Rehabilitation Techniques for Stripped Asphalt Pavements

Final Report

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16. Abstract

Asphalt stripping is a fairly common form of distress for pavements in Montana, particularly for pavements that were surfaced with an open-graded friction course. Currently, the technique for rehabilitating these pavements involves the costly removal of most or all of the stripped material, prior to the placement of an overlay. The goal of this research was to determine whether the stripped material can remain in-place, serving as a structural layer within the rehabilitated pavement. This study has involved the construction of five test sites, which were incorporated into larger overlay projects. At each of these sites, stripped material was removed from a control section and stripped material was left in-place for a test section, prior to the placement of the overlay.

Leaving stripped asphalt concrete surface material in-place during rehabilitation, to be overlayed with new asphalt concrete, did not tend to make the rehabilitated pavement more susceptible to either stripping damage or load-induced damage. Life-cycle cost analyses should consider rate of stripping deterioration (in./year) to new asphalt concrete to be the same, whether or not stripped material is removed prior to placing an overlay. Overlay thickness and mix design methods for resisting stripping are the important factors for extending the life of a rehabilitated stripped asphalt pavement.

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Implementation Statement

This study is sponsored by the Montana Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The major objective of this study is to determine the most cost-effective method of rehabilitating stripped asphalt pavements in the state of Montana. Recommendations from this study will indicate whether or not stripped material should be removed from a pavement surface, prior to the placement of an overlay.

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Abstract

Asphalt stripping is a fairly common form of distress for pavements in Montana, particularly for pavements that were surfaced with an open-graded friction course. Currently, the technique for rehabilitating these pavements involves the costly removal of most or all of the stripped material, prior to the placement of an overlay. The goal of this research was to determine whether the stripped material can remain in-place, serving as a structural layer within the rehabilitated pavement. This study has involved the construction of five test sites, which were incorporated into larger overlay projects. At each of these sites, stripped material was removed from a control section and stripped material was left in-place for a test section, prior to the placement of the overlay.

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1. Introduction

The deterioration of asphalt concrete in the form of stripping is caused by the separation of the asphalt binder from the aggregate. This loss of adhesion causes the asphalt concrete to ravel under traffic loads. Stripping occurs in the presence of water, so it is often referred to as moisture damage.

Currently, the most common method for rehabilitating a stripped pavement in Montana involves the removal and replacement of most of the stripped material. This practice is expensive because it requires milling and hauling operations in addition to normal paving activities. The Montana Department of Transportation (MDT) initiated this project to investigate the possibility of allowing all stripped material to remain in-place. The premise is that the stripped asphalt could retain sufficient integrity to be used effectively within the rehabilitated pavement structure. The original problem statement, submitted by Jim Weaver (former District Engineer in Missoula, Montana), describes the question at-hand precisely:

> “We presently spend millions of dollars to remove pavement that shows signs of stripping because of fear that it may act as ‘marbles’ and destroy subsequent layers of pavement. We should make an attempt to determine if, in fact, the stripped asphalt is a detriment or if it may add value to the surfacing structure. Is the expenditure of millions of dollars to remove stripped asphalt cost effective?”

2. Scope of Work

This study compared two methods of rehabilitating stripped asphalt pavements by comparing the performance of full-scale rehabilitated pavement structures. The first method of rehabilitation involved the removal and replacement of most of the stripped asphalt concrete. Under this approach, the stripped asphalt concrete layer is presumed to have little structural value (i.e. less than the value of an equivalent thickness of standard base course material). The second method of rehabilitation required minimal treatment of the in-place stripped asphalt concrete layer. Only existing surface treatments and open-graded friction courses were removed and then the stripped asphalt concrete layer was overlaid with a new surface course. This approach presumed that the stripped asphalt concrete maintained a structural value at least as high as a standard base course material.

This study began with a review of current practices concerning both the prevention of stripping in asphalt concrete and the rehabilitation of pavements that have experienced stripping-related damage. In addition to a literature search, a brief questionnaire was distributed to the highway agencies in thirteen states from the northwest and north-central United States. The survey solicited information regarding their experiences with the prevention and rehabilitation of stripping damage. Findings from this initial phase are discussed in detail in Sections 3 and 4. As a brief summary, however, most states have experienced stripping problems and most states have handled the problem by removing the stripped material. Some of these states allow the removed material to be recycled as a portion of new hot-mix and some states do not allow the reuse of stripped asphalt concrete. Oregon was the only state to report the practice of placing an overlay on top of stripped material.

The construction phase of this study began with the identification of five stripped pavements among MDT interstate highway resurfacing projects. Plans for each site involved the implementation of the two rehabilitation methods presented previously. Once sites were identified, historical data were obtained for each site regarding soils information, original structural design, and previous overlays. The condition of the pavements needing rehabilitation was also quantified in general terms.

The five rehabilitated pavement test sections were monitored visually, structurally, and in terms of roughness. Monitoring periods lasted for three to five years.

3. Literature Review

This section gives an overview of the state-of-the-art in asphalt stripping science, including causes, identification, and prevention.

3.1 Description

Asphalt stripping is a phenomenon in which the asphalt binder in an asphalt pavement loses its ability to bond to the aggregate and the pavement material loses its structural integrity. The result is a pavement that fails under ordinary traffic loads. These failures manifest themselves in the form of alligator cracking, potholes and surface raveling, typically progressing from the bottom of the pavement layers up to the top (Kandahl 1992).

3.2 Molecular-Level Causes of Stripping

Stripping of asphalt pavements occurs at the molecular level and is not entirely understood in spite of extensive research. It is thought to be associated with either one or both of two phenomena. First, water can interact with asphalt binder to cause a reduction in cohesion with a subsequent reduction in stiffness and strength of the mix. Second, and more commonly believed, water can get between the asphalt film and the aggregate, break the adhesive bond, and strip the asphalt binder from the aggregate (Hicks 1991).

The nature of the adhesive bond between asphalt and aggregate is a subject of some debate. Adhesion is defined as that physical property or molecular force by which one body sticks to another of another nature (Hicks 1991). Several factors affect the adhesion of the asphalt binder to the aggregate, including: interfacial tension between the asphalt binder and the aggregate, chemical composition of the asphalt binder and aggregate, binder viscosity, surface texture of the aggregate, aggregate porosity, aggregate cleanliness, and aggregate temperature and moisture content at time of mixing (Hicks 1991).

Four general theories of adhesion exist to explain the adhesion of asphalt binder to aggregates. These include the Mechanical Interlocking Theory, the Chemical Reaction Theory, the Surface Energy Theory, and the Molecular Orientation Theory. The actual nature of adhesion is not fully explained by any one of these theories, but is partially explained in each theory (Hicks 1991). A brief description of each theory follows.

Mechanical Interlocking Theory. Mechanical interlocking assumes the absence of chemical interaction between binder and aggregate. The bond strength is assumed to be derived from the cohesion in the binder and interlocking properties of the aggregate particles which include individual crystal faces, aggregate porosity, absorption, surface coating, and angularity (Kiggundu and Roberts 1988).

Chemical Reaction Theory. The chemical reaction theory arises from an observation that stripping is more serious in acidic aggregate mixtures as compared to basic aggregate mixtures. It is suggested that the chemical reaction between most asphalt binders and acidic aggregates is not as strong as the reaction between most asphalt binders and basic aggregates (Hicks 1991).

Surface Energy Theory. When asphalt spreads over and wets an aggregate surface, a change in energy takes place. This change of energy, known as adhesion tension, is a surface phenomenon that depends on the closeness of contact and mutual affinity of the asphalt binder and aggregate (Hicks 1991). The adhesion tension for water to aggregate is higher than that for asphalt binder to aggregate, and consequently water has a tendency to displace the asphalt binder from the aggregate.

Molecular Orientation Theory. The molecular orientation theory states that when asphalt binder comes into contact with an aggregate surface, the molecules in the binder orient themselves so as to satisfy the energy demands of the aggregate. Water molecules are dipolar. Asphalt molecules are generally nonpolar although they contain some polar components. Consequently, water molecules, being more polar, may more readily satisfy the energy demands of an aggregate surface (Hicks 1991)

3.3 Macro-Level Mechanisms of Stripping

Many macro-level mechanisms of stripping have been proposed, but the six most commonly accepted mechanisms are detachment, displacement, spontaneous emulsification, film rupture, pore pressure, and hydraulic scouring. Much debate exists as to the relative contribution of each macro-level mechanism to stripping in individual cases. For instance, it is probable that the predominant stripping mechanisms in a hot-dry environment differ from the mechanisms in hot-wet, cold-dry, and cold-wet environments (Kiggundu and Roberts 1988). Each mechanism occurs as a result of one or more of the molecular-level causes described earlier. A brief description of each mechanism follows.

Detachment. Detachment is the microscopic separation of a binder film from the aggregate surface by a thin layer of water with no obvious break in the binder film. The binder will then peel cleanly from the aggregate. The thin film of water probably results from either aggregate that was not completely dried, interstitial pore water which vaporized and condensed on the surface, or possibly water which permeated through the asphalt film to the interface (Kiggundu and Roberts 1988).

Displacement. Displacement occurs when the binder is removed from the aggregate surface by water. In this type of stripping, as compared to detachment, the free water gets to the aggregate surface through a break in the binder coating. The break may be from incomplete coating during mixing or from binder film rupture (Asphalt Institute 1981).

Spontaneous Emulsification. Spontaneous emulsification occurs when an inverted emulsion (water droplets in binder rather than binder droplets in water as found in common emulsified asphalt) is formed. In its emulsified state, the binder is less tenacious. This mechanism seems to be enhanced under traffic on mixtures laden with free water (Kiggundu and Roberts 1988).

Film Rupture. Film rupture, while not a stripping mechanism on its own, is believed to initiate stripping. Film rupture is marked by fissures that occur under stresses of traffic at sharp aggregate edges and corners where the binder film is the thinnest. Once a break in the film is present, water is able to find its way to the interface and initiate stripping (Asphalt Institute 1981).

Pore Pressure. A build-up of pore pressure is another possible stripping mechanism. Stripping from pore pressure build-up begins when water is allowed to circulate freely through the interconnected voids of a high void asphalt mixture. Traffic effects cause the void space to be reduced and passages between voids to be closed thus trapping water. The continued action of traffic can then cause pore pressures to build up to the point of stripping the binder from the aggregate (Asphalt Institute 1981).

Hydraulic Scouring. Hydraulic scouring occurs more in surface courses than the lower courses of an asphalt pavement. When the pavement is saturated, wheel action causes water to be pressed into the pavement in front of the tires and to be sucked out behind the tires. This water tends to strip the binder from the aggregate. This scouring action can be worsened by the presence of abrasives, such as dust, on the surface of the roadway (Asphalt Institute 1981). This form of stripping causes distress in the form of surface raveling.

3.4 Engineering and Construction Considerations

The initiation of one or more of the previously described stripping mechanisms is attributable to engineering and/or construction problems. These problems include, but are not necessarily limited to, inadequate pavement drainage, inadequate compaction, excessive dust coating on the aggregate, inadequate drying of aggregates, weak and friable aggregates, and the use of waterproofing membranes and seal coats (Kandahl 1992). Each factor will be briefly described below, as will several other possible causes.

Inadequate Pavement Drainage. Inadequate surface drainage and/or subsurface drainage allows the water that is necessary for stripping to occur to remain in the pavement system. Water can enter the pavement layers in numerous ways. Surface water can percolate down from the surface, usually through surface cracks. It can also seep in from the sides and bottom from sources such as ditches or high groundwater. Water can also enter the bottom of the pavement system by the upward forces of capillarity or as rising vapor condensation due to water in the subgrade or subbase (Kandahl 1992).

Inadequate Compaction. A high number of air voids present in the asphalt layers allows the movement of water through these pore spaces. Studies have shown that at less than

4% to 5% air void content, the voids are generally not interconnected and therefore impervious to water (Kandahl 1992). While most asphalt mixes are designed to have 3% to 5% air voids, many agencies allow a maximum air void content of 8% at construction assuming that the remaining compaction will occur under 2 to 3 years of traffic. However, partly due to poor quality control at construction, the design air voids content is never reached. If the pavement remains pervious for an extended period of time, stripping is likely to occur due to ingress of water and hydraulic pore pressures induced by traffic (Kandahl 1992).

Excessive Dust Coating on Aggregate. The problem created by excessive dust coating on the aggregate is two-fold. First, the presence of dust and clay coatings on the aggregate inhibits intimate contact and complete wetting of the aggregate by the asphalt cement. Because the asphalt is adhered to the dust coating and not the aggregate itself, the bitumen is easily stripped from the aggregate. Second, the presence of dust particles enhances the action of scouring under the effects of traffic (Kandahl 1992).

Inadequate Drying of Aggregate. Aggregate that absorbs or adsorbs water will strip easily if not properly dried. This results from the asphalt being displaced from the aggregate by the thin layer of water already present. A dry aggregate surface will have increased adhesion with the asphalt cement compared to a moist or wet surface (Kandahl 1992).

Weak and Friable Aggregate. If weak and friable aggregate is used in an asphalt mix, degradation is possible during rolling and subsequently under heavy traffic. Degradation or delamination exposes uncoated aggregate surfaces which will readily absorb water and initiate the stripping process (Kandahl 1992).

Waterproofing Membranes and Seal Coats. If moisture is present beneath the pavement, then sealing the road surface can be detrimental in terms of stripping. A seal coat or membrane, either on or within the pavement layers, acts as a vapor barrier trapping moisture in the asphalt which facilitates stripping (Kandahl 1992).

Additional Factors. Several additional factors have been suggested to also contribute to stripping, including the use of open-graded friction courses (Kandahl 1992), the use of excess anti-strip additives (Asphalt Institute 1981), the use of siliceous, or "water-loving," aggregates (Asphalt Institute 1981), and the use of aggregates that have relatively high surface potentials, those that impart a high pH value to water in contact with their surfaces (Yoon and Tarrer 1988).

Weather conditions during construction have been related to stripping behavior (Hicks 1991). If the weather is cool and wet during construction, moisture damage is more likely to occur. During a pavement's life, environmental factors such as temperature fluctuations, freeze-thaw cycles, and wet-dry cycles have been suggested to influence stripping (Hicks 1991).

The presence of microorganisms in the binder as well as in the surrounding soil may also contribute to stripping (Ramamurti and Jayaprakash 1987, Ramamurti and Jayaprakash 1992, Brown and Pabst 1988). These asphalt-loving bacteria feed on the asphaltic hydrocarbons, thus creating microscopic tunnels through the binder, which allow water access to the binder/aggregate interface. Water access, coupled with the pumping action of repeated wheel loads, can initiate stripping failures.

All other factors being equal, it is suggested that increased repetitions of traffic loadings accelerates stripping (Hicks 1991).

3.5 Prediction and Identification

The asphalt industry has developed two basic types of tests with regard to asphalt stripping: predictive tests that identify stripping potential in asphalt mixes and raw materials, and identification tests that identify whether stripping is in progress. This section will list the test methods currently being practiced in the industry, as well as by MDT.

Prediction. Numerous laboratory tests exist that are designed to identify paving mixtures that are susceptible to stripping. If a material is identified as being prone to stripping, then the necessary actions can be taken to prevent stripping before it starts.

The most basic requirements of a stripping test are that it fail mixes that will strip in service and pass mixes that will perform well in the field. Tests for stripping potential may be divided into three types (Asphalt Institute 1981): 1) those that require visually estimated stripping damage after prescribed conditioning, 2) those that measure the time-to-disruption of mix specimens stressed in some manner in the presence of water, and 3) those that measure the change in mechanical properties of mix specimens exposed to water in some type of conditioning scheme.

The first category includes boiling tests such as ASTM standard method D3625 (ASTM 2000) and static immersion tests such as AASHTO T182 (AASHTO 2000). These methods require visual estimation of stripping after the prescribed conditioning. In ASTM D3625, loose asphalt concrete is placed in boiling water for 10 minutes. After this conditioning, the mix is inspected to determining whether the percentage of aggregate surface that retains its binder coating is above or below 95 percent. AASHTO T182 is similar, but the loose mix is immersed in distilled water at 77°F (25°C) for a period of 16 to 18 hours.

The second category is exemplified by the Texas Freeze-Thaw Pedestal Test (Kennedy, Roberts, and Lee 1983). In this test a small asphalt-aggregate sample (1-5/8 in. diameter x ¾ in. tall cylinder) is cured for 3 days and is then immersed in distilled water, subsequently frozen at 10°F for 15 hours, and then heated to 120°F for 9 hours. This cycle is repeated until visible cracking develops. The mixture is judged to be susceptible to moisture damage if cracking develops in less than 10 cycles (NCHRP 1991).

The third category includes the largest number of tests currently being practiced. In all of these tests, compacted asphalt-aggregate specimens are exposed to prescribed conditioning regimens. The ratio of the value of a specific mechanical property, such as compressive strength or tensile strength, measured after conditioning and before conditioning provides the gauge for stripping damage potential. The Immersion-Compression Test, ASTM D1075 (ASTM 2000), involves conditioning by soaking in hot water (120°F or 140°F) and testing by unconfined compressive strength. The Marshall-Immersion Test is similar, but the mechanical test is achieved with the Marshall stabilometer.

Several tests have been used that involve indirect diametrical tension as the mechanical test. The Lottman version (Lottman 1978, Lottman 1982) requires compaction of nine 4-inch-diameter specimens to a void content similar to that expected in the field. The specimens are divided into three groups of three. Group 1 specimens are subjected to no conditioning. Group 2 specimens are vacuum saturated with water. Group 3 specimens are vacuum saturated similar to Group 2 specimens and then they are subjected to one freeze-thaw cycle. While Group 2 reflects field performance of up to four years, Group 3 reflects field performance up to twelve years. A minimum tensile strength ratio (Group 2 / Group 1 or Group 3 / Group1) of 0.70 is recommended to ensure adequate field performance (Lottman 1982).

Tunnicliff-Root version (Tunnicliff and Root 1984) of this test requires specimens to be compacted to void contents of six to eight percent. The Group 2 specimens are vacuum saturated to levels between fifty-five and eighty percent. They are then soaked for twenty-four hours in 140°F water. A minimum tensile strength ratio of 0.70 is recommended to ensure adequate field performance. This method was standardized as ASTM D4867 (ASTM 2000), with Group 3 (the freeze/thaw group) as an option.

The Modified Lottman Test, which was standardized as AASHTO T283 (AASHTO 2000), combines the two previous methods. Compaction and vacuum saturation are similar to the Tunnicliff-Root version. Group 2 specimens are subjected to one freeze-thaw cycle, as proposed by Lottman. All specimens are tested for indirect tensile strength. A minimum tensile strength ratio of 0.70 is recommended to ensure adequate field performance.

A final test method for this category (involving changes in mechanical properties) was developed during the Strategic Highway Research Program (SHRP) and was given the SHRP test method designation M-006, " Determining Moisture Sensitivity Characteristics of Compacted Bituminous Mixtures Subjected to Hot and Cold Climate Conditions" (SHRP 1994). This test requires the use of the Environmental Conditioning System (ECS) that was also developed during SHRP (Al-Swailmi and Terrel 1992). To represent field conditions, asphalt concrete samples are exposed to both wetting and repeated axial loads. Both warm- and cold-climate conditioning can be performed, depending on the geographic region of interest. Specimens are monitored for changes in resilient modulus (axial), air permeability, and water permeability. At the conclusion of the test, the specimens are tested for indirect tensile strength and they are inspected visually for stripping (by estimating the percent of exposed aggregate surface). As of the year 2002, this test had not been given an AASHTO or ASTM test method designation.

Numerous groups have studied the merits of the various test methods (Kandahl 1992, Kiggundu and Roberts 1988, Parker 1987, Maupin 1989, Shatnawi, Nagarajaiah, and Harvey 1995, and Bruce 1990) have all evaluated the relative merits of some or all of the available test methods. Kiggundu and Roberts (1988) go so far as to rank the methods tested in their study as follows (in decreasing order of usefulness): 1) Lottman test, 2) Tunnicliff-Root test, 3) 10-Minute Boil test, 4) Immersion-Compression test, and 5) Nevada Dynamic Strip test. Kandahl (1992) states that "the Modified Lottman Test is the most appropriate test method available at the present time to detect moisture damage [-potential] in HMA mixes." The Montana Department of Transportation (MDT) primarily uses the Modified Lottman procedure for its own testing.

Identification. Testing of in-situ materials is not standardized throughout the industry and tends to be very subjective. Effective and reliable identification is difficult because the types of pavement distress caused by stripping, such as raveling or rutting, may be caused by other non-hydraulic mechanisms. Also, field data on stripping tends to describe pavement distress in subjective terms that are difficult to normalize between locations when gathered by different individuals. Due to the subjectivity and widespread differences of the available practices, only the MDT method will be described.

The MDT procedure for evaluating stripped asphalt involves visual inspection of asphalt cores. This procedure calls for a core to be taken from the pavement section in question, split diametrically, visually evaluated, and rated on a scale from zero to four. To ensure consistency, the ratings are produced by comparing cored materials to photographs and written descriptions that are associated with each of the five rating numbers. A rating of four indicates a core without stripping damage. The split surface should be shiny and black, with all aggregate particles coated by binder. A rating of zero indicates a core with severe stripping damage. Either binder is absent from most aggregate surfaces or the asphalt material disintegrates during coring operations. This rating procedure is part of Montana test method MT-331, “Method of Sampling and Evaluating Stripping Pavements,” which is included as Appendix A in this report.

3.6 Prevention or Minimization

Whether building a new pavement or overlaying or recycling an old one, measures should be taken to prevent or minimize stripping when stripping occurrence is probable. Prevention or minimization can be achieved by avoiding the causes discussed earlier. Published literature generally categorize stripping precautions into (Asphalt Institute 1981): 1) material selection, 2) construction practices, and 3) the use of anti-strip additives.

By choosing materials that are less prone to stripping, some stripping failures can be averted. Certain types of aggregates are especially prone to stripping (Hicks 1991) and should be avoided. Certain types and grades of asphalt binders as well as some asphalt-aggregate combinations are likely to strip (Hicks 1991) and should also be avoided.

Good construction practices are essential to building pavements that will not strip. Particularly important is the need to ensure thorough compaction of the asphalt mat to minimize pore space and thus permeability (Hicks 1991, Asphalt Institute 1981).

While some agencies specify the use of anti-strip additives, either chemical or lime, in all new asphalt mixes, it is probably only necessary to use additives in those mixes that contain materials that are known to be prone to stripping (Kandahl 1992, Tunnicliff and Root 1984).

3.7 Rehabilitation

Rehabilitation of a stripped pavement currently includes several possible remedial alternatives. The simplest of these is to overlay the stripped pavement as is with a new asphalt mat. Variations of this simple overlay method involve the repair of deteriorated sections of the old pavement, milling of the existing surface course, placement of a leveling course, or placement of a paving fabric, all before an overlay is placed.

The most widely practiced method of rehabilitation involves the complete removal and replacement of the stripped pavement. This is expensive and wasteful, particularly if the material is not used in other paving projects that allow recycling. However, it is the only method that ensures that all stripped and stripping-prone material is removed.

The most important developments in the practice of pavement rehabilitation involve recycling. Recycling is important because of the reduced environmental impacts associated with reusing the failed material. Many methods of recycling are currently being practiced, including various methods of cold and hot recycling. Cold recycling (Epps et al 1980, Canessa 1995, Fanning and Day 1995) can include planing or milling of the existing pavement, followed by reuse of the milled material, typically as a pavement base material.

Hot recycling includes the removal of the failed material, usually by milling. The milled material is then either reused immediately, i.e. hot-in-place recycling (Button et al 1994, Button et al 1995) or it is run back through a hot-mix plant at a future date (Wyoming SHD 1995). Hot in-place recycling is a process that allows the failed pavement to be rehabilitated in-situ. In some cases now, the process can be completed in a single pass. This is beneficial with respect to minimizing disruption of traffic. Current limitations of hot in-place recycle techniques include expensive equipment costs and limited depth of effective rehabilitation.

Some concerns have been raised as to whether the presence of stripped material in a recycled asphalt mix will accelerate stripping in the new pavement. A study by Amirkhanian and Burati (1992) at Clemson University shows that the use of recycled material in an asphalt mix does not create increased risk of moisture damage in the recycled pavement. However, rehabilitation should include determining the cause of stripping, as well as the proper preventive measures for future stripping. Preventive measures will likely involve improving drainage around the site of the stripping failure (Kandhal and Rickards 2001).

4. State Survey

In the early stages of this investigation, questionnaires were sent to state highway agencies in the northwest and north-central United States. The questionnaires pertained to their experiences with asphalt stripping. The purpose was to gain insight as to other state's impressions on the severity of stripping problems, as well as their methods of dealing with the problem.

The sampling of the states was chosen to approximately reflect the type of climate found in Montana, specifically the severe fluctuations between hot and cold. The results were not intended to provide a scientific or statistically significant result, only to provide a brief representation of techniques currently being applied in the area of asphalt stripping. Surveys were sent to thirteen state agencies, of which three did not respond. Responses were received from agencies in the following ten states: Colorado, Idaho, Minnesota, Nebraska, Nevada, North Dakota, Oregon, Utah, Washington, and Wyoming.

This section will briefly summarize the results of the survey and highlight the most interesting feedback.

Question Number 1. The first question inquired if the state had experienced stripping related problems on its roadways and also asked for a brief explanation. North Dakota was the only state that indicated they had experienced no stripping related problems and cited the use of strip resistant aggregates as the reason. All other states indicated at least some degree of experience with asphalt stripping; the reasons cited for the problem varied widely. Colorado and Nebraska indicated that stripping problems were exasperated by moisture trapped beneath seal coats. Other popular causes of stripping included: incompatibility between aggregates and asphalt binder, coating of fine particles on aggregates prior to mixing, incomplete mixing of the aggregate and asphalt binder, and poor compaction of the asphalt concrete mat.

Question Number 2. The second question inquired into a state's current techniques for rehabilitating a pavement that has experienced a stripping failure. The overwhelming response was to remove the stripped asphalt and to replace it with new material using either some or all virgin material. Wyoming and Idaho both hot-recycle stripped asphalt concrete, with Wyoming indicating extensive use. Nebraska and Nevada do not recycle any stripped material. The remaining respondents all indicated some stockpiling and reuse of stripped material, either at some percentage in new asphalt concrete or as base material.

Lime was the most popular additive mentioned to minimize stripping problems in new pavements. Idaho was the only state to indicate the use of a liquid chemical anti-strip additive.

Question Numbers 3 and 4. Questions 3 and 4 asked the states if their chosen method of rehabilitation was effective and what changes they planned to make in the future. Surprisingly, in spite of the on-going problem of stripping, most states expressed satisfaction with their current method of rehabilitation, expected to see full design life from rehabilitated pavements, and indicated no plans to alter their methods in the future.

Question Number 5. Lastly, questionnaire recipients were asked to list any methods for rehabilitating a stripped pavement that they would like to try or like to see tested if the means were available to them. Interestingly, only four agencies replied to this question. All four, however, indicated that they wished to see more done with recycling, and two referred specifically to hot in-place recycling.

Summary. The survey results confirm that stripping is a common form of deterioration for asphalt concrete in the northwestern United States. The phenomenon of stripping is not completely understood. No single rehabilitative method has universal application or acceptance.

5. Test Sites

Five test sites were selected on interstate highways across the state of Montana. All the test sites were constructed as part of larger pavement rehabilitation contracts. Test sites were chosen based on the following criteria:

1. the sites had to be well-distributed across the state,
2. each pavement had to be determined as stripped according to MDT test procedures, and
3. the timing for rehabilitation construction had to be appropriate.

The test sites are listed in Table 1 and their locations are shown in Figure 1. The rehabilitation projects were conducted between 1994 and 1997. The total lengths of the rehabilitation projects ranged from five miles to seventeen miles. Among the chosen test sites, the Rocky Canyon area experiences the most precipitation, while the Lincoln Road-Sieben area experiences the least. The Custer County Line West area experiences the hottest summers. The Tarkio-East area experiences relatively mild winters. The five sites rank in the following order, in terms of both increasing average daily traffic (ADT) and increasing equivalent single-axle loads (ESALS): Custer County Line West, Lincoln Road-Sieben, Tarkio-East, Bearmouth-Drummond, and Rocky Canyon.

Table 1. General Information for Test Sites

Characteristic	Bearmouth-Drummond	Rocky Canyon	Lincoln Road-Sieben	Custer County Line West	Tarkio-East
Project Number	IM 90-3(74) 135	IM 90-6(70) 313	IM 15-4(69) 200	IM 92-4(49) 154	M 90-1(118) 64
County	Granite	Gallatin	Lewis and Clark	Custer	Mineral
Interstate	I-90	I-90	I-15	I-94	I-90
Length of Project (mi)	15.2	5.3	17.1	8.9	10.5
Date of Rehabilitation	1994-95	1995	1996	1996	1996-97
Climatic Data					
Mean Precipitation (in.)	13.5	18.6	11.4	14.0	14.5
Mean Temperatures[a] (ºF)					
7-Day-Average High[b]	91 (3.0)	88 (4.8)	88 (4.8)	97 (4.6)	93 (4.7)
Single-Day Low[b]	-27 (4.2)	-24 (3.4)	-26 (3.1)	-29 (3.8)	-15 (3.5)
Traffic Data					
Average Daily Traffic, ADT (Year)	6800 (1991)	8100 (1991)	3700 (1993)	2800 (1991)	4500 (1991)
ADT[c] (Letting Year), Percent Trucks	7500 (1993), 20.2%	9200 (1996), 20.3%	4000 (1996), 17.2%	3000 (1993), 20.7%	5000 (1993), 23.8%
ADT (2002), Percent Trucks	7360, 26.7%	14480, 14.9%	3600, 20.7%	2870, 32.2%	7150, 25.6%
Average Daily ESALs (2002)	1599	1634	677	748	1602

[a] From SHRP weather database (at least 20 years of data)
[b] mean (standard deviation)
[c] projected

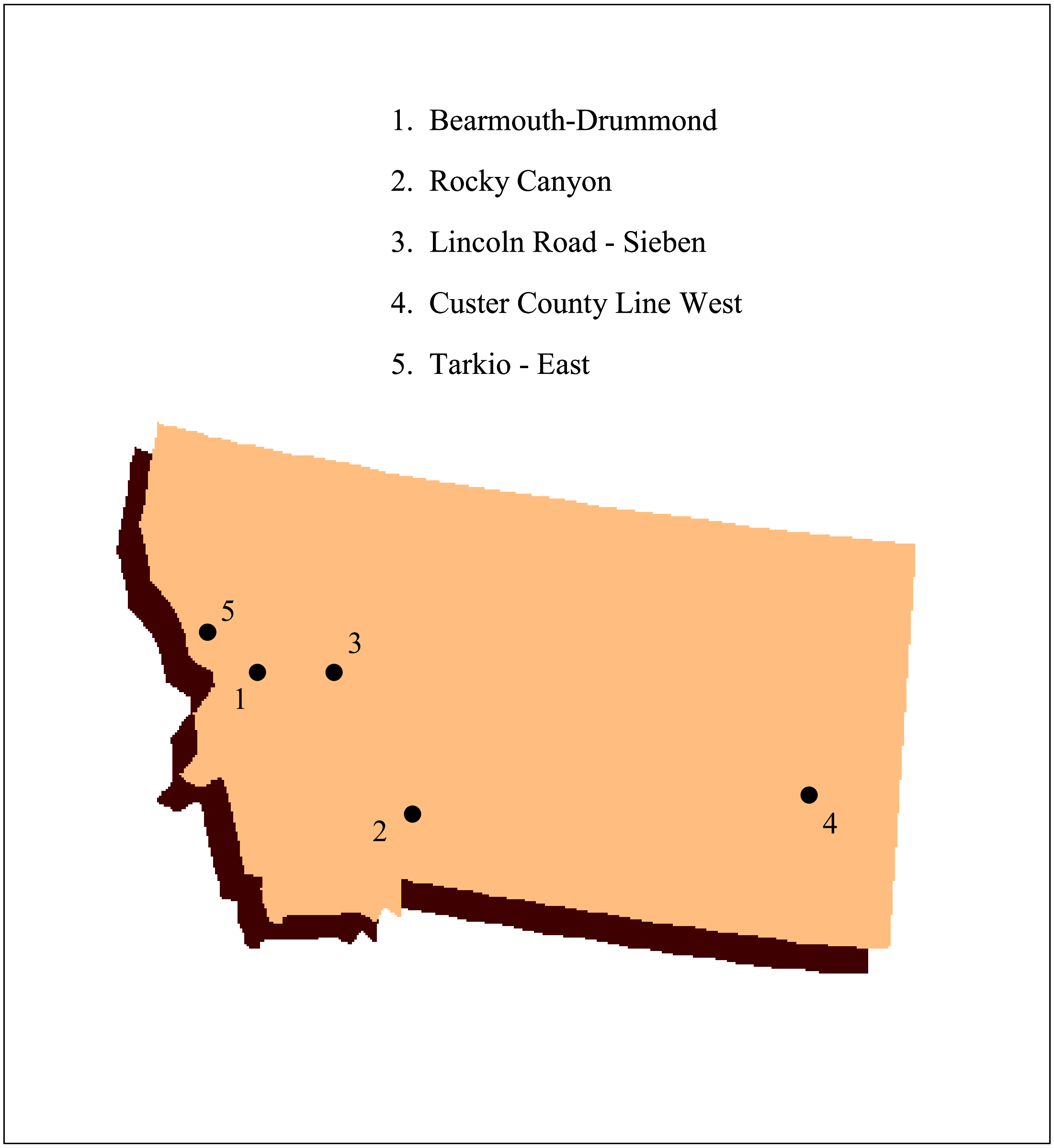

Figure 1. Locations of the Five Test Sites Within the State of Montana

Each test site included one or two test sections and one or two control sections. The control section(s) employed the remove-and-replace method of rehabilitation, while the test section(s) incorporated all the existing stripped asphalt concrete into the new structure. The test and control sections are between 500 and 1370 feet in length and include both the driving and passing lanes. Locating test and control items adjacent to each other minimized unwanted variables minimized by maintaining similarities between:

1. existing soil conditions,
2. existing pavement structure dimensions and properties,
3. drainage,
4. horizontal and vertical road geometry, and
5. types and severity of pavement distress.

The Bearmouth-Drummond test site is located on Interstate 90 in Granite County, just west of Drummond, Montana. The site includes both a control and a test section in each of the eastbound and westbound lanes, as shown in Figure 2. Each control section is west of each test section. Each control section is 500 feet long, starting at milepost 149.76 (STA 842+81.9) and ending at milepost 149.86 (STA 847+81.9). Each test section is also 500 feet long, starting at milepost 149.86 (STA 847+81.9) and ending at milepost 149.96 (STA 852+81.9). A one-inch diameter steel rod was placed at the beginning and end of the control and test sections on the north side of the westbound lane, along the right-of-way fence.

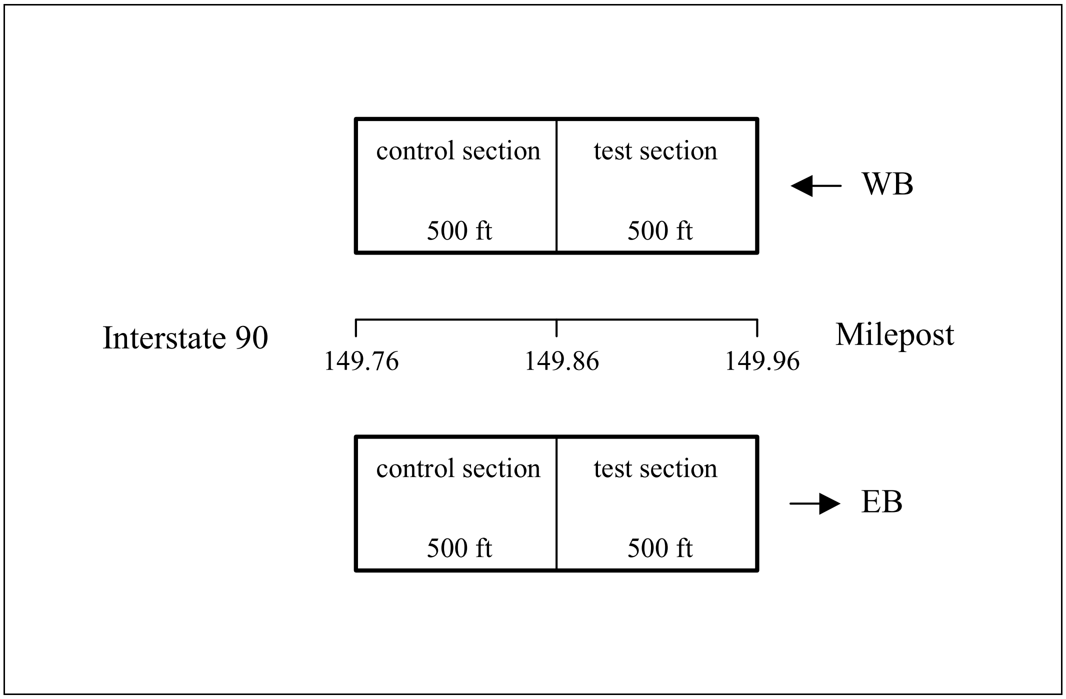

Figure 2. Layout of the Test Site at Bearmouth-Drummond

The Rocky Canyon test site is located on Interstate 90 in Gallatin County, just east of Bozeman, Montana. Both the control and test sections are in the eastbound lane, separated by the bridge at the Bear Canyon Interchange (Figure 3). The test section is west of the bridge, while the control section is east of the bridge. The test section is 1370 feet long, starting at milepost 313.22 (STA 218+77.3) and ending at milepost 313.48 (STA 232+47.7). The control section is also 1370 feet long, starting at milepost 313.50 (STA 233+60.7) and ending at milepost 313.76 (STA 247+30.7). A one-inch diameter steel rod was placed at the beginning and end of the control and test sections on the south side of the eastbound lane, along the right-of-way fence.

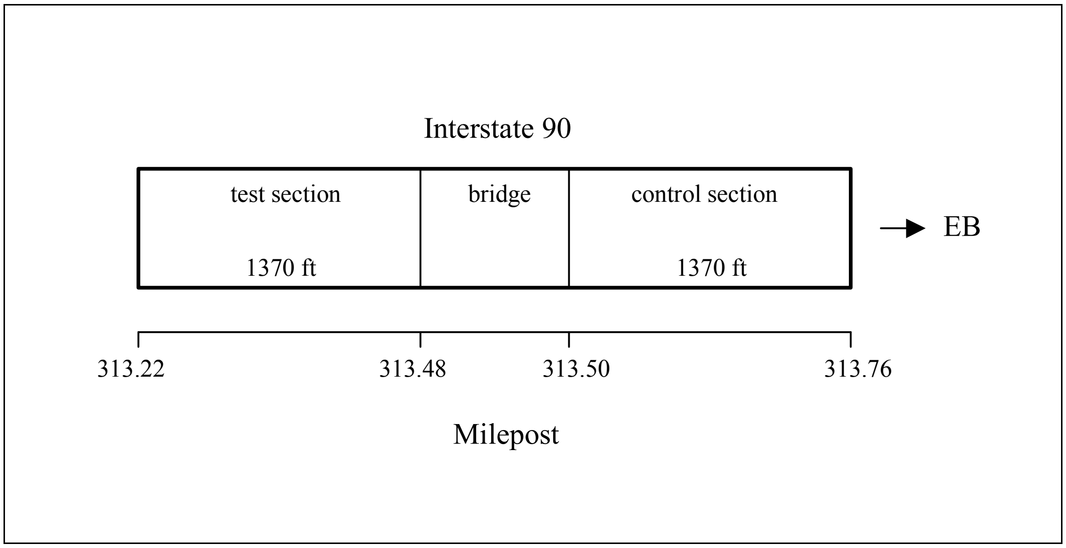

Figure 3. Layout of the Test Site at Rocky Canyon

The Lincoln Road-Sieben test site is located on Interstate 15 in Lewis and Clark County, just north of Helena, Montana. Both the control and test sections are in the northbound lane, as shown in Figure 4. The test section is south of the control section. The test section is 1320 feet long, starting at milepost 201.00 (STA 667+50) and ending at milepost 201.25 (STA 680+70). The control section is also 1320 feet long, starting at milepost 201.25 (STA 680+70) and ending at milepost 201.50 (STA 693+90). A one-inch diameter steel rod was placed at the beginning and end of the control and test items approximately twenty feet off the shoulder.

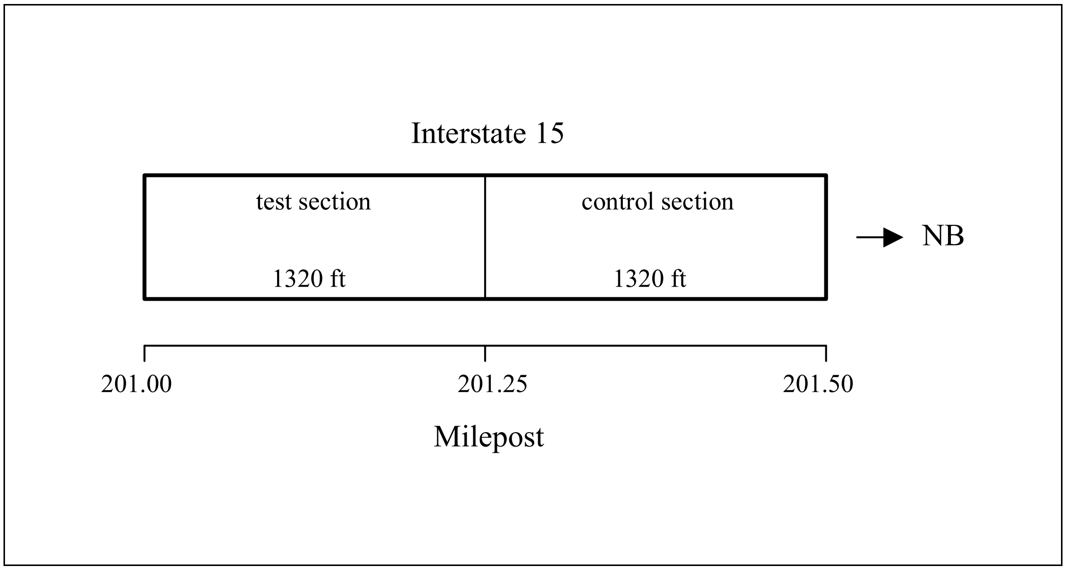

Figure 4. Layout of the Test Site at Lincoln Road-Sieben

The Custer County Line West test site is located on Interstate 94 in Custer County, approximately fifteen miles east of Miles City, Montana. Both the control and test sections are in the westbound lane, as shown in Figure 5. The test section is west of the control section. The test section is 1320 feet long, starting at milepost 157.75 (STA 439+39) and ending at milepost 158.00 (STA 452+59). The control section is also 1320 feet long, starting at milepost 158.00 (STA 452+59) and ending at milepost 158.25 (STA 465+79). A one-inch diameter steel rod was placed at the beginning and end of the control and test items approximately fifty feet off the shoulder.

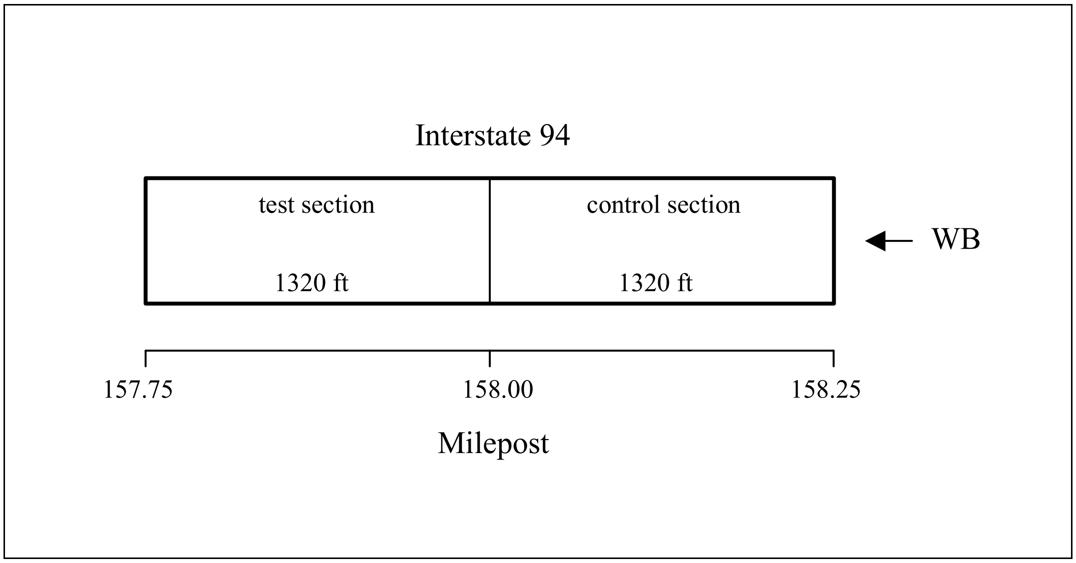

Figure 5. Layout of the Test Site at Custer County Line West

The Tarkio-East test site is located on Interstate 90 in Mineral County, just east of Tarkio, Montana. Both the control and test sections are in the eastbound lane, as shown in Figure 6. The control section is west of the test section. The control section is 1320 feet long, starting at milepost 70.50 (STA 627+18.1) and ending at milepost 70.75 (STA 668+60). Test section is also 1320 feet long, starting at milepost 70.75 (STA 668+60) and ending at milepost 71.00 (STA 681+80). A one-inch diameter steel rod was placed at the beginning and end of the control and test items approximately ten feet off the shoulder.

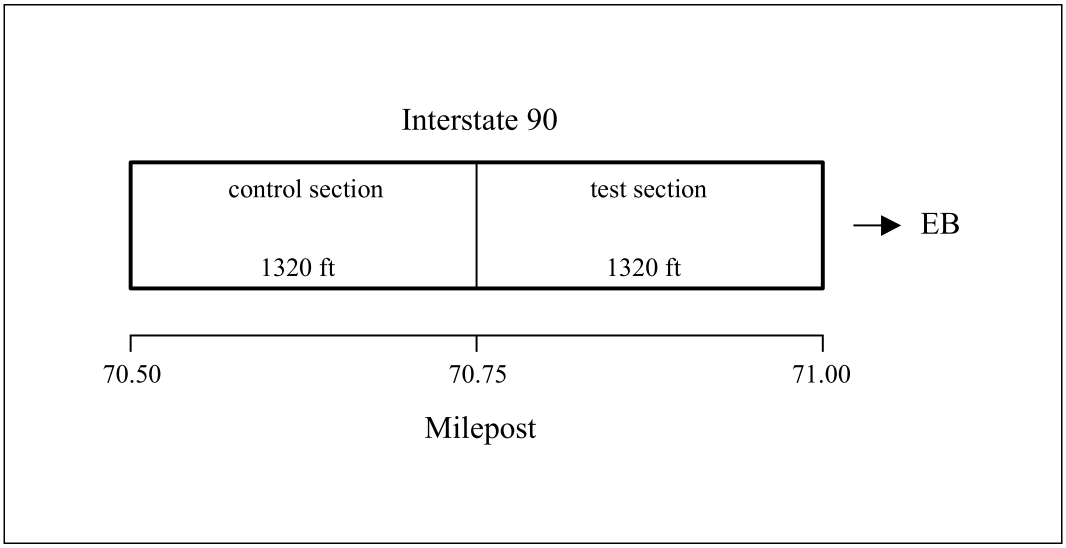

Figure 6. Layout of the Test Site at Tarkio-East

In addition to marking the test sites with steel rods, masonry nails were placed in the roadway at the beginning and end of each experimental section. The nails were placed approximately two feet from the outside shoulder stripe, towards the edge of the paved surface.

Information related to previous construction for the test sites is summarized in Table 2. The original pavement structures were built between 1964 and 1982. Each original structure included two layers over the subgrade: a crushed aggregate base course and an asphalt concrete surface course. All of the reported subgrade types could be expected to provide adequate pavement foundations, without potential for volume changes. Each site had received a single overlay since original construction. The overlays were placed between 1981 and 1985 and each was topped with an open-graded friction course.

Table 2. Pre-Rehabilitation Construction Information for Test Sites

Characteristic	Bearmouth-Drummond	Rocky Canyon	Lincoln Road-Sieben	Custer County Line West	Tarkio-East
Original Pavement					
Date of Construction	1971	1964	1964	1971	1982
AC Thickness (ft)	0.35	0.35	0.50	0.35	0.35
CAB Thickness (ft)	1.7	2.0	2.4	1.5	1.5
Subgrade Type[a]	A-1-a (GW, GP)	A-2-4 (GM, SM)	A-2-4 (GM, SM)	A-1-b to A-4(0) (SW, SP, SM, ML)	A-1-a to A-3 (GW, GP, SP)
First Overlay					
Date of Construction	1985	1983	1983	1981	1985
AC Thickness (ft)	0.15	0.30	0.15	0.15	0.15
OGFC (ft)	0.05	0.05	0.05	0.05	0.05

Note: AC = asphalt concrete; CAB = crushed aggregate base; OGFC = open-graded friction course (approximately ½ in. to ¾ in. thick)

[a] AASHTO Classification with most probable soil in Unified System (after Das 1990) shown in parentheses.

5.1 Pre-Rehabilitation Evaluations

Prior to any rehabilitation project, MDT evaluates the condition of the existing pavement. For the sites included in this project, the non-destructive testing (NDT) team evaluated the structural condition of pavements and documented their impressions on any visual distress. An additional source of pre-rehabilitation information was the MDT pavement management system (PvMS), which retains condition data for all state-maintained pavements within Montana. As a supplement for these sources, representatives of Montana State University (MSU) visited the sites and performed visual inspections specifically for the test sections.

Structural Evaluations. Structural information was obtained with a Road Rater, which provided estimates for the elastic moduli of pavement layers. Details of the method by which the Road Rater estimates moduli will be provided in Section 5.3, titled “Pavement Performance Monitoring.” The reduction of Road Rater data assumes that the pavements consist of multiple layers of linear elastic materials. The reported modulus values, assuming three linear elastic layers for each pavement, are shown in Table 3. These values are conservative estimates for each entire project because they were obtained by subtracting seven-tenths of the calculated standard deviation from the calculated mean. The NDT Team impressions on material adequacy, based on estimated moduli, are also shown in Table 3. Surface moduli were lowest at Custer County Line West and were highest at Bearmouth-Drummond, although all were judged by be either “adequate” or “good.” Base course moduli were lowest at Lincoln Road-Sieben (judged to be “weak”) and were highest at Bearmouth-Drummond. Subgrade moduli were lowest at Bearmouth-Drummond and Tarkio-East (both judged to be “weak”) and were highest at Rocky Canyon. Only two sites had both a base course modulus and a subgrade modulus that were judged to be either weak or marginal: the Lincoln Road-Sieben and the Custer County Line West sites.

Table 3. Pre-Construction Road Rater Evaluations

Characteristic	Bearmouth-Drummond (westbound)	Bearmouth-Drummond (eastbound)	Rocky Canyon (eastbound)	Lincoln Road-Sieben (northbound)	Custer County Line West (westbound)	Tarkio-East (eastbound)
Date Tested	July 1992	July 1992	July 1991	May 1990	Sept. 1992	Aug. 1992
Location of Tests (Range of Mileposts)	135 to 150	135 to 150	313 to 318	200 to 217	155 to 163	64 to 74
Surface Modulus[a] (psi)	324,000 [Good]	216,000 [Good]	248,000 [Good]	216,000 [Good]	191,000 [Adequate]	233,000 [Good]
Base Modulus[a] (psi)	36,700 [Good]	27,900 [Good]	25,100 [Good]	14,600 [Weak]	21,200 [Marginal]	29,800 [Good]
Subgrade Modulus[a] (psi)	5,090 [Weak]	9,750 [Marginal]	11,900 [Good]	6,670 [Weak]	6,420 [Weak]	5,210 [Weak]

Note: Modulus values presented = mean – 0.70(standard deviation)

[a] NDT Team impression on the quality of material is shown in brackets.

Visual Examinations. Visual condition information that was obtained from the MDT is summarized in Table 4. This information reflects the condition of the entire projects; not just the test sections. All sites were experiencing some degree of raveling of the open-graded friction course (OGFC). Bearmouth-Drummond, Lincoln Road-Sieben, and Tarikio-East sites were all found to have transverse cracks. Lincoln Road-Sieben also had longitudinal cracking throughout its length. None of the sites had extensive pothole problems. With exception for Lincoln Road-Sieben, all sites had some measurable rutting. All sites had some degree of fatigue cracking; Tarkio-East had the most.

Table 4. Distress Information for the Total Project Length

Distress	Bearmouth-Drummond	Rocky Canyon	Lincoln Road-Sieben	Custer County Line West	Tarkio-East
Raveling of OGFC	5% coarse	70% medium to coarse	50% fine	10% coarse	40% coarse
Transverse Cracks	100% 1-3 cracks per 100 ft (< ? ” to ¼”)	None reported	100% 5 cracks per 100 ft (? ” to ¼”)	None Reported	40% 2 cracks per 100 ft (? ” to ¼”)
Longitudinal Cracks	2% (? ” to ¼”) centerline	None Reported	100% centerline	None Reported	None Reported
Potholes / Patches	Isolated spots (fair condition)	None Reported	None Reported	< 10% patched (good condition)	None Reported
Ruts	100% (½”)	25% (½” to ¾”)	0%	25% (½” to ¾”)	100% (½” to ¾”)
Fatigue Cracking	2% initial stage	10% at some stage	> 15% initial stage	20% at some stage	40% initial stage
Date of NDT Team Visit	July 1992	July 1991	May 1990	Sept. 1992	Aug. 1992
Note: All percentages indicate percent of total project area or length.					

Pavement condition information, obtained by MSU specifically for the test sections, generally agreed with the MDT findings. The MSU findings are summarized in the following paragraphs. This section will conclude with a characterization of each site in terms of its predominate distress.

At Bearmouth-Drummond, the areas of raveling for the OGFC were approximately five percent for the driving lanes and nine percent for the passing lanes. Relative to the MDT findings for the entire project, rutting within the test sections did not appear to be as severe. Ruts of one-half to three-quarters of an inch were evident in less than five percent of the wheel paths. Alligator cracking was also observed in less than five percent of the wheel paths. Similar to the MDT findings, transverse cracking was found throughout the test sections; it had progressed to moderate severity in many cases. Longitudinal cracks were also found within the test sections, generally of low severity.

By 1995 when MSU performed their visual condition survey, the alligator cracking at Rocky Canyon had worsened from the time of the MDT inspection in 1991. Alligator cracking, or at least longitudinal cracks in the wheelpaths were observed along most of the test site. Transverse cracks and a longitudinal crack between paving lanes were observed, although they were generally low-severity.

A description of the test site at Lincoln Road-Sieben would be similar to that provided by MDT for the entire project. Some slight modifications follow. The longitudinal cracks at the centerline were severe for approximately a third of the project length. Parallel cracks had formed and raveling had become severe within one foot of the longitudinal crack. Low-severity rutting was also noted to have occurred in the driving lane.

Although cracks were not mentioned at the time of the NDT Team visit to Custer County Line West, many transverse cracks had formed within the test site by the year 1995. Moderate- to high-severity cracks were observed on the order of three per 100 feet of pavement length. Isolated potholes, both unimproved and patched, were found in the driving lane. The potholes accounted for less than ten percent of the pavement area. Low-severity rutting was also noted to have occurred in the driving lane.

The condition of the original pavement at the Tarkio East test site was generally poor. Similar to the NDT Team report, raveling and alligator cracking were found to be extensive. Ruts were also found in both the driving and passing lanes. Although MDT had found transverse cracks along the project, they seemed to be nearly absent within the test sections. However, the test site did have longitudinal cracking between paving lanes throughout its length. The longitudinal cracks had promoted additional deterioration in the form of small parallel cracks and intermittent potholes.

Distress types found at the various sites are summarized in Table 5. Those found by MDT during their inspections of the entire projects are indicated with a “P.” Those found by

MSU during their inspections of the test sections are indicated with a "T." Accounting for both MDT and MSU findings, the predominate forms of distress at the various test sites are also summarized in Table 5. Bearmouth-Drummond, Lincoln Road-Sieben, and Custer County Line West had extensive transverse cracking and/or longitudinal cracking. Rocky Canyon and Tarkio-East had extensive rutting and/or fatigue cracking.

Table 5. Summarized Distress Information for Entire Projects and Test Sites

Distress	Bearmouth-Drummond	Rocky Canyon	Lincoln Road-Sieben	Custer County Line West	Tarkio-East
Raveling of OGFC	P, T	P, T	P, T	P, T	P, T
Transverse Cracks	P, T	T	P, T	T	P
Longitudinal Cracks	P, T	T	P, T	None Reported	T
Potholes / Patches	P	None Reported	None Reported	P, T	T
Ruts	P, T	P, T	T	P, T	P, T
Fatigue Cracking	P	P, T	P	P	P, T
Predominate Distress for Test Sections	Transverse Cracking	Fatigue Cracking	Transverse and Longitudinal Cracking	Transverse Cracking	Ruts and Fatigue Cracking

Notes:
P – distress observed during MDT inspection of the entire project
T – distress observed during MSU inspection of the test site

Stripping Evaluations. The MDT procedure for evaluating asphalt cores for stripping involves the visual inspection of core faces produced by indirect tensile splitting. Four-inch-diameter cores were removed from within the projects included in this study in order to quantify the levels of stripping. If a core disintegrated during removal, this condition was noted. If the core remained intact during removal, the various asphalt concrete layers were separated. The core for each layer was then split along its diameter by indirect tension. Finally, the degree of stripping for each lift was estimated by inspecting the two exposed faces. The MDT procedure uses an integer rating scale, ranging from zero to four, as shown in Table 6. To minimize subjectivity, MDT maintains a reference booklet of color photographs showing split core faces, along with their designated ratings.

Table 6. MDT Rating Scheme for Stripping Damage in Cores

Core Rating	Description
0 (no core)	Asphalt is mostly gone from all sizes of aggregate or the core has disintegrated.
1 (severely stripped)	Most of the aggregate is so clean, the colors of the rock are decipherable.
2 (stripping)	In addition to moisture damage, some large aggregate is not coated.
3 (moisture damaged)	Loss of sheen; dull appearance; some smaller aggregate (minus No. 60 sieve) is uncoated.
4 (good core)	The face is shiny and black. All aggregate particles are coated.

Results from the inspections for stripping damage are summarized in Table 7. According to the MDT procedures, all five sites were experiencing severe stripping damage. Generally, the overlays received ratings that were similar to or worse than those for the original asphalt concrete surface layer. There were not substantial differences between the driving lanes and the passing lanes. In terms of severity of stripping, the sites grouped as follows: Custer County Line West and Tarkio-East had the worst ratings, Rocky Canyon and Lincoln Road-Sieben had intermediate ratings, and Bearmouth-Drummond had the least severe ratings.

Table 7. Evaluation of Cores Removed Prior to Rehabilitation

Characteristic	Bearmouth-Drummond (westbound)	Bearmouth-Drummond (eastbound)	Rocky Canyon (eastbound)	Lincoln Road-Sieben (northbound)	Custer County Line West (westbound)	Tarkio-East (eastbound)
Core Stripping Ratings[a]						
Original Surface						
- Driving Lane	2.2 (2-3)	2.0 (2)	1.2 (1-2)	1.8 (1-2)	0.5 (0-2)	1.0 (1-2)
- Passing Lane	2.0 (2)	2.3 (2-3)	1.3 (1-2)	1.7 (1-2)	1.6 (0-3)	0.5 (0-2)
Overlay						
- Driving Lane	1.7 (1-2)	1.2 (0-2)	1.0 (1)	1.3 (1-2)	1.0 (1)	0.6 (0-2)
- Passing Lane	1.8 (1-2)	1.8 (1-2)	1.0 (1)	1.7 (1-2)	0.6 (0-2)	0.2 (0-2)
Number of Cores Evaluated	14	14	12	12	9	32
Additional Core Tests[b]						
Voids Total Mix (%)						
- Original Surface	3.6	3.6	6.1	7.2	3.3	4.0
- Overlay	5.3	6.0	6.6	5.7	7.1	5.4
Binder Content[c] (%)	6.0	5.5	6.1	6.2	5.9	5.5
Location of Cores (Mileposts)	145 to 150	145 to 150	313 to 318	201 to 206	155 to 163	64 to 74

[a] mean (range)
[b] only one test completed for each characteristic
[c] bulk mixture containing both the original mixture and the overlay

Note: Voids total mix were measured using bulk SSD (AASHTO T166) and rice (AASHTO T209) specific gravities. Binder content was measured by solvent-based extractions (AASHTO T164).

In addition to using the cores for stripping damage ratings, a few cores were used to obtain estimates of voids and binder content. Measured void contents for the original asphalt concrete surfaces and the overlays ranged from 3.3 percent to 7.2 percent (see Table 7). Binder contents ranged from 5.5 percent to 6.2 percent (see Table 7). With the few replicates used in this part of the study, it can only be stated that no substantial oddities or differences were found among the test sites.

5.2 Rehabilitation Scenarios

At each site, rehabilitation construction began with milling operations. In the control sections, milling depths ranged from two and a half to five inches, as shown in Table 8. Milling was deep enough to remove the existing OGFC and the existing overlay. With exception for the Tarkio-East site, milling in the control section was deep enough to penetrate into the asphalt concrete that was placed as part of the original pavement. In the test sections, milling was only used to remove the OGFC.

Table 8. Details of Rehabilitation Construction

Distress	Bearmouth-Drummond	Rocky Canyon	Lincoln Road-Sieben	Custer County Line West	Tarkio-East
Control Section					
Cold Mill,[a] ft (in.)	0.30 (3.5)	0.40 (5.0)	0.25 (3.0)	0.25 (3.0)	0.20 (2.5)
Improved Existing Material[b]		0.80 ft CTPB			
New Material (First Lift)	0.15 ft PMS (hot recycle)	0.40 ft PMS (polymer-mod.)	0.15 ft PMS (hot recycle)	0.25 ft PMS	0.15 ft PMS (hot recycle)
New Material (Top Lift)	0.15 ft PMS (polymer-mod.)		0.15 ft PMS (polymer-mod.)	0.40 ft PMS (polymer-mod.)	0.20 ft PMS (polymer-mod.)
Surface Treatment[c]	Seal & Cover	Seal & Cover	Seal & Cover	Seal & Cover	Seal & Cover
Change in Pavement Thickness, ft (in.)	0.00 (0.0)	0.00 (0.0)	+0.05 (0.5)	+0.40 (5.0)	+0.15 (2.0)
Test Section					
Cold Mill,[d] ft (in.)	0.05 (0.5)	0.05 (0.5)	0.05 (0.5)	0.05 (0.5)	0.05 (0.5)
Overlay	0.15 ft PMS (polymer-mod.)	0.30 ft PMS (polymer-mod.)	0.15 ft PMS (polymer-mod.)	0.40 ft PMS (polymer-mod.)	0.20 ft PMS (polymer-mod.)
Surface Treatment[c]	Seal & Cover	Seal & Cover	Seal & Cover	Seal & Cover	Seal & Cover
Change in Pavement Thickness, ft (in.)	+0.10 (1.0)	+0.25 (3.0)	+0.10 (1.0)	+0.35 (4.0)	+0.15 (2.0)

PMS – plant-mix surface
CTPB – cement-treated pulverized base

[a] deep enough to penetrate past the existing overlay and into the original plant mix surface
[b] cement-stabilized the remaining plant-mix surface and part of the gravel base
[c] Grade 4A aggregate
[d] to remove the open-graded friction course

Note: All asphalt concrete mixtures are Grade D, with exception for the surface layer at Custer County Line West, which is Grade S. The design of all asphalt concrete mixtures included the Modified Lottman Test (AASHTO T283) to ensure stripping resistance.

Rocky Canyon was the only site that involved stabilization of material prior to the placement of the overlay. After milling the control section at Rocky Canyon, nine and a half inches of the remaining material was pulverized and stabilized with portland cement. This one and a half inches of material included about three and a half inches of asphalt concrete and about six inches of underlying aggregate base. The test section did not involve any stabilization.

Overlay thicknesses in control sections ranged from 3.5 to 8 inches, as shown in Table 8. Overlay thicknesses in test sections ranged from 2 to 5 inches (Table 8). All top lifts of asphalt concrete were modified with polymers. The Bearmouth-Drummond, Lincoln Road-Sieben, and Tarkio-East projects all used hot recycling for the lower overlay lifts in the control sections. The top lift at Lincoln Road-Sieben was the only case where polymer modification was accompanied by hot recycling. As an added note, the lower lift at Lincoln Road-Sieben contained a relatively high proportion of recycled mix, equal to 34 percent. All asphalt concrete mixtures were MDT Grade D with exception for the top lift at Custer County Line West, which was an MDT Grade S. This Grade S lift extended across both the control item and the test item. All test and control sections were topped with a chip seal (i.e. “seal and cover”) with three eighths of an inch maximum-size aggregate.

The control section at Bearmouth-Drummond did not involve an increase in pavement thickness above subgrade, relative to the original pavement structure (Figure 7). The MDT design personnel did not feel an increase in structural capacity was necessary at this site. The final thickness of the control section at Rocky Canyon was the same as the original structure, but the structural capacity was increased through stabilization (Figure 8). The control sections at Lincoln Road-Sieben, Custer County Line West, and Tarkio-East involved increases in thickness above subgrade of 0.5, 5, and 2 inches, respectively (see Table 8 and Figures 9, 10, and 11).

The structural capacities of the test sections at all sites were increased relative to the original pavement structures. Milling was only deep enough to remove the OGFC, so all overlay lift thicknesses were greater than the depth of removed material. Increases in total thickness above subgrade for the test sections ranged from 1 to 4 inches, as shown in Table 8 and Figures 7 through 11.

In conclusion, stripped asphalt should last longer because they have more structural strength. Therefore, we are not comparing the same structural number between the control and test sections.

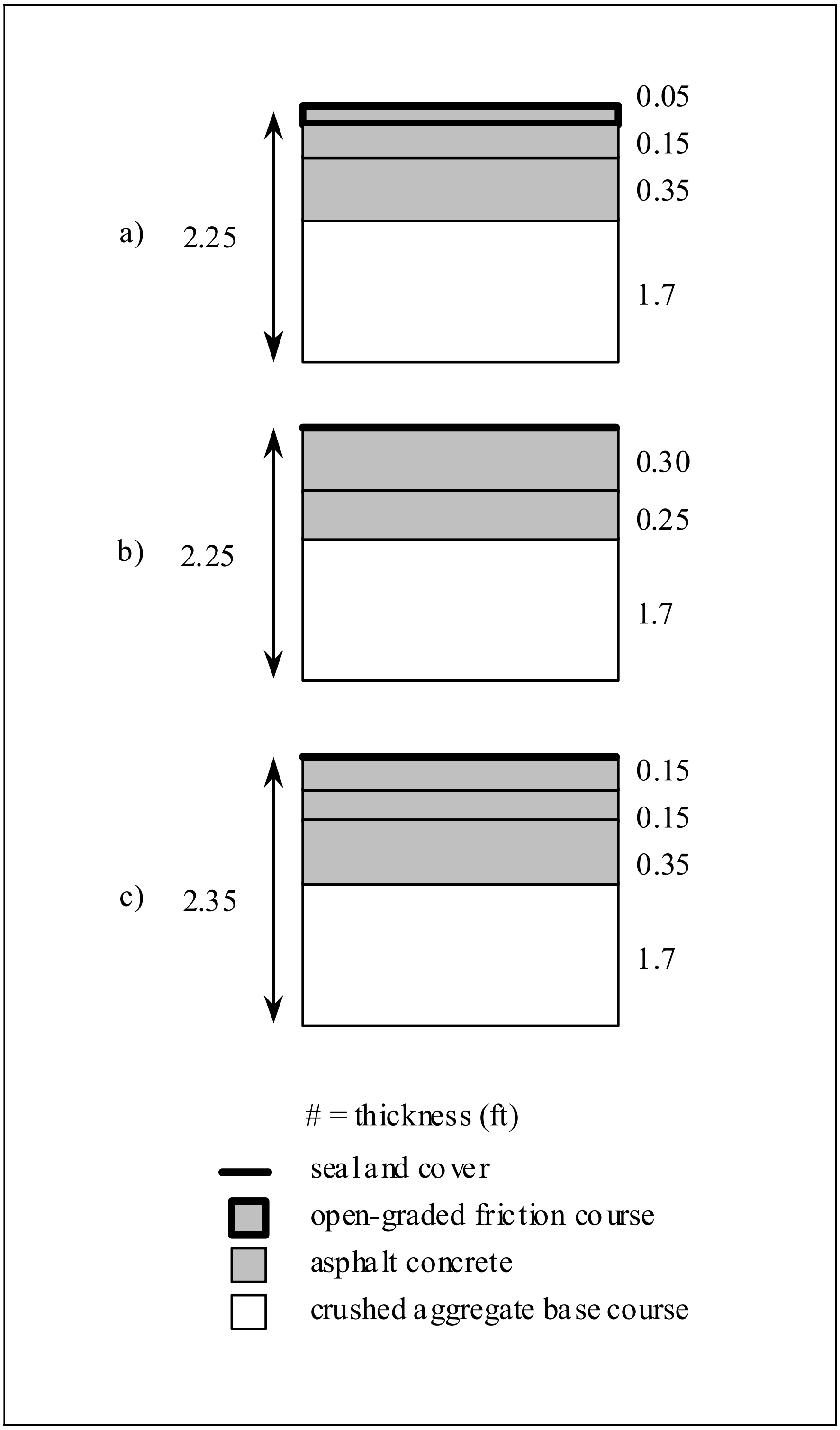

Figure 7. Pavement Cross-Sections at Bearmouth-Drummond: a) original pavement, b) control section, and c) test section

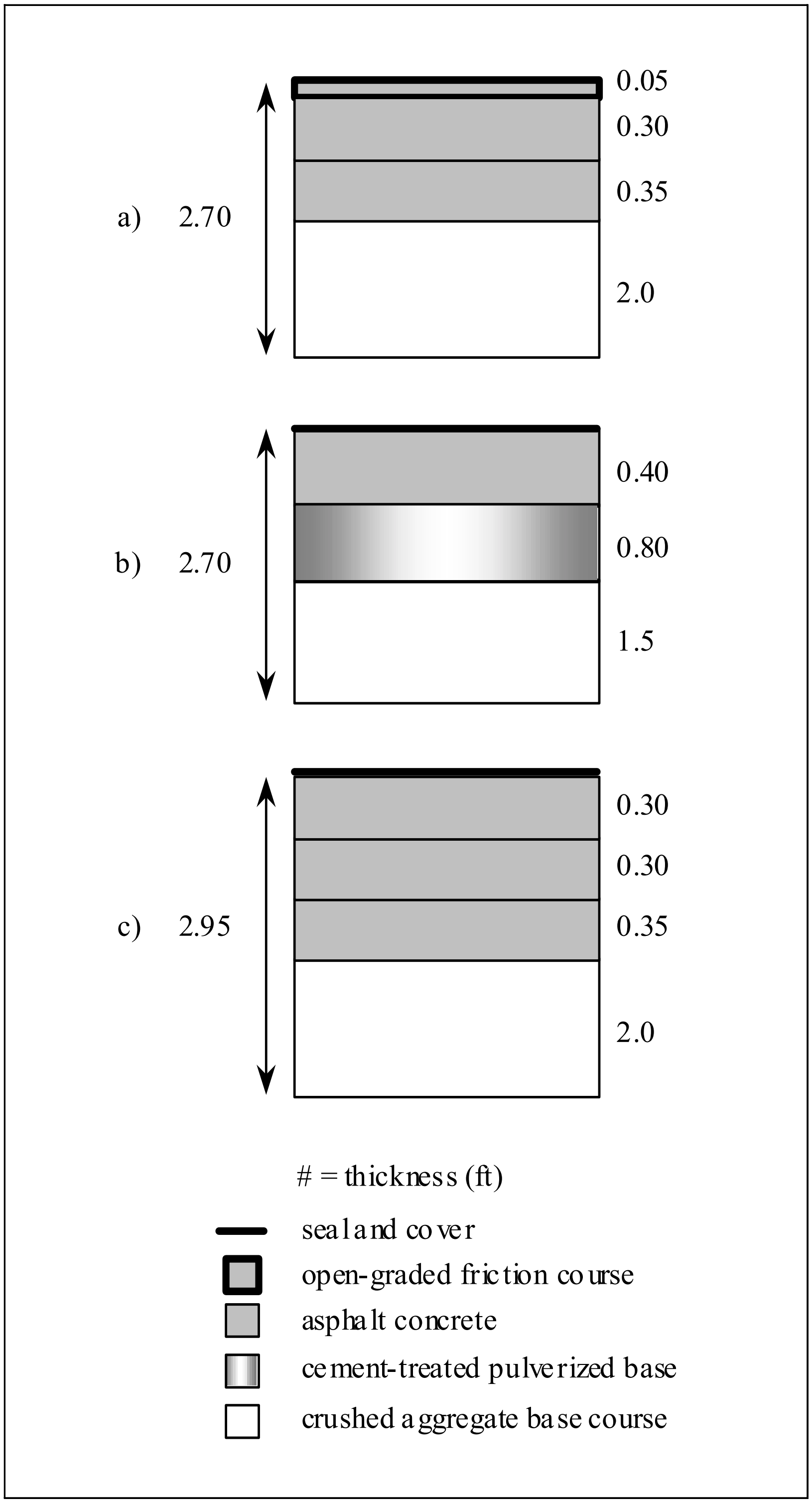

Figure 8. Pavement Cross-Sections at Rocky Canyon:
a) original pavement, b) control section, and c) test section

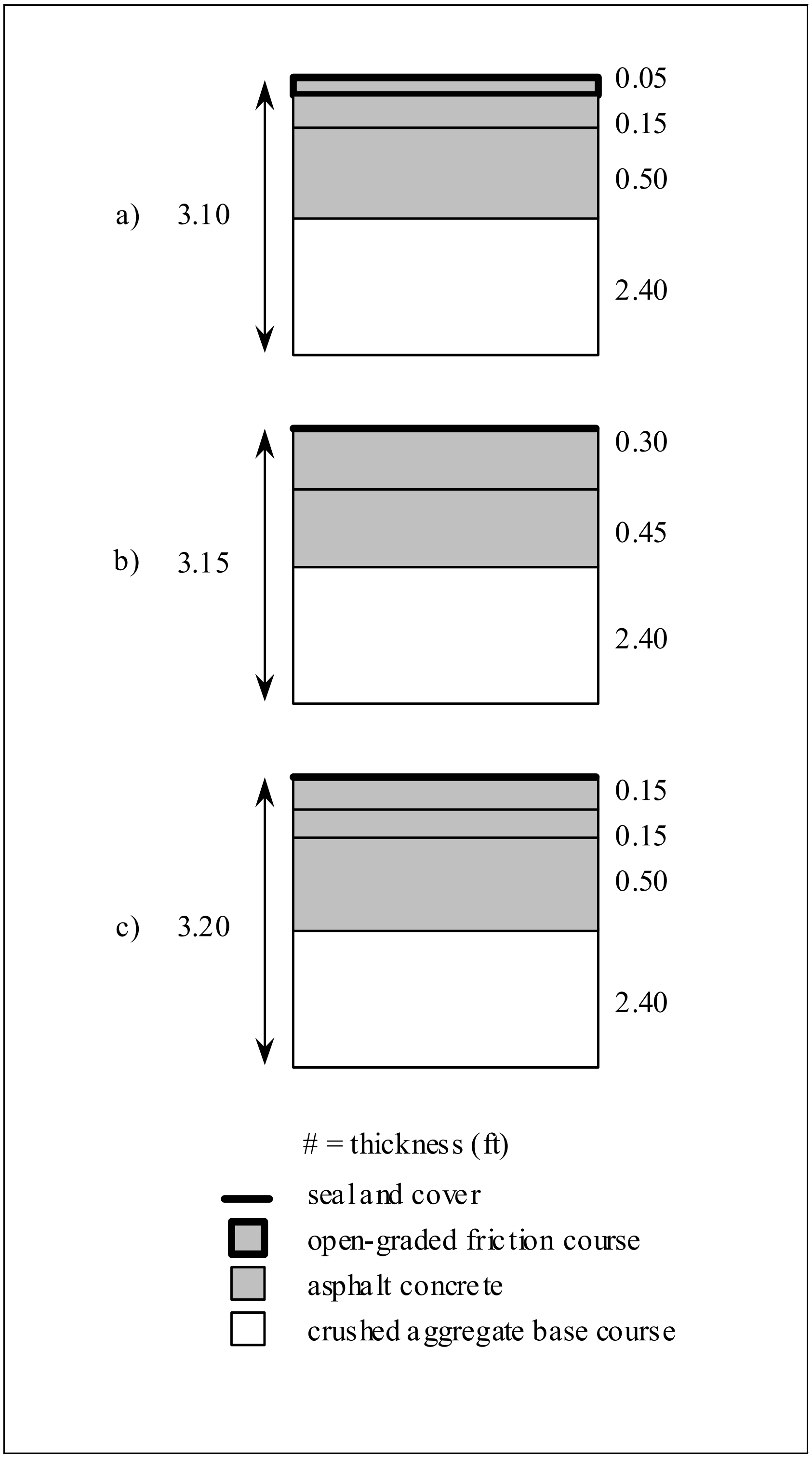

Figure 9. Pavement Cross-Sections at Lincoln Road-Sieben:
a) original pavement, b) control section, and c) test section

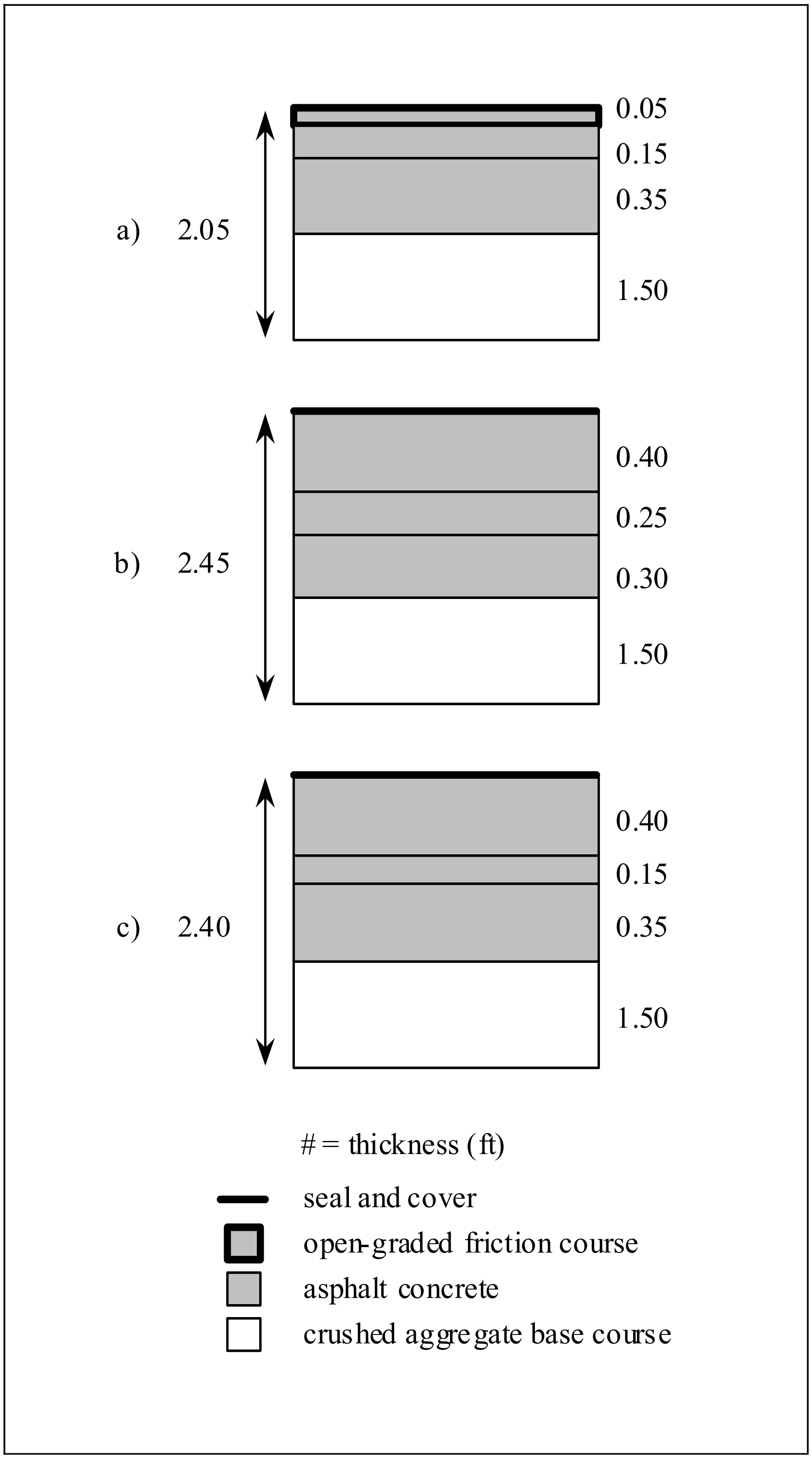

Figure 10. Pavement Cross-Sections at Custer County Line West: a) original pavement, b) control section, and c) test section

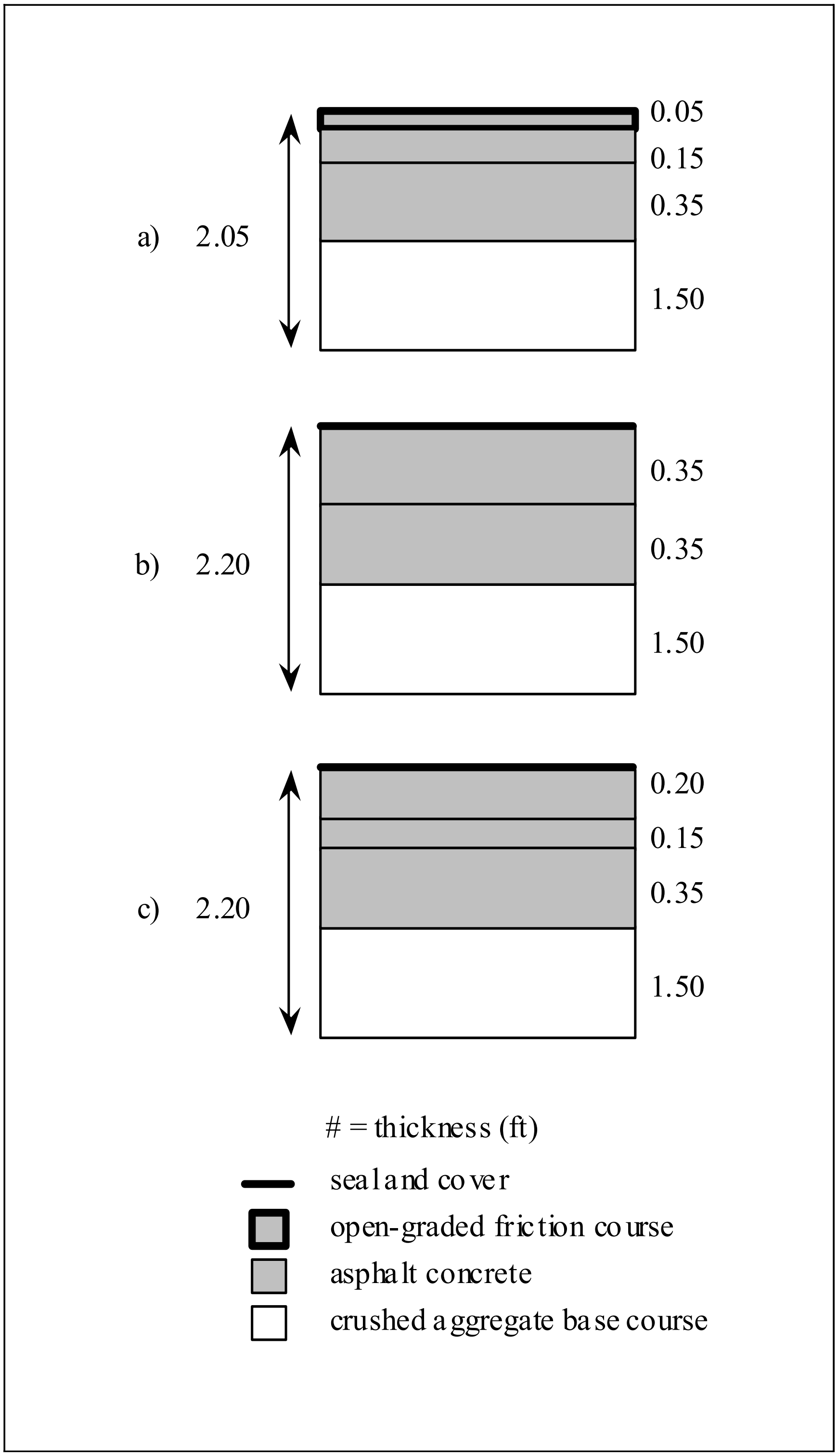

Figure 11. Pavement Cross-Sections at Tarkio-East:
a) original pavement, b) control section, and c) test section

5.3 Pavement Performance Monitoring

The test sections at each site have been monitored annually for changes in structural integrity, roughness, and distress. Structural integrity has been monitored with both a Road Rater and a Jils falling-weight deflectometer (FWD). Roughness has been monitored with both a South Dakota Profilometer and a Rainhart Profilograph. Distress monitoring has involved visual inspections of the road surfaces.

Structural Integrity. The MDT recently transitioned from using a Road Rater to using an FWD. Structural evaluations performed in 1997, or earlier, involved a Road Rater. Road Raters apply a sinusoidal load to the pavement; frequency and peak load are controllable. Frequencies of 12 to 15 Hz and a peak load of 4000 lb were used for the evaluations included in this study. The FWD applies an impact load to the pavement; drop height is controllable, which affects the acceleration of the mass at impact and subsequently, the force at impact. The drop heights used in this study produced forces of approximately 5500, 8000, and 10000 lb. A 9000-lb "seating" load preceded these forces. While the Road Rater was a trailer-type device, the Jils FWD used for this experiment was mounted within the back of a truck.

Both Road Rater and FWD testing were performed every fifty feet within the experimental sections. Most tests are performed in the outer wheelpath of the traveling lane. Every fourth test, however, is performed in the outer wheelpath of the passing lane. Based on an initial inspection of the collected data, differentiating tests by lane for the purpose of analyzing results was not deemed necessary.

During Road Rater and FWD testing, the applied force and the pavement surface deflections were measured. Surface deflections were measured at the following horizontal offset distances from the load: 0, 8, 12, 18 (FWD only), 24, 36, and 48 inches. MDT retains peak loads and peak deflections for analysis purposes. The peak deflections can be used to produce a "deflection basin," such as that shown in Figure 12. The deflection basin, in combination with the known load and assumed layer thicknesses, can be used to estimate the elastic moduli of pavement layers. This process is often referred to as "back-calculation" and usually involves the assumption that all layers are linear elastic. Modeling pavement layers as linear elastic materials provides for an effective method of designing pavements and overlays.

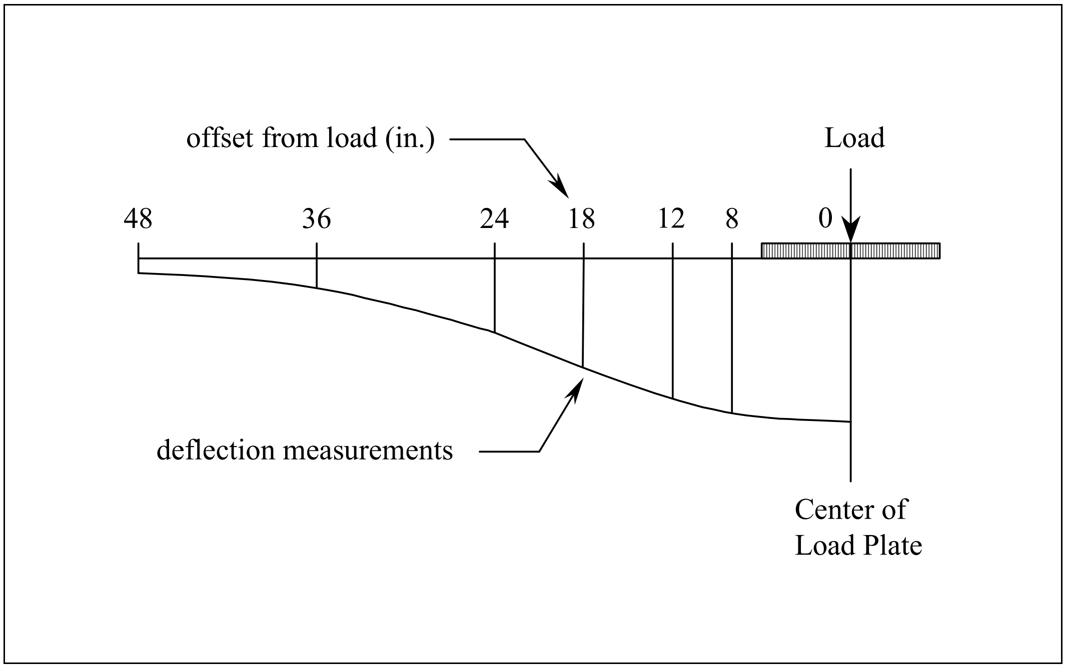

Figure 12. Deflection Basin Obtained During Road Rater and FWD Testing

Additional methods exist for using deflection basin data to characterize pavement materials. For example, the geometric curvature of the deflection basin has been used to deduce general pavement characteristics. Rohde (1990) summarized many techniques for characterizing basin curvature, including the use of deflection ratios, where the deflections are measured at different offset distances from the load. An additional useful pavement parameter is the overall pavement response stiffness, which can be ascertained by simply dividing the applied load by the deflection under the load (offset = 0 in.). The U.S. Army Corps of Engineers (USACE) refers to this parameter as the pavement's impulse stiffness modulus (ISM) and they use it to separate airfield pavements into "features" of relatively uniform structural characteristics (USACE 2001).

Although these simplistic data analysis methods provide less information than the results of back-calculation, they require no assumptions in terms of the number of layers or layer thicknesses. Therefore, the simplistic analysis methods have advantages in cases when pavement layer thicknesses are either unkown or uncertain. In addition, when the primary concern for a transportation agency is detecting changes in pavement properties over time (i.e. deterioration), simplistic monitoring by either overall pavement stiffness or basin curvature is effective.

Roughness and Rut Depth. Roughness monitoring was performed with a South Dakota Profilometer, which is an inertial profiler. The South Dakota Profilometer consists of a truck equipped with accelerometers and lasers. Pavement roughness measurements are obtained at speeds between 20 mph and 65 mph (typically at 60 mph). The accelerometers provide an inertial reference and the lasers are used to measure the distance between the inertial reference and the pavement surface.

Roughness was reported as International Roughness Index (IRI) values, which have units of inches/mile (inches of vertical deviations per mile of road). As a pavement's roughness increases, its IRI increases. MDT ranks the conditions of paved surfaces in terms of IRI as shown in Table 9.

Table 9. MDT Ranking of Pavement Roughness

Condition of Paved Surface	International Roughness Index (in./mi)
Excellent	< 16
Good	16 to 75
Fair	76 to 150
Poor	151 to 225
Very Poor	> 225

The South Dakota Profilometer uses two lasers for measuring pavement surface deviations in order to calculate IRI. These two lasers are attached so that they project into the two wheelpaths. The South Dakota Profilometer has a third laser on the front bumper in order to permit calculations of rut depth. The third laser is attached so that it is aimed at the middle of the lane, centered between the two wheelpaths. As the vehicle travels along the road, twenty to thirty measurements are obtained by each laser per one-foot length of pavement. The differences between the lengths measured by the lasers are used to estimate an average rut depth:

$$average\ rut\ depth = \frac{(h_1 - h_2) + (h_3 - h_2)}{2}$$

where

h_1 and h_3 = distances to the pavement surface in the wheelpaths and
h_2 = distance to the pavement surface at mid-lane

During the last two years of roughness and rut depth monitoring (1999 and 2000), the Rainhart Profilograph was used to measure rut depth because it had demonstrated the ability to supply accurate and reliable data during other recent MDT research studies. The Rainhart Profilograph differs fundamentally from the South Dakota Profilometer because it measures ruts by purely mechanical means. This profilograph consists of a rigid beam, which is supported above the pavement surface by rigid legs. Typically, the beam is positioned transverse to a roadway lane (Figure 13). Upon the beam rests a barrel to which a pen and lined graph paper are attached. As the barrel is moved across the beam, a wheel that rides on the pavement surface transcribes changes in elevation on the paper (Figure 14). The profilograph is placed at discrete locations along the pavement and the surface elevation measurement wheel is moved from one side of a lane to the other. For this research, the wheel was first moved from the shoulder to the centerline. The marking pen was then moved slightly and a second profile of the same station was made from the centerline to the shoulder. Scaling ratios were set so that surface roughness was drawn with a 1:12 ratio (paper:pavement) horizontally and a 1:1 ratio vertically.

Figure 13. Rainhart Profilograph

Figure 14. Moving the Rut-Transcribing Mechanism Across the Rainhart Profilograph

Station locations were chosen randomly for each site. At each site, the station pattern was identical for both the control section and the test section. An analysis of the previous work with the profilograph indicated that a minimum of four stations should be investigated for each experimental section. For four of the five test sites in this study (Custer County, Lincoln Road, Rocky Canyon, and Tarkio), ten stations were chosen for each of the test and control sections. Due to the shorter experimental section lengths at Bearmouth-Drummond, five stations were chosen for each of the test and control sections.

Visual Distress Survey. Visual examinations and the methods for recording distress generally followed the guidelines established by the Strategic Highway Research Program (SHRP 1993).

Some modifications were implemented to meet the specific needs of this study. For example, units of measure have been adjusted in some cases and rutting was not measured during the initial visual inspections because the South Dakota Profilometer provided estimates of rut depth. The final two visual inspections have collected rutting data with the Rainhart Prophilograph. The types of distress that were included in the examinations performed for this study are shown in Table 10.

Table 10. Distress Types Included in the Visual Examinations

Distress Type	Unit of Measure	Comments
Bleeding	Percent Length of Affected Area[a]	Discoloration is reported as low severity bleeding even though it may not have substantial effects on pavement performance.
Raveling	Percent Area	Raveling of the surface treatment is differentiated from raveling of the asphalt concrete.
Transverse Cracking	Number and Density (length/area)	Full-width cracks, which extend from shoulder stripe to shoulder stripe, are differentiated from partial-width cracks.
Longitudinal Cracking at Centerline	Percent Length	None.
Longitudinal Cracking in the Wheelpath	Percent Length	None.
Fatigue Cracking	Percent Area	None.
Potholes	Percent Area	None.
Patches	Percent Area	None.

[a] affected area could be one or both wheelpaths, centerline, or edge of lane; localized bleeding was not a problem for the pavements included in this study

Note: Each distress type can have three levels of severity: low-severity, moderate severity, and high severity. Judgement of severity is based on SHRP (1993) guidelines.

Cores. An extensive coring operation was undertaken during the final year of evaluations for the experimental sections. These cores were necessary to determine the condition of materials located beneath the pavement surface. Specifically, to what extent had moisture damaged underlying layers in the test and control sections, and was there a difference to be found between the two rehabilitation scenarios?

At each test site, cores were removed from the pavements at selected profilograph stations at the time of the last rut depth evaluation. The stations were selected to provide representation throughout each experimental section. Two stations were selected for each of the control and test sections at Bearmouth-Drummond. Three or four stations were selected for each of the control and test sections at the other test sites. At each of these stations three cores were removed: one from the outside wheel path of the traveling lane, one from between wheel paths in the traveling lane, and one from the outside wheel path of the passing lane. The condition of each core was noted as it was removed. Each core was immediately marked for its respective location, including site name, station number, and transverse position on roadway.

Cores were then transported back to the asphalt lab at Montana State University for further analysis. This analysis consisted of measuring the cores for overall length, measuring the thickness of each of the discernable different layers of bound materials, and performing a stripping analysis consistent with Montana Method MT 331.

6. Pavement Performance

Pavement evaluations at the test sites were performed annually for three to five years. As previously stated, evaluations consisted of visual inspections consistent with SHRP methods and standards, FWD or Road Rater evaluations, collection of IRI data, and for the final two years, the collection of Rainhart profiles.

6.1 Structural Condition

Although pavement layer moduli were presented in interim reports for this project, they were eliminated from the final project analyses. The primary reason for elimination is that more simplistic analysis parameters (i.e. overall pavement response stiffness and basin curvature) proved to be equally as effective for detecting changes in pavement properties over time.

Two types of NDT test parameters were selected for presentation in this final report: pavement stiffness, represented as ISM, and basin curvature, represented as deflection ratios. Two deflection ratios were selected: D1/D2 and D1/D3, where D1 is deflection at the center of the load plate, D2 is deflection at 8 inches (0.20 m) offset, and D3 is deflection at 12 inches (0.30 m) offset. An increase in either of these curvature parameters would indicate a relative weakening of layers near the pavement surface. ISM and basin curvature data are summarized in Appendix B.

Comparisons between Road Rater data and FWD data verified that a fundamental difference exists between the results obtained from these two test procedures. This difference was expected because Road Rater machines apply sinusoidal loads and FWD machines apply impulse loads. Only FWD data will be used in the structural analyses for this report.

Using data for all test locations, both ISM and basin curvature were tested for normality and homogeneity of variance. These data generally passed normality tests (Kolmogorov-Smirnov and Shapiro-Wilks), but variance was found to change significantly among sets of treatments, where treatments included different experimental sections (control or test) and different test dates. The transformation of ISM with the natural logarithm was found by the

Levene Test to be effective in stabilizing variance. An effective transformation could not be found for the basin curvature parameters. Therefore non-parametric tests would be necessary for studying differences involving these variables.

The Lincoln Road data was used to develop a routine of data analyses. Only one of the basin curvature parameters was found to be necessary because they were highly correlated with a Pearson correlation coefficient equal to 0.927 (see Figure 15). The D1/D3 parameter was chosen because its use is popular and well documented (e.g. Rohde 1990). Further references to basin curvature will imply the D1/D3 parameter.

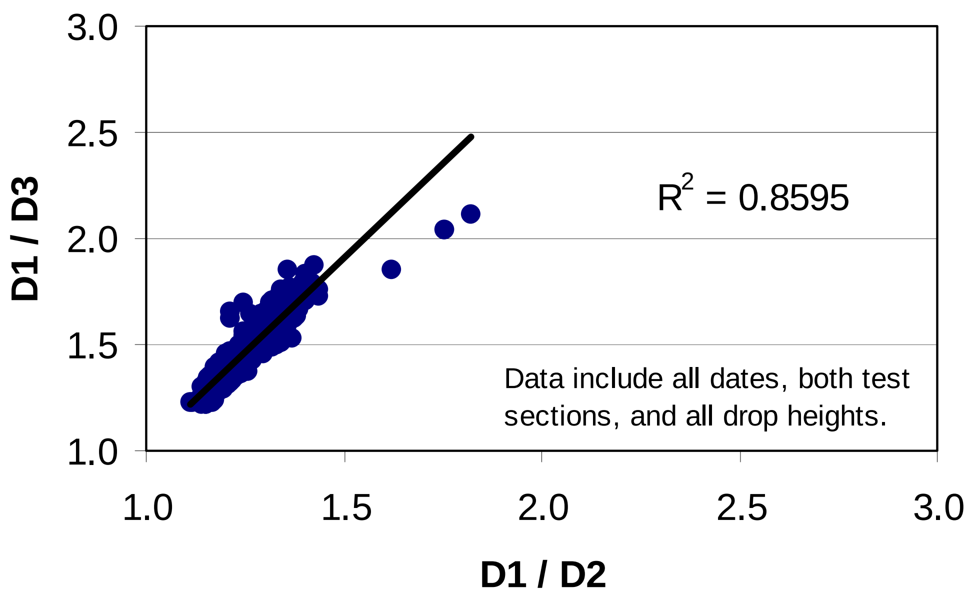

Figure 15. Correlation of Basin Curvature Parameters for Lincoln Road-Sieben Site

The ISM data at Lincoln Road were also used to inspect the importance of considering different drop heights (i.e. load levels) for the falling-weight deflectometer test results. An analysis of variance for transformed ISM data, obtained at Lincoln Road, is shown in Appendix C (Table C1). The independent variables were all significant, including year (i.e. date of evaluation), section (i.e. control and test), and drop height. However, interactions between these variables were not significant. Lack of interaction implied that the effect of drop height was the same for each type of pavement section and that this effect did not change over time (i.e. year).

Due to the consistent influence of drop height over time, the authors decided to select a single drop height for further analyses. Drop height 3 was selected because it produced load levels on the order of 9,000 lb, which is conveniently one-half an equivalent single-axle load (ESAL). While the highest load would be preferable when studying lower pavement layers, it should not be necessary for studying upper pavement layers, as in this study.

Further statistical tests for ISM involved only "year" and "section" as the independent variables. Each site was first analyzed with a two-factor analysis of variance. If the interaction between year and section was not significant, conclusions were based on the analysis of variance alone. If the interaction between year and section was significant, the data were reanalyzed with a combination of one-way analyses of variance and Student's t-tests. Statistical tests for basin curvature required two non-parametric tests: Kruskal Wallis and Mann-Whitney. For each test site, the Kruskal Wallis test was first used to determine whether a significant difference was present between any of the test sites and/or between different evaluation dates. If a significant difference was present, the Mann-Whitney test was used to conduct pairwise comparisons between years and between section types (control versus test). Detailed results from statistical analyses are shown in Appendix C. Important findings from these analyses will be included in the ensuing discussion.

Mean values for ISM for both control and test sections on all FWD test dates are shown in Figures 16 through 21. The range of values for each evaluation of the control section is also shown to provide an indication of data scatter. In each figure, "year" and "section" are each labeled as either significant or non-significant in terms of their effects on the dependant variable (ISM or basin curvature). Significance was gleaned from the statistical tests. For the two-factor analyses of variance, a source of variation was considered significant if the null hypothesis (i.e. equal treatments) could be rejected with a probability of error less than 5 percent. Because the one-way analyses of variance and the Student's t-tests were performed simultaneously, a source of variation was considered significant only if the null hypothesis could be rejected with a probability of error less than 1 percent. This imposed conservatism on each test was necessary to achieve an appropriate experiment-wise error rate.

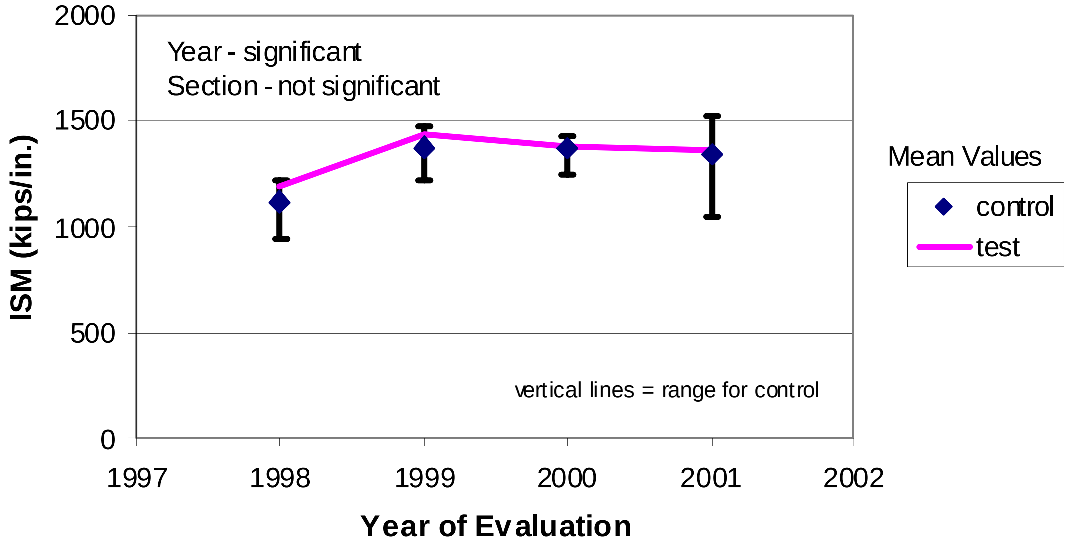

Figure 16. Impulse Stiffness Modulus at Bearmouth Drummond, Westbound Lane

For each test site, ISM changed significantly with time (different evaluation dates). In most cases, the trend was toward increasing stiffness. However, pavement stiffness at Custer County decreased steadily from 1999 to 2001. For each test site, except for Bearmouth Drummond Westbound and Rocky Canyon, ISM was significantly different between pavement sections (i.e. control versus test). At Bearmouth Drummond Eastbound, the test section showed a relatively high ISM in the year 2001 (see Figure 17). However, the control had not decreased from its year 2000 measurements. At each of Lincoln Road, Custer County, and Tarkio East, the test sections were slightly less stiff than the control sections (see Figures 19 through 21).

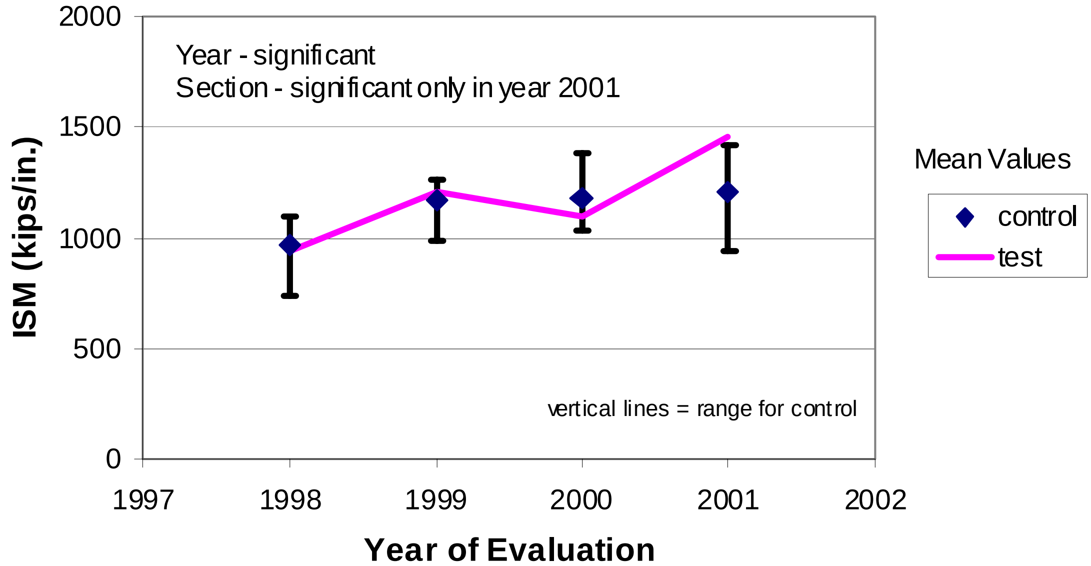

Figure 17. Impulse Stiffness Modulus at Bearmouth Drummond, Eastbound Lane

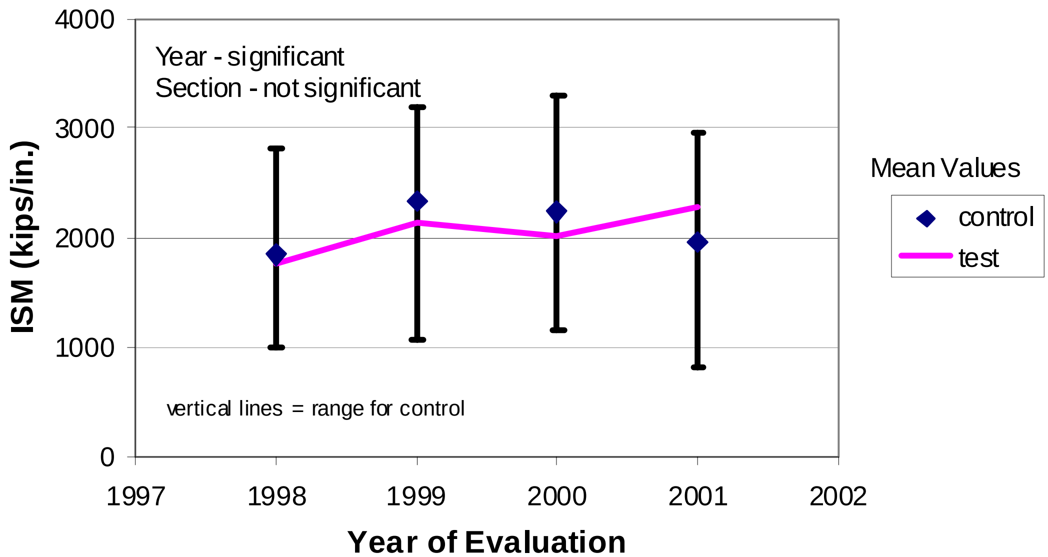

Figure 18. Impulse Stiffness Modulus at Rocky Canyon

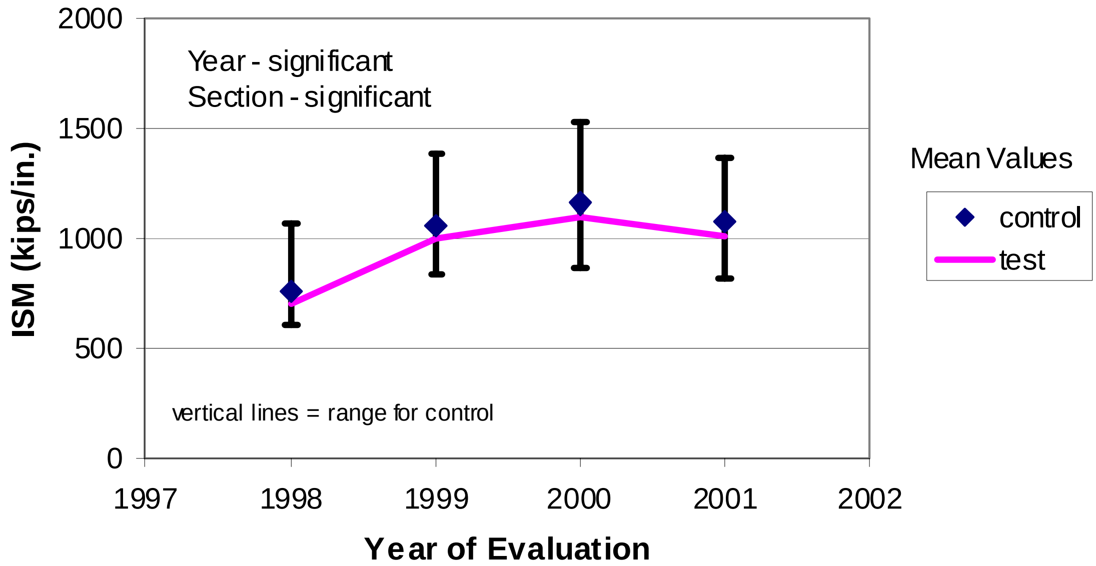

Figure 19. Impulse Stiffness Modulus at Lincoln Road

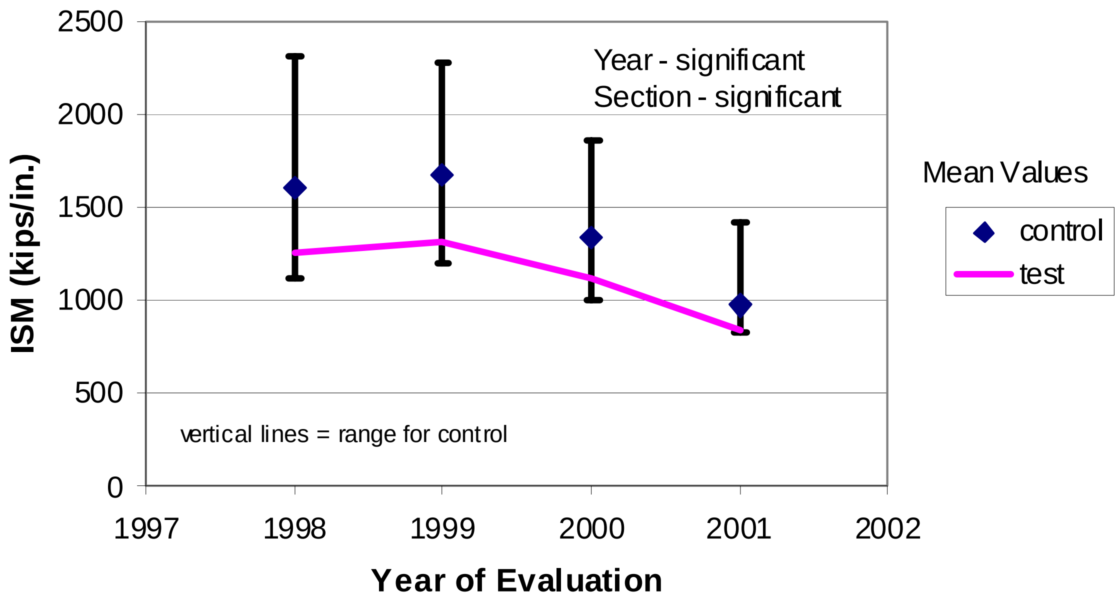

Figure 20. Impulse Stiffness Modulus at Custer County Line

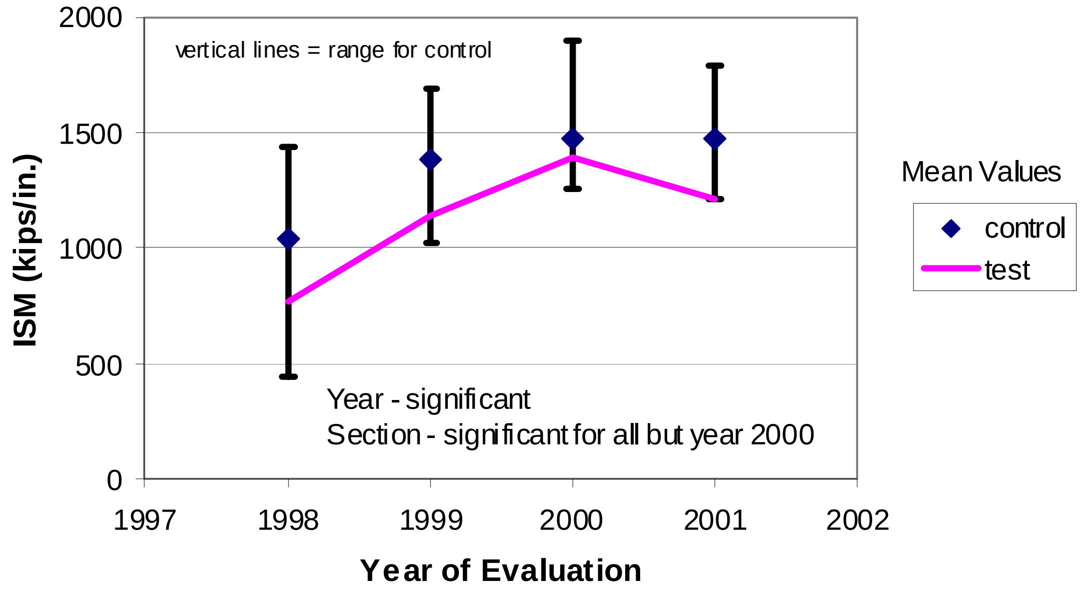

Figure 21. Impulse Stiffness Modulus at Tarkio East

However, these differences did not change over time. These differences should therefore be considered as reflections of different pavement cross-sections, rather than as reflections of the durability of different rehabilitation techniques.

The high variability in ISM for the control section at Rocky Canyon is interesting (see Figure 18). Rehabilitation of this pavement included the placement of 9.6 in. of cement-treated pulverized base (CTPB). The variability in ISM may be a reflection of spatial variability in the strength attained in the CTPB.

Similarly, basin curvature generally changed significantly over time. In most cases, the trend was toward increasing pavement stiffness (see Figures 22 through 27). Custer County was the only test site where curvature increased over time, indicating a weakening of near-surface pavement layers (see Figure 26). This weakening was consistent with ISM results. Differences between control and test sections were not significant.

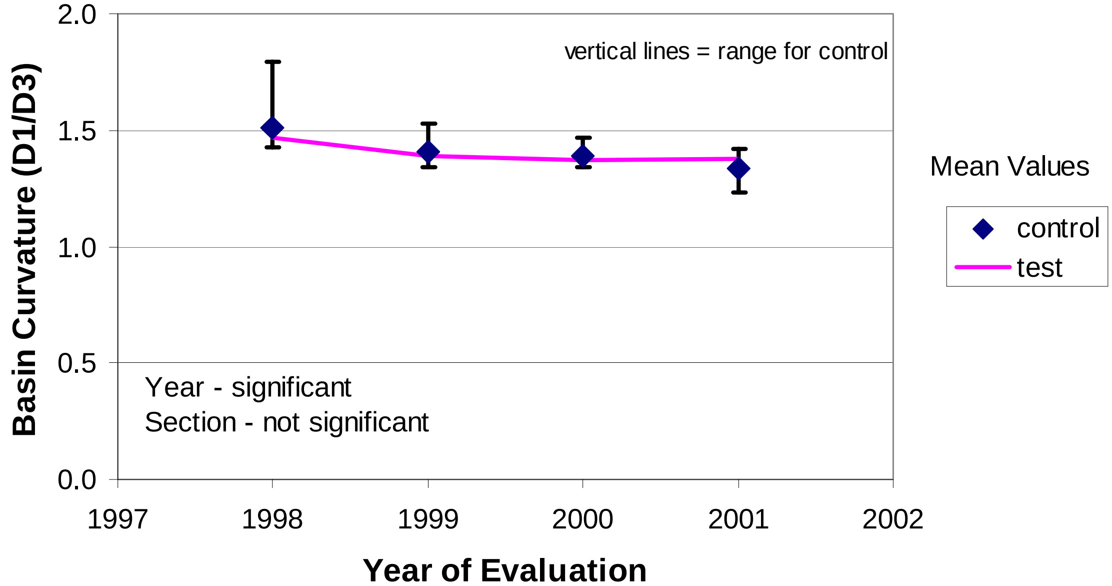

Figure 22. Basin Curvature at Bearmouth Drummond, Westbound Lane

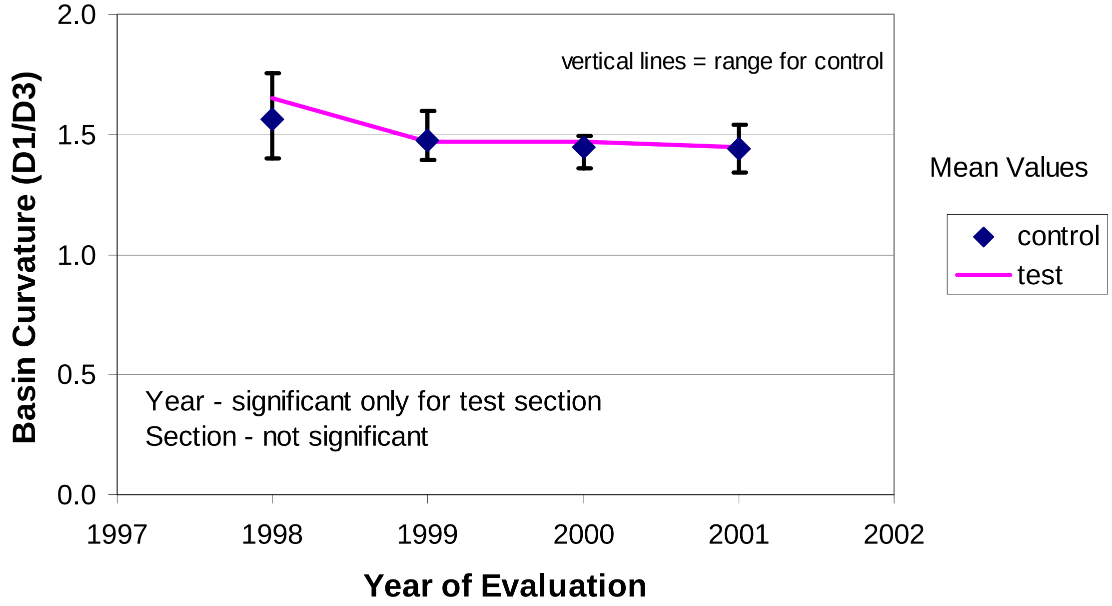

Figure 23. Basin Curvature at Bearmouth Drummond, Eastbound Lane

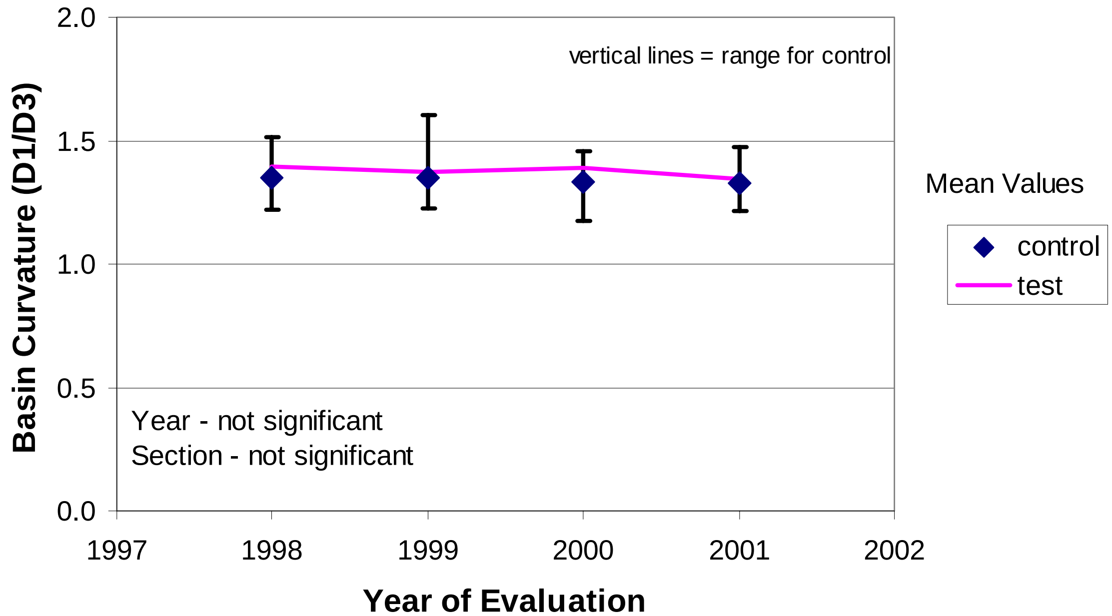

Figure 24. Basin Curvature at Rocky Canyon

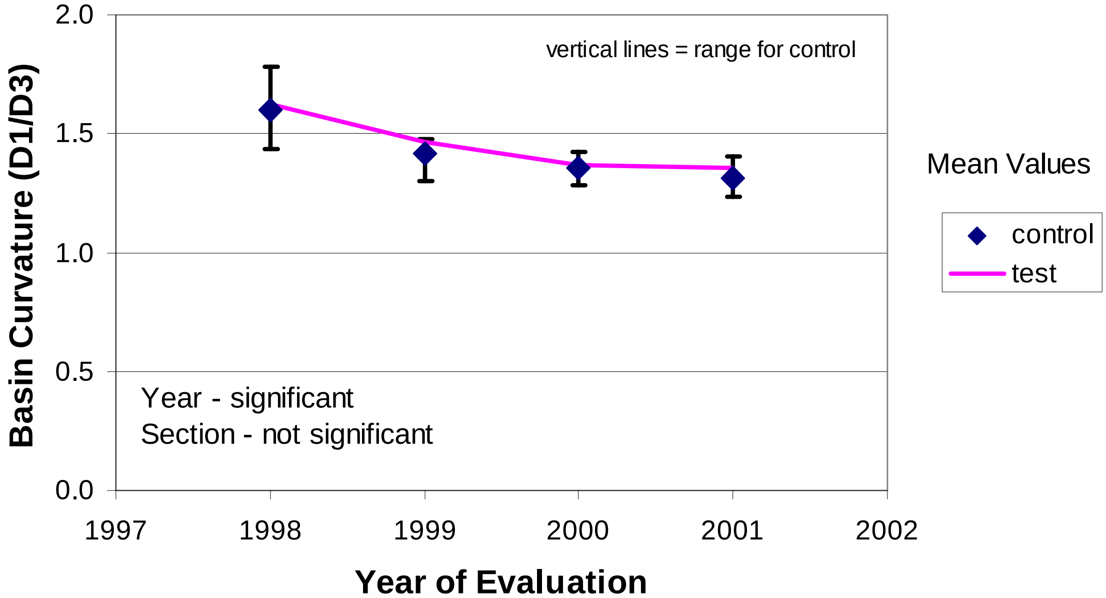

Figure 25. Basin Curvature at Lincoln Road

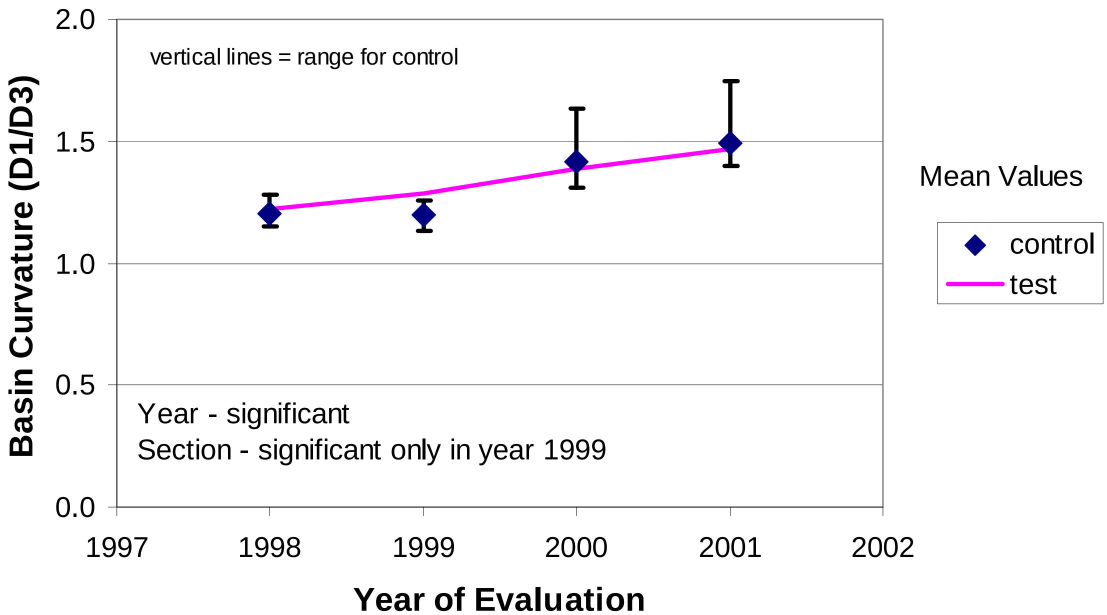

Figure 26. Basin Curvature at Custer County Line

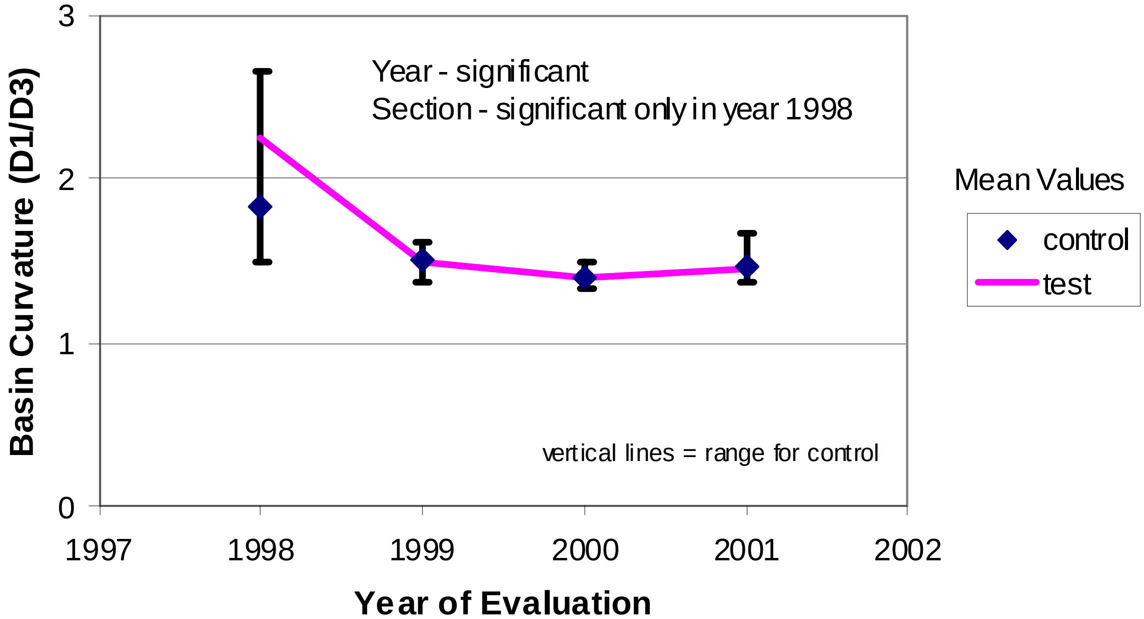

Figure 27. Basin Curvature at Tarkio East

In summary, differences between the structural integrity of control and test sections do not indicate substantially different durability characteristics. Custer County is the only test site where the pavement appears to be deteriorating over time, including both the control section and the test section.

6.2 Roughness and Rut Depth

The experimental pavement sections began their service with roughness values (IRI) of approximately 40 to 100 inches per mile (see Figures 28 through 33 and Appendix D), which correspond to MDT qualitative rankings of good to fair (see Table 9). The IRI, soon after construction, were similar for the control and test sections at Rocky Canyon, Lincoln Road-Sieben, and Custer County Line West (see Figures 30 through 32). The IRI for these sites generally increased with time, slightly more so for the test sections than the control sections. At Bearmouth-Drummond (Westbound) and Tarkio-East, the IRI were higher for the test sections than for the control sections on all evaluation dates (see Figures 28 and 33). At Bearmouth-Drummond (Eastbound), the test section had a higher IRI in 1997, but by 2000 the control section had a higher IRI (see Figure 29).

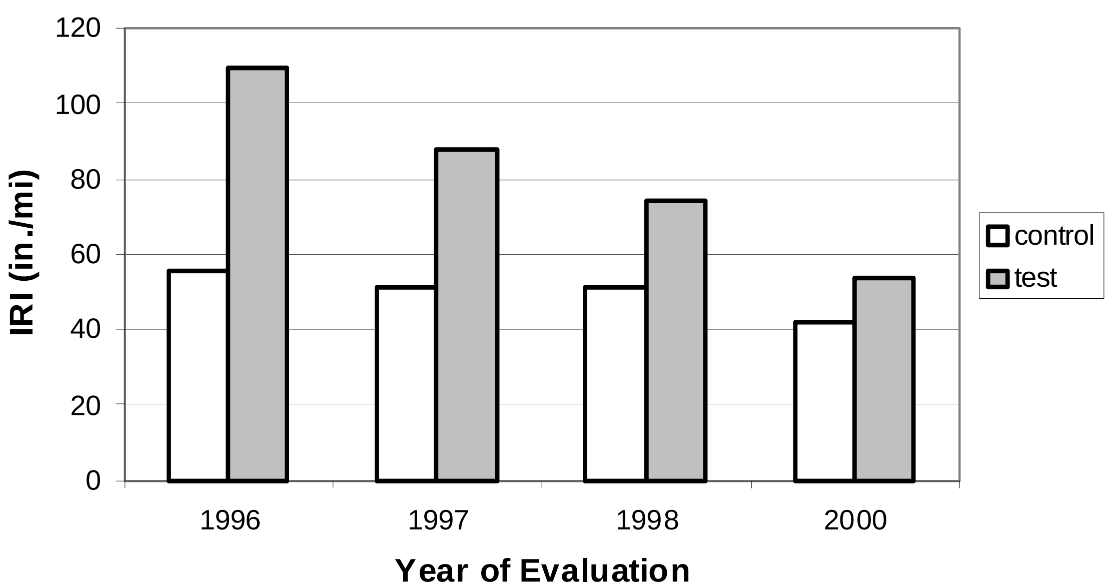

Figure 28. International Roughness Index at Bearmouth Drummond, Westbound Lane

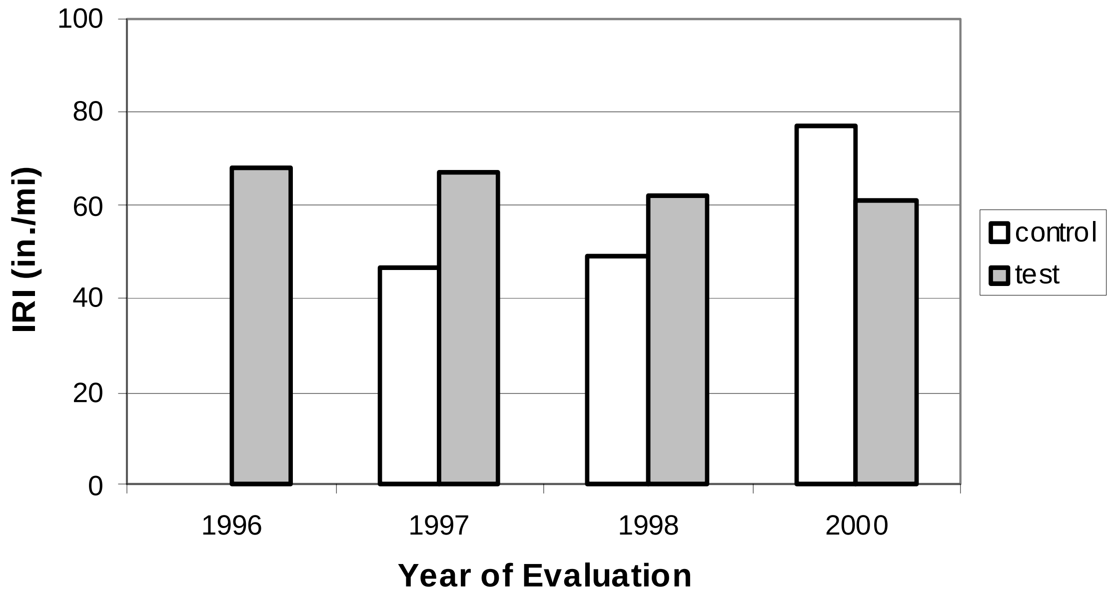

Figure 29. International Roughness Index at Bearmouth Drummond, Eastbound Lane

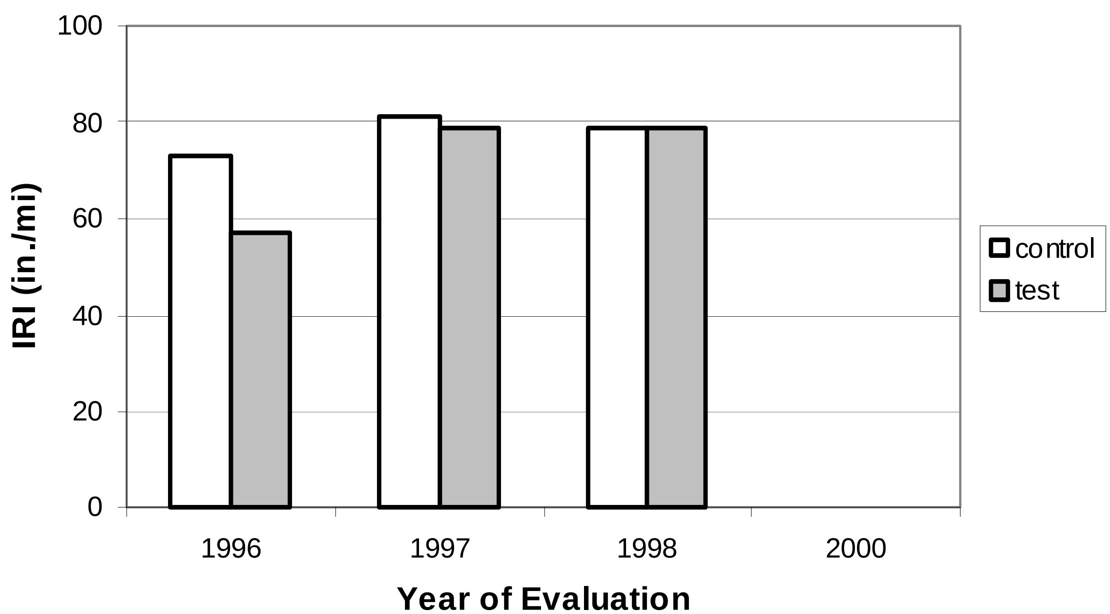

Figure 30. International Roughness Index at Rocky Canyon

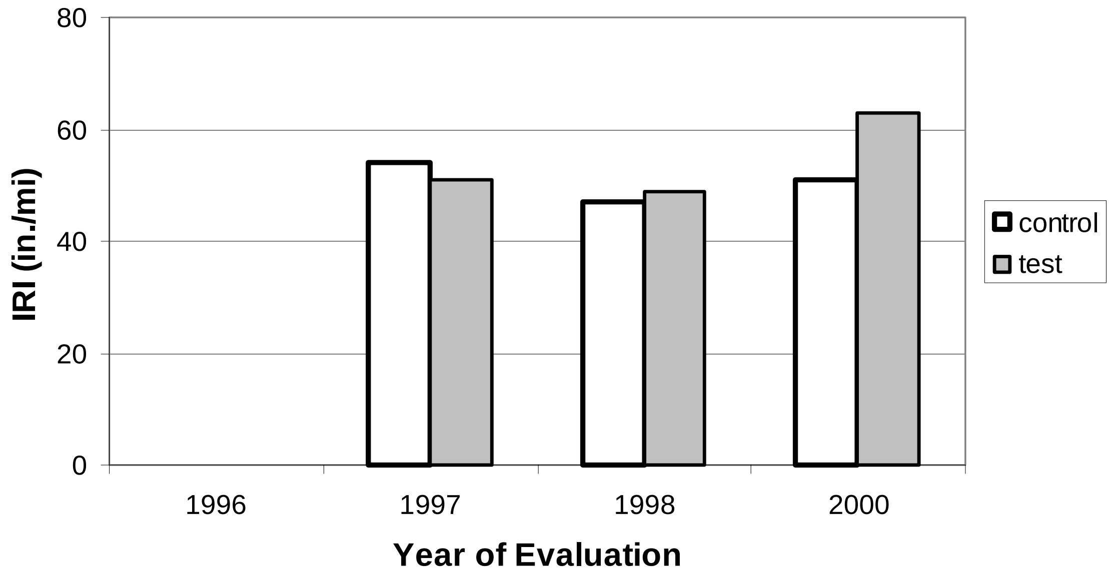

Figure 31. International Roughness Index at Lincoln Road

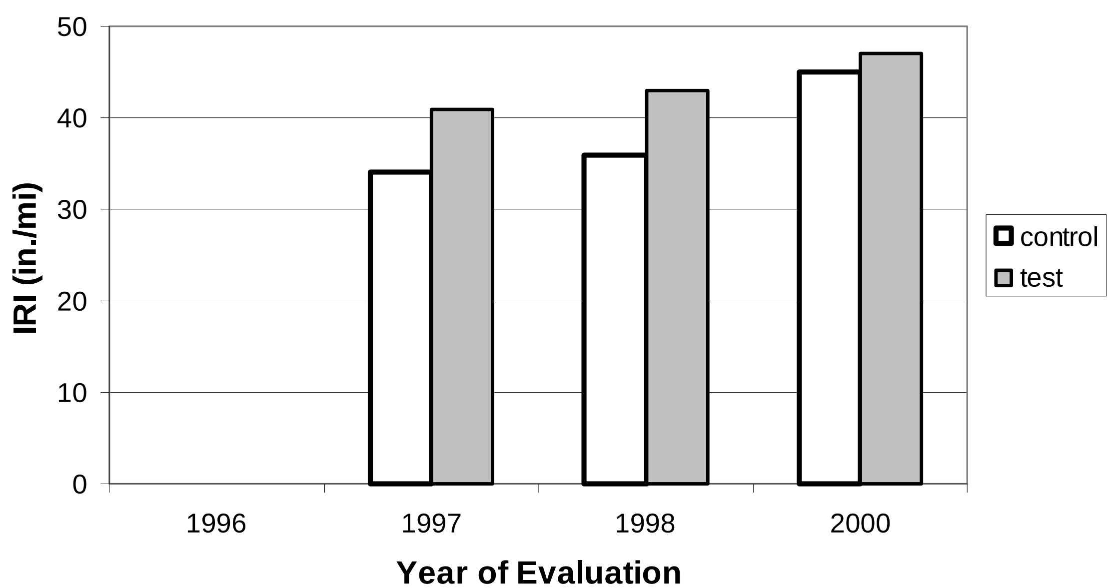

Figure 32. International Roughness Index at Custer County Line

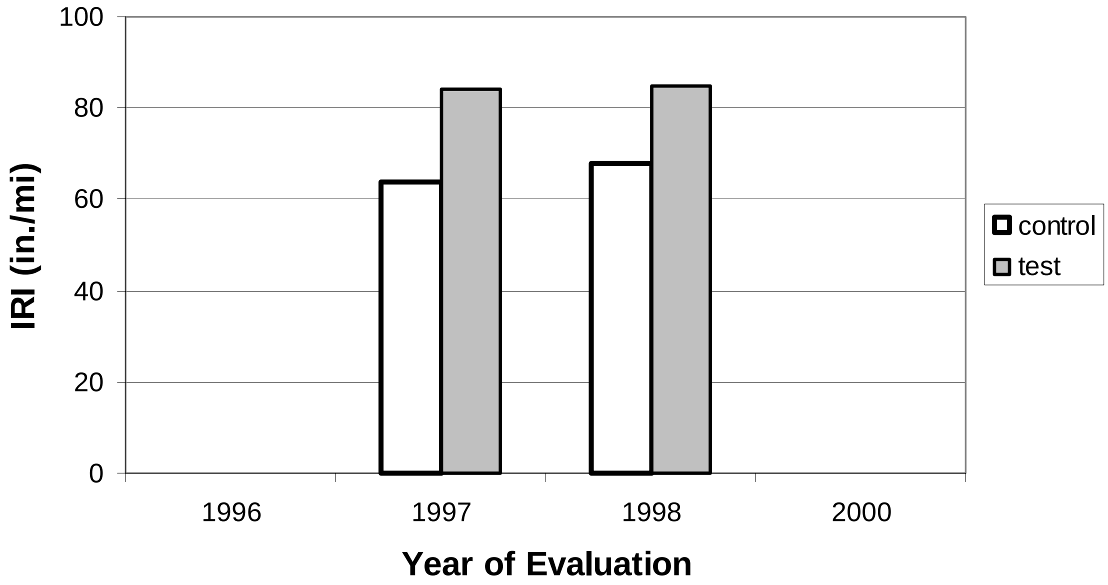

Figure 33. International Roughness Index at Tarkio East

In summary, the control sections were generally smoother than the test sections. The control sections involved more milling and thicker placements of new material, which should promote a smoother product, relative to the thin milling and overlay used for test sections. However, during the last year of evaluations (year 2000), there was no pavement section with an IRI greater than approximately 80 in/mi., which is considered by MDT to be borderline good to fair.

The experimental pavement sections began their service with average rut depths of 0.15 inch or less (Appendix D). At that time, no consistent differences were observed between ruts in the control and test sections. These initial rut measurements were obtained with the South Dakota Profilometer.

Rutting data for each pavement section, for the years 1999 and 2000, were tested for normality and homogeneity of variance. These rut data, which were obtained with the Rainhart Profilograph, were generally found to be normal (as determined by Kolmogorov-Smirnov and Shapiro-Wilks tests). However, at some sites, variances for rutting data were found to be unequal among sections and test dates. Therefore, the raw rutting data (in.) was transformed by the natural logarithm. The transformation was successful in stabilizing variance between treatments (section type and evaluation year).

The analysis of rutting will concentrate on Rainhart Profilograph measurements because they were obtained after the longest duration of trafficking and they were believed to be more reliable than measurements obtained with the South Dakota Profilometer. Similar to the statistical analyses for NDT data, a two-factor analysis of variance was first used to test for significant differences among rutting at different pavement sites, specifically seeking differences between sections (control versus test) and differences between dates of evaluation (year 1999 versus year 2000). A source of variation was considered significant if the null hypothesis (i.e. equal treatments) could be rejected with a probability of error less than 5 percent. Summaries of results from these statistical procedures are included in Appendix C.

In no case was interaction between year and section found to be significant. Therefore, all conclusions for the main effects (section and year) could be obtained from the two-factor analysis of variance. The effect of year (i.e. date of evaluation) on rutting was significant at one-half the sites: Bearmouth Drummond Westbound, Rocky Canyon, and Custer County (see Figures 34, 36, and 38). In each of these cases, rutting increased significantly from 1999 to 2000. At the other sites, rutting increased, but variability in the rut measurements was high relative to the change in mean values (see Figures 35, 37, and 39). The difference between control and test section was significant only for Custer County (Figure 38). In this case, the test section rutted less than the control. By the year 2000, Bearmouth-Drummond (Westbound) had the most rutting (up to about 0.4 in.) and Lincoln Road had the least rutting (up to about 0.2 in.).

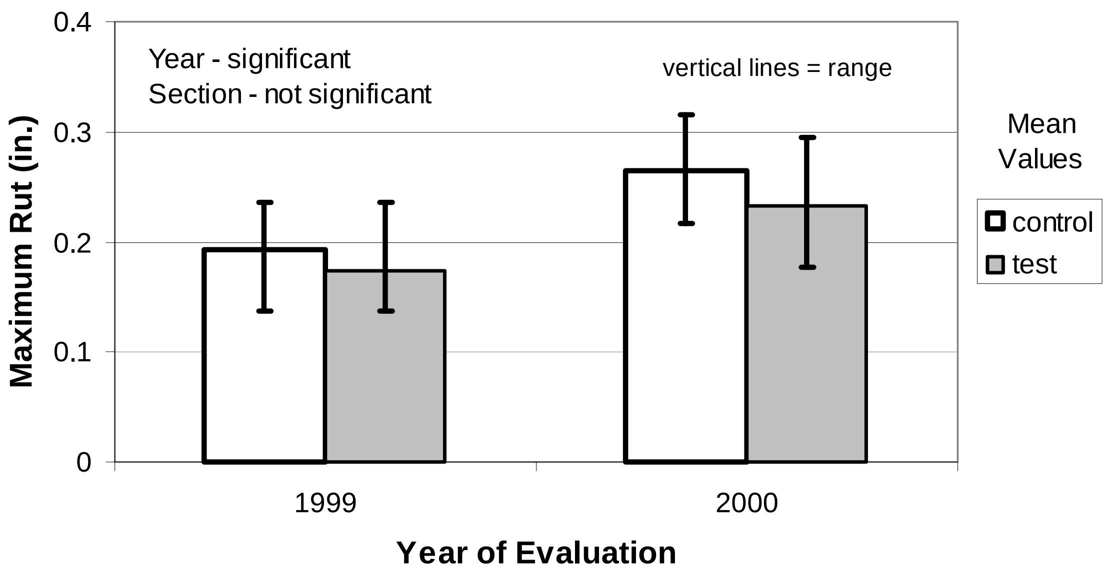

Figure 34. Rutting (Rainhart Profilograph) at Bearmouth Drummond, Westbound Lane

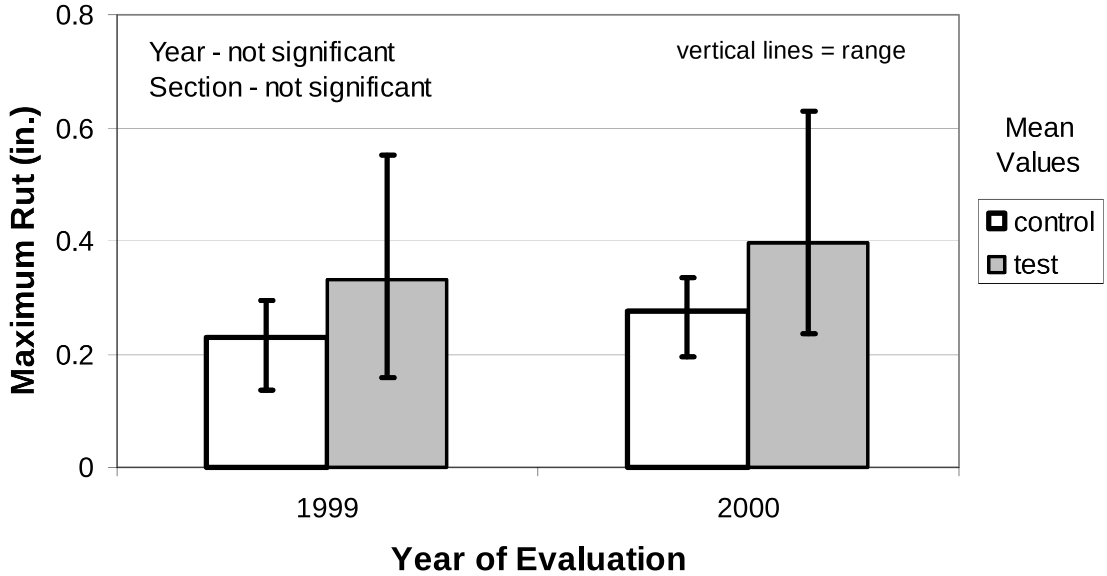

Figure 35. Rutting (Rainhart Profilograph) at Bearmouth Drummond, Eastbound Lane

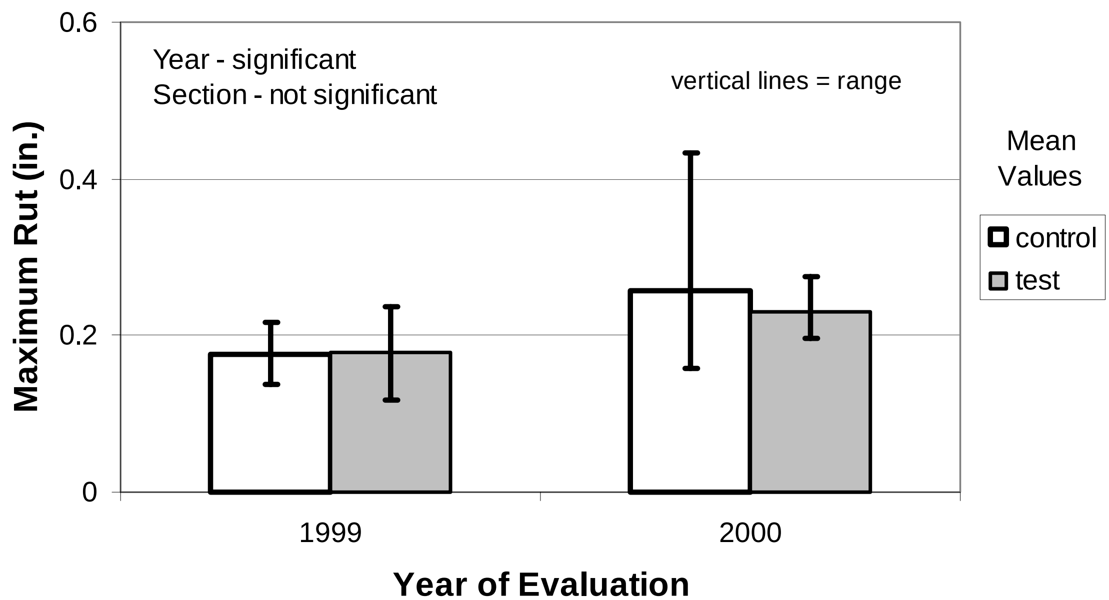

Figure 36. Rutting (Rainhart Profilograph) at Rocky Canyon

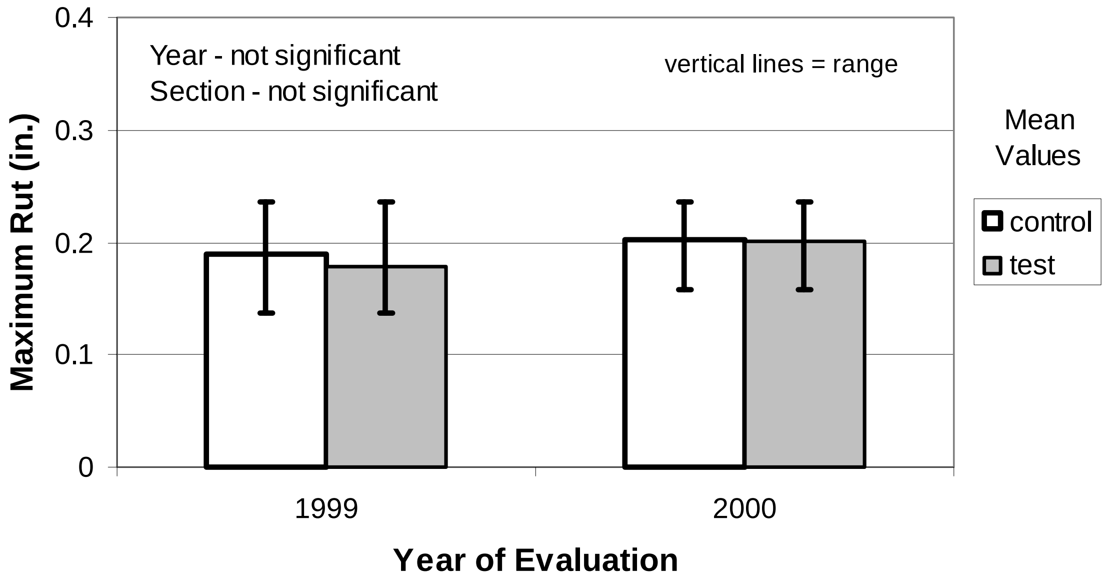

Figure 37. Rutting (Rainhart Profilograph) at Lincoln Road

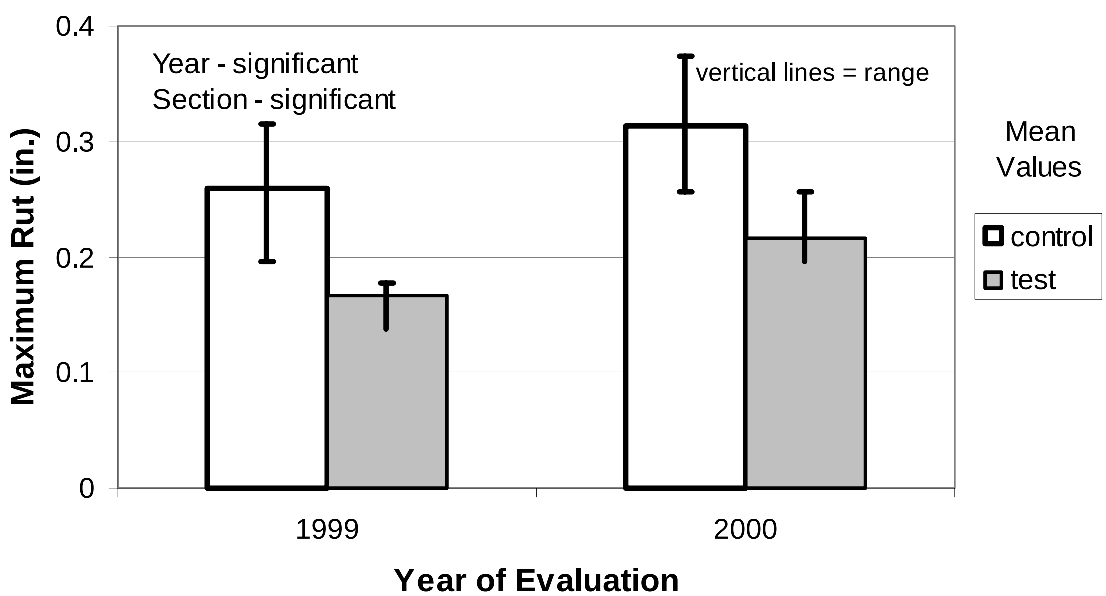

Figure 38. Rutting (Rainhart Profilograph) at Custer County Line

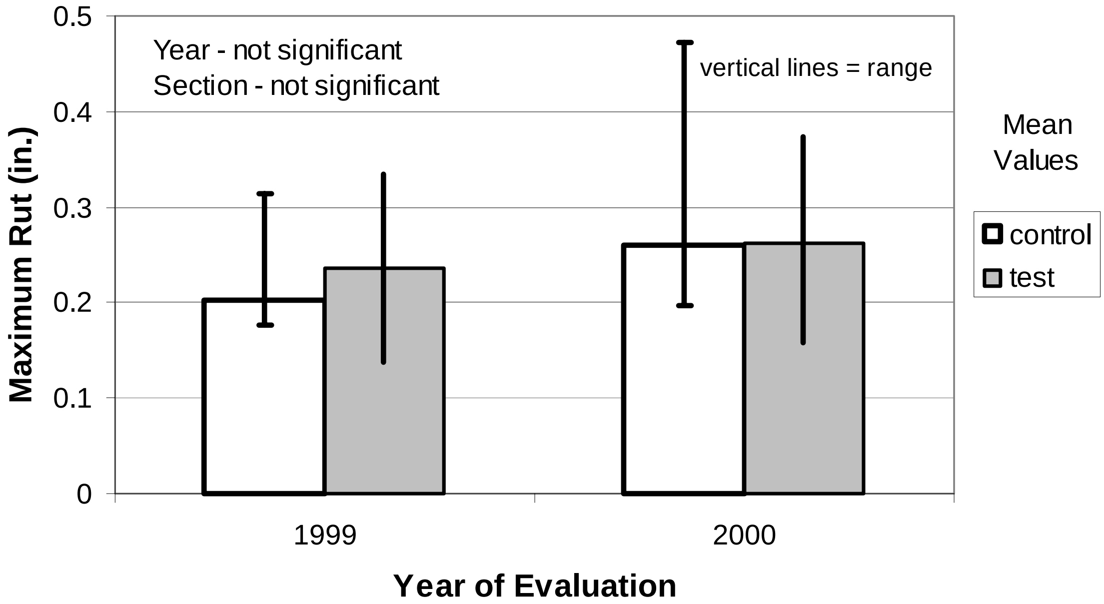

Figure 39. Rutting (Rainhart Profilograph) at Tarkio East

6.3 Visual Distress Survey

Condition surveys were performed annually for the test sites. After three to four years of service (year 2000 evaluation), all of the sites appear to be in good condition. No sites have evidence of fatigue cracking or pothole formation (Appendix E). All the sites have some level of bleeding and/or transverse cracking.

Bleeding is not necessarily a distress that will lead to performance problems. According to SHRP condition survey procedures (SHRP 1993), low-severity bleeding should be recorded if the surface is discolored by excess asphalt. This can occur without affecting skid resistance. If surface texture is affected, the bleeding is labeled as moderate. Most of the bleeding seen at the test sites was low-severity, although Tarkio-East had some moderate-severity bleeding in the traveling lane wheelpaths.

All transverse cracks at the test sites were classified as low-severity. According to SHRP condition survey procedures (SHRP 1993), low-severity transverse cracks are either unsealed with a mean width ≤ 0.25 inch or they are sealed with sealant material in good condition. The only two sites with sealed cracks were Rocky Canyon and Lincoln Road-Sieben. By the year 2000, the Lincoln Road-Sieben site had the highest density of transverse cracks with 17 full-width cracks in the control section and 24 full-width cracks in the test section. The Custer County site has the fewest number of cracks with 2 full-width cracks in the control section and 4 full-width cracks in the test section. It should be noted that the Custer County site is the only experimental pavement site that used a Superpave mix design. Control and test sections experienced similar transverse cracking at most sites. At Rocky Canyon, however, the control section cracked more than the test section: 9 full-width cracks for the control section versus 1 full-width crack for the test section. This difference in observed transverse cracks at Rocky Canyon can be attributed to the installation of a cement-treated base in the control section.

6.4 Cores

Between 6 and 12 cores were removed from each experimental pavement section at each site (see Table 11). Initial inspection of the core data revealed no reason to differentiate between cores removed from different transverse locations on the pavements. Therefore, cores will be referenced only by test site and experimental pavement section type (control or test). The cores verified thicknesses of asphalt concrete that were commensurate with expected thicknesses based on MDT records (see Figure 40). The range of core thicknesses was particularly high for test sections at both Rocky Canyon and Lincoln Road, with ranges equal to 0.8 and 0.5 times the expected values, respectively.

Table 11. Thickness Measurements from Cores

Test Site	Control or Test Section	No. of Cores	Measured Thickness[a] of Asphalt Concrete from Cores (in.)			Expected Thickness (in.) of Asphalt Concrete Based on MDT Records
			Average	Min.	Max.	
Bearmouth – Drummond (West)	C	6	6.7	6.5	7.5	6½
	T	6	7.5	7.1	8.3	8
Bearmouth – Drummond (East)	C	6	6.3	6.1	6.7	6½
	T	6	7.1	6.8	7.1	8
Rocky Canyon	C	6	5.1	5.0	5.5	5
	T	7	12.6	6.3	15.4	11½
Lincoln Road-Sieben	C	12	9.1	7.9	9.8	9
	T	12	7.9	5.0	9.4	9½
Custer County Line West	C	9	12.2	11.5	13.4	11½
	T	9	10.2	8.8	11.4	11
Tarkio-East	C	9	7.5	6.3	7.9	8½
	T	9	7.1	6.8	8.3	8½

[a] excludes asphalt concrete that has insufficient integrity to remain intact during the coring operation

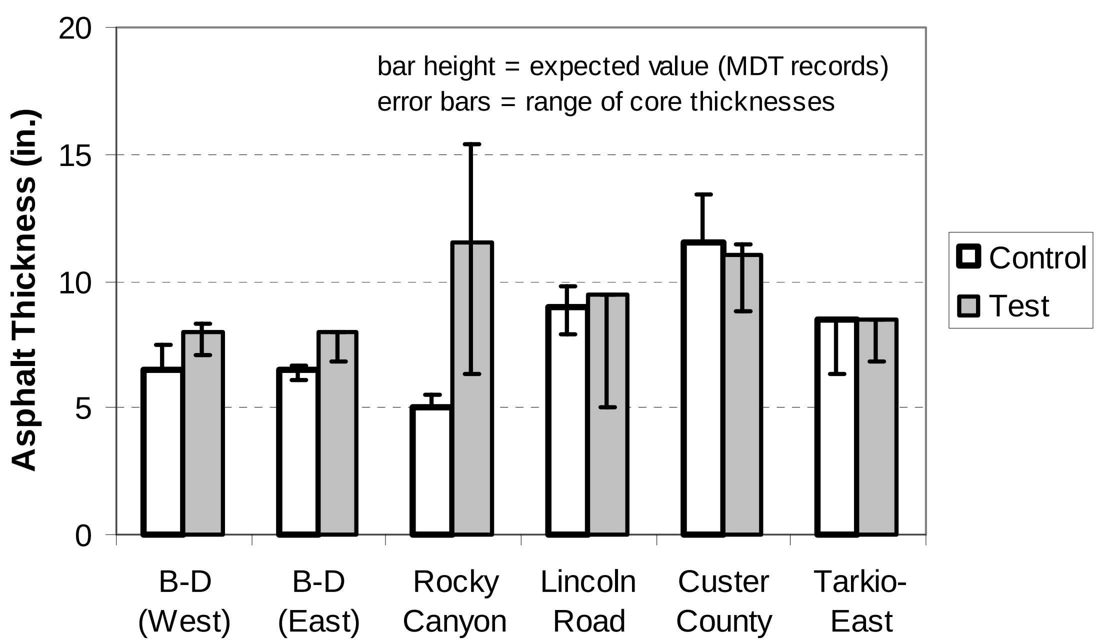

Figure 40. Comparison Between Core Thicknesses and MDT Records

The differences between control and test section thicknesses at individual sites (Figure 40) were to be expected. Relative to the test sections, the control sections received deeper milling and thicker placements of new asphalt concrete. However, only at Tarkio-East were the final thicknesses of asphalt concrete for the control and test section equal in magnitude (see Table 8). At Custer County, the final thickness of asphalt was 0.5 inches higher for the control section. At Lincoln Road, the final thickness of asphalt was 0.5 inches higher for the test section. At Bearmouth-Drummond, the final thickness of asphalt was 1.5 inches higher for the test section. At Rocky Canyon, the final thickness of asphalt was 6.5 inches higher for the test section. (Recall that at Rocky Canyon, the control section was supplied with a cement-treated base [CTPB]).

Stripping evaluations for cores are summarized in terms of thickness ranges for non-stripped material near the pavement surface (MDT stripping rating of 3 or 4) and thickness ranges for stripped material near the bottom of the core (MDT stripping rating of 1 or 2), see Table 12. The thickness ranges for non-stripped material are compared to the thicknesses of asphalt concrete placed during rehabilitation in Figure 41. With exception for the Tarkio site, the thickness of non-stripped material near the pavement surface was similar to the thickness of new asphalt concrete placed during rehabilitation, with allowance for variability associated with construction and coring evaluations. At the Tarkio site, however, the measured thicknesses of non-stripped material were consistently lower than the thickness of new asphalt concrete placed during rehabilitation. Using median values for non-stripped material at Tarkio, it would appear that stripping damage had taken about 1 in. of the new asphalt concrete placed during rehabilitation from both the control and test sections.

Table 12. Stripping Measurements from Cores

Test Site	Control or Test Section	No. of Cores	Thickness[a] (in.) Near Pavement Surface with Stripping Rating[b] of 3 or 4		Thickness[a] (in.) Near Bottom of Core with Stripping Rating[b] of 1 or 2		Thickness (in.) of Asphalt Concrete Placed During Rehabilitation
			Min. (in.)	Max. (in.)	Min. (in.)	Max. (in.)	
Bearmouth – Drummond (West)	C	6	2.5	4.5	2.0	3.9	3½
	T	6	3.7	4.7	2.0	3.9	2
Bearmouth – Drummond (East)	C	6	2.4	3.2	3.5	3.9	3½
	T	6	3.0	4.6	2.4	3.9	2
Rocky Canyon	C	6	5.1	5.5	0.0	0.0	5[c]
	T	7	2.0	6.3	5.9	12.6	3½
Lincoln Road-Sieben	C	12	2.0	5.9	1.6	7.1	3½
	T	12	2.0	2.4	2.4	6.7	2
Custer County Line West	C	9	3.5	11.0	2.0	8.3	8
	T	9	3.5	8.3	2.4	6.7	5
Tarkio-East	C	9	2.0	4.3	2.4	4.7	4
	T	9	0.8	2.4	4.3	6.3	2½

[a] of asphalt-bound materials
[b] based on MDT rating system (MT-331)
[c] 9½ in. of CTPB placed prior to asphalt concrete during rehabilitation
Note: All cores were in good condition (stripping rating 3 or 4) near the pavement surface and most were in poor condition (stripping rating 1 or 2) near the bottom.

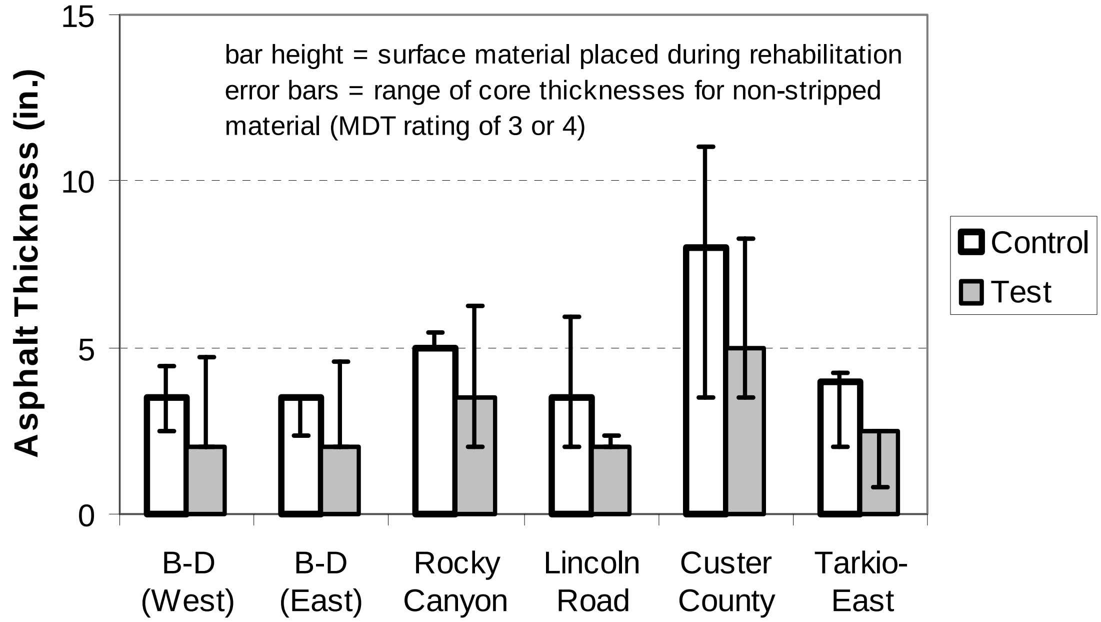

Figure 41. Thickness of Non-stripped Asphalt Concrete Near the Pavement Surface

6.5 <u>Precipitation</u>

Portions of Montana experienced relatively low levels of annual precipitation during this experiment. Lack of moisture would certainly affect a study concerned with stripping damage in asphalt concrete. To investigate the possibility of low moisture at the five test sites, summary precipitation data were collected for the years 1981 through 2000. Table 13 and Figure 42 compare average annual precipitation during the experiment with average annual precipitation for over a decade prior to the experiment. The data for the years prior to the experiment provide an indication of the moisture made available to the pavements during the period that they originally suffered stripping damage (i.e. prior to rehabilitation). The figure shows that while Bearmouth Drummond, Rocky Canyon, and Custer County experienced lower-than-normal levels of rainfall during the experiment (15, 21, and 23 percent low, respectively), Tarkio experienced slightly higher-than-normal levels of rainfall (7 percent high), and Lincoln Road experienced normal levels of rainfall.

Table 13. Precipitation Data Prior To and During the Experiment

Year(s)	Weather Station	Recorded Annual Cumulative Precipitation (in.)			
		Average	Standard Deviation	Minimum	Maximum
1981 to 1993[a]	1. Drummond Aviation	12.6	3.2	6.4	16.6
	2. MSU at Bozeman	12.2	3.0	8.4	18.8
	3. Helena Airport	20.1	2.7	15.6	23.5
	4. Miles City Airport	12.9	4.7	5.3	19.9
	5. Superior	16.2	2.0	12.1	20.4
1994 to 2000	1. Drummond Aviation	10.7	3.3	5.6	15.2
	2. MSU at Bozeman	9.6	2.7	5.1	12.6
	3. Helena Airport	20.5	8.7	11.0	37.6
	4. Miles City Airport	9.9	4.2	2.1	14.2
	5. Superior	17.3	6.2	8.3	25.6

[a] with exception for 1985 and 1987
Note: Weather station proximities to test sections - (1) is near Bearmouth-Drummond, (2) is near Rocky Canyon, (3) is near Lincoln Road – Sieben, (4) is near Custer County Line West, and (5) is near Tarkio East.

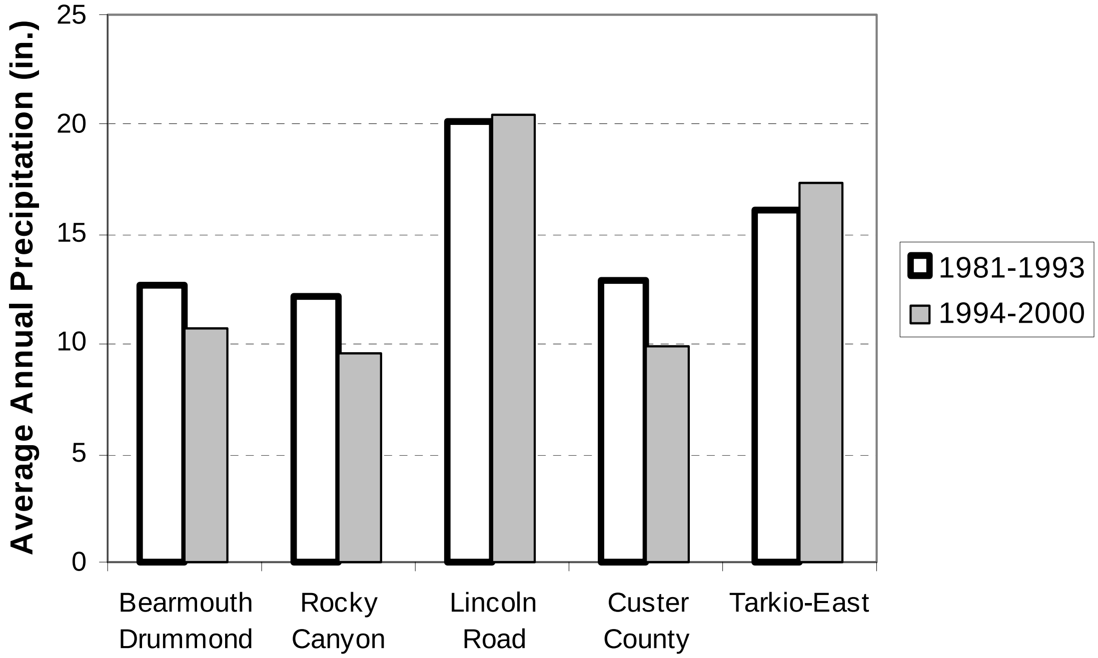

Figure 43. Average Annual Precipitation Near Test Sites During the Experiment

Therefore, it would appear that lack of normal moisture should not have skewed the research results at Tarkio or Lincoln Road. However, lower-than-normal rainfall is a factor that should be taken under consideration when analyzing the results for Bearmouth Drummond, Rocky Canyon, and Custer County. In summary, it does not appear that the reduced precipitation in Montana in recent years should have a substantial effect on this asphalt stripping experiment.

6.6 Test Section Summaries

Bearmouth-Drummond site results: The Bearmouth-Drummond test site (on I-90, near Drummond) includes both a 500-foot control section and a 500-foot test section in each of the eastbound and westbound lanes. In the control section, 3.5 inches of stripped material was milled and replaced with an overlay of the same thickness (half hot-recycled neat binder course and half polymer-modified surface course). In the test section, 0.5 inches of stripped open-graded friction course (OGFC) was removed by milling, followed by an overlay of approximately 1.5 inches of polymer-modified plant-mix.

Annual structural integrity evaluations (both Impulse Stimulus Modulus and Basin Curvature) indicated a trend toward increased stiffness over time with no significant difference in results between the control and test sections (except for a one-time relatively high ISM result in the eastbound lane test section). Roughness evaluations in the westbound lanes indicated decreased roughness over time; however, roughness values were significantly higher each year in the test section than in the control section. In the eastbound lanes, roughness values decreased over time in the test section, and increased over time in the control section, and by the final evaluation year the roughness value in the control section exceeded that of the test section. The Bearmouth-Drummond site had the most rutting (up to about 0.4 inches) of any of the test sites, with a significant increase in the westbound lanes from 1999 to 2000. However, the difference in rutting between the control and test sections was not significant. Visual distress evaluations indicated that the site is in good condition, with no evidence of fatigue cracking or pothole formation. Both westbound and eastbound lanes show low severity bleeding and low severity transverse cracking, but with no significant difference between the control and test sections. Core evaluations suggest that stripping damage has not progressed significantly in either the control or test sections. Finally, precipitation evaluations indicate that the Bearmounth-Drummond site experienced rainfall levels 15 percent lower than normal over the course of this evaluation project; this lack of moisture may have affected the results at this site.

Rocky Canyon site results: The Rocky Canyon test site (on I-90, near Bozeman) includes a 1370-foot control section and a 1370-foot test section in the eastbound lane, separated by a bridge. In the control section, 5 inches of stripped asphalt concrete was removed by milling. The remaining asphalt concrete and part of the unbound aggregate base was mixed in-situ with Portland cement to create a stabilized layer with a thickness of approximately 9.5 inches. The control section overlay included 5 inches of polymer-modified plant-mix. For the test section, the 0.5-inch OGFC was removed by milling, followed by a 3.5-inch overlay of polymer-modified surface course.

The Impulse Stimulus Modulus evaluation indicated a slight overall trend toward increased stiffness over the course of the project in both the control and test sections, although there was a relatively high variability from one year to the next (i.e. an increase one year, a decrease the next). By contrast, Basin Curvature values changed very little at Rocky Canyon, with very close results between the control and test sections. Roughness evaluations indicated increased roughness over time; the roughness value was initially higher for the control section, but by the last evaluation year, the values in the control and test sections were nearly equal. At the Rocky Canyon site, rutting increased significantly from 1999 to 2000; however, the difference in rutting between the control and test sections was not significant. Visual distress evaluations indicated that the site is in good condition, with no evidence of fatigue cracking or pothole formation, and all evidence of bleeding is low severity. However, the control section experienced more transverse cracking than the test section: 9 full-width cracks for the control section versus 1 full-width crack for the test section. Core evaluations suggest that stripping damage has not progressed significantly in either the control or test sections. Precipitation evaluations indicate that the Rocky Canyon site experienced rainfall levels 21 percent lower than normal over the course of this evaluation project; this lack of moisture may have affected the results at this site.

Finally, it is important to note that the Rocky Canyon site was the only site where the control and test sections were separated by a bridge. The experimental sections were carefully analyzed to determine whether the presence of a bridge had an impact on the results. Researchers did not find a disproportionate amount of distress near the bridge, so its presence was judged to be inconsequential.

Lincoln Road site results: The Lincoln Road-Sieben test site (on I-15, near Helena) includes a 1320-foot control section and a 1320-foot test section in the northbound lane. In the control section, 3 inches of stripped asphalt concrete was removed by milling, followed by an overlay of 3.5 inches (half hot-recycled binder course and half polymer-modified surface course, which included the process of hot recycling). In the test section, the 0.5-inch OGFC was removed by milling and the overlay included approximately 1.5 inches of polymer-modified asphalt, which was accomplished with hot recycling. Among the pavements in this experiment, these were the only pavement sections that were surfaced with polymer-modified asphalt that was uniquely accomplished with hot-recycling technology.

Annual structural integrity evaluations (both Impulse Stimulus Modulus and Basin Curvature) indicated a trend toward increased stiffness over time with very close results between the control and test sections. Roughness evaluations indicated a slight decrease in roughness for the control section, but an increase for the test section. The Lincoln Road site had the least rutting (up to about 0.2 inches) of any of the test sites, with no significant difference between the control and test sections. Visual distress evaluations indicated that the site is in good condition, with no evidence of fatigue cracking or pothole formation, and all evidence of bleeding is low severity. However, this site had the highest density of transverse cracks: 17 full-width cracks in the control section and 24 full width cracks in the test section. Core evaluations suggest that stripping damage has not progressed significantly in either the control or test sections. Finally, precipitation evaluations indicate that during this experiment, the Lincoln Road site experienced rainfall levels commensurate with pre-1994 averages.

Custer County site results: The Custer County Line West test site (on I-94, near Miles City) includes a 1320-foot control section and a 1320-foot test section in the westbound lane. In the control section, 3 inches of stripped asphalt was removed by milling, followed by an asphalt overlay with thickness of almost 8 inches (3 inches of unmodified binder course, followed by 5 inches of polymer-modified surface course). In the test section, 0.5 inches of stripped OGFC was removed by milling, followed by an overlay of approximately 5 inches of polymer-modified asphalt plant mix.

Custer County was the only site where annual structural integrity evaluations (both Impulse Stimulus Modulus and Basin Curvature) indicated that pavement stiffness decreased steadily over the course of the project, in both the control and test sections. These results indicate pavement deterioration. Roughness evaluations indicated an increase in roughness over time, slightly more so for the test section than for the control section. At the Custer County site, rutting increased significantly from 1999 to 2000, with the control section rutting more than the test section. Visual distress evaluations indicated that the site is in good condition, with no evidence of fatigue cracking or pothole formation, and all evidence of bleeding is low severity. Custer County also had the lowest incidence of transverse cracking, with all cracks being low severity. Core evaluations suggest that stripping damage has not progressed significantly in either the control or test sections. Finally, precipitation evaluations indicate that the Custer County site experienced rainfall levels 23 percent lower than normal over the course of this evaluation project; this lack of moisture may have affected the results at this site.

Tarkio East site results: The Tarkio East test site (on I-90, near Tarkio) includes a 1320-foot control section and a 1320-foot test section in the eastbound lane. In the control section, 2.5 inches of stripped asphalt was removed by milling, followed by a 4-inch overlay (1.5 inches of hot-recycled binder course, topped with 2.5 inches of polymer-modified surface course). In the test section, 0.5 inches of stripped OGFC was removed by milling, followed by an overlay of approximately 2.5 inches of polymer-modified surface course.

Annual structural integrity evaluations (both Impulse Stimulus Modulus and Basin Curvature) indicated a trend toward increased stiffness over time. The ISM evaluation suggested that the test section was slightly less stiff than the control section, but the Basin Curvature evaluation produced very close results between the control and test sections. The International Roughness Index was higher for the test section than the control section on all evaluation dates. At the Tarkio site, rutting increased slightly, but with little significant difference between the control and test sections. Visual distress evaluations indicated that the site is in good condition, with no evidence of fatigue cracking or pothole formation, and only one transverse crack. The Tarkio site was the only location to show some evidence of moderate severity bleeding. It was also the only site where core evaluations suggested that significant stripping damage had occurred: it appears that stripping damage had taken about 1 inch of the new asphalt concrete placed during rehabilitation from both the control and test sections. Finally, precipitation evaluations indicate that the Tarkio site experienced average rainfall 7 percent higher than pre-1994 averages.

7. Summary

1. This experiment compared the performance of two methods for rehabilitating stripped asphalt concrete pavements: milling to remove most stripped material, followed by an overlay, versus a simple overlay. Mill and overlay is the conventional rehabilitation approach for the Montana Department of Transportation. In the simple overlay approach used in this study, only the porous friction course material was removed from the pavement surface; all the remaining stripped material was assumed to serve as a base course. Five sites at various locations around the state were included in the study. These sites are referred to as Bearmouth-Drummond, Rocky Canyon, Lincoln Road-Sieben, Custer County Line West, and Tarkio-East. Rehabilitations were performed from 1995 to 1997 and the sites were monitored until the year 2000.

2. The original pavements selected for this experiment included 4 to 6 inches of asphalt concrete over 18 to 29 inches of crushed aggregate base. Bearmouth-Drummond and Rocky Canyon were rehabilitated in 1995, Lincoln Road and Custer County were rehabilitated in 1996, and Tarkio was rehabilitated in 1997. The conventional rehabilitation approach (control) involved milling 2.5 to 5 inches and typically placing 3.5 to 5 inches of new asphalt concrete. As one exception, Custer County received 8 inches of new material because the pre-rehabilitation structural evaluation found the pavement to be of marginal quality. The second exception was Rocky Canyon, where 9.5 inches of cement-treated pulverized base (CTPB) was placed prior to the overlay because projected traffic levels at this site were very high. The simple overlay rehabilitation techniques involved placing 2 to 5 inches of new asphalt concrete on top of the stripped asphalt concrete. All asphalt concrete placed during rehabilitations contained lime for improved resistance to stripping.

3. Structural evaluations, provided by a falling-weight deflectometer, revealed that for most sites, the structures increased in stiffness over time. Custer County, as the exception, showed a steady decline in stiffness, but this trend was at least partially attributable to high testing temperatures during the years 2000 and 2001. Structural evaluations at Rocky Canyon showed high spatial variability for the control section, which included the CTPB layer.

4. Roughness values (IRI) were generally lower for the control sections than for the test sections. Roughness did not generally increase with time for either the control sections or the test sections. Almost all IRI measurements were less than 80, which is considered by the MDT to be borderline good to fair.

5. Rutting was monitored with a Rainhart Profilograph. There was not a consistent difference between the performance of the control and test sections. The only statistically significant difference in rutting between control and test sections occurred at Custer County, where the control section rutted significantly more than the test section. A reasonable cause for the higher rutting in the control section for this site could not be determined.

6. The predominant visual distress at the test sites was transverse cracking. By the year 2000, all transverse cracks remained at a low severity level. With the exception of Rocky Canyon, substantial differences between control and test sections were generally not observed. At Rocky Canyon (year 2000) the control section contained nine full-width cracks while the test section contained only one. The CTPB in this case appears to have worsened cracking problems, possibly due to drying shrinkage. The Lincoln Road site had the highest overall quantity of cracks: 17 for the control section and 24 for the test section. This site was the most severely cracked prior to rehabilitation. Also, this was the only site where polymer-modification of asphalt was used in conjunction with hot-mix recycling.

7. During the last year of visual inspections (year 2000), cores were removed from the experimental pavement sections to inspect for stripping damage. With the exception of the Tarkio site, stripping had not progressed significantly. At Tarkio, the newly placed asphalt concrete had stripped from the bottom by a maximum of approximately 2 inches, thus leaving a minimum of 2 inches of intact material in the control section and leaving a minimum of about 0.75 inches of intact material in the test section. Tarkio was the only site that experienced precipitation during the experiment that was high relative to normal, where normal was defined as the average precipitation for the previous 13 years.

8. Primary Conclusion

Leaving stripped asphalt concrete surface material in place during rehabilitation, to be overlayed with new asphalt concrete, did not tend to make the rehabilitated pavement more susceptible to either stripping damage or load-induced damage. Also, in the case of rehabilitating a pavement that had severe transverse temperature cracking, the removal of the top 2.5 to 5 inches of stripped material did not seem to substantially improve resistance to reflective cracking. Milling and placement of new asphalt concrete in two lifts, however, did result in slightly smoother pavements than the direct placement of a single-lift overlay (with only the very thin removal of a porous friction course).

9. Recommendations

1. All sites should continue to be monitored, both structurally and visually. Although the Lincoln Road and Tarkio sites experienced sufficient precipitation to permit conclusive evidence for this study, the other three sites had relatively dry weather during the experiment. Continued monitoring, through what will hopefully be wet years, will provide additional information.

2. Continued structural monitoring for the Custer County site is of particular interest. This site showed signs of structural deterioration for both the control and test sections. If deterioration continues, the falling-weight deflectometer will have successfully identified deterioration at its early stages.

3. Continued monitoring of stripping damage for the Tarkio site is of particular interest. Both the control and the test section have experienced a significant and equal amount of deterioration from the bottom of the newly-placed asphalt concrete. Because the test section received a thinner overlay than the control section (2.5 inches versus 4 inches), the test section has less intact material left near the surface (less than 1 in. in some cases). The test section will, therefore, most likely show stripping damage at the pavement surface before the control section. Life-cycle cost analyses, however, should consider rate of stripping deterioration (in./year) to be the same for either rehabilitation technique (i.e. either mill and overlay or simple overlay). Overlay thickness and mix design methods for resisting stripping are the important factors for extending the life of a rehabilitated stripped asphalt pavement.

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Appendix A

Montana Method MT-331, “Method of Sampling and Evaluating Stripping Pavements”

MT-331

METHOD OF SAMPLING AND TESTING
MT-331
METHOD OF SAMPLING AND EVALUATING
STRIPPING PAVEMENTS
(Montana Method)

1 Scope:

1.1 This method describes the procedure for sampling and evaluating asphalt pavements that are suspected of stripping.

2 Sampling Procedure:

2.1 **4-Lane Roadway**
A minimum of 2 cores per mile of roadway, of which 1 core will be taken in the outside wheelpath of the driving lane and the other core in the passing lane.

Example

		|← 1 Mile →|	|← 1 Mile →|
Traffic ←	D.L.	0 (core)	0 (core)
Traffic ←	P.L.	0 (core)	0 (core)
Traffic →	P.L.	0 (core)	0
Traffic →	D.L.	0 (core)	0 (core)

2.2 **2-Lane Roadway**
A minimum of 1 core per mile of roadway, of which each core will be taken in the outside wheelpath and in alternating lanes.

Example

	|← 1 Mile →|	|← 1 Mile →|
Traffic →	0 (core)	
Traffic ←		(core) 0

2.3 Cores should represent the pavement for the proposed project. Depending upon pavement condition, it may be necessary to modify the sampling frequency and location.

2.4 The exact locations of the cores within the 1 mile section should be determined by a random method.

3 Sample Identification:

3.1 Cores are to be marked for identification. If cores are not intact, rubble from the field has no useful purpose and should not be submitted. Field notes should be kept describing the appearance of the core. Describe where the intact portion was on the core; top, middle or bottom of the plant mix and submit it for stripping evaluation. Also describe the roadway condition and any

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3 Sample Identification: (continued)

other information that would be helpful in evaluating the cores and the in-place pavement.

4 Submitting of Samples:

4.1 The cores are to be submitted to the Materials Bureau for evaluation. Each core is to be accompanied by a completed Form 31. Observations and comments should be included on the Form 31.

5 Evaluation of Cores

5.1 Procedure:

5.1.1 The total core should be evaluated for stripping using the "control photographs" developed by the Materials Bureau. Split cores by indirect tensile loading in a press. Evaluate each lift or distinct layer of plant mix for stripping using the numeric rating system that corresponds to the reference photographs provided by the Materials Bureau.

5.1.2 Extract asphalt and do an abson recovery from a composite sample of plant mix from four complete cores. Determine the viscosity of this asphalt. This value is used in the Road Rater back calculation of Resilient Modulus.

6 Reporting Results:

6.1 At the completion of the evaluation, a report will be written by the Materials Bureau with test results and recommendations concerning extent of stripping, removal of stripped material and other test information. The report should evaluate each lift of plant mix for stripping.

Appendix B

Structural Condition Data

Table B1. Deflection Basin Parameters Calculated from Road Rater Data for Bearmouth-Drummond Westbound Lane (May 22, 1997)

Characteristic	Control Section	Test Section
Surface & Mat Temperature	76 & 72	76 & 72
Number of Test Locations	11	11
ISM [a] (ksi)		
Mean	670	842
Standard Deviation	7.59	243
Skewness	-1.80	1.12
Minimum	651	670
Maximum	678	1240
D1/D2 [b]		
Mean	2.93	2.49
Standard Deviation	0.257	0.793
Skewness	-0.192	-0.510
Minimum	2.56	1.38
Maximum	3.27	3.21
D1/D3 [b]		
Mean	3.65	3.05
Standard Deviation	0.239	0.989
Skewness	-0.199	-0.522
Minimum	3.22	1.75
Maximum	4.08	4.10

Note: Measured peak load was approximately 4000 lb.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B2. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Westbound Lane (April 27, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat	56 & 60			55 & 57		
Number of Test Locations	10			11		
Average Peak Load (kips)	5.8	8.0	9.9	5.8	8.0	9.9
ISM [a] (ksi)						
Mean	1050	1110	1160	1140	1190	1220
Standard Deviation	86.2	90.5	97.2	162	168	172
Skewness	-0.799	-0.670	-0.819	-0.124	-0.405	-0.413
Minimum	875	941	973	892	896	907
Maximum	1150	1220	1260	1370	1430	1460
D1/D2 [b]						
Mean	1.28	1.26	1.23	1.27	1.25	1.23
Standard Deviation	0.048	0.051	0.047	0.046	0.049	0.055
Skewness	1.30	1.23	1.15	1.35	1.66	1.36
Minimum	1.24	1.22	1.19	1.23	1.20	1.16
Maximum	1.38	1.35	1.33	1.38	1.37	1.36
D1/D3 [b]						
Mean	1.53	1.51	1.50	1.48	1.47	1.45
Standard Deviation	0.109	0.112	0.113	0.084	0.080	0.072
Skewness	1.96	2.04	2.03	0.889	0.860	0.894
Minimum	1.45	1.42	1.41	1.36	1.35	1.33
Maximum	1.80	1.79	1.78	1.65	1.63	1.62

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B3. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Westbound Lane (May 5, 1999)

Characteristic	Control Section			Test Section		
Surface & Mat	36 & 40			36 & 40		
Number of Test Locations	11			10		
Average Peak Load (kips)	6.1	8.0	9.9	6.1	8.0	9.9
ISM [a] (ksi)						
Mean	1320	1380	1410	1400	1440	1450
Standard Deviation	70.5	74.2	77.4	164	161	158
Skewness	-0.441	-0.721	-0.577	0.361	0.292	0.056
Minimum	1170	1220	1260	1190	1190	1190
Maximum	1430	1470	1510	1640	1680	1700
D1/D2 [b]						
Mean	1.21	1.21	1.20	1.21	1.21	1.20
Standard Deviation	0.033	0.035	0.037	0.034	0.028	0.022
Skewness	1.25	1.39	1.23	1.14	1.49	0.891
Minimum	1.18	1.17	1.16	1.17	1.18	1.17
Maximum	1.28	1.29	1.28	1.28	1.27	1.25
D1/D3 [b]						
Mean	1.42	1.41	1.38	1.39	1.39	1.36
Standard Deviation	0.051	0.050	0.053	0.053	0.047	0.034
Skewness	1.63	1.55	1.23	0.979	1.35	1.45
Minimum	1.36	1.34	1.31	1.33	1.33	1.31
Maximum	1.55	1.53	1.51	1.51	1.50	1.45

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B4. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Westbound Lane (May 9, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat	48 & 46			47 & 45		
Number of Test Locations	10			10		
Average Peak Load (kips)	6.7	8.2	9.9	7.4	8.8	10.4
ISM [a] (ksi)						
Mean	1320	1360	1400	1360	1380	1410
Standard Deviation	58.8	58.6	58.5	90.8	78.7	82.2
Skewness	-0.782	-0.991	-0.907	0.705	0.383	0.585
Minimum	1210	1240	1300	1260	1270	1290
Maximum	1400	1430	1470	1510	1510	1560
D1/D2 [b]						
Mean	1.17	1.17	1.16	1.18	1.17	1.16
Standard Deviation	0.024	0.022	0.023	0.035	0.033	0.031
Skewness	0.956	0.990	1.13	-1.57	-1.46	-1.41
Minimum	1.15	1.14	1.13	1.10	1.09	1.08
Maximum	1.24	1.22	1.20	1.22	1.20	1.19
D1/D3 [b]						
Mean	1.41	1.39	1.37	1.38	1.37	1.36
Standard Deviation	0.035	0.039	0.032	0.068	0.063	0.063
Skewness	0.500	0.898	0.846	-2.08	-2.03	-1.78
Minimum	1.37	1.34	1.33	1.21	1.21	1.20
Maximum	1.47	1.47	1.43	1.44	1.42	1.42

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B5. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Westbound Lane (April 19, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat	41 & 43			41 & 43		
Number of Test Locations	16			13		
Average Peak Load (kips)	6.0	8.3	10.1	6.1	8.4	10.3
ISM [a] (ksi)						
Mean	1330	1340	1350	1290	1350	1460
Standard Deviation	146	143	148	105	91.3	120
Skewness	-0.940	-0.739	-0.339	-1.04	-0.482	0.344
Minimum	990	1040	1090	1150	1220	1250
Maximum	1540	1510	1550	1390	1480	1730
D1/D2 [b]						
Mean	1.18	1.17	1.16	1.18	1.18	1.18
Standard Deviation	0.020	0.033	0.037	0.015	0.025	0.020
Skewness	0.624	-0.810	-0.144	-0.258	0.370	0.417
Minimum	1.14	1.10	1.09	1.16	1.15	1.16
Maximum	1.22	1.22	1.24	1.20	1.22	1.22
D1/D3 [b]						
Mean	1.35	1.33	1.33	1.39	1.38	1.37
Standard Deviation	0.036	0.050	0.061	0.018	0.031	0.027
Skewness	-0.069	-0.500	0.421	-0.432	1.878	0.657
Minimum	1.30	1.23	1.22	1.37	1.35	1.33
Maximum	1.41	1.42	1.47	1.41	1.45	1.42

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B6. Deflection Basin Parameters Calculated from Road Rater Data for Bearmouth-Drummond Eastbound Lane (May 20, 1997)

Characteristic	Control Section	Test Section
Surface & Mat Temperature	90 & 66	90 & 66
Number of Test Locations	11	11
ISM [a] (ksi)		
Mean	913	951
Standard Deviation	113	97.6
Skewness	-0.374	-0.107
Minimum	686	802
Maximum	1120	1100
D1/D2 [b]		
Mean	1.66	1.82
Standard Deviation	0.064	0.209
Skewness	0.454	1.239
Minimum	1.57	1.61
Maximum	1.76	2.27
D1/D3 [b]		
Mean	2.16	2.28
Standard Deviation	0.095	0.242
Skewness	-0.945	1.309
Minimum	1.98	2.02
Maximum	2.28	2.84

Note: Measured peak load was approximately 4000 lb.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B7. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Eastbound Lane (April 27, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat	50 & 50			51 & 51		
Number of Test Locations	11			10		
Average Peak Load (kips)	5.9	8.2	10.0	5.8	8.0	9.9
ISM [a] (ksi)						
Mean	915	965	1000	916	940	965
Standard Deviation	92.6	97.7	106	105	87.2	107
Skewness	-1.41	-1.54	-1.48	-0.090	0.120	0.397
Minimum	694	730	751	771	823	836
Maximum	1030	1090	1140	1070	1080	1150
D1/D2 [b]						
Mean	1.29	1.27	1.26	1.37	1.36	1.30
Standard Deviation	0.054	0.054	0.057	0.106	0.120	0.061
Skewness	0.473	0.720	0.778	1.87	1.21	0.751
Minimum	1.23	1.20	1.19	1.28	1.24	1.23
Maximum	1.38	1.38	1.37	1.63	1.61	1.42
D1/D3 [b]						
Mean	1.58	1.56	1.55	1.63	1.65	1.59
Standard Deviation	0.121	0.117	0.118	0.133	0.144	0.115
Skewness	0.464	0.371	0.370	1.11	0.288	0.733
Minimum	1.43	1.40	1.40	1.51	1.49	1.46
Maximum	1.78	1.76	1.74	1.90	1.88	1.78

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B8. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Eastbound Lane (May 5, 1999)

Characteristic	Control Section			Test Section		
Surface & Mat	36 & 40			36 & 40		
Number of Test Locations	11			10		
Average Peak Load (kips)	6.1	8.1	10.0	6.1	8.0	9.9
ISM [a] (ksi)						
Mean	1110	1160	1200	1160	1210	1220
Standard Deviation	79.6	85.9	93.2	143	135	135
Skewness	-0.691	-1.11	-1.11	-0.181	-0.494	-0.595
Minimum	949	979	1010	908	951	967
Maximum	1210	1260	1290	1370	1410	1430
D1/D2 [b]						
Mean	1.24	1.24	1.24	1.26	1.26	1.26
Standard Deviation	0.037	0.041	0.047	0.052	0.050	0.044
Skewness	1.10	1.14	0.972	1.09	1.26	0.693
Minimum	1.20	1.20	1.18	1.20	1.21	1.21
Maximum	1.31	1.32	1.33	1.36	1.37	1.34
D1/D3 [b]						
Mean	1.51	1.48	1.45	1.48	1.47	1.45
Standard Deviation	0.086	0.078	0.079	0.092	0.088	0.077
Skewness	0.833	0.856	0.897	1.467	1.642	1.373
Minimum	1.41	1.39	1.36	1.39	1.39	1.37
Maximum	1.65	1.60	1.58	1.69	1.68	1.63

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B9. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Eastbound Lane (May 9, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat	45 & 44			46 & 45		
Number of Test Locations	10			10		
Average Peak Load (kips)	6.6	8.1	9.8	6.6	8.1	9.9
ISM [a] (ksi)						
Mean	1150	1170	1170	1060	1100	1150
Standard Deviation	125	119	107	138	146	160
Skewness	0.546	0.454	0.415	-1.98	-1.95	-1.90
Minimum	1000	1030	1040	714	736	755
Maximum	1350	1380	1350	1170	1220	1280
D1/D2 [b]						
Mean	1.23	1.21	1.20	1.23	1.21	1.20
Standard Deviation	0.030	0.031	0.034	0.024	0.021	0.025
Skewness	-0.250	0.154	0.272	-0.765	-0.774	-0.778
Minimum	1.19	1.17	1.15	1.18	1.16	1.14
Maximum	1.26	1.26	1.25	1.26	1.23	1.23
D1/D3 [b]						
Mean	1.47	1.45	1.43	1.50	1.47	1.46
Standard Deviation	0.040	0.041	0.041	0.050	0.049	0.051
Skewness	-1.06	-1.10	-0.014	-1.13	-1.33	-1.00
Minimum	1.38	1.36	1.36	1.39	1.36	1.34
Maximum	1.52	1.49	1.49	1.57	1.54	1.53

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B10. Deflection Basin Parameters Calculated from FWD Data for Bearmouth-Drummond Eastbound Lane (April 19, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat	41 & 43			41 & 43		
Number of Test Locations	16			11		
Average Peak Load (kips)	6.2	8.5	10.4	6.1	8.3	10.3
ISM [a] (ksi)						
Mean	1150	1210	1240	1410	1450	1500
Standard Deviation	148	122	131	170	165	176
Skewness	-0.341	-1.06	-0.806	-0.287	-0.313	-0.130
Minimum	901	940	966	1130	1180	1210
Maximum	1380	1410	1440	1680	1720	1800
D1/D2 [b]						
Mean	1.21	1.21	1.22	1.22	1.23	1.23
Standard Deviation	0.024	0.031	0.029	0.054	0.050	0.050
Skewness	-0.092	-0.433	-0.139	-0.387	0.214	0.201
Minimum	1.16	1.15	1.17	1.11	1.15	1.16
Maximum	1.24	1.26	1.27	1.31	1.32	1.31
D1/D3 [b]						
Mean	1.43	1.44	1.45	1.44	1.45	1.43
Standard Deviation	0.044	0.052	0.053	0.080	0.065	0.082
Skewness	0.473	0.100	-0.062	-0.562	-0.273	-0.078
Minimum	1.38	1.34	1.34	1.30	1.34	1.31
Maximum	1.51	1.54	1.55	1.54	1.55	1.56

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B11. Deflection Basin Parameters Calculated from Road Rater Data for Rocky Canyon (May 22, 1997)

Characteristic	Control Section	Test Section
Surface & Mat Temperature (ºF)	67 & 80	67 & 80
Number of Test Locations	28	27
ISM [a] (ksi)		
Mean	1690	1620
Standard Deviation	481	310
Skewness	0.553	0.180
Minimum	888	906
Maximum	2750	2450
D1/D2 [b]		
Mean	1.51	1.56
Standard Deviation	0.161	0.267
Skewness	0.691	2.71
Minimum	1.25	1.31
Maximum	1.88	2.64
D1/D3 [b]		
Mean	1.78	1.82
Standard Deviation	0.198	0.307
Skewness	0.655	2.64
Minimum	1.42	1.53
Maximum	2.27	3.05

Note: Measured peak load was approximately 4000 lb.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B12. Deflection Basin Parameters Calculated from FWD Data for Rocky Canyon (April 26, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat Temperature	62 & 82			51 & 80		
Number of Test Locations	25			28		
Average Peak Load (kips)	5.3	8.2	10.0	5.6	7.7	10.1
ISM [a] (ksi)						
Mean	1752	1846	1857	1781	1773	1768
Standard Deviation	425	468	462	259	268	262
Skewness	0.049	0.169	0.156	0.049	0.133	0.050
Minimum	1000	996	997	1313	1297	1297
Maximum	2505	2830	2873	2327	2375	2286
D1/D2 [b]						
Mean	1.244	1.196	1.167	1.263	1.230	1.194
Standard Deviation	0.039	0.043	0.036	0.051	0.051	0.037
Skewness	0.610	0.470	0.256	1.475	1.212	0.921
Minimum	1.178	1.119	1.092	1.210	1.161	1.144
Maximum	1.329	1.295	1.256	1.414	1.367	1.292
D1/D3 [b]						
Mean	1.387	1.352	1.345	1.406	1.393	1.394
Standard Deviation	0.091	0.085	0.082	0.072	0.067	0.068
Skewness	0.125	0.263	0.312	1.224	1.140	1.357
Minimum	1.233	1.219	1.210	1.315	1.300	1.311
Maximum	1.530	1.515	1.503	1.615	1.561	1.586

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B13. Deflection Basin Parameters Calculated from FWD Data for Rocky Canyon (May 20, 1999)

Characteristic	Control Section		Test Section	
Surface & Mat Temperature	44 & 45		45 & 45	
Number of Test Locations	28		27	
Average Peak Load (kips)	7.7	9.6	7.9	9.8
ISM [a] (ksi)				
Mean	2320	2290	2120	2100
Standard Deviation	544	521	408	383
Skewness	-0.164	-0.166	0.071	-0.171
Minimum	1070	1080	1120	1110
Maximum	3180	3080	2970	2900
D1/D2 [b]				
Mean	1.21	1.21	1.21	1.20
Standard Deviation	0.055	0.050	0.032	0.035
Skewness	0.524	0.462	0.760	0.935
Minimum	1.12	1.13	1.16	1.15
Maximum	1.33	1.31	1.29	1.29
D1/D3 [b]				
Mean	1.35	1.34	1.37	1.37
Standard Deviation	0.096	0.086	0.057	0.055
Skewness	0.850	0.672	0.858	0.932
Minimum	1.22	1.24	1.29	1.30
Maximum	1.61	1.56	1.53	1.52

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B14. Deflection Basin Parameters Calculated from FWD Data for Rocky Canyon (May 4, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat Temperature	35 & 48			35 & 49		
Number of Test Locations	27			28		
Average Peak Load (kips)	5.7	7.8	9.6	5.7	7.9	9.7
ISM [a] (ksi)						
Mean	1640	2230	2250	1990	2010	1980
Standard Deviation	252	620	606	307	324	312
Skewness	-0.439	0.111	0.015	-1.36	-0.979	-0.968
Minimum	1120	1150	1170	1040	1050	1060
Maximum	2060	3300	3260	2360	2530	2460
D1/D2 [b]						
Mean	1.19	1.16	1.15	1.21	1.20	1.18
Standard Deviation	0.038	0.031	0.034	0.042	0.038	0.035
Skewness	0.727	-0.252	-0.318	0.455	0.176	-0.114
Minimum	1.14	1.08	1.06	1.14	1.13	1.11
Maximum	1.26	1.22	1.23	1.30	1.27	1.24
D1/D3 [b]						
Mean	1.40	1.34	1.32	1.38	1.39	1.38
Standard Deviation	0.059	0.070	0.069	0.074	0.070	0.068
Skewness	-0.471	-0.278	-0.169	0.697	0.647	0.266
Minimum	1.31	1.18	1.15	1.27	1.27	1.26
Maximum	1.48	1.45	1.45	1.54	1.54	1.50

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B15. Deflection Basin Parameters Calculated from FWD Data for Rocky Canyon (April 24, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat Temperature	49 & 44			49 & 41		
Number of Test Locations	28			28		
Average Peak Load (kips)	5.9	8.0	9.9	6.0	8.2	10.1
ISM [a] (ksi)						
Mean	1820	1960	2000	2310	2280	2320
Standard Deviation	663	567	552	423	372	375
Skewness	0.159	0.207	0.198	-2.58	-1.77	-1.85
Minimum	788	821	847	612	968	982
Maximum	3050	2970	3020	2920	2900	2930
D1/D2 [b]						
Mean	1.17	1.18	1.18	1.18	1.20	1.19
Standard Deviation	0.032	0.030	0.029	0.046	0.049	0.036
Skewness	0.360	0.179	0.634	0.827	0.764	0.672
Minimum	1.12	1.12	1.13	1.13	1.13	1.14
Maximum	1.24	1.23	1.25	1.29	1.33	1.28
D1/D3 [b]						
Mean	1.33	1.33	1.32	1.33	1.34	1.33
Standard Deviation	0.068	0.055	0.054	0.075	0.070	0.060
Skewness	-0.106	0.270	0.526	1.16	0.955	1.07
Minimum	1.18	1.21	1.20	1.24	1.24	1.24
Maximum	1.50	1.47	1.46	1.52	1.51	1.49

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B16. Deflection Basin Parameters Calculated from Road Rater Data for Lincoln Road-Sieben (May 18, 1997)

Characteristic	Control Section	Test Section
Assumed AC Thickness (in.)	9.0	9.6
Assumed Base Thickness (in.)	28.8	28.8
Surface & Mat Temperature (°F)	63 & 65	63 & 65
Number of Test Locations	26	26
ISM[a] (ksi)		
Mean	1030	930
Standard Deviation	165	162
Skewness	0.214	0.656
Maximum	718	716
Minimum	1430	1340
D1/D2[b]		
Mean	1.37	1.36
Standard Deviation	0.146	0.092
Skewness	1.18	0.810
Maximum	1.18	1.25
Minimum	1.69	1.60
D1/D3[b]		
Mean	1.63	1.61
Standard Deviation	0.183	0.120
Skewness	1.05	0.186
Maximum	1.39	1.42
Minimum	2.06	1.84

Note: Measured peak load was approximately 4000 lb.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B17. Deflection Basin Parameters Calculated from FWD Data for Lincoln Road-Sieben (April 26, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat	64 & 98			64 & 98		
Number of Test Locations	27			27		
Average Peak Load (kips)	5.4	8.0	9.8	5.4	8.0	9.8
ISM [a] (ksi)						
Mean	721	754	783	666	703	731
Standard Deviation	123	122	126	100	95.9	93.6
Skewness	1.064	1.157	1.304	1.423	1.573	1.531
Minimum	558	601	628	541	582	617
Maximum	1040	1068	1123	970	1005	1028
D1/D2 [b]						
Mean	1.348	1.324	1.314	1.336	1.314	1.305
Standard Deviation	0.045	0.051	0.056	0.041	0.040	0.035
Skewness	0.036	-0.081	0.271	0.523	0.338	0.108
Minimum	1.260	1.229	1.211	1.276	1.238	1.245
Maximum	1.436	1.413	1.436	1.422	1.404	1.362
D1/D3 [b]						
Mean	1.621	1.602	1.611	1.638	1.623	1.638
Standard Deviation	0.098	0.099	0.103	0.082	0.078	0.084
Skewness	-0.043	0.117	0.073	0.486	0.432	0.322
Minimum	1.454	1.436	1.420	1.473	1.458	1.444
Maximum	1.793	1.778	1.792	1.871	1.835	1.853

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B18. Deflection Basin Parameters Calculated from FWD Data for Lincoln Road-Sieben (May 6, 1999)

Characteristic	Control Section			Test Section		
Surface & Mat	78 & 55			70 & 55		
Number of Test Locations	29			29		
Average Peak Load (kips)	5.7	7.5	9.4	5.8	7.7	9.6
ISM [a] (ksi)						
Mean	1050	1060	1080	973	988	1020
Standard Deviation	166	157	153	167	161	151
Skewness	0.362	0.458	0.523	0.136	-0.014	0.060
Minimum	780	825	853	657	639	731
Maximum	1380	1380	1410	1320	1320	1340
D1/D2 [b]						
Mean	1.22	1.22	1.22	1.24	1.25	1.24
Standard Deviation	0.026	0.027	0.026	0.107	0.119	0.084
Skewness	-0.480	-0.505	-0.498	4.13	4.35	3.56
Minimum	1.16	1.16	1.16	1.16	1.17	1.18
Maximum	1.26	1.27	1.26	1.75	1.82	1.62
D1/D3 [b]						
Mean	1.43	1.42	1.40	1.48	1.46	1.44
Standard Deviation	0.052	0.047	0.045	0.139	0.147	0.109
Skewness	-0.999	-0.994	-1.03	2.77	3.33	2.35
Minimum	1.30	1.30	1.29	1.32	1.32	1.32
Maximum	1.50	1.47	1.46	2.05	2.11	1.85

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B19. Deflection Basin Parameters Calculated from FWD Data for Lincoln Road-Sieben (May 11, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat	51 & 46			48 & 45		
Number of Test Locations	28			28		
Average Peak Load (kips)	6.0	7.9	9.8	6.0	8.0	9.8
ISM [a] (ksi)						
Mean	1150	1151	1174	1080	1090	1110
Standard Deviation	177	167	164	140	131	130
Skewness	0.170	0.251	0.308	0.729	0.789	0.782
Minimum	850	864	889	844	876	888
Maximum	1540	1520	1540	1450	1450	1460
D1/D2 [b]						
Mean	1.18	1.19	1.19	1.18	1.19	1.19
Standard Deviation	0.017	0.020	0.020	0.033	0.033	0.033
Skewness	-0.024	0.308	0.235	0.623	0.519	0.559
Minimum	1.14	1.14	1.15	1.11	1.12	1.13
Maximum	1.22	1.23	1.24	1.27	1.27	1.27
D1/D3 [b]						
Mean	1.36	1.35	1.34	1.38	1.37	1.36
Standard Deviation	0.037	0.036	0.034	0.063	0.061	0.058
Skewness	-0.276	0.151	0.103	0.052	0.110	0.277
Minimum	1.28	1.28	1.27	1.23	1.23	1.24
Maximum	1.43	1.43	1.42	1.52	1.51	1.50

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B20. Deflection Basin Parameters Calculated from FWD Data for Lincoln Road-Sieben (April 26, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat Temperature	67 & 62			61 & 59		
Number of Test Locations	27			27		
Average Peak Load (kips)	6.7	8.7	10.6	6.8	8.9	10.7
ISM [a] (ksi)						
Mean	1040	1070	1120	968	1000	1050
Standard Deviation	147	140	137	138	129	127
Skewness	0.287	0.279	0.257	1.01	1.08	1.08
Minimum	775	813	865	796	856	897
Maximum	1330	1350	1390	1300	1330	1370
D1/D2 [b]						
Mean	1.20	1.19	1.18	1.23	1.21	1.20
Standard Deviation	0.028	0.029	0.024	0.032	0.031	0.031
Skewness	0.400	0.680	0.755	0.193	0.278	0.199
Minimum	1.16	1.14	1.14	1.15	1.14	1.14
Maximum	1.26	1.25	1.23	1.30	1.28	1.27
D1/D3 [b]						
Mean	1.328511	1.312475	1.291453	1.372224	1.354084	1.335683
Standard Deviation	0.052024	0.048478	0.042862	0.087151	0.081989	0.078124
Skewness	0.032054	0.302606	0.46965	1.913506	2.016402	1.843236
Minimum	1.242574	1.232727	1.223473	1.230088	1.230241	1.214697
Maximum	1.417373	1.405263	1.378498	1.695391	1.661316	1.620408

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B21. Deflection Basin Parameters Calculated from Road Rater Data for Custer County Line West (May 21, 1997)

Characteristic	Control Section	Test Section
Surface & Mat Temperature	72 & 70	72 & 70
Number of Test Locations	28	28
ISM [a] (ksi)		
Mean	1250	707
Standard Deviation	326	142
Skewness	0.927	4.16
Minimum	832	642
Maximum	2210	1370
D1/D2 [b]		
Mean	1.51	2.20
Standard Deviation	0.307	0.627
Skewness	0.711	1.45
Minimum	1.12	1.48
Maximum	2.13	3.99
D1/D3 [b]		
Mean	1.67	2.48
Standard Deviation	0.338	0.670
Skewness	0.613	1.35
Minimum	1.23	1.63
Maximum	2.34	4.43

Note: Measured peak load was approximately 4000 lb.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B22. Deflection Basin Parameters Calculated from FWD Data for Custer County Line West (April 28, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat	60 & 67			61 & 70		
Number of Test Locations	26			27		
Average Peak Load (kips)	5.4	7.9	9.8	5.3	7.7	9.5
ISM [a] (ksi)						
Mean	1606	1601	1599	1271	1256	1268
Standard Deviation	281	294	287	228	227	231
Skewness	0.483	0.605	0.600	0.882	0.981	0.970
Minimum	1118	1118	1126	948	921	925
Maximum	2294	2313	2302	1909	1909	1922
D1/D2 [b]						
Mean	1.113	1.116	1.116	1.119	1.119	1.115
Standard Deviation	0.018	0.019	0.020	0.037	0.024	0.020
Skewness	0.328	0.718	0.605	2.979	1.212	0.873
Minimum	1.076	1.079	1.081	1.085	1.084	1.083
Maximum	1.161	1.171	1.165	1.274	1.190	1.168
D1/D3 [b]						
Mean	1.209	1.206	1.202	1.225	1.219	1.208
Standard Deviation	0.029	0.029	0.029	0.054	0.042	0.038
Skewness	0.338	0.504	0.335	1.858	0.909	0.784
Minimum	1.157	1.148	1.154	1.169	1.161	1.151
Maximum	1.266	1.282	1.264	1.412	1.327	1.308

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B23. Deflection Basin Parameters Calculated from FWD Data for Custer County Line West (May 10, 1999)

Characteristic	Control Section			Test Section		
Surface & Mat	60 & 67			61 & 70		
Number of Test Locations	26			27		
Average Peak Load (kips)	6.1	8.1	10.0	6.1	8.1	9.9
ISM [a] (ksi)						
Mean	1680	1670	1670	1320	1320	1340
Standard Deviation	262	271	263	313	317	320
Skewness	0.514	0.585	0.570	0.162	0.333	0.588
Minimum	1200	1190	1200	497	544	628
Maximum	2250	2270	2250	2010	2020	2080
D1/D2 [b]						
Mean	1.11	1.10	1.08	1.17	1.16	1.13
Standard Deviation	0.017	0.019	0.018	0.239	0.198	0.146
Skewness	-0.115	-0.298	0.046	5.17	5.13	5.05
Minimum	1.08	1.06	1.05	1.07	1.07	1.06
Maximum	1.14	1.13	1.12	2.38	2.16	1.87
D1/D3 [b]						
Mean	1.20	1.20	1.19	1.30	1.29	1.26
Standard Deviation	0.029	0.032	0.028	0.283	0.239	0.176
Skewness	0.403	0.200	0.287	4.98	4.84	4.60
Minimum	1.15	1.14	1.14	1.16	1.16	1.15
Maximum	1.25	1.25	1.24	2.72	2.48	2.12

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B24. Deflection Basin Parameters Calculated from FWD Data for Custer County Line West (May 3, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat	90 & 76			92 & 76		
Number of Test Locations	28			26		
Average Peak Load (kips)	5.8	7.7	9.5	5.6	7.5	9.3
ISM [a] (ksi)						
Mean	1350	1330	1330	1130	1110	1100
Standard Deviation	244	251	247	124	122	121
Skewness	0.816	0.786	0.811	1.53	1.58	1.72
Minimum	1020	997	1000	953	937	944
Maximum	1870	1850	1870	1530	1500	1500
D1/D2 [b]						
Mean	1.24	1.24	1.25	1.22	1.23	1.23
Standard Deviation	0.050	0.052	0.053	0.025	0.023	0.032
Skewness	0.852	0.865	0.711	0.578	0.402	0.123
Minimum	1.16	1.17	1.17	1.18	1.19	1.16
Maximum	1.38	1.38	1.37	1.29	1.29	1.30
D1/D3 [b]						
Mean	1.43	1.42	1.41	1.38	1.38	1.38
Standard Deviation	0.087	0.085	0.083	0.038	0.041	0.045
Skewness	0.752	0.854	0.726	-0.046	-0.060	0.068
Minimum	1.31	1.31	1.31	1.32	1.31	1.31
Maximum	1.64	1.64	1.61	1.45	1.45	1.45

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B25. Deflection Basin Parameters Calculated from FWD Data for Custer County Line West (April 25, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat	85 & 70			80 & 68		
Number of Test Locations	24			24		
Average Peak Load (kips)	5.9	8.0	9.8	5.8	7.9	9.7
ISM [a] (ksi)						
Mean	969	966	983	822	837	851
Standard Deviation	135	137	141	101	99.9	114
Skewness	1.83	1.88	1.73	0.381	0.342	0.783
Minimum	824	820	828	677	684	700
Maximum	1410	1410	1420	1010	1020	1130
D1/D2 [b]						
Mean	1.28	1.30	1.29	1.25	1.26	1.26
Standard Deviation	0.048	0.051	0.052	0.026	0.027	0.026
Skewness	1.27	1.31	1.32	0.316	-0.144	0.460
Minimum	1.23	1.24	1.23	1.21	1.20	1.21
Maximum	1.41	1.44	1.44	1.31	1.31	1.32
D1/D3 [b]						
Mean	1.49	1.49	1.48	1.47	1.47	1.46
Standard Deviation	0.082	0.085	0.083	0.054	0.050	0.048
Skewness	1.21	1.31	1.42	0.079	-0.050	-0.195
Minimum	1.40	1.40	1.39	1.37	1.37	1.37
Maximum	1.70	1.74	1.72	1.58	1.57	1.55

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B26. Deflection Basin Parameters Calculated from FWD Data for Tarkio-East (April 27, 1998)

Characteristic	Control Section			Test Section		
Surface & Mat	55 & 54			55 & 54		
Number of Test Locations	27			27		
Average Peak Load (kips)	5.0	8.0	9.8	4.9	7.9	9.7
ISM [a] (ksi)						
Mean	990	1037	1072	717	767	850
Standard Deviation	211	225	227	214	207	194
Skewness	-0.973	-0.852	-0.880	0.093	-0.220	-0.239
Minimum	487	442	472	339	351	486
Maximum	1262	1438	1416	1119	1130	1183
D1/D2 [b]						
Mean	1.514	1.490	1.497	1.917	1.799	1.677
Standard Deviation	0.344	0.316	0.304	0.517	0.474	0.381
Skewness	1.371	1.372	1.188	0.672	1.095	1.246
Minimum	1.248	1.247	1.226	1.355	1.321	1.283
Maximum	2.304	2.320	2.240	2.977	2.897	2.740
D1/D3 [b]						
Mean	1.870	1.831	1.851	2.406	2.243	2.084
Standard Deviation	0.433	0.387	0.366	0.641	0.597	0.456
Skewness	1.441	1.245	0.993	0.704	1.149	1.320
Minimum	1.513	1.487	1.489	1.620	1.547	1.528
Maximum	2.973	2.654	2.580	3.840	3.777	3.396

Note: Loads were applied in the order shown and were preceded by a “seating drop” of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B27. Deflection Basin Parameters Calculated from FWD Data for Tarkio-East (May 5, 1999)

Characteristic	Control Section			Test Section		
Surface & Mat	82 & 50			74 & 50		
Number of Test Locations	28			27		
Average Peak Load (kips)	5.8	7.7	9.6	5.7	7.6	9.4
ISM [a] (ksi)						
Mean	1350	1380	1430	1110	1130	1170
Standard Deviation	146	150	156	114	112	113
Skewness	-0.394	-0.381	-0.501	0.382	0.388	0.372
Minimum	998	1020	1050	916	927	981
Maximum	1650	1690	1720	1340	1370	1400
D1/D2 [b]						
Mean	1.26	1.27	1.27	1.27	1.27	1.26
Standard Deviation	0.027	0.030	0.031	0.029	0.029	0.030
Skewness	0.278	0.339	0.585	-0.367	-0.054	-0.028
Minimum	1.20	1.21	1.21	1.20	1.22	1.20
Maximum	1.31	1.33	1.34	1.32	1.32	1.32
D1/D3 [b]						
Mean	1.51	1.50	1.48	1.52	1.49	1.47
Standard Deviation	0.057	0.060	0.057	0.062	0.063	0.065
Skewness	0.0042	-0.029	0.147	0.102	0.473	0.554
Minimum	1.39	1.37	1.36	1.38	1.36	1.34
Maximum	1.63	1.61	1.60	1.67	1.67	1.66

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B28. Deflection Basin Parameters Calculated from FWD Data for Tarkio-East (May 9, 2000)

Characteristic	Control Section			Test Section		
Surface & Mat	51 & 47			54 & 48		
Number of Test Locations	16			11		
Average Peak Load (kips)	6.5	8.1	9.9	6.5	8.0	9.8
ISM [a] (ksi)						
Mean	1430	1480	1530	1360	1390	1430
Standard Deviation	162	165	171	261	263	272
Skewness	0.993	0.929	1.01	-1.17	-1.25	-1.17
Minimum	1220	1260	1300	788	814	849
Maximum	1860	1900	1970	1700	1730	1790
D1/D2 [b]						
Mean	1.22	1.20	1.18	1.22	1.19	1.19
Standard Deviation	0.019	0.018	0.019	0.032	0.027	0.022
Skewness	-0.048	0.578	0.783	0.031	0.159	0.538
Minimum	1.19	1.17	1.16	1.17	1.16	1.16
Maximum	1.26	1.23	1.22	1.27	1.24	1.23
D1/D3 [b]						
Mean	1.42	1.40	1.39	1.41	1.39	1.39
Standard Deviation	0.036	0.040	0.038	0.065	0.065	0.063
Skewness	0.521	0.462	0.225	0.207	0.173	0.209
Minimum	1.37	1.33	1.32	1.31	1.30	1.30
Maximum	1.49	1.49	1.46	1.53	1.49	1.50

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Table B29. Deflection Basin Parameters Calculated from FWD Data for Tarkio-East (April 30, 2001)

Characteristic	Control Section			Test Section		
Surface & Mat	56 & 56			57 & 56		
Number of Test Locations	28			28		
Average Peak Load (kips)	5.9	8.0	9.9	5.8	7.9	9.7
ISM [a] (ksi)						
Mean	1420	1470	1550	1170	1210	1270
Standard Deviation	142	138	141	140	142	145
Skewness	0.208	-0.018	-0.147	-1.09	-1.17	-1.31
Minimum	1150	1220	1280	762	789	829
Maximum	1790	1790	1840	1380	1420	1490
D1/D2 [b]						
Mean	1.23	1.24	1.23	1.23	1.23	1.23
Standard Deviation	0.040	0.040	0.039	0.030	0.028	0.032
Skewness	1.87	2.23	1.87	-0.254	0.108	0.255
Minimum	1.17	1.18	1.16	1.18	1.19	1.17
Maximum	1.39	1.39	1.37	1.28	1.28	1.29
D1/D3 [b]						
Mean	1.48	1.46	1.44	1.48	1.45	1.44
Standard Deviation	0.069	0.064	0.062	0.058	0.052	0.054
Skewness	1.07	1.09	0.952	-0.369	-0.198	-0.095
Minimum	1.38	1.37	1.35	1.36	1.35	1.34
Maximum	1.69	1.66	1.63	1.58	1.54	1.53

Note: Loads were applied in the order shown and were preceded by a "seating drop" of approximately 9 kips.
[a] ISM = impulse stiffness modulus
[b] D1 = deflection at offset = 0 in., D2 = deflection at offset = 8 in., D3 = deflection at offset

Appendix C

Statistical Test Summaries

Impulse Stiffness Modulus (ISM)

Table C1. Three-Factor Analysis of Variance for ISM at Lincoln Road

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	18.852	3	6.284	316.670	0.000
Section	0.714	1	0.714	36.002	0.000
Drop Height	0.371	2	0.185	9.341	0.000
Year*Section	0.006705	3	0.002235	0.113	0.953
Year*Drop Height	0.07719	6	0.01286	0.648	0.692
Section*Drop Height	0.003688	2	0.001844	0.093	0.911
Year*Section* Drop Height	0.001313	6	0.0002189	0.011	1.000
Error	12.740	642	0.01984		
Corrected Total	32.759	665			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C2. Two-Factor Analysis of Variance for ISM at Bearmouth Drummond, Westbound Lane

Source of Variance	Sum of Squares	Df	Mean Square	F Statistic	P-value[a]
Year	0.529	3	0.176	19.955	0.000
Section	0.02058	1	0.02058	2.328	0.131
Year*Section	0.007474	3	0.002491	0.282	0.838
Error	0.681	77	0.008843		
Corrected Total	1.229	84			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C3. Two-Factor Analysis of Variance for ISM at Bearmouth Drummond, Eastbound Lane

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	1.276	3	0.425	34.824	0.000
Section	0.02068	1	0.02068	1.694	0.197
Year*Section	0.221	3	0.07382	6.046	0.001
Error	1.001	82	0.01221		
Corrected Total	2.428	89			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C4. Student's t-Tests and One-Way Analyses of Variance for ISM at Bearmouth Drummond, Eastbound Lane

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.585	Control	1998 through 2001	0.000
	Test				
1999	Control	0.429			
	Test				
2000	Control	0.230	Test	1998 through 2001	0.000
	Test				
2001	Control	0.000			
	Test				

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments. Student's t-test was performed when grouping variable included two groups. One-way analysis of variance was performed when grouping variable included more than two groups.

Table C5. Two-Factor Analysis of Variance for ISM at Rocky Canyon

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	1.261	3	0.420	7.478	0.000
Section	0.00001186	1	0.0001186	0.000	0.988
Year*Section	0.622	3	0.207	3.689	0.013
Error	11.915	212	0.05620		
Corrected Total	13.798	219			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C6. Student's t-Tests and One-Way Analyses of Variance for ISM at Rocky Canyon

<table>
<tr><td>Year</td><td>Section</td><td>P-value[a]</td><td>Section</td><td>Years</td><td>P-value[a]</td></tr>
<tr><td rowspan="2">1998</td><td>Control</td><td rowspan="2">0.742</td><td rowspan="4">Control</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.007</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">1999</td><td>Control</td><td rowspan="2">0.208</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2000</td><td>Control</td><td rowspan="2">0.236</td><td rowspan="4">Test</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.000</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2001</td><td>Control</td><td rowspan="2">0.013</td></tr>
<tr><td>Test</td></tr>
<tr><td colspan="6">Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments. Student's t-test was performed when grouping variable included two groups. One-way analysis of variance was performed when grouping variable included more than two groups.</td></tr>
</table>

Table C7. Two-Factor Analysis of Variance for ISM at Lincoln Road

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	6.034	3	2.011	102.742	0.000
Section	0.226	1	0.226	11.522	0.001
Year*Section	0.002621	3	0.0008737	0.045	0.987
Error	4.189	214	0.01958		
Corrected Total	10.452	221			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C8. Two-Factor Analysis of Variance for ISM at Custer County Line

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	7.571	3	2.524	86.713	0.000
Section	2.144	1	2.144	73.680	0.000
Year*Section	0.129	3	0.04303	1.479	0.221
Error	5.995	206	0.02910		
Corrected Total	15.741	213			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C9. Two-Factor Analysis of Variance for ISM at Tarkio East

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	6.985	3	2.328	73.210	0.000
Section	1.640	1	1.640	51.548	0.000
Year*Section	0.268	3	0.08940	2.811	0.041
Error	5.884	185	0.03181		
Corrected Total	15.297	192			

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C10. Student's t-Tests and One-Way Analyses of Variance for ISM at Tarkio East

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.000	Control	1998 through 2001	0.000
	Test				
1999	Control	0.000			
	Test				
2000	Control	0.262	Test	1998 through 2001	0.000
	Test				
2001	Control	0.000			
	Test				

Note: Raw data were natural logarithm of impulse stiffness moduli (kips/in.).
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments. Student's t-test was performed when grouping variable included two groups. One-way analysis of variance was performed when grouping variable included more than two groups.

Basin Curvature

Table C11. Mann-Whitney Non-Parametric Test for Basin Curvature at Bearmouth Drummond, Westbound Lane

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.439	Control	1998 through 2001	0.000
	Test				
1999	Control	0.481			
	Test				
2000	Control	0.880	Test	1998 through 2001	0.000
	Test				
2001	Control	0.019			
	Test				

Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.

Table C12. Mann-Whitney Non-Parametric Test for Basin Curvature at Bearmouth Drummond, Eastbound Lane

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.091	Control	1998 through 2001	0.022
	Test				
1999	Control	0.870			
	Test				
2000	Control	0.199	Test	1998 through 2001	0.001
	Test				
2001	Control	0.730			
	Test				

Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.

Table C13. Mann-Whitney Non-Parametric Test for Basin Curvature at Rocky Canyon

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.064	Control	1998 through 2001	0.790
	Test				
1999	Control	0.150			
	Test				
2000	Control	0.025	Test	1998 through 2001	0.011
	Test				
2001	Control	0.635			
	Test				

Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.

Table C14. Mann-Whitney Non-Parametric Test for Basin Curvature at Lincoln Road

Year	Section	P-value[a]	Section	Years	P-value[a]
1998	Control	0.551	Control	1998 through 2001	0.000
	Test				
1999	Control	0.499			
	Test				
2000	Control	0.201	Test	1998 through 2001	0.000
	Test				
2001	Control	0.031			
	Test				

Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.

Table C15. Mann-Whitney Non-Parametric Test for Basin Curvature at Custer County Line

<table>
<tr><th>Year</th><th>Section</th><th>P-value[a]</th><th>Section</th><th>Years</th><th>P-value[a]</th></tr>
<tr><td rowspan="2">1998</td><td>Control</td><td rowspan="2">0.350</td><td rowspan="4">Control</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.000</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">1999</td><td>Control</td><td rowspan="2">0.001</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2000</td><td>Control</td><td rowspan="2">0.426</td><td rowspan="4">Test</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.000</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2001</td><td>Control</td><td rowspan="2">0.587</td></tr>
<tr><td>Test</td></tr>
<tr><td colspan="6">Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.</td></tr>
</table>

Table C16. Mann-Whitney Non-Parametric Test for Basin Curvature at Tarkio East

<table>
<tr><th>Year</th><th>Section</th><th>P-value[a]</th><th>Section</th><th>Years</th><th>P-value[a]</th></tr>
<tr><td rowspan="2">1998</td><td>Control</td><td rowspan="2">0.001</td><td rowspan="4">Control</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.000</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">1999</td><td>Control</td><td rowspan="2">0.649</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2000</td><td>Control</td><td rowspan="2">0.622</td><td rowspan="4">Test</td><td rowspan="4">1998 through 2001</td><td rowspan="4">0.000</td></tr>
<tr><td>Test</td></tr>
<tr><td rowspan="2">2001</td><td>Control</td><td rowspan="2">0.831</td></tr>
<tr><td>Test</td></tr>
<tr><td colspan="6">Note: Raw data were the ratio of deflection at offset = 0 in. to offset = 12 in.
[a] P-value is the probably of being incorrect if one states that a significant difference exists between treatments.</td></tr>
</table>

Rutting (Rainhart Profilograph)

Table C17. Two-Factor Analysis of Variance for Rutting at Bearmouth Drummond, Westbound Lane

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	0.466	1	0.466	10.584	0.005
Section	0.07013	1	0.07013	1.592	0.225
Year*Section	0.0009502	1	0.0009502	0.022	0.885
Error	0.705	16	0.04405		
Corrected Total	1.242	19			

Note: Raw data were natural logarithm of maximum rut (mm).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C18. Two-Factor Analysis of Variance for Rutting at Bearmouth Drummond, Eastbound Lane

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	0.229	1	0.229	1.541	0.232
Section	0.438	1	0.438	2.945	0.105
Year*Section	0.0006441	1	0.0006441	0.004	0.948
Error	2.378	16	0.149		
Corrected Total	3.045	19			

Note: Raw data were natural logarithm of maximum rut (mm).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C19. Two-Factor Analysis of Variance for Rutting at Rocky Canyon

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	1.000	1	1.000	20.781	0.000
Section	0.01448	1	0.01448	0.301	0.587
Year*Section	0.01624	1	0.01624	0.338	0.565
Error	1.732	36	0.04810		
Corrected Total	2.762	39			

Note: Raw data were natural logarithm of maximum rut (mm).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C20. Two-Factor Analysis of Variance for Rutting at Lincoln Road

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	0.09520	1	0.09520	4.261	0.046
Section	0.01032	1	0.01032	0.462	0.501
Year*Section	0.005572	1	0.005572	0.249	0.621
Error	0.804	36	0.02234		
Corrected Total	0.915	39			

Note: Raw data were natural logarithm of maximum rut (mm).
[a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.

Table C21. Two-Factor Analysis of Variance for Rutting at Custer County Line

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	0.496	1	0.496	36.952	0.000
Section	1.597	1	1.597	119.085	0.000
Year*Section	0.01191	1	0.01191	0.888	0.352
Error	0.483	36	0.01341		
Corrected Total	2.587	39			
Note: Raw data were natural logarithm of maximum rut (mm). [a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.					

Table C22. Two-Factor Analysis of Variance for Rutting at Tarkio East

Source of Variance	Sum of Squares	df	Mean Square	F Statistic	P-value[a]
Year	0.268	1	0.268	3.891	0.056
Section	0.07405	1	0.07405	1.076	0.307
Year*Section	0.03526	1	0.03526	0.512	0.479
Error	2.478	36	0.06883		
Corrected Total	2.855	39			
Note: Raw data were natural logarithm of maximum rut (mm). [a] P-value is the probably of being incorrect if the corresponding source of variance is said to be significant.					

Appendix D

Roughness and Rut Depth Data

Table D1. Roughness and Rut Depth for Bearmouth-Drummond (Westbound Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1996			
International Roughness Index (in./mi)	Mean	56	110
	Range	37 to 70	47 to 161
Rut Depth (in.) by South Dakota Profilometer	Mean	0.00	0.00
	Standard Deviation	0.066	0.096
1997			
International Roughness Index (in./mi)	Mean	51	88
	Range	40 to 58	55 to 128
Rut Depth (in.) by South Dakota Profilometer	Mean	0.16	0.20
	Standard Deviation	0.028	0.065
1998			
International Roughness Index (in./mi)	Mean	51	74
	Range	42 to 58	52 to 95
Rut Depth (in.) by South Dakota Profilometer	Mean	0.00	0.10
	Standard Deviation	0.000	0.050
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.19	0.17
	Standard Deviation	0.043	0.038
2000			
International Roughness Index (in./mi)	Mean	42	54
	Range	No data	No data
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.264	0.232
	Standard Deviation	0.049	0.051

Notes:
1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

Table D2. Roughness and Rut Depth for Bearmouth-Drummond (Eastbound Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1996			
International Roughness Index (in./mi)	Mean	No data	68
	Range		50 to 80
Rut Depth (in.) by South Dakota Profilometer	Mean	No data	0.00
	Standard Deviation		0.067
1997			
International Roughness Index (in./mi)	Mean	47	67
	Range	43 to 50	48 to 93
Rut Depth (in.) by South Dakota Profilometer	Mean	0.10	0.17
	Standard Deviation	0.028	0.212
1998			
International Roughness Index (in./mi)	Mean	49	62
	Range	47 to 50	49 to 73
Rut Depth (in.) by South Dakota Profilometer	Mean	0.00	0.03
	Standard Deviation	0.000	0.010
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.23	0.33
	Standard Deviation	0.057	0.173
2000			
International Roughness Index (in./mi)	Mean	77	61
	Range	No data	No data
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.276	0.398
	Standard Deviation	0.050	0.173

Notes:

1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

Table D3. Roughness and Rut Depth for Rocky Canyon (Eastbound Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1996			
International Roughness Index (in./mi)	Mean	73	57
	Range	56 to 88	45 to 74
Rut Depth (in.) by South Dakota Profilometer	Mean	0.00	0.00
	Standard Deviation	0.098	0.058
1997			
International Roughness Index (in./mi)	Mean	81	79
	Range	57 to 113	45 to 117
Rut Depth (in.) by South Dakota Profilometer	Mean	0.16	0.16
	Standard Deviation	0.075	0.049
1998			
International Roughness Index (in./mi)	Mean	79	79
	Range	60 to 108	41 to 121
Rut Depth (in.) by South Dakota Profilometer	Mean	0.19	0.20
	Standard Deviation	0.075	0.033
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.17	0.18
	Standard Deviation	0.028	0.038
2000			
International Roughness Index (in./mi)	Mean	97*	53*
	Range	70 to 143*	49 to 56*
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.260	0.232
	Standard Deviation	0.083	0.032

Notes:
1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

* passing lane

Table D4. Roughness and Rut Depth for Lincoln Road-Sieben (Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1997			
International Roughness Index (in./mi)	Mean	54	51
	Range	43 to 67	44 to 60
Rut Depth (in.) by South Dakota Profilometer	Mean	0.05	0.05
	Standard Deviation	0.022	0.027
1998			
International Roughness Index (in./mi)	Mean	47	49
	Range	41 to 52	41 to 59
Rut Depth (in.) by South Dakota Profilometer	Mean	0.23	0.21
	Standard Deviation	0.026	0.032
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.19	0.18
	Standard Deviation	0.033	0.031
2000			
International Roughness Index (in./mi)	Mean	51	63
	Range	50 to 51	62 to 63
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.205	0.201
	Standard Deviation	0.026	0.024

Notes:
1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

Table D5. Roughness and Rut Depth for Custer County Line West (Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1997			
International Roughness Index (in./mi)	Mean	34	41
	Range	32 to 35	38 to 45
Rut Depth (in.) by South Dakota Profilometer	Mean	No data	No data
	Standard Deviation		
1998			
International Roughness Index (in./mi)	Mean	36	43
	Range	34 to 39	37 to 46
Rut Depth (in.) by South Dakota Profilometer	Mean	0.10	0.10
	Standard Deviation	0.049	0.051
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.26	0.17
	Standard Deviation	0.037	0.014
2000			
International Roughness Index (in./mi)	Mean	45	47
	Range	42 to 48	40 to 53
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.315	0.217
	Standard Deviation	0.040	0.021

Notes:
1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

Table D6. Roughness and Rut Depth for Tarkio-East (Driving Lane)

Characteristic	Statistic	Control Section (500 ft)	Test Section (500 ft)
1997			
International Roughness Index (in./mi)	Mean	64	84
	Range	41 to 90	72 to 94
Rut Depth (in.) by South Dakota Profilometer	Mean	0.08	0.15
	Standard Deviation	0.039	0.078
1998			
International Roughness Index (in./mi)	Mean	68	85
	Range	42 to 98	76 to 100
Rut Depth (in.) by South Dakota Profilometer	Mean	0.17	0.25
	Standard Deviation	0.039	0.083
1999			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.20	0.24
	Standard Deviation	0.050	0.059
2000			
International Roughness Index (in./mi)	Mean	No data	No data
	Range		
Rut Depth (in.) by Rainhart Transverse Profilometer	Mean	0.260	0.264
	Standard Deviation	0.095	0.064

Notes:

1. International Roughness Index (IRI) was measured by a South Dakota profilometer. The raw data included IRI for each wheelpath and for each 0.1-mile length of pavement. The mean and range values reported here for each experimental section were calculated using all the available raw data.
2. Rut depths, as collected by the South Dakota profilometer, included 20 to 30 measurements per linear foot of pavement. The data received from MDT included an average and a standard deviation for rut depth for each 0.1-mile length of pavement. The mean and average standard deviation values reported here for each experimental section were calculated using all the available raw data.
3. Rut depths, as collected by the Rainhart transverse profilometer, included measurements at stations spaced every 100 to 150 ft.

Appendix E

Visual Distress Survey Data

Table E1. Visual Distress Surveys for Bearmouth-Drummond (Westbound Lanes)

Characteristic	Control Section (500 ft)	Test Section (500 ft)
May 20, 1997 (Sunny, Clear, 60ºF)		
Bleeding	None	None
Raveling	None	None
Transverse Cracking	6 FW, 5 PW, (LS)	4 FW, 2 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 28, 1998 (Sunny, Clear, 60ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	6 FW, 5 PW, (LS)	4 FW, 3 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	30%, (LS)	50%, (LS)
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1999 (Partly Cloudy, 57ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	6 FW, 6 PW, (LS)	4 FW, 4 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	55% (LS)	50% (LS)
Longitudinal Cracking in WP	None	1% (LS)
Longitudinal Cracking in Shoulder	None	5% (LS)
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
Continued		
CL = centerline; OE = outside edge IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths TL = travelling lane; PL = passing lane LS = low severity; MS = moderate severity; HS = high severity NS = no sealant; S = sealed; RS = routed and sealed FW = full-width crack (across both TL and PL); PW = partial width crack		

Table E1 (continued). Visual Distress Surveys for Bearmouth-Drummond (Westbound Lanes)

Characteristic	Control Section (500 ft)	Test Section (500 ft)
August 17, 2000 (Sunny, 82°F)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	6 FW, 17 PW, (LS)	4 FW, 5 PW, (LS)
Sealant & Condition	RS (good condition)	RS (good condition)
Longitudinal Cracking at CL	67% (LS)	67% (LS)
Sealant & Condition	RS (good condition)	RS (good condition)
Longitudinal Cracking in WP	None	2.4% (LS)
Sealant & Condition	N/A	RS (1.2%), NS (1.2%)
Longitudinal Cracking in Shoulder	6% (LS)	5% (LS)
Sealant & Condition	RS (good condition)	RS (good condition)
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Table E2. Visual Distress Surveys for Bearmouth-Drummond (Eastbound Lanes)

Characteristic	Control Section (500 ft)	Test Section (500 ft)
May 20, 1997 (Sunny, Clear, 60ºF)		
Bleeding	None	None
Raveling	None	None
Transverse Cracking	3 FW, 1 PW, (LS)	2 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 28, 1998 (Sunny, Clear, 60ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	4 FW, 2 PW, (LS)	2 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1999 (Partly Cloudy, TL-Dry, PL-Wet, 57ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	4 FW, 4 PW, (LS)	2 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	4% (LS)	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
continued		

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Table E2 (continued). Visual Distress Surveys for Bearmouth-Drummond (Eastbound Lanes)

Characteristic	Control Section (500 ft)	Test Section (500 ft)
August 17, 2000 (Sunny, 80ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	6 FW, 5 PW, (LS)	2 FW, 2 PW, (LS)
Sealant & Condition	RS	NS
Longitudinal Cracking at CL	7% (LS)	10% (LS)
Sealant & Condition	RS (good condition)	RS (good condition)
Longitudinal Cracking in WP	None	None
Longitudinal Cracking in WP	None	None
Longitudinal Cracking in Shoulder	None	4% (LS)
Sealant & Condition	N/A	RS (good condition)
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Table E3. Visual Distress Surveys for Rocky Canyon

Characteristic	Control Section (1370 ft)	Test Section (1370 ft)
May 22, 1997 (Sunny, Clear, 70ºF)		
Bleeding	None	None
Raveling	None	None
Transverse Cracking	5 FW, 0 PW, (LS)	0 FW, 1 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1998 (Sunny, Clear, 60ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	7 FW, 4 PW, (LS)	0 FW, 1 PW, (LS)
Sealant & Condition	80% RS (good cond.)	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1999 (Sunny, Clear, 60ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	7 FW, 4 PW, (LS)	0 FW, 1 PW, (LS)
Sealant & Condition	80% RS (good cond.)	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
continued		
CL = centerline; OE = outside edge IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths TL = traveling lane; PL = passing lane LS = low severity; MS = moderate severity; HS = high severity NS = no sealant; S = sealed; RS = routed and sealed FW = full-width crack (across both TL and PL); PW = partial width crack		

Table E3 (continued). Visual Distress Surveys for Rocky Canyon

Characteristic	Control Section (1370 ft)	Test Section (1370 ft)
August 16, 2000 (Smokey, 46°F)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (MS)
Raveling	None	None
Transverse Cracking	9 FW, 5 PW, (LS)	1 FW, 1 PW, (LS)
Sealant & Condition	75% RS (good cond.)	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	21% (LS)	None
Sealant & Condition	NS	N/A
Fatigue Cracking	2%*	None
Sealant & Condition	NS	N/A
Potholes	None	None
Patches	None	None

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack
* Located at the start of the Control Section adjacent to bridge, where vehicles leave the bridge.

Table E4. Visual Distress Surveys for Lincoln Road-Sieben

Characteristic	Control Section (1320 ft)	Test Section (1320 ft)
May 20, 1997 (Sunny, Clear, 70ºF)		
Bleeding	None	None
Raveling	None	None
Transverse Cracking	14 FW, 0 PW, (LS)	20 FW, 3 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1998 (Sunny, Clear, 70ºF)		
Bleeding	25% OE of TL, (LS) 25% CL, (LS)	100% OE of TL, (LS)
Raveling	<10%	10 to 20%
Transverse Cracking	14 FW, 5 PW, (LS)	20 FW, 3 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
May 26, 1999 (Partly Cloudy, 68ºF)		
Bleeding	25% OE of TL, (LS) 25% CL, (LS)	100% OE of TL, (LS)
Raveling	10 to 20%	10 to 20%
Transverse Cracking	17 FW, 13 PW	24 FW, 13 PW
Sealant & Condition	RS (good condition)	RS (good condition)
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
continued		

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Table E4 (continued). Visual Distress Surveys for Lincoln Road-Sieben

Characteristic	Control Section (1320 ft)	Test Section (1320 ft)
August 16, 2000 (Sunny, 85°F)		
Bleeding	25% OE of TL, (LS) 25% CL, (LS)	100% OE of TL, (LS)
Raveling	20%	20%
Transverse Cracking	17 FW, 13 PW	24 FW, 13 PW
Sealant & Condition	RS (good condition)	99% RS (good cond.)
Longitudinal Cracking at CL	None	3.4% (LS)
Sealant & Condition	N/A	NS
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Table E5. Visual Distress Surveys for Custer County Line West

Characteristic	Control Section (1320 ft)	Test Section (1320 ft)
May 23, 1997		
(Sunny, Clear, 70ºF)		
Bleeding	None	None
Raveling	None	None
Transverse Cracking	None	2 FW, 2 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1998		
(Mostly Sunny, 60ºF)		
Bleeding	100% WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	2 FW, 0 PW, (LS)	4 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
May 5, 1999		
(Cloudy, Windy, 48ºF)		
Bleeding	100% of WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	2 FW, 0 PW, (LS)	4 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
continued		

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = traveling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack

Note: An accident caused considerable pavement surface damage during the 1997-1998 winter. Strips of asphalt surface material (about 12 inches long and ½ inch deep) were gouged from the pavement surface every four feet in both the test and control section's TL OWP.

Table E5 (continued). Visual Distress Surveys for Custer County Line West

Characteristic	Control Section (1320 ft)	Test Section (1320 ft)
August 15, 2000 (Sunny, 78°F)		
Bleeding	100% of WP of TL, (LS)	100% WP of TL, (LS)
Raveling	None	None
Transverse Cracking	2 FW, 0 PW, (LS)	4 FW, 0 PW, (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None

CL = centerline; OE = outside edge
IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths
TL = travelling lane; PL = passing lane
LS = low severity; MS = moderate severity; HS = high severity
NS = no sealant; S = sealed; RS = routed and sealed
FW = full-width crack (across both TL and PL); PW = partial width crack
Note: An accident caused considerable pavement surface damage during the 1997-1998 winter. Strips of asphalt surface material (about 12 inches long and ½ inch deep) were gouged from the pavement surface every four feet in both the test and control section's TL OWP.

Table E6. Visual Distress Surveys for Tarkio-East

Characteristic	Control Section (1320 ft)	Test Section (1320 ft)
May 5, 1998 (Mostly Sunny, 70ºF)		
Bleeding	100% WP of TL, (MS) 100% WP of PL, (LS)	100% WP of TL, (MS) 100% WP of PL, (LS)
Raveling	None	None
Transverse Cracking	None	None
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	None	None
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
April 27, 1999 (Overcast, 49°F)		
Bleeding	100% WP of TL, (MS) 100% WP of PL, (LS)	100% WP of TL, (MS) 100% WP of PL, (LS)
Raveling	87% OE of TL, (MS)	100% OE of TL, (MS)
Transverse Cracking	1* FW (LS)	None
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	None	None
Longitudinal Cracking in WP	1% (LS)	>1% (LS)
Sealant & Condition	NS	NS
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
August 17, 2000 (Smokey, 55°F)		
Bleeding	100% WP of TL, (MS) 100% WP of PL, (LS)	100% WP of TL, (MS) 100% WP of PL, (LS)
Raveling	100% OE of TL, (MS)	100% OE of TL, (MS)
Transverse Cracking	1* FW (LS)	None
Sealant & Condition	NS	NS
Longitudinal Cracking at CL	>1% (LS)	>1% (LS)
Sealant & Condition	NS	NS
Longitudinal Cracking in WP	45% (LS)	3% (LS)
Sealant & Condition	NS	NS
Fatigue Cracking	None	None
Potholes	None	None
Patches	None	None
CL = centerline; OE = outside edge IWP = inside wheel path; OWP = outside wheel path; WP = both wheel paths TL = traveling lane; PL = passing lane LS = low severity; MS = moderate severity; HS = high severity NS = no sealant; S = sealed; RS = routed and sealed FW = full-width crack (across both TL and PL); PW = partial width crack * In a transverse construction joint.		