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DARWIN-ME Pavement Analysis and Design Manual for NJDOT

Book 2 – Manual of Practice

Vitillo, Nicholas

Rutgers - CAIT Pavement Resource Program

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Darwin ME

Flexible Pavement Design and Flexible Overlay Design

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Darwin ME – Flexible Pavement Design and Flexible Overlay Design And Material Inputs

The design manual that follows is based on excerpts from the 2007 Manual of Practice for the ME Pavement Design Guide, the Darwin-ME Help files, and NJDOT Research Reports characterizing material inputs for Subgrade, Aggregate Base and Subbase, and HMA materials. Material characterization and design parameters for chemically stabilized soil material and PCC pavements can be found in the 2007 Manual of Practice for the ME Pavement Design Guide, and the Darwin-ME Help files. A new Manual of Practice for the Darwin ME Pavement Design Guide is being developed.

2007 Manual of Practice for the ME Pavement Design Guide

12 Pavement Design Strategies

The MEPDG design process requires the selection of a trial design with all inputs defined. As noted in DARWIN-ME Pavement Analysis and Design Manual for NJDOT Volume 1, the initial trial design may be determined using the 1993 AASHTO Design Guide, other M-E based design procedures, a design catalog, or the user simply identifying the design features and layer thicknesses. This section provides guidance to the designer in developing the initial pavement design strategy for the site conditions and describes new or reconstructed pavement design strategies for flexible and rigid pavements. The designer is referred back to DARWIN-ME Pavement Analysis and Design Manual for NJDOT Volume 1, Section 3 to ensure that the design strategy selected and prepared for analysis is consistent with those calibrated globally or locally in accordance with the MEPDG software.

12.1 New Flexible Pavement Design Strategies – Developing the Initial Trial Design

The MEPDG flexible pavement design procedure allows a wide variety of HMA mixtures, aggregate base layers, and foundation improvements. Specific types of flexible pavement systems that may be analyzed include conventional flexible sections, deep strength sections, full-depth sections, and semi-rigid sections (refer to Figure 6 and 7 under subsection 3.3). The definition for each of these pavement systems was included in Section 3.3.

In setting up an initial new design strategy for flexible pavements, the designer should simulate the pavement structure and foundation as detailed as possible, and then combine layers, as needed. It is recommended that the designer start with the fewest layers as possible to decrease the amount of inputs and time needed to estimate those inputs. Although more than 10 layers may be included in the trial design, the designer needs to limit the number of layer to no more than 6 to begin the design iteration process – 2 HMA layers, an unbound aggregate base, a stabilized subgrade or improved embankment, the subgrade layer, and a rigid layer, if present.

The designer could identify the types of layers and materials to be included in the trial design, and then decide on the inputs for the project site. The following subsections provide some simple rules to start developing the design strategy.

12.1.1 Should the Subgrade Soil be Strengthened/Improved?

The designer needs to evaluate the boring logs and test results prepared from the subsurface or field investigation and determine the subsurface soil strata – the different types of soils, their stiffness, and their thickness. If different soil strata are located with significantly different resilient modulus values along the project, those layers could be included as different soil layers. For example, a wet silty-sandy clay strata with a resilient modulus less than 8,000 psi overlying an over-consolidated, dense clay strata with a resilient modulus exceeding 25,000 psi.

An important step of the new flexible pavement design strategy is to begin with a good foundation for the pavement layers. Proper treatment of problem soil conditions and the preparation of the foundation layer are important to ensure good performance of flexible pavements. Starting with a good foundation that retains good support for the flexible pavement over time cannot be overemphasized and will not require thick paving layers. It needs to be remembered that the MEPDG does not directly predict the increase in roughness or IRI caused by expansive, frost susceptible, and collapsible soils. If these types of problem soils are encountered, treatments to minimize their long-term effects on flexible pavements need to be included in the design strategy.

The designer needs to review the results from the subsurface investigation and provide a foundation layer with a resilient modulus of at least 10,000 psi for supporting any unbound aggregate layer. If the subgrade has a resilient modulus less than 10,000 psi, the designer could consider improving or strengthening the subgrade soils. Different options that may be used depending on the conditions encountered include using select embankment materials, stabilizing the subgrade soil, removing and replacing weak soils, and/or adding subsurface drainage layers.

More importantly, the MEPDG does not predict or consider the lateral flow of subsurface water. If subsurface lateral flow is expected based on the experience of the designer in the area or from observations made during the subsurface investigation, subsurface drainage systems need to be considered to prevent water from saturating the pavement layers and foundation. Saturation of the paving materials and foundation will significantly decrease the resilient modulus of the unbound materials and soils. The MEPDG only predicts the effects of water moving upward into the pavement layers from ground water tables located close to the surface.

In addition, filter fabrics, geotextiles, and geogrids (for example, AASHTO M 288) cannot be directly simulated in the pavement structure. Agencies that routinely use these materials in their standard design sections or strategies need to determine their benefit or effect through the local calibration process for each performance indicator (distresses and smoothness). Manuals and training courses are available for designers to use regarding design and construction guidelines for geosynethics (Holtz, et al., 1998; Koerner, 1998), as well as AASHTO PP 46 – Recommended Practice for Geosynthetic Reinforcement of the Aggregate Base Course of Flexible Pavement Structures.

12.1.2 Is a Rigid Layer or Water Table Present?

A rigid or apparent rigid layer is defined as the lower soil stratum that has a high resilient or elastic modulus (greater than 100,000 psi). A rigid layer may consist of bedrock, severely weathered bedrock, hard-pan, sandstone, shale, or even over-consolidated clays.

If a rigid layer is known to exist along the project boundaries, that layer could be included in the analysis. When a rigid layer is simulated, however, the MEPDG limits the thickness of the last subgrade layer to no more than 100 inches. The designer may need to use multiple subgrade layers when the depth to bedrock exceeds 100 inches. In some areas, multiple-thin strata of rock or hard-pan layers will be encountered near the surface.

The designer could enter an equivalent elastic modulus for this condition and assume that it is bedrock.

Another important point when a rigid layer or rock outcropping is known to exist is the possibility of subsurface water flow above the rigid layer. The designer could have considered this in setting up the subsurface investigation plan for sites with rock outcroppings and rigid layers near the surface. The designer could evaluate the results from the subsurface investigation to determine whether a subsurface drainage system is needed to quickly remove and/or intercept subsurface water flow. This design feature does not relate to the surface infiltration of rainfall water.

When a water table is located near the surface (within 5 feet), a subsurface drainage system is recommended as part of the design strategy (NHI, 1999). The depth to a water table that is entered into the MEPDG software is the depth below the final pavement surface. The designer has the option to enter an annual depth to the water table or seasonal water table depths. The average annual depth could be used, unless the designer has historical data to determine the seasonal fluctuations of the water table depth. If a subsurface drainage system is used to lower that water table, that lower depth could be entered into the program, not the depth measured during the subsurface investigation.

12.1.3 Compacted Embankment or Improved Subgrade Layer Present?

The designer could divide the subgrade into two layers, especially when bedrock or other hard soils are not encountered. Most new alignment projects or new construction projects require that the surface of the subgrade be scarified and compacted after all vegetation has been removed and the elevation has been rough cut. The designer could consider simulating the compacted subgrade as a separate layer, as long as that layer is compacted to a specified density and moisture content that are based on laboratory prepared moisture-density relationships. When used in the trial design, this layer needs to be a minimum of 8 inches thick.

The default values included in the MEPDG software for resilient modulus of unbound materials and soils (refer to subsection 11.5) represent the material placed at optimum moisture content and compacted to its maximum dry unit weight (as defined by AASHTO T 180). If an embankment, improved subgrade, or other material is placed and compacted to a different moisture content and dry unit weight, the default values for resilient modulus need not be used. The design resilient modulus could be determined from an agency's historical database, repeated load resilient modulus tests (performed on test specimens compacted to the agency's specifications), other strength tests (CBR and R-Value), or estimated from regression equations (for example, those developed from the LTPP resilient modulus database [Von Quintus and Yau, 2001]).

12.1.4 Should a Drainage Layer be Included in the Design Strategy?

The use of a drainage system to remove surface water infiltration is dependent on the user's standard design practice. The MEPDG recommends that water not be allowed to accumulate within the pavement structure. Water may significantly weaken aggregate base layers and the subgrade soil, and result in stripping of HMA layers. The MEPDG assumes that all water-related problems will be addressed via the materials and construction specifications, and/or inclusion of subsurface drainage features in the design strategy. NHI Course 131026 provides guidelines and recommendations for the design and construction of subsurface drainage features (NHI, 1999).

The value and benefit of a drainage layer (either an asphalt treated permeable base or permeable aggregate base layer) beneath the dense graded HMA layers is debatable. If an asphalt treated permeable base drainage layer is used directly below the last dense-graded HMA layer, the ATPB needs to be treated as a high quality, crushed stone base layer (refer to subsections 3.5 and 5.2.3). The equivalent annual modulus for an ATPB (high quality aggregate base) that has been used is 65,000 to 75,000 psi. The minimum thickness of an ATPB layer should be 3 inches.

When a subsurface drainage layer is used, it needs to be day-lighted, if possible, or edge drains will need to be placed. The longitudinal, pipe edge drains should have marked lateral outlets adequately spaced to remove the water. A typical edge drain pipe is a 4- inch flexible pipe. Other drainage pipes may consist of rigid, corrugated PVC with smooth interior walls. The back-fill material generally consists of pea gravel or other aggregate materials that have high permeability. The aggregate placed in the trench needs to be well compacted and protected. The use of filter cloth is essential to limit infiltration of fines into the drainage system.

These edge drains need to be inspected after placement and must be maintained over time to ensure positive drainage. The inspection at construction and over time is no different than required for new pavement construction. Mini-cameras may be used to facilitate the inspection and maintenance needs of edge drains. If an agency or owner does not have some type of periodic inspection and maintenance program for these drainage layers and edge drains, the designer could consider other design options, and accordingly reduce the strength of the foundation and unbound layers.

12.1.5 Use of a Stabilized Subgrade – for Structural Design or a Construction Platform?

Lime and/or lime-fly ash stabilized soils could be considered a separate layer, if at all possible. If these layers are engineered to provide structural support and have a sufficient amount of stabilizer mixed in with the soil, they need to be treated as a structural layer.

Under this case, they could be treated as a material that is insensitive to moisture and the resilient modulus or stiffness of these layers can be held constant over time. The National Lime Association manual may be used for designing and placing a lime stabilized layer to provide structural support (Little, 2000). If other stabilizers such as Portland cement and lime-fly ash combinations are used, other manuals could be followed for designing and placing stabilized subgrade layers (PCA, 1995).

On the other hand, when a stabilized subgrade is used as a construction platform for compacting other paving layers, only a small amount of lime or lime-fly ash is added and mixed with the soil. For this case, these layers could be treated as unbound soils. In addition, if these materials are not "engineered" to provide long-term strength and durability, they could also be considered as an unbound material and possibly combined with the upper granular layer.

12.1.6 Should an Aggregate Base/Subbase Layer be Placed?

Unbound aggregate or granular base layers are commonly used in flexible pavement construction, with the exception for full-depth HMA pavements (refer to subsection 3.3).

In most cases, the number of unbound granular layers need not exceed two, especially when one of those layers is thick (more than 18 inches). Sand and other soil-aggregate layers could be simulated separately from crushed stone or crushed aggregate base materials, because the resilient modulus of these materials will be significantly different.

When aggregate or granular base/subbase layers are used, the resilient modulus of these layers is dependent on the resilient modulus of the supporting layers. As a rule of thumb, the resilient modulus entered as the starting value for a granular layer need not exceed a ratio of about 3 of the resilient modulus of the supporting layer to avoid decomposition of that layer. This rule of thumb may apply to all unbound layers. Figure 29 below may be used to estimate the maximum resilient modulus of an unbound layer that depends on its thickness and the resilient modulus of the supporting layers (Barker and Brabston, 1975).

12.1.7 HMA Layers – What Type and How Many?

The number of HMA layers need not exceed three in all cases. As for the unbound materials, similar HMA mixtures could be combined into one layer. Thin layers (less than 1.5 inches in thickness) could be combined with other layers. The minimum lift or layer thickness used for construction may be four times the nominal maximum aggregate size of the HMA mixture.

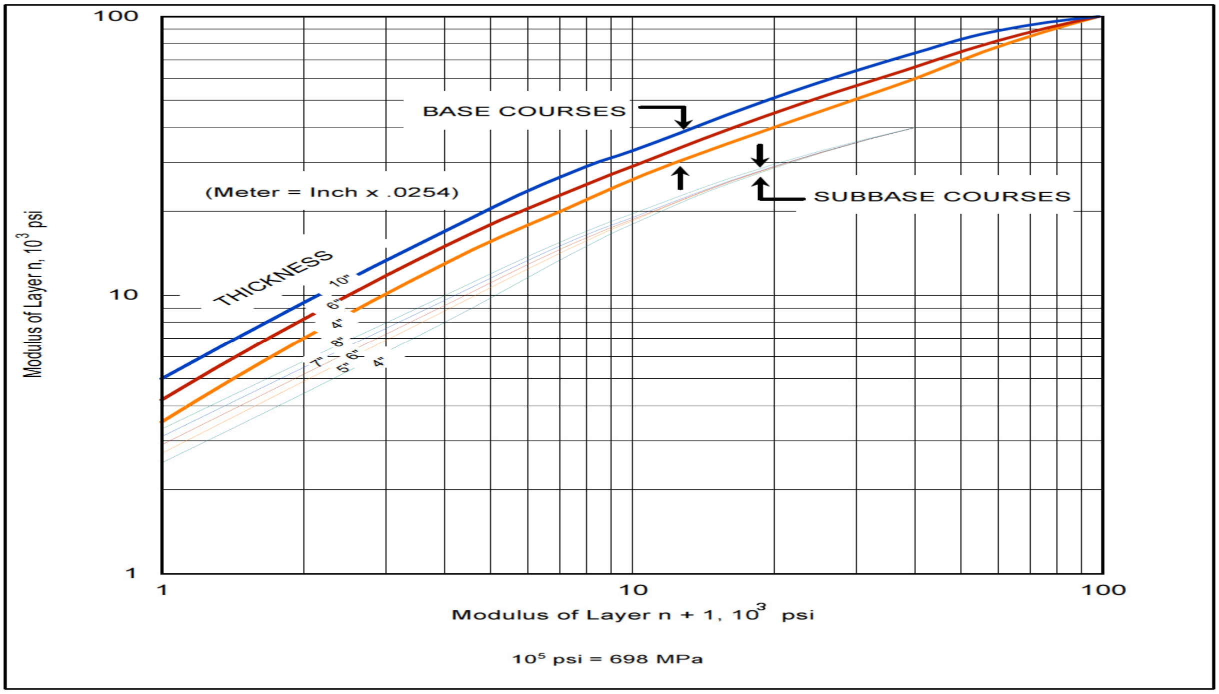


Figure 29. Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers

More importantly, thin wearing courses of a plant seal mix, porous friction course, open-graded friction course and other similar mixtures could be combined with the next layer beneath the wearing surface. The low temperature cracking and load related top-down (longitudinal) cracking models use the properties of the wearing surface in predicting the length of transverse and longitudinal cracks throughout the HMA layers.

Similarly, the alligator cracking model takes the properties of the lowest HMA layer and predicts the percent of total lane area with alligator cracking. As a result, the designer needs to carefully consider the properties being entered into the MEPDG software for the lowest HMA layer and HMA wearing surface.

When multiple layers are combined for the trial design, the volumetric properties (air voids, effective asphalt content, gradation, unit weight, and VFA) entered into the MEPDG software need to represent weighted average values based on the layer thickness of the layers that are combined. A wearing surface greater than 1.5 inches in thickness that has different PG asphalt than the underlying HMA layer needs to be considered as a separate layer. Similarly, a dense-graded HMA base layer (the lowest HMA layer) that is more than 3 inches thick could be considered as a separate layer. All other layers could be combined into the intermediate layer, if possible.

If an APTB layer with high air voids (typically greater than 15 percent) is included as an HMA layer, the high air voids will significantly increase the amount of fatigue cracking of the pavement structure (refer to subsection 12.1.4).

12.1.8 What Initial IRI Value Should be Used?

An initial IRI value is required for each pavement strategy or trial design considered. The initial IRI value could be taken from previous years' construction acceptance records, if available. Not all agencies, however, use IRI in accepting the pavement related to smoothness criteria. The following provides some recommendations for those agencies or users that do not use IRI as a basis for accepting the final surface.

11. Determination of Material Properties for New Paving Materials

The MEPDG procedure requires that all material properties entered into the program for new layers represent the values that exist right after construction. Obviously, the in-place properties for new paving layers will be unavailable to the designer because the project has yet to be built. Thus, most of the material property inputs need to be estimated for most runs (inputs levels 2 or 3). This section provides guidance for estimating the critical properties of the paving layers for new pavement and rehabilitation design strategies.

11.1 Material Inputs and the Hierarchical Input Concept

The general approach for determining design inputs for materials in the MEPDG is a hierarchical (level) system (as defined in Volume 1, Sections 4 and 6). In its simplest and most practical form, the hierarchical approach is based on the philosophy that the level of engineering effort exerted in the pavement design process for characterizing the paving materials and foundation should be consistent with the relative importance, size, and cost of the design project.

Input level 1 involves comprehensive laboratory tests. In contrast, level 3 requires the designer to estimate the most appropriate design input value of the material property based on experience with little or no testing. The major material types for which default values (input level 3) are available in the MEPDG are presented in Table 19 below. Level 2 inputs are estimated through correlations with other material properties that are commonly measured in the laboratory or field. Regardless of input level selected, the program runs the same analysis. As noted above, most of the analysis runs will be completed using input levels 2 and 3, because the paving layers have yet to be placed at the time that the structural analysis is completed.

11.2 HMA Mixtures; Including SMA, Asphalt Treated or Stabilized Base Layers, Asphalt Permeable Treated Base Mixes

Fundamental properties are required for all HMA mixture types or layers to execute the MEPDG. Table 20 lists the HMA material properties that are required for the HMA material types listed in Table 19, as well as identify the recommended test protocols and other sources for estimating these properties. The input properties for all HMA material types may be grouped into volumetric and engineering properties. The volumetric properties include air voids, effective asphalt content by volume, aggregate gradation, mix density, and asphalt grade. The volumetric properties entered into the program need to be representative of the mixture after compaction, before the pavement is opened to truck traffic. Obviously, the project specific values will be unavailable to the designer because the new pavement layers have yet to be produced and placed. However, these parameters could be available from previous construction records.

The engineering or mechanistic properties for HMA materials include the dynamic modulus, creep compliance, and indirect tensile strength. It is recommended that input levels 2 or 3 be used to estimate these properties, unless the agency or user has a library of laboratory test results for different HMA mixtures. The use of library test data is considered input level 2.

Table 19. Major Material Types for the MEPDG

<table>
<tr>
<td>
<u>Asphalt Materials</u>

Stone Matrix Asphalt (SMA)
Hot Mix Asphalt (HMA)

Dense Graded
Open Graded Asphalt
Asphalt Stabilized Base Mixes
Sand Asphalt Mixtures

Cold Mix Asphalt

Central Plant Processed
Cold In-Place Recycling

<u>PCC Materials</u>

Intact Slabs – PCC

High Strength Mixes
Lean Concrete Mixes

Fractured Slabs

Crack/Seat
Break/Seat
Rubblized

<u>Chemically Stabilized Materials</u>

Cement Stabilized Aggregate
Soil Cement
Lime Cement Fly Ash
Lime Fly Ash
Lime Stabilized Soils
Open graded Cement Stabilized Aggregate

</td>
<td>
<u>Non-Stabilized Granular Base/Subbase</u>

Granular Base/Subbase
Sandy Subbase
Cold Recycled Asphalt Mix (used as aggregate)

RAP (includes millings)
Pulverized In-Place

Full Depth Reclamation (In-Place (Cold Recycled Asphalt Pavement; (HMA plus aggregate base/subbase)

<u>Subgrade Soils</u>

Gravelly Soils (A-1;A-2)
Sandy Soils

Loose Sands (A-3)
Dense Sands (A-3)
Silty Sands (A-2-4;A-2-5)
Clayey Sands (A-2-6; A-2-7)

Silty Soils (A-4;A-5)
Clayey Soils, Low Plasticity Clays (A-6)

Dry-Hard
Moist Stiff
Wet/Sat-Soft

Clayey Soils, High Plasticity Clays (A-7)

Dry-Hard
Moist Stiff
Wet/Sat-Soft

<u>Bedrock</u>

Solid, Massive and Continuous
Highly Fractured, Weathered

</td>
</tr>
</table>

Table 20. Asphalt Materials and the Test Protocols for Measuring the Material Property Inputs for New and Existing HMA Layers

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New HMA (new pavement and overlay mixtures), as built properties prior to opening to truck traffic	Dynamic modulus	X		AASHTO TP 62
	Tensile strength	X		AASHTO T 322
	Creep Compliance	X		AASHTO T 322
	Poisson's ratio		X	National test protocol unavailable. Select MEPDG default relationship
	Surface shortwave absorptivity		X	National test protocol unavailable. Use MEPDG default value.
	Thermal conductivity	X		ASTM E 1952
	Heat capacity	X		ASTM D 2766
	Coefficient of thermal contraction		X	National test protocol unavailable. Use MEPDG default values.
	Effective asphalt content by volume	X		AASHTO T 308
	Air voids	X		AASHTO T 166
	Aggregate specific gravity	X		AASHTO T 84 and T 85
	Gradation	X		AASHTO T 27
	Unit Weight	X		AASHTO T 166
	Voids filled with asphalt (VFA)	X		AASHTO T 209
Existing HMA mixtures, in-place properties at time of pavement evaluation	FWD backcalculated layer modulus	X		AASHTO T 256 and ASTM D 5858
	Poisson's ratio		X	National test protocol unavailable. Use MEPDG default values.
	Unit Weight	X		AASHTO T 166 (cores)
	Asphalt content	X		AASHTO T 164 (cores)
	Gradation	X		AASHTO T 27 (cores or blocks)
	Air voids	X		AASHTO T 209 (cores)
	Asphalt recovery	X		AASHTO T 164/T 170/T 319 (cores)
Asphalt (new, overlay, and existing mixtures)	Asphalt Performance Grade (PG), OR	X		AASHTO T 315
	Asphalt binder complex shear modulus (G*) and phase angle (δ), OR	X		AASHTO T 49
	Penetration, OR	X		AASHTO T 53
	Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity, OR	X		AASHTO T 202 AASHTO T 201 AASHTO T 228
	Brookfield Viscosity	X		AASHTO T 316
NOTE : The global calibration factors included in version 1.0 of the MEPDG software for HMA pavements were determined using the NCHRP 1-37A viscosity based predictive model for dynamic modulus (E_{HMA}*).				

If a library of HMA test data has been established, the user could select the test results from previous HMA mixtures most similar to the one being used or use an average of the results from other similar mixtures. The following summarizes the recommended input parameters and values for the HMA mixtures. Refer to the NJDOT HMA inputs Excel Spreadsheet.

• Aggregate gradation; For new HMA mixtures, use values that are near the midrange of the project specifications or use average values from previous construction records for a particular type of mix. For existing HMA layers, use the average value recovered from as built construction records, or if construction records are unavailable, measure the gradation from the aggregates recovered from cores or blocks of the HMA (refer to Section 10).

• Air voids, effective asphalt content by volume, density, voids in mineral aggregate (VMA), voids filled with asphalt (VFA); For new HMA mixtures, use values that are near the mid-range of the project specification or use average values from previous construction records for a particular type of HMA mixture. More detail is provided in the latter part of this subsection for determining the volumetric properties for new HMA mixtures. For existing HMA layers, measure the air voids from cores recovered from the project. The other volumetric properties may be calculated from the in-place air voids and volumetric properties recovered from as built construction records (refer to Section 10). If construction records are unavailable, measure the effective asphalt content, VMA, and VFA from the cores or blocks taken from the project.

• Poisson's ratio; For new HMA mixtures, use the temperature calculated values within the MEPDG. In other words, check the box to use the predictive model to calculate Poisson's ratio from the pavement temperatures. For existing, age-hardened HMA mixtures, use the default values recommended in the MEPDG (refer to Table 21).

• Dynamic modulus, creep compliance, indirect tensile strength; For new HMA mixtures, input levels 2 or 3 could be used, unless the agency has a library of test results. Material properties needed for input levels 2 and 3 include gradation, asphalt PG classification, and test results from the dynamic shear rheometer (DSR; AASHTO T 315). The MEPDG software provides the user with two options for estimating the dynamic modulus; one listed as NCHRP 1-37A viscosity based model and the other listed as NCHRP 1-40D G* (dynamic shear modulus of the asphalt) based model. The global calibration factors for all HMA predictive equations (refer to Subsection 5.2) were determined using the NCHRP 1-37A viscosity based model. The option selected depends on the historical data available to the designer. For existing HMA layers, use input levels 2 or 3 and the backcalculated values from the FWD deflection basins for estimating the dynamic modulus. The creep compliance and indirect tensile strength are not needed for the existing HMA layers.

• Surface shortwave absorptivity; Use default value set in MEPDG, 0.85.

• Coefficient of thermal contraction of the mix; Use default values set in MEPDG for different mixtures and aggregates.

• Reference temperature; 70 °F should be used.

• Thermal conductivity of asphalt; Use default value set in program, 0.67 BTU/frft-°F.

• Heat capacity of asphalt; Use default value set in program, 0.23 BTU/lb-°F.

Although input level 1 is the preferred category of inputs for pavement design, many agencies have yet to acquire the testing capabilities to characterize HMA mixtures. Thus, input levels 2 and 3 are summarized in Table 21. For most analyses, it is permissible for designers to use a combination of level 1, 2, and 3 material inputs that are based on their unique needs and testing capabilities. The following provides more detailed discussion on determining the volumetric properties that may be used to estimate these input parameters for new HMA mixtures.

• Air Voids (AASHTO T 269), V_a: The air voids at construction need to represent the average in-place air voids expected after the HMA has been compacted with the rollers, but prior to opening the roadway to truck traffic. This value will be unavailable during structural design because it has yet to be produced. It is recommended that this value be obtained from previous construction records for similar mixtures or the designer could enter the target value from the project specifications.

• Bulk Specific Gravity of the Combined Aggregate Blend (AASHTO T 84 and T 85), G_{sb}: This value is dependent on the type of aggregates used in the HMA and gradation. Most agencies will have an expected range of this value from previous mixture designs for the type of aggregates used, their source, and combined gradation (type of mixture dependent) specified for the project.

• Maximum Specific Gravity of Mixture (AASHTO T 209), G_{mm}: This value is dependent on the type of aggregate, gradation, and asphalt content used in the HMA. Most agencies will have an expected range of this value from previous mixture designs using the aggregate source and gradation (type of mixture) specified for the project. The maximum specific gravity can be calculated from the component properties, if no historical information exists for the HMA mixture specified for the project.

• Voids in Mineral Aggregate, VMA: VMA is an input to the MEPDG for thermal cracking predictions and determination of other volumetric properties. The mixture VMA needs to represent the condition of the mixture after it has been compacted with the rollers, but prior to opening the roadway to truck traffic. This value will be unavailable during structural design because it has yet to be produced and placed. It is recommended that the value be calculated from other volumetric properties that may be obtained from construction records for similar type mixtures, aggregate sources, and gradations.

• Effective Asphalt Content by Volume, V_{be}: The effective asphalt content by volume needs to represent the in-place asphalt content; after the mix has been placed by the paver. This value will be unavailable during structural design because it has yet to be produced. It is recommended that the value be calculated from the other volumetric properties, as shown below.

Table 21. Recommended Input Parameters and Values; Limited or No Testing Capabilities for HMA (Input Levels 2 or 3)

Measured Property	Input Levels 2 or 3
Dynamic modulus, E_{HMA} (new HMA layers)	No dynamic modulus, E_{HMA}, laboratory testing required: • Use E_{HMA} predictive equation; either the NCHRP 1-37A viscosity based model or 1-40D G* based model. Both predictive equations are included in the software help screens. Inputs are gradation, bitumen viscosity or dynamic shear modulus and phase angle, loading frequency, air void content, and effective bitumen content by volume. Input variables may be obtained through testing of laboratory prepared mixture and asphalt samples or from agency historical records. • Use default *A-VTS-* values included in the software based on asphalt binder grade (PG, viscosity, or penetration grades), as shown below. $LogLog\eta = A + VTS(LogT_R)$ Where: η=Viscosity, cP; T_R=Temperature, Rankine; and *A* & *VTS* are the intercept and slope resulting from a regression of the asphalt viscosity-temperature susceptibility relationship, respectively.
Dynamic modulus, E_{HMA} (existing HMA layers)	No dynamic modulus, E_{HMA}, laboratory testing required: • Use E_{HMA} predictive equation, as noted above. Inputs are gradation, bitumen viscosity or dynamic shear modulus and phase angle, loading frequency, air void content, and effective bitumen content by volume. Input variables may be obtained through testing of cores and asphalt extracted from field samples, or from agency historical records. • Use default *A-VTS-* values based on age-hardened asphalt binder grade (PG, or viscosity, or penetration grades). • Determine existing pavement condition rating (excellent, good, fair, poor, very poor); calculate the modulus from deflection basins.
Tensile strength, *TS* (new HMA surface; not required for existing HMA layers)	Use MEPDG regression equation: $TS(psi) = 7416.712 - 114.016 * Va - 0.304 * Va^2 - 122.592 * VFA + 0.704 * VFA^2 + 405.71 * Log10(Pen77) - 2039.296 * log10(A)$ Where: *TS* = Indirect tensile strength at 14 °F, psi. *Va* = HMA air voids, as-constructed, percent *VFA* = Voids filled with asphalt, as-constructed, percent. *Pen77* = Asphalt penetration at 77 °F, mm/10. *A* = Asphalt viscosity-temperature susceptibility intercept. Input variables may be obtained through testing of lab prepared mix samples, extracted cores (for existing pavements), or from agency historical records.
Creep compliance, *D(t)* (new HMA surface; not required for existing HMA layers)	Use MEPDG regression equation: $D(t) = D_1 * t^m$ $\log(D_1) = -8.524 + 0.01306 * T + 0.7957 * \log10(Va) + 2.0103 * \log10(VFA) - 1.923 * \log10(A)$ $m = 1.1628 - 0.00185 * T - 0.04596 * Va - 0.01126 * VFA + 0.00247 * Pen77 + 0.001683 * T * Pen77^{0.4605}$ Where: *t* = Time, months. *T* = Temperature at which creep compliance is measured, °F. *Va* = HMA air voids, as-constructed, %. *VFA* = Voids filled with asphalt, as-constructed, %. *Pen77* = Asphalt penetration at 77 °F, mm/10. Input variables may be obtained through testing of lab prepared mix samples, extracted cores (for existing pavements), or from agency historical records.
*Note that the MEPDG computes input Levels 2 and 3 dynamic modulus, tensile strength, creep compliance, etc. internally once; all the required input variables required by the various equation are provided.	

Table 21 continued on next page.

<table>
<tr><th>Measured Property</th><th>Recommended Level 3 Input</th></tr>
<tr><td>Air voids</td><td>Use as-constructed mix type specific values available from previous construction records.</td></tr>
<tr><td>Volumetric asphalt content</td><td>Use as-constructed mix type specific values available from previous construction records.</td></tr>
<tr><td>Total unit weight</td><td>Use as-constructed mix type specific values available from previous construction records.</td></tr>
<tr><td>Poisson's ratio</td><td>Use predictive equation based on temperature included in the MEPDG for new HMA mixes; For existing, age-hardened HMA layers, use the typical values listed below:
<table>
<tr><th>Reference Temperature °F</th><th>Dense-Graded HMA (Level 3)
$\mu_{typical}$</th><th>Open-Graded HMA (Level 3)
$\mu_{typical}$</th></tr>
<tr><td>< 0 °F</td><td>0.15</td><td>0.35</td></tr>
<tr><td>0 – 40 °F</td><td>0.20</td><td>0.35</td></tr>
<tr><td>41 – 70 °F</td><td>0.25</td><td>0.40</td></tr>
<tr><td>71 – 100 °F</td><td>0.35</td><td>0.40</td></tr>
<tr><td>101 – 130 °F</td><td>0.45</td><td>0.45</td></tr>
<tr><td>> 130 °F</td><td>0.48</td><td>0.45</td></tr>
</table></td></tr>
<tr><td>Surface shortwave absorptivity</td><td>Use MEPDG default of 0.85, which was used in the global calibration process (refer to Table 20).</td></tr>
<tr><td>Thermal conductivity</td><td>Typical values for HMA range from 0.44 to 0.81 Btu/(ft)(hr)(°F). Use default value set in program—0.67 Btu/(ft)(hr)(°F).</td></tr>
<tr><td>Heat capacity</td><td>Typical values for HMA range from 0.22 to 0.40 Btu/(lb)(°F). Use default value set in program—0.23 BTU/lb.-F</td></tr>
<tr><td>Coefficient of thermal contraction</td><td>Use MEPDG predictive equation shown below:

$$L_{MIX} = \frac{VMA * B_{ac} + V_{AGG} * B_{AGG}}{3 * V_{TOTAL}}$$

Where:
L_{MIX} = Linear coefficient of thermal contraction of the asphalt concrete mixture (1/°C).
B_{ac} = Volumetric coefficient of thermal contraction of the asphalt cement in the solid state (1/°C).
B_{AGG} = Volumetric coefficient of thermal contraction of the aggregate (1/°C)
VMA = Percent volume of voids in the mineral aggregate (equals percent volume of air voids plus percent volume of asphalt cement minus percent volume of absorbed asphalt cement).
V_{AGG} = Percent volume of aggregate in the mixture.
V_{TOTAL} = 100 percent.

Typical values for linear coefficient of thermal contraction, volumetric coefficient of thermal contraction of the asphalt cement in the solid state, and volumetric coefficient of thermal contraction of aggregates measured in various research studies are as follows:
• L_{MIX} = 2.2 to 3.4*10^{-5} /°C (linear).
• B_{ac} = 3.5 to 4.3*10^{-4} /°C (cubic).
• B_{AGG} = 21 to 37*10^{-6} /°C (cubic).</td></tr>
<tr><td colspan="2">*Note that the MEPDG computes input Level 2 and 3 coefficient of thermal extraction, etc. internally; once all the required equation input variables are available.</td></tr>
</table>

11.5 Unbound Aggregate Base Materials and Engineered Embankments

Similar to HMA and PCC, physical and engineering properties are required for the unbound pavement layers and foundation. The physical properties include dry density, moisture content, and classification properties, while the engineering property includes the resilient modulus. These properties and physical condition of the layers need to be representative of the layers when the pavement is opened to truck traffic.

For new alignments or new designs, the default resilient modulus values included in the MEPDG (input level 3) may be used, the modulus may be estimated from other properties of the material (input level 2), or measured in the laboratory (input level 1). For rehabilitation or reconstruction designs, the resilient modulus of each unbound layer and embankment may be backcalculated from deflection basin data or estimated from DCP or CBR tests. If the resilient modulus values are determined by backcalculating elastic layer modulus values from deflection basin tests, those values need to be adjusted to laboratory conditions. The adjustment ratios that need to be applied to the unbound layers for use in design are provided in FHWA design pamphlets FHWA-RD-97-076 and FHWA-RD-97-083 (Von Quintus and Killingsworth, 1997-a and b). Table 26 lists the values recommended in those design pamphlets. If the resilient modulus values are estimated from the DCP or other tests, those values may be used as inputs to the MEPDG, but should be checked based on local material correlations and adjusted to laboratory conditions, if necessary. The DCP test should be performed in accordance with ASTM D 6951 or an equivalent procedure.

Table 26. C-Values to Convert the Calculated Layer Modulus Values to an Equivalent Resilient Modulus Measured in the Laboratory

Layer Type	Location	C-Value or M_r/E_{FWD} Ratio
Aggregate Base/Subbase	Between a Stabilized & HMA Layer	1.43
	Below a PCC Layer	1.32
	Below an HMA Layer	0.62
Subgrade-Embankment	Below a Stabilized Subgrade/Embankment	0.75
	Below an HMA or PCC Layer	0.52
	Below an Unbound Aggregate Base	0.35

Table 27 summarizes the input level 1 parameters required for the unbound aggregate base, subbase, embankment, and subgrade soil material types listed in Table 19. The recommended test protocols are also listed in Table 27. Although input level 1 is preferred for pavement design, most agencies are not equipped with the testing facilities required to characterize the paving materials. Thus, for the more likely situation where agencies have only limited or no testing capability for characterizing unbound aggregate base, subbase, embankment, and subgrade soil materials, input levels 2 and 3 are recommended, which are provided in Table 28. For most analyses, designers will use a combination of level 1, 2, and 3 material inputs based on their unique needs and testing capabilities, which is permissible.

The following summarizes the recommended input parameters and values for the unbound layers and foundation:

• Gradation – For new materials, the mid-range of the material specifications or the average gradation from previous construction records for similar materials is recommended for use as the input values. For existing pavement layers, use the average gradation from as built construction records. If those records are unavailable, use average results from laboratory tests performed on materials recovered during the field investigation. The gradation of the unbound aggregate or embankment soil could be measured in accordance with AASHTO T 88. If sufficient material was not recovered during the field investigation, the default values included in the MEPDG for the material classification could be used.

• Atterberg Limits –For new materials, the mid-range allowed by the material specifications or the average liquid limit and plasticity index from previous construction records for similar materials is recommended for use as the input values. For existing pavement layers, use the average results from the Atterberg limits test for similar materials that were placed using the same material specifications. The liquid limit could be measured in accordance with AASHTO T 89, and the plastic limit and plasticity index determined in accordance with AASHTO T 90. If sufficient material was not recovered during the field investigation, the default values included in the MEPDG for the material classification could be used.

• Dry Density – For new materials, the maximum dry density defined by the material specifications using the compaction effort specified for the project, or the average dry density measured on previous construction projects for similar material is recommended for use as the input value. For existing pavement layers that will remain in-place for the rehabilitation, use the average dry density from as-built construction records or the average value measured during the field investigation. The MEPDG default values for dry density represent the median maximum dry unit weight for specific material classifications. These default values need not be used for existing pavement layers that remain in-place for rehabilitation without confirming those values during the field investigation.

• Moisture Content – For new materials, the optimum moisture content using the compaction effort specified for the project, or the average moisture content measured on previous construction projects for a similar material is recommended for use as the input value. For existing pavement layers that will remain in-place for the rehabilitation, use the average moisture content measured during the field investigation. The MEPDG default values for moisture content represent the median optimum moisture content for specific material classifications. These default values need not be used for existing layers remaining in-place without confirming those values during the field investigation.

• Poisson's Ratio – Use the default values provided in the MEPDG, unless the designer has test data for using different values.

• Resilient Modulus – For new materials, use input levels 2 or 3, unless the agency has a library of test results. Material properties needed for input levels 2 and 3 include gradation, classification, Atterberg limits, moisture content, and dry density. The resilient modulus for the unbound layers and foundation may also be estimated from the CBR test (AASHTO T 193) or the R-Value test (AASHTO T 190).

If resilient modulus tests are available in a library of materials information and data, the designer could use the average value for the in-place material. The resilient modulus may be estimated based on equivalent stress states using the procedure outlined in the FHWA Design Pamphlets noted above (Von Quintus and Killingsworth, 1997-a and b). If input level 3 is used to estimate the resilient modulus from classification tests, these modulus values represent the optimum moisture content and dry density (refer to Table 28). Those default values will need to be adjusted if the in-place layer deviates from the optimum moisture content and maximum dry unit weight, as defined by AASHTO T-180 at the time of construction. Adjustments for lower or higher moisture contents and dry densities can be made using the regression equations derived from the LTPP resilient modulus test results (Von Quintus and Yau, 2001).

Table 27. Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Material Requirements and Test Protocols for New and Existing Materials

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New (lab samples) and existing (extracted materials)	Two Options: Regression coefficients k_1, k_2, k_3 for the generalized constitutive model that defines resilient modulus as a function of stress state and regressed from laboratory resilient modulus tests. Determine the average design resilient modulus for the expected in-place stress state from laboratory resilient modulus tests.	X		AASHTO T 307 or NCHRP 1-28A The generalized model used in M-E PDG design procedure is as follows: $M_r = k_1 p_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$ where M_r = resilient modulus, psi θ = bulk stress $= \sigma_1 + \sigma_2 + \sigma_3$ σ_1 = major principal stress. σ_2 = intermediate principal stress σ_3 = minor principal stress confining pressure τ_{oct} = octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}$ P_a = normalizing stress K_1, k_2, k_3 = regression constants
	Poisson's ratio		X	No national test standard, use default values included in the MEPDG.
	Maximum dry density	X		AASHTO T 180
	Optimum moisture content	X		AASHTO T 180
	Specific gravity	X		AASHTO T 100
	Saturated hydraulic conductivity	X		AASHTO T 215
	Soil water characteristic curve parameters	X		Pressure plate (AASHTO T 99) OR Filter paper (AASHTO T 180) OR Tempe cell (AASHTO T 100)
Existing material to be left in place	FWD backcalculated modulus	X		AASHTO T 256 and ASTM D 5858
	Poisson's ratio		X	No national test standard, use default values included in the MEPDG.

Table 28. Recommended Levels 2 and 3 Input Parameters and Values for Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Material Properties

<table>
<tr><th>Required Input</th><th>Recommended Input Level</th></tr>
<tr><td rowspan="2">Resilient modulus</td><td>Use level 3 inputs based on the unbound aggregate base, subbase, embankment, and subgrade soil material AASHTO Soil Classification. AASHTO Soil Class is determined using material gradation, plasticity index, and liquid limit.
<table>
<tr><th rowspan="2">AASHTO Soil Classification</th><th colspan="3">Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi</th></tr>
<tr><th>Base/Subbase for Flexible and Rigid Pavements</th><th>Embankment & Subgrade for Flexible Pavements</th><th>Embankment & Subgrade for Rigid Pavements</th></tr>
<tr><td>A-1-a</td><td>40,000</td><td>29,500</td><td>18,000</td></tr>
<tr><td>A-1-b</td><td>38,000</td><td>26,500</td><td>18,000</td></tr>
<tr><td>A-2-4</td><td>32,000</td><td>24,500</td><td>16,500</td></tr>
<tr><td>A-2-5</td><td>28,000</td><td>21,500</td><td>16,000</td></tr>
<tr><td>A-2-6</td><td>26,000</td><td>21,000</td><td>16,000</td></tr>
<tr><td>A-2-7</td><td>24,000</td><td>20,500</td><td>16,000</td></tr>
<tr><td>A-3</td><td>29,000</td><td>16,500</td><td>16,000</td></tr>
<tr><td>A-4</td><td>24,000</td><td>16,500</td><td>15,000</td></tr>
<tr><td>A-5</td><td>20,000</td><td>15,500</td><td>8,000</td></tr>
<tr><td>A-6</td><td>17,000</td><td>14,500</td><td>14,000</td></tr>
<tr><td>A-7-5</td><td>12,000</td><td>13,000</td><td>10,000</td></tr>
<tr><td>A-7-6</td><td>8,000</td><td>11,500</td><td>13,000</td></tr>
</table>
</td></tr>
<tr><td>NOTES:
1. The resilient modulus is converted to a k-value internally within the software for evaluating rigid pavements.
2. The resilient modulus values at the time of construction for the same AASHTO soil classification are different under flexible and rigid pavements because the stress-state under these pavements is different. Soils are stress dependent and the resilient modulus will change with changing stress-state (refer to Table 27). The default values included in the MEPDG software were estimated as the median value from the test sections included in the LTPP database and used engineering judgment. These default values can be sued assuming the soils are at the maximum dry density and optimum water content as defined from AASHTO T 180.</td></tr>
<tr><td>Maximum dry density</td><td>Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Optimum moisture content</td><td>Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Specific gravity</td><td>Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Saturated hydraulic conductivity</td><td>Select based on the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Soil water characteristic curve parameters</td><td>Select based on aggregate/subgrade material class.</td></tr>
</table>

For existing unbound layers, use backcalculated modulus values from the FWD deflection basins for estimating the resilient modulus. As noted above, the backcalculated elastic modulus values need to be adjusted to laboratory conditions as input to the MEPDG. However, results from DCP tests on the in-place materials may be used when FWD deflection basin tests have not been performed or were found to be highly variable with large errors to the measured deflection basins.

Saturated Hydraulic Conductivity – For new and existing unbound layers, AASHTO T 215 may be used to measure this input parameter. However, all calibration work completed for version 1.0 of the software was completed using the default values included in the MEPDG software. Use of these default values is recommended.

Soil Water Characteristics Curve Parameters – For new and existing unbound layers, there are AASHTO test standards that may be used to measure these input parameters for predicting the change in moisture content of the unbound layers over time. However, all calibration work completed for version 1.0 was completed using the default values included in the MEPDG software. Use of these default values is recommended.

10 Pavement Evaluation for Rehabilitation Design

Rehabilitation design requires an evaluation of the existing pavement to provide key information. The MEPDG provides detailed and specific guidance for conducting a pavement evaluation program and taking the results from that program to establish inputs to the MEPDG software. The National Highway Institute (NHI) courses on pavement evaluation provide tools that may be followed in planning and executing a pavement evaluation program for rehabilitation design (APT, Inc. 2001.a and b).

It is important to note that the MEPDG inputs of existing pavement layers for overlay design are similar to those required for new or reconstructed pavements except that the values may be different due to load and climate caused deterioration of the existing layers and materials. Determining the extent of damage and material properties of the in-place layers is the most critical challenge in pavement evaluation. This section provides a brief summary of the overall pavement evaluation process followed by guidelines to obtain inputs to the MEPDG for use in rehabilitation design. The test protocols for measuring the material properties are listed in Section 11.

10.1 Overall Condition Assessment and Problem Definition Categories

The first step in the pavement rehabilitation design process involves assessing the overall condition of the existing pavement and fully defining the existing pavement problems. To avoid making an inaccurate assessment of the problem, the engineer needs to collect and evaluate sufficient information about the pavement. High-speed nondestructive testing data, such as GPR and profile testing, could be considered to assist in making decisions related to timing of the improvement and additional data collection effort needed. Overall pavement condition and problem definition could be determined by evaluating the following eight major categories of the existing pavement:

1. Structural adequacy (load related).
2. Functional adequacy (user related).
3. Subsurface drainage adequacy.
4. Material durability.
5. Shoulder condition.
6. Extent of maintenance activities performed in the past.
7. Variation of pavement condition or performance within a project.
8. Miscellaneous constraints (e.g., bridge and lateral clearance and traffic control restrictions).

The structural and material durability categories relate to those properties and features that define the response of the pavement to traffic loads. This data is used in the MEPDG for rehabilitation alternatives. The functional category relates to the surface and subsurface properties that define the smoothness of the roadway, and to those surface characteristics that define the frictional resistance or other safety characteristics of the pavement's surface. Subsurface drainage and material durability may affect both structural and functional condition. Shoulder condition is important in terms of rehabilitation type selection and in affecting project construction cost.

Variation within a project refers to areas where there is a significant difference in pavement condition. Such variation may occur along the length of the project, between lanes (truck lane versus other lanes), among cut and fill portions of the roadway, and at bridge approaches, interchanges, or intersections. Miscellaneous factors, such as joint condition for jointed concrete pavements and HMA reflection cracking for composite pavements, are important to the overall condition of such pavements but only need to be evaluated where relevant.

Table 11 below contains a comprehensive checklist of factors designed to identify the problems that need to be addressed during rehabilitation design. The following provides some guidance on the amount of work or extensiveness of the pavement evaluation plan for determining the input values related to the condition of the existing pavement layers (e.g., if the pavement has over 50 percent high severity load-related cracks, trying to accurately estimate the modulus and volumetric properties of the existing HMA layer is not cost effective for selecting and designing rehabilitation strategies).

• If the pavement has significant and extensive levels of distress that exceed the user's failure criteria or threshold values, extensive field and laboratory testing to characterize the pavement surface layers becomes less important. The condition of the existing pavement may be determined from results of the visual distress surveys.

• If the pavement has exhibited no structural distress, field and laboratory testing become important to determine the condition of the existing pavement layers. For this case, results from the field (deflection basin and DCP tests) and laboratory tests could be used to determine the condition of the existing layers.

• If the pavement has marginal levels of distress, the results from the visual distress survey may be used to determine the location and frequency of the field tests and cores. In this case, both assessments become equally important. The remainder of this section provides a summary of those pavement evaluation activities to determine the existing pavement condition for rehabilitation design with the MEPDG.

10.2 Data Collection to Define Condition Assessment

This subsection summarizes the steps and activities to complete a detailed assessment on the condition of the existing pavement for selecting a proper rehabilitation strategy, as shown in Figure 27. All steps to complete a detailed assessment of the pavement and individual layers are not always needed. Table 12 lists the input levels associated with setting up and conducting a pavement evaluation plan in support of the MEPDG.

Table 11. Checklist of Factors for Overall Pavement Condition Assessment and Problem Definition.

Facet	Factors	Description
Structural Adequacy	Existing Distress	1. Little or no load/fatigue-related distress 2. Moderate load/fatigue-related distress (possible deficiency in load-carrying capacity) 3. Major load/fatigue-related distress (obvious deficiency in current load-carrying capacity) 4. Load-carrying capacity deficiency: (yes or no)
	Nondestructive testing (FWD deflection testing)	1. High deflections or weak layers: (yes or no) 2. Are backcalculated layer moduli reasonable? 3. Are joint load transfer efficiencies reasonable?
	Nondestructive testing (GPR testing)	1. Determine layer thickness 2. Are voids located beneath PCC pavements?
	Nondestructive testing (profile testing)	Determine joint/crack faulting
	Destructive testing	1. Are core strengths & condition reasonable? 2. Are the layer thicknesses adequate?
	Previous maintenance performed	Minor | Normal | Major
	Has lack of maintenance contributed to structural deterioration?	Yes____ No____ Describe ________________________
Functional Adequacy	Smoothness:	Measurement ________________________________
		Very Good | Good | Fair | Poor | Very Poor
	Cause of smoothness deficiency:	Foundation movement Localized distress or deterioration Other
	Noise	Measurement __________________________________
		Satisfactory | Questionable | Unsatisfactory
	Friction resistance	Measurement _________________________________
		Satisfactory | Questionable | Unsatisfactory
Subsurface Drainage	Climate (moisture and temperature region)	Moisture throughout the year: • Seasonal moisture or high water table • Very little moisture • Deep frost penetration • Freeze-thaw cycles • No frost problems
	Presence of moisture-accelerated distress	Yes | Possible | No
	Subsurface drainage facilities	Satisfactory | Marginal | Unsatisfactory
	Surface drainage facilities	Satisfactory | Marginal | Unsatisfactory
	Has lack of maintenance contributed to deterioration of drainage facilities?	Yes | No Describe: ____________________________________

Table 11 continued on the next page.

<table>
<tr><th>Facet</th><th>Factors</th><th colspan="3">Description</th></tr>
<tr><td rowspan="3">Materials Durability</td><td>Presence of durability-related distress (surface layer)</td><td colspan="3">1. Little to not durability-related distress.
2. Moderate durability-related distress
3. Major durability-related distress</td></tr>
<tr><td>Base erosion or stripping</td><td colspan="3">1. Little or no base erosion or stripping
2. Moderate base erosion or stripping
3. Major base erosion or stripping</td></tr>
<tr><td>Nondestructive testing (GPR testing)</td><td colspan="3">Determine areas with material deterioration/moisture damage (stripping)</td></tr>
<tr><td rowspan="2">Shoulder Adequacy</td><td>Surface condition</td><td colspan="3">1. Little or not load-associated/joint distress
2. Moderate load-associated/joint distress
3. Major load-associated/joint distress
4. Structural load-carrying capacity deficiency: (yes or no)</td></tr>
<tr><td>Localized deteriorated areas</td><td>Yes</td><td>No</td><td>Location:</td></tr>
<tr><td rowspan="3">Condition-Performance Variability</td><td>Does the project section include significant deterioration of the following:
• Bridge approaches
• Intersections
• Lane to lane
• Cuts and fills</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Is there a systematic variation in pavement condition along project (localized variation)?</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Systematic lane to lane variation in pavement condition</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Miscellaneous</td><td>PCC joint damage:
• Is there adequate load transfer (transverse joints)?
• Is there adequate load transfer (centerline joint)?
• Is there excessive centerline joint width?
• Is there adequate load transfer (lane-shoulder)?
• Is there joint seal damage?
• Is there excessive joint spalling (transverse)?
• Is there excessive joint spalling (longitudinal)?
• Has there been any blowups?</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td rowspan="7">Constraints</td><td>Are detours available for rehabilitation construction?</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Should construction be accomplished under traffic</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Can construction be done during off-peak hours</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Bridge clearance problems?</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Lateral obstruction problems</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Utility problems/issues</td><td>Yes</td><td colspan="2">No</td></tr>
<tr><td>Other constraint problems</td><td>Yes</td><td colspan="2">No</td></tr>
</table>

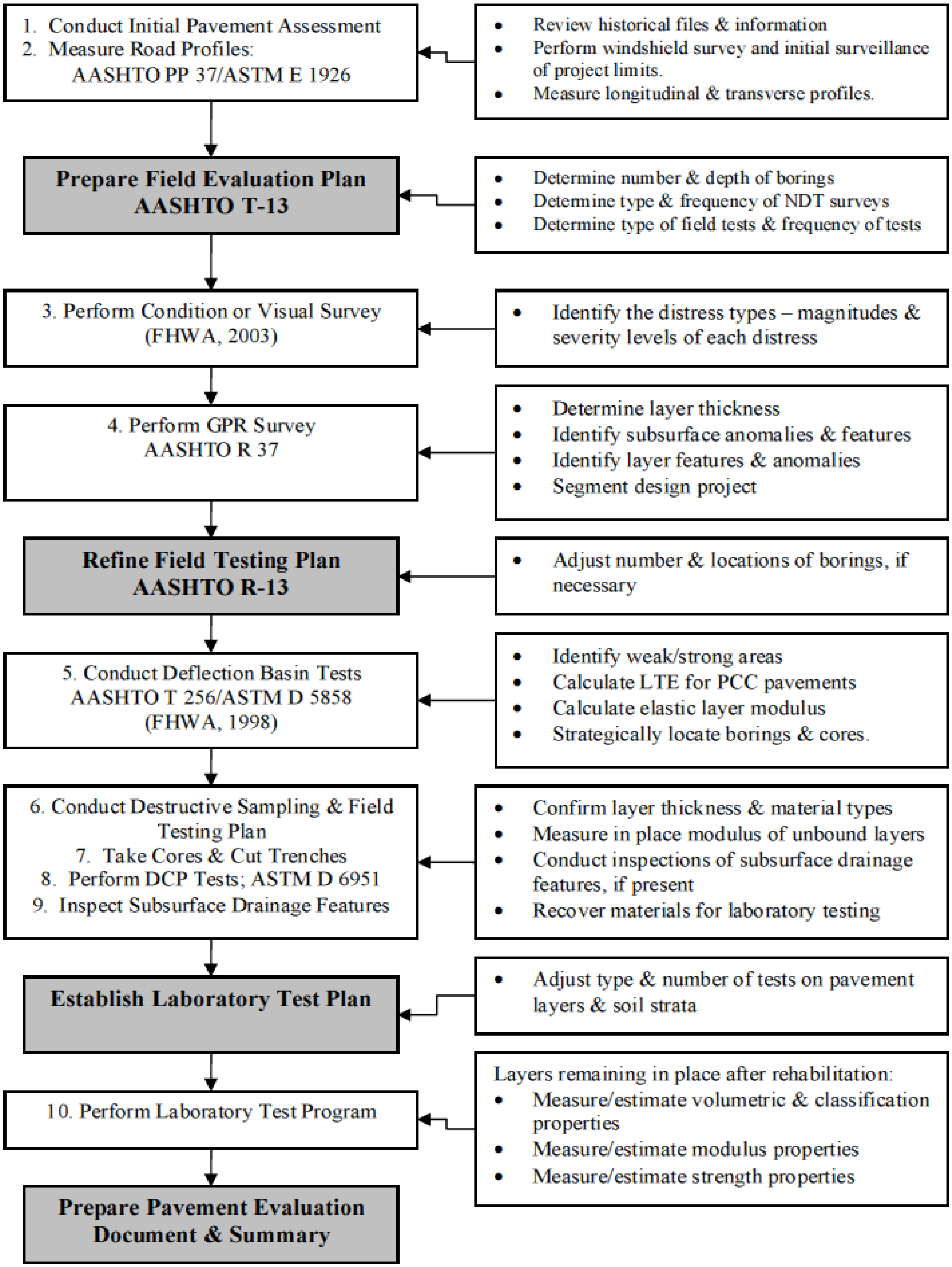


Figure 27. Steps and Activities for Assessing Condition of Existing Pavements for Rehabilitation Design (Refer to Table 12)

Table 12. Hierarchical Input Levels for a Pavement Evaluation Program to Determine Inputs for Existing Pavement Layers for Rehabilitation Design Using the MEPDG

Assessment Activity	Input Level for Pavement Rehabilitation Design			Purpose of Activity
	1	2	3	
1. Initial Assessment; Review files & historical information, conduct windshield survey.	Yes	Yes	Yes	Estimate the overall structural adequacy & materials durability of existing pavement, segment project into similar condition of: • Existing layers • Shoulders, if present • Drainage features (surface & subsurface) • Identify potential rehabilitation strategies
2. Surface Feature Surveys; Measure profile, noise, & friction of existing surface.	Yes, Only Profile	Yes, Only Profile	No	Determine functional adequacy of surface; Profile, friction & noise surveys are only needed to determine if rehabilitation is needed, because the surface will usually be replaced or modified. Profile surveys are used to select a proper rehabilitation strategy – milling depth or diamond grinding, leveling course thickness, or none needed; estimate the initial IRI value after HMA overlay; and CPR appropriateness.
3. Detailed Condition Survey; Determine type, amount, & severity of existing distresses	Yes	Yes	No	Estimate structural adequacy or remaining life & materials durability of existing pavement layers and to select a rehabilitation strategy. • Distortion; faulting of PCC and rutting in HMA • Cracking; non-load related cracks versus fatigue cracks • Material disintegration distresses (raveling, D-cracking, etc.) • Define/segment areas with different distresses
4. GPR Survey; Estimate layer thickness, locate subsurface anomalies & features	Yes	No	No	Determine structural adequacy, subsurface features & anomalies, and materials durability of existing pavement layers: • Estimate layer thickness • Identify potential subsurface anomalies • Locate voids beneath pavement surface • Locate HMA layers with stripping
5. Deflection Basin Tests; Measure load-response of pavement structure & foundation	Yes	Yes	No	Determine structural adequacy and in-place modulus of existing pavement layers and foundation. • Calculate LTE of cracks & joints in PCC pavements • Calculate layer modulus • Locate borings and cores for destructive tests Level 2 – Uniform spacing of deflection basin tests in areas with different distresses. Level 1 – Clustered spacing of deflection basin tests in areas with different distresses along entire project.

Table 12 continued on next page.

Assessment Activity	Input Level for Pavement Rehabilitation Design			Purpose of Activity
	1	2	3	
6. Destructive Sampling; Drill cores & boring to recover materials for visual observation & lab testing	Yes	Yes	Yes	Determine structural adequacy & materials durability. • Visual classification of materials & soils • Confirm layer thickness and material types • Identify/confirm subsurface anomalies – HMA stripping, voids, etc. • Determine depth to rigid layer or bedrock • Determine water table depth • Identify seams with lateral water flow Level 3 – Limited borings in areas identified from the initial pavement assessment activity. Levels 1 & 2 – Boring & cores drilled in each segment identified from the condition survey, deflection basin tests and GPR survey.
7. Field Inspections; Cores & trenches in distressed areas	Yes	No	No	Structural adequacy & rehabilitation strategy selection: • Determine the rutting in each paving layer from the excavated trenches. • Determine where cracking initiated & the direction of crack propagation.
8. Field Tests; DCP tests of unbound layers	Yes	No	No	Determine structural adequacy – estimate the in-place modulus from DCP tests performed on the unbound layer through the core locations.
9. Field Inspections; Subsurface drainage features	Yes	No	No	Subsurface drainage adequacy – Inspecting drainage features with mini-cameras to check condition of & ensure positive drainage of edge drains.
10. Laboratory Tests; Unbound materials & soils, HMA mixtures, & PCC mixtures	Yes	Yes	No	Layers which will remain in place after rehabilitation: Classification tests (gradation & Atterberg limits tests) Unit weight & moisture content tests Coefficient of thermal expansion - PCC Strength tests – PCC & HMA layers Modulus tests – PCC layers only Level 3 – All inputs based on defaults & visual classification of materials & soils; no laboratory tests are performed on layers that will remain in place. Level 2 – Modulus estimated from DCP and deflection basin tests for unbound layers & volumetric properties for bound layers. Level 1 – Laboratory tests listed above

10.2.1 Initial Pavement Assessment

Regardless of the input level adopted for the pavement evaluation, the condition assessment needs to begin with an assembly of historic data. This information may be obtained from a windshield field survey of the entire project followed by a detailed survey of selected areas of the project. The following activities should be performed to assist in preparing the field evaluation plan.

• Review historical records for the roadway segment planned for rehabilitation. The information needed includes the original pavement construction month and year (a required input to the MEPDG), and any preventive maintenance, pavement preservation, or repair activities that have been applied to the roadway segment. The preventive maintenance, pavement preservation, and repair activities are only needed to assist the designer in establishing the condition of the existing pavement and help explain performance anomalies.

• Review construction files and results from previous borings and laboratory results, if available. The Soil Conservation Service Series maps may also be used to ensure that the different subsurface soils along the project are sampled and tested, if needed. These maps were identified and discussed in Section 9 on characterizing the foundation soils for new alignments.

• Review previous distress and profile surveys and pavement management records to establish performance trends and deterioration rates, if available.

• Review previous deflection surveys, if available.

• Perform a windshield survey or complete an initial surveillance of the roadway's surface, drainage features, and other related items. This initial survey may consist of photo logs, low-aerial photographs, and automated distress surveys.

• Segment the roadway or project into areas with similar layer thickness, surface distresses, subsurface features, and foundation soils. As part of the initial condition assessment or the more detailed condition survey (see subsection 10.2.3), longitudinal and transverse profiles may be measured and used to decide on the types of pre-overlay treatments that might be needed.

10.2.2 Prepare Field Evaluation Plan

The engineer needs to prepare an evaluation plan that outlines all activities needed for investigating and determining the causes of the pavement defects observed during the initial surveillance and for selecting and designing an appropriate repair strategy for those defects. The field evaluation plan could consist of a detailed condition survey, nondestructive testing, destructive sampling and testing, and traffic control, as a minimum. Table 13 may be used as an example in setting up the field evaluation plan.

10.2.3 Conduct Condition or Visual Survey

A key factor to determine the condition or strength of the existing pavement layers is the result from a detailed visual survey. Pavement visual surveys are performed to identify the types, magnitudes, and severities of distress. The visual survey needs to be performed on the pavement, shoulders and on any drainage feature along the project site. Automated distress surveys are adequate for rehabilitation design purposes, for most cases.

Table 14 provides a summary of the visual survey data needed for determining the inputs to the MEPDG software related to the condition of the existing pavement. For the MEPDG, distress identification for flexible, rigid, and composite pavements is based on the Distress Identification Manual for the LTPP program (FHWA, 2003). This LTPP manual was used to identify and measure the distresses for all pavement segments that were included in the global calibration process.

Table 13. Field Data Collection and Evaluation Plan

Step	Title	Description
1	Historic data collection	This step involves the collection of information such as location of the project, year constructed, year and type of major maintenance, pavement design features, materials and soils properties, traffic climate, conditions, and any available performance data.
2	First field survey	This step involves conducting a windshield and detailed distress survey of sampled areas within the project to assess the pavement condition. Data required includes distress information, drainage conditions, subjective smoothness, traffic control options, and safety considerations.
3	First data evaluation & the determination of additional data requirements	Determine critical levels of distress/smoothness and the causes of distress and smoothness loss using information collected during the first field survey. This list will aid in assessing preliminarily existing pavement condition and potential problems. Additional data needs will also be addressed during this step.
4	Second field survey	This step involves conducting detailed measuring and testing such as coring and sampling, profile (smoothness) measurement, skid resistance measurement, deflection testing, drainage tests, and measuring vertical clearances.
5	Laboratory testing of samples	This step involves conducting tests such as materials strength, resilient modulus permeability, moisture content, composition, density, and gradations, using samples obtained form the second field survey.
6	Second data evaluation	This involves the determination of existing pavement condition and an overall problem definition. Condition will be assessed and the overall problem defined by assessing the structural, functional, and subsurface drainage adequacy of the existing pavement. Condition assessment and overall problem definition also involve determining material durability, shoulder condition, variability in pavement condition along project, and potential constraints. Additional data requirements for designing rehabilitation alternatives will also be determined during this step.
7	Final field and office data compilation	Preparation of a final evaluation report.

Table 14. Guidelines for Obtaining Non-Materials Input Data for Pavement Rehabilitation

Existing Pavement Layer	Design Input	Measurements and Tests Required for Design Inputs
Flexible pavement	Alligator cracks (bottom-up) cracking plus previous repair of this distress	Level 1 & 2: Conduct visual survey along design lane of project and measure area of all severities of alligator fatigue cracking plus any previous repair of this cracking. Compute percent area affected (cracked and repair).
	Rutting of each layer in the existing pavement	Level 1: Measure from transverse trench data across the traffic lane. Level 2/3: Proportion the total surface rutting to each layer of the pavement and the subgrade. Utilize cores from the wheel path and non-wheel path to help estimate layer rutting.
	Pavement Rating	Level 3: Pavement Rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).
JPCP concrete slab	Cracked (transverse) slabs in design lane plus previous slab replacements	Conduct visual survey along design lane of project and identify slabs with transverse cracking (all severity levels) and slab replacements of transverse cracks. Compute percent slabs affected (cracked and replacements of cracked slabs).
	Joint load transfer (for reflection cracking prediction with HMA overlay)	Use as-built plans to determine if dowels are present and if so, their diameter and spacing. Alternatively, conduct FWD testing of joint to determine joint LTE. If dowels exist, rate joint Good LTE, if not, rate joint Poor LTE. Or, using LTE, rate joint Good LTE if measured LTE is > 60 % when testing @ a temperature < 80°F, or Poor LTE otherwise.
	Thickness of slab	Obtain representative cores and measure for thickness. Input mean thickness.
	Joint spacing & skew	Measure joint spacing & skew in the field. If random spacing, measure spacing pattern. If uniform spacing, enter mean spacing. If joints are skewed, add 2-ft to input joint spacing. Cracking is computed for the longest joint spacing but faulting and IRI for mean spacing.
	Shoulder type	Identify shoulder type (next to design lane), and if PCC determine whether or not it is tied to the traffic lane.
	Pavement Rating (Level 3)	Level 3: Pavement Rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).
CRCP concrete slab	Punchouts (and repairs of punchouts)	Conduct visual survey along design lane of project and identify number of punchouts at Medium and High levels of severity and full depth repairs of punchouts. Compute No. punchouts and repairs of punchouts per mile.
	Longitudinal reinforcement	Use as-built plans to determine bar size and spacing and depth from surface. Compute percent reinforcement of concrete area.
	Thickness of slab	Obtain representative cores (or other method) and measure thickness. Input mean thickness.
	Transverse cracking spacing	Conduct a visual survey along design lane of project and determine mean crack spacing. Include all severity levels of transverse cracks.
	Pavement Rating (Level 3)	Level 3: Pavement Rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).

Some agencies, however, may have to use condition survey data recorded in their pavement management database for establishing the condition of the existing pavements.

ASTM E 1778 **STANDARD TERMINOLOGY RELATING TO PAVEMENT DISTRESS** is another procedure that has been used by some agencies for identifying and measuring pavement distress. It is important that consistency be used to identify and measure pavement distresses. Without re-calibrating the MEPDG to local policies and practices, an agency or designer could use the LTPP Distress Identification Manual for determining the surface condition of the existing pavement. The Standard Practice for Determining the Local Calibration Parameters (NCHRP, 2007.b) addresses the use of condition surveys that have different measures of the distresses and smoothness values included in the LTPP Distress Identification Manual and predicted by the MEPDG.

As part of the condition survey, surface feature surveys may be performed but are not needed to determine the inputs to the MEPDG. These surface feature surveys include profile, friction, and noise measurements that are normally used to determine when a project is in need of repair. Only profile measurements are used in support of the MEPDG (refer to Table 12). The profile measurements are used to determine whether diamond grinding (PCC surfaces) or milling (HMA surfaces), a leveling course and its average thickness, or dense-graded layer are needed to retain the surface profile. The road profiles could be measured in accordance with AASHTO PP 37 or other equivalent procedures (Gillespie et al., 1987; Sayers and Karamihas, 1996; NHT, 1998). For HMA overlays, the number of lifts may be estimated from the existing IRI value – each successive lift of HMA may reduce the IRI value by approximately 70 percent.

10.2.4 Ground Penetrating Radar Survey

GPR is a well-established, high-speed nondestructive technology used to estimate the thickness of different pavement and soil strata layers, and is frequently used to survey areas before destructive sampling takes place. In fact, GPR may be valuable in reducing the number of cores and borings required for a project by segmenting the project based on similar subsurface features or anomalies identified with this technology prior to drilling the borings. Specifically, dielectric and thickness contours may be prepared along the project to locate areas with different structural features and material conditions. GPR data may be collected at highway speeds so that there is no interference with existing traffic.

GPR may also be used to investigate the internal composition of many pavement layers and soils, but is often overlooked or not used as a part of the field evaluation plan. GPR, however, has been used successfully to determine the condition of the existing pavement structure, identify areas with subsurface voids, locate areas with severe stripping in HMA, and locate interfaces with weak bonds between two HMA layers.

10.2.5 Refine Field Testing Plan

Results from the condition and GPR surveys could be used to strategically designate areas along the project for clustered deflection testing, DCP testing, and sampling the pavement layers and foundation soils to minimize the amount of time that the roadway is closed for the field activities requiring lane closure. Deflection basin tests, limited DCP tests, and drilling cores and borings could be located in areas with different surface distress and dielectric readings to ensure that all areas with different physical features and characteristics have been investigated.

10.2.6 Conduct Deflection Basin Tests

Nondestructive deflection testing (NDT) should be an integral part of any structural pavement evaluation for rehabilitation design. NDT could be performed prior to any destructive tests, such as cores and materials excavation, to better select the locations of such tests. The deflection basins are measured along the project at representative locations that vary by pavement type. Deflection basin tests could be performed in accordance with AASHTO T 256 **Standard Method of Test for Pavement Deflection Measurements** and the FHWA Field Operations manual (FHWA, 1998).

The deflection basin data measured along the project is used in several ways to help select adequate rehabilitation strategies and to provide input for backcalculating layer moduli. The backcalculated layer moduli are helpful in establishing the in-place structural condition of the pavement layers. Table 15 lists some of the specific uses of the deflection basin data for eventual inputs to the MEPDG software.

Table 15. Use of Deflection Basin Test Results for Selecting Rehabilitation Strategies and in Estimating Inputs for Rehabilitation Design with the MEPDG

Existing Pavement Layer	Design Input	Measurements and Tests Required for Design Inputs
All types of existing pavements	Deflection or deflection based indexes along the project	Used to select rehabilitation strategies and selection of design sections along project.
HMA	Dynamic modulus, E_{HMA}	Backcalculation of HMA layer modulus.
PCC	Elastic modulus, E_{PCC}	Backcalculation of PCC layer modulus.
	Joint load transfer efficiency (LTE)	Input for determining need for retro fit dowels, and reflection cracking (poor, good)
	Loss of support under corner	Input for determining rehabilitation strategy and repair (subsealing, crack and seat, etc.)
Stabilized base, subbase	Elastic modulus, E_{CTB}	Input for stabilized base or subbase (cement, asphalt, lime, fly ash, etc.).
Unbound materials (base, subbase, subgrade)	Resilient modulus, M_r	Backcalculation of unbound layer and subgrade modulus.

The most widely used deflection testing device is the falling weight deflectometer (FWD). However, the use of seismic testing devices is increasing in popularity and does provide an estimate of the in-place modulus of the pavement layers. Data from both of these types of NDT technologies need to be calibrated to laboratory conditions in providing inputs to the MEPDG procedure. The adjustment to laboratory conditions is discussed in a latter part of this subsection and in Section 11.

Deflection basin tests are suggested over seismic tests because deflections can be measured with different drop heights to evaluate the load-response characteristics of the pavement structure. Four drop heights are suggested for use, similar to the FHWA Field Operations Manual for the LTPP sites (FHWA, 1998). The use of four drop heights does not take much more additional time and may be used to categorize the pavement structure into three distinct load-response categories; elastic, deflection softening, and deflection hardening. These categories and their use are explained in NHI Course 131064 (NHI, 2002).

The spacing of the deflection tests will vary along a project. A closer spacing is suggested for areas with fatigue cracking. In addition, deflection basin tests could be performed in cut and fill areas and in transition areas between cut and fill. The transition areas are where water can accumulate and weaken the underlying soils.

The engineer could also designate a few areas along the project (preferably outside of the traffic lanes), and measure the deflection basins at the same point but during different temperatures (early morning versus late afternoon). The analysis of deflection basin data measured at different temperatures may assist in determining the in-place properties of the HMA and assist in evaluating the support conditions of PCC pavements.

For JPCP, deflections could be measured at the mid-slab (intact condition), along the transverse joints, and along the edge of the slabs to evaluate the load transfer efficiency and check for voids beneath the PCC layer.

10.2.7 Recover Cores and Boring for the Existing Pavement – Destructive Sampling and Testing

Destructive tests require the physical removal or damage of the pavement layer to observe the condition of the material. Tables 12 and 16 provide a summary of the types of destructive testing and their purposes, the procedures used, and the inputs needed for the MEPDG for rehabilitation design.

<u>Cores and Borings</u>

Cores and borings could be located in those areas with different pavement response characteristics and surface conditions. The cores could be used to confirm the layer thicknesses, material types, examine the pavement materials for material durability problems, and collect samples for laboratory tests.

Some cores could be drilled through any cracks observed at the surface of the pavement. These cores could be used to determine the depth of cracking and whether the cracks initiated at the surface. Knowing the depth of cracking and whether they initiated at the surface could be used in selecting a proper rehabilitation strategy for the project. For pavements with excessive rutting (greater than 0.75 inches), trenches may be necessary to determine if the rutting has occurred in the HMA or subsurface layers, in order to select a proper repair strategy. However, trenches are time-consuming and expensive. The engineer could make an assessment of their value and need for selecting a rehabilitation design strategy.

Table 16. Summary of Destructive Tests, Procedures, and Inputs for the MEPDG

Destructive Tests	Procedures	Input for MEPDG
Coring to recover samples for visual inspection & observations and lab testing	Coring & auguring equipment for HMA, PCC, stabilized materials, & unbound materials; DCP for unbound layers	• Thickness of all layers. • HMA durability condition. • HMA layer to layer bonding. • HMA lab testing for asphalt content, air voids, density, gradation. • PCC coefficient of thermal expansion. • PCC modulus of elasticity. • PCC compressive or IDT strength. • Stabilized base compressive strength to estimate the elastic modulus, E. • PCC to stabilized base bonding. • Obtain bulk samples of unbound materials and subgrade for gradation and classification tests. • Resilient modulus for the unbound layers.
Test pit	Saw cut rectangular pit to depth of stabilized materials, obtain samples of all materials	• Test unbound materials in laboratory for Atterberg limits, gradation, water content. • Observe condition of materials in each layer and layer interface bonding. • Beam of PCC for flexural strength testing.
Trenching of HMA pavements (see note 1)	Two saw cuts far enough apart to remove material with available equipment transversely across traffic lane	• Measure permanent deformation at surface and at each interface to determine amount within each layer. • Observe condition of HMA, base, and subbase materials and interfaces to see if HMA layers should be partially or completely removed for rehabilitation purposes.
Milling HMA overlay in composite pavement	Mill HMA down to PCC surface at joints	Observe HMA/PCC interface to determine if bond exists and if any stripping of HMA exists. Determine if HMA overlay should be completely removed for rehabilitation purposes. Observe durability of PCC at joint to determine need for repair or replacement.
Removal of PCC at joint	Full depth saw cut on both sides of joint and lift out joint	Examine condition of dowels, durability of PCC, deterioration of base to determine need for joint replacement.
Note 1: Trenches are expensive and time-consuming. Trenches should only be used in areas where the designer believes that extensive rutting has occurred in the subsurface layers.		

In-Place Strength of Individual Unbound Layers

The DCP may be used in pavement evaluations to measure the strength of unbound layers and materials. It may also be used for estimating soil layer thickness by identifying sudden changes in strength within the pavement structure and foundation. The MEPDG software allows the user to input the DCP test results directly or indirectly depending on the model of choice for converting the raw penetration data into layer moduli. The options include; directly entering the average penetration rate, converting the average penetration rate into a CBR value using locally calibrated models to calculate a CBR value and then entering that CBR value, or converting the average penetration rate into a resilient modulus using locally calibrated models and then entering that resilient modulus.

Interface Friction Between Bound Layers

Layer interface friction is an input parameter to the MEPDG, but is difficult to define and measure. Cores and visual surveys may be used to determine if debonding exists along the project. Slippage cracks and two adjacent layers separating during the coring process may be a result of low interface friction between two HMA layers. If these conditions are found to exist along a project, the designer could consider assuming no bond when those layers are to remain in place and not be milled or removed. All of the global calibration efforts for flexible pavements, however, were completed assuming full friction between all layers – an interface friction value of 1.0 in the MEPDG. This value could be used unless debonding is found. Interface friction values less than 1.0 will increase rutting and cracking of the HMA layers. The increase in rutting and cracking of the HMA is minimal until the condition of no bond, a value of 0, is used. Thus, friction can be defined for just two conditions without significantly affecting the accuracy of the answer; fully bonded (a value of 1.0) or no bond (a value of 0).

JPCP requires a PCC/base contact friction input of months of full contact friction (no slippage between layers). Calibration results for new/reconstructed JPCP showed that full contact friction existed over the life of the pavements for all base types, with the exception for CTB or lean concrete where extraordinary efforts were made to debond the layers. For this situation, the months of full contact friction was reduced to a range of 0 to 15 years to match the cracking exhibited. For new and reconstructed PCC designs, thus, full friction needs to always be assumed, unless debonding techniques are specified and confirmed through historical records.

For rehabilitation of JPCP (CPR and overlays), full contact friction could be input over the rehabilitation design life, when cores through the base course show that interface bond exists. Otherwise, the two layers could be considered as having zero friction over the design life.

Edge Drains

If the existing pavement has subsurface drains that may remain in place, the outlets need to be found and inspected. Mini-camera may also be used to ensure that the edge drains and lateral lines are free-flowing and not restricting the removal of water from the pavement structure.

10.2.8 Laboratory Tests for Materials Characterization of Existing Pavements

Table 16 provided a listing of the materials properties that need to be measured for determining the inputs to the MEPDG relative to the condition of the existing pavement layers. The user is referred to Section 11 for the testing of different pavement layers that is required in support of the MEPDG.

The number of samples that need to be included in the test program is always the difficult question to answer. The engineer needs to establish a sufficient laboratory test program to estimate the material properties of each layer required as inputs to the MEPDG. The following lists the type of samples needed for measuring the properties of the in-place layers (refer to Table 15).

<u>HMA Mixtures and Layers</u>

• Volumetric Properties (air voids, asphalt content, gradation) – If construction data are available from as built project records, air voids (bulk specific and maximum theoretical specific gravities) is the only volumetric property that could be measured on those layers that will remain in place after rehabilitation, as a minimum for input levels 1 and 2 (Table 12). The average effective asphalt content by volume and gradation measured during construction may be used for the rehabilitation design. If this volumetric data is unavailable from construction records, selected cores recovered from the project may be used to measure these properties. Samples recovered from 6-inch-diameter cores should be used to ensure a sufficient amount of material for gradation tests. The NCAT ignition oven may be used to measure the asphalt content (in accordance with AASHTO T 308 **Standard Method of Test for Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method** or an equivalent procedure) and then the gradation can be estimated based on the aggregate remaining (in accordance with AASHTO T 27 **SIEVE ANALYSIS OF FINE AND COARSE AGGREGATES**). The HMA density and VMA may be calculated from the HMA bulk specific gravity (AASHTO T 166 **BULK SPECIFIC GRAVITY OF COMPACTED ASPHALT MIXTURES USING SATURATED SURFACE-DRY SPECIMENS**), maximum theoretical specific gravity (AASHTO T 209 **THEORETICAL MAXIMUM SPECIFIC GRAVITY AND DENSITY OF BITUMINOUS PAVING MIXTURES**), aggregate specific gravity, and asphalt content (refer to subsection 11.2).

• Dynamic Modulus – Use adjusted backcalculated modulus from deflection basin or seismic tests to estimate the amount of damage of the in-place HMA layers. Laboratory dynamic modulus tests are not needed for measuring the in-place modulus because the test needs to be performed on intact, but age-hardened specimens. The resulting modulus values will likely be higher than those for new HMA mixtures, suggesting no damage to the in-place mixture, which may not be the case. Thus, it is recommended that the modulus be determined from the deflection basin tests.

• Creep Compliance – Not needed for the existing HMA layers.

• Indirect Tensile Strength – The relationship between the IDT modulus and tensile strain at failure may be used to estimate the amount of damage of the in-place HMA layer using NCHRP Report 338 (Von Quintus, et al., 1991). If an HMA layer is believed to have exhibited stripping or some moisture damage, indirect tensile tests could be used to measure the strength, tensile strain at failure, and dynamic modulus of moisture-conditioned and unconditioned specimens of the in-place mixtures to confirm the amount of moisture damage that might be present. If moisture damage is found, this finding could be used in establishing the modulus input values and condition to the MEPDG, if that layer is left in place. If stripping is found near the surface, that layer could be considered for removal in the rehabilitation design.

• Asphalt Classification – Extract asphalt from selected cores to determine the performance-grade (PG) of the recovered asphalt (AASHTO M 320 **Standard Specification for Performance-Graded Asphalt Binder**). The asphalt classification and volumetric test results are used to determine the undamaged condition of the HMA layer and compare that value to the average backcalculated value in cracked areas to estimate the amount of damage. Extracting the asphalt from existing HMA layers of flexible pavements is expensive, time-consuming, and becoming problematic because of environmental restrictions. For the projects where asphalt is not extracted, historical information and data may be used to estimate the PG of the age-hardened asphalt for the lower HMA layers that will remain in place after rehabilitation.

PCC Mixtures and Layers

• Elastic Modulus of PCC – Use either the backcalculated modulus values (multiplied by 0.8) to estimate the static modulus, or test for the static modulus of elasticity using a limited number of samples recovered from the coring process. Otherwise, estimate using inputs for flexural strength. The adjustment factor of 0.8 is used to reduce the dynamic modulus value calculated from deflection basin tests to a static modulus value measured in the laboratory.

• Indirect Tensile Strength (for CRCP only) – The indirect tensile strength is measured on samples recovered during the coring process and is used to estimate the flexural strength of the in-place PCC layer. If cores are unavailable, the compressive strength may be used to estimate the in-place flexural strength.

• Flexural Strength – Not needed for the existing PCC layer; the indirect tensile strength or compressive strength may be used to estimate the flexural strength.

Unbound Layers

• Resilient Modulus – The backcalculated modulus values adjusted to laboratory conditions is the preferred and suggested technique for rehabilitation design because the resulting layer modulus value is an equivalent value of the materials that vary horizontally and vertically. The resilient modulus also may be calculated from DCP penetration rates or measured in the laboratory on test specimens prepared and compacted to the in-place moisture content and dry density found during the subsurface investigation. These techniques are not suggested because they do not capture the variability of materials in the vertical and horizontal direction without increasing the test program. The laboratory resilient modulus test represents a discrete specimen in the horizontal and vertical direction, while the DCP test captures the variability vertically, but not horizontally with one test. More importantly, unbound layers and foundations that contain large boulders or aggregates are difficult to test in the laboratory and in-place with the DCP.

• Volumetric Properties – Measure the moisture content and dry density of undisturbed samples recovered during the subsurface investigation. The in-place volumetric properties may be used for estimating the in-place resilient modulus value of the unbound layers from the regression equations developed from the LTPP data, if deflection basin data and DCP test results for estimating in-place modulus values are unavailable (Von Quintus and Yau, 2001).

• Classification Properties – Measure the gradation and Atterberg limits from bulk sample recovered from the subsurface investigation.

10.3 Analysis of Pavement Evaluation Data for Rehabilitation Design Considerations

The pavement structural evaluation for determining the condition of the existing pavement layers is based on an analysis of the visual distress surveys, deflection basin and other field tests, and laboratory tests. It is recommended that the highest input level available be used for rehabilitation design of high volume roadways.

10.3.1 Visual Distress Survey to Define Structural Adequacy

Surface distresses provide a valuable insight into a pavement's current structural condition. Tables 17 and 18 provide a recommended assessment of rigid and flexible pavements, respectively. These two tables relate the condition of the pavement surface as to whether the pavement is structurally adequate, marginal or inadequate. All of the distresses included in Tables 17 and 18 are not predicted with the MEPDG. Adequate implies that the surface condition or individual distresses would not trigger any major rehabilitation activity and the existing pavement has some remaining life; marginal implies that the existing pavement has exhibited distress levels that do require maintenance or some type of minor repairs; and inadequate implies that the pavement has distresses that require immediate major rehabilitation and has no remaining life. Obviously, the values included in these two tables depend on the importance of the distress to an individual agency.

10.3.2 Backcalculation of Layer Modulus Values

Deflection basin data are considered one of the more important factors to assess the structural condition of the pavement. One of the more common methods for analysis of deflection data is to backcalculate the elastic properties for each layer in the pavement structure and foundation. Backcalculation programs provide the elastic layer modulus typically used for pavement evaluation and rehabilitation design. ASTM D 5858, **Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory** is a procedure for analyzing deflection basin test results to determine layer elastic moduli (i.e., Young's modulus).

The absolute error or Root Mean Squared (RMS) error is the value that is used to judge the reasonableness of the backcalculated modulus values. The absolute error term is the absolute difference between the measured and computed deflection basins expressed as a percent error or difference per sensor; the RMS error term represents the goodness-of-fit between the measured and computed deflection basins. The RMS and absolute error terms needs to be as small as possible. An RMSE value in excess of 3 percent generally implies that the layer modulus values calculated from the deflection basins are inaccurate or questionable. RMSE values less than 3 percent should be used in selecting the layer modulus values for determining the minimum overlay thickness.

Table 17. Distress Types and Severity Levels Recommended for Assessing Rigid Pavement Structural Adequacy

Load-Related Distress	**Highway Classification**	**Current Distress Level Regarded As:**		
		Inadequate (Poor)	**Marginal (Fair)**	**Adequate (Good)**
JPCP Deteriorated Cracked Slabs (medium & high severity transverse & longitudinal cracks & corner breaks), % slabs	Interstate, Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10
JRCP Deteriorated Cracked Slabs (medium & high severity transverse cracks & corner breaks), #/lane-mi.	Interstate, Freeway	>40	15 to 40	<15
	Primary	>50	20 to 50	<20
	Secondary	>60	25 to 60	<25
JPCP Mean Transverse Joint/Crack Faulting, in.	Interstate, Freeway	>0.15	0.1 to 0.15	<0.1
	Primary	>0.20	0.12 to 0.20	<0.125
	Secondary	>0.30	0.15 to 0.30	<0.15
CRCP Punchouts (medium & high severity), #/lane-mi.	Interstate, Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10
NOTE: The above distresses can be used to access the condition of the existing rigid pavement, all of which are not predicted by the MEPDG.				

Obviously, the absolute error (percent error per sensor) and RMS error (goodness-of-fit) vary from station-to-station and depend on the pavement's physical features that have an effect on the deflection basin measured with the FWD. For example, thickness variations, material density variations, surface distortion, and cracks, which may or may not be visible at the surface and may cause small irregularities within the measured deflection basin, which are not consistent with the assumptions of elastic layer theory.

Thus, the calculated layer modulus represents an "effective" Young's modulus that adjusts for stress-sensitivity and discontinuities or anomalies (variations in layer thickness, localized segregation, cracks, slippage between adjacent layers, and the combinations of similar materials into a single layer).

Layer thickness is a critical parameter for backcalculating layer modulus values. The use of borings and cores to measure layer thickness becomes expensive, considering traffic control requirements and the time needed for the drilling operation. GPR is another test method that may be used to determine the variation in layer thickness along a project.

Elastic layer modulus (Young's Modulus) values estimated from FWD deflection basin data were used in the MEPDG recalibration effort in NCHRP Project 1-40D. The modulus values for each test section were extracted from the LTPP database (FHWA, 2006) and adjusted to laboratory conditions for the recalibration process. A backcalculation process was used for flexible pavements, while a forward calculation process was used for rigid pavements. Backcalculation means that an iterative, deflection-matching process was used and that there is no unique solution (combination of layer modulus values) for a specific deflection basin. Forward calculation means that the layer modulus values were calculated using specific points along the deflection basin and that a unique set of layer modulus values is determined for each basin. Both approaches have advantages and disadvantages relative to how the results are used with the MEPDG.

Table 18. Distress Types and Levels Recommended for Assessing Current Flexible

Distress Type	Highway Classification	Current Distress Level Regarded As:		
		Inadequate (Poor)	Marginal (Fair)	Adequate (Good)
Fatigue Cracking, percent of total lane area	Interstate, Freeway	>20	5 to 20	<5
	Primary	>45	10 to 45	<10
	Secondary	>45	10 to 45	<10
Longitudinal Cracking in Wheel Path, ft./mi.	Interstate, Freeway	>1060	265 to 1060	<265
	Primary	>2650	530 to 2650	<530
	Secondary	>2650	530 to 2650	<530
Reflection Cracking, percent of total lane area.	Interstate, Freeway	>20	5 to 20	<5
	Primary	>45	10 to 45	>10
	Secondary	>45	10 to 45	<10
Transverse Cracking Length, ft./mi.	Interstate, Freeway	>800	500 to 800	<500
	Primary	>1000	800 to 1000	<800
	Secondary	>1000	800 to 1000	<800
Rutting, mean depth, maximum between both wheel paths, inches.	Interstate, Freeway	>0.45	0.25 to 0.45	<0.25
	Primary	>0.6	0.35 to 0.60	<0.35
	Secondary	>0.8	0.40 to 0.80	<0.4
Shoving, percent of wheel path area	Interstate, Freeway	>10	1 to 10	None
	Primary	>20	10 to 20	<10
	Secondary	>50	20 to 45	<20
NOTE: The above distresses can be used to access the condition of the existing flexible pavement, all of which are not predicted by the MEPDG.				

Flexible Pavements

The elastic modulus of each structural layer typically is calculated using programs based on elastic layer theory that use an iterative technique to match the calculated deflection basin to the measured one. Backcalculation programs that use this iterative technique do not result in a unique solution or set of layer moduli. As such, determining a set of elastic layer moduli to match a measured deflection basin that deviates from elastic theory, for whatever reason, may become difficult and frustrating. As such, it is recommended that the deflection basins be grouped into those that are consistent with elastic layer theory and those that are not. Users may get frustrated in trying to backcalculate elastic layer moduli from deflection basins within an allowable error range that are inconsistent with elastic layer theory. NHI Course 131064 presents the different deflection basin categories (NHI, 2002). There are forward calculation programs that do result in unique layer moduli, but these have not been commonly used and are restricted to three layer structures.

Most backcalculation programs limit the number of layers to five or less. Some of the features of the existing pavements that may be important and have an effect on estimating the elastic modulus of the structural layers include: the depth to a water table and an apparent rigid layer, combining thin layers or adjacent layers of similar materials, transverse and fatigue cracks, and stripping within HMA layers. The NHI Course 131064 **Introduction to Mechanistic-Empirical Design of New and Rehabilitated Pavements** reference manual provides guidance in combining and formulating the structural layers included in the backcalculation process and the number of sensors needed within the backcalculation process.

The other issue regarding backcalculation programs that use an iterative procedure is compensating errors. In other words, the modulus of one layer is continually increased, while the modulus of an adjacent layer continually decreases during the iterative technique in trying to minimize the error term. Compensating errors and their effect also are discussed in the NHI Course 131064 reference manual.

Rigid Pavements

Rigid pavements generally are analyzed as slab on grade with or without a base or subbase. In the past decade, much progress has been made in the development of reliable methods for backcalculation of concrete slab, base layer, and subgrade moduli from deflection measurements. Several methods for backcalculating the PCC slab, base, and subgrade moduli or moduli of subgrade reaction (k-value) are available. Each method has its strengths and its limitations. The following are algorithms specifically developed for rigid pavement; based on slab on elastic solid or slab on dense liquid models:

- AREA method-based procedures.
- Best Fit-based procedures.

Both backcalculation procedures/algorithms are based on plate theory and are used to backcalculate layer material properties—elastic modulus, Poisson's ratio, and modulus of subgrade reaction. The Best Fit method solves for a combination of the radius of relative stiffness, ℓ, and the coefficient of subgrade reaction, k, that produce the best possible agreement between the predicted and measured deflections at each sensor. The AREA method, which was described in the 1993 AASHTO Guide, estimates the radius of relative stiffness as a function of the AREA of the deflection basin. This estimation, along with the subsequent calculation of subgrade k and slab modulus of elasticity, E, is made using simple closed form equations. Both methods are based on Westergaard's solution for the interior loading of a plate consisting of a linear elastic, homogeneous, and isotropic material resting on a dense liquid foundation. To account for the effect of a stabilized base, a ratio of the moduli of elasticity of PCC and base layers should be assumed according to the LTPP guidelines (Khazanovich 1999).

10.3.3 Loss of Support Detection

Detection of loss of support under joints and cracks in rigid pavements is one of the important uses of the GPR and FWD. The FWD deflection data may be analyzed in several ways to estimate the approximate area where loss of support has occurred under a concrete pavement. If extensive loss of support is found along a project this may require subsealing or slab fracturing to establish a uniform layer for an overlay. GPR may also be used to locate areas with this type of anomaly, but it does not provide a quantitative measure of the loss of support.

10.3.4 Joint Load Transfer Efficiency

Deflection testing may also be used to evaluate the LTE of joints and cracks in rigid pavements. This information is used in selecting rehabilitation strategies, needed repair (e.g., retro fit dowels), and in assessing the reflection cracking potential if the jointed concrete pavement is overlaid with an HMA overlay.

10.3.5 Variability Along a Project

Variation along a project creates a much more difficult task to obtain the appropriate inputs for a project. This variability may be quantified based on the field data sets; visual survey, GPR, and deflection basin data. The visual surveys are used to define if there are significant differences in the surface distresses over the length of the project. The deflection basins and GPR readings may also be used to estimate the variability along a project and determine if the load-response or layer thicknesses of the pavement structure are significantly different along the project.

The variation can be handled for cases where large differences occur along the existing project by dividing the project into multiple design sections. The decision as to subdividing the project into two or more design sections could be based on whether or not the recommended rehabilitation work should actually change. For example, one portion of a project may exhibit extensive fatigue cracking, while another portion has only rutting. The overlay design could logically be different for each section, or the possibility of removal and replacement of the existing damaged material may be the deciding factor to subdivide the project.

13 Rehabilitation Design Strategies

13.1 General Overview of Rehabilitation Design Using the MEPDG

A feasible rehabilitation strategy is one that addresses the cause of the pavement distress and deterioration and is effective in both repairing it and preventing or minimizing its reoccurrence. The MEPDG has the capability to evaluate a wide range of rehabilitation designs for flexible, rigid and composite pavements. The MEPDG rehabilitation design process is an iterative, hands-on approach by the designer – starting with a trial rehabilitation strategy. Similar to developing the initial trial design for new pavements, the trial rehabilitation design may be initially determined using the 1993 AASHTO Design Guide, a rehabilitation design catalog, or an agency specific design procedure.

The MEPDG software may then be used to analyze the trial design to ensure that it will meet the user's performance expectations.

A considerable amount of analysis and engineering judgment is required when determining specific treatments required to design a feasible rehabilitation strategy for a given pavement condition. The NHI training course on Techniques for Pavement Rehabilitation provides guidance on selecting repair strategies for different conditions of the existing pavement (NHI, 1998). The MEPDG considers four major strategies, as listed below, which may be applied singly or in combination to obtain an effective rehabilitation plan based on the pavement condition that was defined under Section 9.

- Reconstruction without lane additions – this strategy is considered under new pavement design strategies.
- Reconstruction with lane additions – this strategy is considered under new pavement design strategies.
- Structural overlay, which may include removal and replacement of selected pavement layers.
- Non-structural overlay.
- Restoration without overlays.

The MEPDG provides detailed guidance on the use and design of rehabilitation strategies, depending on the type and condition of the existing pavement, and provides specific details on the use of material specific overlays for existing flexible and rigid pavements. This section provides an overview of strategies for the rehabilitation of existing flexible, rigid, and composite pavements. Figure 30 shows the steps that are suggested for use in determining a preferred rehabilitation strategy.

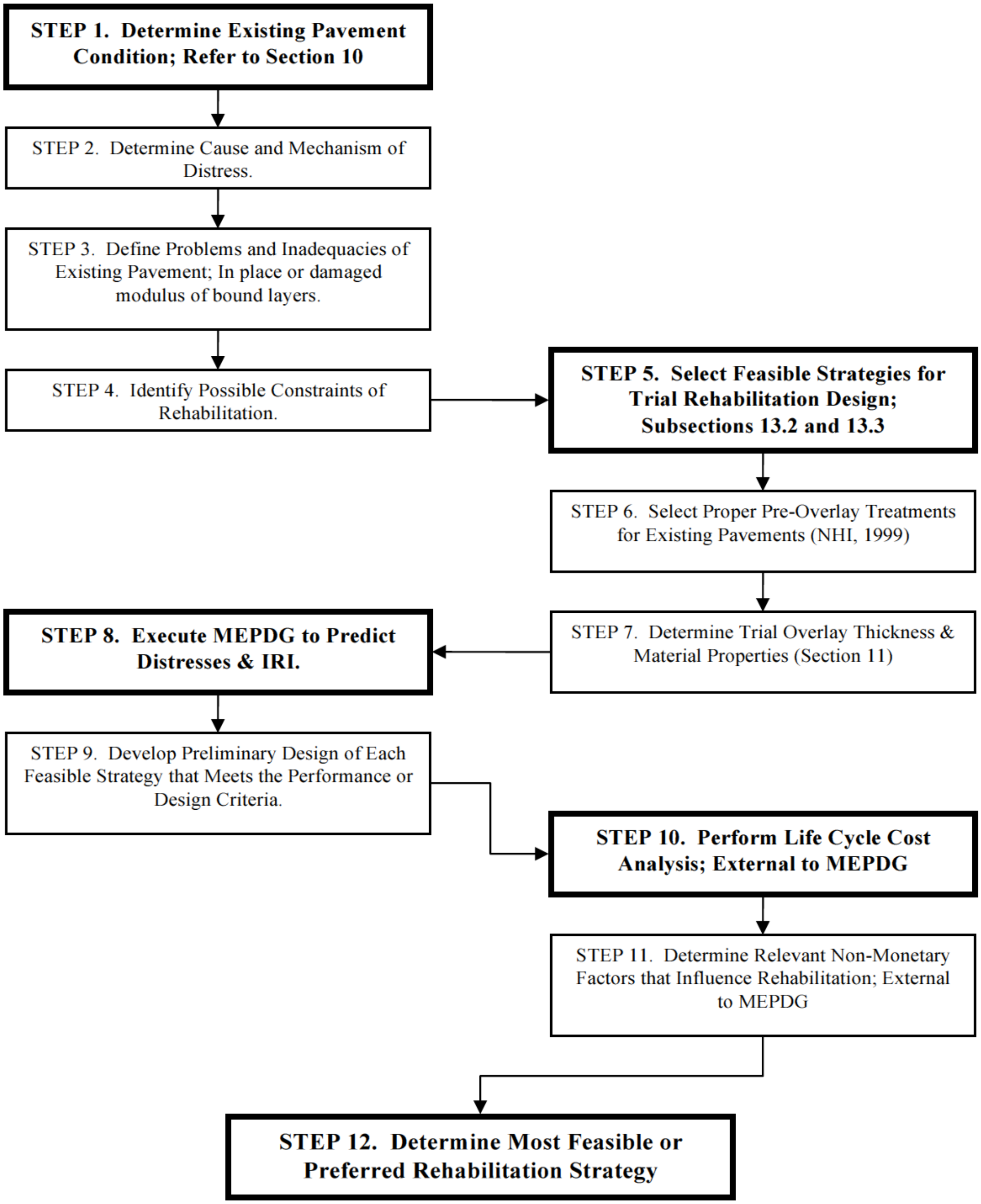


Figure 30. Steps for Determining a Preferred Rehabilitation Strategy

13.2 Rehabilitation Design with HMA Overlays

13.2.1 Overview

The MEPDG includes specific details for selecting and designing HMA overlays to improve the surface condition or to increase the structural capacity of the following pavements (refer to Figure 7 under subsection 3.3).

• HMA overlays of existing HMA surfaced pavements; both flexible and semirigid.

• HMA overlays of existing PCC pavements that has received fractured slab treatments; crack and seat, break and seat, and rubblization.

• HMA overlays of existing intact PCC pavements (JPCP and CRCP), including composite pavements or second overlays of original PCC pavements.

Figure 31 presents a generalized flow chart for pavement rehabilitation with HMA overlays of HMA-surfaced flexible, semi-rigid, or composite pavements, fractured PCC pavements and intact PCC pavements.

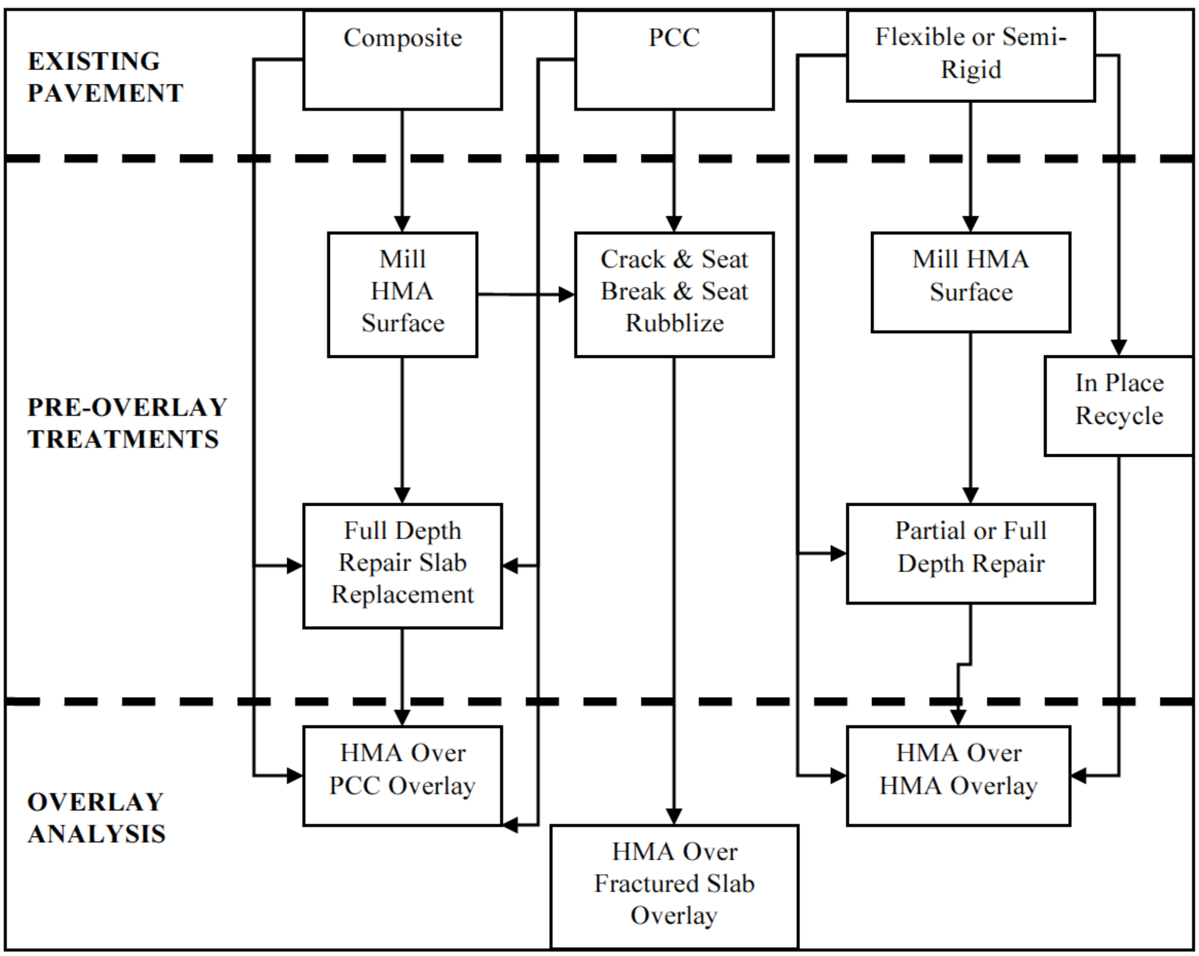


Figure 31. Flow Chart of Rehabilitation Design Options Using HMA Overlays

13.2.2 HMA Overlay Analyses and Trial Rehabilitation Design

For existing flexible or semi-rigid pavements, the designer needs to first decide on what, if any pre-overlay treatment is needed for minimizing the effect of existing pavement distresses on the HMA overlay and select an initial overlay thickness. Pre-overlay treatments may include do nothing, a combination of milling, full or partial depth repairs, or in-place recycling (refer to subsection 13.2.4). In either case, the resulting analysis is an HMA overlay of an existing HMA-surfaced pavement.

Similarly, the analysis for existing PCC pavements may be either an HMA over PCC analysis or an HMA over fractured slab analysis depending on whether or not crack and seat, break and seat, or rubblization techniques are applied to the existing PCC pavement.

Existing composite pavements may result in either an HMA over PCC analysis or an HMA over fractured slab analysis depending on whether or not the existing HMA surface is removed and the underlying PCC pavement is fractured.

The HMA over PCC analysis also considers continued damage of the PCC slab using the rigid pavement performance models presented in Section 5 and subsection 13.2.8. The three overlay analyses also provide the capability to address reflection cracking of joints and cracks in PCC pavements and thermal and load associated cracking in HMA surfaced pavements. However, it needs to be noted that the reflection cracking models incorporated in the MEPDG were based strictly on empirical observations and were not a result of rigorous M-E analyses. Finally, the predicted distresses are linked to estimates of IRI to form a functional performance criterion that may be considered along with the specific distresses in the design-analysis process.

The maximum number of overlay layers that may be specified is four. This includes up to three HMA layers, and one unbound or chemically stabilized layer. The total number of layers of the existing pavement and the overlay is limited to 14. For the initial design, however, it is suggested that the total number of layers be limited to no more than eight to reduce the number of required inputs and run time.

13.2.3 Determine Condition of Existing Pavement

A critical element for determining the HMA overlay design features and thickness is the characterization of the existing pavement, including determination of the damaged modulus of the existing bound layers. General recommendations for evaluating the existing pavement for rehabilitation were included in Section 10. As for new pavement designs, all properties of the existing and new pavement layers need to be representative of the conditions expected right after rehabilitation – when the roadway is opened to traffic.

Table 18 in Section 10 provided general recommendations for assessing the current condition of flexible, semi-rigid, composite, and HMA overlaid pavements, while Table 12 provided the pavement evaluation activities for the different input levels. For input level 3, a generalized rating for the existing pavement is an input to the MEPDG. The designer has five options to select from: Excellent, Good, Fair, Poor, and Very Poor.

Table 29 provides a definition of the surface condition and summarizes the rehabilitation options suggested for each of these general ratings. For input level 1, cores and trenches are used to determine the amount of rutting within each paving layer and whether any cracks that have occurred initiated at the surface or bottom of the HMA layers. For input level 2, cores are used to estimate the amount of rutting within each layer and determine where any load related cracks initiated.

Table 29. Definitions of Surface Condition for Input Level 3 Pavement Condition Ratings and Suggested Rehabilitation Options

Overall Condition (Table 18, Section 10)	General Pavement Condition Rating; Input Level 3		Rehabilitation Options to Consider (With or Without Pre-Overlay Treatments; Subsection 13.2.4)
Adequate (Has Remaining Life)	Excellent	No cracking, minor rutting, and/or minor mixture related distresses (e.g., raveling); little to no surface distortions or roughness.	• Surface repairs without overlays (not analyzed with the MEPDG). • Pavement preservation strategy (not analyzed with the MEPDG). • Non-structural overlay. • Overlay designed for future truck traffic levels.
	Good	Limited load and/or non-load related cracking, minor to moderate rutting, and/or moderate mixture related distresses; some surface distortions & roughness.	• Pavement preservation strategy (not analyzed with the MEPDG). • Overlays designed for future truck traffic levels, with or without milling & surface repairs.
Marginal (May or May Not Have Remaining Life)	Fair	Moderate load and/or non-load related cracking, moderate rutting, moderate amounts of mixture related distresses, and/or some roughness (IRI>120 in./mi.).	Pre-Overlay Treatments Recommended. • Structural overlay, with or without milling & surface repairs. • Remove & replace surface layer prior to overlay. • In place recycling prior to overlay.
Inadequate (No Remaining Life)	Poor	Extensive non-load related cracking, moderate load related cracking, high rutting, extensive mixture related distresses, and/or elevated levels of roughness (IRI>170 in./mi).	Pre-Overlay treatment recommended if not reconstructed. • Structural overlay, with milling or leveling course & surface repairs. • Remove & replace existing layers prior to overlay. • In place recycling prior to overlay. • Reconstruction
	Very Poor	Extensive load related cracking and/or very rough surfaces (IRI>220 in./mi.)	Pre-Overlay treatment recommended if not reconstructed. • Structural overlay with milling & surface repairs. • Remove & replace existing layers prior to overlay. • In place recycling prior to overlay. • Reconstruction.

13.2.4 Decide on Pre-Overlay Treatment

Various pre-overlay treatments and repairs need to be considered to address deterioration of the existing pavement, improve surface smoothness, and provide uniform support conditions for the HMA overlay. For existing flexible or semi-rigid pavements, the pre overlay treatments may include; do nothing, placement of a leveling course, a combination of milling, full or partial depth repairs, or in-place recycling. For existing rigid pavements, the pre-overlay repair may include; do nothing, diamond grinding, full or partial depth slab repair of JPCP and JRCP and punchouts of CRCP, and/or mudjacking the slabs to fill any voids and re-level the slabs. Crack sealing is not a recommended pre-overlay treatment prior to overlay placement because the HMA overlay when placed at elevated temperatures may cause the sealant material to expand creating a bump in the overlay and significantly reducing the smoothness of the final surface.

Determining how much of the distress or damage could be repaired before the HMA overlay is placed requires a careful mix of experience and engineering judgment. Table 30 lists some of the candidate repair or pre-overlay treatments for all types of pavements, while Table 31 lists the major rehabilitation treatments of existing HMA and HMA over PCC pavements. Deciding on the pre-overlay treatment to be used could be based more on experience and historical data, rather than on the distresses and IRI predicted with the MEPDG.

If the distress in the existing pavement is likely to affect overlay performance within a few years, it could be repaired prior to overlay placement. Premature distress in the overlay is often the result of deterioration in the existing pavement that was not properly repaired before overlay placement. NHI Courses 131063 and 131062 provide good reference material for making the decision of what, if any, pre-overlay treatment is needed (APT, Inc., 2001.a and 2001.b).

For HMA surfaced pavements, cold milling and in-place recycling has become common pre-overlay treatments. Cold milling equipment can easily remove as much as 3 to 4 inches of HMA in a single pass. Removal of a portion of the existing cracked and hardened HMA surface by cold milling frequently improves the performance of an HMA overlay – because it provides good interface friction and removes surface defects. Cold milling also increases the smoothness of the existing pavement by removing rutting and other surface distortions. The depth of milling is an input to the MEPDG.

In-place recycling may be considered an option to reconstruction for those cases where an HMA overlay is not feasible due to the extent of repair that needs to be required to provide uniform support conditions. Recent equipment advances provide the capability to recycle pavements in place to a depth of 8 to 12 inches. If the in-place recycling process includes all of the existing HMA layers (defined as pulverization), this option could be treated as a new flexible pavement design strategy. The pulverized layer may be treated as a granular layer if not stabilized or a stabilized layer if asphalt emulsion or some other type of stabilizer is added prior to compaction.

Agencies have used a wide range of materials and techniques as part of a rehabilitation design strategy to delay the occurrence of reflection cracks in HMA overlays of existing pavements. These materials include paving fabrics, stress-absorbing interlayer (SAMI), chip seals, crack relief layer or mixture, cushion course, cold in-place recycling and hot in-place recycling.

Paving fabrics, thin layers, pavement preservation techniques, preventive maintenance activities, and other non-structural layers are not analyzed mechanistically in the MEPDG.

Table 30. Candidate Repair and Preventive Treatments for Flexible, Rigid, and Composite Pavements

Pavement Type	Distress	Preventive Treatments	Repair Treatments
Flexible and Composite	Alligator Cracking	Surface/Fog seal	Full-depth repair
		Surface patch	
	Longitudinal Cracking	Crack sealing	Partial-depth repair
	Reflective Cracking	Rout and seal cracks	Full-depth repair
		Saw & seal cuts above joints in PCC layer	
	Block Cracking	Seal cracks	Chip Seal
		Chip seal	
	Depression	None	Leveling course
			Mill surface
	Rutting	None	Leveling course
			Mill surface
	Raveling	Rejuvenating seal	Chip seal/surface seal
	Potholes	Crack sealing	Full-depth or partial-depth repairs
		Surface patches	
Rigid	JPCP Pumping	Reseal joints	Subseal or mud-jack PCC slabs (effectiveness depends on materials & procedures)
		Restore joint load transfer	
		Subsurface drainage	
		Edge support (tied PCC should edge beam)	
	JPCP Joint Faulting	Subseal joints	Grind surface; Structural overlay
		Reseal joints	
		Restore load transfer	
		Subsurface drainage	
		Edge support (tied PCC should edge beam)	
	JPCP Slab Cracking	Subseal (loss of support)	Full-depth repair
		Restore load transfer	Partial-depth repair
		Structural overlay	
	JPCP Joint or Crack Spalling	Reseal joints	Full-depth repair
			Partial-depth repair
	Punchouts (CRCP)	Polymer or epoxy grouting	Full-depth repair
		Subseal (loss of support)	
	PCC Disintegration	None	Full-depth repair
			Thick overlay

Table 31. Summary of Major Rehabilitation Strategies and Treatments Prior to Overlay Placement for Existing HMA and HMA/PCC Pavements

Pavement Condition	Distress Types	Candidate Treatments for Developing Rehabilitation Design Strategy											
		Full-Depth HMA Repair	Partial-Depth HMA Repair	Cold Milling	Hot or Cold In-place Recycling	Cracking Sealing	Chip Seal	HMA Overlay	HMA Overlay of Fractured PCC Slab	Bonded PCC Overlay	Unbounded PCC Overlay	Subsurface Drainage Improvement	Reconstruction (HMA or PCC)
Structural	Alligator Cracking	✓			✓		✓	✓	✓	✓	✓		✓
	Longitudinal Cracking (low severity)		✓	✓	✓	✓		✓		✓	✓		✓
	Thermal Cracking	✓		✓	✓	✓		✓		✓	✓		✓
	Reflection Cracking	✓	✓	✓				✓	✓	✓	✓		✓
	Rutting - Subsurface			✓	✓			✓		✓	✓		✓
	Shoving - Subsurface	✓						✓					✓
Functional	Excessive Patching							✓			✓		✓
	Smoothness			✓				✓					
Drainage, Moisture Damage	Raveling		✓	✓				✓					
	Stripping	✓	✓					✓		✓	✓	✓	✓
	Flushing/Bleeding		✓				✓	✓					
Durability	Raveling		✓	✓	✓		✓	✓					
	Flushing/Bleeding		✓	✓	✓		✓	✓					
	Shoving - HMA		✓	✓	✓			✓					
	Rutting - HMA			✓	✓			✓					
	Block Cracking			✓	✓	✓	✓	✓					
Shoulders	Same as traveled lanes	Same Treatments as recommended for the traveled lanes.											

The fitting and user-defined cracking progression parameters in the MEPDG empirical reflection crack prediction equation are provided only for the HMA overlay with paving fabrics (refer to Table 1 in subsection 5.2.5). The fitting parameters were estimated from limited test sections with a narrow range of existing pavement conditions and in localized areas. Additional performance data are needed to determine the values for both the fitting and user-defined cracking progression parameters for a more diverse range of conditions and materials.

In the interim, designers may use the default fitting parameters for predicting the amount of reflection cracks over time, but they should not consider the predicted amount of reflection cracks in making design decisions. Design strategies to delay the amount of reflection cracks could be based on local and historical experience, until a reliable M-E based prediction methodology is added to the MEPDG or the empirical regression equation has been calibrated for a more diverse set of existing pavement conditions for the different materials noted above.

13.2.5 Determination of Damaged Modulus of Bound Layers and Reduced Interface Friction

Deterioration in the existing pavement includes visible distress, as well as damage not visible at the surface. Damage not visible at the surface must be detected by a combination of NDT and pavement investigations (cores and borings).

In the overlay analysis, the modulus of certain bound layers of the existing pavement is characterized by a damaged modulus that represents the condition at the time of overlay placement. The modulus of chemically stabilized materials and HMA is reduced due to traffic induced damage during the overlay period. The modulus reduction is not applied to JPCP and CRCP because these type pavements are modeled exactly as they exist.

Cracks in these slabs are considered as reflective transverse cracks through the HMA overlay. Damage of HMA is simulated in the MEPDG as a modulus reduction of that layer.

Results from the pavement investigation need to identify any potential areas or layers with reduced or no interface friction. Reduced interface friction is usually characterized by slippage cracks and potholes. If this condition is found, the layers where the slippage cracks have occurred could be considered for removal or the interface friction input parameter in the overlay design should be reduced to 0 between those adjacent layers.

13.2.6 HMA Overlay Options of Existing Pavements

Table 31 listed different repair strategies for existing HMA and HMA over PCC pavements with different surface conditions that have some type of structural-material deficiency.

HMA Overlay of Existing Flexible and Semi-Rigid Pavements An HMA overlay is generally a feasible rehabilitation alternative for an existing flexible or semi-rigid pavement, except when the conditions of the existing pavement dictate substantial removal and replacement or in-place recycling of the existing pavement layers. Conditions where an HMA overlay is not considered feasible for existing flexible or semi-rigid pavements are listed below.

1. The amount of high-severity alligator cracking is so great that complete removal and replacement of the existing pavement surface layer is dictated.
2. Excessive structural rutting indicates that the existing materials lack sufficient stability to prevent rutting from reoccurring.
3. Existing stabilized base show signs of serious deterioration and requires a large amount of repair to provide a uniform support for the HMA overlay.
4. Existing granular base must be removed and replaced due to infiltration and contamination of clay fines or soils, or saturation of the granular base with water due to inadequate drainage.
5. Stripping in existing HMA layers dictate that those layers need to be removed and replaced.

In the MEPDG, the design procedure for HMA overlays of existing HMA surfaced pavements considers distresses developing in the overlay as well as the continuation of damage in the existing pavement structure. The overlay generally reduces the rate at which distresses develop in the existing pavement. The design procedure provides for the reflection of these distresses through the overlay layers when they become critical. The condition of the existing pavement also has a major effect on the development of damage in the new overlay layers.

HMA Overlay of Intact PCC Slabs

An HMA overlay is generally a feasible option for existing PCC and composite pavements provided reflection cracking is addressed during the overlay design.

Conditions under which an HMA overlay is not considered feasible include:
• The amount of deteriorated slab cracking and joint spalling is so great that complete removal and replacement of the existing PCC pavement is dictated.
• Significant deterioration of the PCC slab has occurred due to severe durability problems.

The design procedure presented in the MEPDG considers distresses developing in the overlay as well as the continuation of damage in the PCC. For existing JPCP, the joints, existing cracks, and any new cracks that develop during the overlay period are reflected through the HMA overlay using empirical reflection cracking models that can be adjusted to local conditions. A primary design consideration for HMA overlays of existing CRCP is to full-depth repair all working cracks and existing punchouts and then provide sufficient HMA overlay to increase the structural section to keep the cracks sufficiently tight and exhibit little loss of crack LTE over the design period. A sufficient HMA overlay is also needed to reduce the critical top of slab tensile stress and fatigue damage that leads to punchouts.

HMA Overlay of Fractured PCC Slabs

The design of an HMA overlay of fractured PCC slabs is very similar to the design of a new flexible pavement structure. The primary design consideration is the estimation of an appropriate elastic modulus for the fractured slab layer. One method to estimate the elastic modulus of the fractured PCC pavement condition is to backcalculate the modulus from deflection basins measured on previous projects (refer to Section 10). The three methods referred to as fractured PCC slabs are defined below:

• Rubblization – Fracturing the slab into pieces less than 12 inches reducing the slab to a high-strength granular base, and used on all types of PCC pavements with extensive deterioration (severe mid-slab cracks, faulting, spalling at cracks and joints, D-cracking, etc.).
• Crack and Seat – Fracturing the JPCP slabs into pieces typically one to three feet in size.
• Break and Seat – Fracturing the JRCP slabs to rupture the reinforcing steel across each crack or break its bond with the concrete.

13.2.7 HMA Overlays of Existing HMA Pavements, Including Semi-Rigid Pavements

HMA overlays of flexible and semi-rigid pavements may be used to restore surface profile or provide structural strength to the existing pavement. The trial overlay and pre-overlay treatments need to be selected considering the condition of the existing pavement and foundation, and future traffic levels. The HMA overlay may consist of up to four layers, including three asphalt layers and one layer of an unbound aggregate (sandwich section) or chemically stabilized layer.

The same distresses used for new flexible pavement designs are also used for rehabilitation designs of flexible and semi-rigid pavements (refer to subsection 5.3). For overlaid pavements, the distress analysis includes considerations of distresses (cracking and rutting) originating in the HMA overlay and the continuation of damage and rutting in the existing pavement layers. The total predicted distresses from the existing pavement layers and HMA overlay are used to predict the IRI values over time (refer to subsection 5.3).

Longitudinal and thermal cracking distresses in the HMA overlay are predicted at the same locations as for new pavement designs. Fatigue damage is evaluated at the bottom of the HMA layer of the overlay using the alligator fatigue cracking model. Reflection cracking is predicted by applying the empirical reflection cracking model to the cracking at the surface of the existing pavement.

The continuation of damage in the existing pavement depends on the composition of the existing pavement after accounting for the effect of pre-overlay treatments, such as milling or in-place recycling. For existing flexible and semi-rigid pavements where the HMA layers remain in place, fatigue damage will continue to develop in those layers in the existing structure using the damaged layer concept. All pavement responses used to predict continued fatigue damage in the existing HMA layers remaining in place are computed using the damaged modulus as determined from the pavement evaluation data using the methods discussed in Section 10. The pavement responses used to predict the fatigue damage of the HMA overlay use the undamaged modulus of that layer.

Plastic deformations in all HMA and unbound layers are included in predicting rutting for the rehabilitated pavement. As discussed in Section 5, rutting in the existing pavement layers will continue to accumulate but at a lower rate than for new materials due to the strain-hardening effect of past truck traffic and time.

13.2.8 HMA Overlays of Existing Intact PCC Pavements Including Composite Pavements (one or more HMA overlays of existing JPCP and CRCP)

HMA overlays may be used to remedy functional or structural deficiencies of all types of existing PCC pavements. It is important for the designer to consider several aspects, including the type of deterioration present, before determining the appropriate rehabilitation strategy to adopt.

Analysis Parameters Unique to HMA Overlay of JPCP and CRCP

Number of HMA Layers for Overlay

The HMA overlay may consist of a maximum of three layers. All mixture parameters normally required for HMA need to be specified for each of the layers.

Reflection Cracking of JPCP through HMA Overlay

The transverse joints and cracks of the underlying JPCP will reflect through the HMA overlay depending on several factors. The empirical reflection cracking models included in the MEPDG may be calibrated to local conditions prior to use of the software (refer to subsection 5.3). They have not been nationally calibrated and thus local calibration is even more important. Both the time in years to 50 percent of reflected joints and the rate of cracking may be adjusted depending on the HMA overlay thickness and local climatic conditions.

It is recommended that reflection cracking be considered outside of the MEPDG by means such as fabrics and grids or saw and sealing of the HMA overlay above joints. The MEPDG only considers reflection cracking treatments of fabrics through empirical relationships (refer to subsection 5.3).

For CRCP, there is no reflection cracking of transverse joints. The design procedures assumes that all medium and high severity punchouts will be repaired with full depth reinforced concrete repairs.

Impact of HMA Overlay on Fatigue Damage

The HMA overlay has a very significant effect on thermal gradients in the PCC slab. Even a thin HMA overlay greatly reduces the thermal gradients in the PCC slab, thereby reducing the amount of fatigue damage at both the top and bottom of the slab. This typically shows that even thin HMA overlays have a sufficient effect as to reduce future fatigue damage in the PCC slab. The extent of reflection cracking, however, is greatly affected by HMA thickness and this often becomes the most critical performance criteria for overlay design.

Estimate of Past Damage

For JPCP and CRCP subjected to an HMA overlay, an estimate of past fatigue damage accumulated since opening to traffic is required. This estimate of past damage is used (along with future damage) to predict future slab cracking and punchouts. For JPCP, the past damage is estimated from the total of the percent of slabs containing transverse cracking (all severities) plus the percentage of slabs that were replaced on the project.

Required inputs for determining past fatigue damage are as follows:

1. Before pre-overlay repair, percent slabs with transverse cracks plus percent previously repaired/replaced slabs. This represents the total percent slabs that have cracked transversely prior to any restoration work.
2. After pre-overlay repair, total percent repaired/replaced slabs (note, the difference between [2] and [1] is the percent of slabs that are still cracked just prior to HMA overlay).

Repairs and replacement refers to full-depth repair and slab replacement of slabs with transverse cracks. The percentage of previously repaired and replaced slabs is added to the existing percent of transverse cracked slabs to establish past fatigue damage caused since opening to traffic. This is done using the MEPDG national calibrated curve for fatigue damage versus slab cracking.

Future slab cracking is then computed over the design period as fatigue damage increases month by month.

Example: A survey of the existing pavement shows 6 percent slabs with transverse cracks and 4 percent slabs that have been replaced. It is assumed that all replaced slabs had transverse cracks. During pre-overlay repair, 5 percent of the transversely cracked slabs were replaced leaving 1 percent still cracked. Inputs to the MEPDG are as follows:

- Six percent slabs with transverse cracks plus four percent previously replaced slabs equals ten percent.
- After pre-overlay repair, total percent replaced slabs equals nine percent. Note that the percent of slabs still cracked, prior to overlay, is therefore 10 – 9 = 1 percent.

For CRCP, the same approach is used. The number of existing punchouts per mile (medium and high severity only) is added to the number of repairs of punchouts per mile. This total punchouts per mile is a required input to establish past fatigue damage caused by repeated axle loads since opening to traffic. This is done using the MEPDG global calibrated curve for fatigue damage versus punchouts. An estimate of future punchouts is then computed over the design period as fatigue damage increases month by month.

Dynamic Modulus of Subgrade Reaction (Dynamic k-value)

The subgrade modulus may be characterized in the following ways for PCC rehabilitation:

1. Provide resilient modulus inputs of the existing unbound sublayers including the subgrade soil similar to new design. The MEPDG software will back calculate an effective single dynamic modulus of subgrade reaction (k-value) for each month of the design analysis period for these layers. The effective k-value, therefore, essentially represents the compressibility of underlying layers (i.e., unbound base, subbase, and subgrade layers) upon which the upper bound layers and existing HMA or PCC layer is constructed. These monthly values will be used in design of the rehabilitation alternative.

2. Measure the top of slab deflections with an FWD and conduct a back calculation process to establish the mean k-value during a given month. Enter this mean value and the month of testing into the MEPDG. This entered k-value will remain for that month throughout the analysis period, but the k-value for other months will vary according to moisture movement and frost depth in the pavement.

Modulus of Elasticity of Existing JPCP or CRCP Slab

The modulus of elasticity of the existing slab is that existing at the point of time of rehabilitation. This value will be higher than the 28-day modulus of course. It is estimated using procedures given in Table 32. This modulus is the intact slab value. It is not a reduced value due to slab cracking as is done for unbonded PCC overlays. This layer is the primary load carrying layer of the overlaid composite pavement structure.

The amount of cracking in the existing slab is accounted for in two ways:
1. Percent of slabs cracked are determined and used to compute past damage which will affect the future cracking of the existing slab.
2. Percent of slabs cracked are considered to reflect through the HMA overlay in a predicted rate thereby affecting the performance through limiting criteria (percent area of traffic lane) and through impacting the IRI.

Table 32. Data Required for Characterizing Existing PCC Slab Static Elastic Modulus for HMA Overlay Design

Input Data	Hierarchical Level		
	1	2	3
Existing PCC slab design static elastic modulus	The existing PCC slab static elastic modulus $E_{BASE/DESIGN}$ for the existing age of the concrete is obtained from (1) coring the intact slab and laboratory testing for elastic modulus or (2) by back calculation (using FWD deflection data from intact slab and layer thicknesses) and multiplying by 0.8 to convert from dynamic to static modulus.	$E_{BASE/DESIGN}$ obtained from coring and testing for compressive strength. The compressive strength value is converted into elastic modulus as outlined in Part 2, Chapter 2. The design elastic modulus is obtained as described for level 1	$E_{BASE/DESIGN}$ estimated from historical agency 28-day values which are extrapolated to the date of construction.

Trial Rehabilitation with HMA Overlays of JPCP and CRCP

A range HMA overlay thickness may be run and the performance projected by the MEPDG. The ability of the overlay to satisfy the performance criteria is then determined. Some general guidelines on criteria are given in Table 33. Note that for some overlay/PCC slab design situations, the structural analysis will show that only a thin HMA overlay is needed (structural adequacy is acceptable). The addition of a relatively thin HMA overlay changes the thermal gradients so much that fatigue damage becomes minimal. In this case, the designer may choose a minimum overlay thickness that can meet all other criteria including (1) the smoothness specification, (2) can be placed and compacted properly, and (3) has adequate thickness to remain in place over the design life. Most highway agencies specify minimum thicknesses of HMA overlays for just this purpose.

Design Modifications to Reduce Distress for HMA Overlays

Trial designs with excessive amounts of predicted distress/smoothness need to be modified to reduce predicted distress/smoothness to tolerable values (within the desired reliability level). Some of the most effective ways of accomplishing this are listed in Table 34.

Table 33. Recommendations for Performance Criteria for HMA Overlays of JPCP and CRCP

Distress Type	Recommended Modifications to Design
Rutting in HMA	Criteria for rutting should be selected similar to new or reconstructed pavement design. This rutting is only in the HMA overlay.
Transverse cracking in JPCP existing slab	The placement of an HMA overlay will significantly reduce the amount of future fatigue transverse cracking in the JPCP slab and this is not normally a problem. A typical limit of 10 percent (all severities) appears to be reasonable in that exceeding this value indicates that the overlaid JPCP is experiencing significant load fatigue damage and a structural improvement is needed.
Punchouts in CRCP existing slab	The placement of an HMA overlay will significantly reduce the amount of future punchout development in CRCP and this is not normally a problem. A typical limit of 5 to 10 per mile (medium and high severity) appears to be reasonable in that exceeding this value indicates that the overlaid CRCP is experiencing significant load fatigue damage and a structural improvement is needed.
Reflection cracking from existing JPCP or CRCP slab	The extent of reflection cracking is dependent on any special reflection cracking treatments that the designer may have specified. Thus, if the designer feels that this treatment will reduce or eliminate reflection cracking from the existing slab then this criterion may be ignored. The MEPDG predicted reflection cracking is from transverse joints and transverse cracks in JPCP but it is converted into a percent area of traffic lane. A maximum recommended value of 1.0 % area is recommended for reflection cracking of all severities (note: this represents 100 transverse cracks per mile or one crack every 53 ft. which creates significant roughness).
Smoothness	The limiting IRI should be set similar to that of new or reconstructed pavements. The only exception to this would be when the existing pavement exhibits a large amount of settlements or heaves that would make it difficult to level out. If this is the case, a level up layer should be placed first and then the designed overlay placed uniformly on top.

13.2.9 HMA Overlay of Fractured PCC Pavements

The objective of rubblizing PCC slabs is to eliminate reflection cracking in an HMA overlay by destroying the integrity of the existing slab. This objective is achieved by fracturing the PCC slab in place into fragments of nominal 3 to 8-inch size or less, while retaining good interlock between the fractured particles. The rubblized layer acts as an interlocked unbound layer, reducing the existing PCC to a material comparable to a high quality aggregate base course.

The rubblization process is applicable to JPCP, JRCP, and CRCP. Reinforcing steel in JRCP and CRCP must become debonded from the concrete to be successful and meet the performance expectations. The purpose of this subsection is to provide guidance on the use of rubblization of PCC pavements to maximize the performance of this rehabilitation option.

Table 34. Recommendations for Modifying Trial Design to Reduce Distress/Smoothness for HMA Overlays of JPCP and CRCP

Distress Type	Recommended Modifications to Design
Rutting in HMA	Modify mixture properties. See recommendations under subsection 13.2.
Transverse cracking in JPCP existing slab	Repair more of the existing slabs that were cracked prior to overlay placement. Increase HMA overlay thickness.
Crack width CRCP	It is desirable to have crack width < 0.020 in over the design period. However, there is not much the designer can do to control this parameter.
Crack LTE CRCP	It is desirable to have crack load transfer efficiency (LTE) greater than 95% over the design period. This will prevent any reflection cracking or punchouts from occurring. The only design feature that will affect this parameter is overlay thickness.
Punchouts in CRCP existing slab	Repair all of the existing punchouts prior to overlay placement. Increase HMA overlay thickness.
Reflection cracking from existing JPCP or CRCP slab	Apply an effective reflection crack control treatment such as saw and seal the HMA overlay over transverse joints. Increase HMA overlay thickness.
Smoothness	Build smoother pavements initially through more stringent specifications. Reduce predicted slab cracking and punchouts.

Project Selection Criteria for Rubblization

Rubblization is an effective reconstruction technique in many situations, but inadequate project scoping may lead to constructability and performance problems. Proper project scoping should follow the following steps, which are illustrated in flow chart form in Figures 32 through 35.

1. Identify roadway site features and conditions that may have a detrimental effect on constructability and performance of rubblized PCC pavements (Figure 32). In general, rubblizing PCC pavements may be considered a viable option when there is no rigid layer within 3 feet, no water table within 5 feet, and no old utility lines within 5 feet of the PCC layer. When these conditions exist, other rehabilitation strategies maybe more appropriate. Rubblization may still be considered for use even under these conditions, but may require more detailed investigations as to the uniformity of the rubblized PCC slabs. In other words, rubblization is not excluded under these conditions, but can be considered with caution.

2. Determine the condition and distresses of the existing PCC pavement (Figures 33 and 34). Rubblization is considered a viable option when the PCC pavement has no remaining life (i.e., when there is extensive structural distress along the project). If horizontal cracks or delamination between different PCC layers has occurred along the project site, however, other rehabilitation options maybe more cost-effective and should be considered.

3. Determine the foundation support conditions and strength (Figure 35). A foundation investigation may be performed using the FWD and DCP tests. The FWD deflection basin and

DCP data are used to determine the elastic modulus of the foundation layers. The frequency of these tests needs to be determined to identify any weak areas along the project. The project engineer may identify areas where the support modulus for the PCC slabs is less than 5,000 psi (34 MPa), based on laboratory measured resilient modulus. A backcalculated modulus value from deflection basin data of 10,000 psi beneath a PCC pavement corresponds to a laboratory measured resilient modulus value of approximately 5,000 psi. Foundation modulus values, backcalculated from deflection basins, less then 10,000 psi may have a detrimental effect on the rubblization process. Rubblization of PCC slabs that are resting directly on a fine grained soil subgrade have experienced significant problems in the vibrating head settling into the fractured slab and into the subgrade.

Design Features for Rubblization PCC Pavements

Installation of Edge Drains

Rubblizing the PCC slabs results in a layer with significant permeability. Any water infiltrating the rubblized layer should be quickly removed through the use of edge drains, especially for pavements supported by fine-grained soils with low permeability. Edge drains are not required in areas with coarse-grained soils that have high permeability. Edge drains may be used in all rubblized projects to drain any saturated foundation layer. These drains may be placed continuously or intermittently along the project. Their use and location could be based on engineering judgment to remove water from the pavement structure. When used, edge drains need to be installed prior to the rubblization process to ensure that there is sufficient time to allow the subbase and subgrade to drain and dry out (usually 2 weeks before rubblization starts).

Leveling Courses

A leveling course is needed to restore the grade and make profile corrections to the surface of the rubblized PCC layer. Leveling course material may consist of crushed aggregate, milled or recycled asphalt pavement (RAP), or a fine-graded HMA mixture that is workable. A 2 to 4-inch leveling course should be included in the design to fill in depressions or low spots along the rubblized surface. This leveling course also acts as a cushion layer for the HMA overlay. If a workable, fine-graded HMA mixture (a HMA mixture with higher asphalt content) is used, the designer could ensure that there is sufficient cover so that rutting does not become a problem within that workable layer.

In many cases, the use of crushed aggregate base materials as the leveling course cannot be used because of clearance or height restrictions at bridges and other overhead structures. HMA leveling courses with specific fracture resistant properties are more beneficial to long term pavement performance. These mixtures could be compacted to in-place air voids less than 7 percent. In either case, leveling courses could be accounted for in the structural design, but not for the sole purpose of reducing the HMA overlay thickness. When HMA leveling courses are used, sufficient HMA overlay thickness needs to be placed to ensure that the heavier trucks will not cause rutting or any lateral distortions in the leveling course.

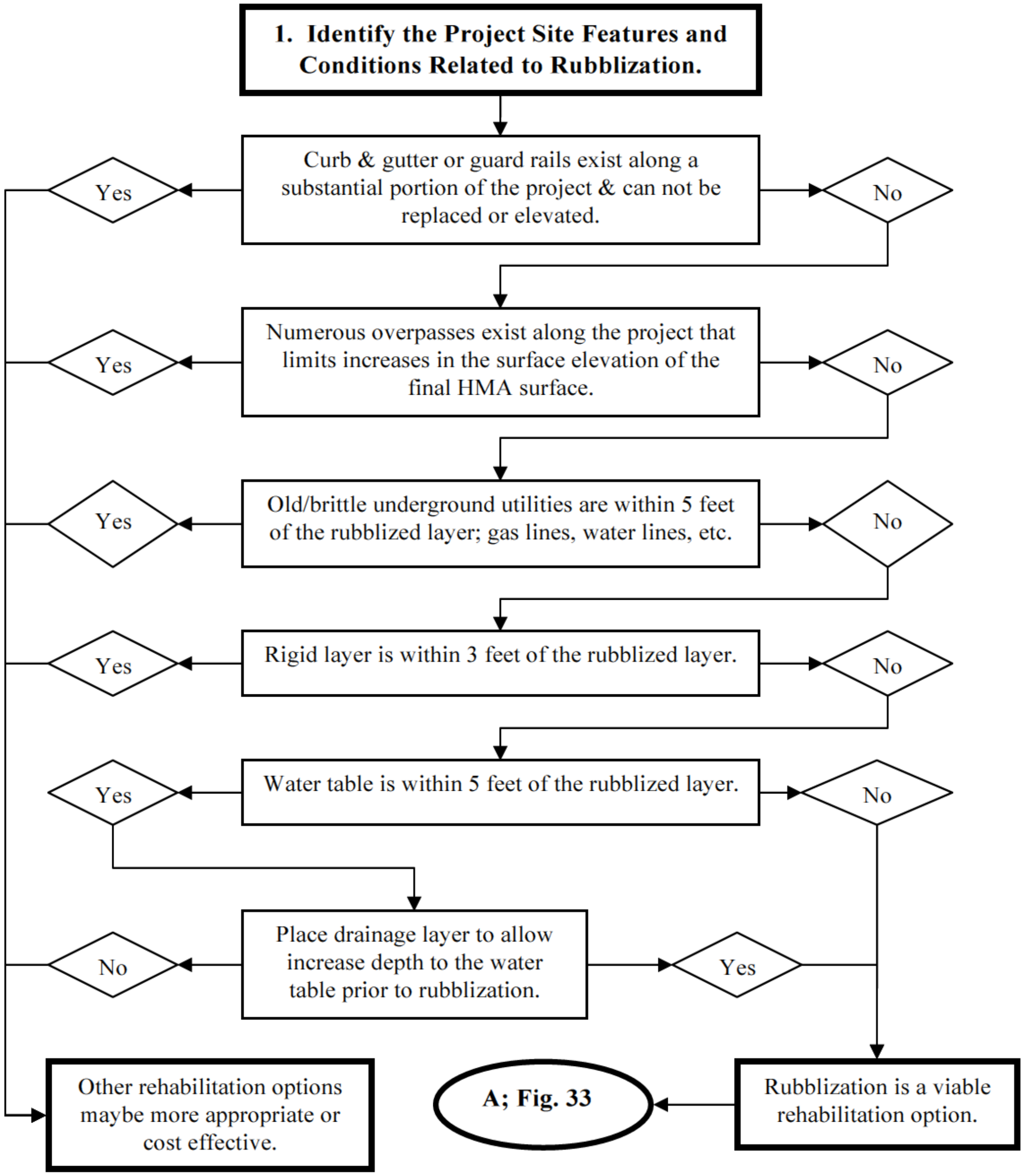


Figure 32. Site Features Conducive to the Selection of the Rubblization Process for Rehabilitating PCC Pavements

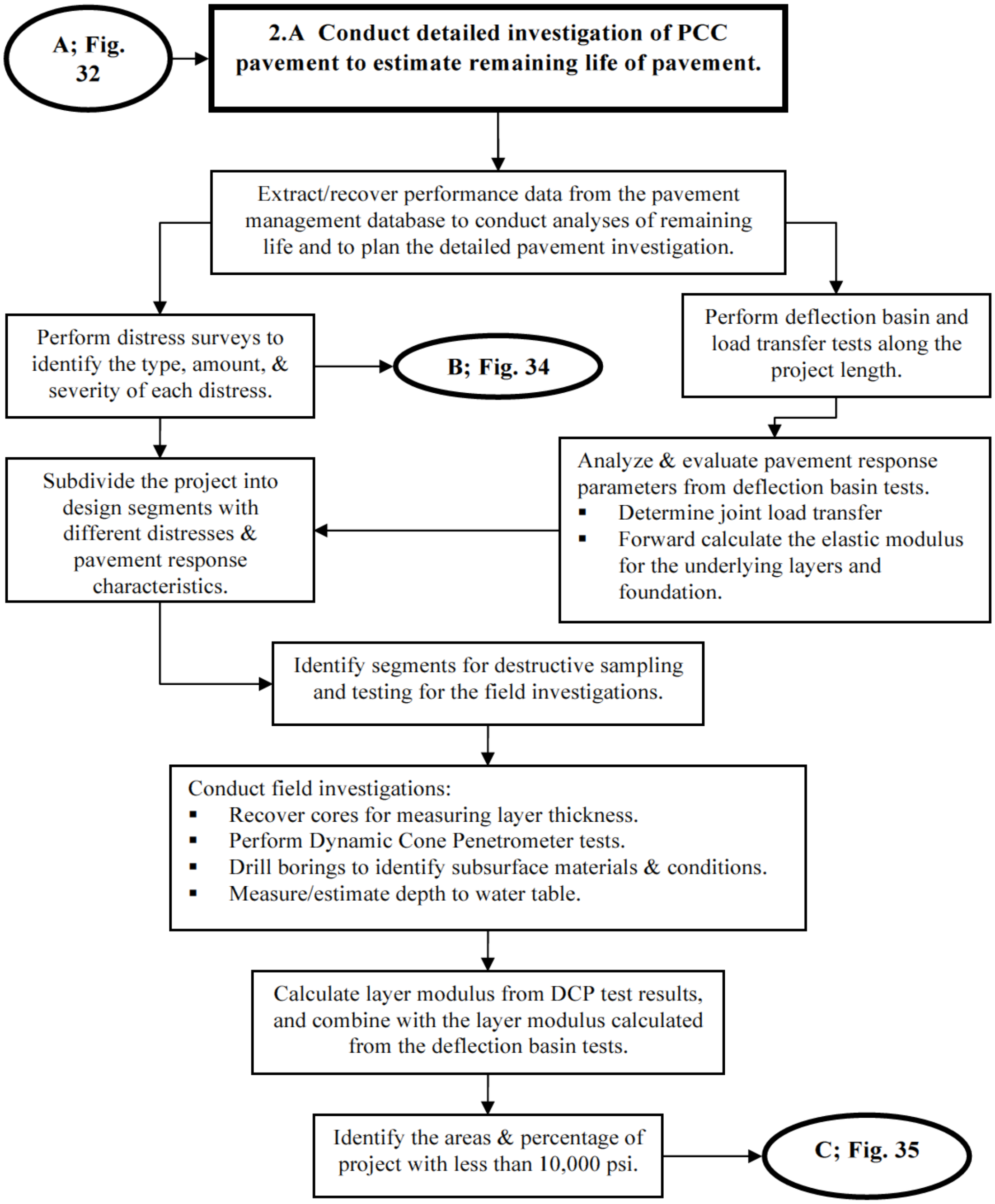


Figure 33. Recommendations for a Detailed Investigation of the PCC Pavement to Estimate Remaining Life and Identifying Site Features and Conditions Conducive to the Rubblization Process

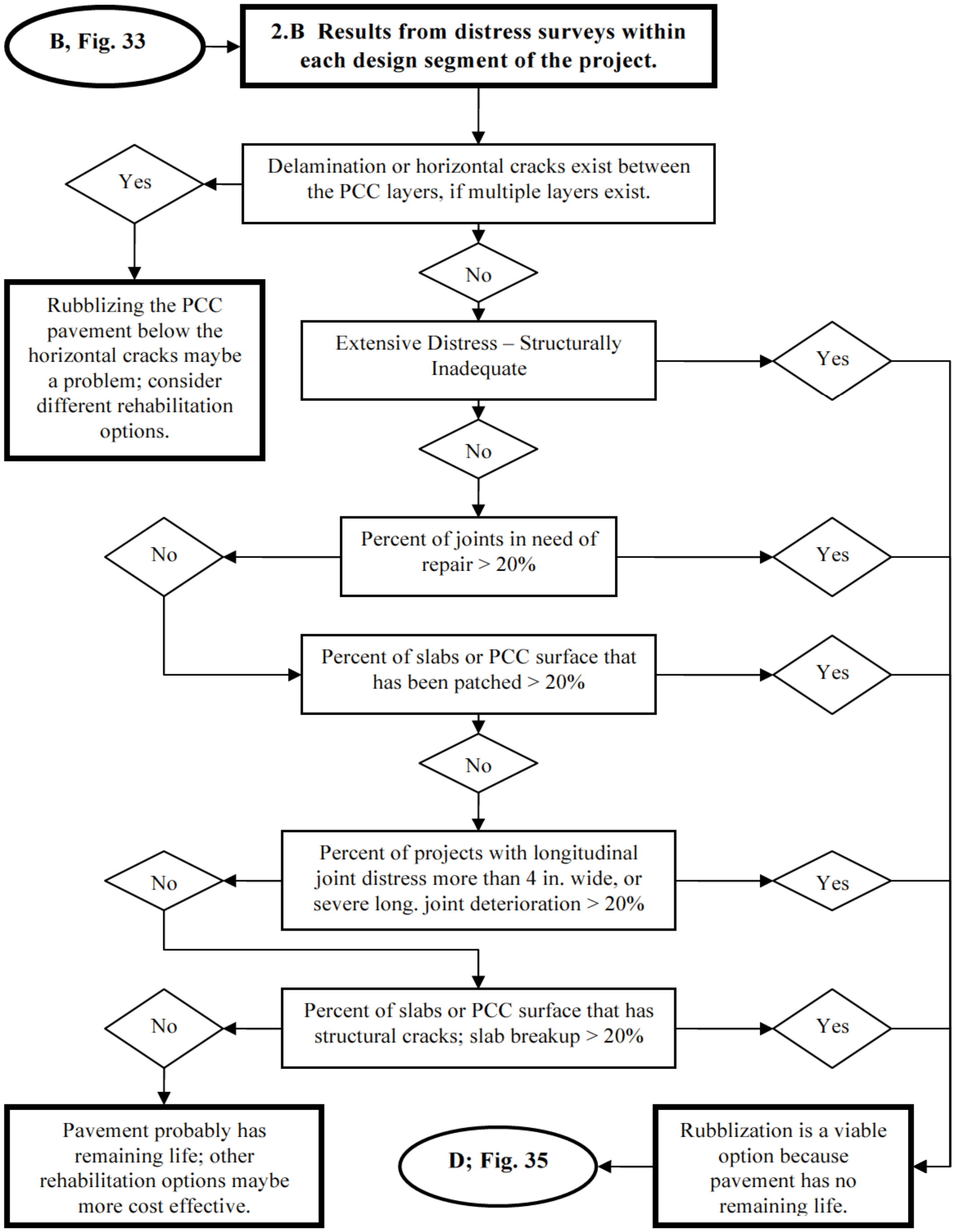


Figure 34. Evaluate Surface Condition and Distress Severities on Selection of Rubblization Option

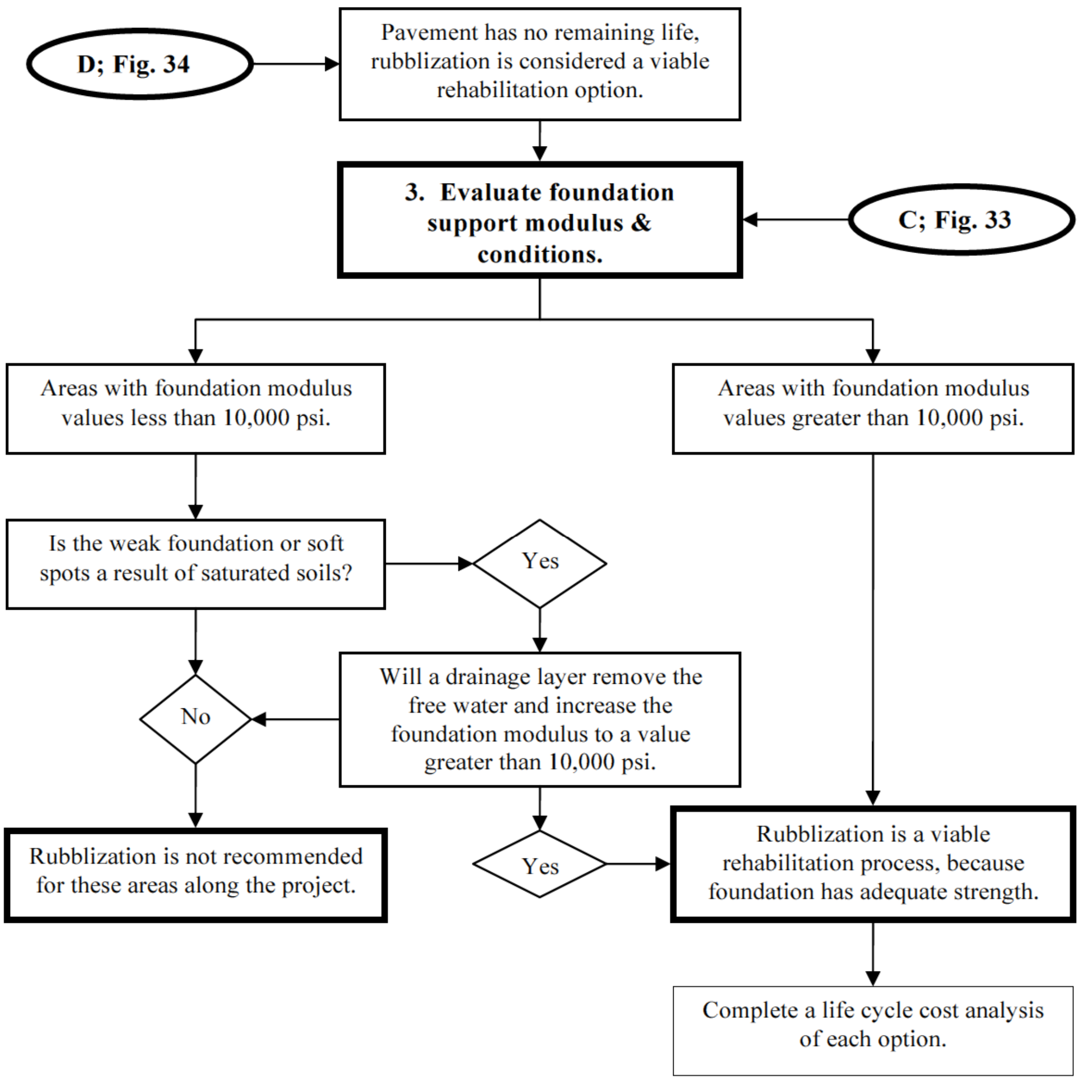


Figure 35. Foundation Support Conditions Related to the Selection of the Rubblization Process

Each design situation and material needs to be evaluated to determine the rehabilitation option that will provide the better long-term performance, while meeting the project requirements. An HMA leveling course could be considered for use on projects where the rubblized pavement must carry traffic temporarily until additional HMA lifts are placed. The thickness of the leveling course and its properties need to be determined to carry the expected traffic during construction.

Minimum HMA Overlay Thickness Above Rubblized PCC Slabs

The minimum HMA overlay thickness placed over rubblized PCC layers from a constructability standpoint is 4 inches. This minimum thickness excludes any HMA leveling course mixture that is placed to correct surface profiles. The performance of a pavement structure is dependent upon the interaction between pavement response and strength of the different layers. Wheel loads induce stresses and strains in each layer, which may result in deformation and cracking of the HMA layer.

The rehabilitation design procedure has to determine the HMA overlay thickness that satisfies both constructability and structural requirements of the rubblized pavement. M-E based design procedures are being used by many agencies, but primarily for forensic studies and post-construction evaluation of the pavement structure. The HMA overlay fatigue considerations control the overlay thickness requirements for rubblized pavement using the M-E based procedures.

Table 23 in Section 11 provided a range of equivalent elastic modulus values that may be used. The equivalent modulus of the rubblized layer is dependent on the agency's specifications for that layer. An elastic modulus value of 65,000 psi (450 MPa) for the rubblized layer is recommended for use in HMA overlay design. This value is less than the value recommended in the NAPA Information Series 117, but is based on back calculation of layer modulus from deflection basin data and performance analyses of rubblized pavements built in around the U.S.

For thick JPCP exceeding 10 inches and JRCP, a large modulus gradient between the surface and bottom of the rubblized layer typically exists because the fractured particle size varies from top to bottom. The designer can subdivide the rubblized layer into an upper and lower portion of the JPCP or above and below the reinforcement of JRCP or just use an average value throughout the fractured slab. Without deflection basin data, it is suggested that an average or equivalent value of 65,000 psi be used for the rubblized layer.

Darwin-ME Material Inputs

Table 19. Major Material Types for the MEPDG

<table>
<tr>
<td>
<u>Asphalt Materials</u>

Stone Matrix Asphalt (SMA)
Hot Mix Asphalt (HMA)

Dense Graded
Open Graded Asphalt
Asphalt Stabilized Base Mixes
Sand Asphalt Mixtures

Cold Mix Asphalt

Central Plant Processed
Cold In-Place Recycling

<u>PCC Materials</u>

Intact Slabs – PCC

High Strength Mixes
Lean Concrete Mixes

Fractured Slabs

Crack/Seat
Break/Seat
Rubblized

<u>Chemically Stabilized Materials</u>

Cement Stabilized Aggregate
Soil Cement
Lime Cement Fly Ash
Lime Fly Ash
Lime Stabilized Soils
Open graded Cement Stabilized Aggregate

</td>
<td>
<u>Non-Stabilized Granular Base/Subbase</u>

Granular Base/Subbase
Sandy Subbase
Cold Recycled Asphalt Mix (used as aggregate)

RAP (includes millings)
Pulverized In-Place

Full Depth Reclamation (In-Place (Cold Recycled Asphalt Pavement; (HMA plus aggregate base/subbase)

<u>Subgrade Soils</u>

Gravelly Soils (A-1;A-2)
Sandy Soils

Loose Sands (A-3)
Dense Sands (A-3)
Silty Sands (A-2-4;A-2-5)
Clayey Sands (A-2-6; A-2-7)

Silty Soils (A-4;A-5)
Clayey Soils, Low Plasticity Clays (A-6)

Dry-Hard
Moist Stiff
Wet/Sat-Soft

Clayey Soils, High Plasticity Clays (A-7)

Dry-Hard
Moist Stiff
Wet/Sat-Soft

<u>Bedrock</u>

Solid, Massive and Continuous
Highly Fractured, Weathered

</td>
</tr>
</table>

9.5.4 Chemically Stabilized Layer

The required inputs for a chemically stabilized layer can be broadly classified as general, strength, and thermal properties. Note that the strength properties required by DARWin-ME are different for flexible and rigid pavements.

Refer to Section 5.5.2 **Chemically Stabilized Layer**

Links to Relevant Section in AASHTO Manual of Practice

Refer to Section 11.4 Chemically Stabilized Materials; Including Lean Concrete and Cement Treated Base Layer.

9.5.5 Non-Stabilized Layer

Non-stabilized materials include AASHTO soil classes A-1 through A-3, as well as those commonly defined in practice as crushed stone, crushed gravel, river gravel, permeable aggregate, and cold recycled asphalt material (includes millings and in-place pulverized material).

Refer to Section 5.5.3 **Non-Stabilized Layer**

Links to Relevant Section in AASHTO Manual of Practice

Refer to Section 11.5 Unbound Aggregate Base Materials and Engineered Embankments.

9.5.6 Subgrade

Subgrade materials include soil classes A-1 through A-7-6 defined in accordance with the AASHTO soil classification system. Inputs required for subgrade materials are same as include physical and engineering properties such as conductivity, specific gravity, soil-water characteristic classification properties, and the resilient modulus.

Refer to Section 5.5.4 Subgrade **Layer**

9.5.7 Bedrock

A bedrock layer, if present under an alignment, could have a significant impact on the pavement's mechanistic responses and therefore need to be fully accounted for in design. This is especially true if backcalculation of layer moduli is adopted in rehabilitation design to characterize pavement materials. While the precise measure of the stiffness is seldom, if ever, warranted, any bedrock layer must be incorporated into the analysis.

Refer to Section 5.5.5 **Bedrock Layer**

9.6 AC Layer Properties

This screen allows you to define other inputs pertinent to flexible pavement design.

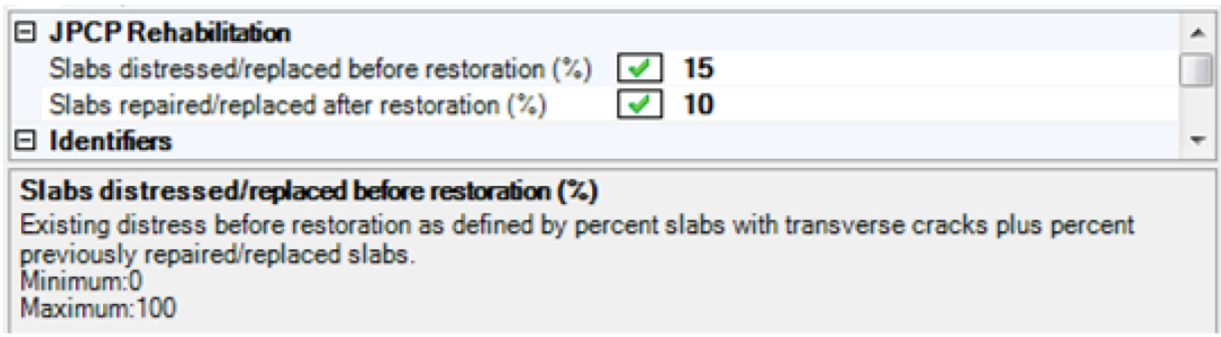
Refer to Section 8.6 AC Layer Properties

9.7 JPCP Rehabilitation

This screen allows you to provide the required DARWin-ME inputs indicative of the existing level of distresses in a restoration or rehabilitation design. The inputs enable DARWin-ME to estimate damage based on the condition of the existing pavement. The extent of repairs undertaken at the time of rehabilitation is used to adjust future performance predictions as needed.

Way to Access this Interface

Open a rehabilitation project with a JPCP layer and select JPCP Rehabilitation in the Layer control.



JPCP Rehabilitation

Ways to Enter Inputs

There are three methods for entering data for the PCC layer:

- Manual entry
- Import from file
- Import from database

Populating the Inputs in this Interface

JPCP Rehabilitation

Slabs distressed/replaced before restoration (%): This control allows you to define the percentage of cracking in slabs before the restoration or rehabilitation. This is essentially the total percentage cracking in the slabs and includes all cracks that were previously.

Slabs repaired/replaced after restoration (%): This option allows you to define the percentage of slabs that were repaired during the restoration or rehabilitation process. The difference between the slabs distressed and slabs replaced is the percentage of slabs that at are still cracked after restoration.

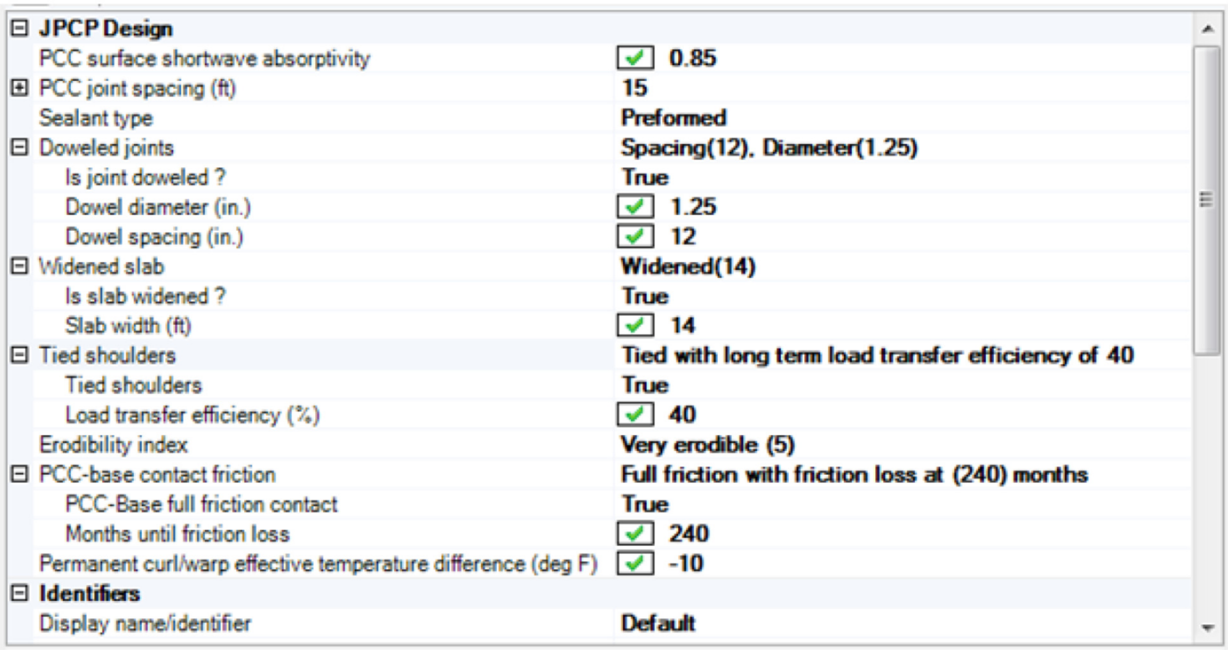
9.8 JPCP Design Properties

JPCP design features and construction practices influence long-term performance. The common design features that are considered by DARWin-ME include widened PCC slabs, joint spacing, shoulder type (tied vs. untied PCC or asphalt concrete), presence and size of dowel bars used for transverse join load transfer, dowel bar spacing, base type and erodibility, (chemically stabilized, asphalt stabilized, and non stabilized aggregate).

Construction practices include PCC curing method (curing compound vs. wet curing), permanent curl/warp effective temperature difference in the PCC, PCC/base layer friction loss age, initial smoothness, and so on. Some of these design features and construction practices have been described in previous sections.

Way to Access this Interface

Open a project with JPCP layer and select JPCP Design Property in the Layer control.



JPCP Design Properties

Ways to Enter Inputs

There are three methods for entering data for the Stabilized (Flexible) layer:

- Manual entry
- Import from file
- Import from database

Populating the Inputs in this Interface

JPCP Design

PCC surface shortwave absorptivity: This control allows you to define the fraction of solar energy (sunshine) at the PCC surface that is absorbed by the PCC. DARWin- ME presents a default value of 0.85. You can override this default.

PCC joint spacing (ft): This control allows you to define whether transverse joints of the trial design are uniformly or randomly spaced.

Is joint spacing random? This control allows you to define whether transverse joints are uniformly spaced or randomly spaced. DARWin-ME allows for the following options:

True: Selecting this option implies transverse joint are randomly spaced.

- ***Spacing of joint 1:*** This option allows you to define the length of the first PCC slab.
- ***Spacing of joint 2:*** This option allows you to define the length of the second PCC slab.
- ***Spacing of joint 3:*** This option allows you to define the length of the third PCC slab.
- ***Spacing of joint 4:*** This option allows you to define the length of the fourth PCC slab.

False: Selecting this option implies transverse joint are uniformly spaced.

Joint spacing: This option allows you to define average length of all the PCC slabs.

Sealant type: This control allows you to select the sealant type applied at the transverse joints. DARWin-ME allows you to choose from the following sealant types:

preformed, liquid, and silicone.

> **Note** Sealant type is an input to the JPCP transverse joint spalling model. Spalling is a key input for JPCP smoothness prediction.

Doweled joints: This control allows you to define transverse joint load transfer mechanism.

Is joint doweled?: This control allows you to select whether transverse joint load transfer mechanism is through the dowel bars:

True: Selecting this option implies transverse joint load transfer mechanism is through dowel bars.

False: Selecting this option implies transverse joint load transfer mechanism is through aggregate interlock only.

Dowel diameter (in.): This option allows you to define the diameter of the dowel bars used for load transfer across the transverse joint. A value of zero implies there are no dowel bars and load transfer is primarily through aggregate interlock.

Dowel spacing (in.): This option allows you to define the center-to-center distance between adjacent dowel bars if used for load transfer across transverse joints.

Widened slab: This control displays if the PCC slabs are widened. Note that the typical JPCP PCC slab width is 12-ft.

Is slab widened? This control allows you to select whether or not the JPCP PCC slab width is widened. DARWin-ME allows you to select one of the following options:

True: Selecting this option implies that the PCC slabs are widened (slab width is greater than 12-ft, typically 13- or 14-ft)

False: Selecting this option implies that the PCC slabs are not widened (slab width is 12-ft)

Slab width (ft.): This option allows you to define the slab width. Note slab width is defined only when widened PCC slab option is applied. Otherwise, DARWin-ME assumes a standard 12-ft slab width.

Tied shoulders: This control displays if tied PCC shoulders are used.

Tied shoulders: This control allows you to select whether or not to use tied PCC shoulders. DARWin-ME allows you to select one of the following options:

True: Selecting this option implies the use of tied PCC shoulders.
False: Selecting this option implies the use of other shoulder types such as PCC (without tie bars), asphalt concrete, or gravel shoulders.

Load transfer efficiency (%): This option allows you to define the long-term or terminal deflection LTE at the lane (PCC outer lane slab) to PCC shoulder longitudinal joint. DARWin-ME provides a default value of 40 percent. Typical long-term LTE are 50 to 70 percent for monolithically constructed tied PCC shoulder and 30 to 50 percent for separately constructed tied PCC shoulder.

Erodibility index: This control allows you to select the resistance of the base course to erosion, using an index on a scale of 1 to 5. Material erosion resistance is determined both by its strength and durability. DARWin-ME allows you to select one of the following options:

Extremely erosion resistant: Select an erodibility index of 1 for extremely erosion resistant base materials such as asphalt concrete and lean concrete materials.

Very erosion resistant: Select an erodibility index of 2 for very erosion resistant base materials such as asphalt treated and cement treated materials.

Erosion resistant: Select an erodibility index of 3 for erosion resistant base materials.

Fairly erodible: Select an erodibility index of 4 for fairly erodible base materials such as weakly stabilized aggregate materials and subgrade soils.

Very erodible: Select an erodibility index of 5 for very erodible base materials such as unstabilized subgrade soils.

PCC-base contact friction: The interface between the underlying base and PCC slab is modeled with or without full friction for JPCP design. DARWin-ME allows you to (1) determine whether or not the PCC slab/base interface has full friction at construction and (2) how long full friction will be available at the interface if present after construction. This control displays options available for modeling PCC slab/base interface condition.

PCC-Base full-friction contact: This option allows you to select whether or not there is full friction at the PCC slab/base interface after construction. DARWin-ME allows you to select one of the following options:

True: Selecting this option implies there is full friction at the PCC slab/base interface after friction.
False: selecting this option implies there is relatively little to no friction at the PCC slab/base interface after friction.

Months until friction loss: This option allows you to define the number of months after which full friction at the PCC slab/base interface is lost. The AASHTO Manual of Practice presents recommendations according to base material type. DARWin-ME allows for a range of 0 to 1200 months.

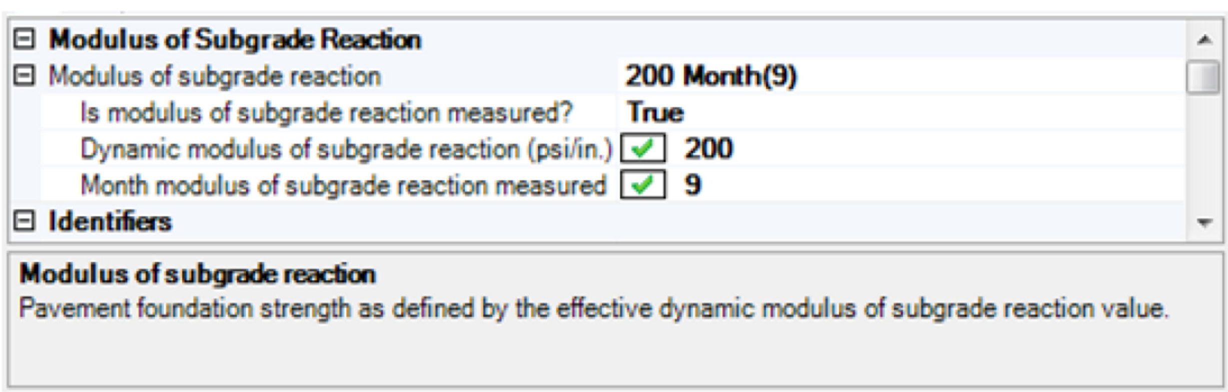
Permanent curl/warp effective temperature difference (deg F): This input describes the combined effect of (1) PCC built-in temperature gradient at time of set, (2) effective gradient of moisture warping in the PCC (dry on top and wet on bottom), (3) long term creep of the PCC slab, and (4) settlement of the PCC into the base. It is defined in terms of equivalent temperature difference. DARWin-ME presents the default value of -10 °F which was established as optimum to minimize error between measured and predicted cracking and measured and predicted faulting during the national calibration.

9.9 Foundation Support

Modulus of subgrade reaction, or k-value, is a soil support parameter. The static k-value is determined from the plate load test and is a measure of the pressure applied on the soil for a unit deformation monitored using a standard size plate. Dynamic k-value is determined through deflection testing and backcalculation. DARWin-ME requires dynamic k-value.

Way to Access this Interface

Open a rehabilitation project with a foundation layer and select Foundation Support in the Layer control.



Foundation Support

Ways to Enter Inputs

There are three methods for entering data for the sandwiched granular layer:

- Manual entry
- Import from file
- Import from database

Modulus of Subgrade Reaction

Modulus of subgrade reaction: This control displays whether the modulus of subgrade reaction is calculated internally by DARWin-ME or user-defined.

Is modulus of subgrade reaction measure?: Select True to define the modulus of subgrade reaction (k-value) and the month of the year it was measured. Select False to allow DARWin-ME to compute this value.

Dynamic modulus of subgrade reaction (psi/in.): This control allows you to define the dynamic k-value.

Month modulus of subgrade reaction measured: This control allows you to define the month of the year when the dynamic k-value was measured.

9.10 Backcalculation

The Backcalculation option allows you to use backcalculated layer moduli of existing pavement layers and subgrade in overlay designs. The moduli typically are obtained using nondestructive deflection basin tests and standard backcalculation procedures. The backcalculated moduli closely match the actual in situ moduli of the existing pavement layers and subgrade. The Backcalculation option allows you to use backcalculated layer moduli of existing pavement layers and subgrade in overlay designs. The moduli typically are obtained using nondestructive deflection basin tests and standard backcalculation procedures. The backcalculated moduli closely match the actual in situ moduli of the existing pavement layers and subgrade.

The most widely used deflection testing device is the falling weight deflectometer (FWD). Backcalculated layer moduli obtained from FWD testing need to be adjusted to laboratory conditions for use in DARWin-ME. The adjustment to laboratory conditions is discussed in the AASHTO Manual of Practice.

Refer to Section 8.7 **Backcalculation**

11 Asphalt Concrete Overlay Design of Fractured JPCP

An Asphalt Concrete Overlay of fractured JPCP is a rehabilitation option considered in DARWin-ME. For this AC overlay design type, the pre-overlay activity is fracturing existing PCC slabs. The objective of fracturing existing PCC slabs prior to AC overlay placement is to eliminate reflection of distresses such as cracking in the existing PCC into the AC overlay. This is done by fracturing the PCC slab in place into small fragments, while retaining good interlock between the fractured particles. In effect the integrity of the existing PCC slab is destroyed and replaced with a strong high-quality interlocked non-stabilized material.

DARWin-ME considers the effect of pre-overlay fracturing of the existing PCC slab through the selection of appropriate fractured PCC properties. DARWin-ME can be used to design and evaluate AC Overlays of Fractured JPCP.

AC Overlay Design of Fractured JPCP Overview

Design Inputs: The topics in this section explain the options DARWin-ME offers for creating and testing AC Overlay Design of Fractured JPCP designs:

General Information: This section allows you to select the basic parameters of an AC Overlay Design of Fractured JPCP design.

Performance Criteria: This section provides a high-level look at the criteria DARWin-ME uses to analyze an AC Overlay Design of Fractured JPCP design.

Traffic: This section provides details on using DARWin-ME's tables and data to examine the effects of traffic loading on pavement design and lifespan.

Climate: This section provides details on working with data from climate data files and then established weather stations for an area and using that data to analyze the effects of climate variables on pavement response and performance.

Pavement Structure Definition and Materials**:** This section explains adding additional layers to a pavement, editing the parameters of those layers, and working with the structure of the pavement as a whole.

Run Analysis: This section details the various analyses DARWin-ME will run to determine if a pavement design is valid.

Reports: This section details the variety of reports DARWin-ME can create on all facets of a project's design.

Structural Response: DARWin-ME's structural model analyzes pavement structure, accounts for discontinuities in that structure, and analyzes the effects of environment and traffic on that structure.

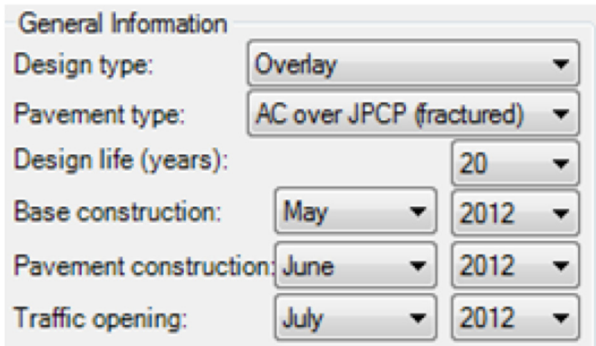
11.1 General Design Inputs

An AC Overlay of Fractured JPCP pavement is a fractured JPCP pavement rehabilitated with a new AC surface.

The following options allow you to establish the basic parameters of your project.

Way to Access this Interface

1. Open a new pavement project. The *General Information* area appears at the top-left corner of the Project Tab.



General Information

Design type: Select *Overlay*

Pavement type: Select *AC over JPCP (fractured)*

In addition to selecting Design and Pavement type, select values for the following basic design parameters:

Design life (years): This control allows you to select from a list the period of time in years from completion of construction where the pavement is expected to perform adequately without significant loss of functional and structural integrity. Pavement performance is predicted over the design life beginning from the month the pavement is open to traffic.

Base Construction: This control allows you to select the month and year when the existing JPCP was fractured.

Pavement construction: This control allows you to select the month and year when the AC overlay is scheduled to be placed.

Traffic opening: This control allows you to select the month and year the pavement is scheduled to be open to traffic. DARWin-ME predicts pavement performance beginning from this month and year.

11.2 Performance Criteria

Performance verification forms the basis of the acceptance or rejection of a trial design evaluated using DARWin-ME. The design procedure is based on pavement performance, and therefore, the critical levels of pavement distresses that can be tolerated by the agency at the selected level of reliability needs to be specified by the user. If the simulation process shows the trial design produces excessive amount of distresses, then the trial design must be modified accordingly to produce a feasible design in the future trials.

The distress types considered in the design of an AC overlay of fractured pavement are total rutting (all layers and subgrade), AC rutting, load-related top-down cracking (longitudinal cracking in the wheel path) and bottom-up fatigue cracking (alligator cracking), and thermal cracking (transverse cracking). In addition, pavement smoothness is considered for performance verification and is characterized using the International Roughness Index (IRI).

Performance Criteria (AC Overlay of Fractured JPCP)

Populating the Inputs in this Interface

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	63	
Terminal IRI (in./mile)	172	90
AC top-down fatigue cracking (ft/mile)	2000	90
AC bottom-up fatigue cracking (percent)	25	90
AC thermal fracture (ft/mile)	250	90
Chemically stabilized layer - fatigue fracture (percent)	25	90
Permanent deformation - total pavement (in.)	0.75	90
Permanent deformation - AC only (in.)	0.25	90

Performance Criteria (AC Overlay of Fractured JPCP)

Populating the Inputs in this Interface

This table allows you to define the limits of critical distresses and smoothness that can be tolerated by the agency at the specified reliability levels. This table has three columns:

Performance Criteria: This column provides a list of performance indicators required to ensure that a pavement design will perform satisfactorily over its design life.

Limit: This column allows you to define the threshold values of these performance indicators to evaluate the adequacy of a design.

Reliability: This column allows you to define the probability at which the predicted distresses and smoothness will be less than the limits over the design period.

> **Note**
>
> You can override the program defaults to enter project-specific threshold limits and reliability values representing agency policies. Refer to Chapter 8 of the AASHTO Manual of Practice for more guidance on selecting design criteria and reliability level.

Initial IRI (in./mile): The limit control allows you to define the expected smoothness immediately after new pavement construction (expressed in terms of IRI). Initial IRI is a very important input as the time from initial construction to attaining threshold IRI value is very much dependent on the initial IRI obtained at the time of construction. Thus, the initial IRI value provided must be what is typically attained in the field. You can override the DARWin-ME default value of 63 in./mi to reflect agency policy and guidelines.

Terminal IRI (in./mile): The limit and reliability controls for this criterion allow you to define the not-to-exceed limit for IRI at the end of the design life at a specified reliability level.
AC top-down fatigue cracking (ft./mile): The limit and reliability controls for this criterion allow you to define the not-to-exceed limit for surface initiated fatigue cracking at the end of the design life at a specified reliability level.

AC bottom-up fatigue cracking (percent): The limit and reliability controls for this criterion controls allow you to define the not-to-exceed limit for bottom-initiated fatigue cracking at the end of the design life at a specified reliability level.

AC thermal fracture (ft./mile): The limit and reliability controls for this criterion allow you to define the not-to-exceed limit for non-load related transverse cracking at the end of the design life at a specified reliability level.

Permanent deformation - total pavement (in.): The limit and reliability controls for this criterion allow you to define the not-to-exceed limit for total rutting at the end of the design life at a specified reliability level. Total permanent deformation at the surface is the accumulation of the permanent deformation in all of the asphalt and unbound layers in the pavement system.

Permanent deformation - AC only (in.): The limit and reliability controls for this criterion allow you to define the not-to-exceed limit for rutting contributed by the AC layers at the end of the design life at a specified reliability level.

11.5 Pavement Structure Definition and Materials

11.5.1 Asphalt Concrete (New) Layer

Refer to section 8.5.1 Asphalt Concrete (New) Layer

11.5.2 Fractured PCC Layer

DARWin-ME considers three methods of fracturing existing pavement PCC as defined below:

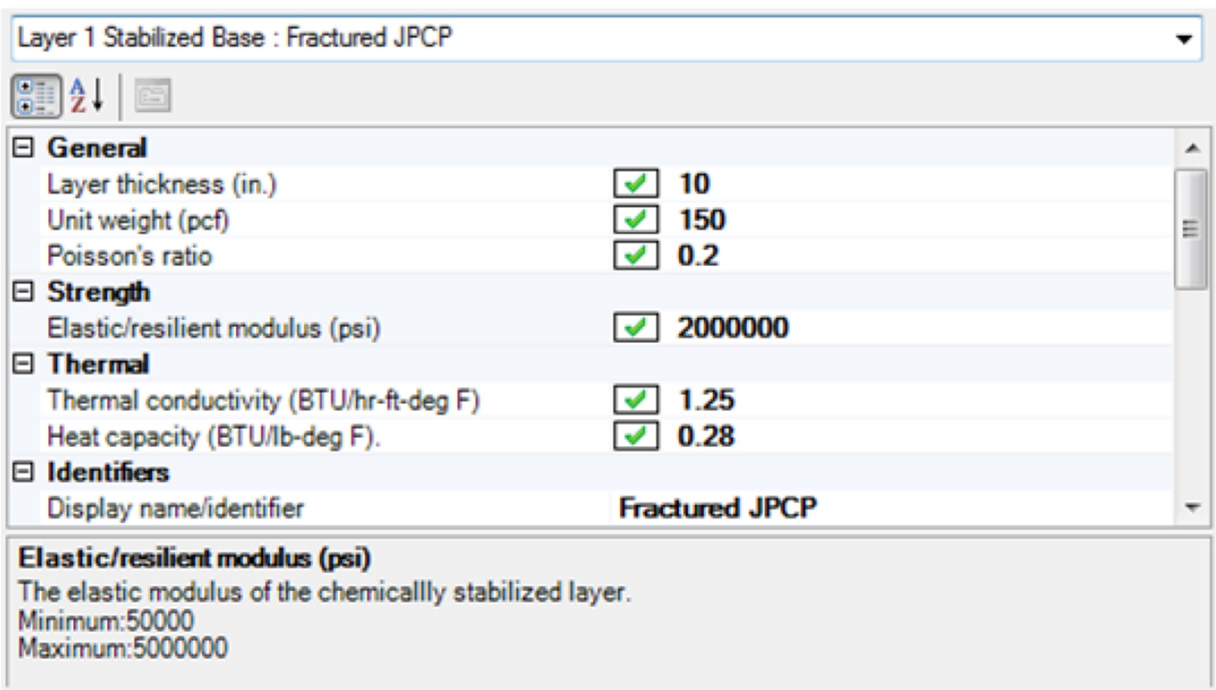
- Rubblization: Applicable to all PCC slabs. Used for fracturing existing PCC slabs into pieces less than 12 inches reducing the existing PCC slab to a high-strength nonstabilized material. Rubblization is commonly applied to PCC layers with extensive deterioration (severe mid-slab cracks, faulting, spalling at cracks and joints, Dcracking, etc.).

- Crack and Seat: Applicable to PCC slabs. Used for fracturing the PCC slabs into pieces typically one to three feet in size.

- Break and Seat: Applicable to jointed reinforced concrete pavement (JRCP) slabs. Used for rupturing the reinforcing steel in JRCP across each crack and breaking its bond with the PCC.

Way to Access this Interface

Open a project with AC over fractured PCC and select fractured PCC in the Layer control.



Fractured PCC Layer Properties

Ways to Enter Inputs

There are three methods for entering data for the fractured PCC layer:

- Manual entry
- Import from file

- Import from database

Populating the Input in this Interface

General

Layer thickness (in.): This control allows you to define the thickness of the fractured PCC layer.

Unit weight (pcf)*:* This control allows you to define the weight per unit volume of the fractured PCC material. DARWin-ME provides a default value of 150 pcf.

Poisson's ratio*:* This control allows you to define the Poisson's ratio of the fractured PCC material. DARWin-ME provides a default value of 0.2.

Strength

Elastic/resilient modulus (psi): This control allows you to define the required elastic/resilient modulus value of fractured PCC materials. One method commonly used to estimate the elastic modulus of the fractured PCC pavement is to perform FWD deflection tests and backcalculate fractured PCC layer modulus from the deflection basins measured. This is mostly done on several similar projects to estimate typical values. Typical moduli values range from 150,000 to 1,000,000 psi for crack/seat and break/seat and 50,000 to 150,000 psi for rubblized PCC.

Thermal

Thermal conductivity (BTU/hr-ft-deg F): This control allows you to define the thermal conductivity of the fractured PCC material. DARWin-ME provides a default value of 1.25 Btu/(ft)(hr)(°F).

Heat capacity (BTU/lb-deg F): This control allows you to define the heat capacity of the fractured PCC material. DARWin-ME provides a default value of 0.28 Btu/(lb)(°F).

Links to Relevant Section in AASHTO Manual of Practice

Refer to Section 11.3 PCC Mixtures, Lean Concrete, and Cement Treated Base Layers

11.5.3 Chemically Stabilized Layer

The required inputs for a chemically stabilized layer can be broadly classified as general, strength, and thermal properties. Note that the strength properties required by DARWin-ME are different for flexible and rigid pavements.

The chemically stabilized materials include lean concrete, cement stabilized, open graded cement stabilized, soil cement, lime-cement-flyash, and lime treated materials.

Refer to Section 5.5.2 **Chemically Stabilized Layer**

11.5.4 Non-Stabilized Layer

Non-stabilized materials include AASHTO soil classes A-1 through A-3, as well as those commonly defined in practice as crushed stone, crushed gravel, river gravel, permeable aggregate, and cold recycled asphalt material (includes millings and in-place pulverized material).

Inputs required for non-stabilized materials include physical and engineering properties such

Refer to Section 5.5.3 **Non-Stabilized Layer**

Links to Relevant Section in AASHTO Manual of Practice
Refer to Section 11.5 Unbound Aggregate Base Materials and Engineered Embankments.

11.5.5 Subgrade

Subgrade materials include soil classes A-1 through A-7-6 defined in accordance with the AASHTO soil classification system.

Inputs required for subgrade materials are same as those of non-stabilized materials, and include physical and engineering properties such as dry density, moisture content, hydraulic conductivity, specific gravity, soil-water characteristic curve (SWCC) parameters, classification properties, and the resilient modulus.

Refer to Section 5.5.3 Subgrade

Links to Relevant Section in AASHTO Manual of Practice

Refer to Section 11.5 Unbound Aggregate Base Materials and Engineered Embankments.

11.5.6 Bedrock

A bedrock layer, if present under an alignment, could have a significant impact on the pavement's mechanistic responses and therefore need to be fully accounted for in design. This is especially true if backcalculation of layer moduli is adopted in rehabilitation design to characterize pavement materials. While the precise measure of the stiffness is seldom, if ever, warranted, any bedrock layer must be incorporated into the analysis.

Refer to Section 5.5.5 **Bedrock**

11.6 AC Layer Properties

This screen allows you to define other inputs pertinent to flexible pavement design.

Refer to Section 8.5.1 and 8.5.2 AC Layer (New) and AC Layer (Existing)

11.7 Backcalculation

The Backcalculation option allows you to use backcalculated layer moduli of existing pavement layers and subgrade in overlay designs. The moduli typically are obtained using nondestructive deflection basin tests and standard backcalculation procedures. The backcalculated moduli closely match the actual in situ moduli of the existing pavement layers and subgrade. The Backcalculation option allows you to use backcalculated layer moduli of existing pavement layers and subgrade in overlay designs. The moduli typically are obtained using nondestructive deflection basin tests and standard backcalculation procedures. The backcalculated moduli closely match the actual in situ moduli of the existing pavement layers and subgrade.

Refer to Section 8.7 Backcalculation

5.7 Thickness Optimization

After establishing a trial design, DARWin-ME allows you to optimize the thickness of any layer above the foundation (semi-infinite thickness). You can only optimize a single layer at a time in 0.5 inch increments.

Optimization is the selection of the lowest thickness within a given range of maximum and minimum thicknesses you define. DARWin-ME first selects the minimum thickness value and determines if the minimum thickness satisfies the performance criteria. If the run is successful, the process stops there. Otherwise, the program selects the maximum thickness value for its next run. If both minimum and maximum thicknesses are unsuccessful, the process stops. If the maximum thickness value is successful, the program selects the mid-point thickness value between the maximum and minimum thickness for its third run. Irrespective of the outcome, the program chooses the mid-point between the thickness of the last successful run and the last unsuccessful run for all further runs.

Way to Access this Interface

In the Explorer tab, expand Project tree and double-click on the Optimization node.

Populating the Inputs in this Interface

The Design Layers table displays the pavement layers (above the foundation layer) of your trial design. This table has five controls (columns):

***Use*:** This control allows you to select a pavement layer of your choice for which the layer thickness will be optimized. You can select only one layer at a time.

***Layer*:** This control displays the layer type as defined in your trial design.

***Default Thickness*:** This control displays the thickness of the given layer you defined in the materials property page.

***Minimum Thickness*:** This control allows you to define the minimum thickness for which the given layer will be optimized.

***Maximum Thickness*:** This control allows you to define the maximum thickness for which the given layer will be optimized.

Click on the Optimize Thickness button found at the bottom of the screen to run the optimization process. In real-time, DARWin-ME displays the history of the optimization process by showing thickness and outcomes of all previous runs. In addition, the program also displays the thickness and status of the current run. At the top left of the screen, the Last Optimized Thickness control updates the results of the optimization process in real time. A green fill displayed in the control box indicates the thickness of the last successful run, while a red fill indicates the last unsuccessful run i.e. none of the previous runs were successful.

1. Select a layer in the list. Check its check box.
2. Enter values for Minimum Thickness and Maximum Thickness.
3. Click on the Optimize button.
4. The application will perform all calculations. When it has found the lowest allowable thickness, it will display in the Last Optimized Thickness field at the top-left of the tab.

Optimization Rules Table

Use: This control, when activated, includes the current row of rule information as part of the optimization parameters.

Property: This control allows you to select a property of the design to modify in order to customize the optimization process.

Rules: This control allows you to enter valid text rules (one per line) to apply to the design property identified in the Property field of this table.

Criteria: This control allows you to enter valid text that establishes the limits (such as upper and lower) of the modification entered into the Rules field.

5.8 Sensitivity Analysis

The Sensitivity Analysis allows you to define minimums and maximum levels for selected parameters to test the design limits in Sensitivity tab.

Way to Access this Interface

Click the Sensitivity node in the Explorer tree.

Populating the Inputs in this Interface ***Run Factorial:*** Enable this control to calculate all permutations of the selected number of increments of each defined property range. Disable this control to prevent DARWin-ME from calculating all permutations of the selected number of increments of each defined property range.

Create Sensitivity: This button creates a sensitivity project based on the parameters that you defined in the Sensitivity Table.

Run Sensitivity: This button calculates the sensitivity project.

View Summary: This button generates a summary report of Pass/Fail results based on the parameters that you defined in the Sensitivity Table.

Sensitivity Table: This table allows you to define the parameters of a sensitivity project.

Use: Enable this control to include the associated layer property in the sensitivity analysis. Disable this control to prevent DARWin-ME from including the associated layer property in the sensitivity analysis.

Property: This column displays the names of properties associated with a particular layer.

Layer: This column displays the names of the layers in the project.

Default: This column displays the default values of associated properties for particular layers.
Minimum: This column allows you to define the lower bound of the range for the property of a particular layer that you want to evaluate.

Maximum: This column allows you to define the upper bound of the range for the property of a particular layer that you want to evaluate.

# of Increments: This column allows you to define how often DARWin-ME will calculate sensitivity between the minimum and maximum range.

Entering Inputs

To run a sensitivity analysis:

1. Enable the Use control associated with a design property in a layer that you want to evaluate.
2. Define Minimum, Maximum, and # of Increments values for the design property.
3. Click Create Sensitivity and wait until the action is complete.
4. Click Run Sensitivity and wait until the action is complete.
5. Click View Summary to open the report.

5.9 Run Analysis

To perform an analysis on a single project:

After you defined the project parameters, click DARWin-ME will perform the analysis and generate reports.

To perform a Batch Run:

1. Right click and select the "Load Projects" control on the Batch Run node on the Explorer Pane.
2. Browse to the folder(s) containing the projects to be run in the batch mode.
3. Select the "Run Batch Projects" control to enable DARWin-ME to run multiple projects in batch. Alternately, the user can also click on the icon on the DARWin-ME Menu run projects in batch.
4. After the analysis is complete, right click and select the "View Batch Report" control on the Batch Run node on the Explorer Pane to generate a summary of the batch project run results. The user can also double click the individual project nodes

14 Interpretation and Analysis of the Trial Design

The MEPDG software predicts the performance of the trial design in terms of key distress types and smoothness at a specified reliability (refer to Section 5). The designer initially decides on a "trial design" for consideration, as discussed in Sections 12 and 13. This trial design may be obtained from the current AASHTO Design Guide, the result of another design program, a design catalog, or a design created solely by the design engineer.

The MEPDG software analyzes that trial design over the selected design period. The program outputs the following information: inputs, reliability of design, materials and other properties, and predicted performance. Each of these outputs needs to be examined by the designer to achieve a satisfactory design as described in this section. An unacceptable design is revised and re-run to establish its performance until all criteria are met. This "trial and error" process allows the pavement designer to "build the pavement in his/her computer," prior to building it in the field to ensure that the performance expectations will be met as economically as possible. The purpose of this section is to provide some guidance on what design features could be revised for the trial design to be accepted.

14.1 Summary of Inputs for the Trial Design

A unique feature of the MEPDG software is that nearly all of the actual program inputs are included in this section of the outputs. Details of the climatic data and the axle load distributions are not included here. The designer needs to review all of these inputs to ensure that no mistake has been made in entering the data. Given the large number of inputs, this check is essential.

14.2 Reliability of Trial Design

Another important output is an assessment of the design reliability, which may be seen under the Reliability Summary tab. The Distress Target and its corresponding Reliability Target are the first right-hand columns listed, followed by the Distress Predicted and the Reliability Predicted. If the Reliability Predicted is greater than the Reliability Target then the pavement passes. If the reverse is true, then the pavement fails. If any key distress fails, the designer needs to alter the trial design to correct the problem. Examples are shown below for a flexible and rigid pavement (Tables 44 and 45, respectively).

- For the flexible pavement example (Table 44), the asphalt concrete (AC) surface down cracking met the reliability criterion (99.92 > 90 %), but terminal IRI did not (52.51 < 90 %). This trial design is not acceptable at the 90% reliability level and needs to be revised.
- For the JPCP example (Table 45), the mean joint faulting met the reliability criterion (98.09 > 95%), but terminal IRI did not (93.98 < 95 %). This trial design is not acceptable at the 90% reliability level and needs to be revised.

Table 44. Reliability Summary for Flexible Pavement Trial Design Example

Project:	US 305				
Reliability Summary					
Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable?
Terminal IRI (in./mi.)	172	90	169.3	52.51	Fail
AC Surface Down Cracking (Long. Cracking; ft./mi.)	2000	90	5	99.92	Pass
AC Bottom Up Cracking (Alligator Cracking; %)	25	90	0.1	99.999	Pass
AC Thermal Fracture (Transverse Cracking; ft./mi.)	1000	90	1	94.16	Pass
Chemically Stabilized Layer (Fatigue Fracture)	25	90	NA	NA	NA
Permanent Deformation (AC Only; in.)	0.25	90	0.58	1.66	Fail
Permanent Deformation (Total Pavement; in.)	0.75	90	0.71	59.13	Fail

Table 45. Reliability Summary for JPCP Trial Design Example

Project	I-999				
Reliability Summary					
Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable?
Terminal IRI	172	95	112.5	93.83	Fail
Transverse Cracking (% slabs cracked)	15	95	21.2	32.9	Fail
Mean Joint Faulting (in.)	0.12	95	0.051	98.09	Pass

14.3 Supplemental Information (Layer Modulus, Truck Applications, and Other Factors)

Another unique feature of the MEPDG software is that the materials properties and other factors are output on a month-by-month basis over the design period. The designer needs to examine the output materials properties and other factors to assess their reasonableness. For flexible pavements, the output provides the HMA dynamic modulus (EHMA) and the resilient modulus (Mr) for unbound layers for each month over the design period. Moisture content and frost condition greatly affects the unbound materials Mr.

The MEPDG provides a graphical output of selected modulus values for the HMA layers. The dynamic modulus for the first quintile of temperatures (the lower temperatures) for each sublayer is plotted over the design life of the pavement. All HMA dynamic modulus values for each temperature quintile and sublayer are included in a tabular format. In addition, the resilient modulus for the unbound layers and foundation are also included in that tabular format for each month over the design life of the pavement.

The designer should examine the monthly output materials properties, number of trucks (Class 4 and higher), and other factors to assess their reasonableness. These are all output at the end of the month.

Flexible pavements key outputs that need to be observed and evaluated include the following.

- HMA Dynamic Modulus (EHMA) of each layer. The software divides each HMA input layer into sublayers and each need to be examined for reasonableness. Materials properties as well as temperature and load speed typically have significant effects on EHMA.
- Unbound material resilient modulus (Mr) for unbound layers for each month over the design period can be examined. The software divides each unbound material input layer (such as a granular base course) into sublayers and each need to be examined for reasonableness. Moisture content and frost condition greatly affects the unbound materials Mr.
- The number of cumulative Heavy Trucks (Class 4 and above) are output shown for the design traffic lane. The total cumulative Heavy Trucks may be examined at the last month of the analysis period. This parameter is a good general indicator of how heavy the truck traffic (volume) is for the design (e.g., 1 million trucks, 20 million trucks, or 100 million trucks is the terminology recommended for design purposes). Note that these may be converted into flexible pavement 18-kip ESALs by multiplying them by an average truck factor, or the actual number of ESALs may be determined by examining an intermediate file by this name that has this information.

14.4 Predicted Performance Values

The software outputs month-by-month the key distress types and smoothness over the entire design period. The designer needs to carefully examine them to see if they appear reasonable and also meet the specified performance criteria.

Flexible pavements.

- Longitudinal fatigue cracking: top down fatigue cracking in the wheel paths. A critical value is reached when longitudinal cracking accelerates and begins to require significant repairs and lane closures.
- Alligator fatigue cracking: traditional bottom up fatigue cracking in the wheel paths. A critical value is reached when alligator cracking accelerates and begins to require significant repairs and lane closures.
- Transverse cracking: caused by low temperatures that result in fracture across the traffic lanes. A critical value is reached when transverse cracking results in significant roughness.
- Rutting or permanent deformation: HMA rutting is only in the asphalt bound layers and total rutting combines all of the pavement layers and the subgrade. A critical value is reached when rutting becomes sufficient enough to cause safety concerns.
- IRI: this index represents the profile of the pavement in the wheel paths. A critical value is reached as judged by highway users as unacceptable ride quality. IRI is a function of

longitudinal cracking, transverse cracking, alligator cracking, and total rutting along with climate and subgrade factors.

- Reflection cracking: reflection cracking occurs only when an HMA overlay is placed over an existing flexible pavement that has alligator fatigue cracking in the wheel paths, or over a jointed rigid pavement where transverse joints and cracks exist and occur. A critical value is reached when reflection alligator cracking results in significant maintenance requirements or when reflection transverse cracking results in significant maintenance requirements or roughness.

14.5 Judging the Acceptability of the Trial Design

While layer thickness is important, many other design factors also affect distress and IRI or smoothness. The designer needs to examine the performance prediction and determine which design feature to modify to improve performance (e.g., layer thickness, materials properties, layering combinations, geometric features, and other inputs). This subsection provides guidance on revising the trial design when the performance criteria have not been met.

The guidance given is distress- specific. The designer needs to be aware, however, that changing a design feature to reduce one distress might result in an increase in another distress. As an example, for excessive transverse cracking of an HMA pavement where the level 3 inputs were used, the user may consider using softer asphalt to reduce transverse cracking, but that will likely increase the predicted rutting. Another option is to use laboratory tests to measure the level 1 inputs, which could reduce or even increase the distress further.

More importantly, some of the input parameters are interrelated; changing one parameter might result in a change to another one. For example, decreasing asphalt content to make the HMA mixture more resistant to rutting will likely increase the in-place air voids resulting in more fatigue cracking. The designer needs to use caution in making changes to individual layer properties. It should be noted that some of these modifications are construction dependent and will be difficult to justify prior to building the pavement or placing the HMA overlay.