MONTANA
MDT
DEPARTMENT OF TRANSPORTATION

PAVEMENT DESIGN MANUAL

2016

Materials Bureau

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Chapter 1 **Preface**

Pavement design is a complex process that requires the designer to analyze data and information from many sources. It is imperative that the designer examine data and design calculations for reasonableness. The guidance provided within this manual is intended to reduce subjectivity in the analysis and design process, but not initiate a "cookbook" approach. It does not nor could it, give inputs for every possible design situation. When the designer varies from these inputs it is important that the change be noted in the design file. This will allow future designers to recreate any given pavement design.

There was no attempt to make this surfacing manual a user's guide for the computer design programs used by the Surfacing Design Unit. There are user's manuals available for that purpose. Additionally, discussion regarding design theory is left to the reference manuals.

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Chapter 2 **Introduction**

Historically, pavement design has been an empirical procedure where engineering judgment and experience have played an integral part. In 1962 the American Association of State Highway Officials (AASHO) issued an interim design guide. This Guide presents an empirical pavement design method based upon the well-known AASHTO road test that took place in the 1950's in Ottawa, Illinois. Updates to the 1962 Guide are described below.

In 1986 the American Association of State Highway and Transportation Officials (AASHTO) issued a more comprehensive guide titled AASHTO® Guide for Design of Pavement Structures. This guide addresses a greater number of design parameters. The idea of mechanistic-empirical (M-E) design procedures was also introduced at this time. Personal computers began to replace nomographs in the design procedures and AASHTO introduced DNSP86, a computerized design procedure to be used in conjunction with the AASHTO® Guide for Design of Pavement Structures. Some state agencies began incorporating non-destructive testing (NDT) in to pavement design.

In 1991 AASHTO released a pavement design software package entitled Design Analysis and Rehabilitation for Windows (DARWin).

A new version of DARWin was released in 1993 along with a revised edition of AASHTO® Guide for Design of Pavement Structures (1993 Guide (AASHTO, 1993)). MDT currently uses a pavement design method based upon the 1993 Guide and DARWin software, with adjustments made based upon MDT's past experience and pavement materials.

Today, the pavement design industry is slowly migrating to M-E pavement design. AASTHO has endorsed a software package entitled AASHTO ME Pavement Design (AASHTO M-E). MDT did a comprehensive study (VonQuitus, 2007) to calibrate the pavement performance models found within a previous version of AASHTO M-E. Unfortunately, the study did not result in a reliable M-E design method. In addition, in the summer 2014, MDT's Surfacing Design Unit did a thorough comparison of the AASHTO 1993 method vs. MEPDG and came to a similar conclusion that at this time MDT is not ready to switch to MEPDG. MDT will continue to work towards utilizing AASHTO M-E or another M-E design method, but for the time being should continue utilizing this manual for routine pavement designs.

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Chapter 3 **Pavement Types**

Montana predominantly utilizes flexible pavements. MDT refers to this as plant mix surfacing (PMS). In rare instances, MDT uses rigid pavement commonly referred to as Portland cement concrete pavement (PCCP). Currently, flexible pavements make up about 97% of MDT's road system (excluding gravel roads).

A flexible pavement can consist of up to four layers -- subgrade, subbase (often omitted), base, and surfacing. These pavement layers are described as follows:

- Surfacing usually consists of PMS, but may consist of a double shot (double chip seal) on very low-volume roads. PMS typically is chip sealed, except when specifically designed to be less permeable such as 3/8" Grade S PMS.
- The base course usually consists of an untreated gravel base or cement-treated base.
- Subbase can consist of a sand surfacing, special borrow, or uncrushed gravel placed on the subgrade.
- Subgrade is the native material beneath the surfacing section.

Rigid pavement is PCCP placed upon a granular, stabilized base, or PMS. MDT currently utilizes jointed plain concrete pavement (JPCP). Rigid pavement is mostly used on roadways with the following:

- High annual daily traffic (ADT) and/or truck traffic,
- Reoccurring PMS rutting problems,
- Slow moving or stop-and-go traffic
- Signalized intersections
- Roundabouts

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Chapter 4 **MDT's Pavement Design Overview**

Effective pavement design is an important element of the overall roadway project design. The pavement is the portion of the highway most obvious to the motorist. From a project design perspective, pavement and related items comprise from 10 percent to 90 percent of highway construction costs. Due to these facts, the designer should not under design pavements, which may lead to:

- Increased user costs (fuel consumption, tires, repair, etc.)
- increased accident cost
- Increased maintenance costs
- Costs of user delays due to reconstruction and maintenance
- Cost of early reconstruction.

Pavement Design is both art and science. Precise pavement design and performance prediction is somewhere between difficult and impossible. The difficulty is due to a number of variables that are difficult to predict including:

- Pavement material properties that change due to climate and over time
- Truck traffic prediction over the 20-year design life
- Construction and Maintenance practices
- Subgrade soil properties

Pavement design should be conservative since the above variables are difficult to predict, but not overly conservative. The cost of overdesign can be substantial due to the high costs of pavement materials, and the fact that the costs of overdesign could be better used on another pavement.

It is believed that MDT's pavement design method has a practical amount of conservatism balanced with cost-effectiveness. It has been adjusted over the years to reflect the improvements made in construction practices, materials, and traffic predictions. But, some level of conservatism is still used in those items that continue to be hard to predict, such as the subgrade soil quality.

4.1 DESIGNING WITH THICK PMS LAYER

MDT pavement designs relatively thick PMS layers relative to other state highway agencies. This reduces the critical tensile stress at the bottom of the PMS layer.

Thicker PMS layers place the critical tensile stress deeper in the pavement reducing its magnitude. Thin PMS layers place the stress higher in the pavement, increasing its magnitude and possibly causing a tensile crack. This is shown in Figure 1 below.

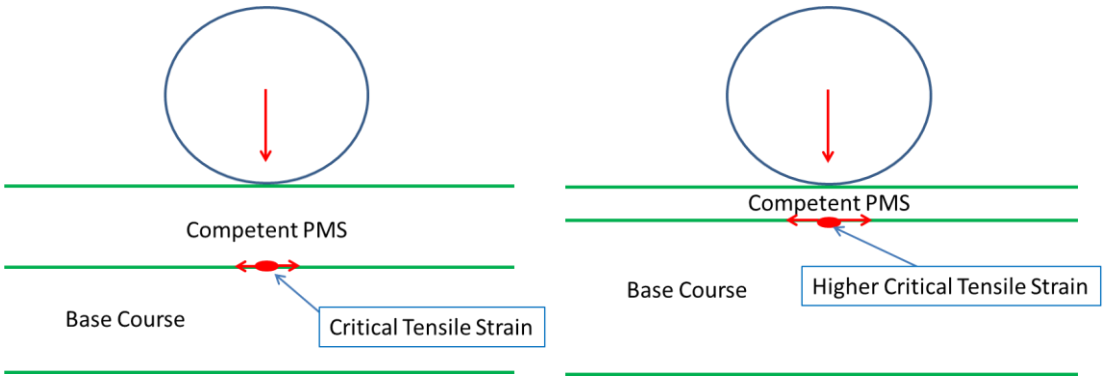

Figure 1: PMS thickness's Effect on Critical Tensile Strain

4.2 PAVEMENT FAILURE TYPES

Generally, most of Montana's hard surfaced pavements are flexible pavements surfaced with PMS. Flexible pavements generally fail as follows:

- **Subgrade Rutting:** Pavements that are too thin do not adequately dissipate tire contact pressures, resulting in high stresses within the pavement subgrade. This stress can result in further consolidation or shoving of subgrade materials, or cause subgrade fines to pump up and into the base course. This problem will result in deep, wide pavement rutting and failure.

  Generally speaking, MDT's pavements rarely fail due to subgrade rutting. It is thought that MDT's pavement design and pavement preservation overlay program result in thick pavements that dissipate pavement stresses to levels that do not cause subgrade damage.

- **Alligator Cracking:** Vehicle loadings cause pavement surface deflection that result in horizontal stresses on the bottom of the PMS layer. These horizontal stresses may exceed the PMS tensile strength, resulting in tensile cracks propagating upwards toward the pavement surface. These cracks will manifest themselves as alligator cracking within the wheel paths.

  Alligator cracking often causes MDT's pavements to fail. **The most effective way to mitigate alligator cracking is designing pavements with adequate surfacing thickness.** This is accomplished by using relatively thick surfacing sections.

- **Low quality Pavement Materials**: These are pavement failures caused by low quality pavement materials. Some examples of this are PMS rutting, stripping, raveling, etc. These pavement distresses can cause pavements to fail even if the pavement is structurally adequate.

Unfortunately, these problems are often unforeseeable and can be the result of poor construction processes. The designer can reduce the failures by specifying the correct pavement materials as discussed in the Manual.

4.3 PAVEMENT OVERLAYS – STAGED CONSTRUCTION

PMS overlays can be used to build thick pavements resistant to alligator cracking. Generally speaking, an overlay will increase a 20-year pavement design to a 30-year pavement design as long as the overlay is placed before alligator cracking occurs. A 2nd overlay may result in a pavement that won't alligator crack, often referred to as a "perpetual pavement" by the asphalt industry.

An example of pavements that have become alligator crack resistant through overlays is MDT's Interstate system. Most of the Interstate was originally built in the 1960's and 1970's with 0.35' PMS. Throughout the years, these pavements have been overlayed multiple times, and most of them are still in-service. Currently, many of these pavements are periodically mill and filled to address rutting or smoothness, but alligator cracking is minimal.

The following guidance will allow the buildup of surfacing thickness through overlays:

- Roads should be constructed with enough pavement width to place at least one overlay upon them.
- Overlays should be placed before alligator cracking occurs. PMS overlays placed on alligator cracked pavement may experience reflective cracking. This may result in alligator cracking of the PMS overlay in a relatively short time period.

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Chapter 5 **Pavement Design Process**

MDT uses Engineering Project Scheduler (EPS) computer software for project management. Designers are EPS Functional Managers, and are responsible for updating the status of the surfacing design activities. Details of EPS are beyond the scope of this manual, however it is important to outline the designer's responsibilities in regards to statusing OPX2, ensuring that:

- There are no late activities on the critical path
- Activities are statused regularly. EPS should be checked and statused no less than every two weeks.
- Do not appear on the Status Compliance Report more than once during a calendar year with activities unstatused for more than two weeks.

The Surfacing Design EPS activities are:

MDT designed projects:

- Activity 600: Prepare Preliminary Surfacing Typical Section
- Activity 602: Provide Deflection Data - NDT unit responsible for this Activity
- Activity 604: Final Surfacing Sections
- Activity 610: Final Surfacing Design Check

Consultant designed MDT projects

- Activity 440: Preliminary Geotech & Materials Review
- Activity 442: Geotechnical & Materials Report Review
- Activity 444: Materials and Geotech Final Review
- Activity 602: Provide Deflection Data - NDT unit responsible for this Activity

The following flowcharts and activity descriptions describe the Surfacing Design Unit's role within the roadway design process. The flowcharts were developed using the EPS flowcharts published by the Engineering Division's Engineering Information Systems Section. The flow charts can be found on the intranet at the following links:

- For in-house projects.
- For consultant projects.

5.1 PROJECT FLOWCHARTS

The following flowcharts describe the surfacing design unit's roles and activities within the overall project design. This does not include pavement preservation and consultant design projects.

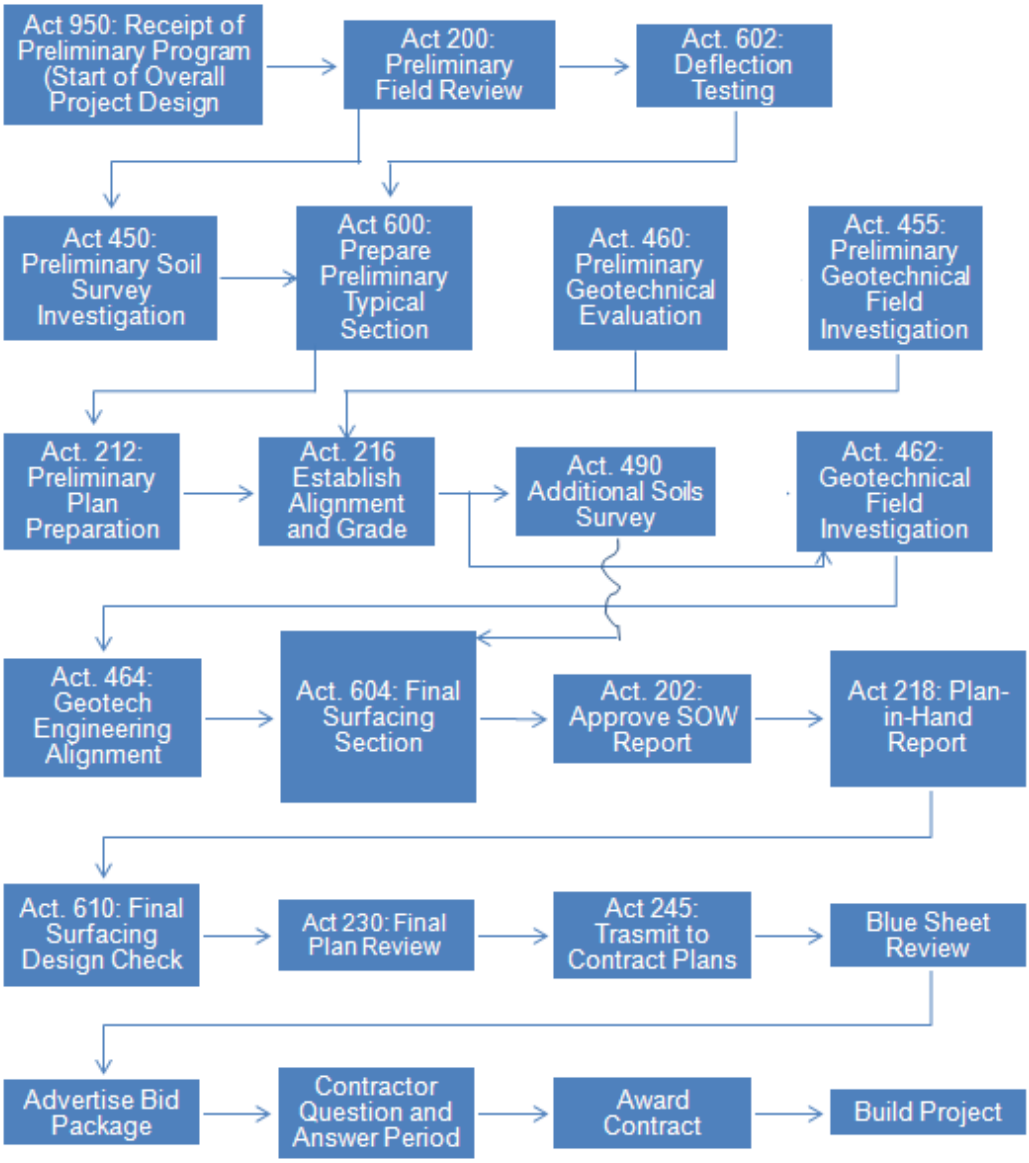

Figure 2: New Construction, Reconstruction, Major Rehabilitation, and Overlay and Widen Design Flowchart

Activity 950: Receipt of Preliminary Program. Shown for information only. This activity establishes a project charge number to monetarily charge MDT design time to, and is the point of time when preliminary engineering begins for a project.

Activity 200: Preliminary Field Review (PFR). A Surfacing Design Unit representative is required to attend all pavement preservation, chip seal, overlay, mill/fill, and minor and major pavement rehabilitation projects. The PFR is an on-site meeting early in the design process where the project scope is preliminarily developed, including the type of pavement treatment to be used. The PFR is attended by Headquarters and District Designers and Engineers, and it culminates in a PFR report that is distributed for comment. The PFR report is considered a "milestone" report.

The designer should gather and examine the following information before the PFR:

- Project Limits
- Pavement Management System (PvMS) information
    - As-built pavement typical section
    - Construction History (Pavement management is one location this information can be obtained)
    - Ride, rut and cracking indices
- NDT information,
    - FWD Results (if applicable)
    - GPR Results (if applicable)
- Soils information
    - Soil survey from previous project (if applicable)

Activity 602: Deflection Testing. This is a prerequisite of Activity 600. This is the Non-Destructive Testing (NDT) Unit's only EPS activity. This activity includes falling weight deflectometer (FWD) and ground penetrating radar (GPR) pavement testing, as well as processing the test data. Processing the data refers to calculating pavement layer resilient modulus (M_R) and thicknesses.

Activity 450: Preliminary Soil Survey Investigation. This is a prerequisite to Activity 600. The preliminary soil survey is done by the District Materials Laboratory, and refers to boring the existing pavement and/or new alignment to determine both thicknesses and quality of the pavement and subgrade materials. Borings are normally done at ½ mile intervals in alternating lanes, but can be reduced to 1 mile intervals when supplemented with FWD and GPR (Activity 602). The soil survey is conducted as specified in MT-207 of the Materials Manual, and reported within the Site Manager computer program, or Lab Form 111.

The project scope will determine the level of soil survey required to develop the project.

- If the project is a reconstruct, the soil survey should primarily consist of a subgrade soil survey. The designer will develop the new typical section based upon the subgrade results. Any information on the base is useful and composite samples should be performed on the base gravel.

- If the project is a pulverization project, the soil survey should consist of both the subgrade and base course information.

Activity 600: Prepare Preliminary Typical Section. This is the Surfacing Design Unit's major design activity. This activity involves retrieving, organizing, and analyzing information to design the preliminary pavement section for a given road project. Details of the process are described throughout this design manual. The conclusion of this activity is sending the preliminary surfacing memo to both Headquarters and District road design staff.

Activities 455 and 465: Preliminary Geotechnical Evaluation and Field Investigation. Shown for information only. This information can be requested from the Geotechnical Section as needed by the Designer to supplement the information provided in Activities 450 and 602.

Activity 212 and 216: Preliminary Plan Preparation and Establish Alignment and Grade. Shown for information only. The preliminary typical section (Activity 600) is a predecessor for both of these Road Design activities. The Designer should be aware the effect that Activity 600 has on these activities and the overall project schedule.

Activity 490: Additional Soils Survey. After the roadway alignment and grade have been set within the Alignment and Grade report, the District Materials Lab may need to retrieve additional soils and pavement information in locations that weren't bored during the preliminary soils survey (Act. 450). The Designer should review the alignment and grade report, and request additional soil survey as needed for the Final Surfacing Section (Activity 604). Usually, the preliminary soil survey is adequate for pavement design, and Activity 490 doesn't occur. Note that it is the ***Designer's responsibility to request additional soils survey*** when needed.

Activities 462 and 464. Geotechnical Field Investigation and Engineering Alignment. Shown for information only. The Designer can request this information from the Geotechnical Section as needed to supplement the information provided in Activities 450, 490, and 602.

Activity 222 and 604. Approve Scope of Work Report and Final Surfacing Section. The Scope of Work (Act. 214) is a project milestone report where the design scope of a given project is defined. The report includes the preliminary typical section. Activity 604 consists of the designer checking the typical section and pavement materials described in the SOW report for accuracy and completion. When the information is correct, the designer should "card-off" the activity in OPX2. If revisions are needed, a final surfacing design memo showing the revised surfacing section is prepared and sent to Headquarters and District Road Design staff.

Activities 218 and 610. Plan-in-Hand Report and Final Surfacing Design Check. The Plan-in-Hand Report (Act. 218) is a project milestone report where the plans are thoroughly reviewed. Activity 610 consists of the designer checking the typical section and pavement materials described in the PIH report for accuracy and completion, especially considering additional

information that became available since the final surfacing design (Act. 604) was completed. When the information is correct, the designer should "card-off" the activity in OPX2. If revisions are needed, an e-mail or memo outlining the revisions is prepared and sent to Headquarters and District Road Design staff.

Activities 230: Final Plan Review. Shown for information only. The final plan review is a mail / e-mail distribution of the project plans, specifications, and cost estimate when the overall design is 90% complete. The distribution should be reviewed for accuracy, but the Designer should not make comments at this stage unless absolutely necessary and after consulting with the Materials Engineer. The reason for this is comments and/or changes made to the plans at this point may delay project delivery.

Activity 245 and Blue Sheet Review. Shown for information only. Preconstruction submits the bid package to contract plans three months before it is scheduled to be released to Contractors for bidding. During the three months, the last project review occurs when the Contract Plans distributes "blue sheet" bid package which is literally the bid package printed on blue paper. The distribution should be reviewed for accuracy, but the Designer should not make comments at this stage unless absolutely necessary and after consulting with the Materials Engineer. The reason for this is comments and/or changes made to the plans at this point may delay project delivery.

Advertise Bid Package, Contractor Question and Answer Period, and Award Contract. Shown for information only. When the project is advertised for award (or "letting") there is a question and answer (Q&A) period where Contractors can ask questions regarding the bid package. The Q&A may result in changes to the bid package. Surfacing Design routinely is involved in answering or advising on questions submitted during the Q&A period. In the event a contractor contacts the designer directly with questions, they need to be directed to submit their questions through the Q/A system for a response. This allows all potential bidders to have the same information.

Build Project. Shown for information only. During construction, surfacing design routinely is called upon to advise regarding surfacing sections and materials. Time devoted to this should be billed to the projects 9402 account with no activity number. Work done on projects under construction should be done quickly and take priority since construction delays are costly to both MDT and Contractors.

5.2 PAVEMENT PRESERVATION FLOWCHART

Pavement Preservation projects are pavement treatments meant to preserve pavements that are in good condition. It is the intent of MDT to nominate, design, and let these projects in less than 2 years in order to build the project with the appropriate scope before the pavement deteriorates into a poorer condition and may no longer be a pavement preservation candidate.

The project is developed as shown in the flow chart and activities below. Surfacing Design normally gets involved in these projects starting with the preliminary field review.

Of particular importance is the method that pavement preservation are chosen. The project treatment should be the same, or one category different (above or below) what is recommended in the Annual Pavement Performance and Condition Report (MDT, 2013). For example, if the Report specifies that a thin overlay is needed, the project treatment should be one step below, the same, or above; or a chip seal, thin overlay, or minor rehabilitation, respectively.

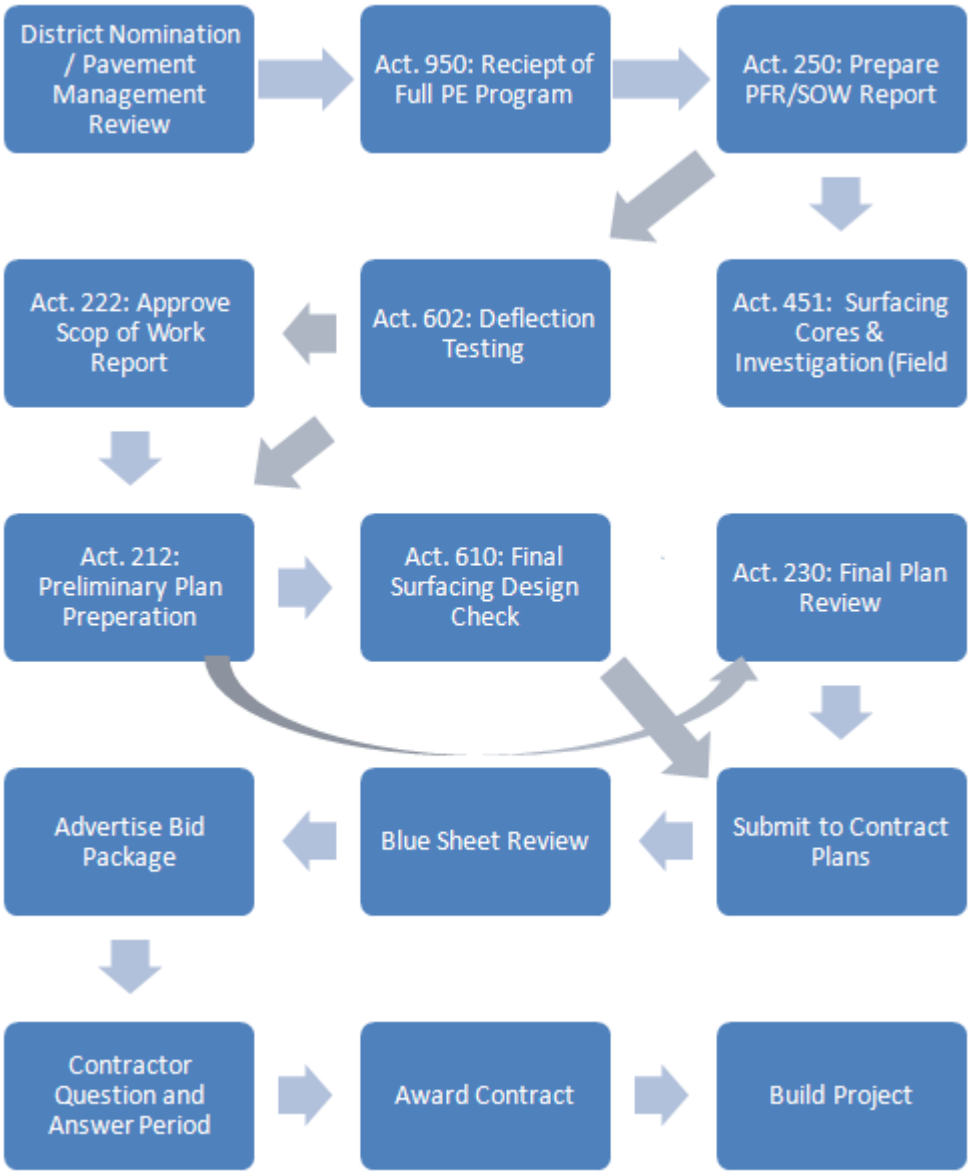

Figure 3: Pavement Preservation Project Flowchart

A more thorough discussion of MDT's policy regarding scoping pavement preservation projects can be found here. Figure 3 contains a flow chart showing the surfacing design related activities that occur during pavement preservation project design. The activities are discussed more in the following paragraphs.

District Nomination / Pavement Management Review. For information only. Generally, the District nominates projects based upon their needs along with the recommended treatment published annually in the Pavement Performance and Condition Report. After the nomination, the pavement management supervisor / engineer inspects the project and approves or disapproves the nomination.

Activity 950: Receipt of Preliminary Program. Shown for information only. This activity establishes a project charge number to monetarily charge MDT design time to, and is the point of time when preliminary engineering begins for a project.

Activity 250: Prepare PFR/SOW Report: Although there is not an EPS activity for it, a preliminary field review occurs on pavement preservation projects. A Surfacing Design Unit representative is required to attend pavement preservation PFRs. The PFR is an on-site meeting early in the design process where the project scope is preliminarily developed, including the type of pavement treatment to be used. For example, if the nomination scope is an overlay, at the PFR the attendees may observe the pavement and define the overlay as a 0.20' Grade S overlay. The PFR is attended by Headquarters and District Designers and Engineers, and it culminates in a PFR report that is distributed for comment, or a combination PFR/SOW (scope of work) report.

Activity 451: Surfacing Cores & Investigation. This activity consists of taking pavement cores and measuring for thickness and for stripping analysis. Plant mix cores should be requested by the designer through the Road Design Project Manager. The cores are usually taken by the District materials lab at ½ mile intervals in alternating lanes. Pavement coring and strip testing are done in accordance with MT 331 within the Materials Manual. As a rule of thumb, cores should be taken as follows:

- All mill/fill projects
- On all pavement preservation overlays and mill/fills on roadways with more than 300 daily ESALs > 300
- On any in place recycling projects (cold in place recycle and hot in place recycle). Double the core frequency and retain ½ the cores for bidding.

The purpose of the cores is to:

- **Specify the milling depth** – it is undesirable to mill PMS just above or below an existing boundary between PMS lifts. This may result in a rough finished milled surface
- **Ensure that milling is feasible** – at least an inch of PMS should remain in place after milling to carry traffic during construction
- **Ensure that milling isn't occurring in overly stripped PMS** – milling into stripped PMS may result in a rough milled surface. Generally, milling should only be done in material with an average stripping test grade ≥ 1.2.

- **Ensure that an overlay isn't being placed directly on overly stripped PMS** – placing PMS overlays on stripped plant mix (stripping grade less than or equal to 1) is not recommended. The underlying PMS may not have adequate strength to support the new overlay. This may not hold true on very low volume roads where overlaying stripped plant mix may be possible due to low truck loading.

- **Allow the recycling contractor to obtain information on existing pavement -** including aggregate size, fracture and oil content. A mix design can also be performed using cores.

Activity 602: Deflection Testing. Often, this activity is only done on mill/fill projects to determine the in-place PMS thickness for specifying milling depths. For pavement preservation projects, FWD data is usually not needed since by definition the pavement should be in good condition.

5.3 CONSULTANT DESIGN PROJECT FLOWCHART

Often, MDT contracts with engineering consultants to design and prepare bid packages for MDT projects. On these projects, a consultant design engineer administers the project and acts as a liaison between MDT and the consultant. That being the case, ***all project communication with the consultant must be done through the consultant design engineer responsible for the project.***

On these projects, surfacing design's duty does not include design work, but rather reviewing the consultant's pavement reports, calculations, plans and specifications to ensure that they are designed and specified in accordance with MDT policies and procedures. Figure 4 contains a flow chart showing the consultant activities integrate with MDT pavement design activities.

Consultant Design EPS activities of direct interest to Surfacing Design include:

- Activity 440: Preliminary Geotech & Materials Review

- Activity 442: Geotechnical & Materials Report Review

- Activity 444: Materials and Geotech Final Review

- Activity 608: Provide Deflection Test Data – NDT Unit responsible for this Activity

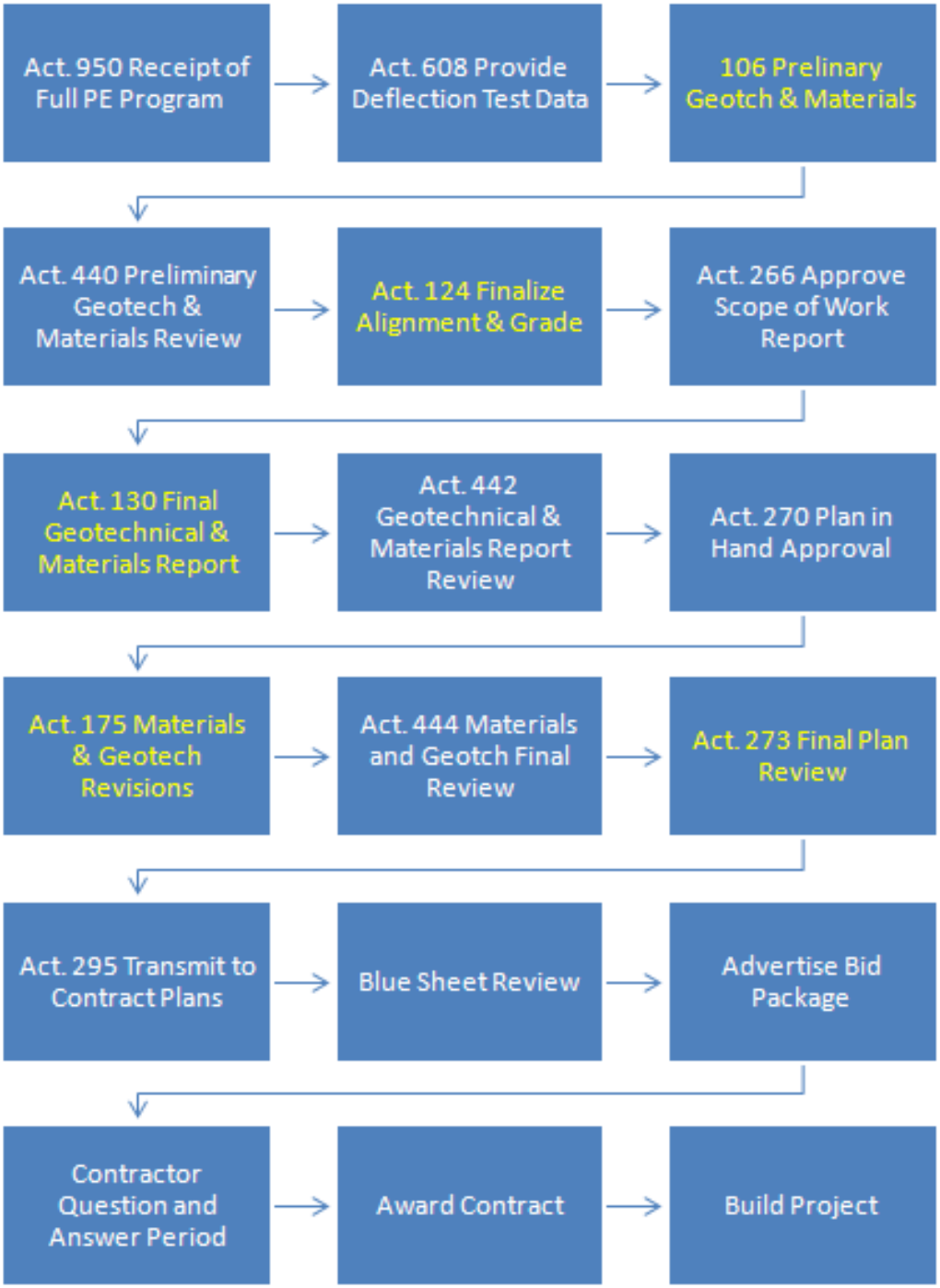

Figure 4: Consultant Design Project Flow Chart (Yellow Font Designates Consultant Activity)

Activity 950: Receipt of Preliminary Program. This activity establishes a project charge number to monetarily charge MDT design time to, and is the point of time when preliminary engineering begins for a project.

Activity 440: **Preliminary Geotech & Materials Review.** The design consultant completes this activity. This activity includes:

- Preliminary Soil Survey Investigation
- Borrow Pit Investigation
- Surfacing Pit Investigation
- Prepare Preliminary Surfacing Typical Sections
- Deflection Analysis
- Preliminary Geotechnical Evaluation

Activity 106: Preliminary Geotech & Materials. This refers to MDT's review of the Activity 440 report for accuracy and completion. The surfacing design unit reviews the pavement related portion of the report including the soil survey investigation, preliminary surfacing typical sections, and the deflection analysis. Comments on the report should be sent via email to the District, Materials Bureau, and the Consultant Design Project Manager.

Activities 124 and 266: Finalize Alignment & Grade and Approve Scope of Work Report. For information only. Defines the project scope.

Activity 130. Final Geotechnical & Materials Report. The design consultant completes this activity including:

- Prepare Final Surfacing Sections
- Primary Soils Survey
- Geotechnical Surveys and Field Investigation
- Geotechnical Engineering – Alignment

Activity 442: Preliminary Geotech & Materials. This refers to MDT's review of the Act. 130 report for accuracy and completion. The surfacing design unit reviews the pavement related portion of the report including the final surfacing sections and primary soils survey. Comments on the report should be sent via email to the District, Materials Bureau, and the Consultant Design Project Manager.

Act. 273: Final Plan Review. Shown for information only. The final plan review is a mail / e-mail distribution of the project plans, specifications, and cost estimate when the design is approximately 90% complete. The distribution should be reviewed for accuracy, but the Designer should not make comments at this stage that significantly alters the design unless absolutely necessary and after consulting with the Materials Engineer. The reason for this is comments and/or changes made to the plans at this point may delay project delivery.

Activity 444: Materials and Geotech Final Review This activity serves as consultation for finalizing any changes for final plan review.

Activity 608: **Provide Deflection Test Data**: The NDT unit is responsible for this activity. Upon request, the NDT unit will provide FWD and GPR data to the design consultant. It should be noted project level NDT data can only be collected when weather permits during spring and fall.

5.4 DESIGN MEMOS

The surfacing designer should be aware of the Design Memos posted here:

Design memos provide additional guidance and take precedence over what is printed in the Road Design Manual and the Surfacing Design Guide.

The surfacing designer should also be aware of the Construction Memos posted here. Construction Memos provide guidance on topics such as chip seals and subgrade sampling.

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Chapter 6 **New and Reconstructed Pavements**

The design method outlined within this section assumes that the existing road will not be used in-place as part of the new pavement. This includes both new and reconstructed pavements defined as follows:

- **NEW PAVEMENT** - A new pavement is a pavement structure that is placed on a previously undisturbed subgrade. It applies to a highway on a new alignment, or to the new part of a widened highway. Often, these projects are needed to update old roadways to new geometric standards.

- **RECONSTRUCTED PAVEMENT** - A reconstructed pavement refers to completely removing an existing pavement structure replacing it with a new pavement structure. This type of work is needed when the existing pavement is in a weakened condition that cannot be salvaged. Also, these projects are often needed to update old roadways to new geometric standards.

MDT's flexible pavement design method is based upon the 1993 AASHTO Guide for the Design of Pavement Structures (1993 Guide) (AASHTO, 1993). MDT has incorporated several modifications to the 1993 Guide to reflect specific Montana conditions and past experience.

6.1 GENERAL CHARACTERISTICS OF MDT PAVEMENTS

The following figures show typical sections from a sample of MDT road plans. In all instances, note that the PMS, base course, and special borrow are uniform thickness across the entire pavement (i.e. the shoulders are not built thinner).

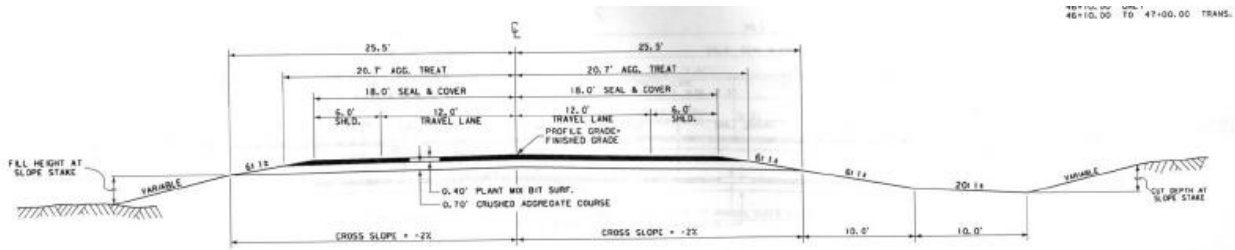


Figure 5: Typical MDT Rural Flexible Pavement

Figure 5 shows MDT's most common pavement type consisting of PMS under laid with crushed aggregate course (CAC). Note that the pavement layers have 2% cross slopes and "daylight" out of the side of the pavement inslopes. The purpose of this is to facilitate the drainage of water laterally towards roadside ditches. Seal & cover is placed on top of most flexible pavements, except those covered with plant mix seal or 3/8" Grade S PMS.

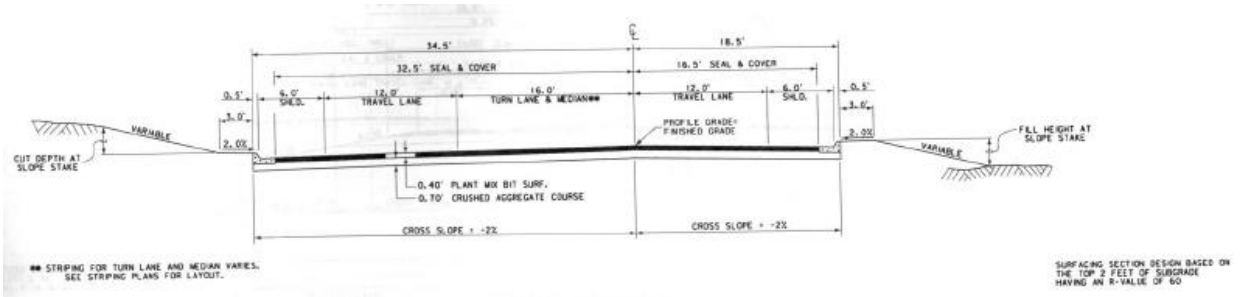


Figure 6: Typical Urban Flexible Pavement

Figure 6 shows the most common urban pavement. Note the urban pavement layers do not "daylight". Edge drains may be used to facilitate drainage depending on soil characteristics.

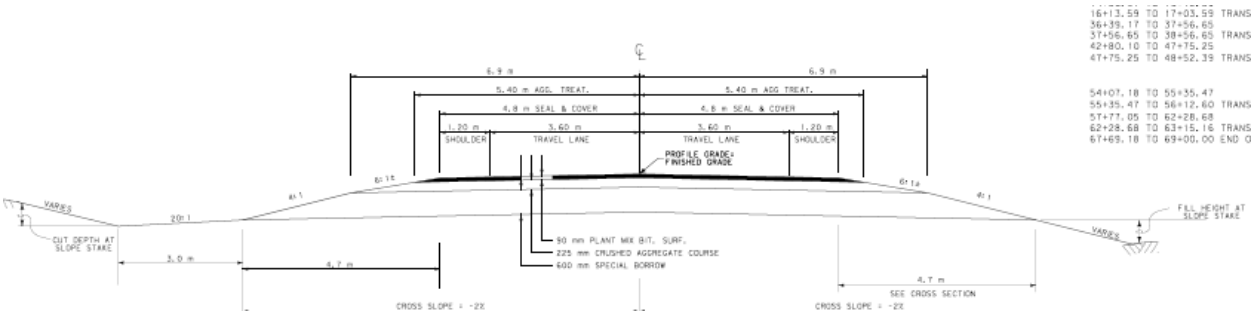


Figure 7: Typical Flexible Pavement with Special Borrow

Figure 7 shows a pavement with a standard 2' layer of special borrow beneath it. Special borrow is often used to decrease CAC thickness or to "bridge" soft subgrades. It usually consists of pit-run gravel, but may consist of other granular materials in gravel-poor areas. **Special borrow pavements are designed using the resilient modulus (M_R) of the special borrow when a minimum of 2 feet of special borrow is used. Typically it is assumed up to five feet of the underlying subgrade will influence the M_R of the special borrow. Care should be taken to select an appropriate (M_R) which accounts for the stiffness of the underlying subgrade.**

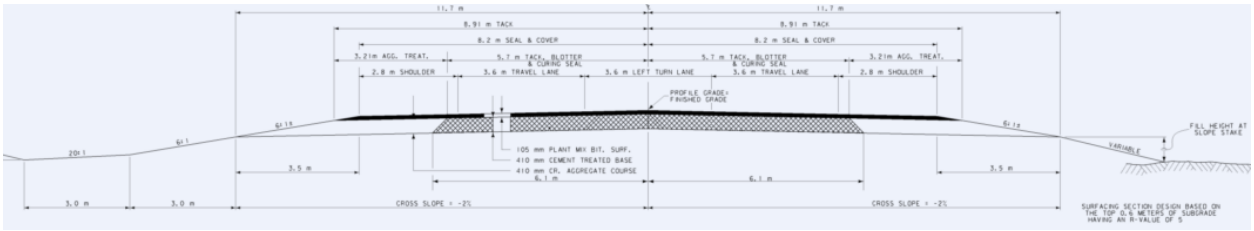


Figure 8: Typical Cement Treated Base CTB Pavement

Figure 8 shows MDT's standard cement treated base (CTB) pavement section. CTB is often used in areas where gravel is scarce. CTB pavements are designed similarly to CAC pavements. Note that the **CTB section extends 1' beneath the adjacent shoulders, and CAC is beneath the shoulders.** This practice is based strictly on economics since CAC is less expensive than CTB.

6.2 FLEXIBLE PAVEMENT DESIGN METHOD

MDT pavement design uses the structural number (SN) approach presented in the 1993 AASHTO Guide. The SN is an abstract number that expresses pavement structural strength required for given combinations of soil support (M_R) and total traffic expressed in ESALs.

Flexible pavement design process consists of the following steps:

Step #1. Determine design inputs to calculate structural number ($SN_{Req'd}$)
Step #2. Calculate $SN_{Req'd}$
Step #3. Design a pavement structure with SN_{des} >= $SN_{req'd}$
Step #4. Specify pavement materials
Step #5. Send pavement design and materials memo

6.2.1 Step #1: Determine Pavement Design Inputs

Most of the work involved in pavement design involves determining the inputs needed to calculate the $SN_{Req'd}$ including:

- Total 18-kip ESALs Over Pavement Design Life
- Initial Serviceability (po = 4.2)
- Terminal Serviceability (pt = 2.5)
- Reliability Level (varies by route type)
- Overall Standard Deviation (S.D. = 0.45)
- Roadbed Soil Resilient Modulus (from R-Value and FWD testing)
- Number of Construction Stages (1)

These inputs are discussed in detail within the following sections.

Traffic

Highway traffic is a combination of many vehicle types, all having different gross weights and axle configurations. To simplify the variety of loadings for pavement design, vehicle loadings are converted to 18-kip equivalent single axle loadings (ESAL). The process of collecting traffic data and converting it to ESAL's is complex and the 1993 Guide presents this in more detail.

It is important to accurately characterize traffic loading for pavement design. Small errors carried through the project design life can produce unexpected results, such as under design and premature failure or overdesign and unnecessary expense.

For pavement design, traffic information is provided by the Traffic Data Unit within the Planning Division. Traffic information is usually requested by the road design project manager after the

preliminary field review, and the pavement designer is copied on this traffic information. The pavement designer can also contact the Traffic Data Unit directly to request traffic information. A sample copy of a traffic report is included in Appendix A.

Annual traffic counts are available on an interactive map found on MDT's website. http://www.mdt.mt.gov/publications/maps.shtml

Flexible pavement design life is 20-years unless specified otherwise by the District. 20-year traffic information is provided by the Traffic Data Unit by default. For designs with other than 20-year design lives, a note should be placed in the project file explaining why another design life length was used.

The design input for pavement design is the total ESALs during the pavement design life. An example of how to calculate this follows.

> ***Example Problem: Calculating Design Life ESAL loading***
> The traffic memo in Appendix A states that the EAL (ESAL) = 23. The EAL is the average daily ESAL loading during the 20-year design life. In this case the total ESALs during the 20-year design life is calculated as follows:
>
> Design Life ESALs = Daily ESALs * Design Life (years) * 365 days per year
>
> = 23 ESALs per day * 20 years * 365 days per year
>
> = 167,900 ESALs over the 20 year design life

Estimating Traffic – Special Cases

Pavements are often designed in areas where the Traffic Data Unit's traffic estimates are unavailable or unable to capture future traffic generators. Some examples of this are Interstate rest areas, energy sectors such as the Bakken oil patch and commodity haul routes near grain elevators. For these situations, the designer should work with the District Traffic Engineer and estimate the daily ESAL's. For special situations such as the Bakken, a report outlining traffic impacts may be available.

Agricultural commodities are a large industry, and a generator of heavy trucks during harvest times. Traffic data may not include commodity hauls in their traffic estimates since traffic counts may have not been done during harvest time. On roadways with commodity hauls, the designer should estimate the increased daily ESALs due to commodity hauls, and add those ESALs to the Daily ESALs provided by the Traffic Data Unit. This is discussed further in the following sections.

Rest Areas

Use the following equation to estimate ESALs for rest area pavements, including approaches and entrance and exit ramps:

Adjacent Mainline Pavement ESALs * 0.25 = Rest Area ESALs

For example, for an interstate pavement with 600 daily ESALs, use 150 ESALs for rest area pavement design.

Sugar Beet Truck Routes

Sugar Beets are grown abundantly in Montana. Figure 9 shows where sugar beet farms are located. The blue dots represent 1,000 acres of beet farms. There are two beet processing facilities, with one located near Hardin and the other near Sydney. There are also a number of storage facilities.

Sugar Beets are a heavy commodity, similar to potatoes. Sometimes harvest takes place during late fall and winter. Transfer of the beets to the processing facility occurs over the winter and into spring. Spring thaws mark the time when pavements are in their weakest state. The designer should consider beet traffic when designing pavements in beet producing areas.

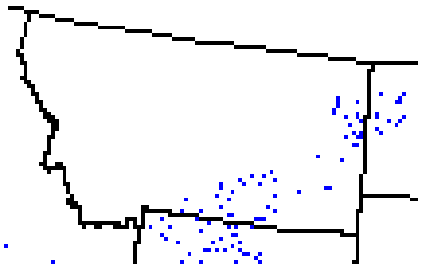


Figure 9: Montana Sugar Beet Farm Locations (1997, US Dept. of Ag.)

Wheat Truck Routes

Wheat is grown abundantly in Montana. At harvest time, wheat is hauled via truck to large silos where it stored in grain elevators to be loaded onto trains. Recently, these grain elevators have been consolidating into very large facilities as shown in Figure 11. These facilities generate a large amount of truck traffic, approximately 60 daily ESALs.

Figure 10: Wheat Farming Areas (US Dept. of Ag., 2002)

Figure 11: Modern Large Grain Elevator

Oil Production and Exploration

There are a number of areas in Montana with oil and gas reserves, but the area where pavements are most currently affected is within the Bakken formation in Northeastern Montana. Truck traffic has increased tremendously within the Bakken within the past 4-5 years and is expected to increase further in the coming years. Designers should consider recommendations in Table 1 when designing pavements within the Bakken Area. The UGPTI traffic predictions are located here.

The surfacing design unit can provide calculated design ESALs for consultant design projects. The report does not provide specific ESAL estimates.

Table 1 Bakken Area Pavement Design Recommendations

Project Type	Recommendation
Pavement Preservation	Utilize the Traffic Data Unit's traffic predictions for pavement design. Do not use UGPTI Traffic Predictions. Consider adding pavement structure utilizing thicker or stronger pavement treatments while still working with pavement preservation guidelines as outlined in the joint agreement.
Minor Rehabilitation	10-year design life using 20-rig UGPTI Traffic Predictions
Major Rehabilitation, Reconstruction and New Construction	20 year design life using 20-rig UGPTI Traffic Predictions

Research to predict future traffic growth within the Bakken has been completed (UGPTI, 2013) and an overview of these predictions is shown in Figure 12. The reference to "20-Rig Scenario" within Figure 12 refers to 20 oil drilling rigs operating continuously within Montana. The 20-rig assumption is thought to be reasonable, and should be utilized for pavement design. Impacts have been seen in surrounding counties not included in the report and as such, care should be taken when estimating ESALs.

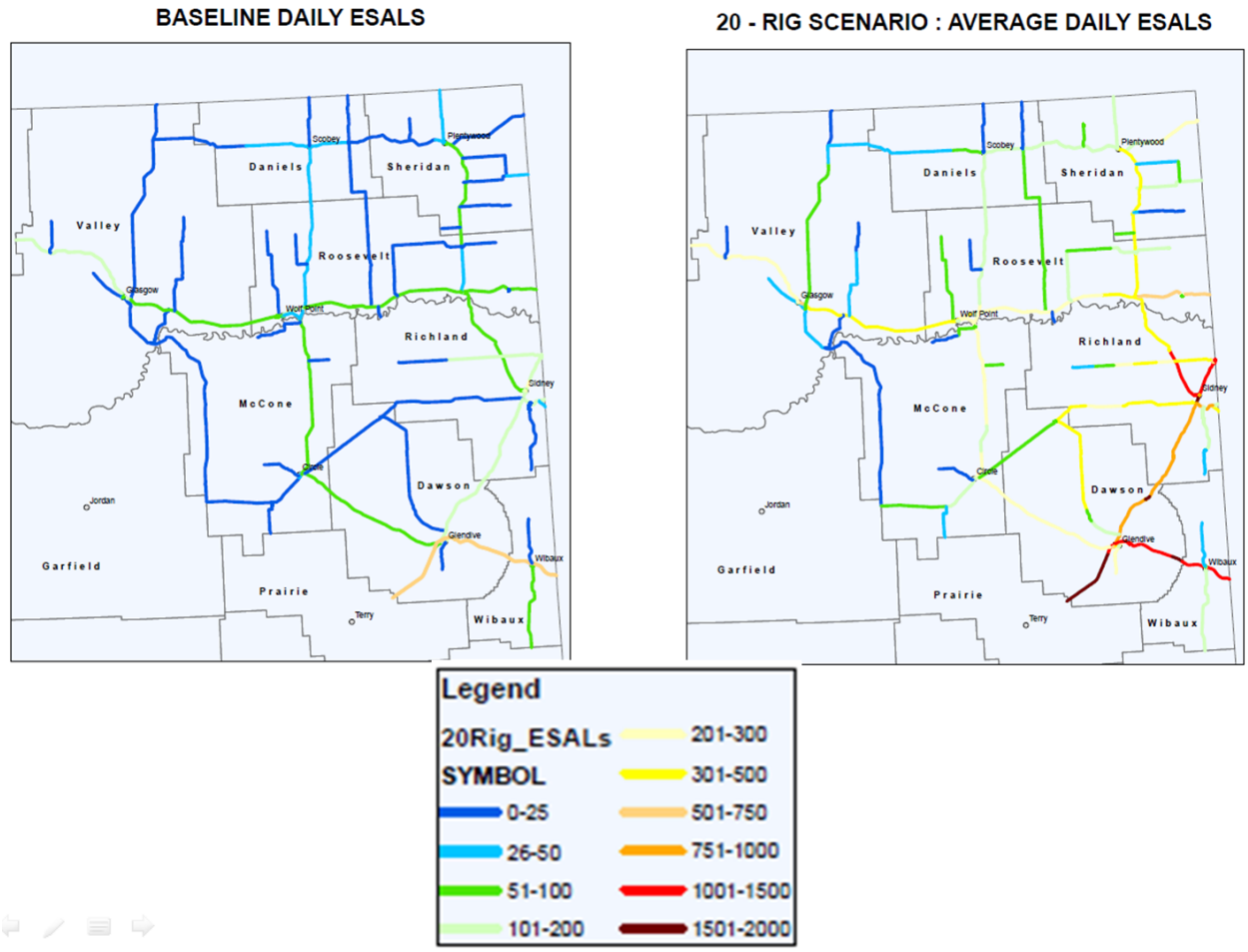


Figure 12: Bakken Area Traffic Increases (UGPTI, 2013)

Most Bakken oil drilling is within North Dakota, but many oil drilling supplies are hauled from Montana to North Dakota. The traffic generators for drilling supplies are shown in Figure 13 .

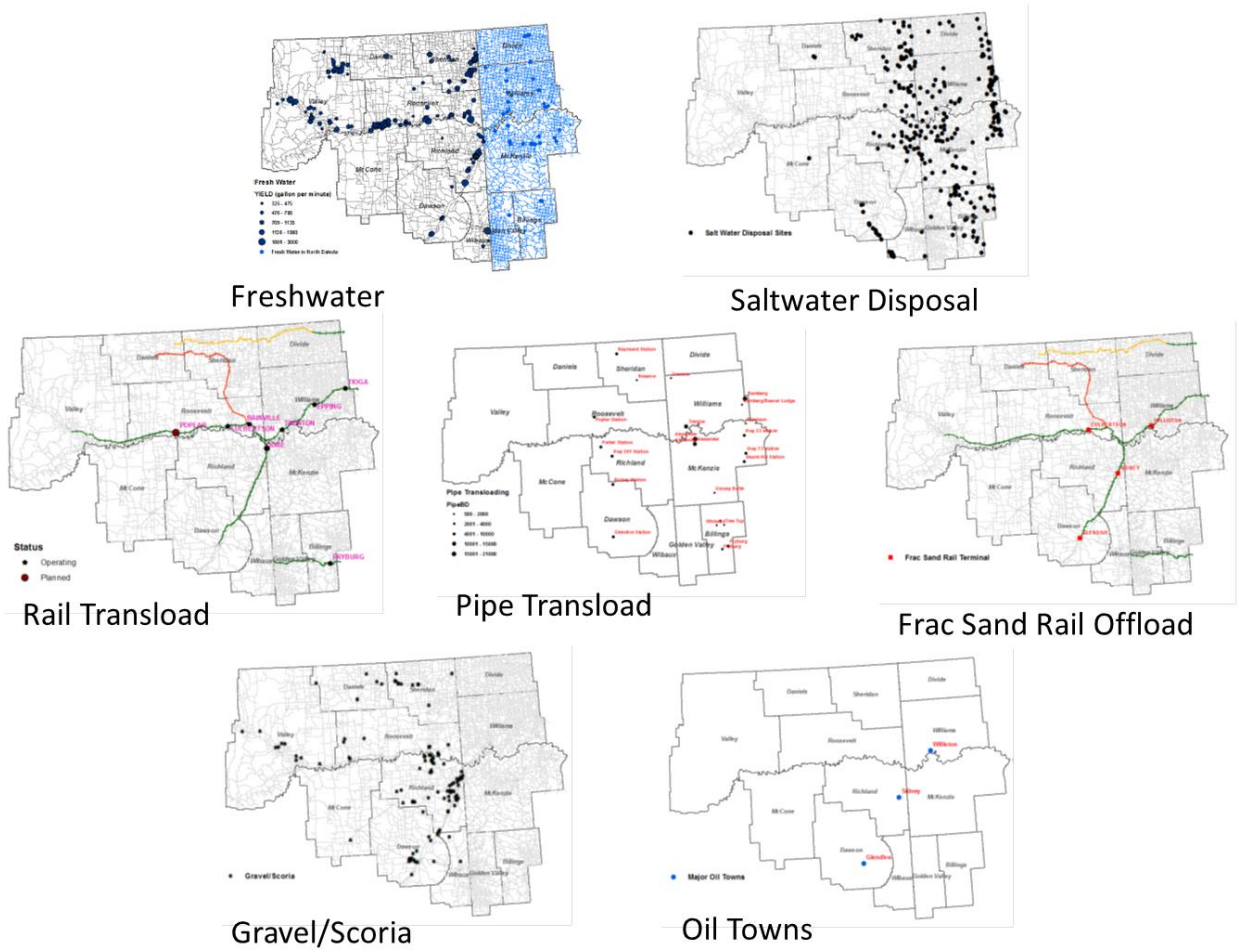


Figure 13: Bakken Oil Traffic Generators (UGPTI, 2013)

Initial and Terminal Serviceability

Pavement serviceability is defined as the pavement's ability to serve the vehicles using the roadway. Serviceability is measured using the Present Serviceability Index (PSI), which ranges from 0 (impassible road) to 5 (perfect road). Initial serviceability is the PSI immediately after a road is reconstructed or rehabilitated, while the terminal serviceability is the PSI where road rehabilitation, resurfacing, or reconstruction becomes necessary. For pavement design, use 4.2 and 2.5 for initial and terminal serviceability, respectively.

PSI is a subjective rating based upon driver's opinion of road conditions. In more recent years, International Roughness Index (IRI) has been used more than PSI to measure road serviceability. IRI is used by the Construction Division for specifying pavement smoothness. PSI of 4.2 and 2.5 correspond to an IRI equal to about 45 and 185 in/mile respectively. Figure 14 shows a common correlation between PSI and IRI

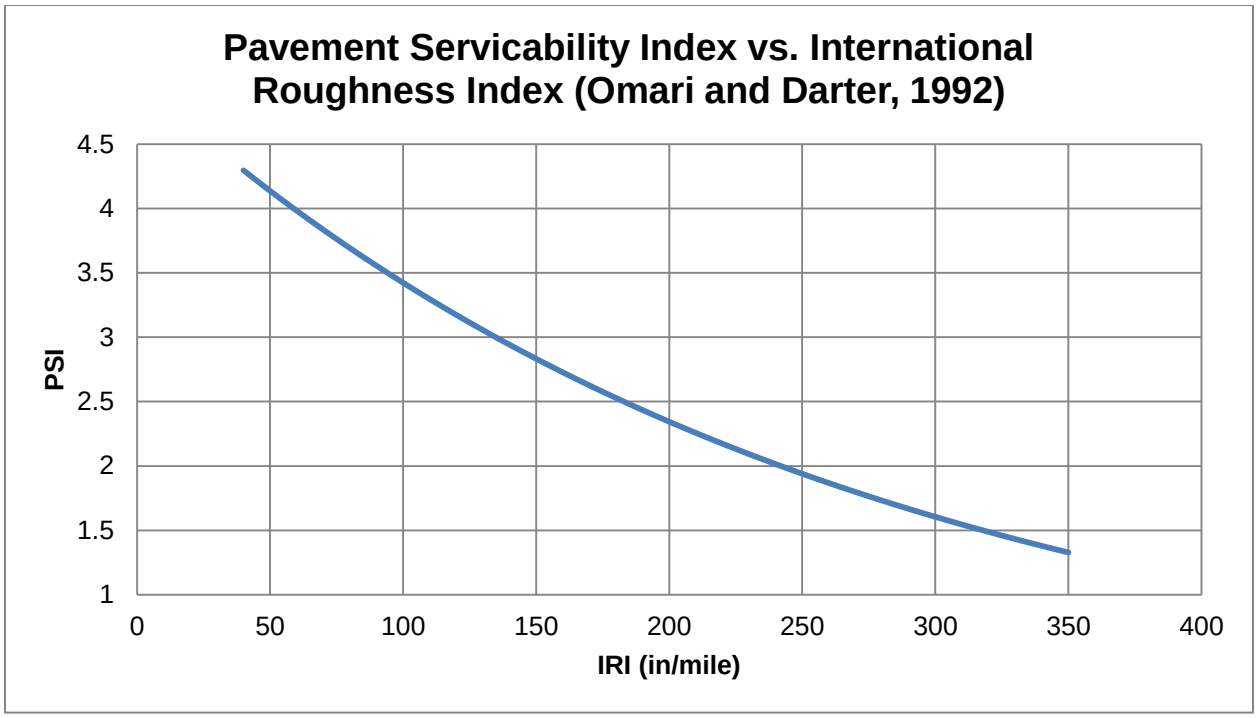


Figure 14: PSI vs. IRI Correlation

Reliability Level and Standard Deviation

The reliability level is a means of incorporating some degree of certainty into the design process to ensure a pavement will last through the design life. The reliability factor accounts for variations in both traffic and performance prediction. Use the following guidance for designating reliability levels.

Interstate and NHS – 85-95% (typically 90%)

Primary Highways – 80-90% (typically 85%)

Secondary, Urban and X-Routes– 70-85% (typically 75%)

Consideration should be given for light vehicle traffic (ADT) and truck traffic in urban areas. For example, a road in downtown Billings with 20,000 ADT should be designed with a 90% reliability because of high user costs incurred during construction.

The standard deviation is used in conjunction with reliability to account for variation in traffic prediction. Use 0.45 for standard deviation.

Drainage coefficients

The drainage coefficient (m_i) is used to increase or decrease pavement layer structural coefficients (a_i) based upon drainage quality. Use 1.0 as a drainage coefficient for all materials. The 1993 AASHTO recommends using 1.0 for stabilized base due to performance history.

Number of Construction Stages

The number of construction stages is required when using the DARWin software. Assume one Construction Stage for all projects.

One Direction Width

The directional width is required when using the DARWin software. Assume 12 feet for all designs.

Subgrade Resilient Modulus

Resilient Modulus (M_R) is a fundamental material property used to characterize unbound pavement materials. It is a measure of material stiffness and provides a means to analyze stiffness under different conditions, such as moisture, density and stress level. With the laboratory, M_R is determined using the triaxial test. The test applies a repeated axial cyclic stress of fixed magnitude, load duration and cycle duration to a cylindrical test specimen. While the specimen is subjected to this dynamic cyclic stress, it is also subjected to a static confining stress provided by a triaxial pressure chamber. It is essentially a cyclic version of a triaxial compression test; the cyclic load application is thought to more accurately simulate actual traffic loading. M_R is defined as a ratio of applied axle deviator stress and axial recoverable strain. MDT does not do triaxial testing at this time.

Pavement sections are designed based upon the subgrade resilient modulus of:

- the top five feet of the subgrade directly beneath the surfacing section in the case of pavements without special borrow or,
- the special borrow with consideration of subgrade stiffness in the case of pavements under laid with two or more feet of special borrow.

MDT utilizes R-Value testing and FWD back-calculation to estimate M_R as detailed later in this section. MDT uses the lesser value of the 85th percentile R-Value and results of FWD test results during spring thaw conditions.

It should be noted that the R-Value test utilizes disturbed specimens while FWD measures undisturbed stiffness. Standard Penetration Test (SPT) blow counts can be used to compare Resilient Modulus (M_R) values based on R-Value, soils class and FWD.

M_R varies over the course of a year, lowering and increasing dramatically during the spring thaw (saturation) and winter freeze, respectively. This change is more pronounced in clay and silts and less so with sand and gravels. For example, the M_R of clay can dip as low as 3,000 psi in the spring and up to 20,000 psi in the winter. Figure 15 illustrates this phenomenon. To account for this variation, the 1993 Guide recommends using an average annual M_R.

MDT does not use an average annual M_R. Instead, MDT utilizes M_R based on spring thaw (worst case) conditions for pavement design. This is a conservative practice but believed to

work since it is difficult to determine the subgrade type, compaction, and loading conditions that will occur during construction.

For these reasons MDT has used the spring thaw M_R for pavement design for a long time, and field performance including lack of subgrade rutting has indicated that it is a reasonable practice.

An in-depth study was conducted on the measurement of subgrade soil parameters. Various relationships of soil class, R-Value, CBR and resilient modulus are evaluated. The study can be found here.

Subgrade M_R determination for pavement design is discussed in the next two sections.

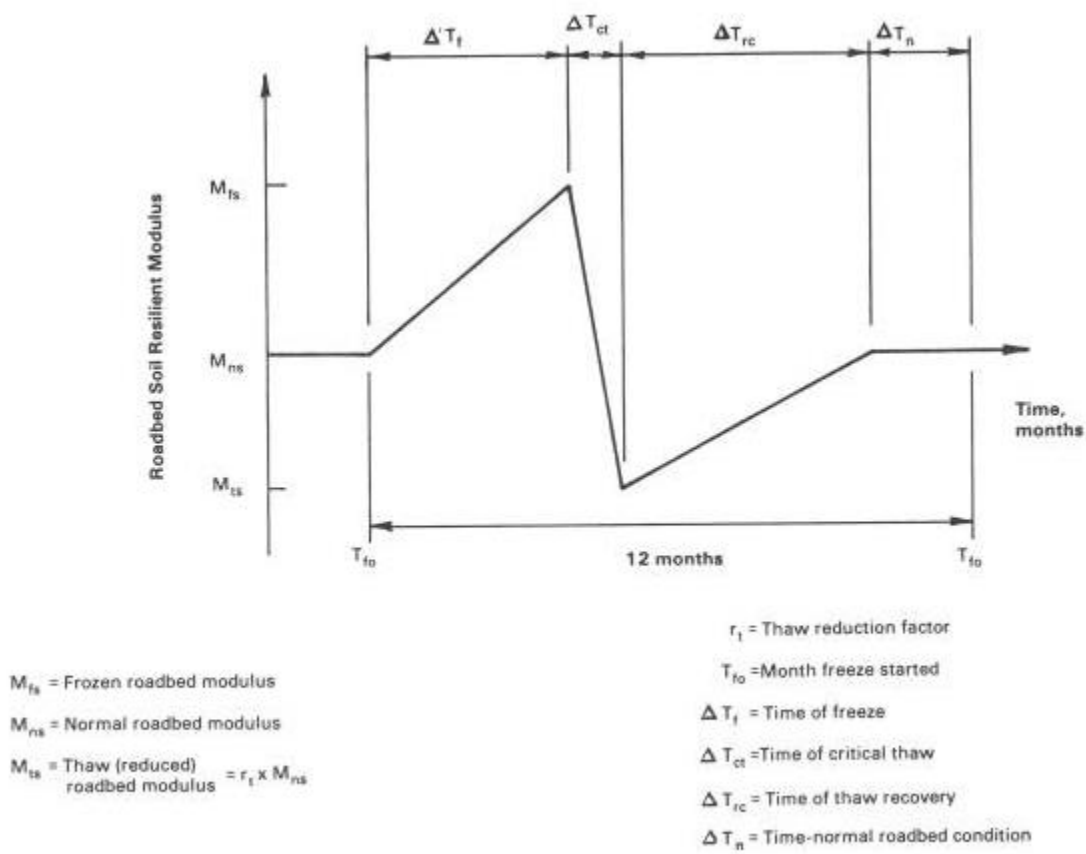


Figure 15: Subgrade Modulus Variations Throughout the Year (1993 Guide, page I-24)

Using R-Value to Estimate $_{MR(Des)}$

R-Value, or resistance value, is a laboratory soil test that measures the support capabilities of subgrade soils. The R-Value tests the soil's saturated condition and is considered to be an estimate of subgrade support capabilities during the spring thaw.

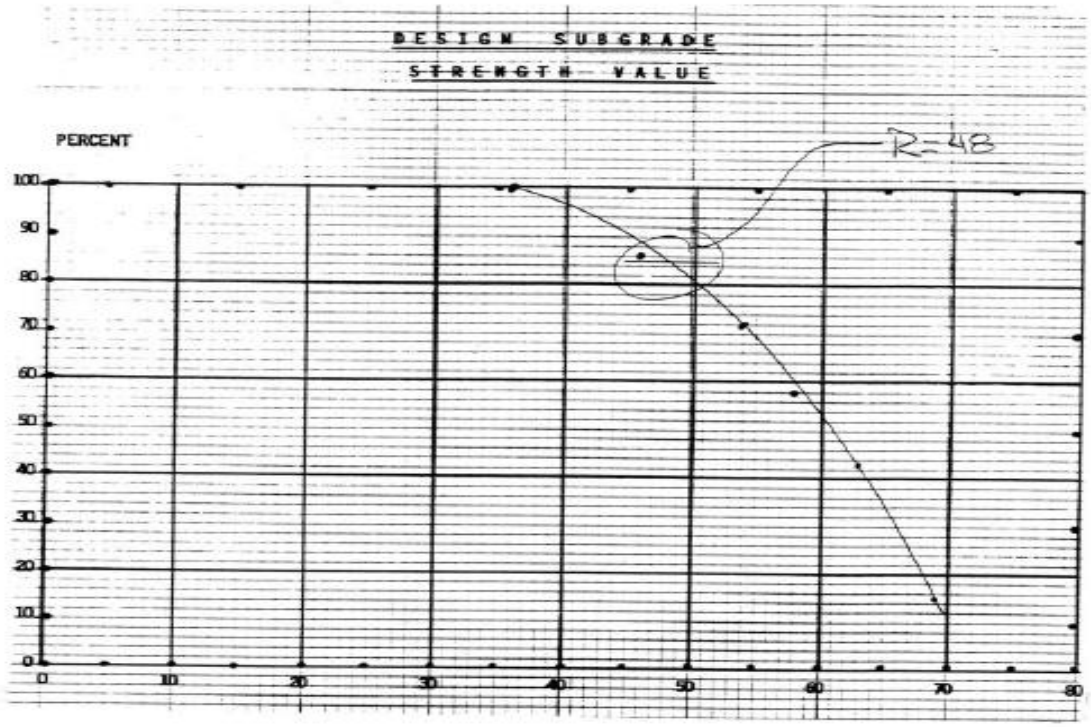
This section discusses determining the subgrade M_R ($M_{R(Des)}$) The following steps describe this process:

1. R-Value soil samples are gathered during the District soil survey (EPS Act. 450) and sent to the Headquarters materials laboratory for R-Value testing.

2. R-Value test results are provided to the Surfacing Design Unit upon the soil survey. The soil survey is provided either via email from the Testing Section's SiteManager software, or via paper copy on materials lab form 111 entitled "MDT Preconstruction Soil Survey Data and Special Recommendations Relative to Subgrade and Road Surface Design"

3. In each soil boring location, determine the R-Value of the material within the top two feet of subgrade.

4. The soil survey should be reviewed to determine major changes in soils and R-Value. For example, it may be clear that for the first three miles of a seven-mile project, soils are fairly uniform and a particular R-Value will represent the area. However, the remaining four miles may be substantially different and a different R-Value may have to be used in the design. If this is the case, two typical sections may be recommended.



R-Value	*Number ≥*	*% ≥*
36	7	(7÷7) x 100 = 100%
46	6	(6÷7) x 100 = 86%
54	5	(5÷7) x 100 = 72%
58	4	(4÷7) x 100 = 58%
63(2)	3	(3÷7) x 100 = 43%
69	1	(1÷7) x 100 = 15%

Figure 16: $R_{85\%}$ Calculation Example

5. The representative R-Values are plotted on a graph to determine the 85th-Percentlie R-Value (R_{85}). Use the following steps along with Figure 16 to determine $R_{85\%}$:

   Step #1: List all the representative R-Values under the R-Value column, beginning with the lowest.

   Step #2: Under the Number => column, list the number of R-Values that are equal to or more than the corresponding R-Value to the left.

   Step#3: Each number in the center column is divided by the total number of R-Values and multiplied by 100. These percentages are plotted and connected with a line. Some interpolation may be necessary. A point where this line crosses 85 percent becomes $R_{85\%}$ that is used to calculate $M_{R(Des)}$.

6. Calculate M_R(Des): The relationship between R85% and M_R(Des) commonly used by MDT is (NCHRP, 2004). This relationship is intended for fine grained soils with an R-Value less than 20:

$$M_{R(Des)} = 1155 + 555*R_{85th\%}$$

Where:

Minimum $M_{R(Des)} =$ 3,250psi
Maximum $M_{R(Des)} =$ 19,000 psi

California Bearing Ratio (CBR) is a soil strength test that is commonly used by MDT consultants. CBR testing can be substituted for R-Value testing. $CBR_{85\%}$ is converted to $R_{85\%}$ as follows (NCHRP, 2004):

$$4.60 * (CBR_{85\%})^{0.64} - 2.08 = R_{85\%}$$

Where:

CBR85% = The 85th Percentile CBR calculated in the same manner as R85% (See Steps #1 - #3 above)

A-1-a subgrade materials should be R-Value tested to determine the design R-Value. However, if A-1-a materials are not R-Value tested, they can be assumed to have an R-Value = 30 (M_R = 12,000 psi) without testing.

Both A-6 and A-7 subgrade soils are not tested for R-Value. An R-Value=5 is assumed for these materials. FWD back-calculated should be reviewed to determine M_R, but in the absence of FWD M_R, an M_R = 3,250 psi can be assumed.

M_R Calculated from Falling Weight Deflectometer

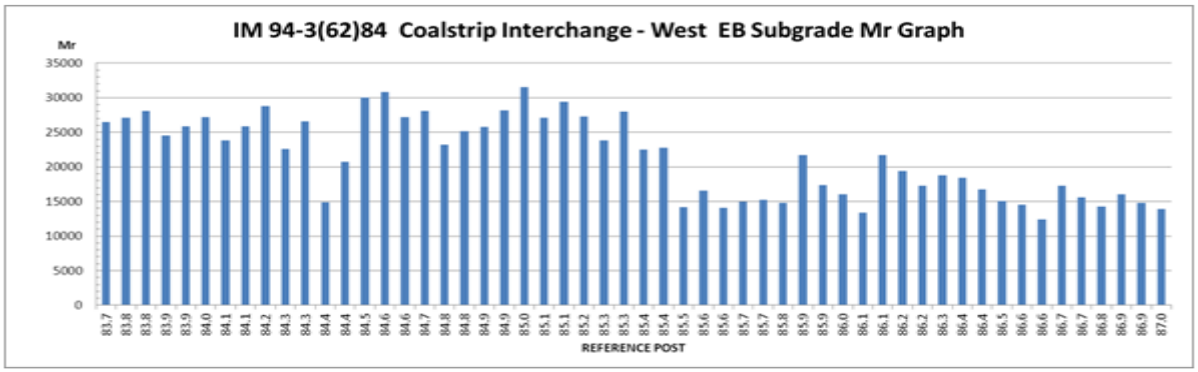
MDT's Non-Destructive Testing (NDT) Unit is responsible for conducting Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) testing. A more thorough summary of this equipment and its use is located in Chapter 7. This section summarizes the use of FWD-generated subgrade M_R for new pavement design.

The NDT unit provides the following FWD testing that can be used to determine subgrade M_R:

Network level FWD/GPR testing is done on all state roadways on a 5-year rotation. Network level testing is done at 820 ft. (250 m) increments within the outside wheel path of the driving lane in one direction only. Network level testing may occur during the summer months which can yield a higher MR than during the period of spring thaw.

Project level FWD/GPR: In addition to Network level testing, project level testing is done prior to road construction projects. Project level testing is done at 330 ft. (100 m) increments within the outside wheelpath, of the driving lane in one direction only. A

second run may be completed (one in spring and one in fall) if the schedule of the project allows. GPR testing is done continuously. This testing is done early in the project design process before Surfacing Design's preliminary surfacing design activity (Act. 600).



6/16/2010	SUB
Average	21446
Std. Dev.	5740
Corrected	17429
Lab	10723

Figure 17: Example FWD Subgrade M_R Printout

After FWD and GPR data is collected, the NDT unit processes the deflection data and backcalculates the in-situ subgrade M_R . Surfacing Design receives the data along with an Excel spreadsheet summarizing the subgrade M_R . Figure 17 shows an example spreadsheet.

Figure 17 shows the *unadjusted* back-calculated subgrade M_R. The table shows 4 different figures:

- Average: This is the average value of all of the unadjusted M_R values.
- Std. Dev: The standard deviation of the unadjusted M_R values.
- Corrected: Calculated as follows:

$$\text{Corrected } M_R = \text{Average } M_R - 0.7 * \text{Std. Dev.}$$

- Lab: This is the laboratory M_R ($M_{R\ Lab}$) converted from the Average M_R calculated as follows:

$$M_{R\ Lab} = \text{Unadjusted Average } M_R * 0.5$$

$M_{R\ LAB}$ should be used for pavement design. The reason is the AASHTO pavement design equation is based upon laboratory calculated Subgrade M_R. The LAB resilient modulus for all three layers was derived by multiplying the AVERAGE by a coefficient. These are modified values from the FHWA-RD-97-076 booklet. This booklet shows what coefficients to use to approximate a laboratory resilient modulus value from a back-calculated resilient modulus

value. The NDT has been instructed to use a coefficient of 0.45 for PMS, a 0.62 for unbound granular bases, and 0.50 for subgrade.

M_R Estimated for Special Borrow Pavements

There will be instances where $M_{R(Des)}$ will need to be estimated. This situation will usually occur when designing special borrow pavements. Recall that special borrow pavements are designed based upon the special borrow M_R, not the subgrade M_R. In this case, the material to be used for special borrow will be unknown since it will be provided by the Contractor.

In these instances the designer will often estimate $M_{R(Des)}$ based upon the special borrow material that is locally available, and/or the type of special borrow the designer determines should be used. Most often, pit run A-1-a material is utilized for special borrow, and a design R-Value = 30 is assumed.

Once the special borrow R-Value is determined / estimated, utilize the following equation to determine $M_{R(Des)}$ (NCHRP, 2004):

$$M_{R(Des)} = 1155 + 555*R_{85th\%}$$

Where:

Minimum $M_{R(Des)} =$ 3,250 psi

Maximum $M_{R(Des)} =$ 19,000 psi

Our current practice utilizes a maximum $M_{R(Des)} =$ 12,000 psi for A-1-a material (R-Value = 30), unless project specific circumstances dictate otherwise. Higher $M_{R(Des)}$ values rely on stiff subgrade underlying the special borrow cap. It should be noted, this situation (placing special borrow on stiff subgrade) rarely occurs. Resilient Modulus is a measure of stiffness of the subgrade to a semi-infinite depth (~around 5 feet deep).

Estimating M_R for Small Projects

Sometimes the Project Designer asks for a pavement design where soil R-Value testing and/or FWD information isn't available and will not be collected. This should occur infrequently, and only for small projects where the pavement quantity is too small to justify the cost of a soil survey, or there is not enough design time to allow for a soil survey.

In these cases, inquire with the NDT unit for FWD back-calculated M_R that may be available from past FWD testing. If FWD information isn't available, the designer should estimate the "worst-case" R-Value that may be encountered on the project and design accordingly. Often, in areas of known fine-grained subgrades, the Minimum $M_{R(Des)} =$ 3,250 psi is assumed for pavement design.

Subgrade Monitoring During Construction

Since subgrade strength is one of the primary inputs to flexible pavement design, it is important to monitor the subgrade during construction. The purpose of this is to ensure that the subgrade

material is of equal or greater quality than the subgrade material that was assumed during the pavement design.

Subgrade and Special Borrow sources are approved prior to construction, and subgrade design checks are done during construction. Either R-Value testing and/or soils classification are used for borrow source approval and subgrade design checks. To ensure that these activities occur, include one of the following special provisions in to the contract when subgrade material is imported to the project:

- MDT Standard Special Provision 106-2a -Borrow Source Approval–Soil Classification,
- MDT Standard Special Provision 106-2b. Borrow Source Approval–Resistance Value, or
- MDT Standard Special Provision 106-2c. Borrow Source Approval-Soil Classification and Resistance Value

Acceptance of subgrade material on the roadbed is generally done by soil classification.

A note showing the design R-Value should be located next to the typical sections within the Plans. The note's purpose is to notify construction personnel of the design R-Value, and help them identify material that does not meet the pavement design requirements.

R-Value testing is used as a final design check during construction. The procedure is described in the Materials Manual. This testing is not a construction contract requirement. It is the final check to ensure the subgrade material meets the properties assumed during the pavement design. MDT's Construction Memo "R-Value Testing of Finished Subgrade" provides guidance on R-Value testing.

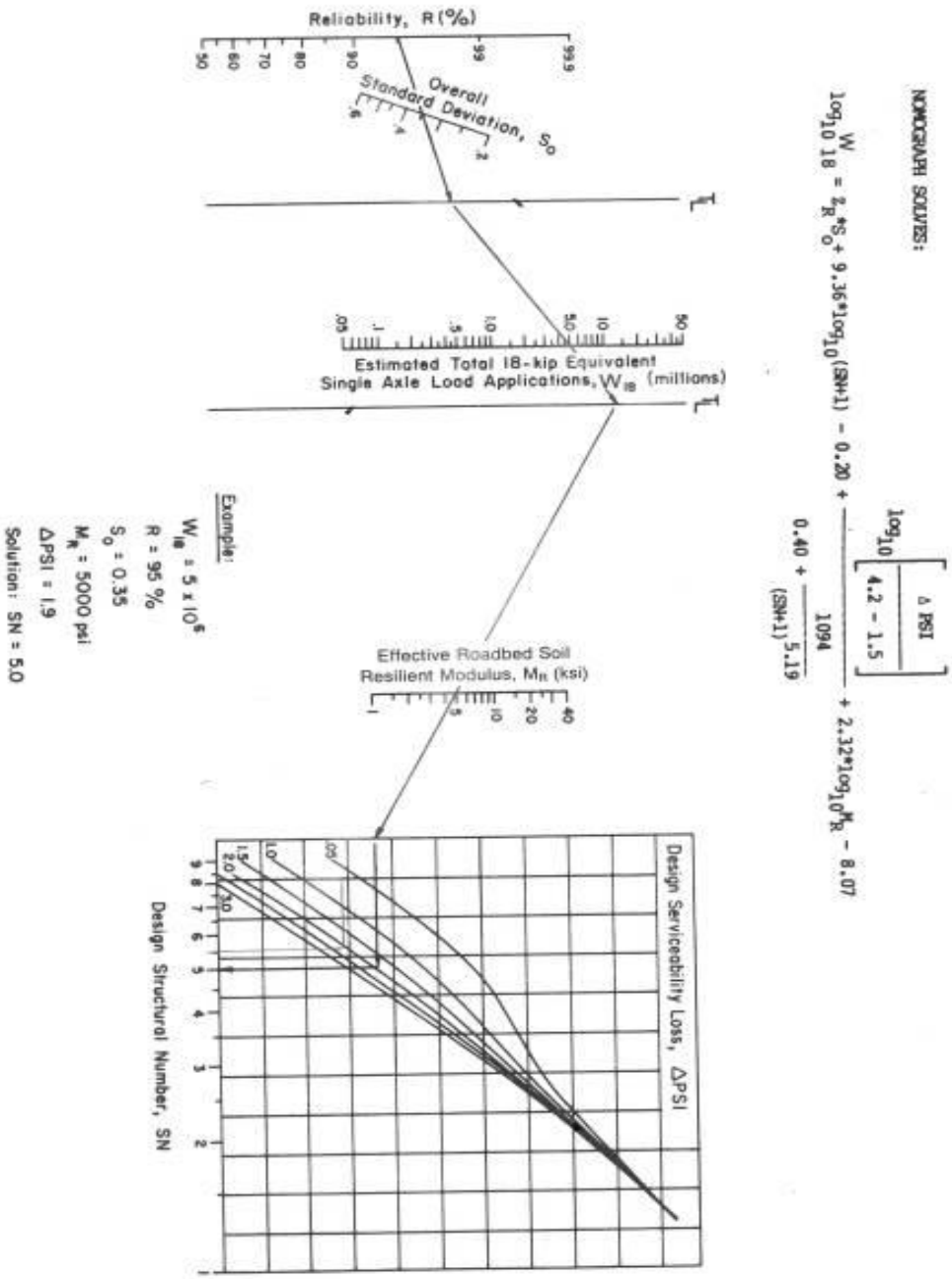


Figure 18: 1993 AASHTO Pavement Design Nomograph and Equation (1993 AASHTO, pg. II-32)

6.2.2 Step #2: Determining required structural number ($SN_{req'd}$)

The design inputs determined in Step #1 are used to calculate $SN_{req'd}$. $SN_{req'd}$ is the SN required for satisfactory pavement performance over the design life. The design equation found in the 1993 AASHTO Guide may be calculated using the nomograph in Figure 18, a spreadsheet or the DARWin software. Utilizing the DARWin software is the preferred method of determining $SN_{req'd}$. A design example usring DARWin is found later in this Section.

6.2.3 Step #3 Design a flexible pavement structure with SN_{des} >= $SN_{req'd}$

Step # 3 consists of choosing the material type and thickness for each pavement layer.

The first task is to choose which materials are to be used for the pavement structure. For flexible pavements, the top layer will always be PMS (PMS). However, the pavement designer may choose to utilize different types of materials to use beneath the PMS. MDT's pavement types are as follows:

- Two layer pavements: PMS under laid by base course (see Figure 19). The base course usually consists of either crushed aggregate course (CAC) or cement treated base (CTB) placed upon the finished subgrade.

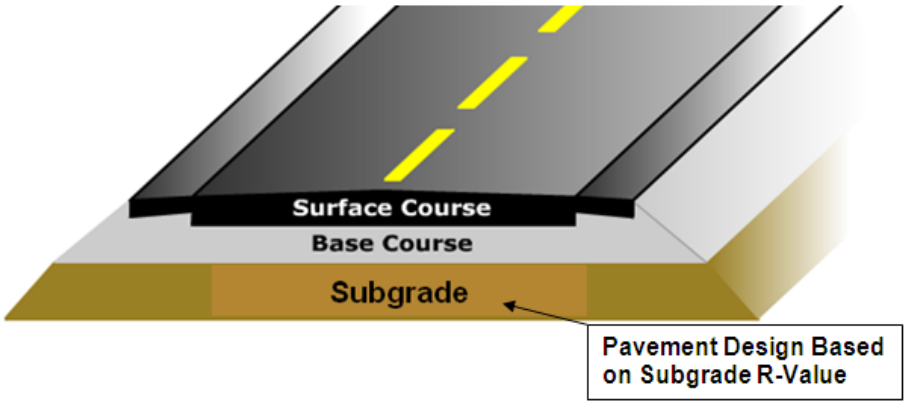


Figure 19; Two Layer Pavement Section

- Three layer pavements: Refers to PMS layer under laid by a base course and subbase course (see Figure 20). The subbase course usually consists of a drainable pit run granular material, but may consist of other granular materials that are locally available.

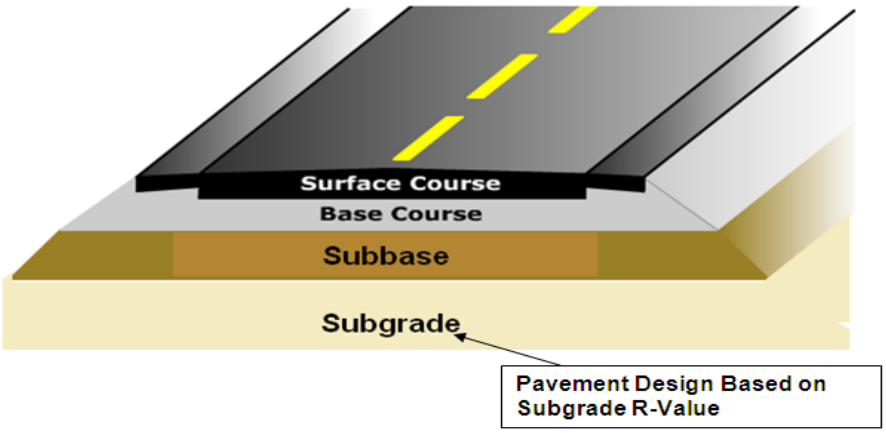


Figure 20: Three Layer Pavement Section

- Special Borrow Pavement: A Special borrow pavement is either a two or three layer under laid with a 2' thick special borrow layer (see Figure 21). Special borrow is usually specified based upon R-Value, and usually consists of a granular material that is both locally available and is better quality than the native subgrade. Special borrow is often used to reduce the thickness of the overlying layers, to aid in constructability by "bridging" underlying weak subgrade soils, or to provide more granular material thickness to mitigate frost heaving.

  There are instances where less than 2' special borrow may be used. When less than 2' special borrow is specified, the special borrow should be treated like a subbase material and designed similar to a three layer pavement.

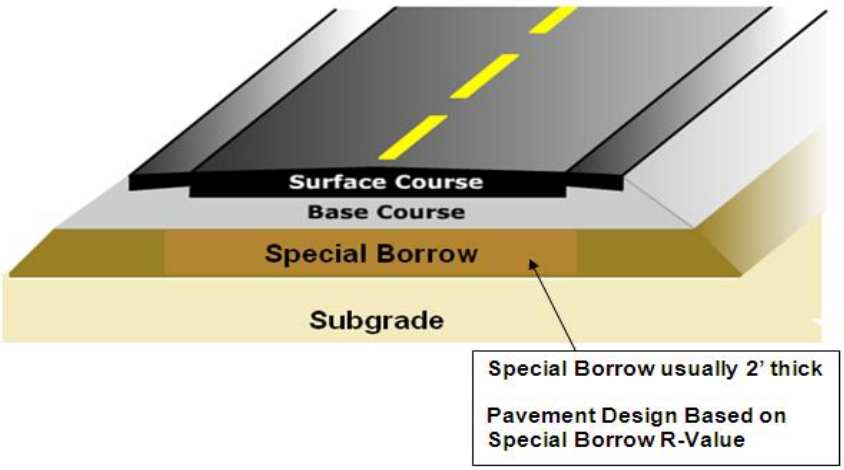


Figure 21: Special Borrow Pavement Section

1993 AASHTO Layered Design Analysis Overview

In the 1993 AASHTO Guide, Section 3.1.5 describes the layered design analysis. The layered design analysis is a procedure used to determine the minimum pavement layer thicknesses needed to "protect" the pavement layers below it. For example, for an Interstate pavement the

layered design analysis may show that the PMS layer must be at least 0.5' thick to "protect" the underlying base layer.

The layered design analysis is a very important procedure to calculate minimum PMS layer thicknesses since PMS layers may crack prematurely if they are too thin. However, the Designer does not have to conduct a layered design analysis. Instead, utilize the minimum PMS, CAC, CTB, and subbase layer thicknesses shown in
Table 2, which were calculated based on layered design analysis.

The designer should design multiple pavement sections for each project. For example, when designing a high-volume, thick pavement section the designer may choose to design all 3 pavement types (two layer, three layer, and special borrow) and provide all designs to the project manager. The project manager can compare all designs while designing the road alignment and grade to determine which option is the most cost effective.

The use of alternate typical sections can increase competition and reduce the possibility of Value Engineering proposals by Contractors. This refers to bid documents that include multiple typical sections with different pavement types or materials, and allowing the Contractor to bid on the alternate that is most cost effective. Examples of alternate typical sections are crushed aggregate course (CAC) vs. cement treated base (CTB) and flexible (asphalt) vs. rigid (concrete) pavements. Due to the variance in typical thicknesses, the designer will need to develop alternate plan and profile sheets to match bridge ends, approaches and other fixed elevations. In addition, alternate typical sections may affect grading quantities, hydraulic features and construction limits. The benefits of alternate sections must be weighed against the additional resources and time required to develop multiple designs.

After the pavement type(s) have been selected, the next step is to design the pavement layer thicknesses. SN_{Des} and pavement layer thicknesses are designed using the following equation:

$$SN_{Des} = a_{PMS*}d_{PMS} + a_{Base*}d_{Base} + a_{Subbase*}d_{Subbase}) \geq SN_{Req'd}$$

Where:

SN_{Des} = SN of design pavement section

$SN_{Req'd}$ = SN required for satisfactory pavement performance over the design life.

a_{PMS}, a_{Base}, a_{Sub} = structural-layer coefficients of PMS, base, and subbase layers, respectively, (

Table 3)

d_{PMS}, d_{Base}, d_{Sub} = thickness of PMS, base, and subbase layers, respectively, (
Table 2)

The following steps are based upon the equation above and can be used to design pavement layers:

Step #1. Select PMS thickness: PMS thickness is chosen based upon daily ESALs and the pavement location as shown in

Step #2. Table 2. Use the following equation to calculate the amount of structure provided by the PMS:

$$SN_{PMS} = \text{PMS structural coefficient} * \text{PMS thickness} = a_{PMS} * d_{PMS}$$

Where: a_{PMS} = PMS structural coefficient (

Table 3)

d_{PMS} = PMS thickness (ft.) (

Table 2)

Step #3. Select Base thickness: After the PMS thickness has been chosen, calculate SN that will be provided by the base course. For pavements without subbase use the following equation:

$$SN_{base} \geq SN_{Des} - SN_{PMS}$$

Calculate base course thickness:

$$d_{base} = SN_{base} / a_{base}$$

Where:

a_{base} = Base course structural coefficient (

Table 3)

d_{base} = Base course thickness (ft.) (

Table 2)

Step #4. Select Subbase Type and Thickness (when applicable): After both the PMS and Base course thicknesses have been designed, calculate the structure that will be provided by the subbase course:

$$SN_{Sub} = SN_{Des} - SN_{PMS} - SN_{Base}$$

Calculate subbase course thickness:

$$d_{Sub} = SN_{Sub} / a_{Sub}$$

Where:

a_{Sub} = Subbase course structural coefficient (

Table 3)

d_{Sub} = Subbase course thickness (ft.) (

Table 2)

Minimum Pavement Layer Thicknesses

The recommended PMS, Base, Subbase, and Special Borrow thicknesses are shown in

Table 2. Plant mix thickness is based on daily equivalent single axle loadings (ESALs) based upon a 20-year pavement design life.

Table 2: Pavement Layer thicknesses

PMS	
ESAL's (Daily)	**PMS Thickness***
>2000	0.70'
1000 – 2000	0.60' – 0.70'
501 - 1000	0.50 – 0.60'
201 - 500	0.40 - 0.50'
101 – 200	0.30 – 0.40'
0 - 100	0.30'*

*In certain cases 0.25' of PMS can be used for ½" or 3/8" mix, when budgets are constrained.

Base Course / Subbase / Special Borrow	
Material	**Thickness**
Crushed Aggregate Course (CAC)	0.65' min.
Cement Treated Base (CTB)	0.65' min.
CAC/CTB pulverized in place	0.50' min.
Subbase Course	0.65' min
Special Borrow	2.0'

Other Situations	PMS Thickness
• Urban Curb & Gutter • Crossovers • Rest Areas	0.40' min.
Mainline Interstate Pavements	0.50' min.
Non-Mainline Interstate Pavement including Interchanges	0.4' min.
Approaches	0.2' min.
Bike Path	0.2' min.

Pavement Layer Structural Coefficients

For New or Reconstructed pavements, use the structural coefficients shown in

Table 3. There may be other materials available or recommended for surfacing or base course. It is up to the designer to investigate these products to determine structural coefficients.

Table 3: Structural Coefficients for New and Reconstructed Pavements

Virgin Materials	Coefficient per in.	Existing Materials	Coefficient per in.
PMS (All Grades)	0.41	PMS (All Grades)**	0.20 - 0.33
Crushed Aggregate Course (CAC)	0.14	Crushed Aggregate Course (CAC)	0.12
PMS / CAC Mixture (pulverized, pugmilled, or mixed in-place)	0.12	PMS / CAC Mixture (pulverized, pugmilled, or mixed in-place)	0.12
Cement Treated Base (CTB)	0.20	Cement Treated Base (CTB)	0.18 #
CTB Pulverized	0.16	CTB Pulverized	0.14 #
Cold Recycled Asphalt (CIR)/(CCPR)	0.30	Cold Recycled Asphalt (CIR)/(CCPR)	0.20
Subbase Material*	0.07 - 0.10	Special Borrow	0.07

* Structural Coefficients for subbase of lesser quality should be reduced as determined by the designer. When less than 2 feet special borrow is specified, it should be treated as sub-base by assigning it a structural number and including it in the pavement structure.

**Coefficient for existing PMS should be reduced based on stripping analysis. See Chapter 7(Table 15).

Higher values may be applied based on unconfined core test results.

6.2.4 Step #4: Specifying Pavement Materials

Specifying the pavement materials to use within the pavement structure is an important part of pavement design. The following section gives guidance on selecting pavement materials.

PMS Type Selection

The Designer is responsible for recommending PMS type, aggregate size, asphalt cement type, asphalt cement quantity, and the use of recycled asphalt concrete. MDT's primary surfacing type is Grade S, where the "S" denotes that the mixture design is done according to the SuperPave mix design procedure. Grade S can consist of three different aggregate sizes; ¾", ½", and 3/8". The size refers to the "nominal maximum aggregate size", or the sieve size one size greater than the first sieve to hold 10% or more aggregate by weight.

PMS type is selected primarily on project quantity (

Table 4) and project type (Table 6).

Table 4: PMS Type Selection Based Upon Project Quantity and Lift Thicknesses

Project Quantity[1]	PMS Type (Bid Item)	Lift Thickness	
		Min ~ Max	Recommended
≥ 5,000 tons[1]	Plant Mix Surf Gr S – 3/8 in	0.10 ~ 0.20'	0.10'
	Plant Mix Surf Gr S – 1/2 in	0.12 ~ 0.25'	0.15'
	Plant Mix Surf Gr S – 3/4 in	0.15 ~ 0.30'	0.15 ~ 0.20'
< 5,000 tons[1]	Commercial Plant Mix-PG 70-28[2]	0.15 ~ 0.30'	0.15 ~ 0.20'
	Commercial Plant Mix-PG 64-28	0.15 ~ 0.30'	0.15 ~ 0.20'
	Commercial Plant Mix-PG 58-28	0.15 ~ 0.30'	0.15 ~ 0.20'
	3/8" Grade S – PG 70-28	0.10 ~ 0.20'	0.10'
	3/8" Grade S – PG 64-28	0.10 ~ 0.20'	0.10'

1 Consider the total contract quantity when multiple projects are tied for letting. Different bid items are used for commercial and non-commercial plant mix. This affects how the oil and lime are paid as well as how Quality Assurance (QA) testing is conducted. QA incentive and disincentive differs for commercial and non-commercial mix.

2 Choose binder type based upon the "PG Binders" section within this manual.

Table 5: Basis of Plan Quantities for Flexible Pavements

<u>BASIS OF PLAN QUANTITIES</u>
(QUANTITIES FOR ESTIMATING PURPOSES ONLY)
COMP. AGGREGATE WEIGHT = 3700 LBS. PER CUBIC YARD
COMP. WEIGHT OF PL. MIX BIT. SURF. = 3855 LBS. PER CUBIC YARD
ASPHALT CEMENT – GRADE S – ¾" AGG. = 5.4% OF PL. MIX BIT. SURF.
ASPHALT CEMENT – GRADE S – ½" AGG. = 5.8% OF PL. MIX BIT. SURF.
ASPHALT CEMENT – GRADE S – 3/8" AGG. = 6.2% OF PL. MIX BIT. SURF.
HYDRATED LIME = 1.4% OF PL. MIX BIT. SURF.
BITUMINOUS MATERIAL = 8.5 LBS. PER GAL.

TACK (ASPHALT SURFACES) = 0.025 GAL. PER SQ. YARD (UNDILUTED)
TACK (ALL OTHER SURFACES) = 0.05 GAL. PER SQ. YARD (UNDILUTED)
SEAL = 0.42 GAL. PER SQ. YARD
COVER = 25 LBS. PER SQ. YARD
BLOTTER = 15 LBS. PER SQ. YARD
FOGSEAL = 0.05 GAL. PER SQ. YARD

Choosing PMS type based upon

Table 4 requires estimating the total PMS tonnage on a project. Use the following equation, along with the Information in Table 5 to estimate the asphalt tonnage on a project:

Project Length (miles) / project * 5280 ft. / mile * Roadtop Width (ft.)

* PMS thickness (ft.) * 1 yd^3/27 ft^3 * 3855 lbs. / yd^3 * 1 ton PMS / 2000 lbs.

= tons PMS / project

There are three major differences between Plant Mix Surf Gr S – #/# in. and Commercial Plant Mix:

Plant Mix Surf Gr S – #/# in. quality control / quality assurance (QC/QA) usually results in a superior, more consistent product. However, the quantity has to be over 5,000 tons to use this QC/QA method. Commercial plant mix utilizes a different method of QC/QA.

For Plant Mix Surf Gr S – #/# in, payment for asphalt cement and hydrated lime are separate from the payment for PMS aggregate. For commercial plant mix, payment for PMS aggregate includes asphalt cement and hydrated lime.

For commercial plant mix, the Contractor may choose to use ¾" Grade S or ½" Grade S. The PMS aggregate type is stipulated in the bid item title.

Do not use Grade B, C, or D. In the past, Grade B was specified in situations where a chip seal would not be placed, such as **bike paths. In these situations 3/8" Grade S would be preferred, but is more expensive than ¾" mix**. In addition, the use of 3/8" mix results in a second bid item.

Table 6: PMS Type Selection Based Upon Project Type

Project Type	PMS layer thickness	PMS Type[1]
Reconstruct, Major Rehab, and Widening	PMS Thickness <0.30'	Plant Mix Surf Gr S – ½ in Commercial Plant Mix-PG ##-##[1] 3/8" Grade S – PG ##-##
	PMS thickness ≥0.30'	¾" Grade S Volumetric[2] Commercial Plant Mix-PG ##-##[2]

Project Type	PMS layer thickness	PMS Type[1]
Overlay or Minor Rehab	PMS thickness ≥0.15'	Plant Mix Surf Gr S – ¾ in[2] Commercial Plant Mix-PG ##-##[2]
	0.15' > PMS thickness ≥ 0.10'	Plant Mix Surf Gr S – ½ in Commercial Plant Mix-PG ##-##[1] Plant Mix Surf Gr S – 3/8 in 3/8" Grade S – PG ##-##
	PMS thickness < 0.10'	Plant Mix Surf Gr S – 3/8 in 3/8" Grade S – PG ##-##

1 When specifying Commercial Plant Mix and ½" Grade S is desired, change Grade S specification language so it only allows ½" PMS aggregate.

2 If consideration is being given to using 3/8" mix, it is generally more cost effective to use ¾" Grade S in the lower lift(s). 3/8" Grade S is structurally equivalent to ¾" Grade S, but is more expensive due to the higher oil content.

Reclaimed Asphalt Pavement

Including recycled asphalt pavement (RAP) within PMS is recommended when it is cost effective and/or to reduce the amount of millings wasted. PMS w/ RAP is typically less expensive then virgin PMS.

RAP refers to asphalt millings produced during cold milling operations. One use for RAP is including it in new PMS, where the RAP is mixed into the PMS within the hot plant. During this process, the asphalt cement within the RAP is heated and blended with the virgin binder. This lowers virgin binder content between 0.2% to 2.2% when using 10 to 50% RAP, respectively.

In the past, MDT has incorporated up to 50% RAP in PMS, usually within the bottom lifts of thick PMS layers. In 2013, MDT began specifying only Grade S PMS. Grade S has tighter specifications than previous PMS mixes, making it more difficult to incorporate RAP. Due to these difficulties, the amount of RAP incorporated in to Grade S mixes is limited to 15% within the top lift, and 30% within the bottom lifts.

The following guidelines are used when specifying RAP:

- MDT's Grade S PMS specification allows up to 15% RAP on all mixes, leaving it up to the Contractor to use up to 15% RAP if they decide to do so on any lift within the PMS layer.

- If the designer wants to dictate the use of RAP on a project use the bid item "Plant Mix Bit Surf Gr S - ## RAP", where ## can be 3/8", ½" or ¾". The amount of RAP to be used in the PMS is specified within the special provision.

- See the PG binder section below and Asphalt Content section for more information on specifying RAP.

On reconstruction projects, RAP may be used in other ways such as:

- Traffic gravel
- Supplement to base gravel (needs to be mixed with crushed aggregate course)
- Digout backfill
- Shoulder gravel
- Guardrail widening
- Detour Surfacing

Performance Grade (PG) Binder

Performance Grade (PG) Binder refers to the type of asphalt cement used within the PMS mixture. Asphalt cements are specified by PG grade, for example "PG 70-28". The numbers, 70 and -28, refer to the pavement temperatures in ºC where the asphalt cement should perform well. For example, a PG 70-28 binder should be expected to be rut resistant at pavement temperatures up to 70 ºC, and be crack resistant down to negative 28 ºC.

Table 7 shows binder types readily available in Montana. Binder grades not shown in the table may be difficult to procure and are not recommended. Use **Table 7 along with the guidance below the table** to specify PG binder types.

Table 7: PG Binder Selection

Design Parameter	Binder Type
Interstate Roadway, Rest Areas and Scale Sites, or Daily ESAL ≥ 400	PG 70-28
Roadway on National Highway System or daily ESALs ≥ 50 or AADT ≥ 1200	PG 64-28
All other roadways	PG 58-28

The following are situations where the **binder grade may be bumped up** from PG 58-28 to PG 64-28 or from PG 64-28 to PG 70-28:

- Construction / Reconstruction projects – often it is desirable to use polymer modified binders for these projects. PG 64-28 and 70-28 are typically polymer modified.

- Urban routes with stop and go traffic and/or high traffic volumes.

- Any route where reoccurring rutting is observed

Consideration should be given to using a lesser PG binder grade in lower lifts when 0.4' or more new PMS is required. As a general rule of thumb, binder grade can be dropped one grade within PMS located more than 0.2' below the pavement surface.

For **PMS with more than 20% recycled asphalt pavement (RAP)** and if the binder required is PG 70-28, then reduce to PG 64-28. Consideration can be given to reducing the PG binder grade to a non-polymer modified binder on low volume roads. This accounts for the stiffening affect caused by the oxidized binder within the RAP.

Asphalt Content Selection

The asphalt cement content, (aka AC content, or "percent asphalt") refers to the amount of asphalt cement within the PMS mixture. Asphalt cement contents for ¾" mix typically range from 4.5 to 6.5% and are measured as a percentage of the total asphalt mixture by weight. Typically, PMS with larger aggregate have lower asphalt quantities than small aggregate. On average ¾" Grade S has 5.4% asphalt, while 3/8" Grade S has about 6.2% asphalt. Table 8 provides guidance for asphalt content selection.

Table 8: Asphalt Content Selection

PMS Type	Asphalt Content			
Commercial PMS [1]	½" Grade S		¾" Grade S	
	5.8%		5.4%	
3/8" Grade S – PG ##-##	6.2%			
Plant Mix Surf Gr S – #/# in	Estimate asphalt content for each individual project. To assist in the selection, a state map showing as-produced asphalt contents from previous construction projects can be found at the following websites: http://mdtinfo.mdt.mt.gov/mdt/docs/percent_asphalt_map.pdf http://www.mdt.mt.gov/other/webdata/external/cadd/design_memos/2011-04-29_PROJECT_SPECIFIC_ASPHALT_CONTENT.PDF The designer can also utilize the map in Figure 22, but the websites have better resolution. Other methods to estimate asphalt content are to speak with District construction/materials personnel or to utilize the QA Suite software.			
Plant Mix Bit Surf Gr S - ## RAP[2]	10% RAP	25% RAP	40% RAP	
	Subtract 0.4%	Subtract 0.9%	Subtract 1.8%	

1 Include all 3 asphalt contents within Basis of Plan Quantities

2 Estimate a project specific asphalt content in accordance with the method above. Subtract the value given unless project specific information is available for the oil content for the RAP that would dictate otherwise.

A few items to consider when utilizing the Project Specific Asphalt Map:

- In 2004 the department began using the Hamburg rut test. The test typically reduced asphalt contents by about 0.3%. As a result, subtract 0.3% to projects built in 2004 and earlier to estimate today's asphalt contents.

- Beginning in 2013, mix design gyrations on high traffic roads (>401 daily ESALs) were lowered to increase asphalt content. This has resulted in an 0.2% increase in asphalt content. As a result, add 0.2% asphalt content to estimate today's asphalt contents (in addition to the guidance given in bullet 1 above).

- There are relatively few ½" Grade S projects on the map. ½" Grade S asphalt contents can be estimated based upon ¾" Grade S asphalt contents. Do this by adding 0.4% to ¾" Grade S projects. For example, if a project on the map indicates a ¾" Grade S project has 5.4% asphalt, assume that the same project would have 5.8% asphalt if it had been ½" Grade S.

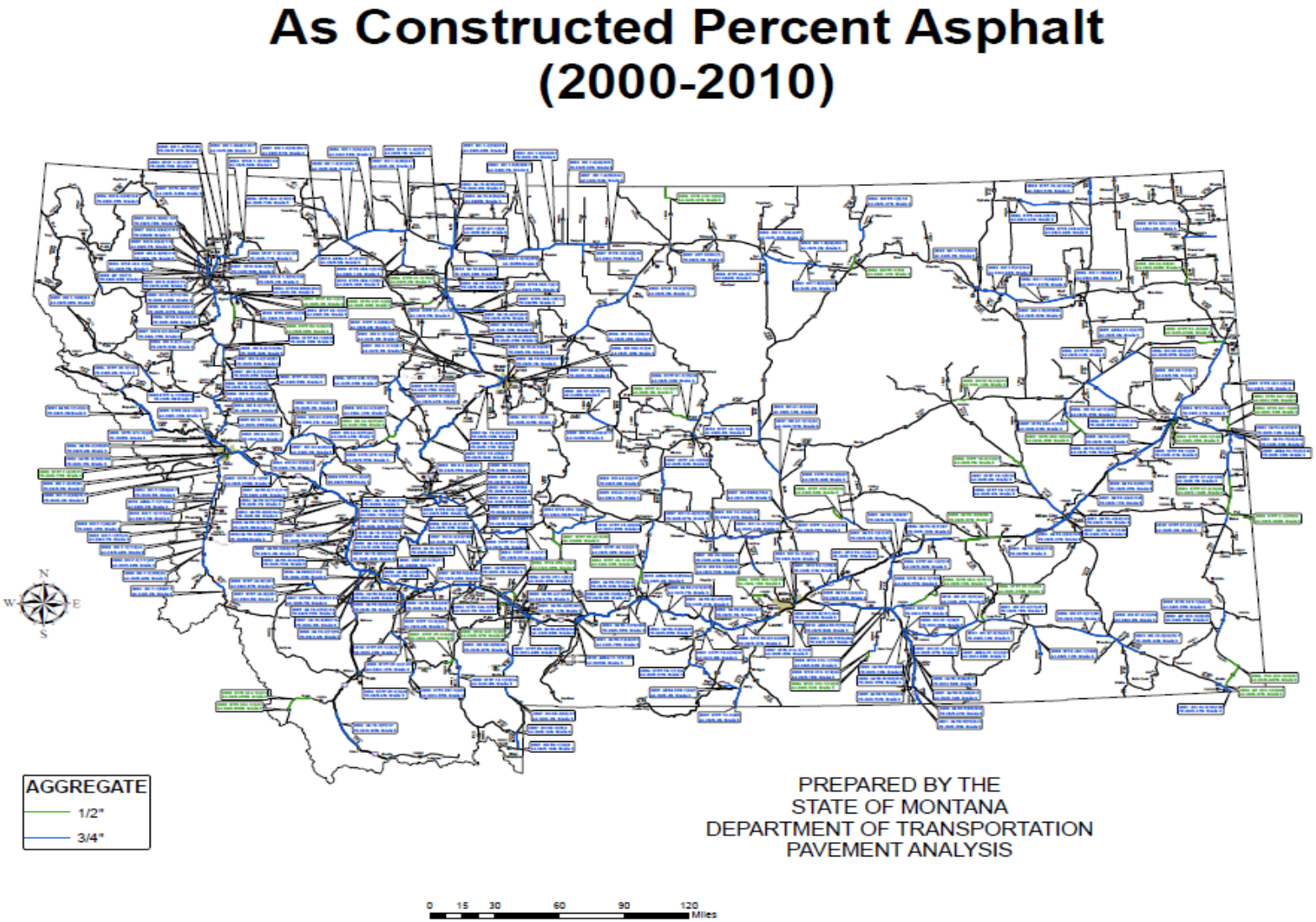


Figure 22: Project Specific Asphalt Map

Pavement Base Course

MDT primarily uses 2 types of base course: untreated crushed aggregate course (CAC) and Cement Treated Base (CTB).

Crushed Aggregate Course

MDT's most common pavement type is PMS under laid with aggregate base. Aggregate Base Course is now referred to as crushed aggregate course (CAC). The current CAC specification allows the Contractor to choose between Crushed Base Course Grade 5A (2"-) and Grade 6A (1.5"-), and also allows the Contractor to choose whether or not to use crushed top surfacing grade 2A (3/4" -) within the top 0.15' of CAC.

MDT minimum CAC thickness is 0.65'. That depth includes crushed top surfacing.

In recent years, recycling RAP into virgin CAC has become more common, even on reconstruction projects. This is a cost effective way to reuse RAP, and results in a competent base course provided it is manufactured as follows:

- It is important to have a uniform blend of CAC and RAP within the pulverized mixture. To achieve a uniform blend, the material can be mixed in either a pugmill (off-site) or with a pulverizer (in-place). Blade mixing is not allowed.

Within the blend, pulverized PMS should comprise a maximum 50% of the pulverized mixture. Because of variability in existing surfacing thickness, some portions of the project may have a higher percentage. MDT research has shown that the mixture strength decreases when RAP comprised more than 60% of the mixture (Mokwa, 2005).

Compacted pulverized material can swell up to 12.5% by volume.

Cement Treated Base

MDT often uses Cement Treated Base Course (CTB) where it is cost effective. CTB courses are generally 2/3 the thickness of comparable CAC courses. CTB is often used in areas without economical access to gravel and/or where a thinner overall pavement section is desirable. The CTB aggregate requires a smaller nominal maximum size (1 inch) than CAC aggregate (1 ½ inch).

For reconstruction or new construction projects, the minimum CTB thickness is 0.65'.

CTB can be manufactured with a pugmill (off-site) or with a pulverizer (in-place). The ability to manufacture CTB with a pulverizer makes it ideal for full depth reclamation (FDR) of existing pavements.

Pavement Subbase Course

While MDT's most common pavement type is a two-layer pavement, three-layer pavements consisting of a surfacing, base, and subbase layer should also be considered. These pavements tend to be more cost effective on projects with:

- CAC layers within two-layer pavements are greater than 1.10' thick.
- Subgrades with soft stiffness (M_R < 4,000 psi) and traffic greater than 50 daily ESALs.
- Subgrades with medium (9,000 psi > M_R > 4,000 psi) stiffness and traffic greater than 800 daily ESALs.

Subbase should consist of material meeting the following requirements:

- 3" maximum aggregate size,
- less than 15% passing the No. 200 sieve, and
- material passing the No. 40 sieve must have a maximum liquid limit and plasticity index of 30 and 6, respectively.

Subbase material meeting these material requirements can be assumed to have a resilient modulus (MR) = 12,000 psi and structural coefficient = 0.07. Higher structural coefficients can be utilized if appropriate testing data is approved by MDT. In locations where material meeting the specifications above is unavailable, other granular materials can be used as subbase material. An example of this sort of material is the sand deposits in eastern Montana. The structural coefficient for these materials will usually range from 0.07 – 0.08/in.

Also, use the guidance below when specifying subbase:

- Adequate base course thickness is needed to "protect" subbase materials from high stresses and strains. The layered analysis approach (Figure 3.2, 1993 Guide) is recommended for determining base course thickness in three-layer pavements.
- Subbase material should daylight out into the pavement inslopes, similar to special borrow.
- The frequency of quality control / quality assurance (QC/QA) testing should be similar to base course material. Special borrow QC / QA should not be used for subbase material.
- Minimum subbase thickness is 0.65'.

6.2.5 Step #5: Send Pavement Design Memo to Project Design Manager.

Now that the thickness design and pavement **materials** have been determined, the last task is to send a paper copy of the surfacing design memo to road design staff via interdepartmental mail.

Montana Department of Transportation
Helena, MT 59620-1001

Memorandum

MASTER FILE COPY

To: Paul Ferry, P.E.
Highways Engineer

Thru: Daniel W. Hill, P.E.
Pavement Design Engineer

From: Stephen A. McEvoy
Surfacing Design

Date: July 24, 2013

Subject: STPP 3-2(77)50
Bynum-South
CN 7637

Listed below are preliminary surfacing typicals for the above reconstruction project. This recommendation completes Surfacing Design activity 600.

Surfacing Alternate No. 1 – CAC Reconstruction
0.30' Plant Mix Bituminous Surfacing
1.25' Crushed Aggregate Course
1.55'
Design R-Value = 5

Surfacing Alternate No. 2 – CAC with Special Borrow Reconstruction
0.30' Plant Mix Bituminous Surfacing
0.65' Crushed Aggregate Course
0.95'
Design R-Value = 30

This design is based on the top two feet of subgrade having an R-value ≥ 30.

Surfacing Alternate No. 3 – CTB Reconstruction
0.30' Plant Mix Bituminous Surfacing
0.80' Cement Treated Base
1.10'
Design R-Value = 5

Surfacing designs are based on 2012 traffic data projecting 21 daily ESALs. Recommendation design life is 20 years in accordance with AASHTO design procedures. PG 64-28 binder, ¾" Grade ***S*** aggregate with an asphalt content of 5.6 % are recommended. PG Binder was determined as per April 7, 2005 Materials Bureau policy memorandum.

R-values representing subgrade soils were used to determine surfacing thicknesses. Soil classes are not shown because R-values within a particular soil type range considerably. Soils within the project area are most commonly classified as A-6, and have been assigned an R-value of 5.

Please contact me with questions you may have concerning this recommendation.

DWH:SAM

cc: D. Wilmot, P.E.
D. Krings, P.E.
RJ Snyder, P.E.
Surfacing File
Materials Bureau File
Geotechnical File

Figure 23: Example Surfacing Design Memo

Figure 23 shows an example surfacing design memo. The following list includes items that are included within the memo:

- To: Highways Engineer
- Thru: Pavement Design Engineer (with signature)

- From: Designer (with signature)
- Date
- Subject Project Number, Name, and CN Number
- Designate whether memo is Preliminary (Act. 600) or Final design (Act. 604)
- PMS, CAC, and Total Thicknesses
- Design R-Value
- Daily ESALs assumed for Pavement Design
- Plant Mix Type
- Use of RAP and PMS RAP content
- Design Life Length (yrs.)
- Construction methods and sequencing for pulverization typical sections
- Binder type
- Asphalt Content
- CC'd to District Administrator, Road Design Engineer, Road Design Project Manager, Bridge Engineer (for Bridge Lead projects), and Surfacing, Materials, and Geotechnical Files

After completing the surfacing design memo with Microsoft Word, save the .doc file using the 7 digit Construction Number (CN) as the file name. Save the .doc files in the following share drive location: \\astro\rdrtr\projects\#######, where ####### is the 7 digit project UPN. Once the memo has been submitted for distribution and saved, the EPS activity may be carded off.

When the paper copy of the memo is distributed, the original copy is stamped "Master Copy" with a green stamp by the Department's mail staff, and sent back to surfacing design. This "Master Copy" is stored in the surfacing design project file.

There will be instances when the design memo is sent electronically via email. When this is necessary, the Word file (.doc) should be converted to an Adobe Acrobat File (.pdf) before sending it. The purpose of this is that a .pdf file cannot be altered.

Low Volume Flexible Pavement Design

Low Volume Roads are defined as those having 137 or less daily ESALs during the 20 year pavement design period. The 1993 AASHTO Low Volume Design (Table 11) should only be used for Secondaries or when a reliability of 75% is deemed appropriate.

The “standard” pavement design method described in the 1993 AASHTO Guide for the Design of Pavement Structures (1993 Guide) tends to be overly conservative when designing low-volume pavements, especially on poor subgrade soils. Part II, chapter 4 of the 1993 Guide page II-77 through II-81 presents an alternative method of designing low volume pavements that is less conservative.

Low Volume Pavement Design and rehabilitation are designed similarly as regularly pavement designs, except that SN_{Des} is calculated differently, as summarized in the rest of this section.

Calculating Low-volume Road SN_{Des} is relatively simple, requiring only the daily ESALs over the design life (typically a 20-year period) and the subgrade soil quality. Use Table 11: (AASHTO 4.7) Low Volume Pavement Design SN to calculate SN_{Des} based upon the traffic and subgrade quality defined in Table 14 and Table 15.

Table 9: (AASHTO 4.2.1) Traffic Level Categories for Low Volume Pavement Design

Category	Traffic Level – Daily ESALs*
High	96-137
Medium	55-82
Low	7-41

*Based upon 20-year Design Life

Table 10: (AASHTO 4.3) Subgrade Characterization M_R (PSI) for Low-volume Pavement Design

U.S. Climatic Region	Relative Quality of Roadbed Soil				
	Very Poor	Poor	Fair	Good	Very Good
I	2,800*	3,700	5,000	6,800	9,500
II	2,700	3,400	4,500	5,500	7,300
III	2,700	3,000	4,000	4,400	5,700
IV	3,200	4,100	5,600	7,900	11,700
V	3,100	3,700	5,000	6,000	8,200
VI	2,800	3,100	4,100	4,500	5,700

* 1: Since Montana is in Region VI, use VI for design.

Table 11: (AASHTO 4.7) Low Volume Pavement Design SN

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very good	High	2.6–2.7*	2.8–2.9	3.0–3.2	2.4–2.5	2.7–2.8	3.0–3.2
	Medium	2.3–2.5	2.5–2.7	2.7–3.0	2.1–2.3	2.4–2.6	2.7–3.0
	Low	1.6–2.1	1.8–2.3	2.0–2.6	1.5–2.0	1.7–2.2	2.0–2.6
Good	High	2.9–3.0	3.0–3.2	3.3–3.4	2.7–2.8	3.0–3.1	3.3–3.4
	Medium	2.6–2.8	2.7–3.0	3.0–3.2	2.4–2.6	2.6–2.9	2.9–3.2
	Low	1.9–2.4	2.0–2.6	2.2–2.8	1.8–2.3	2.0–2.5	2.2–2.8
Fair	High	3.2–3.3	3.3–3.4	3.4–3.5	3.0–3.2	3.2–3.3	3.4–3.5
	Medium	2.8–3.1	2.9–3.2	2.7–3.3	2.7–3.0	2.8–3.1	3.0–3.3
	Low	2.1–2.7	2.2–2.8	2.3–2.9	2.0–2.6	2.1–2.7	2.3–2.9
Poor	High	3.5–3.6	3.6–3.7	3.7–3.9	3.4–3.5	3.5–3.6	3.7–3.8
	Medium	3.1–3.4	3.2–3.5	3.4–3.6	3.0–3.3	3.1–3.4	3.3–3.6
	Low	2.4–3.0	2.4–3.0	2.5–3.2	2.3–2.8	2.3–2.9	2.5–3.2
Very poor	High	3.8–3.9	3.8–4.0	3.8–4.0	3.6–3.8	3.7–3.8	3.8–4.0
	Medium	3.4–3.7	3.5–3.8	3.5–3.7	3.3–3.6	3.3–3.6	3.4–3.7
	Low	2.6–3.2	2.5–3.3	2.6–3.3	2.5–3.1	2.5–3.1	2.6–3.3

Pavement Reconstruction Design Example

This design example summarizes the design of a reconstruction pavement section for the Jct US 2 – North of Chester project. This is a low-volume roadway (less than 137 ESALS per AASHTO), and the example includes both low-volume and conventional design methods. The following information presents a summary of items that may affect the pavement design:

Project Name:	Jct US 2 – North of Chester
Project Number:	STPS 409-1(7)0
Construction Number (CN):	6164
Existing Surface:	Thin gravel on native soil
Project Limits:	RP 0.0 to 0.6
Project Type:	Reconstruction with flexible pavement
District:	Great Falls
Geography:	Flat terrain, plains
Climate:	Low Precipitation, Frozen during winter
Traffic:	Daily ESALs = 54
	Daily AADT (Average Annual Daily Traffic) = 360
Soils:	poor to very poor stiffness, cohesive, and fine grained

Information Available to Designer:

- Soil Survey from District Materials Lab
- Falling Weight Deflectometer Subgrade Resilient Modulus

Step #1: Determine Flexible Pavement Design Inputs

Traffic: The daily was provided by MDT's Traffic Data Unit, the daily ESALs are 54 and the daily AADT is 360 vehicles. Calculate the ESALs during the 20-year Design Life (Estimating Traffic – Special Cases):

(54 ESALs/day)(365 days/year)(20 year/design life) =

394,200 daily ESALs/Design Life

Initial and Terminal Serviceability: 4.2, 2.5, and (Initial and Terminal Serviceability)

Reliability Level and Standard Deviation: 75% and 0.45 (Reliability Level and Standard Deviation)

Drainage Coefficient: 1 (Drainage coefficients)

Number of Construction Stages: 1 (Number of Construction Stages)

Subgrade Resilient Modulus: This pavement design is for the first 0.6 miles of a larger 9.4 mile project. Figure 24 shows the District Soil Survey, where the orange outlining designates the subgrade soils that will likely be encountered.

The 85th percentile R-Value ($R_{85\%}$) is used for design (Chapter 6). In this case there are 5 subgrade R-Value samples, 4 of which are A-6. Recall that A-6 and A-7 soils are automatically given an R-Value = 5. Using the process outlined in Subgrade Resilient Modulus, it is determined that $R_{85\%} = 5$.

$R_{85\%}$ is converted to $M_{R(Des)}$ as follows (Chapter 6):

$$M_{R(Des)} = 1155 + 555*R_{85th\%}$$
$$= 1155 + 555*(5)$$
$$= 3{,}930 \text{ psi}$$

$M_{R(Des)}$ =3,930 psi will be rounded down to $\mathbf{M_{R(Des)} = 3{,}750}$ psi based upon engineering judgment.

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sed 2/8/10

Montana Department of Transportation
Preconstruction Soil Survey Data
and Special Recommendations Relative to Subgrade and Road Surface Design

Project Number STPS 409-1(7)0 PE Project Name Jct US 2-N of Chester Length 9.4 miles County Liberty Date 01/14/13
UPN 6164 Contract No. Stanley Kuntz
Submitted by N. Procto Title Area Lab Supervisor District Materials Supervisor Dist. Preconstruction Engineer

Hole Number	Sample Number	Date	Reference to Centerline – Location of Boring	Depth	Representing Stationing	Soil Class (MT214)	LL	PI	10 Mesh (2.00 mm)	40 Mesh (.425 mm)	200 Mesh (.075 mm)	In Place Density	Specific Gravity	Density Maximum Dry	Moisture Percent Natural	Moisture Percent Optimum	Water Table Depth to	(AASHTO T190) "R" Value
9	2	10/26/12	4.0 ft RT	2.0-5.4 ft	211+17	A-6(9)	31	11	99	99	90.5		2.679	110.1	9.2	15.7		
10	1	10/26/12	7.0 ft LT	0-5.6 ft	237+38	A-4(0)	20	6	99	96	38.8		2.672	118.8	7.6	12.6		38
11	1	10/26/12	13.0 ft LT	0-4.0 ft	264+00	A-6(12)	35	19	96	93	73.7			112.4	12.5	14.7		
11	2	10/26/12	13.0 ft LT	4.0-5.8 ft	264+00	A-6(13)	37	20	97	95	73.0			109.5	12.2	16.2		
12	1	10/26/12	1.0 ft LT	0-2.4 ft	292+15	A-6(1)	32	15	59	52	37.2			135.3	6.5	10.6		
12	2	10/26/12	1.0 ft LT	2.4-5.1 ft	292+15	A-6(14)	40	24	92	87	69.4			110.5	12.7	16.8		
13	1	10/29/12	12.0 ft RT	0-5.3 ft	316+77	A-6(13)	36	20	93	89	70.1			112.1	14.7	15.3		

Figure 24: Sample Soil Survey Form

The other source for estimating Subgrade M_R is utilizing FWD information. This particular project is presently a gravel road, so FWD information is not available.

Step #2: Determine required Structural Number (SN_{des}) for low volume

1993 AASHTO Low-Volume Road Design

This road is considered low volume because it has less than 137 daily ESALs. This being the case, SN_{des} should be determined using both the AASHTO Low-Volume Method below and the conventional flexible pavement design method. This only applies to roads with a reliability of 75% unless case specific conditions dictate a lower value for NHS or primary routes. After calculating SN using both methods, the smaller of the two is chosen for pavement design.

The Daily ESALs fall in the low range of the Medium category, and the Designer made the decision to categorize the subgrade as very poor soils. Montana is entirely within Region VI: Dry, Hard Freeze, and Spring Thaw. Using these inputs in Table 11, an adequate low-volume structural number (SN) is 3.4.

Step #3: Determine the required Structural Number using the DARWin software

1993 AASHTO Conventional Flexible Pavement Design

It should be noted DARWIN runs on Windows XP as of 2014, MDT has converted to Windows 7. Because of this, the in-house designer will have to use a remote desktop connection to open DARWIN on another computer. DARWin is opened by double clicking on the desktop icon labeled "DARWin 3.1". Generally, an error message will occur, and the "Ok" button must be clicked several times before the program opens. The opening screen is below.

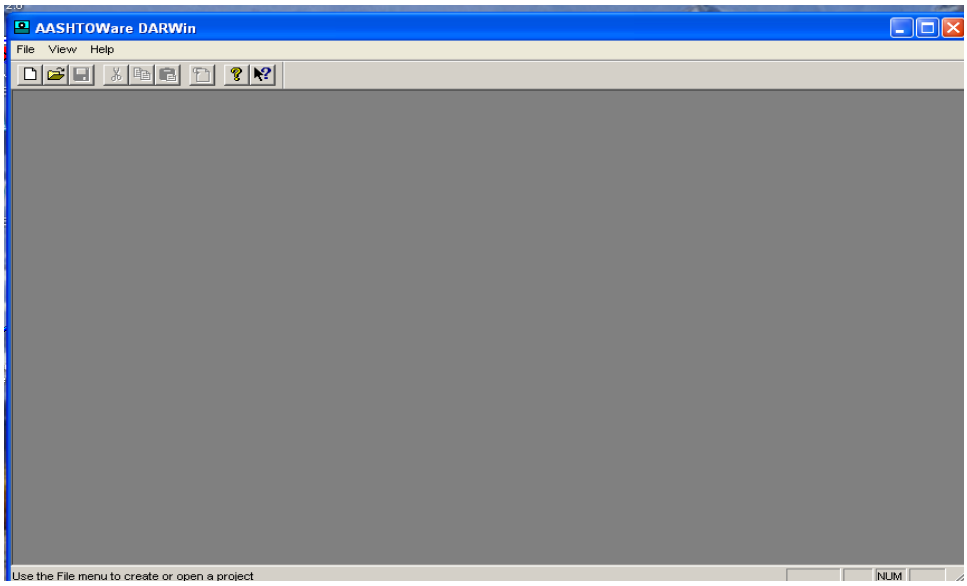


Figure 25: DARWin Opening Screen

Click "File" at the upper left, and select "New" to create a new project. The following screen (Figure 26) will appear. Type the UPN/Control Number for the project in the space under "Project Name", and click OK. This will bring up the template in Figure 27.

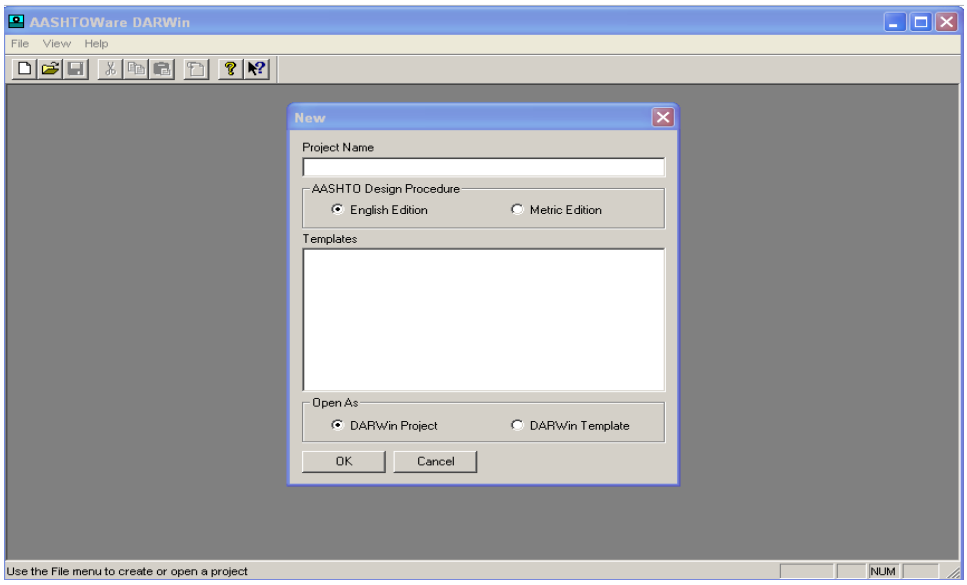


Figure 26: DARWin New project screen

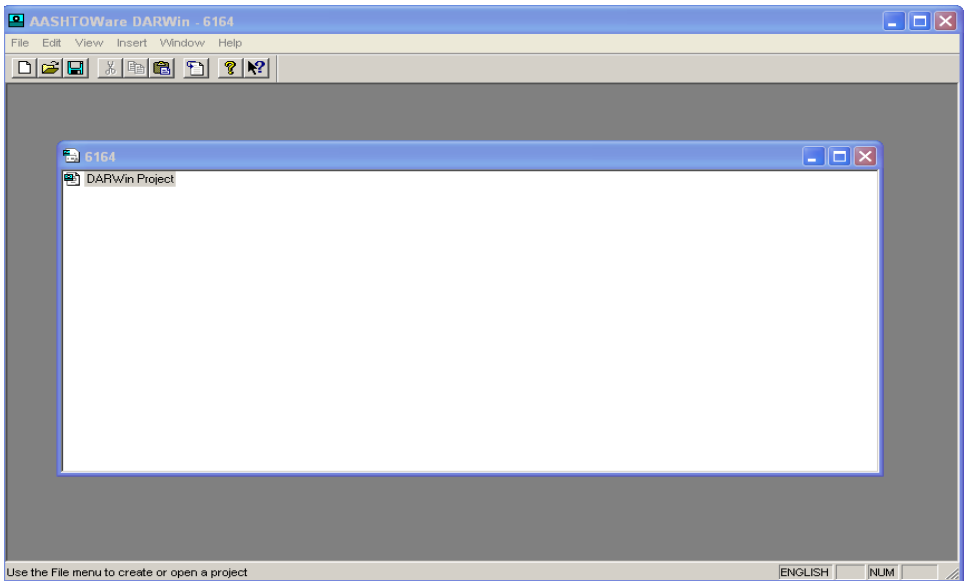


Figure 27: DARWin New project screen template

Click on DARWin Project in the template, click on “Insert” in the header and select “New Module”. The screen will look like Figure 28.

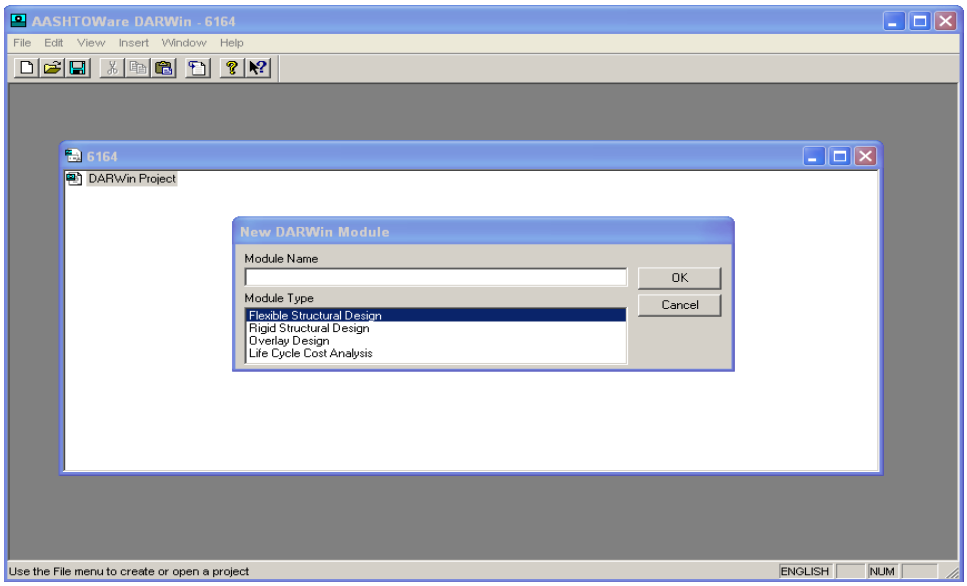


Figure 28: DARWin New Module template

Enter the CN number as the Module Name and click OK: the module has automatically selected Flexible Structural Design. The screen will look like Figure 29.

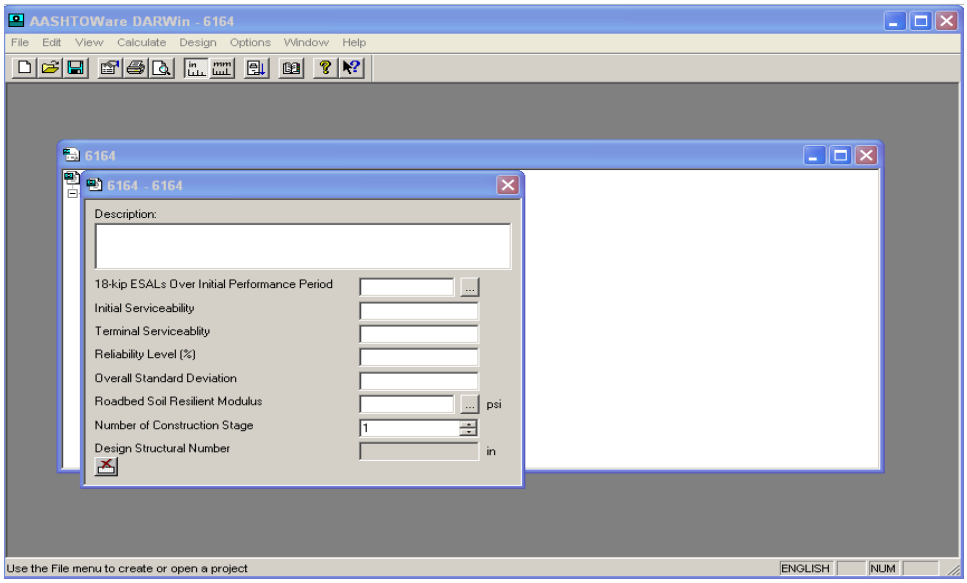


Figure 29: DARWin flexible structural design template

Type the project name and number under "Description." Next, enter design inputs from Step #1, and click the red "X" button. A DARWin **Design Structural Number of 3.46** will appear in the box as shown in Figure 30.

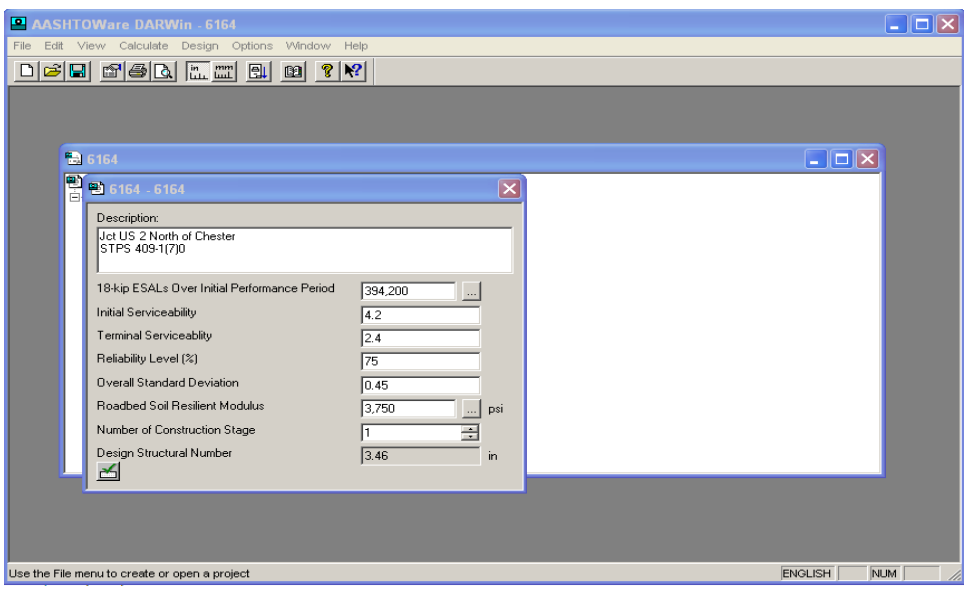


Figure 30: DARWin Completed design template

Recall that SN_{Des} is the lessor SN from the low-volume and conventional AASHTO designs. In this case, the low volume SN = 3.4 will be used for the pavement design.

Step #4: Design a flexible pavement structure with $SN_{Req,d} \geq SN_{Des}$

Click “Design” on the header and select Thickness Design. The template will look like Figure 31. Select the Specified tab at the top, and click the insert layer button indicated in Figure 32 two times to provide PMS and CAC layers.

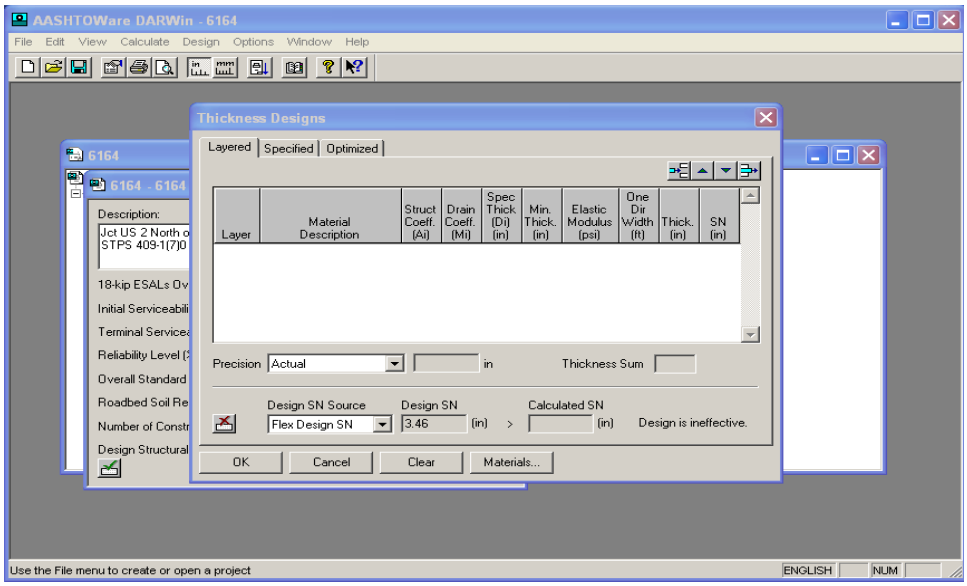


Figure 31: DARWin thickness design template

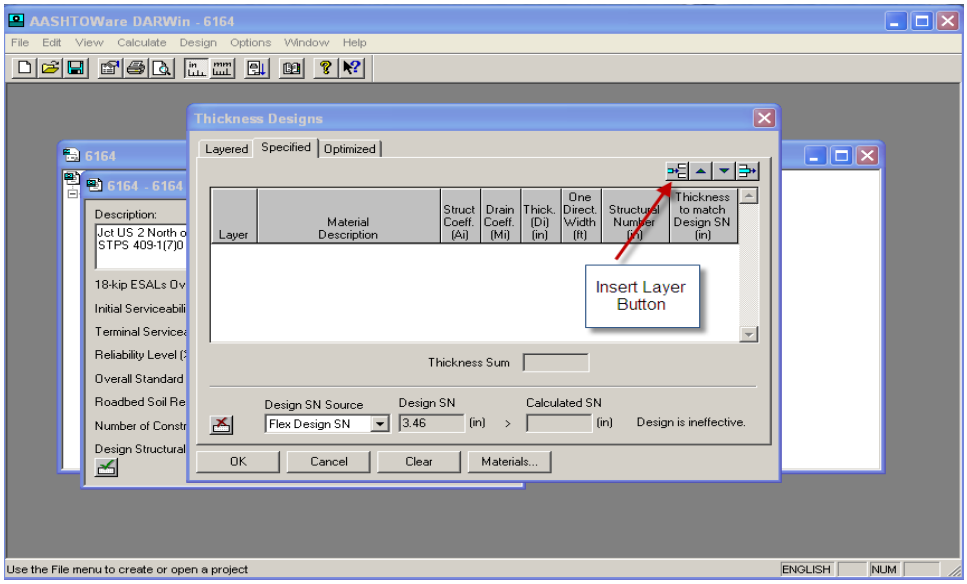


Figure 32: DARWin thickness design template with “Specified” tab selected.

Enter the material descriptions, structural coefficients, drainage coefficients, thicknesses, and one directional width as shown in Figure 33. Structural coefficients for different pavement layers are shown in

Table 3.

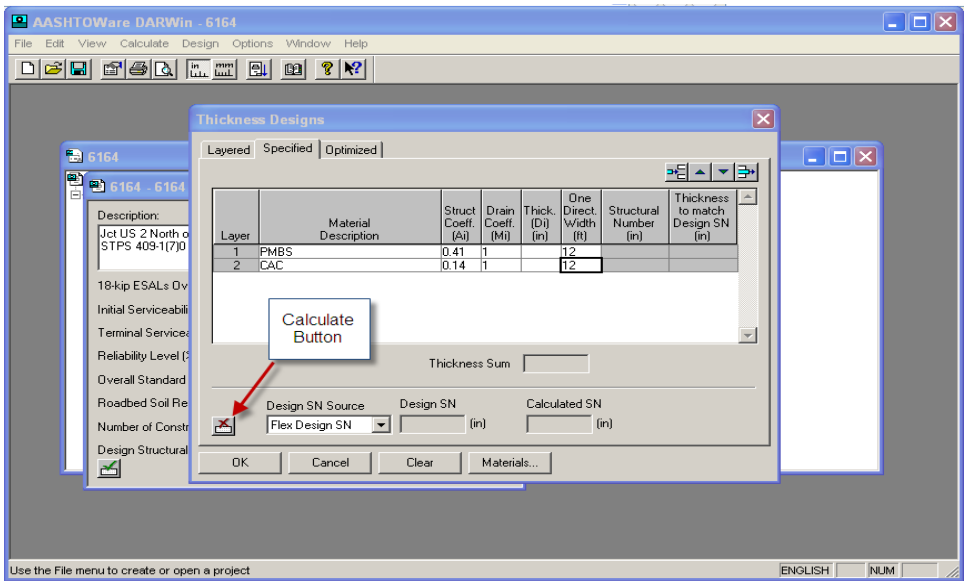


Figure 33: DARWin Partially Completed Thickness Design template

Enter the PMS thickness based upon the minimum PMS thickness shown in

Table 2 below.

Table 12: MDT Structural Coefficients for Different Pavement Layers

MDT Structural Coefficients for Different Pavement Layers	
Layer	a_i
Plant Mix (PMBS)	0.41
CAC gravel	0.14
RAP/ Aggregate	0.12
CTB	0.20

Table 13: Recommended Plant Mix Thicknesses

ESALs (Daily)	**PMS Thickness***
> 2000	0.70’
1000 - 2000	0.60’ – 0.70’
501 - 1000	0.50’ – 0.60’
201 - 500	0.40’ – 0.50’
101 - 200	0.30’ – 0.40’
0 - 100	0.30’*

*In certain cases 0.25’ of PMS can be used for ½” or 3/8” mix, when budgets are constrained.

Use a trial-and-error approach to determine CAC thickness. 13.8” CAC yields an acceptable SN = 3.41 as shown in Figure 31. MDT specifies pavement layer thicknesses in 0.05’ intervals, and rounding 13.8” up to the nearest 0.05’ yields 1.15’. **The resulting pavement design for this project is 0.30’ PMS under laid by 1.15’ CAC.**

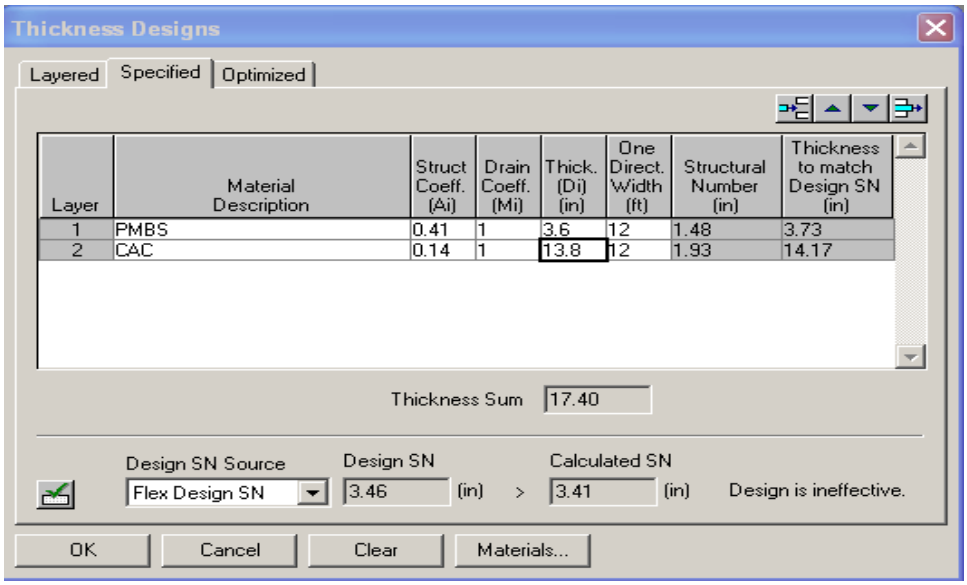


Figure 34: DARWin completed thickness design

It should be noted, in Figure 34: DARWin completed thickness design above, it states the Design is ineffective. The calculated SN of 3.41 exceeds the low volume SN of 3.4 which is found in the low volume table.

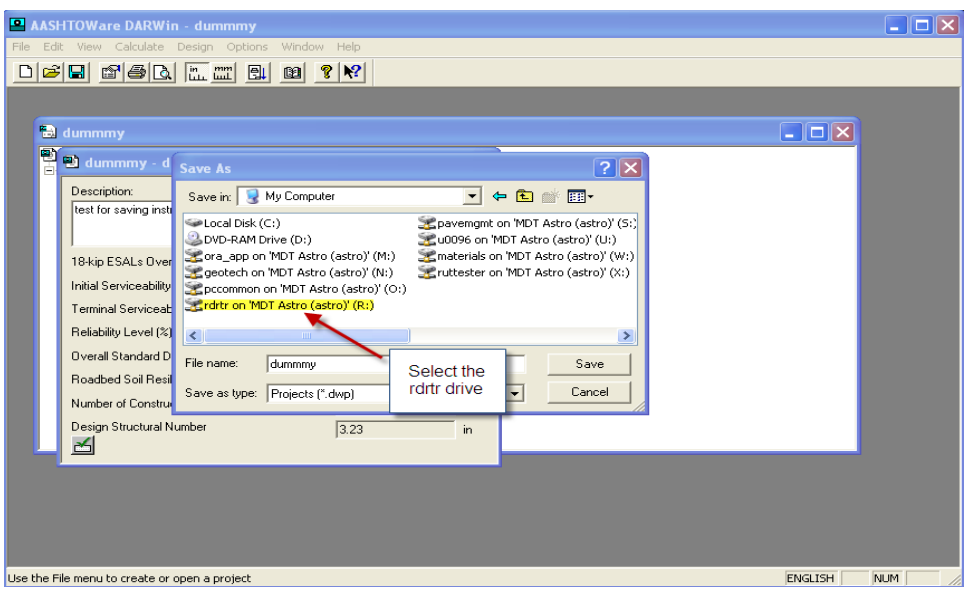


Figure 35: DARWin Pane showing location of rdrtr\projects\7*digitUPN*#.

Saving the DARWin Design File: It is important to save the DARWin design file with the correct file name and location so it can be revisited in the future. Click on "File" in the upper left hand corner of the main pane, and click save. A "Save As" pane will appear, click on "My

Computer", and the selection will change as shown in Figure 35. Click on the "rdrtr on 'MDT Astro (astro)' drive and click on the "DARWIN" folder. The "Save" button will now save the project in the standard location for Surfacing Design projects.

The standard file name for DARWin design files is the 7-digit Construction Number (CN), and DARWin design files are designated by the file extension .dwp. For Jct US 2 – North of Chester, the file name is 6164000.dwp.

Step #5: Specify Pavement Materials

After the thickness design is completed, the next step is specifying pavement materials. This pavement consists of a Surface Treatment (if applicable), PMS and CAC. Regarding the base layer, specifying the base material type is easy because MDT only utilizes CAC for aggregate base. The PMS type needs to be chosen first since that will determine the need for a surface treatment. PMS type is chosen based upon PMS quantity and project type shown in

Table 4 and Table 6 found earlier in this manual.

PMS Type Selection

First, calculate PMS tonnage. Based upon discussion with the Road Design Project Manager, the finished roadway width will be 28'. Use the equation below to calculate PMS tonnage:

Project Length (miles) / project * 5280 ft. / mile * Roadtop Width (ft.)
* PMS thickness (ft.) * 1 yd^3/27 ft^3 * 3855 lbs. / yd^3 * 1 ton PMS / 2000 lbs.
= **tons PMS / project**

OR:

9.0 miles / project * 5280 ft. / mile * 28' * 0.30' * 1 yd^3/27 ft^3 * 3855 lbs. / yd^3 * 1 ton PMS / 2000 lbs.
= **28,496 tons PMS / project**

Based upon 28,496 tons,

Table 4 (found earlier in the manual) specifies that either ½" or ¾" Grade S Volumetric should be specified. Table 6 (found earlier in the manual) specifies ¾" Grade S or Commercial Plant mix. Since both tables allow ¾" Grade S Volumetric, the designer should specify **¾" Grade S Volumetric**. Climate or local project characteristics may lend themselves to ½" Grade S Volumetric mix with higher oil content to combat stripping.

PG Binder Selection

The criteria for picking PG Binder type is found earlier in this manual in Table 7.

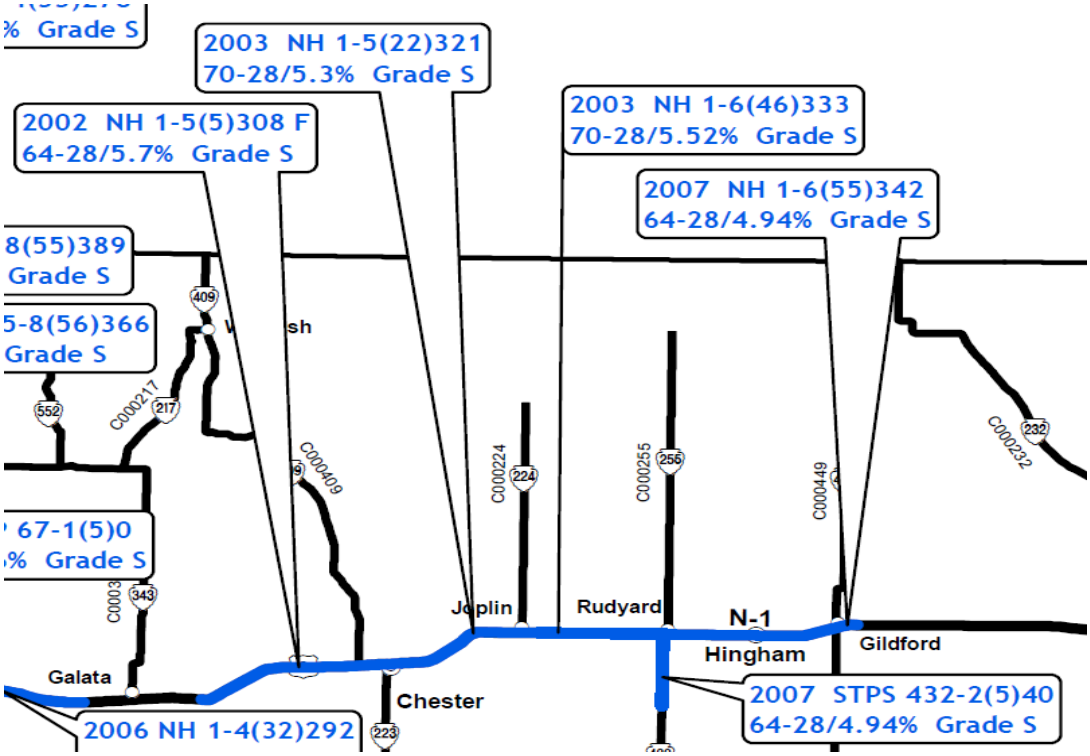
This reconstruction project has more than 50 daily ESALs, therefore PG 64-28 is specified.

Should RAP be used?

Since this is a reconstruction of a gravel road RAP will not be available.

Asphalt Content Selection

Use Table 8 to make an asphalt content recommendation. Project specific asphalt content is determined utilizing the *As Constructed Percent Asphalt* map (see Figure 22). The screen capture below shows asphalt contents from previous projects near the subject project. Based on the screen capture, 5.2**% asphalt content** is estimated.



Selecting Surface Treatment

This project has an AADT = 360 and a 70 mph posted speed. Based upon Table 25, **Type I chip seal** will be specified since this is a newly constructed road with no rutting, high traffic speed, and low traffic volume.

Step #6: Sending the Pavement Design Memo

The last task is to send a paper copy of the surfacing design memo to road design staff via interdepartmental mail. Figure 36 shows the unsigned surfacing design memo for this project. In practice, both the Pavement Design Engineer and Designer sign the memo to the right of their printed names.

After completing the surfacing design memo with Microsoft Word, save the .doc file using the 7 digit Construction Number (CN) as the file name. Save the .doc file in the following locations:

Preliminary Surfacing Design Memo: \\astro\rdrtr\projects\6164000\6164000_600.doc

If changes occur to the Preliminary Surfacing Design Memo, then Surfacing Design will provide a Final Surfacing Design Memo.

Final Surfacing Section Design Memo: \\astro\rdrtr\projects\6164000\6164000_604.doc

Once the memo has been submitted for distribution and saved, the EPS activity may be carded off.

Montana Department of Transportation
Helena, MT 59620-1001

Memorandum

To: Paul Ferry, P.E.
Highways Engineer

Thru: Daniel W. Hill, P.E.
Pavement Analysis Engineer

From: Gregory D. Zeihen, P. E.
Pavement Research Engineering Specialist

Date: 7/18/2013

Subject: STPS 409-1(7)0
Jct U. S. 2 – North of Chester
CN 6164

Preliminary surfacing recommendations (surfacing activity 600) for the above project are listed below:

Surfacing Section No. 1 – Reconstruction, BOP to just past Gavilon Grain Terminal (RP 0.6)
------ Type I Chip Seal
0.30' Plant Mix Bituminous Surfacing, Grade S
1.15' Crushed Aggregate Course
1.45' Design R-value = 5

Surfacing Section No. 2 – Reconstruction, past Gavilon Grain Terminal (RP 0.6) to EOP (RP 9.0)
------ Type I Chip Seal
0.30' Plant Mix Bituminous Surfacing, Grade S
0.75' Crushed Aggregate Course
1.05' Design R-value = 5

Surfacing design sections are based on traffic data indicating 58 ESALs, and 12 ESALs, respectively. Grade S ¾" plant mix surfacing with PG 64-28 binder and 5.2% asphalt content is recommended. R-values taken from the soil survey have been used to determine surfacing thickness. Surface Design recommends the District review soil soil information prior to and during construction. Areas of concern and possible borrow areas should have additional soil samples submitted for R-value testing. Please contact me with questions regarding this recommendation.

DWH:GZ

cc: D. Hand, P.E.
D. Krings, P.E.
S. Prinzing, P.E.
Surfacing File
Materials Bureau File
Geotechnical File

Figure 36: Example Preliminary Surfacing Design Memo

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Chapter 7 **Flexible Pavement Rehabilitation**

Rehabilitation is a strategy to extend a pavement's useful life through pavement structure improvement utilizing the in-place materials. Rehabilitation is considered in two categories: minor and major. These categories are explained further in Table 14.

Table 14: Minor vs. Major Rehabilitation

Category	Minor Rehabilitation	Major Rehabilitation
Surface Engineering	Engineered Design	Engineered Design
Geometric Design Standards	As Built	Ranging from As-Built to Current Standards
Applied treatments	0.2' ≤overlay ≤0.3' Milling ≤ 0.2' No exposure of base gravel	≥0.2' overlay w/ Grading Pulverization Mill, recycle and overlay Exposed gravel may be treated or modified
How Needs are Identified	Observed Distress	Observed Distress Geometrics
Design Life	≥10 years	≥20 years

7.1 MINOR REHABILITATION

The intent of these projects is to rehabilitate the existing pavement surface through an engineered approach that considers the observed pavement distress and in-place materials. Design guidelines for minor rehabilitation projects are defined in MDT / FHWA's *Guidelines for Nomination and Development of Pavement Projects*. This Guideline is available upon request.

Design Method: Engineered judgment, engineered overlay (see Chapter 8), estimated structural number on assumed properties.

Design Life: 10-years design life.

Minor rehabilitation is intended for pavements that are structurally sound to restore the functional condition of the pavement. This usually refers to restoring the ride and rut condition of the pavement. Pavements where there is significant load-associated cracking or obvious base course and subgrade issues should not be minor rehabilitated, as minor rehabilitation of these pavements will probably not last the required 10 year design life.

Some minor rehabilitation treatments that have been utilized by MDT in the past include:

- Asphalt overlay

- Asphalt mill and fill
- Cold in-place recycling overlayed by a chip seal or an asphalt overlay
- Hot in-place recycling with chip seal

The information needed for minor rehabilitation design includes observation of existing pavement distress and in-place materials. Observation includes analyzing PvMS pavement condition data to determine the pavement's rut, ride, and cracking characteristics and visual observation done during the preliminary field review. Characterization of the in-place materials usually pertains to the PMS layer only, and usually consists of pavement cores and GPR analysis to determine PMS condition and thickness.

Since information is not collected for the base course or subgrade materials, it is not usually practical to design these projects using the 1993 AASHTO structural number method. Instead, engineering judgment and past pavement performance are used to determine the pavement treatment that cost effectively provides the desired design life. Using GPR thickness data and FWD subgrade information, a structural number can be estimated.

Minor rehabilitation should be designed to correct the most critical pavement distress. The critical pavement distress for minor rehabilitation is usually minor roughness and alligator cracking <u>or</u> minor to major rutting and transverse cracking. Use Table 15 to help determine the appropriate minor rehab strategy.

Table 15: Minor Rehabilitation Goals and Treatment Selection

Distress	Minor Rehabilitation Goal	Example Treatments
Minor Alligator Cracking	Add pavement structure and seal pavement	Mill <= 0.2', Overlay <= 0.3', cold or hot in-place recycle with overlay
Rutting	Correct or remove pavement ruts	Follow the following guidelines to determine Treatment: 1. Establish that Rutting is confined to upper PMS layer. If rutting exists in lower layers, it is not a Minor Rehabilitation Candidate. 2. Establish whether PMS is stable or unstable. If rutting is an ongoing phenomenon the PMS may be unstable. If rut depth has stabilized and is not becoming deeper, the PMS is stable. This determination is often made by observing PvMS data and graphing rut depth vs. time. **Stable PMS**: Microsurfacing, PMS leveling with overlay, mill/fill **Unstable PMS**: mill/fill as deep as necessary to remove unstable PMS. **Do not use in-place recycling to treat stable or unstable rutting unless an overlay is placed over the recycled material**. Rutting has been shown to reoccur when in-place recycling previously rutted pavements.

Distress	Minor Rehabilitation Goal	Example Treatments
Minor Roughness	Restore ride quality	Overlay that includes a leveling quantity, mill/fill, cold or hot in place recycle with overlay
Minor Transverse Cracking (doesn't affect ride)	Seal pavement with emphasis on transverse cracks	Seal cracks followed by leveling course and overlay, mill fill, hot in-place recycle followed by chip seal, cold in-place recycle followed by chip seal (low-volume only)
Major Transverse Cracking (affects ride)	Restore ride, seal cracks, Add pavement structure	Mill/fill (low volume), Mill/fill w/ additional overlay, cold or hot in-place recycle with overlay

7.1.1 Major Rehabilitation

The intent of these projects is to rehabilitate the existing pavement structure through an engineered approach that considers the observed pavement distress, in-place materials, and roadway geometrics. Milling operations may be > 0.20 ft. and may expose base gravel that can be treated or modified. New right-of-way and utility relocation may be required to improve geometrics, to flatten slopes, or enhance safety. Appropriate soil survey work, subsurface analysis, traffic data and accident data must be collected.

Major rehabilitation usually occurs in situations where:

- An existing pavement is in good condition, but pavement structure needs to be added to accommodate growing traffic levels
- An existing pavement is distressed and the distress cannot be remediated with lesser pavement treatments
- The existing pavement is in good condition but is too narrow for growing traffic levels. In this case, the existing pavement is usually major rehabilitated and widened to meet today's roadway width standards.

The following bullets provide design guidelines for major rehabilitation projects defined in MDT / FHWA's *Guidelines for Nomination and Development of Pavement Projects*.

Design Method: An engineered design based upon a thorough pavement investigation and using the design method described in Chapter 7.

Design Life: 20-years design life.

There are instances where major rehabilitation projects are upgraded to reconstruction projects. This usually occurs where the horizontal or vertical road alignment changes significantly to meet geometric design standards. As a rule of thumb, major rehabilitation projects where more than 25% of the project length needs to be realigned should be upgraded to reconstruction.

In the past, there have been a number of major rehabilitation projects that escalated to reconstruction late in the project design phase. This is usually due to alignment issues rather than pavement issues. Discussions should take place early in the design process to ensure that major rehabilitation is truly possible.

Major Rehabilitation Project Types

Major Rehabilitation pavements are usually designed and built using one of the following methods:

1. Pavement pulverization (also known as Full Depth Reclamation (FDR))
2. Engineered overlays (with or without grade raise)
3. Cold Central Plant Recycling (CCPR)
4. White Topping

Two common pavement types are discussed in the following sections.

7.1.2 Pavement Pulverization

Pavement pulverization is mixing existing PMS and base course together to form a crushed base course for a new pavement. The PMS and base course mixture is known as a "pulverized mixture" and is manufactured using a pavement reclaimer as shown in Figure 37, Figure 38, and Figure 39.

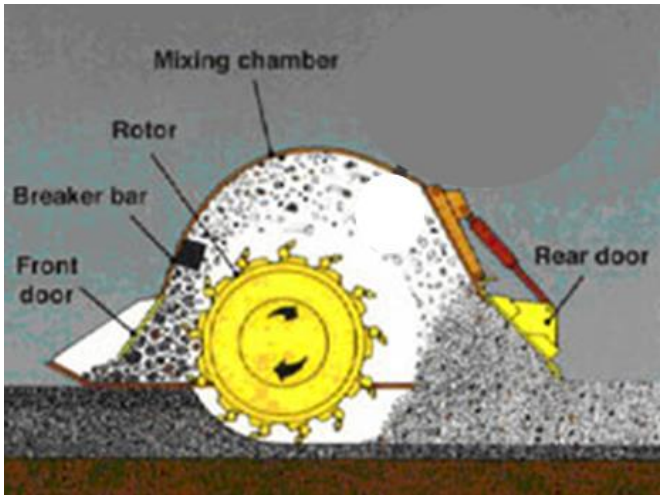

Figure 37: Pavement Pulverization Schematic

This process works well when:

- The horizontal and vertical alignment will be left unchanged, or changed slightly.
- There is a desire to stay within existing Right of Way limits

- The existing PMS is distressed, but the underlying base course is in good condition and has adequate thickness to provide both the required pavement structure and a construction platform for pulverization operations.

Figure 38: Pavement Pulverization

Usually these pavements are built as follows:

Step #1.

A portion of existing PMS may be milled and removed prior to pulverizing. This is done in order to reduce the PMS amount in the pulverized mixture, or to reduce the increase in vertical elevation of the finished pavement.

Step #2.

Virgin CAC may be placed upon the milled or un-milled PMS surface to reduce the PMS amount in the pulverized mixture, or to add pavement structure.

Step #3.

The surface is pulverized, shaped, and re-compacted in-place. Additives such as Portland Cement may be added to increase pavement structure.

Step #4.

After placing aggregate treatment on the pulverized material, PMS is placed to provide the new surfacing course.

Figure 39: Prepulverized Material being Repulverized with Portland Cement

The following are other design details for pavement pulverization:

- 0.65' is the most common pulverization depth which corresponds to MDT's maximum compacted lift depth. Deeper pulverization can be done, but the 0.65' maximum compacted depth must be adhered to.

- It is important to have a consistent blend of pulverized PMS and CAC. Within the blend, pulverized PMS should comprise a maximum 60% of the pulverized mixture. MDT research has shown that pulverized mixture shear strength decreases when PMS comprises more than 60% of the mixture (MDT, Evaluation of the Engineering Characteristics of RAP/Aggregate Blends, 2005).

- Compacted pulverized material should be assumed to have up to 5 - 15% swell factor.

The following figures show typical sections from sample previous MDT Construction plans.

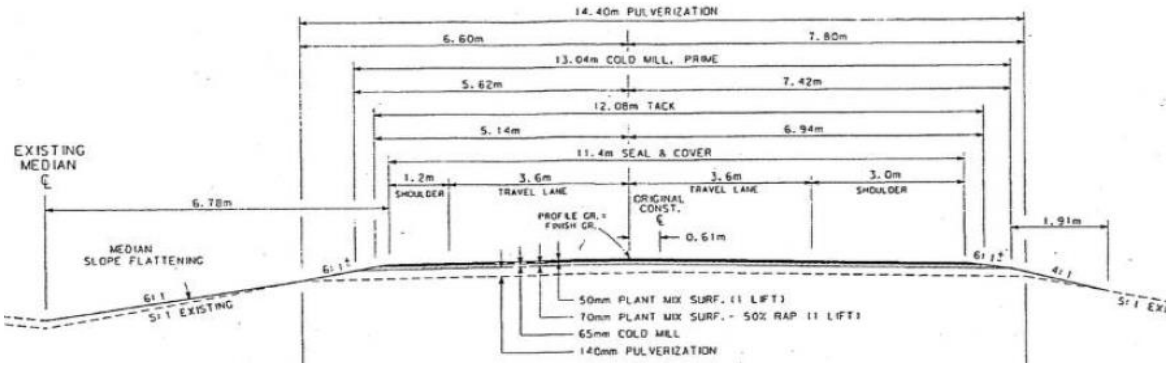

Figure 40: Pavement Pulverization without Widening

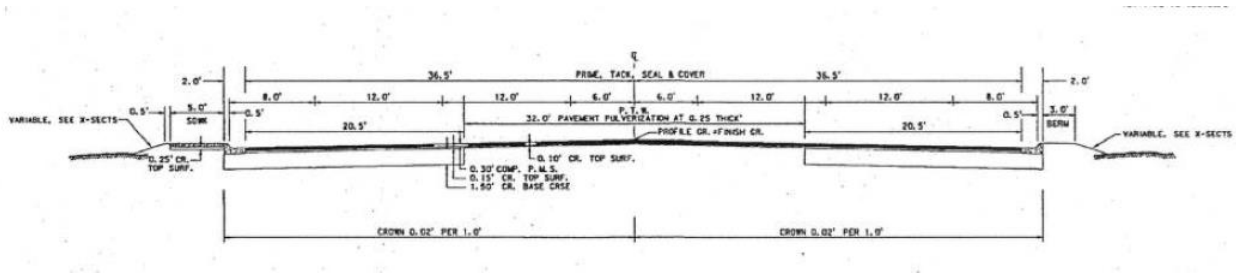

Figure 41: Major Rehabilitation with Pulverization and Major Widening

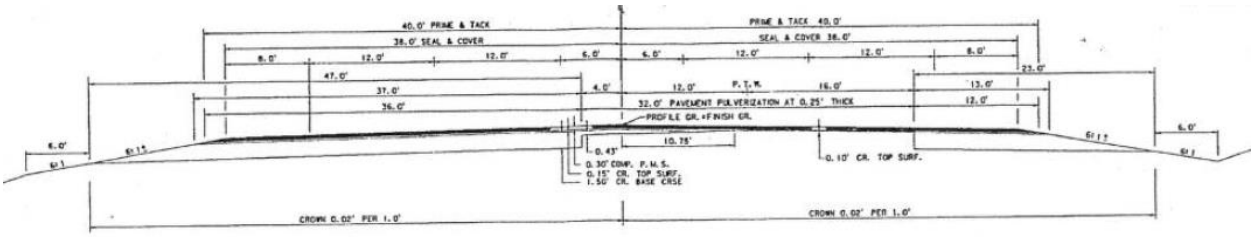

Figure 42: Major Rehabilitation with Widening to One Side of Existing Roadway

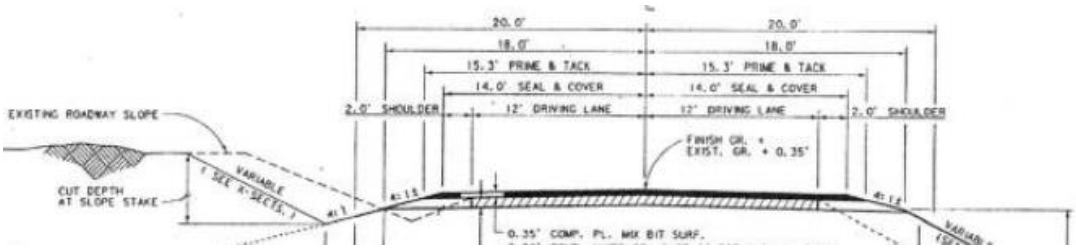

Figure 43: Major Rehabilitation with Minor Widening

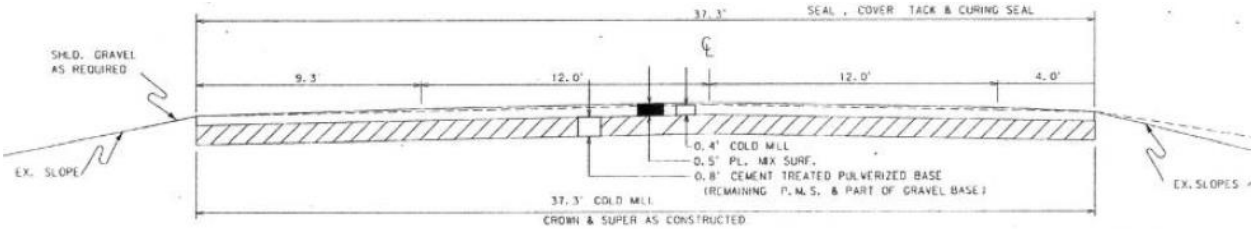

Figure 44: Major Rehabilitation with In-Place Cement Treatment

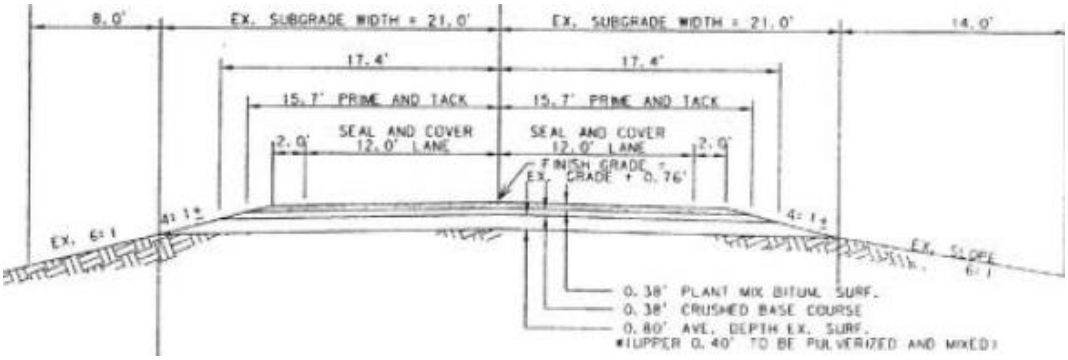

Figure 45: Major Rehabilitation with Addition of Virgin CAC

Project Selection

The appropriate candidate for pulverization needs to be verified early in the design stage. One factor to successful pulverization is to have limited to no change to the vertical alignment. If the existing vertical alignment undulates compared to the corrected vertical profile pulverization is not feasible. The primary reason is due to the conflict where the existing profile protrudes up into the proposed typical section (cut sections).

Pulverization depth is limited by the ability to compact the blended base. MDT specs call for a maximum of 8" compacted lift. Some reclaimers can pulverize up to 20 inches deep, however the maximum pulverization depth is driven by the plant mix thickness. The existing plant mix should not be more than 8 inches (0.65') thick for pulverization. At that maximum, a 16 inch depth would be required to obtain a 50/50 blend. In this case, 8 inches would be required to be bladed off in order to compact the remaining pulverized 8 inches per MDT's specs. This could be a viable option for projects that incorporate widening.

Caution should be used when considering pulverization on projects with existing cement treated base. CTB can be pulverized but can cause difficulties. A combination of milling and pulverization may be necessary. Also, MDT's CTB aggregate gradation allows up to 20% minus 200 mesh, while having a maximum aggregate size of 1 inch. This has been rarely encountered in Montana.

On projects with very weak, soft subgrades and thin existing gravel sections the use of pulverization should be questioned. When utilizing the pulverized base for design, the bottom 0.2' – 0.4' should not be accounted to contribute to the structural number.

When the soil survey encounters high fines in the base, again the designer should question the use of pulverization. Pulverized plant mix typically contains around 8% minus 200 mesh. When blended with base, the plant mix will help reduce the fines content of the mixture. It should be noted that historically MDT's gravel specification allowed for up to 12% minus 200 mesh, whereas the current specification allows for only up to 8% passing the 200 mesh. The estimated weighted average of the pulverized blend should not exceed 10% passing the 200 mesh unless approved by the Pavement Analysis Engineer.

Pulverized Pavement Constructability

MDT has encountered problems with constructability of pulverization projects. The most common problem is the pulverized surface being unable to support traffic before it is paved over. This may result from thin existing pavement thickness and/or fine-grained, saturated subgrade materials. Either of these scenarios may result in subgrade material pumping up into pulverized section and ruining the pulverized material.

A thorough pavement evaluation of the following needs to be completed before pavement design:

> **PMS thickness:** The PMS thickness should be measured continuously along the project length. The purpose of this is to determine the pulverization depth and/or milling

depth needed to meet the blend requirements, and to ensure the pulverizer teeth do not extend into the subgrade material. GPR is an effective way to continuously measure PMS thickness. The soil survey PMS depths can also be used.

CAC thickness and condition: A soils survey (MT-207) should be completed to determine the in-place CAC thickness, condition, and moisture level. Falling Weight Deflectometer data can be used in addition to the soil survey to determine CAC stiffness.

Subgrade Type and condition: Similar to CAC, both a soil survey and FWD testing can be used to identify unstable subgrades. Subgrade moisture contents above optimum may indicate soft, pumpable soils.

After the pavement evaluation has been completed and pavement pulverization is determined to be feasible, follow these design procedures to improve constructability:

- **Do not lower pavement grade**. The finished pulverized surface should be at or above the existing pavement grade. Lowering the grade will reduce pavement structure and reduce construction platform strength.
- **Include contract language to not allow pulverization in the wetter spring months**.
- Include contract language **to limit the amount of time both public and Contractor traffic is allowed to travel on the pulverized material**. Provide alternative haul routes so heavy construction traffic does not travel on pulverized material.

7.1.3 Flexible Pavement Rehabilitation Design

MDT's pavement rehabilitation design is based upon the method presented in the 1993 AASHTO Guide Part III, with modifications presented in this Design Manual.

The following equation is used to design rehabilitation projects:

$$SN_{dgn} - SN_{eff} = SN_{ol}$$

Where:

SN_{dgn} = The structural number required for future traffic loading. Also known as SN_f (Structural number to carry future traffic) in the 1993 AASHTO Guide.

SN_{eff} = Existing pavement structural number.

SN_{ol} = The Structural Number deficiency between the existing pavement and that needed for the future pavement design. This is the amount of pavement structure that needs to be added with an overlay.

Generally, pavement rehabilitation design includes the following steps:

Step #1

Based upon the existing pavement condition, layer thicknesses and quality, geometrics, structural capacity and funding category recommend a rehabilitation method.

Step #2

Determine existing pavement structural capacity (SN_{eff}).

Step #3

Determine future pavement structural capacity (SN_{dgn})

Step #4

Specify construction methods and materials.

Step #5

Send pavement design and materials memo.

Step #1: Determine Pavement Rehabilitation Method

Recommend a rehabilitation method based upon the existing pavement condition, layer thicknesses and quality, geometrics, structural capacity, traffic, and funding category.

Choosing a rehabilitation method is heavily based upon engineering judgment. Each pavement is unique, and there are usually a number of ways to rehabilitate a given pavement. The best method is one that meets the needs of the pavement designer, road designer, planning personnel, traveling public, safety, and the project budget. The designer should communicate with the road designer to ensure the chosen rehabilitation method will fit within the broader project constraints.

Visually evaluating the pavement condition is the single most important input to making this decision. By evaluating the pavement distress, as well as the distress mechanisms that cause the distress, the designer determines whether the pavement distresses are functional or structural:

> **Functional distresses** refer to those distresses that affect the traveling public, such as rough ride or low pavement friction. Those distresses, although a nuisance, can happen on a pavement that is structurally sufficient. Pavements with functional distress tend to need less expensive surface treatments to restore the pavement serviceability and extend its design life. These projects are often referred to as pavement preservation. An example of this would be a chip seal.

Structural distresses, such as deep pavement rutting and alligator cracking, are an indication of inadequate pavement. Structural distress often needs deeper, more expensive rehabilitation in order to meet the desired design life.

Engineered overlays are the preferred method to rehabilitate pavements with functional distress that require additional structure. Milling, pulverization or in-place recycling should be utilized to treat structural distresses. Figure 46 shows a flowchart to serve as a starting point when choosing a rehabilitation method. In addition to the issues defined in the flowchart, some other issues that influence this decision are:

- **Traffic Volume:** There are times when heavy traffic volumes will influence the rehabilitation treatment decision, where the design team may want a more conservative treatment than shown in the flowchart.

  Conversely, on low volume roads there may be a desire to be less conservative in the pavement design. This may mean an engineered solution that effectively “bridges” over a problem pavement rather than treats its structural distress directly. An example of this is designing a thick engineered overlay over a pavement with base course distress.

- **Pavement Grade Raise:** Often, in order to reduce roadside slope work or reduce right-of-way take, the pavement rehabilitation needs to be designed to reduce or eliminate pavement grade raise.

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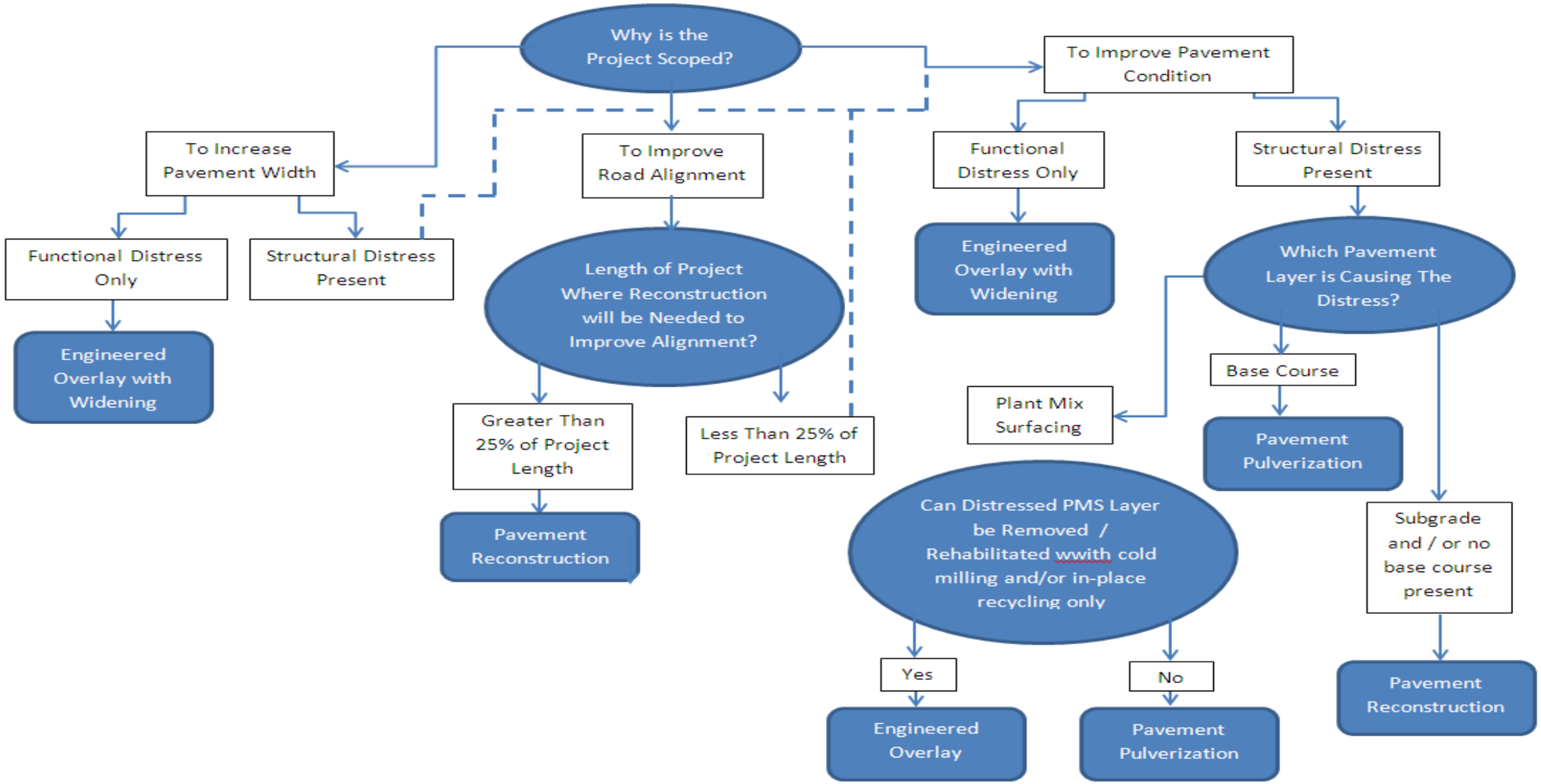

Figure 46: Major Rehabilitation and Engineered Overlay Selection Flowchart

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Step #2: Determine Existing Pavement Structural Capacity (SN_{eff})

MDT usually calculates the structural capacity of the existing pavement (SN_{eff}) by analyzing the existing pavement. This analysis includes calculating the SN of each existing pavement layer, and summarizing the SN of all layers to calculate SN_{eff}. The equation used to determine the structural capacity of the existing pavement is:

$$
\begin{aligned}
SN_{eff} &= SN_{PMS} + SN_{Base} + SN_{sub} \\
&= d_{PMS}\, a_{PMS} + d_{Base}\, a_{base} + d_{sub} a_{sub} \\
&= \sum_{1}^{n} d_{PMSn} a_{PMSn} + d_{Base}\, a_{base} + d_{sub} a_{bub}
\end{aligned}
$$

Where:

SN_{eff} = The structural number of the existing pavement

$d_{PMS}, d_{Base}, d_{Sub}$ = Average thicknesses of the existing PMS, Base Course, and Subbase Course (ft. or in.)

a_{PMS} , a_{base}, a_{sub} = The average effective structural coefficient of the existing PMS, Base Course, and Subbase Course layers

n = Number of individual PMS lifts within the existing pavement. Often, each lift (layer) of PMS have different levels of degradation. In these cases, the SN of each lift is calculated separately.

SN_{eff} = The structural number of the existing pavement

Use the methods presented in the rest of this section to determine both pavement layer depths and structural coefficients.

Existing Pavement Evaluation

The most challenging task in designing a major rehabilitation is determining the structural contribution of the existing pavement, or SN_{eff}. MDT relies on pavement borings and non-destructive testing (NDT) of the existing pavement to estimate both pavement layer thicknesses and structural coefficients. These are expanded on below:

NDT Testing: This refers to the Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) testing done by MDT's NDT unit. FWD testing is analyzed to provide the Resilient Modulus (M_R) of each pavement layer and subgrade material. GPR testing provides a continuous measurement of the PMS thickness. This testing is further discussed in Chapter 7.

Existing Pavement Sampling and Testing: The District conducts a centerline soil survey and PMS core stripping evaluation to measure the thickness and quality of existing pavement layers. The following pavement characteristics are measured:

- PMS, base, and subbase thicknesses,
- Base Course Soils Class, Gradation, Atterburg Limits, Natural (in-place) Moisture Content, Optimum (Proctor) Moisture Content, and R-Value. Usually there is not enough material from any given bore hole, so composite base course samples from a number of bore holes are combined. Note that this results in "average" properties of the in-place base materials.
- Subgrade Soils Class, Gradation, Atterburg Limits, Natural (in-place) Moisture Content, Optimum (Proctor) Moisture Content and R-Value.

The soil sample is done in accordance with MT-207, and the results are presented on Materials Form 111. An example Form 111 is shown in Figure 24.

7.1.4 Non-Destructive Testing Overview

The NDT Unit is responsible for conducting Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR) testing on the state highway network on an ongoing basis. FWD and GPR testing can only occur on bound pavement surfaces. NDT provides analyzed data to both internal and external customers. Testing is done as follows:

Network level FWD/GPR testing is done on all state roadways on a 5-year rotation, meaning that there should be FWD and GPR information available that was collected in the last 5 years for all pavements. Network level FWD testing is done at 820 ft. increments within the outside wheel path of the driving lane in one direction only on 2 lane roads. For Interstate and multi-lane facilities, excluding turning lanes, both directions are tested. GPR is collected continuously.

Project level FWD/GPR: In addition to Network level testing, project level testing is done prior to road construction projects. Project level FWD testing is done at 330 ft. increments within the outside wheel path, of the driving lane in one direction only (or in both directions of Interstate Pavements). GPR testing is done continuously. This testing is done before Surfacing Design's preliminary surfacing design activity (Act. 600). The following paragraphs summarize the testing and reporting of different types of projects.

Reconstruction: The NDT unit analyzes this data and backcalculates the in-situ pavement structure MR (which includes subgrade, base and pavement surfacing). The existing pavement and base layer MR is not needed in the calculations for new pavement structure because it will be eliminated because of a new surfacing section. Surfacing Design receives the data in an Excel spreadsheet summarizing the subgrade M_R.

Major Rehabilitation: The NDT unit analyzes this data and determines the MR of the in-situ subgrade, base course, and surfacing materials. GPR is also used to calculate PMS thickness continuously along the project length. Both GPR and FWD information are analyzed, graphed, and provided to surfacing design in an Excel spreadsheet.

Minor Rehabilitation: The NDT unit also tests Minor Rehabilitation projects for both FWD and GPR. The NDT unit then provides the in-situ pavement structure MR to the surfacing design unit. GPR is analyzed for existing pavement section thicknesses used for back calculation. The analyzed GPR depths may also be used to determine appropriate milling depths.

Pavement Preservation: GPR testing is done on lower traffic volume projects to report PMS thickness to ensure there is adequate PMS thickness for milling operations. GPR testing is not done for pavement preservation projects on the Interstate System due to the known structural integrity of the road. For seal and cover type projects only, GPR testing is not needed. The GPR information is analyzed, graphed, and provided to surfacing design in an Excel spreadsheet.

Falling Weight Deflectometer

A **falling weight deflectometer** (FWD) is a testing device used by civil engineers to evaluate the physical properties of pavements. FWD data is primarily used to estimate pavement structural capacity for 1) overlay design and 2) to determine if a pavement is being overloaded. Use includes (but is not limited to) highways, local roads, airport pavements, and railway tracks.

The FWD is designed to impart a load pulse to the pavement surface which simulates the load produced by a rolling vehicle wheel. The load is produced by dropping a large weight, and transmitted to the pavement through a circular load plate - typically 12 inches in diameter. A load cell mounted on top of the load plate measures the load imparted to the pavement surface. The load plate can be solid or segmented. The advantage of a segmented load plate is that it adapts to the shape of the pavement, giving an even distribution of the load on uneven surfaces. MDT's NDT unit also incorporates a swiveling knuckle to ensure proper surface contact.

There are two different types of load impact systems; single-mass and double-mass. In a single-mass system, which MDT utilizes, a weight is dropped onto a single buffer connected to a load plate, which rests on the surface being tested. The load force is transferred through the plate, and the plate creates a deflection that simulates a wheel load. In the double-mass system, typically used for very thick sections such as airport runways, the weight drops onto a double-buffer system, which includes a first buffer, a second weight, and a second buffer.

Deflection sensors (geophones; force-balance seismometers) mounted radially from the center of the load plate measure the deformation of the pavement in response to the load. MDT's sensor placements are 0", 8", 12", 18", 24", 36", 48", and 60".

FWD data is most often used to calculate stiffness-related parameters of a pavement structure. The process of calculating the elastic moduli of individual layers in a multi-layer system (e.g. asphalt concrete on top of a base course on top of the subgrade) based on surface deflections is known as "back-calculation", as there is no closed-form solution. Instead, initial moduli are assumed, surface deflections calculated, and then the moduli are adjusted in an iterative fashion to converge on the measured deflections. This process is computationally intensive

although quick on modern computers. It can give quite misleading results and requires an experienced analyst.

The FWD process consists of first lowering the mass of weight and the rack of sensors called "geophones" that measure deflections to the roadway surface. The weight and rack of sensors are located in the back of the truck, just behind the rear axle (Figure 47). Next a large weight is dropped which mimics a heavy truck tire, and measures how much the pavement deflects beneath the weight. The weight is approximately 9,000 lbs. which is meant to represent an 18,000 lb. (18-kip) ESAL.

The deflection is measured in 8 locations by geophones located in the following distances from the center of the load plate; D_1 = 0 in., D_2 = 8 in., D_3 = 12 in., D_4 = 18 in., D_5 = 24 in., D_6 = 36 in., D_7 = 48 in., and D_8 = 60 in. All of the deflections measure the deflection basin as shown in Figure 48.

Figure 47 shows one of the two NDT vehicles with both GPR and FWD equipment mounted upon it.

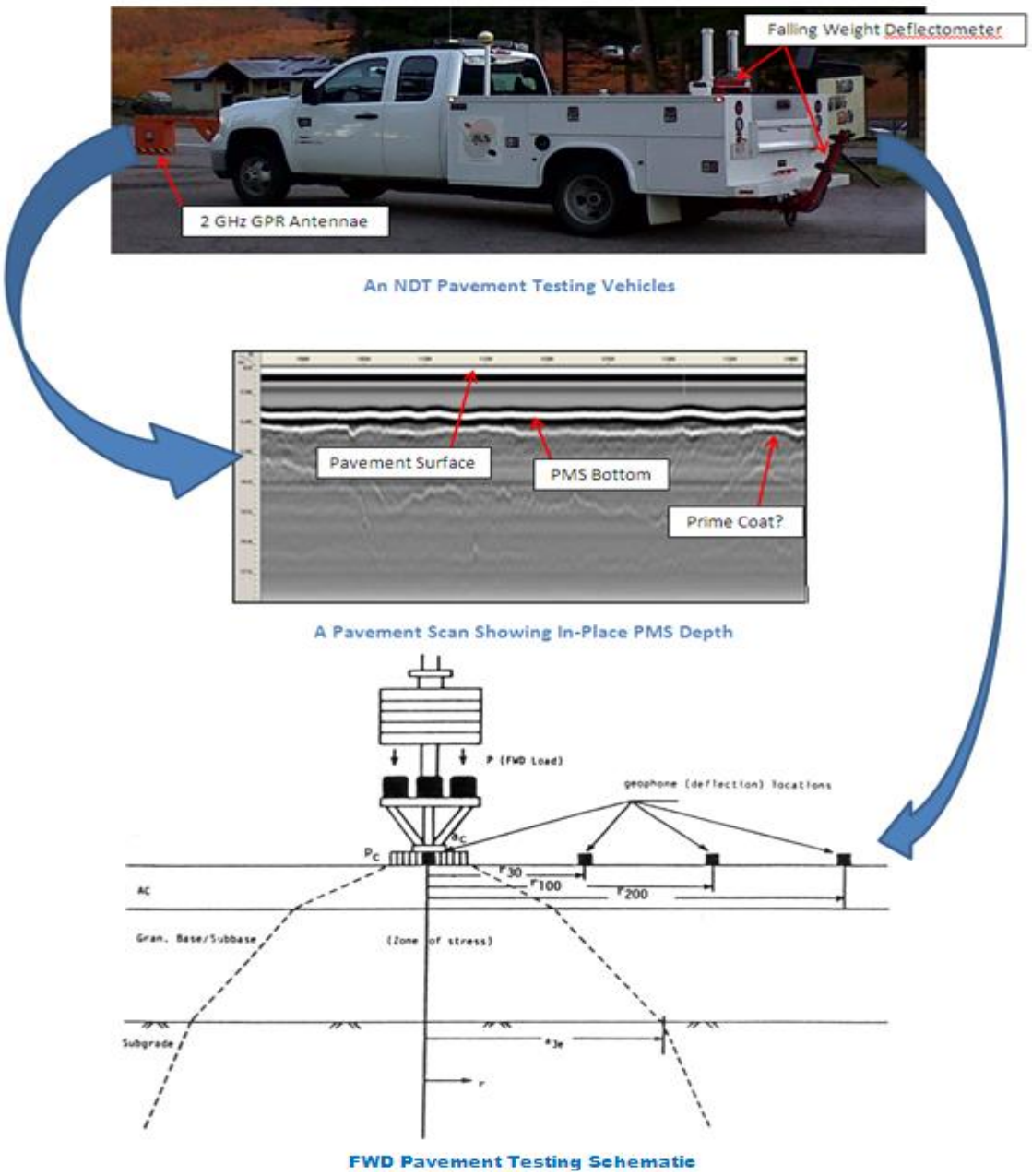


Figure 47: NDT Vehicle with both FWD and GPR

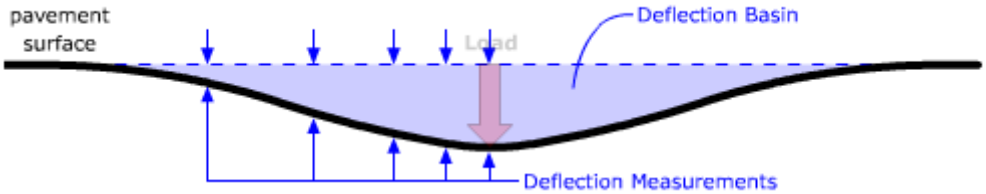


Figure 48: Deflection Basin (Steve Muench (2003))

Analysis of FWD Deflection Basins

The deflection basin and individual deflections are used to make inferences of overall pavement conditions. Here are a few parameters to look at when analyzing deflection basins, although there are others:

- As a rule of thumb, the radial distance from the load center to the geophone represents the depth underground to the material that that geophone is measuring. For example, D_7 is 48 in. from the load center, and its deflection represents the stiffness of the material that is 48" below the pavement surface.
    - By this rule, the D_7 deflection is often used to characterize the subgrade, where larger deflections (>4 mils) often correspond to weak and/or moisture related issues subgrades
    - D_1 is sometimes used to make inferences to the PMS condition, where larger deflections may indicate weak and/or stripped PMS (> 20 mils).
- D1/D7: This ratio can be used to make inferences of the pavement vs. subgrade conditions. For example, a high ratio would indicate a weak pavement on a stiff subgrade, while a low ratio may indicate a pavement and subgrade are both in good condition.
- Other FWD analysis parameters are shown in Table 16.

FWD Back-calculated Pavement Layer Resilient Modulus

The FWD property that is most useful to the pavement designer is the back-calculated layer moduli of each pavement layer and subgrade. Backcalculation is a process where FWD deflections are used to estimate the in-situ elastic modulus (E) of each pavement layer. Backcalculation is beyond the scope of this manual, but the computational method is available from the NDT unit.

Note that backcalculation estimate the elastic (Young's) Modulus, where MDT's and AASHTO's pavement design methods are based upon resilient modulus (M_R). E and M_R are fundamentally different properties, but it is generally believed that E is correlated to M_R for pavement design purposes. Therefore, back-calculated E needs to be converted to M_R using the following conversion:

$$M_R = E_{FWD} * C$$

Where:

M_R = the resilient modulus for any pavement layer or subgrade to be used for pavement design

E_{FWD} = pavement layer or subgrade elastic modulus back-calculated from FWD data

C = Conversion factor (Table 16)

Ground Penetrating Radar Evaluation

GPR is used to measure in-place PMS thickness. MDT research has concluded that GPR can accurately measure PMS thickness to within 95% accuracy, making it a useful tool for project evaluation (Maser, 2011). See example GPR test results in Figure 49.

How GPR works: GPR works by sending a tiny pulse of energy into pavement and recording both the time required for the pulse to reflect out of the pavement and the strength of the reflection. These pulses are collected from a vehicle traveling down a road, and the series of pulses over a road makes up a scan. The scan can be interpreted to determine the thickness of existing pavement layers and soil layers, and can detect most non-soil object (utilities, culverts, etc.) below the ground surface within its depth range. The GPR scan can be processed to determine PMS layer thickness, and those thicknesses are often graphed in a Microsoft Excel format (Figure 49).

MDT's current GPR equipment: The NDT unit has two pavement testing vehicles (Figure 47**).** Each truck is equipped with a 2-GHz and a 400 MHz GPR antennae. These antennas are controlled by a SIR 30 control unit which is connected to a laptop. The 2 GHz antennae are designed to only reach 1foot to 1 ½ foot beneath the pavement surface.

The 400 MHz GPR antennae has the ability to measure deeper into the pavement section and measure the existing base course thickness.

GPR can be particularly useful for optimizing milling depths and pulverization depth on MDT projects, or any other instance where PMS thickness is needed.

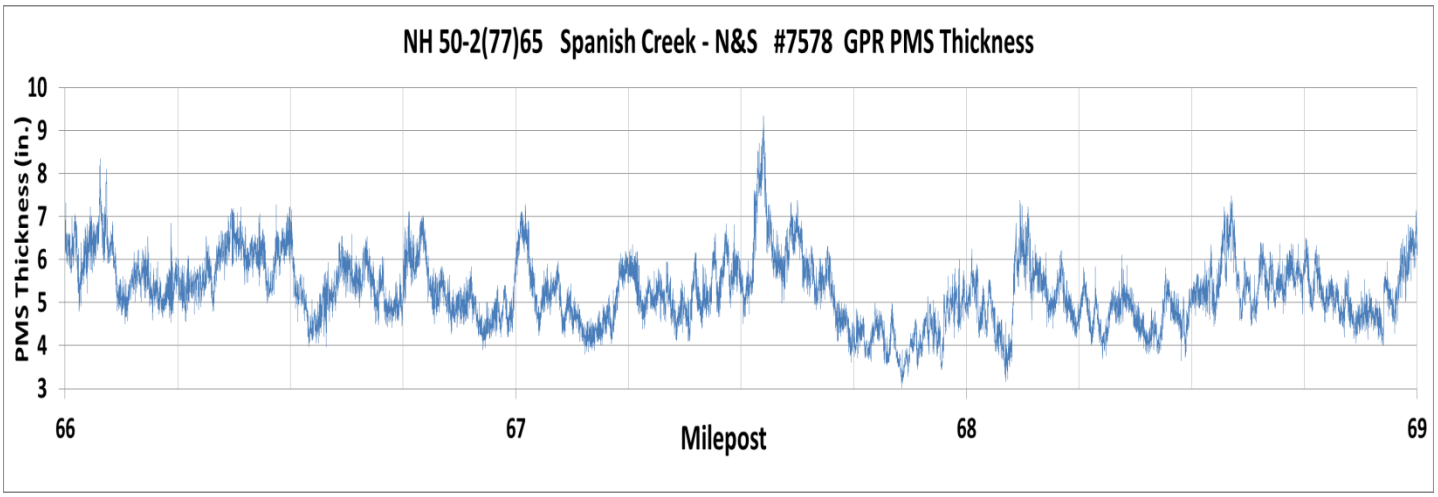


Figure 49: Sample GPR Pavement Thickness Data

Determining PMS, CAC, CTB, and Subbase Structural Coefficient Based Upon FWD Elastic Modulus

Guidance within this section can be used to estimate pavement layer structural coefficients based upon FWD back-calculated elastic modulus (E), as well as other test results as shown within the following figures.

Recall from Chapter 7 that back-calculated E has to be converted to M_R before using it for pavement design purposes. The Designer performs conversion using the conversion factors presented in Table 16. This conversion must be done before utilizing Figure 50, Figure 51, Figure 52, and Figure 53.

One possible exception to Figure 50 is determining structural coefficients for polymer modified Grade D and Grade S PMS. These PMS types became common in the late 1990's through today, and probably have M_R and Structural coefficients higher than shown in Figure 50. It is MDT's policy to use a maximum of 0.33/in for existing PMS surfacing, but the designer should know that this is probably very conservative for polymer modified PMS.

Table 16: Conversion Factors for Converting Back-calculated Layer Modulus to Laboratory Equivalent Resilient Modulus* (VonQuitus, 2007).

Layer & Material Type	Layer Description	Adjustment Factor, C_{FWD} (M_R/E_{FWD})	
		FHWA Pamphlet	Montana Sites
Aggregate Base Layers	Granular base under a PCC surface	1.32	---
	Granular base under a CAM layer; semi-rigid pavement	---	0.75
	Granular base above a stabilized material (a Sandwich Section)	1.43	---
	Granular base under an HMA surface or base	0.62	0.60
Subgrade Soil/Foundation	Soil under a CAM layer; no granular base	---	1.00
	Soil under a semi-rigid pavement with a granular base/subbase	---	0.50
	Soil Under a Stabilized Subgrade	0.75	---
	Soil under a full-depth HMA pavement	0.52	---
	Soil under flexible pavement with a granular base/subbase	0.35	0.50
Cement Aggregate Base Layer	Cement stabilized or treated aggregate layers	---	1.50
HMA Mixtures	HMA surface and base layers; 41 °F	1.00	0.9
	HMA surface and base layers; 77 °F	0.36	0.6
	HMA surface and base layers; 104 °F	0.25	0.5

*CAM = Cement Aggregate Mix = Cement Treated Base

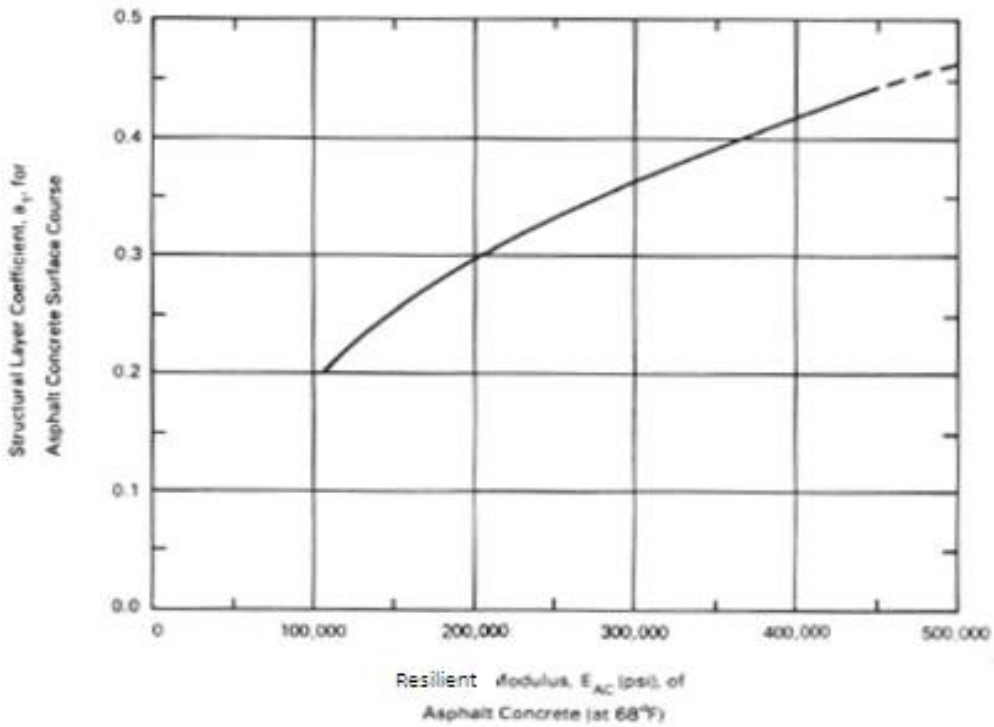


Figure 50: Conversion Graph: FWD M_R to PMS Structural Coefficient (1993 AASHTO)

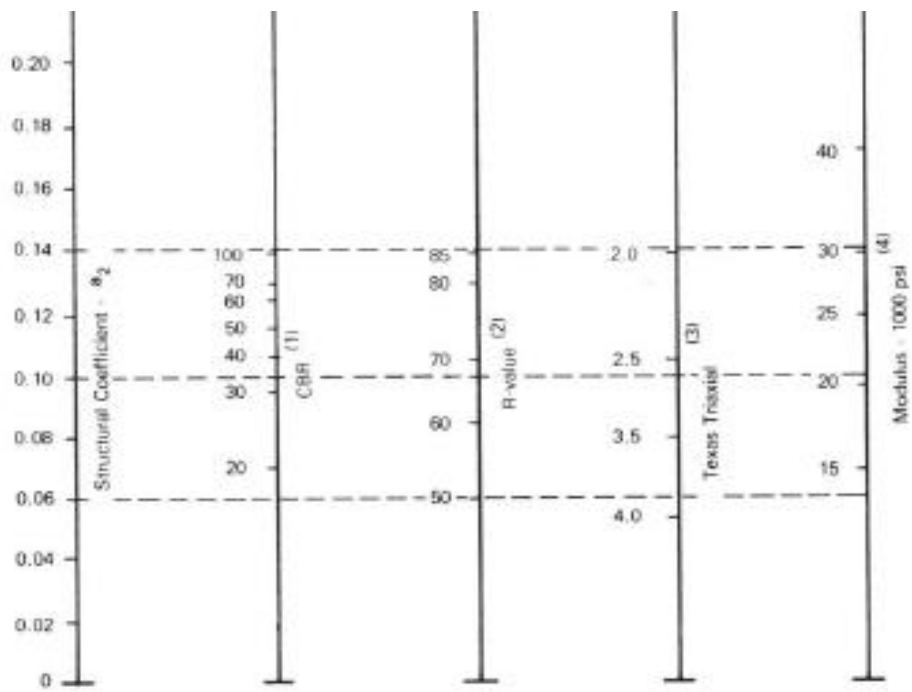


Figure 51: Conversion Graph: FWD M_R to CAC Structural Coefficient (1993 AASHTO)

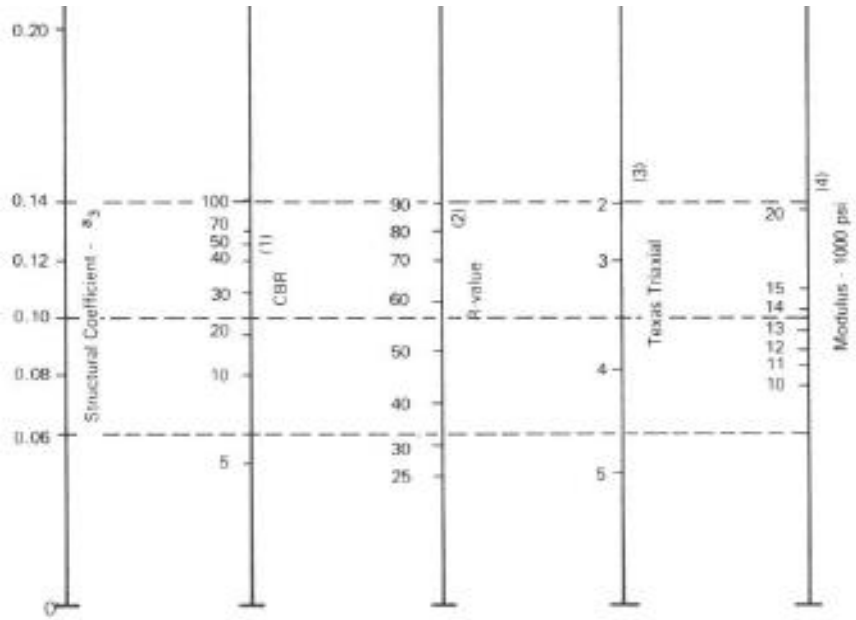


Figure 52: Conversion Graph: FWD M_R to Granular Subbase Structural Coefficient

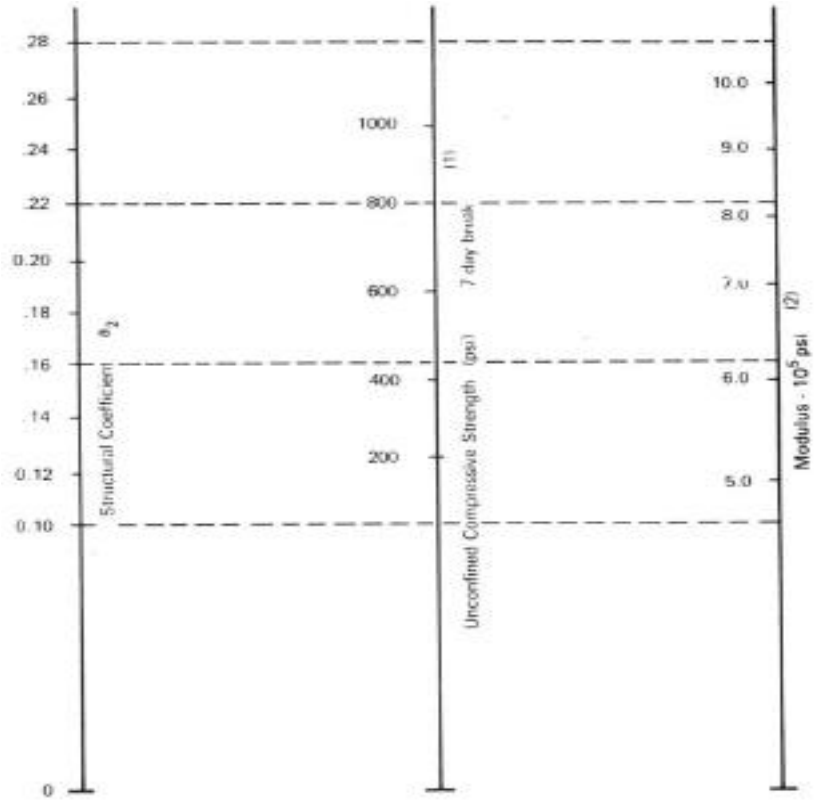


Figure 53: Conversion Graph: FWD M_R to CTB Structural Coefficient (1993 AASHTO)

Using GPR to determine existing PMS depths

GPR results such as those shown in Figure 49 should be used in conjunction with soil survey and core stripping results to determine existing PMS depth for rehabilitation projects. The

designer should use the **average** GPR PMS depth for rehabilitation design. The designer should be aware of thinner locations that may warrant further investigation to ensure there is enough structural integrity to support construction equipment. In situations where GPR data indicates a significant PMS depth change (such as the difference between MP 66.0 and 67.7 and MP 67.8 and 69 in Figure 49) the designer should consult the project manager to consider designing separate rehabilitation typical sections for the differing sections. This may not always be practical and rarely occurs because of the increased complexity of design.

Existing Pavement Sampling and Testing

This section describes how soil survey and PMS curves can be used to estimate pavement layer thickness and condition.

Estimating PMS Structural Coefficient from Core Evaluation

PMS Core evaluation is done in accordance with MT 331, *Method of Sampling and Evaluating Stripping Pavements*. This consists of coring the PMS at 1/2 mile intervals in alternating lanes on 2-lanes roads (also 1/2 mile intervals on 4-lane roads), measuring the cores, and splitting the cores under indirect tension. The split surface is visually characterized for stripping and scored from 0 to 4 as described in Table 17. The control pictures used for the stripping score test are shown in Figure 54. Sample Results from a past project are shown in Table 19. Stripping reports may be distributed in an Excel spreadsheet but are typically always available within Site Manager. Site Manager data may be obtained at the Transport Icon on the Intranet. Using the project UPN, the stripping report (if available) may be found at this link.

Table 17: PMS Core Evaluation

Core Rating:	Description:
4) Good:	- Face shiny, black all aggregate particles are coated.
3) Moisture Damaged:	- Loss of sheen, dull appearance some smaller (-10m) aggregate is uncoated.
2) Stripping:	- In addition to moisture damage some large aggregate is not coated.
1) Severely Stripped:	- Most of the aggregate is so clean the colors of the rock are easily seen.
0) No Core:	- Asphalt is mostly gone from all sizes of aggregate. The core has disintegrated.

Figure 54: Stripping Score Core Pictures

Stripping analysis results are useful as explained below:

- Both the total thickness, and individual lift thicknesses are provided. This is helpful for determining PMS thickness for structural calculations, or to design milling and pulverization depths.

- The stripping scores for each lift are useful for determining structural coefficients to calculate SN_{eff}

- Stripping scores are useful for designing milling depths. Stripping tests can identify poor PMS lifts that should be milled and removed, or ensure that milling operations end in competent plant mix. MDT past experience has shown that PMS with average stripping scores greater than 1.2 are competent enough to mill into, leaving a stable surface.

Milling into PMS with stripping scores less than 1.2 may result in the milled surface being soft, pitted, and too unstable to place new plant mix upon.

The Table 18 shows guidelines for reducing PMS structural coefficients based upon stripping scores. Use the **average** PMS thickness and condition along the project length for rehabilitation design.

Table 18 Guidelines for Adjusting PMS Structural Coefficients Based Upon Stripping Test Scores

Average Stripping Score	Structural Coefficient in. (ft.)
3.0 – 4.0	0.33 (4.0)
1.5 – 3.0	0.27 (3.2)
0 – 1.5	0.20 (2.4)

Table 19: Sample Stripping Analysis Results

Project:	IM 90-2(121)94	Lab#:	u3521119l153538	Date:	10/5/2011				
Termini:	Missoula - West	Evaluated By:	EP/BW						
	Description	Core Length (1/10 Ft)					Rating:		
Sample#	Location-MP	Overall	Chip	Top	2nd	3rd	Top	2nd	3rd
1	Reserve St. EB on Ramp	0.58	0.02	0.35	0.21		2	1	
2	103.9 EBDL RWP	0.41	0.00	0.18	0.22		2	2	
3	104.4 EBDL RWP	0.42	0.00	0.16	0.26		2	2	
4	104.9 EBDL RWP	0.37	0.00	0.17	0.2		2	2	
5	105.63 EBDL RWP	0.33	0.00	0.16	0.17		2	2	
6	104.15 EBPL LWP	0.46	0.00	0.17	0.29		2	2	
7	10407 EBPL LWP	0.47	0.02	0.18	0.27		2	2	
8	105.36 EBPL LWP	0.49	0.01	0.18	0.3		2	2	

Estimating Base and Subbase Structural coefficients from Pavement Soil Survey Results

Pavement soil survey results are useful to the pavement designer as follows:

- **Pavement layer thicknesses:** Use the **average** pavement layer thicknesses along the project length for rehabilitation design.
- **Base Course Properties**
  - **Soil Classification:** Can be used to estimate the amount of subgrade fines that have contaminated the base. MDT has generally used A-1-a(0) crushed gravel for base course, but roads built by other agencies may have used lower quality material. In the past, MDT used base course with up to 12% passing the No. 200

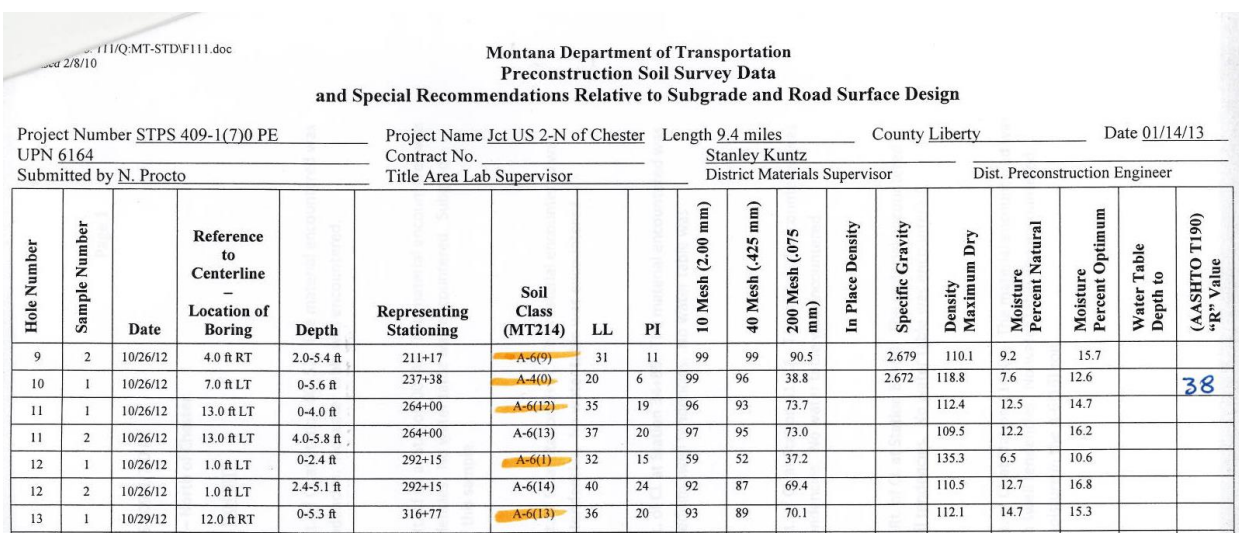
sieve (fines). Since about 2000, the amount of fines has been reduced to 8%. Levels of fines above 12% may indicate base course contamination from the underlying subgrade.

- **Gradation and Atterburg Limits:** The amount of fines and plasticity index can be used to estimate the resilient modulus and structural coefficient as shown later in this section. As fines increases, the base course weakens and may begin to hold more water.

- **Natural and Optimum moisture content:** The moisture content can indicate the amount of contamination that has occurred. Generally, new base course (CAC) has optimum moisture content from about 5-7%. Optimum moisture contents below 5% and above 8% may indicate a very clean base course or a contaminated base course, respectively.

- **Subgrade Properties**

  - **Soil Classification, Gradation and Atterburg Limits:** This may be the best indicator of subgrade quality available. The AASHTO Soil Classification was originally designed to characterize soils for road building, as shown in Figure 56. Generally, as the amount of fines increases, the moisture sensitivity and strength characteristics of the subgrade increase and decrease respectively. As the Plasticity Limits and Liquid Limit increase, its ability to hold moisture increases while constructability decreases.



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Montana Department of Transportation
Preconstruction Soil Survey Data
and Special Recommendations Relative to Subgrade and Road Surface Design

Project Number STPS 409-1(7)0 PE Project Name Jct US 2-N of Chester Length 9.4 miles County Liberty Date 01/14/13
UPN 6164 Contract No. Stanley Kuntz
Submitted by N. Procto Title Area Lab Supervisor District Materials Supervisor Dist. Preconstruction Engineer

Hole Number	Sample Number	Date	Reference to Centerline – Location of Boring	Depth	Representing Stationing	Soil Class (MT214)	LL	PI	10 Mesh (2.00 mm)	40 Mesh (.425 mm)	200 Mesh (.075 mm)	In Place Density	Specific Gravity	Density Maximum Dry	Moisture Percent Natural	Moisture Percent Optimum	Water Table Depth to	(AASHTO T190) "R" Value
9	2	10/26/12	4.0 ft RT	2.0-5.4 ft	211+17	A-6(9)	31	11	99	99	90.5		2.679	110.1	9.2	15.7		
10	1	10/26/12	7.0 ft LT	0-5.6 ft	237+38	A-4(0)	20	6	99	96	38.8		2.672	118.8	7.6	12.6		38
11	1	10/26/12	13.0 ft LT	0-4.0 ft	264+00	A-6(12)	35	19	96	93	73.7			112.4	12.5	14.7		
11	2	10/26/12	13.0 ft LT	4.0-5.8 ft	264+00	A-6(13)	37	20	97	95	73.0			109.5	12.2	16.2		
12	1	10/26/12	1.0 ft LT	0-2.4 ft	292+15	A-6(1)	32	15	59	52	37.2			135.3	6.5	10.6		
12	2	10/26/12	1.0 ft LT	2.4-5.1 ft	292+15	A-6(14)	40	24	92	87	69.4			110.5	12.7	16.8		
13	1	10/29/12	12.0 ft RT	0-5.3 ft	316+77	A-6(13)	36	20	93	89	70.1			112.1	14.7	15.3		

Figure 55: Soil Survey Data

TABLE 1-8 AASHTO SOIL CLASSIFICATION SYSTEM

General Classification	*Granulary Materials (35 % or less passing No. 200)*							*Silt-Clay Materials (More than 35 % passing No. 200)*			
	A-1			*A-2*							*A-7*
Group classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*	*A-4*	*A-5*	*A-6*	*A-7-5, A-7-6*
Sieve analysis, percent passing:											
No. 10 (2.00 mm)	50 max	...	...	...	...	...	...	...	...	...	...
No. 40 (425 μm)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
No. 200 (75 μm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 μm):											
Liquid limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[A]
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General rating as subgrade	Excellent to good							Fair to poor			

[A]Plasticity index of A-7-5 subgroup is equal to or less than *LL* minus 30. Plasticity index of A-7-6 subgroup is greater than *LL* minus 30.

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Figure 56: AASHTO Soil Classification System

- **Natural and Optimum moisture content:** The designer should evaluate whether the natural water content is greater than the optimum water content. When this occurs, it indicates that the subgrade is soft and may not support construction equipment, and that fines may pump into the base course.

- **Maximum Dry Density:** Generally as this amount increases, the soil quality for roadbuilding increases. This property ranges from 80 – 145 lbs./cubicft., where 80 lbs. indicates plastic clay that will hold water and 145 lbs. indicates very dense, drainable gravel.

The existing base or subbase structural coefficient can be estimated using the following equation (NCHRP 1-37a):

$$M_{R(Base)} = 2555 * (75 / (1+0.00728 * PI * P200))^{0.64}$$

Where:

Maximum $M_{R(Base)}$ = 29,000 psi

$M_{R(Base)}$ = Resilient Modulus of the unbound base or subbase course (psi)

PI = Plasticity Index (%)

P200 = Percent Passing the No. 200 Sieve (%)

Use the **<u>average</u>** $M_{R(Base)}$ along the project length to determine the structural coefficient for pavement design using Figure 51. The structural coefficients provided in Chapter 6 should be used unless otherwise approved by the Pavement Analysis Engineer.

Estimating CTB Structural Coefficient

Estimating the structural coefficient of existing CTB is difficult, particularly on thinner pavements. To separate the PMS and CTB layers for backcalculation is difficult. Laboratory testing requires removing an intact CTB core for unconfined compression testing, which MDT does not do.

Fortunately, testing data from prior CTB projects is available to assist designers. Figure 57 shows historical CTB compressive testing results. The figure shows that the average CTB is about 550 psi, which corresponds to a structural coefficient ~0.18. Based upon this, 0.18 is considered to be a good starting point for estimating a CTB structural coefficient. The designer can reduce or increase the coefficient if he/she has better information regarding CTB quality and strength on a given project. The current specification requires CTB to achieve a 7 day compressive strength between 500 – 1500 psi. The structural coefficients provided in Chapter 6 should be used unless otherwise approved by the Pavement Analysis Engineer.

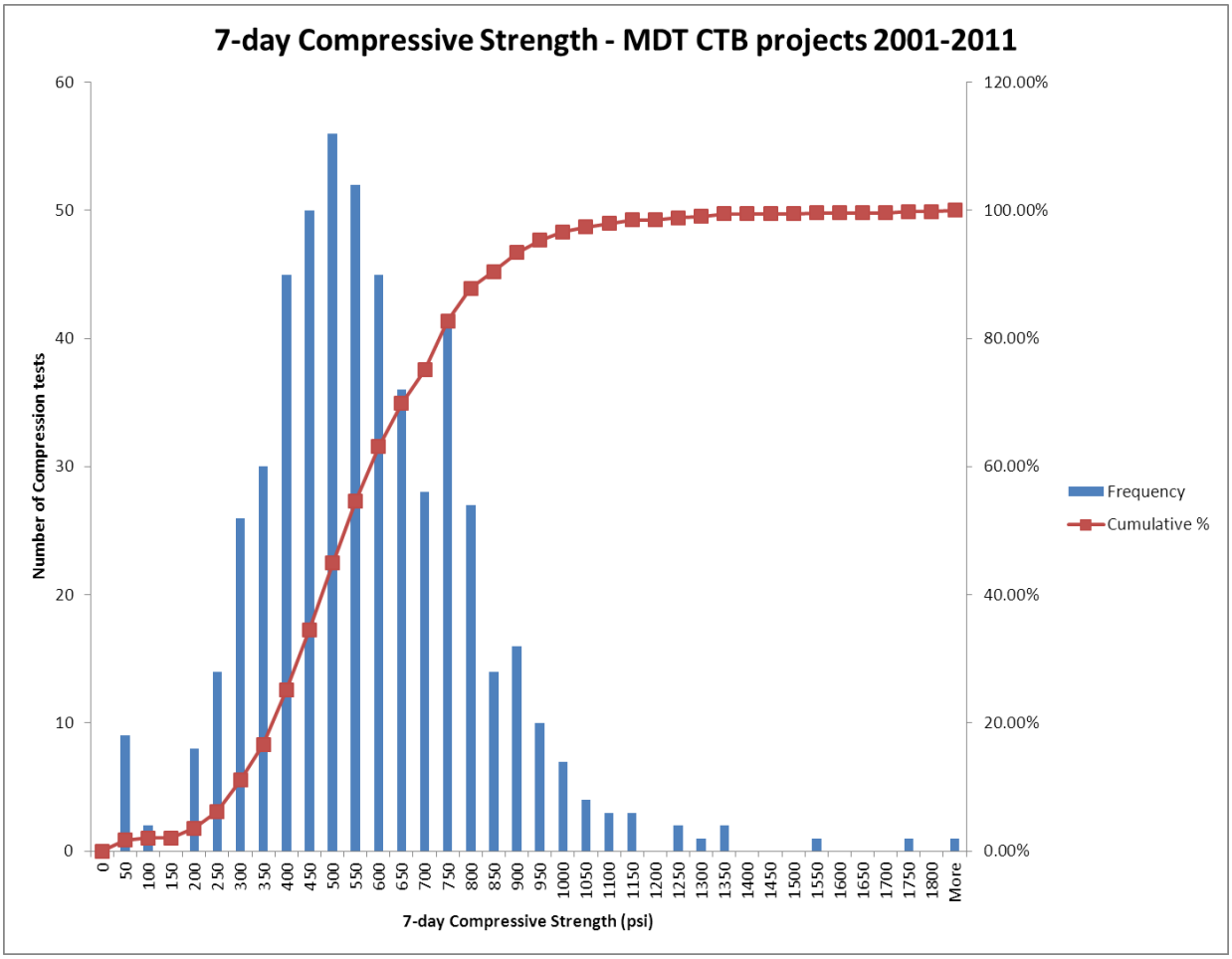


Figure 57: CTB 7-day Compressive Strength Test Records 2001-2011

(Continuing from Step #2: Determine Existing Pavement Structural Capacity (SN_{eff}))

Step #3: Determine Future Pavement Structural Capacity (SNdgn)

Determine SN_{DGN} using the same procedure used for new or reconstructed pavements outlined in Chapter 6.

Step #3B: Complete Pavement Rehabilitation Design

Design a rehabilitation pavement section that increases the existing pavement structure and performs well over the design life. Often, a pavement widening section has to be designed in addition to the rehabilitated section (See the example typical sections in Chapter 7 for more information on widening sections).

Completing the Rehabilitated Pavement Design

Determine the amount of pavement structure that will be added to the existing pavement to perform well over its anticipated design life. This is done using the following equation:

$$SN_{dgn} - SN_{eff} = SN_{ol}$$

Where:

SN_{dgn} = The structural number required for future traffic loading determined during Step #3.

SN_{eff} = The structural number of the existing pavement determined during Step #2.

SN_{ol} = The Structural Number deficiency between the existing pavement and that needed for the future pavement design. This is the amount of pavement structure that needs to be added during pavement rehabilitation.

The additional pavement structure, SN_{ol}, is added by either adding additional pavement thickness, or improving the strength of the existing pavement layers. Additional pavement thickness is usually added by adding additional CAC and/or PMS to the existing pavement. Improving the existing pavement strength can be done by:

- PMS layer:
  - Mill existing weak PMS and replace with new PMS
  - Improve existing pavement strength using in-place recycling.
- Base or Subbase layer:
  - Use pulverization to mix weak and/or contaminated base course with overlying PMS material. The pulverized mixture usually adds strength, drainage, and reduces the amount of fines when compared to the existing base layer.

- Use pulverization to add stabilizing agents (usually Portland Cement) to the existing base.

Completing the Widening Section Design

The pavement widening section design is usually designed similarly to a new pavement. Follow the steps in Chapter 6 *Flexible Pavement Design Method* to design the widening section.

Also consider the following when designing widening sections:

- Consider using the material from the rehabilitation section within the widening section. For example, pulverized material can be bladed over to provide CAC needed for the widening section. Pavement millings can also be blended with virgin CAC to provide CAC for the widening section.

- The widening sections base course should extend to or below the depth of the existing gravel section. This facilitates drainage and avoids designing a “bathtub” pavement section.

- Can the PMS section be thinner than the adjacent rehabilitated section? The designer should approach this with caution, but if the widening section will be a pavement shoulder with little or no traffic loading, consider using a thinner PMS layer upon the widening section. Consider the following guidelines:

  - The minimum shoulder thickness should be 0.30’ PMS in order to provide depth for future milling operations.

  - The designer should not thin shoulders on Interstate Pavements, pavements where future traffic lanes may be placed on the shoulder, or shoulders that may become traveled upon if turning lanes are installed in the future.

Step #4: Specify Pavement Materials and Construction Methods

Specify Materials to be used within the designed pavement using the same procedure used for new or reconstructed pavements. This process is outlined in Chapter 6.

Step #5: Send Pavement Design Memo

Send a surfacing design memo to the road design staff via interdepartmental mail. Figure 23 shows an example surfacing design memo. The following list includes items that are included on the surfacing design memo

- To: Highways Engineer

- Thru: Pavement Design Engineer (with signature)

- From: Designer (with signature)

- Date
- Subject Project Number, Name, and CN Number
- Designate Preliminary or Final and EPS Act. 600 or 604, respectively
- PMS, CAC, and Total Thicknesses for both the rehabilitation and widening sections
- A description of milling and pulverization depths
- Thorough description of the type and order of construction operations needed for pavement rehabilitation
- Description of stabilizers (i.e. portland cement)
- Design R-Value
- The Daily ESALs used for Design
- Plant Mix Type
- Use of RAP and PMS RAP content
- Design Life Length (yrs.)
- Binder type
- Asphalt Content
- CC'd to District Administrator, Road Design Engineer, Road Design Project Manager, and Surfacing Materials, and Geotechnical

An effort should be made to make the surfacing design memos as detailed as needed for the road designer to develop both project reports and construction plans.

After completing the preliminary surfacing design memo with Microsoft Word, save the .doc file using the 7 digit Construction Number (CN) as the file name. Save the file in the project folder on the shared drive. Once the memo has been submitted for distribution and saved, the EPS activity may be carded off.

When the paper copy of the memo is distributed, the original copy is stamped "Master Copy" with a green stamp by the Department's mail staff, and sent back to surfacing design. This "Master Copy" is stored in the surfacing design project file.

There will be instances when the design memo is sent electronically via email. When this is necessary, the Word file (.doc) should be converted to an Adobe Acrobat File (.pdf) before sending it. The purpose of this is that a .pdf file cannot be altered.

Montana Department of Transportation
Helena, MT 59620-1001

Memorandum

To: Paul Ferry, P.E.
Highways Engineer

Thru: Daniel W. Hill, P.E.
Pavement Design Engineer

From: Ed Shea
Surfacing Design

Date: August 13, 2008

Subject: STPP 69-1(9)22
Boulder South
Control #2019

Hereby transmitted, are the revised surfacing sections, for the surfacing structure on the subject project. This recommendation completes the Surfacing Design 604 activity.

Surfacing Section No. 1 - Pulverization – BOP to EOP
0.30' Plant Mix Bituminous Surfacing Grade S
Design R-Value = 27
Pulverize to a depth of 0.60'

Surfacing Section No. 2 –Widening – BOP to EOP
0.30' Plant Mix Bituminous Surfacing Grade S
0.80' Crushed Aggregate Coarse
1.10' Design R-Value = 27

To avoid any bathtub section the embankment in the Widening will be required to have an A-1 soils class. The designs are based on 147 ESAL's from traffic stated in the PFR. The design life of the recommendations is 20 years considering AASHTO design procedures. The Grade of Plant Mix and the PG Binder were determined as per Materials Bureau Policy dated April 7, 2005. The recommended PG Binder is **64-28**. The recommended size of PMS aggregate is 3/4".

The listed R-values of the subgrade soils were used to determine surfacing thicknesses. Soil classes are not shown because of the range of R-values possible within a particular soil type. The R-values used in the AASHTO method are based upon a statistical analysis and the 85th percentile of that analysis. Prior to construction, the District should review the soil survey for R-values. Areas of concern and possible borrow areas should have additional soil samples submitted for R-value testing.

If you have any questions concerning this recommendation, please contact Ed Shea.

cc: J. Ebert, P.;E.
D. Krings, P.E.
J. Davies, P.E.
Surfacing File
Materials Bureau File
Geotechnical File

Figure 58: Example Rehabilitation Design Memo (without signatures)

7.1.5 Flexible pavement rehabilitation design example

The following design example summarizes the design of a major rehabilitation pavement section for the Alberton E. and W. project. This is a high-traffic Interstate, and the example summarizes

a pavement design using the DARWin software. The following information presents a summary of items that may affect the pavement design:

Project Name:	Alberton E. and W.
Project Number:	IM 90-1(196)74
Construction Number(CN):	7523
Existing Surface:	0.2' PMS
	0.15-0.30' Hot Recycled PMS
	0.15-0.2' CTS
	1.25'-1.50' CAC
Project Limits:	RP 74.4 – 84.0
Project Type:	Major Rehabilitation without Added Capacity
District:	Missoula
Geographic Province:	Rolling Terrain - Rural
Weather / Climate:	Moderate Precipitation, Multiple Freeze-Thaw During Spring and Fall, frozen during winter
Traffic information:	Daily ESALs = 1104
Daily AADT (Average Annual Daily Traffic) = 7,090	
Soils:	Mostly Gravel
FWD Subgrade Lab M_R = 12,000 psi	

Determine Pavement Rehabilitation Method

This pavement is in poor condition. The District decided that this pavement needed major rehabilitation because the Maintenance Division was spending a lot of time and money maintaining the roadway.

The road was reconstructed between 1963 and 1979 (projects I-90-1(17), I-90-1(70), I-90-2(51) and I-90-2(21)). In 1998, an overlay and chip seal was applied to the roadway under project IM 90-1(119)74. In 2006, another chip seal was applied under project SFCI 90-1(156)74.

A key design feature on this project is the vertical grade cannot be raised.

Core samples have been obtained from the MDT Missoula District Materials. The following is a summary of the existing surfacing depths.

I-90 mainline

Location	*Surfacing Depth Range*	*Surfacing Depth Average*
Driving Lane	0.58' – 1.05'	0.77'
Passing Lane	0.55' – 0.99'	0.75'
Driving Lane Shoulder	0.34' – 0.94'	0.72'
Passing Lane Shoulder	0.55' – 0.99'	0.75'

I-90 Interchange Ramps

Location	*Surfacing Depth Range*	*Surfacing Depth Average*
Exit 75 (Alberton)	0.56' – 0.76'	0.61'
Exit 77 (Petty Creek)	0.38' – 0.46'	0.42'
Exit 82 (Nine Mile)	0.60' – 0.65'	0.62'

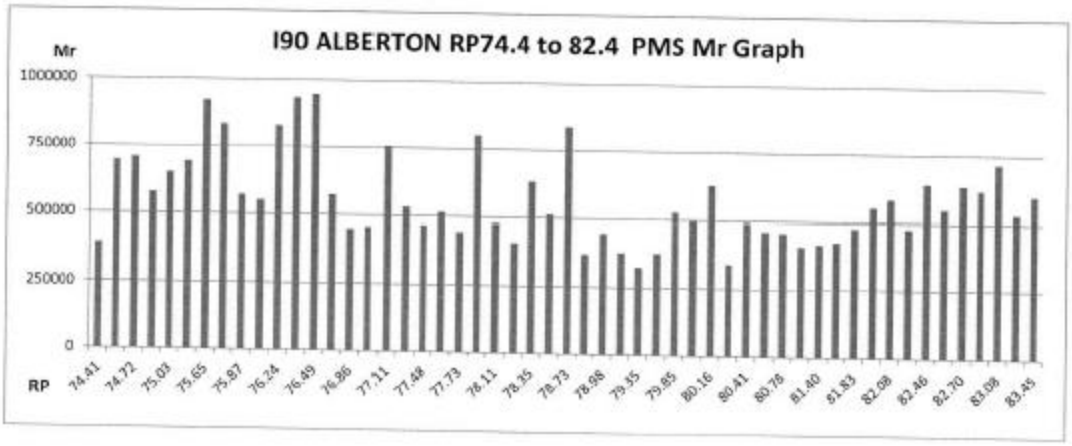
All the cores exhibited stripping in both the top and bottom layers (65 of the 83 cores had a severely stripped top layer).

The first step is to determine what the pavement distress is, and what is causing it. At the PFR, it is noted that there is potholing extending 2-4" into the PMS and raveling within the wheel paths. There does not appear to be rutting or alligator cracking. Based upon that, the pavement appears to be structurally okay, and the problem may be within the PMS layer.

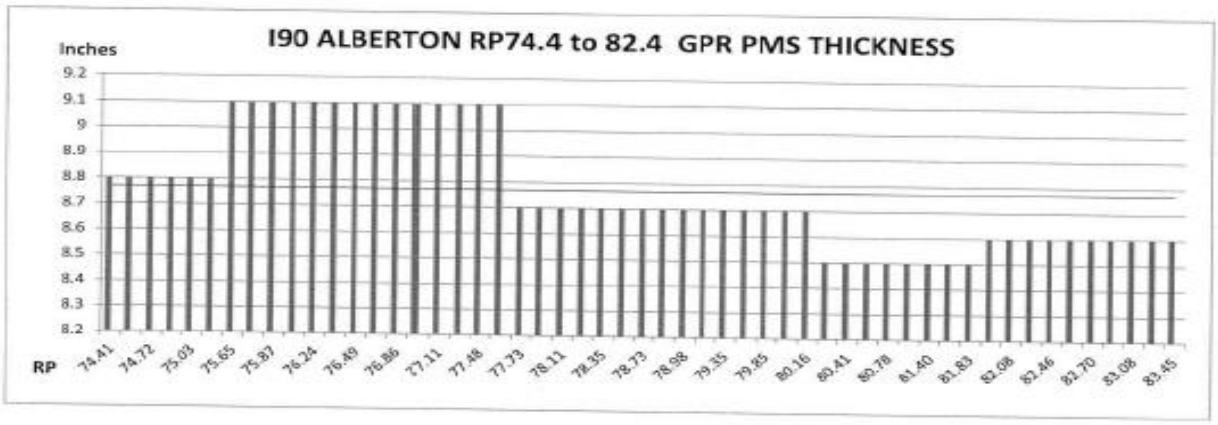
A PMS core evaluation was provided by the Helena Materials lab. A sample of the core report follows. The core report shows that there is moderate to severe PMS stripping occurring throughout the depth of the PMS.

	Description		Core Length (mm)						Rating:				
Sample#	Location-MP	Overall	Chip	Top	2nd	3rd	4th	5th	Top	2nd	3rd	4th	5th
61	RP 80 WB DL SH	0.62 BB	0.03	0.22	0.18				1	2	1		
62	RP 80.5 WB PL	0.96	0.03	0.20	0.20	0.20	0.20	0.13	1	2	2	2	2
63	RP 80.5 WB PL SH	0.93	0.02	0.20	0.20	0.15	0.14	0.22	1	2	1	1	1
64	RP 81 WB DL	0.88	0.02	0.26	0.20	0.28	0.12		1	2	1	2	
65	RP 81 WB DL SH	0.73	0.02	0.18	0.15	0.21	0.17		1	2	1	1	
66	RP 81.5 WB DL	0.85	0.02	0.30	0.17	0.36			1	3	2		
67	RP 81.5 WB DL SH	0.72	0.02	0.18	0.17	0.15	0.20		1	3	2	2	
68	RP 82 WB DL	0.60	0.01	0.30	0.15	0.14			1	2	1		
69	RP 82.25 WB PL	0.74	0.03	0.15	0.22	0.34			1	2	1		
70	RP 82.5 WB DL SH	0.60	0.03	0.28	0.15	0.14			2	2	1		
71	RP 83 WB PL	0.72	0.03	0.16	0.20	0.33			2	2	1		
72	RP 83.5 WB DL	0.58	0.02	0.14	0.23	0.19			1	3	1		
73	RP 84 WB PL	0.74	0.02	0.15	0.22	0.35			2	2	1		
74	RP 84 WB DL	0.72											
75	RP 84 WB DL SH	0.65	0.03	0.22	N/A	N/A			2	0	1		
76	RP 84 WB PL SH	0.67	0.01	0.17	0.33	0.16			2	2	1		
77	EX 75 EB ON RAMP	0.76	N/A	N/A	N/A	0.18			0	0	1		
78	EX 75 EB OFF RAMP	0.56	0.02	0.20	0.20	0.14			1	1	1		
79	EX 75 WB ON RAMP	0.56	0.02	0.28	0.26				1	1			
80	EX 75 WB OFF RAMP	0.56 BB	0.02	0.19	0.22	0.13			1	1	1		

The NDT unit provided GPR and FWD summarizing both the thickness and quality of the pavement layers



.



I90 Alberton RP 74.4 to 84.2

WB

19-AUG-08	PMS	BASE	SUB
Average	568440	88370	26766
Std. Dev.	157352	29024	8423
Corrected	458294	68053	20870
Lab	255798	54789	13383
	8.77"	15.38"	

The FWD Moduli summaries above indicate the PMS M_R is low. The PMS lab M_R is 255,798 psi, which is low for a heavily trafficked Interstate pavement. The GPR information shows the PMS thickness range from 8.5 to 9.1".

Soil Survey information was collected by the District for the base course to identify whether or not the base is in good condition. This information follows:

Reference to Centerline – Location of Boring	Depth	Representing Stationing	Soil Class (MT214)	LL	PI	10 Mesh (2.00 mm)	40 Mesh (.425 mm)	200 Mesh (.075 mm)	In Place Density	Specific Gravity	Density Maximum Dry	Moisture Percent Natural	Moisture Percent Optimum	Water Table Depth to	(AASHTO T190) "R" Value
84.0 WBDL	0.7-2.5		A-1-a(0)	NP	NP	34	22	6.9				2.3			76
82.5 WBSH	0.6-2.4		A-1-a(0)	NP	NP	44	19	8.3				2.4			80
80.5 WBDL	0.9-3.0		A-1-a(0)	NP	NP	38	20	8.6				1.8			75
78.5 WBDL	0.85-3.0		A-1-a(0)	NP	NP	32	21	6.3				2.0			75
75.5 WBDL	0.9-2.5		A-1-a(0)	NP	NP	46	25	8.2				2.1			72
74.5 WBDL	0.9-2.4		A-1-a(0)	NP	NP	47	26	8.2				1.8			75
76.0 EBDL	0.8-1.8		A-1-a(0)	NP	NP	26	16	8.2				3.4			67
76.0 EBDLSH	0.8-1.8		A-1-a(0)	NP	NP	22	14	6.0				3.1			84
78.0 EBDL	0.74-1.74		A-1-a(0)	NP	NP	30	19	8.1				3.4			69
78.0 EBDLSH	0.74-1.74		A-1-a(0)	NP	NP	27	16	6.8				3.5			73
80.0 EBDL	0.8-1.8		A-1-a(0)	NP	NP	44	28	12.2				3.1			77
80.0 EBDLSH	0.8-1.8		A-1-a(0)	NP	NP	43	26	11.9				3.6			79
82.0 EBDL	0.65-2.1		A-1-a(0)	NP	NP	40	20	8.3				2.4			79
82.0 EBDLSH	0.7-2.1		A-1-a(0)	NP	NP	38	16	7.0				2.6			73
84.0 EBDL	0.7-2.1		A-1-a(0)	NP	NP	37	25	8.0				2.6			81

The base course appears to be in very good condition since it all classified as A-1-a(0), is non-plastic, has about 8-9% fines, and has very high R-Values mostly in the upper 70's.

Based upon this information, the Designer decided that PMS stripping is the root cause of the pavement distress, and it should be removed and replaced.

The preliminary rehabilitation recommendation is to remove all PMS. This will be done by milling and removing the majority of the PMS, and pulverizing the remaining PMS into the existing base course. To reduce the project cost, the Designer and the District have decided that the shoulders should be left in place as much as possible.

Step #2 Determine the existing pavement structure's structural capacity (SN_{eff})

Estimating existing structural capacity is done by estimating the average depth and condition of the PMS and base course, and calculating SN_{eff} with the following equation:

$$SN_{eff} = SN_{PMS} + SN_{Base} + SN_{sub}$$

$$= d_{PMS}a_{PMS} + d_{Base}a_{base} + d_{sub}a_{sub}$$

$$= \sum_{1}^{n} d_{PMSn}a_{PMSn} + d_{Base}\, a_{base} + d_{sub}a_{bub}$$

Where:

SN_{eff} = existing pavement structural number

$d_{PMS}, d_{Base}, d_{Sub}$ = Average thicknesses of the existing PMS, Base Course, and Subbase Course (ft. or in.)

a_{PMS} , a_{base}, a_{sub} = The average effective structural coefficient of the existing PMS, Base Course, and Subbase Course layers

n = Number of individual PMS lifts within the existing pavement. Often, each lift (layer) of PMS has different levels of degradation, so the SN of each lift is calculated separately.

It is decided to determine the PMS depth based on the core information due to the abundance of cores. Based upon the core summary, the average PMS depth is about 0.75'

The core information is also used to determine the PMS condition as characterized to the structural coefficient. All lifts of the PMS are stripped and in about the same condition, so the entire PMS layer will be characterized with one structural coefficient rather than specifying a different structural coefficient for each lift. The average stripping score is about 1.5, which corresponds to a structural number 0.20/in or 2.4/ft. (Table 18).

Average Stripping Score	Structural Coefficient in. (ft.)
3.0 – 4.0	0.33 (4.0)
1.5 – 3.0	0.27 (3.2)
0 – 1.5	0.20 (2.4)

The **structural number of the PMS layer is 0.75' * 2.4/ft. = 1.8.**

The base course depth and coefficient is found from the base course soils survey. The average base course depth from the soil survey is 1.77'. The base course condition is also taken from the soil survey. The gravel all classifies as A-1-a(0), the fines average about 9% passing the No. 200 sieve and are non-plastic, and the R-Value average in the upper 70s, all indicating that the base course is in very good condition. The base course M_R is determined using the following equation:

$$M_{R(Base)} = 2555 * (75 / (1+0.00728 * PI * P200))^{0.64}$$

Where:

Maximum $M_{R(Base)}$ = 29,000 psi

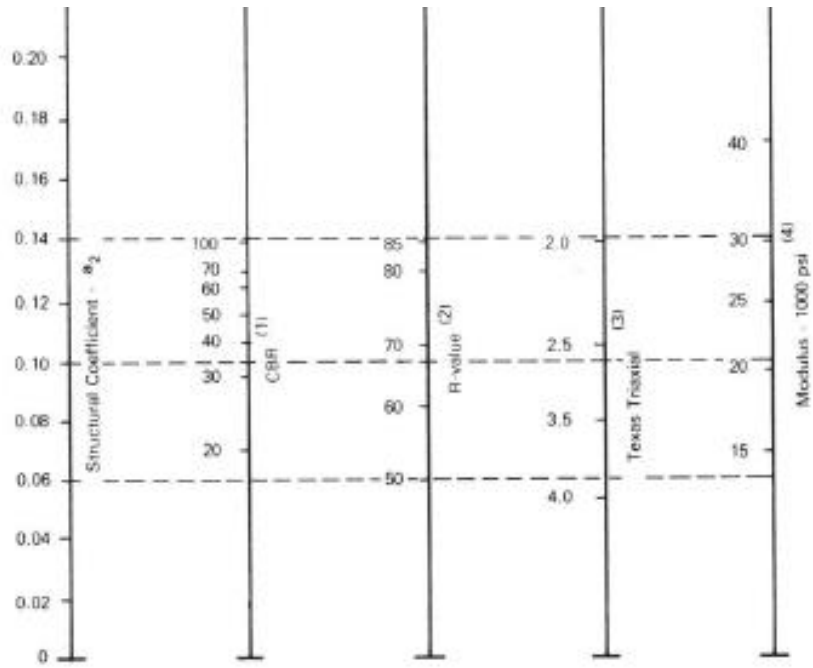
$M_{R(Base)}$ = Resilient Modulus of the unbound base or subbase course (psi)

PI = Plasticity Index (%)

P200 = Percent Passing the No. 200 Sieve (%)

Inserting 8% passing the No. 200 and PI = 0, the estimate base course M_R is 40,500 psi. Based upon Figure 51 (also shown below), 40,000 psi corresponds to structural coefficient = 0.18 / in. However, it is MDT policy to use maximum 0.12 /in for existing base course (

Table 3).



(Figure 47, as shown earlier in the document above.)

The base course SN = 1.77' * 0.12/in * 12 in/ft. = 2.55

The existing pavement structure is:

$$SN_{eff} = SN_{PMS} + SN_{Base} + SN_{sub}$$

$$= 1.8 + 2.55$$

$$= 3.35$$

Step #3 Determine the structural capacity needed for the new pavement (SN_{dgn})

DARWin 3.1 will be used to find SN_{dgn}. This Design is done in accordance with Chapter 6 Flexible Pavement Design Method.

DARWin is opened by double clicking on the desktop icon labeled "DARWin 3.1". Generally, an error message will occur, and the "Ok" button must be clicked several times before the program opens. The opening screen is seen below.

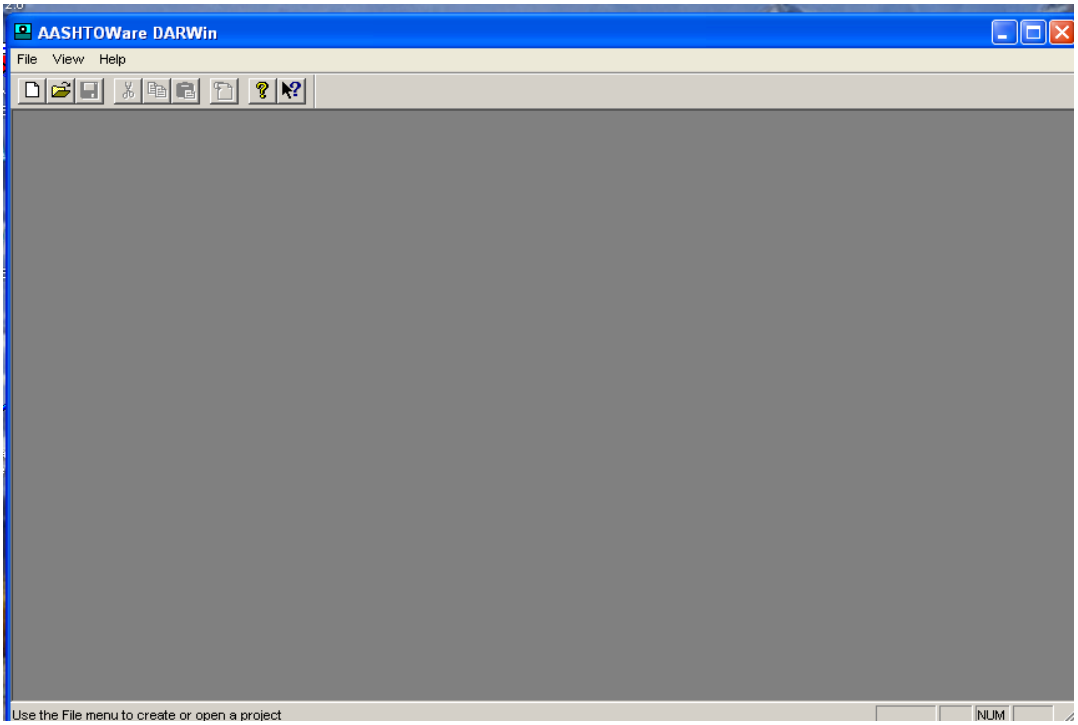


Figure 59 DARWin project screen.

To create a new project, click on “File” at the upper left, and select “New”. The following screen will appear. Type in the UPN/Control Number for the project in the space under “Project Name”, and click OK.

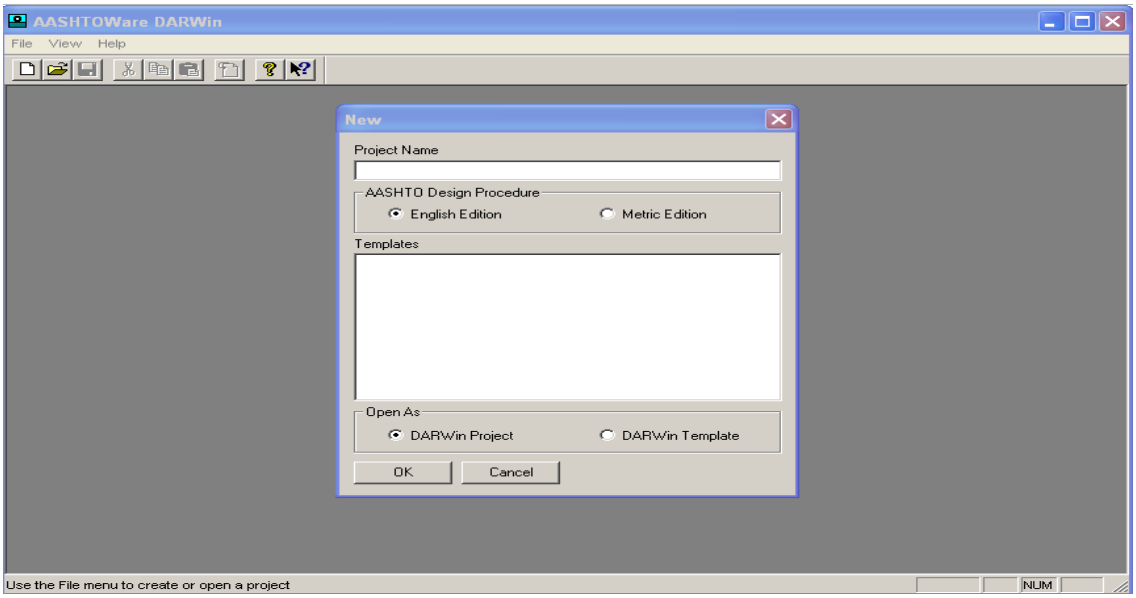


Figure 60 New project screen

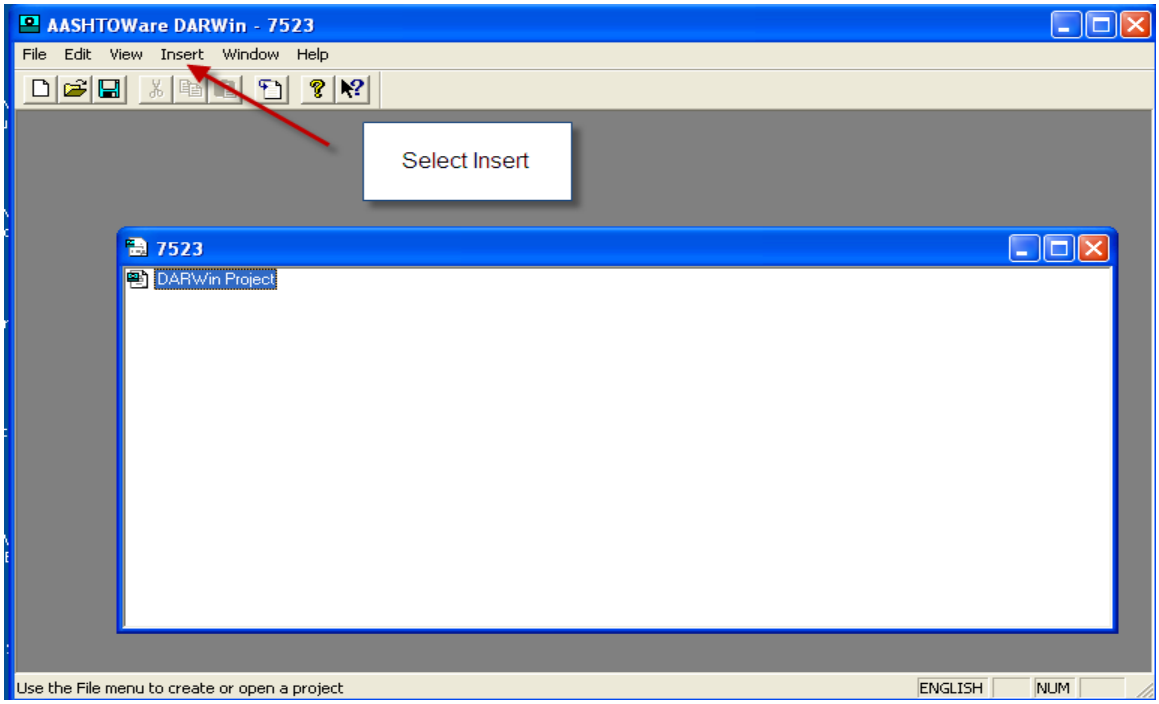


Figure 61 New project screen template.

Click on DARWin Project in the template, and then click on “Insert” in the header and select “New Module”.

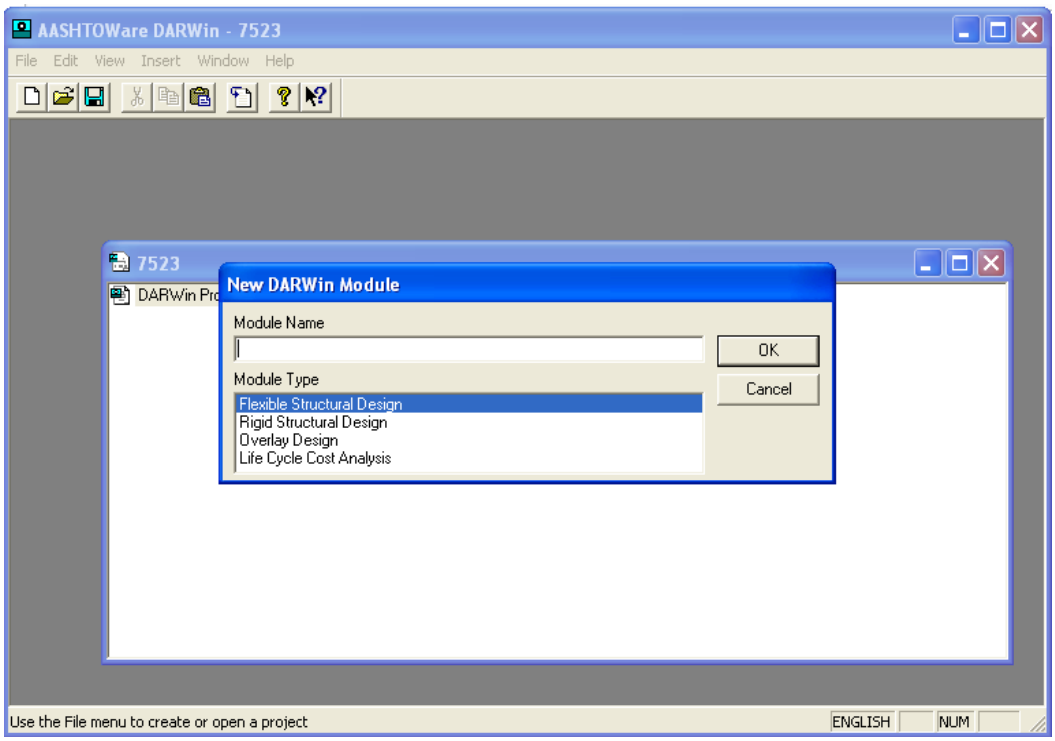


Figure 62 New DARWin Module template.

Enter the CN number as the module name and click OK: the module has automatically selected Flexible Structural Design. Under the Description, enter the project name and number, as well as any other relevant information.

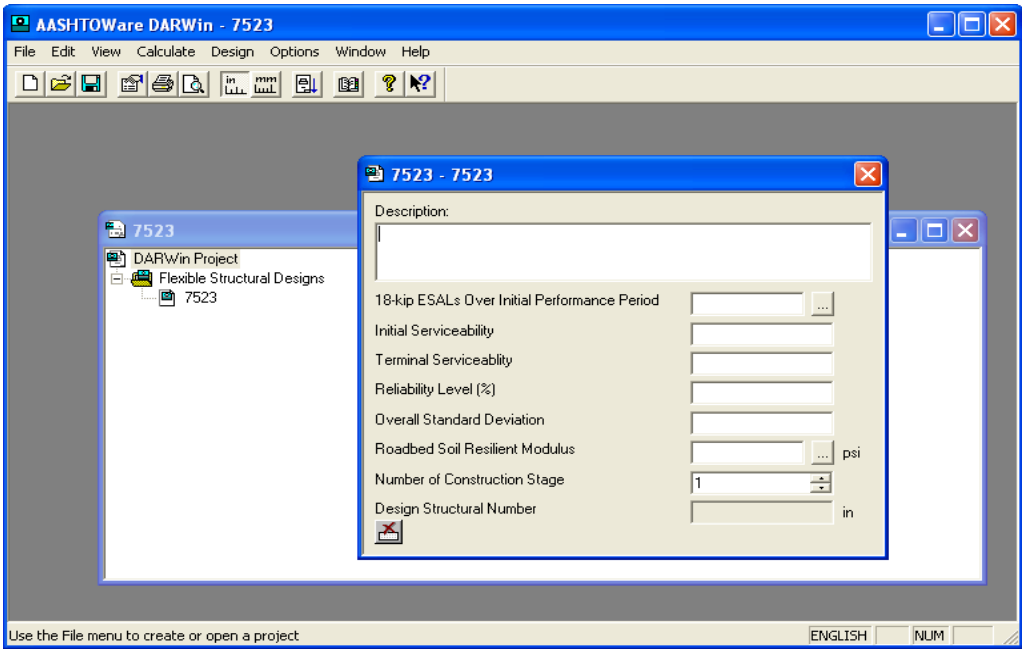


Figure 63 DARWin flexible structural design template.

Enter the Pavement Design Inputs:

Traffic: The traffic memo reports 1,104 daily ESALs. Lifetime ESALs are calculated by multiplying the daily ESALs by 365 days and 20 years design life.

1,104 X 365 X 20 = 8,059,200 lifetime ESALs

Serviceability: Enter 4.5 for Initial Serviceability, and 2.5 for Terminal Serviceability

Reliability: Enter 95% as the Reliability Level

Overall Standard Deviation: Use 0.45 as Overall Standard Deviation

Subgrade Modulus: The average FWD Subgrade lab $_{MR}$ is 13,383 psi. MDT typically uses 12,000 psi for subgrade soils with an R-Value of 30 (2 feet of special borrow). The designer in this example chose to use 12,000 psi for the design. No subgrade sampling was done on this project.

Now click the small button on the lower left with the red "X". A Design Structural Number of 4.10 will appear in the box below. Therefore, **SN_{dgn} = 4.10**.

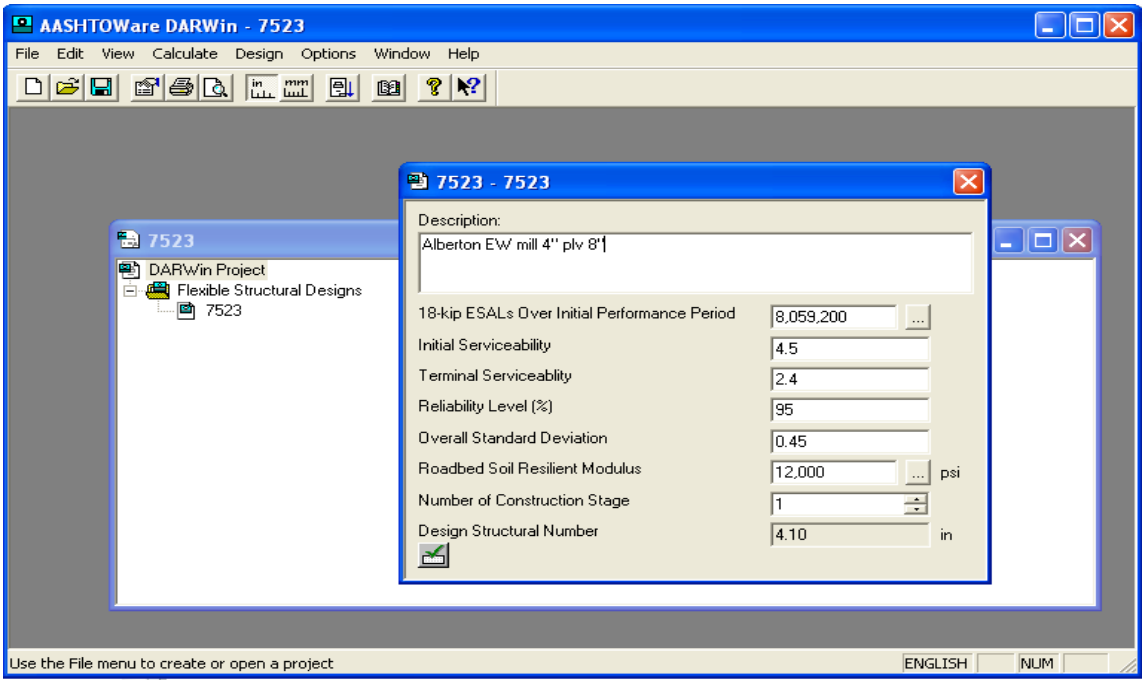


Figure 64 Completed design template with DARWin-calculated Design Structural Number.

Step #4: Complete Pavement Rehabilitation Design

SN_{ol} is the amount of pavement structure that needs to be added to the existing pavement to meet the current demand. SN_{OL} is calculated as follows:

$$SN_{ol} = SN_{dgn} - SN_{eff}$$

$$= 4.10 - 3.35$$

$$= 0.75$$

SN_{ol} is greater than 0, indicating that the existing pavement is structurally inadequate to perform well over the 20-year design period. Recall from Step #1 that the pavement grade cannot be raised, so the existing pavement will need to be strengthened to make up for the structural deficiency. Also recall from Step #1 that the existing PMS is stripped and needs to be removed or rehabilitated.

The rehabilitation design will be done using the DARWin 3.1 software. Click on “Design” on the header, and then select Thickness Design. The template will look like Figure 65 below.

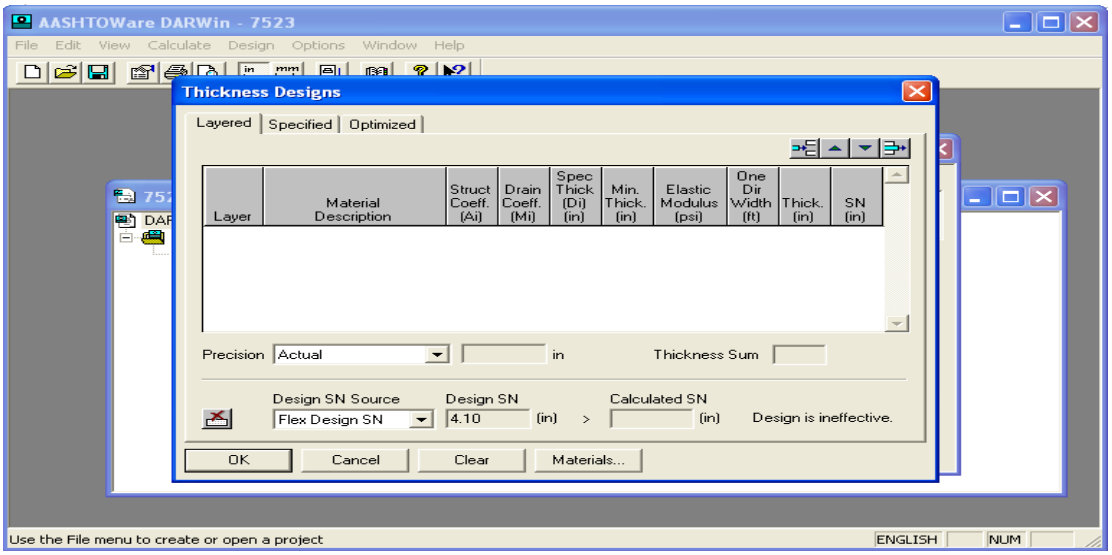


Figure 65 DARWin thickness design template.

Click the Specified tab at the top, and click the insert layer three times to provide for the PMS, pulverized layer, and existing base layer as shown in Figure 66.

Enter the material descriptions (i.e. PMS, CAC, existing CAC), structural coefficients, drainage coefficients, and thicknesses. Structural coefficients for the new PMS and pulverized material are shown in Table 20and structural coefficient for existing material was determined in Step #3. Drainage coefficients are routinely assigned 1.0 by MDT. Minimum PMS design thicknesses are shown in Table 21.

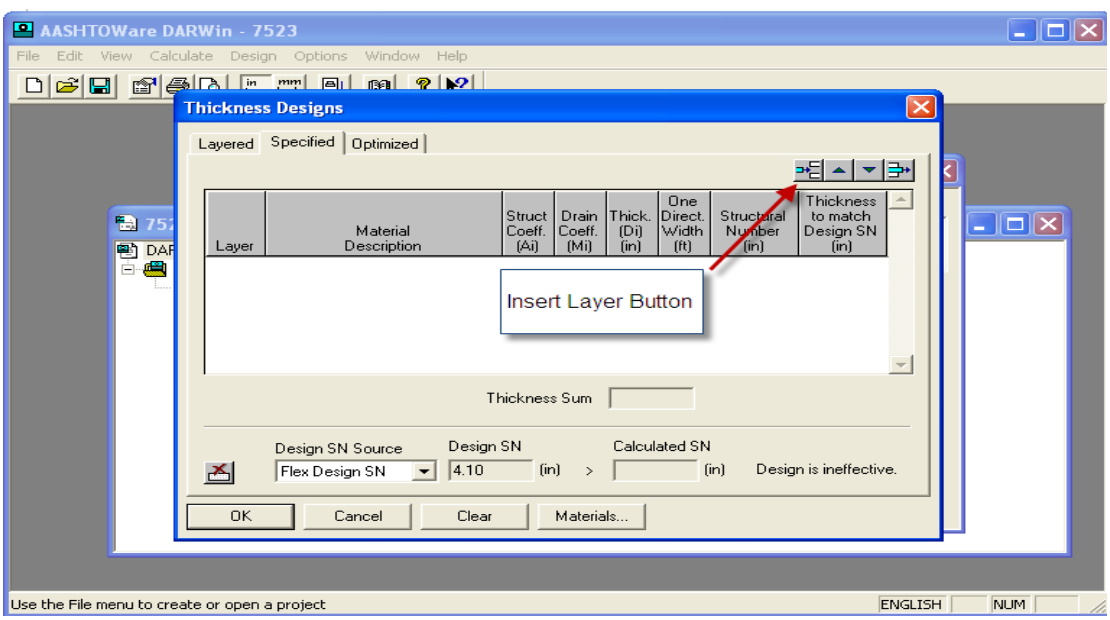


Figure 66 DARWin thickness design template with "Specified" tab selected.

Table 20 – Structural coefficients for pavement design.

MDT Structural Coefficients for Different Pavement Layers	
Layer	a_i
Plant Mix (PMBS)	0.41
CAC gravel	0.14
RAP/ Aggregate	0.12
CTB	0.20

Table 21 – PMS Thickness – Reconstruction, Major Rehab., and Widening

ESALs (Daily)	**PMS Thickness***
> 2000	0.70’
1000 - 2000	0.60’ – 0.70’
501 - 1000	0.50’ – 0.60’
201 - 500	0.40’ – 0.50’
101 - 200	0.30’ – 0.40’
0 - 100	0.30’*
Other Situations	**PMS Thickness**
Urban Curb and Gutter	0.40’ min
Mainline Interstate Pavements	0.50’ min
Rest Areas	0.40’ min

*In certain cases 0.25’ of PMS can be used for ½” or 3/8” mix, when budgets are constrained.

In step #1, it was hypothesized that milling and removing the stripped plant mix followed by pulverization would fully remove or treat the PMS stripping problem. Table 3 indicates at least 0.60’ (7.2”) of plant mix is required for this road’s traffic level. The routine 0.65’ (8”) pulverization depth will be used to mix the remaining PMS and the uppermost CAC together, leaving approximately 0.80’ (10”) existing base. Also recall from Step #1 that the existing shoulders should be utilized as much as possible to reduce project cost. Enter these thicknesses into DARWin as shown in the following Figure 67. This yields an SN of 5.11 which is significantly stronger than the design SN of 4.10.

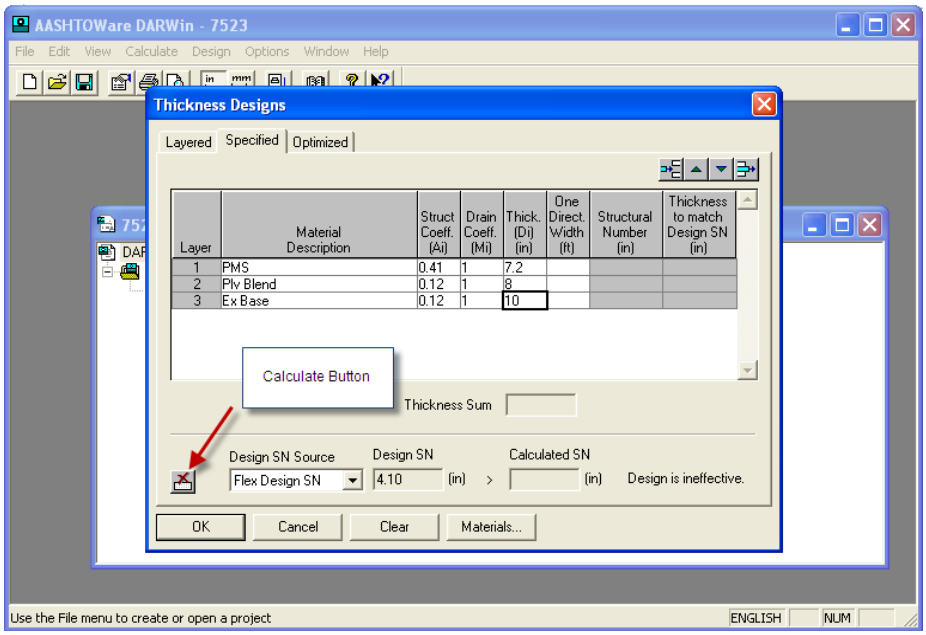


Figure 67 Partially completed Thickness Design template.

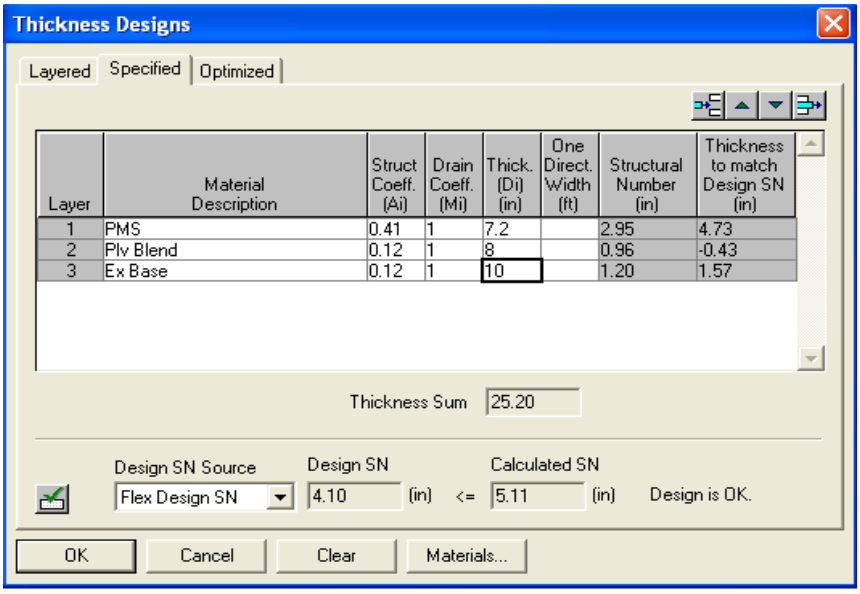


Figure 68 Completed DARWin thickness design.

Regarding the shoulder design, the shoulders will be milled and filled 0.30'. The reasons for this are that within the travel lanes, the top 0.30' will be virgin PMS, while the bottom 0.30' will be PMS w/ RAP. Milling and Filling the entire road width with 0.30' virgin PMS will provide constructability advantages, and will be explained further in Step #5.

Now that the pavement design is complete, click "OK", click on "File" in the upper left hand corner of the main pane, and click save. A "Save As" pane will come up as shown in the following Figure 69, Dialogue for Saving Your Work.

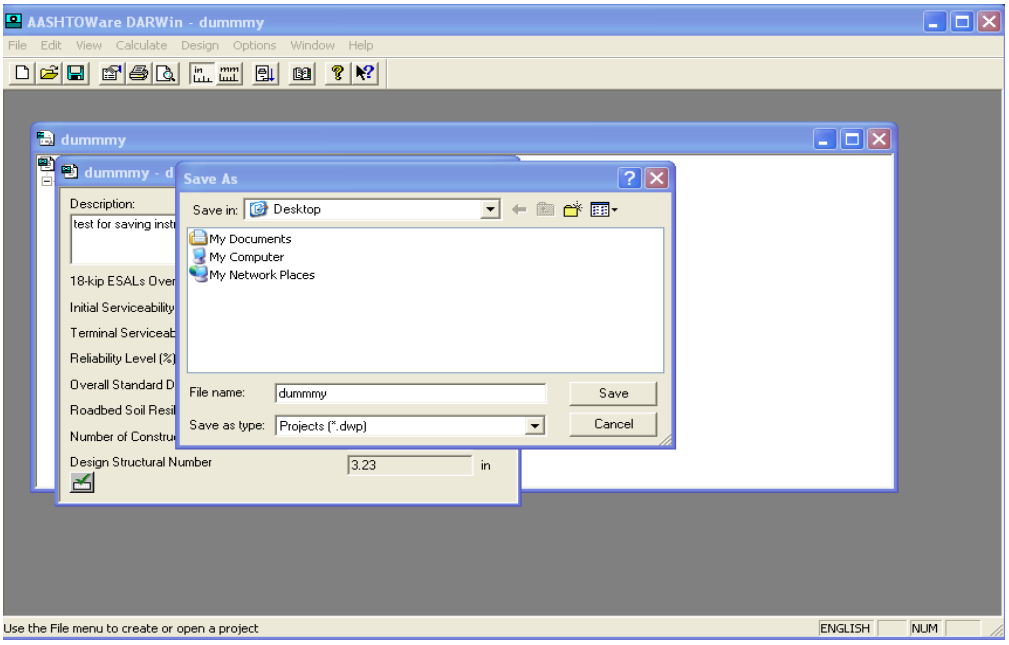


Figure 69 Dialogue for saving your work.

Click on “My Computer”, and the selection will change as shown in Figure 70below. Click on the “rdrtr on ‘MDT Astro (astro)’ drive and click on the “DARWIN” folder, which is the standard location to place DARWin design files. The standard file name for DARWin design files is the 7-digit Construction Number (CN), and DARWin design files are designated by the file extension .dwp. For the Alberton E – W project, the file name will be 7523000.dwp. Click the save button.

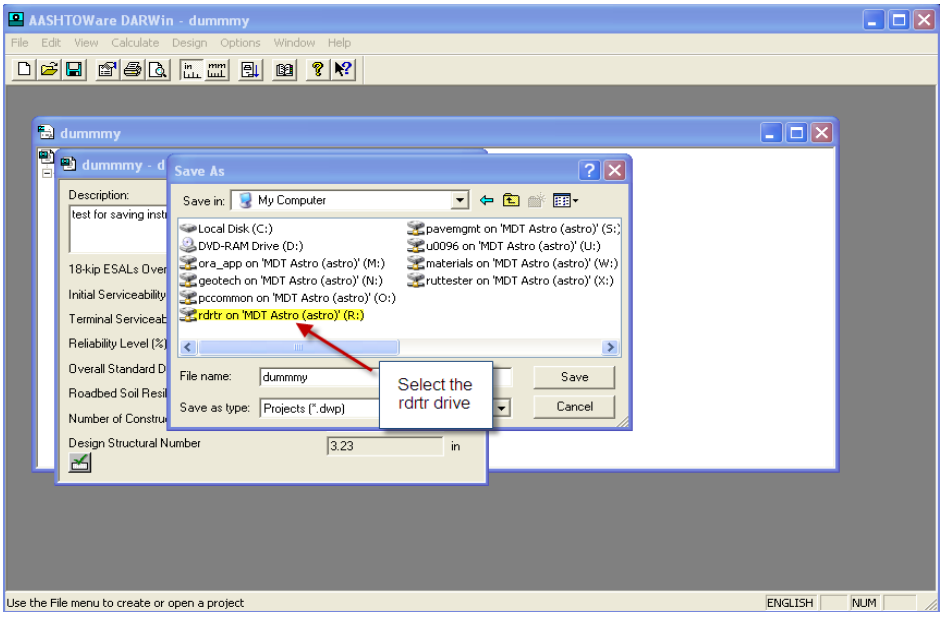


Figure 70 Highlighted location of the rdrtr drive that contains the DARWIN folder that contains surfacing design DARWin files.

Step #5: Specify Pavement Materials and Construction Methods

After the thickness design is completed, the next step is to specify the pavement materials to be within the pavement.

The PMS type needs to be chosen first since that will determine the need for a surface treatment. PMS type is chosen based upon PMS quantity and project type as shown in Table 5 and Table 6.

PMS Type Selection

First, the PMS tonnage is calculated. Recall that this design example is a small portion of a 9.0 mile project. Also, based upon discussion with the Road Design Project Manager, the finished roadway width will be 28'. Use the equation below to calculate PMS tonnage:

Project Length (miles) / project * 5280 ft. / mile * Roadtop Width (ft.)
* PMS thickness (ft.) * 1 yd^3/27 ft^3 * 3855 lbs. / yd^3 * 1 ton PMS / 2000 lbs.
= **tons PMS / project**

OR:

9.6 miles / project * 5280 ft. / mile * (40' pavement width) *(2 sides of Interstate * 0.60' * 1 yd^3/27 ft^3 * 3855 lbs. / yd^3 * 1 ton PMS / 2000 lbs.
= **173,690 tons PMS / project**

Based upon 173,690 tons PMS,

Table 4 specifies that either ½" or ¾" Grade S Volumetric should be specified.

To further refine the PMS type, use Table 6 for this reconstruction project with 0.30' PMS. The table specifies that ¾" Grade S or Commercial Plant should be used. Since both tables allow ¾" Grade S Volumetric, the designer should recommend using **¾" Grade S Volumetric** on the project.

Recall from Step #1 that the District would like to use RAP as much as possible. Chapter 6 indicates that RAP contents up to 15% and 30% in the top and bottom lifts are allowed. On this project it was decided to utilize Virgin PMS on the top 0.3' PMS (as shown above), and PMS containing 30% RAP on the bottom lifts. The PMS for the bottom lifts will be "Plant Mix Bit Surf Gr S – ¾" RAP", and 30% RAP content within the project Special Provisions.

PG Binder Selection

The criteria for picking PG Binder type is in Table 7. This project has more than 400 daily ESALs and is also an Interstate Roadway. For the top 0.30' of virgin PMS, PG 70-28 is selected, which is a polymer modified binder.

The bottom 0.30' will consist of PMS with RAP. Here is the guidance given in Chapter 6 regarding specifying PG binders in PMS with RAP:

Consideration should be given to using a lesser PG binder grade in lower lifts when 0.4' or more new PMS is required. As a general rule of thumb, binder grade can be dropped one grade within PMS located more than 0.2' below the pavement surface.

For PMS with more than 20% recycled asphalt pavement (RAP), reduce the PG binder grade by one binder grade to account for the oxidized binder within the RAP.

Since the PMS with RAP is more than 0.2' below the pavement surface and contains more than 20% RAP, the Binder type can be bumped down twice from what is shown in Table 7 (PG 70-28). That suggests that PG 58-28 should be specified. However, this project is the first project where Grade S PMS with RAP will be used, so the Pavement Design Engineer recommends that **PG 64-28 be used within the PMS with RAP** since it has polymers and will lower the potential risk.

Asphalt Content Selection

The selection of asphalt content is described in Chapter 6. Within Table 8, it is stated that project specific asphalt content should be specified for projects where ¾" Grade S Volumetric is specified. The following website is visited to view the statewide *As Constructed Percent Asphalt* map (Also shown in Figure 22). Based on the figure, 5.2**% asphalt content** is estimated.

Within the **PMS with RAP layer, utilize 4.2% asphalt content.** This is determined by interpolating between the asphalt contents for 25% and 40% RAP within Chapter 6, Table 8.

Surface Treatment Selection

This project has AADT = 7090 and has a 70 mph posted speed. Based upon Table 25, a Type I chip seal should be specified since this is a newly constructed road with no rutting, high traffic speed, and low traffic volume. However, chip seals tend to wear off within the wheel paths within this geographic area. Due to this, a **Type II chip seal** will be specified since it will provide better durability.

Step #6: Sending the Pavement Design Memo

Now that the thickness design and pavement materials have been determined, the last task is to send a paper copy of the surfacing design memo to road design staff via interdepartmental mail. The following pages show the surfacing design memo for this project. Note that the memo is not signed. In practice, both the Pavement Design Engineer and Designer sign the memo to the right of their printed names.

After completing the surfacing design memo with Microsoft Word, save the .doc file using the 7 digit Construction Number (CN) as the file name. Save the .doc file in the projects folder on the shared drive. Once the memo has been submitted for distribution and saved, the EPS activity may be carded.

Montana Department of Transportation
Helena, MT 59620-1001

Memorandum

To: Paul Ferry, P.E.
Highways Engineer

Thru: Dan Hill, P.E.
Pavement Engineer

From: Ed Shea
Surfacing Design

Date: July 31, 2012

Subject: IM 90-1(196)74
Alberton – E & W
Control Number 7523000

Hereby transmitted is the ***preliminary*** surfacing section for the surfacing structure on the subject project. This recommendation completes the Surfacing Design 600 activity.

Surfacing Section No. 1 – Travel Lanes – BOP to EOP
0.30' Plant Mix Bituminous Surfacing Grade S
0.30' Plant Mix Bituminous Surfacing Grade S with 30% RAP
0.60'
Design R-Value = 20

Mill and Remove 0.60' existing PMS from within travel lanes and extending 1' beyond outside shoulder stripe. After milling, pulverize milled surface 0.6' deep and recompact. Place 0.60 PMS as shown above.

Surfacing Section No. 2 - Shoulders – BOP to EOP
0.30' Plant Mix Bituminous Surfacing Grade S

Mill 0.30' prior to placing 0.30' PMS.

Place Type II chip seal upon finished PMS over entire road width.

The designs are based on 1104 daily ESALs from traffic dated Feb. 2012.The design life of the recommendations is 20 years considering AASHTO design procedures.

Within the top 0.30' PMS, utilize ¾" Grade S with 5.2% asphalt content utilizing. PG 70-28 binder. Within the lower 0.30' PMS, utilize Grade S PMS with 30% RAP with 4.2% asphalt content utilizing PG 64-28 binder.

Prior to construction, the District should review the soil survey for R-values. Areas of concern and possible borrow areas should have additional soil samples submitted for R-value testing.

If you have any questions concerning this recommendation, please contact me.

cc:	E. Toavs, P.E.	S. Stack, P.E.	Materials Bureau File
	D. Krings, P.E.	Surfacing File	Geotechnical File

Figure 71 Surfacing Design Memo

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Chapter 8 **Engineered Overlays**

An engineered overlay returns the pavement to a high serviceability level and provides the strength for the pavement design life. It is designed to mitigate existing pavement distress and material defects. For a Minor Rehabilitation project (Work Type 160), a minimum of 10 years for the design life is required. The need for milling is determined by the designer based upon the existing pavement condition. The overlay should do both of the following:

- Provide additional pavement structure needed for the pavement design life,
- Correct existing pavement distresses and/or pavement material problems so those issues do not affect performance of the engineered overlay.
- A grade raise can be accommodated (~0.2' to 0.3')

Engineered Overlays should also be considered on portions of Major Rehabilitation projects where the following conditions are met:

- The horizontal and vertical alignment is not going to change along most of the project length.
- A grade raise can be accommodated (~0.2' to 0.5')
- A pavement design life of 20 years can be achieved.
- The existing pavement, and in particular the PMS layer, is in good condition.
- The existing plant mix does not have excessive cracking.
- Overhead clearance at grade separations needs to be considered on thick overlay projects.

Engineered overlays can be a cost effective and relatively fast method to rehabilitate an existing pavement. They are also particularly useful for pavement widening projects since the existing pavement can be used to carry traffic while the widening section(s) are built.

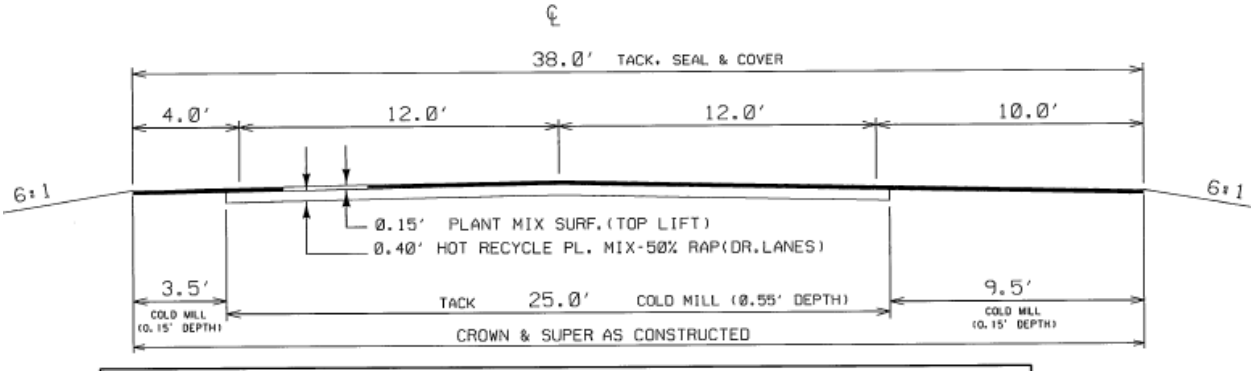

Figure 72: Engineered Overlay Design with Deep Pavement Milling

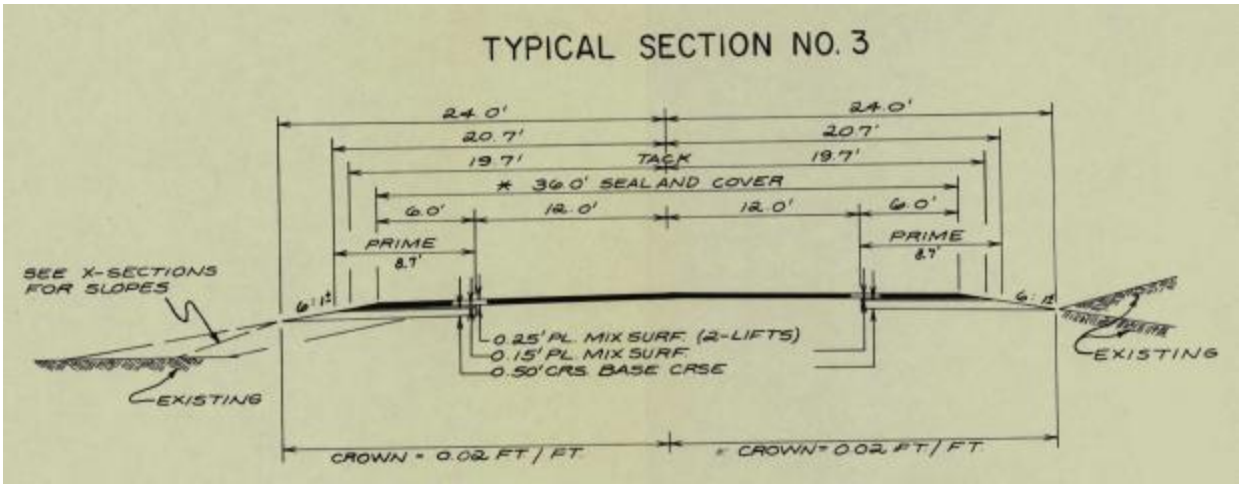

Figure 73: Engineered Overlay with Widening Section

8.1 ENGINEERED OVERLAYS ON HIGH VS. LOW VOLUME ROADS

The previous section states and Figure 75 below state that engineered overlays should not be done on pavement with structural distress. This holds true in most cases, but may not on low volume roads. On low volume roads, a thick, engineered overlay may be able to "bridge over" and protect the underlying, distressed pavement.

An example of this is overlaying stripped PMS on a low-volume road. Normally overlaying stripped PMS is not advised because it moves the critical tensile stress upwards as shown in Figure 74. Moving the critical strain higher in the pavement increases its magnitude, and may result in a tensile crack.

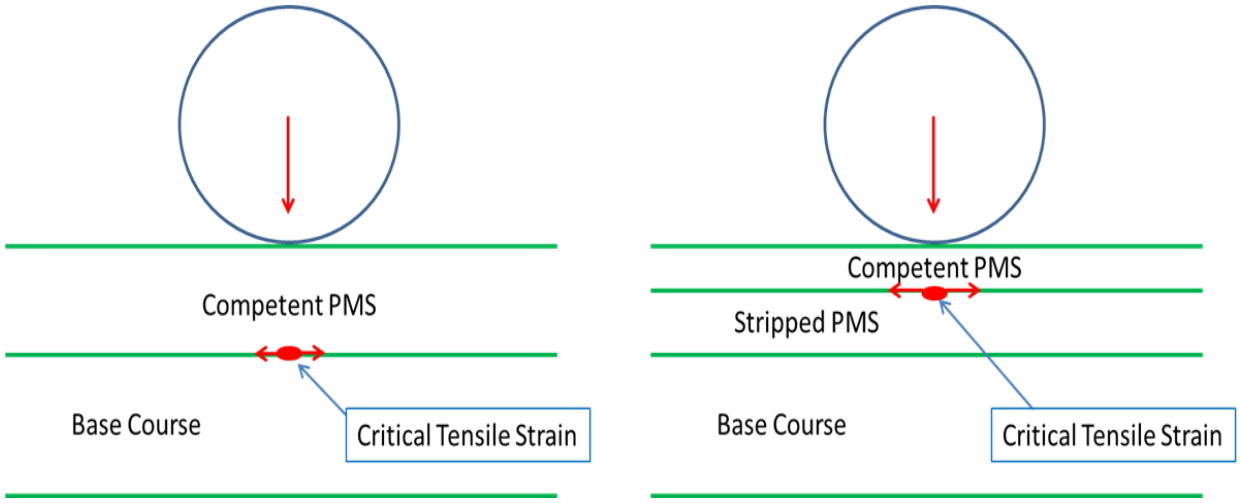

Figure 74: Stripped PMS Effect on Critical Tensile Strain

The phenomenon above results in alligator cracking on high volume roads where repeated loadings cause a tensile crack to initiate at the PMS bottom. However, on low volume roads

where the pavement is loaded relatively few times, a crack may not start. This lends itself to using thick overlays on low-volume roads to bridge structurally deficient pavement. Also, Grade S PMS is a high quality product with superior modulus, tensile strength, and durability; allowing it to bridge structurally deficient pavements more effectively.

8.2 ISOLATION AND LEVELING LIFTS

Isolation and Leveling Lifts can be utilized with engineered or routing PMS overlays to address ride, rutting, and/or crack sealant "bumping" issues.

8.2.1 Isolation Lifts

Crack sealant expands when overlaid with hot PMS. The expansion results in a noticeable bump on the overlay surface. An isolation lift can be used to mitigate this problem. An isolation lift is an additional 0.07' PMS lift placed before the overlay. It is placed only within the travel lanes. Another benefit of an isolation lift is it acts as a leveling course and results in a smoother post-construction ride.

On overlay projects greater than 0.15', it may be desirable to reduce the overlay thickness to 0.15' and place the remaining thickness as an isolation lift. For example, for a 0.20' overlay, the plans should specify a 0.15' overlay and a 0.07' isolation lift. This results in a total thickness of 0.22'. In this case, the isolation lift will provide extra structure and a better post-construction ride.

Note that on engineered overlays where multiple lifts will be placed isolation lifts are not necessary since the bottom lift will serve as an isolation lift. In addition, it should be noted isolation lifts are 0.07' which is thinner than the recommended lift thickness for ¾ inch plant mix (which is 2 to 3 times the nominal maximum aggregate size). Because of this, MDT requires a pneumatic rubber tire roller on the isolation lift to achieve compaction. Currently, there is a special provision that must be inserted in the plans package to outline the requirements and ensure adequate compaction is achieved.

8.2.2 PMS Leveling Quantity

Leveling is almost always necessary on single-lift overlay projects. Leveling serves two purposes. First, it corrects surface defects such as dips, rutting, and otherwise out-of-section pavements. Secondly, it provides a smoother surface to place the overlay upon, resulting in a better post-construction ride.

Review of prior projects has shown that projects with adequate leveling quantities result in smoother overlays. The determination of the proper quantity should be made based on the condition of the existing roadway, discussions with District design and construction personnel, and guidance given in Table 22.

The leveling quantities shown in Table 22 have been shown to result in post-construction rides that meet or exceed MDT's ride specification. If a decision is made to provide significantly less leveling than shown in Table 22, ensure the Ride Specification is excluded from that contract.

There are projects where leveling is needed to address a specific defect, such as a dip or frost heave. On these projects, the defect location and its leveling quantity should be included within the Plans. This ensures the Contractor places leveling in the correct location.

Table 22: Leveling Quantity Selection per 2 lane mile

Crack Sealant & Rut Index / Ride Index[2]	Crack Sealant Present[1]		No Crack Sealant Present[1]	
	Rut ≥ 55 (0.25")	**Rut < 55(0.25")**	**Rut ≥ 55(0.25")**	**Rut < 55(0.25")**
Ride ≥ 76[3]	Isolation Lift	Isolation Lift	50 tons/mile	125 tons/mile
70 ≤ Ride <76			50 -250 tons/mile[3]	125-325 tons/mile[3]
62 ≤ Ride < 70			250 -400 tons/mile[3]	325-475 tons/mile[3]
55 ≤ Ride < 62		Isolation Lift +75 tons/mile	Isolation Lift	Isolation Lift + 75 tons/mile
Ride < 55	Isolation Lift + 150 tons/mile	Isolation Lift + 225 tons/mile	Isolation Lift + 150 tons/mile	Isolation Lift + 225 tons/mile

[1] Include additional leveling to address any significant deformations on all roadways.
[2] Ride and Rut index is found in MDT's Annual Pavement Conditions and Treatments Report on the MDT Intranet, or contact the Pavement Management Unit.
Ride Index=100-(0.2667 X IRI)
3 use Interpolation based on the ride index to determine leveling quantity.

8.3 ENGINEERING OVERLAY DESIGN

MDT's pavement rehabilitation design is based upon the method presented in the 1993 AASHTO Guide Part III, with modifications presented in this Design Manual.

Engineered Overlays are designed using the following equation:

$$SN_{dgn} - SN_{eff} = SN_{ol}$$

Where:

SN_{dgn} = The structural number required for future traffic loading. Also known as SN_f (Structural number to carry future traffic) in the 1993 AASHTO Guide.

SN_{eff} = Existing pavement structural number.

SN_{ol} = The Structural Number deficiency between the existing pavement and that needed for the future pavement design. This is the amount of pavement structure that needs to be added with an overlay.

Generally, pavement rehabilitation design includes the following steps:

Step #1. Determine if Engineered Overlay is Feasible.
Step #2. Determine existing pavement structural capacity (SN_{eff}).
Step #3. Determine future pavement structural capacity (SN_{dgn})
Step #4. Determine additional pavement structure needed for design life (SN_{ol})
Step #5. Specify construction methods and materials
Step #6. Send pavement design and materials memo.

Step #1: Determine if Engineered Overlay is Feasible

Choosing an engineered overlay project is heavily based upon engineering judgment. Each pavement is unique, and there are usually a number of ways to rehabilitate a given pavement. The best method is one that meets the needs of the pavement designer, road designer, planning personnel, traveling public, safety, and the project budget. The designer should communicate with the road designer to ensure the chosen rehabilitation method will fit within the broader project constraints.

Visually evaluating the pavement condition is the single most important input to making this decision. By evaluating the pavement distress, as well as the distress mechanisms that cause the distress, the designer determines whether the pavement distresses are functional or structural:

> **Functional distresses** refer to those distresses that affect the traveling public, such as rough ride or low pavement friction. Those distresses, although a nuisance, can happen on a pavement that is structurally sufficient. Pavements with functional distress tend to need less expensive surface treatments to restore the pavement serviceability and extend its design life. These projects are often referred to as pavement preservation.
>
> **Structural distresses**, such as deep pavement rutting and alligator cracking, are an indication of inadequate pavement. Structural distress often needs deeper, more expensive rehabilitation in order to meet the desired design life.

Engineered overlays are the preferred method to rehabilitate pavements with functional distress while pulverization should be utilized to treat structural distresses. Figure 75 shows a flowchart to serve as a starting point when choosing a rehabilitation method. In addition to the issues defined in the flowchart, some other issues that influence this decision are:

- **Traffic Volume:** There are times when heavy traffic volumes will influence the rehabilitation treatment decision, where the design team may want a more conservative treatment than shown in the flowchart.

  Conversely, on low volume roads there may be a desire to be less conservative in the pavement design. This may mean an engineered solution that effectively “bridges” over a problem pavement rather than treats its structural distress directly.

- **Pavement Grade Raise:** Engineering Overlays require pavement grade raise, usually on the order of 0.2’ to 0.5’. This needs to be considered since it may require roadside

work and/or ROW take to accommodate the grade raise. Engineered overlays will also reduce the top width and should be discussed accordingly as sometimes this may not be a feasible option.

Other design issues that should be considered for engineered overlays:

- The existing pavement has to be characterized using GPR, FWD, soil survey, and PMS cores since these designs rely on the underlying pavement structure.
- Reflection cracking and rutting needs to be mitigated so it does not reoccur on the finished pavement surface. Deep or unstable rutting should be addressed by removing and replacing the unstable PMS layer. Cold in-place recycling should be considered to reduce or delay the onset of reflection cracking. CIR material has a high air void content (~11-14%). The high air void level may retard reflective crack propagation.
- Do not expose Base gravel.
- Ensure that milling is feasible – at least an inch of PMS should remain in place after milling to carry traffic during construction
- Ensure that milling isn't occurring in overly stripped PMS – milling into stripped PMS may result in a rough milled surface. Generally, milling should only be done in material with an average stripping test grade ≥ 1.2.
- Ensure that an overlay isn't being placed directly on overly stripped PMS – placing PMS overlays on stripped plant mix (stripping grade less than or equal to 1) is not recommended (Section 0). The underlying PMS may not have adequate strength to support the new overlay. *This may not hold true on very low volume roads where overlaying stripped plant mix may be possible due to low truck loading.*
- For overlay and widening projects, the widening sections CAC layer should extend to at least the depth of the existing roadway base course. This facilitates lateral pavement drainage and prevents "bathtub" pavement sections.

If milling and overlays are not recommended because of poor stripping analysis results, the project is likely a good candidate for pulverization.

Step #2: Determing Existing Pavement Structural Capacity (SN_{eff})

There are two different procedures for determining SN_{eff}; 1) Determine SN_{eff} from in-situ Destructive Testing, and 2) Determine SN_{eff} from Non-Destructive Testing Results. These procedures are discussed further in the following sections.

8.3.1 Determine SN_{eff} from in-situ Destructive Testing

In the past, MDT has usually determined SN_{eff} utilizing destructive methods to evaluate the existing pavement. Destructive methods refer to methods that damage the pavement such as soil survey boring and PMS coring. The destructive testing information will help characterize

each pavement layer. SN_{eff} is calculated by summarizing the SN of each existing pavement layer. This method is explained in Chapter 7.

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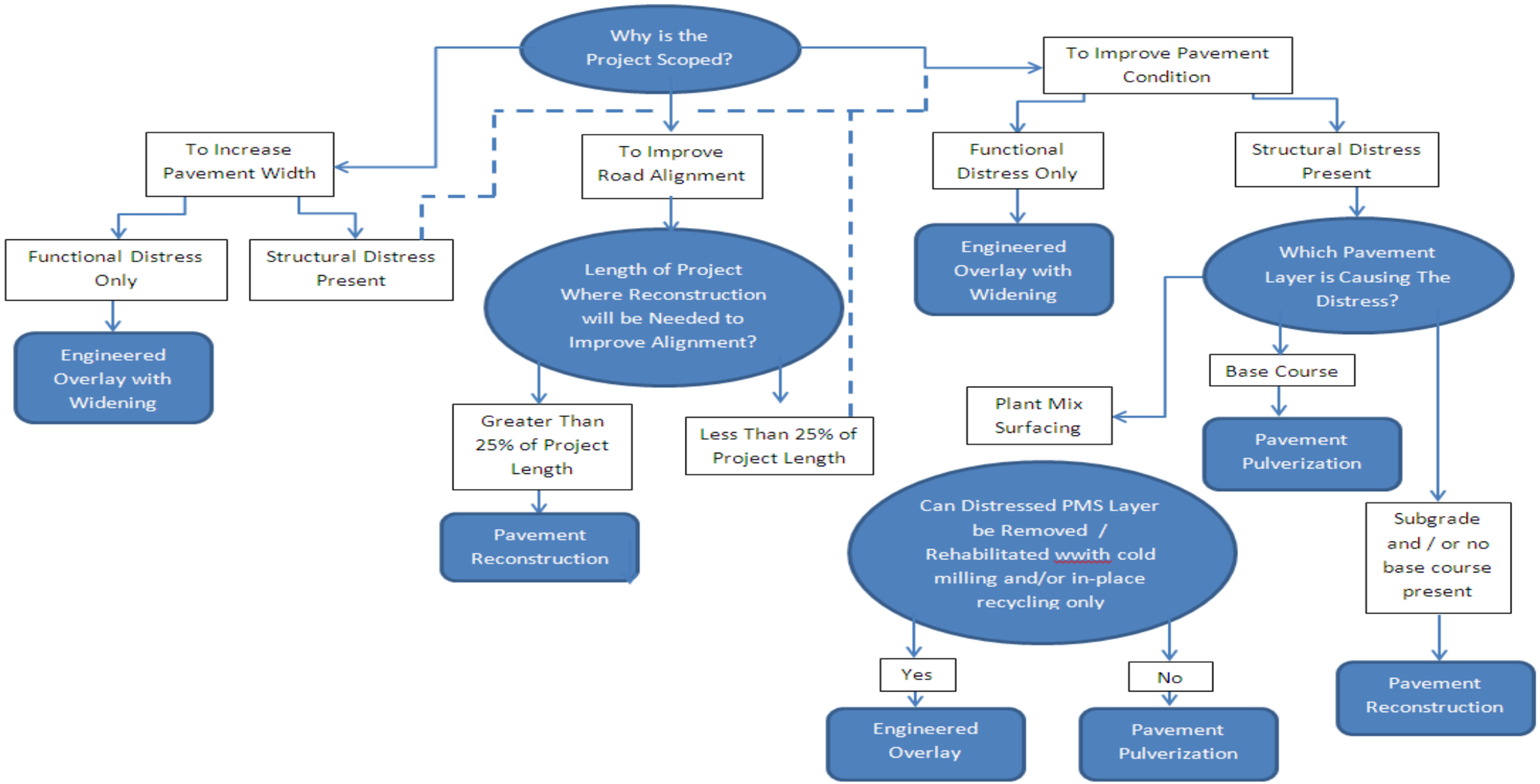

Figure 75: Major Rehabilitation and Engineered Overlay Selection Flowchart

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8.3.2 Determining SN_{eff} from NDT information

This method of determining SN_{eff} is based upon the information presented in 1993 AASHTO, Chapter 5.4 (pg. III-94 to III-102) (AASHTO, 1993), with adjustments based upon MDT experience.

In practice, the SN_{eff} should be calculated at each FWD testing location along the project length. This is most practically done using a spreadsheet discussed later in this section.

This method relies on NDT testing information only to determine SN_{eff}. NDT testing refers to Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR), which are discussed more thoroughly in Chapter 7 subsection *Existing Pavement Evaluation* within this Manual.

The first step to calculate SN_{eff} is to determine $(Ep/Mr)_{nomograph}$

E_p/M_r is determined by using the nomograph in 59. The nomograph requires the following variables:

$M_R * d_o / P$ and D

M_R = Uncorrected FWD back-calculated subgrade M_R. Uncorrected refers to not converting the back-calculated M_R to laboratory M_R (psi)

d_o = The FWD deflection directly under the load plate (mil)

P = The actual FWD load taken from FWD deflection data. Usually ranges from 8,500 to 9,500 lbs.

D = The total thickness of all pavement layers above the subgrade. This information should be determined as follows, in order of decreasing accuracy (in):

GPR PMS depth and soil survey base course depth

Soil survey / core PMS depth and base course depth

TIS Roadlog PMS and base course depth

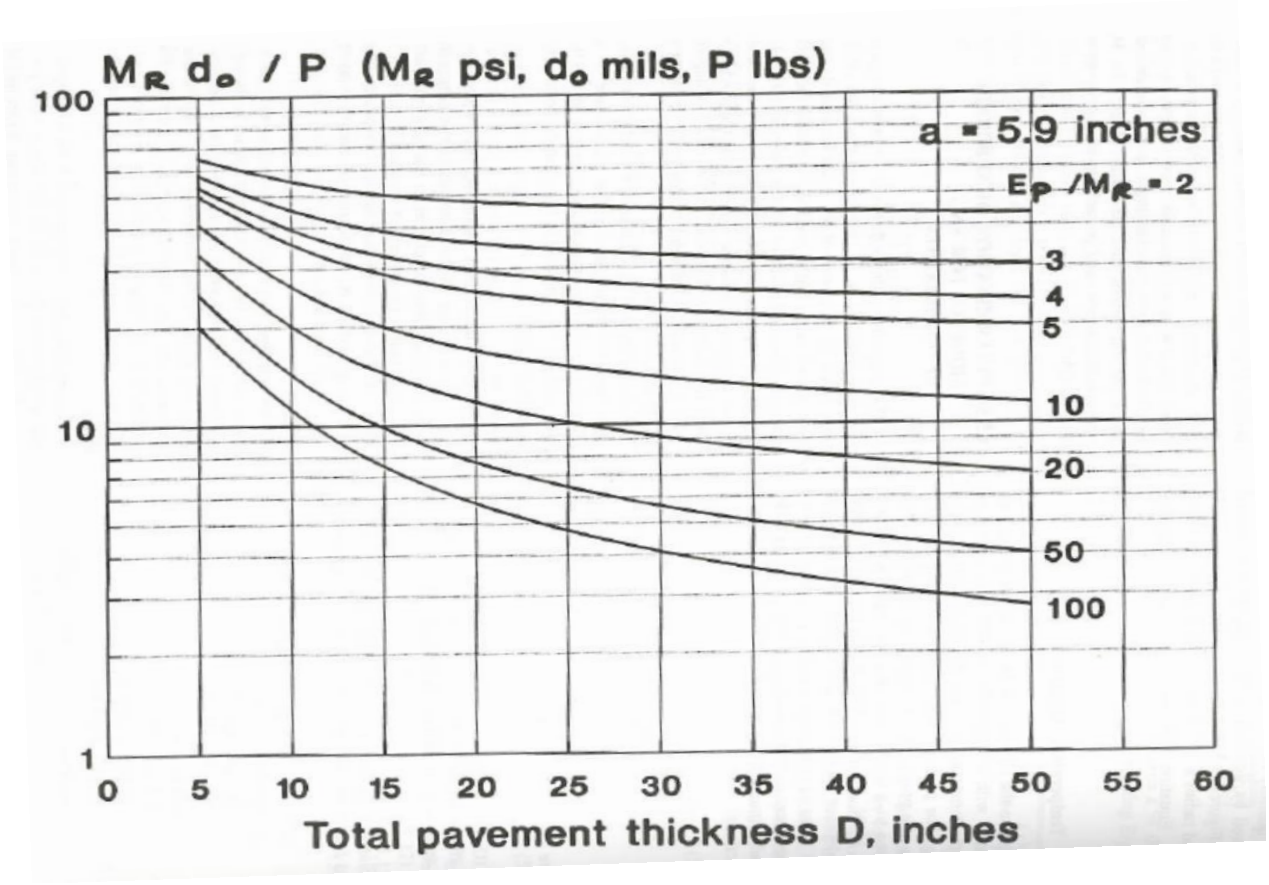

Figure 76: Ep / Mr Nomograph

The second step is to calculate $E_{p\ (\text{modulus of elasticity of the pavement})}$:

$E_p = (E_p/M_R)_{nomograph} * M_R * C$

E_p = Effective Modulus of the pavement layers above the subgrade (psi)

C = Conversion factor to convert FWD back-calculated subgrade modulus to laboratory subgrade modulus. C = 0.5 for use in this equation (dimensionless)

SN_{eff} is calculated as follows:

$SN_{eff} = 0.0045 * D * E_p$^(1/3)

Where:

SN_{eff} = The structural number of the existing pavement

8.4 STEP #3: DETERMINE FUTURE PAVEMENT STRUCTURAL CAPACITY (SN_{DGN})

Determine SN_{DGN} using the same procedure used for new or reconstructed pavements outlined in Chapter 6.

8.5 STEP #4: DETERMINE ADDITIONAL PAVEMENT STRUCTURE NEEDED (SN_{OL})

Determine the pavement structure that needs to be added to the existing pavement to perform well over its anticipated design life. This is done using the following equation:

$$SN_{dgn} - SN_{eff} = SN_{ol}$$

Where:

SN_{dgn} = The structural number required for future traffic loading determined during Step #3.

SN_{eff} = The structural number of the existing pavement determined during Step #2.

SN_{ol} = The Structural Number deficiency between the existing pavement and that needed for the future pavement design. This is the amount of pavement structure that needs to be added during pavement rehabilitation.

The additional pavement structure, SN_{ol}, is added by either adding additional pavement thickness, or improving the strength of the existing pavement layers. Additional pavement thickness is usually added with PMS overlay. Improving the existing pavement strength can be done by:

PMS layer:

- Mill and remove existing PMS and replace with new PMS
- Improve existing pavement strength using in-place recycling

8.5.1 <u>Completing the Widening Section Design</u>

The pavement widening section design is usually designed similarly to a new pavement. Follow the steps in Chapter 6 subsection *Flexible Pavement Design Method* to design the widening section.

Also consider the following when designing widening sections:

- Consider using the material from the rehabilitation section within the widening section. For example, pavement millings can be blended with virgin CAC to provide CAC for the widening section.
- The widening sections base course should extend to or below the depth of the existing gravel section. This facilitates drainage and avoids designing a “bathtub” pavement section.
- The designer should approach widening sections with caution. If the widening section will be a pavement shoulder with little or no traffic, consider using a thinner PMS layer upon the widening section. Consider the following guidelines:

- The minimum shoulder thickness should be 0.30' PMS in order to provide depth for future milling operations (shoulder less than 0.30' may be considered on very narrow widening sections or low-volume roads).
- The designer should not do this on Interstate Pavements, pavements where future traffic lanes may be placed on the shoulder, or shoulders that may become traveled upon if turning lanes are installed in the future.
- Consider constructability when designing widening section PMS thickness. Usually, the engineered overlay is placed over the entire road surface after the widening CAC is placed (Figure 73).

Where more than one PMS lift is placed on the widening section, the bottom lifts are placed flush with the existing surface.

8.5.2 Spreadsheet Solution for Calculating SNeff, SNdgn, and SNol

A spreadsheet has been developed to quickly calculate the structural numbers needed for engineered overlay design. Table 23 shows the spreadsheet with details on its use. The spreadsheet can be found in the Surfacing Design Share Drive.

Table 23: Spreadsheet for calculating SN_{eff}, SN_{des}, SN_{ol}, and Engineered Overlay Thickness

20-year design ESAL	313,900
Average 20-year Daily ESAL	43

																							50% Reliability			95% reliability			
ADAP_ROUTE	ADAP_DATE	ADAP_KM	ADAP_LOAD	ADAP_DEFL1	DEFL2	DEFL3	DEFL4	DEFL5	DEFL6	ADAP_DEFL7	ADAP_SURF_THICK	ADAP_SURF_MOD	ADAP_BASE_THICK	ADAP_BASE_MOD	ADAP_SBGR_MOD	Mr=(.24*P)/(dr*R)	ADAP Subgrade lab mod	Mr*do/P	Total pvmt thickness (in)	Ep/Mr	Ep (psi)	Sneff	Sndes	SN dif	Overlay thick (in)	Sndes	SN dif	Overlay thick (in)	RP
00028	ADAP_DATE	114.8	11.33	14.0	12.2	10.7	8.3	6.9	4.1	2.6	5.3	1,171,783	12	42,900	9,600	21,540	4,800	15	17	18.5	88,800	3.47	2.30	-1.17	-2.86	3.4	-0.07	-0.18	71.29
00028	9/21/2009 0:00	114.9	11.55	14.3	12.5	11.0	8.8	7.4	5.0	3.5	5.3	910,651	12	64,600	10,900	16,453	5,450	17	17	12.5	68,125	3.18	2.30	-0.88	-2.15	3.26	0.08	0.20	71.35
00028	9/21/2009 0:00	115	11.77	17.1	14.0	11.7	8.9	7.1	4.6	3.2	5.3	518,831	12	48,800	12,000	18,565	6,000	23	17	8	48,000	2.83	2.10	-0.73	-1.78	3.04	0.21	0.51	71.42
00028	9/21/2009 0:00	115.1	11.84	14.6	12.3	10.4	7.9	6.2	3.5	2.3	5.3	869,624	12	43,400	10,900	26,195	5,450	18	17	12.5	68,125	3.18	2.30	-0.88	-2.15	3.26	0.08	0.20	71.48

Blue Text refers to items input by the spreadsheet user, Black text refers to variables calculated by the spreadsheet. Note that ADAP inputs are put into the spreadsheet by cut and pasting NDT data from the project spreadsheets provided by the NDT unit.

20-year design ESAL: 20-year design ESAL provided by the Traffic Data Unit for the subject project

Average 20-year Daily ESAL: Calculated based upon 20-year design ESAL / (365 days/year * 20 years)

ADAP_ROUTE, ADAP_DATE, ADAP_KM – The Department Route, Date, and Location (Kilometer Post) where FWD test was done

ADAP_Load – FWD load magnitude (kips)

ADAP_DEFL1 = The deflection directly under the load plate (d_o)

DEFL2, DEFL3, DEFL4, DEFL5, DEFL6, DEFL7 – FWD deflections for information only

ADAP_SURF_THICK, ADAP_BASE_THICK – Usually GPR calculated PMS thickness and TIS Roadlog CAC Thickness

ADAP_SURF_MOD, ADAP_BASE_MOD, ADAP_SBGR_MOD- FWD back-calculated layer moduli

Mr = (.24*P)/(dr*R) = For information only – 1993 AASHTO equation 5.23. This is subgrade M_R calculated based on FWD parameters only. P = FWD load (lbs), dr = DEFL7 (in), R= distance from load to DEFL7 (48")

ADAP Subgrade lab mod = Uncorrected FWD back-calculated subgrade M_R * 0.5

Mr*do/P = Used in nomograh (Figure 76) to calculate $(E_p/M_R)_{Nomograph}$ Where Mr = ADAP_SBGR_MOD (psi), do = ADAP_DEFL1 (mils), and P = ADAP_Load (lbs)

Total pvmt thickness: ADAP_SURF_THICK + ADAP_BASE_THICK

Ep/Mr: $(E_p/M_R)_{Nomograph}$ calculated based on Figure 76 and Mr*do/) and Total pvmt thickness

Ep (psi): Ep/Mr * ADAP Subgrade lab mod

SNeff: =0.0045*(Total pvmt thickness)*Ep^(1/3)

50% Reliability and 90% Reliability SNdes, SN dif, Overlay thick (in): 50% and 90% reliability refers to using 50% and 90% reliability in the AASHTO pavement design equation, respectively. SNdes=SN_{des} calculated from 20-year design ESAL and ADAP Subgrade lab mod, SN dif = SN_{ol} = SN_{des} – SN_{eff}, Overlay thick = SN_{ol} / 0.41 where 0.41 is the structuracl coefficient for new PMS

RP: is the RP that the FWD test was done at. Converted by ADAP_KM * 0.621

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8.6 STEP #5: SPECIFY CONSTRUCTION METHODS AND MATERIALS

Specify Materials to be used within the designed pavement using the same procedure used for new or reconstructed pavements. This process is outlined in Chapter 6 *Step #4: Specifying Pavement Materials.*

8.7 STEP #6: SEND PAVEMENT DESIGN AND MATERIALS MEMO.

Send a surfacing design memo to the road design staff via interdepartmental mail. The following list includes items that are included on the surfacing design memo

- To: Highways Engineer
- Thru: Pavement Design Engineer (with signature)
- From: Designer (with signature)
- Date
- Subject Project Number, Name, and CN Number
- Designate Preliminary or Final and EPS Act. 600 or 604, respectively
- PMS Thicknesses for existing pavement sections
- PMS and CAC thickness for widening section
- A description of milling or in-place recycling depths
- Thorough description of the type and order of construction operations needed for engineered overlay
- Design R-Value
- The Daily ESALs used for Design
- Plant Mix Type
- Use of RAP and PMS RAP content
- Design Life Length (yrs.)
- Binder type
- Asphalt Content
- CC'd to District Administrator, Road Design Engineer, Road Design Project Manager, and Surfacing, Materials, and Geotechnical

An effort should be made to make the surfacing design memos as detailed as needed for the road designer to develop both project reports and construction plans.

After completing the surfacing design memo with Microsoft Word, save the .doc file using the 7 digit Construction Number (CN) as the file name. Save the .doc file in the rdrtr share drive as previously described in Chapter 6. Once the memo has been submitted for distribution and saved, the EPS activity may be carded off.

When the paper copy of the memo is distributed, the original copy is stamped “Master Copy” with a green stamp by the Department’s mail staff, and sent back to surfacing design. This “Master Copy” is stored in the surfacing design project file.

There will be instances when the design memo is sent electronically via email. When this is necessary, the Word file (.doc) should be converted to an Adobe Acrobat File (.pdf) before sending it. The purpose of this is that a .pdf file cannot be altered.

Chapter 9 **Gravel Pavement Design**

MDT typically uses Type B Grade 3 Crushed Top Surfacing for gravel roads. This material is described in Table 701-11 of the 2014 Standard Specifications. Use Figure 77 to determine gravel road thickness.

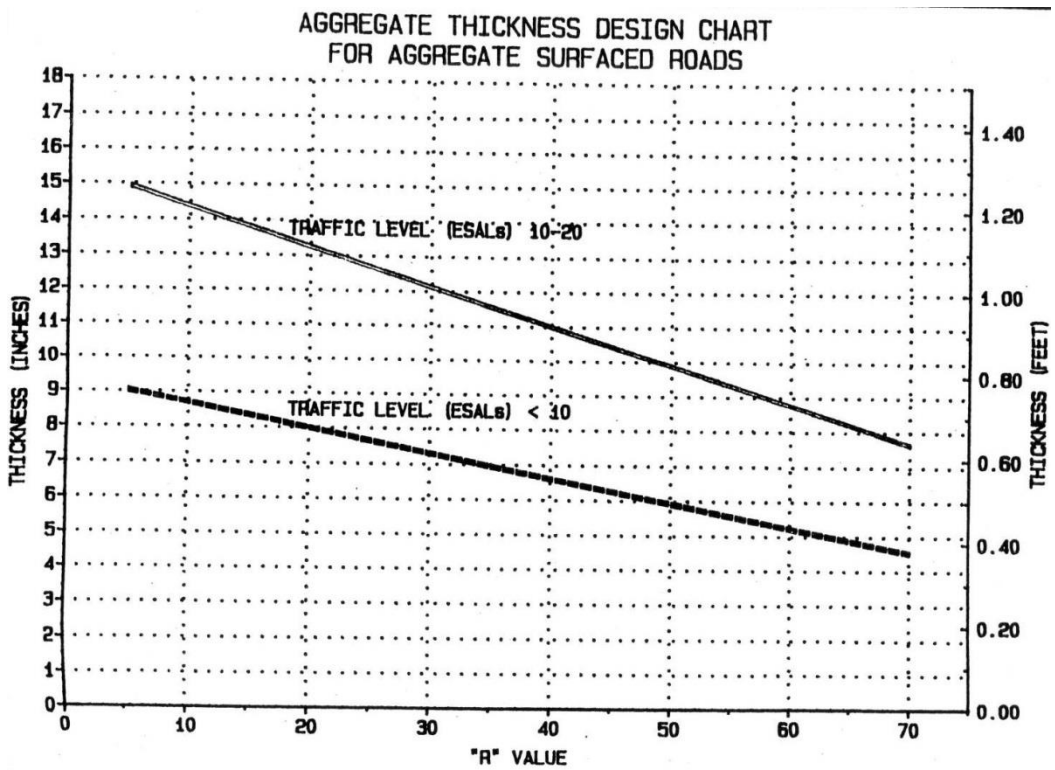

Figure 77: Gravel Road Thickness Design Graph

Detour Design

The following figure (77A) is taken from MDT's 2004 Road Design Manual and is intended as a guide for appropriate surfacing design for a detour.

Current ADT	Duration of Detour Operation			
	< 5 Days	5 - 30 Days	31 Days - 3 Months	> 3 Months
< 500	gravel	gravel	dust palliative	dust palliative
500 - 1499	gravel	dust palliative	dust palliative	PMS
1500 - 6000	dust palliative	dust palliative	PMS	PMS
> 6000	dust palliative	PMS	PMS	PMS

GUIDELINES FOR SELECTION OF DETOUR SURFACING

Figure 77A

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Chapter 10 **New and Reconstructed Rigid Pavement Design**

MDT encourages the use of PCCP where it is cost effective. Usually PCCP has a higher initial construction cost than asphalt pavements. However, PCCP life cycle cost is usually less due to its longer design life and decreased future maintenance. This is particularly true on roadways with relatively thick PMS sections. PCCP should also be considered on urban roadways and intersections where reoccurring asphalt treatments will be a nuisance to the traveling public.

10.1 CONCRETE PAVEMENT DESIGN DETAILS

MDT utilizes jointed plain concrete pavement (JPCP) exclusively. Typically, MDT utilizes a 2-layer concrete pavement consisting of doweled JPCP under laid with a crushed aggregate course subbase. The details for jointed plain concrete are now available in MDT's Detailed Drawings. Additional details and joint plans will be required in non-standard configurations such as roundabouts.

MDT has used continuously reinforced concrete pavement (CRCP) under railroad overpasses to minimize future maintenance near the railroad structure. In these cases, the highway was in a wet environment and consisted of a vertical sag curve under the railroad overpass. The following table shows MDT preferred design parameters. Other design parameters can be considered.

Table 24. Concrete Pavement Design Parameters

Design Parameter	Criteria
Design Life	30 years minimum, 40 years where appropriate
Pavement Design Method	1993 AASHTO, PCAPave, Darwin ME. Consider designing based on an average of these methods. The AASHTO method is often overly conservative.
PCCP Thickness	0.65' minimum
Subbase Thickness	0.50' minimum
Concrete Type*	Class AP or DP
Transverse joints	Non-skewed joints reinforced with corrosion resistant dowel bars. Joints consist of 1/8" saw cuts with crack sealant
Surface Texture	As per MDT Standard Specifications
Slab Length	12' minimum
Longitudinal joints	Reinforced with deformed rebar joints - 1/8" saw cuts with crack sealant or construction joint
Shoulders	Urban sections – Concrete Rural sections – PMS with 1' widened concrete slab along driving lane

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Chapter 11 **Rigid Pavement Rehabilitation**

11.1 MINOR REHABILITATION:

The first post construction treatment for concrete pavements is to perform a minor rehabilitation which typically consists of minor slab replacement, diamond grind and joint resealing. Depending on concrete thickness a concrete pavement may allow for multiple minor rehabilitations within its overall life span.

Another minor rehabilitation treatment for concrete pavements is to perform a diamond grind and then place a plant mix seal on the concrete. This has been done successfully by MDT in the Glendive District.

11.2 MAJOR REHABILITATION:

The next level of post construction treatment for concrete pavements is to perform a major rehabilitation which typically consists of slab replacement, dowel bar retrofit, diamond grind and joint resealing. Many of the concrete pavements in Montana were initially constructed in the 1960's using aggregate interlock between slabs and therefore do not have dowel bars or reinforcement. A successful rehabilitation strategy is to saw cut in dowel bars and grout them in place. Faulting is also corrected with the diamond grinding.

The final level of major rehabilitation treatment for concrete pavements is to perform a crack and seat and then apply a Hot Mix Asphalt (HMA) overlay. The PCCP pavement should be crack and seated prior to placing an HMA to minimize reflective cracking by reducing the size of the PCCP slab. When crack and seated properly, reflective cracking is essentially eliminated.

The PCCP should be cracked at 24 inch intervals to create a uniform pattern of cracking. Once the PCCP is cracked it should be seated with a rubber tired roller weighing at least 35 tons. The seating by the roller pushes down the PCCP pieces and provides an excellent base for the asphalt overlay to be placed directly on. The procedure for the crack and seat should be:

- Crack the PCCP slabs.
- Seat the cracked pieces.
- Remove and repair any soft spots.
- Sweep the project clean.
- Apply a tack coat.
- Place an asphalt leveling course.
- Place the asphalt overlay.

Crack and Seat is a treatment more suitable for the rural environment. When used on the east coast in the urban environment, the process of cracking the existing PCCP broke old water mains and cracked windows and walls in homes adjacent to the roadway.

11.3 OTHER REHABILITATION TREATMENTS:

White Topping can be used successfully on both existing plant mix and concrete pavements. When placed on concrete, this is referred to as a bonded or un-bonded overlay. In the past, MDT has successfully used this in urban areas with existing thick pavement sections. This was primarily done on high AADT routes with bad rutting. This would be considered a major rehabilitation treatment.

Chapter 12 **Pavement Preservation & Scheduled Treatments**

12.1 PAVEMENT PRESERVATION PROJECTS

The intent of pavement preservation projects is to extend the useful life of pavements based upon an observed pavement distress rather than on a scheduled basis. For more information on the pavement preservation process, view the agreement at this link.

The surfacing designer should prepare for the Preliminary Field Review prior to the review by reviewing the project's ride and rut data. In addition, the designer should have a knowledge of the existing typical section thickness and the time and type of the last treatment. The designer can review historical project files, as-built plans, and consult with the District Materials Labs to obtain existing mix and source information.

12.1.1 Surface Treatments

All PMS needs a surface treatment placed on it with the exception of 3/8" Grade S. The primary purpose of the surface treatment is to seal the pavement from water and oxidation. 3/8" Grade S does not need a surface treatment since it is nearly impermeable. MDT's most common surface treatment is seal and cover, also known as a chip seal or seal coat. Seal and cover is the standard surface treatment, but other treatments should be considered on roadways where chip seals are not performing well or are not desired by the traveling public. Table 25 gives guidance on surfacing treatment selection.

Table 25: PMS Surface Treatment Selection Guidelines

Surface Treatment	Treatment Thickness	Traffic Level	Stop-and-go traffic	High Traffic Speed (>55mph)	Heavy Snow-plowing	Other Considerations
Type I Chip Seal[3]	3/8" [1, 2]	<10,000 ADT	No	Yes	No	A chip seal should not be placed on rutted roadways with ruts greater than 0.25".
Type II Chip Seal[3]	3/8-1/2" [1, 2]	>10,000 ADT	Yes	Yes	Yes	Can be considered on roads with <10,000 ADT where previous chip seals have performed poorly(i.e. wear within wheel paths.) Should not be placed on rutted roadways with ruts greater than 0.25".
3/8" Grade S	0.10'	N/A	Yes	No[4]	Yes	Placement on existing Curb & Gutter sections requires taper milling and ADA upgrades. Consider where past chip seals have not performed well (i.e. wear within wheel paths). Can be used to correct rutted pavement
Plant Mix Seal	0.06'[2]	N/A	No	No[4]	No	Placement on existing Curb & Gutter sections does not require taper milling and ADA upgrades Use where pavement noise is an issue. Do not place on rutted pavement.
Microsurfacing	3/8" [1,2]	N/A	No	Yes	No	Not normally placed on new PMS. Can be used to correct rutting on pavements in good condition. Can be used to fill stable ruts up to ¾" deep Ride quality and index should be considered when selecting microsurfacing. For lower ride values (less than 75 IRI), microsurfacing may not improve the ride quality and may not be the best treatment if ride is the primary consideration. After the microsurfacing is placed, the PvMS Condition Report may still trigger a thin overlay as a recommended treatment.

[1] For information only, treatment thickness is not shown in Plans.

[2] Does provide structure to pavement, do not include as part of PMS structural layer.

[3] Type I and Type II chips are Grade 4A and Grade 2A materials, respectively, described in Table 701-12 of the 2006 Standard Specifications

[4] Consideration can be given to using 3/8" Grade S and Plant Mix Seals on high traffic speed roadways if project specific circumstances warrant their use.

12.1.2 Scheduled Rehabilitation Treatments

MDT utilizes pavement management data to select projects for pavement preservation, and identify proper scope for rehabilitations or reconstruction. If a particular road performs well, multiple chip seals may be placed on a segment without the need for more expensive rehabilitation measures.

The following table outlines the triggers for scheduled rehabilitation treatments obtained from pavement management data. This is an automated system which may not always account for all of the factors which should be considered when establishing a project scope.

Table 26: Rehabilitation Triggers

Asphalt Cement AC Treatments	Ride Index
Do nothing	>= 70
AC Crack Seal	
AC Crack Seal & Chip Seal	
AC Thin Overlay	57<=Index<70
AC Thin Overlay Engineered	ESAL >= 300
AC Minor Rehabilitation	30<=Index<57
AC Minor Rehabilitation Rut	
AC Major Rehabilitation	Index < 30
AC Reconstruction	

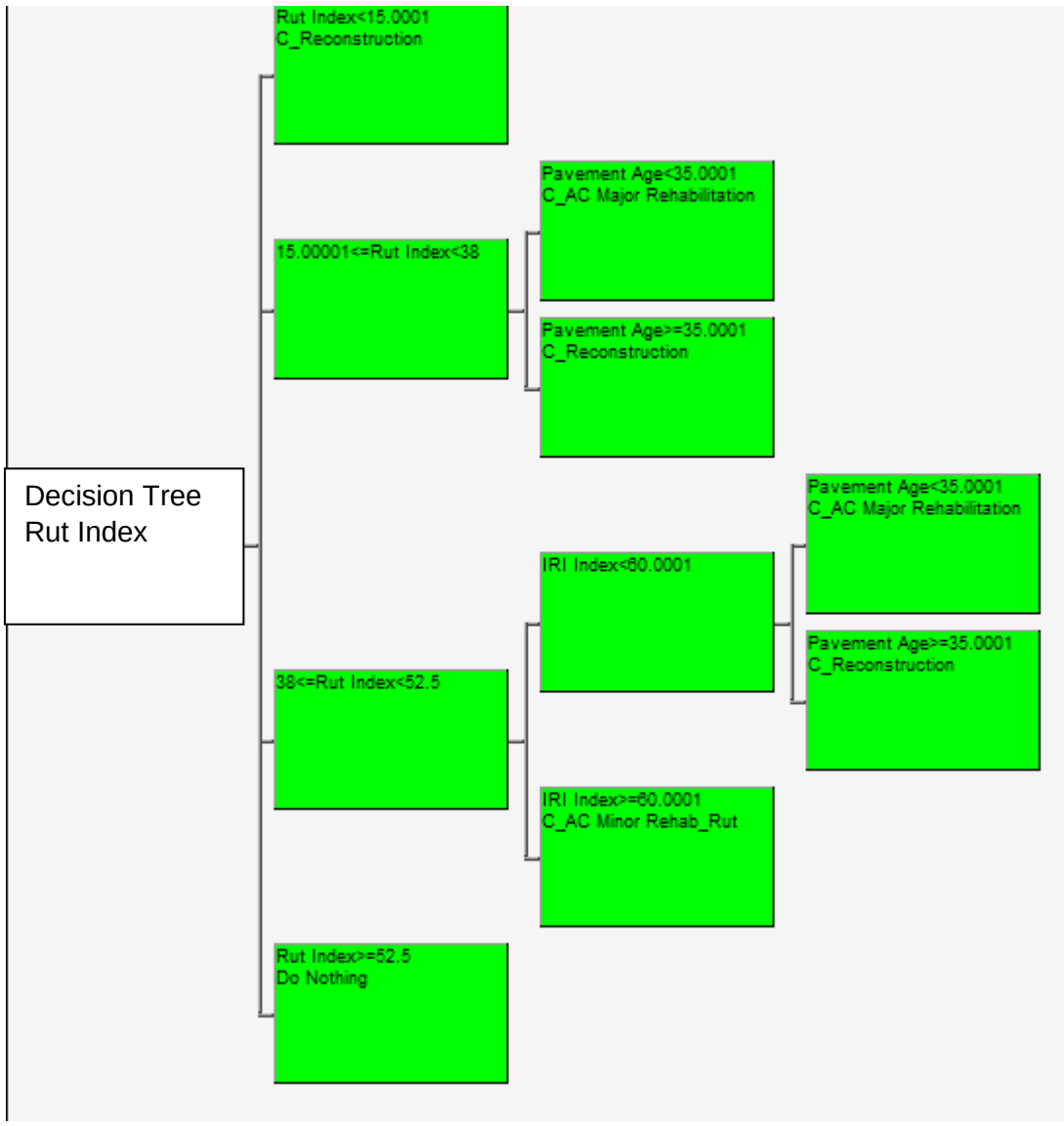

Figure 78 Decision Tree Rut Index

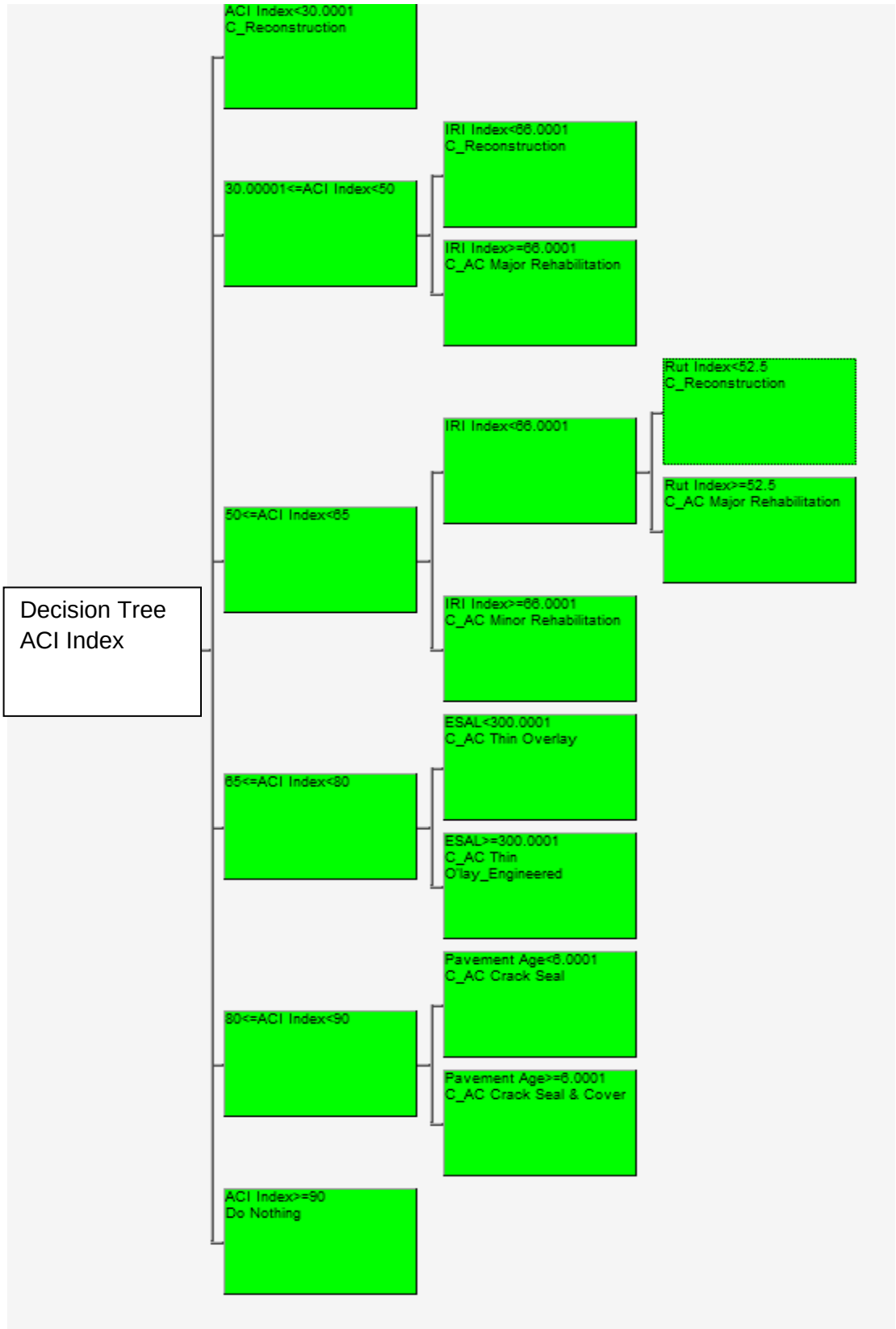

Figure 79 Decision Tree ACI Index

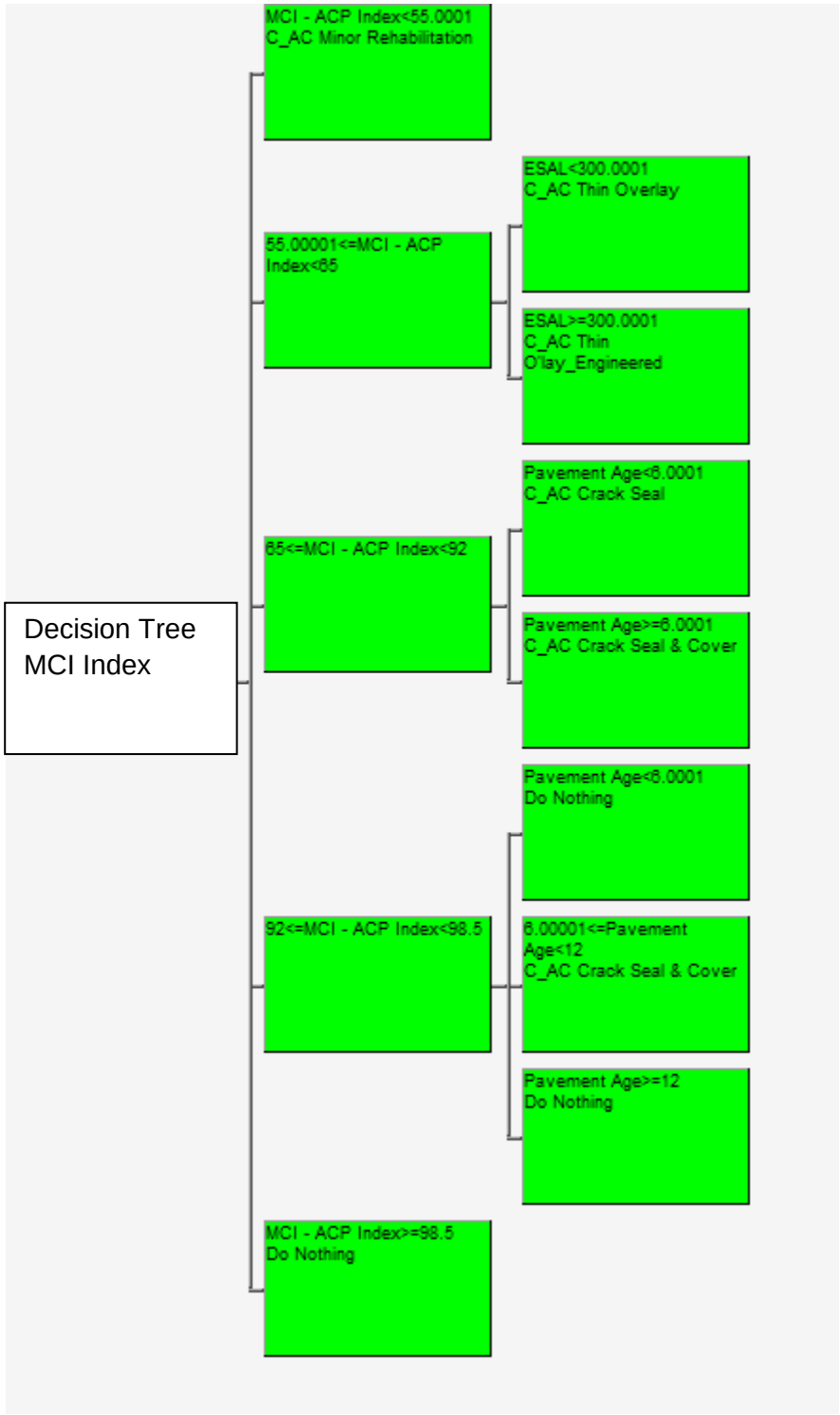

Figure 80 Decision Tree MCI Index

The following table relates the ride index with actual IRI measurements.

Table 27 Ride Index Definition

Ride Index	IRI (in/mile)
100	16
80	75
60	150
0	225

The following table relates the rut index with actual rut measurements.

Table 28 Rut Index Definition

Rut Index	Rut Depth (inches)
100	0.0
60	0.21
40	0.5
0	1.2

A link to the Report Development Section for MDT's Pavement Performance and Condition Report is here.

This section contains detailed definitions and information for the treatments listed above. The indices are broken down into 3 categories: Good, Fair and Poor. The scales for each index are also provided.

In the absence of Pavement Management data, the following may represent a possible maintenance cycle for a flexible pavement:

- A chip seal scheduled for year 7.
- An overlay or mill/fill scheduled for year 14.
- A chip seal scheduled for year 21.
- A minor rehab (overlay or mill/fill) scheduled for year 28.
- A chip seal scheduled for year 35.
- A major rehab at year 42.
- A chip seal scheduled for year 49.
- An overlay or mill/fill scheduled for year 56.

- Reconstruct at year 60.

In the absence of Pavement Management data, the following may represent a possible maintenance cycle for a rigid pavement:

1. A diamond grind, joint seal, and slab replacement (~2%) at year 20.

2. A diamond grind, joint seal, and slab replacement (~2-5%) at year 40.

3. Reconstruct at year 60.

This is a simplistic view of a pavement's life cycle and does not account for projects that may result from other needs such as capacity and safety improvements.

Chapter 13 **Bridge End Pavement Design**

The following guidelines are to assist MDT design staff in scoping and designing 30-year and reinforced bridge end pavements. This refers to the pavement section adjacent to bridge ends or other fixed objects that often deteriorates faster than the mainline pavement.

There are 2 types of bridge end pavement treatments; 30-year and reinforced bridge end pavements. The 30-year bridge end pavement is usually designed on reconstruction and major rehabilitation projects. The reinforced bridge end pavement refers to bridge end treatments done on minor rehabilitation and thin overlay projects.

The following outline shows how these guidelines are organized.

Section 13.1: Explains the bridge end pavement problem and why it occurs.

Section 13.2: Begins with a general overview of the bridge end pavement design process, as well as Table 29 that outlines the design processes for different types of construction projects.

Section 13.3 and 13.4: Include detailed design processes for 30-year and reinforced bridge end pavements, respectively.

13.1 THE BRIDGE END PAVEMENT PROBLEM

Often, the pavement located adjacent to bridge ends has more pavement distress than the mainline pavement. This is because the pavement within about 200' of bridge ends is usually thinner than the mainline pavement.

It is MDT's policy to use a 20-year design life when designing new pavements. The 20-year pavement design life refers to a pavement that lasts for 20 years from initial construction to the point in time where the pavement has deteriorated and has an unacceptably rough ride. The 20-year design life assumes that there is not preventative maintenance applied to the pavement during its design life.

In practice, it is the Department's policy to apply preventative maintenance treatments to pavements so the ride stays smooth. Often the preventative maintenance treatment is an asphalt overlay, which adds pavement thickness to the original pavement. Usually, as a result of asphalt overlays, pavements with 20-year pavement design lives last for 30-years or longer.

This generally presents a problem for the pavement located adjacent to a fixed structure such as a bridge end. In these locations, asphalt thickness cannot be increased since asphalt overlays are taper milled flush with the bridge end. Due to this, the pavement adjacent to the bridge does not receive additional pavement structure, and theoretically should last only 20-years. An example of this is shown in Figure 81.

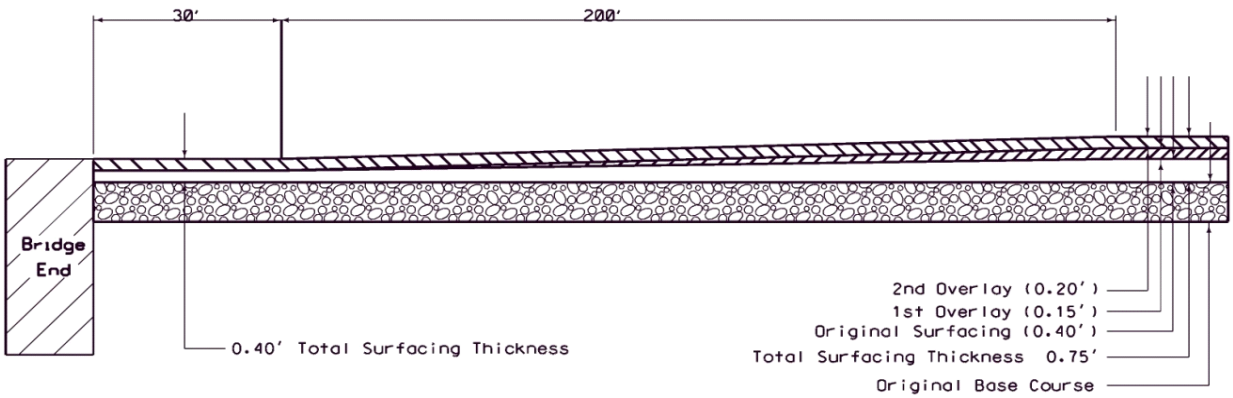

Figure 81: Example of Multiple Overlays Tapering into Bridge End

Since the bridge end pavement only lasts 20-years while the mainline pavement usually lasts 30 or more years, bridge end pavement deteriorates sooner than the mainline pavement.

Bridge end pavement is also subjected to increased truck loadings. Trucks tend to “bounce” when approaching and departing bridges. The bouncing applies additional dynamic forces upon the bridge end pavement as well as the bridge itself. These additional forces can double the amount of pavement damage caused by a normal truck.

This problem is most prevalent on the Interstate system. The majority of Montana’s Interstate pavements were paved with 0.35’ PMS (PMS) when originally constructed. Often these bridge ends exhibit pavement distress since 0.35’ PMS is too thin for today’s traffic loadings. Figure 82 and Figure 83 show examples of Interstate bridge end pavement distress.

The opposite is true on low to moderate traffic roadways which often perform fine, and there is no need to reinforce the bridge end pavement section. A few reasons why these bridge ends perform fine are:

- In the past, many of these pavements were overdesigned.
- Periodic taper mill/filling has added enough pavement structure to refresh the pavement surfaces.
- Many bridges on these roadways are paved over during asphalt overlay projects, so there is no need for taper milling at the bridge ends.
- Fewer truck loadings.

Figure 82: Interstate Bridge Approach Pavement Distress

Figure 83: Interstate Bridge Departure Pavement Distress

13.2 GENERAL BRIDGE END PAVEMENT DESIGN PROCESS

Bridge end pavements will often need a heavier, thicker pavement treatment than the mainline pavement. This will either restore the pavement structure on pavement maintenance projects, or provide a 30-year pavement life in the case of major rehabilitation and road / bridge reconstruction projects.

For design purposes, there are 2 types of bridge end pavement treatments that are utilized by MDT:

- For bridge and/or road reconstruction and road major rehabilitation projects, bridge end pavements should be designed for a 30-year pavement life utilizing a ***30-year Bridge End Pavement Design***. The purpose of this is to design the bridge end pavement section to last as long as the mainline pavement section.

- For existing pavements that are receiving a pavement maintenance treatment, the bridge end pavement should be treated with a ***Reinforced Bridge End Pavement Design*** when necessary. Reinforced Bridge End Pavement Designs refer to designing bridge end pavements so they perform similarly to the existing mainline pavement section.

The intent of the bridge end pavement treatment is to address pavement issues only. It does not address the common bump caused by bridge end pavement settlement. This refers to the bump located at the bridge / pavement interface, caused by the pavement settling lower than the top of the bridge deck. Usually, this settlement is rooted beneath the pavement section. It may be caused by inadequate embankment compaction, or further densification and consolidation of embankment and/or embankment foundation materials occurring after initial construction.

Often in the case of bridge end pavement settlement, settlement occurs shortly after construction and then stops sometime after construction. The settlement stops after the embankment materials have reached equilibrium and their maximum density. In cases where bridge end pavement settlement has either stopped or is not an issue, **care should be taken to not excavate more than necessary to provide for either the 30-year or Reinforced Bridge End Pavement Section**. Excavating deeper than needed may disturb otherwise stable soils, and may result in a new settlement problem when the virgin materials settle after construction.

Table 29: Bridge End Pavement Design Overview

Project Type	Bridge End Pavement Design Type (Design Life)	Recommended Treatment and How to Choose Bridge End Treatment Locations	Design Method and Reporting	Field Testing
Road and Bridge Reconstruction	30-year Bridge End Design (30 Years)	Construct 30 year Bridge End Design at all Bridge Ends	The surfacing design unit will provide Bridge End Pavement Designs. These designs will be provided as part of the preliminary and final surfacing design memos (Activity 600 and 604), or by email when needed.	Road Construction with Existing Bridge – During District Soil Survey add Soil Borings at Bridge Ends Road Construction with New Bridges – Use Soil Survey Borings for Pavement Design, no Additional Borings Needed at Bridge Ends
Major Rehabilitation	30-year Bridge End Design (30 Years)	Deep mill/fill exposing base course Construct 30-year Bridge End Pavements at all Bridge Ends	The surfacing design unit will provide Bridge End Pavement Designs with input from the Geotechnical Section as part of the preliminary and final surfacing design memos (Activity 600 and 604), or by email.	Pavement Major rehabilitation with Existing Bridge - During District Soil Survey add Soil Borings at Bridge Ends Pavement Major Rehabilitation with New Bridges – Use Soil Survey Borings for Pavement Design, no Additional Borings Needed at Bridge Ends
Minor Rehabilitation and Thin Overlay	Reinforced Bridge End Design	Deep mill/fill not exposing base course Bridge End Pavements that have more Load-Associated Pavement Distress than the Mainline Pavement.	If there are bridge ends that are severely distressed and need to be rebuilt or major rehabilitated, follow the guidelines for Major Rehabilitation projects (above). The locations will be determined and "field designed" during the Preliminary Field Review (PFR). The Design Project Manager (DPM) will make the formal request to the District to take bridge end pavement cores at those locations. After core testing is completed, the surfacing design unit will review test results and send final recommendations to the DPM for inclusion into the Scope of Work Report.	Deep mill/fill – Take pavement core at bridge end for thickness measurement and stripping analysis
Surface Seal (Chip Seal, Microsurfacing, etc.)	Do Not Treat Bridge End Pavements Differently than Mainline. However, there may be chip seal projects where bridge end pavements need to be treated with more than a chip seal. Those locations should be identified during the preliminary field review, and designed using the guidance for Major Rehabilitation (for severely distressed pavement) or Minor Rehabilitation/Thin Overlay located in above within this table.			

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13.3 30-YEAR BRIDGE END PAVEMENT DESIGN OVERVIEW

30-year bridge end designs should be included on bridge end pavements where the existing pavement is treated in a manner that provides a 20-year pavement design life. These projects include bridge / road reconstruction, major rehabilitation, and engineered overlay projects.

The purpose of designing bridge end pavements with 30-year design lives is so the resulting pavement performs similarly to the mainline pavement. This should result into more uniform pavement performance between the bridge end and mainline pavements.

30-year bridge end pavements should be designed at all bridge ends as well as other fixed objects that will limit future grade raises such as railroad tracks or cattle guards, but not where the fixed object is an adjacent asphalt pavement.

The surfacing design unit will be responsible for providing the 30-year end pavement section pavement thicknesses, treatment length, and other pavement related issues. Surfacing design will also work with the Geotechnical Section regarding use of Special Borrow and Bridge End Backfill. Surfacing design will verify the 30-year bridge end is incorporated into the plan set at the appropriate time.

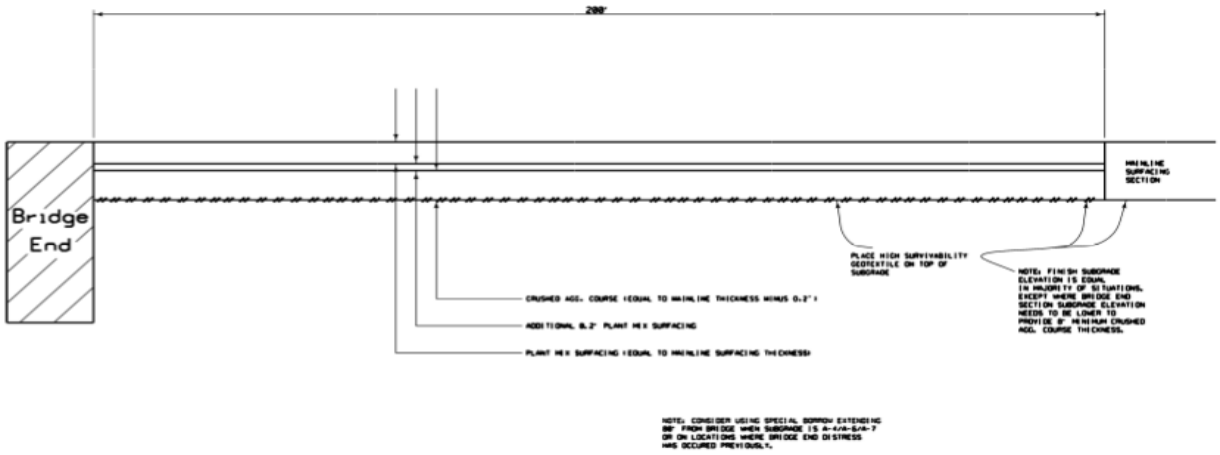
13.3.1 <u>New Bridge and/ or Road Reconstruction</u>

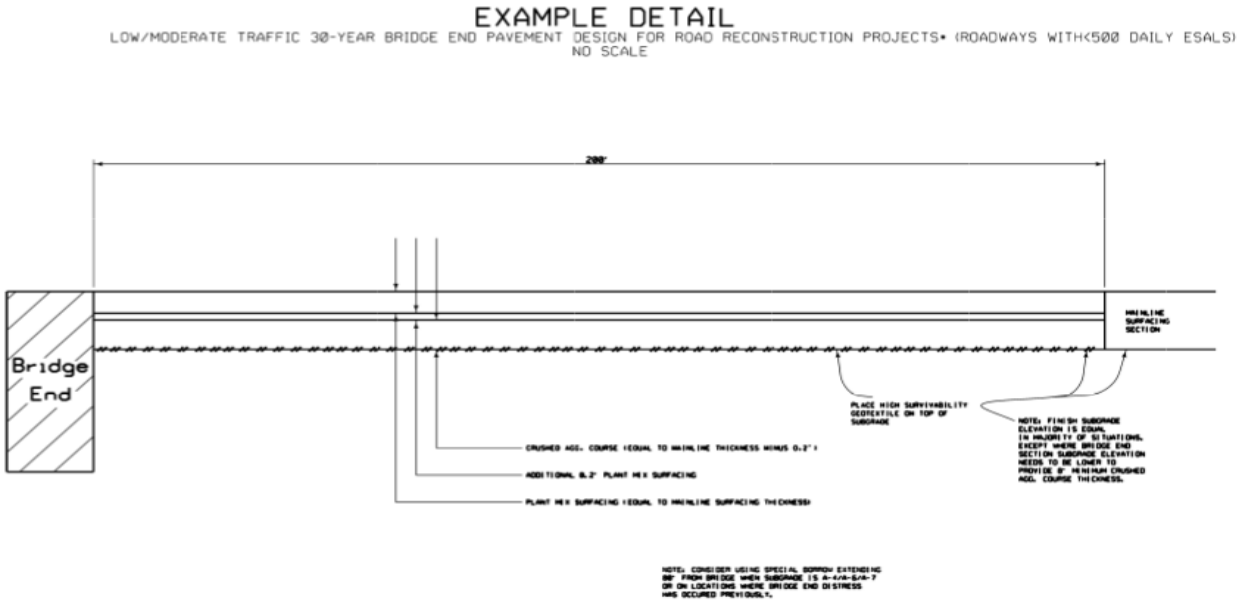
These projects provide the greatest opportunity to design a 30-year bridge end since the pavement will be built new, or removed and replaced within its entirety.

EXAMPLE DETAIL

LOW/MODERATE TRAFFIC 30-YEAR BRIDGE END PAVEMENT DESIGN FOR ROAD RECONSTRUCTION PROJECTS* (ROADWAYS WITH<500 DAILY ESALS)

NO SCALE





Generally,

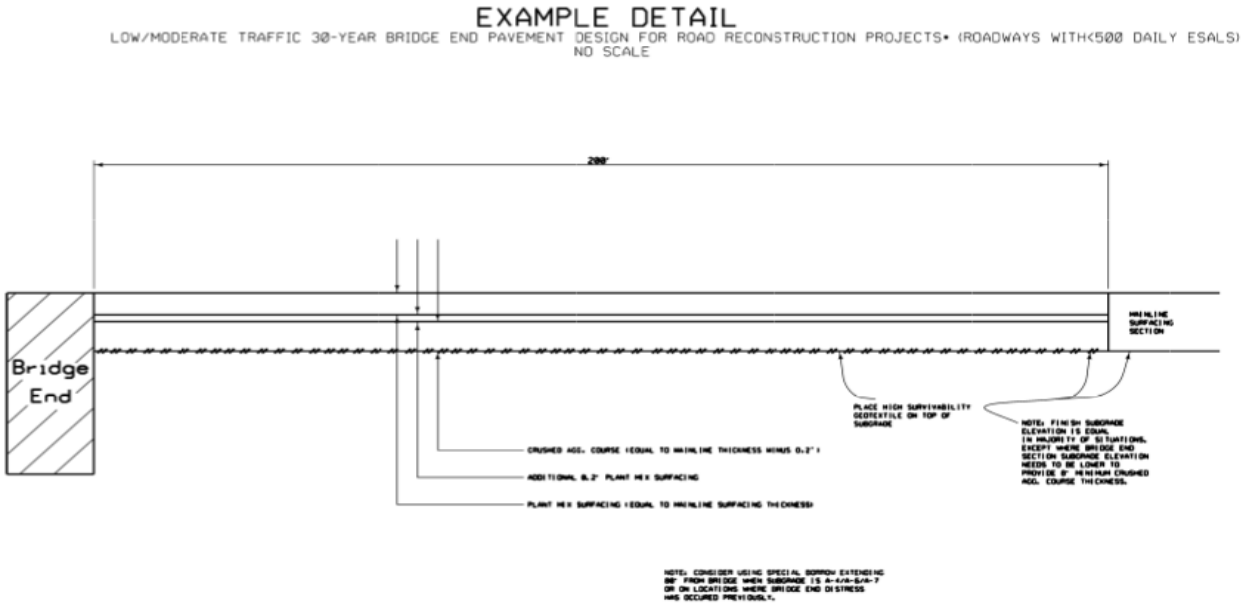


Figure 84 and Figure 85 should be used as a starting point for the 30-year pavement design. These figures are based upon the 20-year pavement design designed for the mainline pavement. Previous Analysis has shown that the additional plant mix thickness shown within the figures will provide a 30-year design life in most situations. Note that the subgrade elevation is the same beneath the bridge end and mainline pavement sections to enhance constructability.

For projects where both the bridges and adjacent pavement are to be reconstructed, a conventional soil survey should be adequate for bridge end pavement design and additional soil survey and geotechnical borings should not be needed at bridge ends. Surfacing Design should

use the same subgrade R-Value for designing both the bridge end and mainline pavement sections.

For projects where the pavement will be reconstructed next to an existing bridge, additional soil survey borings should be taken at the bridge ends. The surfacing design unit will communicate with the District Materials lab to let them know which bridge ends will need borings done. This communication will occur after the preliminary field review.

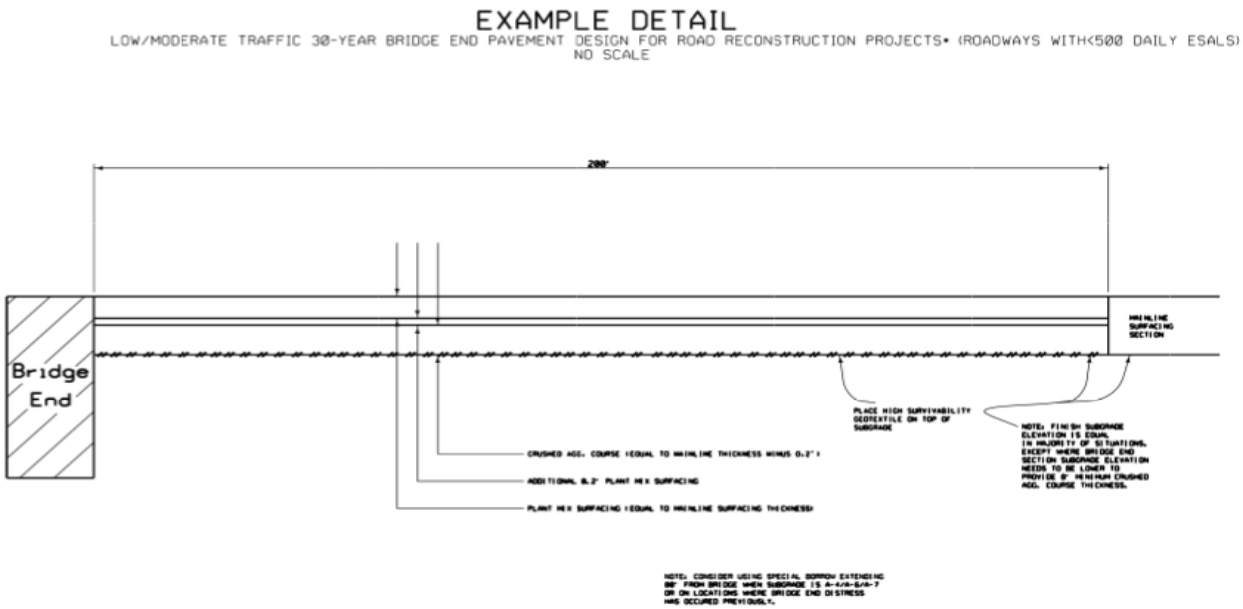


Figure 84: Example 30-year Bridge End Pavement Detail for Low to Moderate Truck Traffic

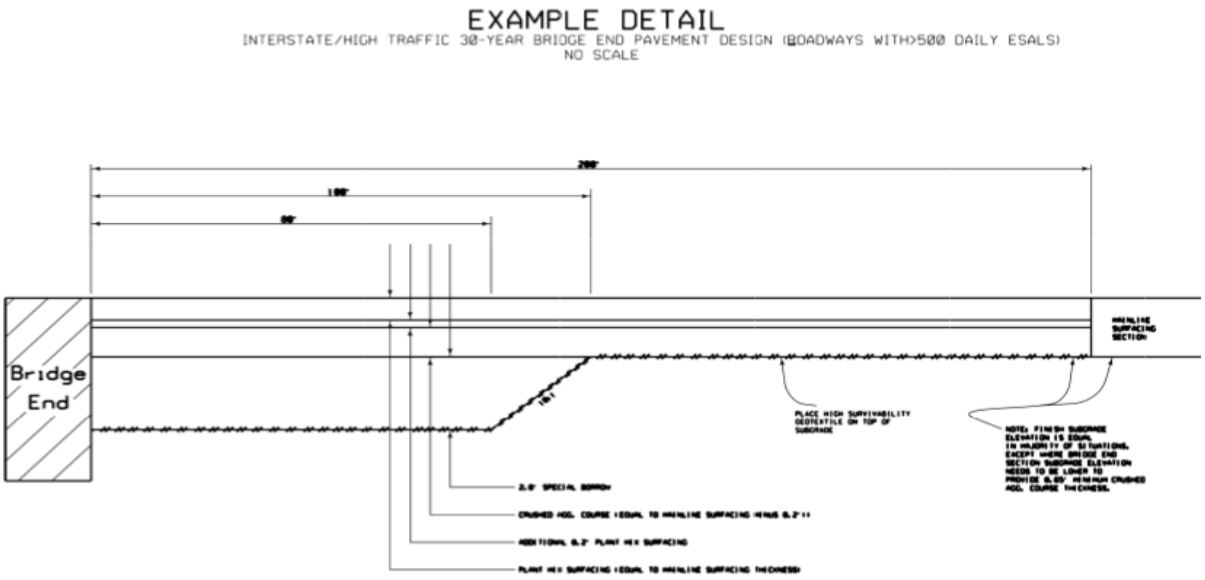


Figure 85: 30-year Bridge End Pavement Detail for Interstates and Roadways with High Truck Traffic

For projects where the pavement will be reconstructed next to an existing bridge and there is severe pavement distress and/or embankment settlement at the existing bridge end pavement sections, a more thorough geotechnical investigation should be considered. The intent of the investigation will be to identify embankment and foundation soil problems beneath the pavement section, and to design a deeper solution to address these problems. These locations will be identified at the preliminary field review by the Surfacing Design Unit and/or Geotechnical Section. Additional borings and evaluation will be done as determined necessary by the geotechnical section.

When necessary, the Geotechnical Section will provide both bridge end backfill and special borrow type, thickness, and extent. If pavement drainage problems are determined to be the cause of bridge end pavement distress in these locations, the Geotechnical Section will provide drainage details.

13.3.2 Pavement Major Rehabilitation

On major rehabilitation projects, often the existing PMS is removed in its entirety or pulverized into the existing base. The mainline pavement is normally designed for a 20-year design life, and the bridge end pavement should be designed with a 30-year design life.

Ideally, the 30-year bridge end should be designed by thickening the PMS section while utilizing the existing base course by leaving it in place. For example, if the mainline pavement is milled, pulverized, and overlaid with 0.4' PMS, the 30-year bridge end pavement may consist of deep milling, pulverizing and placing a 0.6' PMS overlay.

In locations where the existing bridges will remain in place during the pavement major rehabilitation, all bridge ends should be examined during the Preliminary Field Review. In locations where the bridge end pavement appears structurally sound, with no evidence of subgrade failure, the 30-year bridge end pavement should be designed using additional PMS thickness as described above. In locations with moderate to severe pavement distress, pavement subgrade problems, or indications of embankment problems, a deeper 30-year bridge end pavement should be designed.

All bridge end pavements should be bored during the District Material's lab soil survey to provide information for the bridge end pavement design. Bridge end borings should be characterized using the laboratory tests normally done during a soil survey.

13.4 REINFORCED BRIDGE END PAVEMENT DESIGN OVERVIEW

For pavement maintenance / preservation projects, the pavement treatment is not designed for a 20-year design life. Instead the pavement treatment is selected that will provide the most cost-effective design life for the particular pavement. Pavement maintenance projects refer to minor rehabilitation, thin asphalt overlay, chip seal, or other treatments that typically do not expose base course during pavement construction.

On these projects a reinforced bridge end pavement section should be utilized that will provide pavement performance that is similar to the mainline pavement. The resulting treatment will

reduce bridge end pavement maintenance, and will place both the bridge end and mainline pavement on a similar pavement maintenance cycle. This is explained more thoroughly in the following example.

13.4.1 Reinforced Bridge End Pavement Design Example

During a preliminary field review of a thin overlay project it is determined that the bridge end pavements have more fatigue cracking than the mainline pavement. It is probable that if the bridge ends aren't treated differently than the mainline pavement, the bridge ends pavements will degrade faster than the mainline pavement and will need reactive maintenance before the mainline pavement.

During their discussion, the PFR attendees agree that the 0.20' thin overlay project is expected to last 15 years before another pavement treatment is needed.

In this example, the bridge end pavement should be designed so it deteriorates at a similar rate as the mainline pavement which is a 15-year design life. Based upon the pavement condition and engineering judgment, the PFR attendees estimate that a mill/fill 0.40' deep within 200' of the bridge ends should result in the bridge end pavements performing similarly to the mainline pavement.

As a general rule of thumb, bridge end pavements that are in similar condition as the mainline pavement should not receive a reinforced bridge end pavement treatment. In other words, if there isn't anything wrong with the bridge end pavement it should be treated the same as the mainline pavement. However, there will be projects where no bridge end distress is present but an increase in truck traffic is predicted. In these locations, the District may decide to preemptively reinforce the bridge end pavements before in anticipation of the future traffic increase. A few examples are roadways within developing oil fields, or in the vicinity of future gravel pits, mines, or agricultural infrastructure.

Generally, pavement maintenance and preservation projects are usually fast track designs that may not provide adequate time for thorough geotechnical and pavement investigations for designing reinforced bridge end pavements. Project letting dates on these projects should not be pushed out into the future as a result of designing reinforced bridge end sections. **Therefore, in the interest of time, non-engineered designs based upon engineering judgment should be utilized whenever possible.**

Locations and types of reinforced bridge end treatments should be determined during the PFR. During the PFR, the attendees should examine all bridge ends, identify locations where reinforced bridge end treatments are needed. The surfacing representative should field design a *preliminary* reinforced bridge end treatment for inclusion into the PFR report. This will require a surfacing design representative to be present at these PFRs. A geotechnical representative will normally not need to be present, but may be called upon in the event that bridge end pavements are showing distress due to poor subgrade support.

13.4.2 Reinforced Bridge End Pavement Designs – Chip Seal Projects

The intent of a chip seal project is to seal the existing pavement. Often there is not equipment available on these projects to do anything beyond a chip seal on the bridge end pavement. Therefore, the design team should not provide a reinforced bridge end pavement design unless significant pavement distress is observed during the project nomination and preliminary field reviews. However, that should be the exception to the rule.

If bridge end pavement distress is observed and/or is a reoccurring pavement maintenance issue, the design team should coordinate with the District to decide whether reinforced bridge end pavements should be built. If it is decided to reinforce the bridge end pavements, care should be taken to provide a treatment that does not expose base gravel. Also, recognize that adding a reinforced bridge end pavement will significantly increase the overall project cost.

A starting point for reinforced bridge end pavement design on these projects is a mill/fill extending 200' from the bridge end within the travel lanes. Both the mill/fill length and thickness should be designed during the PFR based upon field observation.

Reinforced Bridge End Pavement Designs that expose base course should not be done on chip seal projects unless the District makes a special request.

The geotechnical section will usually not be involved in the bridge end pavement design on these projects. Construction input should be minimal since these treatments usually don't inconvenience the travelling public very much, and constructability issues should be infrequent due to the simplicity of these treatments.

13.4.2 Reinforced Bridge End Pavement Designs – Thin Overlay and Minor Rehabilitation Projects

Reinforced bridge end pavement should be considered on all bridge ends and fixed objects where the bridge end pavement shows load-associated pavement distress or has reoccurring maintenance issues. It is not necessary to provide reinforced bridge end pavements in locations where the fixed object is an asphalt pavement, or in locations where the bridge end pavement is in good structural condition and/or in similar condition as the mainline pavement.

The design life of the reinforced bridge end pavement should match the assumed design life of the overlay. This means that if an overlay is anticipated to last 12 years before the next pavement treatment (excluding surface seals), then the bridge end pavement should be designed to last the same amount of time without maintenance patching and/or alligator crack sealing. Determining design life is subjective, and should be done during the PFR using engineering judgment based upon past performance of the mainline pavement.

The decision point on whether or not reinforced bridge end pavement designs will be built is during the PFR. A surfacing design representative must be present to participate in the review. At the PFR, each bridge end should be evaluated by the design team. If possible, the bridge end pavements should be preliminarily designed at the PFR. Pavement cores should be collected and tested for stripping and thickness to design the mill/fill depth. Care should be

taken to provide a treatment that does not expose base gravel. However, Reinforced Bridge End Pavement Designs that expose base course can be done on a thin overlay if needed to address significant pavement deterioration and/or attain the desired design life.

The standard reinforced bridge end pavement design on these projects should be a deep mill/fill extending 200' from the bridge end within the travel lanes. Both the mill/fill length and thickness should be designed based upon field observation. A "deep" mill/fill is a mill/fill that is thicker than the asphalt overlay. For example, on a 0.20' asphalt overlay project the reinforced pavement bridge end section may consist of increasing the mill depth to 0.4' depth extending 200' from the bridge end.

The geotechnical section will usually not be involved in the bridge end design on these projects. Construction input should be minimal since these treatments usually don't inconvenience the travelling public very much, and constructability issues should be infrequent due to the simplicity of these treatments.

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Chapter 14 **Pavement Economic Analysis**

14.1 INTRODUCTION

A Life Cycle Cost Analysis (LCCA) consists of calculating initial construction costs, future maintenance costs over the LCCA analysis period, and remaining life or salvage value at the end of the analysis period. Based on these calculations, the relative value of each option is determined based upon their net present value. MDT does not currently factor user costs in the analysis.

When deemed necessary to perform an LCCA, a copy should be distributed with the surfacing design memo created under activity 600 (Prepare Preliminary Surfacing Typical Section).

LCCAs are typically performed comparing concrete to plant mix. Currently MDT uses a design life of 30 years for concrete and 20 years for plant mix. A 30 year design life for plant mix should be considered with an adjusted maintenance cycle. A 40 year design life for concrete should be considered in the instances where the minimum typical section will provide that life. The design life is the length of time for which a pavement structure is being designed based on structural distresses and traffic loadings.

MDT currently uses the Net Present Value (NPV) method for LCCA.

14.2 PROJECT SELECTION

A life cycle cost analysis (LCCA) comparing different pavement alternatives should be performed any time there might be economic savings for a project when considering long term construction cycles. In practice it is typically only necessary to do a LCCA on a project that meets all of the following characteristics (or by request from the District or Road Design) :

- Project Scope of Major Rehabilitation or Reconstruction
- High Daily ESALs (plant mix thickness >= 0.40’)
- Substantial Project Length
- Width >= 40 feet

14.3 LCCA TERMINOLOGY

Analysis Period is the service life of the pavement considered in the economic analysis. An analysis period of 40 years or more should be used.

Initial Design Period is the service life of the reconstructed pavement that does not include subsequent pavement preservation or rehabilitation.

The outputs of this analysis include:

Initial Pavement Construction Cost: The cost of initial construction of the pavement. This cost includes pavement related bid items, mobilization, traffic control, and construction engineering. Similar items required in all design options (i.e. striping) are not considered since the cost differences are insignificant.

Maintenance Cycle and Costs: This includes both the predicted year and cost of future maintenance activities. These costs may be calculated using the cost/yd2 prices within the PvMS Annual Report. The PvMS costs include both mobilization and traffic control. A 10% cost factor should be added to account for Construction Engineering (CE).

The PMS options may include:

- A chip seal scheduled for year 7.
- An overlay or mill/fill scheduled for year 14.
- A chip seal scheduled for year 21.
- A minor rehab (overlay or mill/fill) scheduled for year 28.
- A chip seal scheduled for year 35.
- A major rehab at year 42.
- A chip seal scheduled for year 49.
- An overlay or mill/fill scheduled for year 56.
- Salvage value at year 60 for the remaining service life.

The Concrete options may include:

1. A diamond grind, joint seal, and slab replacement (~2%) at year 20.
2. A diamond grind, joint seal, and slab replacement (~2-5%) at year 40.
3. Salvage value at year 60 for the remaining service life.

Net Present Value: The Net Present Value (NPV) is the discounted cost of initial construction, maintenance activities, and salvage value of the pavement option during the LCCA period. Future maintenance activities and salvage value are discounted to today's dollars using the discount rate. In simpler terms, NPV is the amount of money MDT would have to invest in treasury bonds today to pay for all construction activities within the pavement's life cycle analysis period less the salvage value. The pavement option with the lowest NPV is considered the best value.

C-Factor: The C-Factor is used to "level the playing field" between alternate pavement sections (most often PMS vs. PCCP). The C-Factor is a cost adjustment added to the least expensive

option's bid (usually PMS) to account for the decreased life cycle costs associated with pavement options with higher initial costs (usually PCCP). The C-Factor is calculated as follows:

PCCP pavement NPV – PMS pavement NPV = C-Factor

Some additional LCCA terminology is as follows:

Remaining Life / Salvage Value: For pavement LCCA, it is necessary to compare pavements with a different length of service life. In these cases, an additional cost is subtracted during the LCCA's last year to account for either remaining service life or the pavement's salvage value. An example of this is the salvage value of a failed concrete pavement which can still serve as a base for a crack & seat and overlay project.

Discount Rate: The discount rate is the real rate of return that can be expected on a conservative, long-term investment. For pavements, the discount rate is calculated as follows:

30-year Treasury Bond Rate – Average Rate of Inflation = Discount Rate

The official discount rate (real interest rate not nominal) published by the White House Office of Budget and Management (OBM circular A-94) is 1.9% for 30 years or longer for 2014. The official discount rate can be found at this link.

14.3.1 LCCA Process:

The Pavement Design Engineer will perform the LCCA process as follows:

1. Perform a pavement design for concrete and PMS options.

2. Establish project parameters including quantities, length and width.

3. Work with the Road Design to identify unique project constraints such as grade limitations due to approaches, structures, curb and gutter, hydraulic features, etc… Project constraints will dictate the types of future rehabilitations that are possible.

4. Identify maintenance cycles associated with the analysis period for each option.

5. Quantify user costs if considered.

6. Determine materials cost based on project quantities and project location. Verify costs with the respective District Construction personnel and the Engineering Cost Analyst.

7. Calculate the Net Present Value of each option.

8. Summarize the LCCA in a brief report and distribute with the activity 600 memo.

14.3.2 Additional Resources:

A more comprehensive description of LCCAs can be found in the NCHRP Report 703 - Guide for Pavement-Type Selection

FHWA Technical Advisory Use of Alternate Bidding for Pavement Type Selection http://www.fhwa.dot.gov/pavement/t504039.cfm

FHWA Transportation Performance Management:

Chapter 15 **Tools for Pavement Analysis**

Path Web is the current version of the road image viewer: This viewer contains information such as image data, rut, ride, surface rendering, GPS map and transverse profile for all Interstate, NHS, Primary, Secondary and some urban routes. Path Web may not be available without Network access or if you are an external Design Consultant.

The ride and rut links are available at this link: annual treatment and condition reports.

Agile Assets is the comprehensive pavement management tool used by the pavement management unit. Contact the Pavement Management supervisor at 444-6149 for more information.

Path View is an image viewer similar to Path Web that accesses data from a share drive. This viewer provides additional information such as:

- Actual mile posts
- Cross Slope
- Rut Depth
- GPS Coordinates

Contact the Pavement Management supervisor at 444-6149 for more information.

For more information, the following link has been provided to MDT's Pavement Analysis Section.

Additional resources regarding surfacing options can be found at the following link.

Also, additional resources for surfacing options can be found at the Research Link.

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Chapter 17 **Appendix A – Sample Traffic Memo**

Montana Department of Transportation
Helena, Montana 59620

Memorandum

To: Bryce Hove
Great Falls Design

From: Becky Duke, Supervisor
Traffic Data Collection & Analysis Section

Date: January 10, 2013

Subject: STPP 3-2(64)60
Pendroy – N & S
Control No. 4051003

Attached is the traffic information requested in an email dated December 21, 2012. There are no major traffic breaks within the project. Please note that the equivalency factors used to calculate ESAL values are determined using information from our weigh-in-motion sites and reflect a five-year average.

If you have any questions or need further assistance, please contact me at 6122.

CC: Ed Shea, Pavement Analysis and Research - Helena
Project File

RAIL TRANSIT AND PLANNING DIVISION
DATA COLLECTION SECTION
Worksheet for Engineering and Planning Purposes

Project Description: Minor Flexible
STPP 3-2(64)60
Pendroy - N & S
Control No. 4051003
S-219: West of US 89/P-3

Date: 10-Jan-13

					*Truck Distribution**	
				5	20.0 %	3.7 %
2012	AADT=	260	PRESENT	6	12.0 %	2.2 %
				7	0.0 %	0.0 %
2015	AADT=	270	LETTING YEAR	8	4.0 %	0.7 %
2035	AADT=	350	DESIGN YEAR	9	34.0 %	6.3 %
	DHV=	50		10	30.0 %	5.5 %
	D=			11	0.0 %	0.0 %
	T=	18.5%		12	0.0 %	0.0 %
	EAL=	23		13	0.0 %	0.0 %
	AGR=	1.3%				
					100.0 %	18.5 %

2012

AADT*=	260	
BUS=	0.4%	1
COM=	18.5%	48
FUT=	1.3%	
DHV=	14.00%	

* Distribution: Preliminary 2012 Vehicle Class Count (Site ID: 50-1-4)

* AADT and Growth Rate: Preliminary 2012 TYC

PROJECT DESCRIPTION: STPP 3-2(64)60
Pendroy - N & S
Control No. 4051003

DATE: 10-Jan-13

PAVEMENT: RIGID:
FLEXIBLE: X

LETTING YEAR AADT: 270 LETTING YEAR 2015
DESIGN YEAR AADT: 350 DESIGN YEAR 2035

LANE DESIGN FACTOR: 100 %

VEHICLE TYPE	% OF TYPE	LETTING YEAR ADT	DESIGN YEAR ADT	MEAN YEAR ADT	DIRECTIONAL ADT	DESIGN LANE ADT	18K EQUIV RATE FAC	MEAN YEAR ADL
CLASS 1 & 2	39.1	105.57	136.9	121.2	60.6	60.6	0.001	0.06
CLASS 3	42.1	113.67	147.4	130.5	65.3	65.3	0.007	0.46
CLASS 4	0.4	1.04	1.3	1.2	0.6	0.6	0.669	0.40
CLASS 5	3.7	9.97	12.9	11.4	5.7	5.7	0.143	0.82
CLASS 6	2.2	5.98	7.8	6.9	3.4	3.4	0.498	1.71
CLASS 7	0.0	0.00	0.0	0.0	0.0	0.0	0.829	0.00
CLASS 8	0.7	1.99	2.6	2.3	1.1	1.1	0.378	0.43
CLASS 9	6.3	16.95	22.0	19.5	9.7	9.7	1.115	10.85
CLASS 10	5.5	14.95	19.4	17.2	8.6	8.6	1.003	8.61
CLASS 11	0.0	0.00	0.0	0.0	0.0	0.0	0.777	0.00
CLASS 12	0.0	0.00	0.0	0.0	0.0	0.0	0.877	0.00
CLASS 13	0.0	0.00	0.0	0.0	0.0	0.0	1.492	0.00
CLASS 14		0.00	0.0	0.0	0.0	0.0		0.00
CLASS 15		0.00	0.0	0.0	0.0	0.0		0.00
CLASS 16		0.00	0.0	0.0	0.0	0.0		0.00
	18.5	49.85	64.6	57.2				
TOTAL VALUES	100.0							23.34

AVERAGE DAILY 18 KIP EQUIVALENT AXLE LOAD: 23.34

20 YEAR EQUIVALENT AXLE LOAD: 170,351

2012	AADT =	260
2015	AADT =	270
2035	AADT =	350
	DHV =	50
	Direction =	
	Com Trks =	18.5%
	ESAL =	23.34
	AGR =	1.260%

* Equivalency Factors: WIM Data (2007 to 2011)

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