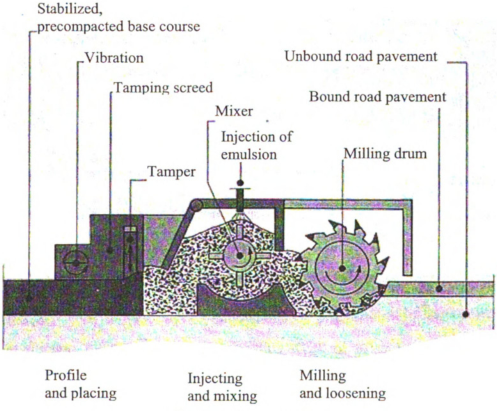


Figure 2.8 Single machine

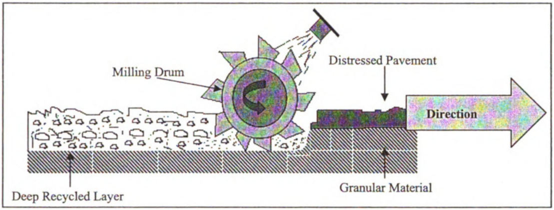


Figure 2.9 Milling process

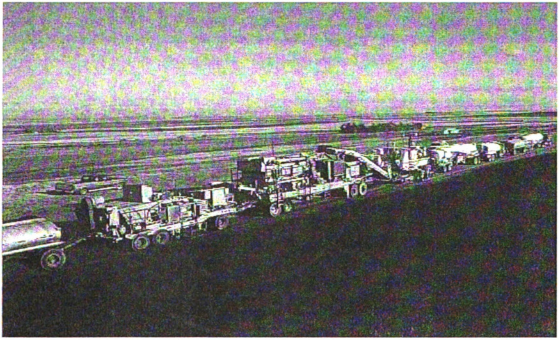


Figure 2.10 Multi unit recycling train (ARRA 2008)

The recycled material is then laid by the paver and finally compacted. Figure 2.11 shows a traditional asphalt paver. Compaction can be performed by pneumatic, static or vibratory rollers or any other combination of rollers. According to ARRA a heavy pneumatic roller of 25 tons and a steel wheel roller are most desirable.

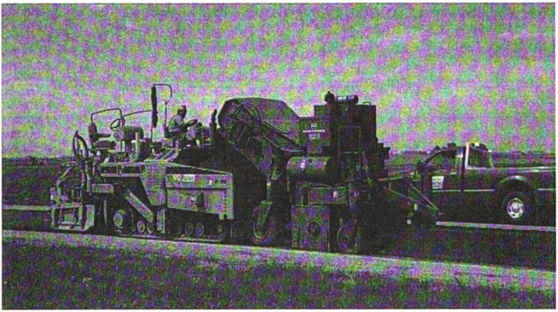


Figure 2.11 Traditional asphalt paver

It is very important to monitor the moisture content of the recycled mix. High moisture can make the mix unstable and low moisture can affect workability. Moisture content is very critical for proper compaction as well. Aeration is generally required in CIR to reduce the moisture and volatile content of the mix, and thus makes it stable. The laying and spreading equipment, which is generally a motor grader, can be used for aeration as well. Curing time is also very critical for the success of a CIR project. New wearing course is laid after the recycled pavement is stabilized after curing. "In a typical CIR process, the curing period requires four to seven days of moderately hot temperatures and no rain events. It is common for the HMA contractor to schedule the HMA overlay two weeks after the CIR is placed," (Heitzman 2007). According to ARRA, the ambient temperature should be 50°F or more during CIR construction. It is advised to cap the cold in-place recycled pavements using virgin asphalt mix. Surface course can be done by chip seat, hot bitumen overlay or any other type of light surfacing.

Chip seal may be done for light traffic and new HMA overlay for heavy traffic. The main purpose of capping is to improve the ride quality.

Additives are needed in CIR to improve the structural capacity of the pavement and its long term performance, by improving the mix properties. New material (binder or aggregate) may or may not be added as per requirements. The basic steps in CIR mix design are as following:

1. Material evaluation of RAP
2. Determination of amount and gradation of virgin aggregates (if required)
3. Selection of type and amount of additive
4. Determination of premix moisture content
5. Testing of trial mixes
6. Establishment of mix

Material evaluation consists of determination of RAP gradation, RAP binder content and aged properties of asphalt binder. Based on the evaluation, the type and amount of new binder is determined, if required. Properties of the binder, like penetration and viscosity are also required for proper mix design. The selection of a recycling agent also depends on the material evaluation. RAP gradation is affected by the fines generated due to milling and underlying layers. Thus new aggregates may be required to provide stability to the mix and to increase the structural capacity.

The most common recycling additive used in CIR is asphalt emulsion. "However, lime slurry, Portland cement, fly ash or a combination of these additives are also used," (ARRA 2008). Sometimes softer grades of asphalt cement and cutback asphalts are used. Emulsions are used readily as they are in liquid state and have good dispersion in the mix. Foamed asphalt cements can be also used. Some amount of water is needed in CIR to facilitate aggregate coating and compaction. This water may be naturally available in the RAP or as part of the recycling agent. It can also be added individually before the recycling agent.

The selection of a particular additive is dependent on the viscosity of the RAP, asphalt demand and time-temperature dependent interaction between the aged asphalt binder and the additives. "At ambient temperature, the softening effect of recycling agent is a time temperature dependent physico-chemical process. The rate of reaction is a function of the properties of the agent and the age of the asphalt cement, and mechanical effects of physical processes such as mixing, compaction, traffic and climatic conditions," (NCAT 1997). As the properties change with moisture, testing of trail mixes is done to determine their initial and final cure properties and moisture sensitivity.

2.2.4.1 Asphalt Emulsion

"Bitumen emulsion is manufactured by emulsifying bitumen and water, using emulsifying agents. Usually the emulsion contains approximately 60% of bitumen and 40% of water," (Lewis & Collings 1999). According to ARRA, asphalt emulsion is added at the rate of 1 to 3% by weight of the RAP. Emulsions can be either anionic or cationic. Anionic emulsions used in CIR are [1]: MS-2 (medium setting), MS-2h, HFMS-2 (high float medium setting), HFMS-2h, HFMS-2s, SS-1 (slow setting) and SS1h. Cationic emulsions are: CMS-2, CMS-2h, CSS-1 and CSS-1h. The selection of a particular asphalt emulsion depends on the aggregate gradation, viscosity requirements of the mix and settling rate requirements. For example medium setting emulsions are used for open or coarse graded material and slow setting for dense graded mix or for mix with more fines. SS can be also used when low viscosity of the mix is required. Polymer modified asphalt emulsion can be used for better strength. "More recently polymer modified asphalt emulsions have been used to take advantage of the greater durability, greater resistance to cracking and lower temperature susceptibility imparted by polymers." (Thomas et al. 2000).

The main advantage of asphalt emulsion is that "it produces a flexible, fatigue resistant layer that is not prone to cracking. Once fully cured, emulsion treated material is resistant to ingress of moisture." (Lewis & Collings 1999). However if the in-situ moisture contents are high, the use of emulsions can be a problematic.

2.2.4.2 Foamed Asphalt

"Foamed bitumen, also referred to as expanded bitumen, is a hot bituminous binder which has been temporarily converted from a liquid to a foam state by the addition of a small percentage of water (typically 2%)," (Muthen 1998). "Controlled flow of cold water is introduced into a hot asphalt stream, passed through a suitable mixing chamber, and then delivered through an appropriate nozzle as asphalt foam," (Kim & Lee 2006). The injection of water in hot asphalt results in instantaneous foaming. This foamed asphalt has a large area and low viscosity which makes it a desirable additive. Figure 2.12 explains the process of asphalt foaming, in which hot asphalt is mixed with cold water and air at particular pressures. This results in foaming and expansion.

Other advantages of foamed asphalt are (Muthen 1998):

- Increases shear strength and reduce moisture susceptibility of granular materials. It is also flexible and fatigue resistant.
- Facilitates construction in cold weather and also light rains

- Allows workability for extended time periods

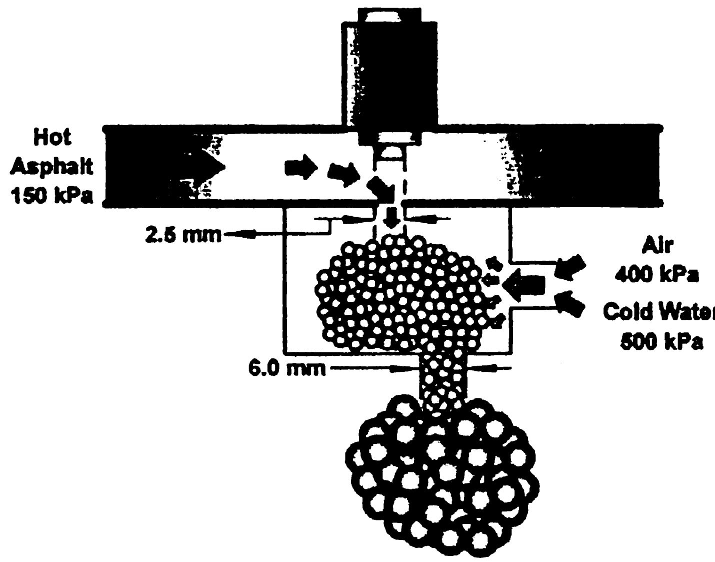


Figure 2.12 Process of asphalt foaming (Kim, 2006)

Expansion ratio and half life are two terms that characterize foamed asphalt. "The expansion ratio of the foam is defined as the ratio between the maximum volume achieved in the foam state and the final volume of the binder once the foam has dissipated. The half-life is the time, in seconds, between the moment the foam achieves maximum volume and the time it dissipates to half of the maximum volume," (Muthen 1998). The properties of foamed asphalt recycled mix depends on foaming water content, temperature, asphalt content, aggregate gradation of RAP and curing. Based on the selected asphalt binder, optimum foaming water content and temperature can be selected to obtain maximum expansion ratio and half life. For example from a laboratory study done by Kim and Lee, 2006, "for the PG 52-34 asphalt binder, the foaming temperature of 170°C was selected with optimum foaming water content of 1.3%." According to ARRA, optimum foamed asphalt content can be determined by Marshall

Stability in dry and wet conditions, indirect tensile strength in dry and wet conditions and resilient modulus testing.

2.2.4.3 Hydrated Lime

Lime can be used as hydrated lime slurry or in dry form as an additive in CIR. According to ARRA the addition of lime varies from 1 to 3% by weight of the RAP. Hydrated lime gives early strength and moisture resistance to the recycled mix. "To prepare 1L of hydrated lime slurry from quicklime, 277.4 g of CaO is mixed with 924.6 g of water. The solid content in the slurry is between 30 to 35%." (Cross 1999). The proportion can vary depending on the design requirements, but the example gives an additive used in Kansas US-283 project. Lime can be also used with asphalt emulsion in the form of hot lime slurry (HLS). HLS results in better material properties of the recycled mix. The addition of increased the unit weight, the tensile strength, and the resilient modulus, regardless of the asphalt emulsion (Cross 1999).

2.2.4.4 Fly Ash

Fly ash is the residue collected from the combustion of coal and is one of the additives in CIR. According to ARRA the application rate for fly ash varies from 5 to 12% of the weight of the RAP. Class C fly ash is recommended by ARRA. Generally fly ash is applied prior to the recycling train by a dry spreading unit. "Fly ash is found to prevent early rutting and raveling, especially when traffic was allowed on the CIR project before the overlay was placed" (Thomas et al. 2000). Thomas et al. also reported that fly ash does not have as good thermal relaxation properties as asphalt emulsions. This may result in stiffer asphalt, which becomes brittle in cold temperatures and causes cracking. "To increase the stability of the CIR mix, the process will require a dry stabilizing agent like cement or fly ash. However, making the CIR mixture stiffer will also reduce the resistance of the mix to reflective cracking. Another option may be the use of a polymer modified asphalt emulsion." (Heitzman 2007). In the study of US-283 in Kansas, the severity of longitudinal cracks with fly ash recycled layer was more than that with asphalt emulsion additive. "This indicates a greater tendency for fatigue associated damage. The tendency of the fly ash section for greater fatigue damage is indicated by the greater stiffness as measured in shear modulus testing" (Thomas et al. 2000). Thus the selection of any additive must be accomplished based on thorough project evaluation to give maximum benefits in the existing conditions.

2.2.4.5 Cement

Cement can be added prior to the recycling unit in a dry state. It can be also added in the form of slurry directly into the mixing unit or can be added by a special cement spreader as part of the recycling unit. According to ARRA cement is generally added at the rate of 1 to 3% by weight of the RAP. The main advantages of using cement are (Lewis & Collings 1999) ease in application as a powder or slurry, generally less expensive than bitumen or emulsion and improvement in resistance to moisture. However its major drawback is the problem of shrinkage cracking. This drawback can be eliminated by using a combination of cement and asphalt emulsion and by proper mix design. This combination however is more expensive than cement or emulsion. Cement can be also used with foamed bitumen as an additive. The choice of an additive depends on the availability, acceptability, material requirement and price.

Table 2.1 provides a summary of the process and benefits of cold milling and the various recycling methods.

Table 2.1 Summary of cold milling and recycling methods

Method	Basic Operation	Benefits
Cold milling	-Controlled removal of HMA by a rotary milling machine	• Can be used up to 10 inches in a single pass • Surface improvements • The roadway is drivable directly after
Hot in-plant recycling	-The existing pavement is heated, softened and then milled or scarified to a desired depth -The material is then mixed and then re-paved onto the road	• Treats 3/4" to 2 inches of pavement at a time • Eliminates hauling expensive • Three main types: surface, remixing and repaving
Cold in-place recycling	-The existing pavement is pulverized and the addition of recycling agents or new materials is added -The material is then mixed and then re-paved onto the road	• Generally treats 3 to 6 inches of pavement at a time • Eliminates hauling expensive • Many additives and procedures can be used based on viscosity of recovered binder: asphalt emulsion, foamed asphalt, fly ash, cement and hydrated lime
Hot in-plant recycling	-Recovered material is transported to the plant where it is mixed at high temperatures with virgin material	• Can be used on any thickness of pavement • Aggregates and binder are broken down to their original states to produce high quality asphalt pavements • Very little modification in plants
Cold in-plant recycling	-Recovered material is transported to the plant where it is mixed with virgin material without the application of heat	• Can be used on any thickness of pavement • Used when high production rates are needed • Used when close control of the mix design is needed

2.3 PAVEMENT EVALUATION FOR THE SELECTION OF ASPHALT PAVEMENT RECYCLING METHOD

Pavement evaluation consists of two activities; distress survey and structural evaluation. The data for the distress survey can be obtained using two methods:

1. Visual examination of the pavement surface by either walking along the road or viewing previously obtained pavement images.
2. Electronic sensors to collect the longitudinal and transverse profile data to assess the ride quality and rut depths, respectively.

The pavement structural evaluation, on the other hand, consists of applying load to the pavement and measuring the pavement response in terms of deflections. The deflection data are then analyzed to determine the moduli of the pavement layers and roadbed soils and to assess the variability of the pavement structural capacity along and across the pavement.

Results of the pavement evaluation are used to determine whether or not the pavement is in need of rehabilitation and for selection of the pavement recycling methods described in Section 2.2. The latter selection is also based on the results of laboratory testing of the asphalt concrete materials to be subjected to recycling.

2.3.1 Surface Distress Survey

In the pavement condition survey, the deteriorated pavement is inspected and the types of distresses are recorded along with their severity and frequency (extent). This is the first step of project evaluation. The distresses in the pavement can be due to inadequate structural design, material deficiencies, traffic effects, construction deficiencies and environmental effects or a combination thereof. Thus it is important to know the problems in the existing pavement, along with the causes of the problem, to be able to remedy it by using appropriate recycling method.

Pavement surface friction and roughness is also recorded as part of the survey. The survey can be carried out manually or by automated survey equipments. The automated devices are better as they give more consistent measurements, have increased safety, less traffic control requirements and increased sample size.

Distresses observed in flexible pavements can be classified as functional or structural distresses. Common functional distresses in flexible pavements are bleeding, corrugation, polished aggregates, raveling, rutting and ride quality. Common structural distresses are alligator or fatigue cracking, block cracking, slippage cracking, and longitudinal cracking (Huang 2004). The causes of these distresses can be load, temperature, moisture, and age. For example, fatigue cracks and rutting is load related, block cracking is age and temperature related, stripping and swelling is moisture related and oxidation is age related distress. More than one cause can also contribute to a particular distress.

Pavement distresses can be also grouped as "surface defects, deformation, cracking, maintenance patching, base/subgrade problems, and poor ride/roughness" (NCAT 1997). Load and non-load related cracks are alligator cracking, transverse, longitudinal and slippage cracks. Rutting and settlement can be due to the asphalt surface, the base/subbase layers and the roadbed soil.

Table 2.2 shows the potential causes for a particular distress. The table is modified from ARRA 2001. A manual like "Techniques for Pavement Rehabilitation" of FHWA, "Distress Identification Manual for the Long Term Pavement Performance

Project" of SHRP, or any other manual can be used to assist in identification of mechanism, causes and severity levels of the observed pavement distresses. The data collected from the survey can be depicted in various forms. "One of the most useful ways is to prepare a strip chart, which shows the various condition deficiencies along the project. These should include the location and severity of surface distress, roughness and surface friction along the project" ("Techniques for Pavement" 1998). All the causes of the distresses observed in the pavement should be used to decide the recycling method. Table 2.3 gives a basic idea of recycling method based on distresses (ARRA 2001).

Additional information is required to quantify the severity and causes of distress. For this purpose cores may be extracted from the place of interest to observe the depth of cracking or stripping. Cores are examined for crack initiation and propagation, as top-down cracks which are caused by the tire-pavement interaction or bottom-up cracks which are due to fatigue. Cores may be also tested in laboratory for further information, if required. Sometimes distresses observed are due to construction, drainage or environmental factors. To analyze these factors the historical data from construction and maintenance records, surface and subsurface drainage and the environment (temperature and rainfall) should be assessed precisely.

2.3.2 Transverse Profile (Rutting) Evaluation

Past research has shown that the width and depth of the rut channel could be correlated to the seat of rutting. For example, a manual published by the Asphalt Recycling and Reclaiming Association (ARRA) (ARRA 2001) states that, "The width of the surface rutting is an indicator of where the pavement weakness may be found. Wide ruts can indicate deeper seated weaknesses, while relatively narrow ruts can indicate

Table 2.2 Potential causes of typical distresses

	Base/Subbase/Roadbed Soil	HMA Properties	Traffic	Environmental	Construction	Pavement Structure
Potholes	2	1	2	2	1	3
Rutting	2	1	2	2	1	2
Fatigue Cracking	1	2	2	3	3	1
Block Cracking	1	1	3	1	3	3
Longitudinal Cracking	3	3	3	3	1	3
Transverse Cracking	3	1	3	3	1	3
Ride Quality	2	2	2	2	2	2

Note: 1 → most likely, 3 → least likely

Table 2.3 Candidate recycling techniques based on distresses

	Milling	HIR	CIR	Thin HMA overlay	Thick HMA overlay	Combination	Reconstruction
Potholes	4	2	1	3	2	1	4
Rutting	2	2	1	4	3	4	1
Fatigue Cracking	4	4	1	4	1	1	1
Block Cracking	4	4	1	4	3	1	2
Longitudinal Cracking	4	2	1	4	3	1	2
Transverse Cracking	4	2	1	4	3	1	2
Ride Quality	3	1	1	3	1	1	4

Note: 1 → most appropriate, 4 → least appropriate upper layer weakness. "

Further, in a study sponsored by the National Cooperative Highway Research Program (NCHRP), a procedure for determining the seat of rutting was developed (NCHRP 2002). The procedure consists of calculating a critical ratio which is defined as the absolute value of the ratio of the total area above the "profile reference line" (positive area) to the total area below the "profile reference line" (negative area). The profile reference line is defined as the line connecting the two endpoints of the transverse profile measurement. Figure 2.13 demonstrates the positive and negative areas, and the profile reference line of the NCHRP procedure.

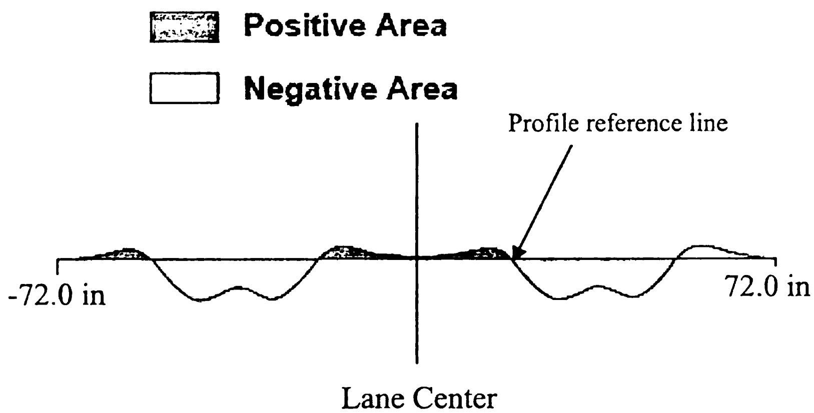


Figure 2.13 Positive and negative areas in the NCHRP procedure

In order to determine the seat of rutting, several calculations have to be made in order to satisfy a set of conditions. The following calculations need to be determined:

$$A = Ap + An \quad (1)$$

$$R = \left|\frac{Ap}{An}\right| \quad (2)$$

$$C1 = (-858.21 \times D) + 667.58 \quad (3)$$

$$C2 = (-1509.0 \times D) - 287.78 \tag{4}$$

$$C3 = (-2120.1 \times D) - 407.95 \tag{5}$$

Where,
A = Total area

Ap = Positive area

An = Negative area

R = Critical ratio

C1 = theoretical average total area for HMA failure, mm^2;

C2 = theoretical average total area for base/subbase failure, mm^2

C3 = theoretical average total area for subgrade failure, mm^2;

D = maximum rut depth, mm

With these calculations known, flow chart show in Figure 2.14 can be used to determine the seat of rutting. Finally, Figure 2.15 can be used as an alternative to Equations 3, 4, and 5.

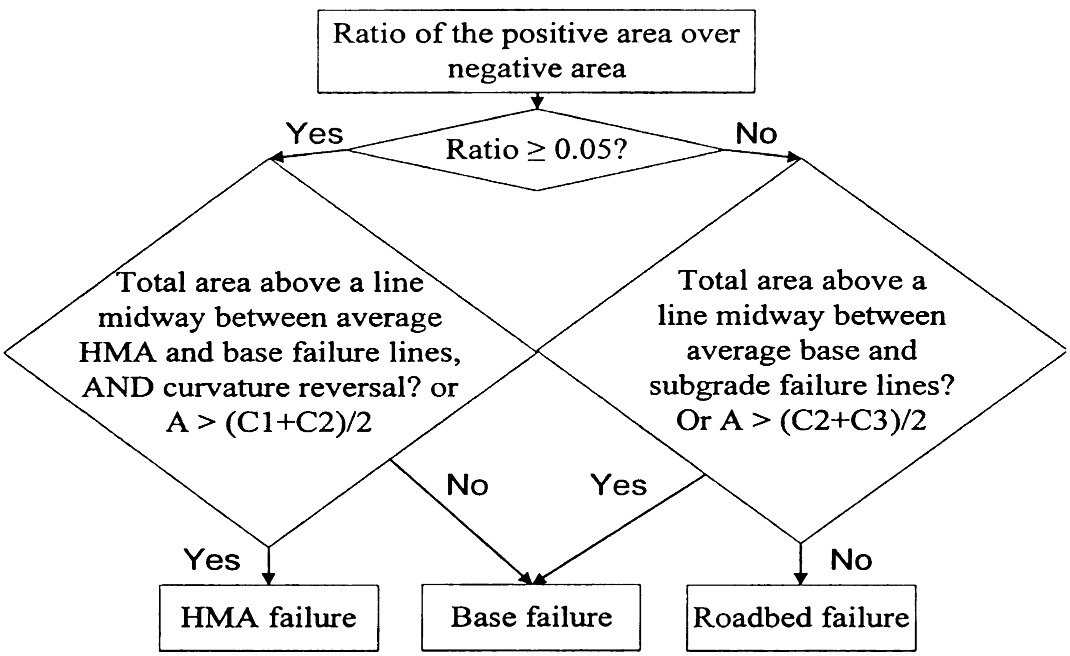


Figure 2.14 Conditions for determining the rutting seat (NCHRP 2002)

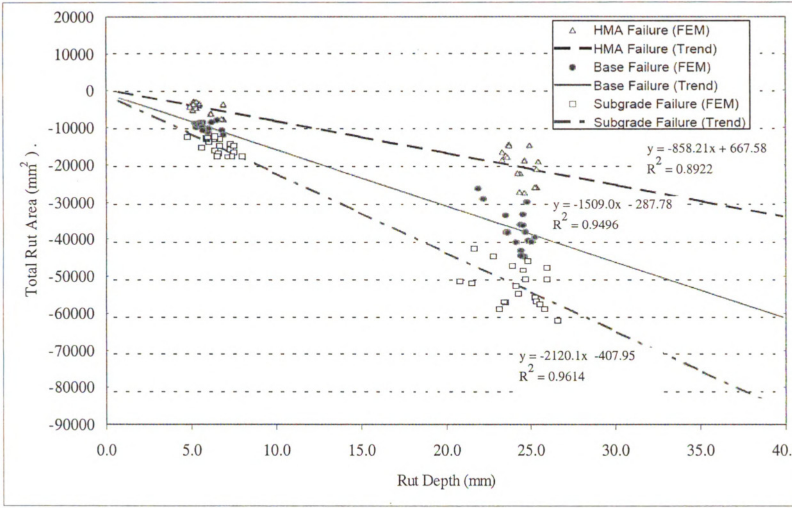


Figure 2.15 Correlation of the type of failure as a function of maximum rut depth and total rut area (NCHRP 2002)

2.3.3 Non-Destructive Deflection Testing (Backcalculation of Layer Moduli)

Destructive or non-destructive field testing can be done to evaluate the strength and properties of the pavement. In case of destructive testing, cores, probe holes or test pits may be used to evaluate the thickness of the different layers of the pavement structure. They are also used to test the strength of the existing pavement by using Dynamic Cone Penetrometer (DCP), Vane Shear test, California Bearing Ratio (CBR) test, etc.

Non-destructive deflection testing (NDT) data can also be used to evaluate the structural capacity of the pavement. "In general, NDT refers to a wide variety of tests that can be executed on any structure in which no physical damage is induced" ("Techniques for Pavement Rehabilitation" 1998). The equipments used can be divided into four categories:

- Static load deflection equipment (slow moving devices)
- Steady-state dynamic load deflection equipment
- Impulse load deflection equipment
- Surface wave propagation equipment

Static deflection devices measure the maximum deflection of the pavement under static or slow moving load. Common devices of this type are Benkelman beam, plate load test and curvature meters. These devices are cheaper as compared to vibratory and dynamic impact devices but are labor intensive and does not simulate moving load. Common steady-state dynamic load deflection devices are Dynaflect, Road Rater and WES heavy vibrator. The equipment applies a static preload and sinusoidal vibration to the pavement. "These devices, which have the ability to measure the deflection basin, as opposed to a single point, are generally easy to operate, acquire data quickly, and have good repeatability, but they are expensive. In addition, the test loads applied to the pavement are generally much lower than actual wheel loads, thus extrapolation of data is necessary." (ARRA 2001).

Impulse load defection or Dynamic impact devices consists of equipments like Falling Weight Deflectometer (FWD). "A weight is lifted to a given height on a guide system and then dropped onto a buffering plate on the pavement. A transient impulse force, which can be changed by varying the magnitude of the falling weight or by varying the drop height is generated" ("Techniques for Pavement" 1998). This method simulates moving wheel load better than any other method. It is very repeatable and provides deflection basin information. The data collected is fast and automated. Figure 2.16 shows a typical FWD. Commercial FWD devices are Dynatest, KUAB, JILS and Phonix falling weight deflectometers. Cores are typically collected from the pavement in order to get an accurate estimate of the thickness of the AC layer.

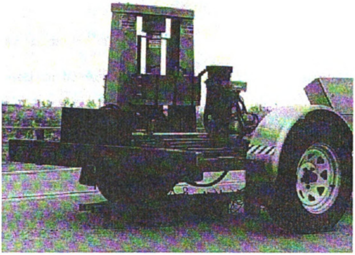


Figure 2.16 Dynatest FWD

FWD records the response of the pavement due to load which can be used to backcalculate the modulus of different layers of the pavement. The subgrade resilient modulus and the presence of stiff layer can be also estimated based on the data collected. Software like MICHBACK, MODULUS, etc can be used for backcalculation. The backcalulated modulus values are indicators of the stiffness of the pavement layers. The information could be used to analyze the causes of observed distresses and to select or design the appropriate pavement rehabilitation method. One disadvantage of the FWD is that it is the most expensive NDT equipment.

The last method is Surface wave propagation. It is based on propagation of stress wave in elastic media. The method uses the concept that the velocity of the stress waves is a direct indicator of the stiffness of the material of different pavement layers.

The information can be NDT survey is generally done prior to destructive testing, in order to identify weak pavement sections. If possible, NDT should be conducted in various seasons, to be able to estimate the layer moduli at different moisture contents. The temperature should be recorded at the time of testing, to be able to obtain the value of the moduli at a standard temperature. Generally NDT are conducted at an interval of 30-150 m along the project.

The NDT data can be used along with distress, material, drainage and subgrade soil tests to determine the uniformity of the project. The recorded deflection values could be used to generate deflection profiles, which give estimates of the locations of stiff and soft areas along the project. This information is important to section the project and to decide the sampling points. NDT data are extremely valuable in assessing; the engineering properties of the various pavement layers, the uniformity of the structural capacity of the pavement section and in identifying weak pavement layers (Haider et al. 2008). If the NDT are conducted at different load levels, the linearity of the material response to load can be analyzed. Evaluation of the load carrying capacity of the subgrade is also very important as it supports the pavement structure.

2.4 MATERIAL CHARACTERIZATION

In order to understand the characterization of RAP, the affects of the percent recovered asphalt binder and aggregate contents on the recycled asphalt mixtures must be understood. There are many laboratory tests that can be conducted to evaluate the characteristics of the recycled binder and aggregate. Also, these laboratory tests can be used to evaluate combinations of the two and how they behave with the addition of virgin materials. It is very important to characterize the behavior of recycled mixtures because the addition of RAP changes the mechanistic properties of the pavement (Daniel and Lachance 2004). The understanding of RAP characteristics is the first step in its proper use.

2.4.1 Material Sampling

Material sampling is the gateway to material characterization. Material is sampled throughout the recycling process and is broken down in this literature review into two sections: virgin aggregate/binder and reclaimed pavement.

Virgin material mixed with the reclaimed material needs to be tested to analyze the properties. Virgin aggregates must be carefully collected at quarries because the aggregate stockpiles may be segrated. Proper methods should be used when sampling from virgin or recovered material stockpiles which is outlined further in stockpiling of RAP found later in this section. Virgin binder samples must also be collected from petroleum refineries. "Almost all paving asphalt binder used today is obtained by processing crude oil" (Roberts el al. 1996). Once these materials are collected commencement of laboratory tests can be utilized to determine their characteristics.

The second type of sampled material in the recycling process is the reclaimed pavement. For this material, there are two types of sampling, roadway sampling and stockpile sampling. Proper road sampling will ensure that necessary laboratory tests can be executed and the material characteristics in question can be determined. Proper stockpile sampling can assure the engineer that the sample is a representative of the type of RAP expected in the HMA mix containing RAP.

Roadway sampling can be done by taking cores, slabs or crushed material from the existing pavement. However, a widely excepted and used method is coring. Cores should be taken randomly throughout the project in order to have a better representation of the existing material. The amount of cores that need to be extracted are relative to the size of the project, however it is recommended that at least three cores needs to be taken every lane-mile (McDaniel & Anderson 2001). The cores can be used to test for aggregate gradation, asphalt content and binder properties. These recovered material properties give an accurate indication of the type of RAP that will be used for an in-place recycling method. If milling is used in the recycling process the RAP gradation is likely altered. It has been a common practice to develop degradation factors based on experience with local equipment and materials.

Stockpile sampling must be taken with caution because of inconsistencies due to variable pavement condition, settling and pile mixing. Stockpiled RAP contains anything that existed on the pavement before removal. Additional material from maintenance such as sealant is often found in the stockpiled materials. Once a stockpile is established, it often becomes stagnate for a period of time allowing the stockpile to settle. Settling causes coarse and fine material to segregate within the pile. It is recommended to remove 6 inches from the outer surface of the pile before any samples are taken.

Another cause of inconsistencies of stockpiles is mixing piles from different projects. In order to save space at plants, plant owners tend to mix RAP's together. This practice is highly discouraged because it causes too much variability. Material variability causes many complications in properly designing HMA mix with RAP. Variability also makes it very hard for contractors and plant owners to meet specifications of mix designs. Proper stockpiling practices along with correct sampling will enable the engineer to obtain the most accurate information of the RAP (McDaniel & Anderson 2001).

In order to design asphalt mix involving RAP, the existing aged binder properties and aggregate need to be quantified. These properties help in selecting the appropriate virgin binder grade and its quantity. Also to study the changes occurring in asphalt as it hardens in the hot-mix plant or roadway and to determine the asphalt content of cores and mixes, asphalt must be extracted from the aggregates. Once the asphalt is extracted, the remainder aggregate can then be tested for its gradation and physical properties, both of which are essential to RAP mix design.

The main obstacle to recycling asphalt binder is the lack of developed science of recycling agent composition and, as a result, optimum agents are not available. An excellent recycling agent should not only be able to reduce the viscosity of the aged material, but it must also be able to restore compatibility (Bullin et al. 1995). Furthermore, the properties of the old material and recycling agent must be compatible to give both good initial properties and long-term aging characteristics, and thus must be understood. Therefore, determination of existing aged binder properties in RAP or cores requires extraction and recovery methods for designing a proper asphalt mix design. This is achieved almost exclusively currently by either the centrifuge or reflux procedure as specified in ASTM D2172. Several studies of solvents and extraction/recovery techniques have been completed (Burr et al. 1990, Burr et al. 1991, Cipione et al. 1991, Burr et al. 1993, Burr et al. 1994, Peterson et al. 1994, Chaffin et al. 1995, Lin et al. 1995). All these studies indicate that the asphalt extraction and recovery procedure is vital in determining the rheological properties of aged binder.

A thorough literature review has been done to determine which extraction and recovery procedure has minimal effect on the extracted asphalt (McDaniel et al. 2000). There are many factors affecting extraction and recovery procedures including selection of solvent, sample size testing time, and testing precision. Extraction procedures for removing asphalt from aggregate in an asphalt mixture typically follow the methods listed in ASTM D2172, Quantitative Extraction of Bitumen from Bituminous Paving Mixtures. There are five test methods listed as Methods A through E. Method A (Centrifuge Extraction) is the most common extraction procedure used by asphalt testing laboratories. Many laboratories also use Method B (Reflux Extraction). Methods C and D are variations of the reflux extraction methods. Method E (Vacuum Extraction) is an option as a third extraction technique.

Recovery of asphalt binder from solution can be obtained by using ASTM D1856 standard. This procedure is based on Abson Method. This test was introduced in 1933 and has been the principal recovery technique used by testing laboratories. Since the

1970's a second recovery procedure has been introduced by some testing laboratories. This ASTM D5404method is titled, Recovery of Asphalt from Solution Using the Rotavapor Apparatus, named for its use of a rotary evaporator as the recovery equipment.

Unfortunately, all previous methods show poor precision and unknown accuracy in measuring the chemical and physical properties of the extracted asphalt (Burr et al. 1993). It is likely that the error in extracted asphalt properties originates from the following problems:

1. Asphalt is not completely or consistently extracted from the aggregate;
2. Residual solvent is left in the asphalt after recovery; and
3. Reaction of asphalt while in solution-solvent aging-can alter properties during both extraction and recovery.

As a part of the SHRP, researchers at Texas A&M University explored asphalt extraction and recovery procedures. Their research led to the development of a new extraction process with a modified recovery procedure. Several reviewed papers detailed the finding of their research (Burr et al. 1990, Burr et al. 1991, Cipione et al. 1991, Burr et al. 1993, Burr et al. 1994, Peterson et al. 1994). These studies provided updated information regarding current (1990s) extraction and recovery techniques while discussing the development of the SHRP extraction and recovery method. This revised method is the preferred method to extract and recover the asphalt binder because the procedure results in less severe changes to the binder properties. The SHRP researchers, in evaluating the effects of solvent hardening, examined several solvents. Their conclusions were that solvent hardening appears to occur to roughly the same degree in most solvents, although they would expect somewhat lower hardening in toluene because it is a poorer solvent that leads to more aggregation or association in solution (Burr et al. 1991). The researchers indicated that while trichloroethylene with 15% ethanol is the most powerful solvent for extracting asphalt, toluene with 15% ethanol works well and has safety advantages (Cipione et al. 1991).

The quantity of RAP in the virgin aggregates varies based on several factors. In addition, the amount of asphalt binder to rejuvenate the asphalt mixture requires a detailed engineering analysis. Therefore, to incorporate different materials in diverse locations involving various types of recycling approaches (as mentioned above); there are numerous mixture design methods were developed. These asphalt mixture design methods involving RAP are presented in the next section 2.5 of this thesis.

2.4.2 Characteristics of Asphalt Binder

Three types of asphalt binder characteristics will be discussed in this literature review: virgin, recovered and blended binder. The characteristics of these three binders are needed for the design of RAPs. "In pavement applications, asphalt binders are specified by their physical properties...the most important physical properties to the engineer are its rheological characteristics" (Lavin 2003). Correctly determining the physical properties and interactions between recovered and virgin binder (blended) lead to a better understanding of the performance of RAP.

2.4.2.1 Virgin Binder

Virgin binder is used in recycled pavements for two purposes, increasing the total asphalt content in RAPs and blending with the aged binder to meet desired specifications (AI MS-20 1986). In order to achieve these two purposes, the characteristics of the binder must be ascertained. The necessary characteristics are established by the type of design method being used. For this report, the traditional (Marshall) and new (Superpave) mix design methods are studied.

For the Marshall mix design method penetration grading and viscosity grading are used. Penetration grading is a very traditional method of testing the consistency of asphalt binder. There are five standard grades: 40-50, 60-70, 85-100, 120-150, and 200-300, with the lowest (40-50) equating to the hardest binder and the highest (200-300) equating to the softest binder. Penetration grading testing procedures are outlined in the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM) testing manuals which can be seen in Table 2.4. Penetration grading is a measure of the distance that a 100g needle penetrates a binder sample at 77°F in 5 seconds. Penetration grade requirements for asphalt binder specifications can be found in Asphalt Institute's "Principles of Construction of Hot-Mix Asphalt Pavements (MS-22)," 1983 on pg. 16. However, because of its empirical nature penetration grading is inadequate with modern technology (AI MS-4 1989).

A more fundamental binder test that is also used in the Marshall mix design method is viscosity grading. When testing viscosity it is important to remember the temperature-viscosity characteristic: "The degree of change in viscosity of asphalt cement with a change in temperature is a very important characteristic. The temperature susceptibility of the asphalt cement is defined as the change in the flow properties of the material with a change in temperature" (USACE 1991). Viscosity grading testing procedures are summarized in Table 2.4. Grading based on the original asphalt cement (abbreviated AC in viscosity charts) is usually conducted at a temperature of 60°C (140°F) but also can be tested at 135°C (275°F) for a minimum viscosity. Viscosity grading can also be based on age residue (abbreviated AR the in viscosity charts) Requirements for the original and residue asphalt binder viscosity grading at 60°C (140°F) can be seen in Asphalt Institute's "Principles of Construction of Hot-Mix Asphalt Pavements (MS-22)," 1983 on pg. 14-15.

Table 2.4 Required test procedures for asphalt binder (AI MS-4 1989)

Test	Test Method	
	AASHTO	ASTM
Viscosity at 60°C (140°F)	T202	D2171
Viscosity at 135°C (275°F)	T201	D2170
Penetration	T49	D5
Flash Point (C.O.C.)	T48	D92
Thin Film Oven Test	T179	D1754
Rolling Thin Film Oven Test	T240	D2872
Ductility	T5	D113

For the Superpave mix design method, the asphalt binder characteristics are determined using a performance-based method. Binders for the Superpave method are selected on the basis of the climate and traffic in which they are intended to serve (AI SP-2 2001). "The Superpave Performance Grade (PG) binder specifications differ from the penetration and viscosity grading systems in that they are based on physical properties that can be directly related to field performance by engineering principles. The basis of the PG binder specifications is that fundamental properties of the asphalt binder are measured at actual pavement temperatures where the critical pavement distress modes occur" (Lavin 2003). In order to determine these properties multiple tests are performed; a list of these tests and purpose can be seen in Table 2.5. More in-depth details of the binder test procedures can be found in Asphalts Institutes' "Superpave Asphalt Binder Specification SP-1." After the tests are performed, the grade (PG) of the binder can be determined.

Table 2.5 Superpave binder test equipment and purpose (AI SP-1 2003)

Equipment	Purpose
Rolling Thin Film Oven (RTFO)	Simulate binder aging (hardening) characteristics
Pressure Aging Vessel (PAV)	
Dynamic Shear Rheometer (DSR)	Measure binder stiffness and elasticity properties at high and intermediate temperatures (G*, λ)
Rotational Viscometer (RV)	Measure binder viscosity at high temperatures
Bendig Beam Rheometer (BBR)	Measure low temperature stiffness and failure properties
Direct Tension Tester (DTT)	Determining the failure properties of asphalt binder at low temperatures

An example of a PG is 64-22, which specifies that the binder must meet high-temperature physical property requirements up to 64°C, and low-temperature physical properties must be met at least down to -22°C. The most common paving grades in the U.S. are PG 64-22, PG 70-22, PG 76-22, PG 58-22, PG 64-28, PG 58-28, and PG 52-34 (AI SP-2 2001). Designers will specify the binder grade based on the climate factors (the seven-day average high and one-day average low temperatures), traffic conditions and desired reliability.

2.4.2.2 Recovered Binder

After the extraction process is complete on a recovered HMA sample the characteristics of the binder can then be determined. Overtime, the binder has aged within the pavement which has caused the binder to harden. If less than 10-20% (depending on recovered binder grade) of RAP is used for a new mix design, the aged binder does not influence the virgin binder and therefore can be ignored (McDaniel et al. 2000). However, if the RAP percentage increases from the 10-20% threshold, the recovered binder begins to have more of an effect on the virgin binder and manipulates the characteristics of the new HMA mix.

Physical properties and critical temperatures of the recovered binder need to be determined just as it was for virgin binder. Characteristics found in Section 2.4.3.1 for virgin binder can be also applied to recovered binder. Recovered binder that will undergo any chemical changes such as additives found in Section 2.2.3 and 2.2.4 will need to be characterized after the additives are added. Emulsified asphalts impart an electric charge on the droplets of asphalt binder which are separated from the emulsifying agent (usually water). This causes the droplets of asphalt to either be electro-negatively charged (anionic) or electro-positively charged (cationic). These binders are further graded according to their "setting" rate, which is rapid, medium or slow setting (Roberts 1996).

2.4.2.3 Blended Binder

Once the properties of the virgin and recovered binders are determined, the characteristics of the blended binder (virgin mixed with recovered) must be determined. This is a very important step in the RAP design process which leads to an understanding of the necessary quantity of RAP to be added along with the required virgin binder grade to meet the design specifications. The blended binder can either be characterized from lab samples or from samples taken after recycling. The samples can then be subjected to the same characterization process that is discussed in Section 2.4.3.1.

There have been many concerns with the actual blending between binders. Questions have been brought in regards to whether or not the recovered binder is 100% working. Research conducted by Al-Qadi et al. (Al-Qadi et al. 2009) examined what extent of RAP binder was being blended at RAP percentages of 20% and 40%. They compared mixes containing normally added RAP to those specifically prepared with a prescribed amount of working stiff RAP binder combined with the virgin binder. The final data revealed that a high percentage of the RAP binder is working. Al-Qadi et al. stated that the assumption of 100% working binder in practice is acceptable. They did recommend that at 40% RAP double bumping of the binder may be needed.

2.4.3 Characteristics of Aggregates

There are also three types of aggregates in the recycling process that need to be characterized: virgin, recovered and blended aggregates. "The physical characteristics of aggregate dictate the final performance of the asphalt pavement. The physical characteristics are very important to the designer and can be apart of the pavement's design and specifications" (Lavin 2003). Physical characteristics of aggregates include gradation, density, toughness, surface texture, particle shape and absorption.

2.4.3.1 Virgin Aggregates

Virgin aggregates are mixed with recovered aggregates so that the desired gradation specification is accomplished for the RAP. The desired specification can be reached with more reliability if proper stockpiling and sampling practices are used. Aggregates should be stockpiled on clean surfaces and precautions should be taken in order to prevent intermixing of adjacent stockpiles. Also, if an aggregate contains a range in particle sizes (both coarse and fine) materials, it is recommended to separate the coarse and fine material and then blend in the proper proportions (AI MS-22 1983). Samples of the aggregate should be taken randomly throughout the stockpile. It is a good practice to remove 6-12 inches from the surface of the pile to get a sample.

Once a sample of the aggregate is obtained, gradation and the maximum and nominal maximum aggregate sizes could be determined using sieve analyses. "The gradation is the most important physical property that an aggregate can contribute to the performance of an asphalt pavement" (Lavin 2003). Typical gradations for aggregate in HMA can be seen in Table 2.6. Procedures for an aggregate sieve analysis can be seen in AASTHO T27.

Table 2.6 Typical specifications of hot mix asphalt

Sieve Size	**Mix Designation and Nominal Maximum Size of Aggregate**				
	1.5 inch (37.5 mm)	1 inch (25.0 mm)	0.75 inch (19.0 mm)	0.5 inch (12.5 mm)	3/8 inch (9.5 mm)
	Total Percent Passing (by weight)				
2" (50 mm)	100	-	-	-	-
1.5" (37.5 mm)	90-100	100	-	-	-
1" (25 mm)	-	90-100	100	-	-
3/4" (19 mm)	56-80	-	90-100	100	-
1/2" (12.5 mm)	-	56-80	-	90-100	100
3/8" (9.5 mm)	-	-	56-80	-	90-100
No.4 (4.75 mm)	23-53	29-59	35-65	44-74	55-85
No.8 (2.36 mm)	15-41	19-45	23-49	28-58	32-67
No. 16 (1.18 mm)	-	-	-	-	-
No.30 (0.60 mm)	-	-	-	-	-
No.50 (0.30 mm)	4-16	5-17	5-19	5-21	7-23
No.100 (0.15 mm)	-	-	-	-	-
No.200 (0.075 mm)	0-5	1-7	2-8	2-10	2-10
Asphalt Cement, weight percent of Total Mixture	3 to 8	3 to 9	4 to 10	4 to 11	5 to 12

The specific gravity of the aggregate must also be determined. It provides a means of expressing the weight-volume characteristic. Also the specific gravity is an aid in calculating the percentage air voids in the final HMA mixes. Since the aggregates contain pores it affects the amount of asphalt that is needed. There are three types of specific gravities that are needed: bulk, apparent and effective specific gravity (AI MS-22 1983). Another characteristic that is needed is the absorption capacity. If the aggregate has a high absorptive capacity it will absorb the binder which was intended to bond the aggregate particles together.

2.4.3.2 Recovered Aggregates

When binder is extracted from the recovered HMA sample, the recovered aggregate is then tested to determine its characteristics. "If a solvent extraction was used to recover the aggregate, the aggregate should be thoroughly dried in an oven or in front of a fan before testing. If the ignition oven was used, the aggregate should be completely cooled before handling" (McDaniel et al. 2000). The gradation of the recovered aggregate is an extremely important characteristic of RAP. The gradation for recovered aggregates degrades during the RAP process either due to milling or mixing. Proper degradation factors, as discussed in Section 2.4.1, need to be used based on local equipment and practices.

2.4.3.3 Blended Aggregates

Blended aggregate samples can either be taken from cores after the recycling process or samples taken during the recycling process that were compacted in the laboratory. The sample of RAP then undergoes the binder extraction process as outlined in Section 2.4.2, and the aggregate's characteristics are then determined as in Section 2.4.4.1.

2.4.4 Characteristics of Hot Mix Asphalt Mixture

It is necessary to determine the characteristics of the asphalt mixture by testing the mixtures response to load, deformation and environment at various rates of loading and temperatures (Roberts et al. 1996). Hence, physical and mechanical tests are typically conducted to determine the physical and engineering properties of the mixtures. Physical properties include density, air voids, voids in mineral aggregate, voids filled with asphalt cement, and aging characteristics. Mechanical properties include Marshall stability and flow, dynamic modulus, resilient modulus, flexural stiffness modulus, indirect tension, creep and moisture susceptibility. The following paragraphs will define and depict these characteristics and identify what methods utilize them.

In the Marshal method, asphalt mixture samples are heated to a specified temperature and then compacted using the Marshall compaction hammer (AASHTO T245) "Resistance to Plastic Flow of Bituminous Mixtures Using the Marshall Apparatus." Compacted test samples are then subjected to the following tests (AI MS-2 1995):

- Bulk specific gravity determination
- Stability and flow tests
- Density and void analysis

The bulk specific gravity test is done in accordance to ASTM D1188, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using Parrafin-Coated Specimens," or by ASTM D2726, "Bulk Specific Gravity of Compacted Bituminous Mixtures Using

Saturated Surface-Dry Specimens." Once the specific gravity has been determined the stability and flow tests are performed. A step by step procedure for the stability and flow tests can be seen in the Asphalt Institute's Manual Series No.2 (AI MS-2 1995). Once the stability and flow test are completed, the density and void analysis can be done. Again a detailed procedure can be found in the Asphalt Institute's Manual Series No.2 (AI MS-2 1995).

The final step in the Marshall design method is interpreting the results from the tests. Individual graphs must be prepared with a best fit curve for the following relationships (AI –MS-2 1995):

- Stability vs. asphalt content
- Flow vs. asphalt content
- Unit weight of total mix vs. asphalt content
- Percent air voids (V_a) vs. asphalt content
- Percent voids filled with asphalt (VFA) vs. asphalt content
- Percent voids in mineral aggregate (VMA) vs. asphalt content

Figure 2.17 shows an example of these relationships. These graphs are then used to determine the design asphalt content of the mix.

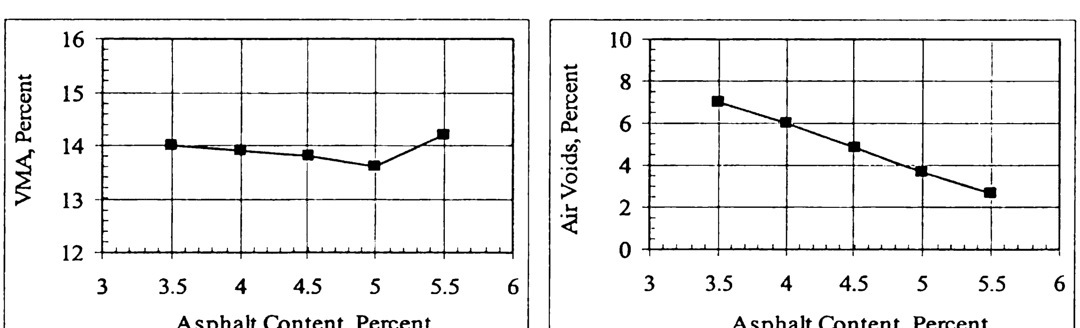


Figure 2.17 Example of test property curves for hot-mix design data for Marshall method

"The Superpave method, like other mix design methods, creates several trial aggregate-asphalt binder blends, each with a different asphalt binder content. Then, by evaluating each trial blend's performance, an optimum asphalt binder content can be selected" (PTC 2009). For the Superpave method, a gyratory compaction method is used rather than the Marshall hammer (Asphalt Institute's SP-2) "Superpave Mix Design." Once the sample is compacted a series of volumetric tests will be conducted to determine the following properties (AI SP-2 2001):

- Bulk specific gravity
- Apparent specific gravity
- Effective specific gravity
- Voids in mineral aggregate
- Effective asphalt content
- Air voids
- Voids filled with asphalt

The trial mixes will also undergo a series of performance tests to determine their mechanical properties that were listed above. However, the most important one, the simple performance test (SPT) is a confined dynamic modulus test. The following section illustrates how characteristics of RAP using the new Superpave method have been addressed by researchers.

2.4.5 Characteristics of Recycled Asphalt Pavements Using the Superpave Method

The literature lacks information and published research results concerning the design of RAP using the Superpave binder or mixture test protocols. However, one can learn from past studies where some of the Superpave procedures were used or where other specifications and test methods were employed to assess the performance of RAP. These included Kandhal et al. (Kandhal et al. 1995), Tam et al. (Tam et al. 1992) and Sargious and Mushule (Sargious & Mushule 1991). Unfortunately the reported results are contradictory at best. For example, Tam et al. found that mixes with RAP are less resistant to thermal cracking than virgin mixtures. Kandhal et al. found that there was no significant difference in cracking performances between recycled and virgin mixtures, while Sargious and Mushule found that the recycled mixture performed better than the virgin mixture in terms of cracking. The mixture behavior is responsive to binder properties at low, intermediate and high temperatures. A binder selected to perform well at high temperatures may not necessarily perform well at low temperatures. The above noted studies were conducted using penetration or viscosity graded asphalts. The Superpave binder system gives a tool to investigate the binder effects over a range of temperatures and aging conditions and should, therefore, allow a better selection of the appropriate binder blend (RAP + virgin) for a given situation. The study by Sargious and Mushule did use a softer asphalt for the recycled mix than for the control mix, which may have rejuvenated the RAP, resulting in the reported improved performance. Resilient modulus has been used in many studies to evaluate RAP mixtures (Epps et al. 1977, Sargious & Mushule 1991, Noureldin & Wood 1900, Amirkhanian & Williams 1993, Terrel & Fritchen 1977). However, variability of the test results, especially between labs, has posed problems in interpreting the data.

Many studies (Hossain et al. 1993, Epps et al. 1977, Paul 1996, Servas et al. 1985) documented the fact that recycled mixtures can perform at least as well as virgin mixtures. Improved extraction, recovery and binder testing procedures may allow even better selection of the right binder for a recycled mixture leading to improved performance.

A variety of methods can be used to simulate mixture aging in the laboratory. Bell et al. (Bell et al. 1994) developed the short and long-term oven aging procedures recommended in Superpave. Ruth and Roque (Ruth & Roque 1995) used the long-term oven aging procedure to fabricate RAP in the laboratory. This type of long-term aging can be used, but testing of actual plant-produced and/or field aged materials is the preferred method. The serious disadvantage for using long-term oven aging when designing a mixture with RAP, is that the mix design process can then be delayed by days or weeks.

Several studies have been directed at examining changes in aggregate properties before and after solvent extraction or burn-off in an ignition oven. Such studies have application to RAP mixtures as well, where the original aggregate properties may be unknown. A study by NCAT (Mallick et al. 1998) showed that there is a significant difference between the virgin and recovered bulk specific gravity (G_{sb}) for three tested aggregates (lime rock fine, trap rock and granite) before and after burn-off. The G_{sb} decreased by 0.021, 0.035 and 0.015 for the above aggregates respectively. The Virginia Transportation Research Center (Prowell & Carter 2000) compared the virgin G_{sb} calibration factor using known asphalt content, and the effective specific gravity (G_{se}) was calculated using an asphalt content determined with the ignition furnace for six aggregates types. The calculated VMA values using the G_{se} were always larger than the specified values. The differences ranged from 0.01 to 0.43 percent.

Chehab and Daniel (Chehab & Daniel 2006) studied the sensitivity of the assumed binder grade on performance prediction of RAP mixtures by utilizing the Mechanistic-Empirical Pavement Design Guide (M-EPDG) software to predict the performance of a specific flexible pavement structure with a RAP modified HMA surface layer. In their study, the RAP content and the effective binder PG grade were the main variables. They concluded that the assumed PG binder grade, particularly the high temperature grade, for the RAP mixtures has a significant influence on the predicted amount of thermal cracking and rutting performances. They added that the predicted performance is sensitive to changes in assumed the PG grade. The results emphasized the importance of determining the effective binder grade of RAP mixtures.

Daniel and Lachance (Daniel & Lachance 2004, Daniel & Lachance 2005) examined how the addition of RAP changes the volumetric and mechanistic properties of asphalt mixtures. In their research, a 19 mm Superpave mixture containing 0% RAP was used as the control mixture to compare the properties of recycled mixtures containing 15, 25, and 40 percent RAP. Testing included dynamic modulus in tension and compression, creep compliance in compression, and creep flow in compression. Using the time-temperature superposition principle, dynamic modulus and creep compliance master curves were constructed to describe the behavior of each mix over a range of temperatures. The VMA and VFA of the RAP mixtures increased at the 25 and 40 percent levels, and there was also an influence of pre-heating time on the volumetric properties. The dynamic modulus of the processed RAP mixtures increased from the control to 15% RAP level, whereas, the 25 and 40 percent RAP mixtures had dynamic modulus curves similar to the control mixture in both tension and compression. The creep compliance curves showed similar trends. A combination of gradation, asphalt content and volumetric properties was identified as the cause of these trends.

Huang et al. (Huang et al. 2005) conducted a laboratory study in which the blending process of RAP with virgin mixture was analyzed through controlled experiments. One type of screened RAP was blended with virgin (new) coarse aggregate at different percentages. A blended mixture containing twenty percent of screened RAP was subjected to staged extraction and recovery. The result from this experiment indicated only a small portion of the aged binder in RAP actually participated in the re-mixing process while other portions formed a stiff coating around RAP aggregates and the RAP functionally acted as "composite black rock".

Huang et al. (Huang et al. 2004) also evaluated the fatigue resistance of HMA mixtures containing No. 4 sieve screened RAP. A typical surface mixture commonly used in the state of Tennessce was evaluated at 0, 10, 20, and 30 percent RAP contents. The fatigue characteristics of mixtures were evaluated through indirect tensile strength, semi-circular bending (SCB) and semi-circular notched fracture resistance tests. It was found that long-term aging does influence the fatigue ranking for mixtures containing different percentages of RAP. Generally, the properties of the long-term aged mixtures resemble the of field mixtures that have been in-service for several years. Also inclusion of RAP into the limestone surface mixture generally increased the tensile strength, reduced the post-failure tenacity, increased mixture's modulus (stiffness), and reduced the viscosity characteristics. They reported that the inclusion of RAP in mixtures resulted in:

- Increases in the SCB fatigue life at stress levels above 20 percent of the SCB tensile strength.

- Improved the mixtures' resistance to fracture failure.
- Mixtures made with less than 20 percent RAP had very limited influence on the mixture stiffness and indirect tensile strength characteristics.
- Mixtures containing 30 percent RAP tended to significantly change the fatigue cracking characteristics.

Kim et al. (Kim et al. 2004) compared the laboratory responses of engineered CIR emulsion and foamed asphalt as a binder for RAP materials collected from the CIR recycling project on US-20 in Iowa. Based on the visual observation of laboratory specimens, as expected, the engineered CIR emulsion coated the RAP materials better than the foamed asphalt. Foamed asphalt instead created a mastic mixture structure to provide better bonding of RAP materials. They found that foamed asphalt mixtures obtained higher density than the engineered CIR emulsion mixtures at the same compaction effort. However, after four hours of curing at the room temperature, the engineered CIR emulsion mixtures showed less raveling than the foamed asphalt mixtures. Both Marshall Stability and indirect tensile strength of foamed asphalt mixtures were about the same as those of engineered CIR emulsion mixtures. However, Marshall Stability and indirect tensile strength of the vacuum-saturated wet samples of foamed asphalt mixtures were lower than those of engineered CIR emulsion mixtures.

Pereira et al. (Pereira et al. 2004) used four point bending and repeated simple shear test at constant height (RSST-CH) to determine the fatigue and rutting resistance in the laboratory, respectively. They concluded that HMA mixtures with reclaimed bituminous material (RBM) having 5% binder content exhibited the best fatigue resistance while for the same recycled mixture at 4.5% binder content showed increased rutting resistance.

Isaa et al. (Issa et al. 2001) conducted a study to characterize the behavior of cold recycled asphalt pavement rejuvenated with high float emulsion and portland cement, producing a cement-emulsion mix. The main objectives were to investigate how the behavior of RAP mixes was affected by the addition of cement and to find the optimum emulsion and cement contents. Test samples were prepared with four different cement contents, three emulsion contents, and one free water content. It was concluded that Hveem stability values increased with cement content but decreased with emulsion content. The addition of Portland cement affected the Hveem stability value of samples cured under soaked conditions more than those cured under dry conditions. Samples prepared with 2 percent emulsion had the highest gain in soaked stability because of the addition of cement. Overall it was concluded that a cold RAP mix containing a low percentage of cement would perform better than conventional cold RAP mixes.

Implementation of the Superpave mix design method has encouraged the use of coarser HMA mixtures, which require tight control of both the overall gradation and the percent passing the 0.075-mm (No. 200) screen. However, there is some concern that use of reclaimed asphalt pavement (RAP) in Superpave mixtures may be seriously limited because stockpiles of RAP may have widely variable gradations as well as high percentages of minus 0.075-mm material (Gardiner & Wagner 1999). They also calculated the possibility of splitting RAP stockpiles by using the coarser RAP fraction in a typical 12.5 mm below the Superpave gradation restricted zone.

In the Gardiner and Wagner study, the finer RAP fraction was used in an above-the-restricted zone 12.5-mm Superpave gradation. Two sources of RAP (Georgia and Minnesota) were used so that a wide range of asphalt and aggregate properties would be represented. Screening the RAP allowed up to 40 percent of the coarse RAP fraction to be used and still meet below the- restricted zone Superpave gradation requirements. This was mainly due to the significant reduction in the finer aggregate fractions, especially the minus 0.075-mm material. They found that the use of RAP in these mixtures resulted in a savings of between 18 and 25 percent in the required virgin asphalt. A noticeable increase in mixture stiffness with as little as 15 percent RAP was observed. This change in mixture properties suggested that a softer grade of neat binder might be needed. A maximum of 15 percent of the fine RAP fraction was used to produce an acceptable above-the- restricted-zone Superpave gradation. The net savings in neat asphalt content was 25 percent. They observed little change in tensile strengths because of the addition of this RAP fraction. However, there was a substantial increase in mixture stiffness at intermediate to warm temperatures. This increase was also observed as a 20 percent reduction in the asphalt pavement analyzer rut depth when RAP was used. The indirect tensile creep compliance decreased when RAP was added.

Solaimanian et al. (Solaimanian et al. 1996) studied production and construction variability of hot-mix asphalt concrete (HMAC) containing large quantities of recycled asphalt pavement (RAP) material. The gradation, asphalt content, air voids, penetrations, viscosities, and stabilities were included in the analysis. They found that in general, projects with high-percentages RAP yield higher variability than a typical virgin HMAC.

The gradations of plant-produced mixtures were finer than the job mix formula target gradations, possibly because of aggregate crushing during the milling operation.

Recently Li et al. (Li et al. 2005) conducted a laboratory study to establish a rational design for asphalt mixture that contains RAP and to change Minnesota DOT's asphalt specification. Ten mixtures, which were the combination of three RAP percentages (0, 20% and 40%), two different virgin asphalt cements (PG 58-28 and PG 58-34), and two different RAP sources (RAP and millings), were studied in this research. RAP material was blended with virgin aggregate such that all samples tested had approximately the same gradation. The Superpave mix design process was used to determine the optimum asphalt content of the mixtures. Dynamic modulus tests (Witczak et al. 2002) were performed at five temperatures (-20, -10, 4, 20, and 40°C) and five frequencies (25, 10, 1, 0.1, 0.01 Hz) and the testing procedure was based on AASHTO TP 62.

Complex modulus master curves were constructed for each mixture, the limited data obtained in this project showed that the addition of RAP increased the complex modulus and that the asphalt binder and RAP source had a significant effect on the mixture modulus. It was also found that the mixtures containing RAP illustrated variability and that the variability increased with the addition of RAP. The creep test data showed that as the percentage of RAP or millings increases, the stiffness increases and that the mixtures with PG 58-34 binder were softer than the mixtures with PG 58-28 binder at -18°C. Asphalt binders extracted from tested dynamic modulus samples were retested at high and low temperatures. Blending charts were constructed based on the test data. The limited test data showed that as the percentage of RAP or millings increased, the stiffness of the extracted binder increased. It was also found that the mixtures with PG 58-28 binders were stiffer than the mixtures containing PG 58-34 binder and the mixtures containing millings were stiffer than those containing RAP, although the effects were less pronounced at low temperatures.

2.5 ASPHALT MIXTURE DESIGN

In this section the different methods of recycled mix design are reviewed. These mix design methods are further divided based on the recycling types: cold recycling and hot recycling. Traditional and new mix design methods are outlined for each type of recycling method.

2.5.1 Mix Design for Cold Recycling

Cold recycling of asphalts can result in a stable pavement at a total expenditure of 40 to 50 percent less than that required by conventional construction methods (ARRA 2001). However, like conventional hot mix asphalt, cold-mix asphalt used for recycling must be designed properly to ensure reliable future pavement performance. The unique features of cold recycled mixes are time temperature effects (curing) due to the presence of the water and/or volatiles and the slower binder softening rate (Kandhal and Mallick 1997). Hence, proper considerations should be given to changes in mixture properties with time and target reduction of aged binder consistency in the mix design.

A standard method for designing cold recycled mixes is not available. However, certain basic steps, as shown in the flow chart in Figure 2.18 (AI MS-21 1983) are included in most mix design procedures used by highway agencies. The first step in mix design is material evaluation. The material evaluation step includes field sampling, determination of aged mix composition, and properties of aged asphalt binder and aggregates. One of the important purposes for this step is to identify the deficiencies of the aged mix and determine the need for virgin material(s). The mix design procedure consists of selection of the recycling agent and the determination of the optimum binder content. These methods are discussed in the following paragraphs.

The Asphalt Institute design method (AI MS-21 1983) recommends the collection of representative samples from the existing pavements before a satisfactory mix design can be accomplished (ASTM D 979 "Sampling Bituminous Paving Mixture"). The sample location should be selected randomly. In addition, the thickness of the asphalt layer along with surface condition assessment is among the required information. A minimum of five samples per kilometer are recommended for laboratory analysis. This design methods recommends that the aggregate and asphalt contained in reclaimed asphalt pavement must be evaluate independently.

The method MS-21 1983 method requires that existing (in-place) aggregates for cold recycling should meet one of the following criteria:

- One measure of the suitability of soil material for cold-mix recycling is that the products PI × PL and PI × P_{200} (where PI = plasticity index, PL = plastic limit and P_{200} = % passing # 200 sieve) should be less than 72.
- Another measure of suitability is the Sand Equivalent Test as per ASTM D 2419 standard. The test is used to detect excessive amount of clay, plastic fines and dust. Generally, materials with a sand equivalent above 30 can be recycled successfully.

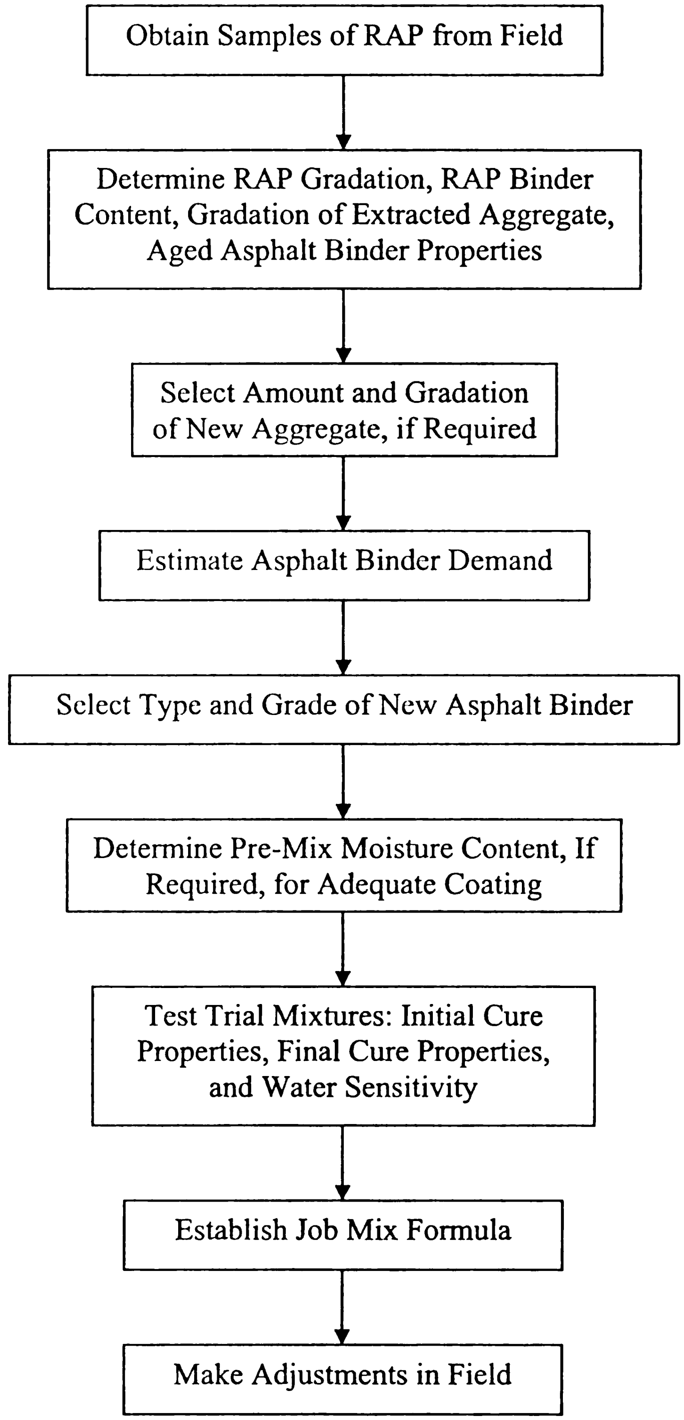


Figure 2.18 Flow chart for mix design of cold recycled mixes

In general, the Asphalt Institute method for mix design calls for blending the reclaimed aggregates with new/virgin aggregates to meet the required specifications. Once the relative aggregate proportions are determined, a grade of new asphalt is selected and new asphalt demand for the blend is calculated by using Equation 6.

$$P_c = \left(\frac{0.035a + 0.045b + Kc + F}{R} \right) \tag{6}$$

where,

P_c = Percent of asphalt material by weight of total mix

a = Percent of mineral aggregate retained on No. 8 (2.36 mm) sieve

b = Percent of mineral aggregate passing No. 8 (2.36 mm) sieve and retained on No. 200 (0.075 mm) sieve

c = Percent of mineral aggregate passing No. 200 (0.075 mm) sieve

K = 0.15 for 11-15 percent passing No. 200 (0.075 mm) sieve
0.18 for 6-10 percent passing No. 200 (0.075 mm) sieve
0.20 for 5 percent or less passing No. 200 (0.075 mm) sieve

R = 1.0 for asphalt cement; 0.6 to 0.65 for asphalt emulsions

F = 0 to 2.0 percent, based on absorption of light or heavy aggregate. The formula is based on an average specific gravity of 2.60 to 2.70. In absence of other data, a value of 0.7 to 1.0 is recommended

The quantity of the new asphalt to be added in the recycled mixtures is calculated using Equation 7.

$$P_r = P_c - \frac{(P_a \times P_p)}{R} \tag{7}$$

where,

P_r = Percent of new asphalt in the recycled mix

P_c = Percent of asphalt by weight of total mix

P_a = Percent of asphalt in the reclaimed asphalt pavement

P_p = Decimal percent reclaimed asphalt pavement in the recycled mix

R = 1.0 for asphalt cement; 0.6 to 0.65 for asphalt emulsions; 0.70 to 0.80 for cutback asphalt.

For in-place recycling, this method recommends to calculate the new asphalt proportions (P_d) based on the weight of aggregate using equation 8.

$$P_d = \frac{100P_r}{100 - P_r} \tag{8}$$

Finally, adjustment of asphalt content can be made by the field engineer to obtain a durable pavement based on the engineering judgment and local experience. This method of mix design is empirical and needs local experience to arrive at a durable mix. In addition, the mix design method does not call for a performance testing to estimate the expected performance of the mix in the field.

The ARRA guidelines (ARRA 2001) indicate three different methods of mix design for cold recycling asphalt pavements. Two of these methods consist of modified Marshall and Hveem for designing cold recycled mixtures with asphalt emulsion or emulsified recycling agent (ERA). The third procedure has been developed by Oregon State University for determination of the required asphalt emulsion content. Only the Marshall mix design method is discussed in this literature review.

In the Marshall mix design method, the mixtures are prepared in such a way to achieve 3 percent total water content (percent emulsion water + percent water remaining in RAP + percent mixture water added). Emulsions are incorporated into the mixtures at the desired content in 0.5 percent increments. Mixtures are then compacted with 50 blows (per face) of the Marshall compacting hammer. The compacted specimens are cured for 6 hours at 60°C (140°F). Next, the specimens are tested for bulk specific gravity, stability (60°C), and flow (60°C). The maximum specific gravity is then determined. Finally, at the optimum additive content, specimens are prepared at additional total water content at 0.5 percent increment (such as 2.0 percent, 2.5 percent, 3.5 percent and 4.0 percent). The average void content for each moisture content is then determined. The recommended mix design parameters include minimum and maximum design air voids of 9 and 14 percent, respectively (ARRA 2001).

Other studies have been done on cold recycling mix design. To improve the consistency of the CIR using foamed asphalt (CIR-foam) mix design process, the critical mix design parameters were identified and the laboratory test procedure was developed (Kim et al. 2006) in Iowa. The developed CIR-foam mix design process was validated against seven different RAP sources collected across the state of Iowa. It was recommended that given the high moisture sensitivity of CIR, the indirect tensile strength test should be performed on the vacuum-saturated mixtures. The was conducted using these RAP materials at five foamed asphalt contents, 1.0, 1.5, 2.0, 2.5, and 3.0 percent, given a fixed moisture content of 4.0 percent. It was found that air voids decreased gradually as the foamed asphalt content increased but the voids did not affect the indirect tensile strength of the foamed asphalt mixtures. Based on the developed mix design procedure, the optimum foamed asphalt contents were consistently found at values between 1.5 % and 2.5 % for seven different RAP materials. Kim et al. recommended that the test samples should be prepared using Gyratory compactor rather than Marshall Hammer because it produced more consistent mixtures for different foamed asphalt contents and curing conditions. It was also found that the indirect tensile strength of CIR-foam samples cured for two days at 60 °C was significantly higher than that of CIR-foam samples cured for three days at 40 °C.

2.5.2 Mix Design for Hot Recycling

The mix design methods for hot recycling can be divided into traditional and new methods. Also, there are different guidelines for each design method for the recycling method in question. The methods in this section are divided for hot recycling mix designs were into:

- Traditional mix design methods for hot in-plant recycling
- Traditional mix design methods for hot in-place recycling
- Superpave mix design

Traditional mix design methods for hot in-plant recycling consist of two main steps: material evaluation and mix design. The objective of the material evaluation process is to determine the important properties of the component materials to come up with an optimum blend of materials to meet the mix requirements. The objective of the mix design step is to determine the type and percentage of asphalt binder with the help of results from compacted test mixes. The specific steps of the material evaluation and mix design process are as follows (Kandhal and Mallick 1997):

1. Obtain representative field samples of the reclaimed material.

2. Perform laboratory analysis:

    a. Determine composition and properties of the RAP

    b. Determine the proper amounts of virgin aggregates to be added

    c. Select the type and amount of the virgin asphalt binder

    d. Mix, compact and test trial mixes

3. Select the optimum combination of mix components that meet the mix design criteria

For the Asphalt Institute method, Figure 2.19 (AI MS-20 1986) shows a flow chart for the recommended steps in the recycled mix design process. Using the gradation of the aggregates from the reclaimed asphalt pavement and new aggregates, a combined gradation meeting the desired specification requirements is calculated. After blending the aggregates (aggregate in the RAP and virgin aggregates) has been established, the amount of new aggregate is expressed as R, in percent. Table 2.7 contains formulas for proportioning materials for recycled HMA mixes where the blend of aggregates in the mix is kept constant.

The next step is to approximate the asphalt binder demand of the combined aggregates. The most practical approach is to assume the asphalt demand of the combined aggregates in the proposed recycled HMA to be equal to the optimum asphalt content of virgin HMA (without any RAP). Therefore, the following procedure for determining the approximate asphalt demand may not be necessary unless no virgin mix design is available for 100 percent virgin mix. The approximate asphalt demand of the combined aggregates may be determined by the Centrifuge Kerosene Equivalent (CKE) test included in the Asphalt Institute Hveem Method of Mix Design, or calculated using the following empirical formula:

$$P = 0.035a + 0.045b + Kc + F \tag{9}$$

where,

P = Approximate total asphalt demand of recycled mix, percent by weight of mix

a = Percent of mineral aggregate retained on 2.36 mm sieve, expressed as a whole number

b = Percent of mineral aggregate passing the 2.36 mm sieve and retained on the 75 μm sieve, expressed as a whole number

c = Percent of mineral aggregate passing the 75μm sieve

K = 0.15 for 11-15 percent passing 75 μm sieve, 0.18 for 6-10 percent passing 75μm sieve, and 0.20 for 5 percent or less passing 75 μm sieve

F = 0 to 2.0 percent, based on absorption of light or heavy aggregate. In the absence of other data, a value of 0.7 is suggested

Table 2.7 Formulas for proportioning materials for hot recycled mixtures (AI MS-20 1986)

Quantity	For Asphalt Content	
	By weight of total mix	By weight of aggregate
% New Asphalt, P_{nb}	$P_{nb} = \frac{(100^2 - rP_{sb})P_b}{100(100-P_{sb})} - \frac{(100-r)P_{sb}}{100-P_{sb}}$	$P_{nb} = P_b - \frac{(100-r)P_{sb}}{100}$
% RAP, P_{sm}	$P_{sm} = \frac{100(100-r)}{(100-P_{sb})} - \frac{(100-r)P_b}{(100-P_{sb})}$	$P_{sm} = \frac{(100-P_{sb})(100-r)}{100}$
% New Aggregate, P_{ns}	$P_{ns} = r - \frac{rP_b}{100}$	$P_{ns} = r$
Total	100	$100 + P_b$
% New Asphalt to total Asphalt Content, R	$R = \frac{100P_{nb}}{P_b}$	$R = \frac{100P_{nb}}{P_b}$

where,

P_{sm} = Percent salvaged mix (RAP) in recycled mix

P_b = Asphalt content of recycled mix, %

P_{sb} = Asphalt content of salvaged mix (RAP), %

P_{nb} = Additional asphalt and/or recycled agent in recycled mix, %

P_{ns} = Percent additional aggregate (new aggregate material)

r = Percent new aggregate material to total aggregate in recycled mix

R = Percent new asphalt and/or recycling agent to total asphalt in recycled mix

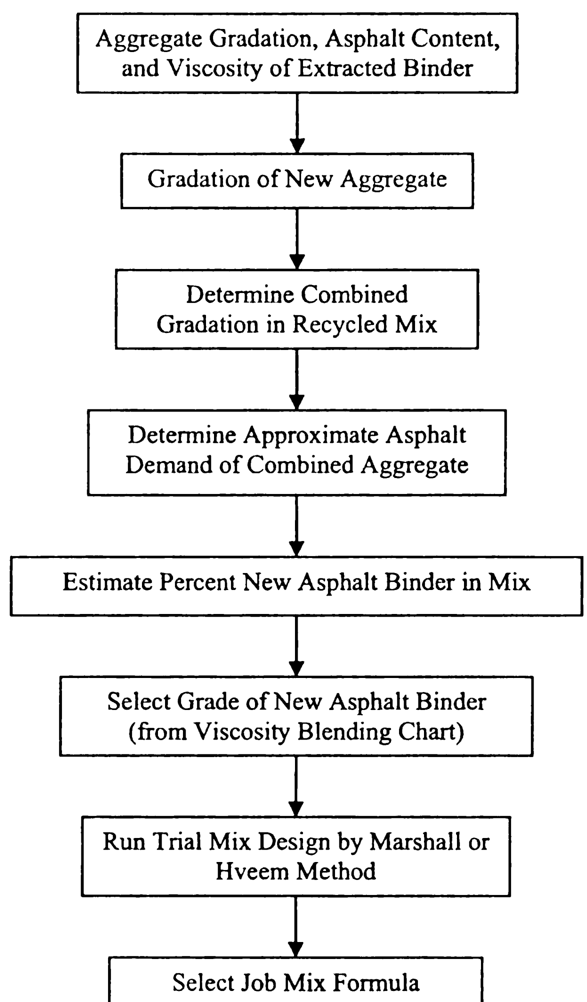


Figure 2.19 Flow chart for mix design procedure

With an approximate asphalt demand established, this will provide a basis for a series of trial mixes for a mix design. Trial mixes will vary in asphalt contents in 0.5 percent increments on either side of the calculated approximate asphalt demand. For example, suppose that the approximate asphalt demand was calculated to be 6.2 percent. A series of trial mixes then range from 5.0 to 7.0 percent or from 5.5 to 7.5 percent are made and tested.

The quantity of the new asphalt binder to be added to the trial mixes of the recycled HMA mixture, expressed as percent by weight of total mix is calculated by the following formula:

$$P_{nb} = \frac{\left(100^2 - rP_{sb}\right)P_b}{100\left(100 - P_{sb}\right)} - \frac{\left(100 - r\right)P_{sb}}{100 - P_{sb}} \tag{10}$$

where,

P_{nb}= Percent of new asphalt binder in recycled mix (plus recycling agent, if used)

r = new aggregate expressed as a percent of the total aggregate in the recycled mix

P_b = percent, estimated asphalt content of recycled mix (assumed to be the same as that of 100 percent virgin HMA mix or determined as an approximate asphalt demand of combined aggregates in the preceding step)

P_{sb}= percent, asphalt content of reclaimed asphalt pavement (RAP) (plus recycling agent, if used)

The percentages of new asphalt binder for any asphalt content may now be readily determined. The formula above is for asphalt content expressed as percent by weight of total mix, the formula expressed by weight of aggregate is:

$$P_{nb} = P_b - \frac{\left(100 - r\right)P_{sb}}{100} \tag{11}$$

The grade of the new asphalt binder must now be selected using Figure 2.20. A target viscosity of the asphalt blend is selected. A commonly selected target point is the viscosity at the mid-range of the specified viscosity-graded asphalt binder. For example, the target for an AC-20 asphalt binder would be 2,000 poises. The percent of the new asphalt, P_{nb}, to the total asphalt content, P_b, is expressed by the following formula:

$$R = \frac{100P_{nb}}{P_b} \tag{12}$$

The grade of new asphalt binder (and/or recycling agent) is determined using a log-log viscosity versus percent new asphalt binder blending chart such as that in Figure 2.20. A target viscosity for the blend of recovered asphalt and the virgin asphalt (and/or recycling agent) is selected. As mentioned earlier, the target viscosity is usually the viscosity of the mid range of the grade of asphalt binder normally used depending on the type of construction, climatic conditions, amount and nature of traffic. A step by step procedure regarding the use of Figure 2.20 is stated below.

- Plot the viscosity of the aged asphalt in the RAP on the left hand vertical scale, Point A, as illustrated in Figure 2.20.
- Draw a vertical line representing the percentage of virgin asphalt binder, R, calculated from equation 12 and determine its intersection with the transverse line representing the target viscosity of 2,000 poises, point B in Figure 2.20.
- Draw a straight line from Point A, through Point B and extend it to intersect the right hand scale, Point C. Point C is the viscosity at 60°C (140°F) of the new asphalt binder (and/or recycling agent) required to blend with the asphalt

binder in the reclaimed asphalt pavement (RAP) to obtain the target viscosity in the blend.

- Select the grade of the virgin asphalt binder that has a viscosity range that includes or is closest to the viscosity at Point C.

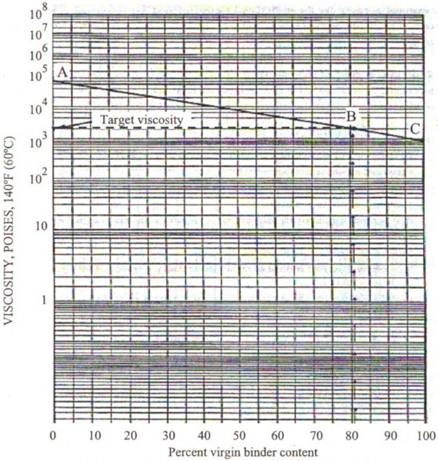


Figure 2.20 Asphalt viscosity blending chart (AI MS-20 1986)

The following guide could be used in the selection of the virgin binder:

- For recycled blends containing up to 15 percent RAP, no change in asphalt binder grade is required (some highway agencies use 20 percent in lieu of 15 percent).

- 16 percent RAP or more = Use asphalt binder one grade softer than that normally specified for 100 percent virgin HMA mix. For example, use AC-10 in lieu of AC-20. Do not change more than one viscosity grade unless the recycled HMA mix is checked for resistance to rutting

Mix designs are then made using the Marshall method. The optimum asphalt content is selected based on the test data obtained in the preceding step. If the Marshall Mix Design procedure is used, the optimum asphalt content is selected to give 4.0 percent air void content (AI MS-20 1986).

Hot recycled mix design involves the determination of the combined gradation of the aggregates and the required amount of new aggregate to meet the target gradation. Next, the amount and type of grade the virgin asphalt binder required in the recycled mix is estimated. Blending charts (based on viscosity or Superpave rutting factor $G^*/\sin\delta$) are then used to select the grade of virgin asphalt binder. A series of trial mixes are then made with different asphalt contents. The optimum asphalt content for the recycled mix is selected based on Marshall or Superpave volumetric mix design procedures.

Traditional mix design methods for hot in-place recycling can be summarized using the following procedure (Kandhal & Mallick 1997):

1. Evaluation of salvaged materials
2. Selection of type and amount of recycling agent
3. Determination of the need for additional aggregates and/or asphalt binder and/or virgin HMA
4. Preparation and testing of paving mixtures

5. Selection of optimum combination of new aggregates, asphalt binder, and recycling agent or virgin HMA

The in-depth steps listed in Table 2.8 are the most important in the mix design. For a detailed discussion on mix design steps, see mix design procedure for hot mix asphalt recycling above. However, it should be noted that the following exceptions are to be dealt with in hot in-place recycling:

Table 2.8 Recycled hot mix design procedures (Kandhal & Mallick 1997)

1. DETERMINE MATERIAL PROPERTIES AND PROPORTIONS	
	a. Obtain representative samples of RAP[a], new aggregates[a], and new asphalt cement selected[b].
	b. Determine asphalt cement content of RAP (including penetration/viscosity of the recovered binder)[b]
	c. Determine gradation of RAP aggregate, including bulk specific gravity.
	d. Determine gradation, percent crushed, bulk specific gravity, and absorption of new aggregates[c].
	e. Determine if adjustments in aggregate gradation are necessary to develop voids in mineral aggregate (VMA) and select as necessary, ensuring adequate stability is maintained[d].
	f. Determine the total aggregate grading, check specification compliance and modify as necessary.
2. PREPARE MATERIALS FOR MIXTURE DESIGN	
	a. Determine increments (range) of total asphalt content required to develop specified parameter plots.
	b. Select recommended grade or preferred penetration/viscosity of new (additional) binder.
	c. Determine mass of RAP, new aggregates, and new binder for each increment.
3. COMPLETE MIXTURE DESIGN	
	a. Prepare compacted briquettes incorporating RAP[e], new aggregates, and new binder.
	b. Test briquettes - bulk specific gravity, maximum specific gravity, stability, flow, air voids, VMA, and appearance.
	c. Report recommended recycled mixture design.
4. QUALITY CONTROL/QUALITY ASSURANCE (QC/QA)	
	a. Similar to conventional hot mixture with addition of monitoring RAP (moisture content, gradation, and asphalt-cement content) and more emphasis on absolute viscosity and penetration of recovered binder.

a: All samples must be representative. Process control data should be used.
b: The new asphalt cement selected must provide properties in the recycled mixture meeting specifications.
c: For new aggregates that have not been used before, factors shuch as petrography and stripping resistance must be considered. This also applies to RAP aggregate, if aggregate-related pavement distress is involved.
d: In order to develop VMA, it is often necessary to incorporate a clean fine aggregate
e: The RAP must be carefully dried during testing to avoid excessive asphalt cement hardening, and then combined with suitably heated new aggregates to give an overall mixing temperature meetin the appropriate combined RAP asphalt cement and new asphalt cement mixing temperature viscosity.

- The amount of RAP is generally between 80 to 100 percent in hot in-place recycling. Where it is only 15 to 20 percent RAP is common in hot in-plant recycling. If 100 percent RAP is used, it is not necessary to determine any combined gradation of RAP and virgin aggregates.
- Air voids in hot in-place recycled mix can be higher than 4 percent. Higher design air voids (as much as 6 percent) have been used successfully in hot in-place recycling in Canada.

2.5.3 Superpave Mix Design for Recycled Asphalt Pavements

From 1987 through 1993, the Strategic Highway Research Program carried out several major research projects to develop the Superpave method for performance-based HMA design. This method has now widely superseded the Marshall and Hveem design methods in the United States and Canada. A distinct shortcoming of the Superpave method was that it did not provide a specific provision for the use of RAP in the mix design process. This shortcoming has hindered RAP use by agencies that have adopted the Superpave mix design method. To remedy this situation, the Federal Highway Administration's Superpave Mixtures Expert Task Group used past experience to develop interim guidelines for the use of RAP in the Superpave method.These guidelines reflect the fact that the effect of aged binder from RAP on the performance properties of the virgin binder depends upon the level of RAP in the HMA. When the level is low, the effect is minimal, and the RAP is likened to a "black rock" that influences the mix volumetric and performance through its aggregate gradation and properties. As the level of RAP in the HMA increases, the black rock analogy breaks down; the aged binder blends with the virgin material in sufficient quantity to significantly affect its performance properties. Under NCHRP Project 9-12, "Incorporation of Reclaimed Asphalt Pavement in the Superpave System," (McDaniel & Anderson 2001, NCHRP 2001) the North Central Superpave Center at Purdue University was assigned the tasks of developing recommended guidelines for incorporating RAP in the Superpave mix design method. The RAP binder evaluation and mix design using Superpave system according to this NCHRP Report 452 can be broken down into the following steps:

1. Determining RAP binder properties
2. Selecting binder grade selection
3. Determining RAP content
4. Developing the mix design
5. Selecting the design aggregate

In order to determine the RAP binder properties, guidelines for whether or not the RAP binder will influence the mix can be found in the NCHRP Report 452. Under the recommended guidelines for using RAP in Superpave mixtures, there are three levels, or tiers, of RAP usage. Table 2.9 shows recommended tiers for Superpave RAP mixtures and the appropriate changes to the binder grade. The limits of these tiers depend on the RAP binder grade. With softer RAP binders, one can use higher percentages of RAP. The first tier establishes the maximum amount of RAP that can be used without changing the virgin binder grade. The second tier shows the percentages of RAP that can be used when the virgin grade is decreased by one grade (a 6-degree increment) on both the high- and low-temperature grades. The third tier is for higher RAP contents; for these higher contents, it is necessary to extract, recover, and test the RAP binder and to construct a blending chart.

Table 2.9 Binder selection guidelines for RAP mixtures

Recommended Virgin Asphalt Binder Grade	RAP Percentage Recovered RAP Grade		
	PG xx-22 or lower	PG xx-16	PG xx-10 or higher
No Change in binder selection	< 20%	<15%	<10%
Select virgin binder one grade softer than normal (e.g., select a PG 58-28 if a PG-64-22 would normally be used)	20-30%	15-25%	10-15%
Follow recommendations from blending charts	>30%	>25%	>15%

To construct a blending chart, the desired final binder grade and the physical properties (and critical temperatures) of the recovered RAP binder are needed, plus one of the following pieces of information:

- The physical properties (and critical temperatures) of the virgin binder
- The percentage of RAP in the mixture

Once the RAP binder has been extracted and recovered, its properties need to be determined. At least 50 g of recovered binder are needed for testing. The RAP binder must be tested in the dynamic shear rheometer (DSR) at a high temperature as if it were original, virgin binder. Then the remaining RAP binder is aged in the rolling thin film oven (RTFO) and is tested in the DSR and bending beam rheometer (BBR). The following steps should be followed to determine the physical properties and critical temperatures of the RAP binder (McDaniel & Anderson 2001).

1. The RAP binder should be recovered using the modified AASHTO TP2 method (described previously) with an appropriate solvent. At least 50 g of recovered RAP binder are needed for testing.

2. Perform binder classification testing using the tests in AASHTO MP1. Rotational viscosity, flash point, and mass-loss tests are not needed.
    a. Perform original DSR testing on the recovered RAP binder to determine the critical high temperature, Tc (High), based on original DSR values where G*/sin δ = 1.00 kPa. Calculate the critical high temperature as follows:
        i. Determine the slope of the stiffness-temperature curve as Δ Log (G*/sin δ)/ΔT.
        ii. Determine Tc (High) to the nearest 0.1°C using the following equation:

$$T_c(High) = \left(\frac{Log(1.0) - \log(G_1)}{a} \right) + T_1 \qquad (13)$$

where,
G1= G*/sinδ value at a specific temperature, T1
a = the slope of the stiffness

3. Perform RTFO aging test on the remaining RAP binder.
4. Perform DSR testing on the RTFO-aged recovered RAP binder to determine the critical high temperature, which can be calculated as follows:
    a. Determine the slope of the stiffness-temperature curve as Δ Log (G*/sin δ)/ΔT.
    b. Determine T_C (*High*), based on RTFO DSR, to the nearest 0.1°C using the following equation:

$$T_c(High) = \left(\frac{Log(2.20) - \log(G_1)}{a} \right) + T_1 \qquad (14)$$

5. Determine the critical high temperature of the recovered RAP binder as the lower of the original DSR and RTFO DSR critical temperatures. Determine the high-temperature performance grade of the recovered RAP binder based on this single critical high temperature.
6. Perform intermediate temperature DSR testing on the RTFO-aged recovered RAP binder to determine the critical intermediate temperature, *Tc* (*Int*), based on pressure aging vessel (PAV) DSR.
   a. Determine the slope of the stiffness-temperature curve as ΔLog (G* sin δ)/ΔT.
   b. Determine T_C (*Int*) to the nearest 0.1°C using the following equation:

$$T_c(Int) = \left(\frac{Log(5000) - \log(G_1)}{a} \right) + T_1 \qquad (15)$$

7. Perform BBR testing on the RTFO-aged recovered RAP binder to determine the critical low temperature, *Tc* (*S*) or *Tc* (*m*), based on BBR stiffness or *m*-value.
   a. Determine the slope of the stiffness-temperature curve as D Log(S)/DT.
   b. Determine *Tc (S)* to the nearest 0.1°C using the following equation:

$$T_c(S) = \left(\frac{Log(300) - \log(S_1)}{a} \right) + T_1 \qquad (16)$$

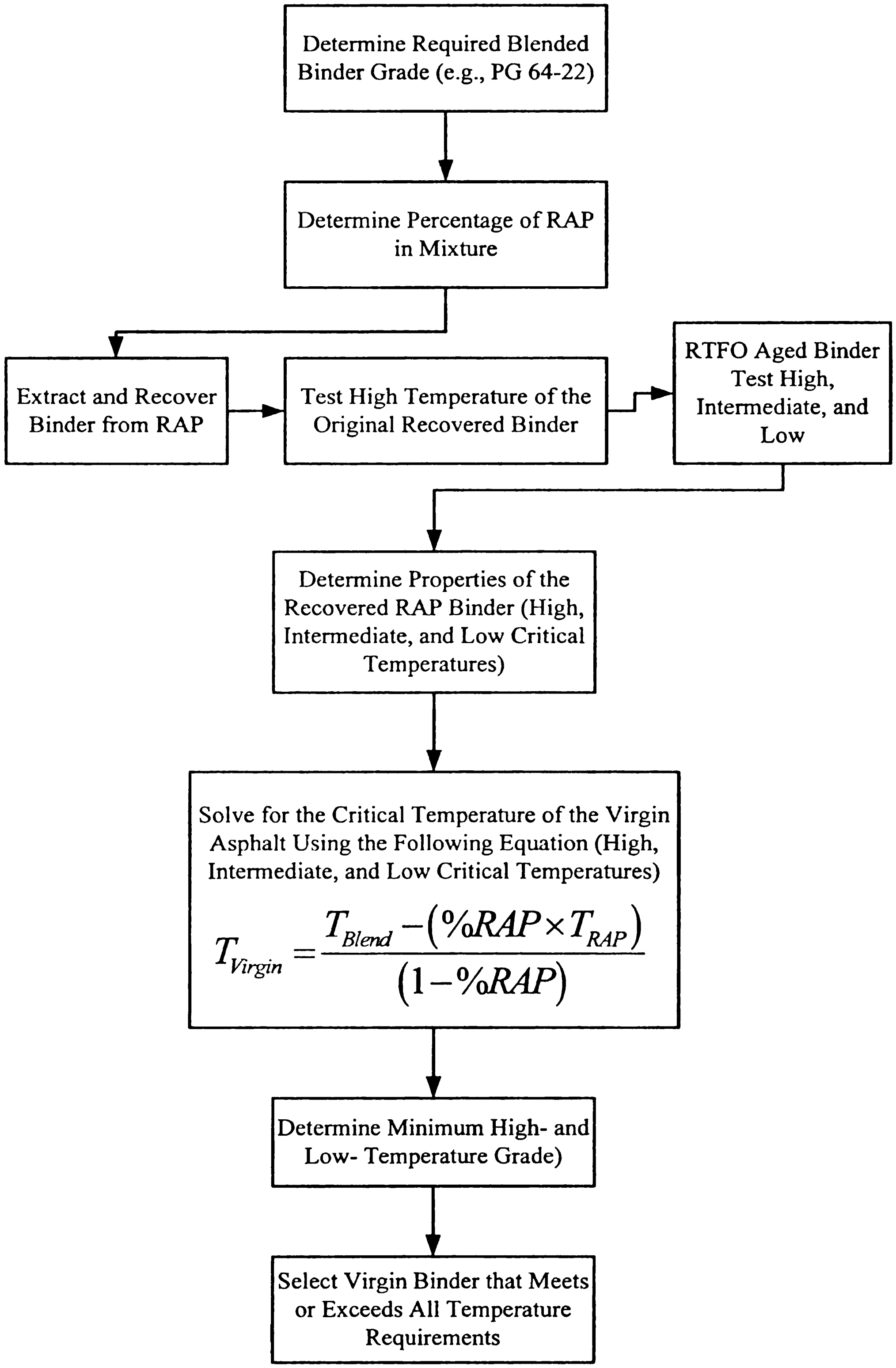


Figure 2.21 Method A: Blending at a known RAP content (virgin binder grade unknown) (McDaniel & Anderson 2001)

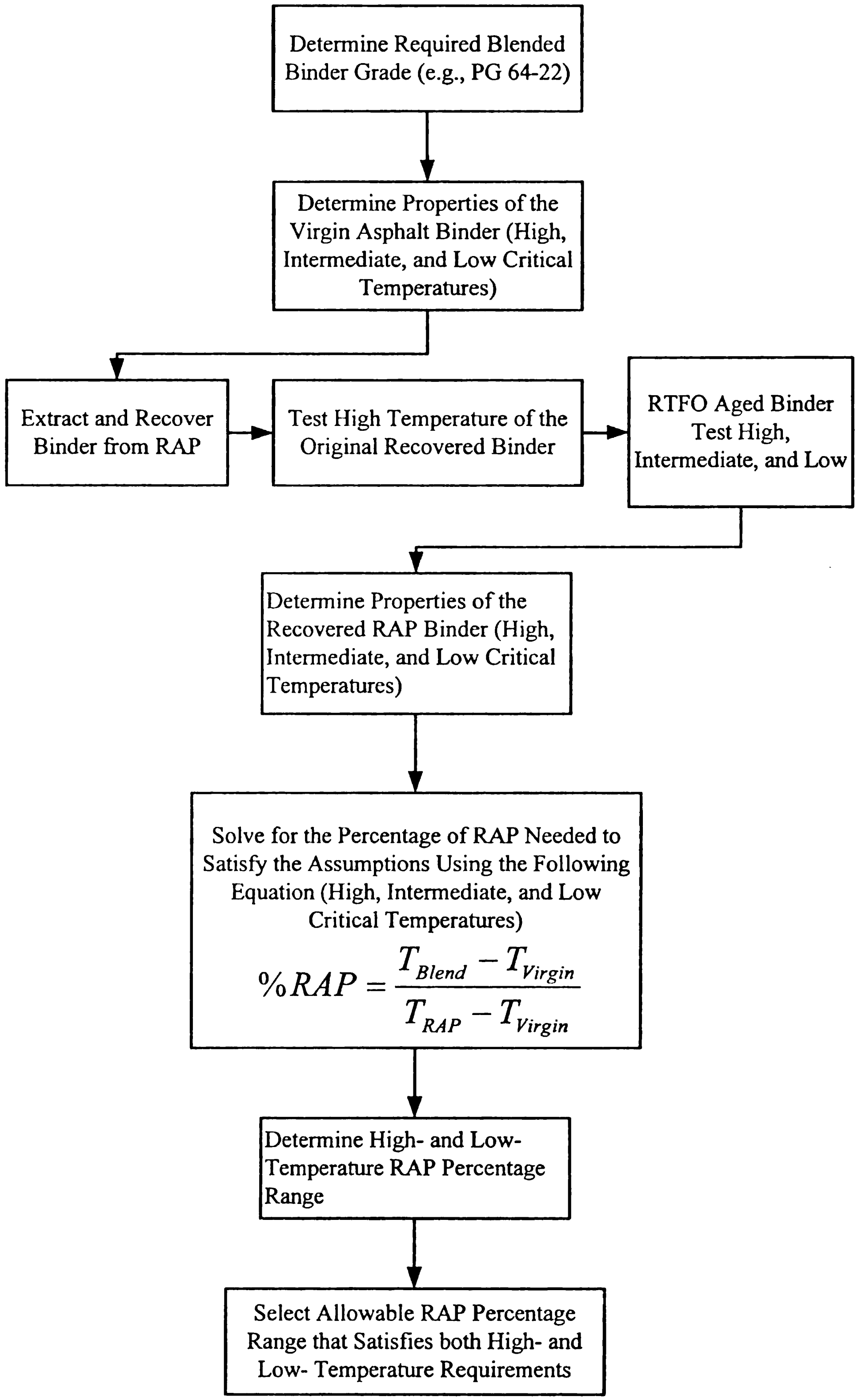


Figure 2.22 Method B: Blending a known virgin binder (RAP content unknown) (McDaniel & Anderson 2001)

c. Determine the slope of the m-value-temperature curve as Δm-value/ΔT.

d. Determine *Tc (m)* to the nearest 0.1°C using the following equation:

$$T_c(m) = \left(\frac{0.300 - m_1}{a}\right) + T_1 \tag{17}$$

e. Select the higher of the two low critical temperatures *Tc* (*S*) and *Tc* (*m*) to represent the low critical temperature for the recovered asphalt binder, *Tc* (*Low*). Determine the low-temperature performance grade of the recovered RAP binder based on this single critical low temperature.

Once the physical properties and critical temperatures of the recovered RAP binder are known, two blending approaches may be used (see Figures 2.21 and 2.22). In the first approach (designated Method A); the percentage of RAP that will be used in an asphalt mixture is known, and the appropriate virgin asphalt binder grade for blending needs to be determined. In the second approach (designated Method B); the maximum percentage of RAP that can be used in an asphalt mixture while still using the same virgin asphalt binder grade needs to be determined. Both approaches assume that the specifying agency will determine the performance grade of the final blended binder. These approaches are discussed further in the following section.

The next step is the binder grade selection. The first method that will be discussed is Method A: Blending at a Known RAP Percentage (Virgin Binder Grade Unknown). In some cases, highway agency may know approximately how much RAP they would like to use in a mixture. For example, agency may want to use all of the millings from a given project, or recycling may be most economical if a certain range of RAP contents is used. In other cases, the gradation or mix properties will limit the amount of RAP that can be used. There also may be specification limits that control how much RAP you can use. In these cases, agency can choose a RAP content, then determine what binder grade is needed to blend with the RAP to get a particular grade for the blend of old and new binder. If the final blended binder grade, percentage of RAP, and recovered RAP properties are known, then the properties of an appropriate virgin asphalt binder grade can be determined based on the following information:

- The specifying agency requires a blended binder grade (e.g., a PG 64-22 or better)
- The RAP percentage in the mixture (e.g., 30 percent)
- The recovered RAP properties.

Using equation 18 for the high, intermediate, and low critical temperatures separately, the properties of the virgin asphalt binder needed to satisfy the assumptions can be determined. (This general equation is a rearranged version of the earlier equations for critical temperatures.)

$$T_{Virgin} = \frac{T_{Blend} - (\%RAP \times T_{RAP})}{(1 - \%RAP)} \tag{18}$$

Where,

T_{Virgin} = critical temperature of the virgin asphalt binder;

T_{Blend} = critical temperature of the blended asphalt binder (final desired);

$\%RAP$ = percentage of RAP expressed as a decimal (i.e., 0.30 for 30 percent); and

T_{RAP} = critical temperature of recovered RAP binder.

The second method determines the RAP content, which uses Method B: Blending with a Known Virgin Binder Grade (RAP Percentage Unknown). If the final blended binder grade, virgin asphalt binder grade, and recovered RAP properties are known, then the appropriate amount of RAP to use can be determined based on the following information:

- The specifying agency requires a blended binder grade (e.g. a PG 64-22 or better)
- The virgin binder grade along with critical temperatures (e.g. a PG 58-28)
- The recovered RAP binder grade and critical temperatures (e.g. a PG 82-10)

Using equation 19 for the high, intermediate, and low critical temperatures separately, the percentage of RAP needed to satisfy the assumptions can be determined (Again, this equation is obtained by rearranging the earlier equations for critical temperatures.)

$$\%RAP = \frac{T_{Blend} - T_{Virgin}}{T_{RAP} - T_{Virgin}} \tag{19}$$

Where,

T_{Virgin} = critical temperature of the virgin asphalt binder;

T_{Blend} = critical temperature of the blended asphalt binder (final desired);

$\%RAP$ = percentage of RAP expressed as a decimal (i.e., 0.30 for 30 percent); and

T_{RAP} = critical temperature of recovered RAP binder.

The next step is developing the mix design. This section describes the step-by-step mix design process as described in NCHRP Report 452 (McDaniel & Anderson 2001). It is important to note that one major decision that must be made early in the process is the approximate amount of RAP to be used. This decision is made based on the prevailing agency specifications, the aggregate gradation and properties, economics, and, sometimes, the binder properties. The amount of RAP to be included in the new mixture may be limited by many factors, including (McDaniel & Anderson 2001):

- Specification limits for mix type, plant type, or other reason
- Gradation
- Aggregate consensus properties
- Binder properties
- Heating, drying, and exhaust capacity of the plant
- Moisture content of the RAP and virgin aggregates
- Temperature to which the virgin aggregate must be superheated
- Ambient temperature of the RAP and virgin aggregate
- Other factors.

These limiting factors could be considered as material and production related factors. The latter factors include the plant capacity for heating and drying the RAP and virgin aggregates. If the ambient temperature is low or the moisture content of the materials is high, it will take more energy to heat and dry the materials. These factors, in turn, will affect the rate of HMA production. Superpave mixtures with RAP will have the same types of production-related limits as those produce by using Marshall or Hveem methods. On the other hand, the material-related restrictions on the amount of RAP are somewhat different for Superpave mixtures than for Marshall or Hveem mixtures because of the differing specification limits. The allowable gradation, for example, may be different for Superpave mixtures; frequently, lower fine contents are required. Also, the blend of virgin and RAP aggregates has to meet the consensus properties, which may be tighter than previous aggregate requirements.

Overall, however, the process of using RAP in Superpave mixtures is similar to that of using RAP in Marshall or Hveem mixtures. The blend of materials has to meet certain properties, and the plant must be capable of drying and heating the materials. Many of the techniques used to evaluate the RAP are similar to previous techniques. A detailed procedure for developing mix design involving RAP along with examples is described in the NCHRP Report 452 (McDaniel & Anderson 2001).

The final step is the selection of design for the aggregate structure. The procedure can be summarized in the following steps, a more detailed procedure is described in the NCHRP Report 452 (McDaniel & Anderson 2001).

1. Establish the trial blend
2. Estimate the trial binder content
3. Calculate batch weights
4. Mix and compact trial blend specimens

In summary, asphalt concrete is a visco-elastic material, which is produced by combining binder (bitumen) and crushed aggregates in an appropriate proportion. The characterization of this material depends on the temperature and rate of loading. If not designed properly, higher temperatures and slow rate of loading will cause permanent deformations (rutting) in the materials while under lower temperatures it may crack (fatigue) under repetitive wheel loads due to brittleness. Therefore, it is imperative to consider both pavement performance and material behavior simultaneously at the design stage.

The traditional methods for designing such materials is based on the Marshall Method, which do not consider the link between design and performance. In 1987, Strategic Highway Research Program (SHRP) was established in USA to further improve the current practices. The objectives of SHRP's research were to improve the performance and durability of roads, extend the life or reduce the life-cycle cost of the asphalt pavements, reduce maintenance costs, minimize premature failures, and to make roads safer. A historic product of SHRP research was a new mix design method referred to as Superpave (Superior Performing Asphalt Pavements).

The SHRP Superpave binder specification was the major ultimate product of the asphalt binder research. It is believed that Superpave technology will result in improved road infrastructure resulting in significant saving in the maintenance costs. In addition, improved infrastructure can indirectly create substantial benefits to the environment, and socio-economic conditions. The characterization of HMA mixtures containing RAP using this new system has been reported in literature and is discussed below.

2.5.4 Summary of Research Studies on Recycled Asphalt Mixtures

Hajj et al. (Hajj et al. 2008) conducted a very extensive literature review on reclaimed asphalt pavements in airfield HMA pavements. They reviewed several reports and studies which are in close parallel to the objectives and tasks for this thesis. Table 2.10 is from the Hajj et al.'s literature review which shows a summary of their findings.

Table 2.10 Summary of findings in literature on recycled asphalt mixtures (Hajj et al. 2008)

Research	Objective	Description	Findings
Minnesota DOT (Li et al. 2005 &2008).	Effect of RAP type and percentage on asphalt mixture properties.	Total of 10 mixes: - %RAP: 0, 20, 40. - Millings binder content 4.3%. - RAP binder content 5.4%. - PG58-28 & PG58-34. - Superpave mix design.	- Mixes TSR at 77°F > 75%. - |E*| increased with RAP. - |E*| affected by RAP source & asphalt binder. - RAP induced higher variability in measured properties & variability increased with RAP content. - Creep stiffness increases with %RAP or millings. - Mixes with PG58-34 binder softer than mixtures with PG58-28 binder at -18°C. - Extracted binder stiffness increased with %RAP or millings.
NCHRP (McDaniel et al. 2000).	Incorporate use of RAP in Superpave HMA mixtures.	- 3 RAP sources: low stiffness RAP (PG82- 22 and 5.9% binder), medium stiffness RAP (PG82-22 and 4.9% binder), high stiffness RAP (PG82-10 and 5.3% binder). - Virgin binder PG52-34 & PG64-22. - %RAP: 0, 10, 20, & 40.	- RAP does not act like a black rock. - Linear blending equations appropriate with some non-linearity above 40% RAP. - Negligible effect of RAP at low RAP content. - At intermediate RAP content, effect of RAP compensated by using virgin binder 1 grade softer on both high & low temperature grades. - At high RAP content: use blending chart. - Properties of low RAP content mix similar to that of no RAP mix. - High RAP content stiffens the mix at high, intermediate, and low temperature. - Higher RAP content exhibits more rutting resistance and lower beam fatigue life when no change made in virgin binder grade.
North Central Superpave Center (McDaniel el al. 2002).	Laboratory performance of Superpave asphalt mixtures incorporating RAP.	- 3 RAP sources: Indiana (4.7% binder), Michigan (3.8% binder), Missouri. (4.4% binder) - RAP content: up to 50%. - Plant produced mix at 15-25% RAP.	- Mixes with up to 50% can be designed under Superpave if RAP gradation and aggregate quality sufficient. - Linear blending charts appropriate in most cases. - Plant mixes showed similar performance as lab mixes except for Indiana mixes. - Increase in RAP content increases rutting resistance when virgin binder unchanged. - Small amount of RAP has low impact on performance - Consider RAP aggregate gradation and quality in mix design.
North Central Superpave Center (McDaniel et al. 2006).	Influence of RAP on mix & recovered binder of plant-produced HMA.	- 15, 25, 40% RAP + PG64-22 virgin binder. - 25% & 40% RAP + PG58-28 virgin binder. - Control mix: PG64-22 & 0% RAP.	- At 15% & 25% RAP, no difference in mean strength at low temperature and |E*|. - Some differences between control & 40% RAP mix at higher test temperature. - adding small amount of RAP may not change mix properties greatly.
Saskat. Highways & Transp. (Puttagunta et al. 1997).	Compare lab fatigue performance & moisture damage of virgin & mixes containing RAP.	- One RAP source with 6.4% binder - 50% RAP core samples from Hwy 11, Canada. - RAP & virgin aggregate were used to prepare 25 & 50% RAP mixes in lab.	- At low temperature virgin mix perform well in fatigue - At high temperature all mixes perform equally in fatigue - RAP mixes perform much better in moisture susceptibility test than virgin mixes

Table 2.10 (continued)

Western Regional Superpave Center (Hajj et al. 2007).	Laboratory evaluation on the use of RAP in laboratory produced HMA mixes.	- 3 RAP sources: 1 plant waste (4.6% binder) and 15 (5.4% binder) and 20 (5.8% binder) years-old reclaimed pavements. - 3 RAP contents: 0%, 15%, & 30%. - 2 target binder grades: PG64-22 and PG64-28 (polymer modified). - Virgin binder grades: selected based on blending charts. - RAP mixes are compared to no RAP mixes	- Marshall Mix Design method can be used to design mixes with 15 and 30% RAP. - Blending chart method conservative & not reliable. - Mixes had Acceptable resistance to moisture damage. - Reduction in the unconditioned & conditioned TS of the 15 & 30% RAP mixes. - PG64-22 mixes: 15% RAP mixes showed higher resistance to moisture damage than 30% RAP mixes. - PG64-28 mixes: 15% RAP mixes had lower resistance to moisture damage than 30% RAP mixes. - In general, PG64-22 RAP mixes showed better or equivalent fatigue resistance to the no RAP mix. - RAP in PG64-28 mixes significantly reduced fatigue resistance. - RAP mixes showed better or equivalent thermal cracking resistance to the no RAP mix.
Western Regional Superpave Center *(*Hajj et al. 2007*)*.	Laboratory evaluation on the use of RAP in field sampled HMA mixes.	- Two field mixes with 15%RAP - Two binders: PG64-22 & polymer modified PG64-28	- PG64-22 mix failed to meet minimum TSR of 70%. - PG64-28 mix barely passed minimum required TSR. - Mixes met the NDOT APA criterion of 8mm. - Polymer modified binder reduced APA rut depth by about 42% compared to neat asphalt binder. - Use of RAP in a polymer modified mix increased mix resistance to fatigue cracking in laboratory test when compared to the neat binder (PG64-22) mix. - Use of RAP in a polymer modified mix reduced the resistance to fatigue cracking in mechanistic analysis when compared to neat binder -Fracture temperature was within 1°C of low performance temperature of corresponding target binder grades (i.e. -22°C & -28°C). - Mixes might show signs of failure in the field due to moisture sensitivity problems. - Attention should be given to moisture resistance of field mixtures.
Daniel and Lachance 2005	Evaluation of volumetric and mechanistic properties of RAP mixtures	-2 RAP sources; processed (3.6% binder of PG94-14) and unprocessed (4.95% binder of PG82-22) - One virgin binder; PG58-28 - 0%, 15%, 25% & 40% RAP contents	- VMA & VFA increase with RAP - RAP preheating time affect the VMA - 15% RAP increased stiffness|, 25% & 40% show similar stiffness as control mix - 15 RAP decreased creep compliance, 25% & 40% showed similar creep compliance as control mix - RAP mixtures show higher variability in compression - Finer gradation, Increased VMA & increased binder content reduce the effects of aged stiffer binder
Xiao et al. *2007)*	Investigation of the use of both RAP and crumb rubber (CR) in HMA mixes.	- Evaluate indirect tensile strength (ITS). - Rutting resistance under APA.	- Higher RAP% in mixes containing CR resulted in higher stiffness & ITS, indicating higher stability. - Increase in rubber content decreased ITS & creep stiffness. - CR effectively increased rutting resistance of mix. - Increasing % of rubber considerably improved ability of mixes to resist deformation.

2.6 PERFORMANCE OF RECYCLED ASPHALT PAVEMENT

Experience gained over the years by different states within USA has shown that asphalt pavement recycling is a technically viable rehabilitation technique (Kandhal & Mallick 1997). Properly designed recycled asphalt pavements have performed similar to and in many cases better than the conventional overlays. A literature search was conducted to collect and summarize the performance of recycled pavement projects in the different states within USA and elsewhere. The objectives of this search are to present the laboratory and field performance data on recycled asphalt pavements and the state-of-practice in recycling techniques in different places.

Hot mix asphalt recycling has been used extensively and routinely in the U.S. during the last several years. It is no longer considered as an experimental operation because recycled HMA pavements have generally performed equal to or better than conventional HMA pavements.

The relative performance of few recycled and conventional HMA pavements has been reported in the literature (Kandhal & Mallick 1997). Kandhal and Mallick mentioned a limited number of recycled projects (mostly constructed in the 1970s) in the FHWA report having excellent performance over the designed life. These projects were located in Florida, Georgia, Kansas, Louisiana, Massachusetts, Minnesota, Washington and Wyoming. This report also highlights the experiences of states for various type of recycling methods. In summary, the experience of the different states indicates that in most cases the performance of the recycled asphalt pavements has been superior to or comparable to conventional asphalt pavements. However, it was also observed that recycled pavements performed well only when:

- Good project selection criteria were followed
- Properly designed and constructed under good quality control and acceptance conditions

Kandhal & Mallick concluded that, as in the case of conventional asphalt pavements, recycled asphalt mixtures must be designed to meet proper specifications, produced with good quality control, and placed properly with no defects or irregularities.

A study was conducted by (Hall et al. 2001) to assess the relative performance of different flexible pavement rehabilitation treatments, including the influence of pretreatment condition and other factors. The Long-Term Pavement Performance Studies' SPS-5 and GPS-6B experiments data were used in this study. The rehabilitation treatments used in the SPS-5 experiment are two-inch and five-inch overlays, with virgin or recycled asphalt concrete mixes, and with or without pre-overlay milling. It was found that overlay thickness and pre-overlay roughness levels were the two most factors that influence the performance of asphalt overlays of asphalt pavements in the SPS-5 experiment, with respect to roughness, rutting, and fatigue cracking. Overlay mix type (virgin versus recycled) and pre-overlay preparation (with or without milling) had slight and inconsistent effects. Regarding virgin versus recycled overlays, no significant differences were found for various performances (fatigue, rutting and roughness), which means a comparable performance between virgin and recycled overlays.

Another study by (Kandhal et al. 1995) evaluated the performance of recycled pavements in Georgia. Performance of recycled pavements was compared with virgin asphalt pavements. No statistical difference in performance was found between two types of projects. Also, statistical tests indicated no difference between the mix properties including air voids, resilient modulus, indirect tensile strength, penetration and viscosity of the recovered binder. A study by Hossain et al. (Hossain et al. 1993) addressed eight test sections in Arizona. Long-term performance of recycled and virgin asphalt concrete overlays was observed. The results for roughness and cracking data analyses indicated that the recycled and virgin asphalt overlays have performed similarly. Also the annual maintenance costs for different test section were found similar.

The California Department of Transportation (Caltrans) conducted a pilot project for evaluating a pulverization process for asphalt concrete to use it as a base material (Bejarano et al. 2003). The process was evaluated in terms of constructability and pavement performance. The process provided lower initial costs than overlay with dig-outs in the wheelpath. Field and laboratory testing were conducted and the results of the investigation indicated that the pulverized base material is expected to perform better than the conventional base materials used in the study. The results also showed greatly improved properties of granular base materials, including this recycled material when the relative density is increased from 95 to 100 percent using Caltrans standard compaction.

The effects of milling and overlaying with a recycled asphalt layer are important factors for pavement engineers to consider when making decisions on rehabilitation and maintenance activities. In Texas, SPS-5 experiment sections were built 10 years ago to address this issue. Other research (Chen & Bilyeu 2000, Chen et al. 2003) has reported that performance data of SPS sections has provided valuable insight that could not have been achieved otherwise. They found that after more than 10 years of service, RAP sections perform as well as the virgin AC section. This indicated that the recycled AC can be effective when used properly. Also, little difference was found in terms of performance on milled and non-milled sections. They also observed that all SPS-5 sections were able to resist underlying cracking when a mixture of 30% RAP and lower viscosity AC was used, with a flexible mixture able to resist cracking. In contrast, cracks came through the remixed sections in just a few weeks for US175 and US84, where low penetration numbers in the range of 20-21 were found. A mixture with 75% RAP was found to be too stiff, because aged binder tends to become brittle, and consequently did a poor job of resisting cracking. Thus, it was concluded that high percents, RAP mixture should not be used on any location with cracking potential. Also, a lower viscosity binder should be added to the RAP to increase its flexibility.

In recent years, asphalt recycling has become a key component of the paving industry in Canada. Other work by (Emery et al. 1992, Emery 1993, Emery 1995) concluded that production of high-quality recycled hot mix incorporating a high content of reclaimed asphalt pavement requires a consistent processed RAP, appropriate new binder properties, appropriate mix design, proper hot-mix plant operations and quality control-quality assurance. For cold or hot in-place asphalt recycling, it was also emphasized the need for evaluation of the existing pavement conditions for suitability and selection of appropriate procedures and materials.

A report by Marquis et al. (Marquis et al. 2003) examined at the early performance of a FDR (incorporating foamed asphalt full depth reclamation) project in Maine. The objectives of this study was to evaluate ME DOT's foamed asphalt design and construction procedures and make appropriate recommendations for improving the performance of foamed asphalt reclaimed pavements. It was concluded that because of the variable nature of thick pavement layers, a high variation in density and asphalt content of foamed asphalt reclaimed layers can be expected, and that variation in density of layers can affect stiffness significantly. Moisture susceptibility testing with cyclic loading in water showed that foamed asphalt mixes were not inferior to emulsion plus lime mixes. Sections with thicker HMA showed significantly lower cost per percent increase in modulus. Back-calculation analysis with falling weight deflectometer deflection data from 10 month old pavement showed a modulus of 1,055 MPa for a foamed asphalt layer of 200 mm thickness, with 60 % recycled asphalt pavement material, 15 % granular material, 25 % crusher dust, 1.5 % cement and with 2.5 % foamed asphalt. Use of heavier rollers for the compaction of foamed asphalt reclaimed layers was recommended.

A study (Aljassar et al. 2005) to improve subgrade soil properties by utilizing RAP in Kuwait showed that mixing RAP with subgrade soils at a ratio of 1:3 by volume, improve the properties of the subgrade soil significantly, in terms of maximum dry density and CBR. However, no laboratory test was performed in this research to assess the performance of these mixes. It was concluded that the use of RAP will lead to a considerable cost savings in pavement rehabilitation projects as well as improved pavement performance.

As mentioned before, cold in-place recycling (CIR) is a pavement rehabilitation method that involves in-situ processes on an existing asphalt pavement i.e., sizes it, mixes in additional asphalt cement, and lays it back down without off-site hauling and processing. The added asphalt cement is typically emulsified asphalt. A recent development in CIR technology in Canada (Lane & Kazmierowski 2005) is the use of expanded (foamed) asphalt, rather than emulsified asphalt to bind the mix aggregates.

This combination of CIR and expanded asphalt technologies results in a 100 percent recycled material termed as cold in-place recycled expanded asphalt mix (CIREAM).

In the evaluation of pavement projects where CIR was employed Lane and Kazmierowski (Lane & Kazmierowski 2005) it was found that CIR mitigated reflective cracking and helped in extending pavement life. By reusing 100 percent of the existing aggregates and asphalt cement, CIR is both environmentally sustainable and cost-effective. CIREAM is a new development in CIR technology that appears to be a promising alternative to the conventional method. CIREAM placement resulted in a fairly smooth, hard, uniform surface suitable for temporary traffic, and provided an excellent platform for HMA paving operations. Short-term results indicated that the CIREAM with a 50 mm HMA overlay provided an equivalent performing pavement structure compared to conventional CIR with a 50 mm HMA overlay at a similar cost. Also, CIREAM appears to provide an acceptable in-place recycling/rehabilitation strategy that conserves resources and provides an economic alternative to conventional CIR, reducing the curing time, and extending the construction season.

In USA, CIR of existing hot mix asphalt materials has been an available for over twenty years. Morian et al. (Morian et al. 2004), evaluated the performance of CIR projects and materials over that period. A total of forty-four pavement sections were constructed by contractors in Northwestern Pennsylvania. Ninety additional sections were recycled as a part of maintenance activities. A subset of these projects was evaluated to determine performance characteristics and cost effectiveness of the treatment and the material. The results of this evaluation indicated that cold in-place recycling has provided an effective means of extending the life of the pavement for highways with up to 13,000

ADT and 200,000 annual equivalent single axle loads. Also cold in-place recycling has provided resistance against reflective cracking between two and three times that exhibited by conventionally resurfaced control sections. This form of recycling has been shown to be very cost effective as a rehabilitation strategy, historically approximately one to two-thirds the local cost of conventional hot mix asphalt material, while providing superior performance. They also concluded that when good construction process control is obtained, cold in-place recycled material develops stiffness values comparable to those expected for conventional hot mix asphalt materials. Also, cold in-place recycled material appeared to be typically a stress sensitive material, providing increased stiffness in response to increased load. This "flexibility" is likely an important factor in the observed good performance of these projects, and the delay in the development of reflective cracking.

CIR and FDR are two pavement rehabilitation strategies that the Nevada Department of Transportation (NDOT) has used for over 20 years (Bemanian et al. 2006). It was reported by Bemanian et al. that these strategies have resulted in saving of over $600 millions over the past 20 years for NDOT as compared to complete reconstruction costs. In addition, traffic interruptions were minimized during construction and natural resources were preserved. Based on the highway performance monitoring system data, NDOT has the highest percentage of its combined National Highway System Interstate and other roadways rated in the “good” category over any other state. The reason for this achievement is that NDOT used a proactive Pavement Management System (PMS) to prioritize its pavement preservation projects. A considerable amount of CIR and FDR rehabilitation work was used in conjunction with the proactive PMS. Since these strategies were more cost-effective than overlay, mill and overlay, or reconstruction, NDOT could rehabilitate more roads with less money. This study (Bemanian et al. 2006) also describes how to select, design, and construct successful CIR and FDR projects. The performances of the strategies were evaluated and life cycle cost analysis was developed to demonstrate the cost benefit of CIR and FDR over conventional rehabilitation strategies.

Full depth reclamation with foamed asphalt and plant mixed recycled asphalt pavement is being used for rehabilitation of asphalt pavements in Maine. Results of a study conducted by Mallick et al. (Mallick et al 2006), for determining layer moduli, comparing moduli from foamed asphalt and plant mixed recycled asphalt pavement layers, developing predictive equation, determining prevalent distress mechanisms and for recommending ways to obtain longer lasting reclaimed pavements indicated that:

- These pavements show non-linear behavior and the moduli can range from 1 GPa to 6 GPa, with most of them showing 1.5 GPa to 3 GPa.
- That the moduli of the foamed asphalt and the plant mixed recycled asphalt pavement layers do not differ significantly – the choice for the specific method depends on practical considerations such as need for realignment/grade change, availability of equipment and time available for applying surface Hot Mix Asphalt, HMA layer.
- Thickness of subbase and reclaimed layers, and binder content can be used for estimation of the moduli of a typical reclaimed layer.

- Thermal cracking was the most prevalent distress in these pavements, which can be prevented by adopting either suitable performance grade asphalt or by increasing the thickness of the HMA layer.
- The use of asphalt binder with low temperature grade of -34, instead of the currently used -28, was suggested for preventing premature thermal cracking in these reclaimed pavements.

Gerbrandt et al. (Gerbrandt et al. 2000) highlighted the dependence of Saskatchewan's (Canada) economic well being on heavyweight trucking industry. Consequently, the condition of the Province's road network is directly linked to its use by the trucking industry. It is to the Province's advantage to research and develop long term preservation strategies with a high benefit to cost ratio to keep the road network in the best state possible. CIR was found to be an attractive alternative for highway rehabilitation because of its economic and structural strengthening advantages. The top of the road surface is milled, recycled with an emulsion and reapplied to the roadbed with an overlay to create a new driving surface. They found that this mixture continues to cure and strengthen structurally as the cohesion increases, CIR mixtures have larger modulus values and significantly greater fatigue lives than standard hot bituminous mixtures and are able to withstand heavier loadings. They also reported that other jurisdictions in Canada have found that CIR is a cost effective rehabilitation method because no aggregate material needs to be transported; it conserves aggregate resources and eliminates many types of paving distresses. Their experience also showed that CIR is an environmentally friendly process that causes a minor inconvenience to the trucking industry when being conducted. The original crown and cross slope of the roadway can also be restored while engineering costs such as design and surveying are reduced.

Lewis and Collings' (Lewis & Collings1999) report of South Africa found that cold in place recycling process is gaining recognition and popularity in that part of the world because it is a cost-effective means of rehabilitating distressed road pavements. Their experience showed that this process requires the use of specially designed recycling machines. The process is carried out in a single-pass operation, enabling a high rate of production to be achieved. It was reported that one of the main benefits of the cold in place recycling process is that the material in the existing distressed road pavement is simultaneously recycled and mixed with the stabilizing agent, enabling the road pavement to be strengthened without the need to import expensive aggregate.

For designing asphalt mix containing RAP, it is essential to determine the properties of both binder and aggregates in RAP. The binder properties in RAP depend on its original grade and aging process over time. Therefore, it is vital to determine these modified binder properties in RAP as these will affect the mixture design attributes.

CHAPTER 3

DATA COLLECTION

Several types of data were collected and analyzed to evaluate the existing pavement surface and structural conditions and to characterize the recycled asphalt materials. The pavement condition data were collected in the field investigation part of the study and the characteristics of the materials were determined in the laboratory investigation. In this chapter the field investigation will be discussed first, followed by the laboratory investigation.

3.1 FIELD INVESTIGATION

The field investigation was designed and completed to evaluate the functional and structural characteristics of the pavements. The field data were analyzed to determine, (a) the quality of the existing pavement, (b) whether or not the pavements are in need of rehabilitation, and (c) if recycling can be used as one of the rehabilitation option. For recycling, the field investigation must be combined with the laboratory investigation for the proper selection of the recycling method and for the recycled asphalt pavement (RAP) hot mix asphalt (HMA) design. In this study, the field investigation comprised of surface distress surveys, pavement coring, longitudinal/transverse profiles and non-destructive deflection tests. These investigations are essential in order to determine the rehabilitation needs, cross-section, ride quality, seat of rutting and structural capacity of the pavement.

3.1.1 Site Selection

In cooperation with the Pakistan National Highway Authorities (NHA) and the USA and Pakistan research teams, twelve asphalt concrete (AC) and thirteen surface treated test sites were selected for this study. The twenty-five test sites represent spectrums of pavement thicknesses, pavement types, pavement surface age and traffic.

Figure 3.1 and Table 3.1 show the location and some parameters of each selected test site. It should be noted that this thesis addresses the AC sites only. The data for the thirteen surface treated sites can be found elsewhere (Baladi et al. 2008). Nevertheless, all asphalt surfaced test sites are located along the national and provincial highway networks in Pakistan.

Table 3.1 Parameters of the selected test sites

Site Number	Site	Age	Traffic	Location
Asphalt concrete test site				
1	N5N-1	< 7 years	Heavy	Nowshera
2	N5N-2	< 7 years	Heavy	Nowshera
3	RRW-3	>7 years	Heavy	Peshawar
4	N5S-4	> 7 years	Heavy	Mandra
5	N5N-5	>7 years	Heavy	Mandra
6	N5S-6	> 7 years	Heavy	Wazirabad
7	N5N-7	< 7 years	Heavy	Sukkar
8	N5N-8	> 7 years	Heavy	Sukkar
9	MNN-9	> 7 years	Light	Murree
10	MJS-10	< 7 years	Light	Murree
11	MLN-11	< 7 years	Light	Murree
12	M2S-12	> 7 years	Medium	Chakri
Surface treated test site				
13	ST-1		Heavy	Nowshera
14	ST-2		Light	Risalpur
15	ST-3		Heavy	Mardan
16	ST-4		Light	Barabanda
17	ST-5		Heavy	Mangla
18	ST-6		Light	Guliana
19	ST-7		Light	Kharian
20	ST-8		Light	Murree
21	ST-9		Heavy	Murree
22	ST-10		Light	Sukkar
23	ST-11		Heavy	Sukkar
24	ST-12		Light	Chakri
25	ST-13		Heavy	Chakri

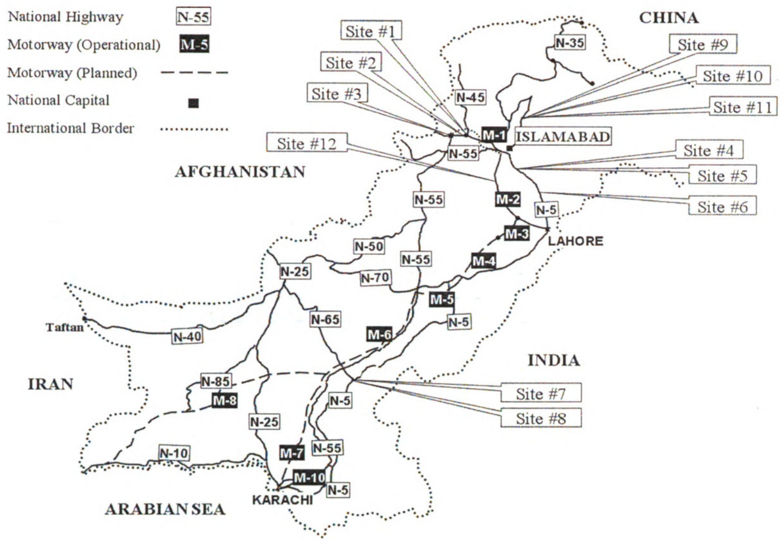


Figure 3.1 Map of Pakistan showing the location of the selected AC test sites

As listed in Table 3.1, each test site is designated by a highway number, traffic direction and site number. For example, the first site along the northbound road on National Highway N5 is designated N5N-1 where the last number represents the site number. All test sites were selected on a 1600 ft straight segment of the road and away from any curve. Each test site was divided into three test sections of 500 ft (150 m) long separated by 30 ft (10 m) distance as shown in Figure 3.2.

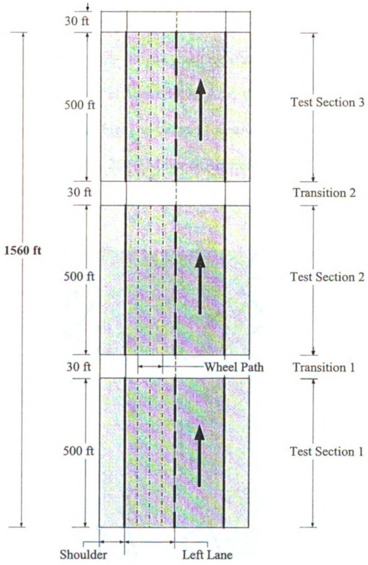


Figure 3.2 Detailed sections of an asphalt concrete site

3.1.2 Pavement Surface Distress Survey

All asphalt concrete (AC) test sites were visually surveyed for surface distresses. The severity and extent information of the following distresses were collected and recorded onto distress maps (an example is shown in Appendix A):

- Cracking (fatigue, transverse, longitudinal, and block)
- Potholes
- Patching

The site distresses were also photographed, for example Figure 3.3 is a photograph showing high severity fatigue cracking along test site 6 (N5S-6), other distress photographs are shown in Appendix A. After the distress maps were produced, the severity (low, medium and high) and extent (occasional, frequent and extensive) levels were established (ODOT 2004; Miller & Bellinger 2003). Table 3.2 represents the criteria for the different severity and extent levels of this study.

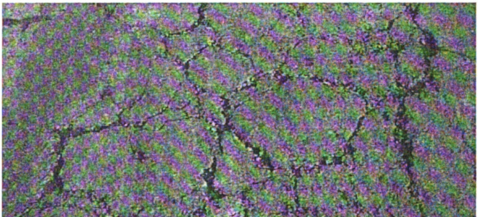


Figure 3.3 High severity fatigue cracking, N5S-6 (test site 6) (Wazirabad)

Table 3.2 Criteria used for the severity and extent levels for different distresses

Distress	Severity			Extent		
	L	M	H	O	F	E
Raveling	Slight loss of sand	Open texture	Rough or pitted	<20%	20-50 %	> 50%
Bleeding	Not rated	Bit and agg visible	Black surface	<10%	10-30 %	> 30%
Pathcing	< 1 ft^2	< 1 yd^2	> 1 yd^2	< 10/mile	10-20/mile	>20/mile
Rutting	⅛" - ⅜"	⅜" - ⅝"	>⅝"	<20%	20-50 %	> 50%
Potholes	Depth < 1" Area < 1yd^2	< 1", >1yd^2 >1",<1yd^2	> 1" and > 1yd^2	< 5/mile	5-10/mile	> 10/mile
Longitudinal Cracking	Single, <¼" No spalling	Single/multiple, ¼" - ¾" Some spalling	Multiple, > ¾" Spalling	< 50' per 100'	50-150' per 100'	>150' per 100'
Transverse Cracking	< ¼ "	¼ to ¾ "	> ¾"	<20%	20-50 %	> 50%
Block Cracking	> 6' by 6'	6' by 6' to 3' by 3'	< 3' by 3'	<20%	20-50 %	> 50%
Fatigue Cracking	Single/multiple cracks < ¼"	Multiple cracks > ¼"	Alligator > ¼" with spalling	<20%	20-50 %	> 50%
Edge Cracking	Tight, < ¼"	> ¼ " some spalling	> ¼" moderate spalling	<20%	20-50 %	> 50%

A summary of the distresses for each AC site was made using the criteria in Table 3.2. A distress summary is listed in Table 3.3.

Table 3.3 Distress summary for AC sites (severity/extent)

Site number	Site	Fatigue cracking	Block cracking	Longitudinal cracking	Transverse cracking	Potholes	Patches
1	N5N-1	L / O	H / O	L / F	L / O	L / O	M / O
2	N5N-2	L / O	H / O	L / F	L / F	L / O	M / O
3	RRW-3	-	-	L / O	-	-	-
4	N5S-4	L / E	M / F	M / O	L / O	L / O	L / O
5	N5N-5	L / O	-	L / O	-	-	M / O
6	N5S-6	H / E	M / F	L / F	M / F	L / O	L / O
7	N5N-7	-	-	-	-	-	-
8	N5N-8	L / O	L / O	M / 0	-	L / O	M / O
9	MNN-9	M / F	-	L / F	L / O	L / O	-
10	MJS-10	L / F	L / O	M / F	L / F	L / O	L / O
11	MLN-11	L / O	-	L / O	L / O	M / O	-
12	M2S-12	-	-	-	-	-	-

Note: L = low, M = medium, H = high; O = occasional, F = frequent, E = extensive (See Table 3.2)

3.1.3 Pavement Sampling and Cross-Section

The pavements of the twelve AC sites were sampled by taking cores. For each section within an AC site, five cores were extracted at the locations shown in Figure 3.4. Three cores were taken from the left wheel-path (LWP), while one core was taken from each the center of the lane (COL) and the right wheel-path (RWP). The cores were used to measure the thickness of the AC layer and to examine the crack depth and any visible distresses within the core. Table 3.4 summarizes the measured AC thickness and crack depth. Photographs of the cores were also taken as a reference. A photograph of a core can be seen in Figure 3.5, other photographs can be seen in Appendix A.

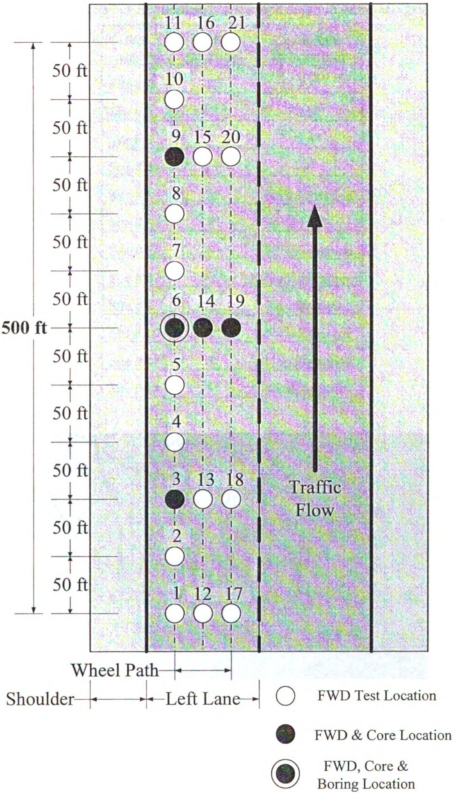


Figure 3.4 FWD test, coring and boring locations for an AC section

Table 3.4 AC thickness and crack depth from cores

Site number	AC thickness	Crack depth from surface
Site # 1	4"	2.5"
Site # 2	4.5"	2.5"
Site # 3	5"	2.5"
Site # 4	6.5"	3"
Site # 5	6"	No information
Site # 6	8"	3 to 5"
Site # 7	6"	No information
Site # 8	7.5"	No information
Site # 9	4.5"	No information
Site # 10	8"	No information
Site # 11	5"	No information
Site # 12	5"	No information

Figure 3.5 Core from test site 4 (N5S-4), section 1 (LWP)

At one location for every AC test section, a bore hole was conducted (its location is shown in Figure 3.4) which extracted one foot into the roadbed soil beneath the pavement. From the bore sample, the type of roadbed soil beneath the pavement was determined. The thickness of the aggregate subbase and base was obtained either from the bore hole or from available cross-section records. Each test site was visited to determine whether the site was on a cut or fill section. The roadbed soil type, base/subbase layer thickness and the embankment height of all test sites are summarized in Table 3.5.

Table 3.5 Layer thicknesses, roadbed soil type and embankment height

Site number	Thickness (inches)		Roadbed soil type	Embankment height(ft)
	HMA	Base/subbase		
1	4	10.5	Clay	3 (Fill)
2	4.5	13.5	Clay	3 (Fill)
3	5	20	Silty Sand	5 (Fill)
4	6.5	20	Silty Sand	4 (Fill)
5	6	20	Gravel	4 (Fill)
6	8	14	Silty Sand	4 (Fill)
7	6	40	Sand	5 (Fill)
8	7.5	22	Clay	4 (Fill)
9	4.5	10	Silty Clay	0 (Cut)
10	8	12	Silty Clay	0 (Cut)
11	5	16	Clay	0 (Cut)
12	5	19.5	Sand	0 (Cut)

3.1.4 Longitudinal Profiles

An AMES profilograph, shown in Figure 3.6, was used on test sites 1 through 6 to determine the longitudinal profile (ride quality) of the pavement along the outer wheel path (left wheel path). The profilograph consists of a 25 ft beam supported by several wheels at each end and one at the center. The vertical deviation (movement) of the center wheel against the reference plane (beam) was recorded. The data from the profilograph was used with a blanking width of 0 mm to calculate the profile index (PI_0) of the test sections. The International Roughness Index (IRI) was then calculated using the PI_0 data and Equation 1 (Smith et al. 2002). The calculated PI_0 and IRI for test sites 1 through 6 are listed in Table 3.6.

$$IRI(mm/km) = 2.66543 \times PI_0 + 213.01 \tag{1}$$

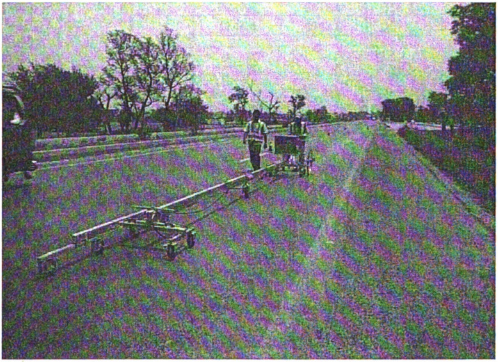


Figure 3.6 AMES profilograph

Table 3.6 Ride quality information for test sites 1 through 6

Site Number	Section	PI_0	IRI	
			mm/km	in/mile
1	1	1348.3	3806.9	241.4
	2	1386.2	3907.8	247.8
	3	1336.9	3776.4	239.4
2	1	1966.7	5455.0	345.9
	2	1860.9	5173.2	328.0
	3	1599.3	4475.9	283.8
3	1	881.8	2563.3	162.5
	2	949.7	2744.3	174.0
	3	643.7	1928.8	122.3
4	1	1547.3	4337.3	275.0
	2	1560.5	4372.5	277.2
	3	2371.5	6534.1	414.3
5	1	1686.8	4708.9	298.6
	2	1789.5	4982.7	315.9
	3	1174.5	3343.6	212.0
6	1	2662.9	7310.8	463.5
	2	2027.8	5618.0	356.2
	3	1146.4	3268.6	207.2

3.1.5 Transverse Profiles

Transverse profiles were measured to determine the rut depth at test sites that showed signs of rutting (test sites 1 through 8 and 12). The transverse profiles were measured with a 14 ft straight edge every 50 ft (15 m) along the sections. Measurements were taken at 12 inch intervals in the transverse direction of the lane. Table 3.7 lists an example of the rut depth measurements taken at test site 7 (N5N-7). The measurements were then plotted along the width of the lane to express a transverse profile, as shown in Figure 3.7. The transverse profiles for test sites 1 through 8 and 12 are shown in Appendix B. The severity and extent of rutting for the test sites using the criteria in Table 3.2 are summarized in Table 3.8.

Table 3.7 Rut depth measurements for test site 7, section 1

Station (ft)	Rut Depth (inches) along Lateral Distance (ft)												
	0	1	2	3	4	5	6	7	8	9	10	11	11.7
0	0	0.01	0.01	-0.43	-0.63	-0.36	-0.09	-0.09	-0.26	-0.42	-0.70	-0.67	0
49.2	0	0.03	-0.05	-0.41	-0.50	-0.31	0.00	-0.28	-0.37	-0.57	-0.78	-0.59	0
98.4	0	0.04	-0.16	-0.52	-0.41	-0.21	-0.06	-0.30	-0.38	-0.66	-0.70	-0.27	0
147.6	0	0.07	-0.13	-0.45	-0.34	-0.11	-0.08	-0.36	-0.33	-0.53	-0.66	-0.31	0
196.9	0	0.04	-0.28	-0.84	-0.76	-0.37	-0.06	-0.30	-0.38	-0.58	-0.70	-0.31	0
246.1	0	0.11	-0.06	-0.50	-0.59	-0.25	-0.10	-0.03	-0.16	-0.68	-0.57	-0.23	0
295.3	0	0.11	-0.22	-0.62	-0.67	-0.17	-0.10	-0.27	-0.36	0.03	-0.81	-0.35	0
344.5	0	0.04	-0.05	-0.40	-0.45	-0.13	0.02	-0.14	-0.34	-0.54	-0.62	-0.27	0
393.7	0	0.11	-0.06	-0.35	-0.43	-0.13	0.06	0.05	-0.24	-0.48	-0.81	-0.43	0
442.9	0	0.14	0.01	-0.36	-0.25	0.01	0.08	-0.09	-0.19	-0.48	-0.65	-0.19	0
492.1	0	0.00	-0.08	-0.39	-0.43	-0.16	0.00	-0.04	-0.12	-0.39	-0.47	-0.16	0

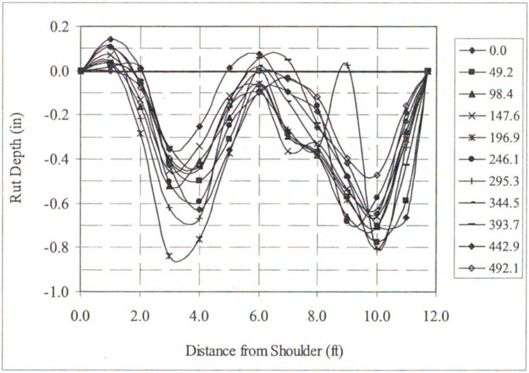


Figure 3.7 Transverse profile for test site 7, section 1

Table 3.8 Severity and extent of rutting on the test sites

Site Number	Site	Section	Rutting
1	N5N-1	1	L / E
		2	L / E
		3	L / F
2	N5N-2	1	M / E
		2	M / E
		3	M / O
3	RRW-3	1	M / E
		2	None
		3	None
4	N5S-4	1	M / E
		2	M / O
		3	M / O
5	N5N-5	1	M / E
		2	M / E
		3	M / E
6	N5S-6	1	M / O
		2	M / F
		3	M / E
7	N5N-7	1	H / E
		2	H / E
		3	H / E
8	N5N-8	1	M / E
		2	M / E
		3	M / F
9	MNN-9	1, 2, 3	None
10	MJS-10	1, 2, 3	
11	MLN-11	1, 2, 3	
12	M2S-12	1	H / E
		2	H / E
		3	H / E

3.1.6 Non-destructive Deflection Testing

Following the research plan, non-destructive deflection tests (NDT) were also conducted using a KUAB falling weight deflectometer (FWD) as a part of the field investigation. All deflection data and the pavement surface and air temperatures were measured and recorded by an on-board computer. NDT results were used to: (a) study the linearity of the pavement response to different load levels and (b) analyze its structural capacity. The sensor spacing of the KUAB FWD is listed in Table 3.9 and is shown in Figure 3.8.

Table 3.9 Sensor spacing and load levels of KUAB FWD

Load Levels (lbs)	Spacing of the FWD Deflection Sensors (inches)								
	D0	D1	D2	D3	D4	D5	D6	D7	D8
6000, 9000, 15000	0	8	12	18	24	36	48	60	72
9000*	0	8	12	18	24	36		60	

*SHRP sensor spacing and load level, and spacing used for backcalculation

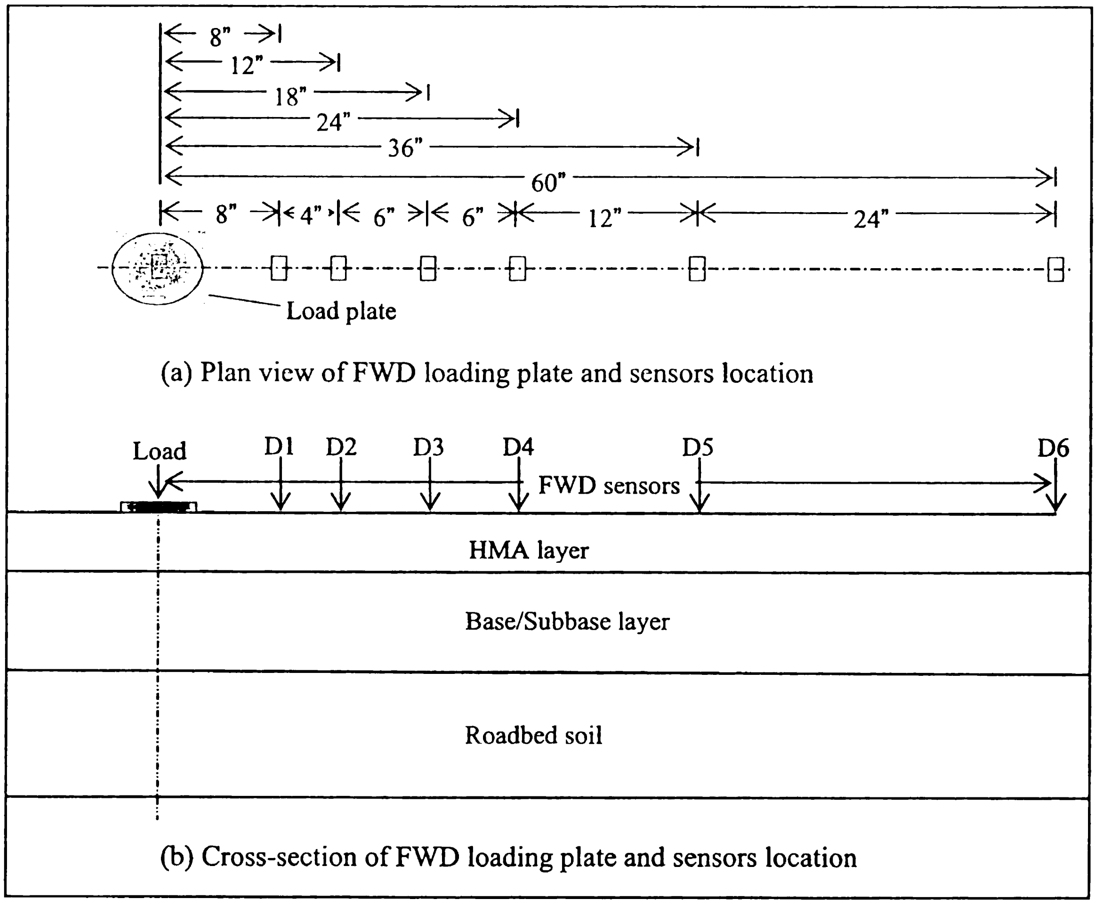


Figure 3.8 Details of the FWD sensor layout and designation

The National Highway Authority (NHA) in Pakistan was requested to conduct all NDT on the twelve AC test sites. For each test site, 21 test locations were marked as seen in Figure 3.4. For each test location, one FWD test was conducted at each load level of 6,000, 9,000, and 15,000 lbs. Each test consisted of three drops, thus a total of 9 drops were conducted at each location. Results from the three load levels were used to study the linearity of the pavement response to load and thus to select the proper analysis model.

An example of the average deflection data for the 9,000 lb load level for test site 1 (N5N-1) can be seen in Table 3.10. The complete data from the FWD tests for every AC site can be seen in Appendix C, Tables C-1 through C-34.

Table 3.10 Measured pavement deflection due to a 9000 lb load for test site 1 (N5N-1), section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)						
		0	7.87	11.81	17.72	23.62	35.43	47.24
		D0	D1	D2	D3	D4	D5	D6
LWP	0.0	13.18	9.25	6.07	3.77	2.53	1.16	0.82
	49.2	13.39	10.52	7.63	5.38	3.74	1.82	1.21
	98.4	15.61	10.99	7.61	5.31	3.64	1.87	1.26
	147.6	8.76	8.03	5.96	4.15	2.81	1.43	1.03
	196.8	12.86	9.02	6.06	3.95	2.79	1.43	1.04
	246.0	9.71	7.09	4.81	3.19	2.18	1.16	0.87
	295.2	13.03	9.83	6.68	4.59	3.09	1.50	0.97
	344.4	11.28	8.31	5.57	3.66	2.51	1.21	0.86
	393.6	13.14	9.77	6.81	4.62	3.02	1.39	0.91
	442.8	11.38	8.37	5.74	3.84	2.56	1.23	0.87
	492.0	11.13	8.54	5.83	3.85	2.55	1.28	0.92
COL	0	12.20	8.34	4.97	3.29	2.33	1.27	0.95
	98.4	13.48	9.60	6.47	4.50	3.17	1.70	1.22
	246	9.88	7.23	4.97	3.32	2.26	1.24	0.93
	393.6	11.52	7.88	4.92	3.33	2.33	1.24	0.91
	492	8.49	7.16	5.05	3.45	2.39	1.32	1.00
RWP	0	11.71	7.89	5.21	3.59	2.48	1.33	0.97
	98.4	13.76	8.85	5.46	3.85	2.72	1.51	1.12
	246	11.19	7.72	5.02	3.45	2.51	1.42	1.05
	393.6	12.41	8.07	5.00	3.34	2.24	1.24	0.96
	492	12.01	8.33	5.59	3.71	2.52	1.36	1.01

3.2 LABORATORY INVESTIGATION

The main objective of the laboratory investigation was to characterize the virgin and the recycled HMA material. The recycled material was made by blending virgin and recovered material (old HMA pavements). Once the characteristics of these materials were known, the impact of the recovered material on the performance of recycled asphalt mixture was determined. The ultimate goal is to create a recycled asphalt pavement that meets the requirements, specifications and performance of a virgin asphalt pavement.

The laboratory investigation addressed both the traditional (Marshall) and the new (Superpave) mix design standards. A detailed chart summarizing the sequence of laboratory tests can be seen in Figure 3.9. The laboratory investigation of the virgin material will first be discussed. In the subsequent section, the properties of the recovered material will be stated. Finally the properties of the blended asphalt material will be presented.

3.2.1 Characteristics of Virgin Binder

Characteristics of the virgin binder were determined for six available binders in Pakistan. Three of the binders were obtained from the Attock Refinery Limited (ARL) in Rawalpindi and the other three from the National Refinery Limited (NRL) in Karachi. These two refineries are the only sources of asphalt binder in Pakistan. ARL is using local heavy crude oil whereas NRL is processing Arabian Light crude oil to produce asphalt binder. One of the binders obtained from ARL was a modified binder containing an Elvaloy polymer of 1.35%. The asphalt binders acquired from ARL and NRL are shown in Table 3.11.

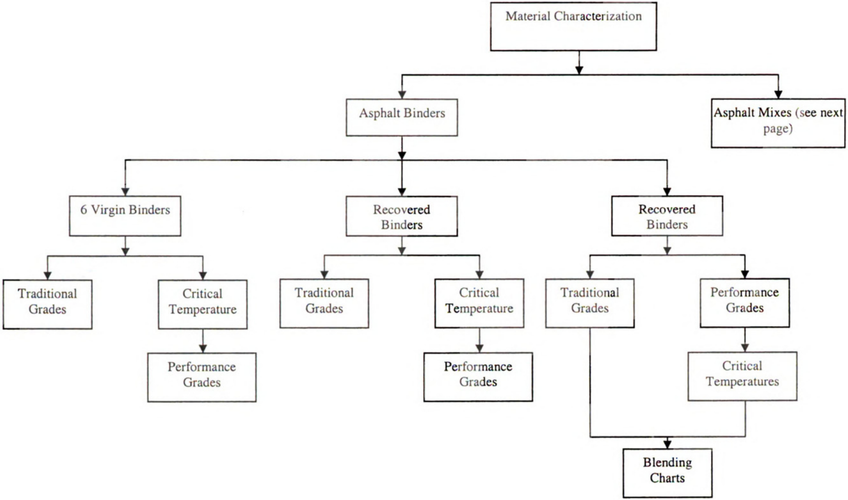


Figure 3.9 Details of material testing and objectives

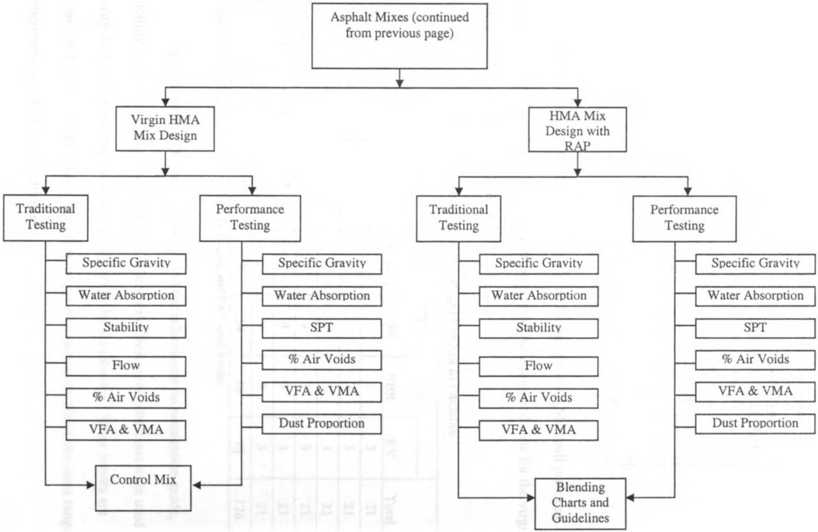


Figure 3.9 (continued)

Table 3.11 Available binder grades in Pakistan

Binder Source	Binder Type (penetration grades)
Attock Refinery Limited (ARL)	60/70
	80/100
	60/70 (P)[1]
National Refinery Limited (NRL)	40/50
	60/70
	80/100

1: Polymer additive binder with 1.35% Elvaloy

Binder tests were performed according to the conventional Marshall protocols and the new practice Superpave protocols. The experiment for completed tests for the virgin asphalt binder can be seen in Table 3.12.

Table 3.12 Test matrix for virgin binders in Pakistan

Binder Source	Binder Grade	Test					Total
		Penetration	Ductility	DSR	BBR	RV	
ARL	60/70	3	3	91	3	3	21
	80/100	3	3	9	3	3	21
	60/70(P)	3	3	9	3	3	21
NRL	40/50	3	3	9	3	3	21
	60/70	3	3	9	3	3	21
	80/100	3	3	9	3	3	21
Total		18	18	54	18	18	126

Note: DSR will be performed on fresh, RTFO aged, and PAV aged binder

The conventional testing results yielded the information on penetration grade, ductility and flashing and softening points. The results from penetration test were used to verify the binder's penetration grades produced by local refineries. These results are shown in Table 3.13. The results show that the penetration values fall in the same ranges which were specified by the refineries.

Table 3.13 Penetration grade verification

Binder Source	Supplier Grade	Penetration Grade			
		1	2	3	Average
ARL	60/70	65	69	64	66
	80/100	81	89	91	87
	60/70 (P)	28	29	27	28
NRL	40/50	44	43	43	43
	60/70	62	64	65	64
	80/100	93	90	92	92

The results of the remaining conventional tests are shown in Table 3.14. The flash points of all the asphalts satisfied the minimum criteria of 230°C by AASHTO M 320. All ductility test results satisfied the AASHTO T51 standard. Similarly, the softening point results also corresponded well with the respective binder grades.

Table 3.14 Conventional binder test results

Binder Source	Binder Grade	Penetration Value (Verification) (ASTM D5)[1]	Softening Point (°C) (AASHTOT53)[1]	Ductility (cm) (AASHTO T51)[1]	Flash Point (°C) (AASHTO T48)[1]
ARL	60/70	66	66.5	>100	255
	80/100	87	51	>100	269
	60/70 (P)	28	68.25	>100	249.5
NRL	40/50	43	79	>100	293
	60/70	64	62	>100	310
	80/100	92	48	>100	295

1 Average of 2-3 tests

The same asphalt binders which were subjected to the conventional tests were also subjected to performance testing. These performance tests were conducted in order to determine the PG grade of the binder under Superpave testing protocols. These performance tests consisted of the Dynamic Shear Rheometer (DSR), Bending Beam Rheometer (BBR) and Rotational Viscometer (RV) tests.

The DSR test was performed on the virgin binders of the different grades to establish their rheological properties (dynamic shear modulus (G*) and phase angle (δ)). The binders were then aged in rolling thin film oven (RTFO) and pressure aging vessel (PAV) to simulate asphalt binder short-term and long-term aging, respectively. The DSR tests were conducted to determine the aged binders' rheological properties. The critical high temperature which meet the Superpave specification for the DSR tests are shown in Table 3.15.

Table 3.15 Critical high temperature of the virgin binders

Property	Critical Temperatures					
	ARL 60-70	ARL 80-100	ARL 60-70 (P)	NRL 40-50	NRL 60-70	NRL 80-100
Original G*/Sinδ $\geq$ 1.00 kPa	62.09	56.38	72.39	72.6	66.56	60.24
RTFO G*/Sinδ $\geq$ 2.20 kPa	62.35	59.59	73.8	72.36	67.3	60.04
PAV G*×Sinδ $\leq$ 5000 kPa	22.21	19.78	23.76	27.83	22.46	19.75

Note: Average value of 2-3 tests

The BBR was used to measure how much a beam, which was made of asphalt binder, deflects under a constant load at a constant temperature. The asphalt binder from the PAV was used in this test. The test was performed at four temperatures (-6,-12,-18, and -24°C). The creep stiffness (s-value) and slope (m-value) were determined from the tests and are listed in Tables 3.16 and 3.17.

Table 3.16 BBR results at 60 seconds for ARL binders

		ARL60/70		ARL80/100		ARL60/70(P)	
	Temp (°C)	1	2	1	2	1	2
Stiffness (MPa)	-6	45.27	46.14	46.35	47.83	58.6	82.28
	-12	105.5	103.55	82.6	99.62	239.02	163.31
	-18	218.76	226.21	154.18	206.52	447.44	464.04
	-24	456.04	412.27	301.15	277.03	-	-
m-value	-6	0.433	0.449	0.457	0.43	0.44	0.402
	-12	0.414	0.383	0.342	0.394	0.32	0.28
	-18	0.256	0.23	0.28	0.295	0.26	0.22
	-24	0.216	0.174	0.169	0.167	-	-

Table 3.17 BBR results at 60 seconds for NRL binders

		NRL40/50		NRL60/70		NRL80/100	
	Temp (°C)	1	2	1	2	1	2
Stiffness (MPa)	-6	89.44	99.54	79.92	49.12	44.22	29.72
	-12	236.98	251.56	159.13	178.44	127.29	143.07
	-18	321.66	347.84	287.49	307.43	257.04	200.9
	-24	472.88	346.09	295.86	435.11	350.74	465.98
m-value	-6	0.435	0.444	0.415	0.542	0.434	0.471
	-12	0.319	0.308	0.307	0.392	0.392	0.381
	-18	0.182	0.217	0.246	0.219	0.329	0.276
	-24	0.131	0.079	0.092	0.108	0.117	0.146

The RV tests were performed on all six asphalt binders at a speed of 20 rpm and at two temperatures, 135 °C and 165 °C. The viscosity was determined in centipoise (cp) and the results obtained are listed in Tables 3.18 and 3.19.

Table 3.18 RV results for 135°C

		Viscosity (cp)			
Source	Grade	1	2	3	Average
NRL	40/50	1481.4	1343	1229.2	1351.2
	60/70	291.4	289	294.2	291.6
	80/100	255	255	251.5	253.8
ARL	60/70	242.9	238	241.9	240.9
	80/100	235.4	223.1	219.2	225.9
	60/70 (P)	1007.9	969	935	970.6

Table 3.19 RV results for 165°C

Source	Grade	Viscosity (cp)			
		1	2	3	Average
NRL	40/50	170	170	163.5	167.8
	60/70	133.6	127.6	127.5	129.5
	80/100	97.1	102	98.1	99.1
ARL	60/70	85	76.5	78.5	80
	80/100	72.9	66.8	66.8	68.8
	60/70 (P)	206.4	204	202.7	204.4

3.2.2 Characteristics of Virgin Aggregate

Of the various available aggregate sources in Pakistan, two dominant sources were selected as apart of this research. Aggregate samples were collected from Margalla (limestone) and Deena (quartzite) quarries. These two sources are considered to be the largest aggregate quarries in Pakistan. Two separate gradations, one for the wearing course (¾ inch (19 mm) nominal maximum aggregate size (NMSG)) and one for the base course (1 inch (25 mm) NMSG) were selected to be control gradations for the mixes. The control aggregate gradations are listed in Table 3.20 and the curves for the wearing and base course are shown in Figures 3.10 and 3.11, respectively. Since two gradations were chosen and two sources of aggregates are used, a total of 8 mixes were designed. Four mixtures for each of the two mix design procedures (Marshall and Superpave) were used.

Specific gravity and absorption tests for coarse (AASHTO T85-91, ASTM C127) and fine (AASHTO T84-95, ASTM C128) aggregates from Margalla (limestone) and Deena (quartzite) were conducted. The bulk (G_{sb}) the saturated surface dry (G_{ssd}) and the apparent (G_{sa}) specific gravities were determined. The specific gravities of the coarse

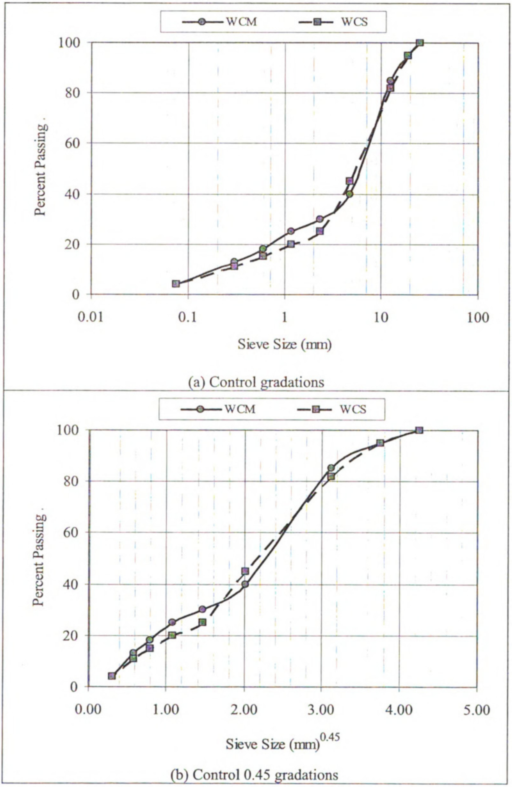


Figure 3.10 Wearing course control gradations (Marshal and Superpave) – 19 mm nominal maximum aggregate size

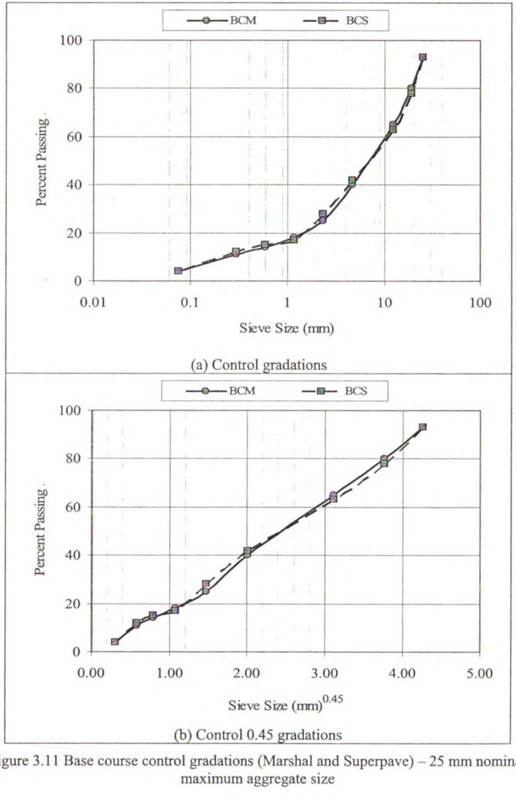


Figure 3.11 Base course control gradations (Marshal and Superpave) – 25 mm nominal maximum aggregate size

Table 3.20 Control gradation for wearing and base courses

Sieve Size (mm)	NMSG ¾ inch (19 mm)		NMSG 1 inch (25 mm)	
	Passing (%)			
	WCM	WCS	BCM	BCS
37.5	-	-	100	100
25	100	100	93	93
19	95	95	80	78
12.5	85	82	65	63
4.75	40	45	40	42
2.36	30	25	25	28
1.18	25	20	18	17
0.6	18	15	14	15
0.3	13	11	11	12
0.075	4	4	4	4

Note: WC = Wearing Course, BC = Base Course, S = Superpave, M = Marshall

aggregates (below 1/2" to above the #4 sieve) and the fine aggregates (below the #4 sieve to above the #200 sieve) were averaged based on the proportions of each sieve size in the proposed aggregate gradation. The proportions used in the gradation were as follows: 60% coarse and 40% fine. A summary of the results for both aggregate sources is shown in Table 3.21. The bulk specific gravity was averaged out based on the appropriate proportion (combined G_{sb}) and was used in determining the volumetric properties of mixtures which are further discussed in subsequent sections.

Table 3.21 Coarse and fine aggregate specific gravities and absorption

Course Type	Agg. Sources	Agg. Type	G_{sa}	G_{ssd}	G_{sb}	Combined G_{sb}	% Absorption
Wearing	Margalla	Coarse	2.705	2.686	2.675	2.653	0.412
		Fine	2.730	2.660	2.621		1.520
	Deena	Coarse	2.743	2.717	2.702	2.685	0.544
		Fine	2.746	2.691	2.659		1.180
Base	Margalla	Coarse	2.705	2.684	2.672	2.651	0.44
		Fine	2.73	2.66	2.621		1.52
	Deena	Coarse	2.764	2.738	2.723	2.697	0.541
		Fine	2.746	2.691	2.659		1.18

The L.A. abrasion test and soundness test (ASTM C88) were also conducted on both aggregate materials. The results of L.A. abrasion test are shown in Table 3.22 whereas Table 3.23 shows the soundness test results.

Table 3.22 Los Angeles abrasion test (ASTM C131-03, AASHTO T 96-87)

Aggregate Source	Abrasion Value (%)	Specification
Margalla (limestone)	16.74	< 30 %
Deena(quartzite)	17.46	< 30 %

Table 3.23 Soundness test results (ASTM C88)

Aggregate Source	Weighted Average (%)	
	Coarse	Fine
Margalla (limestone)	1.526	9.815
Deena (quartzite)	0.458	7.098

3.2.3 Characteristics of Recovered and Recycled Asphalt Binder

The characteristics of the recovered asphalt binder were determined from sources of old HMA pavements. The old HMA pavement (recovered material) was obtained from two sources located near the cities of Mandra and Nowshera along national highway N-5. The binder was extracted from samples that were obtained from the two sources and tested according to conventional and Superpave mix design protocols. The characteristics of the recovered binder must be determined such that when the recovered binder is blended with the virgin binder, forming the recycled binder, it can achieve the desired properties.

The recovered asphalt pavement was separated into a surface portion (¾" NMSG) and a base portion (1" NMSG). In order to assess the binder in the recovered material, asphalt binder extraction was done according to AASTHO T 160, "Quantitative

Extraction of Bitumen from Paving Mixtures" and AASTHO T 170, "Recovery of Asphalt from Solution by Abson Method". It was found that the average asphalt content for the recovered asphalt material of Mandra was 5.1% and for Nowshera was 5.6%. More detailed results can be seen in Appendix D.

Binder blending charts were developed, by blending the recovered binder with the ARL 60/70 virgin binder. A testing matrix was developed, which is shown in Table 3.24, involves laboratory testing outlined in the conventional (penetration and ductility) and Superpave (DSR, BBR and RV) mix design procedures.

Table 3.24 Testing matrix for recovered and blended binders

RAP Source	Ext. Binder (%)	Test					Total
		Penetration	Ductility	DSR	BBR	RV	
Nowshera	100	3	3	3	3	3	15
	90	3	3	3	3	3	15
	80	3	3	3	3	3	15
	70	3	3	3	3	3	15
	40	3	3	3	3	3	15
	0	3	3	3	3	3	15
Mandra	100	3	3	3	3	3	15
	90	3	3	3	3	3	15
	80	3	3	3	3	3	15
	70	3	3	3	3	3	15
	40	3	3	3	3	3	15
	0	3	3	3	3	3	15
Total		36	36	36	36	36	180

The results of the penetration test involving binder blends from the virgin and recovered binder (Mandra and Nowshera) can be seen in Table 3.25. Also the results from the ductility test can be seen in Table 3.26.

Table 3.25 Results from the penetration test on the blended binders

RAP Source	Recovered Binder (%)	Penetration Value			
		1	2	3	Average
Nowshera	100	15	17	14	15.33
	90	20	21	25	22.00
	80	29	31	28	29.33
	70	31	33	31	31.67
	40	35	31	39	35.00
	0 (ARL60/70)	65	69	64	66.00
Mandra	100	8	7	6	7.00
	90	10	11	14	11.67
	80	14	13	16	14.33
	70	35	30	32	32.33
	40	45	52	50	49.00
	0 (ARL60/70)	65	69	64	66.00

Table 3.26 Results from the ductility test on the blended binders

RAP Source	Recovered Binder (%)	Reading (cm)			
		1	2	3	Average
Nowshera	100	13	13	12	12.67
	90	17	18	18	17.67
	80	24.5	24	25	24.5
	70	100	85	70	85
	40	100	90	81	90.33
	0	>100	>100	>100	>100
Mandra	100	15	15	17	15.67
	90	19	20	20	19.67
	80	29	28	29	28.67
	70	40	34	45	39.67
	40	84	23	54	53.67
	0	>100	>100	>100	>100

The extracted asphalt binders and their blends with ARL 60/70 were then subjected to performance testing to determine their PG grades according to AASHTO R

29-02. The tests consisted of the Dynamic Shear Rheometer (DSR), Bending Beam Rheometer (BBR) and Rotational Viscometer (RV). The DSR tests were performed to establish the rheological properties (dynamic shear modulus (G*) and phase angle (δ)) of the blended binders. The results from the DSR tests for the original extracted binders and their blends are shown in Table 3.27.

Table 3.27 DSR results for original blended binders

Source	% Recovered Material	Temp (°C)	G*/Sinδ
Nowshera	0	62.31	1.88
	40	81.06	2.00
	70	88	1.35
	80	88	1.67
	90	88	2.63
	100	88	13.39
Mandra	0	62.31	1.88
	40	74.20	4.38
	70	88	2.76
	80	88	3.74
	90	88	8.31
	100	88	9.64

Note: Temperature and G*Sinδ are average of 2 samples

The BBR tests were used to measure how much a beam deflects under a constant load at a constant temperature. The tests were performed at three temperatures (-6,-12 and -18 °C) in order to determine the binder's susceptibility at low temperatures. The two parameters determined were creep stiffness and m-value, the results of all extracted binders and blends are shown in Table 3.28.

RV tests were performed on all the extracted binders and their blends with ARL 60/70. The viscosity was determined at a speed of 20 rpm and at two temperatures, 135 and 165 °C which covers the compaction and mixing temperature ranges. The results obtained are provided in Table 3.29.

Table 3.28 BBR results for blended binders

RAP Source	Property	Temp (°C)	RAP % 100	90	80	70	40	0
Nowshera	Stiffness (MPa)	-6	101.56	101.41	98.885	84.5	42.71	45.71
		-12	283.69	234.53	225.965	196.32	180.98	104.53
		-18	410.09	395.34	368.26	321.87	280.28	222.49
		-24						434.155
	m-value	-6	0.322	0.296	0.327	0.3475	0.441	0.44
		-12	0.238	0.242	0.2785	0.2925	0.33	0.40
		-18	0.221	0.233	0.247	0.276	0.305	0.24
		-24						0.195
Mandra	Stiffness (MPa)	-6	114.02	74.48	97.24	63.12	65.82	45.71
		-12	297.18	172.275	202.74	171.58	173.55	104.53
		-18	422.17	235.76	311.045	133.79	291.35	222.49
		-24						434.155
	m-value	-6	0.319	0.3305	0.301	0.372	0.399	0.44
		-12	0.213	0.3005	0.299	0.322	0.348	0.40
		-18	0.241	0.258	0.2685	0.128	0.301	0.24
		-24						0.195

Table 3.29 RV test results for blended binders

RAP Source	Temperature (°C)	Viscosity (cp) Extracted Binder (%) 10	20	30	60	100
Nowshera	135	365	394.4	739.7	2668.3	4319.1
	165	90	108.4	128.8	168.3	535.3
Mandra	135	358.1	562.5	656.3	1287.5	3479.3
	165	108.1	111.1	119.4	218.1	507.5

3.2.4 Characteristics of Recovered Aggregates

The gradation tests of the aggregates extracted from the two sources of recovered material (Nowshera and Mandra) were carried out in accordance with AASHTO T 164-93, "Sieve analysis of extracted aggregates." The gradation of the recovered aggregates was controlled by the gradation that was set in Section 3.2.2. The recovered aggregates were then processed and the gradation results are listed in Table 3.30 and shown in Figure 3.12.

Table 3.30 Gradation of recovered aggregates

Sieve size (mm)	RAP source	
	Nowshera	Mandra
	% Passing	
37.5	100	100
25	95	92
19	83	77
12.5	68	60
4.75	39	42
2.36	30	26
1.18	19	20
0.6	13	15
0.3	10	12
0.075	5	8

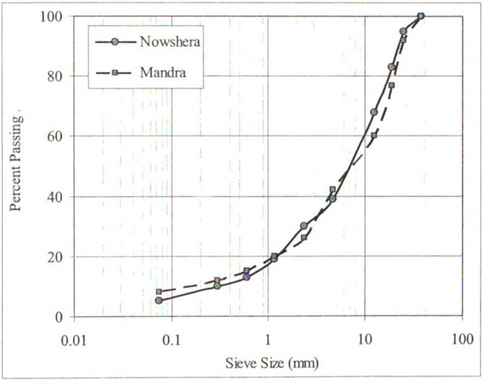


Figure 3.12 Gradation curve of recovered aggregates

The specific gravities of the RAP aggregates were not determined in the laboratory like those of the virgin aggregates. It is state in the literature that it is difficult to use extraction or ignition procedures to remove the asphalt binder from old HMA pavements without changing the aggregate properties (Anderson & Murphy 2005). However, the original source of the aggregates in the Nowshera and Mandra material was known. Nowshera recovered material contains Margalla aggregates while Mandra recovered material contains Deena aggregates. Understanding this relationship, the properties in Table 3.21 can also be used for the recovered aggregates.

3.2.5 Characteristics of Virgin Mixtures

The properties of eight virgin HMA mixes were first determined. Trial virgin mixes were made using the Marshall and Superpave mix design procedures at the binder contents listed in Table 3.31 in order to determine the optimum asphalt content.

Table 3.31 Trial virgin mixtures binder content

Aggregate Type	Aggregate Gradation	Marshall Mix Design at Binder Contents	Superpave Mix Design Binder Contents
Deena (quartzite)	Wearing Course	4, 4.5, 5, 5.5, 6	4.5, 5, 5.5, 6
	Base Course	3.5, 4, 4.5, 5,5.5	3.5, 4, 4.5, 5
Margalla (limestone)	Wearing Course	4, 4.5, 5, 5.5, 6	4.5, 5, 5.5, 6
	Base Course	3.5, 4, 4.5, 5,5.5	3.5, 4, 4.5, 5

Virgin samples for the Marshall and Superpave mix design were made using the Marshall hammer and gyratory compactor, respectively. Since all mixtures were designed for heavy traffic, the Marshall mixtures were compacted using 75 hammer blows on each side of the sample for the wearing courses and 112 times for the base courses. For Superpave mixtures, compaction using the gyratory compactor of 125 (N_{design}) gyrations was used for both wearing and base courses. Once these trial mixtures were produced, the volumetric data of the samples were determined and calculated. The following volumetric information were measured through laboratory testing:

- The theoretical maximum specific gravity (G_{mm}) of the mixtures in Table 3.32 were measured according to AASHTO T 209.

Table 3.32 Trial mixes at which G_{mm} was measured

Design Method	Aggregate type-course type	%AC
Marshall	DW	5, 5.5, 6%
	MW	5, 5.5, 6%
	DB	4, 4.25, 4.5%
	MB	4, 4.25, 4.5%
Superpave	DW	4.5%
	MW	4.5%
	DB	4, 4.25, 4.5%
	MB	4, 4.25, 4.5%

Note: D=Deena, M=Margalla, W=wearing, B=base

- The bulk specific gravity (G_{mb}) for all trial virgin mixtures (shown in Table 3.31) were measured according AASHTO T 166.
- Stability and flow tests were performed according to AASHTO T 245.

With this information available the following volumetric information were determined based on calculations:

- The effective specific gravity of the aggregate was determined using the following equation:

$$G_{se} = \frac{P_s}{\left(\frac{100}{G_{mm}} - \frac{P_b}{G_b}\right)} \tag{2}$$

Where,

G_{se} = effective specific gravity

P_s = percent weight of aggregates

P_b = percent weight of asphalt binder

G_b = specific gravity of the asphalt binder

The G_{se} was determined using Equation 2 for each measured G_{mm}. It was then averaged since the G_{se} is an aggregate property, it can remain constant for all mixes with the same aggregate type and gradation.

- The G_{mm} for all trial mixes (Table 3.31) were then backcalculated using a rearranged form of Equation 2:

$$G_{mm} = \frac{100}{\left(\frac{P_s}{G_{se}} + \frac{P_b}{G_b}\right)} \tag{3}$$

- The percentages of air voids was then determined for all trial mixes using the following equation:

$$AirVoids = 100 \times \left(1 - \frac{G_{mb}}{G_{mm}}\right) \tag{4}$$

- The voids in mineral aggregate (VMA) was then determined using the following equation:

$$VMA = 100 - \left(G_{mb} \frac{P_s}{G_{sb}} \right) \tag{5}$$

- The voids filled with asphalt (VFA) was then determined using the following equation:

$$VFA = 100 \times \left(\frac{VMA - AirVoids}{VMA} \right) \tag{6}$$

Graphs of the percent asphalt content (AC) versus air voids were made. The optimum AC was determined at 4% air voids. See Figure 3.13 for an example. The optimum asphalt content for all eight virgin mixtures can be seen in Table 3.33. The G_{mb}, VFA and VMA for the optimum mixtures were determined using graphs similar to Figure 3.13 and are shown in Table 3.33. The values in Table 3.33 were checked to see if they are within the specifications of each design method. These specification limits can be found in Appendix E and it was concluded that these mixes fall within the specification ranges.

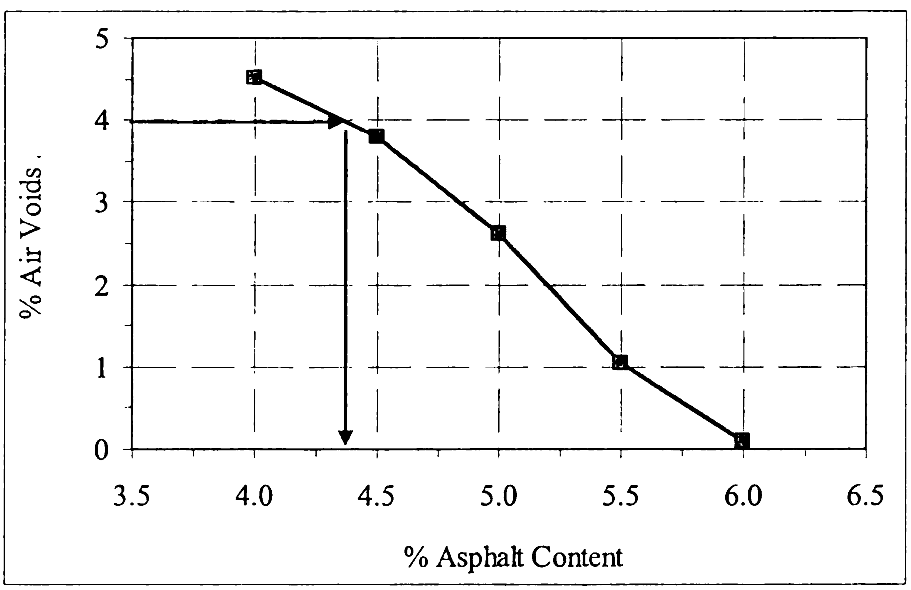


Figure 3.13 Asphalt content versus air voids for Marshall Margalla wearing mixtures

Table 3.33 AC and volumetric data for Marshall and Superpave optimum virgin mixes

Design Method	Course Type	Aggregate Source	Optimum AC (%)	G_{mb}	Air Voids (%)	VFA (%)	VMA (%)
Marshall	Wearing	Deena	4.8	2.37	4	75.5	16.25
		Margalla	4.4	2.38	4	71.5	14.15
	Base	Deena	4.75	2.42	4	73.8	15.25
		Margalla	3.7	2.43	4	68.4	12.66
Superpave	Wearing	Deena	4.8	2.42	4	70.1	13.2
		Margalla	4.2	2.38	4	68.7	14.3
	Base	Deena	4.2	2.42	4	67.3	12.3
		Margalla	3.65	2.42	4	66.8	13.8

The stability and flow properties for the optimum Marshall mixes were also determined using graphs similar to Figure 3.13. These properties can be seen below in Table 3.34 and were also within the Marshall design method specifications.

Table 3.34 Stability and flow for optimum Marshall mixtures

Course Type	Aggregate Source	Optimum AC (%)	Stability (N)	Flow (mm)
Wearing	Deena	4.8	9.89	2.63
	Margalla	4.4	12.65	2.91
Base	Deena	4.75	22.5	4.27
	Margalla	3.7	31.2	3.55

Since the optimum mixes were considered to be within the specifications of their respective mix designs, actual mixtures were made at these optimum asphalt contents.

The G_{mm} of these mixtures were measured and the G_{se} was calculated using Equation 2. This new G_{se} was used with Equation 3 to re-calculate the G_{mm} of all virgin trial mixtures. Then air voids, VMA and VFA of all virgin samples were re-calculated using Equations 4, 5 and 6, respectively. The percent absorbed asphalt (P_{ba}) and the percent effective asphalt (P_{be}) were calculated using Equations 7 and 8, respectively. All volumetric data are shown in Tables 3.35 and 3.36.

$$P_{ba}(\%) = 100 \times \left(\frac{G_{se} - G_{sb}}{G_{sb} \times G_{se}} \right) G_b \tag{7}$$

$$P_{be}(\%) = P_b - \left(\frac{P_{ba}}{100} \right) P_s \tag{8}$$

The optimum Superpave mixtures were also subjected to a simple performance test (SPT) to determine the dynamic modulus (E^*) and phase angle (δ) at three different temperatures (8, 21 and 37°C) at six frequencies (25, 10, 5, 1, 0.5 and 0.1Hz). An example of the SPT results for the optimum mixtures at 8°C can be seen in Table 3.37; the entire results are tabulated in Appendix E.

Table 3.35 Marshall – virgin mixture design volumetric properties

COURSE TYPE	Virgin Aggregate Source	% AC	Gmm	Gmb	Gsb	Gse	Air Voids	VMA	VFA	Pba	Pbe	Stability (kN)	Flow (mm)
WEARING	Deena	4.0	2.541	2.334	2.685	2.710	8.17%	16.6%	50.7%	0.35%	3.66%	10.67	2.35
		4.5	2.522	2.346	2.685	2.710	6.96%	16.5%	57.9%	0.35%	4.17%	9.1	2.45
		5.0	2.502	2.377	2.685	2.710	5.01%	15.9%	68.5%	0.35%	4.67%	10.5	2.75
		5.5	2.483	2.412	2.685	2.710	2.87%	15.1%	81.0%	0.35%	5.17%	10.3	2.82
		6.0	2.465	2.416	2.685	2.710	1.97%	15.4%	87.2%	0.35%	5.67%	9.75	3.01
	Margalla	4.0	2.512	2.383	2.653	2.675	5.15%	13.8%	62.6%	0.32%	3.69%	14.28	2.82
		4.5	2.493	2.383	2.653	2.675	4.42%	14.2%	68.9%	0.32%	4.20%	12.25	2.92
		5.0	2.475	2.394	2.653	2.675	3.25%	14.3%	77.2%	0.32%	4.70%	11.1	2.84
		5.5	2.456	2.415	2.653	2.675	1.67%	14.0%	88.0%	0.32%	5.20%	9.81	2.92
		6.0	2.438	2.420	2.653	2.675	0.73%	14.3%	94.8%	0.32%	5.70%	10.3	3.22
BASE	Deena	3.5	2.588	2.398	2.697	2.740	7.32%	14.2%	48.4%	0.60%	2.92%	29.98	3.79
		4.0	2.567	2.403	2.697	2.740	6.40%	14.5%	55.8%	0.60%	3.43%	25.38	3.43
		4.5	2.547	2.412	2.697	2.740	5.30%	14.6%	63.7%	0.60%	3.93%	23.21	4.06
		5.0	2.527	2.435	2.697	2.740	3.66%	14.2%	74.3%	0.60%	4.43%	22.88	4.35
		5.5	2.508	2.448	2.697	2.740	2.39%	14.2%	83.2%	0.60%	4.94%	28.91	3.94
	Margalla	3.5	2.537	2.412	2.651	2.681	4.92%	12.2%	59.7%	0.43%	3.08%	30.52	3.56
		4.0	2.517	2.437	2.651	2.681	3.21%	11.8%	72.8%	0.43%	3.59%	31.55	3.56
		4.5	2.498	2.415	2.651	2.681	3.34%	13.0%	74.4%	0.43%	4.09%	26.19	3.64
		5.0	2.479	2.436	2.651	2.681	1.77%	12.7%	86.1%	0.43%	4.59%	28.41	4.4
		5.5	2.461	2.448	2.651	2.681	0.51%	12.7%	96.0%	0.43%	5.09%	22.7	5.16

Table 3.36 Superpave – virgin mixture design volumetric properties

COURSE TYPE	Virgin Aggregate Source	% Binder	Gmm	Gmb	Gsb	Gse	Air Voids	VMA	VFA	Pba	Pbe
WEARING	Deena	4.0	2.536	2.42	2.685	2.704	4.58%	13.5%	66.0%	0.27%	3.74%
		4.5	2.517	2.41	2.685	2.704	4.24%	14.3%	70.3%	0.27%	4.24%
		5.0	2.498	2.42	2.685	2.704	3.11%	14.4%	78.4%	0.27%	4.75%
		5.5	2.479	2.44	2.685	2.704	1.56%	14.1%	88.9%	0.27%	5.25%
	Margalla	4.0	2.476	2.36	2.653	2.633	4.69%	15.6%	69.9%	-0.30%	4.28%
		4.5	2.458	2.41	2.653	2.633	1.95%	14.3%	86.3%	-0.30%	4.78%
		5.0	2.440	2.45	2.653	2.633	-0.41%	13.3%	103.1%	-0.30%	5.28%
		5.5	2.422	2.46	2.653	2.633	-1.56%	13.4%	111.6%	-0.30%	5.78%
BASE	Deena	3.5	2.559	2.41	2.697	2.707	5.81%	13.4%	56.5%	0.14%	3.37%
		4.0	2.539	2.42	2.697	2.707	4.68%	13.5%	65.2%	0.14%	3.87%
		4.5	2.519	2.42	2.697	2.707	3.94%	13.9%	71.7%	0.14%	4.37%
		5.0	2.500	2.45	2.697	2.707	2.00%	13.3%	84.9%	0.14%	4.87%
	Margalla	3.5	2.534	2.42	2.651	2.678	4.50%	13.0%	65.4%	0.39%	3.13%
		4.0	2.515	2.43	2.651	2.678	3.37%	13.1%	74.3%	0.39%	3.63%
		4.5	2.496	2.44	2.651	2.678	2.23%	13.2%	83.1%	0.39%	4.13%
		5.0	2.477	2.48	2.651	2.678	-0.12%	12.2%	101.0%	0.39%	4.63%

Table 3.37 SPT results for the optimum virgin mixtures at 8°C

Sample	Property	Frequency (Hz)					
		25	10	5	1	0.5	0.1
MW	E* (ksi)	4,482	3,829	3,792	2,987	3,119	2,095
	δ(Deg)	6.06	6.90	13.93	11.48	8.78	12.22
DW	E* (ksi)	3,399	3,140	2,801	2,162	1,870	1,227
	δ(Deg)	11.63	14.28	14.44	19.83	21.46	26.06
MB	E* (ksi)	4,235	3,895	3,624	3,043	2,721	1,712
	δ(Deg)	6.43	10.97	9.14	14.50	14.72	20.57
DB	E* (ksi)	6,431	4,868	4,544	4,359	4,141	3,103
	δ(Deg)	5.08	3.81	4.54	6.81	9.27	13.71

Note: M=Margalla, D=Deena, W=wearing, B=base

3.2.6 Characteristics of Recycled Asphalt Mixtures

The characteristics of the recycled mixtures could now be determined with the optimum asphalt content of the virgin mixes known. The target asphalt content for the recycled mixtures was the same for the optimum virgin mixtures (shown in Table 3.33). Sixteen recycled mixtures were made using the aforementioned optimum asphalt content. Sixteen recycled mixtures were made [two virgin aggregate sources (Margalla and Deena), two recovered asphalt material sources (Nowshera and Mandra), two course types (wearing and base) and two mix design procedures (Marshall and Superpave)]. For each of these mixtures five blends, combining the virgin and recovered material, and one 100% recovered HMA sample was made. Blends were made at the following percentage of recovered material: 10, 20, 30, 45 and 60%.

The characteristics that were determined through laboratory tests and calculations of the virgin mixtures were repeated for the blended mixtures. The volumetric properties determined for the Marshall samples can be seen in Tables 3.38 and 3.39. Volumetric data for the Superpave samples can be seen in Tables 3.40 and 3.41.

For the Superpave mixtures, the volumetric properties which corresponded to a 4 percent air void also were estimated. These were determined from the following set of equations:

$$P_b, estimated = P_{bi} - (0.4 * (4 - Va)) \tag{9}$$

$$\%VMA, estimated = \%VMA_{initial} + C(4 - Va) \tag{10}$$

$$\%VFA, estimated = 100 \times \frac{\%VMA, estimated - 4.0}{\%VMA, estimated} \tag{11}$$

where,

P_b,estimated = estimated percent binder

P_{bi} = initial percent binder

V_a = percent air voids at Ndesign

%VMAinitial = % VMA at initial binder content

C = 0.1 if V_a is less than 4.0% or 0.2 if V_a is greater than 4.0%

Tables 3.42 and 3.43 show the new estimated volumetric data for the Superpave mixtures.

The eight Superpave recycled mixtures were also run through the SPT to determine the dynamic modulus (E*) and phase angle (δ) of the recycled asphalt mixtures. For these test a low temperature of 4°C was achieved in comparison to 8°C that was achieved for the optimum virgin mixtures. An example of the SPT results is shown in Table 3.44 while the complete results are tabulated in Appendix E.

Table 3.38 Marshall – blended mixture design (wearing course) volumetric properties

Virgin Aggregate Source	RAP Source	RAP	Virgin	Ps %	Pb %	Gmm	Gmb	Gsb	Gse
Deena	Mandra	0%	100%	95.2	4.8	2.51	2.37	2.685	2.710
		10%	90%	95.2	4.8	2.52	2.44	2.685	2.710
		20%	80%	95.2	4.8	2.52	2.41	2.685	2.710
		30%	70%	95.2	4.8	2.46	2.37	2.685	2.710
		45%	55%	95.2	4.8	2.46	2.37	2.685	2.710
		60%	40%	95.2	4.8	2.47	2.36	2.685	2.710
		100%	0%	94.9	5.1	2.48	2.40	2.685	2.710
	Nowshera	0%	100%	95.2	4.8	2.51	2.37	2.685	2.710
		10%	90%	95.2	4.8	2.50	2.44	2.681	2.706
		20%	80%	95.2	4.8	2.51	2.42	2.678	2.703
		30%	70%	95.2	4.8	2.45	2.34	2.675	2.699
		45%	55%	95.2	4.8	2.48	2.38	2.670	2.694
		60%	40%	95.2	4.8	2.46	2.35	2.666	2.689
		100%	0%	94.4	5.6	2.44	2.35	2.653	2.675
Margalla	Mandra	0%	100%	95.6	4.4	2.50	2.40	2.653	2.675
		10%	90%	95.6	4.4	2.47	2.38	2.656	2.679
		20%	80%	95.6	4.4	2.48	2.37	2.659	2.682
		30%	70%	95.6	4.4	2.48	2.31	2.663	2.685
		45%	55%	95.6	4.4	2.47	2.36	2.667	2.691
		60%	40%	95.6	4.4	2.46	2.38	2.672	2.696
		100%	0%	94.9	5.1	2.48	2.40	2.685	2.710
	Nowshera	0%	100%	95.6	4.4	2.50	2.40	2.653	2.675
		10%	90%	95.6	4.4	2.47	2.40	2.653	2.675
		20%	80%	95.6	4.4	2.46	2.36	2.653	2.675
		30%	70%	95.6	4.4	2.48	2.36	2.653	2.675
		45%	55%	95.6	4.4	2.48	2.34	2.653	2.675
		60%	40%	95.6	4.4	2.45	2.35	2.653	2.675
		100%	0%	94.4	5.6	2.44	2.35	2.653	2.675

Table 3.38 (continued)

Virgin Aggregate Source	RAP Source	RAP	Virgin	AV	VMA	VFA	Pba	Pbe	Stability (kN)	Flow (mm)
Deena	Mandra	0%	100%	5.58%	16.0%	65.0%	0.35%	4.47%	9.89	2.63
		10%	90%	3.33%	13.5%	75.3%	0.32%	4.50%	7.54	1.03
		20%	80%	4.44%	14.5%	69.5%	0.28%	4.53%	17.42	2.31
		30%	70%	3.66%	16.0%	77.1%	0.25%	4.57%	10.48	1.91
		45%	55%	3.66%	16.0%	77.1%	0.19%	4.62%	19.55	2.25
		60%	40%	4.26%	16.3%	73.9%	0.14%	4.67%	19.92	2.65
		100%	0%	3.23%	15.2%	78.7%	0.00%	5.10%	13.39	1.96
	Nowshera	0%	100%	5.58%	16.0%	65.0%	0.35%	4.47%	9.89	2.63
		10%	90%	2.56%	13.4%	80.9%	0.35%	4.47%	11.60	1.25
		20%	80%	3.55%	14.0%	74.6%	0.34%	4.47%	7.71	0.64
		30%	70%	4.41%	16.7%	73.6%	0.34%	4.48%	11.74	2.43
		45%	55%	3.84%	15.2%	74.7%	0.34%	4.48%	20.95	2.30
		60%	40%	4.55%	16.1%	71.7%	0.33%	4.48%	9.78	1.11
		100%	0%	3.57%	16.4%	78.2%	0.32%	5.30%	14.98	1.96
Margalla	Mandra	0%	100%	3.88%	13.5%	71.3%	0.32%	4.10%	12.65	2.91
		10%	90%	3.57%	14.3%	75.1%	0.32%	4.09%	10.96	2.29
		20%	80%	4.36%	14.8%	70.6%	0.33%	4.09%	20.22	2.23
		30%	70%	6.82%	17.1%	60.0%	0.33%	4.09%	10.69	2.67
		45%	55%	4.41%	15.4%	71.4%	0.33%	4.08%	9.44	1.45
		60%	40%	3.21%	14.8%	78.4%	0.34%	4.08%	7.82	0.66
		100%	0%	3.23%	15.2%	78.7%	0.35%	4.77%	13.39	1.96
	Nowshera	0%	100%	3.88%	13.5%	71.3%	0.32%	4.10%	12.65	2.91
		10%	90%	2.64%	13.5%	80.5%	0.32%	4.10%	8.59	0.91
		20%	80%	3.95%	15.0%	73.6%	0.32%	4.10%	10.92	1.56
		30%	70%	4.65%	15.0%	68.9%	0.32%	4.10%	18.40	3.07
		45%	55%	5.49%	15.7%	65.0%	0.32%	4.10%	11.73	2.10
		60%	40%	4.04%	15.3%	73.6%	0.32%	4.10%	21.19	1.80
		100%	0%	3.57%	16.4%	78.2%	0.32%	5.30%	14.98	1.96

Note: Ps = percent aggregate, Pb = asphalt content (percent binder), Gmm = theoretical maximum specific gravity of the mix, Gmb = bulk specific gravity of the mix, Gsb = bulk specific gravity of the aggregate, Gse = effective specific gravity of the aggregate, AV = air voids, VMA = voids in mineral aggregate, VFA = voids filled with asphalt, Pba= percent binder absorbed, Pbe = percent effective binder

Table 3.39 Marshall – blended mixture design (base course) volumetric properties

Virgin Aggregate Source	RAP Source	RAP	Virgin	Ps %	Pb %	Gmm	Gmb	Gsb	Gse
Deena	Mandra	0%	100%	95.25	4.75	2.53	2.43	2.697	2.740
		10%	90%	95.25	4.75	2.52	2.30	2.697	2.740
		20%	80%	95.25	4.75	2.52	2.26	2.697	2.740
		30%	70%	95.25	4.75	2.52	2.29	2.697	2.740
		45%	55%	95.25	4.75	2.50	2.29	2.697	2.740
		60%	40%	95.25	4.75	2.48	2.25	2.697	2.740
		100%	0%	94.9	5.1	2.47	2.27	2.697	2.740
	Nowshera	0%	100%	95.25	4.75	2.53	2.43	2.697	2.740
		10%	90%	95.25	4.75	2.47	2.34	2.692	2.734
		20%	80%	95.25	4.75	2.50	2.40	2.688	2.728
		30%	70%	95.25	4.75	2.55	2.45	2.683	2.722
		45%	55%	95.25	4.75	2.55	2.48	2.676	2.713
		60%	40%	95.25	4.75	2.54	2.46	2.669	2.705
		100%	0%	94.4	5.6	2.54	2.38	2.651	2.681
Margalla	Mandra	0%	100%	96.3	3.7	2.52	2.42	2.651	2.681
		10%	90%	96.3	3.7	2.41	2.27	2.656	2.687
		20%	80%	96.3	3.7	2.41	2.28	2.660	2.693
		30%	70%	96.3	3.7	2.43	2.23	2.665	2.699
		45%	55%	96.3	3.7	2.47	2.25	2.672	2.708
		60%	40%	96.3	3.7	2.44	2.28	2.679	2.716
		100%	0%	94.9	5.1	2.47	2.27	2.697	2.740
	Nowshera	0%	100%	96.3	3.7	2.52	2.42	2.651	2.681
		10%	90%	96.3	3.7	2.48	2.39	2.651	2.681
		20%	80%	96.3	3.7	2.49	2.41	2.651	2.681
		30%	70%	96.3	3.7	2.51	2.40	2.651	2.681
		45%	55%	96.3	3.7	2.49	2.42	2.651	2.681
		60%	40%	96.3	3.7	2.48	2.39	2.651	2.681
		100%	0%	94.4	5.6	2.54	2.38	2.651	2.681

Table 3.39 (continued)

Virgin Aggregate Source	RAP Source	RAP	Virgin	AV	VMA	VFA	Pba	Pbe	Stability (kN)	Flow (mm)
Deena	Mandra	0%	100%	3.95%	14.2%	72.1%	0.60%	4.18%	22.50	4.27
		10%	90%	8.55%	18.8%	54.5%	0.60%	4.18%	32.69	7.71
		20%	80%	10.25%	20.2%	49.2%	0.60%	4.18%	33.13	4.40
		30%	70%	9.16%	19.1%	52.1%	0.60%	4.18%	31.99	4.68
		45%	55%	8.55%	19.1%	55.3%	0.60%	4.18%	31.56	4.59
		60%	40%	9.42%	20.5%	54.1%	0.60%	4.18%	29.48	5.41
		100%	0%	8.21%	20.1%	59.2%	0.60%	4.53%	48.26	2.68
	Nowshera	0%	100%	3.95%	14.2%	72.1%	0.60%	4.18%	22.50	4.27
		10%	90%	5.15%	17.2%	70.1%	0.58%	4.20%	21.87	4.46
		20%	80%	4.15%	14.9%	72.2%	0.56%	4.21%	29.83	4.62
		30%	70%	3.88%	13.0%	70.2%	0.55%	4.23%	38.78	4.69
		45%	55%	2.86%	11.7%	75.6%	0.52%	4.25%	38.29	4.63
		60%	40%	3.26%	12.2%	73.3%	0.50%	4.28%	60.63	5.50
		100%	0%	6.26%	15.3%	59.0%	0.43%	5.19%	71.34	4.18
Margalla	Mandra	0%	100%	3.97%	12.1%	67.2%	0.43%	3.28%	31.20	3.55
		10%	90%	5.85%	17.7%	66.9%	0.45%	3.27%	38.24	4.91
		20%	80%	5.35%	17.5%	69.3%	0.46%	3.25%	30.33	3.50
		30%	70%	8.12%	19.4%	58.2%	0.48%	3.24%	28.86	5.06
		45%	55%	8.76%	18.9%	53.7%	0.51%	3.21%	37.05	3.97
		60%	40%	6.71%	18.0%	62.8%	0.53%	3.19%	44.32	3.57
		100%	0%	8.21%	20.1%	59.2%	0.60%	4.53%	48.26	2.68
	Nowshera	0%	100%	3.97%	12.1%	67.2%	0.43%	3.28%	31.20	3.55
		10%	90%	3.75%	13.2%	71.6%	0.43%	3.28%	34.86	3.45
		20%	80%	3.10%	12.5%	75.2%	0.43%	3.28%	30.53	5.03
		30%	70%	4.50%	12.8%	65.0%	0.43%	3.28%	36.63	3.49
		45%	55%	2.62%	12.1%	78.4%	0.43%	3.28%	42.52	3.96
		60%	40%	3.43%	13.2%	74.0%	0.43%	3.28%	47.91	4.71
		100%	0%	6.26%	15.3%	59.0%	0.43%	5.19%	71.34	4.18

Table 3.40 Superpave – blended mixture design (wearing course) volumetric properties

Virgin Aggregate Source	RAP Source	RAP	Virgin	Ps %	Pb %	Gmm	Gmb	Gsb	Gse	AV	VMA	VFA	Pba	Pbe
Deena	Mandra	0%	100%	95.2	4.8	2.51	2.39	2.685	2.704	4.60%	15.2%	69.8%	0.27%	4.54%
		10%	90%	95.2	4.8	2.52	2.42	2.685	2.704	3.98%	14.2%	72.0%	0.27%	4.54%
		20%	80%	95.2	4.8	2.52	2.42	2.685	2.704	4.01%	14.1%	71.6%	0.27%	4.54%
		30%	70%	95.2	4.8	2.53	2.43	2.685	2.704	3.81%	13.7%	72.2%	0.27%	4.54%
		45%	55%	95.2	4.8	2.50	2.43	2.685	2.704	3.05%	13.9%	78.0%	0.27%	4.54%
		60%	40%	95.2	4.8	2.49	2.41	2.685	2.704	2.99%	14.5%	79.3%	0.27%	4.54%
		100%	0%	94.9	5.1	2.45	2.40	2.685	2.704	2.00%	15.1%	86.8%	0.27%	4.85%
	Nowshera	0%	100%	95.2	4.8	2.51	2.39	2.685	2.704	4.60%	15.2%	69.8%	0.27%	4.54%
		10%	90%	95.2	4.8	2.51	2.41	2.681	2.696	3.96%	14.3%	72.4%	0.21%	4.60%
		20%	80%	95.2	4.8	2.49	2.41	2.678	2.689	3.07%	14.2%	78.4%	0.16%	4.65%
		30%	70%	95.2	4.8	2.48	2.42	2.675	2.682	2.73%	14.0%	80.5%	0.10%	4.71%
		45%	55%	95.2	4.8	2.48	2.42	2.670	2.671	2.41%	13.8%	82.5%	0.01%	4.79%
		60%	40%	95.2	4.8	2.47	2.43	2.666	2.661	1.64%	13.1%	87.5%	-0.07%	4.87%
		100%	0%	94.4	5.6	2.55	2.48	2.653	2.633	2.74%	11.6%	76.5%	-0.30%	5.88%
Margalla	Mandra	0%	100%	95.8	4.2	2.47	2.38	2.653	2.633	3.60%	14.1%	74.4%	-0.30%	4.48%
		10%	90%	95.8	4.2	2.46	2.37	2.656	2.640	3.87%	14.7%	73.6%	-0.24%	4.43%
		20%	80%	95.8	4.2	2.47	2.38	2.659	2.647	3.60%	14.3%	74.9%	-0.18%	4.38%
		30%	70%	95.8	4.2	2.47	2.38	2.663	2.654	3.50%	14.3%	75.5%	-0.13%	4.32%
		45%	55%	95.8	4.2	2.45	2.39	2.667	2.664	2.76%	14.3%	80.7%	-0.04%	4.24%
		60%	40%	95.8	4.2	2.45	2.39	2.672	2.675	2.35%	14.3%	83.5%	0.04%	4.16%
		100%	0%	94.9	5.1	2.45	2.40	2.685	2.704	2.00%	15.1%	86.8%	0.27%	4.85%
	Nowshera	0%	100%	95.8	4.2	2.47	2.38	2.653	2.633	3.60%	14.1%	74.4%	-0.30%	4.48%
		10%	90%	95.8	4.2	2.53	2.43	2.653	2.633	4.15%	12.4%	66.6%	-0.30%	4.48%
		20%	80%	95.8	4.2	2.53	2.43	2.653	2.633	4.04%	12.3%	67.0%	-0.30%	4.48%
		30%	70%	95.8	4.2	2.53	2.44	2.653	2.633	3.78%	12.0%	68.4%	-0.30%	4.48%
		45%	55%	95.8	4.2	2.53	2.45	2.653	2.633	3.26%	11.7%	72.1%	-0.30%	4.48%
		60%	40%	95.8	4.2	2.52	2.45	2.653	2.633	2.61%	11.4%	77.1%	-0.30%	4.48%
		100%	0%	94.4	5.6	2.55	2.48	2.653	2.633	2.74%	11.6%	76.5%	-0.30%	5.88%

Table 3.41 Superpave – blended mixture design (base course) volumetric properties

Virgin Aggregate Source	RAP Source	RAP	Virgin	Ps %	Pb %	Gmm	Gmb	Gsb	Gse	AV	VMA	VFA	Pba	Pbe
Deena	Mandra	0%	100%	95.8	4.2	2.53	2.41	2.697	2.707	4.78%	14.4%	66.8%	0.14%	4.07%
		10%	90%	95.8	4.2	2.52	2.43	2.697	2.707	3.81%	13.7%	72.3%	0.14%	4.07%
		20%	80%	95.8	4.2	2.52	2.43	2.697	2.707	3.51%	13.7%	74.3%	0.14%	4.07%
		30%	70%	95.8	4.2	2.51	2.43	2.697	2.707	3.13%	13.6%	76.9%	0.14%	4.07%
		45%	55%	95.8	4.2	2.50	2.44	2.697	2.707	2.56%	13.4%	80.9%	0.14%	4.07%
		60%	40%	95.8	4.2	2.50	2.44	2.697	2.707	2.18%	13.2%	83.5%	0.14%	4.07%
		100%	0%	94.9	5.1	2.50	2.45	2.697	2.707	1.97%	13.9%	85.8%	0.14%	4.97%
	Nowshera	0%	100%	95.8	4.2	2.53	2.41	2.697	2.707	4.78%	14.4%	66.8%	0.14%	4.07%
		10%	90%	95.8	4.2	2.48	2.39	2.692	2.704	3.55%	14.9%	76.2%	0.16%	4.05%
		20%	80%	95.8	4.2	2.47	2.40	2.688	2.701	3.11%	14.6%	78.7%	0.19%	4.02%
		30%	70%	95.8	4.2			2.683	2.698					
		45%	55%	95.8	4.2			2.676	2.694					
		60%	40%	95.8	4.2			2.669	2.690					
		100%	0%	94.4	5.6	2.51	2.46	2.651	2.678	1.79%	12.3%	85.4%	0.39%	5.23%
Margalla	Mandra	0%	100%	96.35	3.65	2.53	2.42	2.651	2.678	4.28%	12.1%	64.5%	0.39%	3.28%
		10%	90%	96.35	3.65	2.50	2.41	2.656	2.681	3.44%	12.5%	72.5%	0.36%	3.30%
		20%	80%	96.35	3.65	2.50	2.42	2.660	2.684	3.28%	12.4%	73.4%	0.34%	3.33%
		30%	70%	96.35	3.65	2.50	2.42	2.665	2.687	2.99%	12.4%	75.8%	0.31%	3.35%
		45%	55%	96.35	3.65	2.49	2.43	2.672	2.691	2.48%	12.5%	80.2%	0.27%	3.39%
		60%	40%	96.35	3.65	2.49	2.43	2.679	2.695	2.16%	12.5%	82.8%	0.24%	3.42%
		100%	0%	94.9	5.1	2.50	2.45	2.697	2.707	1.97%	13.9%	85.8%	0.14%	4.97%
	Nowshera	0%	100%	96.35	3.65	2.53	2.42	2.651	2.678	4.28%	12.1%	64.5%	0.39%	3.28%
		10%	90%	96.35	3.65	2.53	2.43	2.651	2.678	4.15%	11.9%	65.1%	0.39%	3.28%
		20%	80%	96.35	3.65	2.53	2.43	2.651	2.678	4.04%	11.7%	65.4%	0.39%	3.28%
		30%	70%	96.35	3.65	2.53	2.44	2.651	2.678	3.78%	11.4%	66.8%	0.39%	3.28%
		45%	55%	96.35	3.65	2.53	2.45	2.651	2.678	3.26%	11.1%	70.7%	0.39%	3.28%
		60%	40%	96.35	3.65	2.52	2.45	2.651	2.678	2.61%	10.8%	75.9%	0.39%	3.28%
		100%	0%	94.4	5.6	2.51	2.46	2.651	2.678	1.79%	12.3%	85.4%	0.39%	5.23%

Table 3.42 Superpave estimated volumetric data for wearing course at 4% air voids

Virgin Aggregate Source	RAP Source	RAP	Virgin	Pb (est)	VMA (est)	VFA (est)
Deena	Mandra	0%	100%	5.04	15.13	73.56
		10%	90%	4.79	14.25	71.92
		20%	80%	4.80	14.10	71.64
		30%	70%	4.72	13.69	70.79
		45%	55%	4.42	13.98	71.39
		60%	40%	4.40	14.55	72.52
		100%	0%	4.30	15.33	73.91
	Nowshera	0%	100%	5.04	15.13	73.56
		10%	90%	4.78	14.34	72.11
		20%	80%	4.43	14.33	72.09
		30%	70%	4.29	14.13	71.70
		45%	55%	4.17	13.94	71.30
		60%	40%	3.86	13.36	70.07
		100%	0%	5.10	11.77	66.02
Margalla	Mandra	0%	100%	4.04	14.10	71.64
		10%	90%	4.15	14.71	72.81
		20%	80%	4.04	14.37	72.17
		30%	70%	4.00	14.33	72.09
		45%	55%	3.70	14.41	72.23
		60%	40%	3.54	14.45	72.32
		100%	0%	4.30	15.33	73.91
	Nowshera	0%	100%	4.04	14.10	71.64
		10%	90%	4.26	12.40	67.73
		20%	80%	4.22	12.25	67.33
		30%	70%	4.11	11.98	66.61
		45%	55%	3.90	11.75	65.96
		60%	40%	3.64	11.51	65.25
		100%	0%	5.10	11.77	66.02

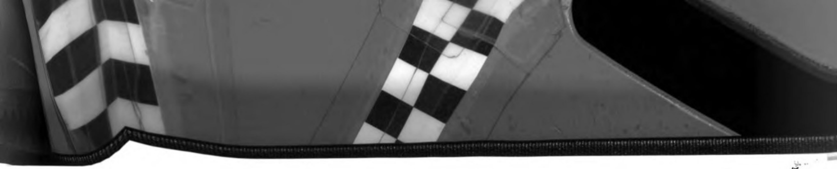
Table 3.43 Superpave estimated volumetric data for base course at 4% air voids

Virgin Aggregate Source	RAP Source	RAP	Virgin	Pb (est)	VMA (est)	VFA (est)
Deena	Mandra	0%	100%	4.51	14.24	71.91
		10%	90%	4.12	13.76	70.94
		20%	80%	4.01	13.72	70.85
		30%	70%	3.85	13.64	70.67
		45%	55%	3.62	13.53	70.44
		60%	40%	3.47	13.43	70.22
		100%	0%	4.29	14.08	71.59
	Nowshera	0%	100%	4.51	14.24	71.91
		10%	90%	4.02	14.99	73.31
		20%	80%	3.84	14.65	72.70
		30%	70%			
		45%	55%			
		60%	40%			
		100%	0%			
Margalla	Mandra	0%	100%	3.76	12.00	66.67
		10%	90%	3.43	12.55	68.12
		20%	80%	3.36	12.42	67.80
		30%	70%	3.25	12.48	67.95
		45%	55%	3.04	12.64	68.36
		60%	40%	2.91	12.69	68.48
		100%	0%	4.29	14.08	71.59
	Nowshera	0%	100%	3.76	12.00	66.67
		10%	90%	3.71	11.84	66.20
		20%	80%	3.67	11.68	65.76
		30%	70%	3.56	11.41	64.96
		45%	55%	3.35	11.18	64.23
		60%	40%	3.09	10.94	63.45
		100%	0%	4.72	12.51	68.02

Table 3.44 SPT results for a recycled Margalla-Nowshera wearing course at 4°C

Sample	Property	Frequency (Hz)					
		25	10	5	1	0.5	0.1
MW+N10%	E* (Ksi)	3,726	3,683	3,665	3,368	3,236	2,706
	δ (deg)	7.91	6.87	6.80	10.59	11.18	12.05
MW+N20%	E* (Ksi)	4,840	4,758	4,196	3,544	3,081	2,532
	δ (deg)	14.14	17.84	16.78	22.40	23.67	26.35
MW+N30%	E* (Ksi)	4,267	3,648	3,472	3,142	2,944	2,381
	δ (deg)	3.03	6.67	7.51	10.87	11.34	13.00
MW+N45%	E* (Ksi)	3,246	4,347	4,096	3,762	3,540	2,805
	δ (deg)	5.71	5.52	5.38	8.10	9.86	13.60
MW+N60%	E* (Ksi)	4,294	4,054	3,843	3,378	3,162	2,681
	δ (deg)	5.58	6.85	7.18	8.97	9.60	11.29
N 100%	E* (Ksi)	4,080	3,679	3,473	3,044	2,877	2,453
	δ (deg)	3.11	5.71	6.67	8.16	8.99	10.79

Note: MW = Margalla Wearing, N = Nowshera



SELECTION OF ASPHALT RECYCLING METHODS AND RECYCLED ASPHALT MIXTURE PROPERTIES

VOLUME II

By

Nicholas James Cerullo

A THESIS

Submitted to
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CHAPTER 4

DATA ANALYSIS

4.1 GENERAL

As stated in Chapter 1, the purpose of this study is to evaluate the field and laboratory information collected during this study to determine whether or not AC recycling can be used to rehabilitate the pavements. Please note that the field data could be used to determine if the pavements are in need of rehabilitation. The decision to use recycling as a rehabilitation technique depends upon the following factors:

- The quality and the structural capacity of the lower pavement layers and the roadbed soils are adequate to withstand future traffic loads. This is especially true for in-place recycling.
- The properties of the asphalt materials, in general, and the asphalt binder, in particular, would support recycling.

In order to make a practical decision, field data were collected on the twelve AC sites and laboratory tests were conducted on virgin binders, virgin mixtures, recycled (blended) binders and recycled mixtures.

In this Chapter, analyses of the field and laboratory data are presented and discussed. Results of the analyses were used to determine the type of recycling methods that can be adopted as a pavement rehabilitation alternative. As explained in the literature review, there are specific conditions and proper applications for various asphalt recycling methods. The basis for deciding which recycling method will be used depends on the following information:

- Thickness of the AC layer

- Type and mechanisms of distress
- Quality of the lower pavement layers
- The age of the asphalt material
- Life cycle costs of applicable rehabilitation alternatives

Results of the data analyses that are related to the above information except for the life cycle cost are summarized in subsequent sections of this chapter. Unfortunately, no cost data were available for the life cycle cost analysis and hence, only guidelines were developed.

4.2 SUMMARY AND EVALUATION OF FIELD DATA

Upon receiving the field data, they were summarized and evaluated as enumerated below.

1. Summary of the distress data
2. Evaluation of the pavement cores and soil borings
3. Evaluation of the longitudinal profiles
4. Analysis of the transverse profiles
5. Analysis of the deflection data

The detailed results of the data analyses are presented next.

4.2.1 Summary of Distress Data (Cracking)

For each of the twelve AC test sites, the pavement surface distress type, severity and extent were summarized. The summary which was presented in Table 3.3 of Chapter 3, is based on the established levels of distress severity and extent presented in Table 3.2. In general, the dominant distresses for the test sites are:

- Fatigue cracking along test sites 4, 6 & 9

- Block cracking along test sites 1, 2, 4 & 6
- Longitudinal cracking along test site 10
- Transverse cracking along test site 6

The pavement conditions of the twelve sites were further used in accordance with other field data analyses to determine candidate recycling methods. This is presented in Section 4.3 below.

4.2.2 Pavement Cores and Soil Boring

As stated in Chapter 3, the thickness of the AC layer was measured from the pavement cores whereas the thicknesses of the base and subbase layers were determined from the soil borings or from construction records. The thicknesses of all pavement layers are provided in Table 3.5. Further, the pavement cores were examined for crack initiation and propagation. Table 3.4 provides a list of the depth of top-down cracks that were measured from cores for test sites 1 through 4.

4.2.3 Evaluation of Longitudinal Profile

The longitudinal pavement profiles that were measured along six of the twelve test sites were analyzed and the profile index (PI) and the International Roughness Index (IRI) were determined. A summary of the calculated PI_O and IRI is provided in Table 3.6. Examination of the data provided in the table indicates that, except for test site 1, the IRI within each test site vary substantially along the site. This observation could be related to the pavement surface distress. For example, the IRI along test site 3 is the lowest compared to the other test sites and the highest along test sites 4 and 6. This was expected and is mainly due to the types of the pavement surface distresses and their severities and extents. The only distress along test site 3 is occasional, low severity longitudinal cracks

(which may not affect pavement roughness); whereas frequent transverse, block and fatigue cracking were observed along the other five test sites (see Table 3.3).

4.2.4 Transverse Profile Analysis

All available transverse profile data (see Appendix B) for test sites 1 through 8 and 12, where rutting was observed, were analyzed. Examination of the data listed in Table 3.8 indicates that test sites 7 and 12 showed extensive, high severity rutting. The other test sites showed marginal to extensive extents and low to moderate severities of rutting. It was also observed that for these seven sites, the shape of the transverse profiles had variability within the test sites and sections. An analysis was completed, using the NCRHP procedure (NCHRP 2002) (see Chapter 2), in order to determine whether or not the transverse profile data could be evaluated to determine the seat of rutting.

The analysis used to evaluate each transverse profile of the above noted nine test sites noted above. The analyses consisted of determining the depth and the width of the rut channel of each profile. This information was then used to calculate the areas of rutting in each wheel path. Figure 4.1 illustrates the NCHRP procedure using a transverse profile measured across the pavement of test site 12. The figure demonstrates the areas of rutting (positive and negative) in respect to the profile reference line. These areas, along with the rut depth were needed in order to determine the seat of rutting and hence, to select the proper recycling method.

A step by step example for the determination of the seat of rutting using the NCHRP procedure (NCHRP 2002) is presented below.

1. Calculate the positive and negative areas of each transverse profile shown in Figure 4.1. For this example, the positive and negative areas are 27.3 in^2

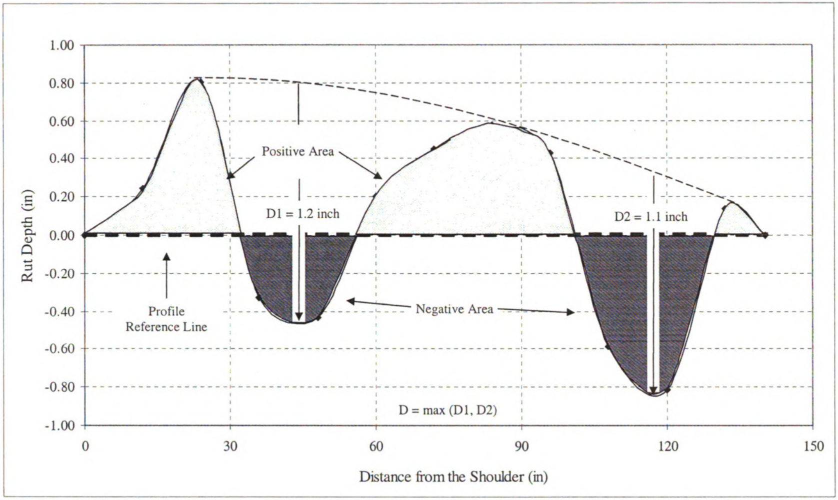


Figure 4.1 Transverse profile of M2S-12 section 2, at 295.3 ft

($17{,}613\ mm^2$) and -19.4 in^2 ($-12{,}516\ mm^2$), respectively.

2. Calculate the total area (A) of the transverse profile by summing the positive and negative areas. For this example, A = 27.3 + (-19.4) = 7.9 in^2 or 5,097 mm^2.
3. Calculate the critical ratio (R) of the absolute values of the positive and negative areas. In this example R = 27.3/19.4 = 1.4.
4. Calculate the maximum rut depth (D) from the transverse profile. For this example, D = 1.2 in or 30 mm.
5. Use equations 1, 2 and 3 to calculate three critical conditions (theoretical average areas); C1 for HMA rutting, C2 for base/subbase rutting, and C3 for roadbed soil rutting.

$$C1 = (-858.21 \times D) + 667.58 \tag{1}$$

$$C2 = (-1509.0 \times D) - 287.78 \tag{2}$$

$$C3 = (-2120.1 \times D) - 407.95 \tag{3}$$

Where, D is in mm, and C1, C2, and C3 are in mm^2. For this example, C1 = -25,079 mm^2, C2 = -45,558 mm^2, and C3 = -64,011 mm^2.

6. Use Figure 4.2 to determine the rutting seat.

As an alternative to these equations, Figures 4.2 and 4.3 can be used to determine the seat of rutting. For this example, R was greater than 0.05 and A was greater than (C1+C2)/2, so the seat of rutting was in the HMA layer. When using Figure 4.3, the point (30, 5097) (D,A) lied above a midway line between the average HMA and base failure lines, so the seat of rutting was again determined to be in the HMA layer.

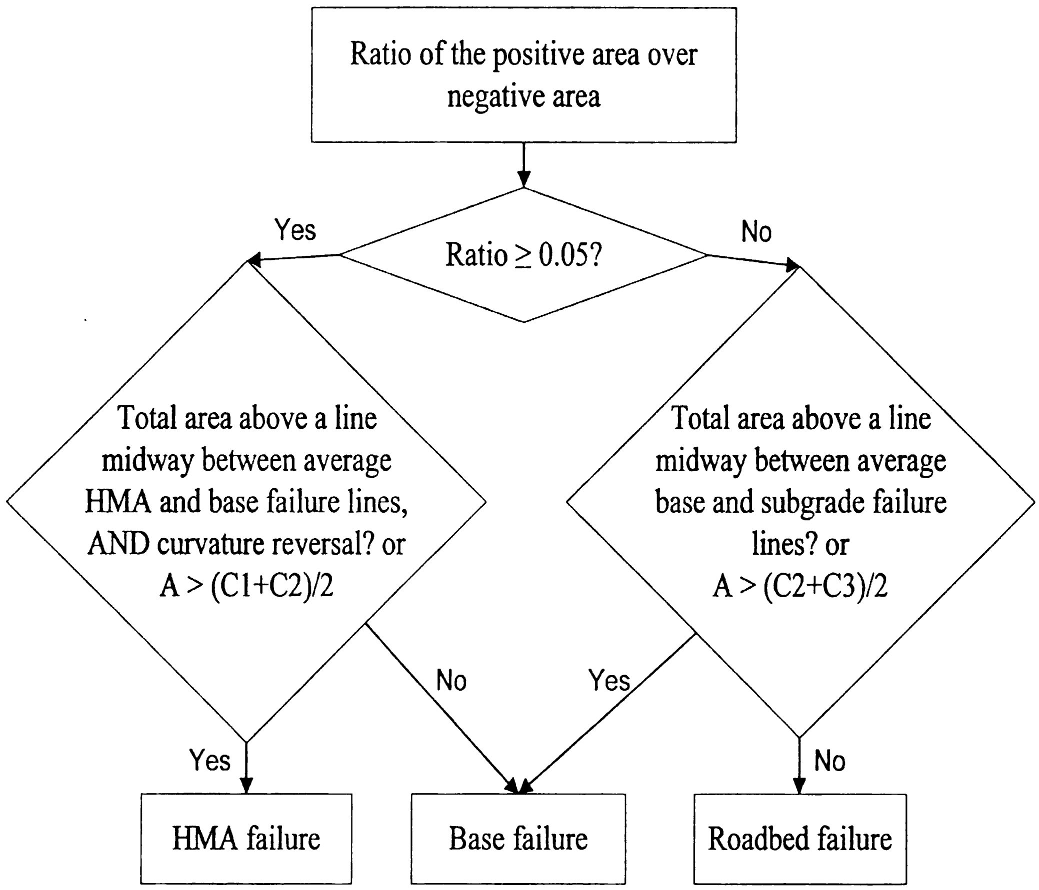


Figure 4.2 Conditions for determining the rutting seat (NCHRP 2002)

The curvature reversal condition (see Figure 4.2) was ignored due to large incremental readings. The NCHRP method requires the rut depth measurements across the pavement to be a maximum increment of 4 inches, while for this study an increment of 12 inches was used. Having a larger increment than what was required may have caused the curvature reversal to be unnoticed within the measurements. A representative transverse profile for the test sites can be seen in Appendix B (Figures B-26 through B-34).

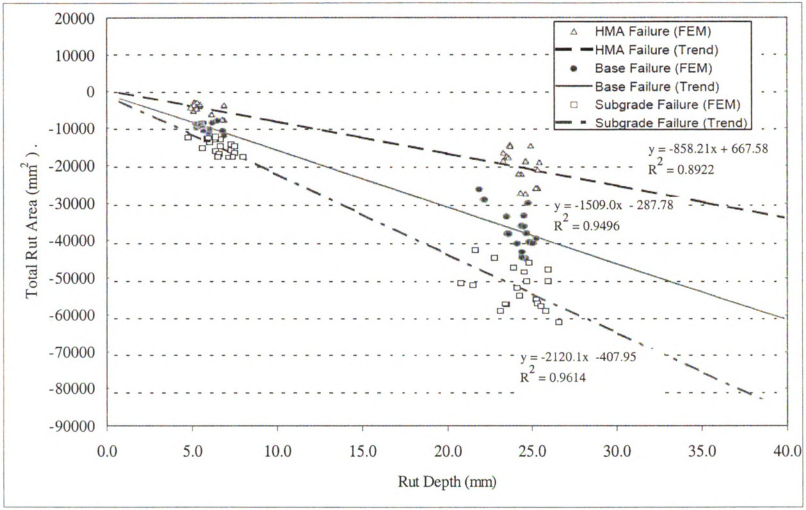


Figure 4.3 Correlation of the type of failure as a function of maximum rut depth and total rut area (NCHRP 2002)

The NCHRP procedure was utilized on all available transverse profiles to identify the seat of rutting. Table 4.1 shows a summary of the results found in Steps 1 through 6 for each test site where rutting was present.

Table 4.1 Summary of failure type using NCHRP rutting procedure

Site #	Average Area[1] (in^2)		Total Area[2]		R[3]	Average rut depth (in)/ (mm)	Theoretical average total areas[4] (mm^2)			Seat of rutting
	(+)	(-)	(in^2)	(mm^2)			C1	C2	C3	
1	8.61	-6.32	2.3	1477.9	1.36	0.33/ 8.4	-6,505	-12,900	-18,128	**HMA**
2	9.34	-16.40	-7.1	-4553.5	0.57	0.5/ 15.8	-12,904	-24,152	-33,936	**HMA**
3	8.37	-17.01	-8.6	-5573.6	0.49	0.5/ 15.2	-13,010	-24,338	-34,197	**HMA**
4	27.02	-0.83	26.2	16902.9	32.75	0.49/ 12.4	-9,942	-18,943	-26,617	**HMA**
5	9.22	-13.40	-4.2	-2696.9	0.69	0.4/ 11.2	-8,976	-17,245	-24,232	**HMA**
6	34.65	-2.31	32.3	20865.1	15.01	0.54/ 14.0	-11,354	-21,425	-30,105	**HMA**
7	0.80	-32.59	-31.8	-20505.4	0.02	0.68/ 16.3	-13,362	-24,957	-35,067	**Base**
8	13.53	-3.65	9.9	6370.4	3.70	0.43/ 10.8	-8,605	-16,591	-23,314	**HMA**
12	20.27	-11.39	8.9	5725.7	1.78	0.74/ 18.9	-15,565	-28,829	-40,508	**HMA**

1. Positive and negative areas were calculated as the average of 33 transverse profiles along one test site.
2. The total area is calculated from the average positive and negative areas (see note 1)
3. The critical ratio (R) was calculated using the average positive and negative areas (see note 1)
4. These areas were calculated using the average rut depth in

The results of the analyses listed in Table 4.1 indicate that the seat of rutting for test sites 1 through 6, 8, and 12 is in the HMA whereas the seat of rutting is in the

base/subbase layer for test site 7. In order to verify the applicability of the NCHRP procedure, the following data were compared:

- The highest and lowest rut depth observed in Pakistan test sites versus those found in the NCHRP report. The former were 0.74 inches (18.9 mm) and 0.33 inches (8.4mm) (See Table 4.1) and the latter were 1.3 inches (33 mm) and 0.25 inches (6mm). . The data shows that the rutting depths that were observed in Pakistan are within the range of rut depths that were analyzed in the NCHRP report.
- The maximum tire pressures and truck loads that are observed in Pakistan versus those that were analyzed in the NCHRP report. It is stated in the NCHRP report that, "...high contact stresses do not affect the shape of the pavement transverse profile if the same rut depth is achieved." Since the rut depth was similar, as noted above, the tire pressures and loads were examined to determine if the contact stresses were different. The maximum analyzed tire load and pressure in the NCHRP report was 5,840 lb and 105 psi, respectively. According to a study done by the National Transport Research Centre (NTRC 1995), the average truck tire pressure in Pakistan was 130 psi while 75% of the single axle loading was at 26,000 lb. For the purpose of this study, the single axle load was increased to 30,000 lb or 7,500 lb per tire. The contact stresses for the load and tire pressure scenario in the NCHRP report was 55.6 in^2 and for the Pakistan scenario it was calculated to be 57.7in^2. This indicates that the contact stresses and rut depth are almost similar and hence, the shape of the pavement transverse profile does not change.

The results must be viewed still be taken with caution because the tire pressure in Pakistan is drastically higher than the tire pressure in the NCHRP report. Further, results in Table 4.1 were compared to the backcalculated layer moduli values in the subsequent section to verify any lower layer weakness.

4.2.5 Deflection Data and Backcalculation of Layer Moduli

As noted in Chapter 3, all twelve AC sites were subjected to deflection tests using a falling weight deflectometer (FWD). The deflection data are listed in Appendix C. Each FWD test was conducted using three load levels (6,000, 9,000, and 15,000 pounds). The deflection data were used to assess the linearity of the pavement responses to load and backcalculate the layer moduli using the MICHBACK computer program.

To analyze the linearity of the pavement response to load, for every test site, the measured peak pavement deflections were plotted against the three applied load levels. Figure 4.4 depicts an example of such plot for test site 1. The plots for the other test sites are shown in Appendix C. Examination of the data shown in Figure 4.4 and in Appendix C indicates that, for all 12 test sites, the pavement response to 6,000, 9,000 and 15,000 pound loads is nearly linear. Given that information, it was decided to use the MICHBACK computer program (a linear elastic layer program) to backcalculate the pavement layer moduli as presented and discussed below.

After studying the linearity of the pavement response to load, the measured deflections and the layer thicknesses data were used to backcalculate the layer moduli using the MICHBACK software. The forward engine of the MICHBACK software is a linear elastic layer computer program known as ChevronX. The MICHBACK program

uses a Newtonian algorithm to converge the calculated deflection values to the measured ones in an iterative manner (Harichandran et al. 1994).

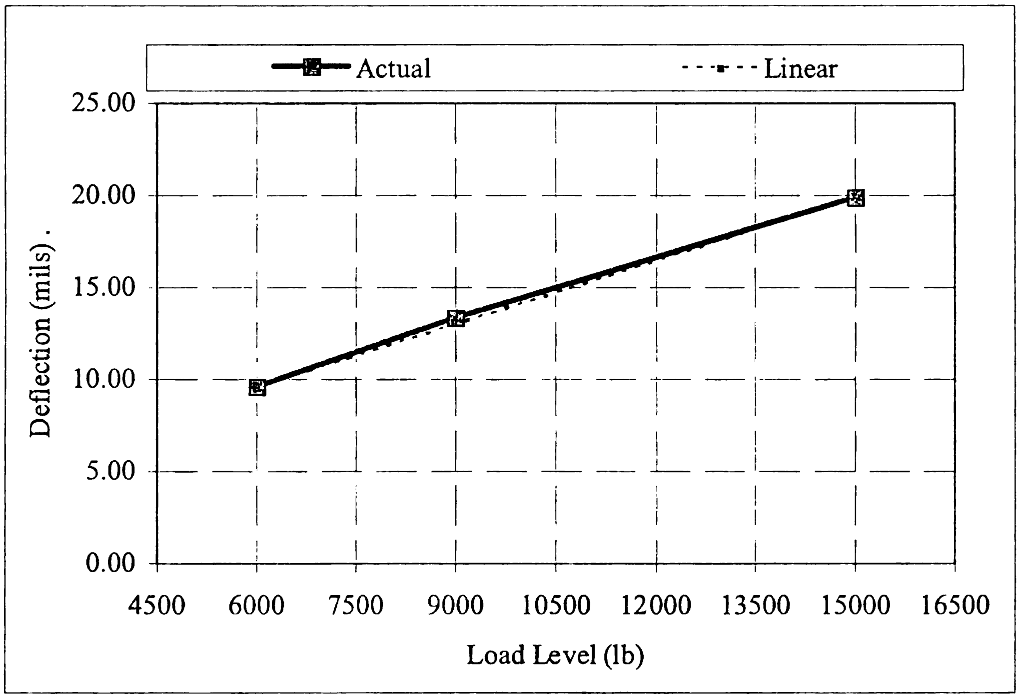


Figure 4.4 Deflection data vs. load level linearity plot for test site 1

At the onset of the backcalculation of the pavement layer moduli, for each FWD test, the measured deflection data were plotted against the radial distance from the center of the loaded area (the deflection basin). The plot was then used to assess whether or not:

1. The deflection data measured by the deflection sensors are consistent (no sand particles under the sensors).
2. The deflection test was not conducted in the vicinity of cracks.
3. A deflection sensor is malfunctioning or out of calibration.

It was found that for all FWD tests, the deflection data measured by the third deflection sensor (D2), which is located at a distance of 11.81 inches from the center of the loaded area was either malfunctioning or out of calibration. The measured deflection data consistently showed abnormality at sensor D2 as shown in Figure 4.5. Hence, the D2 deflection data were eliminated and the resulting deflection basin shown in Figure 4.6 was used in the backcalculation.

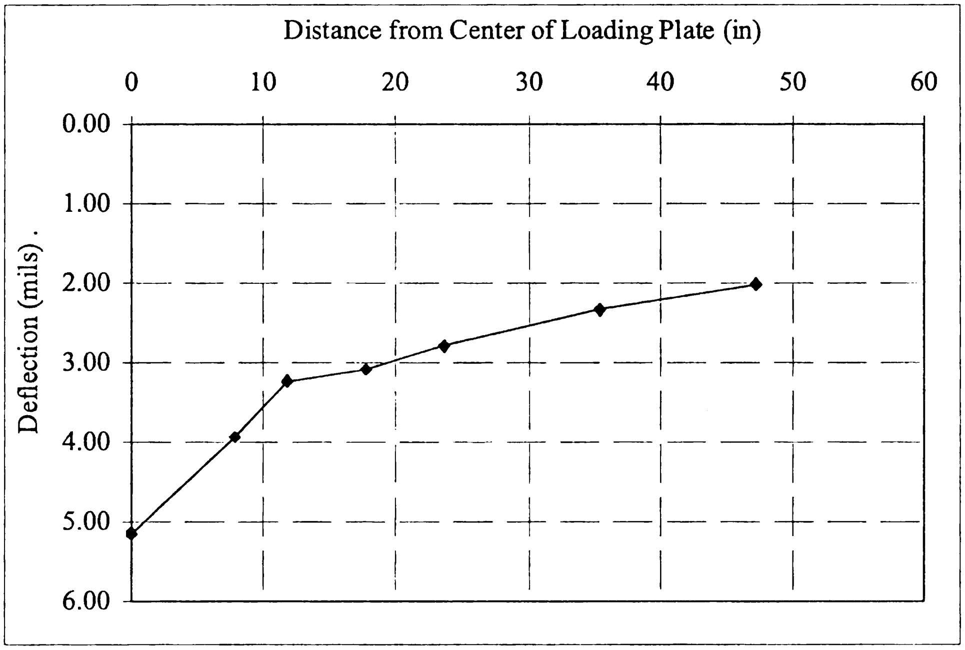
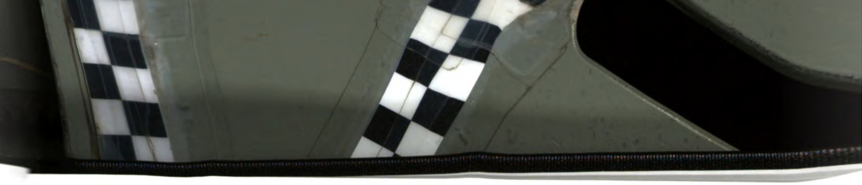


Figure 4.5 Deflection basin of sensors D0 through D6

The measured deflection data at the 9,000 lb load level was used in the backcalculation. Also, as seen in Figure 4.6, the seven sensors that were used in the backcalculation process are D0, D1 and D3 through D7. In the backcalculations a three layer system (AC, combined base and subbase, and roadbed soil) was adopted. The layer thicknesses were determined from the pavement cores and soil borings, as seen in Table 3.5, were used as inputs to the MICHBACK software.



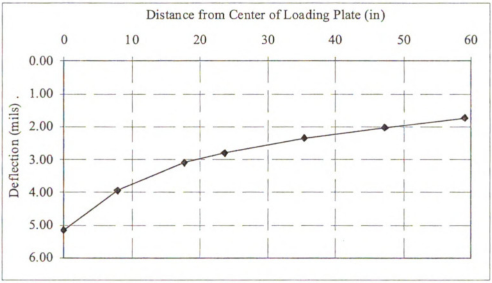


Figure 4.6 Deflection basin of sensors D0, D1 and D3 through D7

In order to determine the presence of a stiff layer, Boussinesq equations (Equations 4 and 5) were used to calculate the equivalent modulus (E_o) of the pavement system using the measured deflection data at each deflection sensor.

$$E_o(0) = 2\frac{(1-\mu^2)\sigma_0 \times a}{d(0)} \tag{4}$$

$$E_o(r) = \frac{(1-\mu^2)\sigma_0 \times a^2}{r \times d(r)} \tag{5}$$

$E_O(r)$ = surface modulus at a distance r from the center of the FWD loading plate

μ = Poisson's ratio (0.5 assumed)

σ_o = contact stress under the loading plate (82 psi)

$d(r)$ = deflection at a distance r (inch)

a = radius of loading plate (5.91 inch)

By plotting E_o against the distance between the sensor and the load, there are four possible outcomes. These possibilities are shown in Figure 4.7 and are as follows:

- Figure 4.5a – Deep stiff layer (720 inches or more)
- Figure 4.5b – Stiff layer at a shallow depth
- Figure 4.5c – Stiff layer at intermediate depth (150-500 inch)
- Figure 4.5d – Deep stiff layer or no stiff layer

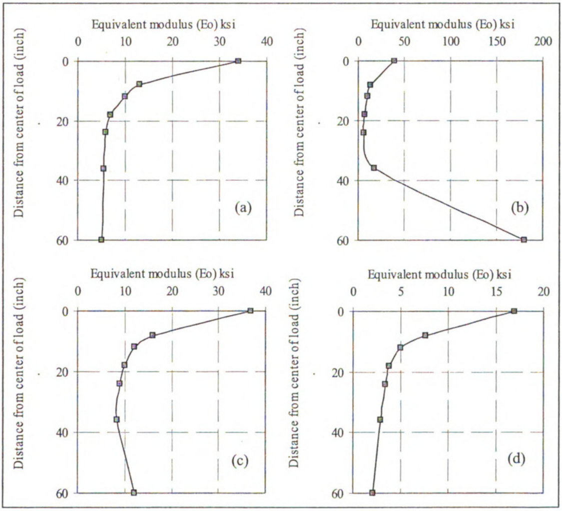


Figure 4.7 Plots used in determining the presence of a stiff layer

The backcalculated moduli results for all FWD tests can be seen in Appendix C, Tables C-1 through C-34. A root mean square (RMS) of less than 5% was used as the criterion for acceptable backcalculated modulus values. The average, standard deviation, maximum and minimum of the accepted moduli values for each AC site and their respective section can be seen in and Tables 4.2 and 4.3. Within each site, an analysis was done on the variability of the backcalculated AC, base/subbase and roadbed soil moduli. An example of the variability of the AC modulus for test site 3 is shown in Figure 4.8, the rest of the figures are shown in Appendix C, Figures C-12 through C-23.

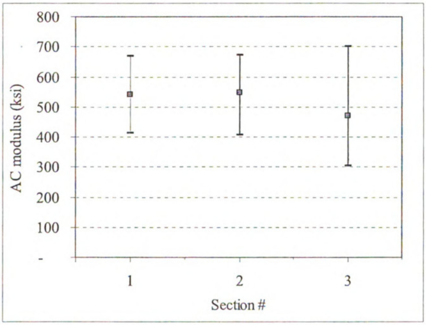


Figure 4.8 Variability of the backcalculated AC modulus for test site 3

Analyzing the information in Tables 4.2 and 4.3 and variability graphs (such as Figure 4.8) the following assessment was determined about the backcalculated AC layer moduli:

- Test sites 1 and 12 show a high variability between their sections.
- The average AC modulus exceeds 1,000 ksi in test sites 1, 5 and 12.

Table 4.2 Average, standard deviation, maximum and minimum modulus values for test sites 1 through 6

Site Number	Layer	Section 1				Section 2				Section 3			
		Modulus Values (ksi)											
		Average	Std. Dev	Maximum	Minimum	Average	Std. Dev	Maximum	Minimum	Average	Std. Dev	Maximum	Minimum
1	AC	564	153	901	278	1,435	851	3,118	193	1,003	569	2,456	229
	B/SB	35	6	44	24	156	57	327	90	94	31	162	50
	RB Soil	34	4	40	26	20	2	24	17	24	4	32	19
2	AC	530	510	2,157	101	470	526	1,970	126	536	380	1,466	123
	B/SB	185	68	338	84	204	58	306	115	129	55	249	67
	RB Soil	74	14	96	57	81	21	125	48	60	13	79	36
3	AC	542	180	669	414	548	83	671	408	471	101	701	304
	B/SB	47	10	54	39	44	13	63	26	45	11	63	29
	RB Soil	57	10	64	50	14	3	19	11	40	5	47	28
4	AC	359	51	426	243	266	40	360	199	317	54	403	201
	B/SB	28	5	38	18	26	5	38	19	25	7	44	17
	RB Soil	27	3	31	23	35	6	50	27	34	2	39	31
5	AC	1,125	411	2,185	608	1,036	390	1,803	490	1,009	187	1,274	633
	B/SB	26	4	34	18	27	5	36	18	28	4	35	19
	RB Soil	16	3	22	12	15	3	21	12	17	2	22	14
6	AC	405	202	839	190	316	131	651	171	316	137	654	148
	B/SB	21	5	30	13	19	3	24	13	26	7	43	16
	RB Soil	16	2	18	12	18	2	20	15	24	4	34	18

Note: AC = asphalt concrete layer, B/SB = base – subbase layer, RB = roadbed soil layer

Table 4.3 Average, standard deviation, maximum and minimum modulus values for test sites 7 through 12

Site Number	Layer	Section 1				Section 2				Section 3			
		Modulus Values (ksi)											
		Average	Std. Dev	Maximum	Minimum	Average	Std. Dev	Maximum	Minimum	Average	Std. Dev	Maximum	Minimum
7	AC	639	239	1,128	248	747	399	1,381	200	774	308	1,524	411
	B/SB	69	20	101	33	78	19	107	51	84	15	111	64
	RB Soil	22	4	30	15	24	3	29	20	23	3	28	17
8	AC	509	173	903	329	712	352	1,629	301	615	272	1,196	222
	B/SB	19	2	23	15	16	3	21	12	16	3	25	12
	RB Soil	14	1	16	11	22	2	27	20	13	1	15	11
9	AC	596	255	858	349	616	172	959	409	825	222	1,322	523
	B/SB	39	7	46	33	41	17	63	15	55	28	130	24
	RB Soil	70	26	90	40	75	17	100	51	87	45	180	47
10	AC	546	137	705	288	447	135	752	277	No measured deflection information			
	B/SB	45	40	156	15	44	34	137	15				
	RB Soil	45	28	94	16	53	28	130	27				
11	AC	769	310	1,280	325	834	338	1,252	317	No measured deflection information			
	B/SB	34	16	58	16	35	31	89	11				
	RB Soil	16	6	25	10	30	14	54	19				
12	AC	2,342	516	2,986	1,787	1,810	778	3,148	851	1,166	289	1,631	711
	B/SB	73	32	150	45	95	56	214	37	104	73	219	38
	RB Soil	120	36	182	44	75	23	125	46	77	15	110	51

Note: AC = asphalt concrete layer, B/SB = base – subbase layer, RB = roadbed soil layer

- The average AC modulus exceeds 250 ksi for test sites 2, 3, 4, 6, 7, 8, 9, 10 and 11.
- Due to a wide range in AC moduli from site to site, there is an indication of a large variability in aging.

From the above assessment, it was determined that the AC layer for test site 1, has undergone major aging and hardening. This is of some concern because the pavement is less than seven years old at this site, compared to older than seven years old for test sites 5 and 12.

The following assessment was determined about the backcalculated base/subbase layer moduli:

- Test sites 1, 2 and 12 show high variability between their sections.
- The minimum base/subbase modulus in test sites 4, 5, 6, 8, 9, 10 and 11 range from 10 to 20 ksi. However for these particular sites, the average modulus exceeded 15,000 ksi.
- The minimum base/subbase modulus in test sites 1, 2, 3, 7, and 12 range from 24 to 67 ksi.

The base/subbase modulus of test site 7 was cautiously analyzed because in Section 4.2.4 it was determined that the seat of rutting was in the base layer. From the backcalculated results, the base/subbase modulus of this layer is relatively high, having an average value of 77 ksi.

The following assessment was determined for the backcalculated roadbed soil layer moduli:

- For all sites, the roadbed soil modulus was relatively high. For every site, the minimum values exceeding 10 ksi.
- For test sites 1, 4, 5, 6, 7, 8 and 11 the average roadbed soil modulus ranged from 14 to 35 ksi
- For test sites 2, 9, 10 and 12 the average roadbed soil modulus ranged from 45 to 125 ksi
- For test site 3, a large variability in the roadbed soil modulus was found. Sections 1 and 3 were relatively similar at 57 and 40 ksi respectively. For section 2 the modulus was backcalculated to be 14 ksi.

The roadbed soil modulus value was determined to be adequate to support recycling. In general, a roadbed soil with a modulus greater than 5 ksi does not require any attention when there is a need to rehabilitate the upper pavement layers.

4.3 LABORATORY DATA EVALUATION

Results from various laboratory tests and climatic data were analyzed to determine whether or not the recovered materials are suitable for recycling. First, historical climatic data in Pakistan were obtained and analyzed to determine the required base performance grade (PG) of each test site. Second, blended binders were tested and analyzed to determine their PG. Based on the results, the percent of recovered material to be used that meet the required base PG of a particular site could be determined. Finally volumetric and simple performance test (SPT) data were analyzed to assess the performance of the recycled asphalt mixtures. The various analyses stated above are detailed in the next subsections.

4.3.1 Evaluation of Climate and Required PG Grade

The climate of Pakistan was evaluated in a previous report (Baladi et al. 2008) by gathering historical temperature data from 21 weather stations within Pakistan. The hottest 7-day period was determined for each year and then the average maximum air temperature for the 7-day periods was calculated for each station. Similarly, the lowest minimum air temperature of the year was determined for each year and was averaged over the available years' data to get an average one-day minimum air temperature. Based on these findings, the minimum required base performance grades (PG) were determined for various geographical regions in Pakistan as shown in Figure 4.9. From the figure it was determined that the minimum required base PG for test sites 1 through 8 and 12 is 70-10, and 64-10 for test sites 9 through 11. The minimum required base PG for each test site was adjusted as a function of traffic speed and volume as recommended by the Asphalt Institute (SP-1, 2003). The results are listed in Table 4.4.

Table 4.4 Base and shifted performance grades according to traffic speed and volume

Test sites	Base grades	Slow speed traffic and 3 to 29.9 million ESALs	Standing traffic and more than 30 million ESALs
9-11	PG 64-10	PG 70-10	PG 76-10
1-8, 12	PG 70-10	PG 76-10	PG 82-10

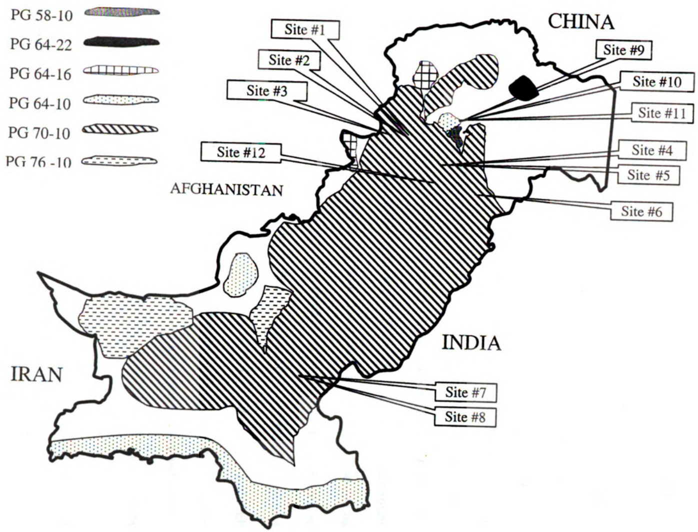


Figure 4.9 Base binder PG for twelve test sites

4.3.2 Analysis of Asphalt Blended Binder

The virgin binder ARL 60/70 was the only binder blended with the recovered binders from Nowshera and Mandra sites. For each site, the blend was made using different percentages of recovered binder. The DSR and BBR test results were then used to determine the critical high and low temperatures and the PG of the virgin and each blended binder. It should be noted that the critical temperatures and the PG of the other five virgin binders were obtained and are listed in Table D-2 of Appendix D.

The DSR results shown in Table 3.27 were analyzed to determine the critical high temperatures. The Asphalt Institute's SP-1 defines the critical high temperature as the temperature at which the ratio $G^*/\sin\delta$ is equal 1 kPa. Since the data in Table 3.27 does not indicate $G^*/\sin\delta$ of 1 kpa, a rule of thumb (MS-25 2007) was used to determine the critical high temperatures. This rule of thumb states the ratio $G^*/\sin\delta$ can be one half if the temperature is increased by 6 degrees or doubled if the temperature is decreased by 6 degrees. For example, if the DSR output showed that at 60 °C, $G^*/\sin\delta$ is equal to 3.5 kPa, then the rule of thumb would estimate $G^*/\sin\delta$ to be 1.75 kPa at 66°C and 7 kPa at 54°C. The rule of thumb was applied to the DSR results in listed Table 3.27. Figure 4.10 shows an example of how the rule of thumb was used for a Mandra blended binder at 70% recovered material. The critical high temperature for all blended binders can be seen in Table 4.5.

The critical high temperature for the 100 percent recovered binder for Nowshera was determined to be inaccurate. This was because the rate of change of $G^*/\sin\delta$ from 90 percent to 100 percent was significantly higher than any other $G^*/\sin\delta$ when the percent recovered binder was increased by 10 percent. The rate of change of $G^*/\sin\delta$ from 90 to

100 percent for the Nowshera blends was 5.1. The rate of change of G*/sinδ for Mandra blends from 90 to 100 percent was 1.2. The highest rate of change of G*/sinδ for any 10 percent increase was determined to be 2.2. This clearly shows there was inaccuracy in the DSR data for the 100 percent recovered Nowshera binder.

In order to determine the critical low temperature, the BBR results (shown in Tables 3.28) were further analyzed. According to the Asphalt Institute (SP-1 2003), the critical low temperature of the binder is at a creep stiffness (s-value) of 300 MPa or less and/or the slope (m-value) is 0.300 or greater. Since the BBR test was only done at three temperatures (-6, -12 and -18°C), the stiffness and m-value were plotted against these temperatures so that the critical low temperature could be extrapolated from the plots. Figure 4.11 shows an example of these plots used to determine the critical low temperature. The critical low temperatures for the stiffness and m-value criteria are listed in Table 4.5.

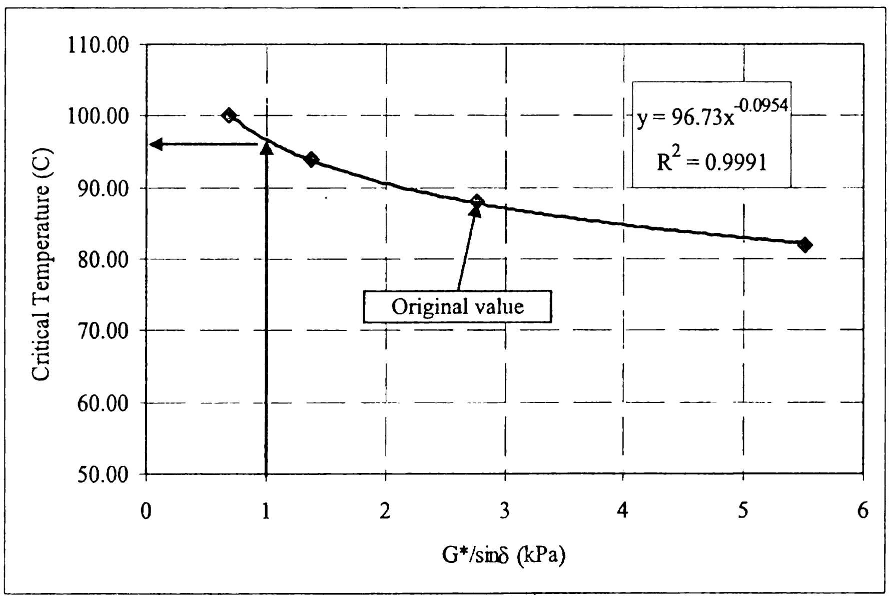


Figure 4.10 Determination of critical high temperature for Mandra 70%

Table 4.5 Critical temperatures and PG of virgin and blended binders

Source	Aging	Property	Critical Temperature (°C)					
			% Recovered (aged) Binder					
			0[a]	40	70	80	90	100
Nowshera	unaged	DSR G*/Sinδ	67.92	87.06	91.11	92.81	96.27	110.67
	aged using PAV	BBR S-value	-19.3	-18.6	-17.0	-15.1	-14.3	-13.3
		BBR m-value	-16.2	-16.6	-12.8	-9.7	-5.0	-7.9
		PG	64-16	82-16	88-10	88-4	94-4	106-4
		PG (SP1)	64-22	82-22	88-22	88-16	94-10	106-16
Mandra	unaged	DSR G*/Sinδ	67.92	86.79	96.70	99.45	106.17	107.56
	aged using PAV	BBR S-value	-19.3	-18.6	-19.1	-17.4	-17.0	-10.2
		BBR m-value	-16.2	-18.0	-10.9	-8.9	-11.4	-8.7
		PG	64-16	82-16	94-10	94-4	106-10	106-4
		PG (SP1)	64-22	82-28	94-16	94-16	106-16	106-16

Note: shaded area was removed from analysis because of inaccurate data from the BBR and DSR tests.
a = 0% recovered binder is ARL 60/70

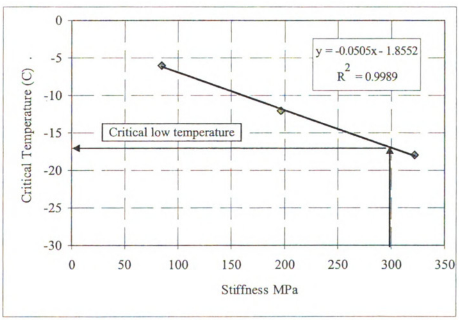


Figure 4.11 Determination of critical low temperature for Nowshera 70%

With the critical high and low temperatures known, the PG grades were determined for the blended binders and are listed in Table 4.5. It should be noted that the criterion for the m-value always controlled the low temperature PG of the binder. Two PGs are shown in Table 4.5; the first is the PG which uses the exact critical low temperature. The second (PG SP1) adds -10°C to the critical low temperature and then re-determines the PG. The second PG was determined to be accurate by the Strategic Highway Research Program (SHRP) researchers. They observed that the BBR test results at 60 seconds were equal to the BBR test results at 2 hours for a 10°C lower sample (SP-1 2003).

With the PG of the blended binders known, the optimum percentage of recovered material to use in a recycled mixture could be determined. Nowshera (which is located near test sites 1 and 2) and Mandra (which is located near test sites 4 and 5) asphalt mixtures require a PG of 82-10. Using the information in Table 4.5, the optimum percentage of recovered material to be used in a recycled mixture containing either Nowshera or Mandra recovered material is 70%.

Figures 4.12 and 4.13 show the trend between the percent recovered binder and the high and low critical temperatures of the blended binder's, respectively. It can be seen from these figures that, in general, as the percentage of recovered binder increased the critical temperatures increased. This indicates a stiffer and less ductile binder when more recovered binder is added. The data in Figure 4.12 indicates that the recovered binder from Nowshera behaves differently than Mandra at 40% blends or higher. Both trends remain relatively linear, however for Mandra blends, the rate of change of the critical high temperature is greater than that of Nowshera.

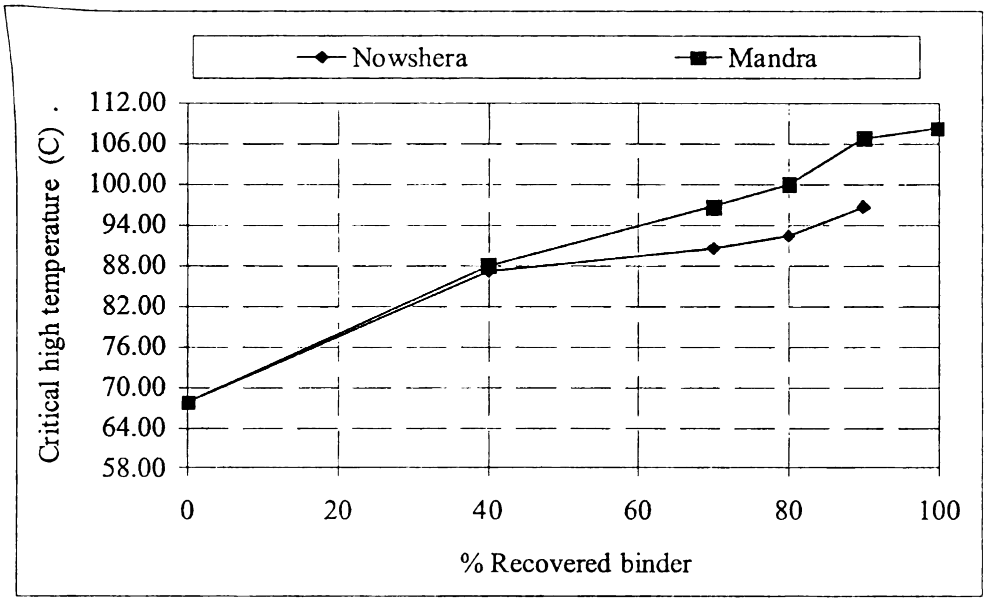


Figure 4.12 Effect of recovered binder on the critical high temperature for Nowshera and Mandra

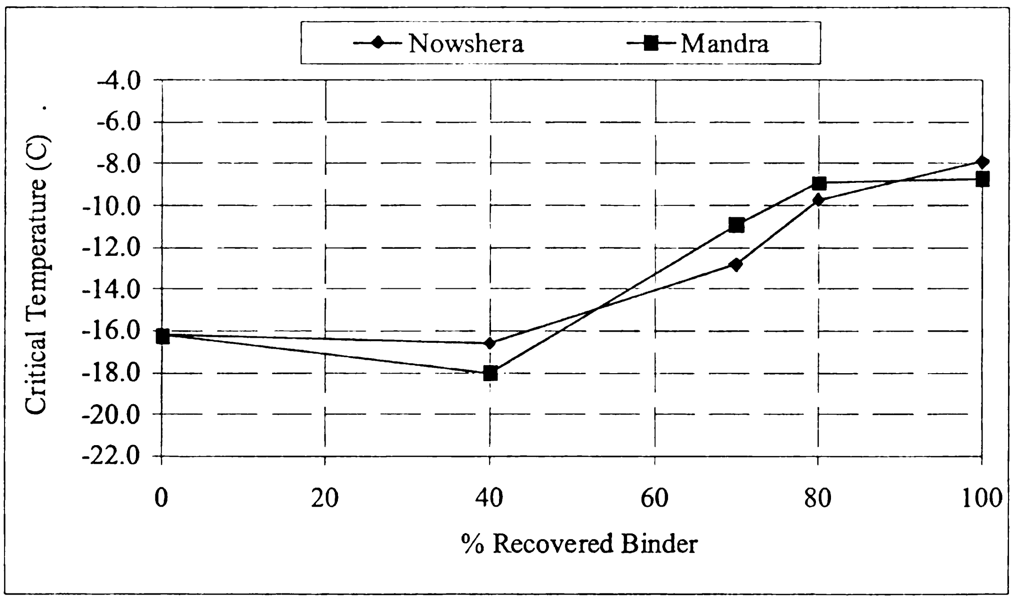


Figure 4.13 Effect of recovered binder on the critical low temperature for Nowshera and Mandra

When the critical low temperatures trends (Figure 4.13) were examined, it was noticed that both types of recovered binder follow the same trend. The figure shows that

up to 40% recovered binder, the critical low temperature stays relatively similar to that of the virgin binder. However, when the percentage of recovered binder increases above 40%, the critical low temperature starts to increase.

When the rate of change was compared for the critical high temperature versus low temperature, it was noticed there was a major difference. For example, when the percent recovered binder increases from 0 to 100, the critical high temperature increased by 40°C while the critical low temperature increased by 8°C.

The critical temperatures in Table 4.5 were compared to Equation 6 which was reported in a NCHRP report (McDaniel & Andersen 2001). Based on this report, the trend of the critical temperature with respect to the percent recovered binder should be linear. Equation 6 was rearranged to predict what the critical temperature would be as a function of percentage of RAP, T_{virgin} and T_{RAP}; this yielded Equation 7.

$$\%RAP = \frac{T_{blend} - T_{virgin}}{T_{RAP} - T_{virgin}} \tag{6}$$

$$T_{blend} = T_{virgin} + \%RAP\left(T_{RAP} - T_{virgin}\right) \tag{7}$$

T_{blend} = critical temperature of blended binder

T_{virgin} = critical temperature of virgin binder

T_{RAP} = critical temperature of recovered binder

%RAP = % recovered binder

The critical temperatures in Table 4.5 were compared with the predicted temperatures from Equation 7. The predicted critical high and low temperature are represented by the dashed line in Figures 4.14 and 4.15, while the squares are the actual critical high and

low temperatures from the DSR and BBR results. It should be noted that for Nowshera critical high temperatures, the linear trend was predicted up until 90 percent recovered binder. The 100 percent recovered binder's critical high temperature was inaccurate. The trend however can be used to reasonably predict thr critical temperature for that blend.

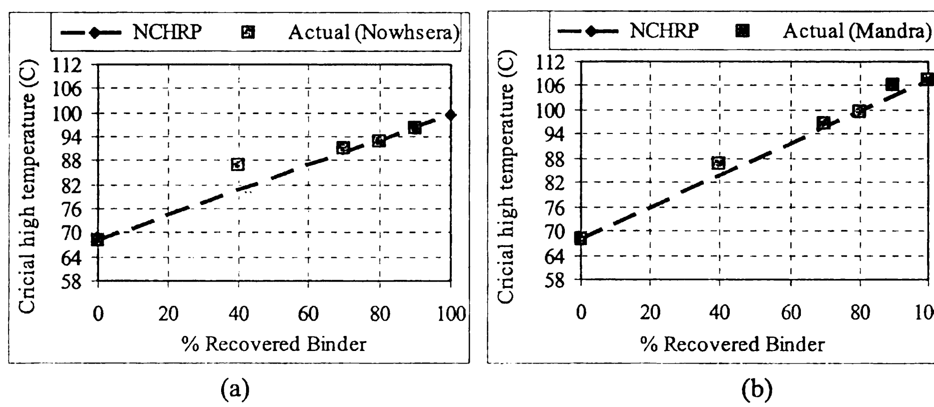


Figure 4.14 Critical high temperatures versus predicted for Nowshera blends (a) and Mandra blends (b)

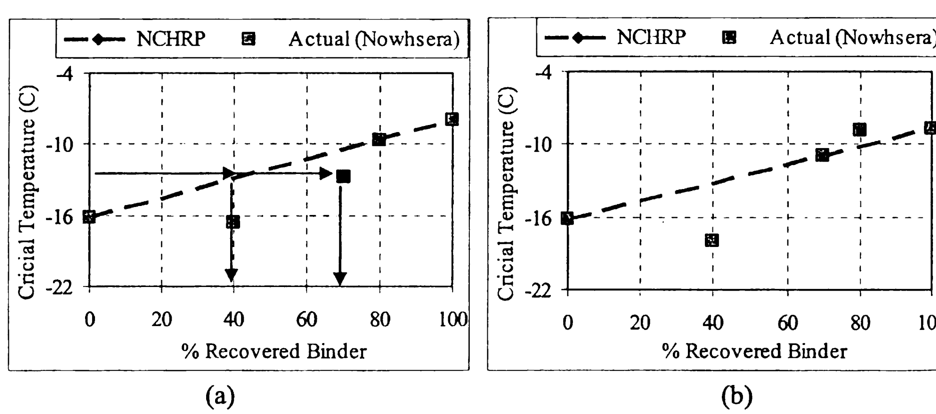


Figure 4.15 Critical low temperatures versus predicted for Nowshera blends (a) and Mandra blends (b)

For the critical high temperatures (Figure 4.14) there is a very strong correlation between the actual temperatures found versus the predicted at 70, 80, 90 and 100 percent recovered binder. For Mandra and Nowshera blends at 40 percent recovered binder, it was noticed that the actual critical low temperature deviates slightly from the linear prediction equation. These actual critical temperatures at 40 percent recovered binder are slightly higher than the predicted by the NCHRP. Overall, the NCRHP linear prediction is fairly accurate and accounts for the effect of the recovered binder on the critical high temperature.

A comparison of the critical low temperature predicted and observed values were also made. At 40% recovered binder, the critical low temperatures of both binders dropped below the virgin critical low temperature. This is considered to be highly unlikely. As the percentage of recovered binder increases, the blended binder properties should not become more ductile. It was also noticed from Figure 4.15 that at higher percentages of recovered binder (≥ 80%), the relationship between critical low temperature and percent recovered binder follows the linear trend of the prediction equation. The relationship at lower percentages of recovered binder (≤ 40%) does not show a linear trend as predicted by the NCHRP equation. However, the difference between the actual critical low temperature at 40 percent recovered binder and the critical low temperature predicted by the NCHRP equation was very small (less than 4°C). Due to only one BBR test result for each blend the small difference can be the cause of testing variability.

If more test results show that the trend which was shown in Figure 4.15 was consistent, then it was analyzed what the effect would be if the critical temperature remained constant at low percents of recovered binder. It was determined from these results that at low percents of recovered binder, the prediction equation over predicts the critical low temperature. For example, if a desired critical temperature is -13°C, the necessary percentage of recovered binder using Figure 4.15a would be 40% using the prediction equation. The actual data shows that at 70% a critical temperature of -13°C was achieved. If 40% was actually used due to lack of blending charts, then from the actual data a critical temperature of -16°C would be achieved. A blended binder with a low critical temperature of -16°C would also have properties that would exceed a binder with a critical temperature of -13°C. Thus the prediction equation was conservative because it over predicts the critical low temperature at low percents of recovered binder.

4.3.3 Evaluation of Recycled Asphalt Mixtures

As mentioned in Chapter 3, sixteen recycled asphalt mixtures were made using two different mix designs (Marshall and Superpave), two virgin aggregate sources (Margalla and Deena), two old HMA pavement sources (Nowshera and Mandra) and two asphalt course types (wearing and base). For each mixture, six blends were made at 10, 20, 30, 45, 60, and 100 percent recovered asphalt material contents. The volumetric information for the Marshall and Superpave mixtures were calculated and then analyzed to determine their relationships to the percent recovered asphalt material contents. Simple performance tests (SPT) were conducted on the Superpave mixtures to measure the mechanical properties of the recycled asphalt mixtures. The results from the SPT test, the dynamic modulus (E^*) and phase angle (δ), were analyzed in order to determine:

- The effects of the percent recovered asphalt material contents on E^* and δ of the recycled mixtures.

- The effect of the percent recovered asphalt material contents on the master curves of the recycled mixtures.

4.3.3.1 Volumetric Analysis

The volumetric data (air voids, VMA and VFA) were analyzed in order to determine the effects of the recovered asphalt material contents on the Marshall and Superpave recycled mixtures. For the Marshall type mixtures, there were insignificant correlations between the percent recovered asphalt material contents and the volumetric data. This was expected due to the variability caused by the Marshall (hammer) compaction method. For example, Figure 4.16 shows the correlation between the percent air voids (AV) and the percent of recovered asphalt material contents for a Marshall wearing course mixture type made with Deena aggregates and Mandra recovered asphalt material. It appears that the AV decrease with increasing percent recovered asphalt material contents.

Indeed, if the percent AV of the virgin mixture is excluded, then the recycled mixtures percent AV is almost constant and independent of the percent recovered asphalt material contents. When the AV of the other seven Marshall recycled mixtures were analyzed, no consistent trends between the percent recovered asphalt material contents and AV were depicted. For example Figure E-6 in Appendix E shows the percent AV increase as the percent recovered asphalt material contents increase. Similar inconsistencies were observed for the voids in mineral aggregate (VMA) and voids filled with asphalt (VFA) data. Hence, the volumetric data for the Marshall recycled mixture types do not support any conclusions.

The Superpave recycled mixture's volumetric data showed more consistency and regularity than the Marshall recycled mixtures. This was mainly due to the more consistent compaction method of the Superpave gyratory compactor. Figure 4.17

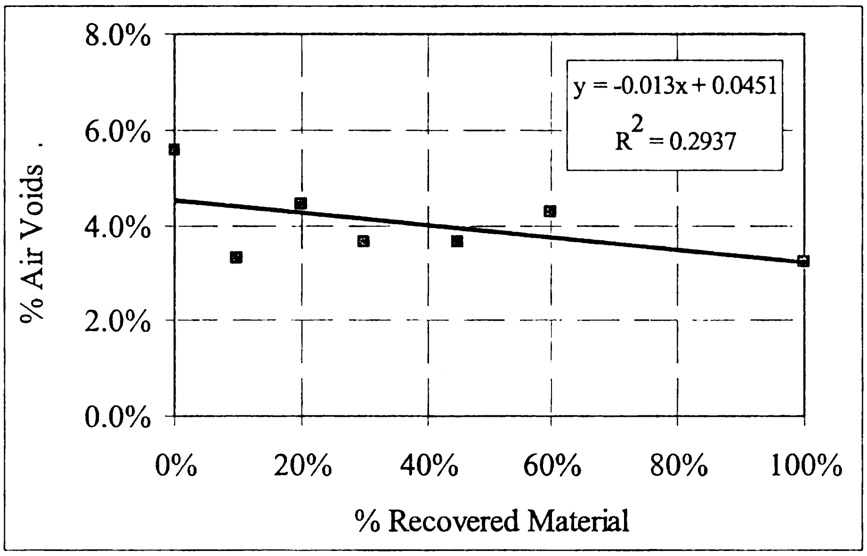


Figure 4.16 Air voids versus percent recovered material for a wearing course recycled mixture with Deena aggregates and Mandra recovered material (Marshall Mixture)

demonstrates the percent AV for the virgin and recycled mixtures. This figure clearly shows that increasing the percent recovered asphalt material contents cause decrease in the percent AV of the recycled mixtures. Further, at low percent of recovered asphalt material contents (10 to 30 percent), the percent AV remain around 4 percent. This trend was consistent for all Superpave recycled mixtures (shown in Figures E8 through E14 of Appendix E). As mentioned earlier, the asphalt binder content of each recycled mixture was same as the optimum asphalt content of the virgin mixtures. The decrease in the percent AV can be attributed to the gradation of the mixtures since the binder content remains the same. In general, recovered asphalt material has a finer gradation. During compaction the finer materials are forced to occupy the space between the coarser material leaving less air voids. This finding was also consisted with that reported by McDaniel et al. (McDaniel et al. 2006). In their report, the percent AV dropped from 3.6% (virgin) to 2.4% (40% recycled mixture).

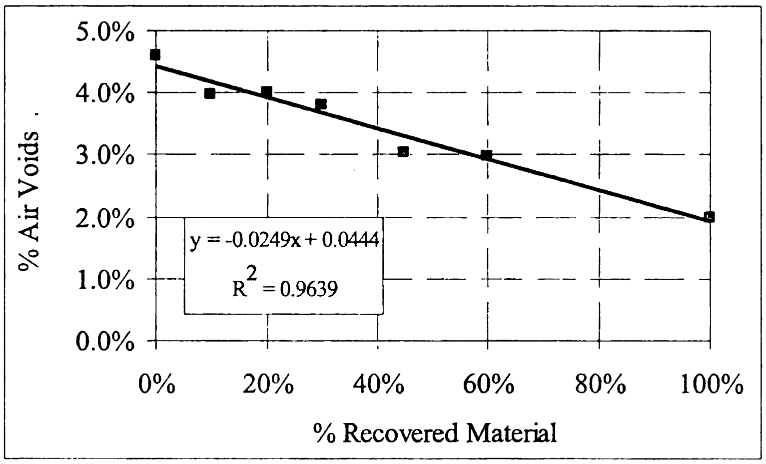


Figure 4.17 Air voids versus percent recovered material for a wearing course recycled mixture with Deena aggregates and Mandra recovered material (Superpave Mixture)

Likewise, the influences of the percent recovered asphalt material contents on the VMA and VFA of the Superpave recycled asphalt mixtures are shown in Figures 4.18 and 4.19. It can be seen that, for a given source of aggregate or aggregate blend, the VMA is almost constant and independent of the percent recovered asphalt material contents. Since VMA is an aggregate gradation property it was expected to remain relatively constant and independent of the percent recovered asphalt material contents. However, this finding contradicts those reported by Daniel and Lachance (Daniel & Lachance 2005). They stated that the VMA increases as the percent recovered asphalt material contents increased from 0 to 25 and to 40 percent. They added that if the recovered material was not adequately heated, the virgin and recovered binders would not

sufficiently blend. When this happens, the recovered material acts like aggregate making the mixture coarser and hence, yielding higher VMA. Their results indicate that there is

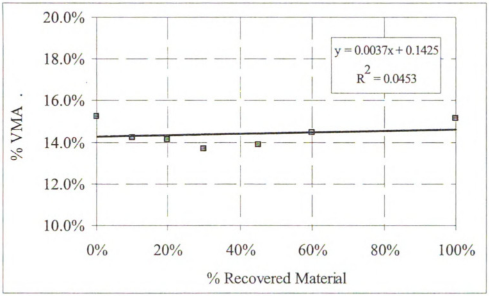


Figure 4.18 VMA versus percent recovered material for a wearing course recycled mixture with Deena aggregates and Mandra recovered material (Superpave Mixture)

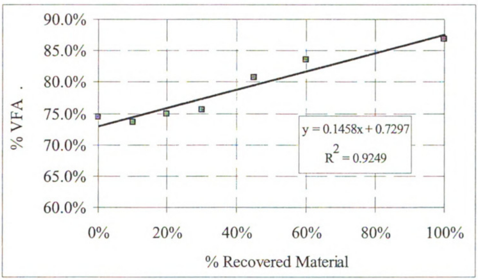


Figure 4.19 VFA versus percent recovered material for a wearing course recycled mixture with Margalla aggregates and Mandra recovered material (Superpave Mixture)

an optimal heating time which allows the recovered material's binder to fully and properly blend with the virgin binder. They recommended that the optimal heating time be determined in the laboratory and be extended to the asphalt mixing plant in lieu of the plant existing practices. It should be noted that, in this study, a preheating time of 2 hours was used.

Finally, as shown in Figure 4.19, it was found that increasing the percent recovered asphalt material contents cause increase in the VFA. For this particular mixture type, which was a wearing course made with Margalla aggregates and Mandra recovered material, the VFA increases from about 73% for the virgin mixtures to about 87% for mixtures made with 100% recovered asphalt materials. The increases in the VFA for all eight Superpave mixtures (see Figures E-9 through E-21 in Appendix E) can be directly related to two factors:

1. Decrease in the AV as shown in Figure 4.17.
2. Decreases in the binder absorption as the percent recovered asphalt material contents increases. Increases in the percent recovered asphalt material contents causes increases in the amount of the aggregate with absorbed asphalt. Less binder absorption implies more effective binder within the mixture and hence higher VFA.

It was also observed, that the Superpave mixtures MW-M, MW-N, DW-M and MB-N, which showed about a 4 percent AV at low recovered asphalt material contents, also showed a relatively constant VFA.

The recycled mixture's volumetric information were compared relative to the Superpave criteria. Said criteria state that:

1. The design percent air void must be at 4% whereas the actual mixture percent air void should be between 3.0 and 5.0%. All recycled asphalt mixtures having high percent of recycled asphalt material contents did not meet this air void criteria.
2. Minimum VMA values of 12 and 13 percent must be achieved for the base and wearing courses, respectively. All Superpave recycled mixtures satisfied the criteria except the MW-N and MB-N mixtures where the VMA were about 12 percent and 11.5%, respectively.
3. The VFA range must be between 65% and 75%. In general, the virgin and the recycled mixtures with 10% and 20%, and some of the 30% recovered asphalt material contents satisfied the criteria. All Superpave recycled asphalt mixtures having higher percentages of recovered asphalt material contents (45, 60, and 100%) did not meet the criteria.

The reason that recycled mixtures at high percent recovered asphalt material contents did not pass the Superpave criteria can be accredited to the increase in the effective binder content. The targeted binder content of the recycled mixtures was the optimum binder content of the virgin mixtures, where part of the binder is absorbed by the virgin aggregate. When the percent recovered asphalt material in the recycled mixture increases, the amount of absorbed binder in the recovered material increases and hence the effective percent binder content increases. The increases in the effective binder content causes increases in the VFA causing the recycled mixtures containing high percents of recovered asphalt material contents to fail the Superpave criteria. The above scenario indicates that the amount of binder absorbed in the recovered material must be accounted for and the binder content in the recycled mixture must be lowered. The new percent binder contents of the recycled mixtures were calculated using the SP-2 2001 An example of the estimated percent binder content for a wearing course Superpave mixture made with Deena aggregates and Mandra recovered material is shown in Figure 4.20. It should be noted that for some cases at high percentages of recovered material, the new estimated binder content could not be achieved. This was due to the fact that the binder content at high percentages of recovered material is higher thatn the targeted binder contents. Nevertheless, the volumetric properties of the recycled mixtures were re-calculated based on the new percent binder content data and 4% air voids. The results are listed in Tables 3.42 and 3.43. The new estimated volumetric information were compared to the Superpave criteria. It was found that:

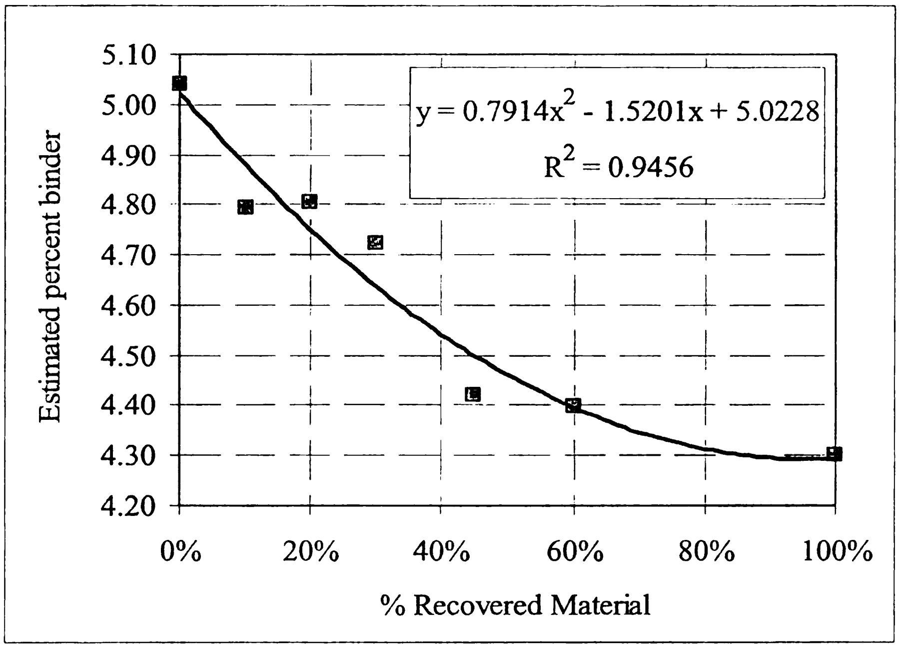


Figure 4.20 Estimated percent binder versus percent recovered asphalt material

- The recycled wearing and base mixtures, MW-N and MB-N, respectively, did not meet the VMA Superpave criteria.

- All recycled mixtures that passed the Superpave criterion for VMA also passed the Superpave criterion for VFA (65-75%). This included recycled mixtures at high percents of recovered asphalt material contents.

It is recommended that the optimum binder content of the virgin mixtures should not be used for recycled mixtures. The optimum binder content of the recycled mixtures should be determined as a function of the amount of recovered asphalt material within the mixture. The Superpave (SP-2) equations should be used as a starting point to determine the optimum binder content.

4.3.3.2 SPT Data Analysis

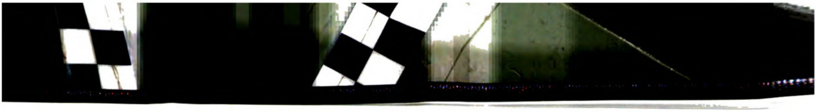
The simple performance test (SPT) data were analyzed to determine the effects of the percent recovered asphalt material contents on the dynamic modulus (E^*) and phase angle (δ). Since it was determined (see Section 4.3.2) that the binder from both sources of recovered material (Nowshera and Mandra) had aged substantially, it was expected that the mixtures dynamic stiffness (E^*) would increase as the percent of recovered material increased. It was also expected that the phase angle, would decrease as the percentage of recovered material increased. Bar charts were made to determine the effect of the percent recycled asphalt material content on E^* and δ for the same frequency and temperature. Figure 4.21 shows an example for a wearing course mixture containing Margalla virgin aggregates and Nowshera recovered material. All other Superpave recycled mixture's bar charts are shown in Figures E-22 through E-28 of Appendix E.

Increases in the percent recovered asphalt material contents effect the E^* and the δ of the recycled mixtures in many ways including:

- At the low temperature of 40°F, the SPT test results indicate that:

- For all eight mixture types, there is no correlation between increases in percent recovered asphalt material content on the E^* and δ. This means the E^* and the δ for all Superpave mixtures is somewhat similar.

- At the mid-range temperature of 70°F the SPT test results indicate that:
  - All eight mixture types behaved the same at low and high frequencies.
  - For mixture types MW-M (Figure E-22), MB-N (Figure E-25) and DB-N (Figure E-27) the dynamic modulus, E^*, stays relatively constant for all percentages of recycled mixtures except the 100 percent.
  - For mixture types MW-N (Figure 4.21), MB-M (Figure E-26) and DB-M (Figure E-28), the E^* decreases when the percent recovered asphalt material content increases from 0 to 20 percent, and then it increases consistently up to 100 percent recovered asphalt material content.
  - For mixture type DW-M (Figure E-24), the E^* consistently increases as the percent of recovered asphalt material content increases.
  - For mixture type DW-N (Figure E-23), no correlation was found.
  - In general, the percent recovered asphalt material content has a minor effect on the phase angle δ.



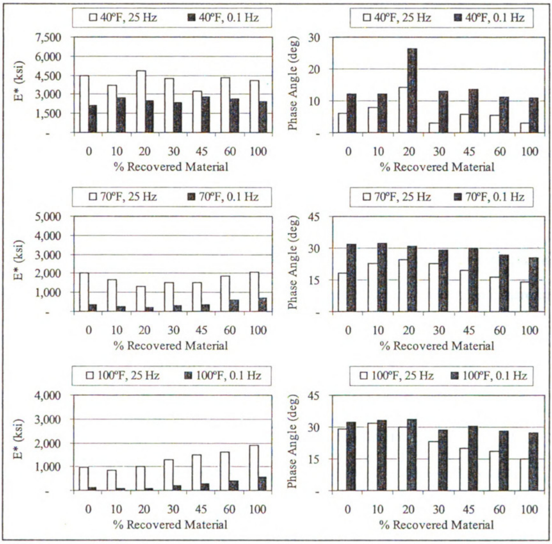


Figure 4.21 Variability of E^* and δ for the same temperature and frequency for wearing course Margalla + Nowshera (MW-N) blends

- At the high temperature of 100°F the SPT test results indicate that:
    - All eight mixtures showed similar trends at both low and high frequencies.
    - For mixture types DB-N (Figure E-27) and MB-N (Figure E-25), E^* increases with increasing percent recovered asphalt material contents.

- For mixture types DW-N and DW-M (Figures E-23 and 24), E^* increases as the percent recovered asphalt material contents increases and remains constant at the high percentages of recovered asphalt material contents.
- For mixture type MW-N (Figure 4.21), E^* initially decreases from 0 to 10 percent recovered asphalt material content and then increases from 10 to 100 percent .
- For mixture type MB-M (Figure E-26), E^* remains constant from 0 to 20 percent recovered asphalt material content, then it increases from 20 to 60 percent and decreases again from 60 to 100 percent.
- Generally, for both frequencies, δ decreases at high percents recovered asphalt material content except for mixture type DB-M (Figure E-28) which show a constant trend.

Further, the test results were analyzed to determine the effects of the virgin aggregate or recovered material sources on E^* and δ. For the wearing course and for all testing temperatures, the mixtures made with Margalla aggregates showed higher E^* and lower δ values than the Deena aggregates. For the base course, at low temperature testing, virgin mixtures made with Deena aggregates had higher E^* and lower δ than virgin mixtures made with Margalla aggregates. At the mid-range and high temperature, E^* and δ were relatively similar for both virgin mixtures. Figure 4.22 shows an example of the effect of aggregate sources on E^* at low temperatures.

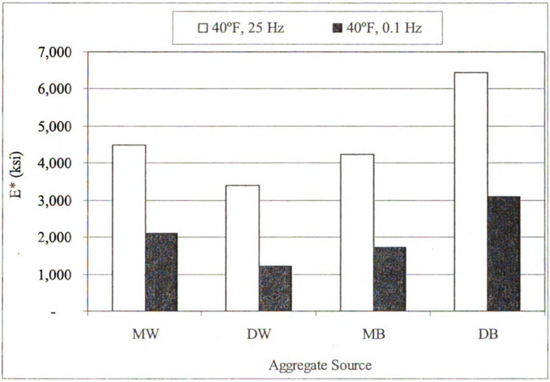


Figure 4.22 E^* versus percent recovered asphalt material for low temperatures

The difference between the SPT data of the two recovered HMA sources (Mandra and Nowshera) was further analyzed. At low and mid range temperatures E^* and δ showed relatively similar results for both Mandra and Nowshera. At high temperature testing, Nowshera mixtures for both wearing and base course showed higher E^* and lower δ values than those of Mandra mixtures. This agrees with similar findings in the blended binder analysis where 100% Nowshera binder was stiffer than that of Mandra.

For all eight recycled mixtures, the dynamic modulus master curves were developed using time-temperature superposition. The dynamic modulus master curve is typically used to accurately account for the effects of temperature and loading rate on the dynamic modulus. In the analysis:

1. A non-linear optimization procedure was used as recommended by Witczak and Sotil (Witczak & Sotil 2004) to develop the master curves.
2. All dynamic modulus data that were obtained from the SPT at the three temperatures of 40, 70, and 100°F and the five frequencies 25, 10, 5, 1, 0.5, and 0.1 Hz were shifted with respect to loading frequency at a specific reference temperature of 70°F (Bonaquist & Christensen 2005).
3. The shift factors, α(T), were determined using Equation 8:

$$\log \alpha(T) = aT^2 + bT + c \tag{8}$$

T = temperature of interest

a, b, and c = regression coefficients

4. The time of loading at the reference temperature, t_R, was determined using Equation 9.

$$t_R = \frac{t}{\alpha(T)} \tag{9}$$

t = time of loading at the given temperature of interest

5. The log time of loading at the reference temperature, log t_R, was determined using Equation 10.

$$\log t_R = \log(t) - \log \alpha(T) \tag{10}$$

6. The dynamic modulus (E*) was then predicted using the sigmoidal Equation 11.

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \times (\log t_R)}} \tag{11}$$

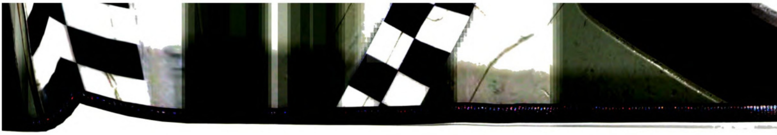
Where δ, α, β, γ = non-linear regression parameters of the sigmoidal function

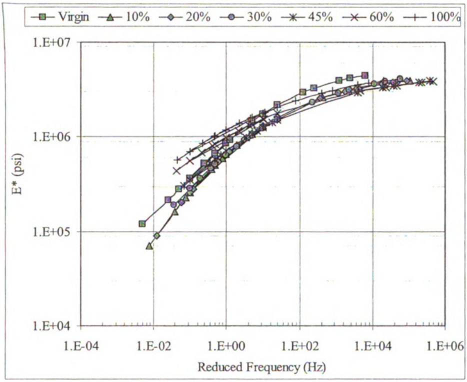
7. The 7 unknown parameters (a, b, c, α, β, γ, and δ) were calculated using the Excel solver function. This function used an iterative optimization process to get the minimum sum of errors squared between the actual dynamic modulus and the predicted dynamic modulus. Note that at the reference temperature the shift factor, log $\alpha(70)$, must be equal to 0.

With the minimum sum of error squared, the master curves for the optimum virgin mixture and recycled mixtures were plotted in the same graph. Figures 4.23 through 4.26 show the master curves for all recycled mixture types. The effect of the recovered asphalt material on the master curves was observed to be relatively the same for all Superpave mixture types.

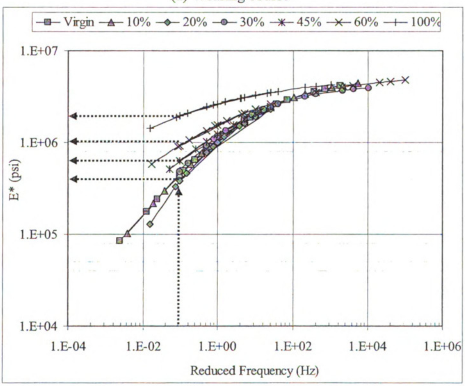
It was determined from the master curves that as the frequency decreased there was a substantial increase in the dynamic moduli at high percents of recovered asphalt material contents. For example Figure 4.23b at 0.1 Hz shows the recycled mixtures made with 45, 60 and 100 percent recovered asphalt material contents correspond to dynamic moduli values of 600,000, 1,000,000 and 2,000,000 psi, respectively. For the optimum virgin mixture and the recycled asphalt mixtures containing up to 30 percent recovered asphalt material, the dynamic moduli at low frequencies remained relatively the same at about 400,000 psi.

For all mixture types at low frequencies, the dynamic modulus of the recycled mixture with 100 percent recovered asphalt material content was always greater than that of the virgin mixtures and recycled mixtures with low percents of recovered asphalt material contents. For the majority of mixture types, the dynamic moduli at low frequencies for the virgin and recycled mixtures at low percents of recovered asphalt



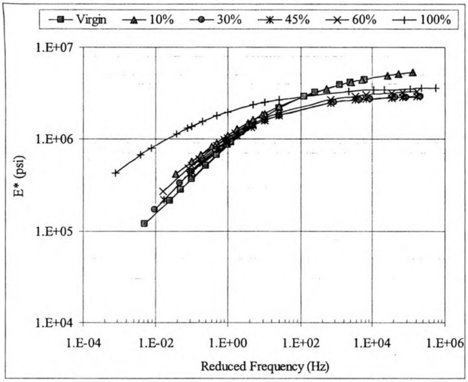


(a) Wearing course

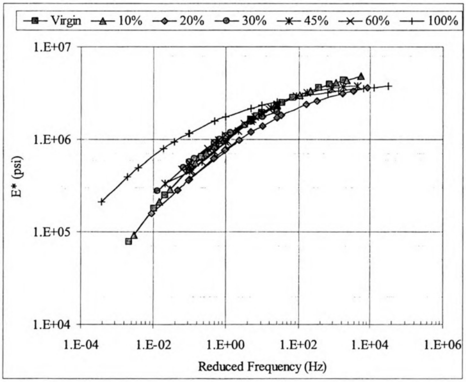


(b) Base course

Figure 4.23 Master curve for the Margalla + Nowshera recycled mixtures

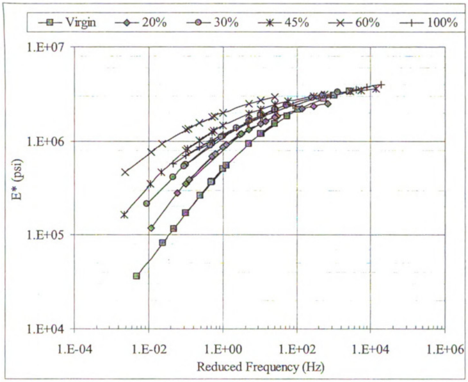


(a) Wearing course

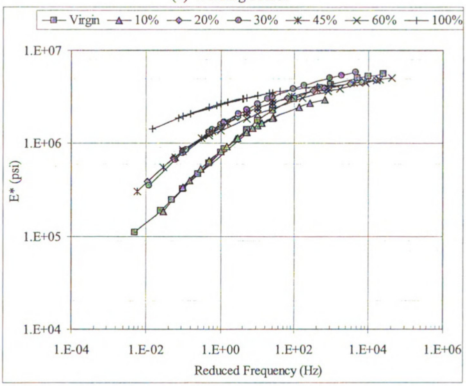


(b) Base course

Figure 4.24 Master curve for the Margalla + Mandra recycled mixtures

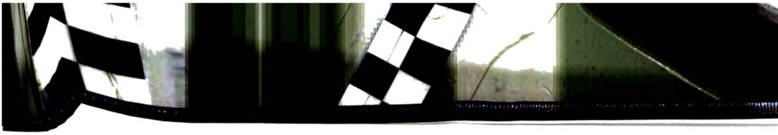


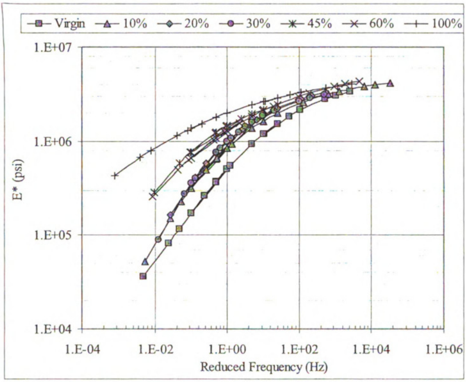
(a) Wearing course



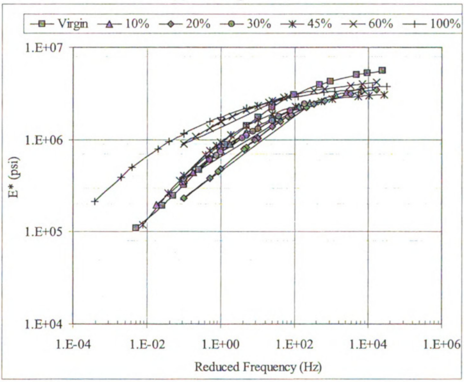
(b) Base course

Figure 4.25 Master curve for the Deena + Nowshera recycled mixtures





(a) Wearing course



(b) Base course

Figure 4.26 Master curve for the Deena + Mandra recycled mixtures

material contents were relatively the same. However, some mixture types show a similarity between virgin mixtures and recycled mixtures with intermediate percentages of recovered asphalt material contents, for example DB-M (Figure 4.25b), MW-M and MB-M (Figure 4.23a and b) mixture types.

The dynamic modulus at low frequencies represents the behavior of the asphalt concrete under a high loading time (slow or standing traffic) and/or the behavior at high temperatures (Daniel & Lachance 2005). Since the dynamic modulus at low frequencies represents the behavior at these conditions, it correlates to the asphalt concrete's ability to resist rutting. It was observed from Figures 4.23 through 4.26 that there is a greater resistance to rutting when greater percents of recovered asphalt material contents (45 percent and greater) are used in the recycled asphalt mixtures.

The dynamic moduli values for virgin and recycled mixtures at high frequencies were also analyzed. At high frequencies, the dynamic moduli values within a mixture type begin to slowly converge to a maximum value. The dynamic modulus at high frequencies corresponds to the behavior at a low loading time (high traffic speed) and/or the behavior at low temperatures. These trends were expected because when asphalt mixtures are at very low temperatures, regardless of the age of the asphalt binder, the behavior is expected to be similar.

The results of a similar study conducted by Daniel and Lachance (Daniel & Lachance 2005) were compared to the results from this study. Their findings contradict the results obtained in this study. For example, they reported that at low frequencies virgin and recycled mixtures (which were made up to 40% recovered material) behaved similarly, while at higher frequencies, the recycled mixtures had a greater dynamic modulus than the virgin mixture. The difference in the dynamic modulus at high frequencies was about 200,000 psi which is relatively low compared to the dynamic moduli values obtained in this study.

On the other hand, the results reported by Al Qadi et al. (Al Qadi et al. 2009) correspond very well to the master curves developed in this study. Al Qadi et al. master curves indicated that at high frequencies, the dynamic modulus values for virgin and recycled mixtures containing 20% and 40% recovered asphalt material do not show any significant difference. However at low frequencies there is a significant increase in the dynamic moduli values for the recycled mixtures versus virgin mixtures.

4.4 GUIDELINES FOR THE SELECTION OF ASPHALT PAVEMENT RECYCLING METHODS

At the outset, these guidelines are general in nature and will be detailed when all test results become available. Further, it is assumed that based on general evaluation (pavement surface distress, ride quality, rut depth and structural integrity) of a pavement project, the project is selected for rehabilitation. Hence, recycling is one of several rehabilitation options that need to be evaluated. The guidelines are presented in a step-wise manner.

1. Determine the structural capacity of all pavement layers using NDT or destructive testing. If the test results showed weak lower pavement layers, the asphalt layer should be evaluated for possible in-plant recycling. No in-place recycling should be undertaken. If the lower layers showed good structural capacity then the asphalt layer could be subjected to in-plant or in-place recycling depending on the results of the other evaluations.

2. Obtain pavement core to:

   a. Determine the thickness of the asphalt layer, the information can be used for backcalculating the layer moduli in step 1 and for determining whether or not in-place recycling can be employed. If the asphalt thickness is more than 6 inches no in-place recycling should be employed.

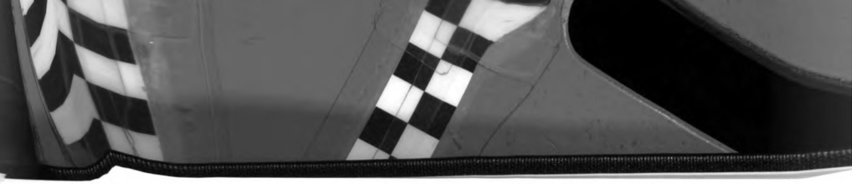
   b. If only a portion of the asphalt layer needs to be recycled then determine the thickness of that portion based on its condition. Two inches or less could be subjected to hot or cold in-place recycling. More than two but less than six inches, it could be subjected to cold in-place recycling.

   c. Regardless of the thickness of the asphalt layer, the layer could be subjected to a combination of milling and in-place recycling provided that the recycled portion is two inches or less of hot in-place and less than six inches for cold in-place recycling.

   d. Regardless of the asphalt layer thickness, the layer could be removed and subjected to in-plant recycling.

3. Examine the core and measure the depth of cracks. If the crack depths are less than two inches, hot in-place recycling is possible. If the crack depths are up to six inches, cold in-place recycling is possible. In-plant recycling can be employed for all crack depths.

4. Obtain samples from the asphalt layer to be subjected to asphalt recycling. Transport the materials to the laboratory and conduct the proper standard tests to determine the recovered aggregate gradation.



5. Blend the recovered binder with virgin binder using different proportions. For each proportion determine the high and low critical temperatures of the blended binder. The data will assist the selection of the proper percent recovered binder to be included in the recycling.
6. Make recycled mixtures using different proportions of recovered and virgin asphalt materials. Determine the volumetric properties of the recycled mixtures.
7. Based on the volumetric properties of the recycled mixtures and on the high and low critical temperatures of the blended binders, select the proper proportion of the recovered asphalt material to be included in the recycled mixtures. Based on this proportion, one could do:
   a. In-place recycling if the percent recovered asphalt material to be included in the recycled mixture is high (>85%).
   b. In-plant recycling if the proportion is less than 85%.
   c. No recycling if the proportion is less than 5%. In this case, the recovered asphalt material could be shattered or fractured and used as unbound base material.
8. Select the required asphalt binder performance grade (PG) to be used under the given environmental condition. Select the proper proportions of the recycled asphalt binder that would yield the required PG using the results from step 5.

9. Do trial recycled asphalt mixtures in the laboratory and determine their volumetric properties. Test whether or not these properties satisfy the requirements of the Superpave mixes.
10. Conduct life cycle cost analysis of all pavement rehabilitation alternatives and all asphalt recycling methods that can be employed for the pavement project in question. Select the most cost effective alternative.

The above stepwise guideline on selecting the proper recycling method was summarized in a flow chart format presented in Figure 4.27.

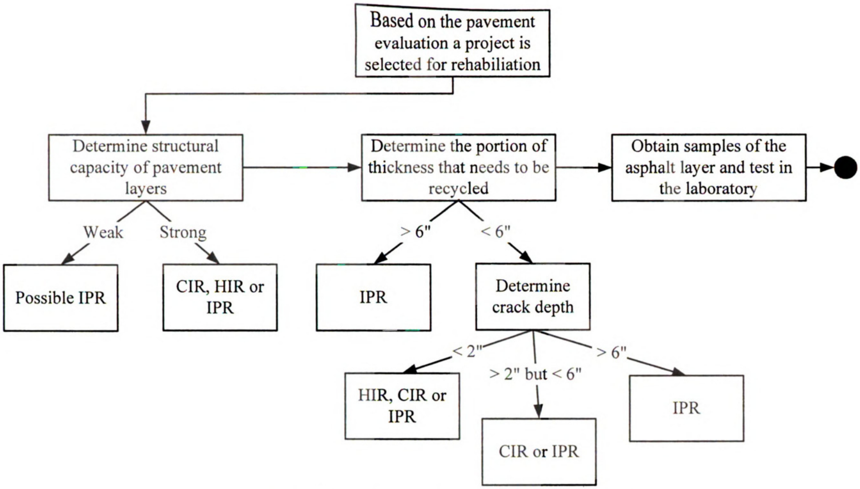


Figure 4.27 Guidelines for selecting appropriate recycling method

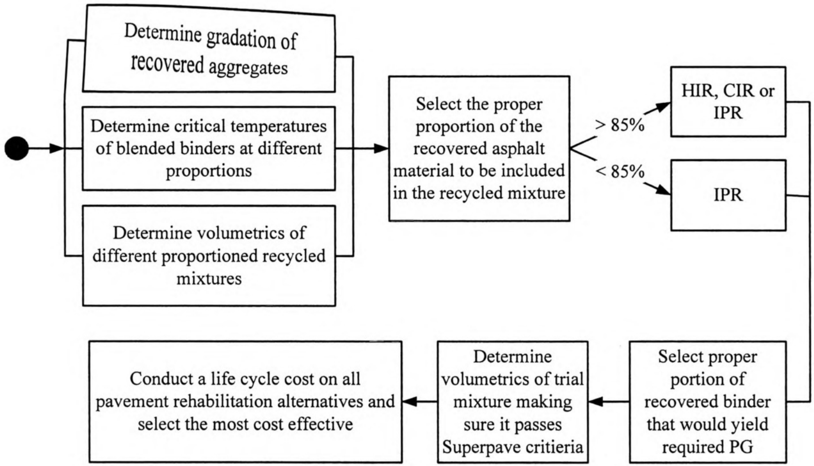


Figure 4.27 (continued)

4.5 CANDIDATE RECYCLING METHODS

Based on the field investigation, and the above guidelines, candidate recycling methods were selected for each AC test site. Since there wasn't laboratory information available for the AC test sites the candidate recycling methods were selected based on the pavements surface condition, crack initiation/propagation, quality of the lower pavement layers and the moduli of the pavement layers. Table 4.6 provides a summary of the pavement condition and recommended recycling methods for the twelve test sites.

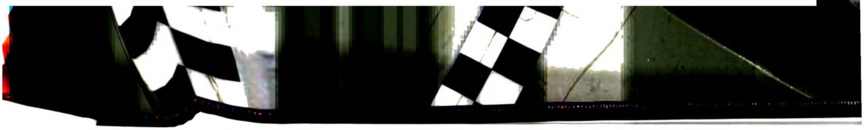


Table 4.6 Candidate recycling methods based on pavement evaluation

Site (location)	AC thickness	Pavement Condition	Traffic	Recycling Method
N5N-1 (Nowshera)	4”	- Severe block cracking and occasional patches. - Partial top-down cracking which has propagated 2.5 inches. -Lower pavement layers have adequate strength.	H	1. CIR 2.5” and cap with 2” fresh mix 2. IPR
N5N-2 (Nowshera)	4.5”	-Severe block cracking and moderate patches and rutting. - Partial top-down cracking which has propagated 2.5 inches. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1.CIR 2.5” and cap with 2” fresh mix 2.IPR
RRW-3 (Peshawar)	5”	-Moderate rutting and longitudinal cracking -Partial top-down cracking which has propagated 2.5 inches. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1.CIR 2.5” and cap with 2” fresh mix 2. HIR 2.5” and cap with 2” fresh mix 3.IPR
N5S-4 (Mandra)	6.5	-Frequent fatigue cracking and severe block cracking, moderate rutting. -Partial top-down cracking which has propagated 3 inches. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1.CIR 3” and overlay with 2” fresh mix 2. Combo of CIR + IPR
N5N-5 (Mandra)	6”	-Moderate patches and rutting. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1. CIR 2” and overlay with 2” fresh mix 2. HIR 2” and overlay with 2” fresh mix 3. IPR
N5S-6 (Wazirabad)	8”	-Severe fatigue cracking and moderate transverse and block cracking and rutting -Partial top-down cracking which has propagated 3 to 5 inches. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1.Top 3” milled and the bottom 5” subjected to CIR. Replace the top 3” with IPR

Table 4.6 (continued)

Site (location)	AC thickness	Pavement Condition	Traffic	Recycling Method
N5N-7 (Sukkar)	6"	-Severe rutting. -Seat of rutting in the base/subbase layer -Lower pavement layers have adequate strength from backcalculation	H	1. CIR 2" and overlay with 2" fresh mix 2. HIR 2" and overlay with 2" fresh mix 3. IPR
N5N-8 (Sukkar)	7.5"	-Moderate longitudinal cracking and rutting. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	H	1. CIR 2" and overlay with 2" fresh mix 2. HIR 2" and overlay with 2" fresh mix 3. IPR
MNN-9 (Murree)	4.5"	-Moderate fatigue cracking. -Lower pavement layers have adequate strength.	L	1. CIR 2" and overlay with 2" fresh mix 2. IPR
MJS-10 (Murree)	8"	-Frequent fatigue cracking and moderate longitudinal cracking. -Lower pavement layers have adequate strength.	L	1. CIR 2" and overlay with 2" fresh mix 2. IPR
MLN-11 (Murree)	5"	-Moderate potholes and occasional cracking. -Lower pavement layers have adequate strength.	L	1. CIR 2" and overlay with 2" fresh mix 3. IPR
M2S-12 (Chakri)	5"	-Severe rutting. -Seat of rutting in the HMA layer -Lower pavement layers have adequate strength.	M	1. CIR 2" and overlay with 2" fresh mix 2. HIR 2" and overlay with 2" fresh mix 3. IPR

Note: For traffic: H > 2million ESALs/year, M = 1 million ESALs/year, L = 0.25 million ESALs/year
CIR = cold in-place recycling, HIR = hot in-place recycling, IPR = in-plant recycling

The reasons from the recommendations for each test site are enumerated below:

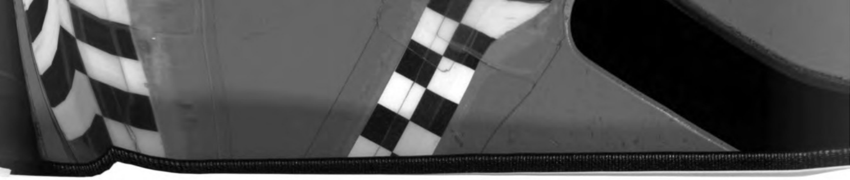
- For test site 1, N5N-1, the following data were available:
    - Distress data showed presence of severe block cracks and occasional patches.
    - Core data indicated that most of these cracks are top-down cracks that have penetrated 2.5 inches in depth.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place asphalt recycling of the top 2.5 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative full depth recycling of the asphalt layer can take place in-plant.

- For test site 2, N5N-2, the following data were available:
    - Distress data showed presence of severe block cracks and moderate patching and rutting.
    - Core data indicated that most of these cracks are top-down cracks that have penetrated 2.5 inches in depth.
    - Horizontal profile data indicate the seat of rutting is located in the HMA layer.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place asphalt recycling of the top 2.5 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative full depth recycling of the asphalt layer can take place in-plant.

- For test site 3, RRW-3, the following data were available:
    - Distress data showed presence of moderate rutting and longitudinal cracking.
    - Core data indicated that most of these cracks are top-down cracks that have penetrated 2.5 inches in depth.

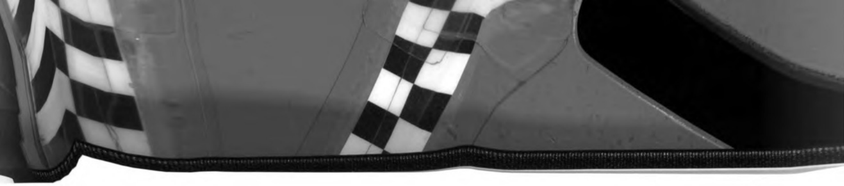


- Horizontal profile data indicate the seat of rutting is located in the HMA layer.
- Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place or hot in-place asphalt recycling of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative full depth recycling of the asphalt layer can take place in-plant.

- For test site 4, N5S-4, the following data were available:
  - Distress data showed presence of severe fatigue and block cracks and moderate rutting.
  - Core data indicated that most of these cracks are top-down cracks that have penetrated up to 3 inches in depth.
  - Horizontal profile data indicate the seat of rutting is located in the HMA layer.
  - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place asphalt recycling of the top 3 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, 3 inches can be milled away and

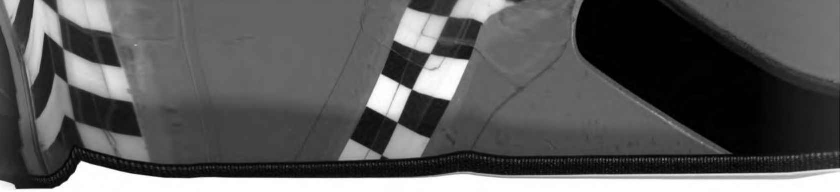


the bottom 2.5 inches of the asphalt layer can be subjected to cold in-place recycling. The top 3 inches can be replaced by in-plant recycling.

- For test site 5, N5N-5, the following data were available:
    - Distress data showed presence of moderate patches and rutting.
    - Horizontal profile data indicate the seat of rutting is located in the HMA layer.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

  Based on the above information, cold in-place or hot in-place asphalt recycling of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, 4 inches can be milled away and re-placed with in-plant recycling.

- For test site 6, N5S-6, the following data were available:
    - Distress data showed presence of severe fatigue cracks and moderate transverse and block cracks and rutting.
    - Core data indicated that most of these cracks are top-down and cracks have penetrated 3 to 5 inches in depth.
    - Horizontal profile data indicate the seat of rutting is located in the HMA layer.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have relatively adequate layer moduli.

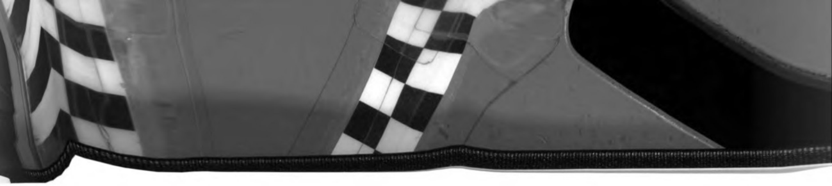


Based on the above information, this section should be subjected to milling of the top 3 inches of the existing asphalt layer. The remaining 5 inches of the asphalt layer should be subjected to cold in-place recycling. The top 3 inches will be replaced by in-plant recycling from the millings.

- For test site 7, N5N-7, the following data were available:
    - Distress data showed presence of severe rutting.
    - Horizontal profile data indicate the seat of rutting is located in the base/subbase layer.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information it was determined that the base and subbase layers are adequate to support recycling from the backcalculated layer moduli. Cold in-place or hot in-place asphalt recycling of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, 4 inches can be milled away and re-placed with in-plant recycling.

- For test site 8, N5N-8, the following data were available:
    - Distress data showed presence of moderate longitudinal cracks and rutting.
    - Horizontal profile data indicate the seat of rutting is located in the HMA layer.



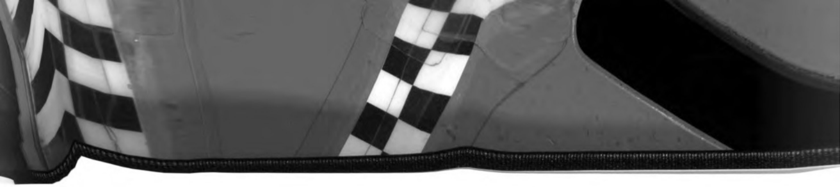
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

  Based on the above information, cold in-place or hot in-place asphalt recycling of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, 4 inches can be milled away and re-placed with in-plant recycling.

- For test site 9, MNN-9, the following data were available:
    - Distress data showed presence of moderate fatigue cracks.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

  Based on the above information, cold in-place of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, the entire asphalt layer (4.5 inches) can be milled away and re-placed with in-plant recycling.

- For test site 10, MJS-10, the following data were available:
    - Distress data showed presence of moderate longitudinal cracks and frequent fatigue cracks.
    - Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.



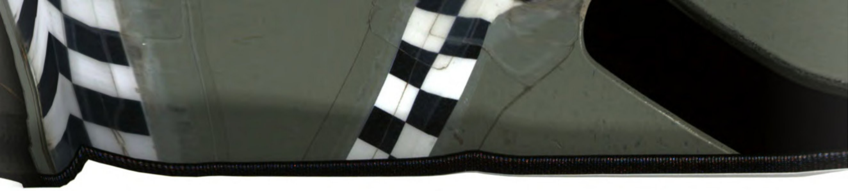
Based on the above information, cold in-place of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, 4 inches can be milled away and re-placed with in-plant recycling.

- For test site 11, MLN-11, the following data were available:
    - o Distress data showed presence of moderate potholes and occasional fatigue, transverse and longitudinal cracks.
    - o Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin mixture is recommended. As an alternative, the entire asphalt layer (5 inches) can be milled away and re-placed with in-plant recycling.

- For test site 12, M2S-12, the following data were available:
    - o Distress data showed presence of severe rutting.
    - o Horizontal profile data indicate the seat of rutting is located in the HMA layer.
    - o Deflection testing and subsequent analysis have indicated that the lower pavement layers (subbase/base and roadbed soil) have adequate residual strengths.

Based on the above information, cold in-place or hot in-place of the top 2 inches (partial asphalt thickness) followed by a 2 inch capping using virgin



mixture is recommended. As an alternative, the entire asphalt layer (5 inches) can be milled away and re-placed with in-plant recycling.

These candidate recycling methods are only suggestions and it is ultimately up to the National Highway Authority in Pakistan to decide on rehabilitation and recycling. Also, other than basing this decision purely off technicality, the availability of technology, field experience and cost should have considerable weight in the final decision.

CHAPTER 5

SUMMARY, CONCLUSIONS & RECOMMEDATIONS

5.1 SUMMARY

The socioeconomic growth of a country depends on the health of its transportation network. To maintain the highway portion of the network, the needs for fixing pavements has to compete with other demands on the limited public funds. Asphalt pavement recycling is one way to stretch existing resources and if adopted properly, it can precipitate in substantial savings in cost, energy and natural resources. However until recently, the Highway Authority in Pakistan had very little knowledge on the proper selection of asphalt recycling methods and the characteristics of the recycled asphalt mixtures. This research addressed those issues by evaluating several pavements test sections in Pakistan and tested recycled asphalt materials.

In this study, methodologies and procedures for the adoption of asphalt pavement recycling in Pakistan were developed based on field and laboratory investigations. Twelve asphalt concrete (AC) pavement test sites were selected and each test site was subjected to the detailed field investigation stated below. In addition, one source of virgin binder, two sources of recovered material, and two sources of virgin aggregates were identified and sampled. The samples were subjected to the lab investigation stated below.

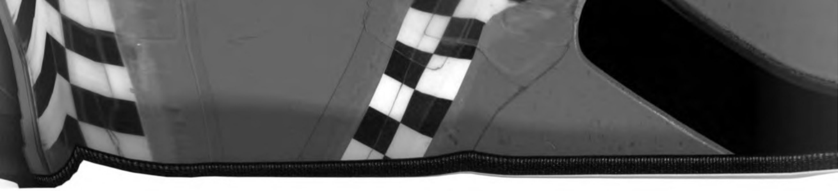
The field investigation consisted of: distress surveys, material sampling, longitudinal and transverse profiles, and nondestructive deflection testing (NDT). The laboratory investigation consisted of characterizing the properties of virgin, blended and recovered materials. Based on the field and laboratory investigations, guidelines for hot in-place (HIR), cold in-place (CIR) and in-plant (IPR) recycling were developed. The

guidelines were implemented to determine the recommended recycling methods for each of the twelve test sites.

5.2 CONCLUSIONS

Based on the field and laboratory investigations and the results of the data analyses, the following conclusions were drawn:

- Proper pavement evaluation is needed to asses the structural (cracking) and functional (IRI) condition of the existing pavements.
- The seat of rutting could be determined using the backcalculated layer moduli from the deflection data and the transverse profiles using the appropriate NCHRP procedure.
- The strength of the lower pavement layers of all twelve test sites is adequate to support recycling.
- Cold in-place and in-plant recycling could be used on all twelve test sites.
- In-place and in-plant recycling requires a full knowledge of the characteristics of virgin, blended and recovered materials.
- The performance grade (PG) of the binder should be selected based on traffic and environmental conditions.
- The McDaniel & Anderson 2001 equation stated in the NCHRP report accurately interprets the effect of increases of the percent recovered binder on the critical high and low temperatures.
- Recycled mixtures designed using the Marshall procedure produce inconsistent volumetric data, whereas Superpave designed mixtures produce consistent data.

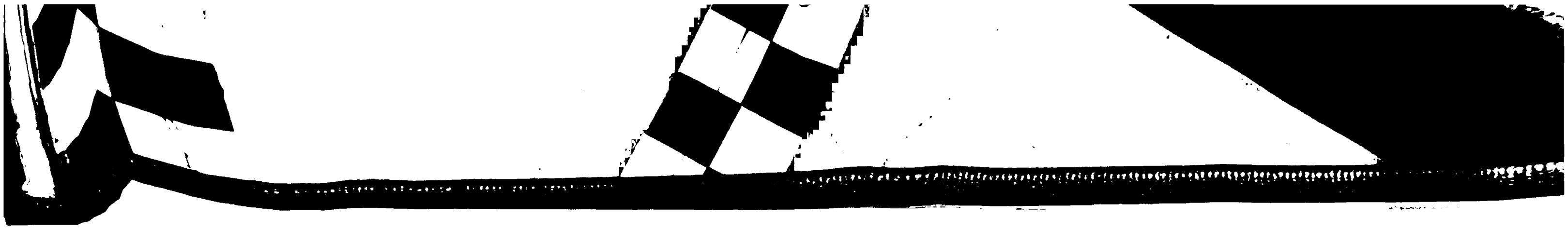


- The percent absorbed binder in the recovered aggregates needs to be accounted for in recycled asphalt mixture design.
- In general, for slow traffic, recycled mixtures containing 45 percent or more recovered materials have higher rutting resistance but they fail the Superpave criteria. Whereas, for fast moving traffic, all recycled mixtures perform similar to the virgin mixture.

5.3 RECOMMENDATIONS

Based on the results of the analysis and the above stated conclusions, the following recommendations are made:

- Superpave testing protocols (SP-2 2001) should be adopted for testing all pavement materials.
- The optimum binder content of the recycled mixtures should be determined based on a four percent air void and the amount of absorbed binder in the recovered aggregates.
- Virgin and recycled asphalt mixtures should be designed using the Superpave mix design method.
- A capping layer should be used on all in-place recycled pavements.



APPENDIX A

Distress map, distress pictures and core pictures

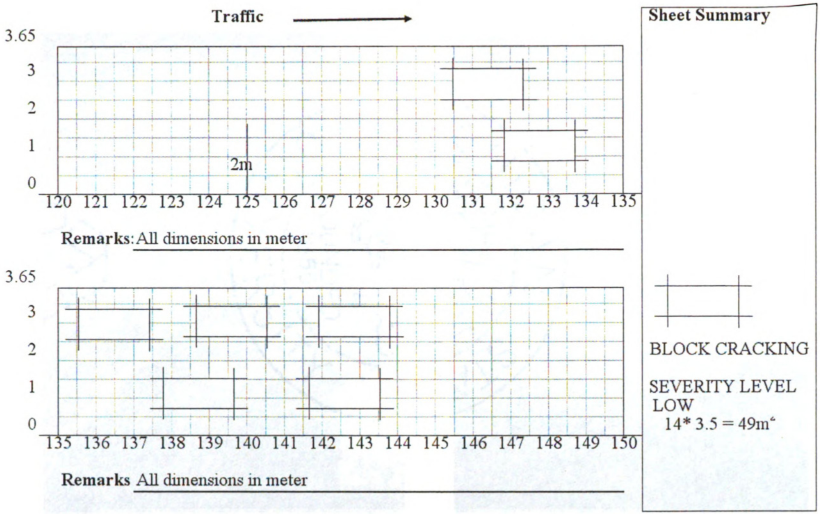
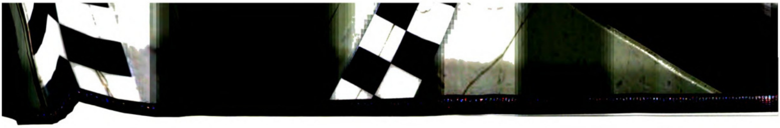


Figure A-1 Distress Map of Site #1 (N5N-1), Section 1 from 0 – 30 m



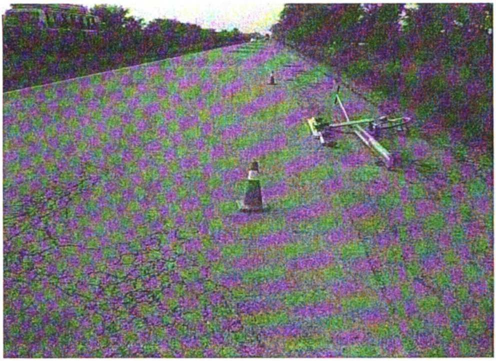


Figure A-2 Alligator Cracking on Site #5 (N5N-5)

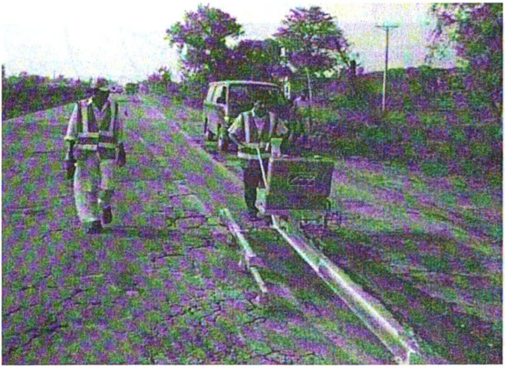
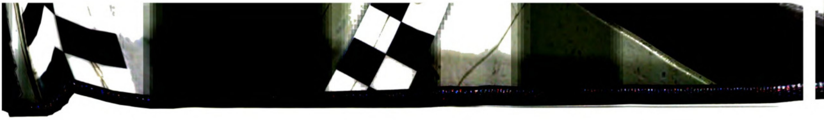


Figure A-3 Alligator Cracking on Site #6 (N5S-6)



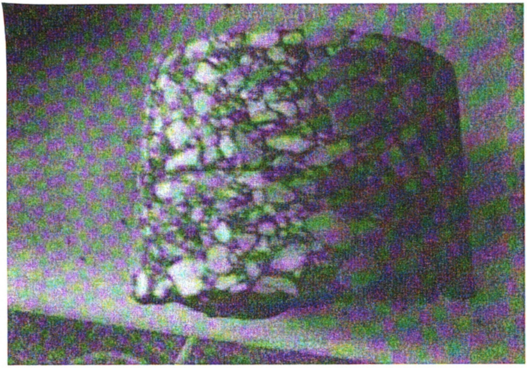


Figure A-4 Core Taken from N5S-4 Section 2 LWP (Top Down Crack)

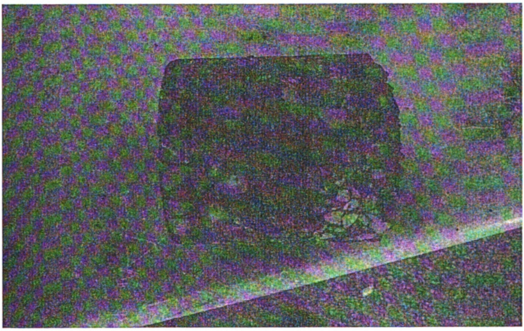
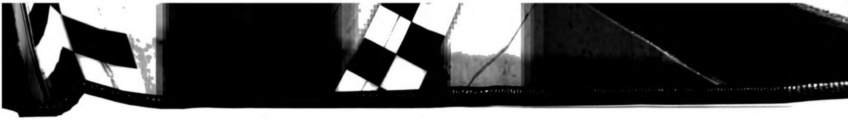


Figure A-5 Core of Site N5S-6 (Wazirabad)



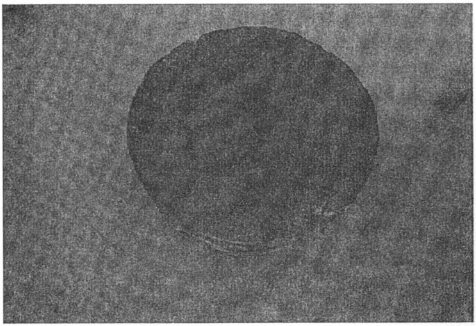
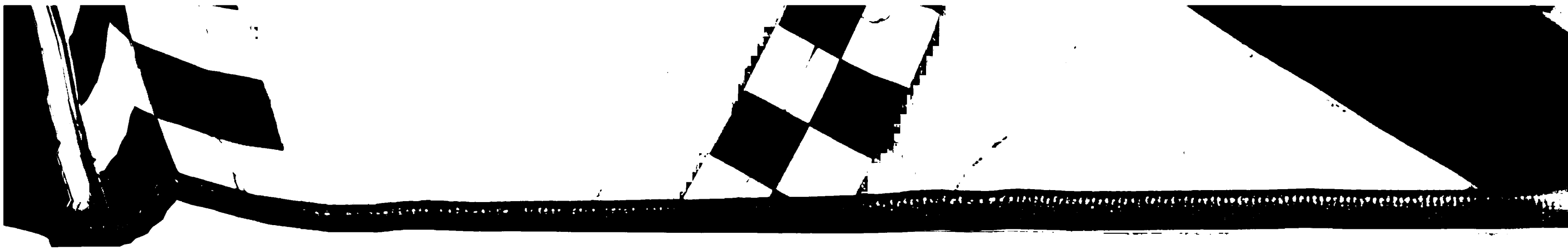


Figure A-6 Core of Site N5S-6 Section 1 LWP



APPENDIX B

Horizontal profiles

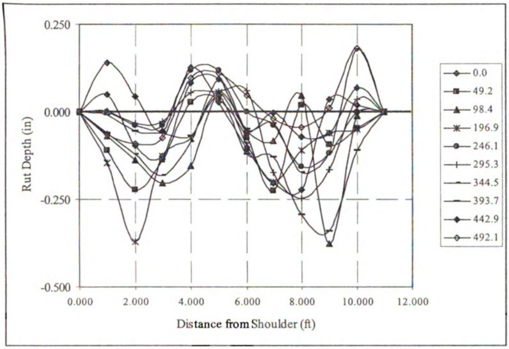


Figure B-1 Horizontal profiles from 0 – 492.1 ft along test site 1, section 1

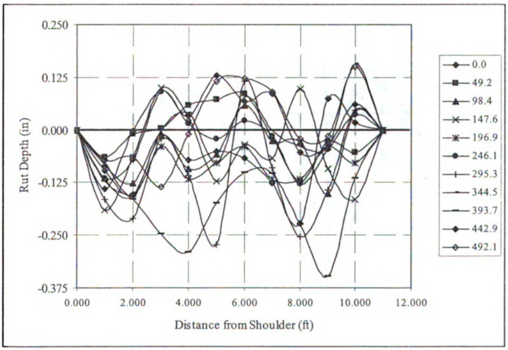
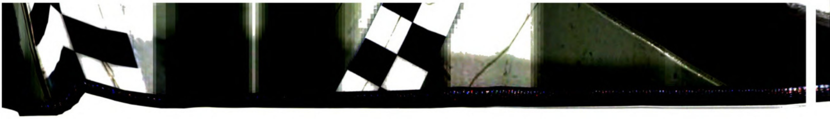


Figure B-2 Horizontal profiles from 0 – 492.1 ft along test site 1, section 2



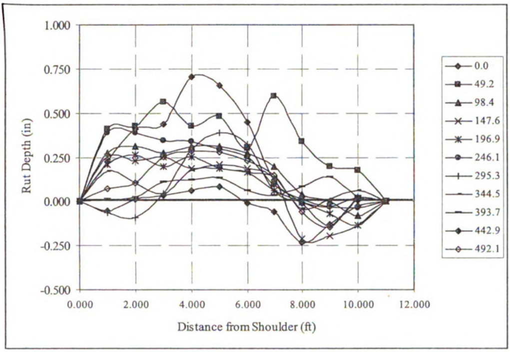


Figure B-3 Horizontal profiles from 0 – 492.1 ft along test site 1, section 3

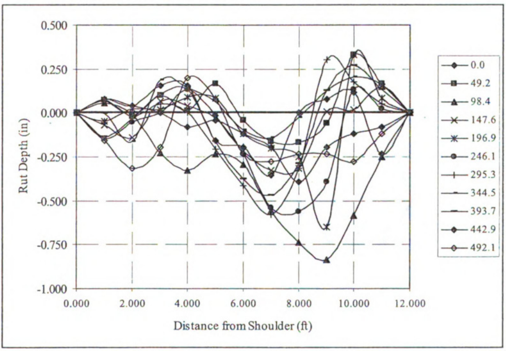


Figure B-4 Horizontal profiles from 0 – 492.1 ft along test site 2, section 1

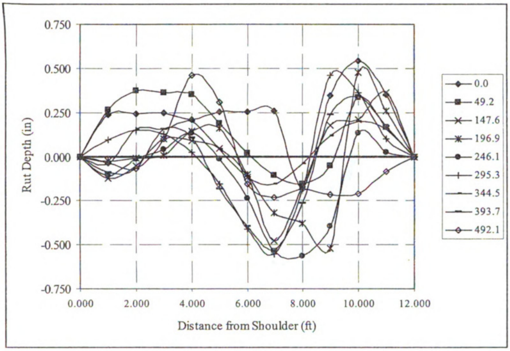


Figure B-5 Horizontal profiles from 0 – 492.1 ft along test site 2, section 2

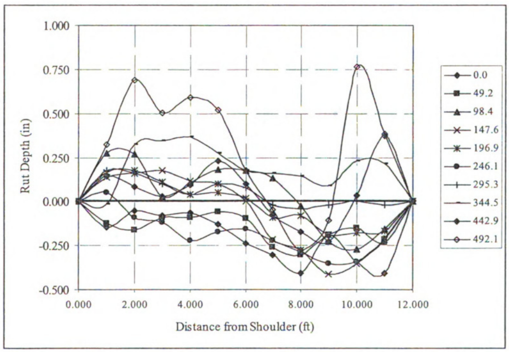
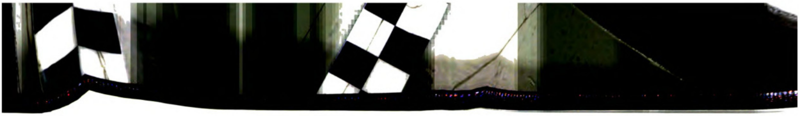


Figure B-6 Horizontal profiles from 0 – 492.1 ft along test site 2, section 3



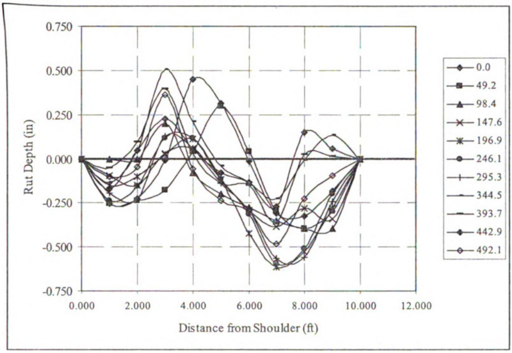


Figure B-7 Horizontal profiles from 0 – 492.1 ft along test site 3, section 1

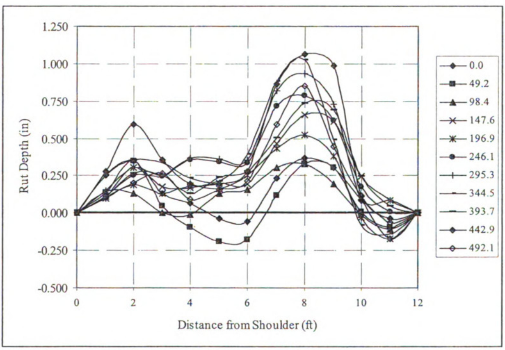


Figure B-8 Horizontal profiles from 0 – 492.1 ft along test site 4, section 1

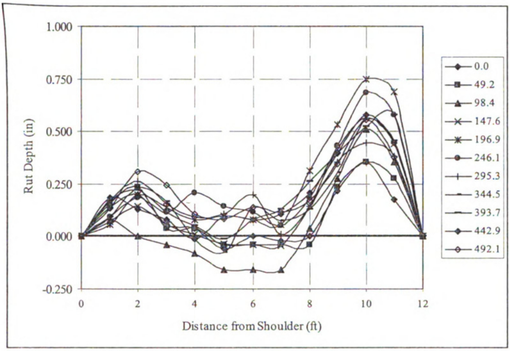


Figure B-9 Horizontal profiles from 0 – 492.1 ft along test site 4, section 2

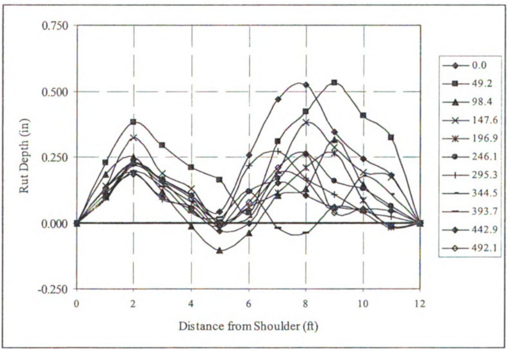
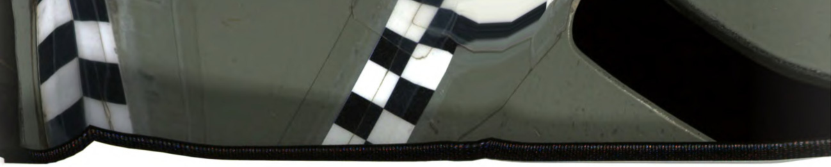


Figure B-10 Horizontal profiles from 0 – 492.1 ft along test site 4, section 3



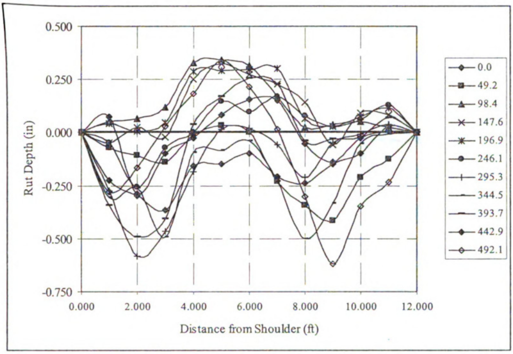


Figure B-11 Horizontal profiles from 0 – 492.1 ft along test site 5, section 1

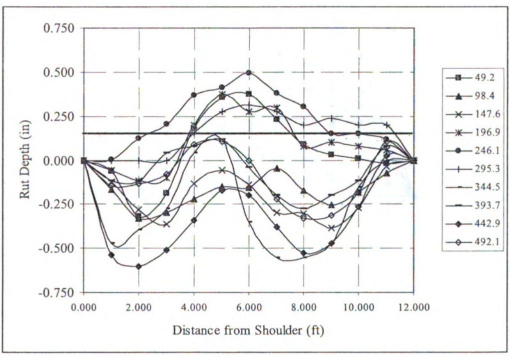
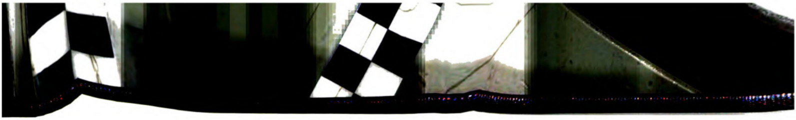


Figure B-12 Horizontal profiles from 0 – 492.1 ft along test site 5, section 2



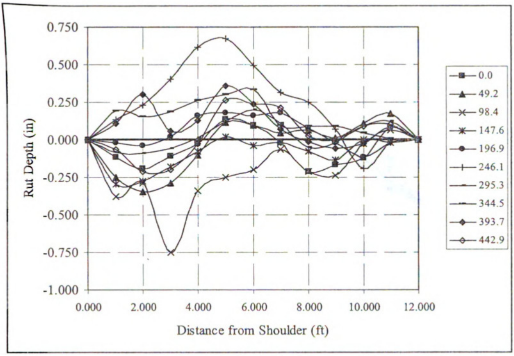


Figure B-13 Horizontal profiles from 0 – 492.1 ft along test site 5, section 3

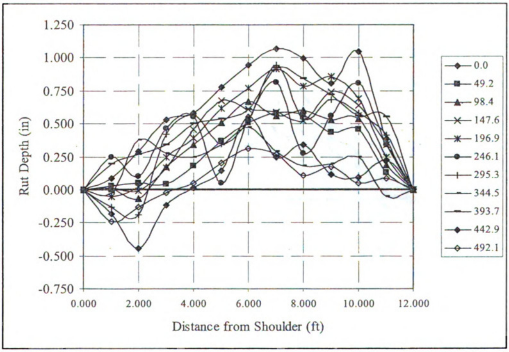
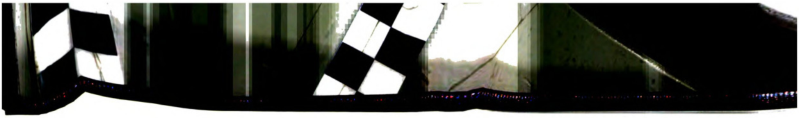


Figure B-14 Horizontal profiles from 0 – 492.1 ft along test site 6, section 1



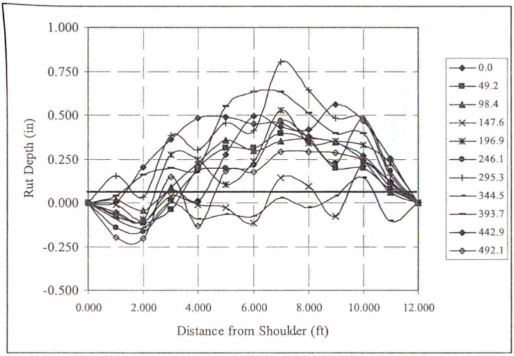


Figure B-15 Horizontal profiles from 0 – 492.1 ft along test site 6, section 2

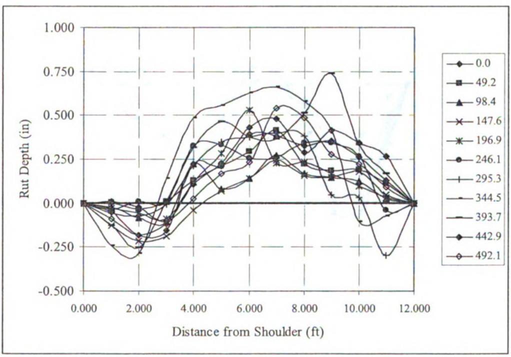


Figure B-16 Horizontal profiles from 0 – 492.1 ft along test site 6, section 3

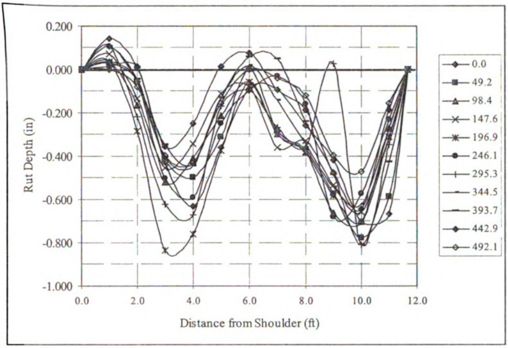


Figure B-17 Horizontal profiles from 0 – 492.1 ft along test site 7, section 1

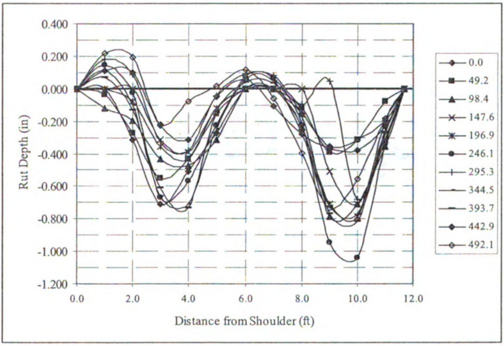


Figure B-18 Horizontal profiles from 0 – 492.1 ft along test site 7, section 2

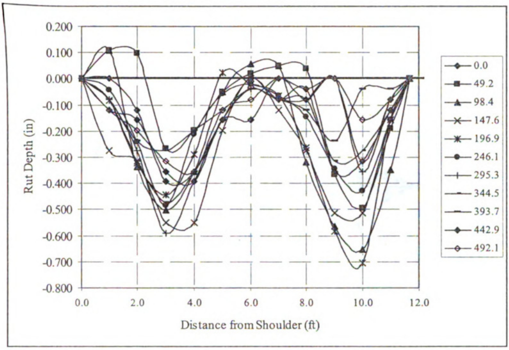


Figure B-19 Horizontal profiles from 0 – 492.1 ft along test site 7, section 3

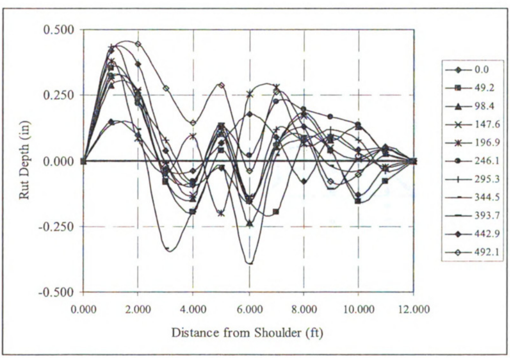
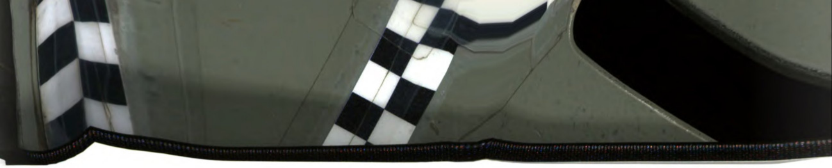


Figure B-20 Horizontal profiles from 0 – 492.1 ft along test site 8, section 1



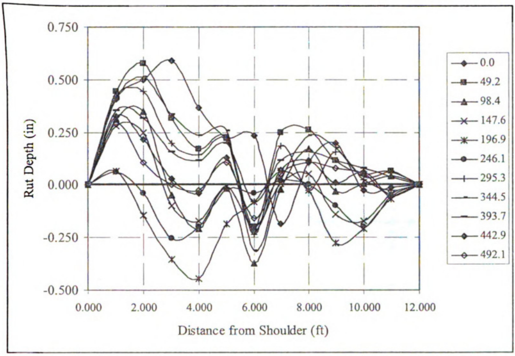


Figure B-21 Horizontal profiles from 0 – 492.1 ft along test site 8, section 2

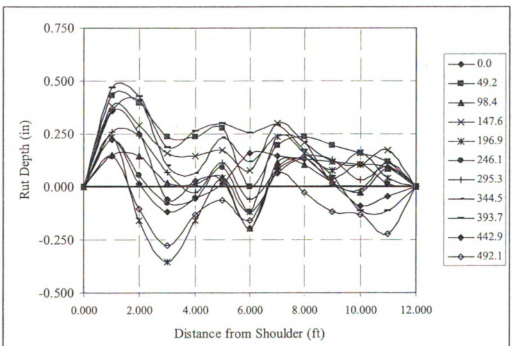
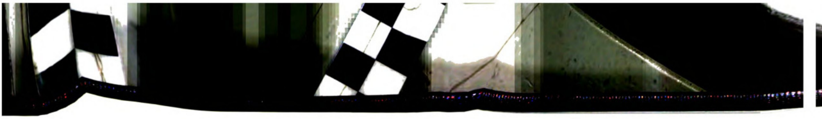


Figure B-22 Horizontal profiles from 0 – 492.1 ft along test site 8, section 3



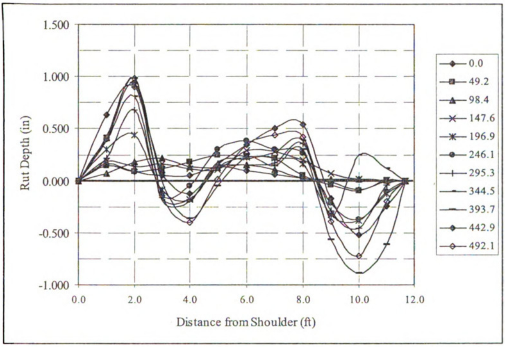


Figure B-23 Horizontal profiles from 0 – 492.1 ft along test site 12, section 1

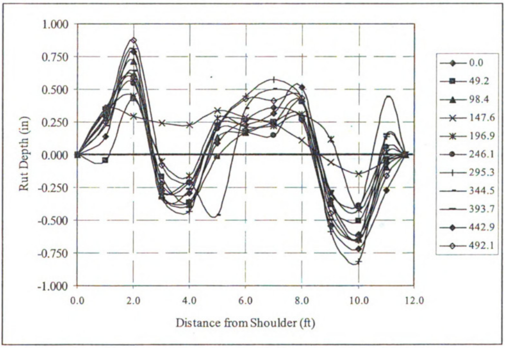
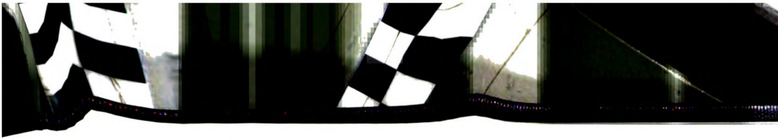


Figure B-24 Horizontal profiles from 0 – 492.1 ft along test site 12, section 2



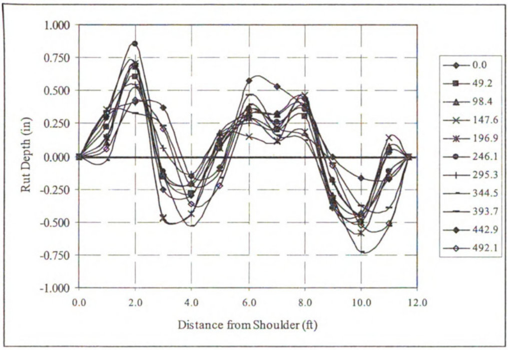


Figure B-25 Horizontal profiles from 0 – 492.1 ft along test site 12, section 3

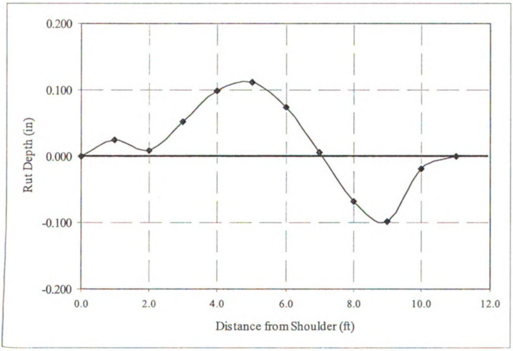


Figure B-26 Average profile of test site 1

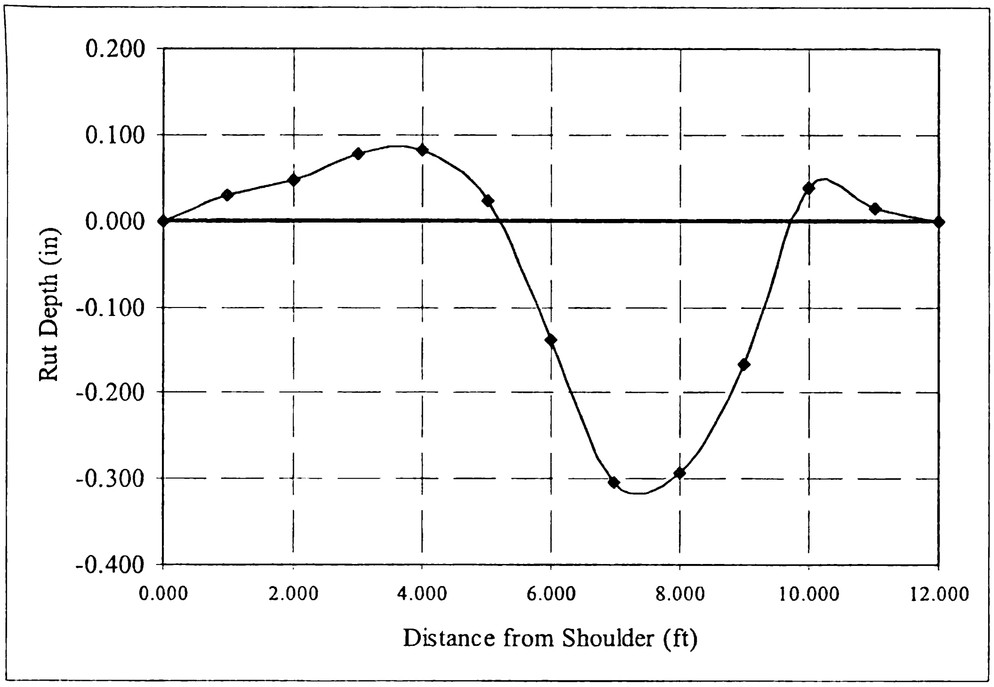


Figure B-27 Average profile of test site 2

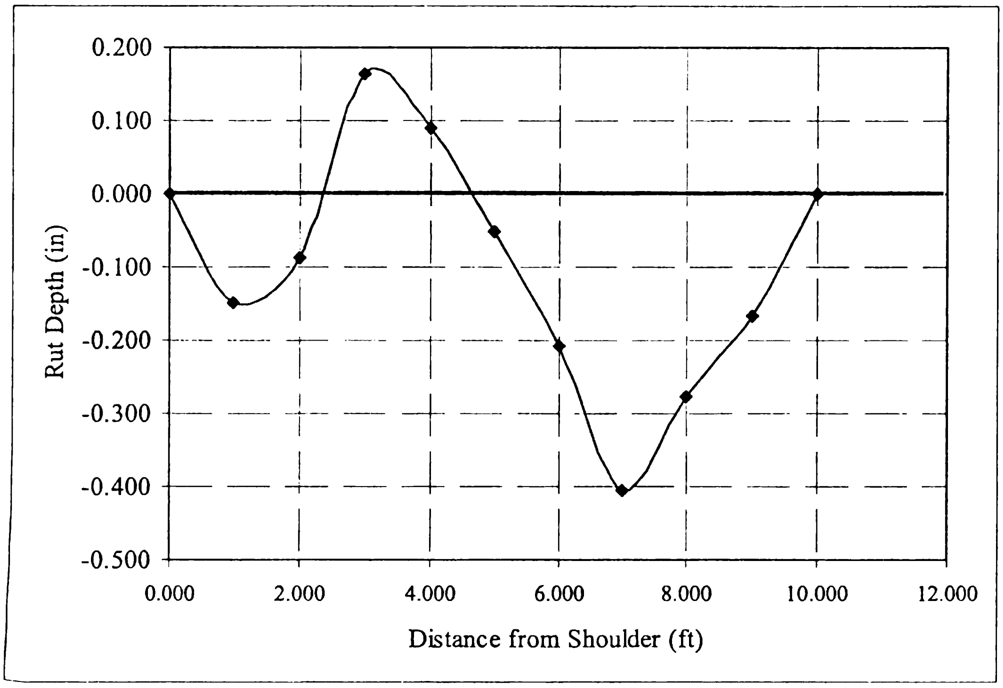


Figure B-28 Average profile of test site 3

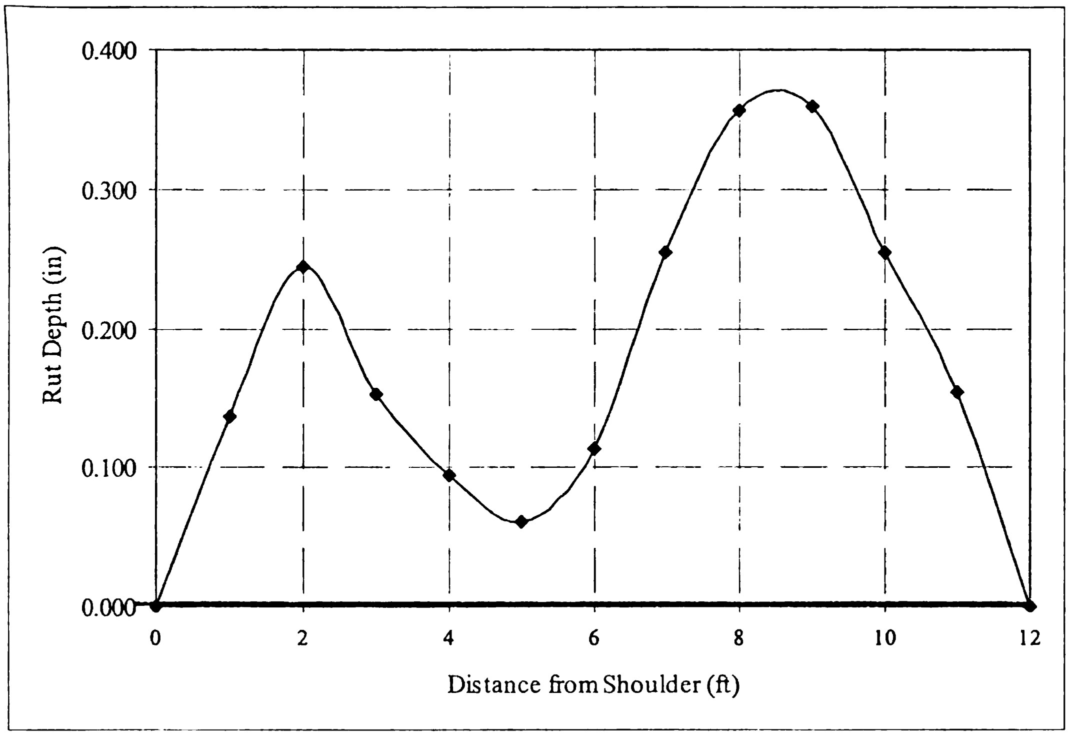


Figure B-29 Average profile of test site 4

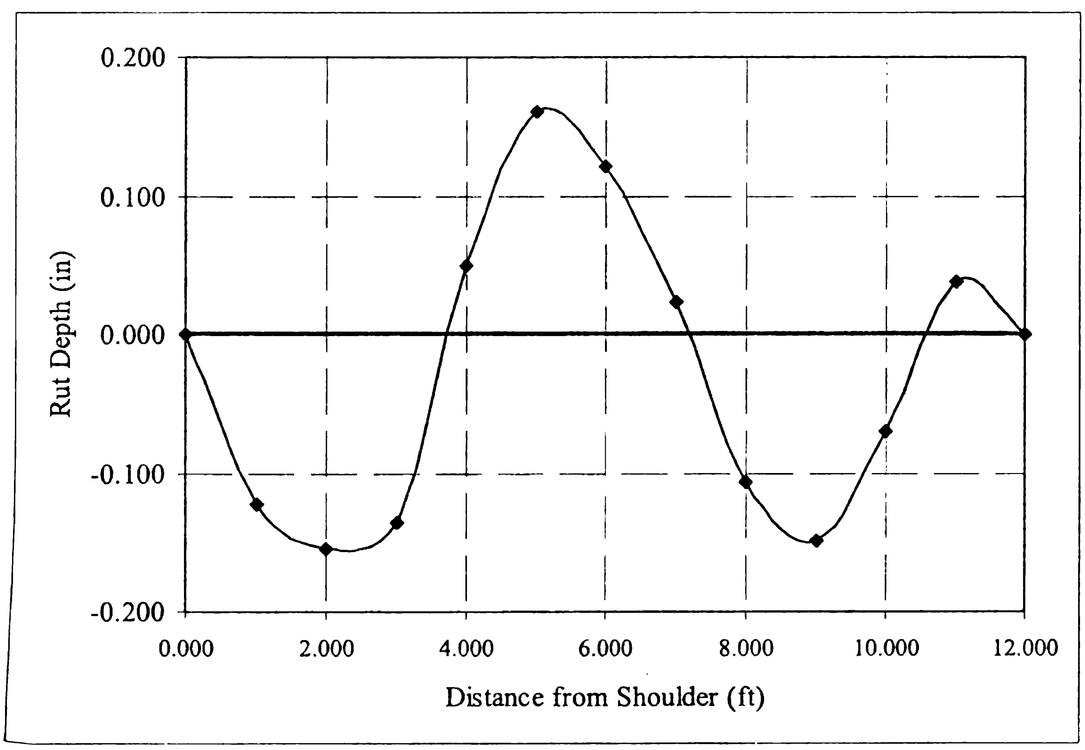


Figure B-30 Average profile of test site 5

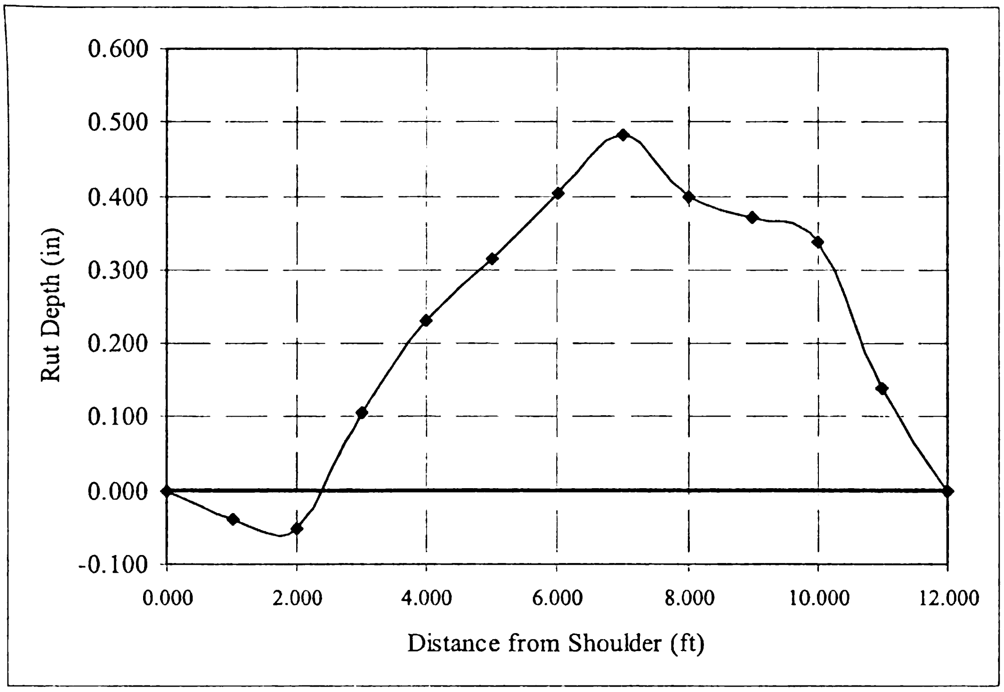


Figure B-31 Average profile of test site 6

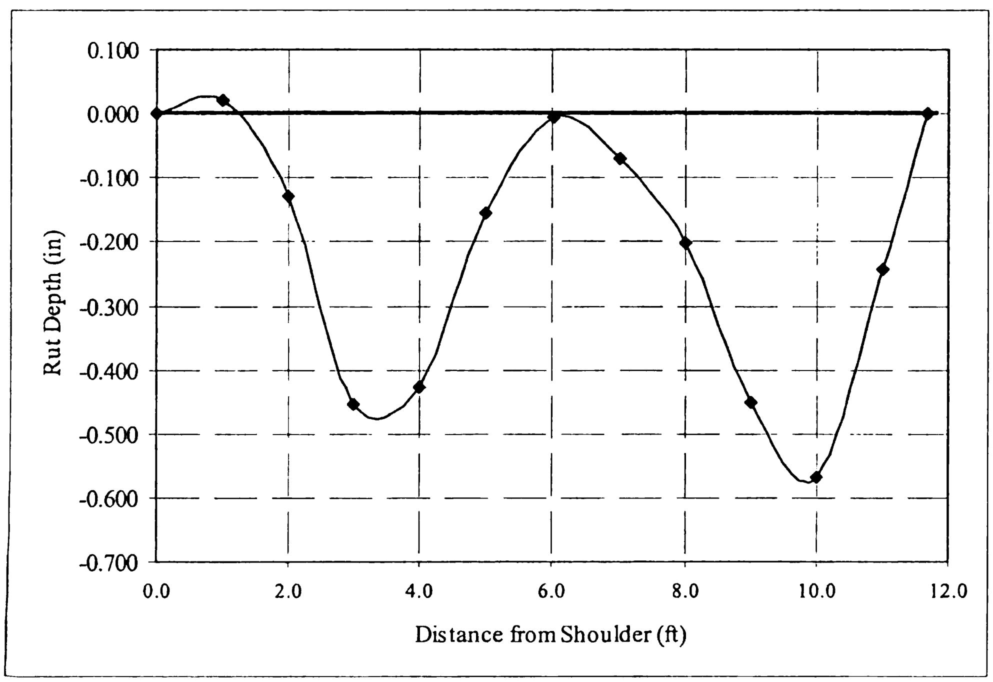


Figure B-32 Average profile of test site 7

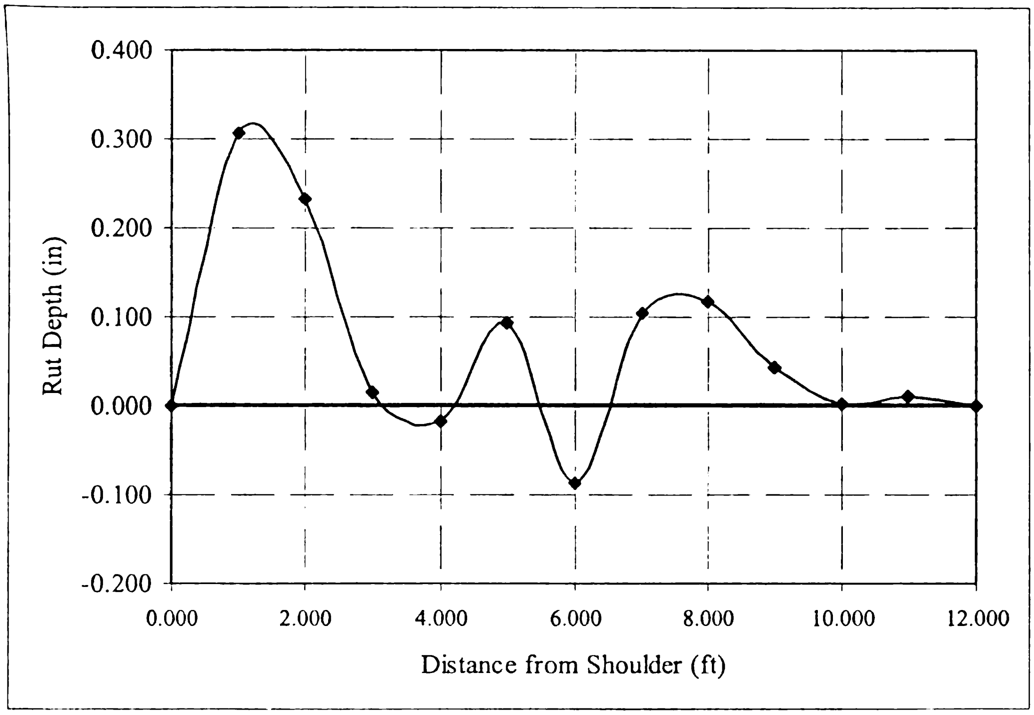


Figure B-33 Average profile of test site 8

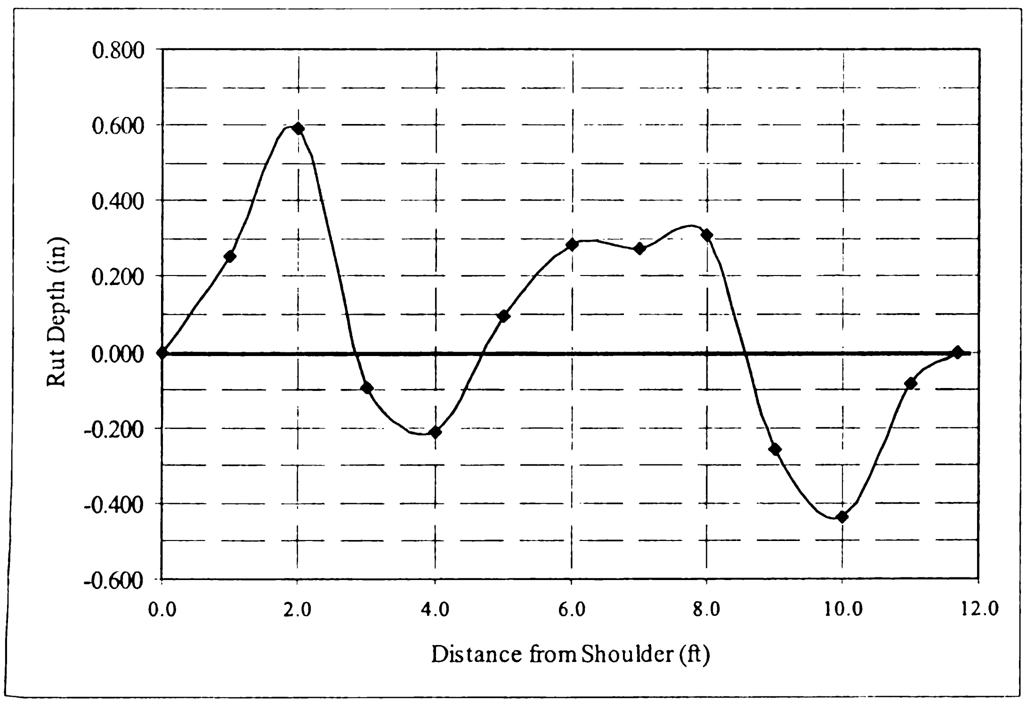
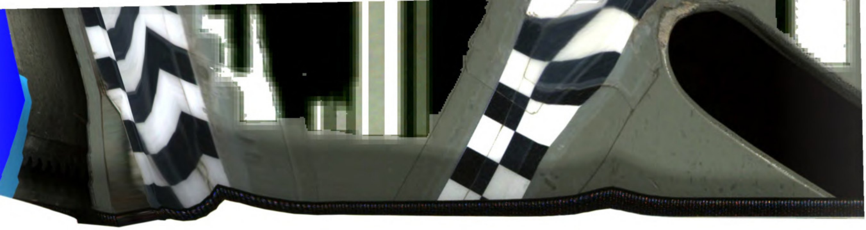


Figure B-34 Average profile of test site 12



APPENDIX C

Deflection data, linearity plots and backcalculation results

Table C-1 Deflection Data and Backcalculated Layer Moduli for Site #1 (N5N-1), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)							Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24				
		D0	D1	D2	D3	D4	D5	D6	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	13.18	9.25	6.07	3.77	2.53	1.16	0.82	434	31	40	4.53
	49.2	13.39	10.52	7.63	5.38	3.74	1.82	1.21	901	24	27	3.15
	98.4	15.61	10.99	7.61	5.31	3.64	1.87	1.26	518	26	26	3.09
	147.6	8.76	8.03	5.96	4.15	2.81	1.43	1.03	1,439	34	33	6.24
	196.8	12.86	9.02	6.06	3.95	2.79	1.43	1.04	470	34	32	3.78
	246.0	9.71	7.09	4.81	3.19	2.18	1.16	0.87	661	44	39	4.15
	295.2	13.03	9.83	6.68	4.59	3.09	1.50	0.97	699	27	33	3.27
	344.4	11.28	8.31	5.57	3.66	2.51	1.21	0.86	627	35	39	4.26
	393.6	13.14	9.77	6.81	4.62	3.02	1.39	0.91	712	26	36	3.36
	442.8	11.38	8.37	5.74	3.84	2.56	1.23	0.87	671	34	38	4.06
	492.0	11.13	8.54	5.83	3.85	2.55	1.28	0.92	683	34	37	4.33
COL	0	12.20	8.34	4.97	3.29	2.33	1.27	0.95	310	41	36	5.18
	98.4	13.48	9.60	6.47	4.50	3.17	1.70	1.22	497	32	27	3.55
	246	9.88	7.23	4.97	3.32	2.26	1.24	0.93	657	43	37	3.86
	393.6	11.52	7.88	4.92	3.33	2.33	1.24	0.91	407	41	37	4.53
	492	8.49	7.16	5.05	3.45	2.39	1.32	1.00	1,209	42	35	5.30
RWP	0	11.71	7.89	5.21	3.59	2.48	1.33	0.97	466	40	34	3.80
	98.4	13.76	8.85	5.46	3.85	2.72	1.51	1.12	278	38	30	4.64
	246	11.19	7.72	5.02	3.45	2.51	1.42	1.05	440	44	32	3.72
	393.6	12.41	8.07	5.00	3.34	2.24	1.24	0.96	288	42	36	5.01
	492	12.01	8.33	5.59	3.71	2.52	1.36	1.01	471	38	34	3.70

Note: LWP = left wheel path, COL = center of lane, RWP = right wheel path

Table C-2 Deflection Data and Backcalculated Layer Moduli for Site #1 (N5N-1), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	6.05	4.74	3.87	3.72	3.30	2.71	2.34	1.98	1.67	2,737	117	19	2.33
	49.2	9.27	6.23	4.90	4.59	3.98	3.22	2.74	2.28	1.97	551	90	17	3.18
	98.4	7.39	5.28	4.20	3.94	3.48	2.87	2.50	2.11	1.80	762	120	18	2.22
	147.6	6.44	4.51	3.51	3.30	2.95	2.46	2.16	1.85	1.60	616	152	21	1.34
	196.8	8.14	5.38	4.10	3.70	3.22	2.55	2.17	1.81	1.54	450	104	22	2.17
	246.0	6.01	4.96	3.95	3.47	2.99	2.42	2.08	1.75	1.50	2,265	105	22	0.74
	295.2	5.89	4.57	3.65	3.39	3.00	2.50	2.21	1.89	1.62	1,568	139	20	1.09
	344.4	6.20	4.54	3.66	3.44	3.07	2.57	2.20	1.89	1.60	1,197	139	20	2.24
	393.6	5.15	3.94	3.24	3.09	2.80	2.34	2.02	1.74	1.46	1,966	166	21	2.46
	442.8	5.96	4.42	3.67	3.54	3.24	2.73	2.36	1.99	1.68	1,574	151	18	3.44
	492.0	9.22	5.60	4.26	4.01	3.61	2.92	2.52	2.11	1.78	193	120	19	2.16
COL	0	6.05	4.67	3.85	3.66	3.30	2.75	2.39	2.02	1.73	1,861	136	18	2.56
	98.4	6.39	4.50	3.69	3.53	3.13	2.65	2.36	2.02	1.74	811	158	19	2.44
	246	4.98	4.07	3.30	3.10	2.78	2.30	2.02	1.73	1.49	3,118	148	21	1.12
	393.6	5.13	4.03	3.25	3.06	2.76	2.33	2.07	1.78	1.53	2,003	168	21	1.22
	492	6.42	4.53	3.65	3.55	3.17	2.67	2.33	1.99	1.71	899	150	19	2.79
RWP	0	6.34	5.03	4.22	3.87	3.47	2.86	2.42	2.02	1.74	2,699	107	18	2.49
	98.4	5.27	4.03	3.41	3.22	2.93	2.51	2.20	1.93	1.68	1,698	184	19	2.05
	246	4.34	3.06	2.56	2.36	2.24	1.96	1.75	1.55	1.34	541	327	24	0.71
	393.6	4.77	3.50	2.82	2.51	2.37	2.07	1.84	1.63	1.41	683	249	23	1.21
	492	4.47	3.52	3.02	2.93	2.72	2.39	2.10	1.84	1.59	1,948	250	18	2.50

Note: Deflections at D2 & D8 were not used for the backcalculation

Table C-3 Deflection Data and Backcalculated Layer Moduli for Site #1 (N5N-1), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	8.82	6.36	4.63	4.09	3.44	2.62	2.09	1.71	1.39	1,026	65	23	2.01
	49.2	13.79	9.01	5.96	4.94	4.17	3.15	2.58	2.09	1.77	251	50	19	1.03
	98.4	8.70	6.51	4.92	4.24	3.46	2.56	2.07	1.67	1.36	1,333	59	24	1.48
	147.6	9.61	6.95	5.31	4.59	3.88	3.01	2.51	2.07	1.73	863	66	19	1.47
	196.8	8.60	6.72	5.20	4.50	3.81	2.88	2.42	1.98	1.65	1,432	64	20	1.11
	246.0	9.03	6.60	4.97	4.25	3.53	2.68	2.18	1.80	1.48	1,016	64	22	1.26
	295.2	8.12	5.90	4.48	3.91	3.33	2.61	2.18	1.82	1.53	956	81	22	1.19
	344.4	7.39	5.53	4.35	3.86	3.26	2.58	2.18	1.84	1.55	1,290	89	21	1.30
	393.6	7.96	5.88	4.51	3.90	3.28	2.48	2.03	1.63	1.35	1,364	70	24	2.09
	442.8	7.16	5.58	4.38	3.93	3.37	2.61	2.16	1.78	1.48	2,026	79	22	1.64
	492.0	7.48	5.35	3.91	3.37	2.87	2.24	1.88	1.56	1.33	796	91	26	0.70
COL	0	10.52	6.36	4.30	3.78	3.23	2.54	2.08	1.70	1.39	229	78	24	1.02
	98.4	7.92	5.52	4.08	3.48	2.85	2.20	1.84	1.52	1.27	698	84	26	1.10
	246	5.98	4.66	3.61	3.21	2.81	2.29	1.96	1.65	1.40	1,569	118	23	0.79
	393.6	7.12	4.90	3.53	3.05	2.65	2.16	1.83	1.52	1.28	508	113	26	0.81
	492	6.23	4.24	3.08	2.65	2.32	1.92	1.68	1.48	1.26	427	149	28	2.23
RWP	0	5.53	4.20	3.47	3.16	2.78	2.21	1.84	1.56	1.27	2,456	118	24	2.63
	98.4	5.99	4.36	3.17	2.58	2.24	1.81	1.52	1.29	1.07	789	122	32	1.65
	246	6.12	4.50	3.50	3.01	2.62	2.13	1.82	1.56	1.30	971	126	25	0.27
	393.6	6.23	4.58	3.51	2.84	2.45	2.00	1.70	1.47	1.22	779	122	28	1.36
	492	6.23	3.99	2.76	2.29	2.06	1.72	1.52	1.34	1.16	294	162	32	4.21

Table C-4 Deflection Data and Backcalculated Layer Moduli for Site #2 (N5N-2), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	6.77	3.35	1.87	1.21	1.01	0.74	0.58	0.49	0.39	150	124	88	6.08
	49.2	5.00	3.61	2.44	1.59	1.23	0.87	0.73	0.63	0.52	572	112	68	5.34
	98.4	5.45	3.82	2.62	1.81	1.40	0.99	0.82	0.73	0.60	603	103	59	4.29
	147.6	5.09	3.77	2.74	1.83	1.37	0.86	0.66	0.55	0.44	1,101	84	75	2.52
	196.8	4.07	3.17	2.37	1.75	1.37	0.87	0.65	0.53	0.42	2,157	93	76	1.48
	246.0	4.39	2.17	1.29	1.02	0.86	0.67	0.53	0.45	0.37	253	202	92	2.17
	295.2	6.48	2.51	1.38	1.21	1.10	0.88	0.73	0.63	0.53	101	208	66	3.01
	344.4	7.46	3.38	1.85	1.04	0.78	0.53	0.41	0.35	0.29	110	118	125	7.71
	393.6	3.65	2.07	1.43	1.19	1.04	0.82	0.68	0.57	0.47	464	229	69	1.40
	442.8	5.96	2.75	1.57	1.11	0.88	0.66	0.54	0.44	0.39	155	152	96	4.00
	492.0	4.37	2.83	1.98	1.50	1.25	0.97	0.81	0.68	0.58	602	155	59	1.38
COL	0	4.82	2.49	1.42	1.06	0.90	0.69	0.55	0.46	0.37	242	173	91	3.88
	98.4	4.65	3.32	2.34	1.71	1.32	0.99	0.83	0.72	0.60	764	124	58	3.21
	246	3.41	1.51	0.98	0.88	0.76	0.61	0.51	0.43	0.34	254	338	92	1.18
	393.6	3.47	1.93	1.35	1.11	0.96	0.75	0.64	0.55	0.45	437	253	73	0.75
	492	5.12	2.71	1.68	1.26	1.07	0.89	0.78	0.65	0.56	209	187	64	5.19
RWP	0	5.34	2.64	1.43	1.01	0.84	0.66	0.53	0.45	0.36	180	168	96	6.36
	98.4	5.43	2.83	1.84	1.43	1.21	0.98	0.84	0.73	0.62	205	176	57	3.09
	246	4.12	1.83	1.14	0.99	0.84	0.67	0.55	0.46	0.39	216	261	87	0.76
	393.6	3.39	2.03	1.38	1.16	0.98	0.75	0.62	0.54	0.44	650	220	75	1.10
	492	4.81	2.71	1.65	1.29	1.10	0.89	0.79	0.65	0.57	271	182	64	4.57

TableC-5 Deflection Data and Backcalculated Layer Moduli for Site #2 (N5N-2), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	6.04	3.41	2.32	1.77	1.41	1.03	0.82	0.69	0.57	302	122	61	1.08
	49.2	5.12	2.96	1.92	1.32	1.01	0.72	0.58	0.46	0.41	362	131	90	2.42
	98.4	11.41	5.01	2.74	1.56	1.03	0.61	0.49	0.40	0.35	80	69	104	6.23
	147.6	3.58	1.92	1.17	0.89	0.79	0.63	0.53	0.45	0.39	300	276	98	5.58
	196.8	4.26	1.91	1.13	0.90	0.75	0.58	0.50	0.41	0.33	189	263	107	4.15
	246.0	10.31	4.17	2.02	1.57	1.30	0.97	0.78	0.64	0.52	68	107	68	5.67
	295.2	4.93	2.81	1.82	1.45	1.20	0.90	0.75	0.62	0.53	328	163	69	1.82
	344.4	3.37	2.14	1.52	1.24	1.06	0.86	0.73	0.62	0.52	593	256	68	1.99
	393.6	3.84	2.14	1.53	1.31	1.08	0.83	0.67	0.56	0.46	442	223	74	1.84
	442.8	2.46	1.86	1.35	1.04	0.85	0.64	0.54	0.43	0.35	1,970	234	94	2.34
	492.0	3.06	2.24	1.60	1.20	0.95	0.70	0.56	0.44	0.36	1,575	173	91	1.92
COL	0	5.56	3.07	1.94	1.52	1.28	0.98	0.82	0.68	0.57	244	155	63	2.94
	98.4	4.75	2.11	1.18	0.85	0.71	0.52	0.42	0.35	0.29	175	209	125	4.72
	246	5.34	2.51	1.52	1.24	1.09	0.91	0.79	0.63	0.57	149	232	68	5.03
	393.6	3.48	1.81	1.26	1.15	0.99	0.76	0.65	0.54	0.46	341	306	77	1.74
	492	5.86	2.51	1.49	1.35	1.18	0.95	0.79	0.65	0.56	126	224	65	1.84
RWP	0	4.52	2.47	1.52	1.26	1.08	0.89	0.78	0.66	0.56	242	232	67	5.16
	98.4	4.46	1.78	1.03	0.83	0.70	0.53	0.43	0.38	0.31	155	279	118	3.76
	246	4.74	2.56	1.64	1.38	1.15	0.87	0.73	0.62	0.52	282	190	70	1.93
	393.6	4.52	2.26	1.44	1.22	1.00	0.80	0.66	0.56	0.46	230	227	77	2.15
	492	7.20	3.94	2.43	2.07	1.73	1.31	1.06	0.86	0.69	207	115	48	0.73

Table C-6 Deflection Data and Backcalculated Layer Moduli for Site #2 (N5N-2), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	4.29	2.94	2.08	1.71	1.45	1.16	0.94	0.80	0.65	729	168	52	1.26
	49.2	6.15	3.97	2.82	2.24	1.80	1.29	1.00	0.84	0.67	612	97	49	1.46
	98.4	6.84	4.30	2.80	2.03	1.65	1.21	1.00	0.84	0.68	277	103	51	3.56
	147.6	7.72	4.68	2.80	2.05	1.63	1.16	0.91	0.74	0.62	272	83	57	2.79
	196.8	4.82	3.77	2.76	1.98	1.54	1.06	0.84	0.70	0.54	1,210	97	60	2.69
	246.0	4.47	3.53	2.58	1.80	1.34	0.88	0.67	0.55	0.45	1,466	92	75	2.68
	295.2	5.46	4.01	2.77	1.89	1.37	0.90	0.70	0.56	0.46	807	85	73	2.76
	344.4	5.82	4.11	2.75	1.83	1.30	0.83	0.64	0.53	0.42	650	82	79	3.32
	393.6	7.65	3.72	2.25	1.57	1.29	0.96	0.78	0.65	0.54	131	119	67	4.40
	442.8	11.69	6.24	3.91	3.01	2.45	1.78	1.43	1.17	0.96	123	67	36	1.30
	492.0	6.36	4.11	3.04	2.60	2.06	1.54	1.29	1.06	0.88	583	104	39	2.21
COL	0	3.77	2.35	1.64	1.45	1.26	1.02	0.89	0.74	0.63	496	249	56	1.34
	98.4	5.53	3.15	2.04	1.54	1.25	1.00	0.85	0.73	0.61	244	159	61	4.96
	246	4.78	2.76	1.60	1.13	0.87	0.66	0.56	0.47	0.40	298	158	95	6.90
	393.6	5.86	3.19	1.95	1.40	1.10	0.83	0.69	0.58	0.48	221	138	76	4.88
	492	5.74	2.79	1.60	1.18	0.97	0.76	0.66	0.55	0.49	154	177	82	6.87
RWP	0	4.18	2.61	1.73	1.47	1.23	0.99	0.85	0.74	0.61	443	207	59	3.19
	98.4	5.61	3.30	1.91	1.36	1.11	0.84	0.71	0.58	0.51	258	136	75	6.27
	246	4.92	2.39	1.39	1.05	0.85	0.65	0.54	0.47	0.38	193	196	96	5.42
	393.6	4.27	2.24	1.50	1.32	1.05	0.81	0.66	0.56	0.47	317	212	76	1.85
	492	5.05	2.73	1.62	1.19	0.96	0.75	0.65	0.56	0.48	223	178	82	7.00

Table C-7 Deflection Data and Backcalculated Layer Moduli for Site #3 (RRW-3), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.84	7.34	5.39	3.70	2.40	1.18	0.83	0.70	0.61	526	36	63	7.05
	49.2	9.67	6.94	4.86	3.26	2.03	1.00	0.71	0.64	0.56	389	41	71	9.22
	98.4	8.63	6.68	4.82	3.18	2.01	1.00	0.74	0.61	0.54	548	42	72	8.14
	147.6	9.52	7.30	5.33	3.65	2.36	1.18	0.78	0.71	0.60	552	37	65	8.08
	196.8	10.13	7.08	5.00	3.41	2.24	1.16	0.83	0.67	0.59	414	39	64	4.79
	246.0	9.27	6.75	4.81	3.19	2.06	1.10	0.84	0.74	0.66	412	44	59	7.87
	295.2	6.88	5.38	3.95	2.81	1.89	1.10	0.83	0.72	0.64	796	54	59	5.50
	344.4	7.48	5.53	4.08	2.95	2.05	1.25	1.01	0.81	0.73	669	54	50	3.53
	393.6	6.18	3.88	2.33	1.31	0.95	0.76	0.65	0.54	0.46	196	115	77	10.61
	442.8	6.76	5.22	3.66	2.41	1.51	0.84	0.68	0.64	0.56	673	59	72	11.49
	492.0	6.94	4.64	3.06	1.88	1.14	0.65	0.53	0.48	0.42	326	73	94	11.08
COL	0	11.34	7.26	5.05	3.46	2.22	1.16	0.88	0.77	0.67	287	40	57	7.60
	98.4	10.64	7.93	5.67	3.63	2.19	1.07	0.76	0.67	0.57	380	36	68	9.94
	246	10.30	7.28	4.90	3.06	1.87	1.02	0.80	0.70	0.64	243	45	63	9.76
	393.6	4.03	2.12	1.45	1.06	0.90	0.74	0.61	0.45	0.38	344	193	78	5.32
	492	6.77	4.61	3.16	2.00	1.24	0.70	0.54	0.48	0.44	393	70	92	8.51
RWP	0	11.56	8.15	5.62	3.79	2.39	1.19	0.87	0.74	0.64	329	35	60	7.91
	98.4	11.94	8.51	5.94	3.79	2.31	1.07	0.74	0.62	0.55	310	33	73	9.09
	246	10.48	7.67	5.41	3.47	2.17	1.14	0.86	0.72	0.64	343	39	61	7.77
	393.6	7.61	5.62	3.49	1.79	1.14	0.83	0.74	0.64	0.55	187	79	68	14.76
	492	7.25	5.42	3.81	2.33	1.39	0.68	0.57	0.37	0.36	567	53	110	10.17

Table C-8 Deflection Data and Backcalculated Layer Moduli for Site #3 (RRW-3), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.38	6.59	4.56	2.93	1.79	0.80	0.55	0.45	0.40	305	46	67	9.84
	49.2	10.24	7.59	5.33	3.50	2.11	0.90	0.59	0.50	0.44	393	37	64	11.13
	98.4	9.69	7.00	4.85	3.13	1.89	0.92	0.68	0.63	0.56	327	45	50	11.99
	147.6	12.13	8.56	5.78	3.58	2.11	1.01	0.69	0.61	0.53	221	37	52	11.14
	196.8	10.97	8.03	5.60	3.56	2.21	1.13	0.86	0.75	0.66	278	40	39	10.14
	246.0	9.02	6.73	4.88	3.29	2.16	1.23	0.99	0.84	0.76	410	49	32	7.56
	295.2	10.72	8.44	6.45	5.14	3.92	2.75	2.15	1.73	1.41	666	38	12	1.00
	344.4	11.96	9.36	6.98	5.18	3.64	2.41	1.92	1.60	1.32	408	36	14	4.07
	393.6	8.38	6.30	4.72	3.64	2.78	2.00	1.65	1.43	1.18	559	59	14	2.59
	442.8	9.66	7.41	5.51	4.09	2.89	1.87	1.48	1.27	1.07	503	44	19	4.37
	492.0	10.69	8.40	6.48	5.12	4.00	2.80	2.21	1.80	1.47	671	39	11	0.74
COL	0	9.28	6.78	4.72	3.14	1.96	0.91	0.62	0.51	0.44	403	43	59	8.63
	98.4	10.26	7.54	5.22	3.32	1.98	0.91	0.67	0.61	0.52	273	42	52	12.95
	246	9.16	6.88	4.92	3.41	2.29	1.35	1.09	0.94	0.82	411	48	28	7.09
	393.6	7.95	5.97	4.42	3.31	2.57	1.84	1.56	1.28	1.09	542	63	16	2.46
	492	11.67	9.17	6.89	5.37	4.09	2.82	2.22	1.77	1.43	569	35	12	1.38
RWP	0	10.45	7.95	5.62	3.74	2.25	0.94	0.61	0.51	0.45	376	35	62	11.25
	98.4	12.57	8.72	5.77	3.50	1.92	0.83	0.61	0.54	0.48	143	39	60	15.75
	246	9.93	7.30	5.12	3.49	2.31	1.40	1.14	1.00	0.84	324	47	27	7.72
	393.6	8.44	6.50	4.70	3.50	2.60	1.81	1.50	1.27	1.09	516	56	17	3.78
	492	14.29	11.24	8.44	6.53	4.85	3.16	2.36	1.82	1.47	496	26	12	1.13

Table C-9 Deflection Data and Backcalculated Layer Moduli for Site #3 (RRW-3), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	7.22	5.54	3.99	2.84	2.05	1.42	1.17	1.02	0.89	569	62	41	3.37
	49.2	8.83	6.76	4.83	3.47	2.60	1.86	1.50	1.28	1.06	481	52	31	2.88
	98.4	8.34	6.30	4.48	3.10	2.16	1.42	1.18	1.02	0.88	478	52	44	4.03
	147.6	8.65	6.59	4.77	3.29	2.21	1.38	1.15	1.03	0.91	508	47	47	4.67
	196.8	11.01	8.46	6.03	4.00	2.60	1.58	1.28	1.13	0.98	367	36	44	5.38
	246.0	11.37	8.82	6.53	4.63	3.15	1.88	1.44	1.21	1.03	498	32	38	3.10
	295.2	12.01	8.87	6.30	4.18	2.73	1.61	1.28	1.10	0.96	304	34	43	4.61
	344.4	8.02	6.22	4.43	2.99	1.98	1.25	1.09	0.98	0.87	478	52	49	6.27
	393.6	7.29	5.48	3.94	2.77	1.96	1.35	1.16	1.06	0.94	523	63	43	4.15
	442.8	8.85	6.82	4.97	3.52	2.46	1.57	1.28	1.11	0.96	563	45	41	3.52
	492.0	8.96	7.19	5.43	4.10	3.04	2.09	1.70	1.44	1.23	701	44	28	2.46
COL	0	8.59	6.14	4.30	2.94	2.04	1.37	1.17	1.01	0.86	385	55	44	4.30
	98.4	7.70	5.89	4.23	3.02	2.20	1.51	1.26	1.11	0.97	568	57	39	3.33
	246	11.38	8.80	6.29	4.35	2.93	1.79	1.41	1.17	1.02	400	34	38	3.97
	393.6	8.34	6.35	4.51	3.15	2.18	1.42	1.18	1.05	0.92	474	52	43	4.18
	492	11.26	8.72	6.34	4.56	3.18	2.02	1.57	1.32	1.12	475	34	33	2.88
RWP	0	9.73	7.36	5.12	3.36	2.20	1.40	1.20	1.04	0.90	363	44	46	6.34
	98.4	9.99	7.85	5.58	3.67	2.47	1.64	1.39	1.20	1.03	386	42	38	6.10
	246	11.58	8.59	6.07	4.05	2.71	1.61	1.28	1.07	0.92	345	35	43	4.14
	393.6	10.30	7.75	5.38	3.59	2.38	1.50	1.21	1.08	0.93	347	41	45	4.86
	492	13.00	10.09	7.20	4.99	3.33	1.96	1.49	1.26	1.05	390	29	37	3.76

Table C-10 Deflection Data and Backcalculated Layer Moduli for Site #4 (N5S-4), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.93	7.34	5.13	4.25	3.30	2.18	1.60	1.28	1.07	383	38	31	0.91
	49.2	11.70	8.64	6.00	5.01	3.79	2.52	1.81	1.39	1.16	335	31	28	1.55
	98.4	13.17	10.52	7.66	6.25	4.66	2.97	2.13	1.62	1.34	362	23	25	1.56
	147.6	11.98	8.96	6.44	5.39	4.12	2.71	1.97	1.51	1.24	370	29	26	1.19
	196.8	12.58	9.41	6.55	5.47	4.17	2.80	2.02	1.52	1.24	329	28	26	2.12
	246.0	12.33	9.90	7.01	5.82	4.38	2.89	2.07	1.58	1.29	380	26	25	1.99
	295.2	11.15	7.89	5.62	4.69	3.56	2.36	1.73	1.36	1.11	338	34	29	1.07
	344.4	10.40	7.86	5.56	4.66	3.53	2.25	1.64	1.27	1.07	411	33	31	0.62
	393.6	13.87	10.52	7.29	5.99	4.34	2.68	1.90	1.45	1.19	285	24	28	1.20
	442.8	10.87	8.25	5.79	4.72	3.56	2.30	1.70	1.35	1.11	347	33	30	1.14
	492.0	13.01	9.59	6.37	5.03	3.76	2.41	1.74	1.34	1.10	243	29	30	1.97
COL	0	11.22	9.11	6.64	5.36	4.01	2.48	1.78	1.40	1.15	424	27	29	1.96
	98.4	14.40	11.25	8.29	7.10	5.29	3.26	2.23	1.63	1.34	398	19	24	0.89
	246	11.51	9.02	6.42	5.50	4.27	2.86	2.10	1.63	1.33	426	29	24	1.23
	393.6	12.42	9.67	6.91	5.70	4.23	2.68	1.96	1.51	1.24	348	26	27	1.30
	492	11.76	9.16	6.80	5.60	4.15	2.58	1.82	1.38	1.17	423	26	29	0.76
RWP	0	11.82	9.22	6.65	5.55	4.12	2.57	1.85	1.43	1.17	391	27	28	0.99
	98.4	16.40	12.77	9.15	7.51	5.52	3.36	2.32	1.72	1.39	288	18	23	1.13
	246	12.05	9.77	6.87	5.70	4.43	2.97	2.18	1.67	1.35	388	27	24	2.30
	393.6	12.23	9.36	6.58	5.34	4.00	2.58	1.89	1.52	1.23	307	29	27	1.41
	492	13.80	10.92	7.88	6.32	4.47	2.61	1.82	1.37	1.16	320	21	29	2.10

Table C-11 Deflection Data and Backcalculated Layer Moduli for Site #4 (N5S-4), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	11.77	7.99	5.13	4.05	3.06	2.00	1.48	1.18	0.99	207	38	34	1.38
	49.2	11.66	8.43	5.85	4.81	3.63	2.34	1.74	1.37	1.13	299	33	29	0.48
	98.4	11.73	8.59	5.87	4.75	3.49	2.14	1.53	1.20	0.98	298	30	34	0.82
	147.6	11.43	7.99	5.16	4.03	2.88	1.60	1.06	0.83	0.67	259	32	49	1.45
	196.8	10.23	7.24	4.92	3.82	2.71	1.55	1.09	0.84	0.68	303	35	48	1.13
	246.0	14.10	10.63	7.26	5.55	3.89	2.21	1.59	1.19	1.00	231	24	34	2.53
	295.2	12.42	8.66	5.71	4.39	3.05	1.63	1.06	0.79	0.64	247	28	50	0.78
	344.4	14.43	10.72	7.45	6.02	4.44	2.61	1.76	1.32	1.06	284	22	30	0.71
	393.6	13.42	9.55	6.54	5.18	3.70	2.20	1.52	1.15	0.93	252	26	35	0.57
	442.8	14.81	11.48	7.92	6.51	4.72	2.69	1.78	1.25	1.00	314	19	31	1.51
	492.0	14.59	11.04	7.43	5.91	4.33	2.50	1.68	1.23	1.02	267	22	32	1.59
COL	0	14.93	10.78	7.51	5.85	4.24	2.52	1.76	1.34	1.09	231	23	30	0.66
	98.4	12.33	8.70	5.94	4.70	3.51	2.16	1.54	1.20	1.02	261	31	33	0.72
	246	14.80	11.31	7.64	5.87	4.16	2.31	1.58	1.22	1.01	229	22	33	2.50
	393.6	13.77	10.30	7.12	5.44	3.78	2.14	1.45	1.08	0.86	251	24	37	1.73
	492	14.64	11.71	8.05	6.26	4.54	2.63	1.75	1.30	1.05	280	20	31	2.40
RWP	0	14.64	11.52	8.33	6.62	4.81	2.85	1.97	1.50	1.25	300	21	27	1.52
	98.4	11.39	8.88	6.33	5.13	3.70	2.20	1.55	1.22	1.02	360	28	33	1.90
	246	15.50	11.94	8.10	6.04	4.13	2.33	1.62	1.24	1.03	199	22	33	3.27
	393.6	13.73	10.73	7.56	5.73	3.98	2.16	1.35	0.98	0.79	294	21	40	1.98
	492	15.94	12.10	8.06	6.29	4.47	2.55	1.69	1.25	1.01	224	20	32	1.87

Table C-12 Deflection Data and Backcalculated Layer Moduli for Site #4 (N5S-4), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	11.65	8.49	6.01	4.84	3.58	2.18	1.53	1.13	0.91	344	29	35	1.22
	49.2	13.04	10.13	7.06	5.65	4.12	2.38	1.65	1.19	0.97	327	23	33	1.77
	98.4	9.70	7.32	5.10	4.05	2.95	1.85	1.34	1.07	0.91	360	37	38	1.67
	147.6	10.32	7.75	5.45	4.42	3.29	2.04	1.49	1.18	1.00	367	34	34	1.05
	196.8	8.70	6.28	4.44	3.67	2.71	1.73	1.30	1.04	0.89	403	44	39	0.83
	246.0	13.90	11.13	7.93	6.47	4.66	2.64	1.74	1.27	1.04	349	20	31	1.54
	295.2	15.79	11.94	8.15	6.63	4.70	2.56	1.61	1.19	0.98	265	18	33	0.97
	344.4	16.77	11.86	7.86	6.27	4.30	2.40	1.56	1.14	0.97	201	20	34	0.75
	393.6	14.37	11.91	8.03	6.21	4.39	2.49	1.60	1.17	0.97	286	20	34	3.34
	442.8	14.34	11.14	7.80	6.33	4.49	2.52	1.64	1.19	0.00	312	20	33	1.22
	492.0	12.92	9.59	6.54	5.26	3.70	2.04	1.36	1.06	0.90	278	25	38	1.85
COL	0	12.80	9.62	6.72	5.23	3.79	2.25	1.52	1.15	0.91	296	25	35	1.25
	98.4	10.75	7.98	5.55	4.62	3.42	2.09	1.50	1.18	0.98	364	32	34	0.65
	246	16.41	12.03	7.94	6.18	4.41	2.54	1.68	1.22	1.00	208	20	32	1.75
	393.6	15.37	11.96	8.45	6.70	4.75	2.68	1.69	1.27	1.02	285	19	32	1.56
	492	14.57	11.13	7.79	6.18	4.42	2.42	1.53	1.16	0.95	286	20	35	1.48
RWP	0	12.54	9.87	6.65	5.05	3.71	2.13	1.42	1.07	0.87	292	25	37	2.67
	98.4	11.36	8.99	6.50	5.26	3.85	2.30	1.62	1.27	1.05	393	27	32	1.79
	246	12.66	9.78	6.82	5.43	3.86	2.25	1.52	1.15	0.98	316	24	35	1.59
	393.6	14.24	11.94	8.85	7.00	4.95	2.67	1.66	1.18	0.97	369	17	34	2.56
	492	13.75	11.13	8.03	6.51	4.53	2.45	1.57	1.13	0.95	349	19	35	2.31

Table C-13 Deflection Data and Backcalculated Layer Moduli for Site #5 (N5N-5), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	12.19	10.41	7.95	7.26	5.88	4.26	3.24	2.53	2.01	812	24	15	1.34
	49.2	12.12	10.04	7.82	7.39	6.20	4.74	3.74	2.91	2.39	921	27	13	2.22
	98.4	9.48	7.97	6.23	5.83	4.80	3.54	2.71	2.15	1.75	1,179	31	18	1.09
	147.6	12.52	10.31	7.80	7.02	5.64	4.02	3.00	2.33	1.89	699	24	17	1.43
	196.8	12.56	10.71	8.14	7.35	5.87	4.22	3.18	2.45	1.93	751	23	16	1.63
	246.0	9.82	8.19	6.49	6.18	5.13	3.79	2.84	2.16	1.71	1,391	26	18	1.96
	295.2	13.41	11.34	8.81	8.09	6.63	4.75	3.59	2.75	2.23	831	20	14	1.37
	344.4	12.65	10.31	8.39	8.04	6.55	4.80	3.74	2.89	2.33	1,023	22	14	2.11
	393.6	13.09	11.00	8.76	8.08	6.51	4.71	3.55	2.83	2.30	843	22	14	0.83
	442.8	12.74	10.75	8.47	7.92	6.55	4.88	3.73	2.92	2.34	939	23	13	1.46
	492.0	16.29	13.51	10.76	9.65	7.72	5.62	4.25	3.33	2.68	608	18	12	1.33
COL	0	11.61	9.92	7.86	7.19	5.89	4.32	3.39	2.70	2.19	914	26	14	0.89
	98.4	9.37	7.85	6.32	5.67	4.59	3.28	2.40	1.86	1.49	1,213	28	21	1.15
	246	8.55	7.17	5.83	5.30	4.38	3.19	2.40	1.87	1.49	1,442	32	21	1.27
	393.6	9.93	8.51	6.92	6.38	5.27	3.93	3.01	2.37	1.92	1,307	28	17	1.13
	492	12.59	10.50	8.36	7.47	6.19	4.69	3.63	2.86	2.32	790	26	13	1.87
RWP	0	10.07	8.62	7.04	6.62	5.58	4.30	3.44	2.72	2.22	1,342	30	14	1.56
	98.4	8.57	7.14	5.99	5.59	4.54	3.30	2.44	1.87	1.52	1,732	28	22	1.68
	246	7.39	6.37	5.24	4.93	4.18	3.13	2.43	1.88	1.49	2,185	34	21	1.38
	393.6	8.66	7.68	6.33	5.92	4.90	3.63	2.79	2.18	1.77	1,816	28	18	0.86
	492	12.71	10.80	8.80	7.87	6.36	4.57	3.51	2.72	2.20	884	22	15	1.11

Table C-14 Deflection Data and Backcalculated Layer Moduli for Site #5 (N5N-5), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	15.48	12.94	10.50	9.50	7.79	5.71	4.38	3.38	2.70	745	18	12	1.56
	49.2	13.79	11.30	8.66	7.84	6.36	4.49	3.37	2.61	2.10	683	21	15	1.30
	98.4	9.92	8.43	6.66	6.28	5.24	4.01	3.18	2.56	2.08	1,154	33	15	1.21
	147.6	13.51	11.19	8.60	7.30	5.96	4.29	3.39	2.71	2.23	512	26	14	1.38
	196.8	10.41	8.95	7.23	6.68	5.55	4.11	3.13	2.46	1.95	1,268	26	16	1.04
	246.0	9.42	7.98	6.41	5.91	4.88	3.50	2.61	1.97	1.55	1,424	26	20	1.42
	295.2	10.17	8.49	6.62	6.04	4.94	3.71	2.91	2.32	1.88	916	33	17	1.37
	344.4	16.36	14.24	10.96	9.53	7.59	5.33	4.10	3.20	2.58	530	18	12	1.73
	393.6	13.31	11.49	9.14	8.39	6.65	4.88	3.71	2.92	2.34	835	21	14	1.36
	442.8	15.98	13.65	10.90	8.74	6.94	4.92	3.71	2.81	2.27	490	19	14	2.46
	492.0	12.87	10.86	8.47	7.51	6.06	4.46	3.40	2.68	2.18	710	24	15	1.50
COL	0	13.55	10.33	8.42	7.64	6.40	4.95	3.95	3.15	2.54	631	29	12	3.02
	98.4	9.14	7.74	6.25	5.74	4.77	3.56	2.79	2.23	1.82	1,243	34	17	1.01
	246	9.52	7.98	6.50	5.92	4.84	3.46	2.59	1.96	1.54	1,326	27	20	1.46
	393.6	11.10	9.37	7.59	6.89	5.70	4.26	3.31	2.61	2.13	1,039	27	15	1.35
	492	10.48	8.63	6.96	6.30	5.25	3.93	3.10	2.48	2.01	971	32	15	1.38
RWP	0	10.42	9.07	7.83	7.35	6.28	4.91	3.93	3.14	2.55	1,768	26	12	1.31
	98.4	7.96	6.75	5.57	5.23	4.41	3.33	2.59	2.08	1.69	1,803	36	19	1.14
	246	9.14	7.76	6.21	5.68	4.66	3.33	2.49	1.90	1.53	1,365	28	21	1.18
	393.6	10.47	9.14	7.42	6.98	5.74	4.30	3.31	2.62	2.16	1,348	25	15	0.97
	492	10.58	8.99	7.16	6.43	5.36	4.02	3.16	2.51	2.03	994	30	15	1.40

Table C-15 Deflection Data and Backcalculated Layer Moduli for Site #5 (N5N-5), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	14.30	12.84	9.78	8.51	6.66	4.57	3.41	2.65	2.16	633	19	15	2.27
	49.2	10.27	8.54	6.78	6.17	5.20	4.01	3.15	2.50	2.04	999	33	15	2.03
	98.4	12.30	10.36	8.14	7.37	5.98	4.41	3.39	2.65	2.16	814	25	15	1.47
	147.6	12.65	10.29	8.16	7.48	6.20	4.56	3.51	2.75	2.20	821	25	14	1.77
	196.8	9.95	8.41	6.75	6.13	5.13	3.90	3.08	2.50	2.07	1,053	34	15	1.09
	246.0	12.12	10.08	7.89	7.09	5.87	4.34	3.33	2.57	2.08	821	25	15	2.00
	295.2	10.46	8.75	7.01	6.30	5.08	3.55	2.63	2.05	1.65	1,033	26	20	0.66
	344.4	10.30	8.79	7.06	6.39	5.19	3.74	2.81	2.24	1.81	1,090	27	18	0.61
	393.6	9.68	8.10	6.44	5.76	4.73	3.51	2.72	2.15	1.78	1,027	33	18	1.39
	442.8	8.89	7.43	5.95	5.47	4.55	3.38	2.64	2.10	1.73	1,239	35	18	1.20
	492.0	9.46	7.86	6.22	5.69	4.73	3.54	2.73	2.15	1.76	1,127	33	18	1.67
COL	0	11.41	9.76	7.69	6.69	5.38	3.97	3.06	2.38	1.96	800	27	16	1.89
	98.4	12.13	10.04	8.00	6.93	5.60	4.08	3.13	2.44	2.00	727	26	16	1.55
	246	10.94	9.13	7.45	6.77	5.56	4.05	3.02	2.33	1.84	1,122	24	17	1.51
	393.6	9.89	8.25	6.67	5.94	4.82	3.46	2.54	1.97	1.57	1,139	27	20	1.28
	492	9.17	7.75	6.23	5.57	4.63	3.40	2.59	2.06	1.68	1,197	32	19	1.09
RWP	0	9.87	8.42	6.79	6.28	5.16	3.83	2.97	2.38	1.96	1,198	30	16	0.82
	98.4	11.68	9.77	7.84	7.13	5.81	4.29	3.31	2.56	2.09	946	25	15	1.63
	246	11.85	10.04	7.94	7.14	5.74	4.08	3.09	2.39	1.90	881	24	17	1.07
	393.6	9.50	8.15	6.58	5.90	4.76	3.29	2.42	1.85	1.49	1,257	26	22	0.60
	492	9.46	8.09	6.49	5.89	4.84	3.56	2.73	2.10	1.71	1,274	29	19	1.52

Table C-16 Deflection Data and Backcalculated Layer Moduli for Site #6 (N5S-6), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	10.24	8.86	6.66	5.97	4.57	3.23	2.45	1.89	1.52	426	28	18	2.90
	49.2	26.79	22.97	17.54	15.59	11.97	7.68	5.19	3.56	2.62	198	8	10	1.86
	98.4	8.31	7.14	5.61	5.35	4.46	3.31	2.51	1.91	1.53	839	30	18	1.07
	147.6	14.06	11.81	9.14	8.25	6.53	4.42	3.04	2.26	1.77	386	16	16	1.00
	196.8	22.42	18.89	14.56	13.16	10.31	6.79	4.71	3.43	2.59	238	10	10	1.12
	246.0	18.96	15.52	11.45	10.27	7.99	5.21	3.79	2.83	2.22	219	14	12	1.49
	295.2	14.71	13.11	10.27	9.51	7.36	5.09	3.46	2.50	1.84	448	13	15	2.15
	344.4	14.48	12.00	8.63	7.40	5.50	3.58	2.59	1.87	1.45	245	19	18	2.98
	393.6	21.27	18.60	14.43	12.26	9.13	5.75	3.79	2.67	2.00	227	10	14	2.78
	442.8	21.84	18.47	13.41	11.45	8.44	5.24	3.68	2.68	2.12	168	12	13	3.18
	492.0	9.51	8.26	6.38	6.10	5.02	3.68	2.74	2.13	1.67	686	26	16	1.03
COL	0	10.26	8.84	6.80	6.59	5.28	3.68	2.63	2.00	1.59	641	21	18	1.13
	98.4	11.49	9.95	7.64	7.24	5.80	4.06	2.93	2.25	1.80	526	20	16	1.31
	246	25.30	20.59	14.42	12.46	9.23	5.68	3.88	2.86	2.22	134	11	12	2.44
	393.6	20.02	16.36	12.07	10.37	7.80	4.83	3.50	2.46	1.94	190	13	14	2.54
	492	10.89	9.29	7.05	6.72	5.42	3.86	2.87	2.23	1.80	519	24	15	1.09
RWP	0	15.44	12.26	8.95	7.91	6.05	3.97	2.85	2.13	1.70	242	19	16	1.30
	98.4	14.65	11.30	7.89	7.13	5.60	3.95	2.96	2.27	1.82	221	24	15	1.57
	246	21.83	17.07	12.29	11.01	8.45	5.58	3.89	2.77	2.17	179	13	12	1.63
	393.6	16.67	13.11	9.55	8.37	6.54	4.43	3.18	2.34	1.78	226	18	14	1.59
	492	13.91	11.11	8.06	7.30	5.63	3.91	2.96	2.27	1.82	261	23	15	1.69

Table C-17 Deflection Data and Backcalculated Layer Moduli for Site #6 (N5S-6), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	10.18	8.84	6.82	6.60	5.39	3.95	2.98	2.34	1.88	651	23	18	1.04
	49.2	12.54	10.04	7.54	7.01	5.65	4.01	3.04	2.39	1.93	361	24	17	0.65
	98.4	11.72	10.16	7.78	7.01	5.64	3.90	2.84	2.16	1.71	467	20	19	1.61
	147.6	21.46	18.44	13.39	10.66	7.73	4.77	3.37	2.52	2.01	149	12	16	4.25
	196.8	14.45	12.12	9.18	8.50	6.68	4.49	3.15	2.38	1.89	370	16	18	0.99
	246.0	11.77	10.17	7.71	7.13	5.68	3.97	2.94	2.22	1.76	470	20	19	1.68
	295.2	23.85	20.88	14.90	13.26	9.51	5.82	3.97	2.97	2.28	165	10	14	4.01
	344.4	18.57	15.85	11.46	9.61	6.75	3.98	2.76	2.01	1.58	180	13	20	4.26
	393.6	19.25	16.31	12.07	10.08	7.56	4.92	3.34	2.48	1.89	205	13	17	2.78
	442.8	15.78	13.00	9.34	8.15	6.24	4.12	2.95	2.17	1.72	247	17	19	2.28
	492.0	13.28	12.28	8.92	8.19	6.23	4.02	2.81	2.15	1.67	385	15	20	3.95
COL	0	12.21	10.31	7.77	7.24	5.79	4.21	3.24	2.46	2.00	424	22	16	1.92
	98.4	15.42	12.25	9.02	8.15	6.36	4.34	3.29	2.53	2.04	253	19	16	1.25
	246	18.11	14.61	10.25	8.78	6.49	4.19	3.08	2.38	1.92	171	17	17	2.83
	393.6	16.17	12.98	9.41	8.21	6.13	4.14	3.05	2.32	1.83	216	18	17	2.25
	492	12.37	10.05	7.33	6.52	5.19	3.62	2.64	2.02	1.64	331	23	20	1.71
RWP	0	16.16	13.13	9.17	8.31	6.42	4.50	3.37	2.64	2.14	218	19	15	2.16
	98.4	12.87	12.67	9.37	8.42	6.61	4.62	3.49	2.70	2.17	427	17	16	4.87
	246	16.50	13.51	9.57	8.53	6.69	4.48	3.25	2.49	1.99	233	17	16	1.81
	393.6	17.34	13.43	9.43	8.30	6.29	4.24	3.08	2.39	1.89	184	18	17	1.58
	492	14.89	11.63	8.27	7.11	5.52	3.76	2.73	2.09	1.63	221	21	19	1.85

Table C-18 Deflection Data and Backcalculated Layer Moduli for Site #6 (N5S-6), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	21.88	17.15	11.43	9.43	6.80	4.21	2.93	2.13	1.64	124	14	19	3.20
	49.2	9.79	8.30	6.08	5.46	4.17	2.71	1.90	1.43	1.14	453	24	29	2.07
	98.4	18.41	14.76	11.03	8.82	6.34	3.89	2.75	2.08	1.61	168	15	20	2.92
	147.6	10.21	8.51	6.34	5.75	4.48	3.05	2.24	1.75	1.40	433	26	23	1.57
	196.8	12.11	9.37	6.93	6.14	4.67	3.09	2.20	1.68	1.33	311	24	24	0.88
	246.0	9.60	7.95	5.91	5.29	4.09	2.66	1.90	1.46	1.18	453	26	28	1.55
	295.2	13.04	10.79	7.88	7.06	5.47	3.71	2.76	2.14	1.71	307	22	19	1.88
	344.4	6.70	5.43	4.01	3.73	2.94	2.03	1.53	1.19	0.96	654	43	34	1.02
	393.6	9.76	8.06	6.10	5.67	4.50	3.14	2.37	1.85	1.47	495	28	22	1.05
	442.8	11.88	9.51	7.08	6.45	5.06	3.46	2.52	1.95	1.58	363	23	21	0.77
	492.0	12.03	9.38	6.16	4.97	3.70	2.50	1.90	1.53	1.23	186	32	27	4.18
COL	0	19.14	14.98	10.17	8.43	6.33	4.14	2.94	2.20	1.71	148	17	18	2.87
	98.4	14.73	11.66	8.15	7.08	5.29	3.52	2.56	1.95	1.54	216	21	21	2.26
	246	14.96	11.72	8.13	6.82	4.92	3.11	2.27	1.73	1.39	186	21	23	2.93
	393.6	11.20	9.20	6.62	6.00	4.63	3.20	2.39	1.86	1.50	348	26	22	1.93
	492	9.37	7.35	5.26	4.58	3.55	2.44	1.88	1.50	1.23	327	37	27	1.83
RWP	0	18.10	14.40	10.06	8.63	6.46	4.21	2.96	2.29	1.76	176	16	18	2.20
	98.4	14.79	12.37	8.53	7.57	5.49	3.60	2.58	1.96	1.55	231	19	21	3.39
	246	16.83	13.86	9.63	7.45	5.60	3.48	2.55	1.92	1.50	158	19	21	4.20
	393.6	13.26	10.40	7.13	6.15	4.64	3.14	2.38	1.86	1.51	212	26	22	2.59
	492	9.05	6.88	4.91	4.45	3.45	2.40	1.85	1.47	1.23	352	39	27	1.05

Table C-19 Deflection Data and Backcalculated Layer Moduli for Site #7 (N5N-7), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	10.19	8.63	7.39	5.97	4.90	3.72	2.87	2.41	2.03	849	33	15	1.35
	49.2	8.36	6.95	5.90	4.68	3.84	2.87	2.23	1.82	1.52	959	40	20	1.54
	98.4	6.88	5.57	4.46	3.43	2.90	2.42	2.01	1.70	1.49	631	72	20	3.01
	147.6	5.62	4.60	3.91	3.19	2.76	2.25	1.90	1.65	1.42	1,128	84	20	1.32
	196.8	9.34	6.81	5.27	3.92	3.24	2.51	2.04	1.75	1.49	338	55	21	1.70
	246.0	6.04	4.78	3.95	3.04	2.54	1.99	1.67	1.45	1.25	769	78	24	1.25
	295.2	7.53	5.99	5.02	4.04	3.24	2.51	2.03	1.70	1.49	806	54	21	1.31
	344.4	8.13	6.42	5.19	3.84	3.08	2.35	1.93	1.61	1.40	520	54	22	2.04
	393.6	6.46	5.32	4.52	3.56	2.95	2.38	1.94	1.64	1.46	968	66	21	1.94
	442.8	6.88	5.74	4.82	3.78	3.07	2.38	1.93	1.65	1.42	912	58	22	1.58
	492.0	7.15	5.75	4.78	3.74	3.03	2.34	1.82	1.68	1.36	753	59	22	2.23
COL	0	10.16	8.18	6.73	5.14	4.31	3.30	2.62	2.12	1.82	575	39	17	2.63
	98.4	6.07	4.55	3.67	2.87	2.54	2.17	1.85	1.59	1.43	544	101	20	2.48
	246	5.78	4.43	3.55	2.69	2.27	1.91	1.63	1.40	1.35	578	98	24	2.48
	393.6	6.21	4.90	4.00	3.07	2.61	2.10	1.77	1.59	1.44	664	83	22	1.75
	492	6.54	5.02	3.97	3.01	2.55	2.00	1.69	1.43	1.33	570	77	24	2.01
RWP	0	9.47	7.48	5.99	4.48	3.68	2.89	2.44	1.91	1.71	454	48	18	3.43
	98.4	7.78	5.20	3.80	2.85	2.44	2.03	1.71	1.45	1.31	248	86	24	2.75
	246	6.10	4.56	3.56	2.62	2.21	1.89	1.65	1.40	1.28	444	100	24	3.41
	393.6	7.73	5.46	4.09	2.94	2.27	1.80	1.57	1.32	1.23	296	74	28	3.40
	492	6.57	4.66	3.55	2.70	2.13	1.60	1.39	1.24	1.07	416	83	30	2.37

Table C-20 Deflection Data and Backcalculated Layer Moduli for Site #7 (N5N-7), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.01	6.60	5.16	3.83	3.10	2.44	1.99	1.63	1.49	366	55	22	2.33
	49.2	8.64	6.86	5.42	3.98	3.21	2.51	2.06	1.65	1.45	474	51	21	3.26
	98.4	8.23	6.62	5.37	4.11	3.28	2.59	2.07	1.80	1.55	572	53	20	2.13
	147.6	6.71	5.78	4.88	3.84	3.13	2.45	1.97	1.54	1.37	1,162	52	23	3.18
	196.8	6.55	5.52	4.65	3.73	3.07	2.40	1.94	1.64	1.41	1,101	59	21	1.58
	246.0	5.90	5.03	4.25	3.48	2.90	2.32	1.91	1.60	1.41	1,327	66	21	1.84
	295.2	5.47	4.53	3.95	3.16	2.60	2.05	1.68	1.45	1.31	1,309	74	24	0.98
	344.4	5.02	4.18	3.54	2.96	2.38	1.87	1.58	1.37	1.20	1,381	82	25	1.32
	393.6	5.08	4.07	3.39	2.69	2.31	1.90	1.62	1.40	1.26	978	100	23	1.63
	442.8	4.98	4.02	3.40	2.70	2.23	1.77	1.56	1.35	1.23	1,025	97	25	1.38
	492.0	6.71	5.21	4.06	3.00	2.32	1.86	1.55	1.27	1.17	514	71	28	3.30
COL	0	7.42	5.44	4.16	3.02	2.57	2.09	1.75	1.46	1.22	361	76	24	3.15
	98.4	7.61	5.89	4.82	3.63	2.89	2.26	1.88	1.56	1.40	561	60	23	1.90
	246	4.99	4.11	3.47	2.76	2.36	1.95	1.66	1.42	1.29	1,140	97	23	1.90
	393.6	4.87	3.86	3.18	2.51	2.14	1.75	1.51	1.30	1.18	918	107	25	1.59
	492	7.80	5.27	3.77	2.58	2.12	1.76	1.51	1.27	1.14	227	83	29	4.60
RWP	0	7.31	5.10	3.83	2.75	2.29	1.88	1.60	1.35	1.23	306	83	26	3.04
	98.4	9.70	6.71	4.89	3.41	2.80	2.38	1.98	1.66	1.45	200	65	22	4.34
	246	5.17	4.06	3.30	2.63	2.23	1.90	1.61	1.38	1.27	810	105	23	2.27
	393.6	5.13	3.98	3.17	2.48	2.07	1.71	1.47	1.26	1.16	722	105	27	2.03
	492	7.52	5.07	3.46	2.50	2.08	1.75	1.47	1.28	1.19	233	88	29	4.53

Table C-21 Deflection Data and Backcalculated Layer Moduli for Site #7 (N5N-7), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	4.65	3.95	3.42	2.72	2.27	1.80	1.52	1.30	1.17	1,524	89	26	1.42
	49.2	4.69	3.96	3.31	2.61	2.21	1.79	1.53	1.31	1.13	1,236	98	25	2.08
	98.4	6.03	4.82	3.95	3.08	2.56	1.99	1.66	1.43	1.30	820	75	24	1.27
	147.6	5.54	4.55	3.77	3.06	2.62	2.13	1.79	1.50	1.33	1,109	82	22	2.12
	196.8	6.47	5.00	3.97	3.08	2.57	2.17	1.84	1.56	1.39	555	84	21	2.59
	246.0	6.57	5.46	4.58	3.69	3.11	2.53	2.12	1.74	1.54	997	65	19	2.48
	295.2	7.36	5.91	4.87	3.84	3.28	2.70	2.32	1.96	1.72	650	69	17	2.13
	344.4	7.40	6.07	4.92	3.69	3.07	2.49	2.10	1.77	1.58	592	64	19	2.90
	393.6	7.14	5.61	4.67	3.66	3.03	2.40	2.01	1.67	1.50	709	64	20	1.73
	442.8	6.20	4.55	3.57	2.67	2.28	1.96	1.71	1.47	1.36	416	103	23	2.92
	492.0	5.36	4.08	3.29	2.56	2.16	1.82	1.56	1.35	1.23	655	106	25	1.95
COL	0	4.65	3.81	3.13	2.50	2.06	1.68	1.39	1.20	1.08	1,178	97	28	1.89
	98.4	6.27	4.89	3.94	2.89	2.35	1.97	1.69	1.36	1.24	544	83	25	3.75
	246	6.93	5.57	4.49	3.56	2.99	2.47	2.10	1.75	1.57	678	71	19	2.53
	393.6	6.32	5.20	4.39	3.46	2.93	2.38	1.98	1.66	1.51	953	70	20	2.17
	492	5.78	4.27	3.35	2.48	2.09	1.85	1.56	1.39	1.25	439	111	25	3.42
RWP	0	5.20	4.08	3.35	2.63	2.17	1.72	1.47	1.24	1.16	902	92	28	1.47
	98.4	7.11	5.23	4.10	3.07	2.46	1.98	1.69	1.43	1.27	411	77	25	2.13
	246	7.26	5.44	4.32	3.47	2.98	2.52	2.14	1.77	1.59	506	77	18	2.82
	393.6	5.91	4.58	3.65	2.83	2.38	2.00	1.73	1.46	1.33	600	94	23	2.56
	492	6.74	4.50	3.12	2.19	1.91	1.65	1.45	1.28	1.17	231	111	28	5.68

Table C-22 Deflection Data and Backcalculated Layer Moduli for Site #8 (N5N-8), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	16.76	14.95	12.90	10.37	7.90	5.03	3.36	2.44	2.03	354	13	11	2.85
	49.2	19.57	16.64	14.27	11.38	8.74	5.53	3.61	2.56	2.11	293	12	10	1.55
	98.4	15.81	13.21	11.54	9.18	7.16	4.77	3.30	2.39	1.98	361	16	11	0.97
	147.6	13.51	11.41	9.89	8.00	6.34	4.24	2.92	2.12	1.79	455	18	12	0.82
	196.8	11.53	9.64	8.46	6.84	5.47	3.78	2.66	1.97	1.67	532	23	13	0.68
	246.0	12.65	10.44	9.06	7.25	5.65	3.83	2.64	1.90	1.61	449	20	14	1.00
	295.2	16.32	13.37	11.49	9.14	6.98	4.46	2.94	2.10	1.76	329	15	13	1.03
	344.4	13.60	11.28	9.84	7.91	6.18	4.16	2.83	2.00	1.65	448	18	13	0.96
	393.6	14.85	12.33	10.61	8.43	6.35	4.06	2.69	1.91	1.64	360	16	14	1.46
	442.8	14.01	11.78	10.09	7.91	6.11	3.96	2.63	1.89	1.57	384	17	14	1.34
	492.0	14.38	11.88	10.08	7.79	5.95	3.78	2.52	1.79	1.54	340	18	15	1.26
COL	0	19.69	16.07	13.98	11.07	8.24	5.02	3.26	2.46	2.10	249	13	11	2.84
	98.4	12.74	10.80	9.48	7.75	6.17	4.25	3.00	2.23	1.88	500	20	11	0.88
	246	13.05	11.04	9.66	7.69	5.89	3.83	2.62	1.90	1.60	435	19	14	1.60
	393.6	11.82	9.98	8.75	7.03	5.53	3.70	2.52	1.86	1.58	513	21	14	1.11
	492	9.20	8.04	7.17	5.98	4.82	3.31	2.30	1.66	1.41	876	23	16	0.89
RWP	0	22.04	18.66	15.42	11.89	8.81	5.00	3.27	2.34	2.00	198	11	12	3.37
	98.4	10.61	9.44	8.48	7.14	5.89	4.28	3.00	2.17	1.85	903	19	12	1.09
	246	12.35	10.08	8.74	6.92	5.46	3.64	2.47	1.79	1.54	449	21	14	0.63
	393.6	10.94	9.52	8.39	6.85	5.51	3.77	2.59	1.86	1.55	671	20	14	0.98
	492	11.05	9.28	8.19	6.56	5.33	3.69	2.48	1.72	1.45	653	20	15	1.49

Table C-23 Deflection Data and Backcalculated Layer Moduli for Site #8 (N5N-8), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.21	8.11	7.19	6.01	4.85	3.35	2.37	1.71	1.48	940	20	25	1.05
	49.2	10.32	8.88	7.71	6.30	5.03	3.48	2.49	1.82	1.52	687	21	23	1.38
	98.4	13.81	11.81	10.22	8.32	6.56	4.41	3.01	2.14	1.73	514	14	20	1.24
	147.6	11.30	10.01	8.86	7.46	5.98	4.29	3.03	2.20	1.84	794	16	20	1.53
	196.8	22.80	18.64	15.31	10.82	7.55	4.02	2.49	1.84	1.57	156	11	22	3.58
	246.0	25.80	21.73	18.15	13.05	7.83	4.07	2.60	1.95	1.66	117	10	21	7.36
	295.2	27.61	22.86	18.67	13.09	8.79	4.37	2.76	2.05	1.79	116	9	20	5.41
	344.4	10.51	9.16	8.04	6.61	5.23	3.53	2.43	1.79	1.49	708	18	24	1.12
	393.6	10.89	9.50	8.24	6.65	5.33	3.56	2.52	1.81	1.49	651	19	23	1.42
	442.8	9.98	8.79	7.79	6.50	5.33	3.70	2.68	1.95	1.64	869	19	22	1.04
	492.0	15.81	13.48	11.49	9.28	7.27	4.61	3.02	2.10	1.74	422	12	20	0.89
COL	0	7.23	6.64	6.01	5.12	4.29	3.13	2.28	1.66	1.37	1,629	21	27	1.24
	98.4	12.40	10.53	9.27	7.51	6.09	4.12	2.85	2.09	1.75	591	16	20	0.65
	246	26.22	22.35	18.98	13.49	9.08	4.36	2.75	2.01	1.73	134	9	20	6.42
	393.6	11.36	9.56	8.35	6.73	5.36	3.58	2.41	1.75	1.45	609	18	24	0.79
	492	15.04	12.87	11.02	8.78	6.83	4.31	2.88	2.02	1.68	421	13	21	1.20
RWP	0	8.20	7.50	6.74	5.73	4.75	3.42	2.44	1.78	1.45	1,340	19	26	1.19
	98.4	13.62	11.90	10.58	8.44	6.54	4.33	2.88	2.08	1.73	520	14	21	1.51
	246	36.62	31.28	25.80	17.38	11.18	5.09	3.07	2.26	1.89	83	7	18	7.55
	393.6	17.87	14.98	12.59	9.82	7.38	4.42	2.82	1.96	1.61	301	12	21	1.36
	492	15.82	13.77	11.90	9.46	7.21	4.46	2.80	2.01	1.68	399	12	22	2.01

Table C-24 Deflection Data and Backcalculated Layer Moduli for Site #8 (N5N-8), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	13.28	11.51	10.12	8.41	6.74	4.49	3.05	2.12	1.75	584	15	13	0.74
	49.2	13.03	11.30	10.07	8.29	6.66	4.45	2.99	2.07	1.67	615	15	13	0.65
	98.4	12.33	10.58	9.37	7.87	6.39	4.43	3.11	2.20	1.77	659	17	12	0.69
	147.6	13.14	11.64	10.19	8.37	6.64	4.46	3.01	2.13	1.75	567	15	13	1.45
	196.8	17.11	15.23	13.39	11.06	8.29	5.24	3.23	2.12	1.68	430	10	14	2.41
	246.0	10.49	9.08	8.08	6.75	5.53	3.90	2.74	1.95	1.58	804	20	14	0.81
	295.2	14.04	11.97	10.45	8.57	6.88	4.70	3.24	2.22	1.75	525	15	12	1.23
	344.4	21.53	18.36	16.03	12.26	8.95	5.25	3.40	2.46	2.04	222	11	11	3.46
	393.6	13.56	11.71	10.25	8.38	6.74	4.36	2.91	2.05	1.71	528	15	13	0.93
	442.8	23.15	19.06	16.56	12.70	9.72	5.91	3.64	2.39	1.90	244	9	11	0.67
	492.0	21.83	17.55	14.25	10.44	7.67	4.47	2.79	2.01	1.66	170	12	13	2.35
COL	0	12.00	11.01	9.73	8.06	6.50	4.31	2.78	1.99	1.66	707	14	15	2.33
	98.4	10.62	9.56	8.68	7.32	6.09	4.32	3.04	2.17	1.74	974	17	13	0.77
	246	8.41	7.50	6.79	5.77	4.80	3.53	2.59	1.88	1.57	1,196	25	14	1.06
	393.6	15.97	13.18	11.39	8.94	6.89	4.35	2.79	1.98	1.69	344	15	13	1.08
	492	16.08	13.09	11.26	8.86	6.79	4.16	2.65	1.87	1.60	330	15	14	1.12
RWP	0	20.34	17.09	14.66	11.49	8.63	5.02	2.99	2.05	1.72	269	10	14	2.34
	98.4	17.02	14.07	12.37	9.95	7.94	5.08	3.31	2.27	1.79	394	12	12	0.22
	246	9.32	8.28	7.40	6.22	5.11	3.66	2.61	1.82	1.50	1,019	21	15	1.34
	393.6	19.67	16.73	14.33	11.01	8.26	4.82	2.96	2.01	1.68	264	11	14	2.20
	492	15.58	13.03	11.20	8.91	6.90	4.30	2.78	1.90	1.58	378	14	14	0.83

Table C-25 Deflection Data and Backcalculated Layer Moduli for Site #9 (MNN-9), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	13.64	9.64	6.38	3.88	2.22	0.86	0.47	0.34	0.28	570	14	44	1.99
	49.2	7.41	4.81	3.08	1.89	1.11	0.50	0.27	0.21	0.17	858	33	80	4.57
	98.4	6.11	3.81	1.97	1.11	0.74	0.49	0.32	0.24	0.16	294	86	88	14.43
	147.6	6.95	4.40	2.69	1.50	0.90	0.43	0.27	0.19	0.13	580	46	90	2.35
	196.8	6.22	3.86	2.27	1.29	0.77	0.38	0.22	0.18	0.13	614	53	103	5.87
COL	0.0	9.14	6.59	4.47	3.06	2.16	1.35	0.94	0.68	0.47	361	95	31	11.39
	49.2	11.01	7.61	5.04	3.06	1.92	1.00	0.64	0.44	0.31	349	38	40	4.86
	98.4	7.57	5.11	3.12	2.05	1.49	0.98	0.68	0.48	0.35	216	138	45	14.03
	147.6	7.00	4.78	3.14	2.06	1.41	0.76	0.41	0.22	0.13	874	49	62	8.09
	196.8	6.12	3.89	2.39	1.38	0.85	0.44	0.27	0.21	0.17	559	63	91	5.77

Table C-26 Deflection Data and Backcalculated Layer Moduli for Site #9 (MNN-9), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	5.72	3.75	2.47	1.57	1.04	0.54	0.34	0.25	0.20	959	57	90	1.91
	49.2	8.11	5.19	3.02	1.56	0.91	0.45	0.29	0.21	0.17	552	32	100	4.93
	98.4	10.52	7.25	4.55	2.62	1.51	0.64	0.36	0.26	0.21	758	17	73	7.18
	147.6	10.71	6.99	3.97	1.94	1.13	0.48	0.31	0.21	0.16	534	20	89	9.93
	196.8	12.47	8.81	5.55	3.20	1.89	0.77	0.47	0.35	0.25	584	15	57	4.27
	246.0	7.22	4.83	2.98	1.84	1.19	0.66	0.45	0.35	0.26	514	55	71	4.54
COL	0.0	5.98	3.98	2.49	1.52	0.99	0.54	0.36	0.27	0.21	670	63	88	2.69
	49.2	8.89	5.73	3.40	1.85	1.15	0.60	0.42	0.31	0.24	409	37	75	2.92
	98.4	9.85	6.85	4.32	2.61	1.56	0.68	0.40	0.28	0.21	806	19	69	5.59
	147.6	9.11	6.26	4.05	2.32	1.41	0.68	0.42	0.29	0.18	739	25	70	4.70
	196.8	8.99	6.22	4.09	2.61	1.72	0.93	0.64	0.47	0.34	497	44	51	3.26
	246	7.14	4.73	2.98	1.91	1.30	0.79	0.56	0.43	0.32	538	68	62	7.82

Table C-27 Deflection Data and Backcalculated Layer Moduli for Site #9 (MNN-9), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	4.43	2.69	1.67	0.99	0.64	0.43	0.34	0.29	0.23	630	108	126	5.73
	32.8	5.62	3.73	2.53	1.78	1.33	0.88	0.64	0.46	0.33	523	130	64	4.19
	65.6	8.25	5.88	4.13	2.88	1.94	1.08	0.71	0.50	0.37	763	46	47	0.69
	98.4	7.83	5.31	3.53	2.24	1.44	0.75	0.48	0.34	0.28	692	41	64	0.73
	131.2	5.82	3.85	2.47	1.55	0.99	0.51	0.30	0.21	0.15	866	55	93	0.89
	164.0	4.89	3.26	2.15	1.43	1.00	0.52	0.30	0.21	0.15	1,056	78	94	1.89
	196.8	15.60	8.70	4.48	1.60	0.53	0.16	0.11	0.09	0.06	252	14	134	4.56
	229.6	7.03	4.53	2.75	1.51	0.76	0.24	0.15	0.13	0.10	919	25	146	1.80
COL	0.0	4.50	2.85	1.65	0.88	0.55	0.34	0.29	0.25	0.21	587	88	147	5.25
	32.8	7.38	5.08	3.46	2.30	1.60	0.96	0.66	0.47	0.36	548	68	54	2.32
	65.6	7.87	5.72	3.89	2.68	1.83	0.98	0.64	0.46	0.36	834	44	51	1.24
	98.4	7.88	5.48	3.69	2.47	1.64	0.89	0.58	0.41	0.32	707	46	55	0.83
	131.2	5.55	3.32	1.84	0.90	0.48	0.22	0.15	0.12	0.11	715	46	180	2.68
	164	4.43	2.80	1.72	1.10	0.71	0.32	0.18	0.13	0.11	1,322	61	140	3.22
	196.8	14.64	6.60	3.04	1.65	1.11	0.55	0.31	0.19	0.16	126	31	75	5.82
	229.6	9.18	6.37	4.49	3.03	1.98	0.89	0.43	0.21	0.10	954	24	53	1.92

Table C-28 Deflection Data and Backcalculated Layer Moduli for Site #10 (MJS-10), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	9.20	7.31	5.79	4.92	3.93	2.74	1.97	1.48	1.16	361	60	22	1.55
	32.8	7.46	6.09	5.07	4.49	3.61	2.58	1.83	1.35	1.01	660	59	24	0.87
	65.6	30.92	26.22	21.59	16.96	12.12	7.60	4.94	3.36	2.36	145	6	9	2.89
	98.4	35.09	29.86	25.05	19.18	13.55	6.88	3.80	2.38	1.68	139	2	11	2.63
	131.2	16.78	13.61	10.62	8.67	6.26	3.59	2.19	1.47	1.03	270	9	19	1.37
	164.0	7.40	5.33	3.94	2.84	1.78	0.72	0.35	0.25	0.22	431	16	94	1.10
	196.8	5.05	3.69	2.76	2.11	1.53	0.90	0.55	0.36	0.23	705	39	74	3.19
	229.6	6.94	5.24	3.91	3.12	2.28	1.30	0.77	0.46	0.29	651	19	56	3.92
	262.4	13.67	10.88	8.66	7.26	5.60	3.80	2.59	1.84	1.33	288	26	16	1.35
COL	0.0	6.78	5.46	4.47	4.04	3.40	2.62	2.02	1.58	1.23	477	156	21	1.61
	32.8	8.47	7.11	5.92	5.13	4.05	2.73	1.90	1.38	1.05	638	34	23	1.10
	62.32	17.63	14.35	11.77	9.98	7.68	4.94	3.25	2.18	1.55	315	10	14	1.23
	98.4	16.29	13.95	11.80	10.53	8.45	6.05	4.27	3.02	2.14	422	17	10	1.25
	131.2	18.08	15.34	12.44	10.51	7.94	5.12	3.30	2.16	1.42	326	8	14	1.96
	164	7.78	6.32	5.14	4.44	3.48	2.35	1.61	1.16	0.83	612	40	27	0.82
	196.8	5.79	4.17	3.10	2.27	1.58	0.86	0.51	0.33	0.24	562	31	77	3.41
	229.6	7.34	5.70	4.38	3.36	2.40	1.27	0.71	0.42	0.27	624	15	60	2.81
	262.4	13.48	10.91	8.18	6.29	4.18	2.07	1.07	0.60	0.36	327	7	39	3.33

Table C-29 Deflection Data and Backcalculated Layer Moduli for Site #10 (MJS-10), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	21.24	17.95	15.07	12.56	9.91	6.35	4.22	2.75	1.83	304	7	11	1.44
	32.8	22.93	19.43	15.94	13.01	9.69	5.51	3.11	1.83	1.14	271	3	16	1.26
	65.6	7.32	5.42	3.83	2.52	1.48	0.52	0.22	0.15	0.14	411	15	130	2.99
	98.4	13.60	11.12	8.67	6.53	4.51	2.34	1.29	0.78	0.52	324	7	32	2.54
	131.2	9.61	7.67	5.87	4.55	3.24	1.83	1.11	0.75	0.55	408	17	36	1.79
	164.0	9.12	7.06	5.38	4.13	3.26	2.20	1.59	1.21	0.89	277	59	27	3.61
	196.8	6.82	4.99	3.63	2.68	1.88	1.07	0.73	0.57	0.44	341	47	54	3.53
	229.6	7.48	5.95	4.63	3.80	2.82	1.77	1.16	0.83	0.59	517	34	36	1.40
	262.4	6.15	4.45	3.34	2.62	1.94	1.21	0.80	0.54	0.36	484	50	52	2.23
COL	0.0	20.34	17.38	14.28	12.26	9.70	6.32	4.14	2.78	1.85	324	7	11	1.02
	32.8	21.89	18.07	14.13	11.61	8.14	4.63	2.84	1.84	1.20	212	6	15	2.32
	65.6	8.99	6.42	4.46	2.96	1.83	0.75	0.40	0.28	0.19	308	16	82	2.76
	98.4	7.75	5.86	4.46	3.46	2.55	1.54	0.95	0.61	0.40	507	24	44	3.30
	131.2	10.15	7.56	5.42	3.93	2.78	1.58	0.99	0.68	0.49	277	23	40	2.77
	164.0	6.04	4.69	3.72	3.22	2.58	1.91	1.43	1.10	0.82	452	137	30	2.41
	196.8	4.73	3.48	2.63	2.11	1.59	1.01	0.68	0.49	0.36	608	77	61	0.84
	229.6	8.29	6.49	5.07	4.14	3.03	1.87	1.22	0.86	0.61	464	29	34	1.28
	262.4	5.17	3.85	2.95	2.39	1.79	1.08	0.67	0.46	0.32	752	41	61	1.63

Table C-30 Deflection Data and Backcalculated Layer Moduli for Site #11 (MLN-11), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	12.40	9.95	7.82	6.29	4.53	2.51	1.48	0.95	0.71	842	25	18	1.64
	32.8	11.81	9.37	7.37	5.95	4.32	2.43	1.45	0.99	0.73	777	31	19	2.85
	65.6	20.57	15.12	10.55	7.11	4.49	1.82	0.79	0.49	0.39	399	10	26	2.68
	98.4	19.36	15.02	11.26	8.42	5.46	2.27	0.99	0.58	0.41	537	9	22	1.66
	131.2	11.10	9.39	7.89	7.17	5.88	4.17	2.93	2.10	1.53	1,280	58	10	1.09
	164.0	21.94	16.43	12.39	9.58	6.70	3.57	2.08	1.40	1.00	325	16	12	2.63
	196.8	19.22	14.86	11.46	8.85	6.05	3.04	1.60	0.97	0.64	541	12	16	1.18
	229.6	11.87	9.01	6.78	5.09	3.47	1.79	1.01	0.67	0.49	642	26	25	2.26
	262.4	11.47	8.99	7.07	6.14	4.77	3.07	2.02	1.39	0.98	749	48	14	1.56

Table C-31 Deflection Data and Backcalculated Layer Moduli for Site #11 (MLN-11), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	55.77	43.79	31.03	20.60	12.44	3.54	1.24	0.80	0.59	169	2	16	2.43
	32.8	19.64	15.96	12.55	9.84	6.78	3.20	1.42	0.75	0.54	818	6	24	1.13
	65.6	14.31	12.10	9.82	8.13	5.90	3.37	1.92	1.17	0.76	1,252	11	19	1.17
	98.4	9.73	7.12	5.29	4.32	3.29	2.35	1.78	1.36	1.05	317	89	21	4.74
	131.2	11.67	9.09	6.97	5.12	3.45	1.79	1.04	0.72	0.51	779	21	31	0.97
	164.0	8.43	5.98	4.29	3.10	2.07	1.01	0.58	0.40	0.29	917	30	54	1.73
	196.8	11.69	9.10	6.94	5.53	4.05	2.28	1.39	0.94	0.70	904	24	25	0.72
	229.6	17.86	13.51	9.88	7.76	5.44	3.14	2.07	1.49	1.18	373	21	16	1.81
	262.4	36.24	28.08	20.44	13.31	7.77	2.42	0.82	0.55	0.47	263	4	24	1.70

Table C-32 Deflection Data and Backcalculated Layer Moduli for Site #12 (M2S-12), Section 1

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	3.76	2.98	2.40	1.70	1.20	0.61	0.37	0.26	0.21	2,986	49	131	1.83
	49.2	3.91	3.18	2.63	1.90	1.34	0.73	0.46	0.35	0.27	2,800	52	105	0.67
	98.4	3.83	3.08	2.46	1.73	1.19	0.59	0.34	0.24	0.19	2,914	45	138	1.68
	147.6	3.34	2.63	2.13	1.46	1.04	0.53	0.34	0.25	0.20	2,957	63	141	1.38
	196.8	3.81	2.95	2.32	1.59	1.08	0.55	0.35	0.27	0.22	2,227	59	132	0.60
	246.0	4.20	3.31	2.63	1.79	1.26	0.69	0.46	0.35	0.28	1,972	60	105	1.24
	295.2	4.47	3.52	2.74	1.85	1.27	0.62	0.41	0.32	0.27	1,787	51	113	1.40
	344.4	3.10	2.44	1.95	1.34	0.92	0.50	0.33	0.26	0.23	2,631	80	143	0.92
	393.6	2.47	1.82	1.36	0.88	0.62	0.36	0.27	0.22	0.20	1,855	150	182	2.37
	442.8	4.89	4.05	3.43	2.69	2.15	1.52	1.17	0.94	0.79	1,825	94	44	0.93
	492.0	3.75	2.86	2.29	1.66	1.26	0.81	0.59	0.46	0.39	1,805	102	86	0.53

Table C-33 Deflection Data and Backcalculated Layer Moduli for Site #12 (M2S-12), Section 2

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	3.38	2.69	2.22	1.66	1.26	0.81	0.56	0.43	0.34	2,594	104	80	0.65
	49.2	3.99	3.30	2.75	2.11	1.62	1.05	0.76	0.60	0.55	2,295	94	59	1.60
	98.4	2.69	2.18	1.88	1.64	1.18	0.88	0.70	0.59	0.50	3,148	214	65	4.73
	147.6	3.48	2.92	2.46	1.94	1.54	1.07	0.81	0.63	0.55	2,670	131	57	1.23
	196.8	3.72	3.00	2.49	1.92	1.51	1.06	0.80	0.66	0.56	1,665	147	56	2.33
	246.0	6.34	4.77	3.91	2.83	2.11	1.34	0.97	0.76	0.65	983	66	46	1.08
	295.2	6.55	4.80	3.65	2.43	1.67	0.92	0.63	0.49	0.43	851	50	67	0.90
	344.4	6.11	4.54	3.52	2.33	1.60	0.81	0.49	0.35	0.29	1,316	40	84	2.46
	393.6	6.12	4.54	3.46	2.25	1.51	0.73	0.42	0.30	0.24	1,312	37	94	2.99
	442.8	6.61	4.75	3.53	2.27	1.50	0.69	0.42	0.30	0.24	1,082	37	94	2.83
	492.0	2.90	2.21	1.73	1.15	0.85	0.50	0.35	0.27	0.22	1,993	124	125	1.48

Table C-34 Deflection Data and Backcalculated Layer Moduli for Site #12 (M2S-12), Section 3

Location within Lane	Station (ft)	Deflections (mils) at the indicated sensor Spacing (inches)									Backcalculated Modulus (ksi)			RMS (%)
		0	7.87	11.81	17.72	23.62	35.43	47.24	59.06	70.87				
		D0	D1	D2	D3	D4	D5	D6	D7	D8	AC	Base/ Subbase	Roadbed Soil	
LWP	0.0	6.68	4.91	3.80	2.47	1.64	0.83	0.54	0.41	0.38	1,062	38	85	1.69
	49.2	5.90	4.43	3.41	2.25	1.52	0.83	0.57	0.46	0.41	1,040	50	82	0.99
	98.4	3.78	2.80	2.18	1.48	1.07	0.63	0.44	0.35	0.30	1,631	88	110	0.76
	147.6	6.97	5.20	4.01	2.63	1.82	0.99	0.66	0.52	0.46	949	41	71	1.16
	196.8	6.75	5.00	3.91	2.68	1.89	1.10	0.77	0.61	0.52	943	48	63	0.27
	246.0	8.41	6.25	4.81	3.28	2.30	1.33	0.95	0.75	0.64	711	39	51	0.61
	295.2	4.77	3.51	2.72	1.92	1.38	0.89	0.68	0.57	0.50	967	93	74	2.82
	344.4	2.88	2.16	1.73	1.29	1.03	0.73	0.59	0.49	0.44	1,425	216	89	2.65
	393.6	3.29	2.54	2.04	1.56	1.27	0.98	0.78	0.64	0.56	1,173	219	67	2.44
	442.8	3.12	2.41	1.97	1.44	1.15	0.84	0.66	0.55	0.48	1,480	193	79	2.45
	492.0	3.87	3.03	2.45	1.79	1.38	0.96	0.73	0.59	0.52	1,449	125	71	1.57

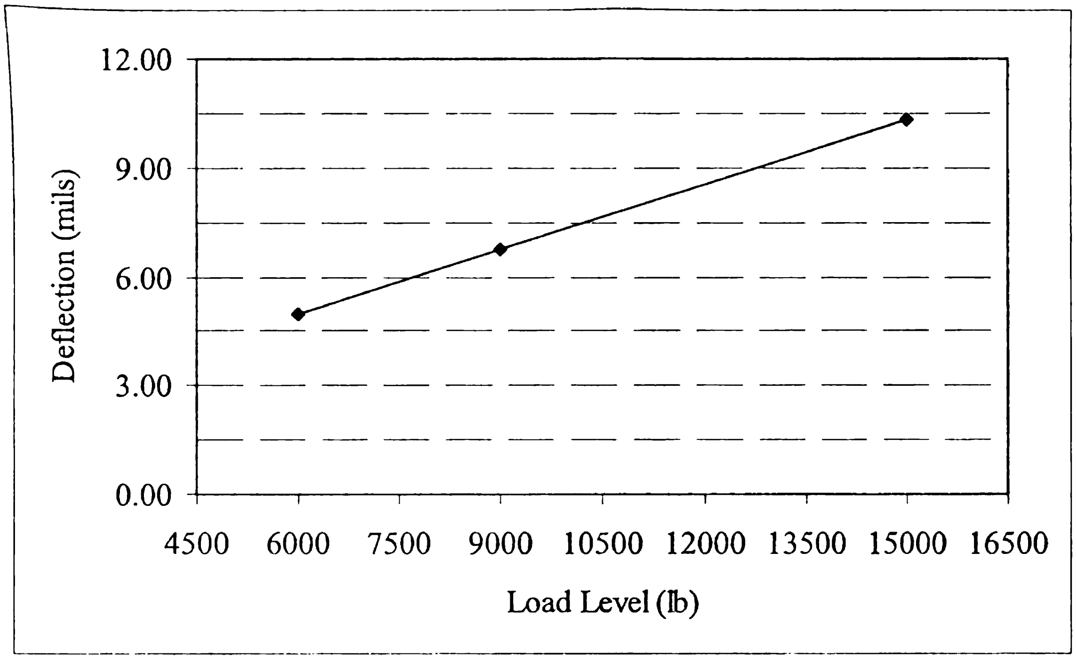


Figure C-1 Deflection Data vs. Load Level Linearity Plot for Site #2

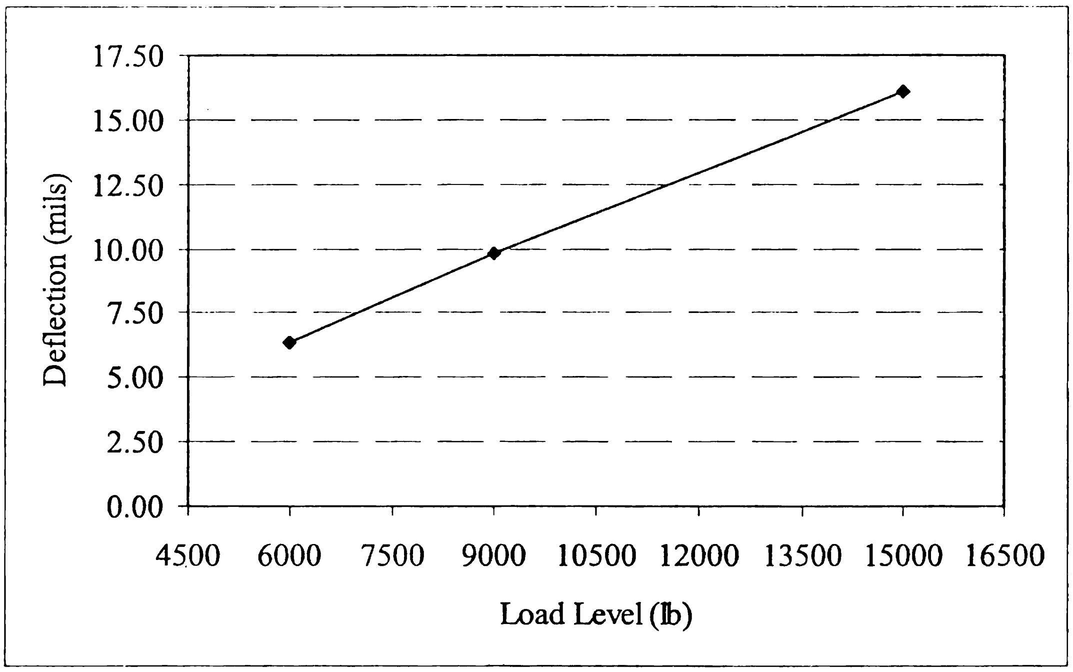


Figure C-2 Deflection Data vs. Load Level Linearity Plot for Site #3

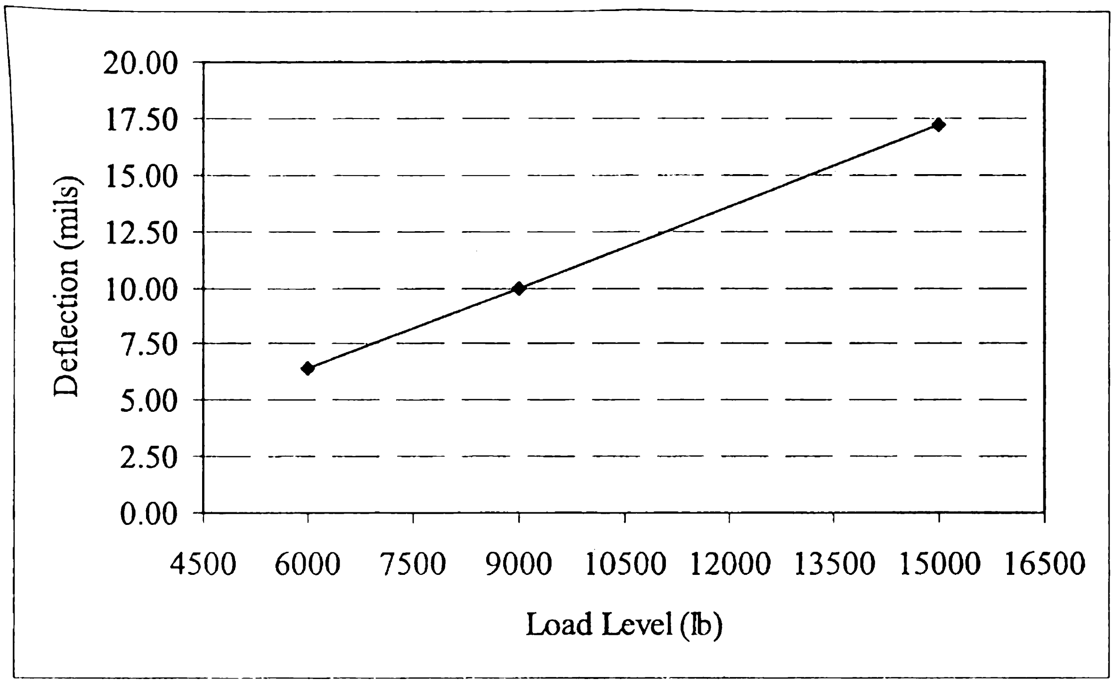


Figure C-3 Deflection Data vs. Load Level Linearity Plot for Site #4

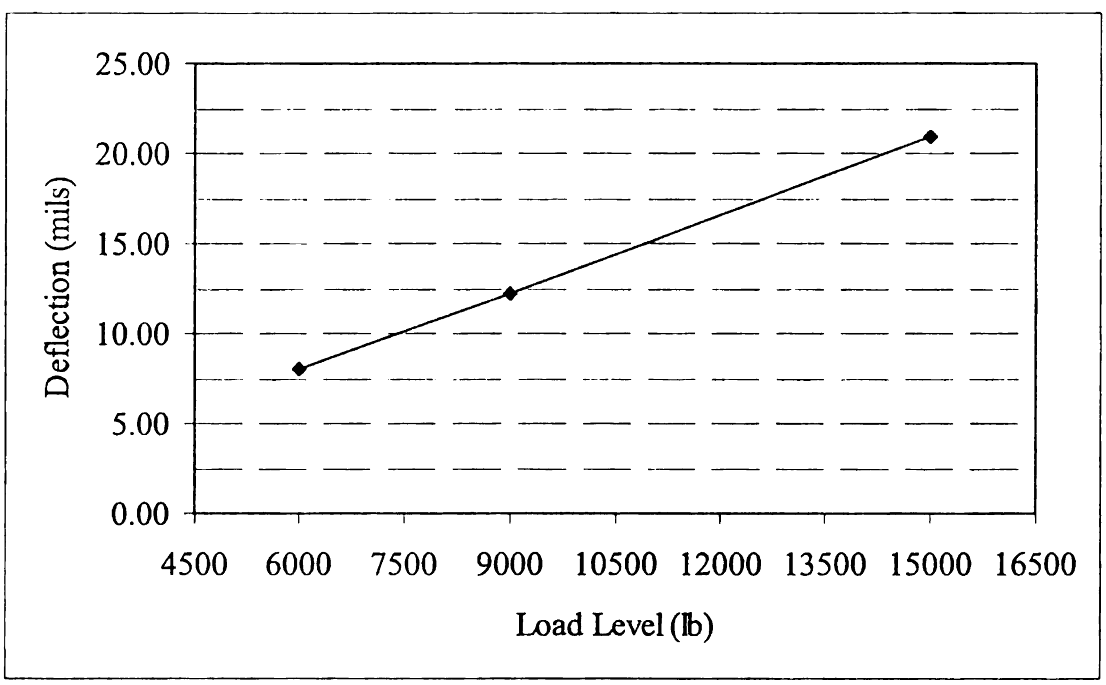


Figure C-4 Deflection Data vs. Load Level Linearity Plot for Site #5

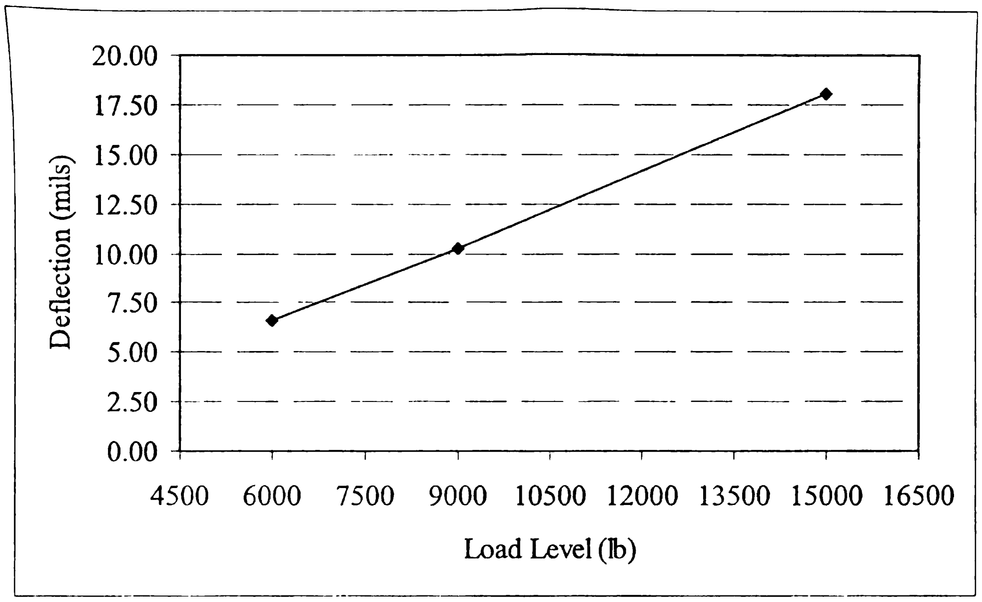


Figure C-5 Deflection Data vs. Load Level Linearity Plot for Site #6

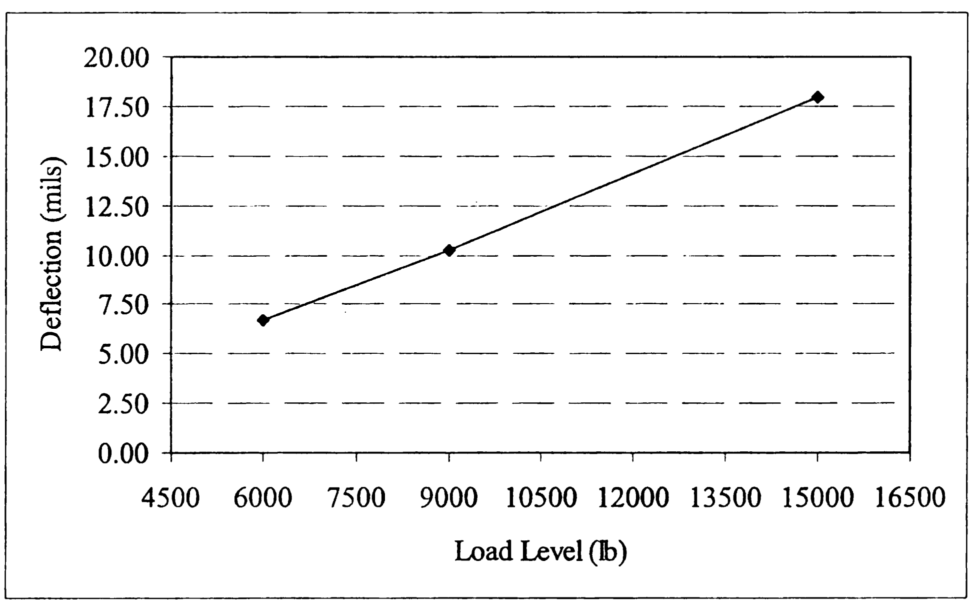


Figure C-6 Deflection Data vs. Load Level Linearity Plot for Site #7

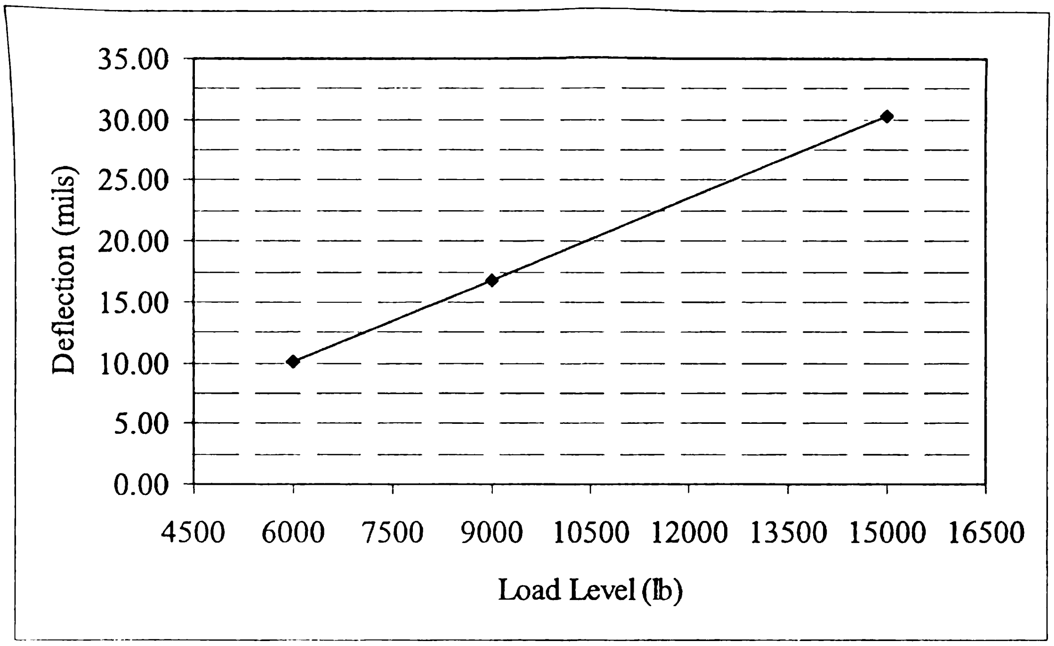


Figure C-7 Deflection Data vs. Load Level Linearity Plot for Site #8

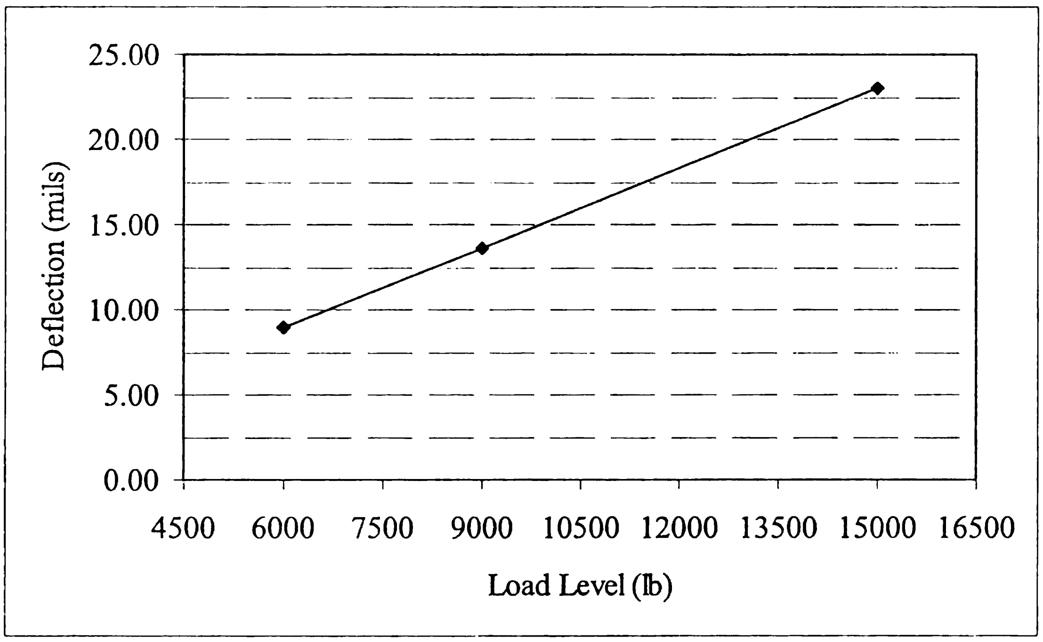


Figure C-8 Deflection Data vs. Load Level Linearity Plot for Site #9

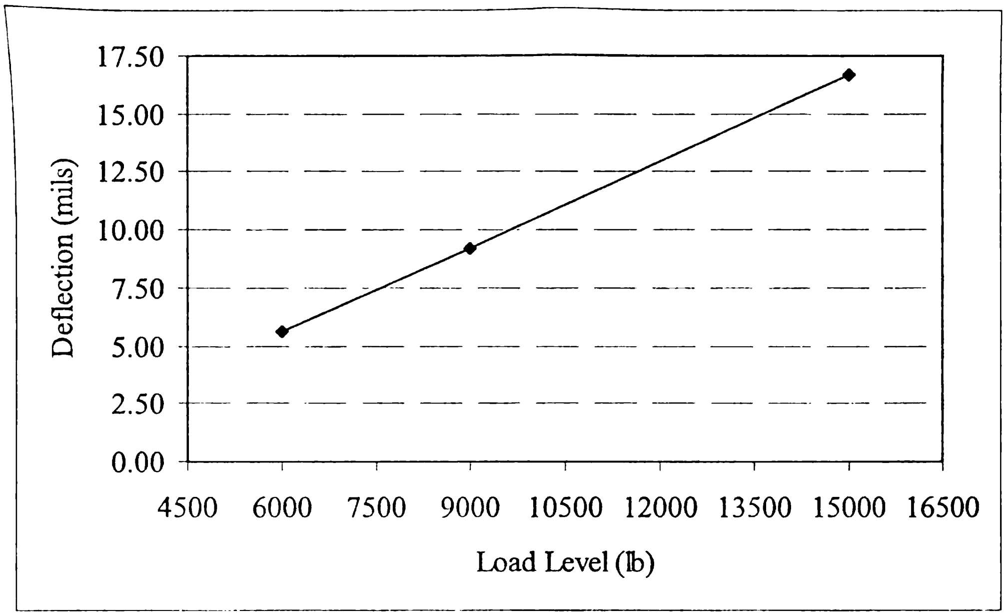


Figure C-9 Deflection Data vs. Load Level Linearity Plot for Site 10

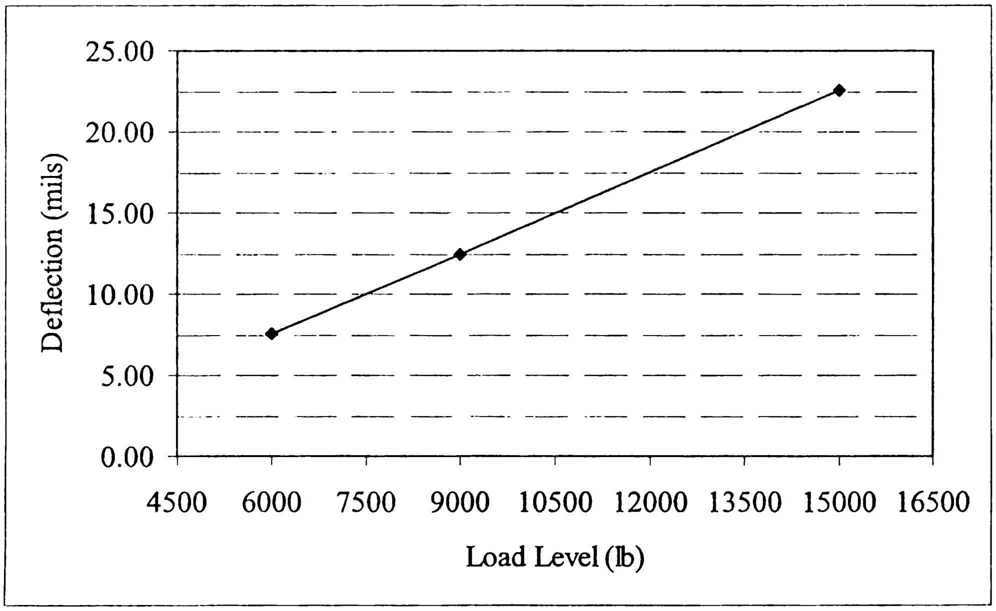


Figure C-10 Deflection Data vs. Load Level Linearity Plot for Site #11

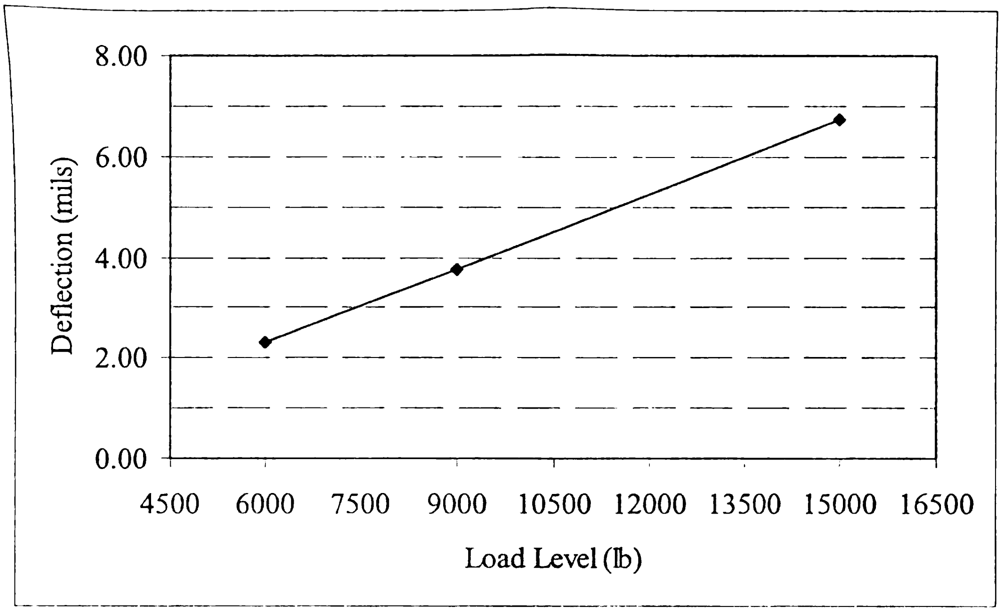


Figure C-11 Deflection Data vs. Load Level Linearity Plot for Site #12

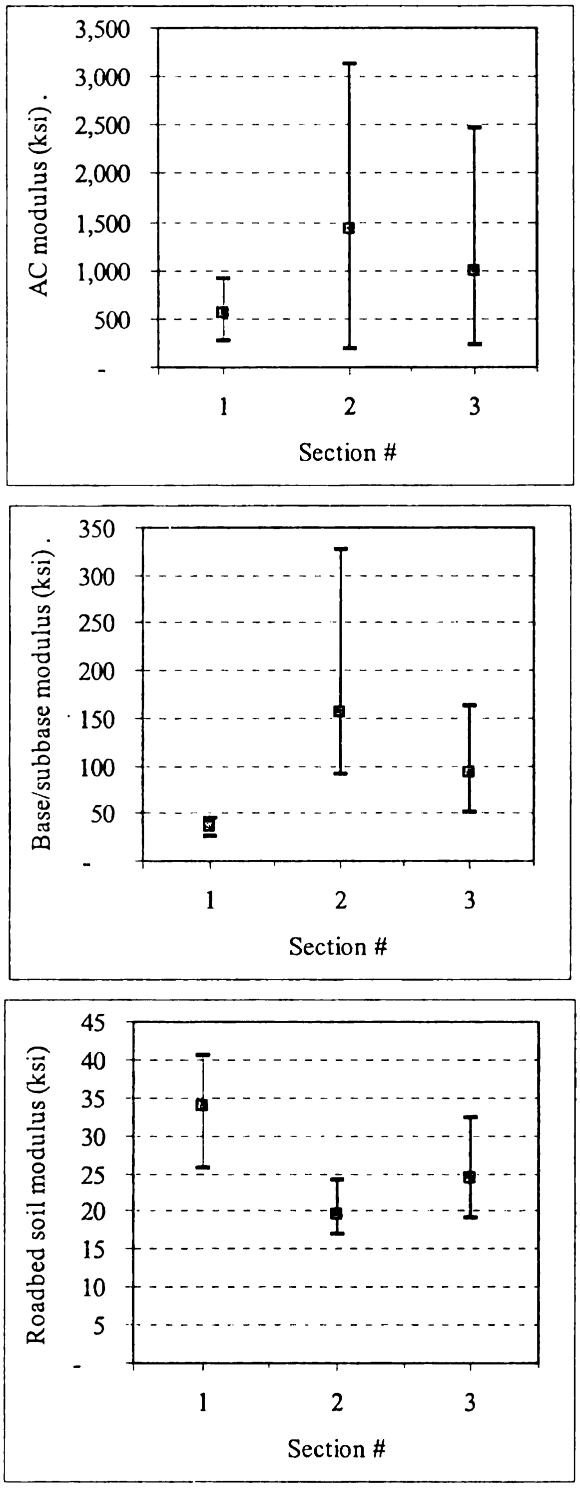


Figure C-12 Variability of the Backcalculated Moduli for Site #1

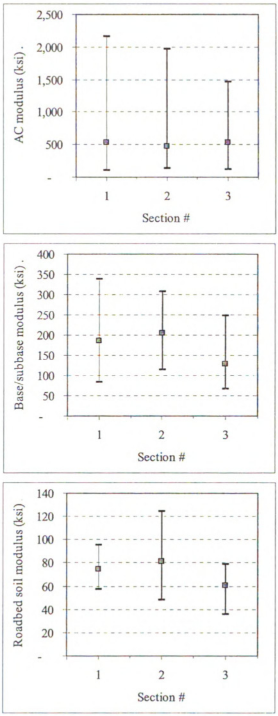


Figure C-13 Variability of the Backcalculated Moduli for Site #2

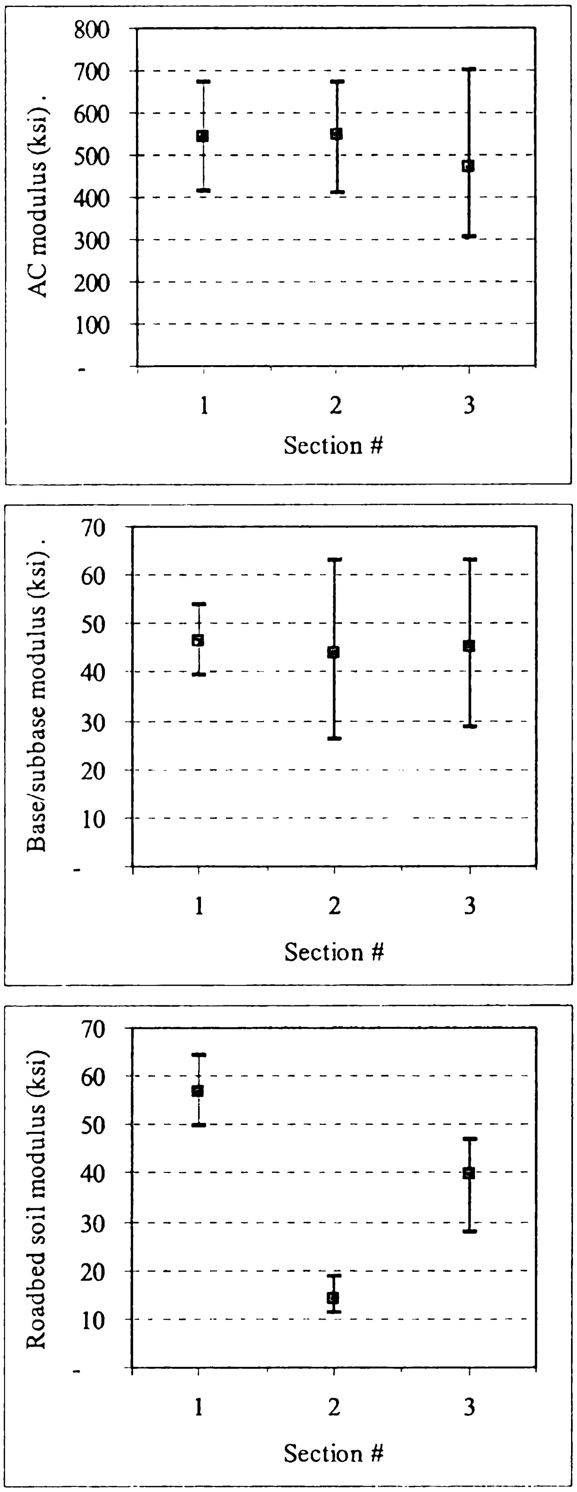


Figure C-14 Variability of the Backcalculated Moduli for Site #3

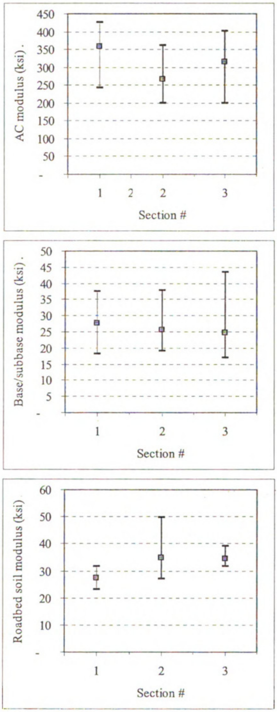


Figure C-15 Variability of the Backcalculated Moduli for Site #4

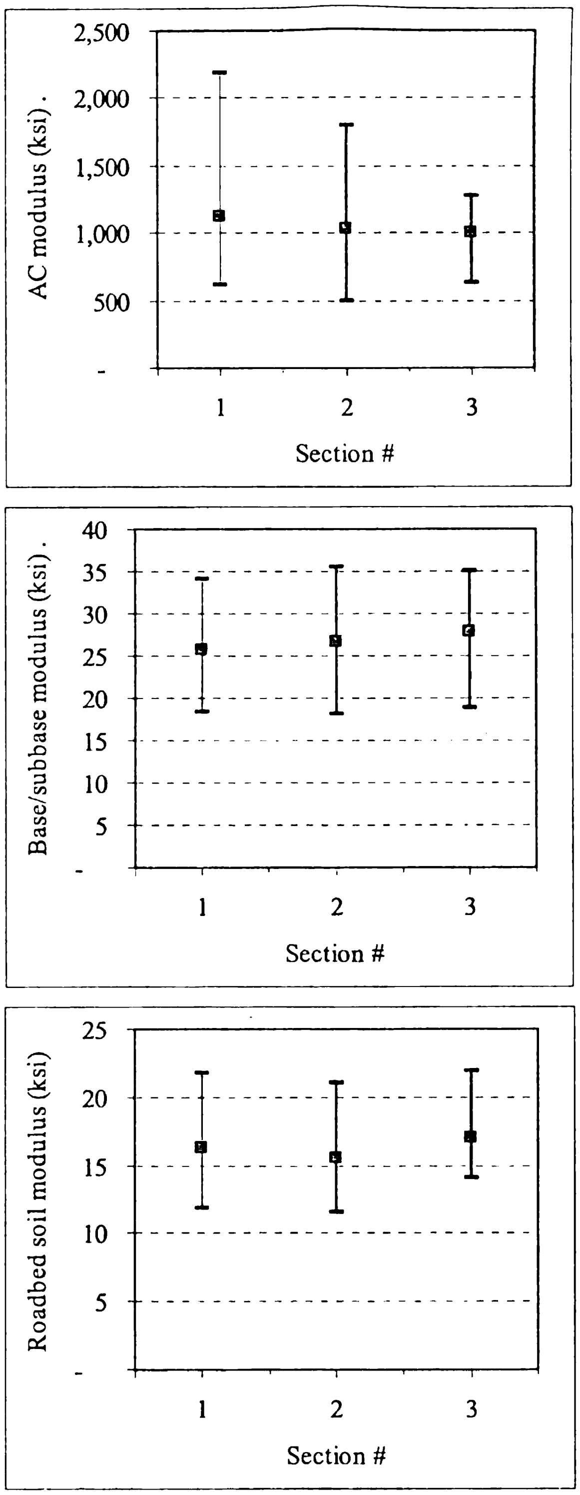


Figure C-16 Variability of the Backcalculated Moduli for Site #5

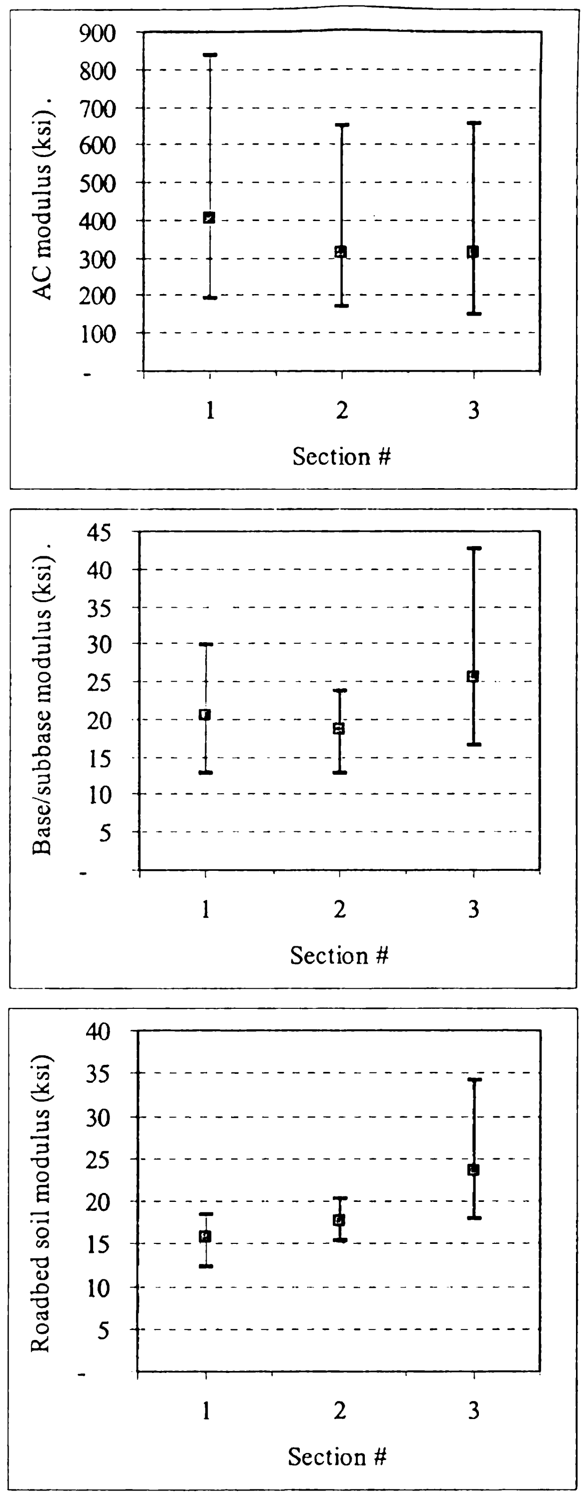


Figure C-17 Variability of the Backcalculated Moduli for Site #6

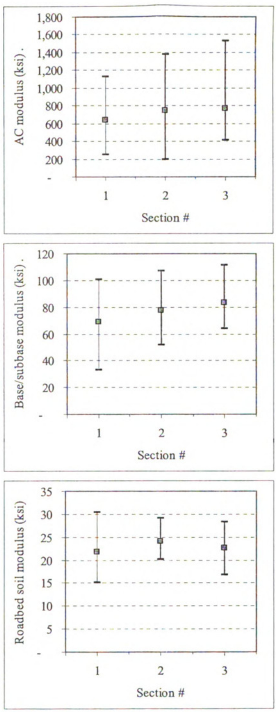


Figure C-18 Variability of the Backcalculated Moduli for Site #7

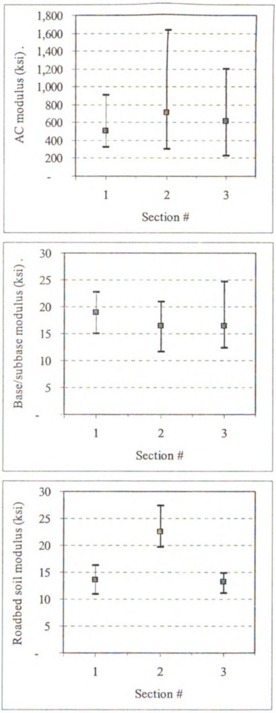


Figure C-19 Variability of the Backcalculated Moduli for Site #8

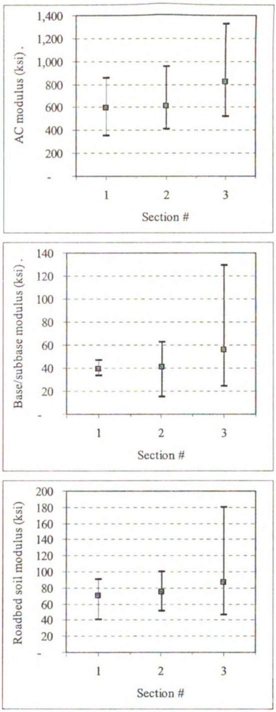


Figure C-20 Variability of the Backcalculated Moduli for Site #9

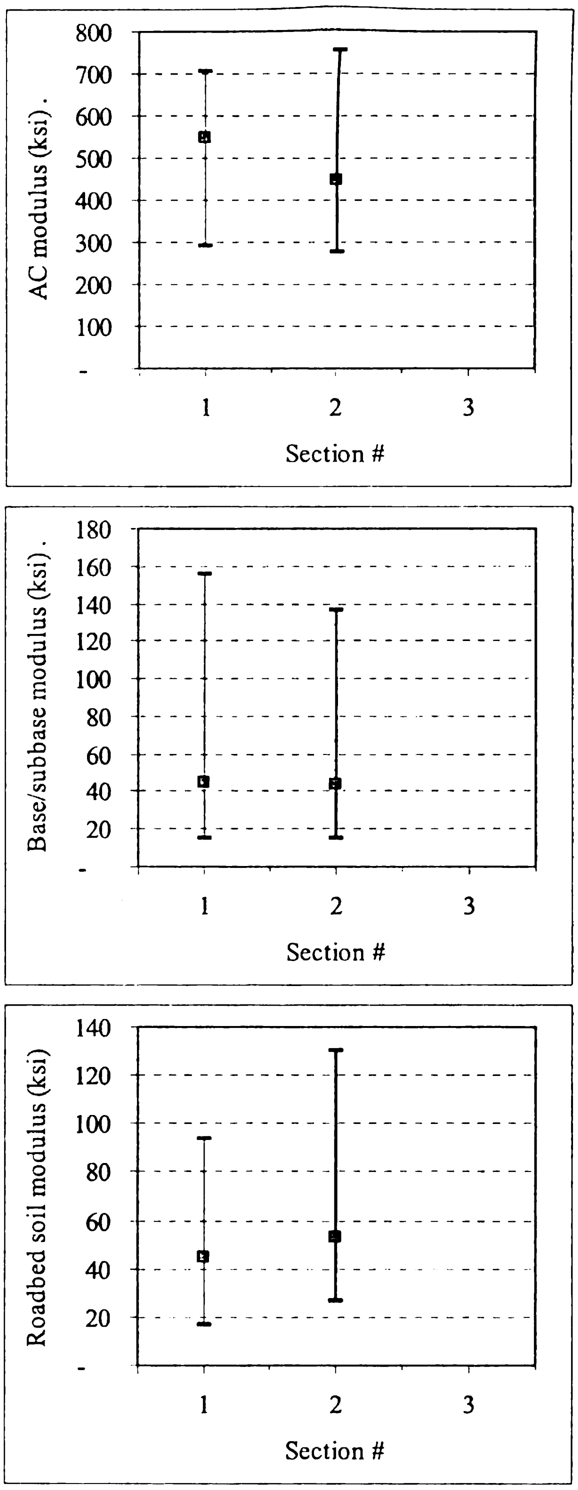


Figure C-21 Variability of the Backcalculated Moduli for Site #10

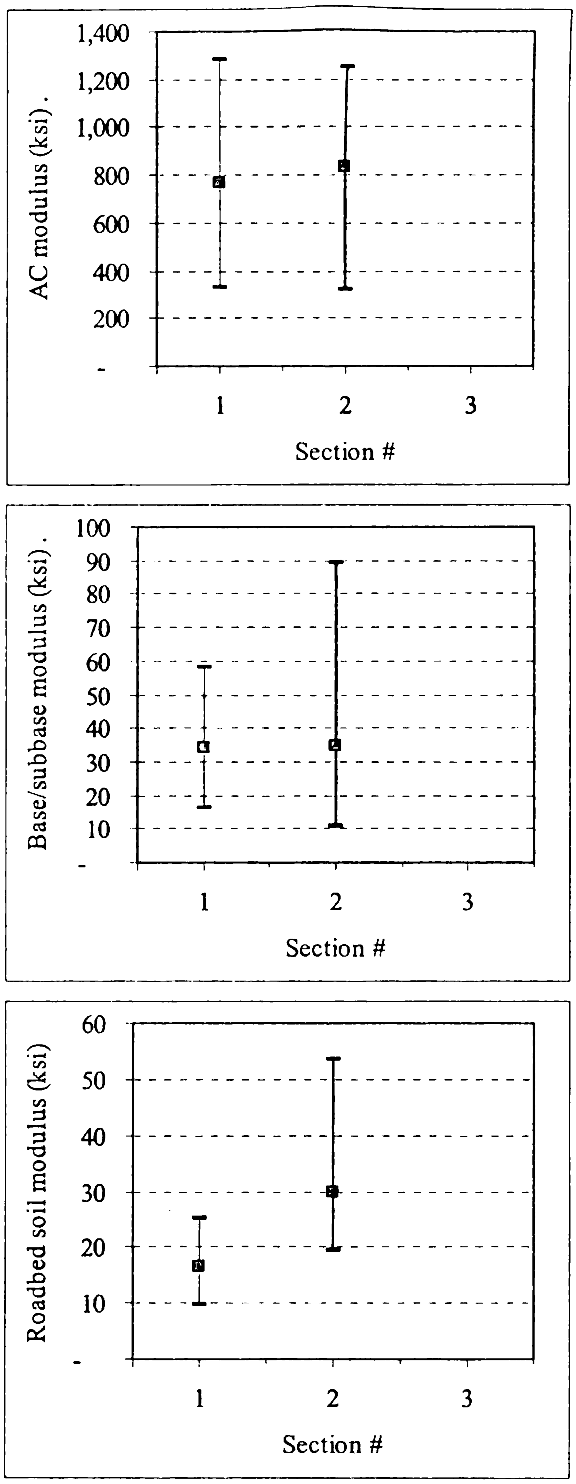


Figure C-22 Variability of the Backcalculated Moduli for Site #11

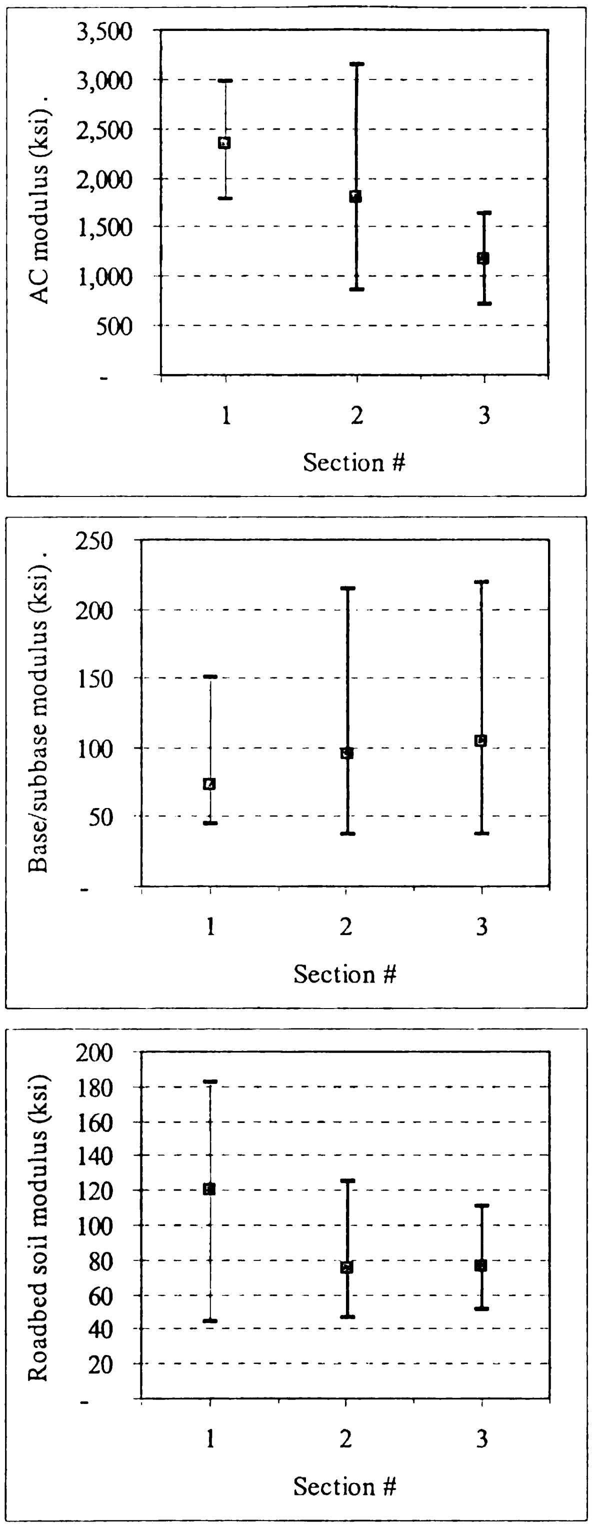


Figure C-23 Variability of the Backcalculated Moduli for Site #12

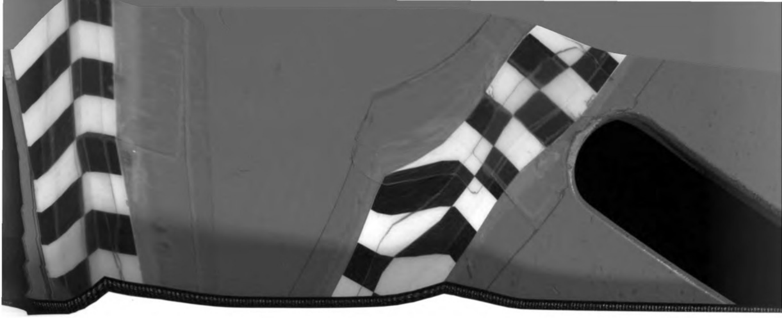
APPENDIX D

Table D-1 Results from binder extraction procedure

RAP Source	Sample	Weight Before Asphalt Extraction (g)	Weight After Asphalt Extraction (g)	Asphalt %
Mandra	1	988	941	4.76
	2	1004	957	5.28
	3	980	930	5.1
	Average Asphalt Content			5.05
Nowshera	1	1006	949	5.67
	2	1002	940	5.59
	3	992	938	5.44
	Average Asphalt Content			5.57

Table D-2 Critical temperatures and PG of the virgin binders

		Critical Temperature (°C)				
Aging	Property	ARL 80/100	ARL 60/70(P)	NRL 40/50	NRL 60/70	NRL 80/100
Original	DSR G*/Sinδ	56.38	72.39	72.60	66.56	60.24
RTFO	DSR G*/Sinδ	59.59	73.80	72.36	67.30	60.04
PAV	DSR G*Sinδ	19.78	23.76	27.83	22.46	19.75
	BBR S-value	-25.6	-13.7	-16.7	-19.4	-19.7
	BBR m-value	-16.1	-13.3	-13.1	-14.5	-16.0
	PG	52-16	70-10	70-10	64-10	58-16
	PG (SP1)	52-22	70-22	70-22	64-22	58-22



APPENDIX E

Virgin and Recycled Mixture Data

Table E-1 Specifications for Mashall mix design

Marshall Mix Design Criteria (MS-2)						
Mix Criteria	Light Traffic		Medium Traffic		Heavy Traffic	
	(< 104 ESALs)		(104 - 106 ESALs)		(> 106 ESALs)	
	Min.	Max.	Min.	Max.	Min.	Max.
Compaction (number of blows on each end of the sample)	35		50		75	
Stability (minimum)	3336 N		5338		8006 N	
	(750 lbs.)		(1200 lbs.)		(1800 lbs.)	
Flow (0.25 mm (0.01 inch))	8	18	8	16	8	14
Percent Air Voids	3	5	3	5	3	5
Percent Voids in Mineral Aggregate (VMA)	See Next Table					
Percent Voids Filled with Asphalt (VFA)	70	80	65	78	65	75

Table E-2 VMA criteria for Marshal mix design

Nominal maximum aggregatesize		Minimum VMA, percent		
		Design air void		
mm	in	3	4	5
1.2	No. 16	21.5	22.5	23.5
2.4	No. 8	19.0	20.0	21.0
4.8	No. 4	16.0	17.0	18.0
9.5	0.375	14.0	15.0	16.0
12.5	0.5	13.0	14.0	15.0
19.0	0.8	12.0	13.0	14.0
25.0	1.0	11.0	12.0	131.0
37.5	1.5	10.0	11.0	12.0
50.0	2.0	9.5	10.5	11.5
63.0	2.5	9.0	10.0	11.0

Table E-3 Specifications for Superpave mix design

Design ESALs (million)	Required Desnity (% of Theoretical Maximum Specific Gravity)			VMA (Percent), minimum					VFA (%)	Dust-to-binder ratio
	Ninitial	Ndesign	Nmax	Nominal Maximum Aggregate Size, mm						
				37.5	25.0	19.0	12.5	9.5		
<0.3	≤ 91.5								70-80	
0.3to <3	≤ 90.5								65-78	
3 to < 10										0.6-1.2
10 to < 30	≤ 89.0	96.0	98.0	11.0	12.0	13.0	14.0	15.0	65-75	
≥ 30										

Table E-4 SPT results for optimum Superpave mixtures at 8°C

Sample	Property	Frequency (Hz)					
		25	10	5	1	0.5	0.1
MW	E* (ksi)	4,482	4,234	3,792	3,457	3,119	2,095
	δ (Deg)	6.06	6.90	13.93	11.48	8.78	12.22
DW	E* (ksi)	3,399	3,140	2,801	2,162	1,870	1,227
	δ (Deg)	11.63	14.28	14.44	19.83	21.46	26.06
MB	E* (ksi)	4,235	3,895	3,624	3,043	2,721	1,712
	δ (Deg)	6.43	10.97	9.14	14.50	14.72	20.57
DB	E* (ksi)	6,431	4,868	4,544	4,359	4,141	3,103
	δ (Deg)	5.08	3.81	4.54	6.81	9.27	13.71

Table E-5 SPT results for optimum Superpave mixtures at 21°C

Sample	Property	Frequency (Hz)					
		25	10	5	1	0.5	0.1
MW	E* (ksi)	1,999	1,646	1,396	886	700	372
	δ (Deg)	18.19	21.40	22.92	27.72	29.27	31.96
DW	E* (ksi)	1,402	1,131	923	509	374	176
	δ (Deg)	24.64	26.13	28.12	32.98	34.61	36.73
MB	E* (ksi)	2,152	1,878	1,614	1,096	877	476
	δ (Deg)	17.55	18.98	20.42	26.10	27.99	32.48
DB	E* (ksi)	2,018	1,658	1,386	835	634	329
	δ (Deg)	19.13	22.78	24.56	29.35	30.91	33.35

Table E-6 SPT results for optimum Superpave mixtures at 37°C

Sample	Property	Frequency (Hz)					
		25	10	5	1	0.5	0.1
MW	E* (ksi)	974	711	544	264	191	128
	δ (Deg)	28.95	31.83	32.89	34.75	34.70	32.07
DW	E* (ksi)	586	388	267	110	75	38
	δ (Deg)	35.16	39.18	40.69	42.31	41.76	37.34
MB	E* (ksi)	948	704	504	225	159	88
	δ (Deg)	36.12	41.88	41.16	41.20	40.07	38.24
DB	E* (ksi)	939	654	482	228	167	119
	δ (Deg)	31.03	33.71	34.69	35.36	34.44	31.07

Note: M=Margalla, D=Deena, W=wearing, B=base

Table E-7 SPT results for blended Superpave mixtures-Margalla wearing + Nowshera

Temp. °C	Sample	Property	Frequency (Hz) 25	10	5	1	0.5	0.1
4°C	MW+N10%	E* (Ksi)	3,726	3,683	3,665	3,368	3,236	2,706
		δ (deg)	7.91	6.87	6.80	10.59	11.18	12.05
	MW+N20%	E* (Ksi)	4,840	4,758	4,196	3,544	3,081	2,532
		δ (deg)	14.14	17.84	16.78	22.40	23.67	26.35
	MW+N30%	E* (Ksi)	4,267	3,648	3,472	3,142	2,944	2,381
		δ (deg)	3.03	6.67	7.51	10.87	11.34	13.00
	MW+N45%	E* (Ksi)	3,246	4,347	4,096	3,762	3,540	2,805
		δ (deg)	5.71	5.52	5.38	8.10	9.86	13.60
	MW+N60%	E* (Ksi)	4,294	4,054	3,843	3,378	3,162	2,681
		δ (deg)	5.58	6.85	7.18	8.97	9.60	11.29
	N 100%	E* (Ksi)	4,080	3,679	3,473	3,044	2,877	2,453
		δ (deg)	3.11	5.71	6.67	8.16	8.99	10.79
21°C	MW+N10%	E* (Ksi)	1,651	1,282	1,040	623	485	275
		δ (deg)	22.89	26.28	27.76	32.79	33.28	32.49
	MW+N20%	E* (Ksi)	1,300	1,000	823	478	371	215
		δ (deg)	24.57	27.94	30.11	33.62	32.95	31.07
	MW+N30%	E* (Ksi)	1,527	1,196	998	620	502	306
		δ (deg)	22.60	25.59	26.89	29.65	29.75	28.88
	MW+N45%	E* (Ksi)	1,504	1,210	1,036	690	579	376
		δ (deg)	19.71	22.17	23.87	27.50	28.23	29.83
	MW+N60%	E* (Ksi)	1,845	1,545	1,346	963	843	588
		δ (deg)	16.33	18.53	19.96	24.04	24.83	26.85
	N 100%	E* (Ksi)	2,085	1,707	1,551	1,166	1,019	729
		δ (deg)	14.31	16.03	16.82	21.35	22.64	25.41
37°C	MW+N10%	E* (Ksi)	865	622	464	217	152	74
		δ (deg)	31.85	33.32	34.35	35.96	36.15	33.35
	MW+N20%	E* (Ksi)	1,002	731	556	269	190	89
		δ (deg)	29.89	31.99	33.11	35.61	35.62	33.56
	MW+N30%	E* (Ksi)	1,289	984	803	461	352	189
		δ (deg)	23.37	26.05	27.28	29.31	29.23	28.85
	MW+N45%	E* (Ksi)	1,482	1,187	1,002	648	524	294
		δ (deg)	19.84	22.19	24.05	27.33	28.22	30.51
	MW+N60%	E* (Ksi)	1,627	1,340	1,168	805	682	420
		δ (deg)	18.64	20.75	22.12	25.35	26.06	28.13
	N 100%	E* (Ksi)	1,889	1,573	1,387	999	862	562
		δ (deg)	15.18	18.29	19.40	23.18	24.08	27.08

Table E-8 SPT results for blended Superpave mixtures-Deena wearing + Nowshera

Temp. °C	Sample	Property	Frequency (Hz)					
			25	10	5	1	0.5	0.1
4°C	DW+N10%	E* (Ksi)	3,736	3,288	3,029	2,496	2,263	1,725
		δ (deg)	9.60	13.04	12.83	17.45	18.55	21.11
	DW+N20%	E* (Ksi)	2,515	2,319	2,161	1,773	1,650	1,220
		δ (deg)	10.04	10.44	11.85	20.99	24.74	28.77
	DW+N30%	E* (Ksi)	3,972	3,074	2,863	2,303	2,100	1,620
		δ (deg)	2.36	9.42	10.01	13.76	14.95	18.69
	DW+N45%	E* (Ksi)	3,816	3,595	3,439	3,269	3,100	2,541
		δ (deg)	11.03	14.47	11.82	7.58	9.12	15.81
	DW+N60%	E* (Ksi)	4,383	4,694	4,258	4,755	4,862	3,862
		δ (deg)	5.05	10.87	9.50	14.08	15.88	18.17
	N 100%	E* (Ksi)	4,080	3,679	3,473	3,044	2,877	2,453
		δ (deg)	3.11	5.71	6.67	8.16	8.99	10.79
21°C	DW+N10%	E* (Ksi)	3,389	2,601	2,426	2,063	1,863	1,185
		δ (deg)	7.26	12.69	14.20	23.20	24.10	27.94
	DW+N20%	E* (Ksi)	1,907	1,511	1,285	836	666	365
		δ (deg)	17.72	20.15	21.61	27.04	28.84	32.17
	DW+N30%	E* (Ksi)	2,024	1,652	1,454	1,083	956	657
		δ (deg)	12.78	15.95	17.47	22.26	23.58	26.08
	DW+N45%	E* (Ksi)	1,824	2,222	2,010	1,533	1,303	792
		δ (deg)	15.08	13.71	14.48	20.91	22.90	28.92
	DW+N60%	E* (Ksi)	2,836	2,762	2,557	1,985	1,756	1,328
		δ (deg)	10.89	7.03	10.11	14.82	16.10	24.23
	N 100%	E* (Ksi)	2,085	1,707	1,551	1,166	1,019	729
		δ (deg)	14.31	16.03	16.82	21.35	22.64	25.41
37°C	DW+N10%	E* (Ksi)	3,083	2,291	2,030	1,452	1,211	634
		δ (deg)	16.24	17.91	18.85	23.75	25.74	31.48
	DW+N20%	E* (Ksi)	1,244	902	731	392	282	116
		δ (deg)	24.70	26.72	28.00	32.80	33.97	34.84
	DW+N30%	E* (Ksi)	1,400	1,156	944	575	430	183
		δ (deg)	22.28	23.98	26.03	30.44	31.81	33.96
	DW+N45%	E* (Ksi)	1,263	1,010	837	489	366	153
		δ (deg)	21.75	25.58	27.37	31.72	32.88	35.26
	DW+N60%	E* (Ksi)	1,947	1,501	1,382	897	757	471
		δ (deg)	36.37	18.70	18.90	24.87	25.72	30.70
	N 100%	E* (Ksi)	1,889	1,573	1,387	999	862	562
		δ (deg)	15.18	18.29	19.40	23.18	24.08	27.08

Table E-9 SPT results for blended Superpave mixtures-Deena wearing + Mandra

Temp. °C	Sample	Property	Frequency (Hz) 25	10	5	1	0.5	0.1
4°C	DW+M10%	E* (Ksi)	4,462	3,970	3,811	3,384	3,186	2,540
		δ (deg)	1.72	5.54	6.95	9.80	10.91	14.86
	DW+M20%	E* (Ksi)	3,580	3,108	2,741	2,112	1,858	1,328
		δ (deg)	7.12	15.79	14.32	20.91	22.57	27.27
	DW+M30%	E* (Ksi)	3,073	2,936	2,627	2,068	1,840	1,321
		δ (deg)	12.30	12.53	13.27	18.79	20.57	25.66
	DW+M45%	E* (Ksi)	5,028	5,893	5,801	6,500	6,556	5,723
		δ (deg)	7.83	6.91	5.04	8.49	10.61	14.92
	DW+M60%	E* (Ksi)	4,261	3,906	3,740	3,309	3,075	2,398
		δ (deg)	4.78	6.91	7.06	10.39	11.95	14.21
	M 100%	E* (Ksi)	3,803	3,942	3,797	3,307	3,118	2,689
		δ (deg)	5.33	6.98	8.01	8.74	9.40	11.23
21°C	DW+M10%	E* (Ksi)	1,772	1,507	1,299	846	685	333
		δ (deg)	16.73	20.27	23.22	30.89	34.04	39.85
	DW+M20%	E* (Ksi)	2,210	1,787	1,508	953	749	374
		δ (deg)	18.04	21.09	22.17	29.93	32.33	36.59
	DW+M30%	E* (Ksi)	2,170	1,813	1,591	986	762	354
		δ (deg)	21.12	24.95	26.59	34.26	36.76	40.32
	DW+M45%	E* (Ksi)	2,699	2,077	1,914	1,446	1,235	758
		δ (deg)	26.43	15.48	17.24	23.98	26.56	30.72
	DW+M60%	E* (Ksi)	2,954	2,006	1,764	1,234	1,055	662
		δ (deg)	8.56	16.52	19.19	23.49	25.31	29.93
	M 100%	E* (Ksi)	2,967	2,662	2,458	1,956	1,751	1,330
		δ (deg)	10.20	12.56	12.98	15.88	16.90	20.73
37°C	DW+M10%	E* (Ksi)	949	695	526	227	148	51
		δ (deg)	29.99	34.01	36.01	40.45	41.31	38.10
	DW+M20%	E* (Ksi)	1,742	1,351	1,096	599	424	155
		δ (deg)	22.62	26.29	28.59	35.08	36.84	39.24
	DW+M30%	E* (Ksi)	1,506	1,109	857	398	263	94
		δ (deg)	26.52	31.22	33.60	39.13	41.11	41.00
	DW+M45%	E* (Ksi)	1,593	1,432	1,235	789	615	278
		δ (deg)	17.95	21.70	24.61	29.19	30.67	35.06
	DW+M60%	E* (Ksi)	1,522	1,210	1,019	637	508	255
		δ (deg)	21.71	24.99	26.54	30.30	31.02	34.17
	M 100%	E* (Ksi)	1,401	1,344	1,186	819	685	422
		δ (deg)	16.90	20.27	21.69	24.91	25.97	27.90

Table E-10 SPT results for blended Superpave mixtures-Margalla wearing + Mandra

Temp. °C	Sample	Property	Frequency (Hz)					
			25	10	5	1	0.5	0.1
4°C	MW+M10%	E* (Ksi)	4,978	5,377	4,898	4,815	4,336	3,329
		δ (deg)	7.38	11.63	11.23	15.53	15.44	16.73
	MW+M20%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	MW+M30%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	MW+M45%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	MW+M60%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	M 100%	E* (Ksi)	3,803	3,942	3,797	3,307	3,118	2,689
		δ (deg)	5.33	6.98	8.01	8.74	9.40	11.23
21°C	MW+M10%	E* (Ksi)	1,976	1,837	1,635	1,144	956	581
		δ (deg)	14.57	17.11	18.79	23.39	24.84	28.23
	MW+M20%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	MW+M30%	E* (Ksi)	1,834	1,615	1,377	921	759	457
		δ (deg)	18.29	20.11	21.77	26.78	28.40	31.67
	MW+M45%	E* (Ksi)	1,542	1,337	1,158	739	610	362
		δ (deg)	18.42	22.66	23.64	27.18	28.14	29.38
	MW+M60%	E* (Ksi)	1,977	1,659	1,441	987	837	552
		δ (deg)	14.92	18.74	20.88	25.65	26.41	29.13
	M 100%	E* (Ksi)	2,967	2,662	2,458	1,956	1,751	1,330
		δ (deg)	10.20	12.56	12.98	15.88	16.90	20.73
37°C	MW+M10%	E* (Ksi)	1,777	1,555	1,331	887	718	374
		δ (deg)	15.81	20.01	21.57	26.27	28.02	31.23
	MW+M20%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	MW+M30%	E* (Ksi)	1,176	957	777	439	332	171
		δ (deg)	23.26	26.82	28.00	30.27	30.30	30.08
	MW+M45%	E* (Ksi)	1,433	1,093	891	524	406	214
		δ (deg)	23.41	26.18	27.99	31.32	31.93	32.74
	MW+M60%	E* (Ksi)	1,486	1,151	975	619	495	262
		δ (deg)	21.42	23.90	26.06	30.32	31.50	33.00
	M 100%	E* (Ksi)	1,401	1,344	1,186	819	685	422
		δ (deg)	16.90	20.27	21.69	24.91	25.97	27.90

Table E-11 SPT results for blended Superpave mixtures-Margalla base + Nowshera

Temp. °C	Sample	Property	Frequency (Hz) 25	10	5	1	0.5	0.1
4°C	MB+N 10%	E* (Ksi)	4,935	3,091	3,027	3,012	2,880	2,321
		δ (deg)	12.56	5.56	7.01	10.11	11.49	16.22
	MB+N 20%	E* (Ksi)	5,872	7,937	7,643	8,858	9,470	9,119
		δ (deg)	29.62	9.86	14.09	3.83	11.50	21.93
	MB+N 30%	E* (Ksi)	3,902	3,778	3,631	3,317	3,148	2,685
		δ (deg)	5.99	6.12	6.72	9.64	10.93	14.41
	MB+N 45%	E* (Ksi)	5,420	5,501	5,429	5,262	5,211	4,505
		δ (deg)	5.01	5.35	4.19	9.19	7.77	9.62
	MB+N 60%	E* (Ksi)	4,889	4,440	4,322	4,538	4,160	3,398
		δ (deg)	6.66	6.66	6.50	11.62	11.82	11.79
	N 100%	E* (Ksi)	4,034	4,002	3,861	3,637	3,511	3,138
		δ (deg)	4.48	3.83	3.78	4.77	5.36	6.58
21°C	MB+N 10%	E* (Ksi)	2,252	1,939	1,646	1,020	767	346
		δ (deg)	18.29	23.07	25.71	35.71	38.66	43.09
	MB+N 20%	E* (Ksi)	2,408	1,860	1,551	972	779	394
		δ (deg)	19.49	22.99	25.29	32.49	34.49	38.26
	MB+N 30%	E* (Ksi)	2,163	1,898	1,632	1,115	922	549
		δ (deg)	16.23	18.38	22.09	30.76	32.49	36.31
	MB+N 45%	E* (Ksi)	2,284	2,023	1,786	1,235	1,029	641
		δ (deg)	16.87	18.28	21.01	26.05	27.89	31.40
	MB+N 60%	E* (Ksi)	2,534	2,327	2,052	1,564	1,364	927
		δ (deg)	13.88	18.04	17.96	25.40	27.66	32.39
	N 100%	E* (Ksi)	4,148	3,191	3,128	2,588	2,345	1,886
		δ (deg)	6.61	8.87	9.34	11.87	12.24	14.29
37°C	MB+N 10%	E* (Ksi)	1,228	880	658	286	192	77
		δ (deg)	33.51	38.51	41.33	45.02	44.50	42.39
	MB+N 20%	E* (Ksi)	1,578	1,144	902	466	332	127
		δ (deg)	27.68	32.04	34.49	40.24	40.96	40.37
	MB+N 30%	E* (Ksi)	2,960	2,497	2,130	1,385	1,080	517
		δ (deg)	39.87	37.71	17.07	4.48	6.14	36.68
	MB+N 45%	E* (Ksi)	2,008	1,860	1,615	1,086	876	475
		δ (deg)	18.71	19.79	21.38	27.09	28.76	33.02
	MB+N 60%	E* (Ksi)	2,023	1,711	1,520	1,075	916	561
		δ (deg)	14.31	17.02	18.64	23.48	25.16	29.42
	N 100%	E* (Ksi)	2,927	2,694	2,500	2,105	1,925	1,459
		δ (deg)	16.75	9.76	10.51	12.58	13.60	16.76

Table E-12 SPT results for blended Superpave mixtures-Deena base + Nowshera

Temp. °C	Sample	Property	Frequency (Hz)					
			25	10	5	1	0.5	0.1
4°C	DB+N 10%	E* (Ksi)	3,102	2,767	2,493	1,842	1,600	1,133
		δ (deg)	11.75	14.33	15.78	20.79	22.31	26.59
	DB+N 20%	E* (Ksi)	4,726	4,664	4,459	3,991	3,726	3,069
		δ (deg)	6.73	7.47	6.64	9.19	10.52	12.93
	DB+N 30%	E* (Ksi)	5,629	5,601	5,295	4,688	3,973	2,697
		δ (deg)	10.29	15.38	13.53	21.41	21.77	22.85
	DB+N 45%	E* (Ksi)	4,758	4,575	4,341	3,789	3,431	2,695
		δ (deg)	8.51	7.96	7.97	11.26	13.15	15.91
	DB+N 60%	E* (Ksi)	4,963	4,824	4,454	4,033	3,718	2,939
		δ (deg)	7.12	9.30	8.43	12.55	13.63	16.05
	N 100%	E* (Ksi)	4,034	4,002	3,861	3,637	3,511	3,138
		δ (deg)	4.48	3.83	3.78	4.77	5.36	6.58
21°C	MB+N 10%	E* (Ksi)	1,784	1,472	1,260	797	653	358
		δ (deg)	18.63	23.52	25.85	31.74	33.38	36.53
	MB+N 20%	E* (Ksi)	2,776	2,370	2,095	1,504	1,292	836
		δ (deg)	14.47	15.97	17.76	21.76	23.79	27.62
	MB+N 30%	E* (Ksi)	2,532	2,615	2,402	1,632	1,317	826
		δ (deg)	13.16	13.84	19.55	24.98	27.22	30.60
	MB+N 45%	E* (Ksi)	2,132	1,827	1,597	1,132	957	600
		δ (deg)	14.48	17.70	19.20	24.69	26.16	29.34
	MB+N 60%	E* (Ksi)	2,351	2,060	1,856	1,386	1,211	852
		δ (deg)	10.66	14.19	16.08	20.51	22.02	25.69
	N 100%	E* (Ksi)	4,148	3,191	3,128	2,588	2,345	1,886
		δ (deg)	6.61	8.87	9.34	11.87	12.24	14.29
37°C	MB+N 10%	E* (Ksi)	1,488	1,172	965	544	399	172
		δ (deg)	22.45	25.78	27.33	31.11	32.33	34.90
	MB+N 20%	E* (Ksi)	1,753	1,592	1,369	896	705	381
		δ (deg)	17.71	21.04	21.88	25.92	26.74	27.99
	MB+N 30%	E* (Ksi)	2,126	1,671	1,416	895	682	322
		δ (deg)	20.32	22.07	23.04	26.67	27.66	29.13
	MB+N 45%	E* (Ksi)	1,814	1,396	1,172	714	542	258
		δ (deg)	17.59	23.35	24.87	30.07	31.51	42.13
	MB+N 60%	E* (Ksi)	-	-	-	-	-	-
		δ (deg)	-	-	-	-	-	-
	N 100%	E* (Ksi)	2,927	2,694	2,500	2,105	1,925	1,459
		δ (deg)	16.75	9.76	10.51	12.58	13.60	16.76

Table E-13 SPT results for blended Superpave mixtures-Deena base + Mandra

Temp. °C	Sample	Property	Frequency (Hz) 25	10	5	1	0.5	0.1
4°C	DB+M 10%	E* (Ksi)	3,237	2,819	2,629	2,226	2,044	1,607
		δ (deg)	2.61	8.13	8.98	12.60	13.49	16.65
	DB+M 20%	E* (Ksi)	3,769	3,275	2,972	2,527	2,364	1,892
		δ (deg)	1.08	10.09	9.39	14.75	16.15	19.68
	DB+M 30%	E* (Ksi)	4,647	5,109	4,874	5,874	5,654	4,263
		δ (deg)	14.44	15.70	14.41	11.57	14.34	23.40
	DB+M 45%	E* (Ksi)	2,112	3,654	3,449	2,937	2,731	2,204
		δ (deg)	17.00	9.65	10.54	12.39	13.76	17.80
	DB+M60%	E* (Ksi)	4,114	3,918	3,879	3,552	3,408	2,967
		δ (deg)	20.10	4.25	5.26	6.58	7.17	9.20
	M 100%	E* (Ksi)	3,703	3,690	3,583	3,413	3,334	2,955
		δ (deg)	4.10	4.99	4.85	7.12	7.97	9.58
21°C	DB+M 10%	E* (Ksi)	1,083	859	715	428	346	208
		δ (deg)	23.82	26.73	28.16	30.43	30.24	29.45
	DB+M 20%	E* (Ksi)	1,324	997	818	491	396	237
		δ (deg)	23.19	26.11	27.51	29.58	29.43	29.04
	DB+M 30%	E* (Ksi)	1,557	1,300	1,105	722	597	384
		δ (deg)	19.78	22.68	24.65	27.53	27.89	27.98
	DB+M 45%	E* (Ksi)	1,847	1,618	1,360	903	740	427
		δ (deg)	19.55	22.29	24.19	29.79	31.06	33.73
	DB+M60%	E* (Ksi)	2,633	2,249	2,011	1,519	1,341	932
		δ (deg)	12.09	14.18	15.72	19.81	21.54	26.10
	M 100%	E* (Ksi)	2,452	2,237	2,095	1,692	1,568	1,240
		δ (deg)	12.83	13.93	13.66	17.70	18.79	22.11
37°C	DB+M 10%	E* (Ksi)	1,127	890	737	433	338	179
		δ (deg)	22.96	25.40	26.76	29.08	29.19	28.38
	DB+M 20%	E* (Ksi)	2,261	1,868	1,599	1,054	821	403
		δ (deg)	16.44	18.81	20.59	25.59	27.21	29.79
	DB+M 30%	E* (Ksi)	2,338	2,075	1,832	1,307	1,077	614
		δ (deg)	15.00	16.75	18.11	21.85	23.57	26.74
	DB+M 45%	E* (Ksi)	1,110	883	699	360	260	119
		δ (deg)	27.01	30.58	32.37	35.38	35.51	34.69
	DB+M60%	E* (Ksi)	2,910	2,585	2,348	1,827	1,600	1,022
		δ (deg)	11.52	13.09	14.37	18.26	19.71	25.60
	M 100%	E* (Ksi)	1,281	945	781	476	385	213
		δ (deg)	26.34	26.82	27.89	29.68	29.53	29.62

Table E-14 SPT results for blended Superpave mixtures-Margalla base + Mandra

Temp. °C	Sample	Property	Frequency (Hz) 25	10	5	1	0.5	0.1
4°C	MB+M 10%	E* (Ksi)	3,703	3,690	3,583	3,413	3,334	2,955
		δ (deg)	4.10	4.99	4.85	7.12	7.97	9.58
	MB+M20%	E* (Ksi)	3,539	3,386	3,150	2,662	2,396	1,789
		δ (deg)	9.47	10.90	11.23	15.35	16.95	20.62
	MB+M30%	E* (Ksi)	4,348	5,035	4,354	5,325	5,600	4,539
		δ (deg)	5.43	8.02	5.59	7.10	9.04	14.33
	MB+M5%	E* (Ksi)	3,949	3,697	3,480	3,146	2,916	2,213
		δ (deg)	4.55	7.17	6.47	11.91	13.53	16.54
	MB+M0%	E* (Ksi)	5,309	5,803	5,470	5,918	5,769	5,137
		δ (deg)	3.03	3.69	3.54	1.56	4.22	9.57
	M100%	E* (Ksi)	3,703	3,690	3,583	3,413	3,334	2,955
		δ (deg)	4.10	4.99	4.85	7.12	7.97	9.58
21°C	MB+M 10%	E* (Ksi)	1,458	1,239	1,057	705	590	369
		δ (deg)	18.76	21.48	23.31	28.97	30.04	31.26
	MB+M20%	E* (Ksi)	1,661	1,391	1,184	760	617	378
		δ (deg)	18.80	23.67	24.89	28.58	29.02	29.96
	MB+M30%	E* (Ksi)	2,004	1,721	1,513	1,070	902	586
		δ (deg)	15.58	18.06	19.40	23.48	24.44	26.24
	MB+M5%	E* (Ksi)	2,187	1,816	1,528	992	795	435
		δ (deg)	18.50	22.51	23.49	29.27	31.04	34.46
	MB+M0%	E* (Ksi)	2,280	1,840	1,596	1,090	927	583
		δ (deg)	16.90	20.13	21.49	27.46	29.55	33.05
	M100%	E* (Ksi)	2,452	2,237	2,095	1,692	1,568	1,240
		δ (deg)	12.83	13.93	13.66	17.70	18.79	22.11
37°C	MB+M 10%	E* (Ksi)	885	661	516	270	208	111
		δ (deg)	29.37	31.06	31.81	32.49	31.45	30.08
	MB+M20%	E* (Ksi)	972	799	640	352	268	160
		δ (deg)	33.29	34.67	33.80	36.40	34.41	38.81
	MB+M30%	E* (Ksi)	1,481	1,197	990	616	494	277
		δ (deg)	24.11	26.99	27.66	29.01	28.97	31.28
	MB+M5%	E* (Ksi)	1,966	1,140	929	534	463	348
		δ (deg)	31.89	32.98	37.00	43.83	45.03	1.79
	MB+M0%	E* (Ksi)	2,256	1,726	1,478	1,000	809	450
		δ (deg)	18.09	21.03	22.96	28.99	30.08	32.67
	M100%	E* (Ksi)	1,281	945	781	476	385	213
		δ (deg)	26.34	26.82	27.89	29.68	29.53	29.62

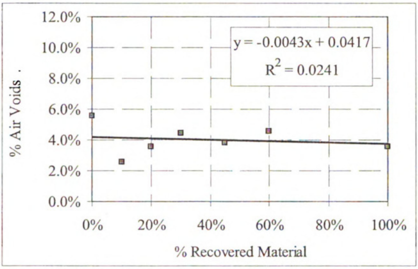


Figure E-1 Air voids versus percent recovered material for a wearing course recycled mixture with Deena aggregates and Nowshera recovered material (Marshall Mixture)

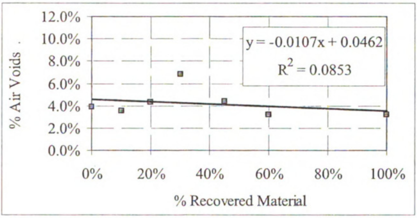


Figure E-2 Air voids versus percent recovered material for a wearing course recycled mixture with Margalla aggregates and Mandra recovered material (Marshall Mixture)

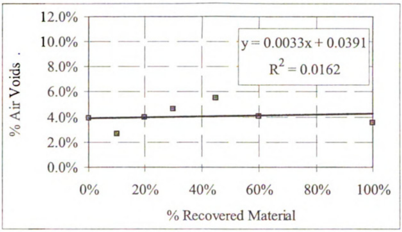


Figure E-3 Air voids versus percent recovered material for a wearing course recycled mixture with Margalla aggregates and Nowshera recovered material (Marshall Mixture)

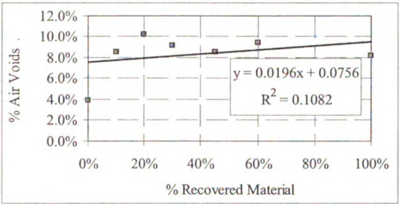


Figure E-4 Air voids versus percent recovered material for a base course recycled mixture with Deena aggregates and Mandra recovered material (Marshall Mixture)

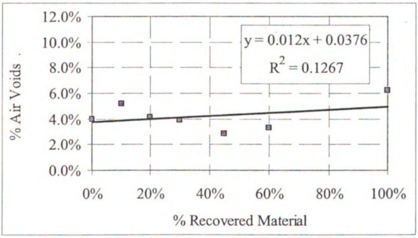


Figure E-5 Air voids versus percent recovered material for a base course recycled mixture with Deena aggregates and Nowshera recovered material (Marshall Mixture)

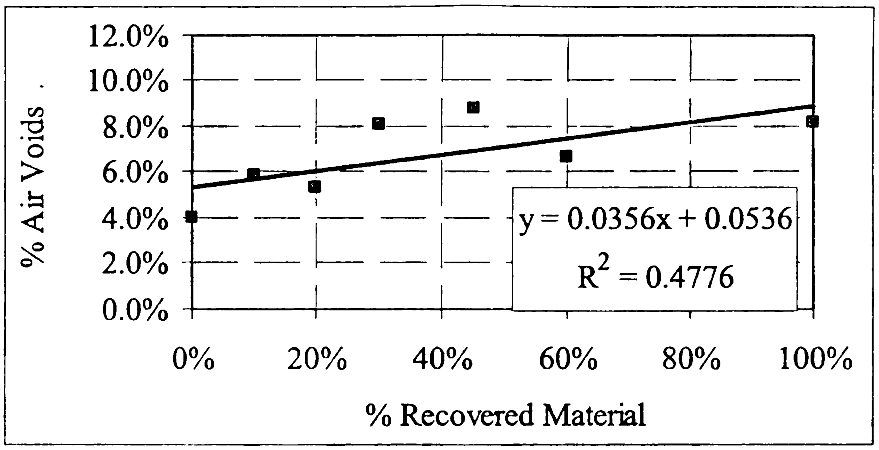


Figure E-6 Air voids versus percent recovered material for a base course recycled mixture with Margalla aggregates and Mandra recovered material (Marshall Mixture)

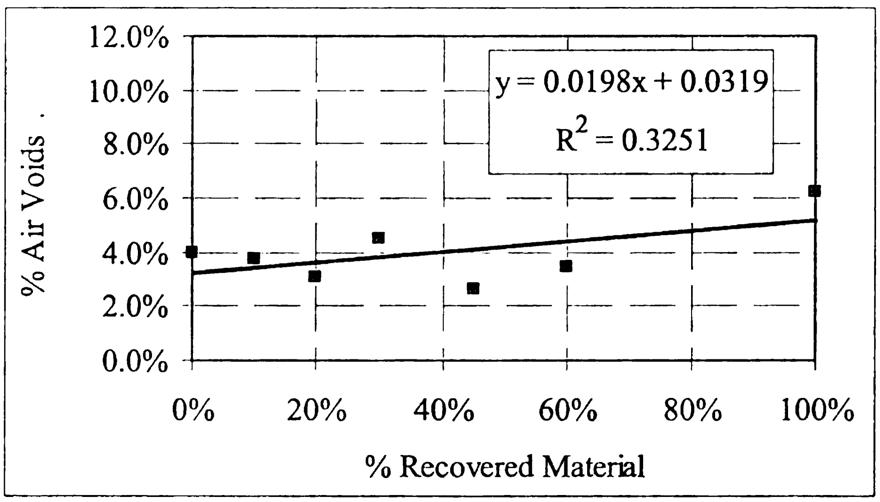


Figure E-7 Air voids versus percent recovered material for a base course recycled mixture with Margalla aggregates and Nowshera recovered material (Marshall Mixture)

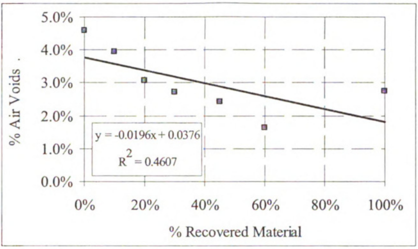


Figure E-8 Air voids versus percent recovered material for DW-N (Superpave Mixture)

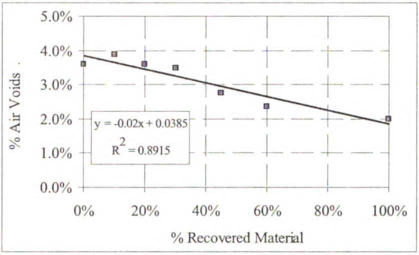


Figure E-9 Air voids versus percent recovered material for MW-N (Superpave Mixture)

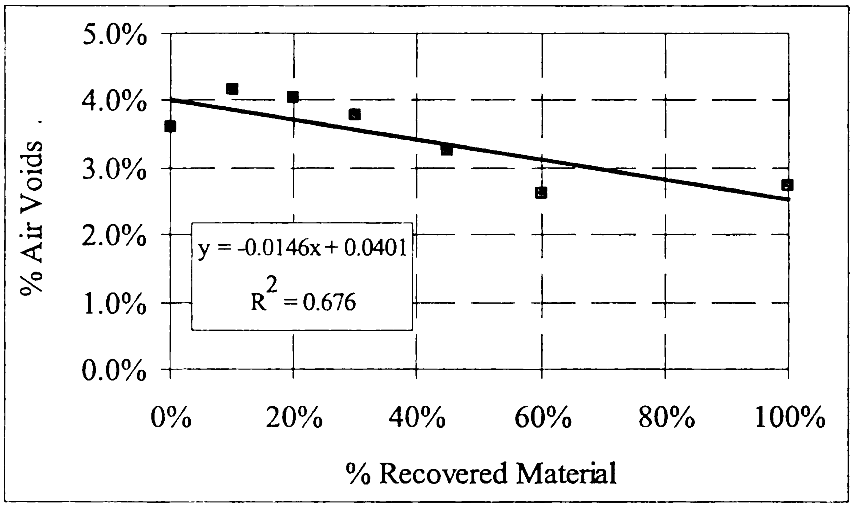


Figure E-10 Air voids versus percent recovered material for MW-M (Superpave Mixture)

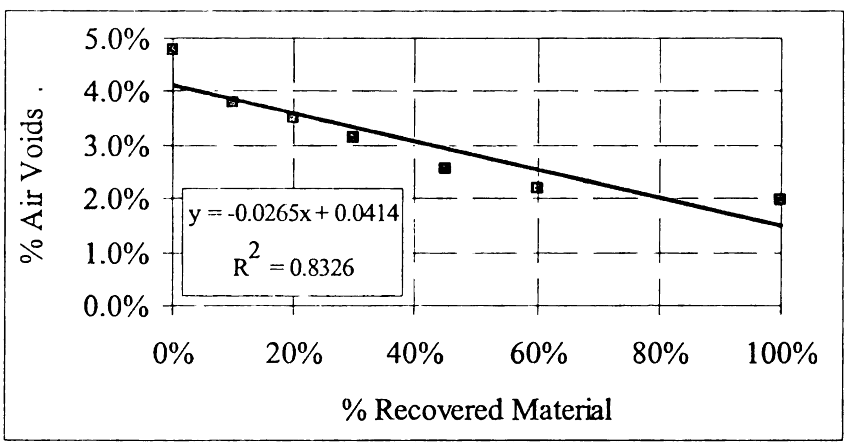


Figure E-11 Air voids versus percent recovered material for DB-M (Superpave Mixture)

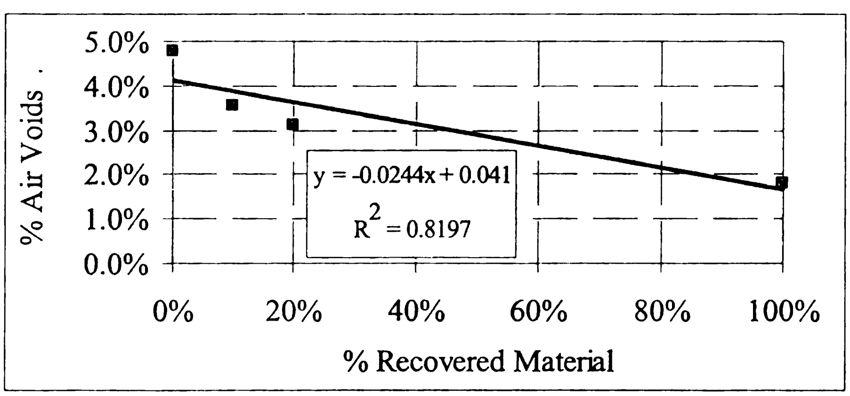


Figure E-12 Air voids versus percent recovered material for DB-N (Superpave Mixture)

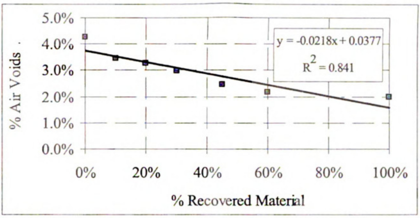


Figure E-13 Air voids versus percent recovered material for MB-M (Superpave Mixture)

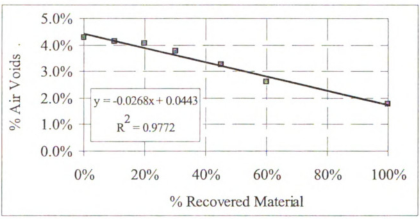


Figure E-14 Air voids versus percent recovered material for MB-N (Superpave Mixture)

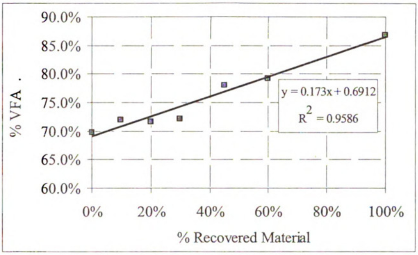


Figure E-15 VFA versus percent recovered material for DW-M (Superpave Mixture)

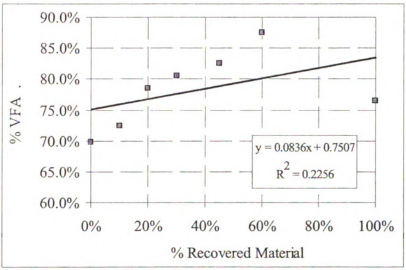


Figure E-16 VFA versus percent recovered material for DW-N (Superpave Mixture)

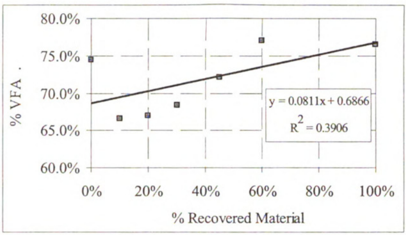


Figure E-17 VFA versus percent recovered material for MW-N (Superpave Mixture)

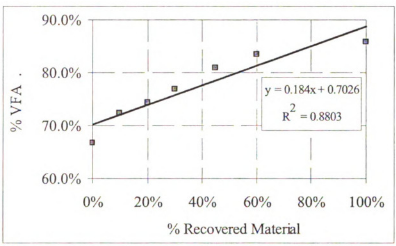


Figure E-18 VFA versus percent recovered material for DB-M (Superpave Mixture)

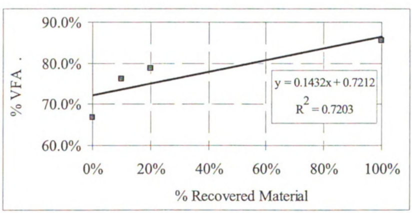


Figure E-19 VFA versus percent recovered material for DB-N (Superpave Mixture)

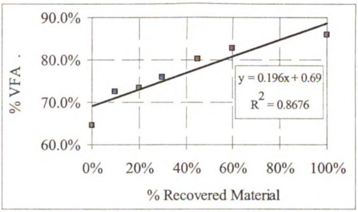


Figure E-20 VFA versus percent recovered material for MB-M (Superpave Mixture)

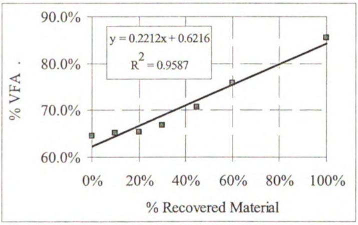


Figure E-21 VFA versus percent recovered material for MB-N (Superpave Mixture)

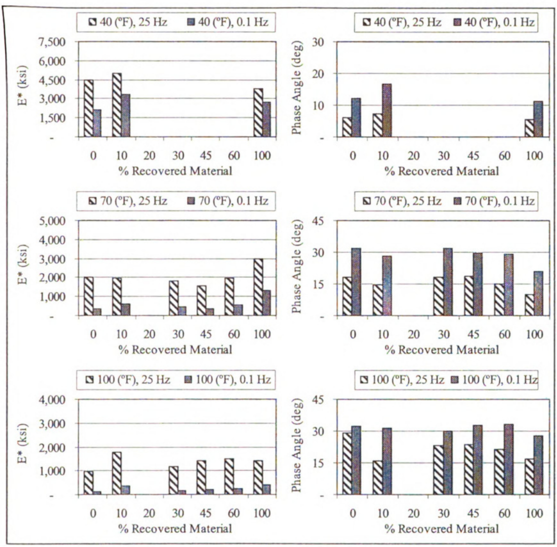


Figure E-22 Variability of E* and δ for the same temperature and frequency for wearing course Margalla + Mandra blends

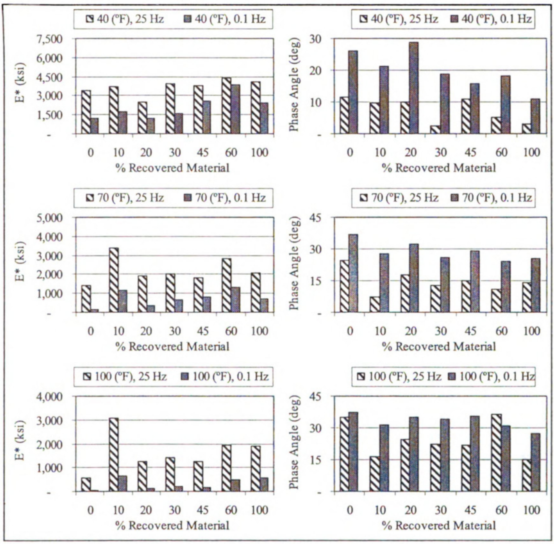


Figure E-23 Variability of E* and δ for the same temperature and frequency for wearing course Deena + Nowshera blends

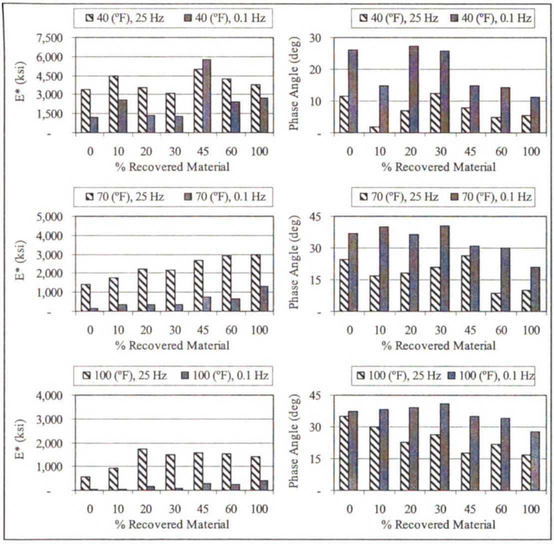


Figure E-24 Variability of E* and δ for the same temperature and frequency for wearing course Deena + Mandra blends

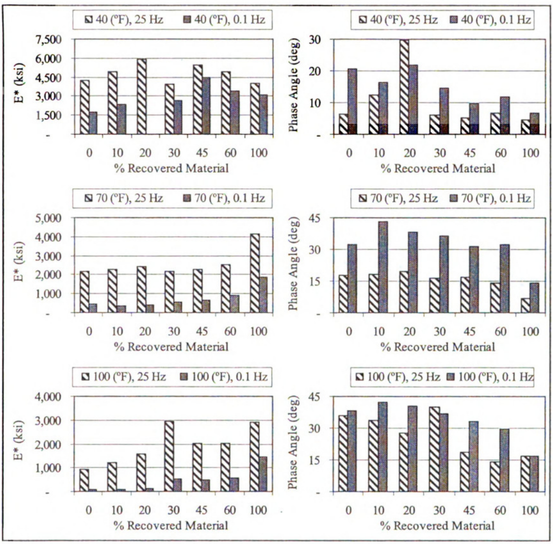


Figure E-25 Variability of E* and δ for the same temperature and frequency for base course Margalla + Nowshera blends

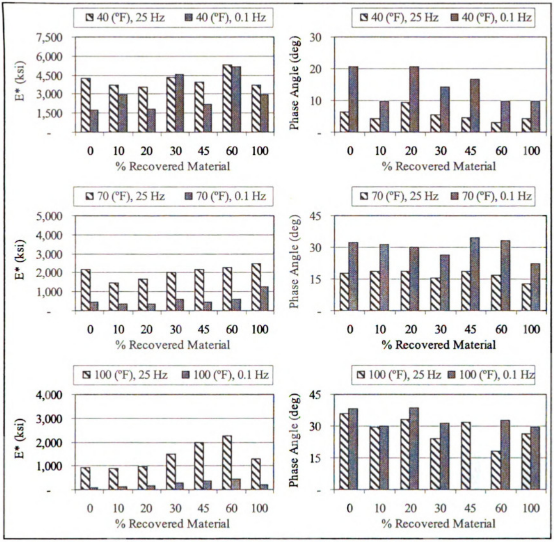


Figure E-26 Variability of E* and δ for the same temperature and frequency for base course Margalla + Mandra blends

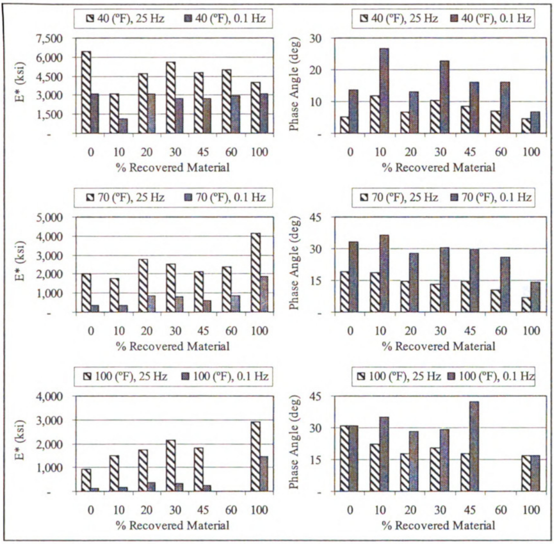


Figure E-27 Variability of E* and δ for the same temperature and frequency for base course Deena + Nowshera blends

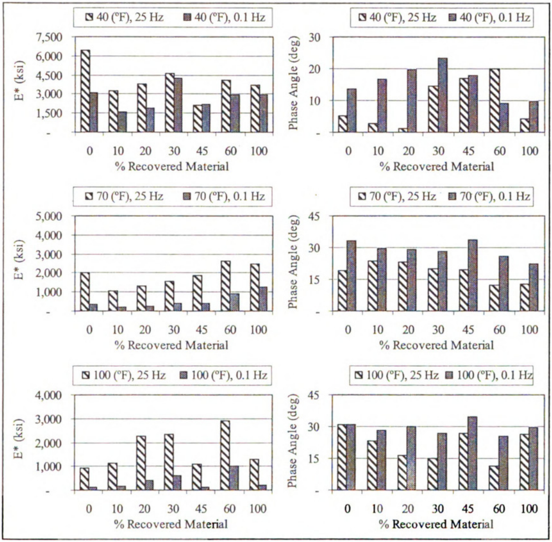


Figure E-28 Variability of E* and δ for the same temperature and frequency for base course Deena + Mandra blends

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