Pavement Design Guide

November 2002

Prepared by the Pavement Division, Office of Materials and Technology.

Maryland Department of Transportation
State Highway Administration

OMT
OFFICE OF MATERIALS & TECHNOLOGY

Forward

There are many different approaches that are used in the pavement industry to develop the pavement design for new roadways and the rehabilitation of existing roadways. In the same manner, there are numerous approaches to evaluating the existing condition of a pavement and project site characteristics. The majority of these existing approaches are based on experience of the individuals responsible for either developing pavement design recommendations or construction of the project. There are very limited documented sources of information for evaluation procedures for pavements and design of pavements. Those documented sources of information are typically not written in a procedural manner and expect the reader to process an experienced level of knowledge about pavement and material engineering. All of these cases have led to a lack of consistency between pavement designers in evaluations, designs, pavement rehabilitation techniques, etc. In addition, all this leads to indecision and confusion for new or inexperienced engineers responsible with developing pavement designs.

Therefore, this guide was written for Maryland State Highway Administration (MDSHA) pavement engineers in order to eliminate the problems discussed previously. This guide will provide MDSHA pavement engineers with a procedural process to complete both the evaluation of the pavement condition and the any pavement design requirements. The policies and procedures included in the guide are written to achieve the Business plan goals of MDSHA.

The individuals listed below provided the development, writing, and review of this document. Any questions or comments concerning this guide should be directed to the individuals below.

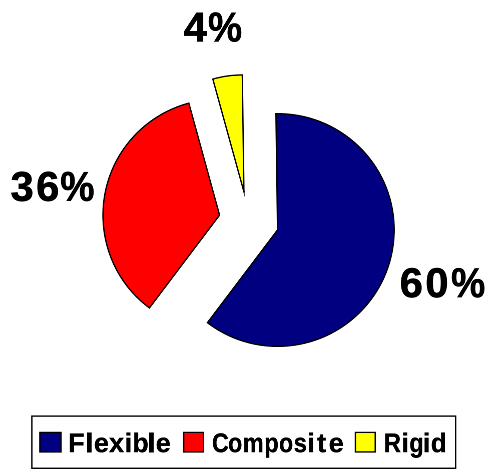
Tim Smith, P.E.
Acting Pavement Division Chief
MDSHA – OMT
2323 W. Joppa Road
Brooklandville, MD 21022
Phone – 410 – 321 – 3110
Fax – 410 – 321 - 3099
Email – tsmith2@sha.state.md.us

Pete Stephanos, P.E.
Director, Office of Materials & Technology
MDSHA – OMT
2323 W. Joppa Road
Brooklandville, MD 21022
Phone – 410 – 321 – 3100
Fax – 410 – 321 - 3099
Email – pstephanos@sha.state.md.us

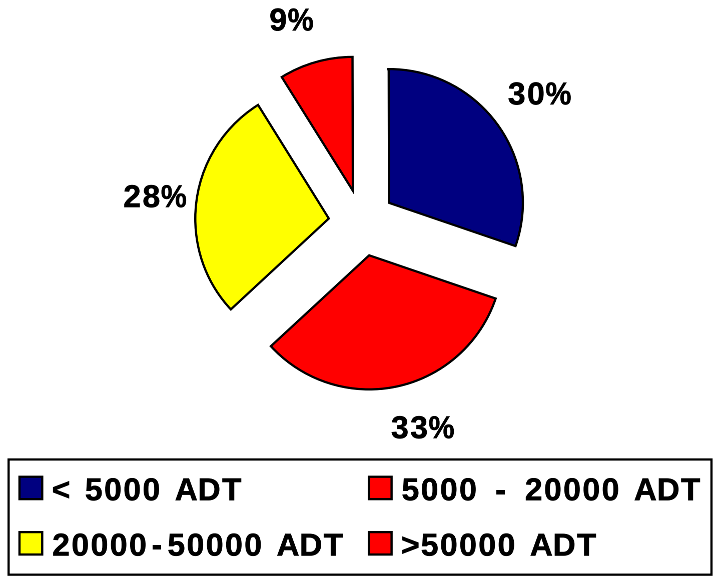
I INTRODUCTION AND BACKGROUND

This Maryland State Highway Administration (MDSHA) Pavement Design Guide provides a comprehensive set of procedures and policies to assist the pavement design engineer in developing recommendations for new construction and pavement rehabilitation projects. The audience this pavement design guide was written for should have a basic background and general understanding of pavement engineering. The purpose of this document is to provide Maryland State Highway pavement designers a guideline to developing pavement recommendations that are consistent and accurate across all pavement engineers. The goal of this document is to supply pavement engineers the guidance to have the ability to provide pavement recommendations that are based on the most effective engineering design considering cost to MDSHA, practicality of construction, and benefit in terms of service life provided to the MDSHA pavement network.

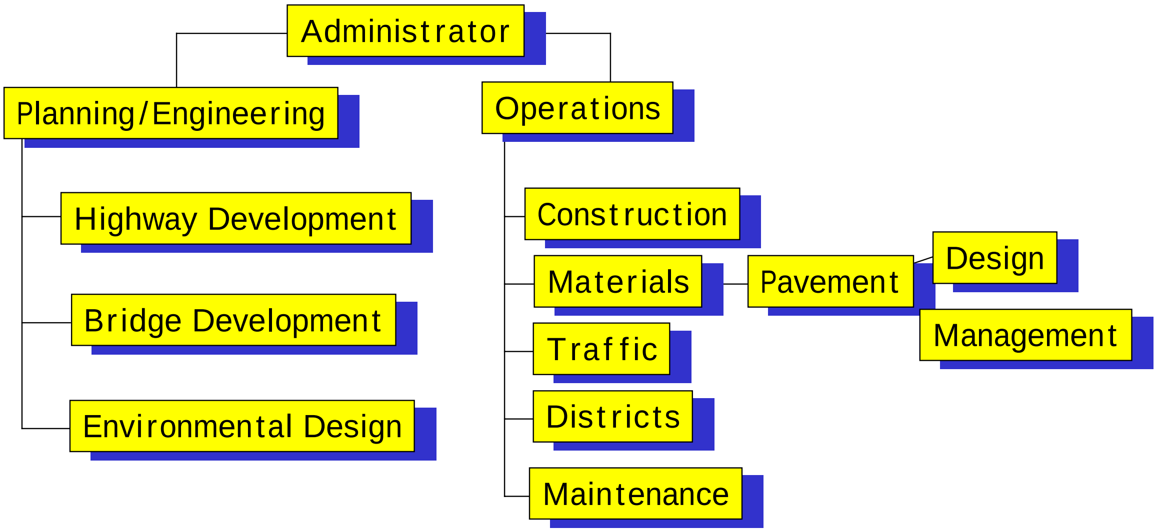
MDSHA is currently responsible for approximately 16,362 lane miles of roadway. Currently, approximately 60% of the pavement network are comprised of flexible pavements, 36% are composite pavements, and less than 5% are rigid pavements.



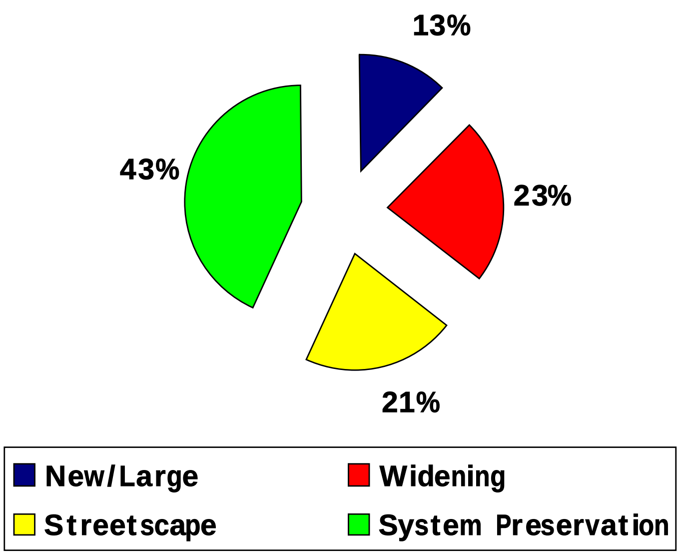
A large portion of the pavement roadway network of MDSHA has a significant traffic volume. The environmental and geological regions of Maryland lend themselves to a wide range of agricultural and industrial commerce. Maryland has three distinct regions that have different traffic and geology/soil conditions. The eastern shore portion of Maryland is dominated with agricultural based commerce and traffic. The soil conditions on the eastern shore are dominated by sandy soils. The central portion of Maryland is strongly metropolitan in business with a high percentage of industrial type traffic motivated by the water ports. This central portion of Maryland is a peidmont area dominated by silty clays, clays, and micateous silts. The western portion of Maryland is dominated by logging, extractive industries (coal, stone, etc.), and agriculture based commerce as well as several major trucking routes that highly influence the traffic mix. The western portion of Maryland is characterized by rocky and silty soils. Although Maryland is a small state, there is a wide range of existing soil and geological conditions, as well as unique traffic volume and weight trends that the MDSHA pavement design engineer needs to possess knowledge of in order to make accurate pavement recommendations.



MDSHA is currently structured into two separate functions for the purpose of completing and maintaining construction projects, Planning/Design and Operations. The MDSHA Administrator is responsible for overseeing and directing both functions to ensure that MDSHA goals are achieved. Each function is completed by the efforts of several offices. There are separate divisions under each office that complete more specific tasks related to the completion of construction projects. The Pavement Division of MDSHA falls under the Office of Materials and Technology (OMT). The OMT is responsible for the design and quality of all materials placed in MDSHA projects. The Pavement Division is responsible for the design of all pavement structures in MDSHA projects. In addition, the Pavement Division is responsible for the data processing and analysis off all the network level data collection for MDSHA roadways and its pavement management system (PMS). Below is a general schematic of the MDSHA structure described previously.



Other State agencies, the Federal government, Counties, Cities, and other local municipalities are responsible for the remaining roadways in Maryland. Frequently in these cases, these other agencies seek the assistance Pavement Division with regard to pavement recommendations. Therefore, in addition to the workload of MDSHA construction projects, the pavement engineers in the Pavement Division are often asked to assist and review other agencies pavement construction projects. Based on the work completed over the last several years and the existing transportation budget, the Pavement Division is responsible for approximately 200 to 250 pavement design recommendations a year, from MDSHA projects alone. The graph below shows a current breakdown of those MDSHA projects:

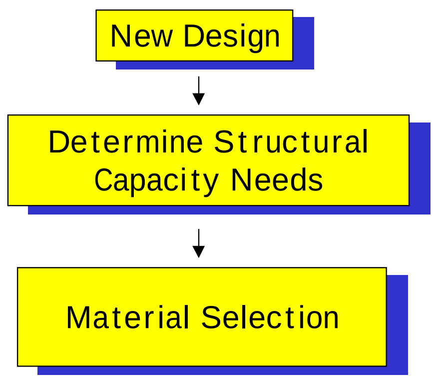


Particular sections of interest in the MDSHA Pavement Design Guide are Section III "Pavement Design Procedures", Section VI " Pavement Design Policies", and Section X "Material Library." These sections provide a procedural guide for developing pavement rehabilitation recommendations as well as a standard Maryland State Highway for pavement design inputs and policies.

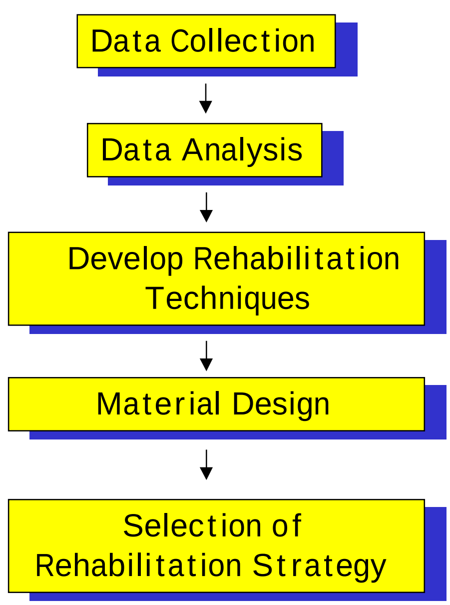
II MDSHA PAVEMENT DESIGN OVERVIEW

The Pavement Division currently utilizes the 1993 “AASHTO Guide for Design of Pavement Structures“ and its subsequent revisions as the framework for its pavement design procedure. The AASHTO guide documents an empirical procedure based on testing and data collection from the AASHO road test in the late 1950’s and from subsequent refinements and revisions. The MDSHA Pavement Design Guide utilizes a majority of the AASHTO Guide for design analysis and has made modifications to that procedure based on local knowledge, available pavement data, material knowledge, past experiences, and knowledge and resource base of pavement engineers.

In the simplest of terms, the goal of the MDSHA pavement engineer is to assess the structural and functional needs of a roadway and develop pavement sections that will meet a desired service life at a specified minimum ride quality and structural capacity. This concept is presented in the following figure:



The individual design tasks required to achieve this goal are more involved than the figure above can demonstrate. In order to determine the structural and functional needs of a roadway, specific pavement engineering design tasks need to be accomplished as presented in the following diagram:



The data collection effort involves the gathering of historical information of the roadway as well as existing pavement and subgrade conditions. The data collection efforts include the following tasks: records review, site inspections, visual condition survey, functional condition data collection, and structural condition data collection. The data analysis efforts involve assessing the functional and structural condition of the existing roadway and subgrade in terms useful life for design and material selection tasks. Data analysis efforts include identifying uniform section, material strengths, existing distress types, and the existing pavement performance. Developing rehabilitation techniques involves selecting repair and rehabilitation techniques to correct existing distress types and meet the structural and functional demands of the roadway. Material design involves the selection of materials to meet the structural and functional demands of the roadway. The selection of a rehabilitation strategy for construction is based on the most effective engineering design considering cost to MDSHA, practicality of construction, and benefit in terms of service life provided to the MDSHA pavement network.

The material selection portion of this pavement design overview requires knowledge of the existing pavement materials available in specific regions of Maryland. A significant amount of effort has been completed to develop policies in this guide for material selection. These policies are intended to keep recommendations and material selection consistent across different pavement design engineers and maintain consistency with current construction issues and concerns. These policies are intended to take into account the different environmental conditions across the state, material availability, material costs, predicted material performance, existing material performance, traffic conditions, and functional use of the roadway in the future.

III PAVEMENT DESIGN PROCEDURES

The project life cycle of a MDSHA project will require completion of numerous procedures by the pavement design engineer that are presented in this section. Several of these procedures can be done concurrently. Although, most of the procedures have a step-by-step outline to follow, the individual pavement design engineer is left to their own responsibility that the most beneficial and economical pavement recommendation is provided for the construction project.

Each procedure in this section is divided into four headings: *purpose, resource requirement, procedure*, and *flowchart*. The *purpose* provides a general understanding and goals of the procedure. The *resource requirement* section provides an approximate amount effort required to complete the task. The *procedure* section details the procedure in a step-by-step process. The *process flowchart* provides a graphical overview of the entire procedure with reference to procedural steps throughout the process.

III.A PRELIMINARY PAVEMENT RECOMMENDATION

III.A.1 Purpose

Preliminary pavement recommendations are developed to:

- Provide a preliminary material and construction cost estimate to Office of Planning and Preliminary Engineering or Highway Design Division based on network level data,
- Provide Access Permits Division with a network level data pavement recommendation,
- Review county projects,
- Provide Districts with a network level data pavement recommendation for fiscal year estimates, and
- Provide a maintenance or operations engineer with a network level data pavement recommendation.

III.A.2 Resource Requirements

The preliminary pavement recommendation process is typically done with data available to the pavement designer in the office coupled with a site visit. Typically, FWD and core results are not available at the time a preliminary pavement recommendation is developed. The pavement design in a preliminary pavement recommendation is based on available network level data and standard design inputs and policies provided in this guide. The preliminary pavement recommendation procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Records Review	1	2
Staff Engineer or Project Engineer	Site Visit	1	4
Staff Engineer or Project Engineer	Data Analysis	1	2
Staff Engineer or Project Engineer	Project Communication	1	2
Staff Engineer or Project Engineer	Pavement Design	1	2
Staff Engineer or Project Engineer	Memo Development	1	2

III.A.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed in a preliminary pavement recommendation. Reference to specific design inputs for rehabilitation or new design development can be located in Section IV "Pavement Design Policies". Numerous steps contained in this procedure can be completed within several software applications that the Pavement Division currently uses and has under development. The software application tool available to the Pavement Division that can be principally used to complete shoulder pavement designs at this time is DARWin. The following procedure was written to provide the design engineer with adequate information to complete a preliminary pavement recommendation without specific knowledge or access to computer software applications, but with the assumption that these tools were available.

Certain steps in the preliminary pavement recommendation process and other processes overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. The procedure described below is conducted after receiving a request from a customer. This request can be completed via a memo request, e-mail request, or verbal request. When this process is completed for a project that will receive a formal pavement recommendation, certain steps will be done with respect to the final design rather than a recommendation provided for a District Maintenance or Access Permits Division project.

Follow the steps below to complete a preliminary pavement recommendation.

Step 1. Complete Step 5 as documented in Section III.F "Pavement Rehabilitation Design." If this project will **not** ultimately receive a final pavement recommendation, a smaller folder without partitions is applicable.

Responsible Party - Engineer

Step 2. Complete Steps 6 through 12 as documented in Section III.F "Pavement Rehabilitation Design." These steps involve collecting all available network level data and in-house data for the project.

Responsible Party - Engineer

Step 3. Complete a site visit to the project. Identify different rehabilitation strategies, causes for existing distress, drainage problems, traffic conditions, and unique site conditions. Complete PD-04 to assist in the documentation of the site conditions.

Responsible Party - Engineer

Step 4. Complete Steps 30 through 32 and 34 as documented in Section III.F "Pavement Rehabilitation Design." These steps involve determining the structural capacities of the pavement, original, effective, and future. Use available records review data.

Responsible Party - Engineer

Step 5. Complete Steps 35 through 38 and Steps 46 through 49 as documented in Section III.F "Pavement Rehabilitation Design." These steps involve determining the required rehabilitation needs. For a new design, complete Steps 24 through 31 as documented in Section III.G "New Pavement Design."

Responsible Party - Engineer

Step 6. Complete Steps 59 through 61 as documented in Section III.F "Pavement Rehabilitation Design." These steps involve determining the pavement recommendation. Depending on the source of the request, a formal memorandum may not be needed. In any case, a documented response shall be provided to the requester and placed in the file.

Responsible Party - Engineer

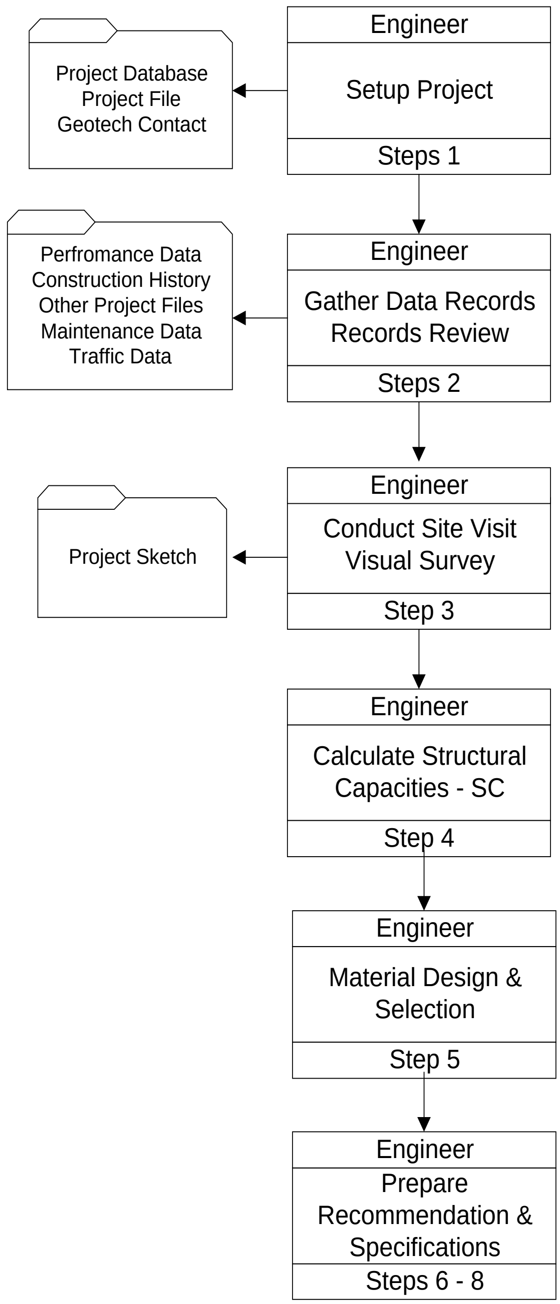
Step 7. Complete Steps 63 and 65 as documented in Section III.F "Pavement Rehabilitation Design." These steps involve QC review.

Responsible Party - Engineer

Step 8. Relay information to the person that requested the recommendation in the appropriate form based on the type of request. This response could be in the form of an e-mail, formal or informal memorandum, and phone conversation. In any case, some form of documentation shall be placed in the project file folder regarding the recommendation that was provided.

Responsible Party - Engineer

III.A.4 Preliminary Pavement Recommendation Flowchart



III.B VISUAL PAVEMENT SURVEY

III.B.1 Purpose

Visual surveys are conducted on rehabilitation designs to:

- Establish an overall condition of the pavement
- Determine estimate of pre-overlay repair needs
- Establish estimate of pre-overlay repair quantities
- Identify predominant distress types

III.B.2 Resource Requirements

The visual survey procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Project or Staff Engineer	Setup Testing	1	4
Survey Technician or Staff Engineer	Data Collection	1	12
Survey Technician or Staff Engineer	Data Entry/Processing	1	4
Survey Technician or Staff Engineer	QC Field Work	1	2
Technician	QC Data Entry	1	1

III.B.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to rate pavement condition for rehabilitation designs. Certain steps in the visual survey for pavement rehabilitation design process and the pavement rehabilitation process overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. The evaluation described below is conducted while walking slowly alongside the highway pavement. Protective clothing must be worn (safety vest and hard hat) at all times when conducting the survey. In addition, a survey vehicle equipped with a mounted flashing light should be parked behind the surveyor at all times.

Step 1. Ride full length of job in each direction slowly in the outside travel lane to identify the following information:

- predominant distresses
- riding quality
- traffic level
- shoulder type and condition
- curb and gutter existence
- curb reveal
- drainage type and condition
- verification of pavement type

Responsible Party – Engineer or SPID

Step 2. Define evaluation sections to rate based on:

- any changes in pavement type

- considerable changes in pavement condition or, more importantly, any changes in the variability of pavement condition
- major changes in traffic level
- major changes in shoulder or drainage structures

Responsible Party – Engineer or SPID

Step 3. Measure pavement lane width.

Responsible Party – Engineer or SPID

Step 4. Determine sample unit length based on the following:

- If lane width = 9 feet then length = 275 feet
- If lane width = 10 feet then length = 250 feet
- If lane width = 11 feet then length = 225 feet
- If lane width = 12 feet then length = 200 feet

"General Rule of Thumb" – Maintain a consistent sample unit length throughout the analysis section. This value will typically be 200 feet. By maintaining a consistent length, the number of sample units can be determined more quickly prior to the starting survey. However, do not have a sample unit with a width greater than 16 feet.

Responsible Party – Engineer or SPID

Step 5. Identify the total number of sample units within each section based on the number of lanes, project length, lane width, and the selected sample unit length; most likely a 200 foot sample unit length. The maximum number of sample units for any section shall be 250. Any sections that have more than 250 sample units following the section criteria shall be broken up to ensure no section has more than 250 sample units.

Responsible Party – Engineer or SPID

Step 6. Assess pavement condition variability and determine sampling rate based on Form PD-03; and discussions with the Assistant Pavement Division Chief, if needed. Determine the number of sample units to be surveyed based on Form PD-03 and the total number of sample units in each section. Complete Form PD-03 with information regarding total number of sample units, estimated standard deviation, and the number of sample units to be survey.

Responsible Party – Engineer or SPID

Step 7. Document observations from Steps 1 through 6 on Form PD-04.

Responsible Party – Engineer or SPID

Examples of Surveyed Sample Units
(Based on 30% Sampling Rate)

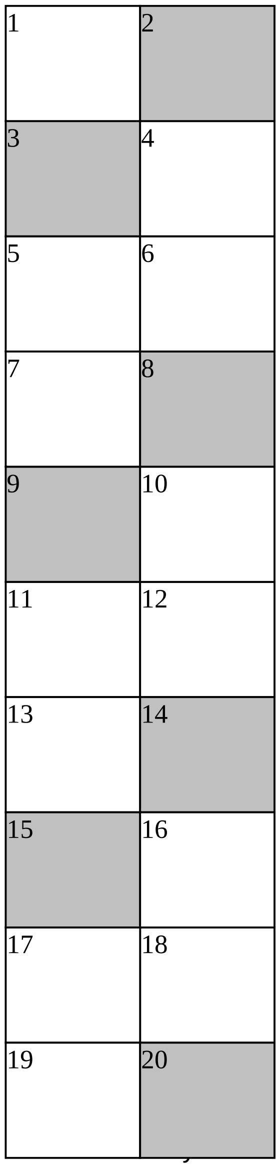
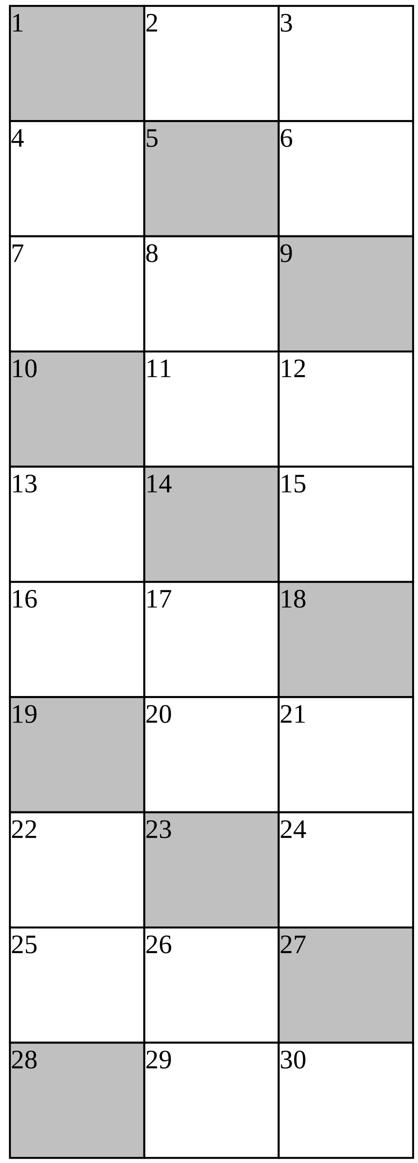


Figure 1



Or
2 Lanes in one Direction

<u>Note:</u> Selection of the first sample unit is random at the start of the section. Assuming 30% sampling, the first sample unit is random among the first three sample units. After the first sample unit is selected, the following sample units to be surveyed will be based on the sampling rate; i.e. every third sample unit for 30% sampling.

Step 8. Complete a Field Work Request Form (Form PD-05) for a visual survey. Provide the detailed information on the form regarding the number of sample units, sample unit dimensions, and number of surveyed sample units. Submit to SPID to schedule testing if SPID has yet to complete visual survey. If SPID has completed visual PCI

survey, verify correct sampling rate was completed, and skip to Step 10.

Responsible Party - Engineer

Step 9. SPID should notify project engineer if requested completion date cannot be accommodated. If so, request should be submitted to consultant as a work assignment.

Responsible Party - SPID

Step 10. Select location of sample units to be surveyed. The selected units should be distributed equally across all lanes and staggered. In addition, the samples should be roughly spaced equal distances from one another. It is important to select the sample units at "random" in order to get and accurate representation of the entire section. Figure 1 diagrams an example of the total number of sample units and the surveyed sample units in a project.

Responsible Party – SPID or Consultant

Step 11. Go to beginning of first sample unit to be surveyed.

Responsible Party – SPID or Consultant

Step 12. Mark beginning location and measure sample length with wheel. Note distress types in sample unit. Record the sample unit location on the "PCI Sample Unit Distress Form" (Form PD-02 or PD-02a).

Responsible Party – SPID or Consultant

Step 13. Record distress types, extent, and severity on the "PCI Sample Unit Distress Form" (Form PD-02 or PD-02a).

Responsible Party – SPID or Consultant

Step 14. Repeat Steps 9 through 13 for the remaining sample units to be surveyed.

Responsible Party – SPID or Consultant

Step 15. Calculate total sum of distress for each distress and severity level for sample unit distress on Form PD-02 or PD-02a.

Responsible Party – SPID or Consultant

Step 16. Provide tallied survey sheets for each sample unit, partially filled out form PD-04 (if applicable), and any additional field notes for the project to the design engineer.

Responsible Party – SPID or Consultant

Step 17. Determine the PCI deduct value for each distress type and severity for each sample unit using the distress deducts curves and record on PD-02 or PD-02a.

Responsible Party – Engineer or Consultant

Step 18. Enter the corrected PCI deduct value and record on For PD-02 or PD-02a.

Responsible Party – Engineer or Consultant

Step 19. Randomly select 5% (at least 2) of the sample units surveyed previously for quality control testing.

Responsible Party – Engineer or Consultant

Step 20. Repeat steps 9 through 13 to record distress quantities and steps 15 through 19 to calculate sample unit PCI values for each of the quality control test sample units. An independent rater should conduct QC surveys.

Responsible Party – Engineer or Consultant

Step 21. Record the quality control PCI values on the original sample unit PCI values on Form PD-06a. Calculate the average difference for all surveyed sample units.

Responsible Party – Engineer or Consultant

Step 22. If the average difference is greater than 10 then discuss results with the project engineer to determine if further testing is necessary. Otherwise, provide project engineer with field data and all forms.

Responsible Party – Engineer or Consultant

Step 23. Calculate PCI and PCI statistics for each section using the PCI software application, (PCItool). It is located on the network at n:omr\everyone\pavedsgn\pci. This software is a MS Access database program. The PCItool software allows you to calculate individual sample unit PCI values, PCI section statistics, and extrapolate distress quantities. At minimum, print out section reports and extrapolated report for each section in the project for the file. Individual sample unit reports are not necessary in the project file, unless otherwise specified.

Responsible Party - Engineer

Step 24. Check the data entry for the PCI input for any errors and document on Form PD-06a.

Responsible Party - Engineer

Step 25. If the sampling rate used for the survey was =< 30% and if the PCI standard deviation is >= 2 from the assumed standard deviation in Form PD-03, then additional sample units need to be surveyed. Determine the additional number of sample units by completing Form PD-03 with the more accurate PCI standard deviation data and request additional testing if necessary.

Responsible Party - Engineer

Step 26. If the sampling rate used for the survey was > 30% and if the PCI standard deviation is >= 3 from the assumed standard deviation in Form PD-03, then additional sample units need to be surveyed. Determine the additional number of sample units by completing Form PD-03 with the more accurate PCI standard deviation data and request additional testing if necessary.

Responsible Party - Engineer

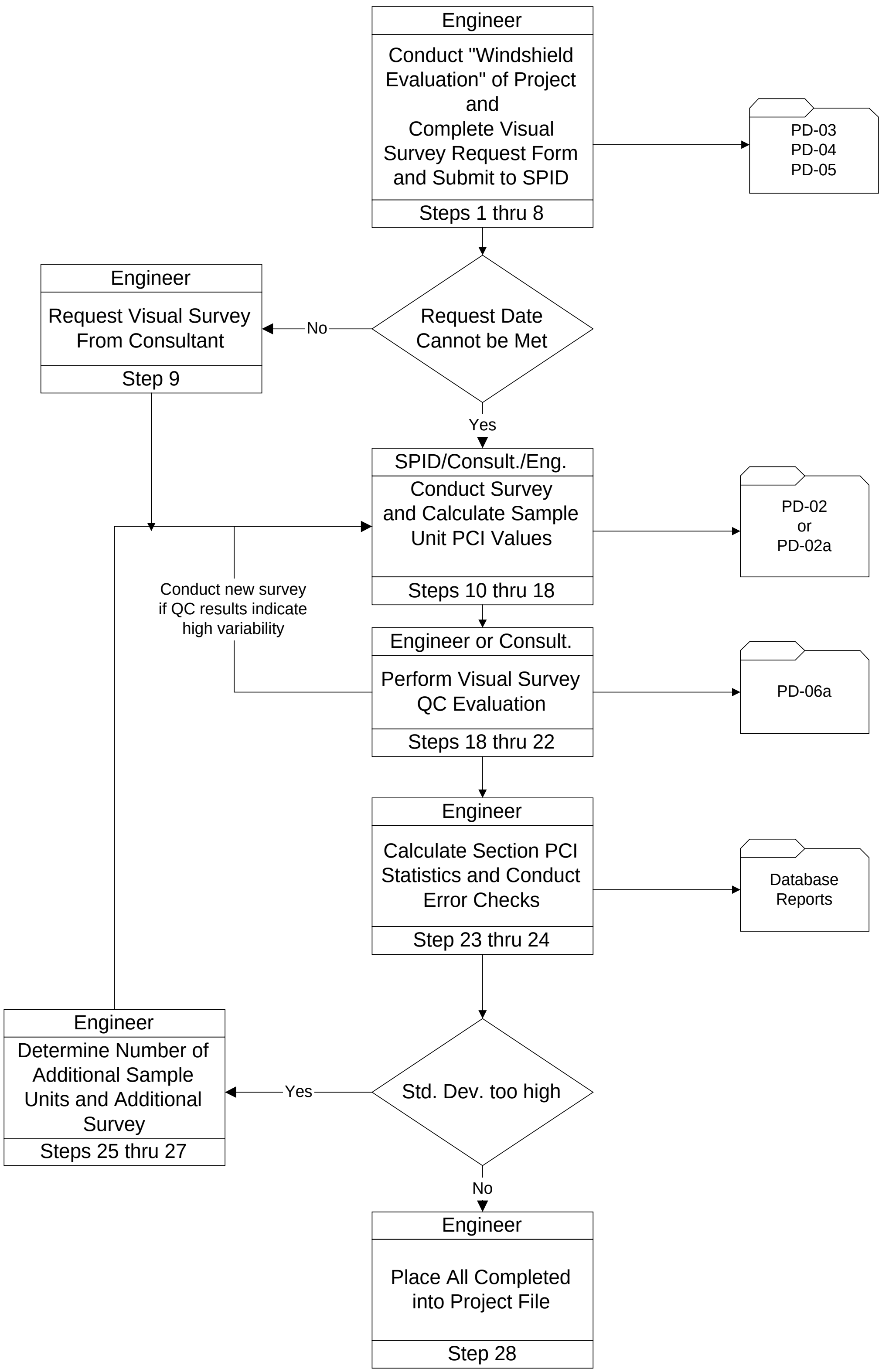
Step 27. If additional sample units are necessary, repeat steps 9 through 13 for each of the additional sample units and recalculate the section PCI following steps 15, 17, 18, and 23. The additional sample units should be selected in locations outside of the original sample locations.

Responsible Party - Engineer

Step 28. Place all completed forms in project file.

Responsible Party - Engineer

III.B.4 Visual Survey Process Flowchart



III.C PAVEMENT STRUCTURE AND SOIL PROPERTY DETERMINATION

III.C.1 Purpose

Pavement structure and soil determination is completed to:

- Identify the pavement structure of the roadway,
- Identify condition of pavement layers beneath the surface,
- Identify weak and thin areas in the pavement structure,
- Determine soil characteristics and existing conditions, and
- Identify depth to water table and bedrock.

III.C.2 Resource Requirements

The actual operation of obtaining the pavement structure and soil properties is completed by OMT field crews from either Geotechnical Explorations or SPID. The pavement engineer is responsible for determining the location and number of areas to collect this information. Determining the number and location of areas to collect pavement and soil data requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Site Visit	1	2
Staff Engineer or Project Engineer	Testing Request	1	2
Staff Engineer or Project Engineer	Field Crew Communication	1	1
Staff Engineer or Project Engineer	Reviewing Data	1	1

III.C.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed in determining pavement structure and soil properties. The primary focus of this section is to provide guidance to the pavement engineer to request the proper number and location of cores/borings. Assistance from the Geotechnical Exploration contact will be required for determining locations of specific soil borings. In most cases requiring a Geotechnical Explorations contact, they will take the lead with regard to requesting and determining test locations and ask for input on roadway borings. Pavement coring operations for pavement structure determination is typically handled by SPID. Soil and pavement borings that collect soil samples are typically handles by the field crews for Geotechnical Explorations Division. The steps identified in this section are to be completed in conjunction with new or rehabilitation pavement design procedures.

Follow the steps below to determine pavement structure and soil property test locations.

Step 1. Determine general areas or sections of roadways where either pavement cores and/or soil borings will be needed to develop pavement recommendations. Pavement cores are needed to identify the existing pavement structure in areas to be rehabilitated or reconstructed. Pavement cores should be obtained to identify the amount of pavement to be removed in cases where the roadway is to be reconstructed; i.e. bridge approaches. Pavement cores should also be obtained in proposed MOT pavement areas. The Geotechnical designer on the project and the existing site conditions will dictate locations for soil borings.

Responsible Party - Engineer

Step 2. Determine the number of mainline cores for the resurfacing portion of a project. Ideally, FWD testing has been completed prior to coring. This information can be used to assist in determining core locations. Changes in deflections, either an increase or decrease, can be used to indicate the need for a core location. Cumulative sum analysis of deflection data can be used to identify different performing sections for core locations. Without the aide of FWD data, a pavement core should be taken in each lane for each pavement structure that exists in the project. The existing conditions of the roadway will assist in determining the frequency of test locations. A consistent pavement type and conditions will typically require a pavement core every 2000 to 4000 feet per lane. If pavement conditions change, then a section could be identified for a core location. Use the same criteria used to determine analysis sections to identify sections for core locations.

Responsible Party - Engineer

Step 3. Determine the number of shoulder cores for the resurfacing portion of a project. A pavement core should be taken in the shoulder (inside and outside) at about half the frequency as that of the mainline, when traffic is not expected to use the shoulder. When traffic is expected to use the shoulder, either for MOT and permanently after construction is complete, a pavement core should be taken in the shoulder (inside and outside) at the same frequency as that of the mainline.

Responsible Party - Engineer

Step 4. Identify any unusual pavement areas and have the pavement structure and materials identified with cores. Examples of unusual pavement areas include the following:

- Obvious widening areas of the older narrower mainline pavement
- Areas of poor performing patches
- Areas of poor condition pavement
- Over poor performing joints in composite pavement
- Areas exhibiting wet conditions on the surface beyond the typical time for drying after rain/snow
- Parking areas / turn lanes / shoulders in projects identified for cross slope adjustment that require the reduction in elevation of the edge of the roadway
- Areas of pavement that appear to have material related problems

Step 5. Determine the number and location of pavement borings for the widening portion of a project. A pavement boring shall be taken in every location in the roadway that currently does not support mainline traffic. These locations include shoulders, gore areas, turn lanes, ramps, and parking lanes. The frequency of the pavement borings will vary depending on the conditions of the pavement, existing site conditions, and project scope. However, for a rule of thumb, a pavement boring should be taken every 1000 to 2000 feet for a consistent appearing pavement structure for a project.

Responsible Party - Engineer

Step 6. Determine the number and location of soil borings for the widening and new construction portion of a project. This decision will reside with the Geotechnical Designer on the project, but occasionally the pavement engineer will be required to make these decisions. Obviously, existing conditions will dictate, but as a guideline, the following locations should have soil borings:

- Every 500 to 1000 feet along the centerline of widening or new construction, and alternative route locations
- Above proposed storm water management facilities
- Above proposed utility locations
- Above proposed storm drain locations
- At the toe of proposed embankments
- At pier or column locations
- At sound wall footing locations

Responsible Party - Engineer

Step 7. Determine the number and location of pavement cores for Bridge Design projects. Bridge projects typically involve some mainline pavement structure identification, but more important, MOT pavement structure identification. This typically requires numerous test locations in the four shoulders near bridges. The frequency of the pavement borings will vary depending on the conditions of the pavement, existing site conditions, and project scope. Obviously, existing conditions will dictate, but as a guideline, the following locations should have soil borings:

- Every shoulder (inside and outside) and both side of the bridge approach shoulders shall have a test location within 50 to 75 feet from the bridge.
- In the mainline on both the approach and leave side of the bridge within 50 feet of the bridge, each lane may not be necessary
- All areas proposed for MOT pavement that currently do not support mainline traffic, every 250 to 500 feet

Responsible Party - Engineer

III.D PATCHING SURVEY

III.D.1 Purpose

Patching surveys are conducted on rehabilitation designs to:

- Identify type and location of patches
- Establish detailed quantity of patching

III.D.2 Resource Requirements

The patching survey procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Survey Technician, or Staff Engineer	Data Collection	1	24
Survey Technician, or Staff Engineer	Data Entry/Processing	1	2
Survey Technician, or Staff Engineer	QC Field Work	1	2

III.D.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to identify patching locations. Certain steps in the patching survey process and the pavement rehabilitation process overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. Locations should be identified after the design customer has selected the preferred design alternate. The evaluation described below is conducted while walking slowly alongside the highway pavement. Protective clothing must be worn (safety vest and hard hat) at all times when conducting the survey. In addition, a survey vehicle equipped with a mounted flashing light should be parked behind the surveyor at all times.

Step 1. Complete a Field Work Request Form (Form PD-05) for a patching survey. Provide the detailed information on the form regarding the estimated partial-depth and full-depth patching in addition to relevant rehabilitation strategies that would effect patching survey. Submit to SPID to schedule survey.

Responsible Party - Engineer

Step 2. SPID should notify project engineer if requested completion date cannot be accommodated. If so, request should be submitted to consultant as a work assignment.

Responsible Party - SPID

Step 3. Obtain the Patching Guidelines forms and talk to the design engineer for specific criteria regarding patching.

Responsible Party – SPID or Consultant

Step 4. The entire length of the job should be evaluated by walking slowly along the pavement. Survey the entire pavement width, including the shoulders, ramps, and intersections to identify:

- patching lane
- location of patch within lane (All, IWP, OWP)
- begin and end locations of patch
- distress type and severity patched
- patch width or size

Responsible Party – SPID or Consultant

Step 5. Document the information recorded in Step 4 on the "Pavement Patching Survey Form" (Form PD-07).

Responsible Party – SPID or Consultant

Step 6. Sum total of patching areas for partial-depth and full-depth patching for entire project. Provide field data and all forms to design engineer.

Responsible Party – SPID or Consultant

Step 7. Randomly select 5% of total area surveyed previously for quality control testing.

Responsible Party – Engineer

Step 8. Repeat steps 3 through 6 to identify patching quantities and locations for 5% of area. An independent rater should conduct QC surveys.

Responsible Party – Engineer

Step 9. Record the QC patching (partial and full-depth) areas and the original sample patching areas on Form PD-06b. Calculate the difference for full-depth and partial-depth patching. Compare patching quantity and locations of original survey with QC survey. If difference in quantities for partial-depth and full-depth patching is greater than 3% of the QC survey area, the patching survey should be re-evaluated to determine reasons for the discrepancies. Also, verify that the locations of patches are not significantly different.

Responsible Party – Engineer

Step 10. Compare patching survey quantities for the entire project with the quantity estimate on the request form (PD-05). If the two patching quantities are significantly different, (+/- 3%) of the total pavement area of the project, then discuss with Assistant Pavement Division Chief. Patching guidelines maybe altered based on discussions and patching survey re-performed.

Responsible Party – Engineer

Step 11. Enter the patching locations and extents in the "Patching Quantity Spreadsheet" (Form PD-08) and print out the patching report to be included into the project files.

Responsible Party – Engineer

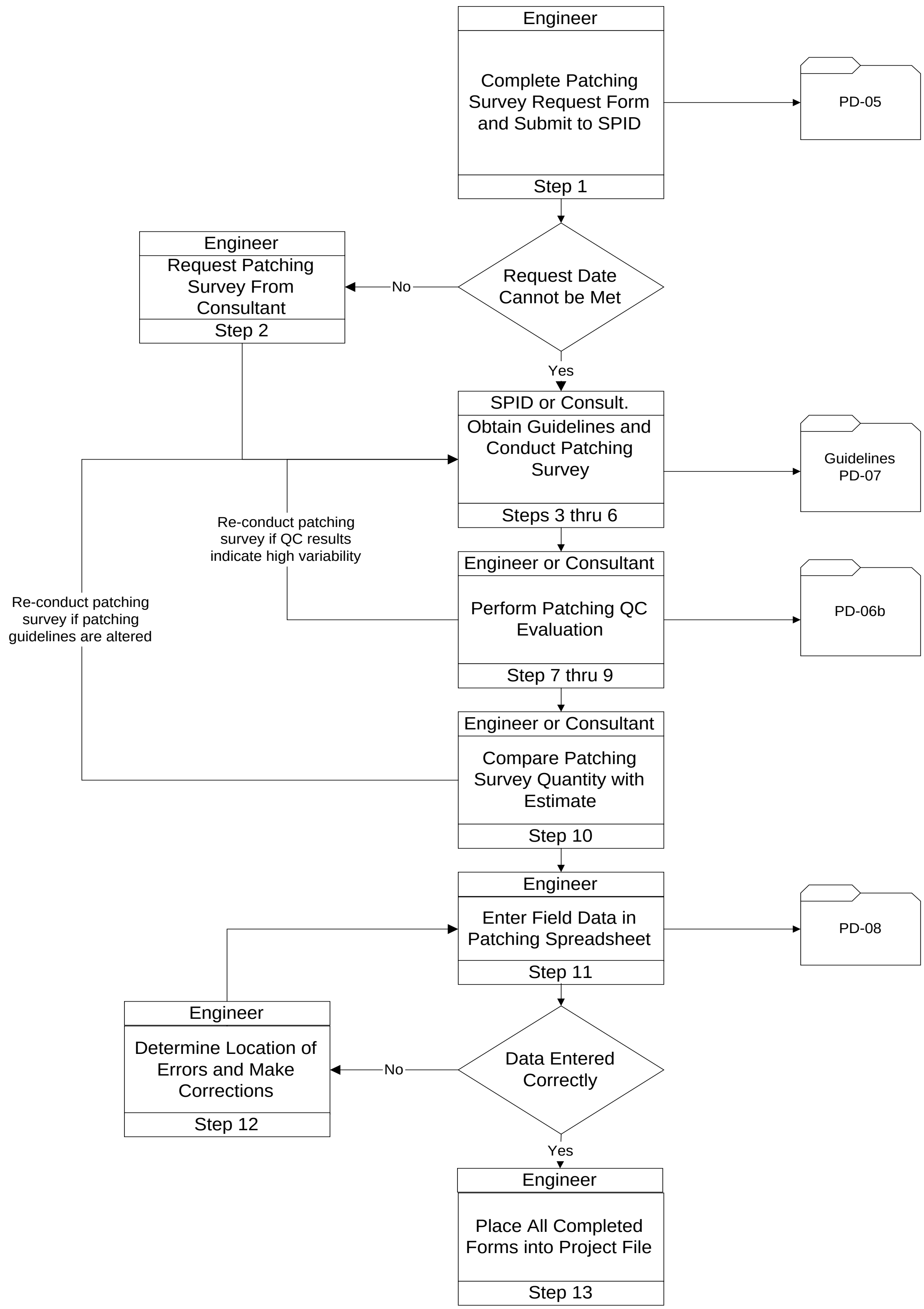
Step 12. The entry of the patching quantities must be checked for input errors by an independent reviewer. Corrections should be made to all errors identified in the quality control process. The reviewer must note when the review process is completed and all corrections are made on the “Patching Survey Quality Control Form” (Form PD-06b).

Responsible Party – Engineer

Step 13. Place all completed forms in project file.

Responsible Party – Engineer

III.D.4 Patching Survey Process Flowchart



III.E TRAFFIC ANALYSIS

III.E.1 Purpose

Traffic Analysis is performed on new and rehabilitation pavement designs to:

- Develop the expected traffic volumes for a pavement section
- Develop the expected truck weight characteristics for a pavement section
- Develop the expected ESAL over the service life of a pavement section
- Develop the expected ESAL for Superpave mix design information
- Calculate the expected ESAL applications to failure for a pavement section
- Calculate the expected ESAL applications since the last major rehabilitation

III.E.2 Resource Requirements

The traffic analysis process documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff or Senior Engineer	Data Collection	1	0.5
Staff or Senior Engineer	ADT Calculation	1	0.5
Staff or Senior Engineer	WIM Site Selection	1	0.5
Staff or Senior Engineer	Truck Factor Development	1	1.0
Staff or Senior Engineer	Future ESAL Calculation	1	0.5
Staff or Senior Engineer	Past ESAL Calculation	1	0.5
Staff or Senior Engineer	ESAL to Failure Calculation	1	0.5

III.E.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to perform traffic analysis. Certain steps in the traffic analysis process and the pavement rehabilitation process overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. Numerous steps contained in this procedure can be completed within several software applications that the Pavement Division currently uses and has under development. The software application tools available to the Pavement Division that can be principally used to complete traffic analysis at this time are DARWin and other in-house programs. The following procedure was written to provide the design engineer with adequate information to complete a traffic analysis without specific knowledge or access to computer software applications, but with the assumption that these tools were available.

Step 1. Retrieve required project level traffic input data needed to initiate the traffic analysis process. This data is typically provided by the Office of Planning and Preliminary Engineering (OPPE), Travel Forecasting Section of the Project Planning Division. If project level traffic input data is not provided by OPPE, it needs to be requested from the OPPE. The traffic input data request is typically handled by the project owner, but on occasion the Pavement Division may need to make a request. The type of project level traffic input data needed is the following:

- Average Daily Traffic (ADT)
- Percent Trucks
- Percent Growth
- Directional Distribution
- Truck Counts – FHWA 13 classes and MDSHA 6 classes

The following hierarchy should be used to obtain traffic data in order of preference:

- OPPE – Travel Forecasting
- Adjacent or Older Projects
- Highway Location Reference Manual
- SHA GIS
- Request from project owner

Responsible Party - Engineer

Step 2. Calculate the ADT and cumulative ADT for future and past years. The growth factor is not always provided specifically as a single value. The OPPE generally provides growth rate in terms of the ADT in the expected construction year of the project and the estimated ADT sometime in the future after construction; typically 20 years. In this case, the growth rate can be calculated with the following equation:

$$r = [(FADT / CADT)^{(1/(FY - CY))}] - 1$$

where:

r = growth rate (in decimal form)
FY = future year
CY = construction year
FADT = future year ADT
CADT = construction year ADT

Occasionally, the growth rate is available and not the future year ADT. Use the following equation to calculate the future ADT given the growth rate:

$$FADT = CADT * (1 + r)^{(FY - CY)}$$

where:

r = growth rate (in decimal form)
FY = future year
CY = construction year
FADT = future year ADT
CADT = construction year ADT

Use the following equation to calculate the total cumulative growth factor:

$$G = [((1 + r)^{(FY - CY)}) - 1] / r$$

where:

G = total cumulative growth factor
r = growth rate (in decimal form)
FY = future year
CY = construction year

Responsible Party - Engineer

Step 3. Identify the traffic group of the roadway at the project site. The MDSHA roadway network is divided into individual traffic groups that are intended to have similar truck weight characteristics. The traffic groups are made up of District, area (rural, urban), functional class (interstate, arterial, and local), and local land use (residential, commercial, industrial, extractive, agriculture, and forest).

Responsible Party – Engineer

Step 4. Select a weigh-in-motion (WIM) that most closely represents traffic group of the roadway at the project site. OPPE provides the classification and number of trucks for a project level design, but the weight of the trucks needs to be obtained from network level weight data. The selection of the WIM site and the resulting truck weight characteristics combined with truck count data will be used to develop a truck factor.

Responsible Party – Engineer

Step 5. Select the terminal serviceability to be used for the pavement design of the roadway. Refer to Section VI.A.a.5 “Pavement Design Policies – Terminal Serviceability” for details of selection.

Responsible Party – Engineer

Step 6. Select the future structural capacity (SC_f) to be used for the pavement design of the roadway. This is an iterative process because in order to select SC_f the engineer needs to know the cumulative ESALs over the service life of the pavement. Since we are following this procedure to develop ESALs, we do not know the cumulative ESALs over the service life of the pavement at this time. There will have to be an original estimate of SC_f to initiate the iterative process. The numerical value of SC_f will vary depending on the pavement type of the roadway.

Responsible Party – Engineer

Step 7. Calculate the truck factor (TF). Inputs needed to calculate a truck factor include the following:

- ADT
- Percent Trucks
- Percent Growth
- Directional Distribution
- Truck Counts – FHWA 13 classes
- Truck weigh data – FHWA 13 classes
- Terminal Serviceability
- Future Structural Capacity (SC_f)

There is an in-house software application to take the data provided above and develop a truck factor.

Responsible Party – Engineer

Step 8. Select the lane distribution of the roadway based on the proposed number of lanes. Refer to Section VI.A.a.9 “Pavement Design Policies – Traffic Lane Distribution” for details of selection.

Responsible Party – Engineer

Step 9. Calculate the future cumulative ESAL applications using the following equation:

Cumulative ESAL = (CADT) * 365 * (%T) * (TF) * (G) * (D) * (L)
where:

CADT = average daily traffic in construction year
%T = percent trucks
TF = truck factor
G = cumulative growth factor
D = directional distribution
L = Lane distribution

The cumulative growth factor (G) in the equation above is calculated in Step 2 of Section III.E “Traffic Analysis.” The (G) is a function of the number of years into the future for the analysis, or design life of the pavement. Therefore, a different (G) must be calculated for each ESAL design life needed in the pavement design process and then used in the equation above to develop the cumulative ESAL. The type of design lives needed for development of cumulative ESAL values are the following:

- Resurfacing Design Life – several values possible
- New/Reconstruction Design Life
- Superpave Mix Design ESAL – 20 years

Place results of ESAL calculation and analysis inputs into project file in a summary sheet.

Responsible Party – Engineer

Step 10. Calculate ADT at time of last major rehabilitation of roadway at project site. Use the following equation to calculate the ADT at a year in the past:

PADT = CADT * (1 + r)^ (CY – PY)
where:

r = growth rate (in decimal form)
PY = past year
CY = construction year
PADT = past year ADT
CADT = construction year ADT

Responsible Party – Engineer

Step 11. The cumulative past ESALS applied to the roadway since the last major rehabilitation can be calculated using the following equation:

Cumulative ESAL = (PADT) * 365 * (%T) * (TF) * (G) * (D) * (L)
where:

PADT = average daily traffic at time of last rehabilitation
%T = percent trucks
TF = truck factor
G = cumulative growth factor
D = directional distribution
L = Lane distribution

The cumulative growth factor (G) in the equation above is calculated in Step 2 of Section III.E "Traffic Analysis." The (G) is a function of the number of years into the past for the analysis, or number of years since last rehabilitation.

Place results of ESAL calculation and analysis inputs into project file in a summary sheet.

Responsible Party – Engineer

Step 12. Calculate ESALs to failure based on the pavement structure at the time of the last major rehabilitation. ESALS to failure is calculated from Figures 3.1 and 3.7 and the nomograph equations on page II-32 and II-45 in the "AASHTO Guide for Design of Pavement Structures" for flexible and rigid/composite sections respectively. Refer to Section VI.A.2 "Pavement Policies – ESALS to Failure" for specific details for inputs to calculate ESALS to failure. The following design inputs are required in order to use the nomograph or equation on pages II-32 and II-45:

- *Flexible Pavements:*
  - SC_o – Original structural capacity of the pavement section following the last rehabilitation.
  - Design Subgrade Resilient Modulus (M_r) – Obtained from either nondestructive testing or geotechnical soils investigation. Section III.J "FWD Data Analysis" will provide further details to this procedure. Section X.B "Material Library – Material Properties" includes default values for various types of subgrade materials.
  - Initial Serviceability – See Section VI "Pavement Design Policies".
  - Terminal Serviceability – See Section VI "Pavement Design Policies".
  - Reliability – See Section VI "Pavement Design Policies".
  - Standard Deviation – See Section VI "Pavement Design Policies".

- *Rigid and Composite Pavements*:
  - SC_o – Original structural capacity of the pavement section following the last rehabilitation.
  - Modulus of Subgrade Reaction (k) – Obtained from either nondestructive testing or geotechnical soils investigation. Section III.J "FWD Data Analysis" will provide further details to this procedure. Section X.B "Material Library – Material Properties" includes default values for various types of subgrade materials. The

modulus of subgrade reaction can also be calculated following the procedures identified in Section 3.2.1 of Chapter II of the "AASHTO Guide for Design of Pavement Structures."

- Initial Serviceability – See Section VI "Pavement Design Policies".
- Terminal Serviceability – See Section VI "Pavement Design Policies".
- Reliability – See Section VI "Pavement Design Policies".
- Standard Deviation – See Section VI "Pavement Design Policies".
- J Factor (J) – Calculate the average load transfer for the design section following Section III.J, "FWD Data Analysis" and select a J Factor as defined in Section VI "Pavement Design Polices"
- PCC Elastic Modulus (E)– Calculate the elastic modulus following Section II.J, "FWD Data Analysis" or select a typical value from Section X.B "Material Library – Material Properties" for concrete materials.
- PCC Modulus of Rupture (S'c)– Calculate the modulus of rupture based on the PCC elastic modulus using the following equation

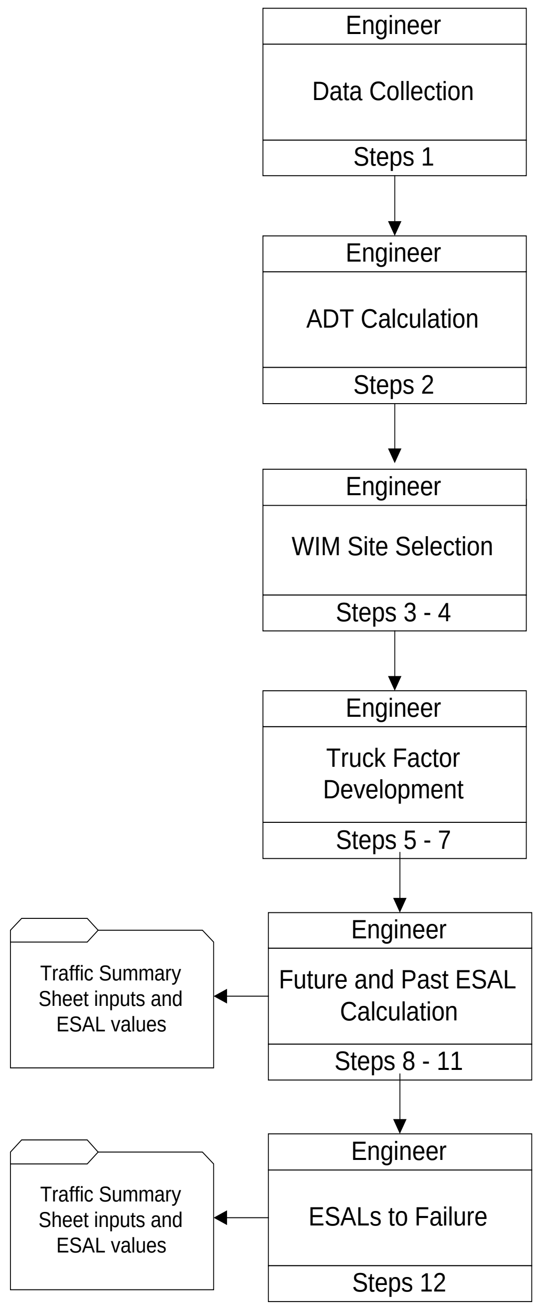
  S'c = 43.5(E)+488.5
  where: E = PCC Elastic Modulus (psi/million)

  Modulus of rupture can also be estimated using Section X.B "Material Library – Material Properties"
- Drainage Factor (C_d) – Select a drainage factor as defined in Section VI "Pavement Design Policies" based on materials and existing conditions

Place results of ESAL calculation and analysis inputs into project file in a summary sheet.

Responsible Party – Engineer

III.E.4 Traffic Analysis Process Flowchart



III.F PAVEMENT REHABILITATION DESIGN

III.F.1 Purpose

Pavement rehabilitation designs are conducted to:

- Identify the exiting condition of the pavement
- Identify the deterioration trend in the existing pavement
- Determine the future structural requirements of the pavement
- Determine the future functional requirements of the pavement
- Determine the necessary improvements to the existing pavement
- Identify the material requirements to improve the pavement following the Pavement Design Policies

III.F.2 Resource Requirements

The pavement rehabilitation design procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Records Review	1	16
Staff Engineer or Project Engineer	Site Visit	1	16
Staff Engineer or Project Engineer	Data Analysis	1	16
Staff Engineer or Project Engineer	Project Communication	1	24
Staff Engineer or Project Engineer	Rehabilitation Design	1	32
Staff Engineer or Project Engineer	Memo Development	1	40

III.F.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed in a typical pavement rehabilitation design recommendation. Reference to specific design inputs for rehabilitation design development can be located in Section IV "Pavement Design Policies". Numerous steps contained in this procedure can be completed within several software applications that the Pavement Division currently uses and has under development. The software application tool available to the Pavement Division that can be principally used to complete pavement rehabilitation designs at this time is DARWin. The following procedure was written to provide the design engineer with adequate information to complete a rehabilitation design without specific knowledge or access to computer software applications, but with the assumption that these tools were available.

Certain steps in the pavement rehabilitation design process and other processes overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. The procedure described below is conducted after receiving a request from a customer. This request can be completed via a memo request, e-mail request, or verbal request. The scope of the project should be provided with the pavement rehabilitation design request. If the scope of the project is not provided in the request, Step 2 may need to be completed prior to Step 1. Otherwise, if the project scope is available with the design request, follow the steps below to complete the pavement rehabilitation design.

Step 1. If the project involves any base-widening, reconstruction, storm water management, embankment slopes, new construction, or has special geotechnical issues in the project, a geotech contact needs to be established for the project. If the Geotechnical Explorations Division is the customer, this step can be skipped. If not, send e-mail to the Geotechnical Exploration Division Chief to request contact for job.

Responsible Party - Engineer

Step 2. Contact project engineer managing the project to request project schedule and to establish contact person from OMT (ask that all future correspondence and requests be sent directly to the OMT contact).

Responsible Party - Engineer

Step 3. Notify Pavement Design Assistant Pavement Division Chief of the recommendation due date with project description, scope, and schedule, if he/she is not already aware of the project.

Responsible Party – Engineer

Step 4. Notify SPID of the project and the potential testing needs. Provide SPID with the route number, county, project limit description, project scope, mileposts, and charge number in addition to the anticipated testing requirements. This is not intended to be a formal testing request, but rather to allow SPID to estimate future testing workload. This will allow SPID will put project into testing schedule and attempt to complete the visual PCI survey as soon as possible by completing steps 1 through 7 and steps 10 through 17 from Section IIIB "Visual Survey for Pavement Rehabilitation Design."

Responsible Party – Engineer

Step 5. Setup a project folder with six sections. The following section headings and contents will be the same for each project. All project documents should be filed in the appropriate categories.

	Section Heading	Typical Contents
1.	Summary	Data Collection and Analysis QC Forms (PD-06,a,b,c) Pavement Rehabilitation Alternative Design Form (PD-10)
2.	Reports	Memorandums or reports issued by OMT
3.	Correspondence	Letters from requesting division Printouts of project related e-mails Comments or supporting information sent to other divisions Project Log-In Sheet Field Work Request Form
4.	Records Review	Traffic and ESAL data Construction history, as- built project data Maintenance History Performance Data (IRI, Rut, etc.) Nearby or previous project data

Section Heading		Typical Contents
5.	Data Analysis	Site visit notes PCI survey forms (PD-02, PD-02a, and PD-06a) Boring logs Patching tables and forms (PD-07, PD-08, and PD-06b) FWD data and forms (PD-06c)
6.	Design	Output from Darwin or other pavement design programs

Responsible Party - Engineer

Step 6. Gather and calculate the following traffic data as described in Section IIIE "Traffic Analysis" of the Pavement Guide and place results in file.

- Service Life Design ESALs
- Superpave Mix Design ESALs
- Cumulative ESALs to Failure
- Cumulative ESALs Since Last Major Rehabilitation
- ADT and ESAL production report

Responsible Party – Engineer

Step 7. Retrieve construction history, performance data (ride quality, friction, rutting, cracking), and other network level data for project site and place in file.

Responsible Party – Engineer

Step 8. Contact District or Highway Designer to identify if any as-built plans exist that include any portion of the project limits. Request any available plans and records relevant to construction history data and place in file.

Responsible Party – Engineer

Step 9. Identify any projects designed by OMT from the "File Room Login/Sign Out Book" that include any portion of the project limits or are adjacent to or near by the project limits. Gather any relevant project data and place in file.

Responsible Party – Engineer

Step 10. Verify with District Maintenance that construction history is accurate and up to date. Record any changes on the construction history print out and return to file.

Responsible Party – Engineer

Step 11. Prepare a sketch of the project to identify any potential sections within the project limits. This sketch is intended to communicate variations within the project with other staff and is not intended to be distributed outside OMT, therefore, the sketch should be hand drawn and rough.

Responsible Party - Engineer

Step 12. Assemble Records Review data and discuss project with Assistant Pavement Division Chief. The discussion will involve identifying problem areas, special concerns, alternative design pavement strategies, and identifying test setups and specific requests. In addition, any geotechnical testing needs will be identified and discussed.

Responsible Party – Engineer

Step 13. Discuss geotechnical testing needs with geotech contact and agree on testing schedule.

Responsible Party – Engineer

Step 14. Determine status of visual PCI survey in the SPID testing schedule. If SPID has not completed the visual PCI survey to date, complete site visit by following Steps 1 through 8 from Section IIIB "Visual Survey for Pavement Rehabilitation Design" of the Pavement Guide. If SPID has already been to the project and completed the visual PCI survey, follow Steps 1 through 7 in order to familiarize the pavement designer with the project pavement conditions and to verify the accuracy of the data gathered by SPID.

Responsible Party – Engineer

Step 15. Conduct PCI Survey following Section III.B " Pavement Visual Survey." This step may be completed before this time as noted in Step 4.

Responsible Party – SPID or Consultant

Step 16. During the site visit, identify potential core locations, special testing needs, different rehabilitation strategies, causes for existing distress, drainage problems, and unique project characteristics. In addition, verify Pavement Management System's estimate for a rehabilitation strategy. Document comments and thoughts onto field notes and place in project file.

Responsible Party – Engineer

Step 17. Complete the Field Work Request Form (Form PD-05) for FWD testing. Coring requests should only be requested at this time if MOT requirements allow only one operation for FWD and coring. Otherwise, coring should be requested at a later time in the design process. Refer to Section III.I "FWD Testing" of the Pavement Guide for guidelines for FWD testing setup. Observations from the site visit will assist in determining testing setup and requirements.

Responsible Party – Engineer

Step 18. Discuss testing set up with SPID and describe special testing needs.

Responsible Party – Engineer and SPID

Step 19. Conduct FWD testing following the procedures outlined in Section III.I "FWD Testing."

Responsible Party – SPID

Step 20. Complete Quality Control of FWD data in field to ensure data is of good quality, checking for sensors out of range and erroneous data outputs from the FWD equipment. Provide data after quality check to engineer once completed.

Responsible Party – SPID

Step 21. Store the FWD data file on the server under the project directory and note on Form PD-06 that the quality checks of the FWD data were conducted following Step 19 in Section III.I “FWD Testing”.

Responsible Party – Engineer

Step 22. Conduct steps in Section III.J “FWD Data Analysis” to process the raw FWD data and to determine erroneous test points and coring needs/locations. In addition, the subgrade behavior (linear/ non-linear), the proper number of sensors to use in analysis, the proper load levels to use in analysis, and the proper test locations to use in analysis will be determined in this pre-processing data procedure.

Responsible Party – Engineer

Step 23. Complete the Field Work Request Form (Form PD-05) for coring. Identify core locations and any special instructions (i.e. coring over a crack or to a specific depth). Information obtained from the pre-processing of the FWD data will assist the engineer to determine coring needs and locations. Refer to Section III.C for assistance and guidelines.

Responsible Party – Engineer

Step 24. Conduct coring operations. Refer to Section III.C for assistance and guidelines.

Responsible Party – SPID

Step 25. Provide coring data to pavement designer.

Responsible Party – SPID

Step 26. Place core results in project file.

Responsible Party – Engineer

Step 27. Provide geotechnical test data to pavement designer.

Responsible Party – Geotech

Step 28. Place geotechnical test data in project file.

Responsible Party – Engineer

Step 29. Determine analysis sections by evaluating the following:

- Sections as defined by FWD testing using the cumulative sum procedure or deflection vs. station procedure following steps in Section III.J “FWD Data Analysis.
- Sections defined by changes in visual condition noted during the windshield survey conducted in Steps 1 through 7 of Section III.B “Pavement Visual Survey.”
- Sections defined for the PCI survey.

- The project engineer should use their judgement to define analysis sections that exhibit consistent condition, pavement structure and traffic level.

  Responsible Party – Engineer

Step 30. Calculate original structural capacity (SC_o) of each pavement analysis section. Use the following guidelines to determine SC_o for each pavement type:

- *Flexible Pavement :* $SC_o = SN_o$

  $SN_o = a_1 {*} d_1 + a_2 {*} d_2 + a_i d_i$

  where:

  d = layer thickness

  a = layer structural coefficient (Follow guidelines established in Section X.C "Material Library – Design Properties" for each pavement layer at the time of the last rehabilitation. Construction History information and core results will assist the pavement designer in determining the age and thickness of pavement layers.)

- *Rigid Pavement :* $SC_o = D_o$

  $D_o = d$

  where:

  d = PCC layer thickness

- *Composite Pavement :* $SC_o = D_o$

  $D_o = d_{pcc} + h_{pcc}$

  where:

  d_{pcc} = PCC layer thickness

  h_{pcc} = The existing AC layer thickness converted to an equivalent PCC layer thickness. Use the following equation for conversion:

  $$h_{pcc} = d_{ac} * [(E_{ac}(1-\mu_{pcc}^2)) / (E_{pcc}*(1-\mu_{ac}^2))]^{1/3}$$

  where:

  d_{ac} = Existing AC layer thickness

  E_{ac} = Existing AC layer elastic modulus (obtained from temperature correction, non-destructive testing, lab results, or typical value, E_{ac} = 750,000 psi.)

  μ_{ac} = Poisson's Ration of AC layer (0.35)

  E_{pcc} = Existing PCC layer elastic modulus (obtained from non-destructive testing, lab results, or typical value, E_{pcc} = 5,000,000 psi.)

  μ_{pcc} = Poisson's Ration of PCC layer (0.15)

Note: $h_{pcc} = d_{ac} / 2$ is a good assumption for composite pavement.

Responsible Party – Engineer

Step 31. Calculate the effective structural capacity (SC_{eff}) of each analysis section following the condition (visual survey) analysis approach. SC_{eff} can be calculated using the visual condition survey analysis approach for flexible, rigid, and composite pavements. Use the following guidelines to determine SC_{eff} following the visual condition survey analysis approach for each pavement type:

- *Flexible Pavement :* $SC_{eff} = SN_{eff}$

  Option 1:

  $$SN_{eff} = SN_o * C_x$$

  where:

  C_x = Condition Factor from visual condition analysis (=< 1.0). To obtain C_x for flexible pavements use the following equation:

  $$C_x = (PCI / 100)^{1/2} > 0.65$$

  where:

  PCI = Average PCI for analysis section obtained following Steps described in Section III.B “Pavement Visual Survey.”

  Option 2:

  $$SN_{eff} = a_1{*}d_1 + a_2{*}d_2 + a_i d_i$$

  where:

  d = layer thickness

  a = layer structural coefficient (Follow guidelines established in by AASHTO found in Table 5.2 on page III-105 in the “AASHTO Guide for Design of Pavement Structures.”

- *Rigid Pavement :* $SC_{eff} = D_{eff}$

  Option 1:

  $$D_{eff} = D_o * C_x$$

  where:

  C_x = Condition Factor from visual condition analysis (=< 1.0). To obtain C_x for rigid pavements use the following equation:

  $$C_x = (PCI / 100)^{1/2} > 0.65$$

  where:

  PCI = Average PCI for analysis section obtained following Steps described in Section III.B “Pavement Visual Survey.”

  Option 2:

  $$D_{eff} = D_o * F_{jc} * F_{dur} * F_{fat}$$

where:

F_{jc} = Joints and cracks adjustment factor. F_{jc} is calculated from Figure 5.12 on page III-124 in the 1993 "AASHTO Guide for Design of Pavement Structures." All of the input values needed to use Figure 5.12 can be tabulated in the field during the pavement designer windshield survey or estimated based on the distress quantities from the PCI visual survey results. Fjc ranges from approximately 0.4 to 1.0. In order to use Figure 5.12 the following items need to be summed:

- Number of unrepaired deteriorated joints per mile
- Number of unrepaired deteriorated cracks per mile
- Number of unrepaired punchouts per mile
- Number of expansion joints, exceptionally wide joints (>1.0"), and full-depth AC patches per mile

F_{dur} = Durability adjustment factor. F_{dur} is a function of the amount of durability problems such as D-cracking in the existing PCC slab. The Durability adjustment factor range is 0.8 to 1.0. The summary results from the PCI survey will provide adequate information to develop an accurate Durability adjustment factor. The 1993 AASHTO Guide for Design of Pavement Structures" provides the following guidelines:

1.00:	No signs of PCC durability problems
0.96 – 0.99:	Durability cracking exists, but no spalling
0.88 – 0.95:	Substantial cracking and some spalling exists
0.80 – 0.88:	Extensive cracking and severe spalling exists

Durability related problems from reactive aggregate are not a prevalent distress in Maryland. The durability adjustment factor is typically close to 1.0 for a majority of Maryland roadways. With extrapolated PCI survey results, use the following equation to calculate F_{dur}:

$$F_{dur} = 1.0 - 0.2 * (A / B)$$

A = Extrapolated number of slabs with D-cracking of any severity

B = Total number of slab in section

F_{fat} = Fatigue damage adjustment factor. F_{fat} is a function of the amount of past fatigue or structural damage that may exist in the PCC slab. The fatigue damage adjustment factor range is 0.9 to 1.0. The summary results from the PCI survey will provide adequate information to develop an accurate Fatigue adjustment factor. The 1993 AASHTO Guide for Design of Pavement Structures" provides the following guidelines for each PCC pavement type:

JPCP (Plain):

0.97 – 1.00	< 5% of slabs are cracked
0.94 – 0.96:	5 to 15 % of slabs are cracked
0.90 – 0.93:	> 15% of slabs are cracked

JRCP (Reinforced):

0.97 – 1.00 < 25 cracks / lane mile

0.94 – 0.96: 25 to 75 cracks / lane mile

0.90 – 0.93: > 75 cracks / lane mile

CRCP (Continuously Reinforced):

0.97 – 1.00 < 4 punchouts / lane mile

0.94 – 0.96: 4 to 12 punchouts / lane mile

0.90 – 0.93: > 12 punchouts / lane mile

The AASHTO criteria to used to develop the F_{fat} factor can be calculated with extrapolated PCI survey results. The extrapolated number of linear cracks, divided slabs, and punchouts can be summed from the PCI results to help to determine the appropriate fatigue factor for the PCC.

- *Composite Pavement :* $SC_{eff} = D_{eff}$

Option 1:

$D_{eff} = D_o * C_x$

where:

C_x = Condition Factor from visual condition analysis (=< 1.0). To obtain C_x for composite pavements use the following equation:

$C_x = (PCI / 100)^{\wedge 1/2} > 0.65$

where:

PCI = Average PCI for analysis section obtained following Steps described in Section III.B "Pavement Visual Survey."

Option 2:

$D_{eff} = (d_{pcc} * F_{jc} * F_{dur}) + (h_{pcc} * F_{ac})$

where:

d_{pcc} = PCC layer thickness

F_{jc} = Joints and cracks adjustment factor. F_{jc} is calculated from Figure 5.12 on page III-124 in the "AASHTO Guide for Design of Pavement Structures." All of the input values needed to use Figure 5.12 can be tabulated in the field during the pavement designer windshield survey or estimated based on the distress quantities from the PCI visual survey results. F_{jc} ranges from approximately 0.4 to 1.0. In order to use Figure 5.12 the following items need to be summed:

- Number of unrepaired deteriorated joints per mile

- Number of unrepaired deteriorated cracks per mile

- Number of unrepaired punchouts per mile

- Number of expansion joints, exceptionally wide joints (>1.0"), and full-depth AC patches per mile

F_{dur} = Durability adjustment factor. F_{dur} is a function of the amount of durability problems such as D-cracking in the existing PCC slab. The Durability adjustment factor range is 0.8 to 1.0. The summary results from the PCI survey will provide adequate information to develop an accurate Durability adjustment factor. The 1993 AASHTO Guide for Design of Pavement Structures" provides the following guidelines:

1.00:	No history or evidence of PCC durability problems
0.96 – 0.99:	Pavement is known to have PCC durability problems, but no localized failures or related distresses are visible
0.88 – 0.95:	Some durability distress, localized failures, is visible at pavement surface
0.80 – 0.88:	Extensive durability distress, localized failures, is visible at pavement surface

This value will be an estimated based on the visual survey and past history of durability problems. For the most part, Maryland does not typically have a reactive aggregate problem. The majority of the problems with composite pavements in Maryland are from reflective cracking, full-depth flexible patches, and flexible pavement widening. PCC durability factor is typically close to 1.0 for a majority of Maryland roadways.

h_{pcc} = The existing AC layer thickness converted to an equivalent PCC layer thickness. Use the equation from Step 30.

F_{ac} = The AC quality adjustment factor. F_{ac} adjusts for the quality of the asphalt concrete (AC) with respect to the effective structural capacity of the composite pavement. F_{ac} is a function of the existing AC material quality related distress in the AC layer. AC material quality distresses include the following: rutting, stripping, bleeding, weathering/raveling. Reflective cracking is not considered an AC material quality related distress for pavement design purposes although the AC material properties may contribute to the development of the distress. F_{ac} ranges from 0.8 to 1.0. The summary results from the PCI survey will provide adequate information to develop an accurate AC quality adjustment factor. The 1993 AASHTO Guide for Design of Pavement Structures" provides the following guidelines:

1.00:	No AC material quality distress
0.96 – 0.99:	Minor AC material quality distress
0.88 – 0.95:	Significant AC material quality distress
0.80 – 0.88:	Severe AC material quality distress

AC material quality factor can be calculated with extrapolated PCI survey results. The calculating procedure to develop the correct extrapolated distress quantities is provided in Section XIII "Pavement Rehabilitation Techniques". Once the proper units for the extrapolated PCI distress quantities, use the following table to determine F_{ac}:

< 25% Weathering		**>= 25% Weathering**	
< 10% Shoving	**>= 10% Shoving**	**< 10% Shoving**	**>= 10% Shoving**
% Rutting	**% Rutting**	**% Rutting**	**% Rutting**

<10%	10-25%	>25%	<10%	10-25%	>25%	<10%	10-25%	>25%	<10%	10-25%	>25%
1.0	0.95	0.80	0.90	0.88	0.80	0.97	0.90	0.80	0.88	0.84	0.80

Not all analysis approaches are valid for every pavement type. The guidelines established in the “AASHTO Guide for Design of Pavement Structures” will be followed by MDSHA. The analysis approaches for determining the existing structural capacity of the pavement structure and the appropriate valid pavement types are listed above.

Responsible Party – Engineer

Step 32. Calculate the effective structural capacity (SC_{eff}) of each analysis section following the traffic analysis approach. The SC_{eff} following the traffic analysis approach can be found for flexible and rigid pavements. Use the following guidelines to determine SC_{eff} following the traffic analysis approach:

$SC_{eff} = SC_o * C_x$

where:

C_x = Condition Factor from traffic analysis (=< 1.0). C_x from traffic analysis is obtained using remaining life (RL) and Figure 5.2 on page III-90 in the “AASHTO Guide for Design of Pavement Structures.” RL is obtained from using the following equation:

$RL = 100 * [1 – (N_p/N_{1.5})]$

where:

N_p = Total number of ESALs applied to pavement since last rehabilitation. Calculate the total number of ESALs since the last major rehabilitation as outlined in steps ? through ? of Section III.E “Traffic Analysis”

$N_{1.5}$ = Total number of ESALs applied to pavement to reach a terminal serviceability of 1.5 since last rehabilitation. To calculate $N_{1.5}$ use the AASHTO flexible or rigid design equation as appropriate with design parameters as defined in Policy VI.A.2 of Section VI “Pavement Design Policies”

<u>Note:</u> For flexible pavements $SC_{eff} = SN_{eff}$

For rigid pavements $SC_{eff} = D_{eff}$

Not all analysis approaches are valid for every pavement type. The guidelines established in the “AASHTO Guide for Design of Pavement Structures” will be followed by MDSHA. The analysis approaches for determining the existing structural capacity of the pavement structure and the appropriate valid pavement types are listed above.

Responsible Party – Engineer

Step 33. Calculate the effective structural capacity (SC_{eff}) of each analysis section following the non-destructive testing analysis approach. The

SC_{eff} following the non-destructive testing analysis approach can be found for flexible pavements only. Use the following guidelines to determine SC_{eff} following the non-destructive testing analysis approach:

Flexible Pavement : $SC_{eff} = SN_{eff}$

where:

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

D = Total thickness of all pavement layers above the subgrade, inches

E_p = Effective modulus of the pavement layers above the subgrade, psi

It is possible for SN_{eff} to be greater than SN_o. If this is true then SN_o should be redefined to be equal to SN_{eff} and Step 31 and Step 32 should be repeated to calculate SN_{eff} based on visual survey results and traffic, respectively. Section III.J "FWD Data Analysis" will provide further details to this procedure.

Not all analysis approaches are valid for every pavement type. The guidelines established in the "AASHTO Guide for Design of Pavement Structures" will be followed by MDSHA. The analysis approaches for determining the existing structural capacity of the pavement structure and the appropriate valid pavement types are listed above.

Responsible Party – Engineer

Step 34. Calculate the required structural capacity (SC_f) for future traffic for each analysis section. SC_f is obtained from Figures 3.1 and 3.7 and the nomograph equations on page II-32 and II-45 in the "AASHTO Guide for Design of Pavement Structures" for flexible and rigid/composite sections respectively. The following design inputs are required in order to use the nomograph or equation on pages II-32 and II-45:

- *Flexible Pavements:*
  - Design Subgrade Resilient Modulus (M_r) – Obtained from either nondestructive testing or geotechnical soils investigation. Section III.J "FWD Data Analysis" will provide further details to this procedure. Section X.B "Material Library – Material Properties" includes default values for various types of subgrade materials.
  - Initial Serviceability – See Section VI "Pavement Design Policies".
  - Terminal Serviceability – See Section VI "Pavement Design Policies".
  - Reliability – See Section VI "Pavement Design Policies".
  - Standard Deviation – See Section VI "Pavement Design Policies".
  - Design ESALs – See Section III.E "Traffic Analysis"

- *Rigid and Composite Pavements*:

    - Modulus of Subgrade Reaction (k) – Obtained from either nondestructive testing or geotechnical soils investigation. Section III.J “FWD Data Analysis” will provide further details to this procedure. Section X.B “Material Library – Material Properties” includes default values for various types of subgrade materials. The modulus of subgrade reaction can also be calculated following the procedures identified in Section 3.2.1 of Chapter II of the “AASHTO Guide for Design of Pavement Structures.”

    - Initial Serviceability – See Section VI “Pavement Design Policies”.

    - Terminal Serviceability – See Section VI “Pavement Design Policies”.

    - Reliability – See Section VI “Pavement Design Policies”.

    - Standard Deviation – See Section VI “Pavement Design Policies”.

    - J Factor (J) – Calculate the average load transfer for the design section following Section III.J, “FWD Data Analysis” and select a J Factor as defined in Section VI “Pavement Design Polices”

    - PCC Elastic Modulus (E)– Calculate the elastic modulus following Section II.J, “FWD Data Analysis” or select a typical value from Section X.B “Material Library – Material Properties” for concrete materials.

    - PCC Modulus of Rupture (S’c)– Calculate the modulus of rupture based on the PCC elastic modulus using the following equation

        S’c = 43.5(E)+488.5
        where: E = PCC Elastic Modulus (psi/million)

        Modulus of rupture can also be estimated using Section X.B “Material Library – Material Properties”

    - Drainage Factor (C_d) – Select a drainage factor as defined in Section VI “Pavement Design Policies” based on materials and existing conditions

    - Design ESALs – See Section III.E “Traffic Analysis”

    Responsible Party – Engineer

Step 35. Calculate the total required HMA overlay thickness without any pre-overlay repair or other rehabilitation alternatives based on the following guidelines:

- *Flexible Pavement*:

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

    Calculate overlay thickness using the following equation:

    $h_{ol} = SN_{ol}/0.44$

- *Rigid and Composite Pavements*

    $h_{ol} = (D_f - D_{eff}) * A$

where:

$A = 2.2233 + 0.0099 * (D_f - D_{eff})^2 - 0.1534 * (D_f - D_{eff})$

h_{ol} = HMA overlay thickness

Responsible Party – Engineer

Step 36. Select feasible rehabilitation alternatives following the guidelines outlined in Section VIII.A, "Pavement Rehabilitation Treatment Methods."

Responsible Party – Engineer

Step 37. Identify pre-overlay needs from Section VIII.B.1 through VIII.B.6, "Pre-Overlay Repair Guidelines", for each selected, applicable rehabilitation alternative. Also, consider drainage treatments that would be appropriate repairs and/or rehabilitation strategies to address the source of the drainage problem.

Responsible Party – Engineer

Step 38. If milling or grinding is to be used as a pre-overlay repair then recalculate SC_o as follows:

- *Flexible pavements*:

  $SC_{o(ar)} = SC_{o(br)} - h_{rem} * a_{ac}$

  where:

  $SC_{o(ar)}$ = Corrected SC_o value after pavement removal
  $SC_{o(br)}$ = SC_o value before pavement removal
  h_{rem} = Depth of pavement removal
  a_{ac} = Layer coefficient of sound asphalt surface material at the time of the last rehabilitation in the past. However, if SC_o was set equal to the SC_{eff} (NDT) in Step 33, then the layer coefficient should be that of the existing deteriorated surface material. (see Section X.C "Material Library-Design Properties")

- *Composite pavements*:

  $SC_{o(ar)} = SC_{o(br)} - h_{rem} / 2$

  where:

  $SC_{o(ar)}$ = Corrected SC_o value after pavement removal
  $SC_{o(br)}$ = SC_o value before pavement removal
  h_{rem} = Depth of pavement removal

Responsible Party – Engineer

Step 39. Change the distresses recorded for each sample unit based on the selected pre-overlay repairs (removal and patching) as defined in Section VIII.B.7, "Pre-Overlay Effect on PCI".

Responsible Party – Engineer

Step 40. Recalculate the densities and corresponding deduct values for each modified distress and sample unit and calculate a new PCI value for each corrected sample unit.

Responsible Party – Engineer

Step 41. Determine the corrected PCI and related statistics for each section based on the modified sample unit PCI values.

Responsible Party – Engineer

Step 42. Recalculate SC_{eff}.as determined from visual survey following the procedures outlined in Step 31.

Responsible Party – Engineer

Step 43. Identify the corrections resulting from the pre-overlay repairs as defined in Section VIII.B.6, "Pre-Overlay Effect on PCI". Use this information to adjust the SC_{eff} calculated based on AASHTO (Option 2). Use Section X.C "Material Library-Design Properties" to assist in determining the correct layer coefficient for each pavement layer following pre-overlay repairs for flexible pavements. Use Tables 5.8 and 5.10 on page III-126 and III-136 in the "AASHTO Guide for Design of Pavement Structures" in determining the existing distresses based on corrections made to the pavement from pre-overlay repairs for rigid and composite pavements.

Responsible Party – Engineer

Step 44. If milling or grinding is to be used as a pre-overlay repair then recalculate SC_{eff} based on traffic analysis approach for flexible pavements using the new SC_o calculated in Step 38 in the following equation.

$$SC_{eff(ar)} = SC_{o(ar)} * C_x$$

where:

$SC_{eff(ar)}$ = Corrected SC_{eff} value after pavement removal

Responsible Party – Engineer

Step 45. If milling or grinding is to be used as a pre-overlay repair then recalculate SC_{eff} based on NDT for flexible pavements as follows:

$$SC_{eff(ar)} = SC_{eff(br)} – (h_{rem} * a_{ac})$$

where:

$SC_{eff(ar)}$ = Corrected SC_{eff} value after pavement removal
$SC_{eff(br)}$ = SC_{eff} value before pavement removal
h_{rem} = Depth of pavement removal
a_{ac} = 0.44 or layer coefficient of existing asphalt surface material (see Section X.C "Material Library – Design Properties")

Responsible Party – Engineer

Step 46. Select the SC_{eff} to use for each rehabilitation design based on the following guidelines.

- *Flexible Pavements*
  - Compare SC_{eff} calculated based on the corrected visual survey (Option 1) to the SC_{eff} calculated from NDT and select the lower of the two. Note: If SC_{eff} was not calculated based

on PCI results (Option 1) then compare the SC_{eff} calculated following AASHTO procedures (Option 2) to the SC_{eff} calculated from NDT.
  - If the SC_{eff} calculated from the corrected visual survey is considerable lower then the SC_{eff} calculated from NDT then determine if the distresses driving the PCI are predominantly functional related. If so, then discuss the selection of the SC_{eff} for design with the Assistant Pavement Division Chief.
  - Compare the SC_{eff} calculated based on traffic and determine if it is considerably different from the selected SC_{eff} for design. If so, discuss with the Assistant Pavement Division Chief.

- *Rigid Pavements*
  - Compare SC_{eff} calculated based on the corrected PCI to the SC_{eff} calculated based on the corrected AASHTO survey and select the lower of the two.
  - If the two SC_{eff} values are considerably different from each other then discuss the selection with the Assistant Pavement Division Chief.
  - Compare the SC_{eff} calculated based on traffic and determine if it is considerably different from the selected SC_{eff} for design. If so, discuss with the Assistant Pavement Division Chief.

- *Composite Pavements*
  - Compare SC_{eff} calcualted based on the corrected PCI to the SC_{eff} calculated based on the corrected AASHTO survey and select the lower of the two.
  - If the two SC_{eff} values are considerably different from each other then discuss the selection with the Assistant Pavement Division Chief.

Responsible Party – Engineer

Step 47. Calculate the total required overlay thickness for all existing flexible pavement sections and appropriate rehabilitation alternatives using the following guidelines:

- *HMA Overlay and Recycled HMA:*

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

Calculate overlay thickness using the following equation:

$$\sum_{i=1}^{n} m_i * a_i * d_i \geq SN_{ol}$$

where:

m_i = drainage coefficient, refer to Section X.C "Material Library – Design Properties"

a_i = layer coefficient, refer to Section X.C "Material Library – Design Properties"

d_i = layer thickness

i = overlay layer

n = total number of layers in overlay

No structural improvement are required for In-Place Recycled HMA to be a viable rehabilitation alternative. Only functional improvements are required in this case.

- *PCC Unbonded Overlay:*

Repeat Step 34 to calculate the required structural capacity (SC_f) using the following design inputs:

- Modulus of Subgrade Reaction (k-value) - Calculate the moduli values for each pavement layer of the existing pavement. This is obtained either following the guidelines outlined in Section III.J "FWD Analysis" or using typical values obtained from Section X.B "Material Library – Material Properties". Use the pavement layer moduli values to obtain the modulus of subgrade reaction (dynamic k-value) following the procedures identified in Section 3.2.1 of Chapter II of the "AASHTO Guide for Design of Pavement Structures."
- Initial Serviceability – See Section VI "Pavement Design Policies".
- Terminal Serviceability – See Section VI "Pavement Design Policies".
- Reliability – See Section VI "Pavement Design Policies".
- Standard Deviation – See Section VI "Pavement Design Policies".
- J Factor (J) – See Section VI "Pavement Design Policies" for new concrete pavement.
- PCC Elastic Modulus (E) - See Section X.B "Material Library – Material Properties" for new concrete pavement.
- PCC Modulus of Rupture (S'c)– Calculate the modulus of rupture based on the PCC elastic modulus using the following equation or estimated using Section X.B "Material Library – Material Properties":

  $S'c = 43.5 * (E) + 488.5$

  where: E = PCC Elastic Modulus (psi/million)
- Drainage Factor (Cd) – Select a drainage factor as defined in Section VI "Pavement Design Policies" based on materials and existing conditions.
- Design ESALs – See Section III.E "Traffic Analysis".

Calculate concrete overlay thickness using the following equation:

$D_{ol} = SC_f$

where:

D_{ol} = thickness of PCC overlay

- *Ultra-Thin Whitetopping*:

  Refer to the attached report "Model Development and Interim Design Procedure Guidelines for Ultra-Thin Whitetopping Pavement" for guidelines regarding Ultra-thin Whitetopping design procedure.

  Responsible Party – Engineer

Step 48. Calculate the total required overlay thickness for all existing rigid pavement sections and appropriate rehabilitation alternatives using the following guidelines:

- *HMA Overlay and HMA Overlay with Saw and Seal*:

  Calculate overlay thickness using the following equation:

  $h_{ol} = (D_f - D_{eff}) * A$

  where:

  $A = 2.2233 + 0.0099 * (D_f - D_{eff})^2 - 0.1534 * (D_f - D_{eff})$

  h_{ol} = HMA overlay thickness

  A minimum of a 4.0" of HMA overlay is required on rigid pavements. The minimum HMA overlay thickness can be reduced if HMA Overlay with Saw and Seal rehabilitation method or joint tape is used.

- *PCC Bonded Overlay*:

  Calculate overlay thickness using the following equation:

  $D_{ol} = (D_f - D_{eff})$

  where:

  D_{ol} = thickness of PCC overlay

  Refer to standard in Section VII "Pavement Design Standards" for steel and joint spacing requirements.

- *PCC Unbonded Overlay*:

  Calculate overlay thickness using the following equation:

  $$D_{ol} = \sqrt{D_f^{\,2} - D_{eff}^{\,2}}$$

  where:

  D_{ol} = thickness of PCC overlay

  Refer to standard in Section VII "Pavement Design Standards" for steel, joint spacing, and bond-breaker requirements.

- *Rubbilize and HMA Overlay and Break and Seat and HMA Overlay*:

Calculate overlay thickness using the following equation:

$$\left(\sum_{i=1}^{n} m_i * a_i * d_i + a_{pcc} * D_{pcc} \right) \geq SC_f$$

where:

mi = drainage coefficient, refer to Section X.C "Material Library – Design Properties"

ai = layer coefficient, refer to Section X.C "Material Library – Design Properties"

di = layer thickness

i = overlay layer

n = total number of layers in overlay

a_{pcc} = layer coefficient of fractured PCC or rubbilized PCC, refer to Section X.C "Material Library – Design Properties"

D_{pcc} = fractured PCC or rubbilized PCC thickness

Responsible Party – Engineer

Step 49. Calculate the total required overlay thickness for all existing composite pavement sections and appropriate rehabilitation alternatives using the following guidelines:

- *HMA Overlay, Recycled HMA Overlay, and HMA Overlay with Saw and Seal*.

Calculate overlay thickness using the following equation:

$h_{ol} = (D_f - D_{eff}) * A$

where:

$A = 2.2233 + 0.0099 * (D_f – D_{eff})^2 – 0.1534 * (D_f – D_{eff})$

h_{ol} = HMA overlay thickness

In order for the rehabilitation alternative HMA Overlay with Saw and Seal to a viable method, the existing HMA material must be entirely removed to expose the bare concrete. The sum of any existing HMA material on the PCC pavement in addition to any resurfacing shall be a minimum of 4.0" in thickness. The minimum HMA overlay thickness can be reduced if HMA Overlay with Saw and Seal rehabilitation method or joint tape is used.

- *PCC Unbonded Overlay*.

Calculate overlay thickness using the following equation:

$$D_{ol} = \sqrt{D_f^{\ 2} - D_{eff}^{\ 2}}$$

where:

D_{ol} = thickness of PCC overlay

Refer to standard in Section VII “Pavement Design Standards” for steel, joint spacing, and bond-breaker requirements.

- *Rubbilize and HMA Overlay and Break and Seat and HMA Overlay:*

Calculate overlay thickness using the following equation:

$$\left(\sum_{i=1}^{n} m_i * a_i * d_i + a_{pcc} * D_{pcc} \right) \geq SC_f$$

where:

mi = drainage coefficient, refer to Section X.C “Material Library – Design Properties”

ai = layer coefficient, refer to Section X.C “Material Library – Design Properties”

di = layer thickness

i = overlay layer

n = total number of layers in overlay

a_{pcc} = layer coefficient of fractured PCC or rubbilized PCC, refer to Section X.C “Material Library – Design Properties”

D_{pcc} = fractured PCC or rubbilized PCC thickness

In order for the rehabilitation alternatives Rubbilize and HMA Overlay and Break and Seat and HMA Overlay to be viable methods, the existing HMA material must be entirely removed to expose the bare concrete.

- *Ultra-Thin Whitetopping*:

Refer to the attached report “Model Development and Interim Design Procedure Guidelines for Ultra-Thin Whitetopping Pavement” for guidelines regarding Ultra-thin Whitetopping design procedure.

Responsible Party – Engineer

Step 50. For new construction or reconstruction rehabilitation alternatives see Section III.G “New Pavement Design.”

Responsible Party – Engineer

Step 51. Complete shoulder pavement design following the guidelines in Section III.L “Shoulder Design” for each rehabilitation alternative.

Responsible Party – Engineer

Step 52. If removal (milling/grinding) was used in pre-overlay repair because of roadway elevation restrictions, determine if corrected overlay thickness is greater than the elevation restrictions. If yes, discuss with the Assistant Pavement Division Chief and the project owner limit to determine if a lower service life is acceptable. If the service life can not be lowered, the rehabilitation alternative should be eliminated from the list of viable candidates.

Responsible Party – Engineer

Step 53. Determine item quantities for pre-overlay repairs and overlay materials. Note: Results from the PCI visual condition procedure will provide quantities of distresses within a sample of an analysis section. These distresses can be extrapolated to provide estimated distress quantities for the entire analysis section. Based on the decisions made in Step 37, the patching quantities can be estimated. This procedure will most likely be handled in the PCI software program. In addition, removal and overlay quantities will need to be estimated based on thickness and coverage area. These quantities should be calculated in the typical unit the value is paid for in a project (tons, square yards, etc.)

Responsible Party – Engineer

Step 54. Develop a cost estimate for the project for each rehabilitation alternative based on the pavement item quantities. Refer to Section XI “Unit Costs” for information regarding the typical cost of items.

Responsible Party – Engineer

Step 55. Complete PD-10 Rehabilitation Alternative Form. Include all possible rehabilitation alternatives in the form. Place form in project file for discussion with the Assistant Pavement Division for Pavement Design and/or Pavement Division Chief at a later time.

Responsible Party – Engineer

Step 56. Compare the cost estimate developed for each rehabilitation alternative with the project budget. This unit cost used to develop a budget is defined in Section VI “Pavement Design Policies” and is a function of the action class established by the Maryland Pavement Management System. If any of the rehabilitation alternatives exceed the budget amount, these alternatives should be removed from the viable options or altered to meet the budget.

Responsible Party – Engineer

Step 57. If pre-overlay total patching quantities (full-depth and partial-depth) for flexible and composite pavements exceeds 30% of the total pavement area or the full-depth patching quantity exceeds 20% of the total pavement area, the feasibility of the rehabilitation alternative should discussed with the Assistant Pavement Division Chief. If pre-overlay total patching quantities (full-depth and partial-depth) for rigid pavements exceeds 5% of the total pavement area, the feasibility of the rehabilitation alternative should discussed with the Assistant Pavement Division Chief.

Responsible Party – Engineer

Step 58. Discuss remaining possible rehabilitation alternatives with Assistant Pavement Division Chief to prioritize rehabilitation alternatives. Note: If the project costs exceeds $20 million then a life cycle cost analysis, as outlined in Section III.H "Pavement Type Selection", shall be performed to select a preferred alternative.

Responsible Party – Engineer

Step 59. Use Section VI "Pavement Design Policies" for assistance in determining specific pavement materials and thickness of lifts to recommend for each rehabilitation alternatives.

Responsible Party – Engineer

Step 60. Discuss rehabilitation alternatives with project owner and appropriate technical teams to select final rehabilitation alternative. The discussion with the project owner should involve input from Construction Division regarding constructability issues with each rehabilitation alternative. The technical material team should provide input for special material requirements in the project.

Responsible Party – Engineer

Step 61. Prepare a memorandum using the typical rehabilitation recommendation memorandum format (n:omr\everyone\pavedsgn\memoformat.doc). Complete the memorandum documenting the selected rehabilitation alternative and using the quantities for pre-overlay developed previously.

Responsible Party – Engineer

Step 62. Gather the appropriate Special Provisions and Detail Specifications to be attached to the recommendation memorandum. Refer to Section VII "Pavement Design Standards" and Section IX "Special Provisions" for appropriate attachments. Also, the engineer should be familiar with the appropriate Special Provisions Inserts that need to be included in the contract documents.

Responsible Party – Engineer

Step 63. Complete the appropriate portions of the Pavement Rehabilitation Design QC Form (PD-6). Complete Project Summary Information form (PD-09). Place forms in project file. Submit project file and memorandum with attachments to Assistant Pavement Division Chief for review of design process and other QC checks.

Responsible Party – Engineer

Step 64. Review design process and analysis for accuracy and quality of work. Complete the Pavement Rehabilitation Design QC Form (PD-6). Submit project file and memorandum with attachments back to Engineer with any comments or notes.

Responsible Party – Assistant Pavement Division Chief

Step 65. Make necessary edits and corrections to rehabilitation design and memorandum. Submit project file and memorandum on letterhead with attachments/specifications to Pavement Chief for final review.

Responsible Party – Engineer

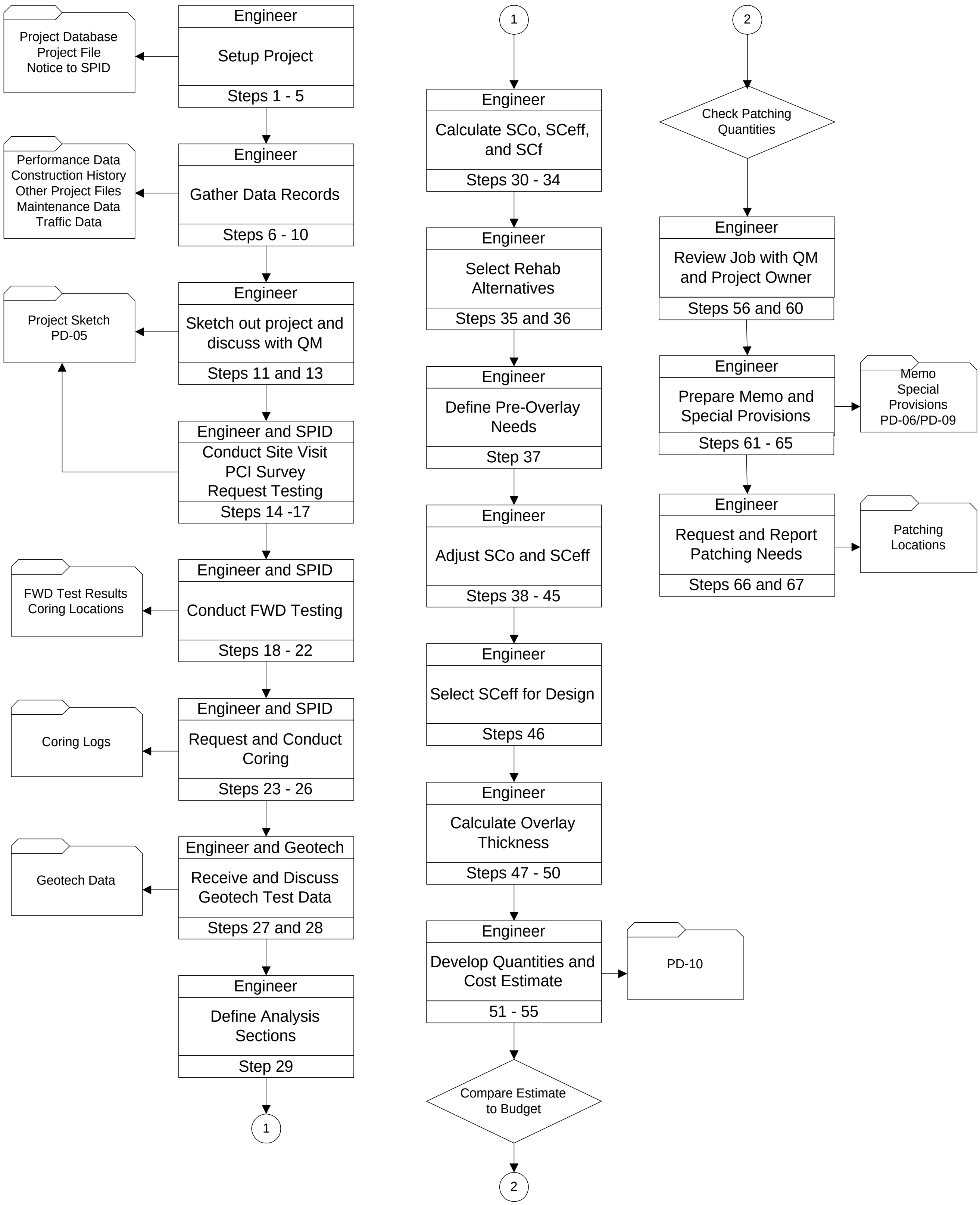
Step 66. Follow the steps outlined in Section III.D “Patching Survey” to request a patching survey to identify patching quantities and locations. Note: This Step should be completed as close to the time of Final Review as possible to accurately define patching quantities.

Responsible Party – Engineer

Step 67. Forward final patching results to project owner for inclusion in contract documents.

Responsible Party – Engineer

III.F.4 Pavement Rehabilitation Process Flowchart



III.G NEW PAVEMENT DESIGN

III.G.1 Purpose

New pavement designs are conducted to:

- Determine the future structural requirements of the pavement
- Determine the future functional requirements of the pavement
- Determine the future structural and functional demands of the new pavement necessary to support the expected traffic volumes
- Identify the material requirements to provide a structurally and functionally adequate pavement system.

III.G.2 Resource Requirements

The new pavement design procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Records Review	1	16
Staff Engineer or Project Engineer	Site Visit	1	16
Staff Engineer or Project Engineer	Data Analysis	1	32
Staff Engineer or Project Engineer	Project Communication	1	48
Staff Engineer or Project Engineer	New Design	1	24
Staff Engineer or Project Engineer	Memo Development	1	40

III.G.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed in a typical new pavement design recommendation. Reference to specific design inputs for rehabilitation design development can be located in Section IV "Pavement Design Policies". Numerous steps contained in this procedure can be completed within several software applications that the Pavement Division currently uses and has under development. The software application tool available to the Pavement Division that can be principally used to complete new pavement designs at this time is DARWin. The following procedure was written to provide the design engineer with adequate information to complete a new design without specific knowledge or access to computer software applications, but with the assumption that these tools were available.

Certain steps in the new pavement design process and other processes overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. The procedure described below is conducted after receiving a request from a customer. This request can be completed via a memo request, e-mail request, or verbal request. The scope of the project should be provided with the new pavement design request. If the scope of the project is not provided in the request, Step 2 may need to be completed prior to Step 1. Otherwise, if the project scope is available with the design request, follow the steps below to complete the new pavement design.

In most cases, a new pavement design is completed in conjunction with a pavement rehabilitation design; i.e. "widening and resurfacing." Therefore, the new and rehabilitation design process will occur concurrently and use the same data and information. The basic difference is that pavement rehabilitation design requires the evaluation and assessment of the existing roadway, both structurally and functionally. New pavement design does not take into account any of the existing conditions of the pavement other than geotechnical and drainage conditions because it is a new design. It is the pavement designers responsibility to use both design processes concurrently where needed and take care to monitor that both designs are agreeable with one another both for design purposes and for construction related reasons.

Step 1. Geotechnical Explorations Division is highly involved in projects requiring a new pavement design. Geotechnical Explorations Division need to be made aware of the project if they have not already been notified by the project customer. Send an e-mail to the Geotechnical Exploration Division Chief to request contact for job if one is not already been provided.

Responsible Party - Engineer

Step 2. Contact project engineer managing the project to request project schedule and to establish contact person from OMT (ask that all future correspondence and requests be sent directly to the OMT contact).

Responsible Party - Engineer

Step 3. Notify Pavement Design Assistant Pavement Division Chief of the recommendation due date with project description, scope, and schedule, if he/she is not already aware of the project.

Responsible Party – Engineer

Step 4. Notify SPID of the project and the potential testing needs. Provide SPID with the route number, county, project limit description, project scope, mileposts, and charge number in addition to the anticipated testing requirements. This is not intended to be a formal testing request, but rather to allow SPID to estimate future testing workload. This will allow SPID will put project into testing schedule and attempt to complete the visual PCI survey, if needed.

Responsible Party – Engineer

Step 5. Setup a project folder with six sections. The following section headings and contents will be the same for each project. All project documents should be filed in the appropriate categories.

Section Heading		Typical Contents
1.	Summary	Data Collection and Analysis QC Forms (PD-06,a,b,c) Pavement Rehabilitation Alternative Design Form (PD-10)
2.	Reports	Memorandums or reports issued by OMT
3.	Correspondence	Letters from requesting division Printouts of project related e-mails

Section Heading		Typical Contents
		Comments or supporting information sent to other divisions Project Log-In Sheet Field Work Request Form
4.	Records Review	Traffic and ESAL data Construction history, as- built project data Maintenance History Performance Data (IRI, Rut, etc.) Nearby or previous project data
5.	Data Analysis	Site visit notes PCI survey forms (PD-02, PD-02a, and PD-06a) Boring logs Patching tables and forms (PD-07, PD-08, and PD-06b) FWD data and forms (PD-06c)
6.	Design	Output from Darwin or other pavement design programs

Responsible Party - Engineer

Step 6. Gather and calculate the following traffic data as described in Section IIIE “Traffic Analysis” of the Pavement Guide and place results in file.

- Service Life Design ESALs
- Superpave Mix Design ESALs
- Cumulative ESALs to Failure
- Cumulative ESALs Since Last Major Rehabilitation
- ADT and ESAL production report

Responsible Party – Engineer

Step 7. Retrieve construction history, performance data (ride quality, friction, rutting, cracking), and other network level data for project site and place in file.

Responsible Party – Engineer

Step 8. Contact District or Highway Designer to identify if any as-built plans exist that include any adjacent projects. Request any available plans and relevant records. Place information in file folder.

Responsible Party – Engineer

Step 9. Identify any projects designed by OMT from the “File Room Login/Sign Out Book” that include are adjacent to or nearby the project limits. Gather any relevant project data and place in file.

Responsible Party – Engineer

Step 10. Prepare a sketch of the project to identify any potential sections within the project limits. This sketch is intended to communicate variations within the project with other staff and is not intended to be distributed outside OMT, therefore, the sketch should be hand drawn and rough.

Responsible Party - Engineer

Step 11. Conduct site visit of project. Identify location of new roadway and needed soil or roadway borings, special soil testing needs, different pavement design strategies, and unique project characteristics. Document comments and thoughts onto field notes and place in project file.

Responsible Party - Engineer

Step 12. Assemble Records Review data and discuss project with Assistant Pavement Division Chief. The discussion will involve identifying problem areas, special concerns, alternative design pavement strategies, and identifying test setups and specific requests. Any geotechnical testing needs and concerns will be identified and discussed. Testing for new roadways is handled through Geotechnical Explorations Division and their drilling crews and roadway borings.

Responsible Party – Engineer

Step 13. Discuss geotechnical testing needs with geotech contact and agree on testing schedule and data to be gathered from the field.

Responsible Party – Engineer

Step 14. Complete the Field Work Request Form (Form PD-05) for FWD testing of roadways that maybe rehabilitated or used for maintenance of traffic (MOT). Typically, shoulders are used for MOT in roadway relocation or widening projects even if no other portion of the roadway is planned for rehabilitation. Refer to Section III.I "FWD Testing" of the Pavement Guide for guidelines for FWD testing setup. Observations from the site visit will assist in determining testing setup and requirements.

Responsible Party – Engineer

Step 15. Discuss testing set up with SPID, if needed.

Responsible Party – Engineer and SPID

Step 16. Conduct FWD testing following the procedures outlined in Section III.I "FWD Testing", if needed.

Responsible Party – SPID

Step 17. Complete Quality Control of FWD data in field to ensure data is of good quality, checking for sensors out of range and erroneous data outputs from the FWD equipment. Provide data after quality check to engineer once completed.

Responsible Party – SPID

Step 18. Store the FWD data file on the server under the project directory and note on Form PD-06 that the quality checks of the FWD data were conducted following Step 19 in Section III.I "FWD Testing".

Responsible Party – Engineer

Step 19. Conduct steps in Of Section III.J “FWD Data Analysis” to process the raw FWD data and to determine erroneous test points. In addition, the subgrade behavior (linear/ non-linear), the proper number of sensors to use in analysis, the proper load levels to use in analysis, and the proper test locations to use in analysis will be determined in this pre-processing data procedure.

Responsible Party – Engineer

Step 20. Conduct pavement and soil borings and provide data to pavement designer and geotech contact. Place boring log into file.

Responsible Party – Geotech and Drillers

Step 21. Conduct soil testing of boring samples. Place geotechnical soil test data in project file.

Responsible Party – Geotech and Engineer

Step 22. Determine analysis sections of sections of the roadway to be used for MOT by following the steps outlined in Section III.F “Pavement Rehabilitation Design” for evaluating existing structural capacity.

Responsible Party – Engineer

Step 23. Calculate the required structural capacity (SC_f) for future traffic for each new pavement section. SC_f is obtained from Figures 3.1 and 3.7 and the nomograph equations on page II-32 and II-45 in the “AASHTO Guide for Design of Pavement Structures”, for flexible and rigid pavement sections respectively. The following design inputs are required in order to use the nomograph or equation on pages II-32 and II-45:

- *Flexible Pavements:*
  - Design Subgrade Resilient Modulus (M_r) – Obtained from geotechnical soils investigation for new pavement designs. Section X.B “Material Library – Material Properties” includes default values for various types of subgrade materials.
  - Initial Serviceability – See Section VI “Pavement Design Policies”.
  - Terminal Serviceability – See Section VI “Pavement Design Policies”.
  - Reliability – See Section VI “Pavement Design Policies”.
  - Standard Deviation – See Section VI “Pavement Design Policies”.
  - Design ESALs – See Section III.E “Traffic Analysis”

- *Rigid Pavements*:
  - Modulus of Subgrade Reaction (k) – Obtained from geotechnical soils investigation for new pavement designs. Section X.B “Material Library – Material Properties” includes default values for various types of subgrade materials. The modulus of subgrade reaction can also be calculated following the procedures identified in Section 3.2.1

of Chapter II of the "AASHTO Guide for Design of Pavement Structures."

- Initial Serviceability – See Section VI "Pavement Design Policies".
- Terminal Serviceability – See Section VI "Pavement Design Policies".
- Reliability – See Section VI "Pavement Design Policies".
- Standard Deviation – See Section VI "Pavement Design Policies".
- J Factor (J) – Calculate the average load transfer for the design section following Section III.J, "FWD Data Analysis" and select a J Factor as defined in Section VI "Pavement Design Polices"
- PCC Elastic Modulus (E)– Calculate the elastic modulus following Section II.J, "FWD Data Analysis" or select a typical value from Section X.B "Material Library – Material Properties" for concrete materials.
- PCC Modulus of Rupture (S'c)– Calculate the modulus of rupture based on the PCC elastic modulus using the following equation:

  S'c = 43.5(E)+488.5
  where: E = PCC Elastic Modulus (psi/million)

  Modulus of rupture can also be estimated using Section X.B "Material Library – Material Properties
- Drainage Factor (C_d) – Select a drainage factor as defined in Section VI "Pavement Design Policies" based on materials and existing conditions
- Design ESALs – See Section III.E "Traffic Analysis"

Responsible Party – Engineer

Step 24. Identify the individual pavement layers and thickness that will provide the structural capacity to satisfy our traffic demands represented by SC_f. Use the following guidelines to determine the individual pavement layers and thickness:

- *Flexible Pavements: $SC_f = SN_f$*

  $SN_f <= a_1*d_1 + a_2*d_2 + a_id_i$

  where:

  d = layer thickness

  a = layer structural coefficient (Follow guidelines established in Section X.C "Material Library – Design Properties" for each new pavement layer.

- *Rigid Pavements: $SC_f = D_f$*

  $D_f <= d$

  where:

d = PCC layer thickness

Responsible Party – Engineer

Step 25. Consider drainage treatments that would be appropriate improvements or repair strategies to address the source of any potential drainage concerns. Refer to Section VIII.B.5 "Drainage Treatment Guidelines." Geotechnical assistance would be appropriate to determine the location and source of any drainage concerns.

Responsible Party – Engineer

Step 26. Use Section VI "Pavement Design Policies" for assistance in determining specific pavement materials and thickness of lifts to recommend for the new pavement design alternatives.

Responsible Party – Engineer

Step 27. If a jointed rigid pavement (JPCP and JRCP) is considered in design process, joint geometry needs to be designed. Use the following guidelines to design the joint geometry:

- *Joint Spacing:*
  Transverse contraction joints are constructed to control natural temperature related cracking in PCC. The construction of the transverse contraction joints is intended to limit the amount of mid slab cracking by controlling where the cracking occurs. For non-reinforced rigid pavements (JPCP), the joint spacing shall not be greater than 15 feet. A general guideline is that the joint spacing in feet should be twice the slab thickness in inches. For reinforced rigid pavements (JRCP), the joint spacing shall not be greater than 30 feet.

- *Joint Layout:*
  Random joint spacing has been attempted in other states in order to eliminate harmonic induced ride quality problems. However, the benefit of reduced harmonic induced ride quality problems has not been proven to account for the added aggravation and random joint spacing in construction. Therefore, consistently spaced joints shall be used in the design of rigid pavement for MDSHA.

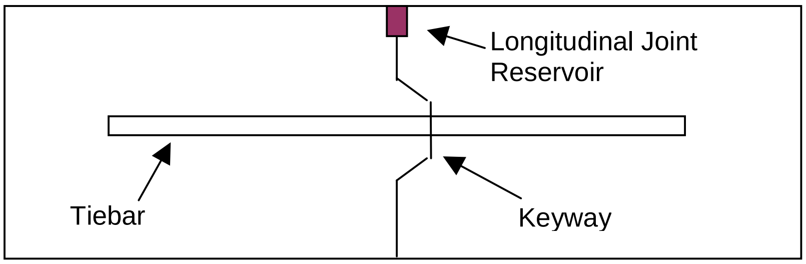
  Skewed joints have also been used by other states. Skewed joints reduce the impact reaction of vehicles and inconclusively have reduced deflection and stress at joints. However, skewed joints pose some repair problems when the joint needs to be patched. Dowels should always be placed parallel to the centerline of the pavement for skewed joints and **not** perpendicular to the angle of the skew.

- *Joint Construction:*
  Joints can be created by sawing, inserts, or forming. The are two general types of joints, transverse and longitudinal. Transverse joints are perpendicular to the travel path of traffic and longitudinal joints are parallel the travel path of traffic. Longitudinal joints tend to be longer than transverse joints because PCC slab are generally rectangular in shape and longer in the longitudinal direction.

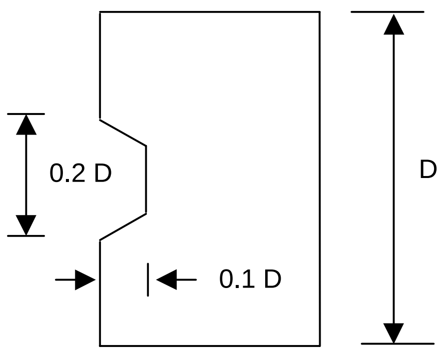
  <u>Transverse Joints</u>: The are two types of transverse joints contraction and expansion. The depth of a transverse joint should be adequate enough to ensure that cracking occurs at the desired location at the joint rather

than in a random pattern. Typically, the depth of the transverse joint should be ¼ of the slab thickness. Timing of sawing of joints is critical to constructing a well performing pavement system. Sawing should be done as soon as possible after adequate compressive strength as been achieved by the PCC slab. Under normal conditions, depending on curing conditions and concrete mix properties, sawing the joints should occur 4 to 12 hours after placement.

Longitudinal Joints: Longitudinal joints are typically formed during PCC placement when lanes are paved in separate passes. Longitudinal joints must also be formed with a keyway. A keyway is a trapezoidal notch and respective tab in adjacent slabs as shown in the figure below:



The keyway shall be located at mid-depth of the slab to provide for maximum strength. Standard dimensions for the keyway are shown in figure below:



Responsible Party – Engineer

Step 28. If jointed (JPCP and JRCP) rigid pavement is considered in design process, steel design and load transfer mechanism design is required. Use the following guidelines to design the steel reinforcement and load transfer mechanism for transverse joints:

- *Load Transfer Mechanism:*
  Unless continuously reinforced concrete pavement (CRCP) is proposed, load transfer mechanisms are required for all rigid pavement designs by MDSHA. Load transfer devices mechanically transfer the load applied to one PCC slab to an adjacent PCC slab. Load transfer devices are placed at the transverse joints between adjacent PCC slabs. Dowel bars are the most widely used type of load transfer device. Typically, one end of the dowel bar is anchored into a PCC slab and the other end of the

dowel bar is lubricated to allow for expansion and contraction of the PCC.

The most commonly used material for dowels is round smooth steel bars, conforming to a Grade 60 or higher. Dowel bar diameter must provide large enough to provide enough mechanical advantage for load transfer, but yet thin enough not to limit the cross section and the structural capacity of the PCC slab. As a guideline, the dowel diameter should be approximately one-eighth (1/8th) that of the slab thickness. The dowel bar length should be adequate to properly distribute the load stresses across slab. Dowel bars are typically 18 inches in length. Dowel bars are typically spaced 12 inches on center and placed 6 inches in from the longitudinal joint. This spacing will provide 12 dowel bars at a transverse joint of a lane 12 feet wide. Dowels are typically coated with epoxy to retard corrosion in the steel. Dowels are placed mid-depth of the PCC slab.

- *Reinforcement Steel Design Inputs:*
  The steel in JRCP consists of either bars or wire mesh that are placed between transverse joints. The steel is to provide addition tension strength to the limited tensile strength of PCC in order to resist mid slab cracking. The longitudinal reinforcement steel is not intended to prevent cracking, but rather to hold the cracks tightly together after their development. Temperature and moisture changes cause volume changes in the PCC. A temperature and moisture decrease will cause the PCC slab to contract and an increase in temperature and moisture will cause the PCC slab to expand. This volume change in combination with the friction and shear resistance of the underlying material creates stresses and strains to the slab in addition to the travelling vehicles. PCC is weak in tensile strength and occasionally with the forces acting upon the PCC in will cause a crack. In this case, the steel reinforcement must carry the tension load without permanent or excessive elongation. Steel reinforcement shall not be placed at a point any higher than mid-slab and any point lower than two-thirds (2/3) into the slab. This range is intended to place the steel in the zone of tension in the slab and also to maintain minimum cover depth around he steel.

  The calculation for steel reinforcement type, size and spacing is based upon steel grade, distance to the closest free edge, slab thickness, friction factor, and the design % of yield strength for the steel. The following are details of these design input material properties.

  Steel Grade: The typical steel grade for reinforcement in Grade 60. Yield strength for Grade 60 steel is 60 ksi.

  Distance to Free Edge: This is defined as the distance from the longitudinal joint of concern to an unsupported PCC slab edge or the slab length for a transverse joint. An edge is considered unsupported if it is not tied to an adjacent lane or shoulder. The outside edge of a tied rigid shoulder is a free edge. The distance to an unsupported longitudinal free edge is needed in determining transverse steel requirements. The slab length is needed when determining longitudinal steel requirements.

  Slab Thickness: Thickness of PCC slab designed previously.

  Friction Factor: The friction factor is a measure of the frictional resistance between the bottom of the slab and the top of the underlying base or subgrade layer. Recommended values for the subgrade and a variety of base materials are provided in Section X "Material Library."

Percent of Yield Strength: To guard against steel fracture and excessive permanent deformation, a limiting stress of 75 percent of the ultimate tensile strength is used in steel design.

- *Steel Reinforcement Requirements:*
  The calculation of the amount of steel required in JRCP, A_s, is based on slab dimensions, steel and other material properties. Both dimensions of the steel requirements need to be estimated, both longitudinal and transverse. The equation is the same to determine both dimensions, the value for the distance to the free edge determines the difference between longitudinal and transverse steel requirements. The amount of steel is designed to carry all the tensile stresses alone without benefit of any PCC tensile strength. Use the following formula to determine the amount of steel requirements:

$$A_s = \left(\frac{W * D * L * F}{2 * f_s} \right)$$

where:

A_s = area of steel (in^2/ft)

W = unit weight of concrete (lb / ft^3) ~ 150 lb/ft^3

D = slab thickness (ft); 9.0" = 0.75 ft

L = distance to closest free edge (ft). For longitudinal steel requirements, this value is the length of the slab. For transverse steel requirements, this value is the distance to an untied longitudinal joint.

F = friction factor

f_s = steel working stress (psi) = Yield strength * % Yield

The steel requirements in JRCP can also be expressed as percent steel as seen in the following equation:

$$P_s = \left(\frac{L * F}{2 * f_s} \right) * 100$$

where:

P_s = percent steel (%)

L = distance to closest free edge (ft). For longitudinal steel requirements, this value is the length of the slab. For transverse steel requirements, this value is the distance to an untied longitudinal joint.

F = friction factor

f_s = steel working stress (psi) = Yield strength * % Yield

The selection of the spacing and size of the steel reinforcement is based on the calculated value for the amount steel required for both dimensions. Refer to the steel bar steel wire mesh properties in Section X "Material Library" for the dimension properties of steel reinforcement.

The selected reinforcement shall have a cross-sectional area greater than the required amount of steel, but not excessively greater. Recommending more reinforcement steel than is necessary is costly and limits the amount of possible cover depth. Typically, steel wire mesh is used as reinforcement for MDSHA. Steel bar reinforcement is typically only used in CRCP and with joint length greater than 40 feet in length. The welded wire mesh can provide for different cross-sectional area in the two different dimensions with different wire diameters and wire spacing.

Responsible Party – Engineer

Step 29. If continuously reinforced rigid (CRCP) pavement is considered in design process, a steel design process unique to CRCP is required to estimate the longitudinal steel requirements. Transverse steel requirements for CRCP shall be designed in the same manner as JRCP. Use the following guidelines to design the longitudinal steel reinforcement for CRCP pavements:

- *Longitudinal Steel Reinforcement Design Inputs:*
  The steel in CRCP consists of reinforcement bars that are designed to provide the PCC with tensile strength when subjected to environmental stresses. Temperature and moisture changes cause volume changes in the PCC. A temperature and moisture decrease will cause the PCC to contract and an increase in temperature and moisture will cause the PCC to expand. This volume change in combination with the friction and shear resistance of the underlying material creates stresses and strains in the PCC in addition to the travelling vehicles. PCC is weak in tensile strength and occasionally with the forces acting upon the PCC in will cause a crack. In this case, the steel reinforcement must carry the tension load without permanent or excessive elongation. Steel reinforcement shall not be placed at a point any higher than mid-slab and any point lower than two-thirds (2/3) into the slab. This range is intended to place the steel in the zone of tension in the slab and also to maintain minimum cover depth around he steel.

  CRCP derives its name, “continuous”, from the longitudinal steel that is placed continuously along the length of the PCC lane. CRCP does not have any traditional contraction transverse joints, only an occasional terminal joint placed at the end of day of paving. The longitudinal steel is not intended to prevent transverse cracks, but rather to hold those cracks tightly together. In fact transverse cracking is desired in CRCP and the spacing is designed to ensure critical stresses in the PCC are not reached. Unlike transverse cracking, longitudinal cracking is not desired in CRCP. Transverse steel is provided in CRCP to prevent longitudinal cracking from developing. CRCP transverse steel reinforcement design is identical to the steel design for jointed PCC. The longitudinal steel requirements for CRCP is discussed in the remaining portion of this step.

  The following are details of the CRCP steel reinforcement design input material properties.

  <u>PCC Indirect Tensile Strength</u>: The indirect tensile strength is tensile strength a measurement of the PCC measured by applying a load along the longitudinal axis of a specimen. Both modulus of rupture and indirect tensile strength our measurements of the tensile strength of PCC.

Typical values for indirect tensile strength are provided in Section X "Material Library."

PCC Shrinkage Factor: This PCC shrinkage factor represents the volume reduction in a PCC mix during curing. The amount of water loss in a concrete mix dictates the shrinkage factor amount. Typical values for the PCC shrinkage factor are provided in Section X "Material Library."

PCC Thermal Coefficient: The PCC thermal coefficient represents the volume change in PCC has a result of changing environmental conditions. Typical values for PCC thermal coefficients are provided in Section X "Material Library."

Reinforcing Bar Diameter: The diameter of the bar used for reinforcement. Typical values for bar diameter are provided in Section X "Material Library."

Reinforcing Bar Thermal Coefficient: The reinforcing bar thermal coefficient represents the volume change in the steel has a result of changing environmental conditions. Typical values for reinforcing bar thermal coefficient are provided in Section X "Material Library."

Design Temperature Drop: The design temperature drop is the difference between the average concrete curing temperature and a design minimum temperature. The average concrete curing temperature may be taken as the average daily high temperature during the month the pavement is expected to be constructed. The design minimum temperature is defined as the average daily low temperature for the coldest month during the pavement life.

Wheel Load Stress: The wheel load stress is the tensile stress developed during initial loading of the constructed pavement. Input factors required to calculate the wheel load stress include the slab thickness, modulus of subgrade reaction, and anticipated wheel load magnitude. The actual wheel load stress can be estimated using the graph presented on page II-55 in the "1993 AASHTO Guide for Design of Pavement Structures."

- *Longitudinal Steel Reinforcement Limiting Criteria:*
  In addition to the design inputs required of longitudinal reinforcing steel, there are three limiting criteria which must be considered: crack spacing, crack width, and steel stress. The limiting criteria are to ensure that neither minimum nor maximum ranges of steel requirements are not violated.

  The following are details of the CRCP steel reinforcement design limiting criteria.

  Crack Spacing: Limiting criteria is placed on crack spacing to limit the potential for spalling and punchouts. To minimize the potential for crack spalling, the maximum spacing between consecutive cracks should be no more than 8 feet. To minimize the potential for the development of punchouts, the minimum desirable crack spacing should be less than 3.5 feet.

  Crack Width: Limiting criteria is placed on crack width to limit the potential for water penetration into the pavement structure and crack spalling. The allowable crack width should not exceed 0.04 inches. Ideally, the predicted crack width should be reduced as much as possible through selection of a higher steel percentage or smaller reinforcing bar diameter.

Steel Stress: Limiting criteria is placed on steel stress to prevent steel fracture and excessive permanent deformation. This limit is established to be 75% of the ultimate tensile strength of the steel.

- *Longitudinal Steel Reinforcement Requirements:*
The calculation of the percent amount of steel required in CRCP, P, is based on the input factors and limiting criteria described above. There are three nomographs in the "1993 AASHTO Guide for Design of Pavement Structures" on pages II-57 through II-59 based on the three limiting criteria that produce the percent of steel required to adequately reinforce the CRCP. The nomographs are derived from the design inputs discussed earlier and customized for each limiting criterion. Each nomograph provides the recommended longitudinal steel percentage based on each limiting criterion. There are several computer software applications available to the Pavement Division that are capable of performing the calculations of the nomographs in electronic form. Once the percent of steel is determined from the limiting criteria, the steel reinforcement specifications can be developed.

The following are the required design steps for the longitudinal steel reinforcement in CRCP. These steps can be completed in the computer software applications available to the Pavement Division.

Step A: Solve for the required longitudinal steel reinforcement (P) to satisfy the limiting criterion using the design charts on pages II-57 through II-59 in the "1993 AASHTO Guide for Design of Pavement Structures."

Step B: If the P based on the crack spacing criterion for the maximum crack spacing (8 feet) is greater than any other limiting criterion P, then adjustments must be made to the design inputs. This step is necessary because the if the P needed for 8 feet crack spacing is greater than another P for a different criterion, then the input parameters used in the design will not provide the desired reinforcement necessary for an adequate pavement design. Therefore, the design inputs must be altered in order for the steel reinforcement to provide the needed tensile strength. Check for data entry errors prior to altering design inputs. After checking for data entry errors, the first iteration is to alter reinforcing bar diameters.

Step C: Determine the range in the number of reinforcing bars needed in the CRCP. Using the following equations to determine that range:

$$N_{\min} = 0.01273 \times P_{\min} \times W_s \times D\phi^2$$

$$N_{\max} = 0.01273 \times P_{\max} \times W_s \times D\phi^2$$

where:

N_{min} = minimum required number of reinforcing bars

N_{max} = maximum required number of reinforcing bars

P_{min} = minimum required percent steel

P_{max} = maximum required percent steel

W_s = total width of pavement section (inches)

D = slab thickness (ft); 9.0" = 0.75 ft

ϕ = diameter of reinforcing bar (inches)

Step D: Select the final steel design by selecting the actual total number of bars (whole number) to be placed in the CRCP, which should be between N_{min} and N_{max}.

Responsible Party – Engineer

Step 30. If any rigid pavement is considered in design process, tie bar steel design is required for longitudinal joints. Use the following guidelines to design the tie bar steel reinforcement for longitudinal joints:

- *Tie Bar Design Inputs:*
  Tie bars are deformed steel bars placed along longitudinal joints to hold together adjacent slabs and provide a load transfer mechanism across longitudinal joints. Tie bars are used to tie adjacent lanes together and rigid shoulders. The spacing of tie bars is significantly greater than dowel bars used in transverse joints because the traffic crosses longitudinal joints far less frequently than transverse joints. In addition, tie bars are used in combination with a keyway to provide additional load transfer capabilities.

  There are three basic methods for tie bar placement for longitudinal joints. The most common method is to place 90 degree bent tie bars into the key during paving. The tie bars are then straightened before paving adjacent lanes. Another method is to drill holes into the keyway after paving and inserting straight tie bars into the keyway and securing with an epoxy grout. The final method uses a two-part threaded tie bar in which the female end is inserted into the keyway during paving. The threaded portion of the tie bar is then placed prior to paving the adjacent lane.

  The calculation for tie bar size and spacing is based upon steel grade, distance to the closest free edge, slab thickness, friction factor, and the design % of yield strength for the steel. The following are details of these design input material properties.

  Steel Grade: The typical steel grade for tie bars in Grade 40. Yield strength for Grade 40 steel is 40 ksi.

  Distance to Free Edge: This is defined as the distance from the longitudinal joint of concern to an unsupported PCC slab edge. A edge is considered unsupported if it is not tied to an adjacent lane or shoulder. The outside edge of a tied rigid shoulder is a free edge.

  Slab Thickness: Thickness of PCC slab.

  Friction Factor: The friction factor is a measure of the frictional resistance between the bottom of the slab and the top of the underlying base or subgrade layer. Recommended values for the subgrade and a variety of base materials are provided in Section X ”Material Library.”

  Percent of Yield Strength: To guard against steel fracture and excessive permanent deformation, a limiting stress of 75 percent of the ultimate tensile strength is used in steel design.

  Bar Diameter: The bar diameter is the dimension of interest for steel design of rigid pavement. Tie bars are typically #4 (0.5” diameter) or #5 (0.625” diameter) size bars. Recommended and common sizes of bars and wires are provided in Section X ”Material Library.”

- *Tie Bar Spacing:*
  There is a formula equation to obtain tie bar spacing given the material properties, but maximum and minimum recommended spacing in addition to engineering judgement should be strongly considered. Typical tie bar spacing is 30 to 40 inches. The maximum allowable tie bar spacing is 48 inches. The calculation of the tie bar spacing requires the calculation of an area of steel, A_s, required to support the slab dimensions given the steel and other material properties. Use the following equation to calculate A_s:

$$A_s = \left(\frac{W * D * L * F}{f_s} \right)$$

where:

A_s = area of steel (in^2/ft)

W = unit weight of concrete (lb / ft^3) ~ 150 lb/ft^3

D = slab thickness (ft); 9.0" = 0.75 ft

L = distance to closest free edge (ft)

F = friction factor

f_s = steel working stress (psi) = Yield strength * % Yield

Once the area of steel for tie bars is determined, the spacing is then determined by dividing the cross-sectional area of the selected tie bar by the area of steel required. An example of this equation is the following:

#4 tie bars = 0.2 in^2

A_s = 0.0675 in^2/ft

Spacing = 0.2 in^2 / 0.0675 in^2/ft = 2.96 ft = 36 in

- *Tie Bar Length:*
  The length of the tie bar is governed by the allowable bond stress between the PCC and the steel. For deformed steel, 350 psi can be assumed for the bond stress. There is an formula equation to obtain tie bar length given the PCC and steel material properties, but in addition to engineering judgement should be strongly considered. A typical tie bar length is 36 inches. The length of the tie bar is calculated by the following formula:

$$t = 0.5 * \left(\frac{f_s * d}{\mu} \right) + 3$$

where:

t = tie bar length (in)

d = bar diameter (in)

μ = bond stress (psi) ~ 350 psi

Responsible Party – Engineer

Step 31. The new pavement design shall be checked for constructability issues. For base widening and new pavement designs, care shall be taken to ensure that the new section does not effect the performance of the existing roadway; i.e. drainage problems from a "bathtub" effect or poor construction joint from different pavement types, etc. In addition, care shall be taken to ensure any pavement rehabilitation design is used in design combination and does not conflict with any new pavement design on the project. For new construction or reconstruction, care shall be taken to ensure that the pavement design can be constructed and perform well over the pavement service life; geometric and grade demands, MOT constraints, regional material constraints, etc.

Responsible Party – Engineer

Step 32. Develop a cost estimate for the project for each new pavement design based on the pavement item quantities. Refer to Section XI "Unit Costs" for information regarding the typical cost of items.

Responsible Party – Engineer

Step 33. Complete PD-10 Rehabilitation Alternative form. Include all the inputs applicable for a new design project. Some areas will be left blank because they are intended for pavement rehabilitation design. Place form into project file for discussion with the Assistant Pavement Division Chief and/or Pavement Chief at a later time.

Responsible Party – Engineer

Step 34. Discuss new pavement design alternative with Assistant Pavement Division Chief. If the project costs exceeds $20 million or if project is deemed to be a major corridor project by Pavement Division Chief, then a life cycle cost analysis, as outlined in Section III.H "Pavement Type Selection", shall be performed to select a preferred alternative.

Responsible Party – Engineer

Step 35. Discuss new pavement design alternatives with project owner and appropriate technical teams to select final design alternative. The discussion with the project owner should involve input from Construction Division regarding constructability issues with each alternative. The technical material team should provide input for special material requirements in the project.

Responsible Party – Engineer

Step 36. Prepare a memorandum using the typical pavement recommendation memorandum format (n:omr\everyone\pavedsgn\memoformat.doc). Complete the memorandum documenting the selected new pavement design alternative.

Responsible Party – Engineer

Step 37. Gather the appropriate Special Provisions and Detail Specifications to be attached to the recommendation memorandum. Refer to Section VII "Pavement Design Standards" and Section IX "Special Provisions" for appropriate attachments. Also, the engineer should be familiar with the appropriate Special Provisions Inserts that need to be included in the contract documents.

Responsible Party – Engineer

Step 38. Complete the appropriate portions for new pavement design of the Pavement Rehabilitation Design QC Form (PD-06). Complete Project Summary Information form (PD-09). Place forms in project file. Submit project file and memorandum with attachments to Assistant Pavement Division Chief for review of design process and other QC checks.

Responsible Party – Engineer

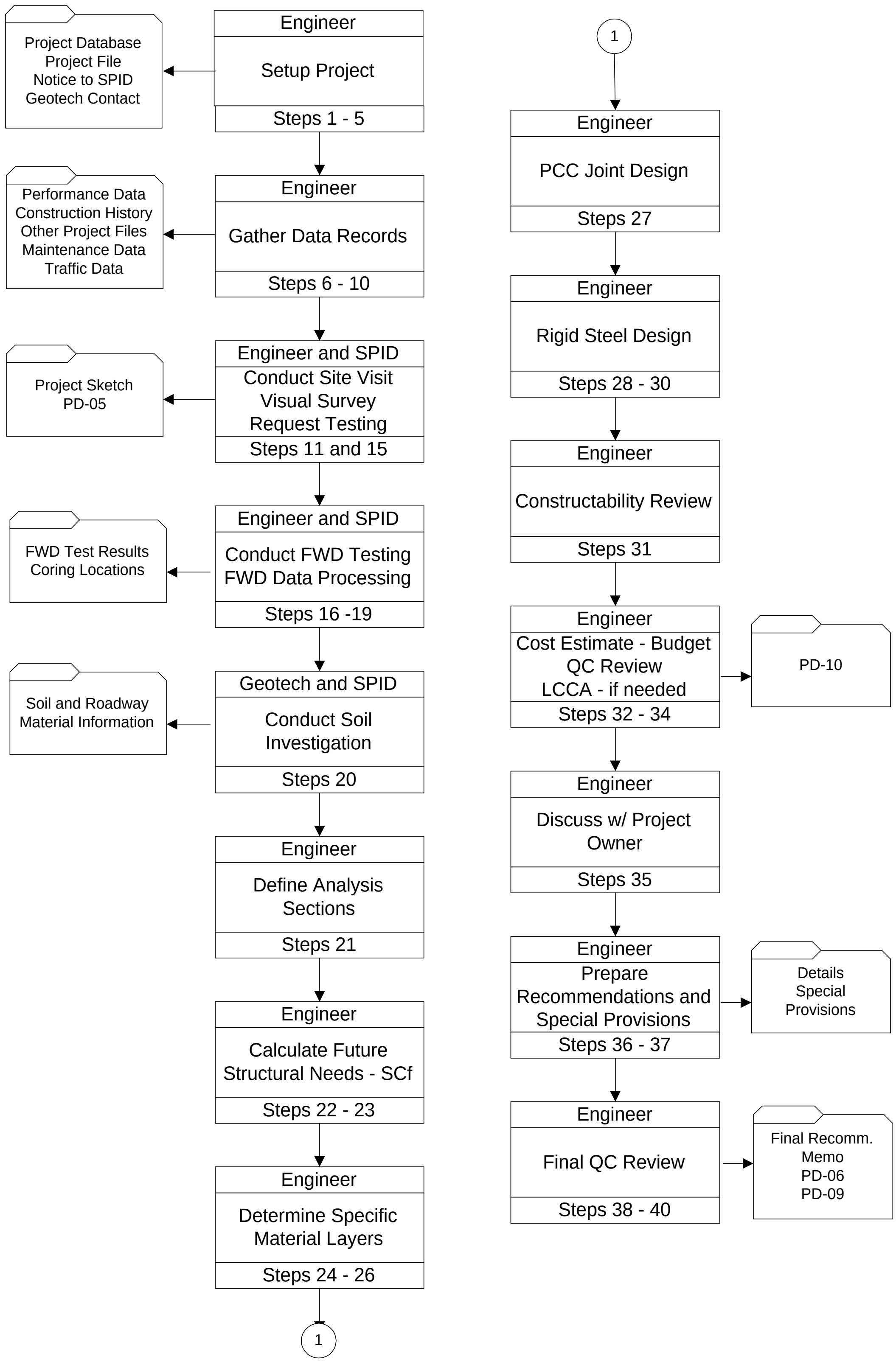
Step 39. Review design process and analysis for accuracy and quality of work. Complete the Pavement Rehabilitation Design QC Form (PD-06). Submit project file and memorandum with attachments back to Engineer with any comments or notes.

Responsible Party – Assistant Pavement Division Chief

Step 40. Make necessary edits and corrections to new pavement design and memorandum. Submit project file and memorandum on letterhead with attachments/specifications to Pavement Chief for final review.

Responsible Party – Engineer

III.G.4 New Pavement Design Process Flowchart



III.H LIFE CYCLE COST ANALYSIS (LCCA)

MDSHA Pavement Division currently uses a probabilistic approach to life cycle cost analysis rather than the traditional deterministic approach. This approach allows for a range of construction and material cost to be used in the analysis rather than a discrete unit cost. The results of this approach provide the pavement engineer the probability of user and agency costs occurring over the analysis period. The probabilistic approach eliminates arguments over unit cost because a reasonable range is used that encompasses variations in the economy and construction costs.

In addition, MDSHA has put a tremendous amount of effort towards developing and calculating user costs involved in construction and rehabilitation projects. Various user delay costs are calculated to develop the overall user cost. These items include, but are not limited to the following: reduced speed costs, slow down costs, queuing costs, idling costs, and acceleration costs. MDSHA has worked closely with FHWA to develop a LCCA approach that uses a probabilistic approach and incorporates detailed user delay costs.

The current LCCA procedure is undocumented at this time. When the procedure has identified steps and required parameters, it shall be incorporated into the MDSHA Pavement Design Guide.

III.I FWD TESTING

III.I.1 Purpose

FWD Testing is conducted to:

- Collect data to assess the structural capacity of pavement structures
- Collect data to determine the soil and subgrade strength
- Collect data to estimate the structural capacity of the pavement structure and the material properties of the individual pavement layers
- Collect data to estimate the load transfer ability of joints in jointed rigid and composite pavements
- Determine statistically different performing pavement sections and sub-sections

III.I.2 Resource Requirements

The FWD testing procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Establish Project Features	1	1
Staff Engineer or Project Engineer	FWD Testing Set-Up	1	2
Project Engineer/ SPID	Testing Communication w/ SPID	2	1
SPID	FWD Testing	1	12*
Staff Engineer or Project Engineer	QC FWD Data	1	2

* This value is dependent upon project size. This is based on 1-day of testing or 150 points/day.

III.I.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to complete FWD Testing. FWD testing shall be required on all projects requiring pavement recommendations unless specified otherwise in this Pavement Guide or as directed by the Assistant Pavement Division Chief or Pavement Division Chief. The procedure described below is conducted within the pavement rehabilitation design process. Certain steps in the pavement rehabilitation design process and the FWD testing process overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations.

There are several assumptions that are made in the development of this process and the values used to establish FWD testing set-ups. The most important item that pavement designers need to be aware of is that is was assumed that the FWD equipment and operator could collect approximately 150 test points a day. This is based on limited travel time, no constricting MOT demands, and no equipment failures. This is the testing schedule that should be used when estimating the testing time demands needed for a particular project. Depending on the size and geometry of some projects, 150 points may be an excessive amount of data; i.e. a short turn lane project. In addition, the testing demands of the FWD occasionally become heavy and the amount of testing for a particular project may need to be reduced in order to provide FWD data to all the projects in the state within a reasonable time. In this case, communication between

SPID and the pavement engineer need to take place to ensure adequate data and time constraints are addressed for all interested parties.

Step 1. Determine the pavement type variations within the project limits. A separate test set-up needs to be established for each pavement type within the project. At this point, the records review and site visit should have provided sufficient information to determine the pavement type variation within a project. Information collected in Forms PD-04, for each project section, will also provide additional refresher information about the pavement types.

Responsible Party - Engineer

Step 2. Determine the project layout and attributes. The project layout will influence the FWD test set-up and pattern of testing. The project sketch developed in the pavement rehabilitation design process to be used in discussions with Assistant Pavement Division Chief will be a useful tool. All state routes, intersections, turn lanes, ramps, service roads, and shoulders within the project limits should be identified and a decision made if FWD testing is required. A well-defined scope is required to make decisions about testing requirements for various project attributes. Discussion with the Assistant Pavement Division Chief maybe needed, if necessary.

Responsible Party - Engineer

Step 3. Determine the size (length and lanes) of each project attribute (mainline, shoulders, ramps, etc.) to receive FWD testing. This will provide the time demands on the FWD for this particular project. These demands are based on the assumptions stated above about the FWD test pace being approximately 150 test points per day.

Responsible Party - Engineer

Step 4. Determine the project geometry. The project geometry will influence the FWD pattern of testing. For example, numerous intersections, closely spaced intersections, and single lane ramps may limit the amount of testing that can be feasibly done because on the demands of the travelling public and MOT. These MOT concerns should be noted and discussed with SPID after the request is provided.

Responsible Party - Engineer

Step 5. Determine FWD test sections. Use the information gathered in Steps 1 through 4 above to determine FWD test sections. A separate FWD test sections should be established for different pavement types and project attributes for a project. Section lengths and project geometry will dictate FWD test set-up in a FWD test section.

Responsible Party - Engineer

Step 6. Determine FWD test type for each section. There are three basic types of FWD tests: basin, joint, corner. Basin testing is used to evaluate the structural strength of the pavement structure and subgrade in flexible and rigid pavements. The primary use of basin testing in composite pavements is to establish a HMA compression or bending factor. Basin testing can also be used to produce a general estimate of the structural strength of the pavement structure and subgrade in composite pavements. Specific structural characteristics of composite pavement are difficult to obtain with a great deal of confidence with the current backcalculation analysis methods available today.

Basin testing can also be done on subgrade and base materials. Basin testing on subgrade or unbound base materials should be completed with the larger load plate (radius of 9", 230 mm) to prevent damage to the underlying layers. All other testing should be done with the smaller load plate (radius of 6", 150 mm).

Joint testing is used to assess the performance of existing joints to transfer loads across slabs in rigid and composite pavements. In addition, joint testing can be used to determine the presence of voids under the slabs at the joints. Corner testing is used to assess the performance of existing corners (most highly stressed area of the slab) to transfer loads across slab corners in rigid pavements. In addition, corner testing can be used to determine the presence of voids under the corners of the slab. Use the table below to determine the appropriate FWD test type for a given section. Figure 2, 3, and 4 present the three FWD test types for the various pavement types.

Pavement Type	Test Type	Load Plate Type	Test Point Location in Lane
Flexible	Basin	Small (6" radius)	Right Wheel Path
Rigid	Basin	Small (6" radius)	Right Wheel Path / Mid slab
	Joint	Small (6" radius)	Right Wheel Path / At joint
	Corner	Small (6" radius)	Corner of Slab / At joint
Composite	Basin	Small (6" radius)	Right Wheel Path / Mid slab
	Joint	Small (6" radius)	Right Wheel Path / At joint
Subgrade/Unbound Base	Basin	Large (9" radius)	Right Wheel Path

Responsible Party – Engineer

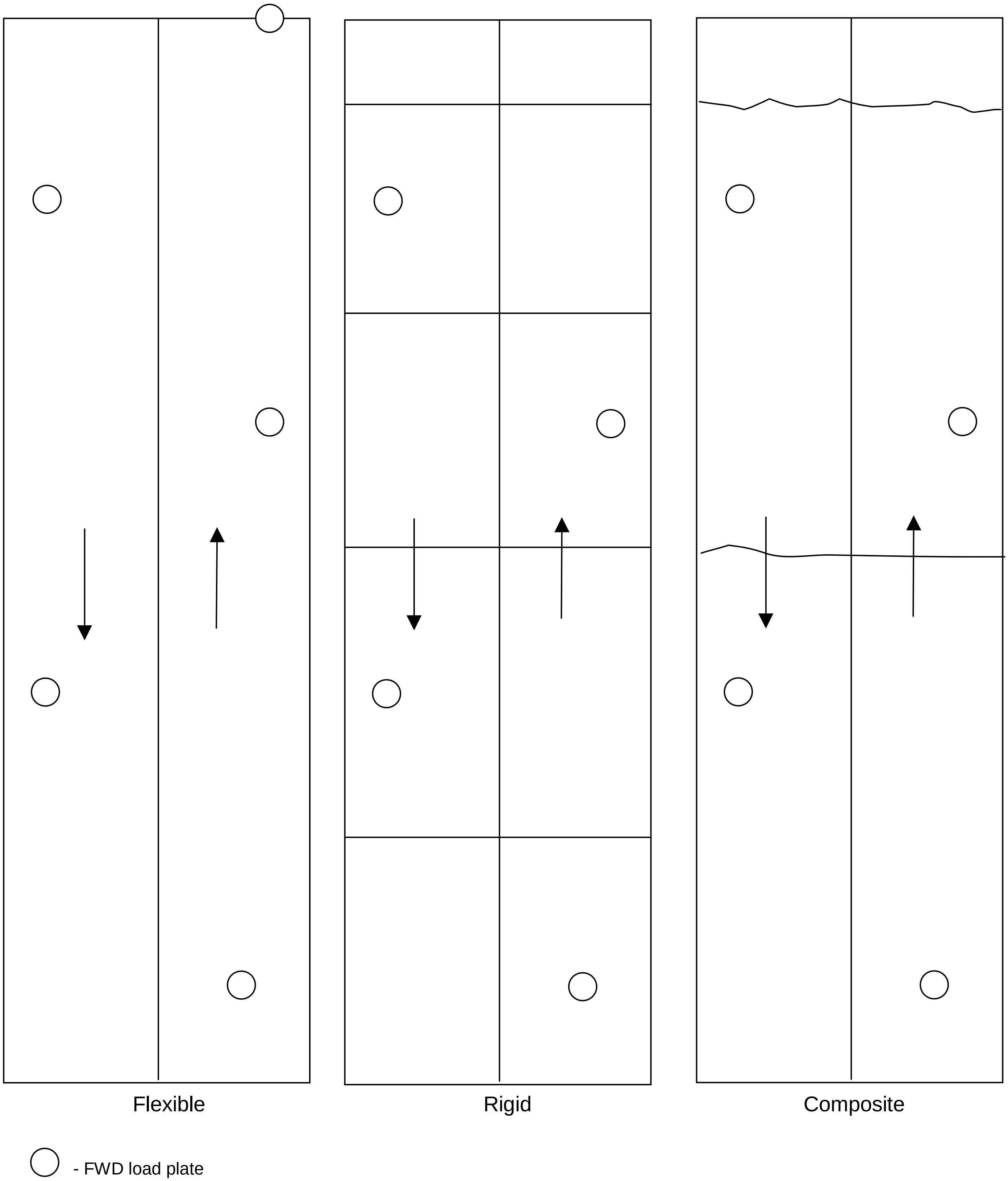
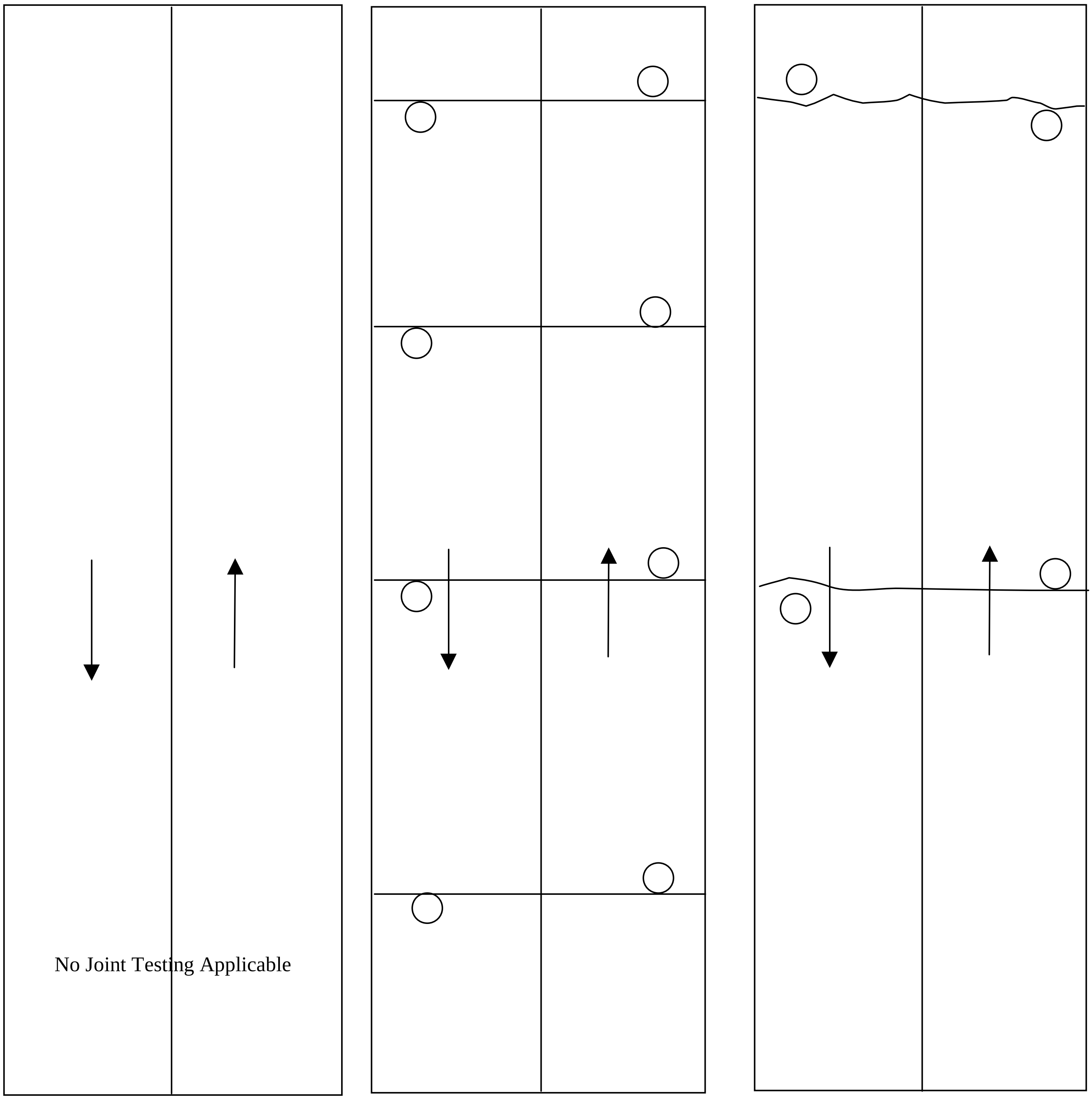


Figure 2 – FWD Basin Testing



Flexible

Rigid

Composite

No Joint Testing Applicable

- FWD load plate

Figure 3 – Joint Testing

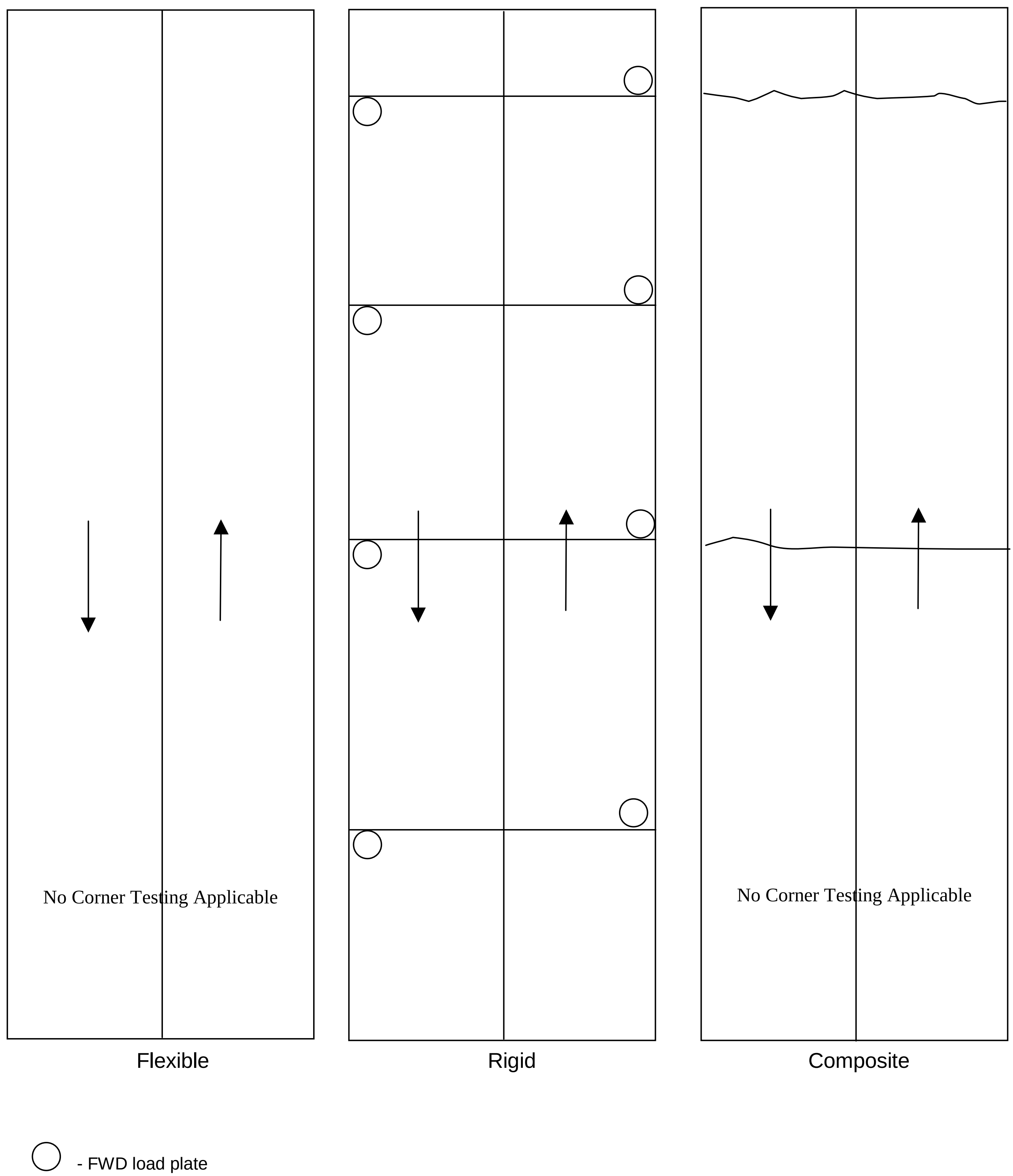


Figure 4 – Corner Testing

Step 7. Determine sensor spacing of geophones on the FWD. There are four basic sensor spacing configurations, two for basin testing and two for joint/corner testing. In basin testing, the critical locations for the sensors are on and next to the load plate and those furthest away from the load plate. The sensor on the load plate provides layer material characteristic information about the entire pavement structure. The further away you move from the load plate those sensors provide information about pavement layers deeper in the pavement structure. The outer most sensors provide information about the subgrade performance. One of the two basin test sensor spacing configurations shown below is for typical basin testing, the other is for basin testing on subgrade or unbound base layers.

There are only two critical sensors locations for joint testing. Those critical locations are the sensors on either side of the joint. If only joint testing is required without basin testing, then one sensor can be placed at (–12”) and the other at 0”. If basin testing is to be completed in combination with joint testing, the typical basin testing sensor spacing configuration can be modified to allow for both testing in a single pass of the FWD by moving the sensor at 8” to 18”. An example of both of the joint testing sensor spacings is presented in Figure 5a/b and Figure 6a/b.

The table below lists the four basic sensor spacing configurations. This table is based on a FWD configuration with 7 sensors. The latest Dynatest FWD equipment configuration has nine sensors. If MDSHA upgrades its FWD equipment the sensor configuration will be changed to reflect the additional two sensors.

Test Type	Sensor Spacing
Basin	0”, 8”, 12”, 24”, 36”, 48”, 60”
Basin/Subgrade	0”, 12”, 18”, 24”, 36”, 48”, 60”
Joint	-12”, 0”, 12”, 24”, 36”, 48”, 60”
Joint/Basin	0”, 12”, 18”, 24”, 36”, 48”, 60”

The sensor at 60” in basin testing is used to determine the subgrade characteristics beneath the pavement structure. If an unusually thick pavement structure is existing, moving this sensor out to 72” is appropriate. An unusually thick pavement structure would be defined as a flexible pavement with > 16.0” of HMA, a rigid pavement with greater than 11.0” of PCC, or a composite pavement with a combination of thickness greater than 14.0”.

Responsible Party - Engineer

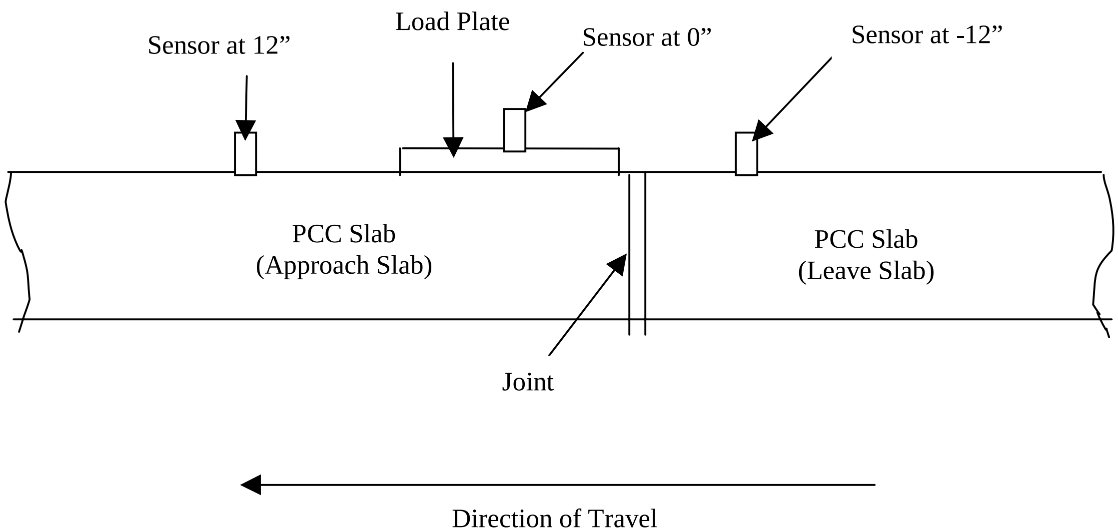


Figure 5a : Joint Load Transfer Testing Sensor Spacing ID #3

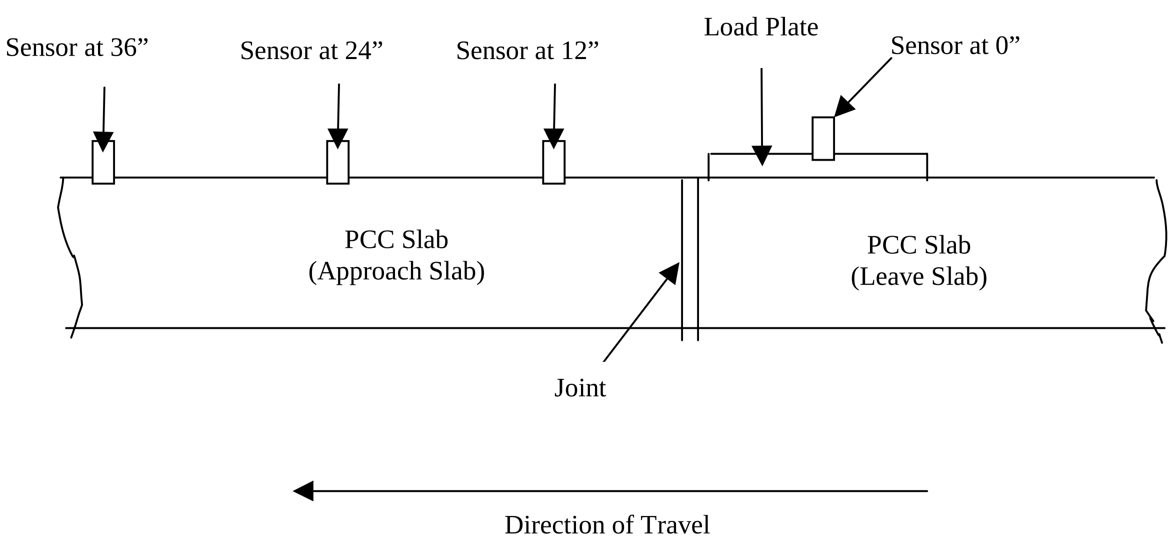


Figure 5b : Joint Load Transfer Testing Sensor Spacing ID #3

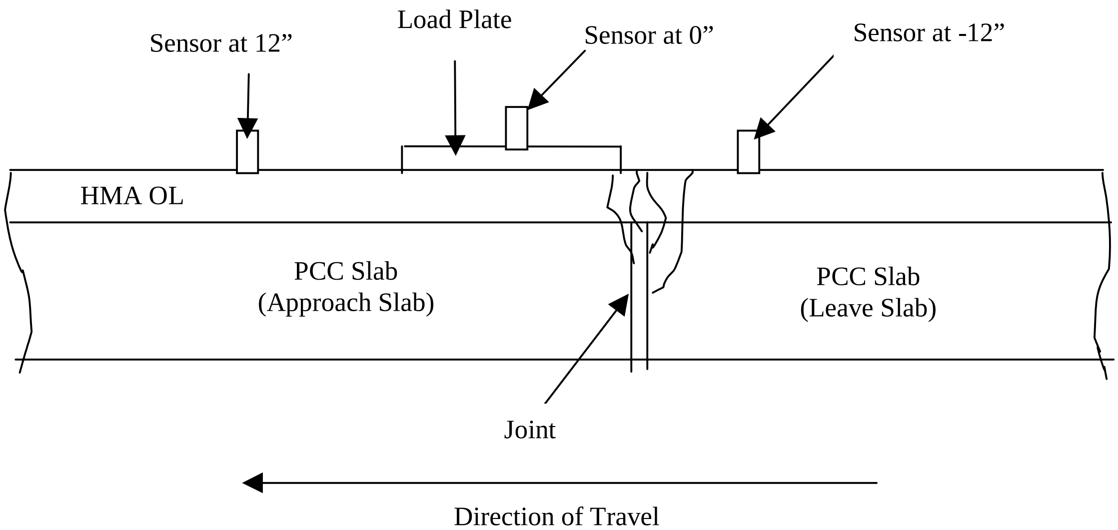


Figure 6a : Joint Load Transfer Testing Sensor Spacing ID #7

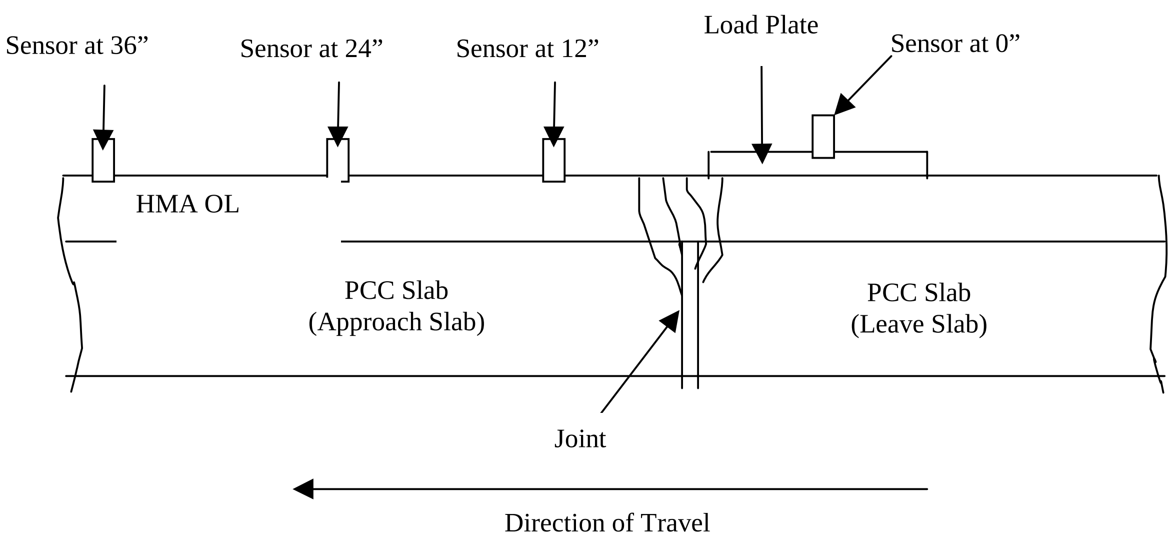


Figure 6b : Joint Load Transfer Testing Sensor Spacing ID #8

Step 8. Determine the FWD Test Set-Up Identification number. This selection is based on the results from Step 6 and 7. The ID number will be used in the Field Work Request Form PD-05. This FWD Test Set-Up ID number will provide information concerning the test type, pavement type, and sensor spacing. Use the table below to select the proper FWD Test Set-Up ID number.

Test Set-Up ID	Pavement Type	Test Type	Sensor Spacing
1	Flexible	Basin	0”, 8”, 12”, 24”, 36”, 48”, 60”
2	Rigid	Basin	0”, 8”, 12”, 24”, 36”, 48”, 60”
3	Rigid	Joint	-12”, 0”
4	Rigid	Joint	0”, 12”
5	Rigid	Corner	-12”, 0”, 12”, 24”, 36”, 48”, 60”
6	Composite	Basin	0”, 8”, 12”, 24”, 36”, 48”, 60”
7	Composite	Joint	-12”, 0”
8	Composite	Joint/Basin	0”, 12”, 18”, 24”, 36”, 48”, 60”
9	Subgrade	Basin	0”, 18”, 12”, 24”, 36”, 48”, 60”

Responsible Party - Engineer

Step 9. Determine the FWD load package and drop sequence identification number. This decision will dictate the force of the load applied to the pavement, the number of drops, the number of seating drops, and the sequence of the drops. A seating drop is a load applied to the pavement to seat the load plate, perform a quality check on the sensors and load cell, and to warm the buffers beneath the weights on the FWD properly. No deflection or load data is recorded during a seating drop. It is typical to have at least one seating drop prior to any testing at a new test point and one seating drop prior to each drop height change. The height the weights of the FWD are raised dictate the amount of load applied to the pavement. The are four basic drop heights known as 1, 2, 3, and 4. These heights typically result in a load applied to the pavement of 6000 lbs., 9000 lbs., 12000 lbs., or 18000 lbs. The seating drops are done at the same drop heights, but are know as A, B, C, and D. Therefore a load package and drop sequence written as “BC3D4” would result in the following:

- a seating drop at 9000 lbs. for a new test point,
- a seating drop at 12000 lbs. for a new drop height,
- a recorded drop at 12000 lbs.,
- a seating drop at 18000 lbs. for a new drop height, and
- a recorded drop at 18000 lbs.

The pavement type and thickness and FWD test type dictate the load package. Use the table below to select the proper load package and drop sequence for your particular section within a project. Additional drops and sequences can be selected, the list in the table below is only a typical range of acceptable load packages and sequences.

Load Package ID	Pavement Type	HMA Thickness	Test Type	Load Package and Sequence
1	Flexible	< 4”	Basin	BA1B2
2	Flexible	> 4” and < 8”	Basin	BB2C3
3	Flexible	> 8”	Basin	BB2D4
4	Rigid	N/A	Basin	BC3D4
5	Rigid	N/A	Joint	BB2C3D4
6	Composite	N/A	Basin	BC3D4
7	Composite	N/A	Joint	BB2C3D4
8	Composite	N/A	Joint/Basin	BB2C3D4
9*	Subgrade	N/A	Basin	AA1B2

Note: Every effort should be made to ensure that at least 18,000 lbs. is achieved at drop height 4 for rigid and composite pavements. In some cases, exceptionally thick or strong rigid or composite pavements, it may be necessary to apply greater than 18,000 lbs. to the weight by adding additional weight to the FWD.

* The weights may need to be removed completely if FWD testing is to be completed on exceptionally weak soils or subgrade.

Responsible Party - Engineer

Step 10. Determine the test point spacing for each FWD test section. The length of the project, pavement type, and FWD test type dictates the test point spacing. The basic goal of SPID and the FWD team is to collect 150 test points per day. Our test spacing should be based on a test collection pace of 150 test points per day and the assumption that on average 1 project is completed per work-day. The table below shows general guidelines for how those 150 test points should be distributed among the different FWD test types.

	Pumping Evident			No Pumping		
Pavement Type	**Basin**	**Joint**	**Corner**	**Basin**	**Joint**	**Corner**
Flexible	150	N/A	N/A	N/A	N/A	N/A
Composite	25	125	N/A	50	100	N/A
Rigid	25	75	50	50	50	50

A typical point test spacing for a two lane undivided roadway is provided in the table below. This is to be used as a guideline, but numerous other engineering, FWD testing, MOT, and schedule factors may alter the test point spacing. The typical test point location is shown in Figure 2, 3, and 4. Testing done in both directions should be staggered as shown in figures 2 through 4 to ensure a more efficient coverage of the project length. The test point spacing in the table below is based on completing all FWD testing in one workday.

		Test Point Spacing				
		Flexible	Composite		Rigid	
Project Length (Centerline)	Test Type		No Pumping Evident	Pumping Evident	No Pumping Evident	Pumping Evident
3 miles	Basin	250’	640’ / 16th slab	1280’ / 32nd slab	640’ / 16th slab	1280’ / 32nd slab
2 miles	Basin	175’	400’ / 10th slab	840’ / 21st slab	400’ / 10th slab	840’ / 21st slab
1 mile	Basin	100’	200’ / 5th slab	400’ / 10th slab	200’ / 5th slab	400’ / 10th slab
< 1 mile	Basin	50’	160’ / 4th slab	200’ / 5th slab	160’ / 4th slab	200’ / 5th slab
3 miles	Joint	N/A	280’ / 7th slab	240’ / 6th slab	640’ / 16th slab	400’ / 10th slab
2 miles	Joint	N/A	200’ / 5th slab	160’ / 4th slab	400’ / 10th slab	280’ / 7th slab
1 mile	Joint	N/A	120’ / 3rd slab	80’ / 2nd slab	200’ / 5th slab	120’ / 3rd slab
< 1 mile	Joint	N/A	80’ / 2nd slab	40’ / every slab	160’ / 4th slab	80’ / 2nd slab
3 miles	Corner	N/A	N/A	N/A	640’ / 16th slab	640’ / 16th slab
2 miles	Corner	N/A	N/A	N/A	400’ / 10th slab	400’ / 10th slab
1 mile	Corner	N/A	N/A	N/A	200’ / 5th slab	200’ / 5th slab
< 1 mile	Corner	N/A	N/A	N/A	160’ / 4th slab	160’ / 4th slab

At minimum, at least 30 test points should be collected per direction for any project of reasonable length. Thirty test points in one direction provide the minimal amount of information about the stretch of roadway to identify weak/strong areas and adequately characterize the structural capacity of the pavement. Projects longer than 3 centerline miles should be considered more than a more day operation for FWD testing.

Projects containing several different sections because of different pavement types should have testing broken down according to the percentage of each pavement type. Therefore, if half the project is composite and half is flexible, a one-day test point spacing should be 50% of the test point spacing typically done for each pavement type.

Responsible Party - Engineer

Step 11. Complete the Field Work Request Form (PD-05) using the information gathered in Steps 1 through 10 of this section, FWD Testing. The work completed Step 8 provided the information for FWD test set-up, Step 9 provided information for the load package, and Step 10 provided information about the test point spacing. On Form PD-05 complete the date of the request and the requested completion date for the FWD testing. Once the form is filled out, give the request form to SPID.

Responsible Party - Engineer

Step 12. Schedule FWD testing and MOT for testing for the submitted request. Every attempt should to schedule the testing in order to have it completed by the requested completion date.

Responsible Party - SPID

Step 13. The pavement design engineer and SPID should meet and discuss the testing to ensure that all facets of the FWD testing are understood. Specific details about the project, testing, and other concerns can be addressed at this time.

Responsible Party – SPID and Engineer

Step 14. Set-up the Dynatest FWD software for the FWD testing of the specific section in the project. Use the following criteria when establishing the software test set-up:

- Always store the length measurement in the FWD software in feet/stations, not miles, unless requested otherwise.
- Store FWD test type in the Lane field in the software, in addition to the lane number; i.e. using “B” for basin, “J” for joint testing, and “C” for corner testing. For instance, joint testing in lane 1 would have “J1” in the lane field and basin testing in lane 2 would have “B2” in the lane field. Testing in the shoulder will not have a number, but rather a “S” in place of the number. Therefore, basin testing in the shoulder will be “BS”.
- For load transfer testing, the sensor spacing will dictate the population of the test type field in the Dynatest software. When joint testing with a sensor at (–12) inches, it shall be entered as “Approach” joint testing. When joint testing with a sensor at (+12) inches, it shall be entered as “Leave” joint testing.
- Create a separate electronic file for each direction for each section in the project, unless requested otherwise.

- Name the electronic file with the FWD data by the route, route number, route suffix, and direction. For instance testing on MD 32 would have a filename of "MD32WB.FWD" and "MD32EB.FWD"

  Responsible Party - SPID

Step 15. Complete the FWD testing following the guidelines established in the operations manual of Dynatest. Specific test set-up is provided in the field work request form, Pavement Guide, and discussions with the pavement design engineer.

Responsible Party - SPID

Step 16. Measure the surface and air temperature regularly during FWD testing and record it in the Dynatest FWD software. Make sure to indicate in the software that the temperature recorded is either surface or air. Only classify the temperature as mid-depth temperature when the pavement surface is drilled, filled with mineral oil, and measured with a temperature probe.

Responsible Party – SPID

Step 17. Proper Quality Control shall be followed during the testing to ensure that sensors and load cells stay within tolerable limits. Comments and field notes about a particular point should be placed in the comments field in the Dynatest software. A separate hand written sheet is only needed if requested by the pavement design engineer when the temperature and field notes are added into the Dynatest software program.

Responsible Party – SPID

Step 18. Provide the FWD file and hard copy printout of the deflection/load results to the responsible pavement design engineer after FWD testing is completed.

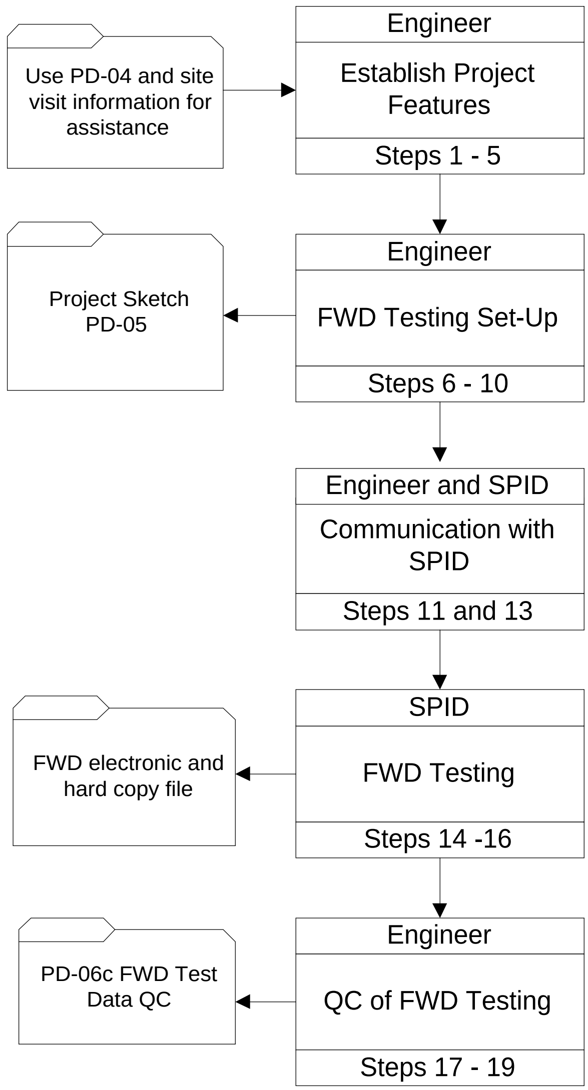
Responsible Party - SPID

Step 19. Quality Assurance shall be completed on the FWD data to ensure the data is a good quality and free of data collection errors prior performing data analysis. It is planned to develop a software program to assist in the QA of the FWD data. Complete FWD data QA Form (PD-06c) after reviewing the FWD data for QA. The QA will involve, but not limited to the following:

- Eliminate test points with non-decreasing deflections
- Eliminate test points with sensors with "Out of Range" errors.
- Review recorded loads to ensure that erroneous measurements are not recorded.

  Responsible Party – Engineer

III.I.4 FWD Testing Process Flowchart



III.J FWD DATA ANALYSIS

III.J.1 Purpose

FWD Data Analysis is conducted to:

- Verify the accuracy of FWD Data
- Sub-section the FWD data in analysis sections
- Determine general characteristics of analysis sections
- Determine the structural capacity of the pavement structure and individual pavement layers
- Estimate the load transfer ability of joints in jointed rigid and composite pavements
- Report FWD analysis results in a form conducive to pavement rehabilitation design

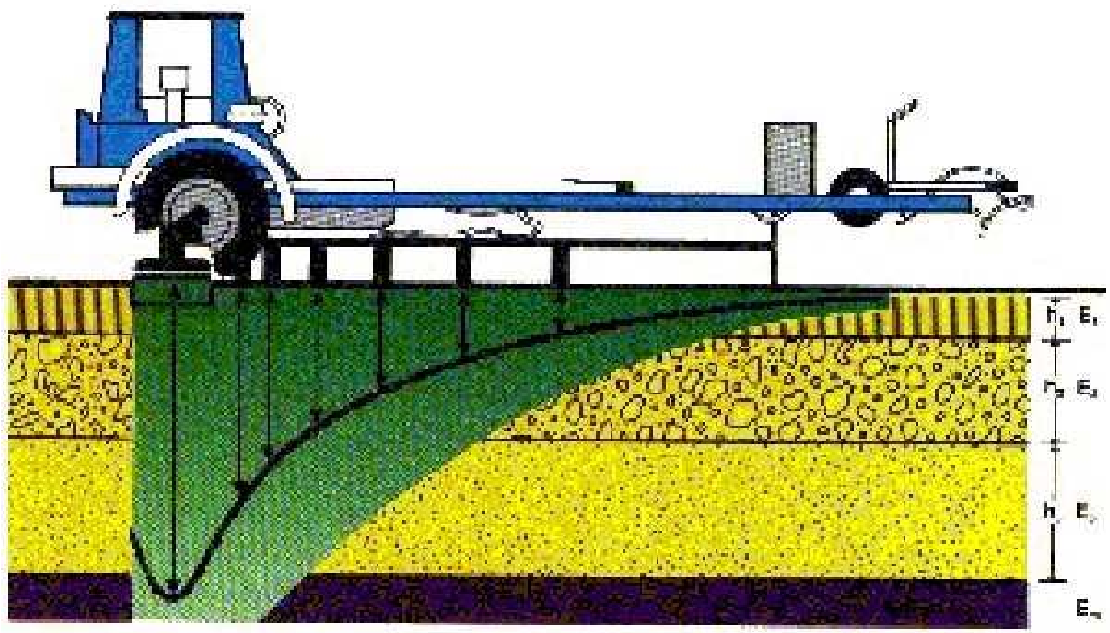
III.J.2 Resource Requirements

The FWD Analysis procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Quality Control of FWD Data	1	1
Staff Engineer or Project Engineer	Pre-Process FWD Data	1	1
Staff Engineer or Project Engineer	Backcalculation Analysis of FWD Data	1	6
Staff Engineer or Project Engineer	Quality Assurance of Backcalculation Analysis	1	2
Staff Engineer or Project Engineer	Reporting/Summary of Analysis Results	1	3

III.J.3 Procedure

The basic concept of FWD testing is to simulate typical traffic loads and monitor the deflection response of the pavement structure from that load. FWD analysis is the science of taking recorded deflections and loads and using knowledge of pavement material characteristics to estimate the existing material properties of the pavement structure. The primary data collected from FWD testing is a measured vertical load and the resulting deflections in the pavement surface measured at radial distances from the load. The largest deflection from the load occurs under the load plate and decreases outwardly from the load plate. Deflections under the load plate provide an indication of the material properties of all the layers at that location. As recorded deflections move further away from the load plate, the data is representative of the layers deeper into the pavement structure with less influence from the layers near the surface. This concept is presented in the following figure.



The process of developing pavement layer properties from FWD data requires knowledge of the art/science of analyzing loads and deflections and a good understanding of pavement engineering and material properties. The MDSHA Pavement Division's process for checking FWD data quality, developing material properties for pavement rehabilitation design, and quality checks of the analysis are described in this Section. It is the desire of the Pavement Division to develop a software tool to assist pavement designers to perform the majority of the steps involved in the FWD Analysis Section. At this time, that software tool does not exist for MDSHA. Therefore, this Section will not be written with specific step-by-step instructions, but rather objectives and concepts needed by the pavement design engineer to accomplish the task of FWD data analysis. Several of the general headings in the FWD Data Analysis Section at this time will need to be completed by hand, through engineering judgment, or using an assortment of software tools available to MDSHA at this time. The AASHTO pavement design software (DarWin), the backcalculation analysis software MODULUS 5.1, and several in-house software applications are capable of assisting the engineer perform some of the sections described below. The following outline and description should be used to assist the engineer following the necessary process to complete FWD data analysis.

1.0 Quality Assurance of FWD Data
- 1.1 Non-Decreasing Deflections
- 1.2 Out of Range Errors
- 1.3 Error Comments

2.0 Pre-process FWD Data
- 2.1 Select FWD Load (drop height/s)
  - Normalize Deflection Data
- 2.2 Composite Modulus Plots
  - Non-linear Soils
  - Erroneous Sensor Readings
  - Select # of Sensors to Analyze
- 2.3 Select Distinct Sections
  - Recorded Distinctions: Test Type, Lane, and Field Comments
  - Sensor/Deflection vs. Station
  - Cumulative Sum

3.0 Analysis of FWD Data
- 3.1 Flexible Standard Calculations:
  - Resilient Modulus of the Subgrade
  - Composite Modulus of Pavement Layers Above Subgrade
  - Effective Structural Number
  - Individual Layer Coefficients
  - Temperature Correction Factor
- 3.2 Rigid/Composite Standard Calculations:
  - Slab Bending Factor / AC Compression Factor
  - Load Transfer Efficiency (LTE)
  - Void Detection
  - Area Method
  - Radius of Relative Stiffness
  - Modulus of Subgrade Reaction
  - PCC Elastic Modulus
- 3.3 Backcalculation Software
  - Use Modulus, WESDef, Modcomp, EverCalc, or other tools

4.0 Quality Control of FWD Analysis
- 4.1 Percent Sensor Error
- 4.2 Layer Moduli Reasonable Ranges
- 4.3 Data Reasonableness Checks

5.0 Reporting/Summary of FWD Data Analysis Results

III.J.3.1 Quality Assurance of FWD Data

The accuracy and reasonableness of the FWD analysis is only as good as the quality of data collected during testing. There are several internal quality control checks (QC) in the FWD testing software to flag erroneous data. In these cases, the data is still collected are stored in the data file, but coupled with an error message. The FWD QC checks are focused on obvious sensor and load cell errors and other equipment malfunctions. Data reasonableness also needs to be verified through quality assurance (QA) by the pavement design engineers after the data has been collected and submitted. This sub-section describes the steps needed to check the FWD data for errors, data reasonableness, and quality. The following headings should be viewed, as the general steps needed to assure the quality of the FWD data.

III.J.3.1.1 Non-Decreasing Deflections

An accurate FWD test data point has sensor readings with deflections that should typically continually decrease as the sensors are placed further away from the load plate. The case where the sensor readings do not decrease, i.e. (in mils) - 20, 15, 10, 5, 8, 4, 2, the FWD test point is considered to have non-decreasing deflections. There are several reasons for non-decreasing deflections including the following:

- Sensor reading error,
- Sensor on piece of debris or crack,
- Severely deteriorated pavement surface,
- Pavement change boundary within range of sensors.

The FWD test points with non-decreasing deflections need to be eliminated as outliers and not used any further in the analysis. Looking through the raw FWD data can complete the QA check for non-decreasing deflections. This is a QA check that will be completed with the future FWD analysis software tool.

III.J.3.1.2 Out of Range Errors

Each sensor on the FWD that records deflection has a total range of vertical movement available for recording data. An “out of range” error occurs when the maximum vertical movement of the sensor has been exceeded. The FWD software that collects the test data also recognizes the fact that the maximum sensor measurement has been exceeded and tags an “out of range” error along with the specific test point data for that sensor. There are several reasons for “out of range” deflections including the following:

- Sensor reading error,
- Severely deteriorated pavement surface,
- Weak structural capacity of the pavement structure,
- Load applied to pavement is too great for existing structure.

The FWD test points with an “out of range” error need to be eliminated as outliers and not used any further in the analysis. If a significant number of out of range errors are encountered during testing, the load package may need to be lowered because the load may be too great for the existing pavement structural capacity. If the load package is not altered in the field and cores indicate a thin pavement structure, additional testing at a lower load may need to be completed. Looking through the raw FWD data can complete the QA check for out of range errors. This is a QA check that will be completed with the future FWD analysis software tool.

III.J.3.1.3 Field Error Comments

The FWD operator shall record any important unusual field information for each test point in the comment field in the FWD data collection software. Occasionally, the FWD operator will provide comments that indicate the test point may need to be eliminated from data analysis. The FWD test points with error comments need to be eliminated as outliers and not used any further in the analysis. Looking through the raw FWD data can complete the QA check for error comments. This is a QA check that will allow the field comments to be viewed with the future FWD analysis software tool, but the engineer will make the final determination of whether the data point is to be used in analysis.

III.J.3.2 Pre-Process FWD Data

Once FWD data has been ensured for reasonableness and accuracy, the data is further processed to arrange it into logical and manageable sub sets that will be used to complete the data analysis. The outline steps in this sub-section are designed to assist the pavement designer to filter out the most beneficial FWD test data and establish sections for further analysis. Analysis sections are pavement sections with similar pavement structures and similar performing pavement materials properties.

III.J.3.2.1 Select FWD Load

The typical roadway load plate is approximately 5.91” in radius and is designed to mimic the contact area patch of a dual tire load from an 18-kip axle. Therefore, to simulate an 18-kip axle load, the 9,000-pound load level or FWD drop height #2 should be used in FWD analysis. In some cases in the MDSHA roadway network, the pavement structure is significantly thick and a 9,000-pound load will not allow a sufficient amount of load to be transferred to the lower layers in the pavement structure. In these cases, a larger load applied should be analyzed in addition to the to the 9,000 pound load. Guidelines for the designer to select the appropriate FWD load package are outlined in Section III.I “FWD Testing”.

A deflection versus station plot allows the designer to identify weak areas and to delineate sections that are performing differently. In order to view a deflection versus station plot accurately, every load should be normalized to the same standard load. Although the same drop height maybe applied to consecutive points, the actual measured applied load may vary due to slight variations in the pavement structure and FWD mechanical equipment. Normalizing the deflections will eliminate these slight changes in load allow numerous test points to be compared equally. The collected deflection is converted into normalized deflection by equating the measured load to a normal load. The following equation shows the process to normalize a deflection to 9,000 pounds:

Measured Load = 9,252# *Measure Deflection* = 15.2 mils

Normal Load = 9,000#

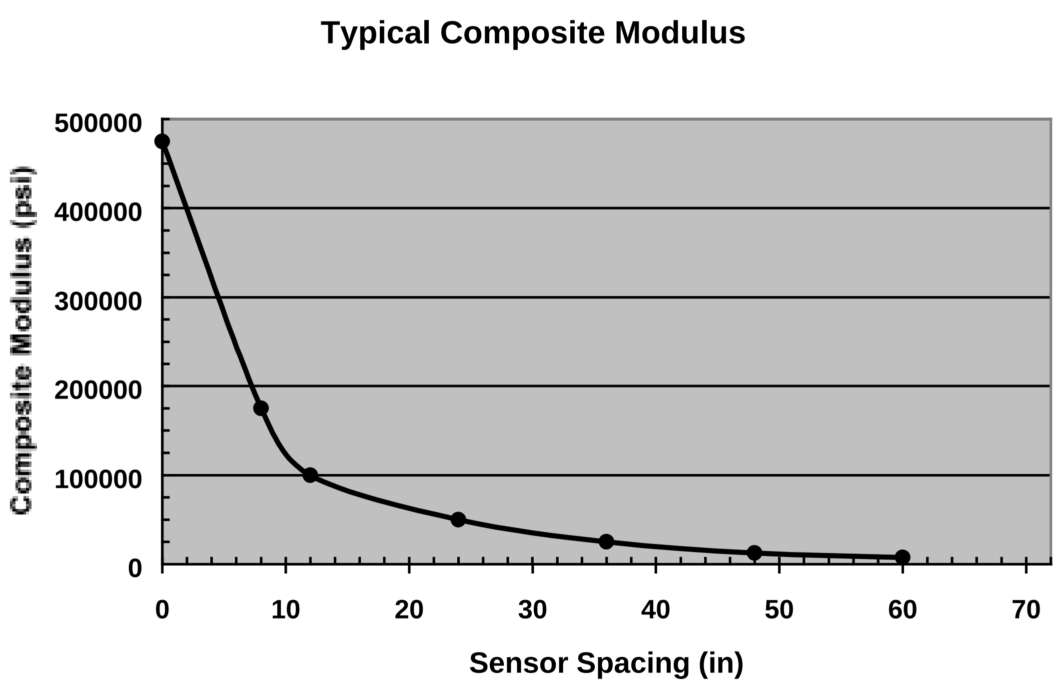
Normalized Deflection = 15.2 * (9,000 / 9,252) = 14.8 mils

Several different drop heights are typically recorded at each test point. Some materials behave differently depending on the amount of load that applied; these are called “stress sensitive" or "non-linear" materials. Applying more than one load provides an opportunity for the pavement design engineer to identify this type of material. Therefore, a stress-sensitive pavement structure can be identified if two separate FWD data analysis runs with different load levels result in different material properties at the same test point. If different loads result in similar FWD data analysis results, then the pavement layers can be considered not to contain stress sensitive material. In either case, the selection of one, several, or all the load levels applied to the pavement via the FWD need to be selected and filtered for analysis.

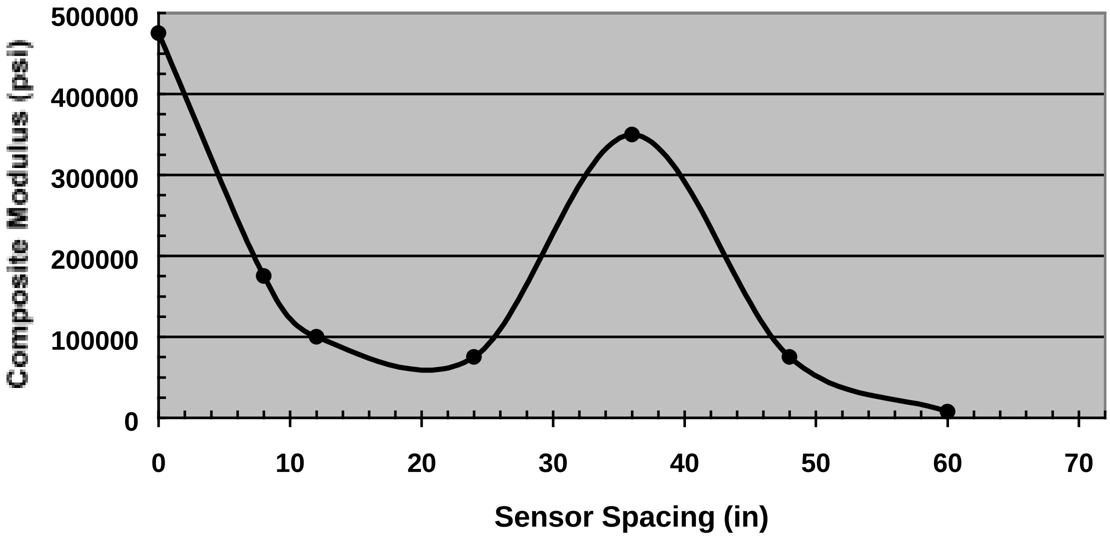
III.J.3.2.2 Composite Modulus Plots

The composite modulus of a pavement structure is a single-value representation of the overall pavement stiffness based on the modulus of elasticity of all the pavement layers at a given point and radial distance from the load plate. The composite modulus is a computed value based on the applied load, plate radius, sensor spacing, measured deflection, and an assumed Poisson ratio of the entire pavement structure. A composite modulus can be calculated for every test point at each sensor. The composite modulus is typically reported graphically to allow the designer to identify erroneous data or non-linear subgrade.

The composite modulus derived from the sensor directly under the load provides a representation of the stiffness of all the pavement layers as a single layer system. The composite moduli derived from the outer sensors lose the contributing effect of the stronger upper layers of the pavement while maintaining the influence of the subgrade and lower layers. Therefore, the composite modulus plot of all of the sensors from a test point should have a general decreasing or "half bowl" shape as the distance from the load increases. Sharp fluctuation or an irregular shaped composite modulus plot may indicate an erroneous test point. The following two plots show a typical and an irregular shape composite modulus plot.

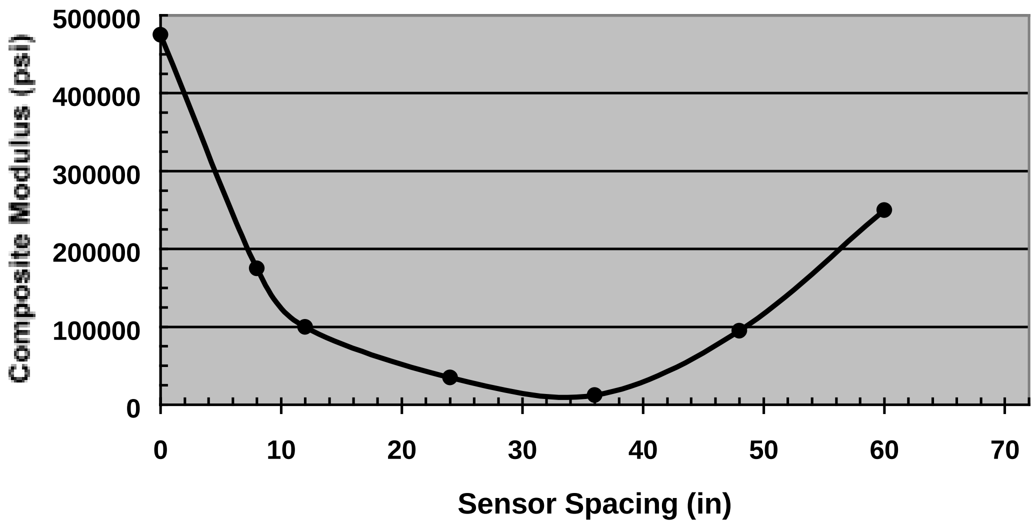


Irregular Composite Modulus



The shape of the composite modulus plot can help to identify the presence of non-linear materials and subgrade. This is capable because the sensors further away from the load provide an indication of the material properties and behavior of the lower pavement layers and subgrade. The amount of load experienced by the outer sensors decreases the as the distance from the load increases. Stress sensitive materials can either increase or decrease in strength as the stress applied to the material increases. Typically with the materials in Maryland, the strength or stiffness of a stress sensitive or non-linear subgrade is inversely proportional to the amount of the applied load. Therefore, a non-linear subgrade will appear to have a higher modulus with a lighter load and as the applied load increases, the weaker the non-linear subgrade behaves. The composite modulus plot allows the designer to identify non-linear subgrade because the amount of load decreases as the distance from the load increases and because the composite modulus is a numerical representation of the pavement stiffness at varying depths. Therefore, if at the outer sensors the composite modulus begins to increase, it is an indication of a non-linear subgrade. This is because the load is decreasing at the outer sensors, yet the composite modulus is increasing. The following composite modulus is an example of plot displaying a pavement structure with a non-linear subgrade.

Non-Linear Composite Modulus Plot



Knowledge of the expected FWD data analysis tool and its specific technique for estimating the subgrade modulus are necessary when evaluating composite modulus plots. Most FWD data analysis tools use some type of combination of the outer sensor deflection and algorithms to estimate the subgrade modulus. Some FWD data analysis tools use the farthest sensor from the load to estimate the subgrade strength. In the case that a non-linear subgrade is present, the estimated subgrade modulus would be too great if the last sensor were used in the calculation. The last sensor reading from a test point with a non-linear subgrade would be falsely greater because of the soil behaving differently at varying loads. In the case of a pavement structure with a non-linear subgrade as we have in the previous plot, the subgrade modulus estimated from data collected from the sensor at 36" would produce a more representative subgrade modulus. The sensor reading at 36" provides a deflection based on the soil behaving with the highest encountered load. In some cases like this, another FWD data analysis approach may be needed to sufficiently estimate an accurate subgrade modulus. In either case, the designer needs to have the ability to eliminate sensor reading from the analysis to develop accurate pavement properties through FWD data analysis. Therefore, in the case of the non-linear subgrade shown in the previous plot, the last 2 sensors would not be used in the analysis to develop layer properties. Typically, all software applications used for FWD data analysis need at least sensors out to 36" in order to perform their internal algorithms.

III.J.3.2.3 Delineate Distinct Sections

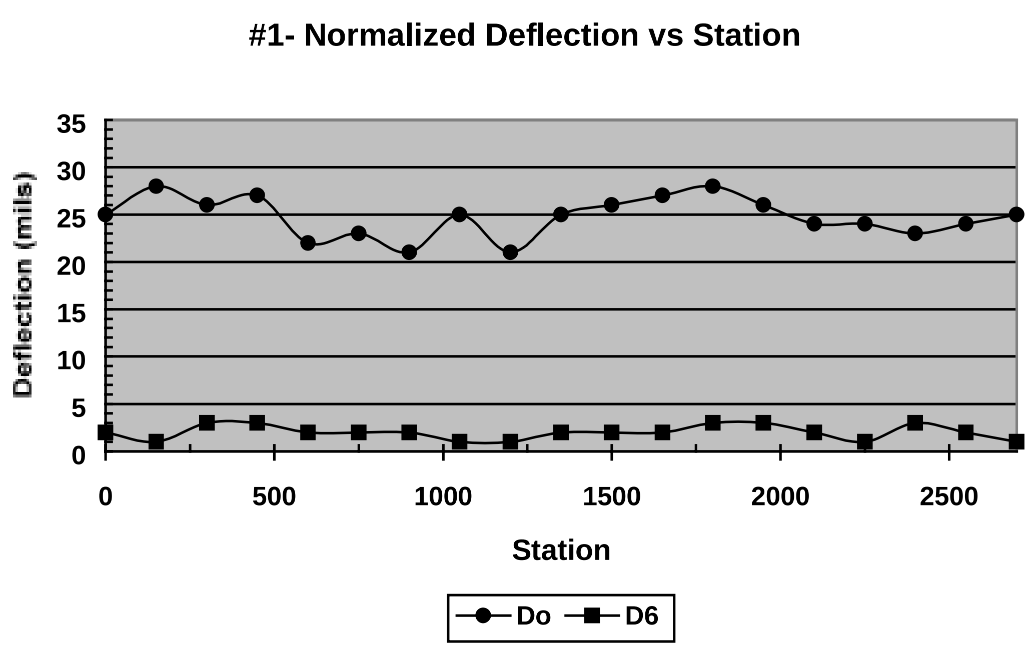
The FWD data collection software stores both test type and specific test point header information. The header information can be used to sort out specific details of a particular test point or a group of similar test points. The ability to sort out particular tests allows the pavement designer to analyze specific data points or areas of interest for rehabilitation design. This type of header information includes, but is not limited to, the following:

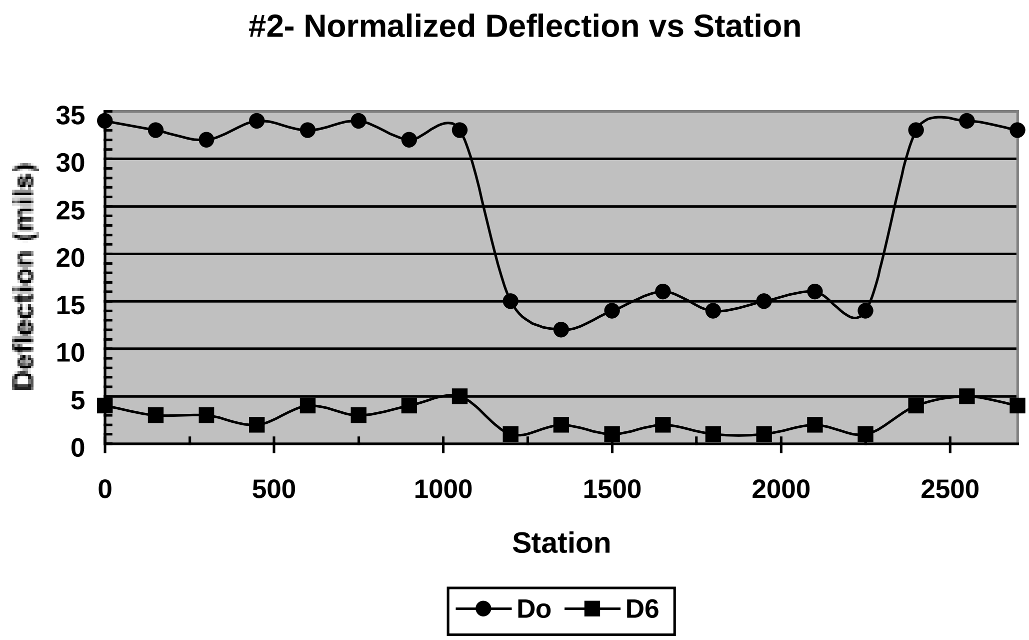
- Test Type – Joint, Basin, Void, Subgrade

- Test Lane – 1, 2, 3, …..
- Field Comments – FWD operator provides input about particular test point operations and pavement conditions

Refer to Section III.I "FWD Testing" for details about how header information is entered and stored.

In addition to header information, the normalized deflection data can be used to delineate similar sections of project for FWD data analysis. The response of a pavement structure to a simulated traffic load can be used to identify thickness variations and areas of variability in material strengths. Variability in the pavement layer properties can be observed by observing a plot of the normalized deflections versus station. To better view the variations in material properties, each sensor can be viewed in a plot or only selected sensors. The most critical sensors to view in a normalized deflection plot are the sensors at the load plate and the sensor farthest away from the load. These two provide an indication of the overall pavement structure (load plate sensor) and the only the subgrade (farthest sensor). The following plots are examples of normalized deflection plots versus station.

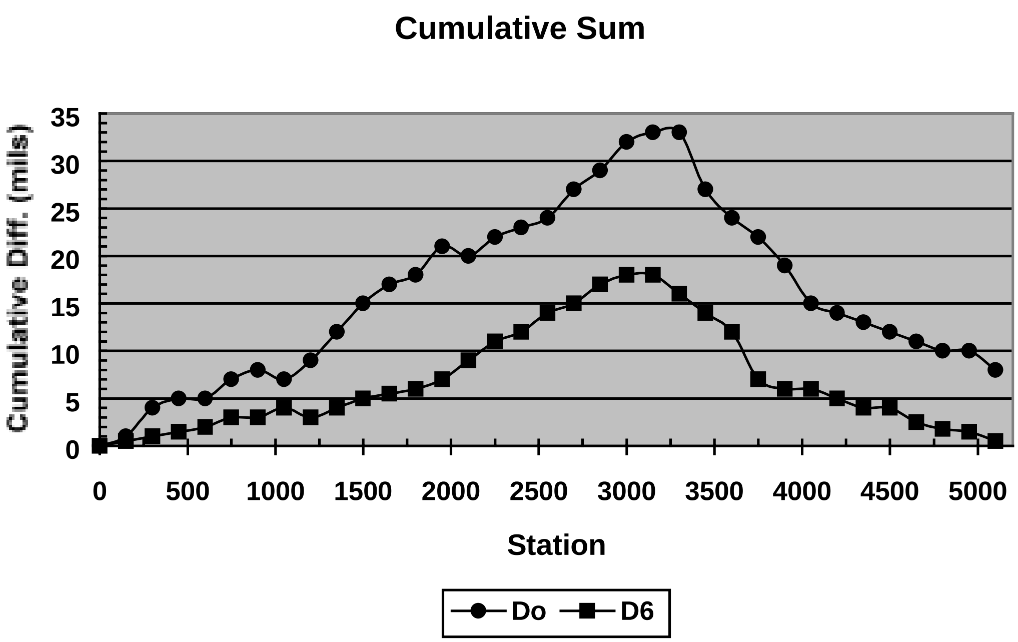




The #1 normalized deflection plot is a typical plot for a consistent pavement structure throughout the testing limits. The #2 normalized deflection plot is a roadway that has a distinct pavement thickness or pavement material strength change from approximately Station 10+00 to 24+00. The pavement thickness is thicker or the material strength is greater from Station 10+00 to 24+00 than the surrounding pavement, based on the deflections in this area being significantly less. The pavement designer can select more accurate and beneficial locations for coring with this type of deflection plot information. In addition to assisting in the determination of core locations, this section of roadway maybe be designed and analyzed separately than the rest of the roadway and possibly have a separate pavement rehabilitation strategy.

In many cases, the normalized deflection versus station plot does not show a pavement material variation as distinctly as in plot #2 from above. The normalized deflection plots can have more data noise and variation from point to point making identification of a separate performing section difficult, especially with numerous consecutive data points. However, there is an algorithm using the average deflection within a test section to develop a plot to determine unique pavement sections. The algorithm method is called cumulative sum. The cumulative sum of deflection method is recommended by AASHTO to delineate section based on FWD data and is outlined in Appendix J of the "1993 AASHTO Guide for the Design of Pavement Structures."

The cumulative sum algorithm produces a function for interpreting trends in the pavement structure from FWD deflection data. Calculating the average deflection for each sensor for all the test points within a test section is the first step in the cumulative sum algorithm. A substantial number of test points, at least 30, are needed in order to develop a stable average and make the algorithm statistically significant. The next step involves stepping through each test point and summing up the deflections. At each test point and sensor, the cumulative sum of the deflection is compared to the average deflection for that sensor multiplied by the total number of test points at that location. This cumulative difference is then plotted versus station. Upon viewing this plot, it is highly probable that there is a change in pavement properties at locations in the plot where the slope significantly changes. The following plot shows is an example of typical cumulative sum plot.



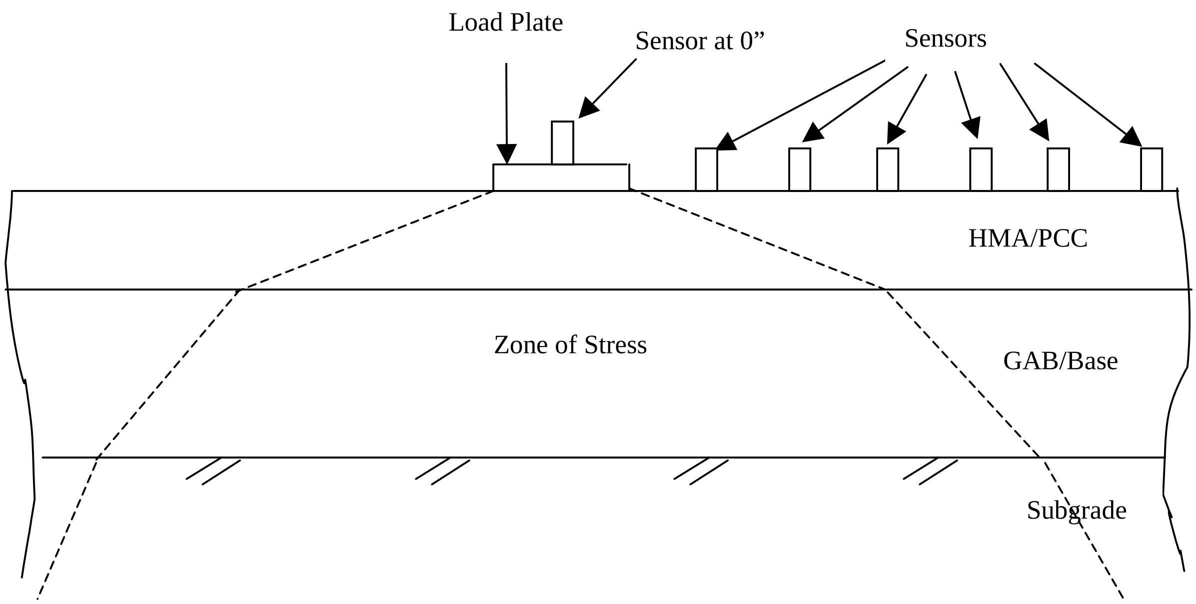
As with the normalized deflection plot, the cumulative sum plot can be done for any of the sensor locations, but the sensor at the load and farthest from the load are the most important to view initially. In the case of the plot above, two statistically homogeneous units can be identified. The change in pavement material properties is at approximately station 32+50. This change could be a difference in thickness or material strength properties. In either case, cores taken in both sections would verify the case and allow the pavement designer to analyze these sections separately for design.

III.J.3.3 Analysis of FWD Data

The analysis of FWD data involves evaluating the measured load and resulting deflections to determine the material properties of the pavement structure. A typical “forward” material calculation requires knowledge of the material properties and the deflection is estimated based on the applied load. The use of FWD requires a “back” calculation method because the deflections and load are known and the material properties are estimated. This method of obtaining the material response and estimating the material properties is called “backcalculation.”

The FWD applies a simulated dual tire load to the pavement through a 12.0” diameter circular load plate for paved surfaces. The amount of load a particular pavement layer experiences is based on the depth of the layer in the pavement structure. The deeper a pavement layer is in the pavement structure, the less the amount of stress the layer is subjected to from the load. This decreasing stress is a result of the vertical load being distributed outwardly in a stress zone down and out from the load plate into the pavement structure.

The distribution of FWD load allows the individual pavement layer properties to be estimated through backcalculation. From the load plate, the load is distributed outwardly and down into the pavement structure at approximately a 30° angle. This angle varies depending on the various layer properties. Directly under the load plate, the sensor deflection reading provides an indication of the layer properties of all the pavement layers at that point. The sensor deflection readings further away from the load plate provide an indication of the layer properties of the pavement layers deeper into the pavement at that point. This concept is presented in the following figure.



III.J.3.3.1 Flexible Standard Calculations

The type of data analysis is dependent on the pavement type. More information about the structural capacity of the pavement can be obtained through FWD testing of flexible pavement. Subgrade strength values and overall composite modulus of the pavement structure can be obtained from flexible pavement FWD testing. These values can be used directly in pavement rehabilitation design to assist in determining the effective structural capacity. The following sub-sections describe the FWD data analysis procedures for flexible pavements.

III.J.3.3.1.1 Resilient Modulus of Subgrade

Efforts shall be made to verify the subgrade resilient modulus calculated from FWD testing is consistent with laboratory testing similar to that used to develop the subgrade values in the AASHO Road Test subgrade. Typically, subgrade resilient modulus results obtained from dynamic field testing, FWD testing, will be less conservative and give higher results than those obtained in the laboratory. This is because laboratory testing used for the subgrade at the AASHO Road Test was static load testing, CBR testing. The difference in testing mechanisms between static laboratory testing and dynamic field testing will provide different strength properties of the subgrade. Therefore, any dynamic testing must be correlated to laboratory testing.

The AASHTO equation to calculate the subgrade resilient modulus from FWD testing is the following:

$$\mathbf{M_r = (0.24 * P) / (d_r * r)}$$

where:
M_r = resilient modulus of the subgrade
P = applied load , pounds
d_r = measured deflection at radial distance r, inches
r = radial distance of measured deflection, inches

The key to this equation is selecting the radial distance to ensure the measured deflection is due to the subgrade and not other layers, but yet close enough to the load plate to ensure data accuracy. The minimum distance can be determined using the following AASHTO equation:

$$\mathbf{r >= 0.7 * a_e}$$

where:
a_e = radius of the stress bulb at the subgrade-pavement interface, inches

The stress bulb is calculated using the following AASHTO equation:

$$\mathbf{a_e} = \sqrt{\left\{\mathbf{a}^2 + \left(\mathbf{D} * \sqrt[3]{\frac{\mathbf{E_p}}{\mathbf{M_r}}}\right)^2\right\}}$$

where:
a = FWD load plate radius, inches
D = total pavement thickness of pavement layers above the subgrade, inches
E_p = effective modulus of all pavement layers above the subgrade, psi

Based on the equations provided above, the resilient modulus can not be calculated without the effective modulus of all pavement layers above the subgrade. Yet, the effective modulus of all pavement layers above the subgrade cannot be calculated without the subgrade resilient modulus. Therefore, an iterative process is required to obtain both values using the equations provided above. However, engineering judgement can be used coupled with information from composite modulus plots to estimate the sensor to use to obtain the resilient subgrade modulus. To ensure that the sensor is only recording the effect from the subgrade, use the furthest sensor away from the load plate that does not show effects of a non-linear subgrade to calculate the subgrade resilient modulus. Composite modulus plots and non-liner subgrade items were discussed in a previous section about pre-processing FWD data.

The resilient subgrade modulus calculated from the equations above provides the results as measured through dynamic testing at not correlated to laboratory testing. The correct method is develop a local agency correlation factor by performing a study to develop a relationship between subgrade resilient results obtained from FWD data analysis and those from laboratory testing. However, AASHTO provides a general guideline of one-third the calculated subgrade resilient modulus from FWD data analysis shall be used for the design subgrade resilient modulus. Therefore, the AASHTO equation to calculate subgrade resilient modulus results in the following:

$$\mathbf{M_r = C * ((0.24 * P) / (d_r * r))}$$

where: C = field to laboratory correlation for subgrade resilient modulus. This value is typically **0.33**.

III.J.3.3.1.2 Composite Modulus of Pavement Layers Above Subgrade
The composite modulus, or the effective modulus, is the measure of the strength of all the pavement layers above the subgrade in the pavement structure. This value provides an indication of the overall strength of the pavement structure at the test point. This can be used to compare with other sections to identify change in pavement structure or pavement layer material strengths. The composite modulus is also needed to calculate both the radius of the stress bulb at the subgrade-pavement interface and the effective structural capacity (structural number – SN) of the pavement structure. In order to calculate the composite modulus, both the total thickness of all the layers above subgrade and the subgrade resilient modulus need to be known. This information and the deflection from FWD testing directly under the load plate, is used in the following equation to calculate the composite modulus:

$$\mathbf{d_o = 1.5 * p * a *} \left[\frac{1}{M_r * \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_p}{M_r}} \right)^2}} + \frac{\left(1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}} \right)}{E_p} \right]$$

where: d_o = deflection measured at the center of the load late (adjusted to a standard temperature of 68° F), inches
p = FWD load plate pressure, psi
a = FWD load plate radius, inches
D = total thickness of pavement layers above subgrade, inches
a = FWD load plate radius, inches

The calculation of composite modulus is also an iterative process with the temperature-corrected deflection under the load, based on the equation above. All deflections need to be adjusted to a reference temperature of 68° F for comparison purposes and to also follow the AASHTO deflection analysis procedure.

III.J.3.3.1.3 Effective Structural Number
The effective structural number for flexible pavements is calculated based on the assumption that the structural capacity of a pavement is a function of the total thickness and the composite modulus of the pavement structure above subgrade. Use the following equation to calculate the effective structural capacity for flexible pavements:

$$SN_{eff} = 0.0045 * D * \sqrt[3]{E_p}$$

where: SN_{eff}= effective structural capacity.

III.J.3.3.1.4 Individual Layer Coefficients

In some cases, the individual layer modulus will be backcalculated from deflection data. From the individual layer modulus, the AASHTO layer coefficient can be calculated. The layer coefficients for each pavement layer are estimated based on the ratio of a typical asphalt concrete layer modulus 930,000 psi, with a layer coefficient of 0.44. The individual layer coefficient, for a given layer modulus, is calculated with the following equation:

$$\mathbf{a_i = 0.44 * \sqrt[3]{\frac{M_i * (1-\mu_i^2)}{930000(1-0.35^2)}}}$$

where: a_i = layer coefficent
M_i = layer modulus
μ_i = Poisson's ratio of layer

III.J.3.3.1.5 Temperature Correction Factor

In order to allow to equal comparisons between FWD test points done at different under varying weather and temperature conditions, all test data needs to be corrected to a reference temperature. The material properties of a majority of pavement materials are temperature dependent, some material like hot mix asphalt are more temperature susceptible than other materials. The same flexible pavement structure may provide drastically different strength characteristics in Maryland at mid-day testing in August than in the morning testing in late October. Therefore, all FWD testing data must be corrected to a reference temperature of 68° F for valid comparison. There are two basic ways to correct for temperature, either the measured deflection can be corrected or the resulting calculated modulus could be corrected to the reference temperature. The equations used by AASHTO, and those used in DarWin software, require the correction to occur for the deflection.

Temperature correction curves for deflections are contained on pages III-99 and III-100 in the 1993 "AASHTO Guide for Design of Pavement Structures." DarWin uses these equations within the software calculations. These curves will be used to temperature correct deflection for MDSHA pavement design projects. The BELLS equation will be used to temperature correct the backcalculated modulus for MDSHA pavement design projects. The BELLS equation calculates the mid-depth HMA temperature and calculates an adjustment factor to be applied to the uncorrected layer modulus. There are several other methods to temperature correct modulus values that are used by MDSHA, including Asphalt Institute and Washington DOT.

III.J.3.3.2 Rigid/Composite Standard Calculations

The type of data analysis is dependent on the pavement type. More information about the material properties can be obtained through FWD testing of rigid and composite pavement. Subgrade strength values, slab bending /compression factors, joint load transfer efficiency, void detection, and PCC material properties can be obtained from rigid and composite pavement FWD testing. These values can be used directly in pavement rehabilitation design to assist in determining the effective structural capacity and used to assist in making rehabilitation strategy decisions. The following sub-sections describe the FWD data analysis procedures for composite and rigid pavements.

III.J.3.3.2.1 Slab Bending Factor / AC Compression Factor

The slab bending factor / AC compression factor (B) is used in the calculation of load transfer efficiency of joints in rigid and composite pavements; slab bending factor for rigid pavements and AC compression factor for composite pavements. This factor corrects for the differential bending of the slab under vertical loading in rigid and composite pavements. It also accounts for the compression of the AC material under loading in composite pavements. This correction is needed because of the slight slab bending that occurs even when testing sound concrete at the center of the slab. Even the deflections measured at the first 2 sensor locations at the center of a slab, with theoretically 100% load transfer efficiency, would not be equal. Several basin tests at the mid-slab away from the joints are needed to develop a reasonable bending factor (B). The bending/compression correction factor is obtained from the ratio of deflection under the load plate to the deflection 12.0" away from the center of the load plate, for a typical center slab location. Typical values for B range from 1.05 and 1.20. Values above 1.25 should be reconsidered prior to using for analysis based on reasonable data. It is reasonable and conservative to base all load transfer analysis on a bending factor equal to 1.0.

III.J.3.3.2.2 Load Transfer Efficiency

Load transfer efficiency is used in the AASHTO rigid and composite pavement design procedures. MDSHA Pavement Division also uses load transfer efficiency to assist in determining the type of pavement rehabilitation and pre-overlay repair strategies. Load transfer efficiency is calculated by taking a percentage value of the deflection measured on the unloaded side of a joint to the deflection measured on the loaded side of a joint from FWD testing. Load transfer efficiency (LTE) is calculated with the following equation:

$$\mathbf{LTE = (d_1 / d_0) * 100}$$

where: LTE = load transfer efficiency, percentage
d_1 = deflection measured on unloaded side of joint, mills
d_0 = deflection measured on unloaded side of joint, mills

Load transfer efficiency can also be calculated using the slab bending / AC compression factor (B), discussed previously. Load transfer efficiency (LTE) with the bending factor (B) is calculated with the following equation:

$$\mathbf{LTE = B * (d_1 / d_0) * 100}$$

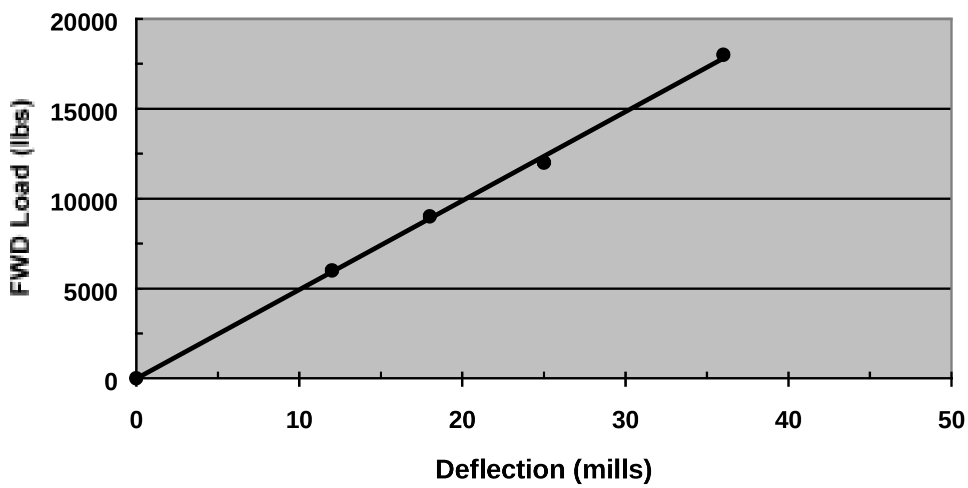
where: B = bending / compression factor

Load transfer efficiency is the widely used tool to assess the joint performance. The same deflection data can be used to calculate **differential deflection** which maybe more relevant to the rate of deterioration of joints and cracks, and to the likelihood of reflection cracking in asphalt overlays. Differential deflection is the absolute difference between the loaded joint side deflection minus the unloaded joint side deflection. Even if a definitive differential deflection value has yet to be established, poorer performing joints can definitely be distinguished from those that are performing well.

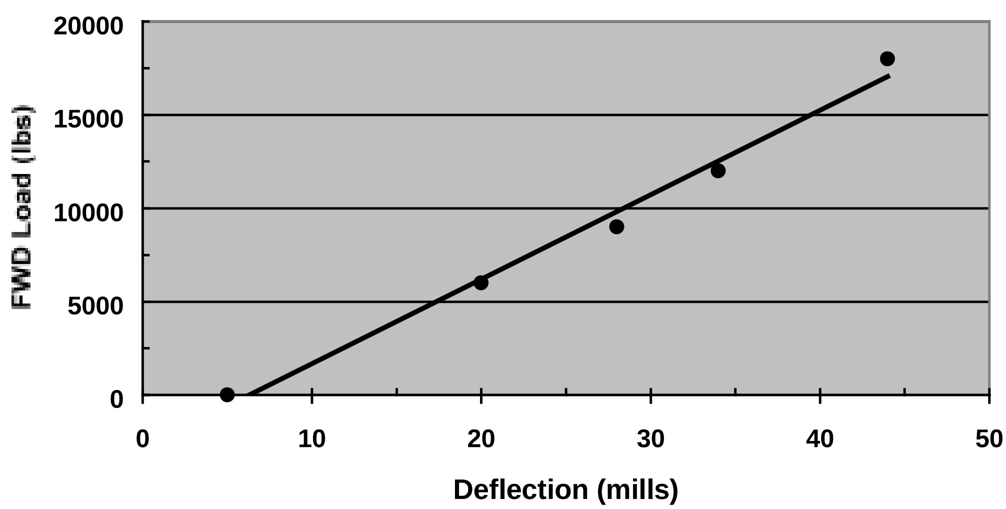
III.J.3.3.2.3 Void Detection

Void detection analysis is used to identify joints that have lost base material under the concrete slab. AASHTO identifies three ways to assist in determining voids under slab; MDSHA Pavement Division will only use one of those methods for recommendation production means. FWD testing for void detection involves corner testing at three load levels, typically 6, 9, and 12 kips; greater loads can be used depending on the thickness and type of pavement structure. The deflection versus load plot is produced for each test location in order to identify the presence of voids. If no voids exist, the slope of the line between the three load drop points will cross very near the origin (less than 2 mills). When the slope of the line crosses further away from the origin (greater than 2 mills), the presence of a void is expected. This method identifies the presence of a void, but does provide any indication as to the size of the void. The graphs below demonstrate this concept.

Example without Void



Example with Void



III.J.3.3.2.4 Area Method

A simple two-layer pavement system approach to backcalculation of FWD data that is used to determine the structural capacity of rigid and composite pavements is the AREA method. Each FWD test measures the load and resulting deflection basin from that applied load. The size and shape of the deflection basin provides an indication of the structural capacity of the pavement structure at the test point. Therefore, the AREA method uses the deflection basin to calculate the pavement structural capacity. For **rigid pavements**, AREA is calculated with the following equation:

$$AREA_7 = 4 + 6*\left(\frac{d_8}{d_0}\right) + 5*\left(\frac{d_{12}}{d_0}\right) + 6*\left(\frac{d_{18}}{d_0}\right) + 9*\left(\frac{d_{24}}{d_0}\right) + 18*\left(\frac{d_{36}}{d_0}\right) + 12*\left(\frac{d_{60}}{d_0}\right)$$

where: d_0 = deflection in center of load plate, inches
d_i = deflection at 0, 8, 12, 18, 36, and 60 inches from load plate center, inches

AREA has units of length, not area, since each of the deflections is normalized with respect to the deflection under the load plate in order to remove the effect of different load levels and to restrict the range of values. For sound concrete, the AREA will range from 29 to 32. For **composite pavements**, AREA is calculated with the following equation:

$$AREA_5 = 3 + 6*\left(\frac{d_{18}}{d_{12}}\right) + 9*\left(\frac{d_{24}}{d_{12}}\right) + 18*\left(\frac{d_{36}}{d_0}\right) + 12*\left(\frac{d_{60}}{d_{12}}\right)$$

where: d_0 = deflection in center of load plate, inches
d_i = deflection at 0, 8, 12, 18, 36, and 60 inches from load plate center, inches

The deflection at 12" rather than under the load plate is used for composite pavements due reduce the effect of the compression of the HMA that might occur under the load plate during loading.

III.J.3.3.2.5 Radius of Relative Stiffness

For a given load radius and sensor configuration, a unique relationship exists between AREA and the dense liquid radius of relative stiffness (**l**) for the pavement system. The radius of relative stiffness is basically a measure of the structural capacity of the pavement structure based on the dense liquid theory for PCC. The value by itself does not provide any beneficial qualifier of the structural capacity, but it is used in other equations to calculate subgrade and pavement layer strengths. Calculating radius of relative stiffness is an iterative process, with the starting point being the assumption that the PCC slab has an infinite size. With this assumption in mind, the initial calculation of the radius of relative stiffness is an estimate; $\mathbf{l_{est}}$. For **rigid pavements**, the initial estimated radius of relative stiffness ($\mathbf{l_{est}}$) is calculated with the following equation:

$$l_{est} = \left[\frac{\ln\left(\frac{60 - AREA_7}{289.708} \right)}{-0.698} \right]^{2.566}$$

For **composite pavements**, the initial estimated radius of relative stiffness (**l_{est}**) is calculated with the following equation:

$$l_{est} = \left[\frac{\ln\left(\frac{48 - AREA_5}{157.40} \right)}{-0.476} \right]^{2..220}$$

III.J.3.3.2.6 Modulus of Subgrade Reaction

The modulus of subgrade reaction (**k**) represents the support provided by all pavement layers, including aggregate base and subgrade, under the PCC slab. The resilient modulus (M_r) of the subgrade is used in analysis for flexible pavements and the modulus of subgrade reaction (**k**) is used in analysis for rigid and composite pavements. The initial calculation of the modulus of subgrade reaction (**k**) is an estimate because it is calculated with estimated radius of relative stiffness (**l_{est}**), based on the assumption that the PCC slab of infinite size. For **rigid pavements**, based on infinite slab assumption, the initial estimated modulus of subgrade reaction (**k_{est}**) is calculated with the following equation:

$$k_{est} = \frac{P * d_{oc}}{d_o * (l_{est})^2}$$

where:

k_{est} = estimated backcalculated modulus of subgrade reaction
P = load, pounds
d_o = deflection measured at the center of load plate, inch
l_{est} = estimated radius of relative stiffness, inches (see above)
d_{oc} = nondimensional coefficient of deflection at center of plate

$$d_{oc} = 0.1245 * e^{\left[-0.14707 * e^{\left(-0.07565 * l_{est} \right)} \right]}$$

For **composite pavements**, based on infinite slab assumption, the initial estimated modulus of subgrade reaction (**k_{est}**) is calculated with the following equation:

$$k_{est} = \frac{P * d_{12c}}{d_{12} * (l_{est})^2}$$

where: d_{12} = deflection measured 12" from center of load plate, inch
d_{12c} = nondimensional coefficient of deflection 12" from center of plate

$$d_{12c} = 0.12188 * e^{\left[-0.79432 * e^{\left(-0.07074 * l_{est}\right)}\right]}$$

As discussed previously, all the previous equations were based on the assumption of an infinite slab. The correction factor for the slab size is identified as, **L**. The next step is to correct for the size of slab in the pavement structure. If the slab length is less than or equal to twice the width, **L**, is the square root of the product of the slab length and width, with all dimensions in inches. If the PCC slab is greater than twice the width, **L**, is the product of the square of the root of two and the slab length, all dimensions in inches. This logic is identified in the following equations:

If length <= 2 * width, $\mathbf{L} = \sqrt{\text{Length} * \text{Width}}$
If length > 2 * width, $\mathbf{L} = \text{Length} * \sqrt{2}$

There are two resulting adjustment factors based on the slab size correction factor, AF_{do} and AF_l. The correction factor for the deflection under the load plate AF_{do}, for both rigid and composite pavements is the following equation:

$$AF_{d_o} = 1 - 1.15085 * e^{\left[-0.71878\left(\frac{L}{l_{est}}\right)^{0.80151}\right]}$$

The correction factor for the radius for relative stiffness AF_l, for both rigid and composite pavements is the following equation:

$$AF_l = 1 - 10.89434 * e^{\left[-0.61662\left(\frac{L}{l_{est}}\right)^{1.04831}\right]}$$

The dynamic modulus of subgrade reaction, **k**, for both composite and rigid pavements, corrected for slab size, is the following equation:

$$k = \frac{k_{est}}{AF_{l}^{2} * AF_{d_{o}}}$$

Using the FWD data and performing a backcalculation analysis to obtain the modulus of subgrade reaction provides a result based on a dynamic load applied in the field. All AASHTO design equations were based on static loads applied in a laboratory. Therefore, to obtain a modulus of subgrade reaction design value for pavement design, the results obtained from backcalculation analysis must be corrected. Use the following equation to determine the design modulus of subgrade reaction (**k_{design}**)

$$k_{design} = \frac{k}{2}$$

III.J.3.3.2.7 PCC Elastic Modulus

The PCC elastic modulus is a required input for the AASHTO pavement design process. Use the following equation to calculate PCC elastic modulus:

$$E_{pcc} = \frac{l_{est} * \left(12 * \left(1 - 0.15^{2}\right) * k_{design}\right)}{PCC^{3}}$$

where: PCC = thickness of PCC slab, inches
E_{pcc} = Elastic modulus of PCC layer, psi

III.J.3.3.3 Backcalculation Software

There are several backcalculation software applications available on the market today. MDSHA currently primarily uses the backcalculation algorithms in the DarWin software, which is based on the AASHTO deflection analysis procedures. Most of these procedures have been documented in this section earlier and can be replicated by hand without the assistance of a software application. The AASHTO deflection analysis procedures do not produce individual pavement layer moduli for flexible pavement, only a composite modulus for all layers. Other software applications are required if individual pavement layers strengths are desired.

The most used software application by MDSHA, and industry wide, for the estimation of individual pavement layer strength is MODULUS. The MODULUS algorithm is based on trying to match the collected FWD deflection basin with a database of deflection basins with known layer moduli. The accuracy of the backcalculated layer moduli is based on the percent sensor error between the measured deflection and the predicted deflection from the MODULUS algorithm.

There is several other software applications for backcalculation available on the market today. Some applications perform an iterative process to estimate the layer modulus, others use the database approach and match measured deflections, while others use a finite element analysis approach to solving for individual layers. Some of the other backcalculation software applications available today include the following: WESDEF, MODCOMP, ILLI-BACK, ELMOD, ELSYM, KENLAYER, KENSLAB, and ISLAB.

III.J.3.4 Quality Control of FWD Analysis

In the same manner the quality of the data collected during deflection testing needs to be controlled for quality to ensure accurate pavement design recommendations, so does the results of the backcalculation analysis. FWD deflection analysis produces estimates of subgrade strengths, composite layer modulus (all layers), individual layer moduli, load transfer efficiencies, void detection, and etc, all of which need to be verified for logical and reasonable results. This sub-section describes the steps needed to check the FWD deflection analysis for backcalculation errors, data reasonableness, and quality. The following headings should be viewed, as the general steps needed to control the quality of the FWD deflection analysis.

III.J.3.4.1 Percent Sensor Error

Backcalculation software applications, like MODULUS, compare measured deflection basins with estimated deflection basins. These comparison results in the introduction of the error involved with matching deflection basins. Therefore, the percent sensor error becomes a critical element in determining the accuracy the results from backcalculation analysis. The percent sensor error is the percent difference between the measured and expected deflection. Errors less than 5% are considered to produce reasonable results. Errors between 5% and 10% should be questioned for validity. Errors greater 10% are not valid and should not be used in analysis for the development of pavement rehabilitation recommendations.

III.J.3.4.2 Reasonable Layer Modulus Range

The results produced from backcalculation analysis need to be reasonable for the type of material under investigation. The Material Properties section of the Material Library, Section X.B, of this pavement design guide should be used to determine the reasonable strength range for pavement materials used by MDSHA. In addition, these ranges should be used to confirm the reasonableness of analysis results involving the combining several pavement layer materials into one value, i.e. composite modulus.

III.J.3.4.3 Data Reasonableness Checks

Other results produced from backcalculation analysis, like load transfer efficiency (LTE), need to be reasonable for the type of material under investigation. LTE greater than 100% is not reasonable under typical environmental and loading conditions and values greater than 100% shall be used in the pavement data analysis.

III.J.3.5 Reporting/Plots of FWD Data Analysis Results

It is beneficial to present the results of FWD data analysis in a form that can be easily interpreted and presented to others not privy to the details of FWD data analysis. In addition, the task of formalizing the data into a presentable form will force the pavement designer to look over the data for errors and compile numerous data points into beneficial summary data results. The two most powerful presentation tools for FWD data analysis results are plots and tables or charts. Plots allows the pavements designer and others to view the all the data in a section or project which provides focus on segments performing differently that the majority of the project or section. Tables and charts provide a summary of an entire project or section in just a few descriptive material property items. Statistical analysis results in tables and charts provide additional summary of the performance of the project or analysis section.

The following material properties from FWD data analysis are beneficial and informative for flexible pavements when plotted versus station:

- subgrade resilient modulus (Mr),
- effective structural capacity of pavement – structural number (SN),
- backcalculated individual layer moduli.

The following material properties from FWD data analysis are beneficial and informative for rigid or composite pavements when plotted versus station:

- modulus of subgrade reaction, k
- strength of PCC - elastic modulus of PCC elastic - E_{pcc}
- strength of PCC - modulus of rupture of PCC - S'c
- strength of PCC – Area,
- load transfer efficiency – LTE % or load transfer deficit
- backcalculated layer moduli

The following material properties from FWD data analysis are beneficial and informative for flexible pavements when presented in table form and broken out into different analysis sections or segments:

- mean, minimum, maximum, standard deviation of Mr, SN, and backcalculated layer moduli.

The following material properties from FWD data analysis are beneficial and informative for rigid or composite pavements when presented in table form and broken out into different analysis sections or segments:

- mean, minimum, maximum, and standard deviation of k, Epcc, S'c, Area, LTE, and backcalculated layer moduli.

III.K PRELIMINARY ENGINEERING COST ESTIMATE

III.K.1 Purpose

Preliminary Engineering (PE) Cost Estimate is completed to:

- Provide the Project Owner an estimate of the of the cost required to complete the review, testing, design, and analysis of the project
- Estimate the amount of testing required on the project, for Pavement Division, Geotechnical Investigation, Geology, and the testing laboratory
- Estimate the man days required to complete testing and design of the project

III.K.2 Resource Requirements

The PE cost estimate procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Overview of Project	1	1
Staff Engineer or Project Engineer	Assess Testing Needs	1	1
Staff Engineer or Project Engineer	Assess Man-hour Requirements	1	1
Staff Engineer or Project Engineer	Verify with other Divisions	1	2
Staff Engineer or Project Engineer	Develop Cost Estimate	1	1

III.K.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to complete a PE cost estimate. The lead Division (Pavement or Geotechnical Explorations) should be the division responsible for developing and providing the PE cost estimate to the project owner. However, an attempt to verify that the correct number of man-hours and testing needed by each Division should be completed prior to submitting the PE cost estimate memorandum. The end result of the PE cost estimate process is a 1-page memorandum to the project owner that describes the estimated engineering design costs needed to provide recommendations for a given project.

The majority of the effort in this process is to develop the cost estimate value is done within the PE Cost Estimate Form (PD-13). This effort will involve entering estimated hours directly into the spreadsheets in the MS Excel file named PD-13.xls, for form PD13. This will result in a monetary value for the PE cost estimate.

In the PD-13.xls Excel file and form, the number of working days is requested in several locations. Working days are the number of days it would take one individual to complete each item. If more than one person is expected to work on that item, increase the number of working days by the number of individuals. The cost of travel and out of town expenses are already figured into the calculation in the MS Excel file PD-13.xls.

NOTE: If it is anticipated that consultants will be assisting on any particular item in the PE Cost estimate, the amount of workdays required be the consultant should be tripled (multiply by 3) to account for consultant overhead and profits costs. Therefore, 1 workday should be estimated to be 3 workdays if a consultant is to perform the item.

Step 1. Receive a request for a PE cost estimate from project owner. If this project is a project that is new to OMT then, Steps 1 through 12 need to be followed in Section III.F "Pavement Rehabilitation Design." By completing these steps in Section III.F it will provide with adequate information to complete the PE cost estimate process. In some cases, not all the steps in Section III.F can be completed by the due date of the PE cost estimate. In this case, certain assumptions and a more conservative PC cost estimate needs to be completed.

Responsible Party - Engineer

Step 2. Start the cost estimating process after collecting the data from the records review, open the MS Excel file named PD-13.xls located on the network at n:\omr\everyone\pavedsgn\guide\forms.

Responsible Party - Engineer

Step 3. Open the Summary sheet in the PD-13 Excel file and enter the project description information.

Responsible Party - Engineer

Step 4. Select the region that the project is located from the table below. The cost for certain items, i.e. testing and travel, will vary depending on the regions of the state. This region will be used in the development of the cost estimate. Enter the region in the summary sheet in the PD-13.xls file.

Region	County
1	Caroline, Cecil, Kent, Queen Annes, Talbot
	Dorchester, Somerset, Wicomico, Worcester
2	Anne Arundel and Calvert
	Baltimore, Baltimore City, Harford
	Carroll, Frederick, Howard
	Montegomery, Prince George's
3	Charles and St. Mary's
4	Allegany, Garrett, Washington

Responsible Party - Engineer

Step 5. Open the Pavement sheet in the PD-13 Excel file and enter the number of working days for each of the items under the Meetings Design/Analysis, and Report headings. The values entered in the Pavement sheet are the expected working days required for the Pavement Division to complete the project accurately and thoroughly. A summary of the total working days and cost will result from the data entry for each heading and the Division. Print out the Pavement sheet when all the required fields are populated.

Responsible Party - Engineer

Step 6. Open the Geotechnical sheet in the PD-13 Excel file and enter the number of working days for each of the items under the Meetings Design/Analysis, and Report headings. The values entered in the Geotechnical sheet are the expected working days required for the Geotechnical Explorations Division to complete the project accurately and thoroughly. A summary of the total working days and cost will result from the data entry for each heading and the Division. Print out the Geotechnical sheet when all the required fields are populated.

Responsible Party - Engineer

Step 7. Open the Geology sheet in the PD-13 Excel file and enter the number of working days or other information for each of the items under the Meetings and and Design/Analysis headings. The values entered in the Geology sheet are the expected working days and other information required for the Geology Division to complete the project accurately and thoroughly. A summary of the total working days and cost will result from the data entry for each heading and the Division. Print out the Geology sheet when all the required fields are populated.

Responsible Party - Engineer

Step 8. Open the SPID sheet in the PD-13 Excel file and enter the number of working days for each of the items under the Testing heading. The values entered in the SPID sheet are the expected working days required for the SPID to complete the project accurately and thoroughly. A summary of the total working days and cost will result from the data entry for the Division. Print out the SPID sheet when all the required fields are populated.

Responsible Party - Engineer

Step 9. Open the Drillers sheet in the PD-13 Excel file and enter the total length of the project and the number of foundation borings for each of the items under the Soil Borings and Foundation Borings headings. The values entered in the Drillers sheet are the expected information required for the Drillers to complete the project accurately and thoroughly. Foundation borings are necessary in cases that a structure (bridge) is involved in the project or exceptionally deep utilities need to be placed. It is typical to have 5 foundation borings per normal structure. A summary of the cost will result from the data entry for each heading and the Division. Print out the Drillers sheet when all the required fields are populated.

Responsible Party - Engineer

Step 10. Open the Lab sheet in the PD-13 Excel file and enter the number of working days for each of the items under the Testing heading. The values entered in the Lab sheet are the expected information required for the Lab to complete the project accurately and thoroughly. A summary of the total cost will result from the data entry for the Division. Print out the Lab sheet when all the required fields are populated.

Responsible Party - Engineer

Step 11. Open the Summary sheet in the PD-13 Excel file and view the estimated PE cost of the project. Print out the Summary sheet.

Responsible Party - Engineer

Step 12. Provide a copy of the PE cost estimate to each Division and allow them to review their respective sections for verification of information and costs. Give the Geotechnical, Drillers, and Lab sections to the Geotech contact for review. Give the Geology sections to the Geology Division Chief (Dave Martin) for review. The Pavement and SPID sections are the responsibility of the pavement design engineer.

Responsible Party - Engineer

Step 13. After the respective Divisions have provided verification, make any correction, if necessary. Print out all sections of Form PD-13 and place in project file.

Responsible Party - Engineer

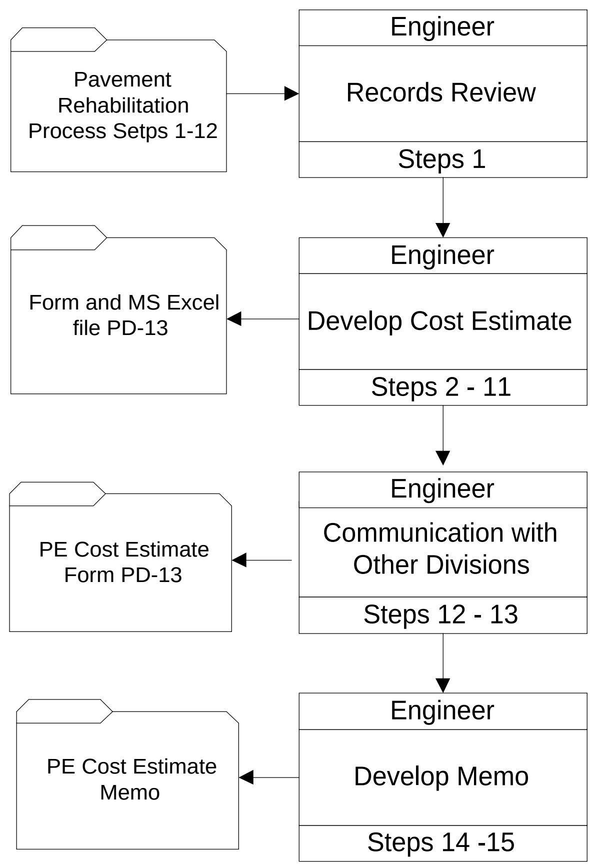
Step 14. Prepare a PE cost estimate memorandum on MDSHA letterhead following the typical format (n:\omr\everyone\pavedsgn\pecostformat.doc).

Responsible Party - Engineer

Step 15. Submit PE cost estimate memorandum to Pavement Chief for approval and signature.

Responsible Party - Engineer

III.K.4 PE Cost Estimate Process Flowchart



III.L SHOULDER DESIGN

III.L.1 Purpose

Shoulder designs are conducted to:

- Determine the future functional and structural requirements of the pavement,
- Match the existing mainline pavement structure to provide adequate drainage characteristics, and
- Identify the material requirements for future demands of shoulder.

III.L.2 Resource Requirements

The shoulder design process is typically done concurrently with a new pavement design for the mainline. The majority of the data collection and design effort done for the mainline shall complete a majority of the steps needed for shoulder design. The shoulder design procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Records Review	1	1
Staff Engineer or Project Engineer	Site Visit	1	0.5
Staff Engineer or Project Engineer	Data Analysis	1	0.5
Staff Engineer or Project Engineer	Project Communication	1	0.5
Staff Engineer or Project Engineer	Shoulder Design	1	1
Staff Engineer or Project Engineer	Memo Development	1	0.5

III.L.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed in a typical shoulder pavement design recommendation. Reference to specific design inputs for rehabilitation or new design development can be located in Section IV “Pavement Design Policies”. Numerous steps contained in this procedure can be completed within several software applications that the Pavement Division currently uses and has under development. The software application tool available to the Pavement Division that can be principally used to complete shoulder pavement designs at this time is DARWin. The following procedure was written to provide the design engineer with adequate information to complete a shoulder design without specific knowledge or access to computer software applications, but with the assumption that these tools were available.

Certain steps in the shoulder pavement design process and other processes overlap. It is important to keep in mind that although these processes are broken out and written in separate sections, they are a part of an overall process to provide logically and technically sound recommendations. The procedure described below is conducted after receiving a request from a customer. This request can be completed via a memo request, e-mail request, or verbal request. The scope of the project should be provided with the shoulder pavement design request. In most cases, a shoulder pavement design is completed in conjunction with a pavement rehabilitation design; i.e. “widening and resurfacing.” Therefore, the new, shoulder, and rehabilitation design process will occur concurrently and use the same data and information. The basic difference is that pavement rehabilitation design requires the evaluation and assessment of the existing roadway, both structurally and functionally. Shoulder and new pavement design does not take into account any of the existing conditions of the pavement other than geotechnical and drainage conditions because it is a new design. It is the pavement designers responsibility to use all design processes concurrently where needed and take care to monitor that both designs are agreeable with one another both for design purposes and for construction related reasons.

The following guidelines should be considered when designing the shoulders:

- Median shoulders less than 4.0’ in width are designed with the same pavement structure as the mainline roadway.
- Median shoulders greater than 4.0’ in width are designed with 2.0’ of the shoulder next to the mainline having the same pavement structure as the mainline roadway. The remaining width of the shoulder is designed following shoulder design guidelines.
- Outside shoulders with a requiring a ESAL range of 3 (greater than 3 million ESAL) or greater for Superpave mix design shall be designed with 2.0’ of the shoulder next to the mainline match the pavement structure of the mainline roadway. The remaining width of the shoulder is designed following shoulder design guidelines.

Follow the steps below to complete the shoulder pavement design.

Step 1. Complete Steps 1 through 5 as documented in Section III.G “New Pavement Design.”

Responsible Party - Engineer

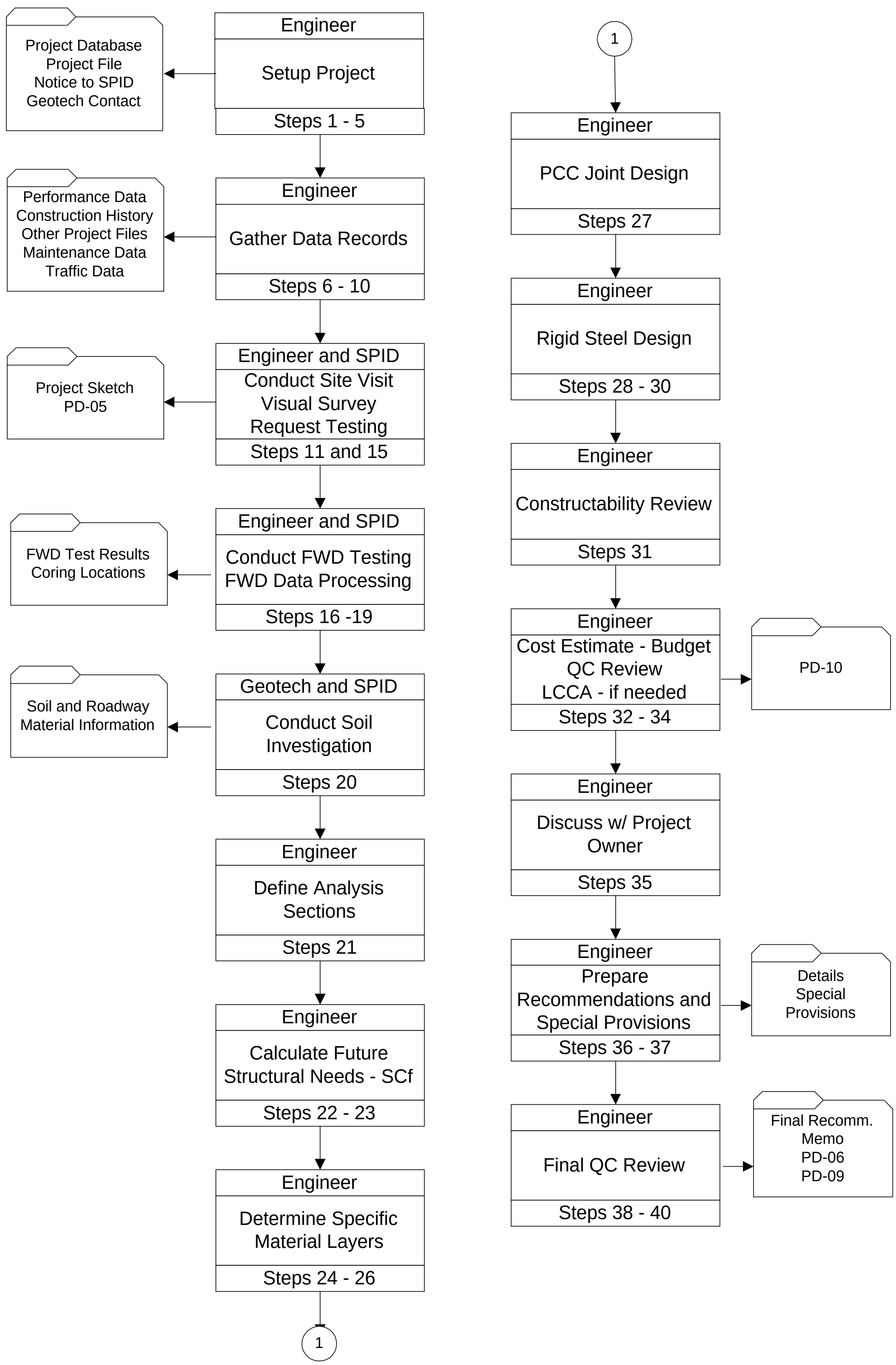
Step 2. Complete Step 6 as documented in Section III.G “New Pavement Design” using the ESAL adjustment factor documented in Section VI. A.3.9 “Pavement Design Policies.”

Responsible Party - Engineer

Step 3. Complete Steps 7 through 40 as documented in Section III.G “New Pavement Design.”

Responsible Party - Engineer

III.L.4 Shoulder Design (New Design) Flowchart



III.M FINAL REVIEW

III.M.1 Purpose

The Final Review process is a critical step in the design process to ensure the design effort is documented properly and to ensure designs from other divisions are incorporated accurately into the final advertised package. A Final Review process ensures the accuracy and the correctness of the contract documents for a project prior to that project being advertised for bids. The review verifies that the designs, plans, specifications, bid quantities, and estimates are accurate and the most recent. In addition, it is an opportunity to verify that designs from different design groups are brought together in an accurate MDSHA project design. For Pavement Division to prepare for Final Review, one must conduct a review or verify the following:

- Verify the accuracy of plans and specifications
- Ensure proposed pavement recommendations can be physically constructed within the constraints of the project
- Verify quantities provided in pavement recommendation are accurately included into the advertised documents
- Verify the pavement recommendations provided are presented accurately in the advertised documents
- Allow our Specification Team to check the entire package prior to advertisement
- Reduce the number Addendum and Extra Work Orders

III.M.2 Resource Requirements

The Final Review procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Verify Recommendations	1	1*
Staff Engineer or Project Engineer	Review Plans	1	2*
Staff Engineer or Project Engineer	Internal Constructability Review	1	1*
Staff Engineer or Project Engineer	Review Specifications	1	1*
Spec Team	Review Entire Package	1	8*
Staff Engineer or Project Engineer	Review Estimate and Quantities	1	2*

* The time required would vary depending on the size and complexity of a project.

III.M.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed for a typical final review of a project. Several steps in the Final Review process may be done out of sequence. It is important that all the steps are completed and not necessarily followed in the order in which they are presented in this section. The Final Review process begins when OMT receives the notice for final review meeting and the final review package. The contract document package could include plans and specifications or in the case of a “proposal” only advertisement, details should be included in the specifications. On projects involving the Geotechnical Explorations Division, their Division should conduct their own review of the contract documents concentrating on Geotechnical issues. There will be some overlap between the Pavement and Geotechnical review, but it is important to concentrate on our own area of responsibility and be aware of other’s areas. All divisions at OMT should work as a team to ensure that all area's OMT is responsible for are done accurately and are sound designs. The review should begin with the plans first and then the specifications.

Step 1. Verify with Geotechnical Explorations Division has received the final contract document package. Discuss with Geotech contact the areas each division would review and attendance at the Final Review meeting. In addition, discuss the time to provide the Specification Team a set of the contract documents to review.

Responsible Party – Engineer and Geotech Contact

Step 2. Verify the existing conditions of the roadway have not significantly changed since the pavement recommendations were developed. These changes include, but are not limited to the following: project scope, alignment, distress conditions, traffic levels, other geometric improvements. This step is critical for review of "shelf" projects and projects done by others.

Responsible Party - Engineer

Step 3. Verify pavement recommendations used to develop final review package meet our current MDSHA Pavement Division design standards and policy for structural and functional condition. This step is critical for review of "shelf" projects and projects done by others.

Responsible Party – Engineer

Step 4. Verify the material used in the project follows MDSHA Pavement Division current material policies. This step is critical for review of "shelf" projects and projects done by others.

Responsible Party – Engineer

Step 5. Review the Title sheet of plans to ensure that the traffic data provided is correct and matches the data used in the rehabilitation design. Verify the soil legend provided on the Title sheet is accurate. Verify the correct contract number and job description is shown.

Responsible Party - Engineer

Step 6. Review the typical section sheets of the plans. Each of the following items shall be reviewed for accuracy and correctness in the typical section sheets:

- Base-widening locations and dimensions shown.
    - Width of widening shown accurately.
- Reconstruction limits and dimensions shown.
- Pavement details shown or referred to on typical sections.
- Correct typical sections shown within specific station limits.
    - Curb and gutter shown in correct location.
    - Varying roadway or widening widths shown as separate typical sections.
    - Longitudinal underdrain (LUD) shown correctly.
    - Guardrail placements interfere with LUD.

Responsible Party - Engineer

Step 7. Review the pavement details sheets of the plans. Each of the following items shall be reviewed for accuracy and correctness in the pavement details sheets:

- Pavement details match our recommendations.
- Correct nomenclature and category code numbers.
- Legend items refer to proper material in detail.
- Notes for details correct.
    - Steel reinforcement and load transfer.
    - Joint dimensions and layout.
    - Particular patching requirements.

Responsible Party - Engineer

Step 8. Review the plan sheets of the plans. Each of the following items shall be reviewed for accuracy and correctness in the plan sheets:

- Rehabilitation strategy shown correctly with shading/legend of plan sheets.
- Identify possible conflicts with utility or hydraulic construction that may have been added since our recommendations were provided.
    - Drain pipes in roadway, wheel path. Construction parallel to roadway or crossing roadway in transverse direction. Excavation needs and dimension. Reconstruction needs or utility patch. Utility patch details provided.
    - Bridges – end walls/wingwalls. Pavement replacement detail.
    - Guardrail or other structure placement interferes with pavement section or construction.
    - Identify between existing, new, and relocated utility lines/drains.
- Project scope changes or limit adjustments. Additional roadway area inclusion after recommendations provided.

Responsible Party - Engineer

Step 9. Review the profile sheets of the plans. This area of the review is the responsibility of the Geotech contact, but the pavement engineer shall be familiar and knowledgeable with the information on the profile sheets. Each of the following items shall be reviewed for accuracy and correctness in the profile sheets:

- Centerline roadway elevation changes – increase or decrease. Verify structural capacity needs.
- Identify low areas or sump areas for longitudinal underdrain need areas.
- Pavement/soil borings shown with results of field and laboratory testing.

Responsible Party – Engineer and Geotech Contact

Step 10. Review the cross section sheets of plans. Some items in this area of the review is the responsibility of the Geotech contact, but the pavement engineer shall be familiar and knowledgeable with all the information on the cross section sheets. Each of the following items shall be reviewed for accuracy and correctness in the cross section sheets:

- Base-widening locations and dimensions shown.
    - Width of widening shown accurately.
- Reconstruction limits and dimensions shown.
- Full width roadway elevation changes – increase or decrease. Check at mainline/shoulder edge for reduction in pavement thickness.
- Cross slope improvements – removal of material to correct cross slope. Verify structural capacity needs. Reconstruction may be needed for reduction requirements.
- Identify fill/cut areas. Identify grading slopes beyond roadway. Verify Class 1A needs based on fill/cut areas. Geotech is responsible for this item.

Responsible Party - Engineer

Step 11. Review the grading chart of the plans, applicable in any new construction or widening projects. This area of the review is the responsibility of the Geotech contact, but the pavement engineer shall be familiar and knowledgeable with the information on the grading chart. Each of the following items shall be reviewed for accuracy and correctness in the grading chart:

- Ensure quantities from cross sections are transposed correctly.
- Verify calculation of quantities is correct.

Responsible Party - Engineer

Step 12. Review the Maintenance of Traffic (MOT) sheets of the plans. Each of the following items shall be reviewed for accuracy and correctness in the MOT sheets:

- Verify duration of MOT pavement sections.
- Verify use of traffic bearing pavement sections exist during all phases of MOT.
- Ensure construction sequence does not conflict with the use of traffic bearing pavement sections.
- MOT pavement sections to be used after construction project complete.
- MOT pavement sections able to be constructed.

Responsible Party - Engineer

Step 13. Review the landscaping sheets of the plans. Each of the following items shall be reviewed for accuracy and correctness in the landscaping sheets:

- Verify any cross walk is traffic bearing and at minimum has the same service life of the pavement rehabilitation. Use of mortar or bedding sand for paver blocks.
- Verify that any unique landscaping, curbs, or other visual enhancements do not adversely effect pavement performance.

Responsible Party - Engineer

Step 14. Review the hydraulic sheets of the plans. Some items in this area of the review is the responsibility of the Geotech contact, but the pavement engineer shall be familiar and knowledgeable with all the information on the hydraulic sheets. Each of the following items shall be reviewed for accuracy and correctness in the hydraulic sheets:

- Ensure storm water management facilities do not adversely effect structural capacity of pavement section. Change of water flow or movement.

Responsible Party - Engineer

Step 15. Review project for construction issues. The construction review shall used to verify pavement section can be constructed given the existing conditions, geometry, MOT phasing, and materials. Every unique aspect of each construction project can not be documented into one procedure. However, several key construction items are consistent among the majority of the projects. The following construction items shall be ease of construction:

- Depth of pavement sections compared to adjacent existing or new pavement sections. Eliminate “bathtub” conditions. Be consistent with the thickness and material type with adjacent pavement sections.
- Utility patch and limit of utility work in roadway. Amount of excavation needed for placement and effect on roadway.
- Sequence of construction relative to placement of different pavement sections. For example, ramps, mainlines, and intersecting roadways.
- Sequence of construction relative to MOT and structural capacity of pavement sections. For example, shoulders and turn lanes used for mainline traffic. Also, grinding, patching, and widening operations relative to functional and structural capacity.
- Width of paving less than 4.0’. Areas less than 4.0’ in width develop compaction issues with HMA material.
- Location and placement of longitudinal underdrain.
- Percentage of reconstruction and resurfacing on a project with respect to the final ride quality and the structural integrity of the project.
- Joint layout and steel design and construction of rigid pavements.

Responsible Party - Engineer

Step 16. Review the specifications. Each of the following items shall be reviewed for accuracy and correctness in the specifications:

- Ensure specifications provided by Pavement Division are included and unedited.
- Verify specifications in 500 and 900 section have the most recent and accurate information and revisions dates.
- Verify correct nomenclature used in "other" special provisions provided other divisions.
- If no plans will be used for advertised package, follow steps 2 through 4 above with the details provided in the specifications.

Responsible Party - Engineer

Step 17. Provide Specification Team with your edited copy of plans and specifications for thorough review. Provide Specification Team with charge number and due date for their comments; preferably a few days prior to the Final Review meeting.

Responsible Party - Engineer

Step 18. Review plans and Specifications in their entirety. Provide comments to engineer by requested due date.

Responsible Party – Specification Team

Step 19. Review bid estimate and quantities. Communication with the project owners is needed in this step to ensure the same correct area values are used to develop quantities. Each of the following items shall be reviewed for accuracy and correctness in the bid estimate and quantities:

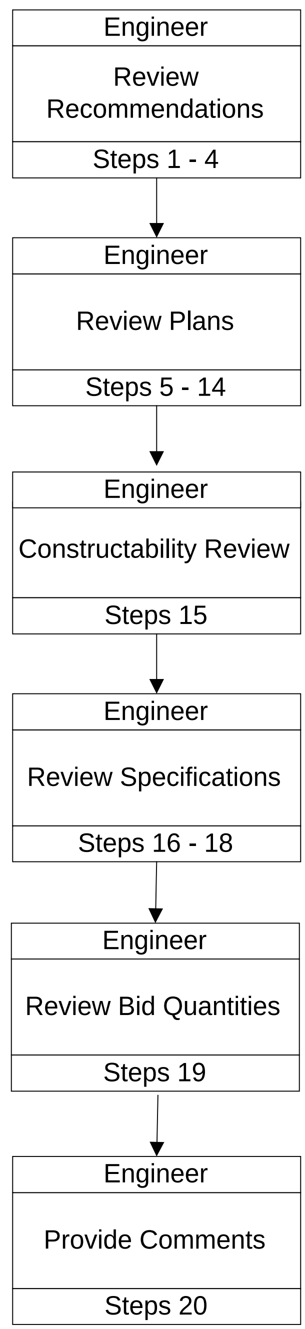
- Verify all quantities needed for pavement rehabilitation are identified and presented accurately.
- Verify actual quantity provided for each item in recommendation is presented accurately. For example patching, Class 1A, removal/replacement of unsuitable material, and longitudinal underdrain.
- Verify nomenclature of material types is correct and consistent with plans and specifications.
- Verify unit costs for paving items are reasonable and accurate.
- Verify actual quantity calculated by project owner provided for each item in the recommendation is presented accurately. For example HMA tonnage or area, grinding, joint tape, etc.

Responsible Party – Engineer

Step 20. Provide oral comments from design team (Pavement, Geotech, and Geology) review and specification team review at final review meeting. If attendance at meeting is not possible, provide written comments from design and specification team to project owner. If comments are significant or project is large and complex, attendance at meeting and written comments are recommended. Comments can be provided via e-mail or memorandum depending on significance of project. Place written comments into project file folder.

Responsible Party – Engineer

III.M.4 Final Review Process Flowchart



III.N VERIFICATION OF CLASSIFICATION DATA

III.N.1 Purpose

On-site classification counts are done to:

- Verify that the classification data provided for design is reasonable
- Verify that the predominant trucks from the selected WIM site are those for the project

III.N.2 Resource Requirements

The classification verification procedure documented below requires the following staffing for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Project or Staff Engineer	Setup Testing	1	1
Survey Technician or Staff Engineer	Data Collection	1	2
Survey Technician or Staff Engineer	Data Entry/Processing	1	1

Reference: Federal Highway Administration's Traffic Monitoring Guide; discussion of vehicle classification system.

III.N.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to obtain a one-hour single lane count. It is expected that this count will be included as a part of the routinely scheduled site survey activities and not as a separate visit. The evaluation described below is conducted from the roadside and preferably off the roadway to avoid distorting lane distribution patterns. Parking in adjacent parking lots or sitting on bridges above the project is recommended. The surveyor is responsible for obtaining any permission required to be on private property and to notify any local authorities of roadside activities as necessary. If the survey is conducted within the right-of-way, the surveyor should be in a survey vehicle equipped with a mounted flashing light.

Step 1. Obtain a copy of the classification data used for traffic forecasts on the project. If by lane data is not available, use data aggregated by by direction.

Responsible Party – Engineer

Step 2. Determine the underlying counts for the WIM location used for ESAL estimation. If by lane data is not available, use the aggregated weight data.

Responsible Party – Engineer

Step 3. Determine the direction convention used for the classification count. Identify the corresponding direction for the project. Both must be consistent with Highway Location Reference. Any discrepancies must be resolved before commencing field work.

Responsible Party – Engineer

Step 4. Ride the full length of the project plus a mile in each direction to determine if a major truck generator such as one of those listed below will influence the project loading estimate:

- Industrial park with a majority of warehousing or manufacturing tenants
- Factory
- Quarry
- Batch mix plant
- Truck stop or rest area
- Freeway exit

Responsible Party – Engineer

Step 5. Select data collection site based on:

- safety
- proximity to an identified major truck generator
- visibility of the roadway
- relative truck volumes (either the heavier direction or larger volume end on the project)

Responsible Party – Engineer

Step 6. Do a 1-hour classification count of the outside lane nearest the data collection location using Form PD-xx. Counts should begin on the hour and end on the hour. All vehicles in the lane are included in the count. For high volume sites, or sites where bi-directional count verification is needed due to the presence of more than one major truck generator, an hour of video tape may be taken for subsequent reduction in the office.

Responsible Party – Engineer

Step 7. Total the observed hourly volumes for each class on Form PD –xx.

Responsible Party – Engineer

Step 8. Enter the reported total daily lane (direction) volumes by class from the classification count used for the traffic forecasts on Form PD-xx.

Responsible Party – Engineer

Step 9. Enter the reported total daily lane (direction) volumes by class from the WIM count used for the ESAL estimate on Form PD-xxx.

Responsible Party – Engineer

Step 10. Compute the comparison percentages as number of a given class over the total number in classes 6 through 13 * 100 rounded to the nearest integer.

Responsible Party – Engineer

Step 11. Determine whether appropriate matches exist. The same classes should be present as 10 percent or more of the truck population in classes 6 through 13 for a match to exist.

Responsible Party – Engineer

Step 12. If a match does not exist consult with MD SHA Traffic Forecasting Section the possible source of differences.

Responsible Party – Engineer

Step 13. Place all completed forms in project file.

Responsible Party - Engineer

III.N.4 Classification Counting Training and Qualification

An individual is considered capable of conducting classification counts if they have a 95% accuracy rate for a one-hour real time classification count. Training in classification counting is done on a self-paced basis and consists of a five-step process including two separate tests. Tests may be repeated until passed.

Step 1. Introduction to Classification Counting – A short PowerPoint slide show on the training process and the classification system.

Responsible Party – Team Leader and Team Member

Step 2. Practice identifying vehicles by class using flash cards. Cards should be shuffled and reviewed randomly.

Responsible Party – Team Member

Step 3. Take a quiz using 25 randomly selected classification cards. There is no time limit. Cards should not be shuffled between selection and scoring of test. A passing score is 23 or more correctly identified.

Responsible Party – Team Leader and Team Member

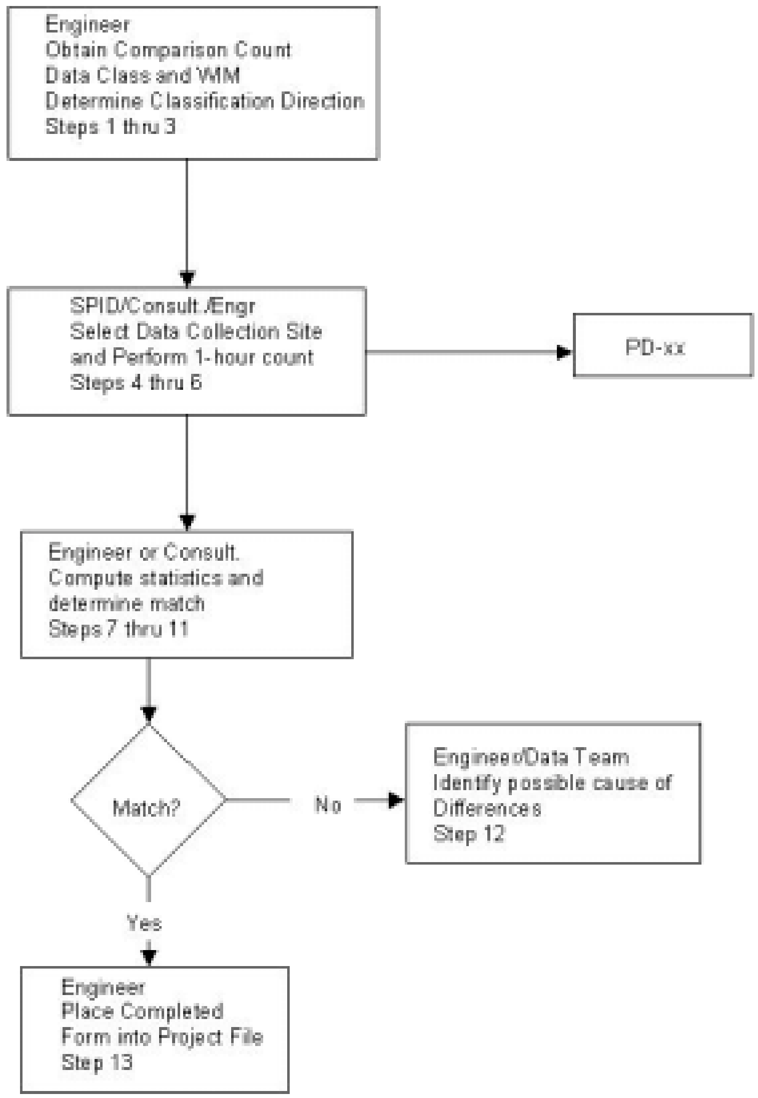
Step 4. Practice identifying vehicles in real time using one or more CD-ROM videos with sites which have volumes consistent with most projects in the individual's district. Five minute volumes are available to check answers.

Responsible Party –Team Member

Step 5. Take a quiz using one or more CD-ROMs and four 15-minute intervals. A passing score is 95% or more correctly counted and identified.

Responsible Party – Team Leader and Team Member

III.N.5 Verification of Classification Data Process Flowchart



III.O PAVEMENT MARKING ERADICATION

III.O.1 Purpose

The type of lane marking and the condition of the lane marking will effect pavement rehabilitation recommendations. Identifying the type and condition of the lane marking will aid in the following:

- Establishing the overall condition of the pavement markings
- Determining if the pavement markings need to be removed
- Determining the procedure for removing pavement markings

III.O.2 Resource Requirements

The pavement marking visual survey procedure documented below requires the following staffing needs for a typical job:

Position	Function	Resources	Effort Level (man-hours)
Staff Engineer or Project Engineer	Site Visit	1	4
Staff Engineer or Project Engineer	Data Collection	1	1
Staff Engineer or Project Engineer	Determination of Lane Marking Eradication	1	1

III.O.3 Procedure

The procedure presented in the attached flowchart and described in the following text should be followed to rate the pavement marking condition for all pavement related projects. The visual survey to determine the type of lane marking and its condition should be performed during the Visual Survey for Pavement Rehabilitation Design. It is important to identify the pavement marking material and condition of the pavement markings because it may affect the Pavement Rehabilitation Recommendations. The evaluation described below is conducted while walking slowly alongside the highway pavement. Protective clothing must be worn (safety vest and hard hat) at all times when conducting the survey. In addition, a survey vehicle equipped with a mounted flashing light should be parked behind the surveyor or at a location visible to traveling motorists.

Step 1. Ride full length of job in each direction in the slowest travel lane to identify the overall condition of the pavement lane markings.

Responsible Party – Engineer

Step 2. Identify the following:

- The pavement marking material, which may be one or a combination of the following materials (see the chart on Page II.M-2 for definitions of pavement marking materials):
  1. Paint (see Figure 1)
  2. Thermoplastic (see Figure 2)
  3. Paint over Thermoplastic

4. Patterned or Non-Patterned Tape (see Figures 3 and 4)
5. Epoxy

Pavement Marking Material	***Material Description***
Paint	Water-borne, non-durable pavement markings composed of a mixture of pigments, resins, fillers, and water.
Thermoplastic	Durable, high-quality pavement marking. The 100% solid mixture formula is composed of either maleic-modified glycerol resin esters or hydrocarbon resins with plasticizers, and glass beads that become liquid with heat.
Patterned and Non-Patterned Tape	Durable pavement marking that is cold applied. Typically, tape is comprised of three layers: adhesive layer, backing material, and pigment and glass bead layer.
Epoxy	Durable, high quality pavement marking that is 100% solid. Two-component material composed of a pigmented resin and a catalyst for hardening.

Step 3: Determine the percentage of bead coverage (see Figures 5 and 6) :

1. Low Bead Percentage (5%-50%)
2. Medium Bead Percentage (50%)
3. High Bead Percentage (50%-100%)

Rule of Thumb: Look at the pavement markings with your back to the sun (the shadow of your head should be near the movement marking) to determine the percentage of bead coverage.

Responsible Party – Engineer or (SPID)

Step 4. Determine the wear condition of the pavement lane markings. The condition of the lane markings will have severity level similar to that of pavement distresses. See Table 1, "Pavement Marking Wear Condition Guidelines", for guidelines to determine pavement lane marking condition.

Responsible Party – Engineer or SPID

Step 5. Determine how the lane markings should be removed. **Keep in mind that the lane markings should be removed in the method that is least damaging to the pavement.** Use the following guidelines to determine the method of removal for pavement markings.

Pavement Marking Removal Guidelines:

Paint:

Low Severity Paint Markings on Asphalt Surface and adding an asphalt lift </=1"

- Hydroblasting
- Hydroblasting with grit

Note: Removal is necessary since an asphalt lift of </=1" will not adequately cover paint markings.

Low Severity Paint Markings on Asphalt Surface and adding an asphalt lift >1"

- Removal is not necessary. An asphalt lift of >1" will adequately cover paint markings.

Medium and High Severity Paint Markings

- Removal is not necessary

Thermoplastic:

Low to High Severity Thermoplastic

- Hydroblasting
- Hydroblasting with grit
- Bead/Pellet blasting less than 1/8 in. depth
- Grinding

Tape:

Low to High Severity Tape

- Manually Scrape
- Hydroblasting
- Hydroblasting with grit
- Bead/Pellet blasting less than 1/8 in. depth
- Grinding

Epoxy:

Low to High Severity Epoxy

- Hydroblasting
- Hydroblasting with grit
- Bead/pellet blasting less than 1/8 in. depth
- Grinding

Responsible Party – Engineer or SPID

Step 5. Information from Steps 2-5 should be gathered and Form PD-XX should be completed. Place the completed forms in project file.

Responsible Party - Engineer

Step 6. Follow the Steps in Section III.F "Pavement Rehabilitation Design" and consider the pavement marking types and removal while developing rehabilitation recommendations. If applicable, include the Pavement Eradication Specification.

III.O.4 Pavement Marking Photos

Figure 1. High Severity Wear Paint Lane Marking.

Figure 2. High Severity Wear Thermoplastic Lane Marking.

Figure 3. Medium Severity Wear Patterned Tape Lane Marking.

Figure 4. High Severity Wear Non-Patterned Tape Lane Marking.

III.O.5 Percentage of Bead Coverage

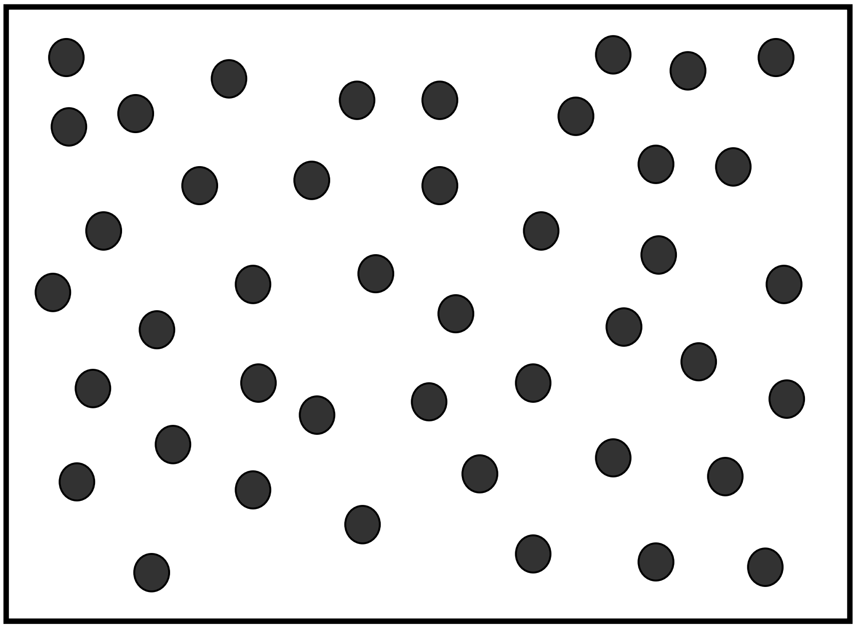


Figure 5. 5% Bead Coverage

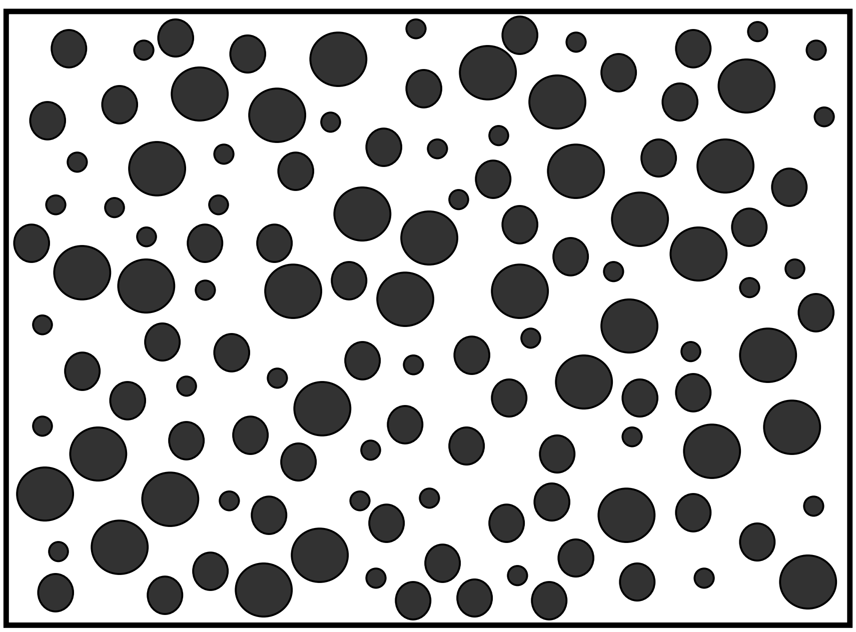


Figure 6. 50% Bead Coverage

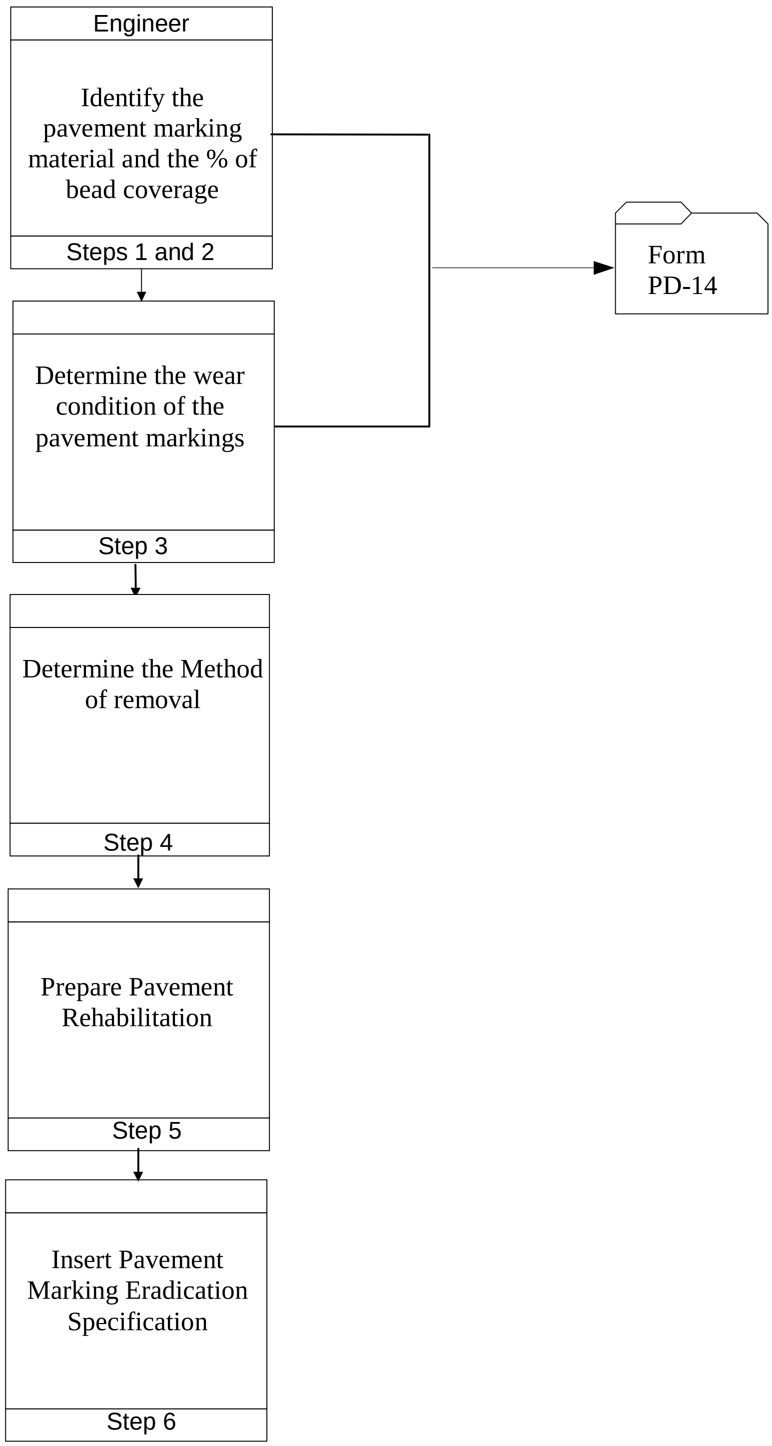
III.O.6 Pavement Marking Condition Guidelines

Table 1.

Pavement Marking Condition	***Paint and Thin Thermoplastic***	***Thermoplastic***	***Paint over Thermoplastic***	***Tape***	***Epoxy***
Low Severity Wear	• Paint is highly visible • High bead percentage • Paint and beads are reflective	• Lane markings adhere to the pavement • Lane markings are not delaminating	• Lane markings are highly visible • Lane markings are adhering to the pavement • Paint is reflective • Lane markings are not delaminating	• Lane markings adhere to the pavement • Lane markings are not delaminating	• Epoxy is highly visible • High bead percentage • Epoxy and beads are reflective
Medium Severity Wear	• Paint is starting to wear away from the pavement surface • Paint is fading and not highly reflective • Medium bead percentage	• Lane marking is starting to chip away from the pavement surface	• Paint is fading and not highly effective • Lane markings are starting to chip away from the pavement surface	• Lane marking is starting to chip away from the pavement surface	• Epoxy is starting to wear away from the pavement surface • Epoxy is fading and not highly reflective • Medium bead percentage
High Severity Wear	• Paint is not easily visible • Paint is not reflective	• Lane markings are peeling away from the pavement surface • A lot of delamination is present	• Paint is faded and not highly reflective • Lane markings are pealing away from the pavement surface • A lot of delamination is present	• Lane markings are peeling away from the pavement surface • A lot of delamination is present	• Epoxy is not easily visible • Epoxy is not reflective

Responsible Party – Engineer or SPID

III.O.7 Visual Survey Process Flowchart



IV FORMS

In order to standardize pavement data collection analysis efforts, several forms were developed for pavement designers to use while developing pavement recommendations. These forms are referred to and described in the procedures in Section III of this pavement design guide. The following table lists all the current forms, their purpose, and whether the form currently is used as part of the standard operating procedure:

Form	Form Name	Purpose	Used?
PD-01	FY Project Estimate Summary Form	Estimate	Yes
PD-02	Flexible/Composite Pavement Visual Survey Form	PCI Surveys	Yes
PD-02a	Rigid Pavement Visual Survey Form	PCI Surveys	Yes
PD-03	Sampling Rate Form	PCI Surveys	Yes
PD-04	Rehabilitation Design Condition Summary Form	Estimate	Yes
PD-05	Field Testing Request Form	Request	Yes
PD-06	Pavement Rehabilitation Design QC Form	QC	Yes
PD-06a	PCI Visual Survey Quality Control Form	QC	Yes
PD-06b	Patching Survey Quality Control Form	QC	Yes
PD-06c	FWD Data Quality Control Form	QC	
PD-07	Pavement Patching Survey Form	Patching	Yes
PD-08	Patching Quantity Spreadsheet	Patching	Yes
PD-09	Project Summary Information Form	Design	Yes
PD-10	Pavement Rehabilitation Alternatives Form	Design	Yes
PD-11	Ride Quality Specification Guideline Form	Design	Yes
PD-12			
PD-13	PE Cost Estimate Form	Estimate	Yes

The remaining portion of this section provides copies of the current forms used by the pavement design unit of the Pavement Division of OMT.

V UNIT COST

The cost of materials and construction operations is a critical element in determining the most beneficial and cost effective pavement recommendation for MDSHA projects. The unit cost of particular construction materials and operations has been used extensively in life cycle costs analysis (LCCA) procedures like the MDSHA process described in Section III.H. The MDSHA Pavement Division of OMT has taken the unit cost information beyond LCCA for use in determining the most economical rehabilitation alternatives for MDSHA. This approach is documented in several areas of Section III.

MDSHA regularly produces a "Price Index", that provides the specific bid prices of a majority of the construction materials and operations used in MDSHA projects. The contract numbers, quantity, and bid price of all advertised contract materials and operations are stored in the MDSHA Transport system. A category code number references each item in the database. Price differences based on quantity amounts can be calculated from this data. Regional and local price difference can also be observed with this type of data storage. Not all items have category code numbers that would allow the data to be collected and stored, therefore some "write-in" items will not be available in the "Price Index." An electronic form of this data can also be obtained by using the software program "Estimator." This program provides the same information as the "Price Index", but allows greater flexibility with the type and number of filters on the requested information. Both of these items are readily available to all pavement designers who are expected to use the information in the development of their pavement recommendations.

The Pavement Management Unit of the Pavement Division also keeps a database of pavement-related materials and operations. These include special items and "write-in" items to recorded in the other two sources for unit prices. All unit prices are extremely dependent on the quantity of material in the project. Actual prices will vary widely depending on the contractor and their approach to bidding and figuring profit in various project items. In the tables below are some common pavement materials and operations and the current range of unit prices:

HMA Items	Typical Unit Price	Reasonable Range
HMA 12.5 mm Gap Graded PG 76-22	$50 / ton	$40 – 65 / ton
HMA 12.5 mm Gap Graded PG 70-22	$45 / ton	$35 – 55 / ton
HMA 12.5 mm PG 76-22, 8 PV	$48 / ton	$40 – 55 / ton
HMA 12.5 mm PG 70-22, 8 PV	$42 / ton	$37 – 50 / ton
HMA 12.5 mm PG 64-22	$36 / ton	$30 – 40 / ton
HMA 19.0 mm or 25.0 mm PG 64-22	$32 / ton	$25 – 36 / ton
PCC Items	Typical Unit Price	Reasonable Range
New PCC with no steel – 10.0"	$40 / sy	$30 – 50 / sy
New PCC with steel reinforcement – 10.0"	$50 / sy	$40 – 60 / sy
PCC Patching	$125 / sy	$100 – 200 / sy

Note: At the time of the developing of this page, there has been no difference in price between PG 70-22 versus PG 64-22. All evidence has shown that PG 64-28 is also the same price, or $1 or $ 2 more a ton. There has been no significant difference in contract price for different HMA traffic levels. All HMA prices are highly dependent on oil prices.

Geotechnical Items	Typical Unit Price	Reasonable Range
Common Borrow	$7 / cy	$5 – 10 / cy
Select Borrow	$10 / cy	$8 – 12 / cy
Capping Borrow	$12 / cy	$10 – 15 / cy
Modified Borrow	$14 / cy	$12 – 17 / cy
Graded Aggregate Base – 6.0”	$ 7 / sy	$4 – 10 / sy
Unsuitable Material Replacement	$33 / cy	$25 – 40 / cy
Class 1 Excavation	$10 / cy	$6 – 15 / cy
Class 1A Excavation	$16 / cy	$10 – 25 / cy
Class 2 Excavation	$13 / cy	$10 – 20 / cy
Pre-Overlay/Maintenance Items	Typical Unit Price	Reasonable Range
HMA 4.75 mm PG 64-22, Wedge/Level	$40 / ton	$30 – 50 / ton
HMA 9.5 mm PG 64-22, Wedge/Level	$35 / ton	$25 – 40 / ton
HMA 19.0 / 25.0 mm PG 64-22, Partial-Depth Patching	$45 / ton $25 / sy	$35 – 55 / ton $15 – 35 / sy
HMA 19.0 / 25.0 mm PG 64-22, Full-Depth Patching	$55 / ton $30 / sy	$45 – 65 / ton $20 – 40 / sy
Milling / Grinding	$1.50 / sy	$1.00 – 2.00 / sy
Latex Modified Slurry Seal – One application	$1.00 / sy	$0.75 – 1.25 / sy
Novachip	$4.00 / sy	$3.00 – 5.00 / sy
Typical HMA Overlay – 1.0”	$2.00 / sy	$1.75 – 2.25 / sy

VI PAVEMENT DESIGN POLICIES

Pavement recommendations for MDSHA construction projects require both pavement and material design. Pavement design and material policies were developed to provide guidance to and consistency between MDSHA pavement designers. The following is a list of the current pavement design and material policies and their appropriate sections:

VI.A DESIGN INPUT POLICIES

The MDSHA pavement design process requires several AASHTO and MDSHA standard design inputs. These inputs vary depending on the type of design that is developed, new and rehabilitation design, ESALs to Failure, shoulder design, and temporary road design. These design inputs will provide a level of consistency between designers and across the state of Maryland.

VI.A.1 New and Rehabilitation Pavement Design

The design inputs required for new and rehabilitation pavement design vary depending on the existing conditions at the project site, the functional classification of the roadway, and the pavement type. These design inputs follow the same guidelines as those developed by AASHTO, but modified for local conditions. The design inputs in Section VI.A.1 are needed to develop new and rehabilitation pavement design recommendations.

VI.A.1.1 Design Life (New/Rehab)

If a project is assigned a design life through the use of the tools in the Maryland Pavement Management System (PMS), then the selected design service life shall be used in pavement design. Otherwise, if the PMS did not assign the project an action class, select the design service life based on the following table:

Pavement Rehabilitation Technique	Desired Design Service Life	Minimum Design Service Life
HMA Overlay	12	8
PCC Bonded Overlay	15	12
PCC Unbonded Overlay	20	20
Rubblize and HMA Overlay	15	15
Break and Seat and HMA Overlay	15	12
Saw and Seal with HMA Overlay	12	8
Ultra-Thin Whitetopping	10	8
In-Place Recycled HMA Overlay	8	5
Recycled HMA Overlay	12	8
CPR	8	5
New Flexible Construction and Widening	15	15
New Rigid Construction and Widening	25	25

VI.A.1.2 Reliability, R (New/Rehab)

Use the following table to select the appropriate Reliability based on the functional class of the roadway:

	Urban		Rural	
Functional Class	**Desired Reliability**	**Minimum Reliability**	**Desired Reliability**	**Minimum Reliability**
Interstate	95	90	95	85
Freeways and Expressways	90	85		
Principal Arterial – Other	90	85	90	80
Minor Arterial	90	80	85	80
Major Collector			80	75
Minor Collector			80	75
Collector	85	75		
Local	80	70	75	70

VI.A.1.3 Standard Deviation, S (New/Rehab)

Use the following table to select the appropriate Standard Deviation based on the pavement type of the roadway:

	Standard Deviation (SD)	
Pavement Type	**Desired SD**	**Minimum SD**
Flexible	0.49	0.45
Rigid	0.39	0.35
Composite	0.39	0.35

VI.A.1.4 Initial Serviceability, PSI_I (New/Rehab)

Use the following table to select the appropriate Initial Serviceability based on the resulting pavement surface material, the type of construction (rehabilitation / new construction), the use of the ride incentive specification, and the required ride quality after resurfacing of the project:

Final Surface Material	Type of Construction	Required Profile Index	Initial Serviceability
HMA	New	N/A	4.2
HMA	Rehabilitation	7	*
		10	*
		12	*
		15	*
		Ride Spec Waived	*
PCC	New	N/A	4.5
PCC	Rehabilitation		4.5

* A research study is currently underway to determine a correlation between IRI, profile index, and the ride quality of the final resurfacing. Until those values become available use the initial serviceability for new construction.

The selection process for initial serviceability will be an iterative process based on the thickness of the required overlay and the selection of pre-overlay treatments. The number of lifts placed and the type of pre-overlay treatments dictate the required profile index to the existing roadway. The selection of the initial serviceability may dictate the thickness of the overlay and type of pre-overlay treatments. Therefore, an iterative process may be required.

VI.A.1.5 Terminal Serviceability, PSI_t (New/Rehab)

Use the following table to select the appropriate terminal serviceability based on the functional class of the roadway:

	Terminal Serviceability	
Functional Class	**Desired**	**Minimum**
Interstate	3.0	3.0
Freeways and Expressways	2.9	2.8
Principal Arterial – Other	2.9	2.7
Minor Arterial	2.8	2.5
Major Collector	2.6	2.4
Minor Collector	2.6	2.4
Collector	2.6	2.4
Local	2.4	2.2

VI.A.1.6 Load Transfer Coefficient, J (New/Rehab)

Use the following table to select the appropriate load transfer coefficient for new rigid pavement design based on the type of pavement and shoulders:

Shoulder Type	**Asphalt Concrete**				**Tied PCC**			
Load Transfer Devices	**Yes**		**No**		**Yes**		**No**	
Pavement Type	**JPCP/ JRCP**	**CRCP**	**JPCP/ JRCP**	**CRCP**	**JPCP/ JRCP**	**CRCP**	**JPCP/ JRCP**	**CRCP**
Load Transfer Factor, J	3.2	2.9 to 3.2	3.8 to 4.4	N/A	2.5 to 3.1	2.3 to 2.9	3.6 to 4.2	N/A

Use the following table to select the appropriate load transfer coefficient for pavement rigid or composite pavement rehabilitation design based on the load transfer capability of the joints in the roadway:

Average Load Transfer in Section	**Load Transfer Coefficent, J**
> 70%	3.2
50% to 70%	3.5
< 50%	4.0

VI.A.1.7 Loss of Support, LS (New/Rehab)

Use the following table to select the appropriate loss of support value for determining the modulus of subgrade reaction (k) without FWD testing based on the base material types in the pavement structure:

Treatment	All Rehab Methods	New PCC w/ AC or PCC treated base	New PCC w/ lime stabilized subgrade or cement soil	New PCC w/ unbound GAB	New PCC w/ no base material
	0	0	1	2	3

VI.A.1.8 Overall Drainage Coefficient, Cd (New/Rehab)

Refer to Table 2.5 on page II-26 in the AASHTO "Guide for Design of Pavement Structures". Confer with some from Geotechnical Explorations to provide a more detailed description for moisture and soil conditions particular to Maryland.

VI.A.1.9 Traffic Lane Distribution (New/Rehab)

Use the following table to select the appropriate traffic lane distribution factor based on the number lanes in each (design) direction.

Number of Lanes in Design Direction	Range of Percent of ESAL in Design Lane	Desired Percent of ESAL in Design Lane
1	100	100
2	80 – 100	90
3	60 – 80	80
4	50 – 75	70
5+	40 – 70	60

VI.A.2 ESALs to Failure

ESALs to failure is needed to calculate the effective structural capacity of a pavement structure based on traffic remaining life analysis. ESALs to failure is the number of 18-kip axle applications required to fail a given pavement structure, a terminal serviceability of 1.5. This number of applications is then compared to actual number of applications to a pavement structure.

VI.A.2.1 Reliability, R (ESALs to Failure)

All pavement types are designed with a reliability of **50%** for ESALs to failure.

VI.A.2.2 Standard Deviation, S (ESALs to Failure)

Use the following table to select the appropriate Standard Deviation based on the pavement type of the roadway:

	Standard Deviation (SD)	
Pavement Type	**Desired SD**	**Minimum SD**
Flexible	0.49	0.45
Rigid	0.39	0.35
Composite	0.39	0.35

VI.A.2.3 Initial Serviceability, PSI_I (ESALs to Failure)

Use the following table to select the appropriate Initial Serviceability based on the resulting pavement surface material of the project:

Final Surface Material	Initial Serviceability
HMA	4.2
PCC	4.5

VI.A.2.4 Terminal Serviceability, PSI_t (ESALs to Failure)

The terminal serviceability for all pavement types is specified to be **1.5**.

VI.A.2.5 Load Transfer Coefficient, J (ESALs to Failure)

Use the following table to select the appropriate load transfer coefficient based on the load transfer capability of the joints, existing pavement type at the time of the last rehabilitation, type of shoulder, and the existence of the load transfer devices in the roadway:

Existing Pavement Type at Time of Last Rehabilitation	Type of Shoulder	Load Transfer Device	Load Transfer Coefficent, J
JPCP, JRCP	AC	Yes	3.2
JPCP, JRCP	AC	No	3.8 – 4.0
JPCP, JRCP	Tied PCC	Yes	2.8 – 3.1
JPCP, JRCP	Tied PCC	No	3.6 – 3.8
CRCP	AC	N/A	2.9 – 3.2
CRCP	Tied PCC	N/A	2.6 – 2.9
Composite Pavement	N/A	N/A	3.2 – 3.5

VI.A.2.6 Overall Drainage Coefficient, Cd (ESALs to Failure)

Refer to Table 2.5 on page II-26 in the AASHTO "Guide for Design of Pavement Structures". Confer with someone from Geotechnical Explorations Division to provide a more detailed description for moisture and soil conditions particular to Maryland.

VI.A.2.7 Traffic Lane Distribution (ESALs to Failure)

Use the following table to select the appropriate traffic lane distribution factor based on the number lanes in each (design) direction.

Number of Lanes in Design Direction	Range of Percent of ESAL in Design Lane	Desired Percent of ESAL in Design Lane
1	100	100
2	80 – 100	90
3	60 – 80	80
4	50 – 75	70
5+	40 – 70	60

VI.A.3 Shoulder Design

The design inputs required for shoulder pavement design vary depending on the existing conditions at the project site, the functional classification of the roadway, and the pavement type. These design inputs follow the same guidelines as those developed by AASHTO, but modified for local conditions. The design inputs are similar to those for new and rehabilitation design with the exception of the ESAL adjustment factor.

VI.A.3.1 Design Life (Shoulder)

If a project is assigned a design life through the use of the tools in the Maryland Pavement Management System (PMS), then the selected design service life shall be used in pavement design. Otherwise, if the PMS did not assign the project an action class, select the design service life based on the following table:

Pavement Rehabilitation Technique	Desired Design Service Life	Minimum Design Service Life
HMA Overlay	12	8
PCC Bonded Overlay	15	12
PCC Unbonded Overlay	20	20
Rubblize and HMA Overlay	15	15
Break and Seat and HMA Overlay	15	12
Saw and Seal with HMA Overlay	12	8
Ultra-Thin Whitetopping	10	8
In-Place Recycled HMA Overlay	8	5
Recycled HMA Overlay	12	8

Pavement Rehabilitation Technique	Desired Design Service Life	Minimum Design Service Life
CPR	8	5
New Flexible Construction and Widening	15	15
New Rigid Construction and Widening	25	25

VI.A.3.2 Reliability, R (Shoulder)

Use the following table to select the appropriate Reliability based on the functional class of the roadway:

	Urban		Rural	
Functional Class	**Desired Reliability**	**Minimum Reliability**	**Desired Reliability**	**Minimum Reliability**
Interstate	95	90	95	85
Freeways and Expressways	95	90		
Principal Arterial – Other	95	85	90	85
Minor Arterial	90	80	85	80
Major Collector			80	75
Minor Collector			80	75
Collector	85	75		
Local	80	70	75	70

VI.A.3.3 Standard Deviation, S (Shoulder)

Use the following table to select the appropriate Standard Deviation based on the pavement type of the roadway:

	Standard Deviation (SD)	
Pavement Type	**Desired SD**	**Minimum SD**
Flexible	0.49	0.45
Rigid	0.39	0.35
Composite	0.39	0.35

VI.A.3.4 Initial Serviceability, PSI_I (Shoulder)

Use the following table to select the appropriate Initial Serviceability based on the resulting pavement surface material, the type of construction (rehabilitation / new construction), the use of the ride incentive specification, and the required ride quality after resurfacing of the project:

Final Surface Material	Type of Construction	Initial Serviceability
HMA	New	4.2
HMA	Rehabilitation	4.2
PCC	New	4.5
PCC	Rehabilitation	4.5

* A research study is currently underway to determine a correlation between IRI, profile index, and the ride quality of the final resurfacing. Until those values become available use the initial serviceability for new construction.

The selection process for initial serviceability will be an iterative process based on the thickness of the required overlay and the selection of pre-overlay treatments. The number of lifts placed and the type of pre-overlay treatments dictate the required profile index to the existing roadway. The selection of the initial serviceability may dictate the thickness of the overlay and type of pre-overlay treatments. Therefore, an iterative process may be required.

VI.A.3.5 Terminal Serviceability, PSI_t (Shoulder)

Use the following table to select the appropriate terminal serviceability based on the functional class of the roadway:

	Terminal Serviceability	
Functional Class	**Desired**	**Minimum**
Interstate	3.0	3.0
Freeways and Expressways	2.9	2.8
Principal Arterial – Other	2.9	2.7
Minor Arterial	2.8	2.5
Major Collector	2.6	2.4
Minor Collector	2.6	2.4
Collector	2.6	2.4
Local	2.4	2.2

VI.A.3.6 Load Transfer Coefficient, J (Shoulder)

Use the following table to select the appropriate load transfer coefficient for new rigid pavement design based on the type of pavement and shoulders:

Shoulder Type	Asphalt Concrete				Tied PCC			
Load Transfer Devices	Yes		No		Yes		No	
Pavement Type	JPCP/ JRCP	CRCP	JPCP/ JRCP	CRCP	JPCP/ JRCP	CRCP	JPCP/ JRCP	CRCP
Load Transfer Factor, J	3.2	2.9 to 3.2	3.8 to 4.4	N/A	2.5 to 3.1	2.3 to 2.9	3.6 to 4.2	N/A

Use the following table to select the appropriate load transfer coefficient for pavement rigid or composite pavement rehabilitation design based on the load transfer capability of the joints in the roadway:

Average Load Transfer in Section	Load Transfer Coefficent, J
> 70%	3.2
50% to 70%	3.5
< 50%	4.0

VI.A.3.7 Loss of Support, LS (Shoulder)
Use the following table to select the appropriate loss of support value for determining the modulus of subgrade reaction (k) without FWD testing based on the base material types in the pavement structure:

Treatment	All Rehab Methods	New PCC w/ AC or PCC treated base	New PCC w/ lime stabilized subgrade or cement soil	New PCC w/ unbound GAB	New PCC w/ no base material
	0	0	1	2	3

VI.A.3.8 Overall Drainage Coefficient, Cd (Shoulder)
Refer to Table 2.5 on page II-26 in the AASHTO “Guide for Design of Pavement Structures”. Confer with some from Geotechnical Explorations to provide a more detailed description for moisture and soil conditions particular to Maryland.

VI.A.3.9 ESAL Adjustment Factor (Shoulder)
The design ESALs for the pavement design of the shoulder will be based on a percentage of the design ESALs for the mainline roadway. Use the following table to select the appropriate ESAL adjustment factor based on the functional class of the roadway:

Functional Class	Percent of Design ESALs

Interstate	100%
Freeways and Expressways	100%
Principal Arterial – Other	100%
Minor Arterial	10%
Major Collector	10%
Minor Collector	10%
Collector	10%
Local	10%

VI.A.3.10 Traffic Lane Distribution (Shoulder)

Use the following table to select the appropriate traffic lane distribution factor based on the number lanes in each (design) direction.

Number of Lanes in Design Direction	Range of Percent of ESAL in Design Lane	Desired Percent of ESAL in Design Lane
1	100	100
2	80 – 100	90
3	60 – 80	80
4	50 – 75	70
5+	40 – 70	60

VI.A.4 Temporary / Detour Road Design

The design inputs required for temporary / detour road pavement design vary depending on the existing conditions at the project site, the functional classification of the roadway, and the pavement type. These design inputs follow the same guidelines as those developed by AASHTO, but modified for local conditions. The design inputs vary from new and rehabilitation design to account for the abbreviated service life and into account that the pavement is taken to failure at the conclusion of the service life.

VI.A.4.1 Design Life (Temporary)

The design life for a temporary or detour road shall be the maximum of 1 of the following:

- **MOT construction duration – verify with highway design and construction inspection**
- **2 years**

VI.A.4.2 Reliability, R (Temporary)

Temporary / detour roads shall be designed with a Reliability of **50%.**

VI.A.4.3 Standard Deviation, S (Temporary)

Use the following Standard Deviation to design temporary / detour roads:

Standard Deviation (SD)	
Desired SD	**Minimum SD**
0.49	0.45

VI.A.4.4 Initial Serviceability, PSI_I (Temporary)

Temporary / detour roads shall be designed with an Initial Serviceability of **4.2**.

VI.A.4.5 Terminal Serviceability, PSI_t (Temporary)

Temporary / detour roads shall be designed with a Terminal Serviceability of **2.0**.

VI.A.4.6 Other Factors (Temporary)

Temporary / detour roads shall be designed with the goal to minimize the amount of HMA material placed in the temporary pavement structure. The goal should be to utilize graded aggregate base to the fullest extend to keep material costs for a temporary pavement structure to a minimum.

VI.A.4.7 Traffic Lane Distribution (Temporary)

Use the following table to select the appropriate traffic lane distribution factor based on the number lanes in each (design) direction.

Number of Lanes in Design Direction	Range of Percent of ESAL in Design Lane	Desired Percent of ESAL in Design Lane
1	100	100
2	80 – 100	90
3	60 – 80	80
4	50 – 75	70
5+	40 – 70	60

VI.B MATERIAL SELECTION POLICIES

There was a need to develop a section of material selection policies because of the numerous pavement material designations available to a MDSHA pavement designer. Policies for material selection provide both guidance to and consistency between MDSHA pavement designers. This need is especially true with the adoption of the Superpave HMA mix design criteria and the numerous types of HMA mixes that were generated. In addition, policy and guidance for wearing coarse selection and reflective crack control are also addressed in this section.

VI.B.1 Superpave HMA Criteria

This adoption of the Superpave HMA mix design criteria and the numerous types of HMA mixes that were generated sparked the need for policy in material selection. Material selection items crucial to the Superpave HMA mix criteria include the performance grade binder, traffic or compaction level (ESAL category), and HMA mix size. In addition, policies regarding the type and number of HMA mixes on a typical construction project are also provided.

VI.B.1.1 Performance Grade Binder Selection

The base performance grade (PG) Binder for the state of Maryland is PG 64-22. All HMA mixes, with the exception of the wearing coarse, shall always be designed with PG 64-22 as the binder, unless the Pavement Design Policies or the Asphalt Team or site conditions dictate otherwise. The wearing coarse PG binder can vary from PG 64-22 based on the following conditions: geographic area, ESAL category, and rutting conditions. Use the following table to select the appropriate PG binder for the project:

Wearing Surface for All Counties except Garrett:

		< 0.3		0.3 to 30		> 30	
		No Rut	Rut	No Rut	Rut	No Rut	Rut
< 1000 tons	Standard	64-22	64-22	64-22	70-22	70-22	70-22 P
	Slow	64-22	64-22	70-22	70-22 P	70-22	70-22 P
	Standing	64-22	70-22	70-22 P	70-22 P	70-22 P	70-22 P
> 1000 tons	Standard	64-22	64-22	64-22	70-22	70-22	76-22
	Slow	64-22	64-22	70-22	76-22	70-22	76-22
	Standing	64-22	70-22	76-22	76-22	76-22	76-22

Standing Traffic - where the average traffic speed is less than 12 mph (20 km/h).

Slow Traffic - where the average traffic speed ranges from 12 to 43 mph (20 to 70 km/h).

Standard Traffic - where the average traffic speed is greater than 43 mph (70 km/h).

Wearing Surface for Garrett County only:

		< 0.3		0.3 to 30		> 30	
		No Rut	Rut	No Rut	Rut	No Rut	Rut
< 1000 tons	Standard	64-28	64-28	64-28	64-28	70-22	70-22 P
	Slow	64-28	64-28	70-22	70-22 P	70-22	70-22 P
	Standing	64-28	64-28	70-22 P	70-22 P	70-22 P	70-22 P
> 1000 tons	Standard	64-28	64-28	64-28	64-28	70-22	76-22
	Slow	64-28	64-28	70-22	76-22	70-22	76-22
	Standing	64-28	64-28	76-22	76-22	76-22	76-22

Standing Traffic - where the average traffic speed is less than 12 mph (20 km/h).

Slow Traffic - where the average traffic speed ranges from 12 to 43 mph (20 to 70 km/h).

Standard Traffic - where the average traffic speed is greater than 43 mph (70 km/h).

PG 70-22 is not recommended for application on composite pavements because of the high potential for reflective cracking. Either PG 64-22 should be considered if no rutting exists, otherwise PG 76-22 should be used.

VI.B.1.2 Traffic Level Selection

There are five traffic levels or compaction levels in MDSHA HMA Superpave mix design. The different traffic levels represent a specified number of gyrations in the gyratory compactor in HMA Superpave mix design. From a practical and engineering standpoint, each traffic level indicates a HMA designed for a certain volume and type of traffic loading. The selections of the ESAL category for Superpave mix designs are based on the cumulative 20-year design ESAL from traffic analysis. The category code system is established with every Superpave ESAL category; it is broken into five ESAL categories: 1, 2, 3, 4 & 5. The Superpave mix design ESAL category is determined by 20-year cumulative ESAL as seen in the following table:

		Compaction Parameters			
Compaction Level	**Design ESALs[1] (Millions)**	**$N_{initial}$**	**N_{design}**	**N_{max}**	**Typical Roadway Application [2]**
1	< 0.3	6	50	75	Applications include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways would be considered local in nature, not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas, parking lots and driveways may also be applicable to this level.
2	0.3 to < 3.0	7	75	115	Applications include many collector roads or access streets. Medium traffic city streets and the majority of county roadways may be applicable to this level.
3	3.0 to < 10	8	100	160	Applications include many two lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, some rural interstates and US highways.
4	10 to < 30	8	100	160	Applications include many two lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, some rural interstates and US highways.
5	> 30	9	125	205	Applications include the vast majority of the US Interstate system, both rural and urban in nature. Special applications such as truck weighing stations or truck climbing lanes on two lane roadways may also be applicable to this level.
GAP	> 3.0	8	100	160	Gap Graded mixes are used for special application final wearing surfaces. Applications include many two lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked state routes, rural and urban interstates and rural and urban US highways.

Note 1 – Design ESALs are the anticipated project traffic level expected on the design lane over a 20 year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.

Note 2 – When the top of the layer being designed is ≥ 100 mm from the pavement surface and the estimated design traffic level > 0.3 million ESALs, decrease the estimated design traffic level by one, unless the mixture will be exposed to significant traffic prior to being overlaid. If less than 25% of the layer is within 100 mm of the surface, the layer should be considered to be below 100 mm for mixture design purposes.

VI.B.1.3 HMA Mix Selection

Use the table to select the appropriate HMA Superpave mix based on the application of that mix in the pavement structure:

Lift Thickness				Mix Size	Design Application
Minimum	Preferred	Maximum	Total Maximum		
4.0”	5.0”	6.0”		37.5 mm	Base, Deep Patching
3.0”	3.5”	5.0”	10.5”	25.0 mm	Base, Patching
2.0”	2.5”	4.0”	9.0”	19.0 mm	Surface, Base, Patching
2.0”	2.5”	2.0”	2.5”	19.0 mm	GAP Graded
1.5”	2.0”	3.0”	6.0”	12.5 mm	Surface
1.5”	1.5”	2.0”	2.0”	12.5 mm	GAP Graded
1.0”	1.5”	2.0”	3.0”	9.5 mm	Surface, Leveling
1.0”	1.0”	1.5”	1.5”	9.5 mm	GAP Graded
0.5”	0.75”	0.75”	0.75”	4.75 mm	Surface, Rut Fill

VI.B.1.4 Number of ESAL Categories per Mix

One (1) ESAL category will be specified for each mix within a contract. Exceptions to this policy will be granted if the quantity of the mixes exceed 1,000 tons for each ESAL category. The Asphalt Team must approve any exception to this policy.

VI.B.1.5 Number of PG Binders per Contract

Two (2) PG binders will be specified as the maximum number of PG Binders allowable within a contract. If a minimum PG binder can be specified for a particular project, the contractor can provide a higher grade PG binder without objection. The temperature range for the PG Binder has to be the same or greater than that specified in the contract specifications.

VI.B.1.6 Number of HMA Mixes per Contract

At a maximum, the goal is to have three (3) HMA Superpave mixes per contract. Exceptions to this policy will be granted if the quantity of the mixes exceed 1,000 tons for HMA mix.

VI.B.1.7 Placement of Final Surface Material

The final surface layer shall be placed on a HMA Superpave 19.0 mm base layer or finer material when grinding is not included in the project. This surface layer policy also includes the placement of the surface layer over full-depth and partial-depth patch areas.

VI.B.2 Wearing Coarse Selection

The wearing coarse in an HMA pavement can vary depending on the existing site conditions, anticipated traffic conditions, and material availability. Some of the decisions made about the wearing coarse selection can greatly alter the cost of the materials in a particular project. For that reason and to provide a level of consistency through the state, material selection policies were developed for a wearing coarse.

VI.B.2.1 Specifying Gap Graded HMA Mixes

The selection of a Gap Graded HMA mix should be discussed with the Asphalt Team when the criteria for selecting a Gap Graded HMA mix is met, but other conditions may have more influence in the material selection. The following conditions need to the present prior to considering the selection of a Gap Graded HMA mix for the wearing coarse:

- **20,000 ADT (Two-way) in the design year, and**
- **ESAL category 4 or higher, and**
- **With a functional class of Freeway/Expressway or greater.**

VI.B.2.2 Specifying High Polish Value HMA Mixes

The selection of a High Polish HMA mix should be discussed with the Asphalt Team when the criteria for selecting a High Polish HMA mix are met, but other conditions may have more influence in the material selection. The following conditions need to the present prior to considering the selection of a High Polish HMA mix for the wearing coarse:

- **ESAL category 3 or lower, and**
- **With a functional class of expressway/freeway or lower, and**
- **At least 1 of the following items:**
  - **20,000 ADT (Two-way) in the design year, or**
  - **Skid Number values < 40, or**
  - **Greater than 25% of the mainline area is exhibiting polished aggregate as a distress.**

VI.B.3 Reflective Crack Control

In 2001, composite pavements make up the largest percentage of MDSHA's pavement network. Reflective cracking is the most predominant distress present in composite pavements in Maryland. For this reason, several policies were developed in an attempt to limit the amount of reflective cracking in Maryland's composite pavements.

VI.B.3.1 Minimum HMA Overlay Thickness

Four (4.0") inches is the minimum thickness for a plain HMA overlay, without the use of any other material to limit reflective cracking, on bare rigid pavement.

VI.B.3.2 Joint Tape

Joint tape consists of a stress relieving and crack reduction mastic layer with polyester webbing. Joint tape is intended to reduce the amount of stress transferred to upper pavement layers from the horizontal environmental movement of PCC at the joints. The success of joint tape has been mixed to numerous reasons ranging from placement to appropriateness of joint tape for existing conditions. Theoretically, joint tape should provide some stress relief for movement in the horizontal direction, but it would provide benefit with vertical movement at the joints resulting from poor load transfer across joints.

VI.B.3.2.1 Application of Joint Tape

All of the following criteria need to be met in order for joint tape to be an appropriate rehabilitation tool for joint reflective cracking:

- **In composite pavement, at least 25% of the joints are exhibiting at least low severity joint reflective cracking**
- **Load transfer at the joints is greater than 70% on average**
- **Joint tape is only applied to joints with greater than 70% load transfer**

VI.B.3.2.2 Resurfacing with Joint Tape

Use the following criteria when determining the HMA overlay on a project that has joint tape specified as a rehabilitation tool:

- **Minimum of 2 lifts**
- **Minimum of 3.0" overlay over bare rigid pavement**

VI.B.3.2.3 Location of Joint Tape

Joint tape will **only be placed at transverse joints**. Joint tape will not be placed longitudinal joints. Joint tape shall only be **placed directly on PCC**, not HMA.

VI.C LAY-DOWN OPERATIONS

Construction specifications can significantly effect the lay down operations of a HMA mat and ultimately the performance of the pavement. MDSHA pavement designer's have control over some of those specifications and should use consistent policies to ensure that quality material is placed to provide the potential for most improvement to the MDSHA pavement network.

VI.C.1.1 Material Transfer Vehicle (MTV)

An MTV shall always be used for placement of Gap Graded Mixes. For other cases, the Asphalt Team, District Construction, and Construction Inspection Division need to be consulted about the necessity for a MTV. Criteria to consider when selecting the potential need for a MTV include the following:

- Amount of material being placed (greater than 20,000 ton for the surface material)
- Interstate resurfacing
- Other special construction needs

VI.D RIDE QUALITY POLICIES

Material acceptance specifications can significantly effect the lay down operations of a HMA mat and ultimately the performance of the pavement. MDSHA pavement designer's have control over some of those specifications and should use consistent policies to ensure that quality material is placed to provide the potential for most improvement to the MDSHA pavement network. The ride quality of the roadway is typically one of the first criteria the travelling public notices about a roadway.

VI.D.1 Ride Specification

Currently, MDSHA is in the process of implementing a new ride quality specification. In the past, MDSHA used the California Profilograph to measure ride quality with a 0.2" blanking band. The contractor was responsible for performing quality control of ride quality for paving projects and MDSHA would complete ride quality assurance on an as-needed basis. The new ride specification eliminates the blanking band, allows the use of high speed profilers, specifies more aggressive quality assurance practices, and modifies the practices of determining incentive, disincentive, and corrective action. The use of the California Profilograph is currently planned to be phased out of use after two years, but that is dependent on the success of the implementation of the new ride specification.

The new ride specification, 535 – Pavement Surface Profile, requires the pavement design engineer to make decisions concerning the required ride quality for the specification. The maximum limit of ride quality to determine for full pay, corrective action, and disincentive vary depending on the constraints on the project, the materials being placed, and the proposed construction operations. The pavement design engineer shall use Form PD-11, Section IV, to assist in the decision process. The following factors are the type of factors used in Form PD-11 to assist the engineer to determine ride quality limits.

- Required to match existing curb and gutter grade
- Considerable manholes and other utilities
- Intersections, entrances, driveways that must be tied into
- Night time paving with reduced closure times
- Use of PG 76-22 late in the season
- Work area confined with barrier wall or other obstacles
- Existing pavement in poor condition and no w/l or removal
- Crack sealing in the pavement surface

VI.D.1.1 *Incentive & Disincentive Payments*

Use the following table to select the appropriate Incentive (P_{max}) and Disincentive (P_{min}) based on the functional class of the roadway:

	Urban		Rural	
Functional Class	**Desired Reliability**	**Minimum Reliability**	**Desired Reliability**	**Minimum Reliability**
Interstate	500	-500	500	-500
Freeways and Expressways	450	-450		
Principal Arterial – Other	450	-450	450	-450
Minor Arterial	450	-400	400	-400
Major Collector			400	-400
Minor Collector			400	-400
Collector	350	-350		
Local	350	-350	350	-350

VII Pavement Design Standards

The pavement design standards contained in this section are to be used by Pavement Division of the OMT only and are not considered to be part of the MDSHA “Book of Standards.” The standards are intended to maintain consistency in the distribution of recommendations from the Pavement Division. The following standards are provided in this section:

A. Roundabout

1. Truck Apron
    1) Material and Thickness Design
    2) Joint Design and Spacing
    3) Steel Design
2. Curb and Truck Apron Tie
3. Travel Way

B. Park and Ride

1. Passenger Vehicle
2. Bus Lane
3. Bus Stop Pad

C. Narrow Base Widening with Curb (< 4 ft)

D. Composite Patch (HMA and PCC)

VII.A ROUNDABOUT

A roundabout is a circular intersection intended to regulate traffic flow with the use of a signalized lighting. A roundabout consists of travel way, which maybe either a flexible or rigid pavement, and a truck apron at the inner radius of the circle intended for truck with trailers incapable of making a tight radius turn. A mountable curb resides between the paved travel way and the truck apron. Typically some type of curb or wall at the boundary with the truck apron with landscaping at the very center of the roundabout. The mainline travel way shall be design following the guidelines.

VII.A.1 Truck Apron

The truck apron of a roundabout shall be Jointed Reinforced Concrete Pavement (JRCP) to limit pavement distress from the slow moving, turning truck traffic. The inner and outer radius of the truck apron will differ because in is a circle. The joints in the truck apron shall be doweled to provide for load transfer. Depending on the dimensions of the roundabout, truck aprons are not typically reinforced. Truck aprons are typically constructed in conjunction with a Type ‘C” mountable curb that is 8.0” to 10.0” in depth with a 3.0” high mountable lip.

VII.A.1.1 Material and Thickness Design

The truck apron shall be constructed with the following pavement structure:

9.0” Portland Cement Concrete – Mix #7
8.0” Graded Aggregate Base

The thickness of the lifts of the graded aggregate base (GAB) will vary depending on the thickness of the Type “C” curb, but the overall GAB thickness shall remain at 8.0”. The thickness of the lifts will vary because of the construction sequence to place a base on which to construct the Type “C” curb. Given the 9.0” thickness of the JRCP layer in a truck apron, the Type “C” curb shall be either 9.0” or 10.0” in depth. If a 9.0” Type “C” curb is placed, the GAB shall be placed in one 3.0” lift and one 5.0” lift. If a 10.0” Type “C” curb is placed, the GAB shall be placed in two one 4.0” lifts. This information can be seen in the detail at the end of this section.

VII.A.1.2 Joint Design and Spacing

All longitudinal joints on a truck apron shall be either formed or constructed with a slip form paver. All transverse joints shall be contraction joints with the exception of two joints on opposite side of the truck apron. There shall be two expansion joints (only two) on opposite sides of the truck apron to allow for the expansion of the unique geometry of a circular truck apron. The maximum joint spacing at the outer radius of the truck apron shall be 30 feet. Every attempt shall be attempt to have equal and consistent joint spacing around the truck apron. The need for mid-slab reinforcement can be eliminated if joint spacing at the outer radius is limited to a maximum of 15 feet.

VII.A.1.3 Steel Design

It may be more advantageous to limit the joint spacing at outer radius of the truck apron to be less than is 15 feet because mid slab steel reinforcement would not be necessary. If the joint spacing is greater than 15 feet at the outer radius, then mid-slab steel reinforcement is required. The dimensions of the truck apron may warrant a shorter joint spacing to eliminate the additional construction issues involved with mid slab reinforcement. In cases where steel design is required, the truck apron shall be constructed with the following mid slab steel design:

6 X 6 – W4 X W4

The dowel bars shall have the following design:

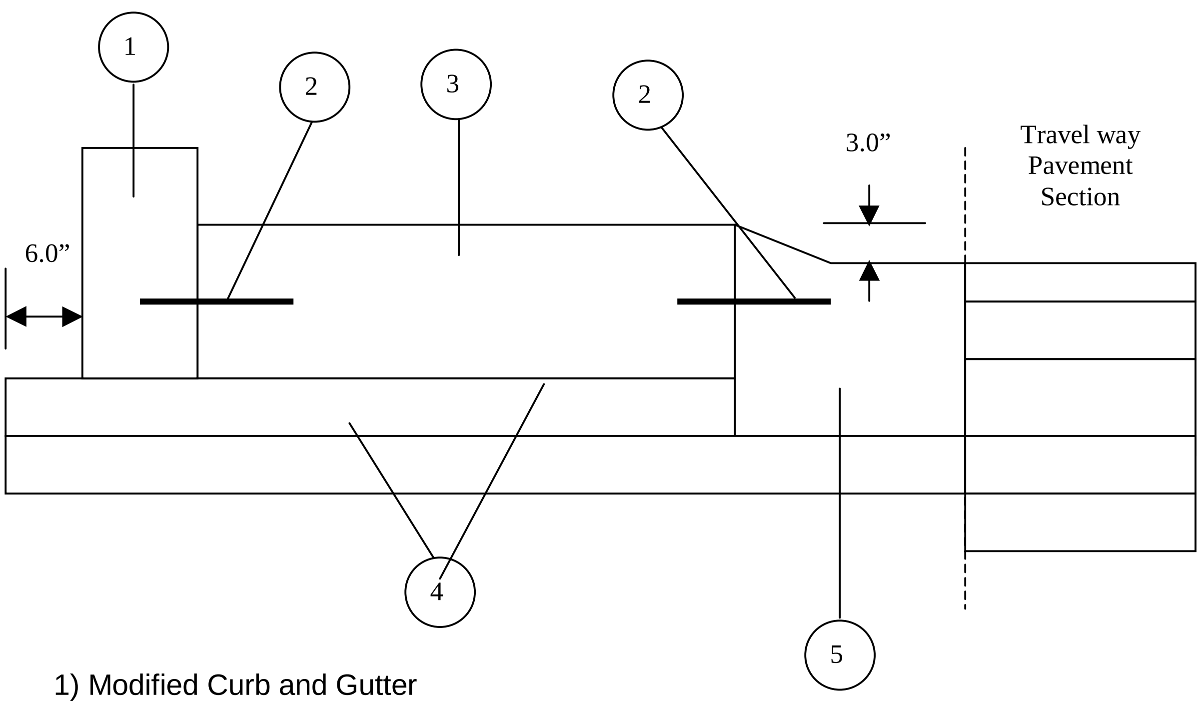
#8 bars – 18.0" long, 12" on center, placed 6 " from longitudinal joint

Longitudinal tie bars shall be placed at all longitudinal joints and slab /curb interfaces and have the following design:

#4 bars – 14.0" (J) Bar, placed 36" on center

VII.A.2 Curb and Truck Apron Tie

The truck apron and curb shall be tied together with the specified steel design as described above. The "j" shaped longitudinal devices shall be placed with the bend in the horizontal plane, i.e. the entire bar is at the same elevation. The following detail is provided:



1) Modified Curb and Gutter
2) Longitudinal Tie Devices - #4 "J" Bar, 14.0" long, placed on 36.0" center
3) 9.0" Portland Cement Concrete – Mix #7
4) 4.0" Graded Aggregate Base
5) Type "C" Curb and Gutter – 10.0" Depth

VII.A.3 Travel Way

The material type and thickness of the travel portion of the roundabout shall be designed as new pavement section is typically designed in accordance to Section III.G "New Design." An attempt shall be made to match the thickness of the aggregate base layer in the travel way with that of the truck apron, for ease of construction. This is demonstrated in the pavement detail for the truck apron shown above.

VII.B PARK AND RIDE

The Park and Ride as three possible pavement sections: passenger vehicle area, bus lane area, and bus stop pad area. Adjustments to the design can be made to suit local traffic patterns and regional climates; for example, the use of PG 64-28 in Garrett County or the use of polyolifin for I-95 park and ride lots. The typical pavement sections for all three cases are shown in the following section.

VII.B.1 Passenger Vehicle

The passenger vehicle area shall be designed with the following pavement section:

2.0” HMA Superpave 9.5mm for Surface – PG64-22, Level 2
3.0” HMA Superpave 19.0mm for Base – PG64-22, Level 2
6.0” Graded Aggregate Base

VII.B.2 Bus Lane

The bus lane area shall be designed with the following pavement section:

2.0” HMA Superpave 9.5mm for Surface – PG64-22, Level 2
5.0” HMA Superpave 19.0mm for Base – PG64-22, Level 2
(2 – 2.5” lifts)
6.0” Graded Aggregate Base

VII.B.3 Bus Stop Pad

The bus stop pad area shall be designed with the following pavement section:

9.0” Portland Cement Concrete – Mix #7
6.0” Graded Aggregate Base

The joint spacing shall be **15 feet or less**. This length of joint spacing allows for the placement of unreinforced PCC slabs. Therefore, there will **be no mid slab steel reinforcement**.

The dowel bars shall have the following design:

#8 bars – 18.0” long, 12” on center, placed 6 “ from longitudinal joint

Longitudinal tie bars shall be placed at all longitudinal joints and slab /curb interfaces and have the following design:

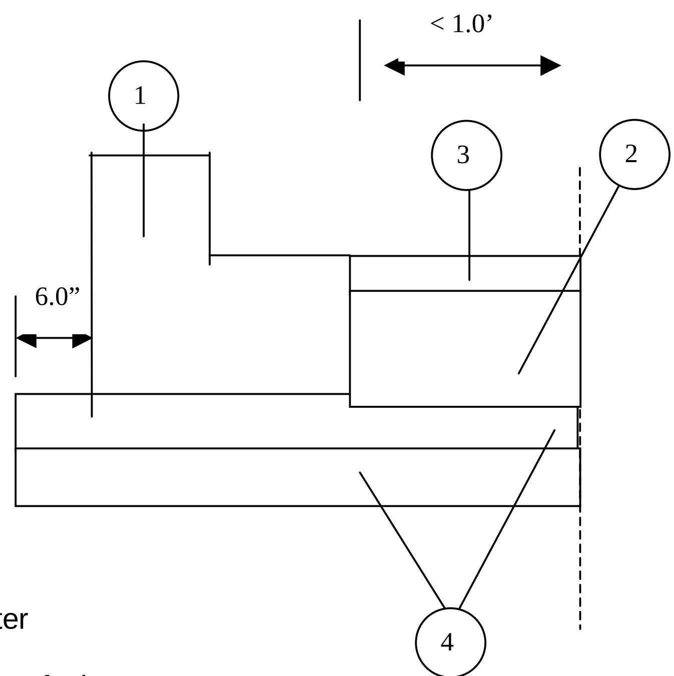
#4 bars – 14.0” (J) Bar, placed 36” on center

VII.C NARROW BASE WIDENING WITH CURB (LESS THAN 4 FEET)

This entire issue arose from the concern that it is extremely difficult to achieve adequate compaction of HMA in tight, narrow areas like in the case of narrow base widening with curb. The lack of available equipment to compact in such area less than 4 feet has led to the selection of different materials and new details. There are two cases when this occurs, either MDSHA is placing new curb or the roadway is actually being widened less than 4 feet. In the case with new curb, the contractor can either saw cut the existing pavement and place the new curb directly against the sawed edge (skilled contractor). The alternative is remove a portion of the existing pavement to allow for placement of forms for the new curb and that area needs to be replaced after form removal. With this option, the contractor will complete the removal and replacement incidental to the placement of the new curb.

VII.C.1 New Curb Placement

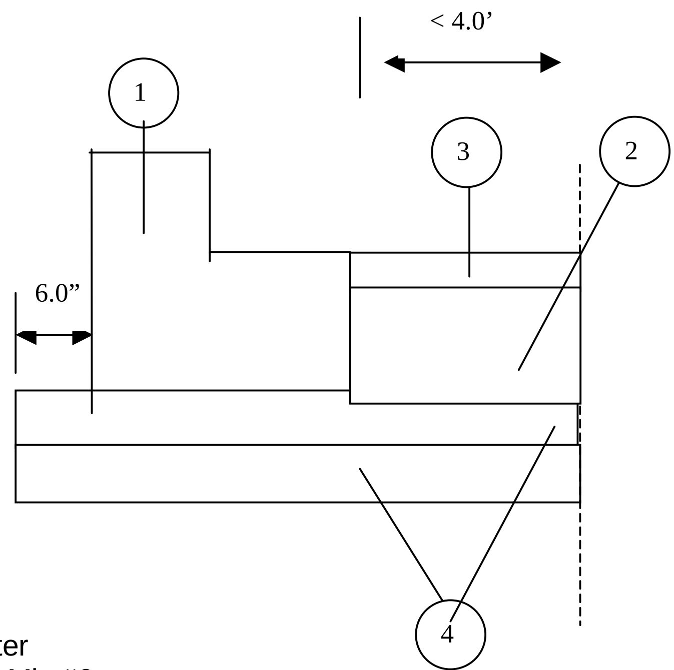
The contractor may elect to remove a portion of the existing pavement for placement of forms for the new curb. The replacement of this area is too small for placement of HMA based on District Construction's opinion. Therefore, PCC is used as a replacement material. PCC mix #2 is typically used for curb placement and either modified PCC mix #6 or PCC mix #7 for pavement. When dealing with new curb placement, it reasonable to use PCC mix #2 for pavement placement. The joints in the narrow widening for curb placement shall match those placed for the new curb. No steel reinforcement or dowel bars shall be used in the PCC used in the pavement replacement for the new curb. The following detail is provided:



1) Curb and Gutter
2) PCC Mix #2
3) Final HMA Resurfacing
4) Graded Aggregate Base Layer

VII.C.2 Narrow Base Widening (less 4 feet)

In this case, the widening is a capacity or geometric design improvement. The replacement of this area is too small for placement of HMA based on District Construction's opinion. Therefore, PCC is used as a replacement material. PCC mix #2 is typically used for curb placement and either modified PCC mix #6 or PCC mix #7 for pavement. In this case, engineering judgement and the quantity of widening will dictate the selection of PCC mix. If there is a limited amount of widening and the width is small, using PCC mix #2 is reasonable. If there is substantial quantity of widening and the width is 3 plus feet, then modified mix #6 is appropriate. The quantity is critical because it will involve the introduction of another material on the project. The joints in the narrow widening shall match those placed for the new curb. No steel reinforcement or dowel bars shall be used in the PCC for narrow widening. The following detail is provided:

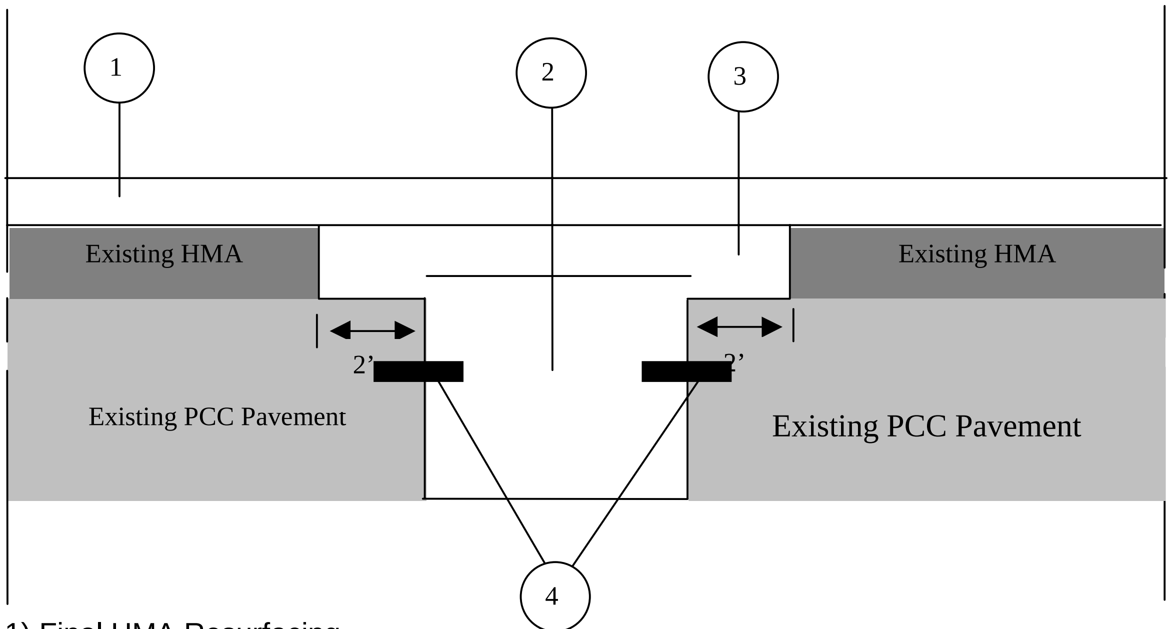


1) Curb and Gutter
2) Modified PCC Mix #6
3) Final HMA Resurfacing
4) Graded Aggregate Base Layer

VII.D COMPOSITE PATCH (HMA OVER PCC)

The composite patch is placed in exiting composite pavement to repair an existing pavement distress. The success of composite patches has been mixed because of the dimension details of the patches, type of materials, and construction techniques used in the past. These patches involve the use of modified PCC mix #7, dowel bar placement, and HMA placement above he PCC. The MDSHA "Book of Standards" shall be used to repair different dimensions of distress in the PCC layer. This is true for both jointed and continuous PCC pavement.

Typically, the majority of distress types in jointed rigid pavements occur at the joint. The PCC patch standards in the "Book of Standards" and the construction specification in Section 522 shall control the PCC portion of the patch construction. The HMA placement shall be 2.0 feet greater than the underlying PCC patch to limit the length of the shear plane and limit the potential for "sinking" patches. The following detail is provided:



1) Final HMA Resurfacing
2) Modified PCC Mix #6 – steel reinforcement dependent on size and location of patch and type of PCC
3) HMA Patching
4) Dowel bar or tie bar – dependent on size and location of patch

VII.E DRIVEWAYS & BIKE PATHS

The typical pavement sections for both cases shall be designed with the following pavement section:

1.5" HMA Superpave 9.5mm for Surface – PG64-22, Level 1
2.5" HMA Superpave 19.0mm for Base – PG64-22, Level 1
4.0" Graded Aggregate Base

In cases where roadway resurfacing is to be done in the same contract, make an effort to use the same mixes that are to be used on the roadway resurfacing.

VIII PAVEMENT REHABILITATION TECHNIQUES

The purposes of this section is to assist the pavement design engineer identify the rehabilitation technique best suited to correct existing distress types and achieve the functional and structural condition objectives established by MDSHA. Rehabilitation techniques encompass all operations from partial-depth patching, grinding, functional HMA overlay to reconstruction. Several rehabilitation techniques are often used on same project as part of the pavement rehabilitation project strategy.

This section provides numerous rehabilitation methods and a description and definition of each technique. In addition, this sections provides assistance in the form of guidelines as to when certain rehabilitation techniques are most appropriate. This section also provides input into several data analysis steps for pavement rehabilitation recommendations involve tasks involving the selection of rehabilitation techniques.

VIII.A PAVEMENT REHABILITATION METHODS

Rehab Technique	Pavement Type			Functional Class		Extended Lane Closure Required	Applicable PCI Range	Service Life (yrs)	Applicable Uses
	Rigid	Flexible	Composite	Interstate, Princ. Art.	All Others				
HMA Overlay	✔	✔	✔	✔	✔		> 25	8 - 12	• structural or functional improvement
PCC Bonded Overlay	✔			✔		✔	≥ 70	12 – 15	• PCC pavement in very good, sound condition • minimal to no pre-overlay required • structural overlay required
PCC Unbonded Overlay	✔	✔	✔	✔		✔	< 55	≥ 20	• PCC pavement in poor condition • structural overlay required • adequate vertical clearances
Rubblize and HMA Overlay	✔		✔	✔	✔	✔	< 55	15	• PCC pavement in poor condition • joint deterioration prevalent • removal of AC overlay required • adequate vertical clearances
Break & Seat and HMA Overlay	✔		✔	✔	✔	✔	≤ 70	12 - 15	• PCC pavement in fair to poor condition • Removal of AC overlay required
HMA Overlay with Saw & Seal	✔		✔	✔	✔		> 70	8 - 12	• joints in good working condition • load transfer > 70% on average • minimal joint patching exists or required • removal of AC overlay required
Ultra-Thin Whitetopping		✔	✔		✔	✔	> 55	< 10	• rutting exists throughout project limits (> 50% of the project length) • rutting resulting from material related problem • pavement structurally sound

Rehab Technique	Pavement Type			Functional Class		Extended Lane Closure Required	Applicable PCI Range	Service Life (yrs)	Applicable Uses
	Rigid	Flexible	Composite	Interstate, Princ. Art.	All Others				
In-place Recycled HMA		✔	✔	✔	✔		≥ 70	5 - 8	• minimal structural distress • minimal patching required • limited to approx. 3 inches in depth • good surface materials • not recommended for heavily oxidized material
Recycled HMA		✔	✔	✔	✔		> 25	8 - 12	• structural or functional improvement • good material to recycle • not recommended for heavily oxidized material
Concrete Pavement Restoration	✔			✔	✔		> 55	5 - 8	• joints in good condition • load transfer > 70% on average • no high steel problems • existing structure is adequate
AC Reconstruction	✔	✔	✔	✔	✔	✔	< 40	15	• significant structural improvement required • no allowance to increase grade • extensive (>40%) patching required
PCC Reconstruction	✔	✔	✔	✔	✔	✔	< 40	20	• significant structural improvement required • heavy truck loadings expected (potential to rut) • no allowance to increase grade • extensive (>40%) patching required
Pre-Overlay Techniques	✔	✔	✔	✔	✔				See Section VIII.B
Maintenance Treatments	✔	✔	✔	✔	✔				See Section VIII.C

VIII.A.1 Treatment Method Definitions

Rehabilitation Technique	Definition
HMA Overlay	Placement of a HMA overlay on any pavement type (Flexible, rigid, or composite). Within this rehabilitation technique there are numerous combinations of HMA overlay thickness along with varying levels of percent patching and surface removal (milling/grinding).
PCC Bonded Overlay	Placement of a PCC overlay on a rigid pavement. Additional care and construction practices are taken to ensure a good bond is established between the old PCC and the new PCC overlay. The pavement design is based on the premise that two PCC layers will perform as one homogeneous layer.
PCC Unbonded Overlay	Placement of a PCC overlay on a rigid pavement. Prior to placement of the PCC overlay a bond breaker is placed to isolate the two PCC layers. The bondbreaker is typically a 1.0” to 2.0” HMA overlay directly on the old PCC pavement prior to the placement of new PCC overlay. The bondbreaker is designed to allow the two PCC layers to move independently and limit the amount of reflective cracking.
Rubbilize and HMA Overlay	Placement of a HMA overlay on a rigid pavement that has been structurally altered, rubbilized. Rubbilization involves breaking the PCC layer into smaller pieces of less than 6.0” in diameter. This effectively creates a cement treated aggregate base from the rigid pavement. Rubbiliztion is generally completed to eliminate the potential for reflective cracking. Rubbiliztion is generally completed on rigid pavement s that are beyond their structurally life.
Break and Seat and HMA Overlay	Placement of a HMA overlay on a rigid pavement that has been altered, break and seat. The break and seat method differs from rubbilization in that the a large percentage of the structural integrity of a rigid pavement remains following a break and seat operation. The break and seat operation involves breaking the existing slab in much shorter slabs followed by compaction of those smaller slabs to “seat” them. This operation is design to limit the potential for reflective cracking while maintaining a majority of the structural capacity of the rigid pavement.
HMA Overlay with Saw & Seal	Placement of a HMA overlay on a rigid pavement with joints formed into the HMA overlay to match the underlying PCC joints. This method is designed to limit the damage created by reflective cracking distress by pre-selecting the locations for the reflective cracks in the HMA overlay. The challenging portion of this technique is matching the HMA sawed joint over the underlying PCC joint. The HMA joints need to be sealed and maintained to prevent the infiltration into the pavement structure.
Ultra-Thin Whitetopping	Placement of a thin (≤ 4.0”) PCC overlay on a flexible or composite pavement. The ultra-thin PCC overlay is cut into small slabs or panels of less than 6 feet in any dimension. Whitetopping is designed to limit the potential for rutting.
In-Place Recycled HMA	Placement of an HMA overlay on a flexible or composite pavement using the existing HMA surface as a portion of the overlay. Generally, no significant structural improvements are required with this rehabilitation technique, only a functional improvement. The existing HMA surface is removed, then rejuvenated with additional asphalt cement and HMA, and then placed back onto the exiting roadway surface all in one operation.

Rehabilitation Technique	Definition
Recycled HMA	Placement of an HMA overlay on a flexible or composite pavement using a HMA material comprised of previously placed HMA material that has been removed. This technique differs from in-place recycling in that this technique does not have to use HMA material from this specific project site, it could be from a stock pile.
Concrete Pavement Restoration (CPR)	Non-overlay option used to repair isolated areas of distress and restore ride quality in rigid pavements. CPR is generally the first rehabilitation procedure applied to a rigid pavement in reasonably good condition with only slight deterioration. CPR generally involves both partial-depth and full-depth PCC patches, slab stabilization, diamond grinding, followed by joint and crack sealing.
AC Reconstruction	Complete reconstruction of the existing pavement type with a flexible pavement design.
PCC Reconstruction	Complete reconstruction of the existing pavement type with a rigid pavement design.
Pre-Overlay Techniques	Completing improvements to the existing pavement in order to restore a consistent structural integrity throughout the project or to improve the functional condition. These techniques are completed prior to the placement Pre-overlay techniques include, but are not limited to partial-depth and full-depth patching, removal (milling/grinding), wedge/level, etc.
Maintenance Treatments	Completing improvements to the existing pavement in order to restore a consistent structural integrity throughout the project or to improve the functional condition. Both corrective (correcting an existing problem) and preventive (limiting the potential problems) for maintenance methods will be included. Maintenance treatments include but are not limited to crack sealing, slurry seal, patching, microsurfacing, grinding, etc.

VIII.BPRE-OVERLAY REPAIR GUIDELINES

VIII.B.1 Application for Removal (Milling/Grinding)

Patching operations shall proceed all other construction operations within the existing roadway is the basic assumption that shall be followed when considering criteria to determine the need for removal. Therefore, some distress types shall be repaired through patching and effect the total distress quantities used in the guidelines for removal criteria.

Criteria	Depth of Removal
h_{ol} > proposed roadway elevation[1]	Overlay thickness
Medium and high severity rutting > 30% of the length	Depth of max rutting
Functional distress > 50% of area[2]	Depth of last surface layer[3]
> 30% of joints are medium or high severity joint reflection cracking	Depth of last surface layer[3]
Patching > 25% of area	Depth of last surface layer[3]
IRI > 170 in/mile for interstates	Depth of last surface layer[3]
IRI > 220 in/mile for non-interstates	Depth of last surface layer[3]

1 – If the removal and replacement thickness does not provide adequate structural capacity for the expected service life, additional discussion will needed to determine if a lower service life is acceptable. If not, this rehabilitation technique alternative should be eliminated from the viable alternatives.
2 – Functional distress consists of bleeding (sf), block cracking (sf), bumps/sags [(lf) * 6ft] = (sf), depression (sf), joint reflective cracking [(lf) * 6ft] = (sf), polished aggregate (sf), swell (sf), and weathering and raveling (sf).
3 – The depth of removal could be increased based on the depth of the cracks and the thickness of the existing HMA. It is not recommended to leave less than 1.5" of HMA material above PCC pavement. If less than 1.5" of HMA material will be left following required removal, all of the HMA material should be removed to the bare concrete surface.

The thickness of partial-depth, full-depth, and Type A partial depth patches should be evaluated following the determination for the need for removal process. The need for partial-depth patching could vary depending on the results of the removal selection process. The thickness of partial-depth and full-depth patches could vary depending on the results of the removal selection process, especially in composite pavements.

The thickness of the existing shoulders should be considered when determining the depth of removal. Shoulders with thin pavement structures may dictate minimum removal depths or the need for other alternative rehabilitation techniques.

Reasons to specify grinding over milling:

- recommend if only one lift is to be placed above removal area
- recommend if removing for poor ride conditions only

- recommend if removed pavement section is to be opened to traffic

Removal of the pavement by milling or grinding effects the existing condition of the pavement in addition to affecting the existing structural capacity of the pavement. Specific distresses are reduced or eliminated by removal of the pavement. Please refer to Section VIII.B.7 for information regarding the effect of pavement removal has on distresses based on the visual PCI procedure. In addition, the original structural capacity (SC_o) of the pavement is reduced as a result of milling or grinding. The calculation to adjust SC_o based on a removal depth is described in Section III.F.

VIII.B.2 Wedge/Level Guidelines

Applications for Wedge/Level Layer

Criteria	Comments
Patching > 25% of project area	Full-depth and partial-depth
Any severity rutting > 30% of area	Based on length not area
Joint tape is used with a single HMA lift[1]	
Designed elevation increase in roadway[2]	Superelevation or cross slope
> 30% of joints are medium or high severity joint reflection cracking	Compared to total # of joints
IRI > 170 in/mile for interstates	
IRI > 220 in/mile for non-interstates	

1 – The existing composite or rigid pavement is structurally adequate (SC_{eff} > = SC_f). The wedge/level layer is intended to provide an additional layer between the joint tape and the final HMA overlay layer, resulting in two lifts of HMA between the joint tape and the final roadway surface.
2 – If a significant increase in elevation is required (>2.0"), then the 19.0 mm should also be used in combination with the 9.5 mm for wedge/level.

The use a wedge/level layer is dictated by the amount the roadway elevation can be raised. If there is restrictions regarding the roadway elevation, removal (milling/grinding) should be the selected alternative. Otherwise if there are no roadway elevation restrictions, wedge/level is the preferred alternative. District Construction should be consulted to confirm the preference of removal versus wedge/level.

The type of mix generally used for wedge/level is the HMA Superpave 9.5 mm. In the case of a significant increase in roadway elevation (> 1.5"), the HMA Superpave 19.0 mm mix should be used instead of or in addition to the 9.5 mm, depending on the total increase in elevation. The ranges for lift thickness for the wedge/level mixes are the same as the typical range for each mix and are in the following table:

Recommended Wedge/Level Lift Thickness		
HMA Superpave Mix Type	**Minimum Thickness**	**Maximum Thickness**
9.5 mm	0.75”	1.5”
12.5 mm	1.5”	2.0”
19.0 mm	2.0”	3.0”

VIII.B.3 Patching and Joint Tape Guidelines for PCC Surfaces

Distress	Severity			Comments
	Low	Medium	High	
Blow-Up	Full	Full	Full	
Divided Slab	None	Full	Full	
Corner Break	None	Full	Full	
Durability	Full	Full	Full	
Faulting	None	Full	Full	Consider grinding or undersealing the joint to remove fault.
Linear Cracking	None	Partial	Full	Consider grinding, undersealing or crack sealing for Low and Medium Severity.
Patching	None	Full	Full	
Pumping	None	None	Full	Consider undersealing to correct Pumping
Punchout	Full	Full	Full	Type II patch if punchout greater that 6' long
Spalling	AC	Partial	Full	Clean out spalled area and replace with AC prior to overlay.

Type I PCC Patch is less than 15 feet in length

Type II PCC Patch is greater than 15 feet in length

Note: Joint sealant damage, lane/shoulder drop off, polished aggregate, popouts, RR Xing, Map Cracking, and shrinkage cracking are not patched.
If LTE < 70% then a flexible full-depth patch is not recommended (Use rigid patch).
If M_r subgrade is weak - PCC patch required.
If Pumping is evident - PCC patch required.

VIII.B.4 Patching and Joint Tape Guidelines for AC Surfaces

		Milling (1" - 2")			**Comments**
Distress	**Severity**	**No**	**Yes**		
			AC Material Thickness		
			> 6"	**< 6"**	
Fatigue Cracking	**Low**	None	None	None	
	Medium	Partial	Partial	Full	
	High	Full	Full	Full	
Rutting areas < 500' long	**Low**	None	None	None	Soil/Subgrade could be cause for rutting.
	Medium	Partial	None	None	
	High	Partial	Partial	Partial	
Linear Cracking Edge Cracking	**Low**	None	None	None	If the crack depth is greater than ½ AC layer thickness, then use full-depth patch.
	Medium	Partial	None	None	
	High	Partial	Partial	Partial	
Potholes/Failures	**Low**	Partial	None	None	
	Medium	Partial	Partial	Full	
	High	Full	Full	Full	
Block Cracking Slippage Crack Shoving Corrugation	**Low**	None	None	None	Full-depth patching may be required if distress is significantly deep (>1/2 AC).
	Medium	Partial	None	None	
	High	Partial	Partial	Partial	
Depression Bumps/Sags Swelling	**Low**	None	None	None	Soil/Subgrade could be cause of depression, bumps/sags, and swelling.
	Medium	Partial	None	None	
	High	Full	Full	Full	
Patches	**Low**	None	None	None	
	Medium	Partial	Partial	Full	
	High	Full	Full	Full	
Joint Reflection Cracking LTE >70% - Good	**Low**	None	None	None	Potential to use Joint Tape (Type A Patch) remove to PCC
	Medium	Partial	Partial	Partial	
	High	Partial	Partial	Partial	
Joint Reflection Cracking LTE < 70% - Bad	**Low**	None	None	None	Patch with PCC.
	Medium	Partial	Partial	Partial	
	High	Full	Full	Full	

Note: 1) Polished aggregate, bleeding, RR Xing, weathering, and lane/shoulder drop off are not patched.
2) Minimum patch size is generally 6 ft x 6 ft
3) Partial depth patching should not exceed 50% of the pavement in depth

Section VIII.B.3 and VIII.B.4 are to be used as guidelines to determine specific patching requirements. These guidelines can be customized to specific project conditions and demands. Any major changes to the patching guidelines should be discussed with the Assistant Pavement Division Chief.

VIII.B.5 Drainage Treatment Guidelines

Drainage Tool	Applicable Use

Longitudinal Underdrain (LUD)	• Used for roadways in areas with fine grain frost susceptible or high capillary soils.
	• Place continuously through all cuts and fills.
	• On divided highways use 4 lines; with paved median use 3 lines (one under barrier).
	• For closed sections place only through non-piped areas, i.e. opposite the storm drains.
	• Always use on PCC pavements, pavements open graded base courses and Interstate pavements.
	• Use LUD on other pavements if the following conditions exist:
	  • Presence of high water tables
	  • High moisture conditions in the soil (saturated soils)
	  • Poorly draining soils encountered on the project
	  • Topography is condusive to placing longitudinal underdrain
	• Do not use LUD when the following conditions exist:
	  • Generally in rock cuts
	  • Projects that are mostly fill and have low water tables
	  • Projects that are very dry and have easily drainable soils
	  • Projects located where sand capping material is available and moisture conditions are not extreme
	  • Projects with poor topography (i.e. flat)

Pre-Fab Edge Drains	Application similar to Longitudinal Underdrain but usually where anticipated water amounts are low, or as retro-fits.
Cap with Select or Capping Borrow	• Use a 6" to 12" layer; daylight or to the back face of curbs. • Use Capping Borrow beneath pavement in areas having "dead" A-3 soils. Note: capping may not drain as well as is expected.
Open-Graded Base (OGB)	• OGB should be used to control water infiltrating through the pavement surface only. It should not be used to correct a high water table problem. • All PCC pavements will be considered for OGB and Longitudinal Underdrain. Generally OGB shall not be used with flexible pavements. • Truck traffic and Soil type shall also be considered when evaluating the need for OGB. Silty and clayey soils are prime candidates for use of OGB. • Good vertical drainage, clean sands and gravels for a considerable depth at subgrades may diminish the need for OGB provided that the base course used is not erodible and permeable(greater than $1x10^{-3}$ cm/sec). • Use 4" layer beneath PCC roadway placed on a Graded Aggregate Base. • Use longitudinal underdrain to provide an outlet for water in the OGB • OGB is a stabilized mix using either asphalt or cement concrete.

Drainage Tool	Applicable Use
Subgrade Drains	• Construction expedient • Adequate to use where anticipated moisture is not serious. • Use should be specified through limits where rock exists at subgrade.
Underdrain	• Usually placed beneath side ditches in wet cuts. • Used in areas of excessive moisture or high water tables. • Specify continuous line(s), 1 or 2, through limits.
Spring Control	• List in report springs located by soil survey and recommend designer to increase bid quantity of underdrain to provide for spring control.

VIII.B.6 Other Pre-Overlay Techniques

VIII.B.6.1 Slab Stabilization

- Use if FWD testing indicates a large presence of voids, see Section III.J, "FWD Data Analysis"
- Consider if a large presence of corner breaks or pumping exist on a project.

VIII.B.7 Pre-Rehabilitation Effect on PCI

The PCI calculated from the visual survey is based on the existing distresses in the roadway. The PCI is then used as an indication of the effective structural capacity of the pavement structure. Pre-rehabilitation repairs alter the existing distresses and the PCI and therefore effect the existing structural capacity of the pavement structure. For example, if high severity alligator cracking were repaired with a full-depth patch, that distress would be changed to a low severity patch. Depending on the distress type and severity, pre-rehabilitation repairs may increase the PCI value and subsequently the structural capacity. Therefore, certain pre-rehabilitation repair strategies will influence the type and thickness of the pavement rehabilitation. A significant step in the pavement rehabilitation process is to re-calculate PCI for each sample unit based on the selected pre-overlay repairs. The PCItool program, as of 8/1/2001, does not have the capability to re-calculate PCI based on pre-rehabilitation decisions. Therefore, this step will need to be done by creating a "dummy" project and re-entering the PCI data with the pre-rehabilitation repair adjustments. This function has already been designed for PCItool and should be implemented in the future. Patching and removal (milling/grinding) can both be completed as pre-rehabilitation repairs. The sequence of those operations will effect the outcome of the PCI data analysis. It is established in MDSHA specifications that patching is to proceed removal operations. Use this sequence when determining your pre-rehabilitation repairs and calculating the effect PCI data. The following table shows the effect of different pre-rehabilitation repairs on PCI for flexible/composite and rigid pavements.

Pre-Overlay Repairs for AC Surfaced Pavements			
Distress	**Partial Depth**	**Full Depth Patch**	**Removal**
Alligator	Low patch	Low patch	-1 Severity
Bleeding	No	No	No Distress
Block	Low patch	Low patch	-2 Severity
Bumps/Sags	Low patch	Low patch	-2 Severity
Corrugation	Low patch	Low patch	-2 Severity
Depression	Low patch	Low patch	-1 Severity
Edge Cracking	Low patch	Low patch	-1 Severity
Joint Reflection	Low patch	Low patch	-1 Severity
Ln/Shld Drop	No	No	No
Linear Cracking	Low patch	Low patch	-1 Severity
Patch	Low patch	Low patch	-1 Severity
Polished Agg.	No	Low patch	No Distress
Pothole	Low patch	Low patch	-1 Severity
RR Crossing	No	No	No
Rutting	Low patch	Low patch	-2 Severity
Shoving	Low patch	Low patch	-2 Severity
Slippage	Low patch	Low patch	No Distress
Swell	Low patch	Low patch	-1 Severity
Weathering	No	No	No Distress

Pre-Overlay Repairs for PCC Surfaced Pavements			
Distress	**HMA Patch**	**PCC Patch**	**CPR**
Blow-up	Low patch	Low patch	Low patch
Corner Break	Low patch	Low patch	Low patch
Divided Slab	Low patch	No Distress	No Distress
“D” Cracking	Low patch	Low patch	Low patch
Faulting	Low patch	Low patch	Low patch
Joint Seal Damage	No	No	Low Severity
Shoulder Drop off	No	No	No
Linear Cracking	Low patch	Low patch	Low patch
Patch – Large and Small	Low patch	Low patch	Low patch
Polished Aggregate	No	No	No Distress
Popouts	No	No	No Distress
Pumping	Low patch	Low patch	Low patch
Punchouts	No	Low patch	Low patch
RR Crossing	No	No	No
Scaling	No	No	No Distress
Shrinkage Cracks	No	No	No Distress
Spalling, Corner	Low patch	Low patch	Low patch
Spalling, Joint	Low patch	Low patch	Low patch

VIII.B.8 Estimating Patching Quantities from PCI Data

The distress quantities summed from the PCI visual condition survey can be used to develop estimates for patching quantities. The patching quantity developed from a PCI survey is not intended to be the final quantity used in the contract documents, but rather an estimate to be provided to the project owner early (pre-semi-final) in the project design schedule. The extrapolated distress quantities are to be used in the development of the patching quantity estimate. The specific dimensions of the PCI distresses are not directly transferable into patch area. Besides the obvious PCI distresses measured in linear feet, the distresses measured in square feet are not always equivalent to a patch area. This is because MDSHA typically has standard minimum patch sizes and the PCI distress can be a small as 1 square foot. Therefore, in order to develop patching quantity estimates from extrapolated PCI distresses, a conversion factored is necessary. The following tables show the conversion factors to convert extrapolated PCI distresses into a patch estimate.

Flexible / Composite Pavements

Distress Type	Patch Area (sf)
Alligator	Extrapolated Area * 2
Bleeding	No Patch
Block	Extrapolated Area * 2
Bumps and Sags	Extrapolated Length * 6
Corrugation	Extrapolated Area * 2
Depression	Extrapolated Area * 2
Edge Cracking	Extrapolated Length * 6
Joint Reflection	Extrapolated Length * 6
Lane/Shld Drop off	No Patch
Linear Cracking	Extrapolated Length * 6
Patching	Extrapolated Area * 2
Polished Aggregate	No Patch
Pothole	Extrapolated Number * 72
RR Crossing	No Patch
Rutting	Extrapolated Area * 2
Shoving	Extrapolated Area * 2
Slippage	Extrapolated Area * 2
Swell	Extrapolated Area * 2
Weathering/Raveling	No Patch

Rigid Pavements

Distress Type	Patch Area (sf)*
Blow Up	480
Corner Break	36[#]
Divided Slab	480
Durability	No Patch
Faulting	72
Joint Seal Damage	No Patch
Lane/Shld Drop Off	No Patch
Linear Crack	72
Patch / Large	72
Patch / Small	36
Polished Aggregate	No Patch
Popouts	No Patch
Pumping	72
Punchout	72
RR Crossing	No Patch
Scaling	No Patch
Shrinkage	No Patch
Spall / Corner	36[#]
Spall / Joint	36[#]

* Based on a 12' x 40' slab. Three basic patch type sizes: entire slab, joint, and partial width. Exact area may change if slab dimensions are different.
[#] Partial width patches may not be recommended depending on PCC type, steel configuration, distress extent, and expected rehabilitation strategy.

IX CONSTRUCTION AND MATERIAL SPECIFICATIONS

A pavement designer in the Pavement Division of OMT must be familiar with construction operations and the specifications required by MDSHA on the contractor performing the work. The most beneficial and economical pavement recommendation is useless if it cannot be constructed or built with the existing construction or material specifications. Therefore, the pavement designer must be intimately familiar with existing materials and construction specifications and be possess the ability to develop new specifications.

MDSHA currently has two documents that all dictates the specifications and standards by which all projects shall be constructed, “Standard Specification for Construction and Materials” book and “Book of Standards.” Obviously, other specification documents are occasionally required for construction, but the initial reference and binding documentation are the two books previously mentioned. The “Standard Specification for Construction and Materials” book controls material and construction specifications. The “Specification” book is broken out into a Terms and Condition (TC) section and a Technical Requirements section. The TC section establishes the scope of work, definitions of terms, legal relations, restrictions and permits, and payment practices. The Technical Requirements section is broken out into the following category items:

- Category 100 – Preliminary
- Category 200 – Grading
- Category 300 – Drainage
- Category 400 – Structures
- Category 500 – Paving
- Category 600 – Shoulders
- Category 700 – Landscaping
- Category 800 – Traffic
- Category 900 – Materials

The “Book of Standards” controls typical details and dimensions of items. The “Book of Standards” is broken out into the same category section as the Technical Requirements section of the “Specification” book. Category 500 and 900 shall be of particular interest, and familiar, to the pavement designer in the Pavement division of the OMT.

Typically, MDSHA provides contractors desiring to bid on a construction project a proposal book and plans as part of the invitation for bid package. The plans provide a plan view of the project, typical sections, and specific details of construction. The proposal book provides additions to the “Standard Specification for Construction and Materials” and specific specification unique to the project. Special Provisions Inserts (SPI) are specifications that are revisions or an addendum to the current “Standard Specification for Construction and Materials” book. A SPI is a general specification that is applicable to all construction projects. A Special Provision (SP) is also a specification that is a revision or an addendum to the current “Standard Specification for Construction and Materials” book. However, a SP is specific to a particular project and is not applicable to all construction projects.

When a particular item or subject is not covered by the plans or proposal book, then the "Standard Specification for Construction and Materials" book and the "Book of Standards" are the controlling articles of documentation. The following is the order in which all construction and material specification / standard documentation shall be viewed, increasing in importance and controlling factor downward:

- Book of Standards & Specification Book
- Special Provision Inserts (SPI)
- Plans
- Special Provisions (SP)
- Addendum
- Redline Revision

The Specification Team of the Pavement Division of the OMT provides regular updates of all current SPIs and potential SPs to all pavement designers. Included in this section is a copy of the specification update provided by the Specification Team.

X MATERIAL LIBRARY

The Material Library provides the pavement design engineer with information about the construction materials used by MDSHA. The Material Library provides both the laboratory material properties and the AASHTO Pavement design properties of the construction materials

X.A MATERIAL DESCRIPTION

The following table provides a list and description of the material types used to construct pavements by MDSHA:

Material	Description
PCC	Surface material comprised of aggregate, portland cement, water, and potential admixtures.
HMA Superpave 4.75 mm	Surface or base materials comprised of aggregate, asphalt cement, and potential admixtures. Gap graded mix do not have a dense graded gradation curve because a band of aggregate sizes are excluded from the HMA mix to prevent rutting.
HMA Superpave 9.5 mm Gap Graded	
HMA Superpave 9.5 mm	
HMA Superpave 12.5 mm Gap Graded	
HMA Superpave 12.5 mm	
HMA Superpave 19.0 mm Gap Graded	
HMA Superpave 19.0 mm	
HMA Superpave 25.0 mm	
HMA Superpave 37.5 mm	
Microsurfacing	Surface sealer comprised of sequential applications of asphalt and stone chips in either single or repetitive layers. Traditional seal coating done by agencies.
Slurry Seal	Surface sealer comprised of sand-sized aggregate mixed with diluted asphalt emulsion, squeegeed onto the pavement surface. Usually 3/8" thickness.
Novachip	Surface sealer that places an ultra-thin coarse aggregate hot mix over a special polymer modified liquid asphalt membrane
Cement Treated Granular Base	Base material comprised of aggregate mixed with portland cement, typically 3.25% to 5% by weight. Generally mixed in plant. Target strength of 750 psi for unconfined compressive strength.
Asphalt Treated Aggregate Base	Base material comprised of aggregate mixed with asphalt cement, typically 3.25% to 5% by weight.
Penetration Macadam	Commonly used prior to the 1950's. A stone matrix of large stones at the bottom of the lift choked with increasingly larger aggregate sizes. A crude form of asphalt cement, tar, was then poured over the stone matrix after compaction.

Material	Description
Macadam	Commonly used prior to the 1950's. A stone matrix of large stones at the bottom of the lift choked with increasingly larger aggregate sizes. The term use used for a wide variety of materials that may meet this description; i.e. old bituminous concrete.
Soil Cement	Base material with granular soil mixed with portland cement, typically 8% to 10% by weight and mixed in place. Target strength of 450 psi unconfined compressive strength.
Cement Modified Subgrade	Subgrade material of on-site non-plastic soil mixed with portland cement, typically 3% to 5% by weight and mixed in place. Generally a poorer quality than Soil Cement with a target strength of 300 psi for unconfined compressive strength.
Graded Aggregate Base (GAB)	Base material of graded aggregates from crushed stone. GAB has narrow and defined sieve control points compared to CR-6. Typically of higher quality than CR-6 and more coarse.
Geosynthetically Stabilized Subgrade Using Graded Aggregate Base (GSS w/ GAB)	GAB material placed on geotextile that is placed at the aggregate-soil interface. Used in placed of undercutting in potential Class 1A excavation areas. Not typically considered part of the pavement section, but as a construction platform.
CR-6 – Crusher Run	Base material of graded aggregates from crushed stone. CR-6 has wider sieve control points and typically of poorer quality and finer than GAB.
Gravel – Bank Run Gravel	Base material typically rounded and uncrushed. Usually from riverbed areas.
Soil Contaminated Aggregate Base	Base material that has been contaminated from the soil beneath the layer or an aggregate layer that has been pushed into a soft subgrade.
Common Borrow	A soil and aggregate mixture with a minimum dry density of 100 pcf.
Select Borrow	A soil and aggregate mixture with A-2, A-3 or A-2-4 material with a maximum dry density of 105 pcf.
Capping Borrow	A soil and aggregate mixture identical to select borrow with the exception that when A-3 material has less than 10% retained on #10 sieve, at least 15% shall pass the #200 sieve. This specification is to ensure that the capping borrow has a broader band gradation.
Modified Borrow	A soil and aggregate mixture with a minimum of 25% retained on #10 sieve, liquid limit not greater than 30 and a plasticity index not greater than 9. Minimum maximum dry density of 115 pcf.

The following table provides a list and description of the various pavement types used by MDSHA:

Material	Description
Flexible Pavement	Pavement structure composed of hot mix asphalt material without any portland cement concrete layers.
Rigid Pavement	Pavement structure composed of portland cement concrete material on the surface.
Composite Pavement	Pavement structure composed of hot mix asphalt material layer above a portland cement concrete material layer.
Jointed Plain Concrete Pavement (JPCP)	Rigid pavement built with transverse joints, but without any steel reinforcement.
Jointed Reinforced Concrete Pavement (JRCP)	Rigid pavement built with transverse joints and with steel reinforcement between the joints.
Continuously Reinforced Concrete Pavement (CRCP)	Rigid pavement built without transverse joints and with steel reinforcement throughout the length of the pavement.

The following table provides a list and description of the soil types identified by MDSHA:

Material	Description
A-2	Sand and Fines
A-2-4	Silty Sand
A-2-7	Clayey Sand
A-3	Sand
A-4	Silt
A-4-2	Sandy Silt
A-4-7	Clayey Silt
A-5	Mica, Diatoms, Decomposed Rock
A-6	Colloidal Clay
A-7	Clay
A-7-2	Sandy Clays
A-7-4	Silty clay
A-8	Swamp Muck
Rock Refusal	

X.B MATERIAL PROPERTIES

Material properties are general material characteristics of common MDSHA pavement materials. Typical material property values are obtained from laboratory testing and assumed from engineering experience of the materials commonly used by MDSHA.

Elastic modulus and resilient modulus provide an indication of the strength of the material and are fundamentally the same material property. The higher the elastic or resilient modulus the stronger the material will behave under traffic loading. Elastic modulus defined is the slope of the stress-strain curve of a material specimen under loading. Elastic modulus is based on the principle the material does not permanently deform under loading. This is not true for a majority of the paving materials with the exception of maybe portland cement concrete (PCC). Resilient modulus is also defined as the slope of the stress-strain curve of a material specimen under loading. However, resilient modulus is based on the principle that the material may permanently deform under loading. This permanent deformation is monitored during the laboratory testing. Therefore, resilient modulus testing is basically elastic modulus testing while monitoring permanent deformation.

X.B.1 Elastic Modulus / Resilient Modulus

The following sections provide the pavement designer a frame of reference for the respective strengths of common pavement materials. This information can be used in determining the application of different materials based on a strength comparison. In addition, the ranges of elastic moduli values for the pavement materials in the following sections shall be used in backcalculation analysis of FWD data.

X.B.1.1 Surface / Base Elastic Modulus

The elastic modulus of surface and base materials are relatively independent of the subgrade strength. The weaker the surface and base material, the more its strength will be influenced by the strength of the underlying subgrade soil. The following table provides typical elastic modulus values for surface and base materials typical for MDSHA. The materials with asphalt concrete are temperature dependent materials. Hot mix asphalt will have a higher elastic modulus in colder temperatures and a weaker elastic modulus when exposed to warmer temperatures. Some of the manufactured or produced materials listed below are general ranges because they very dependent on the production method and the care taken during construction.

Surface / Base Elastic Modulus

Material	Minimum Modulus (psi)	Typical Modulus (psi)	Maximum Modulus (psi)
Portland Cement Concrete[1]	3,000,000	5,000,000	8,000,000
HMA, Surface[2]	250,000	750,000	1,500,000
HMA, Base[2]	200,000	600,000	1,250,000
Break/Crack and Seat Portland Cement Concrete	50,000	150,000	500,000
Cement Treated Granular Base[3]	50,000	1,000,000	1,500,000
Asphalt Treated Aggregate Base	50,000	150,000	500,000
Penetration Macadam	30,000	75,000	250,000

Material	Minimum Modulus (psi)	Typical Modulus (psi)	Maximum Modulus (psi)
Rubbilized Portland Cement Concrete[3]	25,000	75,000	250,000
Macadam	25,000	30,000	75,000
Graded Aggregate Base, Stone	15,000	25,000	45,000
Soil Cement[3]	15,000	400,000	750,000
Gravel	10,000	15,000	30,000
Cement Modified Subgrade[3]	5,000	250,000	500,000
Soil Contaminated Aggregate Base	3,000	10,000	20,000
Capping Borrow	10,500	10,500	10,500

1- American Concrete Institute correlation between elastic modulus and compressive strength is the following:

$$\text{Elastic Modulus (psi)} = 57000 * (\text{Compressive Strength (psi)})^{0.5}$$

2 - Temperature dependent

3 – Construction/Production dependent

X.B.1.2 Subgrade Resilient Modulus (M_r)

MDSHA Pavement Division records and designs flexible pavements based on the subgrade strength reported in terms of resilient modulus. Several other tests can be used to assess the strength of the subgrade; i.e. CBR. However, all analysis, designs, and reports will be done in terms of resilient modulus for the soil subgrade of flexible pavements. A simple conversion from CBR to resilient modulus for soil subgrade is the following equation:

$$M_r = 1500 * CBR$$

This equation has been questioned in terms of accuracy and properly identifying the relationship between M_r and CBR by the technical pavement industry. The highest level of confidence for this equation is for low CBR values, typically under a CBR value of 10.

The subgrade resilient modulus used for new or rehabilitation design shall be the average modulus for a particular section, not the lowest. If distinct smaller sections can be identified with different subgrade resilient moduli within a longer section, then the sections shall be broken out as separate sections and pavement rehabilitation designs shall be done for each section. It is the policy of MDSHA Pavement Division to have a maximum design subgrade modulus of soil conditions in their natural state without any treatment or modification. The maximum design M_r is 10,500 psi or a CBR of approximately 7. Occasionally, higher subgrade moduli values are found throughout the state, but because of the rather moist climate and the natural soils of Maryland, it has been decided to cap the maximum design subgrade modulus. The following table provides typical resilient modulus values for materials typical for MDSHA.

Subgrade Resilient Modulus

Material	General Strength	Typical Modulus (psi)	Typical CBR
Silts and Clays (w/ high compressibility)	Very Low	1,000 – 2,000	Less than 2
Fine Grain Soils with Silts and Clays (w/ low compressibility)	Low	2,000 – 3,000	2 to 2.5
Poorly Graded Sands	Medium	3,000 – 4,500	2.5 to 3
Gravely Soils, Well Graded Sands, and Sand/Gravel Mixtures	High	4,500 – 10,500	3 to 7

The values in the table above shall be used when FWD analysis data, laboratory information, or expert Geotechnical opinions are not available. The subgrade strength is an extremely influential parameter in the AASHTO pavement design process and every effort shall be completed to obtain an accurate and specific value for each section.

X.B.2 Modulus of Subgrade Reaction (k)

MDSHA Pavement Division records and designs rigid and composite pavements based on the subgrade strength reported in terms of modulus of subgrade reaction (k). In the same manner as flexible pavements, several other tests can be used to assess the strength of the subgrade; i.e. CBR. However, all analysis, designs, and reports will be done in terms of resilient modulus for the soil subgrade of flexible pavements and in terms of modulus of subgrade reaction for rigid and composite pavements.

In the case of flexible pavements, the resilient modulus is simply a material property of the soil subgrade. In the case of rigid and composite pavements, the modulus of subgrade reaction is a material property of the subgrade and aggregate base material as a whole. The aggregate base layer in AASHTO pavement design procedures does not contribute to the structural capacity of the pavement structure, it provides additional support to the subgrade. Therefore, subsequent rehabilitation strategies involving a HMA overlay of a rigid pavement, or additional HMA overlays of a composite pavement, the aggregate base layer and subgrade are viewed and analyzed a single material property, referred to a modulus of subgrade reaction. Unlike the resilient modulus of the subgrade in flexible pavements, the effect of k value is minimal in terms of structural improvements required for rigid and composite pavements.

A relationship has been developed between resilient modulus of the subgrade and the modulus of subgrade reaction of the subgrade/aggregate base combination. That relationship is represented in the following equation:

$$k = M_r / 19.4$$

An approach to evaluating the k value from FWD data is described in Section III.J "FWD Data Analysis". The AASHTO DarWin software program typically provides a reasonable k value for rigid and composite pavement sections. Occasionally, the pavement structure deflection behavior of the pavement structure does not provide a reasonable k value from the FWD analysis in DarWin. A typical range a k values for MDSHA pavement materials is from 100 pci to a maximum of 600 pci. Modulus of subgrade reaction greater than 700 should be closely evaluated to assure that any error in testing or analysis has not occurred. The following table provides typical ranges of k values for subgrade and aggregate materials.

Modulus of Subgrade Reaction (k)*

Material	Aggregate Thickness Greater than 6.0"	Minimum Value (pci)	Typical Value (pci)	Maximum Value (pci)
Silts and Clays (w/ high compressibility)	No	50	75	100
	Yes	75	100	150
Fine Grain Soils with Silts and Clays (w/ low compressibility)	No	100	125	150
	Yes	125	150	180
Poorly Graded Sands	No	150	175	220
	Yes	175	200	250
Gravely Soils, Well Graded Sands, and Sand/Gravel Mixtures	No	220	350	550
	Yes	250	400	600

* Bituminous or Cement treated bases or soils will obviously provide a greater amount support and a resulting higher k value.

X.B.3 Possion Ratio

The Poisson Ratio is a measure of a material's lateral strain compared to the axial strain. In other words, it the ratio of lateral movement compared to vertical movement of a material under a vertical load. Therefore, very stiff materials (PCC) will have a smaller Poisson Ratio compared to those materials that are elastic and have a flowable nature (soil and HMA). Poisson ratio can measure during resilient modulus testing.

Poisson Ratio is a critical material property when performing backcalculation analysis. Poisson Ratio allows the pavement designer to model the behavior of a material under loading. The following table provides typical ranges of Poisson Ratio for common MDSHA pavement materials.

Poisson Ratio

Material	Minimum Value	Typical Value	Maximum Value
Portland Cement Concrete*	0.15	0.15	0.20
Break/Crack and Seat Portland Cement Concrete	0.15	0.20	0.30
Rubbilized Portland Cement Concrete	0.15	0.25	0.30
Cement Treated Granular Base	0.15	0.20	0.30
HMA, Surface	0.30	0.35	0.40
HMA, Base	0.30	0.35	0.40
Soil Cement	0.20	0.30	0.40
Asphalt Treated Aggregate Base	0.20	0.35	0.40
Penetration Macadam	0.20	0.35	0.40

Material	Minimum Value	Typical Value	Maximum Value
Macadam	0.20	0.40	0.45
Graded Aggregate Base, Stone	0.20	0.40	0.45
Gravel	0.30	0.40	0.45
Soil Contaminated Aggregate Base	0.30	0.40	0.50
Sandy Soil Subgrade	0.30	0.45	0.50
Silt or Clay Soil Subgrade	0.40	0.45	0.50

X.B.4 Tensile Strength Modulus of PCC

There are two methods to measure the tensile strength of PCC, modulus of rupture (S'_c) and indirect tensile strength (f_t). The modulus of rupture (S'_c) is an indication of the flexural strength of PCC and is used in the AASHTO pavement design procedure to calculate slab thickness. The indirect tensile strength (f_t) is used in the AASHTO pavement design procedure to calculate steel reinforcement requirements. Typical values of modulus of rupture are from 550 psi to 850 psi for sound PCC. A good estimate for pavement design purposes is 650 psi. The modulus of rupture of jointed rigid pavement can be calculated using the following equation:

$$S'_c = 43.5 * (\text{Elastic Modulus of PCC} / (1*10^6)) + 488.5$$

The indirect tensile strength should be consistent with the modulus of rupture. For the AASHTO pavement design procedure, the indirect tensile strength will normally be about 86 percent of the concrete modulus of rupture.

X.B.5 Coefficient of Thermal Expansion

The coefficient of thermal expansion is a measure of the change in volume of a material subjected to a change in temperature. There are two areas interest with regard to thermal expansion for pavement design purposes, PCC and steel.

The PCC coefficient of thermal expansion varies with such factors as the water-cement ratio, concrete age, cement content, and relative humidity. However, the coarse aggregate thermal properties are the most significant influence on the thermal expansion and contraction of a PCC slab. The recommended PCC thermal coefficients of expansion are presented in the following table:

Type of Aggregate	Concrete of Thermal Coefficient ($1x10^{-6}$ in/in/°F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

The steel coefficient of thermal expansion should be dependent on the steel type manufacturer. Without that knowledge, the designer should use the following value for the reinforcing steel coefficient of expansion:

5.0 X 10^{-6} in/in/°F

X.B.6 Drying Shrinkage Coefficient of PCC Slab

The shrinkage that occurs in a curing PCC slab needs to be considered in the design of the longitudinal reinforcement steel design in CRCP and the joint reservoir in jointed rigid pavements. Drying shrinkage is a result of water loss from curing PCC that is affected by cement content, the types of admixtures, the curing method, the aggregates, and curing conditions. The value of shrinkage at 28 days is used for the design shrinkage value. The shrinkage factor of a PCC slab is inversely proportional to the to the strength. Therefore, the more water that is added to a PCC mix the greater the potential for shrinkage becomes and the strength of the PCC will decrease. So, the 28-day indirect tensile strength can be used as a guide in selecting a drying shrinkage coefficient. The recommended drying shrinkage coefficients for PCC slabs are presented in the following table:

Indirect Tensile Strength (psi)	Shrinkage (in/in)
300 or <	0.0008
400	0.0006
500	0.00045
600	0.0003
700 or >	0.0002

X.B.7 Steel Reinforcement Bar Dimensions

The following is a listing of the typical reinforcement bar dimensions in terms of diameter and cross sectional area for dowels and tie bars.

Bar Size	Diameter	Cross-sectional Area (in^2)
#3	0.375	0.11
#4	0.500	0.20
#5	0.625	0.31
#6	0.750	0.44
#7	0.875	0.60
#8	1.000	0.79
#9	1.128	1.00
#10	1.270	1.27
#11	1.410	1.56

X.B.8 Steel Mesh Dimensions

The following is a listing of the typical welded wire mesh dimensions in terms of spacing between wires and the cross sectional area for both smooth and deformed wire mesh.

Wire Size #		Diameter (in)	Cross-sectional Area (in²/ft)				
			Center-to-Center Spacing (in)				
Smooth	Deformed		4	6	8	10	12
W31	D31	0.628	0.93	0.62	0.465	0.372	0.31
W30	D30	0.618	0.90	0.60	0.45	0.36	0.30
W28	D28	0.597	0.84	0.56	0.42	0.336	0.28
W26	D26	0.575	0.78	0.52	0.39	0.312	0.26
W24	D24	0.553	0.72	0.48	0.36	0.288	0.24
W22	D22	0.529	0.66	0.44	0.33	0.264	0.22
W20	D20	0.504	0.60	0.40	0.30	0.24	0.20
W18	D18	0.478	0.54	0.36	0.27	0.216	0.18
W16	D16	0.451	0.48	0.32	0.24	0.192	0.16
W14	D14	0.422	0.42	0.28	0.21	0.168	0.14
W12	D12	0.390	0.36	0.24	0.18	0.144	0.12
W11	D11	0.374	0.33	0.22	0.165	0.132	0.11
W10.5		0.366	0.315	0.21	0.157	0.126	0.105
W10	D10	0.356	0.30	0.20	0.15	0.12	0.10
W9.5		0.348	0.285	0.19	0.142	0.114	0.095
W9	D9	0.338	0.27	0.18	0.135	0.108	0.09
W8.5		0.329	0.255	0.17	0.127	0.102	0.085
W8	D8	0.319	0.24	0.16	0.12	0.096	0.08
W7.5		0.309	0.225	0.15	0.112	0.09	0.075
W7	D7	0.298	0.21	0.14	0.105	0.084	0.07
W6.5		0.288	0.195	0.13	0.097	0.078	0.065
W6	D6	0.276	0.18	0.12	0.09	0.072	0.06
W5.5		0.264	0.165	0.11	0.082	0.066	0.055
W5	D5	0.252	0.15	0.10	0.075	0.06	0.05
W4.5		0.240	0.135	0.09	0.067	0.054	0.45
W4	D4	0.225	0.12	0.08	0.06	0.048	0.04

Welded wire mesh is prefabricated reinforcement consisting of parallel series of high-strength, cold drawn wires welded together in square or rectangular grids. The nomenclature for specifying wires is identified by the spacing between the wires and then the wire size number as follows; 6 X 12 – W8 X W6. The spacing of the longitudinal wires is the first number (6) and the spacing of the transverse wires is the second number (12). The size of the longitudinal wire is the third number (W8) and the size of the transverse wire is the last number (W6). The size of the wires also indicates the cross-sectional area per foot of wire; i.e. W8 is equivalent to 0.08 in^2 per foot.

X.B.9 HMA Unit Weight

The material unit weight is critical information in order to develop accurate quantities to be placed into contract documents. The unit weight of HMA is the most used unit weight when developing material quantities. At this time, the only substantial historical information concerning measured unit weights of HMA mixes is for the older Marshall mix design mixes. Historical unit weight information for placed HMA Superpave mixes will be available in the near future. Until that information becomes available, the Marshall mixes will be adequate for estimating purposes.

Several factors contribute to the unit weight of HMA that are beyond the control of the designer developing the material quantities. These factors include, but are not limited to the following:

- Specific gravity of aggregate – varies by supplier
- Compaction effort by contractor
- Climatic conditions at time of construction
- Construction practices of contractor

The following table presents the historical (1999) unit weights of Superpave mixes developed from maximum specific gravity and based on a 4% air void assumption.

Material	Average Unit Weight (lbs/ft^3)
4.75 mm	153.2
9.5 mm	147.5
12.5 mm	148.5
19.0 mm	149.9
25.0 mm	150.9
12.5 mm Gap Graded	152.1
19.0 mm Gap Graded	150.2
Non GG Surface Mixes	**149.7**
Base Mixes	**150.4**
All Mixes	**150.3**

The majority of the pavement design analysis effort is to develop a thickness of rehabilitation and an area of the rehabilitation treatment. The following equation shows a simplified method of developing HMA quantities knowing the thickness of the material and the predicted area of the treatment.

$$\text{HMA Quantity (tons)} = [\text{Area (yd}^2\text{)} * \text{Thickness of Lift (inches)}] / 17$$

The simplified equation is developed from the following equation that involves conservative unit weight estimate and material unit conversions.

$$\text{HMA Quantity (tons)} = \text{Area (yd}^2\text{)} * \text{Thickness of Lift (inches)} * (1 \text{ yd} / 36 \text{ inches}) * (27 \text{ ft}^3 / 1 \text{ yd}^3) * \text{Unit Weight HMA (\#/ft}^3\text{)} * (1 \text{ ton} / 2000 \text{ \#})$$

A denominator of 17 in the simplified equation is equivalent to a HMA unit weight of 156.9 lbs/ft^3. This value is slightly higher than our historical average for HMA unit weights shown in the table above, but this is designed to be a conservative estimate and it is determined this simplified equation provides an adequate quantity.

X.C DESIGN PROPERTIES

Design properties are the input parameters of pavement materials used in new or pavement rehabilitation design procedures. Design properties are based on the material properties, but are expressed in a form to be used in the pavement design process. This Section describes the design input properties used by MDSHA Pavement Division.

X.C.1 Drainage Coefficient Factor

The drainage coefficient or any variation of the structural coefficient based on saturation levels shall only be considered for unbound material layers. The drainage coefficient for bound materials shall be equal to one (1.0). MDSHA has adopted an approach to adjust the structural coefficient of unbound layers based on the saturation level rather than introduce a drainage coefficient. The following equations present the relation between the subgrade strength, degree of saturation, and material type to the structural coefficient of unbound layers.

Graded Aggregate Base, Stone

HMA Thickness <= 5.0"

$$a_2 = (0.14 + 0.0029 * (\text{Subgrade (CBR)}) + f_s$$

HMA Thickness > 5.0"

$$a_2 = (0.14 + 0.0029 * (\text{Subgrade (CBR)}) – 0.035 + f_s$$

f_s = 0.0 for dry conditions
f_s = -0.033 for wet conditions

Gravel

HMA Thickness <= 5.0"

$$a_2 = (0.08 + 0.0064 * (\text{Subgrade (CBR)}) + f_s$$

HMA Thickness > 5.0"

$$a_2 = (0.10 + 0.0021 * (\text{Subgrade (CBR)}) + f_s$$

f_s = 0.0 for dry conditions
f_s = -0.046 for wet conditions

Wet conditions are defined as when the unbound material is saturated more than 60% of the time. Based on the climatic conditions and the natural soils in Maryland, it is recommended to assume that wet conditions exist unless evidence can be provided to show that the unbound base is dry a majority of the time. These special cases would only exist where an open-graded base was used or in topographical conditions with good vertical drainage and clean sands/gravel exists for several feet beneath the top of subgrade.

X.C.2 Friction Factor

The following are recommended friction values for the subgrade and a variety of base materials to be used in rigid pavement design:

Material Beneath Slab	Friction Factor
Surface Treatment	2.2
Lime Stabilization	1.8
Asphalt Stabilization	1.8
Cement Stabilization	1.8
River Gravel	1.5
Crushed Stone	1.5
Sandstone	1.2
Natural Subgrade	0.9

X.C.3 *Design Properties for Pavement Materials*

The following table presents numerous design parameters for materials commonly used by MDSHA.

Material	Design Use	Structural Coefficient Range for New Material	Desired Structural Coefficient	Min. Lift Thickness	Desired Lift Thickness	Max. Lift Thickness	Structural Coefficient Range After Deterioration	Structural Coefficient for Deteriorated Material	Drainage Coefficient
PCC	Surface	N/A	N/A	6.0”	N/A	14.0”	N/A	N/A	N/A
Break/Crack and Seat PCC	Base	0.20 – 0.35	0.25	6.0”	N/A	14.0”	N/A	N/A	1.0
Rubbilized PCC	Base	0.15 – 0.30	0.20	6.0”	N/A	14.0”	N/A	N/A	1.0
HMA Superpave 4.75 mm	Surface	0.44	0.44	0.5”	0.75”	0.75”	0.3 – 0.44	0.38	1.0
HMA Superpave 9.5 mm Gap Graded	Surface	0.44	0.44	1.0”	1.5”	1.5”	0.3 – 0.44	0.38	1.0
HMA Superpave 9.5 mm	Surface, W/L	0.44	0.44	1.0”	1.5”	2.0”	0.3 – 0.44	0.38	1.0
HMA Superpave 12.5 mm Gap Graded	Surface	0.44	0.44	1.5”	2.0”	2.0”	0.3 – 0.44	0.38	1.0
HMA Superpave 12.5 mm	Surface	0.44	0.44	1.5”	2.0”	2.5”	0.3 – 0.44	0.38	1.0
HMA Superpave 19.0 mm Gap Graded	Surface	0.44	0.44	2.0”	2.0”	2.5”	0.3 – 0.44	0.38	1.0
HMA Superpave 19.0 mm	Base, Surface	0.40	0.40	2.0”	2.5”	4.0”	0.3 – 0.40	0.36	1.0
HMA Superpave 25.0 mm	Base	0.40	0.40	3.0”	3.5”	5.0”	0.3 – 0.40	0.36	1.0
HMA Superpave 37.5 mm	Base	0.38	0.38	4.0”	5.0”	6.5”	0.3 – 0.38	0.34	1.0
Cement Treated Granular Base	Base	0.15 – 0.30	0.25	4.0”	6.0”	6.0”	0.15 – 0.30	0.25	1.0
Asphalt Treated Aggregate Base	Base	0.10 – 0.25	0.20	4.0”	6.0”	6.0”	0.10 – 0.25	0.20	1.0
Penetration Macadam	Base	0.10 – 0.25	0.20	3.0”	6.0”	8.0”	0.10 – 0.25	0.20	1.0
Macadam	Base	0.10 – 0.20	0.15	3.0”	6.0”	8.0”	0.10 – 0.20	0.15	*
Soil Cement	Base	0.15 – 0.25	0.20	4.0”	6.0”	6.0”	0.15 – 0.25	0.20	1.0

Material	Design Use	Structural Coefficient Range for New Material	Desired Structural Coefficient	Min. Lift Thickness	Desired Lift Thickness	Max. Lift Thickness	Structural Coefficient Range After Deterioration	Structural Coefficient for Deteriorated Material	Drainage Coefficient
Cement Modified Subgrade*	Subbase	0.05 – 0.07	0.06	4.0”	6.0”	8.0”	0.05 – 0.07	0.06	1.0
Graded Aggregate Base, GAB	Base	0.08 – 0.14	0.12	3.0”	6.0”	6.0”	0.08 – 0.14	0.11	*
Bank Run Gravel	Base	0.06 – 0.12	0.10	3.0”	6.0”	6.0”	0.06 – 0.12	0.10	*
CR-6 Crusher Run	Base	0.06 – 0.12	0.09	3.0”	6.0”	6.0”	0.06 – 0.12	0.09	*
GSS w/ GAB	Base	0.05 – 0.10	0.08	3.0”	6.0”	12.0”	0.05 – 0.10	0.08	*
Soil Contaminated Aggregate Base	Base	0.05 – 0.10	0.08	3.0”	6.0”	6.0”	0.05 – 0.10	0.08	*
Common Borrow	Subbase	0.03 – 0.06	0.04	3.0”	6.0”	8.0”	0.03 – 0.06	0.04	*
Select Borrow	Subbase	0.04 – 0.08	0.05	3.0”	6.0”	8.0”	0.04 – 0.08	0.05	*
Capping Borrow	Subbase	0.04 – 0.08	0.06	3.0”	6.0”	8.0”	0.04 – 0.08	0.06	*
Modified Borrow	Subbase	0.05 – 0.09	0.07	3.0”	6.0”	8.0”	0.05 – 0.09	0.07	*

* MDSHA has adopted an approach to adjust the structural coefficient of unbound base rather than introduce a drainage coefficient. The structural coefficient of the unbound layer is effected by the thickness of the overlying material and degree of saturation of the base layer. This section further describes this relationship between degree of saturation and the effect on structural coefficient.