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Cost-Effective Rut Repair Methods

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Wheelpath rutting prevents rapid drainage of water from the pavement surface and causes hydroplaning. Deeply rutted pavements are associated with driver fatigue, vehicle steering problems, and vehicle wear. This report reviews state-of-the-practice methods for repairing rutted asphalt concrete pavements. Information sources included technical literature, paving industry representatives and highway agency personnel. The report describes methods for repairing ruts, but does not address all rutting mechanisms. It covers tire-abrasion rutting and plastic deformation of asphalt concrete. Rutting from deformation of unbound layers is not addressed. Reasons for targeting specific rut damage types are explained. The report presents:

1. Concepts regarding selection of rut-resistant materials
2. Descriptions of materials and construction methods for five repair methods:

Micro-Surfacing, Ultra-Thin Bonded Wearing Course—NovaChip®, Ultra-Thin Bonded Wearing Course - Ultra-Thin Whitetopping, Stone Matrix Asphalt, Conventional Overlays Using High Quality Mixes and Materials.

3. Basic cost estimates and discussions of important economic principals involved in selecting one of the repair types
4. Extensive references separated as to both past and current rut repair research

Recommendations support constructing experimental sections for evaluating the performance of each rut repair method in Alaska, followed by large-scale trials of viable methods.

14. KEY WORDS

Alaska, asphalt, aggregate, pavement, rutting, studed tire, plastic deformation, damage, rehabilitation, repair, maintenance, milling, micro-surfacing, SMA, ultra-thin whitetopping, UTW, NovaChip, overlay, LCCA, life-cycle

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Disclaimer

Abstract

Wheelpath rutting prevents rapid drainage of water from the pavement surface and causes hydroplaning. Deeply rutted pavements are associated with driver fatigue, vehicle steering problems, and vehicle wear. This report reviews state-of-the-practice methods for repairing rutted asphalt concrete pavements. Information sources included technical literature, paving industry representatives and highway agency personnel.

The report describes methods for repairing ruts, but does not address all rutting mechanisms. It covers tire-abrasion rutting and plastic deformation of asphalt concrete. Rutting from deformation of unbound layers is not addressed. Reasons for targeting specific rut damage types are explained.

The report presents:

1. Concepts regarding selection of rut-resistant materials
2. Descriptions of materials and construction methods for five repair methods
    - Micro-Surfacing
    - Ultra-Thin Bonded Wearing Course—NovaChip®
    - Ultra-Thin Bonded Wearing Course—Ultra-Thin Whitetopping
    - Stone Matrix Asphalt
    - Conventional Overlays Using High Quality Mixes and Materials
3. Basic cost estimates and discussions of important economic principals involved in selecting one of the repair types
4. Extensive references separated as to both past and current rut repair research

Recommendations support constructing experimental sections for evaluating the performance of each rut repair method in Alaska, followed by large-scale trials of viable methods.

Key Words

Alaska, asphalt, pavement, rut, rutting, stud, tire, plastic, deformation, damage, rehabilitation, repair, maintenance, cost, economic, mill, micro-surfacing, stone, matrix, mastic, asphalt, SMA, ultra-thin, whitetopping, UTW, NovaChip, overlay, LCCA, life-cycle, aggregate

SUMMARY OF FINDINGS

This report reviews several state-of-the-practice methods for repairing rutted asphalt concrete pavements. Information was assembled from numerous sources including a comprehensive literature review as well as personal contacts with pavement industry and highway agency representatives. The report does not address all rutting problems. Repairs covered in this report are for rutting caused by tire abrasion and/or plastic deformation of the asphalt concrete layer. Specifically <u>not</u> addressed here is rutting caused by deformation of the base course or other unbound aggregate layers. It is imperative that anyone concerned with repairing a rutted pavement investigate the mechanism behind the rutting. Reasons for targeting specific types of rut damage are explained in Chapter 1 of this report.

The report provides general concepts pertaining to selecting rut-resistant materials, and then specifically describes materials and construction methods covering five (5) state-of-the-practice repair methods. The report provides some basic cost estimates and discusses important economic principals involved in selecting one of these repair types. Finally, the report contains extensive references separated as to both past and current rut repair research. Repair methods discussed in this report are:

- Micro-Surfacing
- Ultra-Thin Bonded Wearing Course—NovaChip®
- Ultra-Thin Bonded Wearing Course—Ultra-Thin Whitetopping (UTW)
- Stone Matrix Asphalt (SMA)
- Conventional Overlays Using High Quality Mix Design Methods and Materials

The report concludes that each of the repair methods probably merit trials as experimental features in Alaska, perhaps with a reservation concerning the applicability of UTW overlays. As explained in the report text, UTWs require a more substantial asphalt concrete layer than would be available for most Alaskan pavements, after preparation for overlay. SMAs and conventional asphalt concrete mixes should be evaluated using the most rut-resistant components obtainable, i.e., very hard aggregates and polymer-modified asphalt cements.

The report presents basic information concerning various levels of complexity for life-cycle cost analysis. The report recommends applying life-cycle cost analysis when selecting a rut repair method for full-scale projects.

The report recommends small-scale performance trials to evaluate the performance of each rut repair option under Alaskan conditions. Full-scale rut repair projects would follow construction and careful evaluation of the smaller test sections. One or more repair options should be selected for the full-scale rut repair projects. Base the selection on realistic life-cycle cost analyses, but only after enough performance data is collected from the experimental sections to justify such analyses.

1 - CHAPTER 1 – INTRODUCTION AND RESEARCH APPROACH

1.1 The Rut Problem In Alaska

Ruts are indentations within the wheelpath areas, parallel to the centerline, caused by repeated wheel loads. Ruts as deep as ~1/8 inch (3 mm) appear on almost all normally constructed pavements in Alaska within a year after construction. This amount of rutting is expected and considered acceptable. It is produced by the post-construction compaction of materials by the kneading action of vehicle tires. In terms of DOT&PF standards, rut depths remaining less than 0.30 inch (7.6 mm) are of no concern. The rut condition category assigned to such roadways is "good." Intermediate rut depth categories include those 0.30 inch (7.6 mm) to 0.39 inch (10.0 mm) and 0.40 (10.1 mm) to 0.49 inch (12.5 mm) deep. These are classified as "fair" and "marginal" respectively, and receive increasingly serious attention in DOT&PF's pavement management system. Pavements rut depths of 0.50 inch (12.6 mm) or more are classed as "poor," and are repaired or replaced as soon as possible within the economic confines of DOT&PF budgeting. Deeply rutted pavements [(= 0.50 inch) (=12.6 mm)] can be dangerous. Ruts often form barriers preventing rapid drainage of water from the pavement surface during rainstorms—a situation that can lead to loss of vehicle control through hydroplaning. Rutting decreases driver satisfaction and comfort; deep rutting noticeably affects vehicle steering response, thus creating or intensifying driver fatigue. Rutting also increases mechanical wear on vehicle suspension and steering components, thereby increasing vehicle user costs.

This report presents methods that may be used to repair rutting of asphalt concrete pavements in Alaska. In general terms, ruts are a form of pavement damage caused by: 1) deformation of one or more of the layers of the pavement structure or 2) material removal from the pavement surface (usually through abrasive wear of the pavement surface by studded tires). Hansen [1] describes in some detail the critical nature of investigations to determine the cause of rutting at a particular location before contemplating repair options. He states "You can't fix a pavement if you don't know what's wrong. It would be like trying to fix a car transmission by changing the tires". Hansen identifies key elements, each of which has to be carefully addressed before developing a rut repair strategy.

- Traffic Data
- Site Investigation
- Structural Analysis
- Materials Selection
- Construction Methods

For reasons discussed below, repairs addressed in this report are for rutting caused by tire abrasion and/or plastic deformation of the asphalt concrete layer. Specifically not addressed herein is rutting caused by deformation of the base course or underlying unbound aggregate layers. Justification for disregarding repairs of rutting caused by deformation of unbound layers is presented in the following paragraph and Table 1. The listing of repair methods described in this report was obtained from a general search of the available literature. Only a few of the methods have yet been tried in Alaska. "Hard" information regarding costs, constructability and performance of each method will remain speculative until each is tried (in the form of a construction experimental feature) and performance-monitored within the State. The report also describes methods by which a particular rut repair method can be optimally selected (in terms of life-cycle costs and required performance) from a catalog of repair methods for use on a specific pavement repair project.

Statewide, the Alaska Department of Transportation & Public Facilities (DOT&PF) spends an estimated $5 million dollars annually on rut repairs. Considering the 3,160 paved centerline miles (5,086 km) in Alaska, such repairs average nearly $1,600 per mile per year. The abrading action of studded tires and deformation of the asphalt concrete material have been identified as the predominant rut causing mechanisms in DOT&PF's Central and Southeast Regions. These Regions include the rather densely populated Anchorage and Juneau areas respectively. Most rutting in the Northern Region has been related to soft aggregate layers beneath the asphalt concrete pavement. The relative economic importance attached to repairing the rutting in various areas of Alaska is apparent in Table 1. These data were extracted from the 1999 DOT&PF Pavement Management System reports.

Table 1: Rutting Problem Shown by DOT&PF Region (in centerline-miles, -km)

DOT&PF Region	**Condition**			
	Good *Ruts < 0.30" (<7.6 mm)	**Fair 0.30" to 0.39" (7.6 to 10.0 mm)**	**Marginal 0.40" to 0.49" (10.1 to 12.5 mm)**	**Poor =0.50" (=12.6 mm)**
Northern	1,482 miles (2,385 km)	50 miles (80 km)	16 miles (26 km)	~2 miles (3 km)
Central	995 miles (1,601 km)	138 miles (222 km)	70 miles (113 km)	67 miles (108 km)
Southeast	250 miles (402 km)	32 miles (51 km)	22 miles (35 km)	36 miles (58 km)

* Average rut depths—based on electronic measurements selected from the most deeply rutted of two measured wheelpaths and averaged over 1/10 mile (0.16 km) intervals of roadway.

The Northern Region, with its predominately aggregate-related rutting, contains about half of the State's paved centerline miles but very little unacceptable rutting. In fact, almost 96% of Northern Region's pavements are in "good" condition in terms of rutting. On the other hand, nearly a quarter of the combined Central and Southeast Region miles (with their asphalt concrete-related rut problems) are rated "fair" or worse, with more than 6% in the "poor" category. This report is a first step in addressing rut repairs in Alaska, and the above statistics justify the emphasis on repairing deformation and/or abrasion of the asphalt concrete.

1.2 Alaska-Specific Rut Research

Two studies covering the (then) state-of-the-art research with respect to rutting in Alaska were published in 1990. The first, titled "Wheel Track Rutting Due to Studded Tires," [2] deals with the technology of studded tires (including safety benefits), pavement wear rates, factors affecting rates of pavement wear, the economics of studded tire use, and proposed restrictions on studded tires. The second, titled "State-of-the-Art on Rutting in Asphalt Concrete," [3] covers rutting not associated with surface abrasion (studded tire wear). This report discusses the test properties of rut-resistant materials and methods that can be used to predict rut formation in pavements. The report also presents asphalt concrete mix design methods and mechanistic pavement design methods intended to limit rutting in new pavements or overlays. Some of the information in each of these reports is useful when considering materials or design methods for improved rut resistance, although this subject is addressed using more recent references under following headings. Neither report specifically covers the subject of repairing already-damaged pavements.

Another report was produced (remains unpublished) by the DOT&PF, Central Region Materials Laboratory in 1995. Titled simply "Rut Study," [4] the report covers monitored field sites on two Anchorage roadways, the New Seward Highway and on Tudor Road. The report examines seasonal aspects of accumulating rut damage, rut development on a short Portland cement concrete section (Tudor Road), and rut development on three types of stone matrix asphalt (SMA) pavement (test sections on the New Seward Highway). This report contains no information addressing repairs of rutted pavements.

1.3 Rut Repair Methods—General Concepts & Basic "Tools"

The assumption here is that the repair method will fix a rut-damaged asphalt concrete layer, and that all layers below the asphalt concrete are of adequate design and construction. The author suggests several categories of rut repairs based on many literature sources and common sense. Conceptually, the general categories of rut repairs are:

Repair Catagory 1. *Fill-Width Replacement*—Remove and replace the existing asphalt concrete pavement layer.

Repair Catagory 2. *Full-Width Overlay*—Place a leveling course of material on top of the existing layer.

Repair Catagory 3. *Full-Width Mill & Replacement*—Remove a portion of the thickness of the existing pavement material; then replace with new material.

Repair Catagory 4. *Rut-Width Mill & Replacement*—Remove a thickness of material only from within the width of each rutted area; then replace with new material.

Repair Catagory 5. *Rut-Width Shimming of Low Areas*—Add fill material to the volume of each rut in order to bring the pavement surface back to proper cross section (analogous to the use of "Bondo®" to fill small dents in automobile body work).

Methods of categories 1, 2, and 3 involve pavement overlay or replacement thicknesses of 1 inch (25 mm) or more, and have traditionally been the "big hammers" of rut repair. These categories provide the opportunity to apply many recycling options as part of particular repair strategies. They usually involve most of if not the entire driving surface, and are therefore inherently expensive in terms of simply repairing ruts. However, depending on how much material is removed and replaced, each can take care of ruts as well as most other structural or surface damage problems (cracking, bleeding, potholing, raveling, shoving, etc.). The Alaska DOT&PF has used each of these methods for repairing ruts specifically, but usually in conjunction with repairs for other types of damage. A subsidiary benefit is normally gained from using these methods even where other damage types may not be prominent or even visible. These subsidiary benefits are real and can be accounted for in DOT&PF's Pavement Management System. For example, network-level pavement management may recommend prophylactic repairs, i.e., preventive maintenance, of seemingly undamaged pavements. The objective is to improve long-term minimization of total costs throughout the roadway network (optimization). Rut distress may be a primary or secondary target of pavement management project scheduling.

Methods in categories 4 and 5 have not been tried in Alaska. In fact, although category 4 repairs are possible, no examples of rut-width "mill and fill" projects could be found either in the literature or through personal contacts. If rutting were the <u>only</u> problem needing repair, methods 4 and 5 would offer an apparent economic advantage since they involve adding or replacing materials only within the rutted wheelpaths. If additional problems (besides rutting) exist, these methods would be unsuitable. Also, since special equipment or materials are required, the expected economic advantage may not actually exist. Finally, category 4 and 5 rut repairs would have almost no subsidiary value in terms of preventing other damage types—as seen in the big picture of pavement management optimization.

Some of the newest methods of rut repair involve categories 2, 3 and 5. The new methods are essentially overlay techniques even though category 3 requires surface preparation by milling. These methods are innovative because they utilize new rut-resistant materials that can be placed in thin layers. Some of the new slurry asphalt materials can be placed in layers as thin as about 1/2 inch (12.5 mm). New hot asphalt materials have been developed that can remain stable and rut resistant also when placed as thin as about 3/8 inch (10 mm). Where no milling preparation is desired, these materials can fill wheelpath rut depressions while greatly conserving on material across the remainder of the roadway width. The newest slurry materials can be applied in such a way as to fill very deep ruts [2 inches (50 mm) claimed] with no milling preparation—a category 5 repair.

1.4 Aiming at Rut-Resistant Materials—General Concepts

There is no intention to make this report into a design guide for rut-resistant paving materials. However, some background and reference materials are presented that give the reader a general perspective about how appropriate (rut-resistant) materials might be selected, and incorporated into the asphalt concrete mix design process. The section also discusses tests to determine the rut resistive potential of asphalt concrete material in the laboratory. This information will be food for thought when comparing various repair alternatives without the benefit of long-term field test data from Alaskan sites.

Keep in mind that the special (and perhaps very costly) rut-resistant materials used for rut repairs become just a single layer of a new pavement structure. New and old layers, must therefore function as an acceptable new structural system. The new system will fail if the choice of repair methods and materials is wrong. Repairs made with even the best materials will fail if they are placed on "poor" materials. The "poorness" or "appropriateness" of materials is of course relative. Resolve the question using a valid method to design the pavement structure, as a system of layers, based on expected traffic and environmental loadings. The structure must meet performance requirements not only for predicted rutting, but predicted cracking, and roughness as well. Finally, be careful in selecting a pavement design method. Get advice; verify that your design method accurately models the performance of every layer of material within the pavement structure.

If a design method is not available that models performance of that new high-tech rut repair material you've selected, then your rut repair project is an experiment! If this is the case, try to include one or more control sections within the project limits, make a concerted effort to monitor performance (usually 2 to 5 years or more), and keep good notes. Eventually, develop or modify a design method to properly model the new materials.

1.4.1 Selecting Aggregate for Rut Repair Materials

What about the relative importance of aggregate used in rut repair materials? The aggregate component is about 95 percent (by weight) of an asphalt concrete mixture. It's the aggregate that: form strong particle-to-particle contacts to support the vehicle load, must remain stable to resist permanent deformation, and must be hard enough to resist surface wear. Since asphalt concrete is almost entirely aggregate, can rut performance of candidate repair materials be predicted simply through one or more aggregate tests? The answer is no, but aggregate tests are indeed useful and are discussed below.

Although aggregate comprises almost the entire asphalt concrete mass, aggregate tests are problematic. Aggregate tests usually come up short on their ability to quantify asphalt concrete performance. An ideal aggregate test or suite of tests would allow researchers to derive a quantitative function (a "calibration") between test values and field performance. The interplay of the many variables in asphalt concrete/environmental systems pretty much guarantees that such functions are not derived. The bottom line is that aggregate test results are used for their qualitative "index" value (able to discriminate between several quality levels), but will not quantitatively predict performance. For example, DOT&PF has assumed for a long time, as have many other agencies, that gradation is strongly related to rut susceptibility of the pavement. Although the DOT&PF considers gradation information generally "useful" and coarser aggregate gradations in particular to be beneficial, the Department does not estimate rut development based on gradation variables.

NCHRP Report 405 relates aggregate test properties to pavement performance [5]. It identifies aggregate tests that appear to best correlate with how the asphalt concrete resists plastic deformation in laboratory performance tests of the asphalt concrete mix. Standard test methods are identified in the publication. Modified or new test procedures are described in a report appendix. The following tests were identified as correlating best with permanent (plastic) deformation.

- Aggregate Gradation Determined by Sieve Analysis
- Uncompacted Void Content of Coarse Aggregate
- Flat or Elongated Particles in Coarse Aggregate (2:1 ratio)
- Uncompacted Void Content of Fine Aggregate
- Methylene Blue Test of Fine Aggregate*
- Analysis of P200 (-0.074 mm) Size Fraction for Determining D60 and D10 Sizes*
- Methylene Blue Test of P200 (-0.074 mm) Size Fraction*

* *Permanent deformation may be due to stripping and loss of material rather than plastic deformation*

The findings of NCHRP Report 405 were from aggregate and mix performance tests run in the laboratory. The report recommends field trials to validate and quantify the link between aggregate test data and actual pavement performance.

DOT&PF adopted a Finnish test method, the so called "Nordic Abrasion Value" test in an attempt to characterize the resistance of Alaskan aggregates to tire-stud abrasion. It is not a standard DOT&PF test, and is therefore not contained in publications of Alaska Test Methods (ATMs). However, the test method (DOT&PF version) can be obtained by contacting the Statewide Materials Engineer at the DOT&PF Materials Laboratory in Anchorage, Alaska (907-269-6200). Nordic Abrasion test values are in terms of aggregate weight percent lost under wet-abrasion conditions, i.e., a rotating drum containing a mixture of aggregate, steel balls and water. DOT&PF cannot estimate rut development based on Nordic Abrasion data. Nordic abrasion criteria are as follows (obtained from Alaska DOT&PF Statewide Materials Laboratory):

Table 2: Nordic Abrasion Values Versus Allowable Traffic

Class	Nordic Abrasion Value	Average Daily Traffic/Lane
I	= 7	=10,000
II	=10	= 5,000
III	=14	= 2,500
IV	=17	=1,500

The Micro Deval test is not used in Alaska, but is very similar to the Nordic Abrasion Test and therefore worth mentioning. It also measures the degradation resistance of aggregates subjected to the abrasive action of steel balls in water. The Micro Deval abrasion test was developed in France in the 1960s. Canadian researchers conducted extensive research to correlate Micro Deval test results with field-performance levels of asphalt concrete pavements[6]. The Micro Deval test is presently available for evaluation in the U.S. as a provisional AASHTO method (TP58-00).

1.4.2 Latest Mix Design Technology (includes selecting asphalt cement) for Rut Repair Materials

Can new mix design methods help create an asphalt concrete material that resists rutting? This section presents some aspects of standard asphalt concrete materials and mix design technology that are considered state-of-the-art in producing rut-resistant asphalt concrete. This information is applicable to repair categories 1 through 3 and may be generally useful when evaluating the rut-resistive potential of paving materials proposed for new construction. Fill materials used in category 5 (and potentially useful in category 4) repairs are special material types that are appropriately discussed as these materials are introduced in the report.

Superpave (SUperior PERformance asphalt PAVEment) binder selection and mix design methods were developed as a principal product of the Federal Highway Administration's Strategic Highway Research Program. Superpave binder selection and mix design methodology is supposed to produce the best pavement serviceability for a given level of traffic in terms of all common types of damage—with an emphasis on rut resistance. Manuals published by The Asphalt Institute (TAI) have become the standard Superpave references. SP-1 covers Superpave asphalt binder selection and testing [7], while SP-2 covers mix design [8]. Problems were noticed on some new roadway pavements and test sections based on Superpave designs. The "Superpave Mixture Design Guide," recently issued by engineers of the Federal Highway Administration (FHWA), addresses the observed problems and provides an extremely useful commentary on the Superpave method [9]. The design guide supplements the TAI publications as well as other Superpave literature, and incorporates the experience of engineers across the country to date.

Superpave methods are complicated compared to DOT&PF's present selection of an AC-graded asphalt cement based on experience, and the use of the Marshall mix design method—this is something of an understatement. At the date of this writing, DOT&PF is leaning toward its first step into the world of Superpave mixes via proposed use of PG (Performance Graded) asphalt cement specifications. DOT&PF occasionally uses the Superpave gyratory compactor as a way of gaining additional information about otherwise Marshall-designed asphalt concrete.

1.4.3 The Use of Polymer Additives in Rut Repair Materials

How about the use of polymer modifiers in rut repair materials? Many types of polymer additives have been placed on the market with advertising that suggests almost miracle power to prevent rutting and cracking problems. With so many additives, so many claims, and so much recent research on polymer modifiers for asphalt concrete mixes, only a few words are appropriate here. Getting past the "hype" and after some experimentation in Alaska, it appears that a polymer additive may have worked very well at limiting plastic deformational-type ruts on some roadways. DOT&PF's Central Region Materials Engineer (personal contact) is of the opinion that a 3 percent addition (by weight of asphalt cement) of Styrene-Butadiene-Styrene (SBS), to the AC-5 asphalt cement component of an otherwise standard hot mix, has helped limit deformational rutting on some of Anchorage's busiest streets. His observations suggest that conventional asphalt concrete modified with SBS seems to resist plastic deformation about as well as stone matrix asphalt (SMA) mixes tried in Alaska to date. SBS additives increase asphalt cement viscosity at high temperatures.

1.4.4 Performance Testing of Asphalt Concrete Used as Rut Repair Materials

Can special testing of asphalt concrete materials help determine the potential of the mix to resist rutting? Short of large-scale field-testing with real vehicle loadings, laboratory performance testing on samples of the asphalt concrete provides some idea of the rut-resistant properties of the mix as a whole. The "Superpave Mixture Design Guide" referenced above [9] contains an appendix discussion of performance test methods. Performance test methods discussed in the Design Guide are aimed at determining the potential for rutting caused by plastic deformation of the mix. Performance test methods discussed are:

- *Marshall Mix Design Method*—Flow value can indicate if mix is over-asphalted and therefore susceptible to rutting. The Design Guide notes that stability and flow values obtained from Alaska's Marshall mix design method are useful indicators of rut potential as long as aggregate size is smaller than 1.5 inches (38 mm) and the mix is well graded.
- *Hveem Mix Design Method*—Stability values can indicate if the mix is over-asphalted and therefore susceptible to rutting.
- *Gyratory Testing Machine (GTM)*— A compaction equipment type developed by the U.S. Army Corps of Engineers. During GTM compaction of an asphalt concrete sample, the angle of gyration is measured as the number of gyratory cycles increase (see ASTM D3387). The gyratory shear index is defined as the initial angle of gyration divided by the maximum angle. Shear indices above 1.1 indicate an unstable mix, while values approaching 1.0 indicate increasing rut resistance.
- *Wheel-Track Testers*—The French LCPC Tester, the Georgia Loaded-Wheel Tester (GLWT) and the Hamburg Wheel-Tracking Device (HWTD) are each discussed. With each machine, a rolling load is applied to a laboratory specimen of asphalt concrete. However, they differ in design as well as the load and test conditions used. Also, each has a different criterion for whether the sample passes or fails. The Design Guide comments that the French and Georgia testers may be non-conservative for some mix types, perhaps leading to placing poor mixes. On the other hand, the severe conditions applied by the Hamburg tester result in a conservative selection process that may reject reasonably serviceable mixes.
- *Superpave Shear Tester (SST)*—This is a complicated device with several modes of testing available. The bugs have not been worked out; the Design Guide advises that test data cannot be used to predict rut performance at this time.
- *Creep Tests*—Tests mentioned include the standard creep and creep-creep recovery (CCR) tests done under triaxial stress conditions, as well as the Static Creep/Flow-Time test performed with or without confining stresses. Standard creep and CCR data have been applied in one computer program for modeling rut development [10], otherwise the application of these data is still being researched. The Static Creep/Flow-Time test shows promise according to researchers on NCHRP Project 9-19. The Design Guide suggests that data from this test can be used to evaluate both the asphalt cement content and the interlocking structure of the aggregate particles. Test data may be used as a rut

performance criterion or simply to compare the shear resistance properties of different mixes.

- *SRK Test*—Tests resistance to wear by an abrasive mechanism applied around the curved periphery of an asphalt concrete cylinder. This European testing procedure [11] measures wear resistance based on the depth of groove worn into the side of the cylinder. The method is currently available in draft form as prEN 12697-16, Method B. SRK criteria are as follows (obtained from Alaska DOT&PF Statewide Materials Laboratory):

Table 3: SRK Value Verses Allowable Traffic.

Pavement Class	SRK Value	Average Daily Traffic/Lane
1	= 25	= 10,000
2	= 35	5,000 – 10,000
3	= 45	1,500 – 5,000
4	= 60	= 1,500

- *Prall Test*—Tests resistance to wear by an abrasive mechanism applied to the top surface of an asphalt concrete cylinder. This European testing procedure is Method A of prEN 12697-16 cited above. It measures wear resistance based on the amount of material removed from the top of the cylinder. Prall test performance criteria are not available at the time of this writing.

2 - CHAPTER 2 – RUT REPAIR METHODS & REPAIR COSTS

This research identified the following as representing the "latest" of reasonably mature, verifiably reliable (state-of-the-practice) technologies for repairing rut-damaged asphalt concrete pavements:

- *Micro-surfacing*—an asphalt slurry surfacing
- *Ultra-Thin Bonded Wearing Courses*—includes thin hot mix asphalt concrete and Portland Cement concrete overlays (sometimes called "inlays" if milling is done first)
- *Stone Matrix Asphalt (SMA) pavements*—a coarse textured hot asphalt concrete pavement type
- *Rut-Resistant Conventional Hot Asphalt Pavements*—involves "high tech" modifications of conventional hot asphalt concrete mixes (using improved mix design methods, asphalt cement additives, and special abrasion-resistant aggregates)

These technologies are new enough that they remain under evaluation, and are the subjects of ongoing research efforts by many researchers. Research in progress is documented in Appendix A.

Each rut repair technology is discussed below in enough detail that the reader will understand basic concepts regarding materials and construction methods. This section also provides unit cost and performance-life estimates for each of the repair methods discussed.

Depending on the thickness of the existing rutted pavement, rut depth, presence or absence of curb/gutter and other factors, it may be necessary to remove a portion (sometimes all) of the rutted surface using milling equipment. A discussion of the cold milling operation and associated costs is therefore included for the sake of completeness.

2.1 Surface Preparation (Milling)

A National Highway Institute (NHI) course on pavement rehabilitation described cold milling as one of the most common ways to remove a portion of an asphalt concrete pavement surface prior to overlay work [12]. Cold milling uses carbide steel cutting bits, mounted on a rotating drum, to chip off a selected thickness of the pavement surface. Milling is normally used in combination with an overlay technique to maintain curb lines while removing some or all of the rutted pavement thickness. With conventional milling equipment it is possible to remove as much as 3 to 4 inches (75 to 100 mm) of pavement during a single pass, so the inch or less of removal often required by the thin overlay techniques is easily done. Hot milling is an alternative to cold milling, although heater-milling units are most commonly used as equipment elements in a "paving train" recycling operation [13].

Besides reducing the pavement thickness, a milling operation can produce a rough surface texture that helps ensure a good bond between the old pavement and the overlay material. The best bond occurs when the overlay work is done within a day of milling. Longer waiting periods usually require careful cleaning of the milled surface prior to placing the overlay. However, if there is an appreciable time lapse between milling and overlay work, the side benefit is that the rough-textured milled surface provides interim skid resistance. In addition to on-grade benefits, the asphalt concrete chunks produced as mill cuttings are very desirable for a number of applications. Common applications are as a recycled component in asphalt concrete mixtures, as aggregate for road surfacing, and as an excellent base course aggregate.

On open stretches of pavement, the cold milling operation is usually done as a series of adjoining passes that parallel the roadway centerline. Seal or patch cracks larger than hairline size [~ 1/4 inch (6 mm) width or wider] and potholes prior to milling. During milling, hold variations in the longitudinal profile and cross section to the same tolerances required for new construction.

Although this report is concerned mainly with ruts generated through plastic deformation or abrasion of the asphalt concrete, Noureldin [14] provides insight into the thickness of pavement that must be replaced to contend with other sources of rutting.

2.2 State-of-the-Practice Repair Treatments

2.2.1 Micro-surfacing

Micro-surfacing is described by the International Slurry Surfacing Association (ISSA) [15,45] as a polymer modified slurry paving system appropriate for repairing a broad range of problems. Like normal slurry seal material, micro-surfacing is mixture of dense-graded aggregate, asphalt, water, and mineral fillers, although it contains higher quality aggregates and additives. Micro-surfacing was developed in Germany during the 1960s and 1970s, and was introduced into the U.S. in1980. Micro-surfacing materials evolved from conventional slurry during German engineers' efforts to come up with economical rut fillers. Their mixture of selected aggregate, emulsified asphalt and polymers is highly stable relative to normal surface treatment materials, i.e., micro-surfacing is stable when placed in multi-stone thicknesses. Micro-surfacing has been adopted as a fairly common way of repairing ruts in the U.S., and the performance record accumulated over the past 20 years has been generally good in all climate areas. The ISSA is the chief technical organization providing micro-surfacing design methods and specifications to the paving construction industry. Several references covering the performance of micro-surfacing are included in the bibliography.

Micro-surfacing aggregate can be either of two similar gradations, both with a maximum size near 3/8 inch (9.5 mm). Using the largest aggregate and the rule-of-thumb that a paving material can be placed at a minimum thickness of about 1.5 times the maximum aggregate size, micro-surfacing can be placed as thin as just over 1/2 inch (12.7 mm). With the slightly smaller aggregate the material can actually be placed as thin as 3/8 inch (9.5 mm). Micro-surfacing components are mixed, as needed during the paving process, in the machine that applies the micro-surfacing. For lane-width surfacing, the machine moves forward as the components are mixed then fed into a full-width "surfacing box" that spreads the micro-surfacing across the width of the lane. Shallow ruts can be filled as part of lane-width placement. Supplier's literature claims that continuous-load slurry pavers can lay up to 500 tons (450 Mg) of micro-surfacing per day. A tack coat is usually not required.

Deep ruts are another matter. In terms of maximum thickness, micro-surfacing has been placed using special, narrow spreader boxes "rut boxes" that can deliver a rut-width band of material thick enough to fill 1.5 inch (38 mm) ruts [up to 2 inches (50 mm) is sometimes claimed]. Each rut is filled separately. The specially engineered rut box delivers the largest aggregate particles into the deepest part of the rut to promote maximum stability. The rut box is adjusted to leave a slight crown to compensate for initial compaction by traffic. The micro-surfacing edges are automatically feathered. Deep ruts can be filled with no further paving required. However, the usual process is that the rut fill micro-surfacing serves as a "scratch" (leveling) course, followed by a wide and very thin micro-surfacing layer. This 2-step process provides better aesthetics than simply filling the ruts, and has some other advantages as far as reestablishing a generally smooth pavement surface.

The ISSA provides information for estimating the amount of micro-surfacing required to fill a rut of specific depth (average rut shape assumed) [18].

Table 4: Micro-Surfacing Quantity Needed for a Given Rut Depth.

Rut Depth	Micro-surfacing Quantity Needed
0.50 – 0.75 inch (12.7 – 19 mm)	20 – 30 lb/yd^2 (11– 16 kg/m^2)
0.75 – 1.00 inch (19 – 25 mm)	25 – 35 lb/yd^2 (14 – 19 kg/m^2)
1.00 – 1.25 inch (25 – 32 mm)	28 – 38 lb/yd^2 (15 – 21 kg/m^2)
1.25 – 1.50 inch (32 – 38 mm)	32 – 40 lb/yd^2 (17 – 22 kg/m^2)

Even thickly placed micro-surfacing is stable after curing. As with other emulsified asphalt-based surface treatments, micro-surfacing is initially a dark brown color then soon changes to black as the emulsion breaks and the water is ejected to the surface. Curing sufficient to withstand traffic can occur within about 1 hour with moderate humidity and a warm temperature [= 75° F (24° C) and = 50% relative humidity]. Micro-surfacing cannot be applied if the air temperature or the pavement surface is below 50º F (10° C) and falling, but can be applied if these temperatures are above 45º F (7° C) and rising. Micro-surfacing cannot be applied is there is a possibility of freezing within 24 hours. Rain or threat of imminent rain will prevent micro-surfacing. The old pavement surface must be clean (normal sweeping) and dry prior to the micro-surfacing application.

A number of literature sources provide micro-surfacing specifications as represented by those obtained from Sealcoating Inc. [16], the Virginia DOT [17], and the ISSA [18]. The specifications differ mainly on the subject of mix design. Information extracted mainly from the ISSA specification summarizes materials and mix design requirements from the perspective of the leading slurry paving trade organization.

CSS-1h is modified with polymer and used as the micro-surfacing binder. The polymer modifier must be blended into the asphalt or emulsifier solution prior to the emulsification process. The minimum amount and type of modifier is determined by laboratory testing during a mix design process. Various modifiers can be used in micro-surfacing. A particular polymer additive is selected based only on the requirement that the micro-surfacing mix must pass the battery of mix design tests. Usually, about 3% modifier, by weight of the asphalt cement, is considered minimum. CSS-1h quality is determined using AASHTO M208, T59, T53 and T49.

The required aggregate is 100% freshly fractured, hard stone material. Total fracture is assured; all aggregate must be crushed from source material larger than the largest stone in the specified gradation. Aggregate quality tests include Sand Equivalent, AASHTO T176 (=65), Magnesium-Sulfate Soundness, AASHTO T104 (= 25), Los Angeles Abrasion test, AASHTO T96 (= 30), and the aggregate must also meet state-approved hardness tests. Gradation tests are according to AASHTO T27 and T11. Commonly used gradations (standard ISSA gradations I and II) are shown in Table 5.

Table 5: Micro-Surfacing Gradations

Sieve Size	Type II * (Percent Passing)	Type III ** (Percent Passing)	Stockpile Tolerance
3/8 inch (9.51 mm)	100	100	
# 4 (4.76 mm)	90 – 100	70 – 90	± 5%
# 8 (2.38 mm)	65 – 90	45 – 70	± 5%
# 16 (1.19 mm)	45 – 70	28 – 50	± 5%
# 30 (0.595 mm)	30 – 50	19 – 34	± 5%
#50 (0.297 mm)	18 – 30	12 – 25	± 4%
#100 (0.149 mm)	10 – 21	7 – 18	± 3%
# 200 (0.074 mm)	5 – 15	5 – 15	± 2%

* Type II normally used for urban and residential area applications.
** Type III normally used for primary and interstate routes (most common micro-surfacing gradation).

The percent passing each sieve cannot vary more than the indicated tolerance after the target gradation has been selected and the mix design has been done.

The mix design process may require the addition of mineral filler. Mineral filler (if required) is non-air entrained Portland cement or lump-free hydrated lime. The mineral filler quantity is considered part of the aggregate gradation.

Water requirements are as normal for emulsified asphalt work, i.e., use potable water.

Special additives may be used to control the curing rate of the micro-surfacing. Special additives must be evaluated as part of the mix design process to ensure compatibility with all other components of the mix.

The mix design process is fairly involved, and the ISSA can supply a list of companies qualified to do micro-surfacing mix designs. The mix design is performed using a series of seven ISSA testing procedures, including: ISSA TB-139 (wet cohesion), TB-109 (excess asphalt by LWT sand adhesion), TB-114 (wet stripping), TB-100 (wet-track abrasion loss), TB-147 (lateral displacement), TB-144 (classification compatibility), and TB-113 [mix time @ 77º F (25° C)]. Detailed discussion of these tests is beyond the scope of this report. The mix design report will list percentages of each material used in the mix, although some adjustments might be needed in the field.

2.2.2 Ultra-Thin Bonded Wearing Course—NovaChip®

NovaChip® is usually a 1/2 to 3/4 inch (12.7 to 19 mm) thick, open-graded hot mix asphalt concrete overlay surfacing placed on a heavily applied polymer-modified asphalt cement "membrane" tack coat [19,20] NovaChip® is a registered trademark of Societe Internationale Routiere, a subsidiary of Screg Routes STP, France. Koch Materials Company Inc. licenses the proprietary paving "system" in the U.S. This overlay process was developed in France in 1986 to increase skid resistance and to seal old pavement surfaces. It has since been found useful for restoring surface smoothness and as a rut repair method. Koch representatives state that NovaChip® can be used to fill ruts up to about 1/2 inch (12.7 mm) deep without first having to mill the surface or fill the ruts with another form of scratch course.

A Minnesota Technology Transfer newsletter describes the NovaChip® process [21]. When completed, a NovaChip® pavement appears similar to a conventional asphalt concrete pavement with an open-textured surface. The NovaChip® paver is a large and specialized affair. In addition to simply applying the paving mixture, the machine includes a tank containing NovaBond®, the polymer-modified emulsion tack coat/membrane. Because the heavily modified emulsion is applied almost simultaneously with NovaChip® mix, the emulsion can be applied much thicker than a typical tack coat. The heavy application seals the entire surface including most small cracks, and it ensures that NovaChip® bonds well to the old pavement surface. Surface preparation involves simply removing loose debris. The paving equipment makes only a single pass, during which the heat from the hot mix wicks the thick tack coat upward where it permeates a portion of the hot mix. Because NovaChip® is a hot mix material, there are no loose aggregate "chips" when paving is completed. The new pavement can withstand full traffic flow usually in less than an hour.

DeMartino [22] described NovaChip® construction in more detail (Note: numerical values in the following paragraphs have been updated, from the DeMartino reference, based on personnel communications with the Koch, Inc. representatives). The mixes are produced at relatively high temperature [330º to 350º F (166 to 177° C)] in either batch or continuous (drum) plants, and conventional trucks usually deliver the material to the paver within a temperature range of 300º to 330º F (149 to 166° C). After mixing, the material should not be stored overnight—primarily due to the possibility of asphalt cement drain-down.

The NovaChip® paving machine's four augers move delivered material from the front to the rear of the machine with minimum segregation. At the rear of the paver, the mix is distributed to the full width of the screed by conventional augers (the paver uses a vibrating, heated screed). The polymer-modified emulsion tack coat, stored in the paver's on-board tank, is sprayed onto the road surface immediately in front of the hot mix application. Spray bar and screed extensions are designed to work as a unit, extending and retracting together. NovaChip® can be applied at a rate of 30 to 90 ft/minute (9.1 to 27.4 m/minute).

NovaChip® construction involves placement of the polymer-modified tack followed by the hot mix. The application rate of tack is about 0.2 gallons/yd^2 (0.9 l/m^2). Tack is sprayed from the paver from a location behind the rear wheels so that it is not wheel-tracked. The temperature of the emulsion tack ranges between 120º and 180º F (49° and 82° C). Requirements are for placement of the hot mix on the tack within 5 seconds, although placement is almost immediate since the paver dispenses both materials. The paver spreads NovaChip® hot mix on the tack coat with a hot mix temperature between 290º and 330º F (143° and 166° C) (screed exit temperature), and then the heavily–applied tack draws upward into the gap-graded mix. Water evaporates as a portion of the hot mix combines with a portion of the tack coat. Some of the tack is therefore incorporated into about the bottom third of the NovaChip® layer. Approximately the top two thirds of the hot mix layer remains porous, thus providing the textured driving surface.

Rolling must be done very quickly, before the temperature falls below 195° F (91° C), because the thin layer cools rapidly. The vibratory screed provides breakdown rolling. Final rolling consists of one or two passes with a 10-ton (9.1 Mg) roller (no vibration), operating very closely behind the paver. There are no minimum density specifications—as final rolling is said to simply "seat" the aggregate particles. The new pavement can be opened to traffic after rolling and when the mat has cooled below 185º F (85° C).

Reports of NovaChip® long-term performance in colder areas of the U.S. are limited but favorable. A 1999 report by the Pennsylvania DOT [23] discusses monitoring of four projects over a five-year period. These sections performed well, and NovaChip® is now considered a viable maintenance option on highly trafficked roads. Personal communications with Missouri and Michigan DOT engineers indicated successful experimentation with NovaChip® surfaces, although both suggested possible problems related to the open texture of the material. Problems appear to involve some freeze/thaw damage due to trapping of water and/or removal of aggregate through snowplow operations.

DeMartino [22] and Oregon researchers [45] outlined NovaChip® materials requirements and construction basics. Gap-graded aggregate is produced from a mixture of several stockpiles using 100% crushed materials. The mix requires hard aggregates. A licensed laboratory performs the proprietary mix design. Performance graded asphalt cement (a Superpave grading) is used in the mix. Traffic (ESALs), pavement condition, climate, and aggregate type are factors considered to determine whether the NovaChip® mix requires polymer modifiers, and if so, the amount and type.

Requirements for coarse aggregate [retained on #4 (4.76 mm) sieve] include Los Angeles Abrasion, AASHTO T96 (= 35), the Micro Deval % loss, provisional AASHTO TP58-00 (= 18), a Magnesium-Sulfate Soundness % loss, AASHTO T104 (= 18), [or Sodium Sulfate Soundness % loss =12], Flat & Elongated Ratio, ASTM D 4791 (=25% @ 3:1), Single Face % Crushed, ASTM 5821 (=95) and Multiple-Face % Crushed, ASTM 5821 (=85).

Requirements for fine aggregate [passing #4 (4.76 mm) sieve] include a Sand Equivalent, AASHTO T176 (= 45), Methylene Blue, AASHTO TP57-99 [=10 on materials passing #200 (0.074 mm) sieve], and an Uncompacted Void Content, AASHTO T304 (=40).

Gradation tests are according to AASHTO T27 and T11. Commonly used gradation and asphalt content requirements are shown in Table 6.

Table 6: Novachip® Aggregate Gradations

Sieve Size	#4 (4.76 mm) Type A (Percent Passing)	3/8" (9.51 mm) Type B (Percent Passing)	1/2" (12.7 mm) Type C (Percent Passing)
¾ inch (19 mm)			100
½ inch (12.7 mm)		100	85 – 100
3/8 inch (9.51 mm)	100	85 – 100	60 – 80
# 4 (4.76 mm)	40 – 55	25 – 38	25 – 38
# 8 (2.38mm)	22 – 32	22 – 32	22 – 32
# 16 (1.19 mm)	15 – 25	15 – 23	15 – 23
# 30 (0.595 mm)	10 – 18	10 – 18	10 – 18
# 50 (0.297 mm)	8 – 13	8 – 13	8 – 13
#100 (0.149 mm)	6 – 10	6 – 10	6 – 10
# 200 (0.074 mm)	4 – 7	4 – 7	4 – 7
Asphalt Content, %	5.0 – 5.8	4.8 – 5.6	4.6 – 5.6

The mix design has a drain down requirement for the loose mixture, AASHTO T305 (=0.10%) and must pass a tensile strength requirement, AASHTO T283 (=80%).

A 1997 National Center for Asphalt Technology (NCAT) study [24] provides the following aggregate gradations and asphalt content information for NovaChip® pavement sections constructed in Alabama.

Table 7: Job Mix Information for Alabama Projects.

Sieve Size	Percent Passing		
	Tallapossa Project Crushed Gravel	Tallapoosa Project Crushed Granite	Talladega Project Crushed Granite
1/2 inch (12.7 mm)	100	100	100
3/8 inch (9.51 mm)	88	95	99
# 4 (4.76 mm)	36	35	40
# 8 (2.38 mm)	28	24	25
# 16 (1.19 mm)	21	18	15
# 30 (0.595 mm)	15	12	13
# 50 (0.297 mm)	11	9	10
#100 (0.149 mm)	8	7	8
# 200 (0.074 mm)	5.0	5.2	5.3
% Asphalt Content	4.8	5.2	5.2

2.2.3 Ultra-Thin Bonded Wearing Course—Ultra-Thin Whitetopping (UTW)

Ultra-thin whitetopping (UTW) is a relatively thin, high strength Portland cement concrete (PCC) overlay placed on a milled surface of asphalt concrete pavement. Whitetopping is usually 2 to 4 inches (50 to 100 mm) thick and is constructed with closely spaced joints. The American Concrete Pavement Association (ACPA) is the chief technical organization providing whitetopping design methods and specifications to the paving construction industry. Recent ACPA publications cover essentially all aspects of UTW technology [25,26].

Recent literature [27] gives a state-of-the-practice overview of the technology. Overlays of PCC on asphalt concrete have been constructed for nearly 60 years. Conventional whitetopping pavements were first constructed in 1944, and usually have a minimum thickness about 5 inches (125 mm)—enough thickness that no bond between the overlay and the underlying asphalt concrete is necessary. Conventional whitetopping has been applied for many years to combat rutting of asphalt concrete pavements on roads carrying heavy truck traffic.

UTW is different from conventional whitetopping because techniques are used to ensure that the UTW bonds to the underlying asphalt concrete. The bonded layers form a composite pavement section that reduces stresses within the UTW layer to a structurally acceptable level. The UTW joint spacing is critical and must be kept short. Joint spacings that performed well on UTW projects are between 2 and 5 feet (0.61 and 1.52 m). The maximum joint spacing is 12 to 15 times the UTW thickness in each direction. UTW contains no steel reinforcing elements, but the PCC mix usually contains reinforcing *fibers*. Although UTW can be constructed with or without fibers, the ACPA recommends their inclusion. The fibers prevent spalling and loss of concrete pieces when inevitable cracks occur. The asphalt concrete under the UTW must be appropriately stiff and thick. ACPA representatives recommend an asphalt concrete thickness of no less than 3 inches (75 mm)—and the stiffer the asphalt concrete the better.

The UTW concept was developed for low-volume pavements, such as city streets, where rutting, washboarding and shoving were the main problems. Earliest trials of whitetopping were in Kentucky (1988) and Colorado (1990). Apparently, the first heavily monitored UTW overlay project was constructed in 1991 in Kentucky, where 2 inch and 3.5 inch (50 and 90 mm) thicknesses were tested. The test section was intentionally placed at a location highly trafficked by trucks to provide an accelerated-wear test section. The experimental UTW sections were exposed to 400 to 600 heavy trucks per day, and reportedly carried traffic successfully for nearly a year. As of 1998, about 300 sections of UTW overlay have been constructed in 25 states including California, Minnesota, South Dakota, Tennessee, and Pennsylvania. Most of the sections have not existed long enough to establish a long-term performance record for UTW.

The author contacted a representative of the ACPA regarding the question of expected service life. Having observed the UTW performance of many UTW sections since the first experiments, the representative speculated that a 10-year+ service-life for city streets might be reasonably expected. FHWA's Turner-Fairbank Highway Research Center reported on accelerated testing of UTW that began in 1998 at its McLean, Virginia pavement testing facility [28]. The test sections included eight, 48-foot-long (14.5-m-long) lanes. The objective was to help State and local highway agencies make decisions about UTW applications by validating ACPA design methods. UTW thicknesses of 2.5 inches and 3.5 inches (65 and 90 mm) were, depending on the test section, subjected to either 53,200 ESALS or 126,700 ESALS per week using a loaded tire speed of about 11.5 mph (19 km/hr). Testing was done between May 1998 and November 1999. Actual loading of individual sections varied between 7 and 30 weeks. These accelerated rut tests are discussed in a paper presented by J. A. Sherwood at the spring 2001 American Concrete Institute (ACI) convention [29]. The paper indicates that mechanistic models will be developed for designing the UTW layer thickness. The mechanistic design will determine a UTW modulus and thickness combination that minimizes UTW tensile stresses to control cracking. Design inputs will include traffic loadings and the elastic modulus values of asphalt concrete and aggregate layers that will underlie the UTW. Additional reports should become available by mid 2002.

There are three steps in constructing a UTW overlay [27]:

1. Prepare the surface by milling and cleaning.
2. Place/finish/texture.
3. Saw joints.

A mechanical bond must be established between the overlay and overlaid material through intimate contact of the two materials, since no additional agent, additive, etc. is used to promote bonding. Therefore, a freshly milled surface probably promotes the best bonding between the UTW and asphalt concrete. Allow no appreciable time lag (no more than a day) and no traffic on the milled surface between final cleaning and UTW paving. Dust or debris that accumulates on the prepared asphalt concrete surface will diminish the bond.

Use conventional paving methods for the UTW, including slip-form, fixed-form pavers and normal hand equipment. Handle finishing and texturing in the normal way.

Proper curing is the key to a successful UTW; this avoids shrinkage cracking, and loss of the asphalt/PCC bond. The UTW has a high surface-area/volume ratio so water will be lost very rapidly during curing. Curing compound should be heavily applied at about 11 yd^2/gal (2.4 m^2/l). The compound must be applied to the surface and all exposed edges without running. Curing compound slopped onto the asphalt concrete surface will decrease the bond.

To limit cracking, joints are sawn as quickly as possible to a depth of about 1/4 to 1/3 the thickness of the UTW, and the sawn joints are usually left unsealed. A 3 mm (about 1/8 inch) joint width is standard, i.e., narrow enough to keep most incompressible particles out of the joint.

A mix design is done for a particular project according to traffic load requirements and the length of time before the road must be reopened to traffic. For example, early trafficking requires PCC that will cure to at least a 3,000 lb/in^2 (20.7 MPa) compressive strength within 24 hours. The usual mix design addresses cement, coarse aggregate, fine aggregate, an air entraining agent, synthetic fibers (usually polypropylene), and water reducers or plasticizers to promote a low water/cement ratio. ACPA publications [25,26] are the recommended sources of mix design information. For a realistic idea of materials proportioning actually used, Table 8 contains a sampling of recent mix designs [27].

Table 8: UTW Mix Components From Three Construction Projects

Material Proportions (per cubic yard)	State Route 21, Iowa, 1994	Leawood, Kansas, 1995	Tennessee & Dekalb County, Georgia
Cement, lb (kg)	573 (260)	610 (277)	799 (363)
Coarse Aggregate, lb (kg)	1,662 (755)	1,694 (769)	1,699 (771)
Fine Aggregate, lb (kg)	1,364 (619)	1,320 (599)	1,230 (558)
Water, gal—@8.34 lb/gal (liters— @ 1.00 kg/l)	29.2 (110.5)	26.9 (101.8)	33.5 (126.8)
Air Content (%)	6	6.5	Unknown
Water/Cement Ratio	0.43	0.37	0.35
Synthetic Fibers, lb (kg)	2.3 (1.0)	2.3 (1.0)	2.3 (1.0)
Compressive Strength, @ 24 hours, lb/in^2 (MPa)	unknown	3,000 [design] (20.7)	3,000 (20.7) [design] 5,000 (34.5) [actual]

An ACPA representative suggested estimating mix costs assuming 7-sacks/yd^3 (9.2-sacks/m^3) of high strength-early set cement, 3 lb/yd^3 (1.8 kg/m^3) of polypropylene fibers, plus accelerators. The author contacted Alaskan PCC suppliers who estimated that such concrete in Alaska would fetch about $125.00 to $150.00/yd^3 ($163.00 to $196.00/m^3).

2.2.4 Stone Matrix Asphalt (SMA)

SMA is a hot-mixed asphalt concrete containing relatively high proportions of large aggregate and asphalt cement. SMA mixes have garnered a reputation for providing tough, stable, rut-resistant pavements. The SMA concept relies on stone-on-stone contact to provide loading bearing strength and a rich binder for durability (resistance to oxidation). SMA contains a gap-graded aggregate that is held together by a rich matrix of mineral filler, fiber, and polymer-modified asphalt cement [30]. Based on Alaskan experience, the asphalt cement content of an SMA usually runs about 1% higher than a conventional asphalt concrete. The National Asphalt Pavement Association (NAPA) is the chief technical organization providing SMA design methods and specifications to the paving construction industry. Recent NAPA and the FHWA publications cover design and construction with SMA mixtures [31,32].

According to Brown [33], SMA was developed in West Germany, has been used in Europe since about 1972, and was first developed to resist studded-tire wear. SMAs fell out of favor in some European countries as studded tires were banned, then again came into more popular use as tire pressures, wheel loads, and traffic volumes increased. Besides Germany, SMA came to be used extensively in Sweden, Denmark, Norway, Finland, Austria, France Switzerland, and the Netherlands. A team of U.S. technical representatives visited Europe in the fall of 1990, found the SMA concept to be attractive, and made plans for constructing the first SMA pavement in Michigan in 1991.

Performance of SMA pavements in the U.S. is summarized in a 1997 NCAT report [34]. The principal conclusion was that the increased cost of an SMA pavement is more than offset by increased performance. Other performance-related conclusions drawn from more than 100 projects were:

- Rut depths less than 0.2 inch (5 mm) on 90% of the selected SMA projects constructed before 1996.
- No measurable rutting on 25% of the SMA projects.
- Relatively little thermal and reflective cracking compared with conventional pavements.
- No raveling, i.e., loss of surface aggregate on SMA pavements.
- Most frequent problems are "fat spots" (flushing) blamed on aggregate segregation, draindown of asphalt cement, high asphalt cement content, and improper amount and/or type of stabilizer. Draindown refers to the downward flow asphalt cement through the SMA aggregate structure while the mix is hot—AASHTO test method T305.

The Alaska DOT&PF has constructed several SMA pavement sections using Alaskan specification special provisions [35]. As mentioned previously, Alaska DOT&PF experience indicates less than spectacular performance. Some DOT&PF materials personnel hold the opinion that normal asphalt concrete mix modified with 3% SBS resists rutting caused by plastic deformation about as well as the stone matrix asphalt (SMA) mixes evaluated so far in Alaska.

A Washington DOT article discusses general aspects of SMA specifications [36]. Gap-graded aggregate used for SMA is on the coarse side of the maximum density line when the gradation curve is plotted on a 0.45 power gradation chart, i.e., plotted gradation line is concave upward. The SMA aggregate structure is different from most open-textured mixtures because most of the voids between the coarse aggregate are filled with mineral filler and binder. SMA air void contents are usually 2 to 4%. The addition of too much asphalt cement can cause a drastic loss of shear strength and resistance to rutting. Too little asphalt cement increases the air voids to a point where the mix is vulnerable to oxidation (accelerated aging), moisture damage, and perhaps freeze/thaw induced damage. SMA mixtures normally contain about 10% fines, i.e., "dust" [-#200 (0.074 mm) sieve fraction], and have a dust/asphalt cement ratio of about 1.5. Desirable aggregates are:

- Highly cubical with rough surface texture to resist rutting
- Hard as possible to resist polishing as well as fracturing and abrasion under wheel loads

Stabilizing additives such as fibers (e.g., cellulose and rock wool), polymers, carbon black, artificial silica, etc. are often added to stiffen the mix. Although not needed in all mixes, these help form a thick "mastic" binder that allows high asphalt cement contents without draindown immediately after mix production and/or post construction bleeding problems when the pavement becomes hot.

Information extracted from a Michigan DOT's SMA specification [37] summarizes requirements from the perspective of a typical northern tier state. Indicative of the recent improvements in asphalt concrete mix design and materials technology, Michigan refers to its SMA as a " bituminous mixture gap-graded Superpave with cellulose, composed of coarse aggregate, fine aggregate, mineral filler, cellulose fibers and asphalt cement."

Michigan's mix design procedure is AASHTO PP41-00, and the mix must meet the requirements of Table 9.

Table 9: Michigan SMA Mix Design Criteria*

Design Parameter	Specification Limits
Percent of Maximum Specific Gravity (%G_{mm}) at 100 gyrations using the Superpave Gyratory Compactor (SGC)	96%
VMA, minimum % @ N_{100}	17.0
VCA_{mix}, minimum %	Less than VCA_{drc}
Tensile Strength Ratio (TSR), minimum %	70
Draindown at Production Temperature, maximum % (AASHTO T305)	0.30
Asphalt Content, minimum %	Ranges between 5.5 to 6.8 % for aggregate specific gravities between 3.00 and 2.40 respectively (note asphalt cement requirement is inversely proportionate to Sp.G.)

* Criteria as listed in the specification. Contact Michigan DOT Materials Engineer to identify test method standard and number.

There is a Superpave performance graded asphalt cement requirement, i.e., PG 70-22 for "Metro" regions, otherwise PG 70-28.

Mineral filler must be fine mineral matter such as rock dust, or crushed limestone. The material can contain no measurable organics and have an AASHTO T90 Plasticity Index = 4.

The cellulose fiber (added to prevent draindown) must conform to several tests that are listed here without detail. There are fiber-length requirements involving sieve analyses. In addition, the cellulose fibers must meet: Ash Content, pH, Oil Absorption, and Moisture Content requirements.

Coarse and fine aggregates are defined as being retained or passing the # 4 (4.76 mm) sieve. Fine aggregate must meet requirements for Angularity (ASTM C 1252, Method A = 45) and must be non-plastic according to AASHTO T90.

Coarse aggregate must consist of 100% crushed material. Coarse aggregate must meet Michigan DOT Test Method (MTM) or ASTM requirements for: Percent Abrasion Loss (MTM 102, = 30), Percent Crushed Particles (MTM 117, 100% for 1-face fracture, = 90% for 2-face fracture), Percent Soft Particles (MTM 110, = 3.0), Percent Absorption (MTM 320 & 321, = 2.0), and Percent Flat & Elongated Particles (ASTM. D 4791, = 20 for 3-to-1, = 5 for 5-to-1.

Table 10 shows the required combined aggregate gradation.

Table 10: Aggregate Gradation for Michigan SMA

Sieve Size	Percent Passing
3/4 inch (19 mm)	100
1/2 inch (12.7 mm)	90 – 99
3/8 inch (9.51 mm)	50 – 85
# 4 (4.76 mm)	20 – 40
# 8 (2.38 mm)	16 – 28
# 200 (0.074 mm)	8 – 12

Compaction requires that rolling start immediately behind the paver. A breakdown roller (may be vibratory type) is followed by at least three passes of 12-ton (11 Mg), non-vibratory rollers. Finish rolling before the SMA temperature drops below 230º F (110º C).

2.2.5 Conventional Overlays Using High Quality Mix Design Methods and Materials

This brief discussion provides recommendations derived, conceptually, from the Chapter 1 section titled "Aiming at Rut-Resistant Materials" and the performance history of standard asphalt concrete materials in Anchorage and Juneau, Alaska. Unlike the previously described methods, the one described here simply involves modifying DOT&PF's present mix design practice and materials selection rather than trying to achieve a new form of asphalt concrete.

Apply standard Marshall asphalt concrete mix design methods and the Alaska DOT&PF Construction Standard Specification 401 with the following exceptions:

1. Select a high quality aggregate based on a Nordic Abrasion Value of = 7.
2. Use the appropriate Superpave performance-graded asphalt cement, and add additional requirements for Softening Point, Toughness, and Tenacity. A Superpave-type PG 58-28 is very similar in nature to standard AC-5 asphalt cement modified with the addition of 3% SBS additive. Refer to Table 11 for the necessary PG grading specification additions (courtesy DOT&PF Central Region Materials Engineer).

Table 11: Recommended Modifications for PG-Grade Asphalt Cement

	Performance Graded Asphalt Cement Type		
	PG 52-28	**PG 58-28**	**PG 64-28**
Softening Point (min.) AASHTO T-53	(none)	120° F (49º C)	125° F (52º C)
Toughness (min.) AASHTO D5801	(none)	9.1 ft-lb (12.4 N-m)	9.1 ft-lb (12.4 N-m)
Tenacity (min.) AASHTO D5801 (9-18-00) R244M98	(none)	6.3 ft-lb (8.5 N-m)	6.3 ft-lb (8.5 N-m)

2.3 Estimating Alaskan Costs

Unit costs (installed material) for the various rut repair options were obtained from numerous information sources that included the literature as well as personal contacts with government agencies, producers, suppliers, and trade organizations. Alaska DOT&PF sources supplied cost information for standard asphalt concrete, asphalt cement (standard and polymer modified), and informed speculation regarding premium aggregate costs. The R.S. Means "Heavy Construction Cost Data" [38] manual provided estimates of cost differentials between Alaska and locations in the continental U.S.

Table 12 summarizes these costs and contains other useful information regarding application and the estimated performance life of each option. Since several of the treatments addressed in Table 12 have never been tried in Alaska (Whitetopping, Micro-surfacing, and Ultra-Thin Bonded Wearing Courses) cost estimates are, of course, subject to argument. Of necessity, such cost estimates are also subject to substantial revision with time. Be aware that such factors as revised costs, improvements in equipment and changes in materials requirements may completely change the relative economic ordering of the various rut repair options. Methods for comparing the relative economics of various options involve life-cycle cost analysis and optimization. These techniques are discussed in Chapters 3 and 4.

For Table 12, "super aggregate" is defined as aggregate suitable for use in pavements subjected to ADTs = 10,000. Such materials would meet either one of the following values:

- Nordic Abrasion test requirement for a Class I material, i.e., an Abrasion Value of = 7
- SRK test requirement for a Pavement Class 1 material, i.e., an SRK Value of = 25

The few samples of normal Alaskan aggregates subjected to these tests met Scandinavian criteria for ADT levels of about 2,500 to 5,000, i.e., about 25 to 50% the level of the proposed super aggregate material.

Table 12: Rut Repair Methods & Costs

Method	Action	Unit Cost, $/yd^2-inch ($/m^2-25 mm)	Time Before Traffic	Expected Life (years)	General Comments
Micro-surfacing	Repair	$6.80 A* ($8.16A) $13.50 J** ($16.20 J)	1 hour	7	Smooth surface. Can be used to fill ruts as deep as about 1.5" (38 mm).
Micro-surfacing w/Super Aggregate	Repair	$9.20 A ($11.04 A) $15.25 J ($18.30 J)	1 hour	15	Same as above.
NovaChip®	Repair	$6.80 A ($8,16 A) $13.50 J ($16.20 J)	when cool	7	Open surface texture. Maximum rut fill about 0.5" (12.7 mm) w/ nominal 0.75" (19 mm) overlay.
NovaChip® w/Super Aggregate	Repair	$9.20 A ($11.04 A) $15.25 J ($18.30 J)	When cool	15	Same as above.
Ultra-Thin Whitetopping	Repair	$7.00 A ($8.40 A) $14.00 J ($16.80 J)	8 – 12 hours	10+	Smooth/rough surface texture options. Can be used to fill deep ruts. Must be placed over minimum 3" (75 mm) of asphalt concrete pavement.
Ultra-Thin Whitetopping w/Super Aggregate	Repair	$8.40 A ($10.08 A) $15.00 J ($18.00 J)	8 – 12 hours	15+	Same as above.
Stone Matrix Asphalt (SMA)	Repair	$3.90 A ($4.68 A) $5.60 J ($6.72 J)	When cool	7	Open surface texture. Maximum rut fill about 0.5" (12.7 mm) w/ nominal 1.5" (38 mm) overlay.
SMA w/Super Aggregate	Repair	$6.20 A ($7.44 A) $7.30 J ($8.76 J)	When cool	15	Same as above.

Table 12: Rut Repair Methods & Costs

Method	Action	Unit Cost, $/yd^2-inch ($/m^2-25 mm)	Time Before Traffic	Expected Life (years)	General Comments
Conventional Asphalt Concrete	Repair	$2.20 A ($2.64 A) $4.30 J ($5.16 J)	When cool	4	Smooth surface. Maximum rut fill about 0.5" (12.7 mm) w/ nominal 1.5" (38 mm) overlay.
Conventional Asphalt Concrete W/3% SBS	Repair	$2.80 A ($3.36 A) $4.50 J ($5.40 J)	When cool	7	Same as above.
Conventional Asphalt Concrete w/3% SBS + Super Aggregate	Repair	$5.00 A ($6.00 A) $6.20 J ($7.44 J)	When cool	15	Same as above.
Cold-Mill Planer (surface milling)	Prepare Surface	$1.00 A ($1.20 A) $1.60 J ($1.92 J)	N/A	N/A	N/A

*A = Anchorage, **J = Juneau

During the course of this study it became apparent that large variations in component costs of paving materials may exert a relatively minor influence on the total, as-placed cost of those materials. It is worth considering the affect of differing aggregate costs on the total cost per yd^2 –inch (or per m^2 –25 mm) of, for example, normal polymer-modified hot mix asphalt concrete in Alaska. Table 13 shows how a large increase in the cost of asphalt concrete aggregate causes a disproportionately small rise in the total cost of the hot mix.

Table 13: Aggregate Cost Versus Total In-Place Cost of Polymer-Modified HMA

Cost of Aggregate, Per Ton (Per Mg)	Cost of Asphalt Concrete, Per Ton (Per Mg)	Cost Multiple for Aggregate Per Ton (or Per Mg) of Aggregate	Cost Multiple for Hot Mix Per Ton (or Per Mg) of Hot Mix
$8.00 ($8.81)	$2.80 ($3.08)	1.0	1.0
$16.00 ($17.62)	$3.23 ($3.56)	2.0	1.2
$50.00 ($55.07)	$5.04 ($5.55)	6.0	1.8
$80.00 ($88.11)	$6.64 ($7.31)	10.0	2.4

Basic costs shown in the above table (top row of values) are those expected in Anchorage, Alaska using ordinary aggregate and polymer-modified asphalt cement. The cost of polymer-modified asphalt concrete is about $50.00/ton ($55.07/Mg), i.e., about $2.80/yd^2-inch ($3.35/m^2-25 mm) (as indicated in Table 12). At ~$40 - $50/ton (~$44 - $55/Mg), very high quality aggregate can be imported to Juneau or Anchorage (respectively) from Washington State. Such materials should meet "super aggregate" requirements described previously as characterized by Scandinavian test methods and therefore withstand perhaps 2 to 4 times the studded tire applications of normal Alaskan aggregates (according to Scandinavian ADT criteria and testing done on Alaskan aggregates). Assuming only a doubling of allowable studded tire passes—a very conservative assumption—is the extra cost of the aggregate justified? Table 13 shows the cost of polymer-modified asphalt concrete containing $50.00/ton ($55/Mg) aggregate (termed "super aggregate" in Table 12) to be less than twice the cost of modified asphalt concrete containing standard aggregate. With respect to studded tire wear, the additional aggregate cost is certainly justified.

A breakdown of the $2.10/yd^2-inch ($2.51/m^2-25 mm) base cost for polymer-modified asphalt concrete may be of interest. The polymer modification normally used in Alaska at the present time consists of Styrene-Butadiene-Styrene (SBS) additive at 3% by total weight of modified asphalt cement. The unit weight of the hot mix is assumed at about 150 lb/ft^3 (2.4 Mg/m^3), i.e., about 113 lbs/yd^2-inch (61.4 kg/m^2-25 mm). The hot mix contains about 5.5% asphalt cement (by total weight of the hot mix). The cost of the polymer-modified asphalt cement is about $225.00/ton ($248.00/Mg), i.e., about $0.70/yd^2-inch ($0.84/m^2-25 mm). The cost of the standard aggregate component in this material is about $8.00/ton ($8.81/Mg), i.e., about $0.43/yd^2-inch ($0.51/m^2-25 mm). Besides aggregate and the modified asphalt cement, the remainder of the $2.10/yd^2-inch ($2.51/m^2-25 mm) asphalt concrete cost goes to labor, overhead and profit.

3 - CHAPTER 3 – LIFE-CYCLE COSTS BASED ON PRESENT VALUE ESTIMATES

The following discussion of life-cycle cost analysis (LCCA) is from an FHWA bulletin titled "Life-Cycle Cost Analysis in Pavement Design." [39] Life-cycle cost analysis (LCCA) is founded on the principals of economic analysis that are used to evaluate competing alternative investment options. The objective is to identify the best value, i.e., the lowest long-term cost option that satisfies required performance requirements. In the most general form it incorporates all initial and discounted future agency costs, user costs, and any other relevant costs over the life of the alternative investments. Within the framework of LCCA, present value analysis (sometimes called present worth analysis) is one of the most common ways of summing all costs for a given option, regardless of when the cost occurs during the LCCA analysis period. In present value analysis, all costs associated with a specific option are reduced to a single value (the present value) at the beginning of the LCCA analysis period. LCCA then compares the present values of each option to determine which one is most economical.

The following two sections discuss applications of LCCA principals to the cost data contained in Table 12. The first section provides a simplified example of LCCA analysis. The next section briefly discusses the more general form of LCCA and points out problems associated with the simplified approach. The general LCCA method considers user costs in addition to materials and construction-related costs. A generalized LCCA often greatly accentuates cost differences between options, or may cause a reshuffling of the options in terms of relative economic benefit.

3.1 Simple Present Value Estimates Based on Agency Costs of Repairs & Timing

This section provides examples of present value life-cycle costs of alternatives using only agency cost components from Table 12. This method corresponds to that often used by the Alaska DOT&PF since it does not directly consider user costs. User costs, if considered, are usually treated subjectively, and not computed as specific elements of the total life-cycle present value.

The following example compares the present value of four repair scenarios. These scenarios are only a small sampling of the many combinations of repair strategies that *could* be generated based on Table 12. This example uses an interest rate (discount rate) of 4% and an analysis period of 35 years. A 35-year analysis period is based on the minimum 30 to 40 year analysis period recommended in the FHWA bulletin cited above. In the bulletin, FHWA also recommends a 3 to 5% *real* discount rate (interest rate) because it reflects the historic average for a non-inflated return on investment. A *real* interest rate of 4% is used in the Table 14 examples below. Finally, the FHWA bulletin recommends that future actions (future costs) should be estimated in constant dollars and discounted to a present value using the *real* interest rate. This means that the estimated cost of some future maintenance or reconstruction action remains the same as today's estimated cost throughout the entire analysis period. The present value (Net Present Value) computation as follows:

Equation 1: Net Present Value Computation.

$$NPV = \text{Initial Cost} + \sum_{k=1}^{N} \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right]$$

where: i = discount rate and

n = year of expenditure

The total required summation of all cost events (through "*N*" cost events) includes present values for each cost event (*k*) incurred during the 35-year (minimum) analysis period.

Treatment of Salvage Value—If salvage value exists at the end of the analysis period, (35 years in the following examples), the summation includes the "cost" of the present value of salvage, calculated from year 35, as a negative value. This treatment of salvage value follows the FHWA recommendation that "salvage value should be based on the remaining life of an alternative at the end of the analysis period as a prorated share of the last rehabilitation cost".

The repair unit costs are Anchorage estimates from Table 12 (marked "A"). That table also lists the expected performance life for each treatment. Tables 14a through 14d present descriptions and costs in terms of \$/lane-mile (lane-mile assumed @12 ft by 5,280 ft, i.e., 7,040 yd^2). The lane-mile units are for illustrative purposes and therefore do not include shoulder widths.

The following is an example of present value computations used for Table 14b. Table 14b is used because it contains a salvage value:

Calculation for present value for 3rd row of Table 14b (end of year 15) —

1. Using a cost at time of action = \$40,500, i = 4%, and n = 15, Equation 1 yields a present value for the rehabilitation action of \$40,500 x 0.556 = \$22,500

Calculation for present value of salvage for 5th row of Table 14b (end of year 35) —

1. A salvage of 10/15ths of the 15-year life of the last maintenance action (an action performed at year 30) remains at year 35. Therefore, calculate a prorated value for the last rehabilitation cost of (10/15) x \$40,500 = \$27,000.

2. Then, using the prorated cost at time of action = \$27,000, i = 4%, and n = 35, Equation 1 yields a present value for the salvage of \$7,000 (rounded from \$6,842). Note that salvage becomes a negative value in the following tables, i.e., a negative cost in the final summing of all present values in Equation 1.

Note: $/lane-km units are also provided in the following tables (where the lane width is assumed to be 3.66 m, i.e., 12 ft). A lane-km of pavement is assumed to cover about 3,658 m^2, i.e., about 4,375 yd^2.

Table 14a: Scenario No. 1 Present Value Life-Cycle Cost Analysis

Time of Action	Action	Cost at Time of Action $/lane-mile ($/lane-km)	Present Value of Action $/lane-mile ($/lane-km)	Cumulative Present Value of Action $/lane-mile ($/lane-km)
Start of Year 1	Micro-surfacing rut fill with a 3/8" (10 mm) thick, lane-width overlay (estimate 5/8" (16 mm) total thickness including rut fill)	$30,000 ($18,600)	$30,000 ($18,600)	$30,000 ($18,600)
End of Year 7	Same as above	$30,000 ($18,600)	$23,000 ($14,300)	$30,000 + $23,000 = $53,000 ($32,900)
End of Year 14	Same as above	$30,000 ($18,600)	$17,500 ($10,900)	$53,000 + $17,500 = $70,500 ($43,800)
End of Year 21	Same as above	$30,000 ($18,600)	$13,000 ($8,100)	$70,500 + $13,000 = $83,500 ($51900)
End of Year 28	Same as above	$30,000 ($18,600)	$10,000 ($6,200)	$83,500 + $10,000 = $93,500 ($58,100)
End of Year 35	Estimate salvage	No salvage value	$ 0.00	**$93,000 + $0.00 =$93,500 ($58,100) Total to year 35**

Table 14b: Scenario No. 2 Present Value Life-Cycle Cost Analysis

Time of Action	Action	Cost at Time of Action $/lane-mile ($/lane-km)	Present Value of Action $/lane-mile ($/lane-km)	Cumulative Present Value of Action $/lane-mile ($/lane-km)
Start of Year 1	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$10,500 ($6,500)	$10,500 ($6,500)
Start of Year 1	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS + super aggregate	$53,000 ($32,900)	$53,000 ($32,900)	$63,500 ($39,400)
End of Year 15	Micro-surfacing w/super aggregate rut fill with a 3/8" (10 mm) thick, lane-width overlay (estimate 5/8" (16 mm) total thickness including rut fill)	$40,500 ($25,200)	$22,500 ($14,000)	$86,000 ($53,400)
End of Year 30	Same as above	$40,500 ($25,200)	$12,500 ($7,800)	$98,500 ($61,200)
End of Year 35	Estimate salvage @ 10/15 x $40,500 = $27,000 (10/15 x $25,200 = $16,800) *Note: This is a negative cost.*	-$27,000 (-$16,800)	-$7,000 (-$4,400)	**$91,500 ($56,800) Total to year 35**

Table 14c: Scenario No. 3 Present Value Life-Cycle Cost Analysis

Time of Action	Action	Cost at Time of Action $/lane-mile ($/lane-km)	Present Value of Action $/lane-mile ($/lane-km)	Cumulative Present Value of Action $/lane-mile ($/lane-km)
Start of Year 1	Micro-surfacing w/super aggregate rut fill with a 3/8” (10 mm) thick, lane-width overlay (estimate 5/8” (16 mm) total thickness including rut fill)	$40,500 ($25,200)	$40,500 ($25,200)	$40,500 ($25,200)
End of Year 15	Same as above	$40,500 ($25,200)	$22,500 ($14,000)	$63,000 ($39,200)
End of Year 30	Same as above	$40,500 ($25,200)	$12,500 ($7,800)	$75,500 ($47,000)
End of Year 35	Estimate salvage @ 10/15 x $40,500 = $27,000 (10/15 x $25,200 = $16,800) *Note: This is a negative cost.*	-$27,000 (-$16,800)	-$7,000 (-$4,400)	**$68,500 ($42,600) Total to year 35**

Table 14d: Scenario No. 4 Present Value Life-Cycle Cost Analysis

Time of Action	Action	Cost at Time of Action $/lane-mile ($/lane-km)	Present Value of Action $/lane-mile ($/lane-km)	Cumulative Present Value of Action $/lane-mile ($/lane-km)
Start of Year 1	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$10,500 ($6,500)	$10,500 ($6,500)
Start of Year 1	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS	$30,000 ($18,600)	$30,000 ($18,600)	$40,500 ($25,100)
End of Year 7	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$8,000 ($5,000)	$48,500 ($30,100)
End of Year 7	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS	$30,000 ($18,600)	$22,500 ($14,000)	$71,000 ($44,100)
End of Year 14	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$6,000 ($3,700)	$77,000 ($47,800)
End of Year 14	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS	$30,000 ($18,600)	$17,500 ($10,900)	$94,500 ($58,700)
End of Year 21	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$4,500 ($2,800)	$99,000 ($61,500)
End of Year 21	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS	$30,000 ($18,600)	$13,000 ($8,100)	$112,000 ($69,600)
End of Year 28	Mill 1.5" (38 mm) of existing pavement	$10,500 ($6,500)	$3,500 ($2,200)	$115,500 ($71,800)
End of Year 28	Place 1.5" (38 mm) thick hot mix asphalt concrete w/ 3% SBS	$30,000 ($18,600)	$10,000 ($6,200)	$125,500 ($78,000)
End of Year 35	Estimate salvage	No salvage value	$0.00	**$125,500 ($78,000) Total to year 35**

Results of the example show the following ranking:

1. Best Scenario No. 3 at $68,500/lane mile ($42,600/lane-km), Relative Cost Factor = 1.0
2. Scenario No. 2 at $91,500/lane-mile ($56,800/lane-km), Relative Cost Factor = 1.3
3. Scenario No. 1 at $93,500/lane-mile ($58,100/lane-km), Relative Cost Factor = 1.4
4. Worst Scenario No. 4 at $125,500/lane-mile($78,000/lane-km), Relative Cost Factor = 1.8

Scenario 3 requires three applications of micro-surfacing at 15-year intervals, at a total cost of less than 2/3 that of scenario 4—a substantial savings. This apparent savings is only the tip of the iceberg though. Note that scenario 3 requires three construction events versus five for scenario no. 4. It should be obvious without calculation that two less construction projects means substantial savings in terms of design work, construction mobilization/ demobilization, construction management, construction-site accidents, construction site environmental problems, and the many costs associated with the highway users and their vehicles.

3.2 Advanced Present Value Estimates Based on Agency Costs of Repairs, User Costs, Sensitivity Analysis & Probability [39,40]

User Costs—A more valid form of present value analysis considers agency costs plus as many of the user costs as can be realistically determined. Agency costs are those associated with materials, construction and significant maintenance actions throughout a defined life-cycle period. The present value analysis in the above section used only agency costs. User costs are defined for a particular section of roadway under construction, during each construction event. They are the costs incurred by the owners/operators as a result of using that particular section of roadway.

User costs include: vehicle operating costs, crash costs, and user delay costs associated with each repair project. In projects where traffic flow is seriously impeded, delay costs are arguably the largest and most easily definable of the user costs [39]. At the low end of the scale, the time delay cost for passenger vehicles is estimated between $10 and $13/hour. At the top of the scale are the time delay costs for large trucks estimated between $21 and $24/hour. In 1998, the author attended an FHWA life-cycle cost estimating class where the instructors stated that, for some highway projects, user costs are estimated to have exceeded the cost of the project plus the estimated value of project benefits for the design-life of the project. Also, potential user costs can grow exponentially with time due to highway crowding caused by population increases. User costs shouldn't be ignored. They can be a significant and perhaps critical cost contributor whenever a particular repair scenario requires a relatively large number of construction events over an LCCA analysis period.

Sensitivity Analysis—Analyzing the influence of key variables in the present value calculations is a time consuming but justifiable part of a good LCCA. The principle of sensitivity analysis is to assign a range of values to one of the variables. Present value computations are done using, in turn, each new value of the variable while holding all other variables constant. Variables often tested include timing of construction events, and the discount rate. Sensitivity analyses might also be a useful way of investigating variations in construction scheduling that would affect user costs.

Use of Probability (Risk Analysis)—Risk analysis can be considered as sensitivity analysis taken to the highest possible degree. Risk analysis assumes that some or all of the variables used in a present value analysis can assume any one of many values within a definable range. Many if not most variables used in present value analyses can be characterized as possessing a statistical distribution of expected values that can be defined by a mean and standard deviation. Using a technique known as Monte Carlo Simulation [39,41], it is possible to randomly draw a sample value for each variable (from within each variable's statistical distribution). A present value computation is then performed using the new set of values. On a computer, Monty Carlo Simulation is repeated a very large number of times, thus supplying new sets of variables for a very large number of present value computations. After the present value computation is repeated enough times, the present value itself will exhibit a distribution of values that gives the engineer a realistic look at the "real" range of costs possible for each repair scenario. Handle this kind of analysis using spreadsheet add-in programs such as @RISK and Crystal Ball [39] (can be used with Microsoft Excel spreadsheets).

4 - CHAPTER 4 – OPTIMIZING THE SELECTION OF RUT REPAIR TREATMENTS & PAVEMENT MANAGEMENT METHODS

In a much higher level of economic analysis the cost of rut repairs is placed within the context of repairing all pavement damage types while staying within required economic constraints. Pavement management systems (PMS) perform this function. Conceptually they simultaneously examine the competing economics of repairing each damage type at different points in time. Haas, Hudson and Zaniewski offer comprehensive coverage of PMS principals in a textbook format [42].

A PMS usually first requires assessing the condition of a particular roadway section (or an entire roadway system, i.e., road network) in terms of several damage types such as rutting, cracking, roughness, flushing, raveling, etc. Then, based on the present condition, and perhaps information regarding the structural composition of various pavement sections, pavement management computer programs predict the worsening of the various damage types with time. A good PMS works from a large catalog of options that cover repairs of numerous damage types. Such a catalog can include the rut repair options covered earlier in this report. The PMS is able to estimate the performance life of each option, through the programming period, with respect to all damage types, for expected levels of traffic. The PMS then selects repair methods and timing that usually conform to one or the other (selected by the agency) of two economic constraints:

1. Requires that the roadway section or system be kept at the highest possible level of condition within a maximum budget limit.
2. Requires that the roadway section or system be kept above a specific condition level at the lowest cost.

To reiterate, the main point is that a PMS juggles the economics of repairing all damage types simultaneously, and then optimizes the choice of repair options.

It is worth discussing a specific pavement management computer program since it has been licensed for use by the Alaska DOT&PF, and can be used to make project-level decisions. Dynatest Inc. created the "Performance and Economic Rating System" (PERS) program. It is capable of selecting repair methods from a large catalog of options, and can be applied at either the project level or for an entire road network [43,44]. Although the Alaska DOT&PF presently uses another Dynatest product, The "Dynatest Management System (DMS)," as the basis for its statewide pavement management system, PERS is a completely separate program that can be run independently as a tool for project level work. The following information is extracted, with permission, from Dynatest literature.

"*The three elements of PERS are:*

- *Models for predicting (or forecasting) the pavement performance based on mechanistic (analytical) principals*
- *Models for quantifying the economic effects of pavement conditions*
- *Methods for selecting the optimal combination of maintenance and rehabilitation alternatives over a number of budget years (optimization)*"

As explained in the Dynatest literature: "*PERS makes use of an incremental-recursive approach for calculating pavement performance. For each increment of time (normally one season) the damage caused by traffic loading and by time related effects is calculated, and the new pavement condition is then used recursively as input for the next time increment.*"

PERS can be run on a desktop computer, and uses the same kind of data files as the statewide DMS system. PERS looks at structural deterioration, rutting, roughness, skid resistance and surface wear (caused by studded tires) as criteria for selecting repair methods. PERS adjusts the models used for estimating future damage based on actual observations in the field, i.e., the models are self-training to some extent. The program estimates user costs during the service life of the applied repair options. It also calculates agency costs in addition to fluctuations in the capital value due to pavement improvement or deterioration.

5 - CONCLUSIONS

Literature sources indicate five (5) methods that may have greatest applicability for repairing rut-damaged pavements in Central and Southeast Regions of Alaska. Methods discussed in this report are directed at repairing ruts due to plastic deformation of the asphalt concrete layer and/or tire-stud abrasion of the pavement surface. These methods are:

- Micro-Surfacing
- Ultra-Thin Bonded Wearing Course—NovaChip®
- Ultra-Thin Bonded Wearing Course—Ultra-Thin Whitetopping (UTW)
- Stone Matrix Asphalt (SMA)
- Conventional Overlays Using High Quality Mix Design Methods and Materials

Only SMAs and polymer-modified conventional asphalt concrete mixes have been tried in Alaska in an attempt to reduce rutting. Performance observations so far indicate moderate success. DOT&PF engineers specializing in pavement design and materials speculate that much greater success might be obtained if higher quality (harder) aggregates are used. The other three methods have not been tried in Alaska, although there may be significant economies of materials inherent in using thin, rut-resistant layers to repair rutted pavements.

Of the methods listed above, UTW seems least attractive in Alaska at the present time. To achieve a high level of performance the UTW system should be placed on a 3-inch (75-mm) thick high-modulus asphalt concrete. The required substantial asphalt concrete supporting layer plus the normal UTW thickness of 2 to 4 inches (50 to 100 mm) would eliminate the UTW option for almost all Alaskan pavements.

Cost estimates for each of the methods are presented in the report. Understand that these cost figures are intended to provide the reader with some degree of economic insight. They were derived as a reasonably estimated “snapshot” in time. Do not expect to see these particular values show up as actual bid prices on a particular project. And of course the relative cost picture may also change substantially at a future time. Such changes will follow changes in the availability of special materials as well as the maturity and more common use of one or more of the technologies.

Various methods of life-cycle cost estimation, ranging from very simple to complex, are available for deciding which repair option to select for a particular location. First, however, it will be necessary to determine, by field experiment, which of the repair options are viable given Alaskan conditions.

A few words of caution are in order about projecting cost elements of small-scale experimental sections to life-cycle costs for full-scale projects. Small-scale field trials often imply a substantial cost penalty inherent in the use of small quantities. The reverse can be true (more rare) if a materials supplier or contractor is anxious to generate interest in a particular technology, and is willing to subsidize costs of the experimental section. Neither pricing situation provides useful information for future “real” projects.

6 - RECOMMENDATIONS

Conduct Small-Scale Tests (Experimental Phase of Evaluation)—Establish the performance viability of each repair option by constructing experimental sections in the Anchorage and Juneau areas. These experimental sections can be installed as part of a normal construction project under the *Experimental Features in Construction* program. Contact the DOT&PF Research Section for more information about initiating, monitoring, and reporting requirements for experimental features done under this program. The possible exception to this recommendation is perhaps the UTW option since most Alaskan pavements do not provide adequate support for this type of overlay. Micro-surfacing, NovaChip, and SMA type repairs appear to show promise. SMAs have been tried in Alaska and have not exhibited impressive performance. Therefore, SMA pavements recommended here should be designed to incorporate very hard aggregates. Also include test sections of conventional asphalt concrete with the aggregate and asphalt cement modifications suggested in this report. If the performance of the test sections is not carefully documented, life-cycle cost calculations for future projects will not be valid! The DOT&PF Research Section will supervise and archive all performance reporting done for *Experimental Features in Construction* projects. Evaluate the performance of each experimental section for a minimum of two to five years—the longer the better.

Meaningful construction and performance evaluations for each rut repair technique require that each be constructed to some usefully representative minimum length. In other words, sections that are too short may not lead to generally useful conclusions regarding construction problems or performance. The recommended minimum length for each repair treatment is 200 feet (60 m), although section lengths of 500 feet (150 m) or longer would be ideal.

Conduct Full-Scale Repair Trials (Implementation Phase of Evaluation)—Implementation will involve applying one or more rut repair methods (selected after phase-1 evaluations) to the entirety of a 3-R project. The intention here is to apply repair methods to sections large enough that bid prices will reflect the use of large materials quantities.

After establishing the performance viability of the repair options as *Experimental Features in Construction*, select several viable rut repair options for potential implementation. Produce competing candidate designs and specifications, i.e., bid options for a moderate to large size 3-R design project. The contractor bid package will include solicitations for construction bid prices for all options. The designer will use submitted bid prices to evaluate the relative cost of each rut repair option using life-cycle cost methods for each rut repair option. A realistic life-cycle analysis will be possible only if the performance of each option can be realistically modeled (how long will each repair treatment last?), and actual bid prices are used. Repair section lengths of 1 mile (1.6 km) or more are appropriate for this stage of implementation.

Although these are likely not experimental sections per se, formal performance evaluations should be done. The project designer and/or DOT&PF Research Section staff member should be designated to evaluate the performance of each experimental section for a minimum of two to five years. Assign the job of doing performance evaluations to a specific individual(s), and set minimum requirements for documentation, or the monitoring will not be done. Formal reporting should not be necessary—annual performance descriptions (with photos) should suffice. The DOT&PF Research Section should archive copies of all performance documentation. Again, the longest possible evaluation period is desirable.

In General—Avoid rejecting the use of high quality asphalt concrete components out of hand. This may be a false economy. The report describes an example where expensive, very high quality aggregates could significantly improve pavement performance while increasing the total cost of the paving mixture a relatively small amount. The example reveals that a five-fold increase in the cost of aggregate produces only a 60% increase in the hot mix price. At a fractional cost increase for the mix, such aggregate may increase the ESAL capacity of the pavement by a factor of three to four with respect to rutting—a good value. False economies can be exposed in the light of a good life-cycle analysis where user costs and the time-value aspects of improved long-term performance are realistically modeled.

APPENDIX A

Research in Progress

Table A-1 summarizes research efforts reported as active by the Transportation Research Board as of August 2001. The projects listed below are those that appeared most closely associated with various rut repair technologies.

Table A-1: Current Research.

Agency *	Project Title/Description	Contact
Alabama	"Evaluation of the NovaChip process in Alabama" / Investigates NovaChip construction process and evaluates performance at regular intervals.	Holman, F., 205-242-6539
California	"Micro-Surfacing Mix Design Procedure" / Establish optimal mix components and predict performance.	Mann, Gary, 916-227-7049
Colorado	"Development of Design Guidelines for Thin Whitetopping overlays" / Develop new guidelines based on observations from existing whitetopping projects.	Ardani, Ahmad, 303-757-9978
Colorado	"Validation of the Thin Whitetopping Design Procedures in Colorado" / Part of FHWA national effort to validate new procedures for designing and constructing whitetopping pavements. Existing whitetopping will be examined in the field and laboratory studies will be conducted.	Ardani, Ahmad, 303-757-9978
Colorado	"Wearing Surfaces" / Investigating longer lasting wearing surfaces to be used for pavement rehabilitation.	Harmelink, D., 303-757-9518
Colorado	"Stone Mastic (Matrix) Asphalt Flexible Pavements" / Compare performance of SMA with conventional pavements. Will look at life-cycle costs, polymer additives, and a particular cellulose fiber additive.	Harmelink, D., 303-757-9506
Florida	"Field Assessment and Analytical Modeling Ultra-Thin whitetopping" / Assess rehabilitation strategy of laying whitetopping over old pavement.	Tawfig, K., Tallahassee, Fl., No telephone number provided
Illinois	"Ultra-Thin Whitetopping of Pavements" / Will document performance of whitetopping pavements in Illinois.	Volle, Tessa, 217-782-7200
Illinois	"Evaluation of Stone Matrix Asphalt In-Situ" / Determine constructability and applicability of SMAs using typical Illinois materials and construction practices.	Rademaker, M., 217-782-1056
Indiana	"Concrete Overlays as a Maintenance Option for Distressed Asphalt Intersections" / Investigate factors affecting performance of concrete overlays. Ultra-thin whitetopping (UTW) will be tested using slow-moving loads under laboratory conditions to determine performance.	Partridge, Barry, 765-463-1521
Iowa	"Bond Enhancement Techniques for PCC Whitetopping" / Determine techniques that will ensure the bond between the old asphalt pavement and the whitetopping overlay.	Harris, G., 515-239-1382

Table A-1: Current Research.

Agency *	Project Title/Description	Contact
Kansas	"Evaluation of Rutting Potential of Superpave Mixtures Using the Asphalt Pavement Analyzer" / Correlate laboratory wheel-load testing of asphalt concrete specimens with field measurements of rut depth for the same mixes. Develop test method for evaluating rut potential of Kansas asphalt concrete pavements.	Fager, Glenn, 785-291-3843
Louisiana	"Laboratory Evaluation of Stone Mastic (Matrix) Asphalt Pavement Mixtures" / Evaluate SMA pavements using local materials and investigate influence of fiber additives and asphalt cement modifiers on SMA performance.	Paul, H., 504-767-9124
Michigan	"Evaluation of Whitetopping as a Pavement Rehabilitation Technique" / Determine cost effectiveness of a PCC overlay for rehabilitating asphalt concrete pavements. Study will be based on observations of actual constructed projects.	No contact information
Mississippi	"Evaluation of E-Krete for Filing Ruts" / E-Krete is a locally produced PCC material that may find application as a type of whitetopping. The material will be tested as a rut filler on a section of asphalt concrete pavement.	Battey, R., 601-359-7650
Missouri	"Ultra-Thin Whitetopping" / Construct whitetopping test sections in Missouri. Evaluate the performance of the newly overlaid sections.	Cook, N., 573-526-4320
North Carolina	"Thin Bonded Overlay and Surface Laminates for Pavements and Bridges" / Place and evaluate whitetopping overlays on several roads and bridges, then evaluate performance.	Biswas, M., 919-715-2465
North Dakota	"Micro-surfacing—A Rut Resisting Material Used as an Asphalt Rut Filler" / Evaluate micro-surfacing as an effective rut filler and monitor its ability to resist further rutting.	Kuntz, C., 701-221-6910
Oregon	"Repair of Rutting Caused by Studded Tires" / Conduct literature review and perform laboratory testing (using Scandinavian test methods) to determine best aggregate for producing SMA pavements.	Edgar, R., 503-986-2846
South Carolina	"Ultra-Thin White Top for Distressed Intersections" / Evaluate performance of whitetopping at intersections that exhibit shoving and rutting problems.	No specific contact person, 803-737-6687
Wisconsin	"Evaluation of Stone Mastic (Matrix) Asphalt (SMA)" / Evaluate a range of different SMA types. Will evaluate both organic and inorganic fibers as well as plastic and elastomer additives. Evaluate constructability and performance.	Schmeidlin, R., 608-246-7950
Wisconsin	"Performance Evaluation of Rut Resistant Asphaltic Concrete Pavement Overlays in Wisconsin" / Monitor long term performance of rut-resistant asphalt concrete mixtures and compare performance with standard asphalt concrete pavements.	Okpala, D.C., 608-246-7953
National Center for Asphalt Technology (NCAT)	"Evaluation of Fine (4.75mm) SMA Mixes" / Evaluate and develop mix design procedures for fine SMA mixes.	No specific contact

Table A-1: Current Research.

Agency *	Project Title/Description	Contact
NCAT	"Evaluation of Field Performance of SMA and Superpave Pavements" / Evaluate SMA and Superpave pavements for performance throughout the U.S.	No specific contact
National Cooperative Highway Research Program (NCHRP)	"Relationship Between Superpave Gyratory Properties and Permanent Deformation of Pavements in Service" (NCHRP 9-16) / Evaluate use of Superpave gyratory compactor to predict rutting.	Anderson, The Asphalt Institute, No additional contact information
NCHRP	"Accelerated Laboratory Rutting tests: Asphalt Pavement Analyzer" (NCHRP 9-17) / Evaluate Asphalt Pavement Analyzer tests for predicting rutting potential of asphalt concrete pavements.	Kandhal, NCAT, No additional contact information

* State Department of Transportation agencies unless otherwise noted.

APPENDIX B

Some Basic Information Used for Estimating Costs

In 1995 the Oregon Department of Transportation published a literature review covering the repair of ruts caused by studded tires [45]. The Oregon report proved especially useful to this literature review. It was the only research document that contained a fairly complete comparison of costs for various rut repair scenarios that represent state-of-the-art methods and materials. The information has been updated wherever possible based on general literature sources available as of this writing.

Rule of Thumb: The minimum placement thickness of all asphalt/aggregate mixtures is about 1.5 to 2 times the largest aggregate size (1.5 factor is used most often).

Key data used in constructing portions of the Chapter 2 table titled "Rut Repair Methods and Costs" are:

- Unit weight of asphalt concrete-type materials is about 150 lb/ft^3 (2.40 Mg/m^3) (includes standard asphalt concrete, micro-surfacing, SMA and NovaChip)—according to various information sources.
- Unit weight of the Portland cement concrete used as Ultra-thin whitetopping is about 165 lb/ft^3 (2.64 Mg/m^3)—according to various information sources.
- Estimates of cold-mill planer (surface milling) costs for Juneau and Anchorage—according to Alaska DOT&PF sources.
- Regular asphalt concrete contains 5.5% asphalt cement (by total weight of mix)—according to various information sources.
- SMA contains 6.5% asphalt cement (by total weight of mix)—according to Alaska DOT&PF sources.
- Juneau standard asphalt cement @ $380.00/ton ($419.00/Mg), and asphalt cement w/3% SBS polymer additive @ $450.00/ton ($496.00/Mg) (estimated)—according to Alaska DOT&PF sources.
- Juneau standard asphalt concrete @ $55.00/ton ($61.00/Mg) + cost of asphalt cement—according to Alaska DOT&PF sources.
- Juneau asphalt concrete aggregate @ $10.00/ton ($11.00/Mg)—according to Alaska DOT&PF sources.
- Super aggregate in Juneau @ $40.00/ton ($44.00/Mg) (from Washington State) —according to Alaska DOT&PF sources.
- Anchorage standard asphalt cement (PG 52-28) @ $150.00/ton ($165.00/Mg), and asphalt cement w/3% SBS polymer additive (PG 58-28) @ $360.00/ton ($396.00/Mg)—according to Alaska DOT&PF sources.
- Anchorage standard asphalt concrete @ $30.00/ton ($33.00/Mg) + cost of asphalt cement—according to Alaska DOT&PF sources.
- Anchorage asphalt concrete aggregate @ $8.00/ton ($9.00/Mg)—according to Alaska DOT&PF sources.

- Super aggregate in Anchorage @ $50.00/ton ($55.00/Mg) (from Washington State) — according to Alaska DOT&PF sources.
- Costs for SMA [Anchorage: $45.00/ton ($50.00/Mg) and Juneau: $70.00/ton ($77.00/Mg)] were according to Alaskan DOT&PF sources.
- Costs for micro-surfacing and NovaChip were estimated according to literature sources and recommendations given by suppliers. Costs include estimated Alaskan adjustments.
- Costs of whitetopping were estimated according to literature sources, recommendations given by the American Concrete Pavement Association (ACPA) representatives, and quotes from Alaskan PCC suppliers.

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