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PAVEMENT DESIGN MANUAL OF INSTRUCTION

Oct 2016

Introduction

The Utah Department of Transportation (UDOT) has changed the way that Pavement designs are done on all projects except Federal aided Local Government projects.

UDOT has adopted the new AASHTO pavement design software called Pavement ME Design or ME Design, sold through AASHTOWare, as its official pavement design method. Federal aid Local Government projects may use other means to design their pavement designs at this time, such as the AASHTO 1993 pavement design software or other approved methods. All pavement design will be reviewed and approved by the Region Pavement Design Engineer. This will allow the Local Governments time to make the transition from the way they currently do pavement designs to the new ME Design software within the next three years (by 2015). All UDOT projects will use ME Design software for all pavement designs at that time.

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PART 1 – GENERAL

Section 1
Pavement Management

1.1 Introduction

A pavement management system is defined as a set of tools or methods that assist decision makers in finding cost-effective strategies for maintaining the roadway system in optimal condition. The detailed structure of a pavement management system is separated into two levels – system (network) level and project level.

1.2 Organization

Pavement Management is an ongoing effort throughout the Utah Department of Transportation (UDOT). Organizationally the Central pavement management group is under the direction of the Director of Asset Management. Each Region has a pavement management engineer within its Materials group. The Central pavement management group is responsible for the network level program and the Regions are responsible for the project level programs.

1.3 Network Level Pavement Management

Administrative decisions are made that affect the programs for the entire roadway system. The management system considers the needs of the network as a whole and provides information for a statewide program of preservation and rehabilitation. The goal of this level of analysis is to optimize the use of funds over the entire system. The system compares the benefits and costs for several alternative programs and then identifies the program or budget that will have the greatest cost/benefit ratio over the analysis period.

The principal products of network level pavement management include:

1. Lists of recommended candidate projects
2. Reports on the current state of the highway network
3. Recommended budget levels to meet system level goals and strategies

A number of separate activities help to produce these products.

Network Definition

The UDOT pavement management system can be set up for any number of highway networks. State roads typically analyzed include all state roads, state roads on the National Highway System, the Interstate System, the Level 1 system, or the Level 2 system. Subsets are also set up to do Region level analysis. The first step in an analysis is to define which set of roads will be analyzed.

Analysis Sections

Once the network is defined it must be divided into analysis sections that are ideally homogeneous in terms of age, pavement type, pavement condition, and are at an appropriate length for a paving project. These are typically based on the construction history. The Region Pavement Management Team manages the sectioning.

Data Collection

Three types of data are important to the pavement management system:

1. **Construction history data** provides the year/age for the last surface seal and last major rehab. This data is managed at the Region level. Good age data is essential to the performance of the pavement management models that include age as one of the criteria for recommending treatments. The Regions also maintain data for the surface area for each section.
2. **Inventory data** provides information about the roads that include pavement type, number of lanes, functional classification, and administrative items. This data is kept current by the Planning Statistics Unit. Traffic data is also collected and maintained by this group.
3. **Condition data** includes annually collected pavement condition data. The Asset Management group collects data for pavement roughness, rut depth, environmental cracking, structural cracking, slab cracking, joint spalling and joint faulting. This data is summarized and reported on one-tenth mile intervals and included as part of the inventory data. Skid number and deflection data is also collected by this group. Most of this data is currently collected under contract and includes roadway and pavement images.

Evaluations of Current Conditions and Trends

Reports are produced to show the current state of the network based on the condition data. The current year condition is compared with previous years to show trends. These trends can help managers determine the effectiveness of the overall pavement management strategies.

Deterioration Model

UDOT has used the Deighton dTIMS CT pavement management system since 1992.

The dTIMS database integrates the condition data with the inventory, traffic, and construction history data. The system then calculates and inserts this data for each pavement section.

The model can analyze an unlimited number of scenarios for the statewide network or for each Region, including:

1. Predict future network or section condition at various funding levels.
2. Identify the funding required to maintain the network in a status quo condition or the funding needed for various condition levels.
3. Recommend candidate preservation and rehabilitation projects based on a given funding level.

The model recommends the appropriate preservation and rehabilitation treatments based on the provided funding for each section. These recommendations are provided to the Regions and reviewed by the Asset Management Group to develop the Statewide Transportation Improvement Program (STIP).

The development of our model requires that many basic parameters be defined.

1. Condition Indices are used to define the pavement condition, trigger treatments, and report overall network condition. These are based on a 100 to 0 scale with 100 being ideal and 50 being unacceptable. UDOT uses qualitative measures for Good (80 to 100), Fair (50 to 80), and Poor (0 to 50) condition. Ride based on IRI data, rutting, environmental cracking, and wheel path cracking for asphalt pavements are used. Ride, joint faulting, joint spalling, and slab cracking for concrete pavements are also used. An overall condition Index is also used that is an average of the other four primary measures.
2. Performance curves for each condition index are developed to predict how a pavement will behave over time.
3. Preservation and rehabilitation treatments with unit costs each have defined benefits for each of the condition measures. The benefits are based on resetting the condition level different amounts.
4. A set of expressions are defined to “trigger” the various treatment strategies at different condition levels.

Substantial changes have been made to the original model and the UDOT overall pavement management philosophy. Refer to the documentation manual for the current model configuration and other pavement management manuals. The model documentation details how the index values are calculated from the condition data, the deterioration curves for these condition indexes, the treatments being recommended, the trigger expressions for selecting appropriate treatments, and the treatment costs.

1.4 Project Level Pavement Management

Project level pavement management is where technical decisions are made for specific projects. Detailed consideration is given to alternative design, construction, preservation, and rehabilitation activities for specific projects. This is accomplished by comparing cost/benefit ratios of several design alternatives and picking the design

alternative that provides the desired benefits for the least total cost over the projected life of the project.

UDOT's principal pavement management tool at the project level is its Plan for Every Section (PFES) database. This has a new web based format incorporated with the Agile Asset's OMS database developed for the Pavement Management Operations group. This database is used to manage the section locations, information about the sections, and some of the construction history. It was originally developed to produce time based treatment plans for each section. The limited funding made it difficult to achieve an ideal program and most sections were falling behind. Now it includes the recommended treatment from dTIMS.

Each of the Region Pavement Management Engineers utilizes the condition data, dTIMS recommendations, PFES data, and their observed ground truth to develop annual preservation and rehabilitation programs. Additional project level testing such as coring and deflection testing assist with determining the most appropriate level of treatment. Many other considerations are included in making specific surfacing decisions.

1.5 UDOT Approach

UDOT prioritized our statewide network in 2008 with the designation of the Interstate system, Level 1 system, and the Level 2 system. This resulted from a formal task force charged with looking into ways to address the reduced funding available for preservation and rehabilitation needs. This was adjusted with the MAP 21 NHS in 2012.

Traffic volumes were used to define the systems with the Level 1 routes being designated as having over 1,000 AADT or over 200 large trucks daily. This resulted in 935 Interstate miles, 1,720 NHS miles, 1,330 Level 1 miles, and 1,880 Level 2 miles. The Level 2 system has 32 percent of the mileage but < 5 percent of the VMT.

The NHPP funding will be programmed for the Interstate & NHS. The STP funding will be used for the Level 1 system. Funding for the Level 2 system is limited to the Maintenance program code 1 funds.

The available funding is allocated the Preservation program, Rehabilitation program, and" major rehab/reconstruction program. The model is used to recommend how much funding needs to be available for each program. The model is also used to recommend how the funding will be allocated for each Region. The UDOT Senior Leadership ultimately makes these decisions with Commission approval.

Preservation Program: This program is designated funding for preservation projects. A block of funding is programmed in the STIP for each category. Each Region provides a prioritized list of eligible projects and Programming matches up available funds with as

many projects as possible.

Rehabilitation Program: This program is designated for funding rehabilitation type projects. A block of funding is programmed in the STIP for each category similar to the Orange book program. Each Region provides a prioritized list of eligible projects and Programming matches up available funds with as many projects as possible. The prioritized list is provided to the Commission for review. This program was first funded for FY 2011 but many projects were advanced and the program was funding projects sooner.

Reconstruction Program: This is the traditional construction project program. Each project is listed in the STIP as it becomes funded. Paving projects that are in this program are outside the scopes for the Preservation and Rehabilitation Programs.

Section 2
Pavement Design Coordination

2.1 Introduction

The UDOT Design Process creates a timeline for planning, creating, submitting, and reviewing a project pavement design. This process uses standard project programming techniques to schedule and organize a project through the design phase. In-depth pavement designs can require up to eight months to complete due to the implementation of soils investigations, traffic investigations, field and laboratory tests of existing pavement, project scope review and modification, and review and selection of the appropriate alternatives. The timetable outlined in this section is intended to delineate a process that has been accepted by the UDOT Materials Division that will coordinate the actions of all the parties involved in completing the pavement design in a timely manner and with a minimal amount of confusion.

2.2 Design Network

The Design Network is a planning and scheduling tool used by the Preconstruction Division to organize the design of all state projects. The process includes the division of any particular project into a large assortment of relatively small activities. Each activity is defined by the work it encompasses, the time frame expected to complete the project, the responsible parties, and a list of those who will receive copies of the final document. The pavement structure design requires the completion of two separate activities.

<u>Activity 1</u>

1. Assess existing pavement condition (activity 2MI – Develop Pavement Design and Report) to investigate and summarize the existing data.
2. Prepare a preliminary testing strategy to supplement existing data.
3. Review ground penetrating radar information to aid in selecting core locations.
4. Take 10-15 cores, slabs, or both from the project area and perform minimal lab testing to determine the condition of the pavement including:
    a. Pre-testing (pavement management data) including project level coring and material testing
    b. Falling weight deflectometer or structural measurements
    c. Rut measurements
    d. Ride index/IRI
    e. Cracking type and extent
    f. Skid index
    g. Traffic data
    h. Economic factors for life cycle cost analysis.
    i. Core data
5. Determine possible project alternatives. Review corridor pavement type strategy, project specific constraints, and life cycle cost analysis.

6. Recommend alternative solutions to be further evaluated during activity #2MI along with any significant factors to be considered in the alternative analysis such as additional testing needed to assess pavement conditions or selected alternatives.

Activity 2

1. Develop and confirm pavement including the actual in-depth study of the pavement condition and development of the appropriate solution to reconstruct or rehabilitate.
2. Develop a pavement testing strategy, proposed design life, and any additional testing required to complete an approved pavement design. Evaluate potential material sources and additional parameters such as noise consideration or drainage requirements.

   Testing

   a. Perform additional site investigation/testing as needed.
   b. Core and trench for thickness and condition of existing pavement.
   c. Extraction/gradation on cores.
   d. Strength and stripping tests.
   e. Trench for subbase and subgrade samples.
   f. Concrete pavement evaluation.
   g. Falling weight deflectometer testing.
   h. Centerline soil survey report
      1) Soil classification – plastic limit
      2) Soluble salts
      3) Resistivity
      4) PH
3. Select pavement type. Review existing regional pavement corridor plan and choose the appropriate pavement type. Review preliminary testing and provide the most likely design alternatives for both portland cement concrete and hot mix asphalt pavements. Perform a pavement type LCCA comparing both options according to UDOT Pavement Management and Pavement design MOI recommendations. Recommend base on lowest LCC solution for pavement. Verify pavement type section is consistent with the corridor plan.
4. Final Pavement Design Report will include existing pavement conditions, pavement construction and maintenance history, recommend pavement type, rehabilitation strategy, life-cycle cost, initial cost estimate, additional project acceptance or verification testing requirements, and material descriptions as needed.

Submit the pavement design recommendation to the Region Materials Engineer for approval. Perform cost estimate, develop applicable special provisions, and identify pay items to be used in the project. Summarize into a document the following:

1. Construction cost estimate
2. LCC evaluation

3. Additional project acceptance or verification testing required
4. Material description
5. Pavement history and condition information
6. Design calculations and assumptions
7. Cost information
8. List of potential issues for the field.

2.3 Design Build Projects

Design Build projects follow the same general requirements for the technical portions of the pavement design. Coordination of the design with the Department occurs according to the stipulations of the project RFP.

Deviations from the Pavement Management and Pavement Design Manual of Instruction (MOI) for Design Build projects must be coordinated with the Department project representatives according to the RFP.

Section 3
Pavement Design Considerations

3.1 Introduction

The basic elements of the paving structure include the surfacing, base course - stabilized or unstabilized, and subbase courses as required. Pavement structures are divided into two general classifications – flexible and rigid – based on the type of pavement structure. Flexible pavements have some type of bituminous surfacing and rigid pavements have a surfacing of portland cement concrete (PCC). Discussion of the following considerations can be found in later sections of this manual.

3.2 Purpose of Surfacing and Base Courses

The surfacing and base courses are layers of high stiffness and density. Their principal purpose is to distribute the wheel load stresses within the pavement structure and thus protect the subgrade soils against excessive deformation or displacement. Use free-draining base material where water is expected to accumulate.

3.3 Frost Action

Greater depths of base or selected free-draining borrow materials are usually necessary in areas where frost action is severe or the subgrade soil is extremely weak. The total depth of the pavement structure is extremely important in high frost penetration areas. Additional thickness of non-frost susceptible base or subbase materials is often effectively used to combat this problem. An effective measure is to have the pavement structure at least equal to ½ the maximum expected depth of freeze when the subgrade is classified as a frost-susceptible soil. Various methods for estimating the maximum expected frost depth are discussed in Section 5.

3.4 Design Period

The design period is the time from original construction to a terminal condition for a pavement structure. AASHTO defines design period, design life, and performance period as the same terms. AASHTO defines an analysis period as the time for which an economic analysis is to be conducted. The analysis period can include provisions for periodic surface renewal or rehabilitation strategies that will extend the overall service life of a pavement structure before complete reconstruction is required.

The design periods used by UDOT are chosen so the design period traffic will result in a pavement structure sufficient to survive through the analysis period. Intermittent treatments may be needed to preserve the surface quality and ensure that the structure lasts through the analysis period.

Table 3 – 1

Required Design Period		
Highway Description Design Period	**Design Period Rigid**	**Design Period Flexible**
Interstate and Principal Arterial	40 years	20 years
Major Arterial Collector with ESALs greater than 100,000 per year	40 years	20 years
Minor Arterial Collector with ESALs less than 100,000 per year	20 years	20 years

The 40-year design periods can be reduced for unique, project specific conditions such as temporary HOV lane pavements, future planned realignment, temporary detours, or grade changes. The 20-year design period can be increased for those routes with future, large expected increases in traffic (ESALs), and anticipated functional class changes. Use the base design periods and equal performance periods when performing Life Cycle Cost Analysis (LCCA) for flexible and rigid pavement comparisons. Include the cost of annual maintenance and periodic rehabilitation activities.

Doubling the design period traffic adds about one or two inches of HMA or PCC to the initial structural thickness of a flexible or rigid pavement design.

3.5 Traffic

The volume and character of traffic, expressed in terms of axle spectra for structural design purposes strongly influences pavement structural requirements. Both flexible and rigid pavement structures can be designed to meet most axle spectra requirements but this does not imply similar maintenance and resurfacing requirements.

3.6 Subgrade Soils

The characteristics of native soils directly affect the pavement structure design. A careful evaluation of soil characteristics is a basic requirement for each individual pavement structure design. Resilient modulus is the primary material characteristic used in the AASHTO Mechanistic/Empirical Pavement Design Guide (MEPDG) as well as the UDOT HMA overlay design procedure. Use of Level II inputs into the ME Design software allows for the conversion of common laboratory values such as CBR or R-Value to approximate the resilient modulus of the subgrade.

3.7 UDOT Pavement Design Practices

<u>Pyramid Design</u>
UDOT pavement design practices employ typical good design practices, including the use of a pyramid design approach for base layers. The pavement section is designed with thicker lower layers and thinner upper layers in the base for the pyramid approach.

Each successive base layer should be equal to or thicker than the layer above it. Design surface layers so their thickness is based on the support of the material beneath it and can be thicker than the initial base layer below.

Materials Layer Requirements

Design individual layers according to placement of the materials that make up the layer. Use the following layer limits designing pavement sections. Discuss and get approval of the UDOT Pavement Management Engineer (PME) for any deviation.

Table 3 – 2

Material Layer Requirements		
Material/Layer	**Minimum Thickness (in)**	**Maximum Thickness (in)**
HMA Surface	3	10
PCC Surface	7	14
HMA Overlay (single lift)	1.5*	3.5
PCC Overlay	4	14
Untreated Base Course	4	12
Granular Borrow	6	24
Treated Base	4	8
Permeable Base	4	6
CIR	3	6
FDR	6	12

*1.5” thickness based on ½” nominal mix design. ¾” nominal mix design requires a minimum thickness of 2.25”.

Materials Call-outs/Specifications

Build pavement layers with materials that conform to the latest UDOT Standard Specifications or project specific special provisions. Tie each layer identified within the pavement section to a specification or special provision within the pavement design report. HMA, PCC, and Base Course specifications allow options for things such as gradation, stability, asphalt, or portland cement type. Identify each of these items within the pavement report. Discuss and verify with the Region PME or Region Material Engineer material details and specific concerns related to each material layer and that all specific details have been addressed in the design report.

Drainable Layer Use

Specify drainable layers with thickness and permeability requirements (k-value). Typical k-values for use in Utah are 200 - 250 ft/day. Specific pavements may require higher drainage requirements. Discuss these with the Region PME. Drainable layers can be bound or unbound materials.

Include a drain system to remove the water from the drainable layer for any drainable layer that is either day-lighted or tied to the project storm drainage system. The FHWA Report, Guidelines for the Design of Subsurvace Drainage Systems for Highway

Structural Sections *(FHWA_RD-72-30)* is a good resource for understanding and designing the pieces of a pavement drainage system.

Surface Course Use as Part of Initial Design

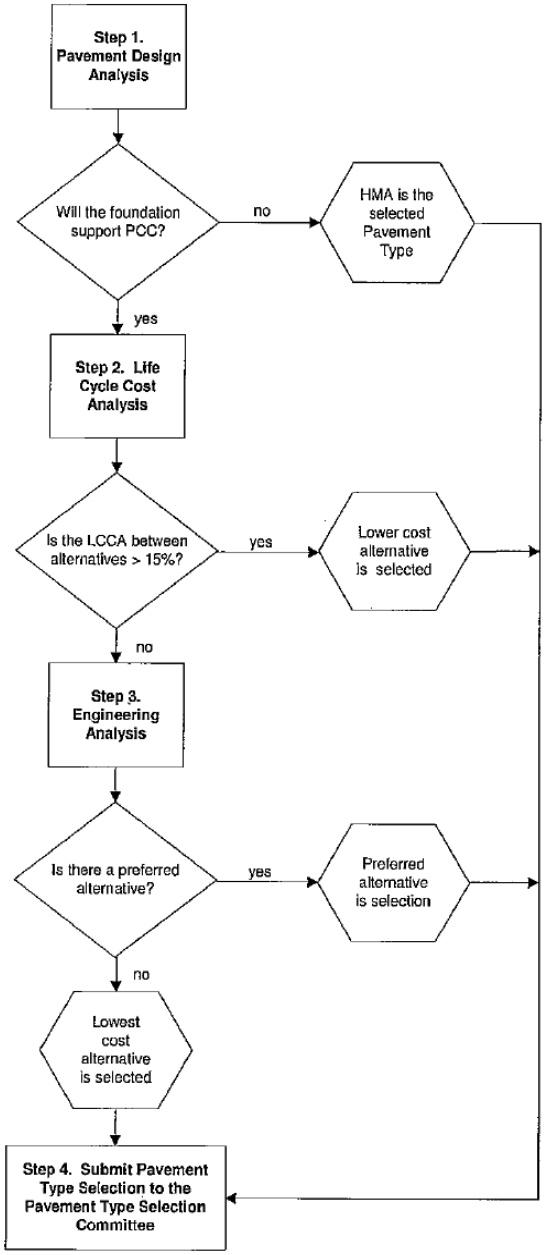
Long term durability performance of a pavement relies on both the materials selection for the pavement layers and the use of appropriate preventative maintenance practices for the pavement structure. This includes the use of a protective surface course included with the initial pavement construction. These protective surfaces can be Stone Matrix Asphalt (SMA), Open-Graded Surface Courses (OGSCs), Micro Surfacing, Bonded Wearing Course, or surface seals such as chip seals, slurry seals, or sand seals.

Surface seals are not given structural value but SMAs and OGSCs can contribute to the structure of the section. Design pavement sections with a protective surface included in the design and selection. Discuss the proper surface and the appropriate materials to be used for that layer with the Region PME.

Section 4
Pavement Type Selection

4.1 Introduction

The areas of pavement design, life cycle cost, and engineering analysis have significant impact on the selected pavement type. Use the following chart for roadways not already designated as a rigid or flexible corridor roadway.



The Region PME will determine the pavement type at the project level with assistance

from the Asset/Pavement Management Group and the Region Project Managers. Pavement type determination will consist of four steps:

1. Corridor Determination
2. Pavement Design Analysis with Project Specific Constraints
3. Life Cycle Cost Analysis (LCCA)
4. Engineering Analysis

4.2 Corridor Determination

The first step is to check to see if the project is on the Regional Corridors list of designated rigid or flexible pavement found in Section 7.2. Your pavement type is chosen for you if your project is on this list. The Asset Management group champions this process by working closely with Region Pavement Managers and Central Materials to select pavement types for UDOT. These selections are based on an overall corridor analysis including life cycle, maintenance consistency, and geographic constrains. Apply a general LCCA based on total costs that include initial construction maintenance, rehabilitation, and user cost. Develop maps and documents that will show designated corridors for use in pavement design. Pavement types will remain unchanged once determined unless documentation can be shown for safety issues, minimal, or temporary performance period requirements. Not all pavements and corridors will be identified as one or the other. These selections will follow the process in the diagram of Section 4.1.

Corridor Criteria for Determining Pavement Type

1. Corridor Criteria (Entire State – See VII for individual region)
    a. Truck Volumes
    b. Traffic Counts
    c. Truck Speeds
    d. Consistency – Maintenance and Pavement
    e. Life Cycle Costs – User Impacts Costs and Maintenance Impacts
    f. Subgrade Conditions
    g. Construction Materials
    h. Climate data – Canyon Areas where concrete will not be used (safety)
    i. Context Sensitive Solution
2. Concrete roads, high truck traffic, and ADT – See VII for individual roads.
    a. Most interstates through the Wasatch Front
    b. Most major collectors
3. Asphalt Roads, bad subgrade and low truck traffic – See VII for individual Roads.
    a. Local roads – mainly cars
    b. Rural roads

4.3 Pavement Design Analysis with Project Specific Constraints

Identify project specific constrains that will define one specific pavement type over another. Perform the pavement design first since the results may preclude the need to continue with the remainder of the pavement type protocol process including life cycle cost analysis and engineering analysis. These items can drive a need that may not show in a Life Cycle Cost Analysis (LCCA).

The review and analysis includes:

1. Subgrade Competency
2. Traffic Analysis
3. Materials
4. Climate/Drainage
5. Other Pavement Design Factors

4.3.1 Subgrade Competency

Decisions about the subgrade are made during the Pavement Design Analysis. HMA performs better in situations where long-term settlement due to simplified patching and level up overlay opportunities is expected compared to PCC pavement. The pavement design analysis might eliminate the need to continue with a life cycle cost analysis and the engineering analysis if subgrade conditions do not allow the use of PCC pavement. The pavement type selection is complete if the engineering evaluation of the subgrade concludes that PCC pavement cannot be placed or will not perform successfully and HMA is the selected pavement type. Proceed with the remainder of the pavement type selection process if the engineering evaluation of the subgrade concludes that either pavement type can be used successfully.

4.3.2 Traffic Analysis

Pavements are divided into different traffic classes depending on extremely light to extremely heavy traffic. Flexible and rigid pavements can be designed to accommodate these wide traffic ranges. Traffic can be quantified for each of the pavement classes by the number of trucks (classes 4 through 13) per day or per year or over the design life, or by traffic volume such as annual average daily traffic (AADT), annual average daily truck traffic (AADTT), or percent trucks. The design traffic loading can be estimated over the structural design period or the analysis period based on the traffic volume and traffic growth rate. The design traffic loading in the design traffic lane determines the pavement thickness needed to support the traffic loading over the structural design period.

Correctly estimating design traffic is crucial to selecting an appropriate pavement. It is necessary to estimate present traffic loading in order to calculate the total design traffic per lane that a pavement will carry over its structural design life. Use traffic growth rates to estimate future traffic loadings. Conduct a sensitivity analysis to compare

growth rates and the impact of the growth rate on pavement thickness if the roadway segment has significant importance.

4.3.3 Materials

Select materials for a road pavement design by the availability of suitable materials, environmental considerations, construction methods, economics, and previous performance. Evaluate the materials that best suit the conditions during the design to ensure a whole-life cycle strategy. Select materials whose properties are in line with values used in the design and meet UDPT specifications.

Availability and Performance

Most road construction materials have been assigned a classification and have specifications prepared for each class. Every road pavement is subjected to certain traffic loads and environmental factors independent of its type and applied materials. These factors create various deterioration modes under in-service conditions. Deterioration modes and the susceptibility to various deteriorating factors depend on the type of pavement and materials applied. The table below shows the pavement deterioration modes for HMA and PCC pavements.

Table 4 – 1

Pavement Deterioration Modes	
HMA Pavements	**PCC Pavements**
Surface deterioration	Surface deterioration
Decrease in friction	Decrease in friction
Rutting	Surface cracking
Surface cracking	Curling and warping
Raveling (stripping)	Joint raveling
Roughness	Roughness
Studded tire wear	Studded tire wear
Structural deterioration	Structural deterioration
Base and subgrade rutting	Fatigue transverse Cracking
Fatigue cracking in wheelpaths	Longitudinal cracking
Reflective cracking	Pumping
Transverse cracking	Joint Faulting

Pavement surface defects from environmental factors may only require surface course maintenance or rehabilitation.

Structural deterioration (in the wheel paths) is a defect of the whole pavement structure and treating it may require more extensive pavement rehabilitation. Knowing the difference between these two types of deterioration is important to maintaining and properly understanding pavement durability or pavement life.

Consider past performance with a particular material along with applicable traffic and environmental factors. Also consider the performance of similar pavements or materials under similar circumstances. Information from pre-existing designs, material tests, and pavement management data can help characterize the stability of a material for pavement applications.

Recycling

Consider using recycled materials in roadway construction to enhance sustainable development. Incorporate recycled materials whenever possible for future rehabilitation or maintenance treatments according to specifications. Use proper laboratory values to represent recycled materials used in the design.

Studded Tire Wear on PCC

Studded Tire Wear on HMA

4.3.4 Climate and Drainage

Consider both surface runoff and subsurface water control. Effective drainage design prevents the pavement structure from becoming saturated. Effective drainage is essential for proper pavement performance and is assumed in the structural design procedure.

4.4 Pavement Design Factors

UDOT pavements are designed according to AASHTO ME Design Guide and this Pavement Design manual. The following issues related to surfacing type should also be addressed.

4.4.1 PCC Issues

UDOT has demonstrated that the PCC pavements constructed in the late 1950s and 1960s are able to obtain a 40-year or more pavement life as long as joint faulting can be overcome. The ability to provide adequate joint design to minimize joint faulting is

addressed by requiring the use of non-erodable bases and dowel bars at every transverse joint. Default dowel bar sizes are based on a minimum ⅛ inch of dowel diameter for every 1 inch of pavement surface thickness. Increase dowel bar sizes appropriately to accommodate higher volumes of truck traffic based on ME Design design performance. The use of epoxy-coated dowel bars, both locally and nationally, does not guarantee a 40-year performance life. Several states have observed that the corrosion of epoxy coated dowel bars occurs within 15 to 20 years.

Minnesota Department of Transportation has conducted a study of in-service concrete pavements that were constructed with epoxy-coated steel dowel bars at transverse joints and has determined that significant corrosion has occurred in the dowel bars. The result of this study indicates that the corrosion of epoxy coated dowel bars results in a pavement life of less than 20 years. Dowel bar corrosion leads to joint deterioration which requires either complete replacement of the concrete pavement or a dowel bar retrofit. California Department of Transportation has conducted a study on the corrosion rates of a variety of different dowel bars including epoxy coated, solid stainless steel, and stainless steel clad and has found that the epoxy coated bars failed the corrosion testing while the stainless steel bars both clad and solid experienced no corrosion. Washington State Department of Transportation currently requires the use of stainless steel clad dowel bars on all newly constructed concrete pavements in western Washington and mountain passes and corrosion resistant alternatives for eastern Washington. The anticipated life of a PCC pavement is an expected 40 years. PCC pavement rehabilitation will potentially require diamond grinding at the 20 to 30 year range to address studded tire wear.

4.4.2 HMA Issues

Design pavement thickness for heavy-traffic roadways such as Interstates and principal arterials to a depth so that future roadway reconstruction is not necessary and 20 years of traffic will not generate significant bottom-up fatigue cracking. Use thicker base courses to achieve longer-term roadway performance with future mill and fills or HMA overlays to address surface distress such as rutting, top-down cracking, and HMA surface aging.

4.5 Construction Considerations

Pavement construction issues are an important component of the selection of pavement type. These issues can include:

1. Pavement thickness constraints – Utilities below the pavement and overhead clearances may have an impact on the layer thickness, type, and may limit future overlay thickness.
2. Effects on detours, bypasses, and alternate routes – Consider the geometric and structural capacity of detours, bypasses, and alternate routes to accommodate rerouted traffic.
3. Effects of underground pipes and services on performance – Determine

the impact of existing utilities and future utility upgrades on initial and future rehabilitation treatments.

4. Anticipated future improvements and upgrades – Consider if the pavement type restricts or minimizes the ability to efficiently and cost effectively upgrade or improve the roadway width, geometry, or structural support.
5. Impact on maintenance operations including winter maintenance – Will the selected pavement type have impacts on surface and full-depth due to freeze-thaw or snow and ice removal?
6. Grades, curvature, and unique loadings such as slow-moving vehicles and starting and stopping – How will steep grades, curvature, and unique loadings impact pavement performance. Slow moving vehicles will generate increased strain levels in the HMA pavement structure and these strains can significantly impact pavement performance through rutting and cracking.
7. Scheduled analysis – Conduct analysis to determine critical construction features for things such as haul truck access, traffic control constraints, road closures and their impact on the project. Include staging analysis for multiple projects within the project corridor to ensure that alternate routes are free of traffic delay due to construction activities.
8. As-constructed smoothness – Smoother pavements last longer. Make every effort to build smoother pavements.

4.5.1 Other Factors

The second step is to identify project-specific constraints that will define one specific pavement type over another. These items can drive a need that may not show up in a LCCA. Typical items are listed below and remain minimal. A Region can develop a staged construction plan showing interim projects that will help meet an applicable corridor pavement strategy if it determines that a specific pavement type is not feasible. The following are situations that can fall under this procedure.

1. Historical Materials Performance Issues
    a. Stripping – susceptible aggregates
    b. Settlement problems – the region can place an asphalt road until settlement is removed if concrete is determined to be the final pavement type.
2. Performance Period Required
3. Utility Construction Issues
4. Research Scope within project
5. Existing Surface – the region can use overlay principles and change pavement types when the life of the pavement is exhausted.

4.6 Life Cycle Comparison – Alternate Bid Determination

Pavements that do not fall into a defined corridor or have specific constraints that prevent one type or the other will be addressed through a project specific LCCA in the

UDOT design process. Details of the UDOT LCCA process are listed below.

Results that show greater than 15 percent benefit in LCCA will be used to determine pavement type. This type will be carried through the design process and bid in the normal approach.

Results that show a 15 percent or less benefit in LCCA will facilitate the use of an alternate bid process. One rigid design and one flexible design will be carried through design and into the bid process. This will require items such as duplicate summary sheets, typical sections, and quantity determinations. The time and cost for duplication will vary based on project size. A typical interstate rehab on I-80 was about four weeks and $5,000. Typical projects that fall into this category are thick and thin overlays, major reconstruction, and new construction. Others will be dependent on project specific issues addressed in Section II.

LCCA provides a useful tool to assist in the pavement type selection. Only consider differential factors. The alternative resulting in the lowest net present value or annualized cost over a given analysis period is considered the most cost efficient. Life cycle costs refer to all costs that are involved with the construction, maintenance, rehabilitation, and associated user impacts of a pavement over a given analysis period. Life cycle cost analysis is an economic comparison of all feasible construction or rehabilitation alternatives evaluated over the same analysis period. A feasible alternative is one that fits with the required constraints such as geometric, construction time, traffic flow conditions, clearances, right-of-way, and maximum funds available. Evaluate at least one HMA and one PCC alternative. The total cost including initial construction, maintenance, rehabilitation, and user costs of each design alternative can be compared based on the present value or equivalent uniform annual cost. Conduct the life cycle cost analysis using the FHWA life cycle cost analysis software available at http://www.fhwa.dot.gov/infrastructure/asstmgmt/lccasoft.cfm

Determine user costs from an acceptable delay program such as the FHWA RealCost program with rates based on UDOT calculations. A simplified version that includes direct delay of travelers based on speed reduction through the work zone is most appropriate because it is defendable. The majority of full-blown user cost analyses are very time consuming and results can vary significantly to the point where they no longer make sense.

The FHWA policy on life cycle cost analysis is that it is a decision support tool and the results of the life cycle cost analysis are not decisions in and of themselves. The logical analytical evaluation framework that life cycle cost analysis fosters is as important as the life cycle cost analysis results themselves. Net present value is the economic efficiency indictor of choice. The annualized method is appropriate but derived from the net present value. Computation of cost/benefit ratios is not recommended because of the difficulty in sorting out costs and benefits for use in the ratios. Estimate future costs in constant dollars and discount to the present using a discount rate. The use of

constant dollars and discount rates eliminates the need to include an inflation factor for future costs.

4.6.1 Annualized Method

UDOT uses the annualized method for calculating LCCA. The annualized method is an economic procedure that requires converting all present and future expenditures to a uniform annual cost. This method reduces each alternative to a common base. The costs are equated into uniform annual costs through the use of an appropriate discount rate. Recurring costs such as annual maintenance are already expressed as annual costs. A given future expenditure such as a pavement overlay must first be converted to its present value before calculating its annualized cost. The general form of the Annualized cost equation is as follows:

$$A = PV \quad \frac{i(1 + i)^n}{(1 + i)^{n -} -1}$$

where

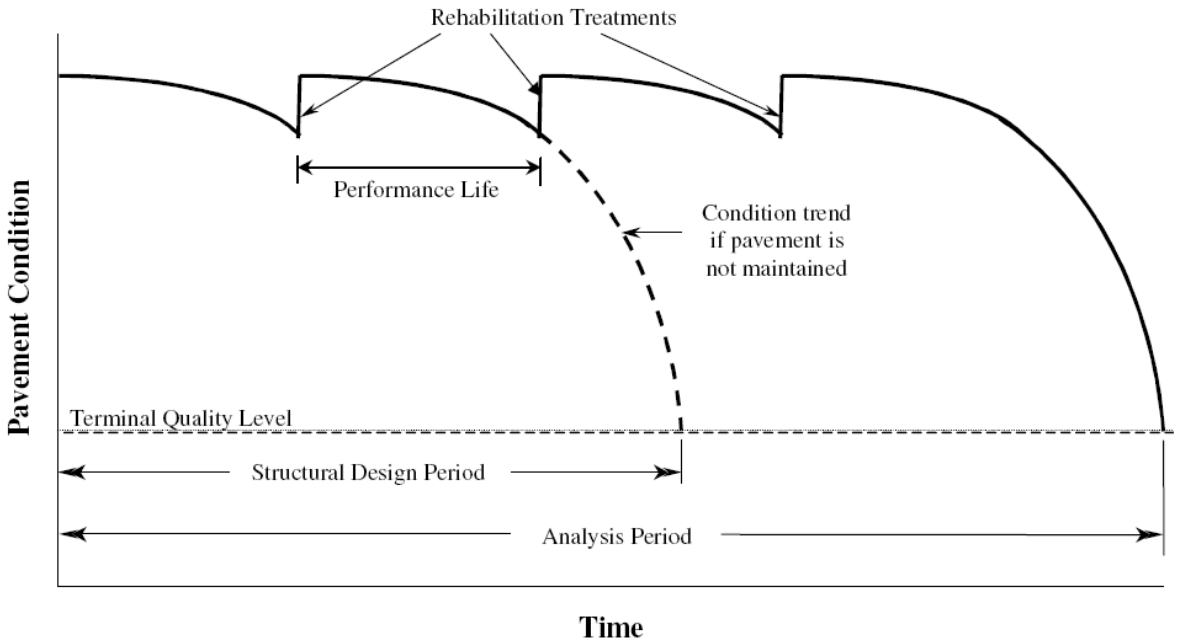
A = Annual cost
PV = Present Value
N = Number of years
I = Discount rate

4.6.2 Economic Analysis

The costs included in the analysis are costs related to planning, working on, and maintaining the pavement during its useful life. Account for all costs attributed to the alternative and that differ from one alternative. These include costs to the highway agencies and user costs.

4.6.3 Performance Period

Pavement condition gradually deteriorates as it ages to the point where some type of rehabilitation treatment is necessary. The timing between rehabilitation treatments is defined as the performance life. The diagram below illustrates performance life for the initial pavement design and subsequent rehabilitation activities have a major impact on life cycle cost analysis results.



Determine the performance life of the various rehabilitation alternatives on past performance history when available. Coordinate the performance life with improved engineering and technologies in instances where the anticipated performance life is not well established.

4.6.4 Initial Construction Costs

Unit costs vary according to location, material availability, project scope, and compliance standards. They can be estimated based on previous experiences by averaging the bids submitted for recent projects of similar scope. Typical item costs can be located in bid item tabulations. The bid item costs may need to be adjusted according to local availability and work constraints. Mobilization, engineering, contingencies, and preliminary engineering can be excluded for the initial construction cost estimate since these costs are similar for HMA and PCC. UDOT calculates each project based on current unit bid pricing and perceived increase due to material availability and labor cost.

4.6.5 Life Cycle Cost Analysis Process

1. UDOT uses the Annualized Method for calculating Life Cycle Costs.

$$A = P \times \left[\frac{I(1+I)^n}{(1+I)^{n-1}} \right]$$

A = Annualized Costs
P = Present Worth or Current costs
n = Number of years
I = Interest Rate (Discount Rate)

$$UCR = \left[\frac{I(1+I)^n}{(1+I)^{n-1}} \right]$$

2. UDOT recommends a four percent discount rate through simplifying the use of a variable rate based on economic values.
3. Current costs will consist of using the following portions and equations:
   a. Initial Construction Costs
      1) These items consist of either alternate bidding from contractors or estimating construction costs through the Pavement Management Engineer.
      2) UDOT focuses on what items need to be considered when calculating initial construction costs
   b. Rehab Costs
      1) UDOT uses plans for every section (PFES) to control what treatments to use and when each treatment will take place. Each road section has different criteria based on type, traffic volume, and use.
      2) UDOT is creating standard items to be included when determining the costs for each rehab project. This will allow PMEs to use similar rehab costs when determining projects throughout the state.
   c. User Costs
      1) Use the following as a minimum to calculate user costs. Each PME can look at other models based on items such as delay and alternate routes.

2) Equation:

$$UC = (AVT)\left[\frac{L}{RS} - \frac{L}{IS}\right](ADT)(PT(CP))$$

Where:

UC	=	User Cost (total project costs)
AVT	=	Value of Delay Time – From Tables Below
L	=	Project Length
RS	=	Reduced speed through construction zone
IS	=	Initial speed prior to construction zone
ADT	=	Average daily traffic in current year (only portion of ADT affected by the project
PT	=	Percent of the traffic affected by the construction project.
CP	=	Construction period in days

d. UDOT calculates each project based on current unit bid pricing and recalculates through the design phases.

4.6.6 Maintenance and Rehabilitation Costs

The type and frequency of future maintenance and rehabilitation operations vary according to the pavement type being considered. Knowing how a particular pavement type performed in the past is a valuable guide in predicting future performance. Determine costs as realistically and accurately as possible based on local context and specific project features.

The total construction cost should include items such as the cost of pavement resurfacing or PCC rehabilitation, milling or diamond grinding, shoulders, pavement repair, drainage and guardrail adjustments, and maintenance and protection of traffic. Include mobilization (5 percent), engineering and contingencies (15 percent), and preliminary engineering in all rehabilitation costs.

Table 4 – 2

Typical Maintenance Requirements During the Life of a Pavement		
Year	**All PCC Pavement**	**Interstate HMA Pavement**
0	Construction or Reconstruction	Construction or Reconstruction
6		Surface Seal (SS)
10	Joint Seal, Diamond Grinding, and Joint Repair	
12		SS
15		2.5 inch HMA overlay and SS
20	Partial and Full Depth Slab Repair	
21 and 24		SS
30	Joint Seal, Grinding, and Joint Repair	
36		SS
40	New Pavement	
42		SS
45		2.5 inch HMA overlay and SS
50		New Pavement

Table 4 – 3

Typical Maintenance Requirements During the Life of a Pavement		
Year	**HMA – High Volume (AADT > 5000)**	**HMA Pavement – Low Volume (ADDT < 5000)**
0	Construction or Reconstruction	Construction or Reconstruction
6		Surface Seal (SS)
8	Surface Seal	
12		SS
15	Structural Overlay and SS	
18		SS
23	SS	SS
30	Structural Overlay and SS	SS
38		SS
40		New Pavement
48	New Pavement	

Construction duration reflects the actual construction time required for each pavement type. Construction durations consider improvements, proposals, or innovative contracting procedures in construction processes. Include any difference that exists in routine maintenance costs between the various alternatives and include them in the life cycle cost analysis. The following table contains a probable scenario corresponding to average traffic and climate conditions assuming that state-of-the-art practices have been followed during construction and that maintenance and rehabilitation projects are carried out efficiently and on schedule.

Table 4 – 4

Typical Maintenance Requirements During the Life of a Pavement		
Year	**PCC Pavement**	**HMA Pavement**
0	Construction or Reconstruction	Construction or Reconstruction
15		2.5 inch rotomill and HMA overlay
20	Diamond Grinding	
30		2.5 inch HMA overlay
40	Diamond Grinding	
45		2.5 inch rotomill and HMA overlay
50	Salvage value if applicable	Salvage value if applicable

4.6.7 Salvage Value

Salvage value is the asset value at the end of the analysis period. The difference between the salvage values of the various alternatives for a project can be small because discounting can considerably reduce this value but the size of this reduction is influenced by the chosen discount rate. Predicting the proportion of pavement materials to recover or recycle at the end of the analysis period is uncertain. An alternative is considered to have no remaining salvage value if it has reached its full life cycle at the end of the analysis period. An alternative is given a salvage value if it has not completed a life cycle which is determined by multiplying the last construction or rehabilitation cost by the ratio of the remaining expected life cycle to the total expected life.

$$\text{Salvage Value} = CC \times TEL$$

where

CC = Last construction or rehabilitation project costs
ERL = Expected remaining life of the last construction or rehabilitation project
TEL = Total expected life of the last construction or rehabilitation project

4.6.8 User Costs

It is difficult to determine whether or not one rehabilitation alternative results in a higher vehicle operating cost than another. The user costs associated with each rehabilitation alternative is determined using only costs associated with user delay. This is based on the construction periods and the traffic volumes that are affected by each of the rehabilitation alternatives. Several studies have been performed that associate cost with the amount of time the user is delayed through a construction project. The method used is not as important as using the same method for each of the alternatives. The costs associated with user delays are estimated only if the effects on traffic differ among the alternatives being analyzed. User costs associated with delays for future rehabilitation work can be substantial for heavily travelled roadways, especially when

work is frequent. The recommended mean values and ranges for the value of time shown in the table below are reasonable but there are several different sources for the dollar value of time delay.

Table 4 – 5

Recommended Dollar Values per Vehicle Hour of Delay in 2012 Dollars		
Vehicle Class	**Value Per Vehicle Hour**	
	Value	**Range**
Passenger Vehicle	$13.96	$12 to $16
Single-Unit Trucks	$22.34	$20 to $24
Combination Trucks	$26.89	$25 to $29

The following equation is the minimum when calculating user costs. The PME can look at other models based on delay, alternate routes, or other items.

$$UC = (AVT)\left[\frac{L}{RS} - \frac{L}{IS}\right](ADT)(PT(CP))$$

Where:
UC = User Cost (total project costs)
AVT = Value of Delay Time
L = Project Length
RS = Reduced speed through construction zone
IS = Initial speed prior to construction zone
ADT = Average daily traffic in current year
(only portion of ADT affected by the project)
PT = Percent of the traffic affected by the construction project.
CP = Construction period in days

Surfacing types and characteristics influence the noise emitted on tire-to-pavement contact. The cost of a noise attenuation structure is planned must be included in the treatment costs of the alternative being analyzed. All safety issues can be addressed similarly.

4.6.9 Discount Rate

A discount rate is needed to compare costs occurring at different points in time in a life cycle cost analysis. The discount rate reduces the impact of future costs on the analysis, reflecting the fact that money has a time value. The discount rate is defined as the difference between the market interest rate and inflation using constant dollars.

Table 8 shows recent trends in the real treasury interest rates for various analysis periods published in the annual updates to OMB Circular.

Table 4 – 6

Real Treasury Interest Rates					
Year	**3-Year**	**5-Year**	**7-Year**	**10-Year**	**30-Year**
1979	2.8	3.4	4.1	4.6	5.4
1980	2.1	2.1	2.9	3.3	3.7
1981	3.6	3.9	4.3	4.4	4.8
1982	6.1	7.1	7.5	7.8	7.9
1983	4.2	4.7	5.0	5.3	5.6
1984	5.0	5.4	5.7	6.1	6.4
1985	5.9	6.5	6.8	7.1	7.4
1986	4.6	5.1	5.6	5.9	6.7
1987	2.8	3.1	3.5	3.8	4.4
1988	3.5	4.2	4.7	5.1	5.6
1989	4.1	4.8	5.3	5.8	6.1
1990	3.2	3.6	3.9	4.2	4.6
1991	3.2	3.5	3.7	3.9	4.2
1992	2.7	3.1	3.3	3.6	3.8
1993	3.1	3.6	3.9	4.3	4.5
1994	2.1	2.3	2.5	2.7	2.8
1995	4.2	4.5	4.6	4.8	4.9
1996	2.6	2.7	2.8	2.8	3.0
1997	3.2	3.3	3.4	3.5	3.6
1998	3.4	3.5	3.5	3.6	3.8
1999	2.6	2.7	2.7	2.7	2.9
2000	3.8	3.9	4.0	4.0	4.2
2001	3.2	3.2	3.2	3.2	3.2
2002	2.1	2.8	3.0	3.1	3.9
2003	1.6	1.9	2.2	2.5	3.2
2004	1.6	2.1	2.4	2.8	3.5
2005	1.7	2	2.3	2.5	3.1
2006	2.5	2.6	2.7	2.8	3
2007	2.5	2.6	2.7	2.8	3
2008	2.1	2.3	2.4	2.6	2.8
2009	0.9	1.6	1.9	2.4	2.7
2010	0.9	1.6	1.9	2.2	2.7
2011	-0.1	0.4	0.8	1.3	2.3
Average	**3.0**	**3.3**	**3.6**	**3.9**	**4.2**

UDOT has selected a discount rate of four (4.0) percent.

4.6.10 Analysis Period

The analysis period is the time used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated. The recommended analysis period coincides with the useful life of the most durable alternative. Past pavements were designed and analyzed for a 20 year performance period since the original Interstate Highway Act in 1956 required that traffic be considered through 1976. Longer analysis periods are now recommended for evaluation of long term strategies based on life cycle costs. Consider extending the analysis period to include one rehabilitation as shown in the following table.

Table 4 – 7

Rehabilitation Period	
Traffic Level	**Analysis Period (years)**
Interstate or Principal Arterial	40-50
Minor Arterial or Major Collector	20-30

4.7 Engineering Analysis

Conduct an engineering analysis when there are two viable alternatives from the completed pavement design analysis and the life cycle cost analysis. The Region provides its engineering analysis supporting the pavement type selection after the HMA and PCC alternatives are approximately equivalent. The fact that these are not easily quantified does not lessen their importance. These factors may be the overriding reason for making the final pavement type selection. Carefully review and consider these decision factors according to the corridor and the surrounding environment.

Do not use reasons or examples that have already been taken into account within the pavement design analysis when offering the engineering analysis for pavement type selection or the life cycle cost analysis. Do not use these examples in the engineering analysis:

1. Funds are available for more expensive pavement type.
2. Support for pavement type based on ESALs or ADT already accounted for in the life cycle cost analysis.
3. Support for the choice for pavement type based on user delay already accounted for in the life cycle cost analysis.

The Region includes the engineering reasons that drive the selection of one pavement type over another given that the life cycle costs are approximately equivalent. Not all factors will apply on every project nor will all factors have equal weight or importance on each project. Many factors are synergistic, combining or subtracting, depending on the selection and many of the factors are interrelated. Staff intimately familiar with the design goals of the entire project or entire corridor will make the engineering analysis evaluations.

Air pollution impacts – Will effects on traffic or effects during production affect the project or future preservation efforts?

Non-user impacts – How are surrounding neighborhoods affected by the project? How do these impacts vary depending on the type of pavement selected? What are the impacts at the point of production?

Haul routes through neighborhoods – Consider the impacts both during initial construction and future preservation projects.

Future ability of plants to operate at night in urban areas and associated cost increases. Where are typical production plants located? Will night work continue to be feasible in the area of plant production or will urban growth affect this? What possible effect will urban growth have on making production plants move further away from the corridor?

Neighborhood impacts due to trip diversion during preservation projects. – When a highway closure impacts the traveling public, many will divert to other routes to avoid delays. These diversions have associated costs. Some of the costs are user impacts that come from backups and delays on diversion routes. Some costs come from impacts to neighborhoods through increased traffic, noise, congestion, air pollution, safety, and accident risks. Consider the level of user delays, the likelihood that diversions will occur, and the level of impact these diversions will have on non-highway users.

Business impacts due to reduced or restricted access. – This impact is caused by diversions and direct impacts to users. The magnitude grows as an area urbanizes and increases the number of businesses that stay open for extended evening hours. Diversion through a neighborhood with extensive commercial business can greatly impact those businesses.

Effect of nighttime noise variances and risk of noise variance approval. – These two items tie in with reduced or restricted access. Nighttime noise variances become more difficult to obtain and more restrictive in their limitations as urban areas grow. Review the corridor in question and the expected growth projections to develop an idea of the risk associated with this non-user impact. Noise restrictions can limit hours of operations to the point of preventing work or they can restrict noise levels to below what is achievable by construction equipment. Noise restrictions apply also to vibration and noise generated by vibratory equipment and these restrictions can prevent the use of particular equipment within selected urban corridors. A single resident affected by nighttime noise can and has shut projects down forcing a move to day work and creating huge impacts on highway users through delay and impacts.

Noise

1. Pavement noise – Pavement surface texture effects noise through a

corridor. Some pavement surfaces are measurably quieter than others.

2. Noise walls – Evaluate whether the corridor already has noise walls or is expected to have noise walls by project construction time and the impacts of having or not having the walls might have on non-users and residents for construction noise and pavement noise.
3. Night haul through neighborhoods – Consider the impact of night hauling. Sparsely populated areas will obviously have a smaller impact from haul vehicle noises than densely packed urban areas.
4. Noise from diverted traffic and other impacts – Diverted traffic must drive through someone's neighborhood. Diverted night traffic, especially large trucks, can have a significant noise impact on neighborhoods.
5. Noise generation during preservation projects – Preservation projects to any pavement generates noise and impacts the local community including the noise from the initial construction.

Safety Concerns

1. Public exposure to traffic control during lane closures.
2. Worker exposure to traffic during lane closures.
3. Lane closures are a safety risk for workers and the traveling public. Limited vision, nighttime lighting, temporary traffic control, and other factors increase the risk of accidents to motorists and to workers within the work zone. Evaluate the risk to both based on the nature of the corridor, ADT, the degree of urbanization, and the complexity of the facility.
4. Maintenance crews performing routine maintenance activities.

Pavement Type

Maintain continuity within a corridor. Use similar architectural choices for structures and wall types and landscape architecture within a corridor. Do not switch pavement types over relatively short stretches of highway. Maintenance and preservation needs change for each pavement type. Change in pavement type impacts the public in various ways including aesthetically.

Environmental Effects

Evaluate runoff temperature due to heating on various pavement types while designing the storm sewer system.

Section 5
Site Condition Evaluation

5.1 Introduction

This section identifies UDOT standards, practices, and references and provides guidelines and work aids for site evaluations for new pavement design. It applies to site investigations for new construction performed for pavement thickness design including the development of performance related earthwork or pavement drainage designs. This category of site investigations primarily obtains information regarding soil support conditions and drainage requirements. Site investigations for geometric roadway locations, excavation or embankment design, slope stability analysis, structure design, construction material sources or pavement rehabilitation are not part of this section.

5.2 Scope of Site Evaluations

Site evaluations for new pavement design and construction are performed following a project specific testing plan that helps identify the necessary conditions to ensure the required information is obtained in the field.

Required Design Information

Pavement design site investigations are not intended to identify all conditions that can affect construction. Specific projects may require only some of the information listed below or may require additional information. Determine the elimination or addition of information during the initial consultant design-concept meeting.

Typical Site Investigation Data

1. Soil types exposed at the ground surface.
2. Soil conditions within a depth of 3 to 5 feet below the pavement subgrade including:
    a. Soil classification units
    b. In-place soil moisture content and density
    c. The occurrence of bedrock or boulders
    d. The occurrence of swelling soils
    e. Soil plastic and liquid limits
    f. Moisture-density compaction curves
    g. The occurrence of moisture induced collapsing soils
    h. Potential sources of settlement
    i. Laboratory and field CBR values
3. The depth to groundwater below the pavement subgrade
4. Subgrade support variability
5. The approximate vertical distance of the pavement surface above or below the adjacent ground surface

6. Soft or weak soils that will not support or will limit the size of earthwork equipment
7. Vegetation, debris, and other deleterious material that may affect pavement support
8. A hazard rating for frost damage
9. Water hazards
10. Performance of nearby pavements
11. Design CBR for road sections

Site Evaluation/Testing Plans

Develop site evaluation and testing plans with the Regional Pavement Management Engineer. Coordinate with the UDOT Geotechnical section when the project includes soils investigations for purposes other than pavement design. These plans must include exploration and sampling strategy, preliminary boring or test pit locations, and testing criteria. Use the information listed in the preceding section as a check list for developing plans.

5.3 Field Investigations

Field investigations include planning, locating, drilling, and collecting borings and samples for laboratory testing. Obtain information regarding the alignment and preliminary grade before a field investigation begins to assist in the proper location of the test holes. Sample the soils to determine both the vertical and horizontal boundaries of each significant soil type affecting the pavement support.

Objectives for sampling include:

1. Classifying soils
2. Obtaining material for laboratory testing
3. Characterizing material pavement support characteristics
4. Delineating soil boundaries with varying pavement support values or verifying or documenting uniformity. The number and size of the samples are adequate for determining soil classification, moisture-density curves, CBR values, and other laboratory testing appropriate for a specific project. Develop a preliminary laboratory testing plan before the field investigation is completed and integrate into the field testing plan to assure that the number and size of samples is adequate. Take contingency samples to accommodate unexpected variations of laboratory test results.

5.4 Boring or Sample Locations

Determine location of the borings and samples based on the centerline location of the planned road, width of the road, and the expected soil conditions for the area. Spacing of the test holes will be controlled by the type and profile of the soil at each location. UDOT includes using 200 feet as a starting interval for long road sections for

exploration locations and varying this interval up to a maximum of 1,000 feet for uniform conditions. Investigate intermediate locations if the soil types significantly change between test holes. Extend exploration depth to 10 feet below the final pavement elevation.

The number and location of samples or borings will determine the reliability of the pavement design and the cost-effectiveness of the investigation. Locate borings, samples, and other explorations so the sites can be found during construction. Reference the locations to:

1. A construction station
2. Road centerline
3. Elevations or road grades where possible

5.5 Sampling Techniques

There are two basic sampling techniques that can be applied to the investigations – systematic or representative. UDOT basic practice is to use systematic sampling.

Systematic Sampling

Systematic sampling is performed at uniform horizontal or vertical intervals. Intermediate locations are sampled when varying conditions are encountered. All the soil samples are classified either visually or in the laboratory.

Representative Sampling

Representative sampling and testing consists of taking samples that are believed to represent the typical or conservative soil support values. This type of sampling is based primarily on personal judgment, involves a few samples, and generally is only appropriate for small projects or verifying existing data.

5.6 Laboratory Investigations

Laboratory investigations involve the performance of all required testing and classification necessary to adequately identify the soil conditions for the entire project. Test according to the appropriate AASHTO, ASTM, and UDOT procedures and fully document.

Soil Classification

Soils are classified to group material with similar engineering properties and to identify soil types specified in construction contracts. Classify all soil samples according to the AASHTO Soil Classification System. This classification system is used in UDOT construction specifications. Use the Unified Soil Classification System to help further identify soil classes with smaller ranges of CBR values than the soil classifications in the AASHTO system. The Unified Soil Classification System can be found in the Appendix C.

The **AASHTO Soil Classification System** was developed by the American Association of State Highway and Transportation Officials, and is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction purposes. The classification system was first developed by Hogentogler and Terzaghi in 1929, but has been revised several times since.

Table 5 – 1

AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282)

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[1]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Note (1): Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

5.7 Selection of Design CBR

Determine subgrade soil CBR values using samples compacted at optimum moisture content to 100 percent of the maximum density obtainable by the AASHTO T-99D method of compaction. Perform CBR tests according to AASHTO T-193 except use a standard surcharge weight of 10 pounds for soaking and the penetration test of all samples.

Use a CBR for pavement design purposes with a confidence level of 90 percent for a normal distribution of values. The lowest CBR value will be used for design if it is determined that there is an insufficient number of CBR tests. This will be determined by the Regional Pavement Management Engineer. The following table indicates the number of CBR tests that will provide a 90 percent confidence level that the average test value is within plus or minus 1 unit of the average for a normal t-distribution of values.

Table 5 – 2

CBR Test						
CBR Test Range	1	2	3	4	5	6
Number of Required Tests	2	3	4	5	6	8-9

Correlation of Range vs. Number of Tests for 90 percent Confidence

Reliability (safety) is a specific input factor in the AASHTO ME Design Design Guide. Use of the AASHTO design procedure reliability factor in the structural thickness design will account for any additional variability in subgrade support values relative to the selected value used in the design.

5.8 Data Presentation

Document and tabulate the results of the site investigation for use and review in the pavement design process as part of the project pavement design. Include the following site information with a pavement design report:

1. A summary of the soil types and test results on site plans that show the sample or test-hole locations.
2. CBR test results including load-penetration curves, moisture contents at set up/after swell/top 1 inch after penetration, percent relative compaction, compaction method, percent swell, AASHTO soil classification, liquid limit and plastic limit.
3. Verify during construction any conditions that controlled the pavement design such as soil type and subgrade moisture conditions. Review the pavement design and modify if conditions are significantly different from those determined by the site evaluation.
4. Climatic conditions that will require changes or alterations to the structural pavement sections. These items include such issues as frost hazards, water hazards and unstable sub grades.

Example Table

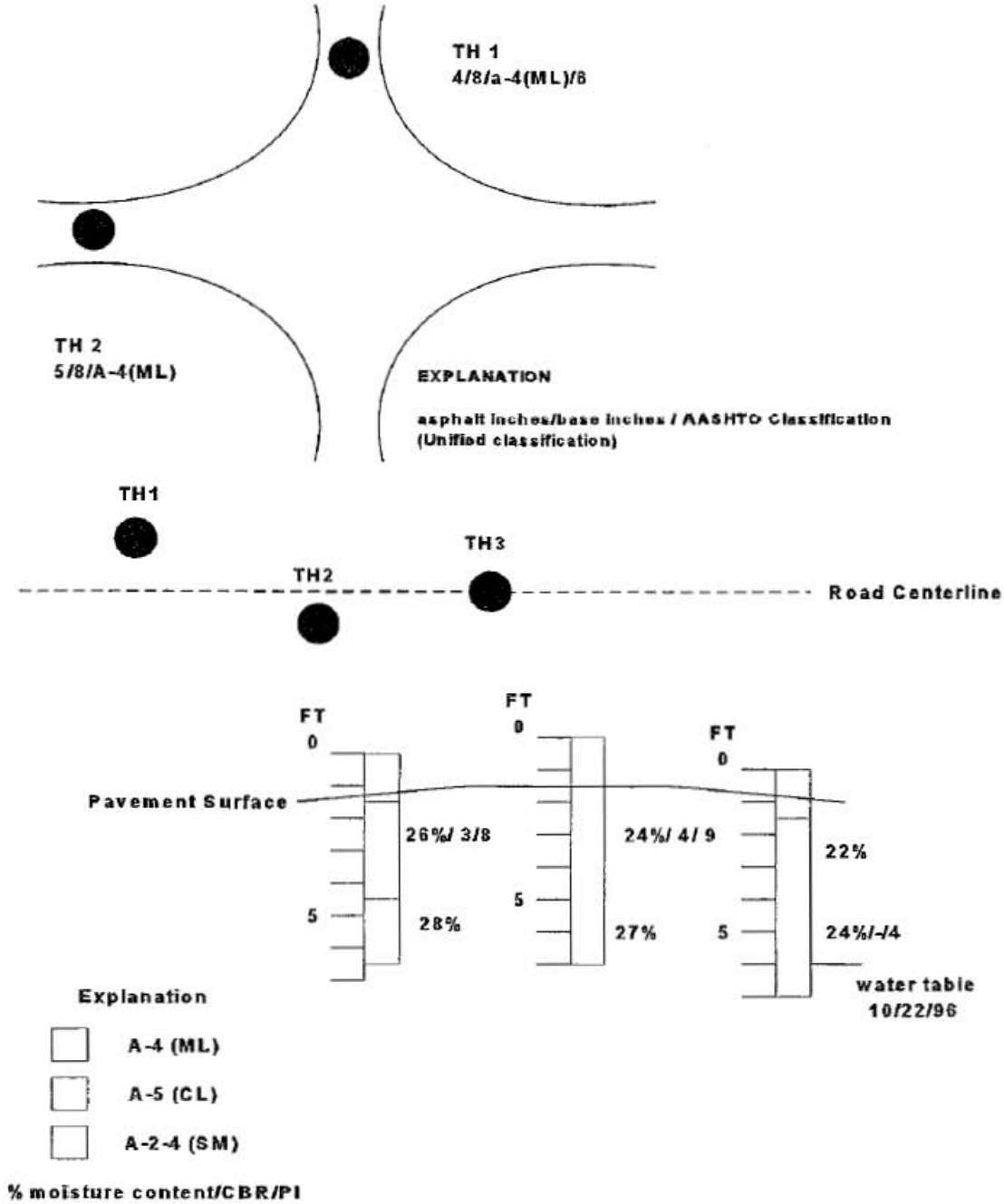
Coring Information

Date: 2/19/08

Project Name: MP 12.6 – 15.9 Station Location: Weather: cold and clear

Core #	Mile Post, Lane, Wheel Path	Location	Depth Inches	Description of Core, Road Surface, Water Table
1	NB lane mp 12.6 OSWP		8½	
2	SB lane mp12.85 OSWP		7¾	
3	NB lane mp 12.1 ISWP		8½	
4	SB lane mp 13.35 ISWP		8¾	
5	NB lane mp 13.6 OSWP		8¾	
6	SB lane mp 13.85 OSWP		8¼	
7	NB lane mp 14.25 ISWP		10½	
8	SB lane mp 14.75 ISWP		10	
9	NB lane mp 15.25 OSWP		9¼	

Comments: Overall condition of road fairly good, some minor rutting.



Example Subsurface Summaries

Frost Hazard Identification

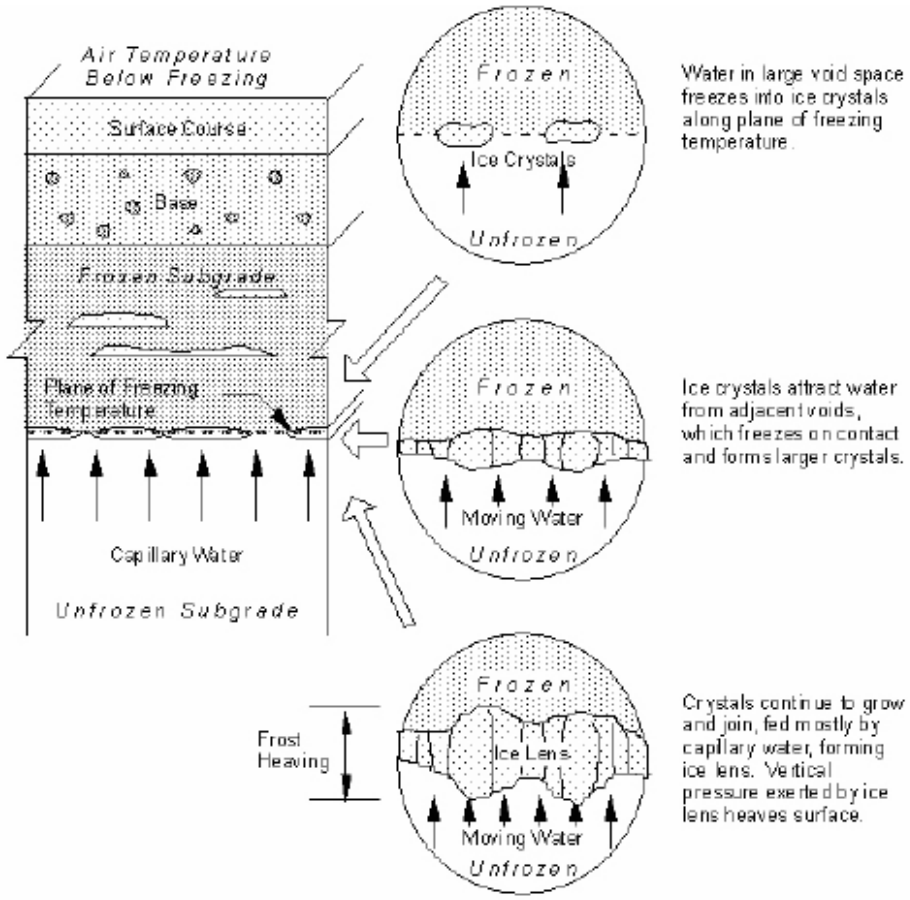
Frost action creates a hazard to pavement performance by causing the surface to heave and by decreasing the soil support by increasing the moisture content above the normal saturation content.

Frost Action

Frost action can be detrimental to pavements and can refer to the two separate but related processes of frost heave and thaw weakening.

Frost Heave

Frost heave is an upward movement of the subgrade resulting from the expansion of accumulated soil moisture as it freezes. Frost heaving of soil is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins, or other ice masses. An ice lens grows through capillary rise and thickens in the direction of heat transfer until the water supply is depleted or until freezing conditions at the freezing interface no longer support further crystallization. The overlying soil and pavement will heave up potentially resulting in a cracked, rough pavement as the ice lens grows. This problem occurs primarily in soils containing fine particles often called frost susceptible soils, while clean sands and gravels are non-frost susceptible (NFS). The degree of frost susceptibility is mainly a function of the percentage of fine particles within the soil. Many agencies classify materials as being frost susceptible if 10 percent or more passed a 0.075 mm (No. 200) sieve or 3 percent or more passed a 0.02 mm (No. 635) sieve. The following diagram illustrates the formation of ice lenses in a frost susceptible soil.



The three elements necessary for ice lenses and frost heave are:

1. Frost susceptible soil – significant amount of fines.
2. Subfreezing temperatures – freezing temperatures must penetrate the soil and the thickness of an ice lens will be thicker with slower rates of freezing.
3. Water must be available from the groundwater table, infiltration, an aquifer, or held within the voids of fine – grained soil.

Remove any of the three conditions above and frost effects will be eliminated or at least minimized. Heaving will be uniform if the three conditions occur uniformly otherwise differential heaving will occur resulting in pavement cracking and roughness. Differential heave is more likely to occur at locations such as:

1. Sub grades that change from clean, not frost susceptible (NFS) sands to silty, frost susceptible materials.
2. Abrupt transitions from cut to fill with groundwater close to the surface.
3. Excavation that exposes water-bearing strata.
4. Drains and culverts with different backfill material or compaction.
5. Open buried pipes change the thermal conditions such as removing heat resulting in more frozen soil.

Additional factors that will affect the degree of frost susceptibility or ability of a soil to heave:

1. Rate of heat removal
2. Temperature gradient
3. Water mobility and soil permeability
4. Water table depth
5. Soil type and condition including density, texture, and structure

The Casagrande Criterion

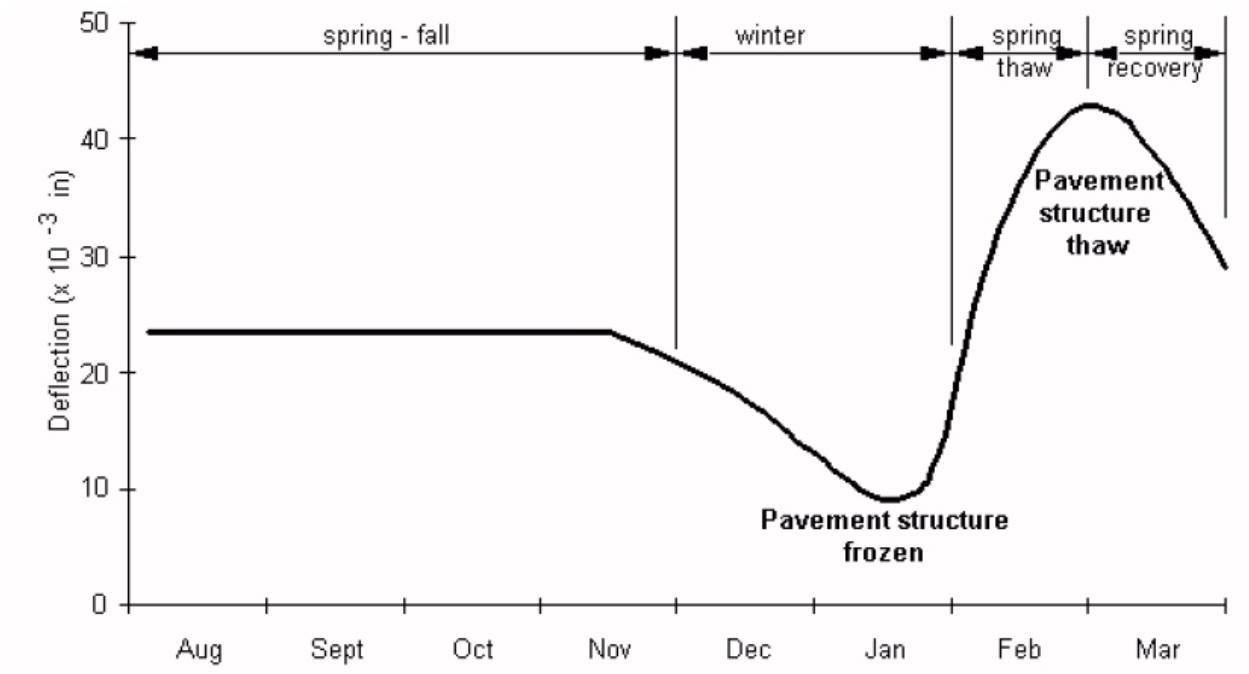
In 1932, Dr. Arthur Casagrande proposed the following widely known criterion for identifying potentially frost susceptible soils:

"Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3 percent of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

Application of the Casagrande criterion requires a hydrometer test of a soil suspension in water to determine the distribution of particles passing the 0.075 mm sieve and to compute the percentage of particles finer than 0.02 mm. UDOT uses untreated base course (UTBC) as a frost resistant aggregate because it has a maximum of only 7-11 percent passing the 0.075 mm (No. 200).

Thaw Weakening

Thaw weakening is a subgrade condition resulting from soil saturation as ice within the soil melts. The ice cannot drain out of the soil fast enough as it turns to liquid and the subgrade becomes substantially weaker and less stiff and loses bearing capacity. Loading that will not normally damage a given pavement may be detrimental during thaw periods. The follow diagram is an example of typical pavement deflection changes throughout the year caused by winter freezing and spring thawing.



Thawing can proceed from the top downward, from the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take.

The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition. This observation shows that:

1. The reduced density of base or subgrade materials helps to explain the long recovery period for material stiffness or strength following thawing.
2. Refreezing following an initial thaw can create the potential for greater weakening when the final thaw does occur.

Water Sources

The two basic forms of frost heave and thaw both require water. Water sources can be separated into two broad categories:

1. Surface water – Enters the pavement primarily by infiltration through surface cracks and joints and through adjacent unpaved surfaces during periods of rain and melting snow and ice. Many crack-free pavements are not entirely impermeable to moisture.
2. Subsurface water – Comes from three primary sources:
   a. Groundwater table (or perched water table).
   b. Moisture held in soil voids or drawn upward from a water table by capillary forces.
   c. Moisture that moves laterally beneath a pavement from an external source such as a pervious water bearing strata.

The frost hazard may be rated as high or low for frost susceptible soils within the frost zone according to the following conditions. An unknown rating may be appropriate when conditions for both high and low ratings occur and cannot be resolved or when little or no information is available. The inclusion of a frost hazard rating in the site evaluation documentation verifies that an evaluation of frost action has been attempted and has not been overlooked. When the rating is unknown, a decision to include frost action mitigation measures in a design will be based more upon the unacceptableness of frost damage than the probability of occurrence.

Estimating Pavement Freeze or Thaw Depths

UDOT estimates detailed freezing or thawing depth in pavement structures in the following ways. Use one of these methods to determine the estimated frost depth as a beginning point for the calculate value in ME Design.

1. Check the Western Regional Climate Center website for historical weather data from data collection sites nearest your project.
   http://www.wrcc.dri.edu/CLIMATEDATA.html
   Click
   – Historical Climate Information
   – Western US Historical Summaries (individual stations)
   – Utah Climatological Data Summaries
   – Monthly Temperature Listing
2. Transfer data to UDOT Excel spreadsheet called Frost Depth Calculation that can be downloaded from
   http://www.udot.utah.gov/main/f?p=100:pg:0:::1:T,V:1893,

   Enter the following information
   – # of day/month
   – Average maximum temperature for each month during the year

– Average minimum temperature for each month during the year
Calculate:
+ N = Average maximum temperature plus August minimum temperature

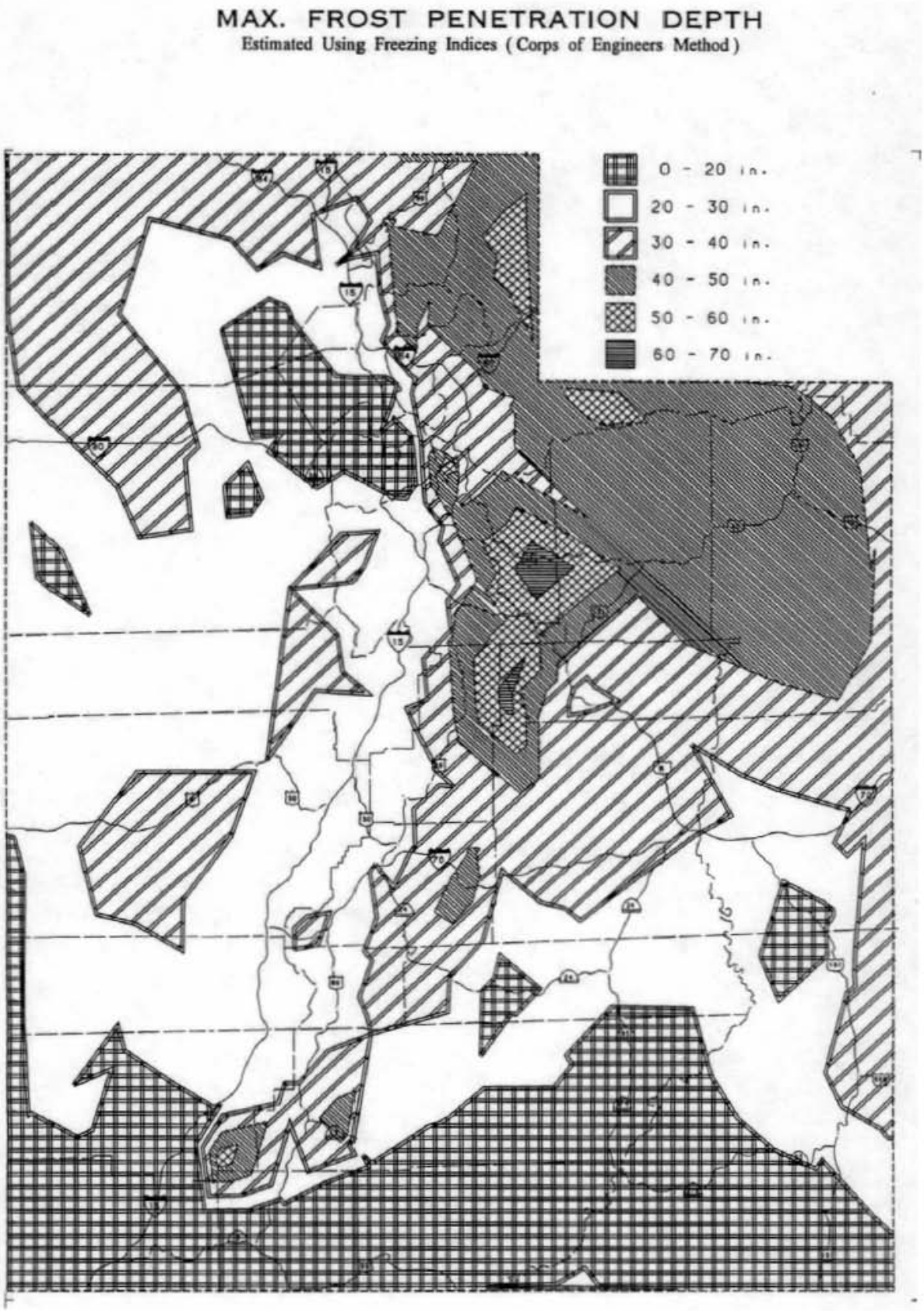
Freezing index = ∑ (32 – N) (# day) =

This is the sum of all months with freezing temperatures below 32 degrees. If N < 32 then (32 – N) * (# days) is calculated for each month and then added together to get total degree days

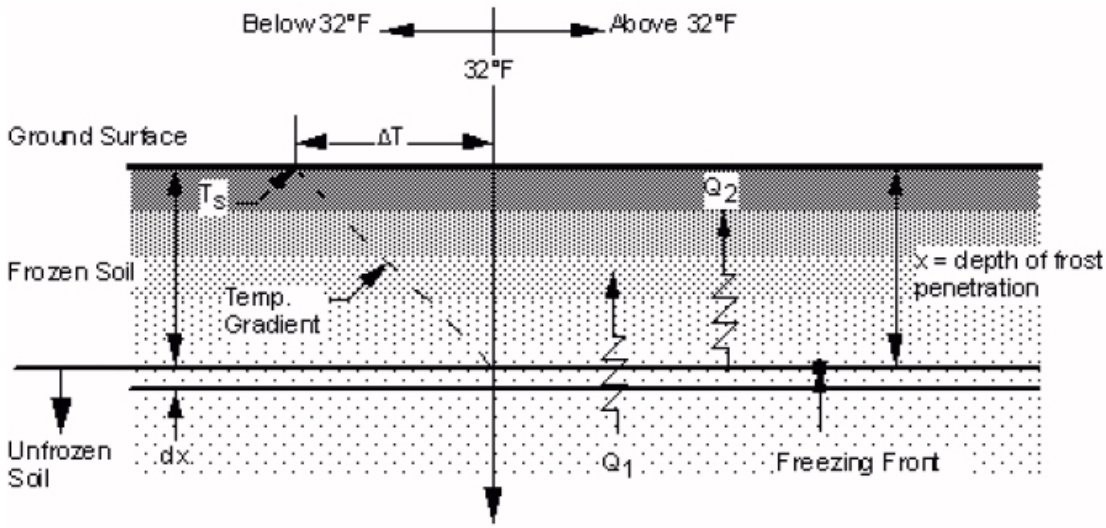
Frost Penetration is approximately equal to 1.482 (freezing index)$^{0.4911}$

This frost depth information should be used in the formulation of initial ME Design pavement sections and selection of base and subbase materials. ME Design computes the frost depth over the years of climate data available and this information may be useful in formulating a design.

The following map shows typical depths to the frost line at varying locations within the State of Utah and can be used for a quick reference to estimated frost depths. Many of the soils above the frost line in Utah may be classified as frost susceptible according to grain size classification criteria. The most distinguishing factor for identifying pavement frost hazards is water supply.


MAX. FROST PENETRATION DEPTH
Estimated Using Freezing Indices (Corps of Engineers Method)
0 - 20 in.
20 - 30 in.
30 - 40 in.
40 - 50 in.
50 - 60 in.
60 - 70 in.

A freeze depth estimate is helpful in designing for frost conditions but oversimplifies the complex conditions that accompany various pavement materials, freeze depths, and water sources. Some of the first studies of freeze/thaw depth were made by Josef Stefan in 1889, in connection with ice formation and melting in the Polar oceans. In this formula it is assumed that the latent heat of soil moisture is the only heat that must be removed when freezing the soil. Thermal energy stored as volumetric heat and released as soil-temperatures drop to and below freezing is not considered. The Stefan Formula tends to overestimate frost depth in temperate zones because volumetric heat is neglected. The latent heat supplied by the soil moisture as it freezes a depth dx in time dt = rate at which heat is conducted to the ground surface. This is illustrated in the sketch below:



Heat removal process can be represented by:

$$Q_1 = L\frac{dx}{dt}$$ Heat released by freezing a layer of soil dx thick in time dt

$$Q_2 = \frac{\Delta T}{R} = \frac{\Delta T}{\left(x/k_f \right)}$$ Heat conducted through frozen layer

And $Q_1 = Q_2$ so $L\frac{dx}{dt} = k_f \frac{\Delta T}{x}$

By integrating and solving for x

$$x = \sqrt{\frac{2k_f}{L}\int \Delta T dt}$$

Where $\int \Delta T dt$ is in units of ºF ● hr and is called surface freezing index. The freezing index normally expressed as ºF ● days. Rewrite the equation and add an "n" factor which results in

$$x = \sqrt{\frac{48 k_f n F I}{L}}$$

Climate and Drainage

Consider both surface runoff and subsurface water control. Effective drainage design prevents the pavement structure from becoming saturated. Effective drainage is essential for proper pavement performance and is assumed in the structural design procedure.

The conditions associated with a <u>high</u> frost hazard potential include:

1. Water table within 10 feet of the pavement surface
2. Observed frost heaves in the area
3. U.S. Army Corps of Engineers considers inorganic soils containing more than 3 percent (by weight) or more grains finer than 0.02 mm in diameter to be frost susceptible for pavement design purposes.
4. Potential for the ponding of surface water and the occurrence of soils between the frost zone beneath the pavement and the surface water with permeabilities high enough to enable seepage to saturate soils within the frost zone beneath the pavement during the term of ponding.

The conditions associated with a <u>low</u> frost hazard potential include:

1. Water table greater than 20 feet below the pavement surface
2. Natural moisture content in the frost zone low versus the saturation level
3. See page barriers between the water supply and the frost zone
4. Existing pavements or sidewalks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have not experienced frost damage
5. Pavements on embankments with surfaces more than 3 to 6 feet above the adjacent grades

Mitigating Frost Action

Mitigating frost action and its detrimental effects generally involves structural design considerations as well as other techniques applied to the base and subgrade to limit the effects of frost action. The basic methods used can be broadly categorized into the following techniques:

Limit the depth of frost into the subgrade soils. This is typically accomplished by specifying the pavement depth to be a minimum percentage of the frost depth. The depth of frost-susceptible subgrade under the pavement between the bottom of the pavement structure and frost depth is reduced by extending the pavement section well into the frost depth. The assumption is that a reduced depth of soil under frost action will cause correspondingly less damage.

Remove and replace frost-susceptible subgrade. Ideally the subgrade will be removed at least down to the typical frost depth. Removing frost-susceptible soils removes frost action.

Design the pavement structure based on reduced subgrade support. This method simply increases the pavement thickness to account for the damage and loss of support caused by frost action. This method can be susceptible to frost heave related distresses.

Providing a capillary break. Frost action will be less severe by breaking the capillary flow path because frost heaving requires substantially more water than is naturally available in the soil pores.

Freezing and Thawing Implications for Maintenance Operations

Use the calculated freezing index (FI) and thawing index (TI) to estimate the depth of freeze at a specific site and the resulting thaw. Maintenance personnel can use the TI to assess the need for seasonal load limits. Consider the following general guidelines relative to spring highway load restrictions:

Where to apply load restrictions. Spring thaw deflections greater than 45 to 50 percent of summer deflections suggest a need for load restriction. Consider freeze depth, pavement surface thickness, moisture condition, type of subgrade, and local experience. Subgrades with Unified Soil Classifications of ML, MH, CL, and CH will result in the largest pavement weakening.

Amount of load reduction. The minimum load reduction level is 20 percent. Load reductions greater than 60 percent generally are not warranted based on potential pavement damage. A load reduction range of 40 to 50 percent will accommodate a wide range of pavement conditions.

When to apply load restrictions. Apply load restrictions after accumulating a Thawing Index (TI) of about 25 degree F-days and applied at a TI of about 50 degree F-days both based on an air temperature datum of 29 degree F. Corresponding TI levels are less for thin pavements such as two inches of HMA and six inches of aggregate base or less.

When to remove load restrictions. Two approaches based on air temperatures are

recommended. The duration of the load restriction period can be directly estimated by the following function of Freezing Index (FI):

Duration (days) = 25 + 0.01 (FI)

The duration can also be estimated by use of TI and the following rough relationship:

TI = 0.3 (FI)

Frost Action Summary

Frost action is a critical pavement structural design concern in parts of the country that regularly experience ground freezing. Severe frost action can destroy a new pavement in a matter of one or two years without proper precautions. There are two basic types of frost action:

1. **Frost heave**. Results from accumulation of moisture in the soil during the freezing period. These accumulations (ice lenses) expand perpendicular to the direction of heat flow and push the pavement up, often causing severe cracking.
2. **Thaw weakening**. Once a subgrade is frozen it can be severely weakened when it thaws, usually in the spring time. Loading that will not normally damage a pavement may be detrimental during thaw periods.

Frost action can be further characterized by the typical depth to which the subgrade freezes in a particular area. This depth can be used as a pavement structural design input to mitigate the detrimental effects of frost action once this depth is known. Mitigation techniques can be classified into four broad categories:

1. Limit the depth of frost-susceptible material under the pavement structure.
2. Remove and replace the frost-susceptible subgrade.
3. Design the pavement structure based on reduced subgrade support.
4. Force a break in the groundwater's capillary path.

Severe pavement damage in the case of frost heave or a loss of bearing capacity in the case of thaw weakening can result if frost action cannot be adequately mitigated. Maintenance options to correct these problems are limited to pavement repair or replacement or limiting pavement loading during spring thawing.

5.9 Water Hazards

Water hazards are those conditions that may require the design of drainage provisions to improve pavement performance.

Conditions that may be classified as water hazards include:

1. Potential for surface water to pond in close proximity to pavement
2. Water table within a 6 foot depth of the pavement surface.
3. Spring beneath the pavement
4. Intermittent sources of water that can cause seepage beneath and within a 5 foot depth of the pavement surface.

Section 6
Traffic Data Analysis for Pavement Design and Evaluation

6.1 Introduction

UDOT follows FHWA Highway Performance Monitoring System (HPMS) requirements. The HPMS system requires UDOT to set up a statewide sampling frame. The volume is collected at over 5,000 sites with one third collected each year. The volume is collected for a minimum of 48 hours at three sites. Three hundred random sites were selected for a traffic classification count from these volume sites. Counts are performed over a three year period with 100 of these classifications counts performed per year. Ninety sites were selected out of the 300 classifications sites in order to meet HPMS requirements for collecting WIM data and calculate truck factors for functional classes. It is this sampling process that produces the design factors and growth rates for volume, vehicle classifications, and weigh-in-motions data for defining axle load spectra for each vehicle classification used in the design process.

The axle load spectra, in conjunction with the measured or calculated AADT and truck percentages, are direct inputs into ME Design. These files can be electronically imported and requested from UDOT. Use of the default distributions contained within ME Design may be used for smaller projects but should be discussed with the Region PME prior to implementation. ME Design calculates the total number of trucks in the design lane over the design period. The total number of trucks is a good overall index of the level of truck traffic for both flexible and rigid pavements. The pavement should be considered as very heavily loaded for example if the number of trucks over the design life in the design lane is over 25 million. The design is considered as low traffic if the number of trucks is less than 100,000.

All project traffic information can be requested through UDOT for all state routes where traffic information is available using the Traffic Request Form available at http://www.udot.utah.gov/main/f?p=100:pg:1594831487200005106:::1:T,V:507. Make requests through the Region Pavement Management Engineer and the Planning Division statistical engineer. The traffic information will be supplied in most cases by UDOT but there will be a number of limited projects that UDOT will not have the desired traffic information. It will be these projects that require manual project design AADT and truck percentage calculations to be performed by the design consultant, and then used in conjunction with one of the default axle distributions in ME Design. The design AADT and truck percentage calculations are coordinated and approved through the Region Pavement Management Engineer and performed by a consultant.

The Region Pavement Management Engineer meets with the consultant during a mandatory initial consultant design concept meeting to discuss many design considerations including any traffic calculations. The initial concept meeting includes

the Region Pavement Management Engineer, design consultants, and other UDOT representatives to discuss project specific details. The consultant submits a letter for approval to the Region Pavement Management Engineer outlining the pavement design details agreed upon in the initial consultant design-concept meeting. This approval is project specific and is done on a project-by-project basis. A pavement design performed without an approved traffic collection methodology may be rejected. The approved method may contain one or more of the methodologies defined or discussed in the traffic data analysis section of this manual.

PROJECT TRAFFIC DATA
REQUEST FORM

State Route Number:_____ Project Number (if programmed):_____
Project Description:_____
Milepost - Begin:_____ End:_____
Construction Year:_____
Number of Lanes (after construction-both directions):_____
Design Period (check two): ☐ 5 years ☐ 10 years ☐☐ 20 years
☐ 30 years ☐ 40 years ☐ ___ years

Project Scope (check any potential strategies):

☐ Overlay ☐ Reconstruction Widen-Add Lanes

☐ New Alignment ☐ Bridge Replacement

☐ Inlay ☐ Recycle

Other:

Pavement Type: ☐ Bituminous ☐ Concrete ☐ Both Required

Any Special Requirements or Comments:
_____
_____
_____
_____

Requested By:_____
Date of Request:_____
If Urgent Call, Planning Statistics Engineer at 964-4552.

6.2 Analysis Period

The analysis period refers to the total period of time that will cover the design life of the particular pavement design strategy. The design life will be determined by the UDOT Regional Pavement Management Engineer at the initial concept meeting when consultants perform pavement designs for UDOT. The performance period must take into account many design strategy considerations in addition to the pavement section in question which may or may not be known to a design consultant. The Regional Pavement Management Teams determine pavement design life by incorporating the knowledge of planned future corridor projects, future widening and overlay projects, safety projects as well as information from other management systems besides pavement management, while still balancing the design and capacity needs with funding.

6.3 Manual Traffic Counts

Some UDOT Local Government projects can have volume information provided by HPMS. There are pavement design projects where UDOT does not have the traffic information required to calculate the project-specific accumulative design period Equivalent Single Axle Loads. Manual counts may be required in these instances as the only manner available to calculate the percent of truck traffic on any given pavement section.

Many local roads will not have a great deal of truck traffic. On some routes, such as I-15, it may be possible to need as many as three people to collect data on a manual count. This is a rare case where UDOT may not have traffic data on a route with three lanes in one direction.

1. **Volume** – Collect the volume using tubes or permanent counters over a minimum of 48 hours period of time. These short time counts need to be adjusted to an Average Annual Daily Traffic (AADT). UDOT System Planning and Programming can provide the factors.

2. **Classification** – A manual classification count is probably necessary in order to collect classification information. WIM equipment can be used if it is available, the use of two tubes, or the use of Permanent Traffic Recorders (ATR's classify by length). The classification requires a minimum of a 7 hour count, making sure it includes either an a.m. or p.m. peak hour. These short time counts require adjustments to an annual average. Contact UDOT Planning Division for the factors.

3. **Growth Rates For Design Year Traffic** – Contact UDOT Planning Division to obtain the growth factors needed to estimate the 48 hour volume data and the 7 hour classification data into a full, one-year traffic count (current design year traffic count).

4. **Growth Rates For Volume, Classification, and Axle Configuration** – The Regional Pavement Management Engineer can give guidance when needed in the areas where there are no growth rates available for volume, classification, and axle configurations.

6.4 Vehicle Volume and Classification

Network volume and classification data are generally compiled according to rural or urban area type; functional classification such as interstate, principal arterial, minor arterial, or collector; and volume group as defined by AASHTO highway functions. This type of data may be adequate for planning and preliminary design purposes but it may produce ESAL estimation errors when used for the design or evaluation of individual projects. The type of data collected will be determined in the initial concept meeting when a consultant is providing project specific traffic data.

Project-specific data that is collected over a short period of time must be adjusted for hourly, daily, and monthly or seasonal variations as determined from continuous count data. These variations include but are not limited to highway functional class, geometric type, vehicle volume, vehicle classification, vehicle axle configurations, and growth rates for all vehicle information types.

6.5 Vehicle Classification

Vehicles are typically classified according to whether or not they carry passengers. Further classification is based on gross weight and axle configuration such as three-axle single unit trucks, six-axle multi-trailer trucks, etc. FHWA has adopted the following 13 vehicle classifications that are grouped into five categories and used in the process of calculating design traffic when design traffic is not available through UDOT.

Vehicle Classes

Category I – Two Axle Vehicles

Class 1	Motorcycles
Class 2	Passenger Cars
Class 3	Other Two-Axle, Four-Tire, Single-Unit Vehicles

Category II – Buses

Class 4	Buses

Category III – Single Unit Trucks

Class 5	Two-Axle, Six-Tire, Single-Unit Trucks
Class 6	Three-Axle, Single-Unit Trucks
Class 7	Four or More Axle Single-Unit Trucks

Category IV – Single Trailer Trucks

Class 8	Four or less axle single trailer combinations
Class 9	Five-axle single trailer combinations
Class 10	Six or more axle single trailer combinations

Category V – Multi-trailer Trucks

Class 11	Five or Less Axle Multi-Trailer combinations
Class 12	Six-axle, multi-trailer combinations
Class 13	Seven or more axle multi-trailer combinations

6.6 Equivalent Single Axle Loads (For AASHTO 1993 Design Process Projects

Different wheel loads and load configurations produce different stresses and strains in various layers of a pavement structure, with larger and more concentrated loads producing larger stresses and strains. The repeated application of these stresses and strains causes load-related pavement deterioration and fatigue. Equivalent single axle loads (ESAL) factors were developed from observing load-related distresses. An ESAL Factor is the ratio of the number of applications for a given load and axle configuration required to produce the same deterioration as a number of applications of a standard load and axle configuration. ESAL factors vary according to flexible or rigid pavement type, pavement structure including structure number or slab thickness, axle load, axle configuration, tire pressure, terminal serviceability level, serviceability loss, and route functional class.

NOTE: Traffic analysis is based on ESAL factors for pavement structural design. See ESAL Calculations Table and UDOT Functional Classification Code table.

ESAL Calculations

Project Description:

State Route	
Begin MP	
End MP	
Project Scope	
Region	
Rigid or Flexible	
Construction	
Functional Class	
Design Period	
SN	

Vehicle Types	AADT for Base Year Current Traffic (A)	Growth Factors (B)	Design Traffic Column A * B (C)	ESAL Factor (D)	Design ESAL Columns C * D (E)
Category 1 Axel Class 1-3					
Category II Axel Class 4					
Category III Axel Class 5-7					
Category IV Axel Class 8-10					
Category V Axle Class 11-13					
Total Design ESALs					

Table 6 – 1

UDOT Functional Classification Code	
RURAL	
01	Interstate System
02	Other Principal Arterials
06	Minor Arterial System
07	Major Collector
08	Minor Collector
09	Local System
URBAN	
11	Interstate System
12	Other Freeways and Expressways
14	Other Principal Arterials
16	Minor Arterial Systems
17	Collector System
19	Local System

6.7 ESAL Computations

Use the ESAL computations table to compute historical traffic levels or forecast ESALs for a given pavement section. Complete the Project Traffic Data Request Form as described below. Include the growth factors for vehicle volume, vehicle classification, and axle configuration in the approved design concept plan.

1. Enter the average daily traffic volume for the base year (first year) in the design/analysis period in column A for each vehicle classification or group being considered in the traffic analysis and the design location.
2. Compute or select a growth factor from the Annual Growth Rate Table for each vehicle classification or group and entered in column B. Each growth factor is a function of the design/analysis period and the annual growth rate of that vehicle classification. These growth factors vary from one vehicle type to another and must be taken into account in the calculations.

Table 6 – 2

Annual Growth Rate								
Annual Period Years	**No Growth**	**2**	**4**	**5**	**6**	**7**	**8**	**10**
1	1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20	24.3	29.78	33.06	36.79	41.00	45.76	57.28
25	25	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35	49.99	73.65	90.32	111.43	138.24	172.32	271.02
40	40	60.40	95.03	120.80	154.76	199.64	259.06	442.59

$$Factor = \frac{((1+g)^n)^{-1}}{g} \quad where \ g = \frac{rate}{100} \quad g \neq 0$$

3. Column C is obtained by multiplying column A by column B by 365 (days per year). This column contains the estimated number accumulated applications of specific vehicle types during the design/analysis period. Consider classification and volume growth rates.
4. Enter the truck load factor (ESAL per truck) for each vehicle classification or group in column D from the Distribution and Directional Factors Table. Consider the ESAL factor growth rate.

Table 6 – 3

Distribution and Directional Factors						
	Functional Class - *Refer to Table 3B-3					
	1		2		6	
Axle Class	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.22	0.88	1.28	0.88	0.83	0.88
5-7	0.5064	0.4718	0.2065	0.1996	0.3061	0.2896
8-10	4.8749	2.8744	2.9648	1.7796	2.7212	1.641
11-13	5.2436	3.6942	2.2342	1.3596	2.6832	1.7199

5. Column E is the product of column D and column C and represents the total number of ESALs applied by each vehicle classification/group during the design/analysis period. The sum of column E produces the total number ESAL applications that have been or will be applied during the analysis/design period. This number must be corrected for directional and lane distributions if such corrections were not incorporated in the volume estimates in column A.

Table 6 – 4

Distribution and Directional Factors						
Axle Class	Functional Class - *Refer to Table 3B-3					
	11		12		14	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.2	0.88	1.2	0.88	1.08	0.88
5-7	0.3827	0.3529	0.3827	0.3529	0.1993	0.1912
8-10	2.8934	1.6884	2.8934	1.6884	3.0104	1.8133
11-13	3.6508	2.5203	3.6508	2.5203	3.1270	1.9288

Table 6 – 5

Distribution and Directional Factors						
Axle Class	Functional Class - *Refer to Table 3B-3					
	16		17		19	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.07	0.88	1.07	0.88	1.07	0.88
5-7	0.3827	0.3529	0.3827	0.3529	0.1993	0.1912
8-10	4.2476	2.6028	4.2476	2.6028	4.2476	2.6028
11-13	5.2762	3.3584	5.2762	3.3584	5.2762	3.3584

The growth factors are equal to the analysis period if Annual Growth is zero.
The above growth factors multiplied by the first year traffic estimate will give the total volume of traffic expected during the analysis period.

Lane Distribution and Directional Factors

Design Lane ESAL = base year vol.
X directional factor x lane factor (F) x 365 days x midpoint truck factor x growth factor.

Where F = 1, for 3 lanes or less
F = 2.275 $(AADT)^{-0.1054}$ for 4 or 5 lanes
F = 2.484 $(AADT)^{-0.1312}$ for 6 lanes or more
I = annual growth rate = 10 exp. {[log(f/b]/n}-1
n = design period (yrs.)
growth factor = $\{(1+I)^{n-1}\}/I$

Direction factor defaults to 0.5 for two way and 1.0 for one way traffic if unknown.

Midpoint truck factor	= truck factor	:Class 1-4
	= truck factor + 0.1(n/2)	:Class 5-7
	= Truck factor + 0.3(n/2)	:Class 8-13

PART 2 – DEFAULT PAVEMENT DESIGN

Section 7
Default Pavement Design Alternatives

7.1 Introduction

The Region Pavement Management Engineer will determine the pavement type at the project level with assistance from the Asset/Pavement Management Group and the Region Project Managers. Pavement type determination will consist of three steps:

1. Corridor Determination – Check the Designated Corridor List in Section 7.2.
2. Project Specific Constraints
3. Life Cycle Analysis

7.2 Corridor Determination

The first step is to determine if specific corridors will be rigid or flexible pavement and conform to the corridor designations. The Asset Management group champions this process by working closely with region pavement managers and central materials to select pavement types for UDOT. Base this selection on an overall corridor analysis including life-cycle, maintenance consistency, and geographic constraints. Apply a general LCCA that includes initial construction, rehabilitation, user costs, and maintenance. Develop maps and documents that will be published for use in pavement design. Pavement type will remain unchanged once determined by corridor unless a safety issue or minimal (temporary) performance period requirement (preservation-type) is presented. Not all pavements and corridors will be identified as one or the other. These will continue on in the process. Summaries of the corridors criteria are listed below. See Appendix A for a detailed list of each Region's Designated Pavement Corridors.

Corridor Criteria for Determining Pavement Type

1. Corridor Criteria (Entire State – See individual region)
    a. Truck Volumes
    b. Traffic Counts
    c. Truck Speeds
    d. Consistency – Maintenance and Pavement
    e. Life Cycle Costs – User Impacts Costs and Maintenance Impacts
    f. Subgrade Conditions
    g. Construction Materials
    h. Climate data – Canyon Areas where concrete will not be used (safety)
    i. Context Sensitive Solution
2. Concrete roads, high truck traffic, and ADT – See individual roads.
    a. Most interstates through the Wasatch Front
    b. Most major collectors

3. Asphalt Roads, bad subgrade, and low truck traffic – See individual Roads.
    a. Local roads – mainly cars
    b. Rural roads

7.3 Project Specific Constraints

The second step is to identify project-specific constraints that will define one specific pavement type over another. These items can drive a need that may not show up in a LCCA. Typical items are listed below and remain minimal. A Region can develop a staged construction plan showing interim projects that will help meet an applicable corridor pavement strategy if it determines that a specific pavement type is not feasible. The following are situations that can fall under this procedure.

1. Historical Materials Performance Issues
    a. Stripping-susceptible aggregates
    b. Settlement problems – the region can place an asphalt road until settlement is removed if concrete is determined to be the final pavement type.
2. Performance Period Required
3. Utility Construction Issues
4. Research Scope
5. Existing Surface – the region can use overlay principles and change pavement types when the life of the pavement is exhausted.

7.4 Life Cycle Comparison – Alternate Bid Determination

Pavements that do not fall into a defined corridor or have specific constraints that prevent one type or the other will be addressed through a project specific LCCA in the UDOT design process. Details of the UDOT LCCA process are listed in sections 8, 9, and 10.

Determine user costs from an acceptable delay program with rates based upon UDOT calculations. A simplified version that includes direct delay of travelers based on speed reduction through the work zone is most appropriate because it is defendable. The majority of full-blown user cost analyses are very time consuming and results can vary significantly to the point where they no longer make sense.

Results that show greater than 15 percent benefit in LCCA will be used to determine pavement type. This type will be carried through the design process and bid in the normal approach.

Results that show a 15 percent or less benefit in LCCA will facilitate the use of an alternate bid process. One rigid design and one flexible design will be carried through design and into the bid process. This will require duplicate summary sheets, typical sections, and quantity determinations. The time and cost for duplication will vary based on project size. A typical interstate rehab on I-80 came to about four weeks and $5,000. Typical projects that fall into this category are thick and thin overlays, major

reconstruction, and new construction. Others will be dependent on project specific issues addressed in Section 4.

7.5 Pavement Concept Modifications

Pavement type is considered a defined concept within a project scope as determined by the above process. Changes to this concept or pavement type need to be brought through a similar process to a typical design exception. Minimize changes to the pavement type due to scope modifications or need for additional funds. The long-term ramifications to the overall state system are large and need to be addressed by the UDOT Deputy Director.

7.6 Miscellaneous Items to Address

1. Apply the process retroactively to any projects that are still in the scope.
2. Adopt the process as a UDOT program/procedure with Technical Committee approval.
3. Set up Commission Approved Amounts to allow for either pavement type when appropriate. Consult current bid abstracts in your region for accurate estimates and remember to perform a LCCA to compare PCC to HMA. Communicate this information to PMs.
4. Modify dTIMS runs to allow pavement type flexibility. Asset Management Group is aware of this and will address when they can.
5. Leave escalators for binder out of the project or add escalators for portland cement. True comparison and low-bid results cannot be accomplished without these items and this will be the biggest contention at the industry level. Try both methods for the first several alternate bids to see what happens.

PART 3 – MECHANISTIC PAVEMENT DESIGN

Section 8
ME – Design Guide and Software Installation

8.1 ME Design Software Installation

ME Design is the next generation of AASHTOWare® pavement design software, that builds upon the mechanistic-empirical pavement design guide and expands and improves the features in the accompanying prototype computational software. ME Design supports the AASHTO Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice. ME Design is a production-ready software tool to support the day-to-day operations of public and private pavement engineers.

http://www.me-design.com

Pricing and information are available at the following web link:

http://www.aashtoware.org/Pavement/Pages/default.aspx

ME Design now includes an enterprise option for saving, searching and loading projects utilizing a relational database. This feature allows users to store and retrieve data at varying degrees of granularity, from entire projects down through data from individual objects such as pavement layers, materials, traffic, climate, backcalculation, etc. This section describes how to set up a ME Design database in both MS SQL and ORACLE environments. Blank ME Design databases for MS SQL and Oracle can be found from Database Resource Documents section at http://www.me-design.com/.

8.2 Utah Local Calibration Coefficients

Two distress prediction models have modified local calibration coefficients. These include (1) flexible pavement rutting, and (2) rigid pavement cracking and faulting.

Rutting Model Revised Calibration Coefficients
The 2009 local calibration in Utah established that the flexible pavement rutting model was biased or over predicted actual rutting in the field most of the time so new State calibration coefficients were established. A recalibration was conducted in 2013 with additional rutting performance data. These new 2013 values are presented here. They are entered into ME Design Explorer window in the upper left hand of the main screen. Note that there are two ways to enter a new calibration coefficient.

1. Current Project Specific Calibration Factors – Incorporating the new Utah rutting local calibration factors here will only work for the current project.
2. ME Design Calibration Factors (Recommended) – Incorporating the new Utah rutting local calibration factors here will work for all future projects.

Be sure to add them for both New Flexible design and Rehabilitation Flexible overlay design. Also be sure to check the PDF output for every design to ensure that these local calibration coefficients are used.

The Utah local rutting calibration factors are shown in Table 8-1. This information is output in the detailed Excel file and in the output PDF file.

1. New Flexible Pavement

   AC Rutting

   BR1 = 0.58 instead of 1.00

   Standard Deviation = 0.0694*Pow(RUT,0.2439)+0.001

   Base Rutting

   Granular Subgrade Rutting BS1 = 0.71 instead of 1.00

   Standard Deviation = 0.1439*Pow(BASERUT,0.6711)+0.001

   (Note: these are actually the base course (UTBC) only.)

   Subgrade Rutting

   Fine Subgrade Rutting BS1 = 0.28 instead of 1.00

   Standard Deviation = 0.0531*Pow(SUBRUT,0.5012)+0.001

   (Note: these are actually the subgrade (coarse or fine grained) only

2. Rehabilitation Flexible Pavement

   AC Rutting

   BR1 = 0.58 instead of 1.00

   Standard Deviation = 0.0694*Pow(RUT,0.2439)+0.001

   Base Rutting

   Granular Subgrade Rutting BS1 = 0.71 instead of 1.00

   Standard Deviation = 0.1439*Pow(BASERUT,0.6711)+0.001

   (Note: these are actually the base course (UTBC) only.)

   Subgrade Rutting

   Fine Subgrade Rutting BS1 = 0.28 instead of 1.00

   Standard Deviation = 0.0531*Pow(SUBRUT,0.5012)+0.001

   (Note: these are actually the subgrade (coarse or fine grained) only

Be sure to verify that the local flexible pavement calibration factors are used for each run by checking the ME Design output PDF file as shown in Table 8-1.

Table 8 – 1
Illustration of Utah Specific Local Calibration Coefficients for Flexible Pavement ME Design.

Calibration Coefficients

AC Fatigue

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 \beta_{f3}}$$

$$C = 10^M$$

$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

k1: 0.007566	
k2: 3.9492	
k3: 1.281	
Bf1: 1	
Bf2: 1	
Bf3: 1	

AC Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$

$$C_1 = -0.1039 * H_a^2 + 2.4868 * H_a - 17.342$$

$$C_2 = 0.0172 * H_a^2 - 1.7331 * H_a + 27.428$$

Where:

$H_{ac} = total\ AC\ thickness(in)$

$\varepsilon_p = plastic\ strain(^{in}/_{in})$
$\varepsilon_r = resilient\ strain(^{in}/_{in})$
$T = layer\ temperature(^{\circ}F)$
$N = number\ of\ load\ repetitions$

K1: -3.35412	K2: 1.5606	K3: 0.4791
Br2: 1	Br3: 1	Br1: 0.58
AC Rutting Standard Deviation		
0.0694*Pow(RUT,0.2439)+0.001		

Thermal Fracture

$$C_f = 400 * N\left(\frac{\log C / h_{ac}}{\sigma}\right)$$

$$\Delta C = (k * \beta t)^{n+1} * A * \Delta K^n$$

$$A = 10^{(4.389 - 2.52 * \log(E * \sigma_m * n))}$$

$C_f = observed\ amount\ of\ thermal\ cracking(ft/500ft)$
$k = refression\ coefficient\ determined\ through\ field\ calibration$
$N() = standard\ normal\ distribution\ evaluated\ at()$
$\sigma = standard\ deviation\ of\ the\ log\ of\ the\ depth\ of\ cracks\ in\ the\ pavments$
$C = crack\ depth(in)$
$h_{ac} = thickness\ of\ asphalt\ layer(in)$
$\Delta C = Change\ in\ the\ crack\ depth\ due\ to\ a\ cooling\ cycle$
$\Delta K = Change\ in\ the\ stress\ intensity\ factor\ due\ to\ a\ cooling\ cycle$
$A, n = Fracture\ parameters\ for\ the\ asphalt\ mixture$
$E = mixture\ stiffness$
$\sigma_M = Undamaged\ mixture\ tensile\ strength$
$\beta_t = Calibration\ parameter$

Level 1 K: 1.5	Level 1 Standard Deviation: 0.1468 * THERMAL + 65.027
Level 2 K: 0.5	Level 2 Standard Deviation: 0.2841 *THERMAL + 55.462
Level 3 K: 1.5	Level 3 Standard Deviation: 0.3972 * THERMAL + 20.422

CSM Fatigue

$$N_f = 10^{\left(\frac{k_1 \beta_{c1}\left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{c2}}\right)}$$

$N_f = number\ of\ repetitions\ to\ fatigue\ cracking$
$\sigma_s = Tensile\ stress(psi)$
$M_r = modulus\ of\ rupture(psi)$

k1: 1	k2: 1	Bc1: 1	Bc2:1

Subgrade Rutting

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right|$$

$\delta_a = permanent\ deformation\ for\ the\ layer$
$N = number\ of\ repetitions$
$\varepsilon_v = average\ veritcal\ strain(in/in)$
$\varepsilon_0, \beta, \rho = material\ properties$
$\varepsilon_r = resilient\ strain(in/in)$

Granular		Fine	
k1: 2.03	Bs1: 0.71	k1: 1.35	Bs1: 0.28
Standard Deviation (BASERUT) 0.1439*Pow(BASERUT,0.6711)+0.001		Standard Deviation (BASERUT) 0.0531*Pow(SUBRUT,0.5012)+0.001	

AC Cracking

AC Top Down Cracking				AC Bottom Up Cracking		
$FC_{top} = \left(\frac{C_4}{1 + e^{(C_1 - C_2 * \log_{10}(Damage))}}\right) * 10.56$				$FC = \left(\frac{6000}{1 + e^{(C_1 * C_1' + C_2 * C_2' \log_{10}(D * 100))}}\right) * \left(\frac{1}{60}\right)$ $C_2' = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$ $C_1' = -2 * C_2'$		
c1: 7	c2: 3.5	c3: 0	c4: 1000	c1: 1	c2: 1	c3: 6000
AC Cracking Top Standard Deviation				**AC Cracking Bottom Standard Deviation**		
200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))				1.13+13/(1+exp(7.57-15.5*LOG10(BOTTOM+0.0001)))		

Note: Utah specific local calibration factors are included for the rutting model AC Br1=0.58, Granular base: Bs1=0.71, Subgrade: Bs1=0.28.

Concrete Pavement Cracking and Faulting Model Revised Calibration Coefficients
The concrete pavement coefficient of thermal expansion (CTE) measured by FHWA/LTPP in previous years was found to be in error (approximately $0.8*^{10-6}$/F too high). This error was corrected and a new national calibration was completed in 2011 under NCHRP 20-07. The new national calibration coefficients were recently validated for Utah concrete pavement transverse cracking and joint faulting and found to be unbiased. Thus, these calibration coefficients that utilize the correct CTE values measured in the UDOT lab can be used in Utah.

Fortunately, UDOT has recently sponsored an extensive lab testing program of the CTE of concrete produced from about 20 pits across the State and these properly measured CTE values are lower as described above. The validation of the new national calibration coefficients was conducted using Utah data and the correct (lower) CTE values was used for each Utah JPCP section. The validation study showed that NCHRP 20-07 models for JPCP transverse cracking and transverse joint faulting are unbiased and adequate for Utah conditions. This makes it possible to use a UDOT lab measured CTE (AASHTO T 336, Coefficient of Thermal Expansion of Hydraulic Cement Concrete) directly in the design of JPCP without modification.

The Utah validated NCHRP 20-07 calibration factors are shown below:

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP Transverse Cracking Model	C4	0.6
	C5	-2.05
	PCC Cracking Standard Deviation	Pow(57.08*CRACK,0.33) +1.5

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP Transverse Joint Faulting Model	C1	0.5104
	C2	0.00838
	C3	0.00147
	C4	0.008345
	C5	5999
	C6	0.8404
	C7	5.9293
	C8	400
	PCC Faulting Standard Deviation	0.0831*Pow(FAULT,0.3426)+0.00521

The IRI model indicated no bias and was not changed from the original calibration coefficients.

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP IRI Model	J1 (for Cracking)	0.8203
	J2 (for Spalling)	0.4417
	J3 (for Faulting)	1.4929
	J4 (for Site Factor)	25.24
	PCC IRI Standard Deviation	5.4

The Utah local calibration coefficients are shown below in Table 8 – 2 for cracking and faulting of JPCP. These should be checked after each run of the program.

Table 8 – 2
Illustration of Utah specific local calibration factors for JPCP transverse cracking and transverse joint faulting.

Calibration Coefficients

PCC Faulting

$$C_{12} = C_1 + (C_2 * FR^{0.25})$$
$$C_{34} = C_3 + (C_4 * FR^{0.25})$$
$$FaultMax_0 = C_{12} * \delta_{curling} * \left[\log(1 + C_5 * 5.0^{EROD}) * \log\left(P_{200} * \frac{WetDays}{p_S}\right)\right]^{C_6}$$
$$FaultMax_i = FaultMax_0 + C_7 * \sum_{j=1}^{m} DE_j * \log(1 + C_5 * 5.0^{EROD})^{C_6}$$
$$\Delta Fault_i = C_{34} * (FaultMax_{i-1} - Fault_{i-1})^2 * DE_i$$
$$C_8 = DowelDeterioration$$

C1: 0.5104	C2: 0.00838	C3: 0.00147	C4: 0.008345
C5: 5999	C6: 0.8404	C7: 5.9293	C8: 400

PCC Reliability Faulting Standard Deviation

0.0831*Pow(FAULT,0.3426)+0.00521

IRI-jpcp

C1 - Cracking	C1: 0.8203	C2: 0.4417
C2 - Spalling	C3: 1.4929	C4: 25.24
C3 - Faulting	**Reliability Standard Deviation**	
C4 - Site Factor	5.4	

PCC Cracking

$$\log(N) = C1 \cdot \left(\frac{MR}{\sigma}\right)^{C2}$$
$$CRK = \frac{100}{1 + C4\, FD^{C3}}$$

Fatigue Coefficients		**Cracking Coefficients**	
C1: 2	C2: 1.22	C4: 0.6	C5: -2.05
PCC Reliability Cracking Standard Deviation			
Pow(57.08*CRACK,0.33) + 1.5			

Section 9
Data Input

9.1 Design Life

The design life of a new, reconstructed, overlaid, or restored pavement is the time from initial construction until the pavement has structurally deteriorated to the point when significant rehabilitation or reconstruction is needed. The design life is defined by the initial pavement conditions until the specified critical pavement condition has been reached at a selected level of reliability. This definition means that for a 20 year design life and a design reliability of say 90 percent, the flexible pavement should not develop wheel-path fatigue cracking, total rutting, transverse cracking, and IRI levels above the selected criteria 90 percent of the time. Note that other distresses (e.g., raveling, cold joint deterioration, potholes) could develop that would require maintenance and rehabilitation sooner.

The software can handle design lives from 1 year for a detour to over 50 years. UDOT recommends the time periods shown in Table 17 for design life. Exceptions may be considered for unique situations. Contact the Region PME where the project is located to obtain the desired design life for the specific pavement project.

Table 9 – 1

UDOT Recommendations For Pavement Design Life		
Pavement Type	**Functional Class**	**Design Life, Years**
New or reconstructed hot-mix asphalt (HMA)	Interstate or other freeway	20
	Secondary or Urban street or highway	20
New or reconstructed concrete pavement (JPCP)	Interstate or other freeway	40
	Secondary or Urban street or highway	40
HMA Rehabilitation	Any functional class	20
PCC Rehabilitation	Any functional class	20

9.2 Construction and Traffic Opening Dates

Construction and traffic opening dates (month/year – see Table 9 – 2) are keyed to the monthly traffic loadings, monthly climatic inputs, and certain material properties that affect all future monthly layer and subgrade modulus values. Aging of asphalt materials is keyed to the construction date. Construction and traffic opening dates have additional effects on concrete pavements:

1. Construction month affects the “zero stress” solidification temperature of the concrete (the temperature that concrete becomes a solid and starts to shrink). Construction during warmer months will result in wider joints and more faulting than during cooler months but proper sized dowels can be used to offset this effect.
2. Traffic opening is affected by the curing time, strength and modulus, and

slab cracking potential over time. The longer time allowed from slab construction to opening to traffic, the stronger the concrete and less cracking that will occur over the design period.

The designer will select the most likely month for construction and for opening to traffic. Different months can be tried if these are unknown and the one resulting in the most distress can be selected for design.

Table 9 – 2

Construction and Traffic Opening Date Description	
Activity	**Best Estimate**
Base/Subgrade construction (flexible pavement only)	Month/Year - program begins with first day of the month to calculate moisture content in unbound layers
Pavement construction month	Month/Year - program assumes first day of month. Selecting August will result in the August climate being used and the August 1, date for timing material properties.
Traffic opening date	Month/Year - program begins computing damage on first day of month. Selecting June will start traffic on June 1. June will be the first month listed in the ME Design output.

9.3 New and Reconstructed Pavement and Rehabilitated Pavement Types Considered by ME Design

New and reconstructed pavements are described in Tables 9 – 3 through 9 – 7.

Table 9 – 3

Description of New Pavement Types Considered by the ME Design	
Type of Pavement	**Description**
Flexible pavement	HMA of all types including conventional thin HMA, deep strength HMA, and full depth HMA
Rigid Pavement	JPCP with or without dowels at joints
	Continuously reinforced concrete pavements (not used in Utah) has no regular transverse joints

Table 9 – 4

Description of Restored JPCP	
Type of Pavement	**Description**
Existing JPCP	Engineered design that may include cracked slab replacement, joint spall repair, shoulder replacement, dowel bar retrofit, and requires diamond grinding

Table 9 – 5

Description of HMA and PCC Overlays	
Type of Overlay	**Existing Pavement**
HMA Material	Existing flexible pavements Existing intact JPCP Existing JPCP that has been cracked and seated or rubblized
PCC Material	Existing flexible pavements (conventional PCC overlays, 6-inch minimum only) Existing intact JPCP (separated overlay) Existing JPCP that has been cracked and seated or rubblized

Section 10
Performance Criteria Input

Performance criteria called Analysis Parameters on the software window are used to make sure a pavement design performs satisfactorily over its design life. Critical limits are selected and used by the designer to judge the adequacy of a design. The International Roughness Index (IRI) criteria are similar to the 1993 AASHTO Design Guide use of the initial and terminal serviceability. Other important distress types have been added. These criteria must be selected in consideration with the Design Reliability in Section 4. Selection of too tight of a criterion such as 0.1 inch rutting at a very high reliability such as 97 percent may make it impossible to obtain an acceptable design or the design may be excessively costly.

These criteria represent the pavement condition at the time of rehabilitation at a given level of design reliability. Selecting a percent fatigue cracking for either HMA or PCC pavements of 10 percent at a 90 percent reliability level indicates that the designer is specifying this amount of fatigue cracking that requires rehabilitation such as structural improvement in this case and does not want this amount of cracking to be exceeded in 9 out of 10 projects over the specified design period.

Newly constructed HMA with initial IRI

Use the criteria presented in Table 10 – 1 to determine if a pavement design meets the minimum performance standards during its design life for a given level of reliability shown in Section 11. These values are tentative and may need revision after a period of use by UDOT.

HMA with Alligator Cracking

Additional guidance is provided for flexible and rigid pavements for use in design. For flexible pavement, the design must be sufficient to meet alligator wheel path fatigue cracking and transverse cracking over the design period (e.g. 20-years). However, if it is common to require an AC overlay at say 15 years from other causes, then the rutting and IRI design reliability criteria need only meet the 15 year time period.

HMA with transverse cracking

For rigid pavement, the design must be

sufficient to meet transverse fatigue cracking and joint faulting over the design period (e.g., 40 years). However, if it is common to require a diamond grind at say 30 years, then the IRI design reliability criteria need only meet the 30 year time period.

Initial IRI is selected at a value being achieved regularly in construction with the UDOT smoothness specifications. The initial IRI values for HMA, JPCP, and CPR of JPCP projects were examined and an average obtained for each value presented in Table 10 – 2. Unusual conditions for HMA overlays or CPR of an existing pavement with heaves or settlements may require a higher value if the effects of the existing pavement settlements or heaves are not removed by the overlay or CPR.

Table 10 – 1

Suggested Performance Criteria for Use in Pavement Design		
Pavement Type	**Performance Criteria**	**Maximum Value at End of Design Life at Design Reliability*** **
HMA pavement and overlays	HMA bottom up fatigue cracking (alligator cracking)	Interstate: 10 percent lane area Primary: 15 percent lane area Secondary: 25 percent lane area
	HMA longitudinal fatigue cracking (top down)**	UDOT does not use in design due to deficiencies in the model. Enter 20,000 ft/mile to avoid triggering reliability.
	Total permanent deformation (rutting of both wheel paths)	Interstate, Primary: 0.75 inch mean
	AC permanent deformation	Interstate, Primary: 0.75 inch mean
	Thermal fracture (transverse cracks)	Interstate: Crack spacing > 70-ft Crack length < 905-ft/mile Primary/Secondary: Crack spacing > 50-ft Crack length < 1,267-ft/mile
	IRI	Interstate/Primary: 170 inch/mile maximum* Secondary: 170 inch/mile maximum*
JPCP new, CPR, and JPCP overlays	Mean joint faulting	Interstate: 0.15 inch mean all joints Primary: 0.25 inch mean all joints Secondary: 0.25 inch mean all joints
	Percent transverse slab cracking	Interstate: 10 percent Primary: 15 percent Secondary: 20 percent
	IRI	Interstate: 170 inch/mile* Primary/Secondary: 170 inch/mile maximum*

*Initial IRI for HMA and JPCP pavements is set within the range of 50 in to 60 in/mile.
**Top down longitudinal HMA cracking can be examined but is not currently used in Utah.
***At levels of reliability given in Section 11 Design Reliability Input. The values are tentative and may need revision after a period of use by UDOT.

Table 10 – 2

Initial IRI Values for New and Rehabilitated Pavement Design			
Pavement Type	**IRI – inch/mile**		
	Average	**Minimum**	**Maximum**
New HMA and HMA/HMA	50	32	106
New JPCP	60	52	116
JPCP subjected to CPR	74	65	85

Section 11
Design Reliability Input

The design reliability is the probability that the pavement will not exceed specific performance criteria over the design period. A design reliability of 90 percent for rutting represents the probability in 9 out of 10 projects that the mean rutting for the project will not exceed the mean 0.50 inch criteria for example.

Design reliability must be selected for each distress and IRI performance criteria and they can vary between types. Select the design reliability that is best balanced with the performance criteria. Selecting a high design reliability level such as 99 percent for example and a very low performance criterion like 3 percent alligator cracking will result in an overly conservative design and make it impossible or very costly to obtain an adequate design see Tables 11-2 and 11-3.

A general rule is that design reliability should increase with traffic level since the consequence of exceeding a performance criteria may require lane closures to repair or overlay which may result in congestion. Farm to market highways are typically designed at lower reliability levels than urban Interstate or freeway pavements because the consequence of early maintenance and rehabilitation is less serious for lower traffic roadways. The selection of a very high level of design reliability (greater than 99 percent) is not recommended. This may significantly increase costs. The consequences of a project exceeding a performance criteria usually requires earlier than programmed maintenance or rehabilitation. It does not have dire structural collapse consequence and that is why pavements are designed at a much lower reliability level than bridges or retaining walls.

Use the following recommended values for UDOT designs with the performance criteria selected in Section 9. Use the same level of reliability for all distress types and IRI. Higher design reliability will require more substantial designs for such things as thicker or improved materials. Further implementation studies may show that these will need to be adjusted.

Table 11 – 1

Recommended Level of Reliability	
Functional Classification	**Reliability**
Interstate Urban and Rural	95
Non Interstate Principal Arterials, Collectors, Local	90

Table 11 – 2

Illustration of the effect of design reliability on JPCP fatigue cracking. Notice the very large effect when reliability approaches 99.8 percent.

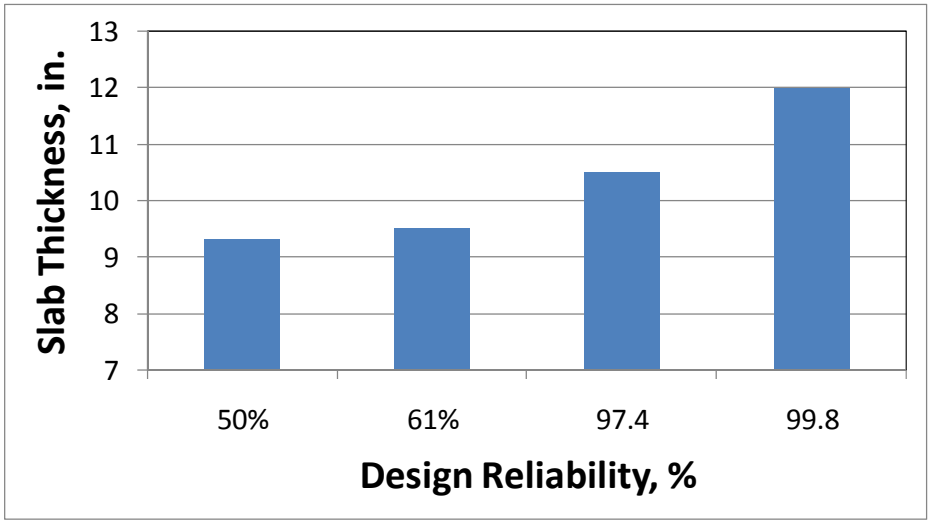
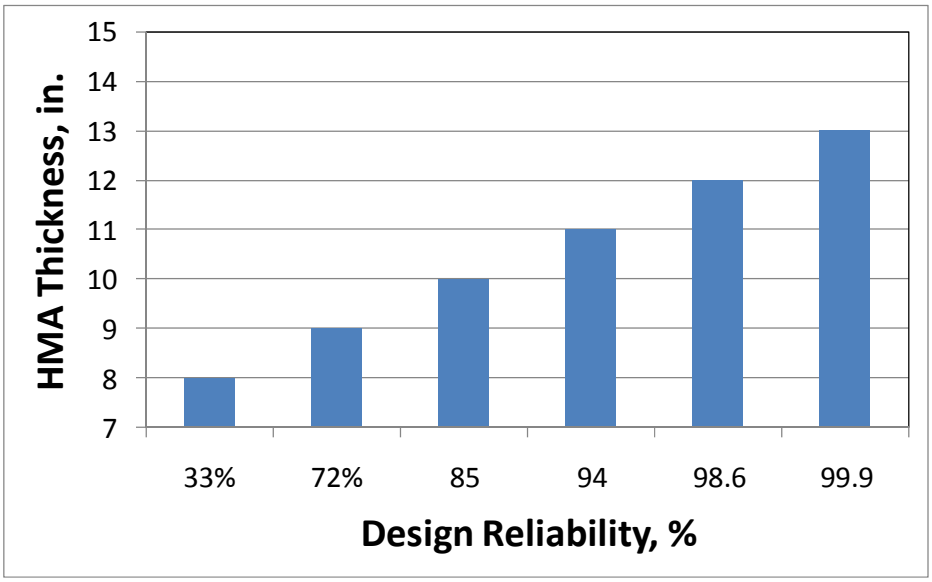


Table 11 – 3

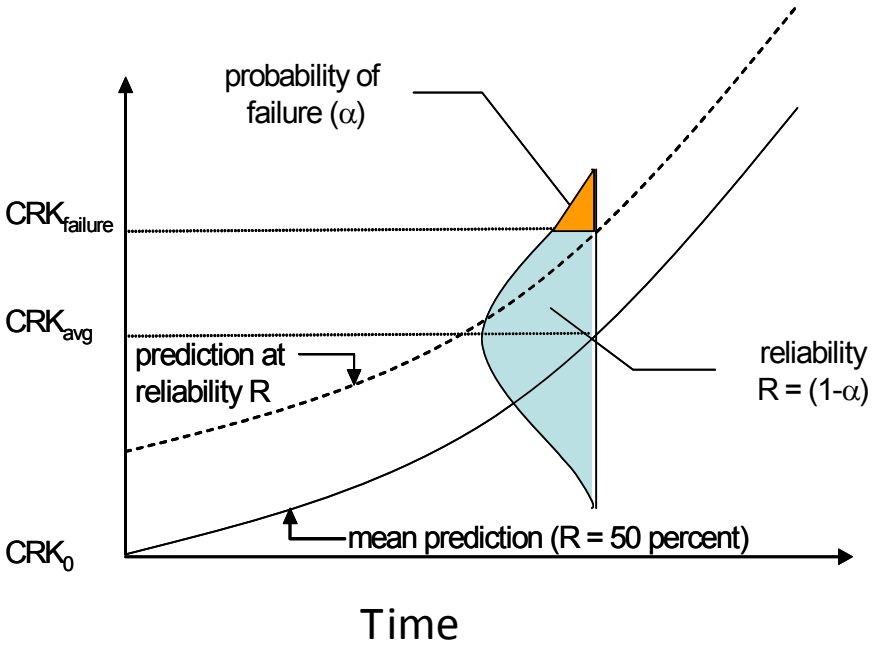
Illustration of the effect of design reliability on HMA fatigue cracking. Notice the very large effect when reliability approaches 99.9 percent.



The standard error (or deviation) of prediction is another required input along with Design Reliability. The standard deviation of prediction represents the magnitude of possible error in prediction of rutting, fatigue cracking, faulting, or IRI. The standard deviation for each prediction model is provided as a default in the software and should not be changed. The basic concept of design reliability and standard deviation (or error) of prediction is shown in Table 11-4. As the standard deviation of prediction increases, the ME Design will require a thicker pavement for the same reliability.

Table 11-4

Illustration of design reliability and standard deviation for distress prediction over time.



Section 12

Traffic Inputs For ME Design

12.1 Introduction

Several inputs are required for characterizing traffic for ME Design. The ME Design contains default traffic distributions for all required inputs with the exception of initial truck volume and future truck volume growth estimates that must always be project specific. UDOT collects truck traffic data among other vehicles types at automatic traffic recorder (ATR) stations. UDOT has three different traffic types of ATR stations that measure:

1. Volume only
2. Volume by length
3. Volume by vehicle class

UDOT has a total of 90 working ATR stations across the state as of 2008/2009 from which valuable truck traffic type and volume data are collected.

UDOT collects axle load data from 15 permanent Weigh-In-Motion (WIM) sites including nine piezoelectric sites and six load cell sites across and around the State. All sites are under the jurisdiction of UDOT with the exception of the I-80 Evanston and I-70 Loma sites, which are maintained by the Wyoming and Colorado Departments of Transportation. Data collected at each of the WIM site include a listing of time and date for each vehicle as well as detailed classification data, vehicle length, aggregate vehicle weight, disaggregate axle spacing, and disaggregate axle weight for each vehicle that crosses the WIM location.

Combining information from the ATR and WIM sites in Utah provided traffic data in sufficient detail for developing ME Design traffic inputs for several pavement sites across the State as part of ME Design implementation. Pavement designers can obtain level 3 inputs for preliminary designs from this database of default ME Design traffic inputs from sections shown in Tables 12 – 1 through 12 – 5 and other information in this Guide. There was consistent agreement between the UDOT traffic measurements and the national ME Design defaults including such inputs as axle load distribution, vehicle class distributions, and axle spacings.

The level 3 inputs must be selected based on similarity of pavement project characteristics such as functional class and location. A description of the default pavement project sites is presented in Table 12 – 1. Default data is presented throughout this section. Obtain level 1 or 2 traffic inputs from UDOT traffic engineers for final designs and special projects with unique needs.

Table 12 – 1

Location of Highway Sites With ME Design Traffic Inputs					
Project ID*	Highway	County	Mile Post		Location
			Begin	End	
CPR1	I-80	Salt Lake	121.0	126.0	State Street to 2300 East
CPR2	I-15	Juab	211.2	216.0	Diamond grind at Levan Ridge South of Nephi
CPR3	I-70	Sevier	42.0	48.0	North Richfield to Sigard
CPR4	SR-120	Sevier	1.0	3.4	SR-120 MP 1 to MP 3
CPR5	I-84	Weber	42.0	44.0	Riverdale to Uintah junction
CPR6	I-15	Box Elder	354.2	364.8	Hot Springs to Brigham
CPR7	I-215	Salt Lake	3.7	5.7	5600 S to 4500 S. Salt Lake East Side
CPR8	I-15	Juab	216.0	230.0	S Nephi to N. Nephi
CPR9	I-70	Sevier	7.0	17.0	Clear Creek Canyon MP 7 to 17
JPCP1	I-15	Davis	315.0	321.0	Pages Lane lagoon
JPCP2	I-84	Morgan	112.3	102.2	Morgan to Summit county
JPCP3	I-80	Summit	191.9	196.7	Wahsatch to WY State line
JPCP4	I-80	Summit	181.0	196.7	Wahsatch to Castle Rock
JPCP5	I-15	Salt Lake	293.0	309.0	10800 South to 500 N. SLC valley
JPCP6	I-215	Salt Lake	13.3	17.0	Redwood Rd. to 4700 South, Salt Lake West Side Belt
JPCP7	I-80	Summit	181.0	196.7	Wyoming state line to Castle Rock
JPCP10	US-89 & US-50	Sevier	194.8	195.6	Salina Main Street
JPCP11	I-15	Millard	188.0	194.0	Scipio to Juab County
JPCP13	I-70	Sevier	17.0	31.0	Belknap to Elsenor
JPCP14	I-70	Sevier	31.0	37.7	Elsenor to South Richfield
JPCP15	1-70	Sevier	37.8	46.8	North Richfield to Sigard
JPCP16	I-15	Box Elder	382.0	388.5	Plymouth to Idaho
JPCP17	I-15	Box Elder	387.0	396.7	Riverside to Plymouth
HMA-R1 01	SR-226	Weber	0.0	3.2	Snow Basin Rd.
HMA-R1 02	US-89	Cache	392.7	397.8	Logan Canyon; Tony Grove to Franklin Basin
HMA-R1 03	SR-104	Weber	0.0	0.7	Wilson Lane in Ogden; SR-126 to I-15
HMA-R1 04	I-15	Weber	346.9	352.0	450 North to Hot Springs
HMA-R2 01	SR-248	Summit	1.4	30.7	High School to US-40
HMA-R2 02	SR-224	Summit	6.0	9.4	Bear Hollow to SR-248
HMA-R2 03	SR-71	Salt Lake	16.7	14.1	700 East; 6300 S. to 6000 S.
HMA-R2 04	SR-36	Tooele	62.1	65.6	Mills Junction to I-80, Tooele Co
HMA-R3 01	SR-73	Utah	20.8	31.9	Tickville Wash to Fairfield
HMA-R3 02	SR-73	Utah	31.5	36.5	Tickville Wash to SR-68
HMA-R3 03	I-15	Utah	285.9	282.7	I-15, Point of Mountain to Lehi
HMA-R3 04	I-15	Juab	200.1	211.2	Sevier River to Mills
HMA-R4 01	US-89	Sanpete	204.6	207.9	US-89; Centerfield to Gunnison
HMA-R4 02	SR-10	Emery	48.4	53.4	SR-10; Huntington to Poison Springs Bench
HMA-R4 03	SR-56	Iron	56.0	57.5	I-15 to Iron Springs, Iron Co
HMA-R4 04	US-191	Grand	125.0	132.0	Moab to I-70 at Crescent Junction
HMA-OVLY_1	I-15	Washington	0.0	6.0	Arizona State Line to Bluff Street MP 0-6
HMA-OVLY_2	I-15	Millard	138.6	143.9	Dog Valley through Baker Canyon
HMA-OVLY_3	US-191	San Juan	86.0	89.0	Junction SR-211 to RP 93 North of Monticello
HMA-OVLY_4	SR-10	Sevier	0.0	7.0	Fremont junction to Quitchupah Hill, Emery

*Project ID's or locations in this document with the prefix CPR, JPCP, or HMA indicate UDOT pavement management system (PMS) projects.

12.2 Traffic Volume

Initial Volume

Current and future truck traffic volumes are estimated using the parameters presented in Table 9.

Table 12 – 2

Current and Future Truck Traffic Volumes Estimates for Pavement Design	
Traffic Input	**Recommended Value**
Initial two-way Average Annual Daily Truck Traffic (AADTT—class 4 and above)	Projected for month of opening to traffic from measured historical data at site is desirable.
Number of lanes in design direction	Actual, from design plans.
Percent of two-directional trucks in design direction (%)	50%, unless higher truck volume is measured in design direction (note this is volume, not weight).
Percent of trucks in design lane (% of all trucks in design direction in design lane. For example, of 100% of trucks in design direction, 60% may be in design lane, the other 40% in other lanes)	Actual measured in design (heaviest truck volume) lane over 24-hours, otherwise use the following based on Utah measurements: • 100% for 1 lane in design direction • 90% for 2 lanes in design direction • 60% for 3 lanes in design direction • 50% for 4 or more in design direction For unusual truck traffic situations (mountainous terrain or urban usage complexity), conduct on site truck lane usage counts over 24-hour period.
Operational speed (mph)	Posted or Design Speed

12.3 Traffic Volume Adjustment Factors

These are adjustment factors used to distribute annual truck traffic estimates by month. The distribution is made for each truck class type. Table 12 – 3 presents default monthly truck distributions for three sites. Data for all the sites listed in Table 12 – 1 are available in electronic format and can be obtained from the UDOT Traffic Statistics office.

Monthly Adjustment Factors
Level 1 is the actual measured site data and must be used for highways with heavy seasonal recreational and agricultural traffic. Obtain Levels 2 and 3 in electronic format from the UDOT Traffic Statistics office at 801-965-4737. Base site selection input on project location and functional class as a minimum. Use the ME Design default of 1.0 where no data in the default tables are suitable.

Table 12 – 3

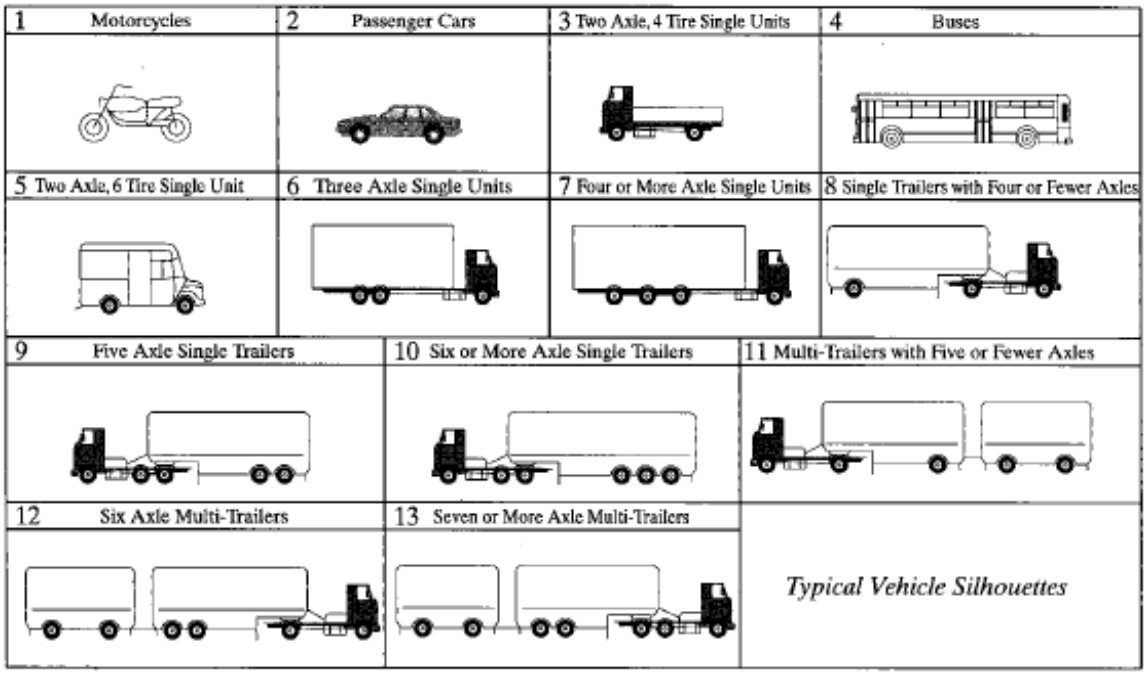
Examples of Default Monthly Adjustment Factors for Pavement Design											
Location	**Month**	**Class 4**	**Class 5**	**Class 6**	**Class 7**	**Class 8**	**Class 9**	**Class 10**	**Class 11**	**Class 12**	**Class 13**
HMA-R1 01	January	1.82	1.59	1.68	1.57	1.72	0.46	0.67	1.07	0.77	0.78
	February	1.86	1.56	1.68	1.54	1.73	0.52	0.75	1.13	0.86	0.87
	March	1.73	1.37	1.51	1.34	1.58	0.51	0.67	1.11	0.75	0.76
	April	0.82	0.94	0.89	0.95	0.87	0.58	0.46	0.67	0.43	0.43
	May	0.48	0.72	0.63	0.74	0.58	0.68	0.65	0.54	0.63	0.63
	June	0.54	0.64	0.6	0.65	0.58	0.92	1.07	0.71	1.11	1.11
	July	0.6	0.79	0.72	0.8	0.68	1.02	1.54	0.71	1.69	1.7
	August	0.66	0.8	0.75	0.81	0.72	0.99	1.46	0.73	1.6	1.61
	September	0.68	0.77	0.74	0.77	0.72	2.3	1.76	1.68	1.55	1.53
	October	0.59	0.63	0.62	0.64	0.62	2.18	1.18	1.66	0.82	0.8
	November	0.6	0.61	0.61	0.61	0.61	1.16	0.56	1.01	0.36	0.35
	December	1.61	1.58	1.59	1.58	1.59	0.67	1.23	0.99	1.43	1.45
JPCP16	January	0.65	0.76	0.71	0.77	0.72	0.96	0.92	0.96	0.76	0.72
	February	0.69	0.8	0.75	0.82	0.76	1.01	0.96	1.01	0.8	0.75
	March	0.89	1.02	0.96	1.04	0.95	1.05	1.03	1.05	0.95	0.93
	April	0.99	1.07	1.03	1.07	1.02	1.06	1.05	1.06	1.02	1.02
	May	1.21	1.43	1.32	1.45	1.25	1.05	1.07	1.05	1.17	1.19
	June	1.48	1.84	1.67	1.88	1.53	1.03	1.1	1.03	1.32	1.38
	July	1.68	2.04	1.87	2.08	1.71	1.02	1.1	1.03	1.39	1.47
	August	1.31	1.23	1.27	1.23	1.25	1	1.02	1.01	1.09	1.11
	September	0.98	0.54	0.75	0.49	0.85	0.98	0.97	0.99	0.92	0.91
	October	0.86	0.49	0.67	0.44	0.77	1.04	1.03	1.04	0.99	0.97
	November	0.7	0.43	0.56	0.4	0.65	0.94	0.92	0.94	0.84	0.82
	December	0.55	0.37	0.46	0.35	0.53	0.86	0.83	0.85	0.74	0.72
CPR7	January	0.68	0.78	0.73	0.79	0.72	0.84	0.74	0.85	0.65	0.64
	February	0.68	0.79	0.73	0.8	0.71	0.77	0.71	0.77	0.65	0.64
	March	0.76	0.82	0.79	0.82	0.78	0.9	0.84	0.9	0.79	0.79
	April	1.02	1.05	1.03	1.06	1.03	1.07	1.02	1.08	0.97	0.97
	May	1.16	1.07	1.12	1.06	1.13	1.11	1.11	1.12	1.11	1.11
	June	1.12	1.02	1.07	1.01	1.09	1.1	1.2	1.09	1.29	1.3
	July	1.38	1.22	1.3	1.19	1.32	1.09	1.23	1.09	1.35	1.36
	August	1.31	1.22	1.27	1.21	1.27	1.07	1.23	1.07	1.35	1.37
	September	1.14	1.08	1.11	1.07	1.12	1.09	1.12	1.09	1.15	1.15
	October	1.15	1.2	1.18	1.21	1.16	1.01	1.07	1	1.12	1.13
	November	0.9	0.94	0.92	0.94	0.91	0.91	0.86	0.92	0.82	0.82
	December	0.71	0.82	0.76	0.83	0.76	1.02	0.87	1.02	0.74	0.73

Each column in Table 10 must add to 12 regardless of monthly variation. A zero value for the month must be entered if no trucks use the facility for a given month. Most highways will be reasonably uniform across months. Exceptions may be highways used for heavy seasonal recreational and agricultural purposes.

12.4 Vehicle Class Distribution

Vehicle class types are defined according to FHWA and AASHTO definitions as shown in Table 12 – 4. Vehicle class distributions for the ME Design are basically adjustment factors used to distribute annual truck traffic estimates by vehicle/truck type.
Table 12 – 5 presents default truck class distributions. Each row must add to 100 regardless of truck class variation. The distribution factor for a truck class that does not use the highway facility is zero.

Table 12 – 4
Illustration of FHWA/AASHTO Vehicle Class Type Description



Vehicle Class Distribution
Level 1 is the actual measured site data over 24-hours and must be used for highways with heavy seasonal recreational and agricultural traffic (see Traffic Statistics office). Level 2 is Utah average values for highway class (see Table 12 - 5). Selection of the appropriate site input must be based on project location and functional class as a minimum. Use Level 3 that is the appropriate ME Design default Truck Traffic Class (TTC) group where no data is in the default tables is suitable.

Table 12 – 5

Examples of Default Vehicle Class Distribution Factors for Pavement* Design										
Location	**Class 4**	**Class 5**	**Class 6**	**Class 7**	**Class 8**	**Class 9**	**Class 10**	**Class 11**	**Class 12**	**Class 13**
CPR1	2.1	46.7	2.6	0.0	9.7	22.8	0.2	0.3	3.8	11.8
CPR2	1.0	18.5	1.1	0.0	5.0	62.9	0.4	0.8	3.0	7.4
CPR3	0.9	14.6	1.0	0.0	4.8	66.4	0.4	0.9	3.1	7.8
CPR4	2.4	66.5	2.9	2.6	11.0	4.2	0.4	4.3	3.2	2.4
CPR5	7.5	37.7	4.5	0.3	14.8	25.8	1.1	2.0	1.4	5.0
CPR6	1.4	21.6	1.5	0.0	6.4	51.4	0.3	0.7	4.3	12.4
CPR7	1.4	32.8	1.8	0.0	6.5	25.8	0.3	0.3	7.4	23.7
CPR8	1.0	18.5	1.1	0.0	5.0	62.9	0.4	0.8	3.0	7.4
CPR9	2.0	12.4	1.4	0.1	20.3	46.0	2.1	4.9	3.0	7.8
JPCP1	1.8	41.6	2.3	0.0	8.3	30.5	0.2	0.4	3.7	11.2
JPCP10	1.9	29.5	2.7	0.1	13.9	15.2	1.1	1.2	0.9	33.4
JPCP11	0.9	19.9	1.1	0.0	4.9	59.0	0.4	0.8	3.6	9.5
JPCP13	6.7	26.7	6.6	0.1	26.7	20.8	0.3	4.0	2.0	6.1
JPCP14	6.7	26.7	6.6	0.1	26.7	20.8	0.3	4.0	2.0	6.1
JPCP15	16.7	34.5	3.2	0.1	29.9	4.4	0.1	7.0	2.0	2.1
JPCP16	1.1	32.9	1.6	0.0	5.8	45.3	0.3	0.6	3.3	9.2
JPCP17	1.1	32.9	1.6	0.0	5.8	45.3	0.3	0.6	3.3	9.2
JPCP2	1.0	29.9	1.4	0.0	5.4	48.7	0.3	0.7	3.4	9.3
JPCP3	1.1	8.1	0.9	0.0	5.1	81.1	0.4	1.1	1.3	0.9
JPCP4	0.9	14.6	1.0	0.0	4.8	66.4	0.4	0.9	3.1	7.8
JPCP5	1.5	66.3	4.2	1.2	9.1	14.2	1.5	0.6	0.1	1.3
JPCP6	1.5	66.3	4.2	1.2	9.1	14.2	1.5	0.6	0.1	1.3
HMA-OVLY1	0.9	14.0	0.9	0.0	4.5	66.0	0.4	0.9	3.5	9.1
HMA-OVLY2	0.9	19.9	1.1	0.0	4.9	59.0	0.4	0.8	3.6	9.5
HMA-OVLY3	0.9	37.5	1.1	0.1	10.1	41.7	0.3	0.9	2.6	4.8
HMA-OVLY4	1.3	10.1	0.6	0.1	9.7	24.9	0.8	1.5	1.5	49.5
HMA-R1 01	1.5	80.9	4.4	0.0	11.0	0.7	0.7	0.0	0.0	0.7
HMA-R1 02	2.3	65.1	3.2	0.0	11.2	14.5	0.1	0.2	0.9	2.4
HMA-R1 03	2.7	49.0	7.1	2.0	17.3	7.8	1.9	2.1	1.9	8.2
HMA-R1 04	1.0	20.0	1.2	0.0	5.0	54.1	0.4	0.7	4.6	13.2
HMA-R2 01	3.0	54.8	16.9	0.0	17.8	3.8	2.4	0.1	0.0	1.1
HMA-R2 02	3.1	54.0	11.3	3.0	16.6	3.8	1.0	3.1	2.3	1.8
HMA-R2 03	2.1	81.1	3.5	0.0	11.1	2.2	0.0	0.1	0.0	0.0
HMA-R2 04	6.9	54.5	6.4	1.2	12.4	5.4	3.0	3.7	1.6	5.1
HMA-R3 01	2.1	57.1	3.8	0.4	17.1	11.6	2.0	1.0	0.5	4.4
HMA-R3 02	2.1	66.7	3.1	0.0	10.5	11.5	0.1	0.2	1.4	4.3
HMA-R3 03	1.4	35.9	5.0	1.2	10.5	34.5	2.2	0.7	1.0	7.7
HMA-R3 04	1.0	18.5	1.1	0.0	5.0	62.9	0.4	0.8	3.0	7.4
HMA-R4 01	1.7	23.5	1.8	0.0	19.7	13.1	0.8	2.2	2.9	34.4
HMA-R4 02	1.2	39.2	3.2	0.3	11.4	5.5	1.4	0.3	0.7	37.0
HMA-R4 03	5.9	56.7	5.3	0.3	15.6	10.0	1.4	1.1	0.4	3.3
HMA-R4 04	1.4	27.6	1.6	0.0	6.6	55.2	0.3	0.8	2.0	4.5

*Note that each row must add to 100.0 percent.

12.5 Hourly Truck Distribution

This input is only needed for concrete pavements and is keyed to climatic inputs over 24 hours.

Hourly Truck Distribution
UDOT specific defaults are not available at this time. Use of ME Design defaults for highways with heavy seasonal recreational and agricultural traffic (see Traffic Statistics office).

12.6 Truck Traffic Growth Factor

These inputs are unique to a given pavement project and only site specific inputs must be used.

Truck Traffic Growth Factor

- Vehicle class specific traffic growth
  - Leave blank. Assume all vehicle classes grow equally unless knowledge of growth variation is available.
- Default Growth Function
  - Use either linear or compound growth. Base decision on historical growth trends or additional information for site.
- Default Growth Rate
  - Varies long term from 0 to 10 percent on Utah highways. Base estimate on historical growth or modify based on additional information for site.

12.7 Axle Load Distribution

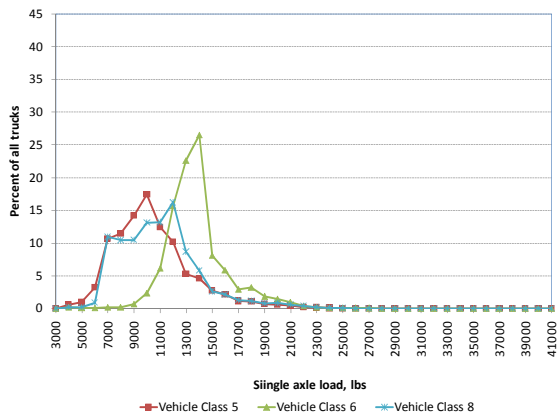
Axle load distributions are adjustment factors used to distribute the total number of axles for each axle type (single, tandem, tridem, and quad) considered by the ME Design into axle load groupings as shown below:

1. Single axles from 3,000 to 41,000 lbs in 1,000 lbs increments
2. Tandem axles from 6,000 to 82,000 lbs in 2,000 lbs increments
3. Tridem axles from 12,000 to 102,000 lbs in 3,000 lbs increments
4. Quad axles from 12,000 to 102,000 lbs in 3,000 lbs increments

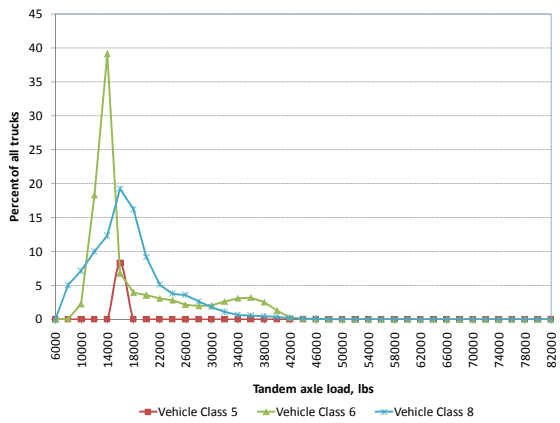
Tables 12 – 6 through 12 – 8 show single, tandem, tridem, and quad axle distributions for three sites in Utah including urban Interstate, rural Interstate, and rural local route. Note that axle load distributions vary between urban and rural sites because rural trucks are nearly all loaded whereas a significant proportion of urban trucks are partially empty.

Table 12 – 6

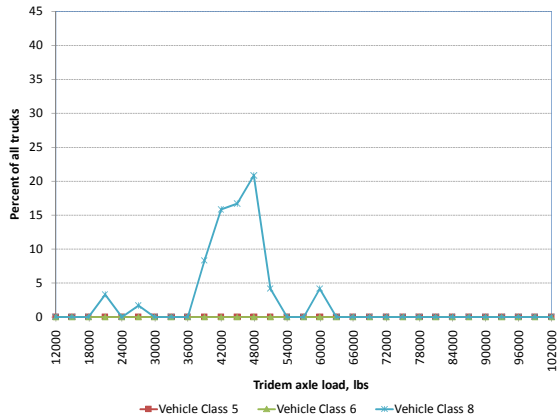
Example of Axle Load Distributions for Site HMA_R1 01 (Local Route)



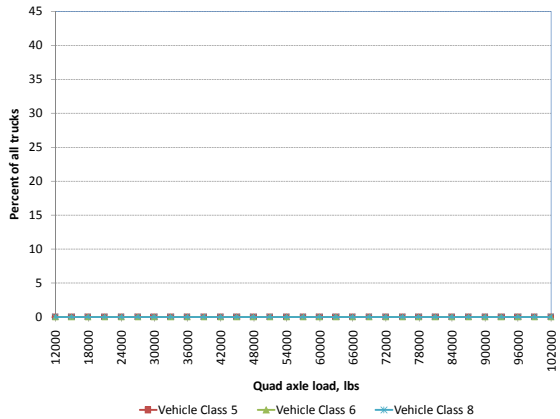
(Single Axle)



(Tandem Axle)



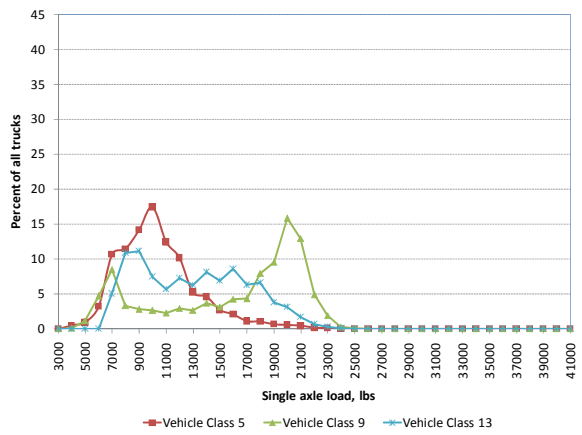
(Tridem Axle)



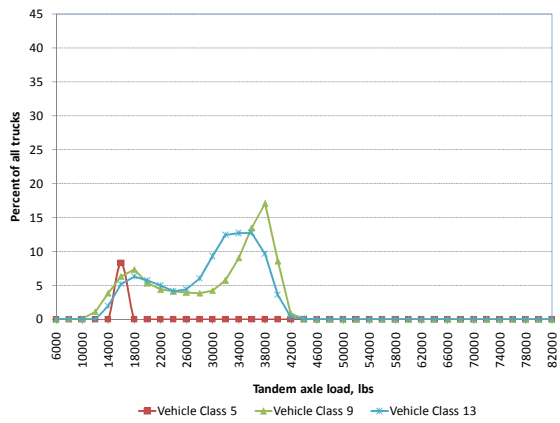
(Quad Axle)

Table 12 – 7

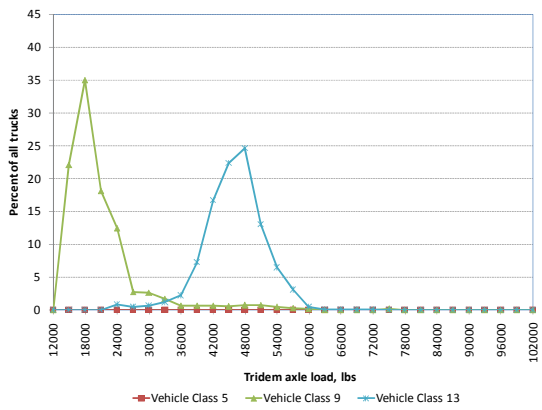
Example of Axle Load Distributions for Site JPCP16 (Rural Interstate)



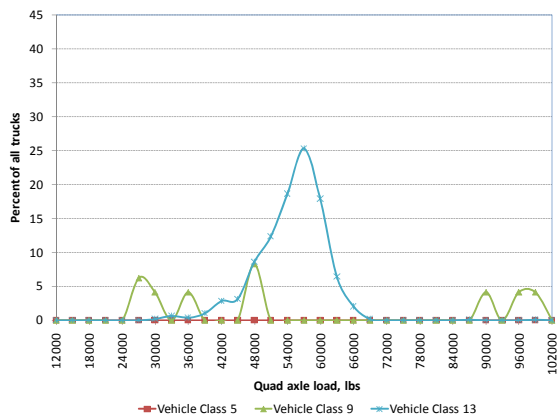
(Single Axle)



(Tandem Axle)



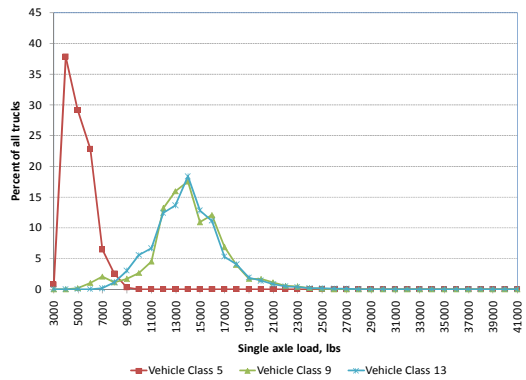
(Tridem Axle)



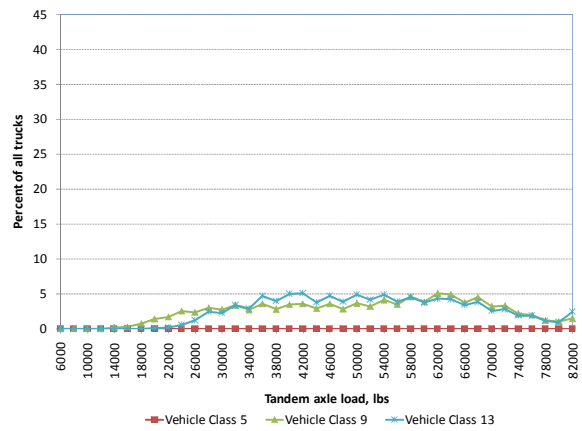
(Quad Axle)

Table 12 – 8

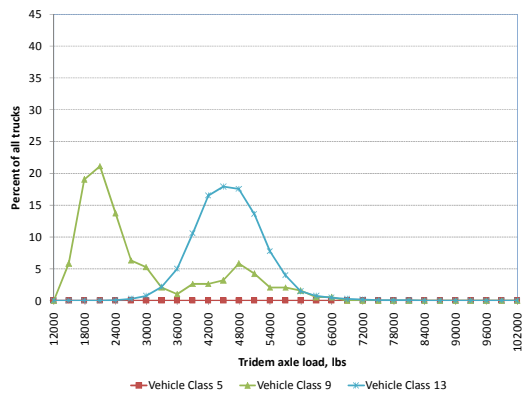
Example of Axle Load Distributions for Site CPR7 (Urban Interstate)



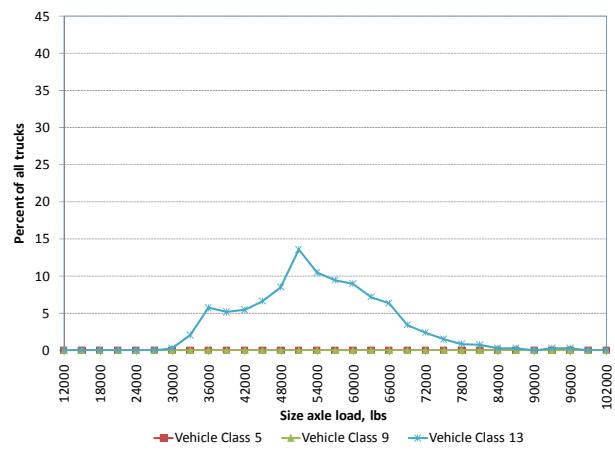
(Single Axle)



(Tandem Axle)



(Tridem Axle)



(Quad Axle)

12.8 Number of Axles per Truck Type/Class

Number of single, tandem, tridem, and quad axles per truck are basically adjustment factors used to estimate the total number of single, tandem, tridem, and quad axles for a given distribution of truck traffic. Table 12 – 4 shows each truck class type has a unique range of combination of axle types. The average number of single, tandem, tridem, and quad axles per truck varies from site to site based on the distribution of truck traffic types and volume.

Axle Load Distribution
Level 1 is the actual measured site data and must be used for highways with unique traffic characteristics such as mining, recreational, and agricultural routes (see Traffic Statistics office).
Level 2 is Utah average axle load distribution factors for highway sites described in Table 12 – 4. Graphical examples are provided in Tables 12 – 6 through 12 – 8. Electronic versions of the Level 2 axle load distribution factors can be obtained from the UDOT Traffic Statistics office. Selection of the appropriate site input must be based on project location and functional class as a minimum. Use the ME Design default axle load distribution (Level 3) where no data in the default tables are suitable.

Table 12 – 9 presents default number single, tandem, tridem, and quad axles per truck distributions for three sites within Utah. Data for all the sites presented in Table 12 - 8 can be obtained in electronic format from the UDOT Traffic Statistics office.

Table 12 – 9

Number Single, Tandem, Tridem, and Quad Axles per Truck for Pavement Design – Default Level 2

Location	Vehicle Class	Single	Tandem	Tridem	Quad
R101	Class 4	1.66	0.33	0.01	0
	Class 5	2	0	0	0
	Class 6	1	1	0	0
	Class 7	1.16	1.2	0.38	0.36
	Class 8	2.13	0.87	0	0
	Class 9	2.33	1.83	0	0
	Class 10	1.03	0.96	0.99	0.05
	Class 11	4.88	0.01	0.03	0
	Class 12	3.93	1	0.03	0
	Class 13	2.27	2.18	0.41	0.03
JPCP16	Class 4	1.38	0.62	0	0.07
	Class 5	2	0	0	0
	Class 6	1	1	0	0
	Class 7	0.83	0.39	0.57	0.23
	Class 8	2.28	0.72	0	0
	Class 9	1.28	1.86	0	0
	Class 10	1.08	0.79	0.94	0.26
	Class 11	4.89	0.01	0.03	0
	Class 12	3.44	0.89	0.09	0.13
	Class 13	3.02	1.73	0.44	0.04
CPR7	Class 4	1.58	0.42	0	0
	Class 5	2	0	0	0
	Class 6	0.84	1	0	0
	Class 7	0.49	0.91	0.27	0.26
	Class 8	2.09	0.82	0.01	0
	Class 9	1.3	1.84	0.005	0
	Class 10	1.02	0.97	0.98	0.04
	Class 11	3.52	0.28	0.32	0
	Class 12	2.69	1.07	0.14	0.19
	Class 13	2.19	1.28	0.89	0.04

Number of Axles per Truck Type/Class
Level 1 is the actual measured site data and must be used for highways with heavy seasonal mining, recreational, or agricultural traffic (see Traffic Statistics office). Level 2 is Utah average values for specific sites in Utah (see Table 12 – 1). Level 2 data can be obtained from UDOT Traffic Statistics office. Selection of the appropriate site input must be based on project location and functional class as a minimum. Use ME Design national default (Level 3) when no data in the default tables is suitable.

General Traffic Inputs

- Mean wheel location in inches, see Figure 13a
  - 18 inches from edge of lane stripe to outside of dual tires. Reduce to 12 inches if traffic lane width is greater than 12 feet
- Traffic wander standard deviation in inches
  - 10 inches lateral wander standard deviation
- Design lane width in feet, see Figure 13d
  - 12 feet – this value is not slab width, it is measured between lane longitudinal paint stripes.
- Axle Configuration, see Figure 13b

Average axle width – feet	8.5 (outside to outside of truck tires)
Dual tire spacing – inch	12
Dual Tire Pressure – psi	120
Tandem Axle Spacing – inch	51.6
Tridem Axle Spacing – inch	49.2
Quad Axle Spacing – inch	49.2

- Wheelbase , see Figures 13b and 13c

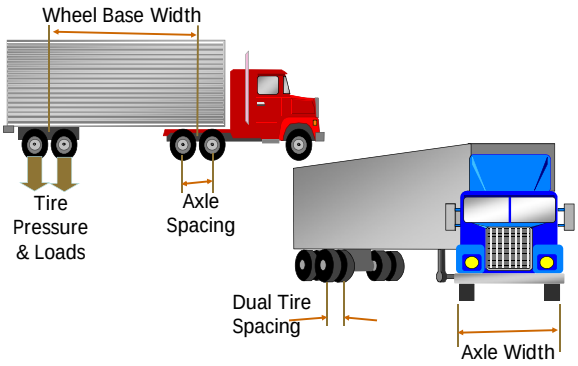
Wheelbase	Short	Medium	Long
Average Axle Spacing in feet	12 (10 to 13.5)	15 (13.5 to 16.5)	18 (16.5 to 20.0)
Percent of trucks (%)	17*	22*	61*

*Based on US national data.

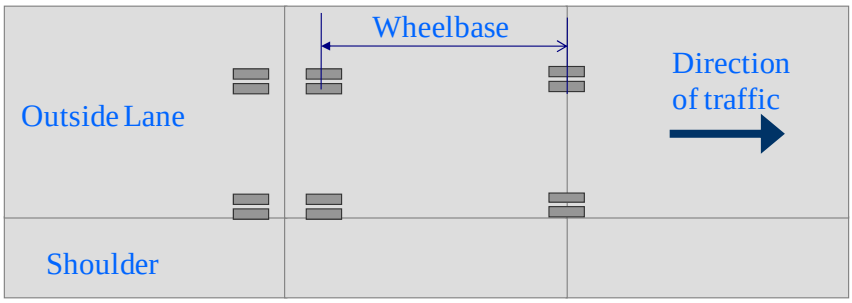
Table 12 – 10

Schematic Illustration of Mean Wheel Location

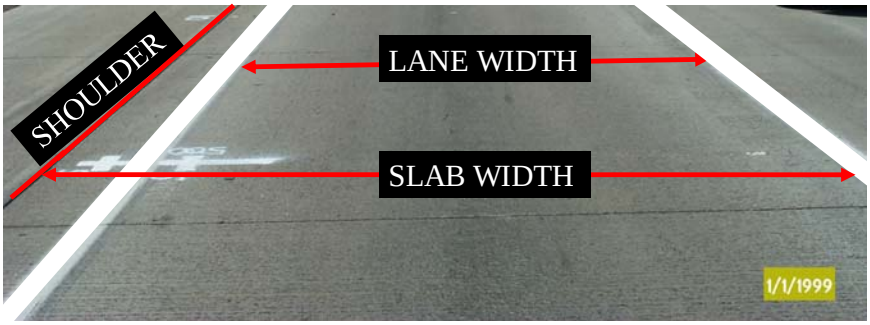
a



b



c



d

a. Wheel Location
b. Axle configuration and wheelbase
c. Axle configuration and wheelbase
d. Lane width

12.9 Traffic Sensitivity Analysis

Examples of the impact of key traffic inputs on JPCP and HMA pavement predicted performance.

Table 12 – 11

Effect of Axle Load Distribution for Urban and Rural on JPCP transverse "fatigue" cracking.

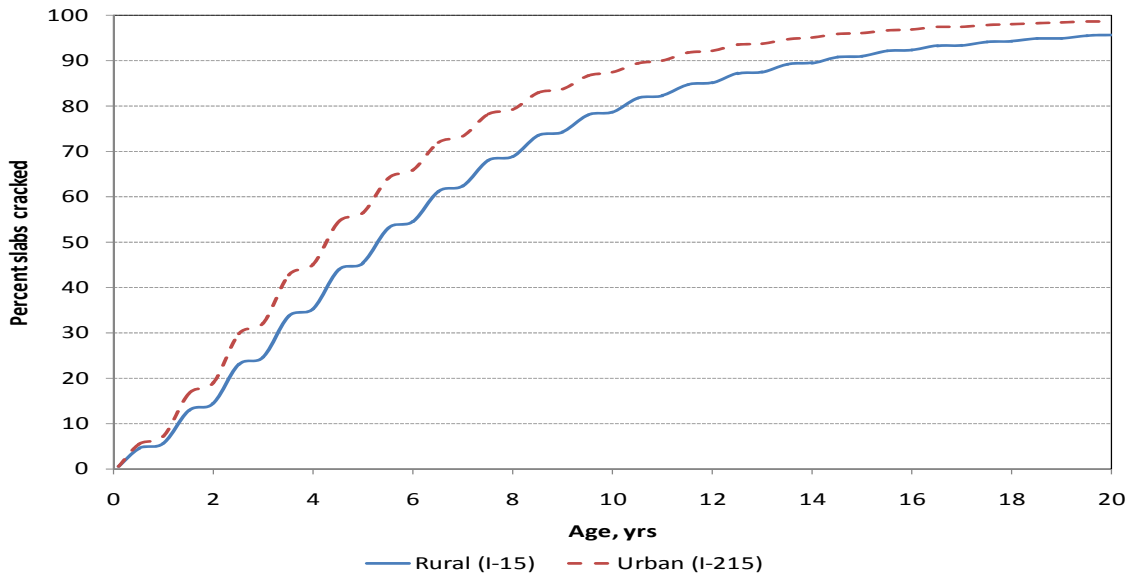
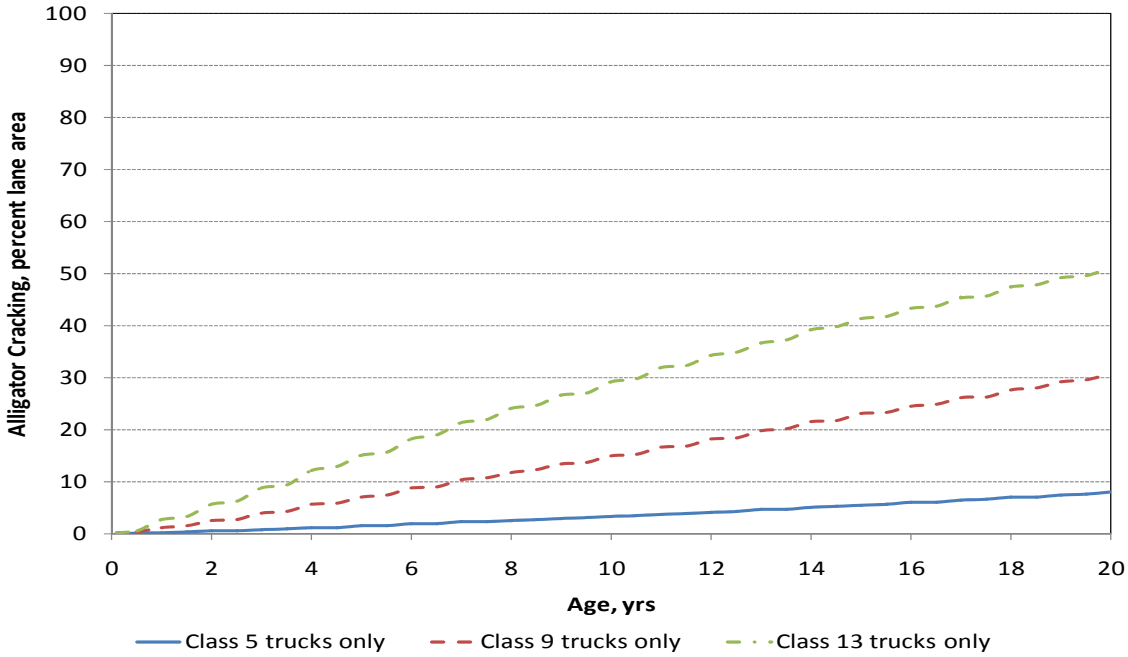


Table 12 – 12

Effect of Vehicle Class Distribution, Same Number of Class 5, Class 9, and Class 13 trucks, HMA fatigue cracking



Section 13
Climate Inputs

One or more weather stations are selected as close to the project as possible to provide hourly temperature, precipitation, wind speed, relative humidity, and cloud cover information. The project is located based on longitude and latitude in decimals of degrees (such as 90 degrees and 30 minutes is entered as 90.50 degrees). The software will identify the closest weather stations. The Utah weather stations presented in Table 13 – 1 contain up to 9 years of data but some are less. The data is currently available in the ME Design software. Table 13 – 2 shows the location of weather stations.

There are several weather stations in surrounding states of Wyoming, Colorado, New Mexico, Arizona, Idaho, and Nevada for projects located near the state lines. A single weather station can be selected when the project is within reasonable proximity or up to six surrounding weather stations can be selected and combined into a virtual weather station for a project. This is all done automatically by the software after selection by the user. The use of more than one weather station is recommended so that a better estimate of the climate at the project site can be obtained. Use caution when creating virtual weather stations and pay attention to microclimates and differences when combing stations at significantly different elevations.

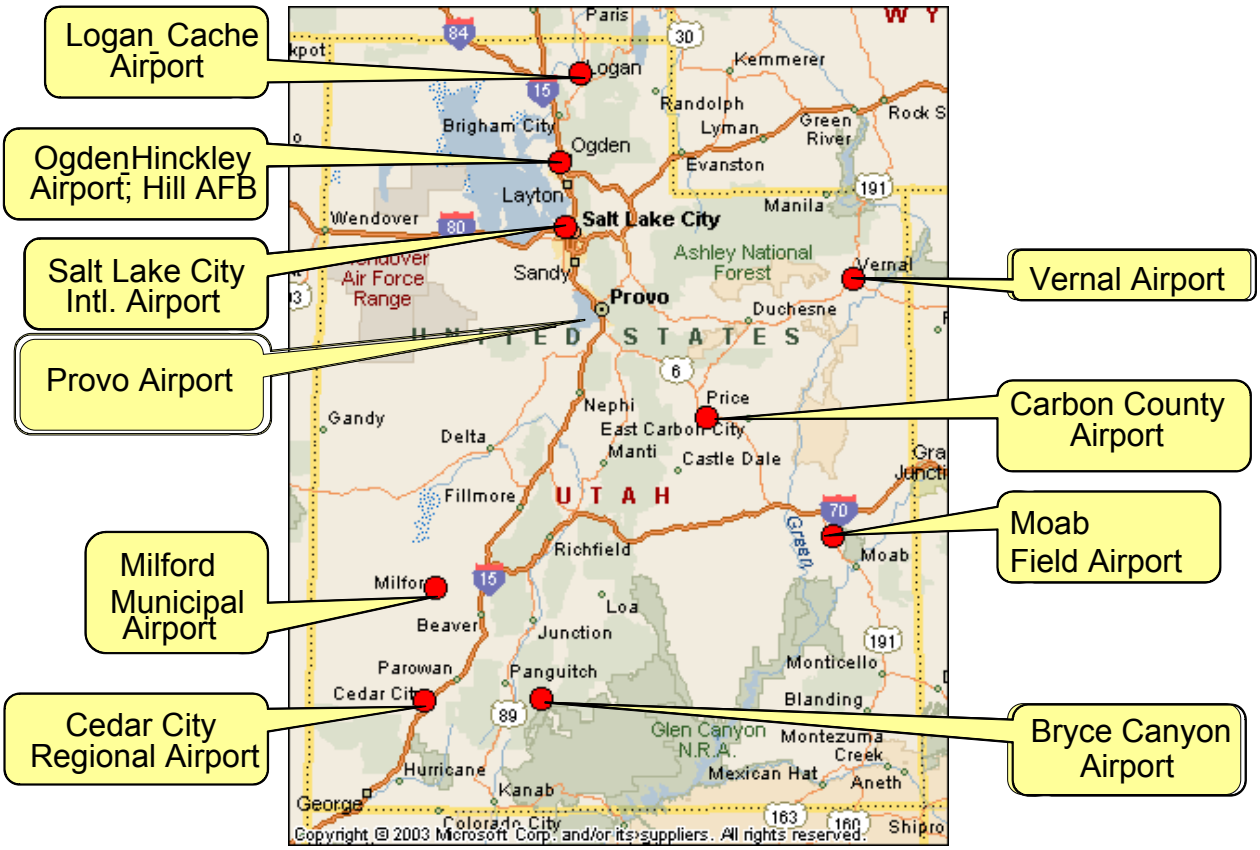
Idaho	Burley, Pocatello, Twin Falls
Wyoming	Evanston, Rock Springs
Colorado	Cortez, Durango, Grand Junction, Montrose
New Mexico	Farmington
Arizona	Page, Flagstaff, Grand Canyon
Nevada	Las Vegas, Elko, Ely

Table 13 – 1

Weather Stations with Default Climate Data for Utah Pavement Design				
Climate Station	**Mean annual air temperature (ºF)**	**Mean annual rainfall (inch)**	**Freezing index (ºF-days)**	**Average Annual Number of Freeze/Thaw Cycles**
Bryce Canyon Airport	41.6	10.1	3590	185
Cedar City Regional Airport	50.6	11.1	1753	140
Logan - Cache Airport	46.8	14.9	2081	107
Milford Municipal Airport	51.2	7.8	1765	133
Moab - Canyonlands Field Airport	55.3	6.5	1200	113
Ogden-Hinckley Airport & Hill AFB	52.40	11.2	1016	73
Price-Carbon County Airport	49.8	8.2	1658	116
Provo Municipal Airport	51.40	19.90	1211	97
St. George Municipal Airport	64.2	10.9	156	35
Salt Lake City International Airport	53.2	14.6	909	75
Vernal Airport	47.0	8.1	2298	113
Wendover, UT	53.6	3.6	1104	88

Table 13 – 2

Utah Weather Stations Used to Obtain Climate Data for Pavement Design



Climate Inputs

Climate Inputs	Definitions
Weather station within 50 miles	Import specific weather station
Weather station more than 50 miles	Create virtual weather station from 2 + surrounding weather stations
Depth of water table (feet)	Actual – see County Soil Reports* or project geotechnical reports or estimate based on area, typically ranges from 3 to 40 ft

* The United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database. Note that another available resource for estimating depth of water table for a project site is the Utah Division of Water Rights well drilling database and geologic well logs available online at http://www.waterrights.utah.gov/wellinfo/default.asp.

Selection of Weather Stations for Pavement Design

The following indicates weather stations that are in or near each Utah county. Some may not be appropriate throughout the county.

Utah County	Potential Weather Station(s)
Beaver	Milford
Box Elder	Wendover, Ogden, Logan, Burley, Pocatello, Twin Falls.
Cache	Logan
Carbon	Price
Daggett	Vernal
Davis	Ogden, Salt Lake
Duchesne	Price, Vernal
Emery	Price, Moab
Grand	Moab, Price, Grand Junction
Garfield	Bryce Canyon
Iron	Cedar City
Juab	Milford, Ely, Provo
Kane	Cedar City, Page
Millard	Milford, Ely
Morgan	Ogden, Hill AFB
Piute	Cedar City, Milford, Moab
Rich	Ogden, Logan, Evanston, Rock Springs
Salt Lake	Salt Lake City
San Juan	Moab, Cortez, Farmington
Sevier	Milford, Moab, Price
Summit	Vernal, Ogden, Evanston, Rock Springs
Sanpete	Price
Tooele	Wendover, Salt Lake City, Elko
Uintah	Vernal
Utah	Provo, Salt Lake City
Wasatch	Provo, Salt Lake City, Evanston
Washington	Cedar City, Las Vegas
Wayne	Moab
Weber	Ogden

Tables 13 – 3 through 13 – 5 show the effect of climate on pavement performance. These graphs show that climate has a very significant effect on flexible and rigid pavement performance in Utah. It is important to select a representative weather station for the project under design.

Table 13 – 3

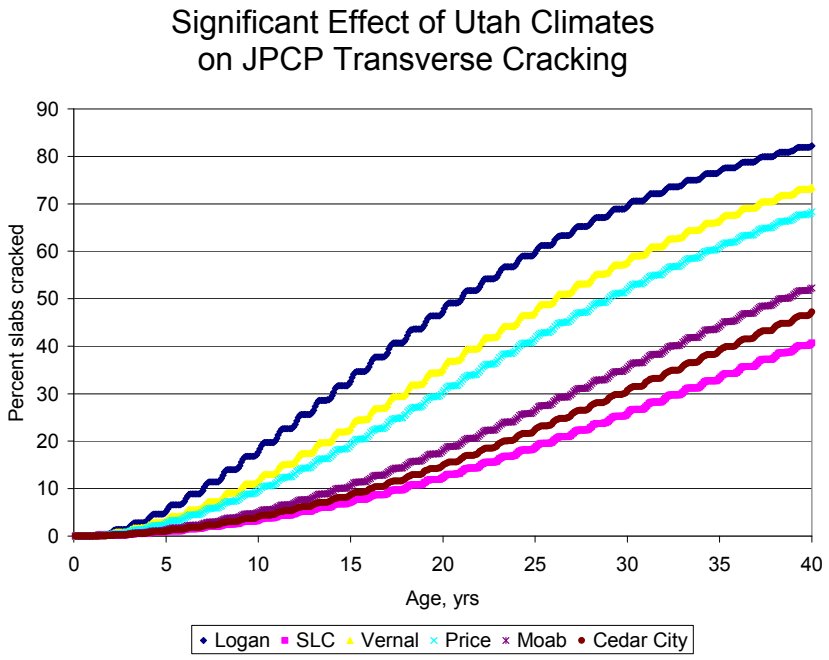


Table 13 – 4

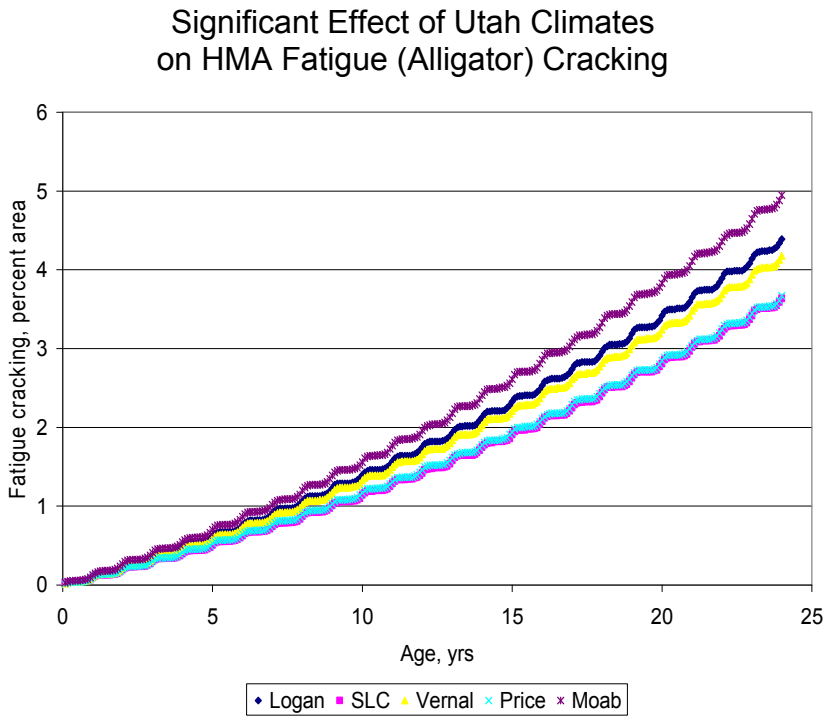
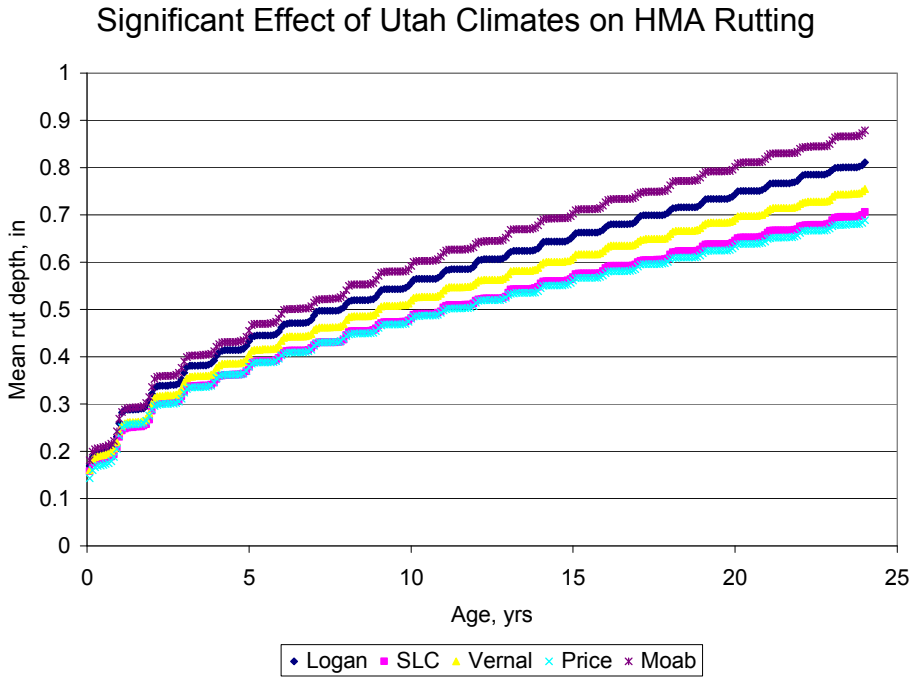


Table 13 – 5


Significant Effect of Utah Climates on HMA Rutting
Mean rut depth, in
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
5
10
15
20
25
Age, yrs
Logan
SLC
Vernal
Price
Moab

Section 14
Structures and Materials Inputs

14.1 Introduction

The design engineer must select a trial design that is analyzed by the ME Design software for adequacy. This trial design can be based on the current 1993 AASHTO Design Guide as used by UDOT or an alternative of interest to the designer. The inputs required are a range of possible layer thicknesses plus joint design for JPCP and material properties for the following material types:

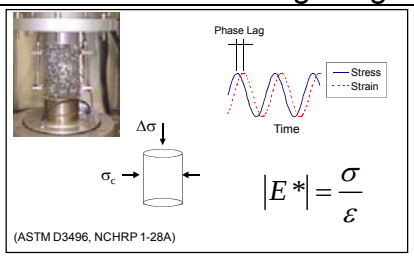
1. Asphalt materials including new and existing dense-graded and open-graded HMA materials.
2. Concrete materials including new and existing PCC.
3. Chemically stabilized materials for base and subbase.
4. Unbound aggregate layers and embankment and subgrade soils.

14.2 Recommended Level 1 Lab Testing for Charactering New and Existing Materials

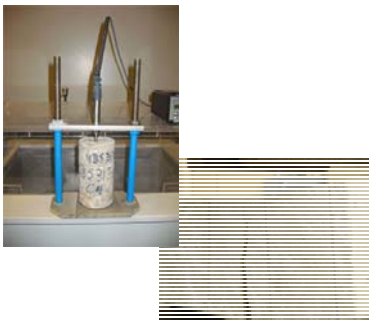
Tables 14 – 2 through 14 – 5 summarize all the **level 1 inputs** testing required for the HMA, PCC, chemically stabilized, and unbound aggregate and soils material types listed above. Table 14 – 1 shows some common HMA, PCC, and unbound materials/subgrade tests performed in the lab as part of level 1 testing.

Table 14 – 1

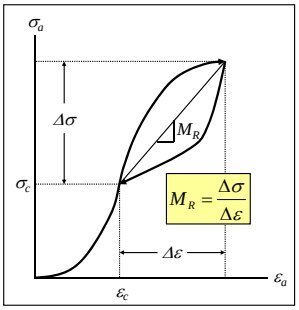
Level 1 Material Testing Program



HMA dynamic modulus testing



PCC thermal expansion and flexural strength testing



LTPP P-46/AASHTO T307
NCHRP 1-28A

Unbound granular material and subgrade soil resilient modulus testing

Table 14 – 2

Asphalt Materials Level 1 Input Requirements and Corresponding Testing Protocols for New Asphalt, Asphalt Overlays and Existing Asphalt Materials				
Design Type	**Measured Property**	**Source of Data**		**Recommended Test Protocol and Data Source**
		Test	**Estimate**	
New asphalt and asphalt overlay mixtures	Dynamic modulus (E*) *(new asphalt as-constructed)*	X		AASHTO TP62
	Tensile strength	X		AASHTO T322
	Creep Compliance	X		AASHTO T322
	Effective asphalt content *(new as-built)*	X		AASHTO T308
	Air voids	X		AASHTO T166
	Voids filled with asphalt (VFA)	X		AASHTO T209
Existing asphalt layer mixture	FWD backcalculated pavement modulus	X		ASTM* D4694 (in-situ) and backcalculation
	Asphalt content	X		AASHTO T164 (cores)
	Gradation	X		AASHTO T166 (cores)
	Air voids	X		AASHTO T209 (cores)
	Asphalt recovery	X		ASTM D5404 (cores)
New asphalt, asphalt overlays, and existing asphalt mixture	Unit weight	X		AASHTO T166
	Short term oven aging	X		AASHTO R30
	Poisson's ratio		X	Select ME Design defaults
	Surface shortwave absorptivity		X	National test protocol not available. Estimate using agency historical data or select ME Design defaults
	Thermal conductivity	X		ASTM E 1952
	Heat capacity	X		ASTM D 2766
	Coefficient of thermal contraction		X	Estimate using prediction equation or with other historical input data (see level 2 and 3 recommendations)
Asphalt binder (new, overlay, and existing mixtures)	Asphalt binder complex shear modulus (G*) and phase angle (□) OR Penetration OR Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity OR Brookfield Viscosity		X	AASHTO T315 AASHTO T49 OR AASHTO T53 AASHTO T 202 AASHTO T201 AASHTO T228 OR AASHTO T316
Existing asphalt (surface) layer	FWD backcalculated pavement modulus *(existing in-place)*	X		ASTM D4694 and backcalculation

* ASTM stands for the American Society for Testing and Materials.

Table 14 – 3

PCC Materials Level 1 Input Requirements and Corresponding Testing Protocols for New PCC, PCC Overlays and Existing PCC				
Design Type	**Measured Property**	**Source of Data**		**Recommended Test Protocol and/or Data Source**
		Test	**Estimate**	
New PCC and PCC overlays and existing PCC when subject to a bonded PCC overly	Elastic modulus	X		ASTM C469
	Poisson's ratio	X		ASTM C469
	Flexural strength	X		AASHTO T97
	Indirect tensile strength (CRCP only)	X		AASHTO T198
	Unit weight	X		AASHTO T121
	Coefficient of thermal expansion	X		AASHTO TP60
	Surface shortwave absorptivity		X	Estimate using agency historical data or select ME Design defaults
	Thermal conductivity	X		ASTM E 1952
	Heat capacity	X		ASTM D 2766
	PCC zero-stress temperature		X	National test protocol not available. Estimate using agency historical data or select ME Design defaults
	Cement type		X	Select based on actual or expected cement source
	Cementitious material content		X	Select based on actual or expected concrete mix design
	Water to cement ratio		X	Select based on actual or expected concrete mix design
	Aggregate type		X	Select based on actual or expected aggregate source
	Curing method		X	Select based on agency recommendations and practices
	Ultimate shrinkage		X	Testing not practical. Estimate using prediction equation in ME Design
	Reversible shrinkage		X	Estimate using agency historical data or select ME Design defaults
	Time to develop 50 percent of ultimate shrinkage		X	Estimate using agency historical data or select ME Design defaults
Existing intact and fractured PCC	Elastic modulus	X		ASTM C469 (extracted cores) ASTM D4694 (non-destructive deflection testing)
	Poisson's ratio	X		ASTM C469 (extracted cores)
	Flexural strength	X		AASHTO T97 (extracted cores)
	Unit weight	X		AASHTO T121 (extracted cores)
	Surface shortwave absorptivity		X	National test protocol not available. Estimate using agency historical data or select ME Design defaults
	Thermal conductivity	X		ASTM E 1952 (extracted cores)
	Heat capacity	X		ASTM D 2766 (extracted cores)

Table 14 – 4

Chemically Stabilized Materials Level 1 Input Requirement and Corresponding Testing Protocols for New and Existing Chemically Stabilized Materials					
Design Type	**Material Type**	**Measured Property**	**Source of Data**		**Recommended Test Protocol and Data Source**
			Test	**Estimate**	
New	Lean concrete & Cement-treated aggregate	Elastic modulus	X		ASTM C 469
		Flexural strength (Required only when used in HMA pavement design)	X		AASHTO T97
	Lime-cement-flyash	Resilient modulus		X	No test protocols available. Estimate using levels 2 and 3
	Soil cement	Resilient modulus	X		Mixture Design and Testing Protocol (MDTP) in conjunction with AASHTO T307[3]
	Lime stabilized soil	Resilient modulus		X	No test protocols available. Estimate using levels 2 and 3
	All	Unit weight		X	No testing required. Estimate using levels 2 and 3
		Poisson's ratio		X	No testing required. Estimate using levels 2 and 3
		Thermal conductivity	X		ASTM E 1952
		Heat capacity	X		ASTM D 2766
		Surface short wave absorptivity		X	No test protocols available. Estimate using levels 2 and 3
Existing	Lean concrete & Cement-treated aggregate	FWD backcalculated modulus	X		ASTM D4694
	Lime-cement-flyash	FWD backcalculated modulus	X		ASTM D4694
	Soil cement	FWD backcalculated modulus	X		ASTM D4694
	Lime stabilized soil	FWD backcalculated modulus	X		ASTM D4694
	All	Unit weight		X	No testing required. Estimate using levels 2 and 3
		Poisson's ratio		X	No testing required. Estimate using levels 2 and 3
		Thermal conductivity	X		ASTM E 1952 (cores)
		Heat capacity	X		ASTM D 2766 (cores)
		Surface short wave absorptivity		X	No test protocols available. Estimate using levels 2 and 3

Table 14 – 5

Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Materials Level 1 Input Requirements and Corresponding Testing 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)	Regression coefficients k_1, k_2, k_3 for the generalized constitutive model that define resilient modulus as a function of stress state	X		AASHTO T307 or NCHRP 1-28A The generalized model (NCHRP 1-28A) used in 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 □□□□ = bulk stress = □$_1$ + □$_2$ + □$_3$ □$_1$ = major principal stress. □□□□$_2$ = intermediate principal stress □□□□$_3$ = minor principal stress confining pressure □□□□$_{oct}$ = 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
	Maximum dry density	X		AASHTO T99
	Optimum moisture content	X		AASHTO T180
	Specific gravity	X		AASHTO T100
	Saturated hydraulic conductivity	X		AASHTO T215
	Soil water characteristic curve parameters	X		Pressure plate (AASHTO T99) OR Filter paper (AASHTO T180) OR Tempe cell (AASHTO T100)
Existing in situ material	FWD backcalculated modulus	X		ASTM D4694 and backcalculation of layer moduli and modulus of subgrade reaction

Level 1 inputs are the preferred inputs for pavement design. Most agencies are not equipped with the testing facilities required for materials testing and developing level 1 inputs. Level 2 and 3 inputs are recommended for the more likely situation where agencies have only limited or no testing capability for characterizing materials. It is noted that for most situations designers used a combination of levels 1, 2, and 3 material inputs based on their unique needs and testing capabilities. Designers must

use most likely values for inputs such as averages from previous projects since level 1 inputs are generally not available prior to construction.

14.3 Recommended Levels 2 and 3 HMA Inputs

New HMA Dynamic Modulus (E*) Recommended Level 2 or 3 Input

- No E* laboratory testing required.
- Use E* predictive equation. Inputs are gradation, bitumen viscosity, loading frequency, air void content, and effective bitumen content by volume. Input variables can be obtained through testing of lab prepared mix samples or from agency historical records. See recommendations below.
- Use typical Ai-VTS- values based on asphalt binder grade (PG, or viscosity, or penetration grades).

Recommended Typical Utah HMA Mix Gradations Input

Gradation Mix Designation	Percent Retained				Percent Passing
	¾ inch Sieve	½ inch Sieve	⅜ inch Sieve	#4 inch Sieve	#200 Sieve
1 inch	15	30	48	62	4
¾ inch	5	20	40	58	5
½ inch	0	5	25	52	6
⅜ inch	0	0	5	45	6

Recommended Typical Utah HMA Mix VMA and Binder Content

Gradation Mix Designation	In-situ VMA, percent	In-situ Effective Binder Content, percent by volume
1inch	16.5	10.0
¾ inch	18.0	11.5
½ inch	19.5	13.0
⅜ inch	21.0	14.5

Recommended Typical SMA Mixture Properties

SMA Input	Value
¾ Inch Sieve	0 Percent Retained
3/8 Inch Sieve	38 Percent Retained
#4 Sieve	76 Percent Retained
#200 Sieve	9 Percent Passing
Binder Grade	PG 70-28
Effective Binder Content, Percent by Volume	14.6
In Situ Percent Air Voids	6
In Situ Unit Weight	152 pounds per cubic foot

Asphalt Binder Grades
Level 3
Use PG grade defaults: PG 58-34, PG 64-34, PG 70-34, PG 64-28, PG 70-28, PG 76-28, PG 70-22, PG 76-22

As-Built Air Voids (Not mixture design)
This is the in situ field air voids at construction, NOT mixture design air voids. Based on percent compaction in specifications. Percentage is **Critical**

- Range 3.5 to 9.5 (90.5 to 96.5)
- Target 6.5 (93.5)

Recommended Input: 6.5%

As-Built Unit Weight
Actual, typical
Range 142 to 155, 148 typical dense graded

Existing HMA Dynamic modulus, E*

- No E* laboratory testing required.
- Use E* predictive equation. Inputs are gradation, bitumen viscosity, loading frequency, air void content, and effective bitumen content by volume. Input variables can be obtained through testing of extracted cores or from agency historical records
- Use typical Ai-VTS- values based on asphalt binder grade (PG, or viscosity, or penetration grades).
- Determine existing pavement condition rating (excellent, good, fair, poor, very poor)

Other New and Existing HMA Properties

Tensile Strength

Use the relationship below (developed under NCHRP 1-37A)

$$TS(psi) = 7416.712 - 114.016 * Va - 0.304 * Va^2 - 122.592 * VFA + 0.704 * VFA^2 + 405.71 * Log10(Pen77) - 2039.296 * log10(A)$$

where:

TS = indirect tensile strength at 14 °F

Va = as construction HMA air voids, percent

VFA = as construction voids filled with asphalt, percent

Pen77 = binder penetration at 77 °F, mm/10

A = viscosity-temperature susceptibility intercept

Input variables can be obtained through testing of lab prepared mix samples, extracted cores (for existing pavements), or from agency historical records.

Creep Compliance D(t)

Use the relationship below (developed under NCHRP 1-37A)

$$D(t) = D_1 * t^m$$

$$\log(D_1) = -8.524 + 0.01306 * Temp + 0.7957 * \log10(Va) + 2.0103 * \log10(VFA) - 1.923 * \log10(A)$$

$$m = 1.1628 - 0.00185 * Temp - 0.04596 * Va - 0.01126 * VFA + 0.00247 * Pen77 + 0.001683 * Temp * Pen77^{0.4605}$$

where:

t = time

Temp = temperature at which creep compliance is measured, °F.

Va = as construction air voids, %

Air Voids (Not mixture design)
Use as-constructed mix type specific values available from previous construction.

Volumetric Binder Content
Use as-constructed mix type specific values available from previous construction.

Total Unit Weight
Use as-constructed mix type specific values available from previous construction.

Poisson's Ratio
Use calculated values (takes temperature into account).

Surface Shortwave Absorptivity
Use ME Design default of 0.85.

Thermal Conductivity
Typical values for asphalt concrete range from 0.44 to 0.81 Btu(ft)(hr)(°F). Use default value set in program – 0.67 Btu(ft)(hr)(°F).

Heat Capacity
Typical values for asphalt concrete range from 0.22 to 0.40 Btu(lb)(°F).Use default value set in program – 0.23 Btu/lb. °F

Coefficient of Thermal Contraction

Use the relationship below (developed under NCHRP 1-37A).

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

where

LMIX = linear coefficient of thermal contraction of the asphalt concrete mixture (1/°C)

Bac = volumetric coefficient of thermal contraction of the asphalt cement in the solid state (1/°C)

BAGG = 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)

VAGG = percent volume of aggregate in the mixture

VTOTAL = 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:

LMIX = 2.2 to 3.4*10-5/°C (linear)

Bac = 3.5 to 4.3*10-4/°C (cubic)

BAGG = 21 to 37*10-6/°C (cubic)

Notes on HMA Levels 2 and 3 Inputs

1. The ME Design computes level 2 and 3 dynamic modulus, tensile strength, and creep compliance internally once all the required input variables required by the various equation are provided.
2. The ME Design computes level 2 and 3 coefficient of thermal contraction internally once all the required equation input variables are available.

Special Notes on HMA Transverse Cracking Model Inputs

Use levels 1 and 2 inputs only as described below:

Level 1: Lab testing of creep compliance at 3 temperatures, indirect tensile strength of first HMA layer only.
Level 2: Lab testing of creep compliance at one temperature, indirect tensile strength of first HMA layer only.

Transverse cracking predicted using default ME Design Level 3 inputs (calculated from mix volumetrics) was found to be inadequate for HMA mixes with conventional binders. The outcome was tentative for HMA mixes with SuperPave binders with early predictions being reasonable and no clear long term assessment.

Perpetual Pavement. There is also a box to check to set a fatigue analysis endurance limit for bottom up alligator cracking (see Table A-7). This is intended for perpetual pavement design but has never been calibrated and thus cannot be used. ME Design can handle a perpetual HMA pavement (or JPCP) design very effectively in the following way: structurally design the pavement for a long life such as 50 years for low fatigue cracking criteria such as 3 percent at a typically high level of reliability such as 95 percent or greater. The result will be a highly structurally adequate pavement whose surface can be renewed as needed but will not show bottom up fatigue cracking.

Special Notes on Flexible Pavement Local Calibration of the Rutting Model

The local calibration in Utah established that the flexible pavement rutting model was biased or over predicted actual rutting in the field most of the time. New Utah calibration coefficients were established in 2009 and again in 2013 (see Table 8-1). These new values are entered into the ME Design Explorer window in the upper left hand of the main screen. Note that there are two ways to enter a new calibration coefficient.

1. **Current Project Specific Calibration Factors**: Incorporating the new Utah rutting local calibration factors here will only work for the current project.
2. **ME Design Calibration Factors (Recommended)**: Incorporating the new Utah rutting local calibration factors here will work for all future projects. Be sure to add them for both New Flexible design and Rehabilitation Flexible overlay design.

14.4 Recommended Levels 2 and 3 PCC Inputs

Elastic Modulus and Flexural Strength

New PCC (Input the Mean Values, Not Specification Limits)

	28-day Modulus Elasticity, psi*	28-day Flexural Strength, psi**	28-day Compressive Strength, psi*
Mean	3,952,229	723	5,027
Minimum	3,268,113	632	4,389
Maximum	5,399,955	866	5,771
Std Dev	601,071	87	437
No. of samples	14	10	10

*Measured at long-term age and adjusted to 28-day value
**Estimated from measured compressive strength value.

Existing Intact PCC

- Determine the overall condition of the existing pavement using the guidelines presented in Section 5.
- Select typical modulus values from the range of values given below based on the pavement condition:

Qualitative Description of Pavement Condition	Typical Modulus Ranges, psi
Good/Adequate	3 to 4 x 10^6
Marginal	1 to 3 x 10^6
Poor/inadequate	0.3 to 1 x 10^6

Existing Fractured PCC

The two common methods of fracturing PCC slabs include rubblization and crack and seat. Rubblized PCC is considered equivalent to a high quality crushed stone whose modulus does not change over time. Cracked and seated PCC layers have a higher modulus and also do not change over time (see EXCEL output Sublayer Modulus tab). Recommended modulus values are given below:

Fractured PCC Type	Fixed Modulus, psi
Crack and seat PCC	125,000*
Rubblized PCC	60,000*

*Use of too high of modulus will prevent obtaining bottom up fatigue based design which makes it impossible to design an HMA overlay. It is recommended to use modulus values no higher than these values for design to ensure sufficient overlay thickness to prevent bottom up fatigue cracking and rocking of cracked and seated pieces.

Poisson's Ratio

New, Existing Intact, and Fractured PCC

Poisson's ratio (μ) for new PCC typically ranges between 0.11 and 0.21 and values between 0.15 and 0.18 are typically assumed for PCC design.
See table below for typical Poisson's ratio values for PCC materials.

PCC Material Type	Level 3 μ typical
PCC Slabs (newly constructed or existing)	0.20
Fractured Slab	
Crack/Seat	0.20
Break/Seat	0.20
Rubblized	0.30

Unit Weight

New, Existing Intact, and Fractured PCC

Select agency historical data or from typical range for normal weight concrete: 140 to 150 lb/ft^3

Surface Shortwave Absorptivity (New and Existing Intact PCC)
Use level 3 ME Design default of 0.85

Thermal Conductivity (New and Existing Intact PCC)
Typical values for PCC range from 0.2 to 2.0 Btu/(ft)(hr)(^{o}F). Use default value set in program – 1.25 Btu/(ft)(hr)($^{\circ}$F).

Heat Capacity (New and Existing Intact)
Typical values for PCC range from 0.1 to 0.5 Btu/(lb)(^{o}F). Use default value set in program – 0.28 Btu/lb. $^{\circ}$F

Typical Utah PCC Mix Properties

	PCC Unit Weight, pcf	PCC Poisson's Ratio	PCC Cementitious Material Content, lb/yd^3	Water-to-Cement Ratio
Average	142.8	0.175	574	0.419
Minimum	137.5	0.110	513	0.381
Maximum	152.0	0.210	612	0.500
Std Dev	4.8	0.035	43	0.042

Data obtained from testing several pavements across Utah

Cement Type
Estimate based on agency practices

Aggregate Type
Estimate based on agency practices

Curing Method
Determine based on agency practices

Coefficient of Thermal Expansion (New and Existing Intact PCC)

Select agency historical values or national ME Design typical values based on PCC coarse aggregate type. Use the values in these tables with the new recommended local calibration coefficients for cracking and faulting.

Utah Defaults
Recommended Level 3 concrete CTE values based on LTPP and Utah measurements.

Concrete Coarse Aggregate Type	Concrete Coefficient of Thermal Expansion ($10^{-6}/^{o}F$)
Basalt (UDOT)	4.5*
Diabase	5.2**
Granite	4.8**
Schist	4.4**
Chert	6.1**
Dolomite	5.0**
Limestone (UDOT)	4.6*
Quartzite (UDOT)	5.6*
Sandstone	5.8**

*Average measured concrete coefficient of expansion in UDOT labs.
**Average of measured (correctly) concrete coefficient of expansion in LTPP/FHWA labs.

Use ME Design default value of $5.6*10^{-6}/^{\circ}F$ where coarse aggregate type is unknown.

Level 2 Concrete CTE values measured from specific aggregate locations in Utah.

Pit Location	Concrete Supplier	Avg. CTE (x 10-6 in/in/ °F)	Primary Aggregate Classification
Moab	LeGrand Johnson	4.42	Granite/Quartzite
Monticello	Sonderegger Inc.	5.33	Quartzite
Cedar City	Sunroc Corp.	4.33	Limestone
Hurricane	Interstate Rock Products	4.27	Volcanic
St. George	Sunroc Corp.	4.63	Limestone
Tooele	Harper Ready Mix	5.96	Quartzite
Pt. of Mountain	Geneva	5.79	Quartzite
Mouth of Big Cottonwood Canyon (Walker Pit)	Binggeli Rock Products	5.24	Quartzite
Heber City	Binggeli Rock Products	6.02	Quartzite
Brigham City 7.8 Bag Mix	JBP	6.08	Quartzite
Highland	Westroc	4.60	Limestone/Dolomite
Vernal	Binggeli Rock Products	5.47	Quartzite
Randlett	Tri-County Concrete	6.08	Quartzite
South Weber	Geneva Rock	6.16	Quartzite
Nephi	Staker Parsons	5.13	Quartzite
Brigham City 6.5 Bag Mix	JBP	6.02	Quartzite
Elsinore	Western Rock	4.64	Volcanic
Nibley	LeGrand Johnson ?	5.15	Limestone
Fruitland	Cross Roads Concrete	5.93	Quartzite

Notes on PCC Levels 2 and 3 Inputs

Project specific testing is required for level 2 but is not required at level 3. Historical agency test values assembled from past construction with tests conducted using the list of protocols provided earlier are all that is required for level 3.

Zero-Stress Temperature (New and Existing Intact PCC)

Zero stress temperature, Tz, can be input directly or can be estimated from monthly ambient temperature and cement content using the equation shown below:

$$Tz = (CC*0.59328*H*0.5*1000*1.8/(1.1*2400) + MMT)$$

where

Tz = zero stress temperature (allowable range: 60 to 120 °F)
CC = cementitious content, lb/yd^3
H = -0.0787+0.007*MMT-0.00003*MMT^2
MMT = mean monthly temperature for month of construction, °F

An illustration of the zero stress temperatures for different mean monthly temperatures and different cement contents in the PCC mix design is presented below:

Mean Monthly Temperature	H	Cement Content lbs/cy			
		400	500	600	700
40	0.1533	52*	56	59	62
50	0.1963	66	70	74	78
60	0.2333	79	84	88	93
70	0.2643	91	97	102	107
80	0.2893	103	109	115	121
90	0.3083	115	121	127	134
100	0.3213	126	132	139	145

*Mean PCC temperature in degrees F.

Ultimate Shrinkage (New)

Computed based on cement type, cementitious material content, w/c ratio, curing type, and compressive strength. It is desirable to keep this as low as possible.

Reversible Shrinkage (New)

Use ME Design default of 50 percent unless more accurate information is available

Time to Develop 50 Percent of Ultimate Shrinkage (New)

Use ME Design default of 35 days unless more accurate information is available.

14.5 Recommended Levels 2 and 3 Inputs for Chemically Stabilized Materials

Elastic Modulus
Use level 2 or 3 inputs – compressive strength of lab samples or extracted cores converted into elastic modulus or select typical E and M_r values in psi as follows:

Chemically Stabilized Material Type	E or M_r, psi
Lean concrete	2,000,000
Cement stabilized aggregate	1,000,000
Open graded cement stabilized aggregate	750,000
Soil cement	500,000
Lime-cement-flyash	1,500,000
Lime stabilized soils	45,000

Flexural Strength
Use level 2 or 3 inputs – compressive strength of lab samples or extracted cores converted into flexural strength OR select typical M_r values in psi as follows:

Chemically Stabilized Material Type	M_r, psi
Chemically stabilized material placed under flexible pavement (base)	750
Chemically stabilized material used as subbase, select material, or subgrade under flexible pavement	250

Poisson's Ratio
Select typical Poisson's ratio values as follows:

Chemically Stabilized Material Type	Value
Lean concrete & cement stabilized aggregate	0.1 to 0.2
Soil cement	0.15 to 0.35
Lime-Fly Ash Materials	0.1 to 0.15
Lime Stabilized Soil	0.15 to 0.2

Unit Weight
Use default ME Design values of 150 pcf

Thermal Conductivity and Heat Capacity
Use default ME Design values of 1.25 Btu/hr.-ft-F and 0.28 Btu/lb. °F respectively.

14.6 Recommended Level 2 and 3 M_r Input Parameters for Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Materials

Level 2 FWD Deflection, Backcalculation, and Adjustment of M_r:
FWD deflection testing along the project, back calculation of the situ elastic modulus, and then an adjustment to optimum moisture and "lab" condition is the most practical and accurate way to establish a Level 2 subgrade resilient modulus (M_r). This approach is highly recommended. The procedure to obtain an appropriate input to the ME Design is given in Section 17. Rehabilitation Inputs, Table 17 – 1 for HMA pavements and Table 19 for JPCP.

Level 2 Testing of Untreated Aggregate Base Course (UTBC):
Recent extensive lab testing of the M_r of Utah untreated aggregate base course (UTBC) has indicated a typical value of 25,000 psi at optimum moisture and density (note: resilient modulus testing conducted at BYU labs). This M_r value has been added to the table below for UTBC.

Level 2 Correlation of Subgrade M_r with CBR:
The subgrade design input resilient modulus can also be roughly estimated using CBR. The ME Design software provides the following equation to estimate the resilient modulus.

$$\mathbf{M_r = 2555\ (CBR)^{0.64}} \quad [CBR < 30\%]$$

An evaluation of this equation indicated that the resulting resilient modulus (M_r) for a CBR <30 are similar to those in Section 14 at optimum moisture content. For example, an A-4 soil with a CBR = 15 results in an estimated M_r = 14,457 psi (at optimum). The mean value for an A-4 soil is 15,000 psi (rigid). However, this approach can result in serious errors in the estimation of subgrade support and is recommended only for preliminary estimation of subgrade M_r. FWD testing and backcalculation is the recommended procedure.

Level 3 Soil Classification Correlation with M_r:
The Level 3 M_r values provided in the table below are recommended for base (UTBC), subbase (GB), and embankments/subgrades if FWD testing and back calculation is not feasible.

- Untreated aggregate base course (UTBC): Recent extensive lab testing of the M_r of Utah untreated aggregate base course (UTBC) has indicated a typical value of 25,000 psi at optimum moisture and density (note: resilient modulus testing conducted at BYU labs). This M_r value has been added to the table below for UTBC.
- Subgrade/embankment: M_r values for subgrade/embankment were established during the national calibration of the ME Design using the procedure described in Section 17, Tables 17 – 1 and 17 – 2. This procedure involved FWD testing and back calculation of an elastic modulus which was then adjustment from "field to lab" and "in situ to optimum moisture." The values in the table represent the

mean M_r values for each AASHTO soil class for embankment/ subgrade. These were validated for Utah conditions by FWD testing all of the HMA and JPCP sections and back calculation and adjustment as described in Section 17. A note of caution, these mean values are computed from a wide range of project values that typically range from ½ to twice these values. Thus, it is always much better to FWD test a given project if possible.

Resilient Modulus (at optimum moisture and density)

Approximate M_r of 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.

Resilient Modulus Table

AASHTO Soil Classification	Resilient Modulus at Optimum Moisture, psi		
	Base/Subbase for Flexible and Rigid Pavements	Embankment and Subgrade for Flexible Pavements	Embankment and Subgrade for Rigid Pavements
A-1-a (UTBC)	25,000	25,000	18,000
A-1-b (UTBC)	25,000	25,000	18,000
A-2-4	NA	21,500	16,000
A-2-5	NA	21,000	16,000
A-2-6	NA	20,500	16,000
A-2-7	NA	16,500	16,000
A-3	NA	24,500	16,500
A-4	NA	16,500	15,000
A-5	NA	15,500	8,000
A-6	NA	14,500	14,000
A-7-5	NA	13,000	10,000
A-7-6	NA	11,500	13,000

***Note: Ratio 2-3 max unbound layers (e.g., if subgrade resilient modulus is 10,000 psi, the next unbound aggregate layer above should be assigned a modulus less than 30,000 psi).

Notes on Unbound Aggregate Base and Subgrade Soil Materials Inputs

Note 1
These resilient modulus values represent the mean recommended Level 3 input resilient modulus at **optimum moisture content and density** for a specific AASHTO soil classification required by the ME Design software. They represent the mean values used in the national calibration of the distress and IRI models. These values were compared to the results obtained from 50 Long Term Pavement Performance (LTPP) and UDOT PMS calibration sections using FWD back calculation and adjustment described in Section 17 to obtain appropriate input M_r (M_r at optimum moisture and density) for each section. The Utah specific values agreed with the national calibration. Note that the UTBC, GB, and subgrade input resilient modulus may be higher than UDOT has used in the AASHTO 1993. This is because the proper input for the ASHTO 1993 is a resilient modulus one or two percentage points above optimum moisture.

Note 2
As soon as the initial month of the design period begins, the ME Design adjusts the resilient modulus to a new value depending upon the moisture content or saturation level. This continues on month after month and it is not unusual for the input resilient modulus to drop to ½ or less of its original value especially if there is a water table near the surface.

Note 3
Bedrock can be considered in the back calculation if a very stiff layer exists within 20 feet of the surface. Bedrock may exist if the back calculated modulus is much higher than those provided above. ME Design can add bedrock as the lowest layer. The above M_r values were derived by considering bedrock wherever it existed. Use of M_r input for a project that is very different than these recommendations may result in erroneous predictions.

Note 4
The subgrade can be represented by more than one layer for example, A-1 can be an embankment 4 feet thick that exists over an A-6 subgrade. The program divides the pavement/subgrade into many sublayers and occasionally this becomes greater than 20, the maximum possible. The designer will have to select a composite M_r for the composite “subgrade” between the two values if this occurs.

Note 5
Place certain limitations on input modulus values when two or more unbound aggregate or soil layers are part of the pavement structure and the resilient modulus much be selected for each. The upper layer should be not more than 2 to 3 times the lower layer for any two layers to avoid decompaction of the upper layer and loss of modulus. Unbound materials do not have tensile strength and this is one simple way to keep the layer resilient modulus within reason of field conditions.

Note 6
Occasionally a subgrade soil exists that has a very low stiffness or modulus. This could be defined as any resilient modulus, Mr, less than about 10,000 psi at optimum moisture and density. This represents a very poor foundation on which to build a pavement and use of the ME Design may show seemingly unrealistic thicknesses required for long term design. This is due to the minus #200 sieve showing high levels of passing such as greater than 50 percent. This may be a good indication that the top of the subgrade should be treated with lime or cement to provide a more stable construction platform and long term performance foundation. If this is done, such as an 8-in lime stabilized subgrade, then the percent passing the #200 sieve can be reduced to the lowest allowed for the AASHTO classification. For a fine grained soil, this would be a reduction from say 80 percent to 36 percent (lowest for a fine grained soil). This gradation can be modified in the inputs with no other changes needed and should provide a more realistic design. This should only be done for fine grained soils.

Maximum Dry Density
Compute using ME Design predictive equations based on the following inputs – gradation, plasticity index, and liquid limit.

Optimum Moisture Content
Compute using ME Design predictive equations based on the following inputs – gradation, plasticity index, and liquid limit.

Specific Gravity
Compute using ME Design predictive equations based on the following inputs – gradation, plasticity index, and liquid limit.

Saturated Hydraulic Conductivity
Compute using ME Design predictive equations based on the following inputs – gradation, plasticity index, and liquid limit.

Soil Water Characteristic Curve Parameters
Select based on aggregate/subgrade material class.

Guidance on Coding Unbound Aggregate for Base and Subbase Layer Properties into the ME Design

Unbound Material	Crushed Stone, Gravel, or AASHTO Class A-1-a through A-3
Thickness in inches	Actual
Strength Properties Input Level	Level 3
Poisson's ratio	0.35
Coefficient of lateral pressure	0.5
Compacted unbound material or uncompacted natural unbound material	Click on "Compacted" option for all base/subbase layers
Resilient Modulus M_r (psi) at optimum moisture content	See table of recommended M_r values for base/subbase. Base/subbase to subgrade M_r ratio should be between 2 and 3 to prevent decompaction of the base/subbase. **[Critical]**
Plasticity Index, PI	Actual or default **always use 1 minimum, even if non-plastic for drainage reasons [Critical]**
Liquid Limit, LL	Actual or default
Gradation	Actual or use defaults for soil class or use UDOT table below
User Override Index Properties Unit maximum dry unit weight, specific gravity, sat. hydraulic conductivity, optimum gravimetric water content, degree of saturation at optimum	User may enter specific values for these parameters if available. Measured values will be more accurate than these estimated values.

UDOT Untreated Base Course (UTBC) Specifications (Adapted from UDOT 2008 Standard Specifications for Road and Bridge Construction)

Aggregate Properties				
	Aggregate Class			
	A	B	C	
Dry Rodded Unit Weight	Not less than 75 lb/ft^3			AASHTO T 19
Liquid Limit/Plastic Index	Non-plastic		PI < 6	AASHTO T 89 AASHTO T 90
Gradation	See below			AASHTO T 11 AASHTO T 27
CBR with a 10 lb surcharge measured at 0.2 inch penetration	70 percent minimum		N/A	AASHTO T 193

Gradation Limits		
Sieve Size	**Job Mix Gradation Target Band**	**Job Mix Gradation Tolerance**
1½ inch	100	
1 inch	90 - 100	±9.0
¾ inch	70 - 85	±9.0
½ inch	65 - 80	±9.0
⅜ inch	55 - 75	±9.0
No. 4	40 - 65	±7.0
No. 16	25 - 40	±5.0
No. 200	7 - 11	±3.0

Guidance on Coding Unbound Soils for Embankment and Subgrade Layer Properties into ME Design

Unbound Material	Actual, from soil report AASHTO Class A-1-a through A-7-6
Thickness (in)	Actual or Semi-infinite if Last Layer
Strength Properties Input Level	Level 2
Poisson's ratio	0.4
Coefficient of lateral pressure	0.5
Compacted unbound material or uncompacted natural unbound material	Click on "Uncompacted" option for subgrade regardless if top is compacted. **[Critical]**
Resilient Modulus M_r (psi) at optimum moisture content and density	See table of recommended M_r values for subgrades. **[Critical]**
Plasticity Index, PI	Actual or use default for soil classification Note: Use **PI = 1** for drainage reasons if non-plastic **[Critical]**
Liquid Limit, LL	Actual or use default for soil classification
Gradation	Actual or use defaults for soil classification
User Override Index Properties (Unit maximum dry unit weight, specific gravity, sat. hydraulic conductivity, optimum gravimetric water content, degree of saturation at optimum)	Enter specific values for these parameters if available. Measured values will be more accurate than these estimated values.

Section 15
Sensitivity Analysis

A sensitivity analysis was conducted to determine the impact of all key inputs on predicted pavement performance. Results are presented in this section. Tables 15 – 1 through 15 – 9 show the effect of material properties on predicted HMA pavement performance. Tables 15 – 10 through 15 – 14 show the effect of material properties on the predicted PCC (JPCP) pavement performance.

Summary of ME Design Sensitivity Results for Utah New/Reconstructed HMA Pavements

Design/Material Variable	Distress/Smoothness			
	Alligator Fatigue Cracking	Rutting	Transverse Cracking	IRI
HMA thickness	XXX	XX	X	XX
Tire load, contact area, and pressure	XX	XXX		
HMA Tensile Strength			XXX	
HMA Coefficient of Thermal Contraction			XX	
Mixture Gradation	XX	XXX		
HMA air voids in situ	XXX	XX	XX	XX
Effective HMA binder content	XXX	XX	XX	X
HMA binder grade	XX	XX	XXX	XXX
Bonding with base	XXX	X		
Base type/modulus	XXX	XX		
Base thickness	X			
Subgrade type/modulus	XX	XX		
Ground water table	X	X		
Climate	XX	XX	XXX	X
Truck volume	XXX	XXX		
Truck axle load dist.	XX	XX		
Truck speed	XX	XXX		
Truck wander	XX	XX		
Initial IRI				XXX

Key: X has small effect on distress/IRI
XX Factor has moderate effect on distress/IRI
XXX Factor has large effect on distress/IRI
O means no effect (add to key) Put O in blank area

Summary of ME Design Sensitivity Results for Utah New/Reconstructed JPCP

Design/Material Variable	Distress/Smoothness		
	Transverse Joint Faulting	Transverse Cracking	IRI
PCC thickness	XX	XXX	XXX
PCC modulus of rupture & elasticity		XXX	XX
PCC Coefficient of thermal expansion	XXX	XXX	XXX
PCC unit weight	X	XX	X
Joint spacing	XX	XXX	XX
Joint load transfer efficiency	XXX		XXX
Edge support*	XXX	XXX	XX
Permanent curl/warp	XXX	XXX	XXX
Zero-stress temp	XX		X
Friction between slab & base		XXX	XX
Base type	XXX	XX	X
Climate	XXX	XXX	XXX
Subgrade type/modulus	X	XX	X
Ground water table	X	X	X
Truck speed		X (with HMA base only)	
Truck axle load distribution	X	XX	X
Truck Volume	XXX	XXX	XXX
Tire pressure		X	
Truck lateral offset	XX	XXX	XX
Truck wander		XX	X
Initial IRI			XXX

Key: X Factor has small effect on distress/IRI
XX Factor has moderate effect on distress/IRI
XXX Factor has large effect on distress/IRI
O has no effect (put O in blank areas)

***Free edge vs. tied shoulder vs. widened lane**

Table 15 – 1

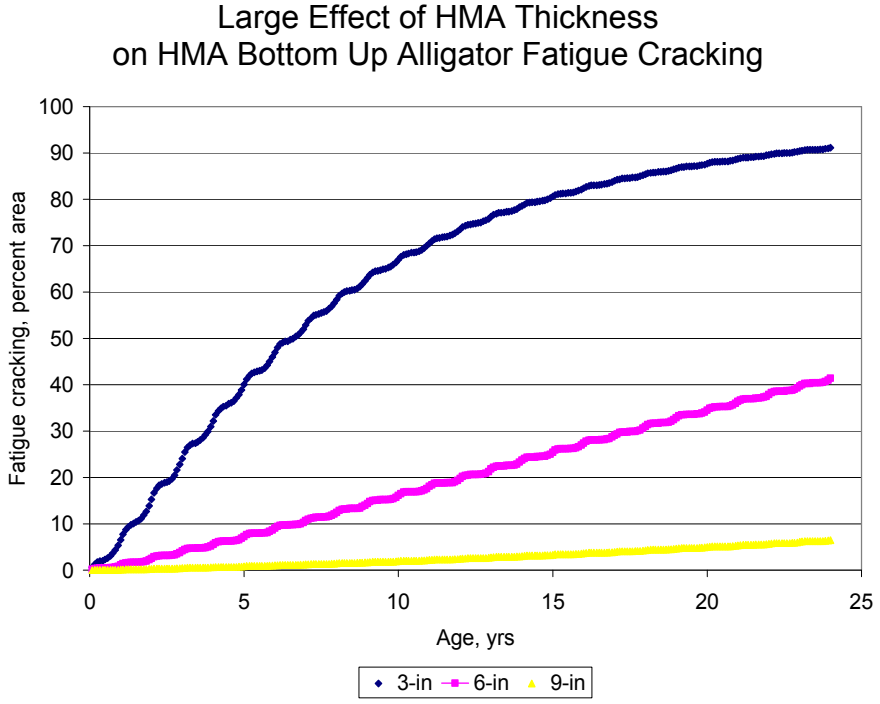


Table 15 – 2

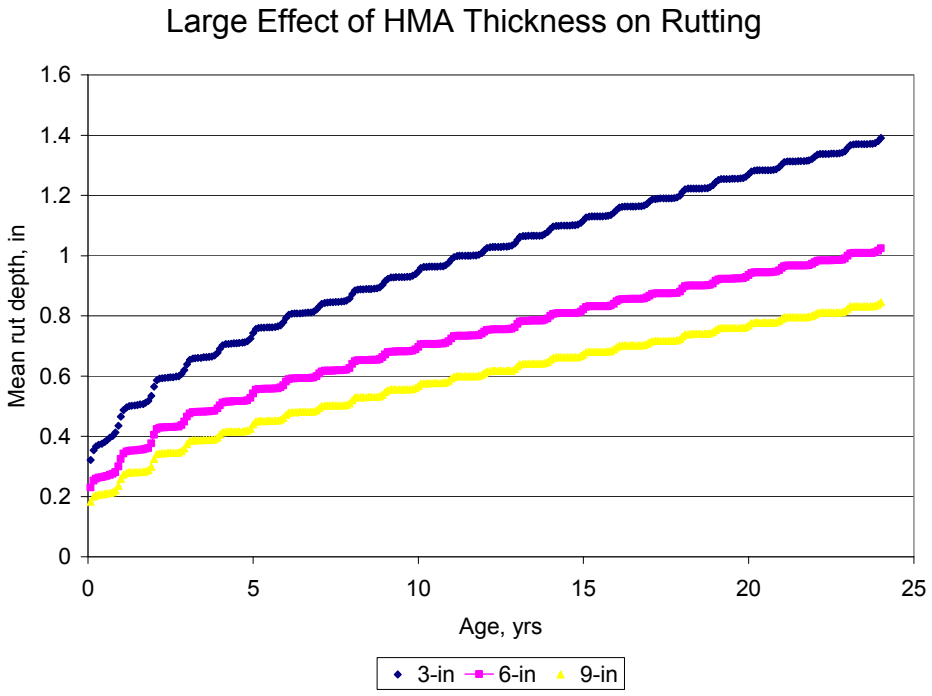
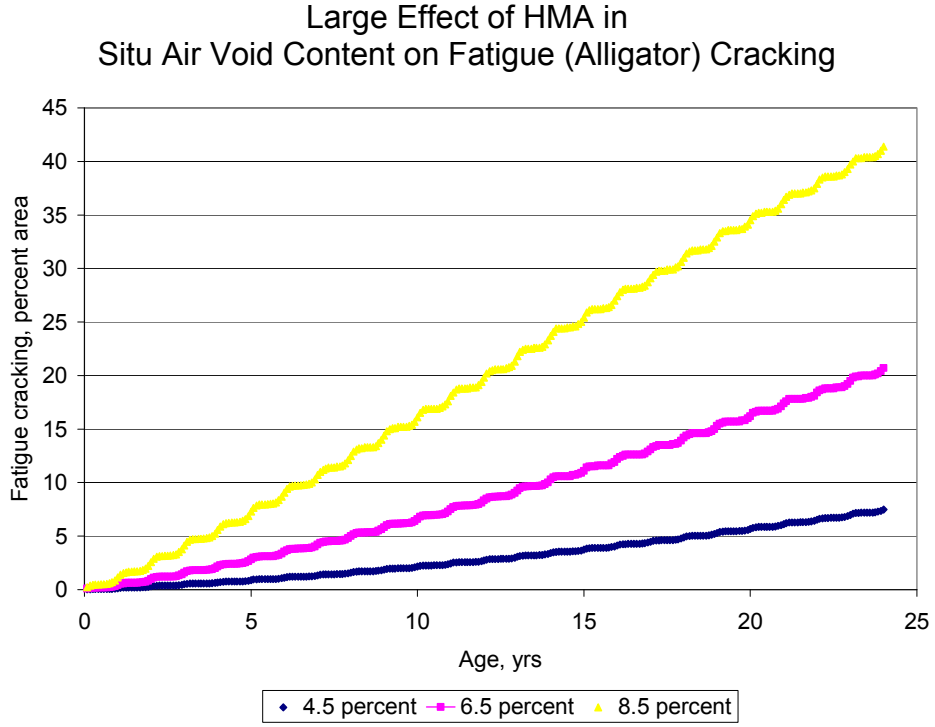


Table 15 – 3



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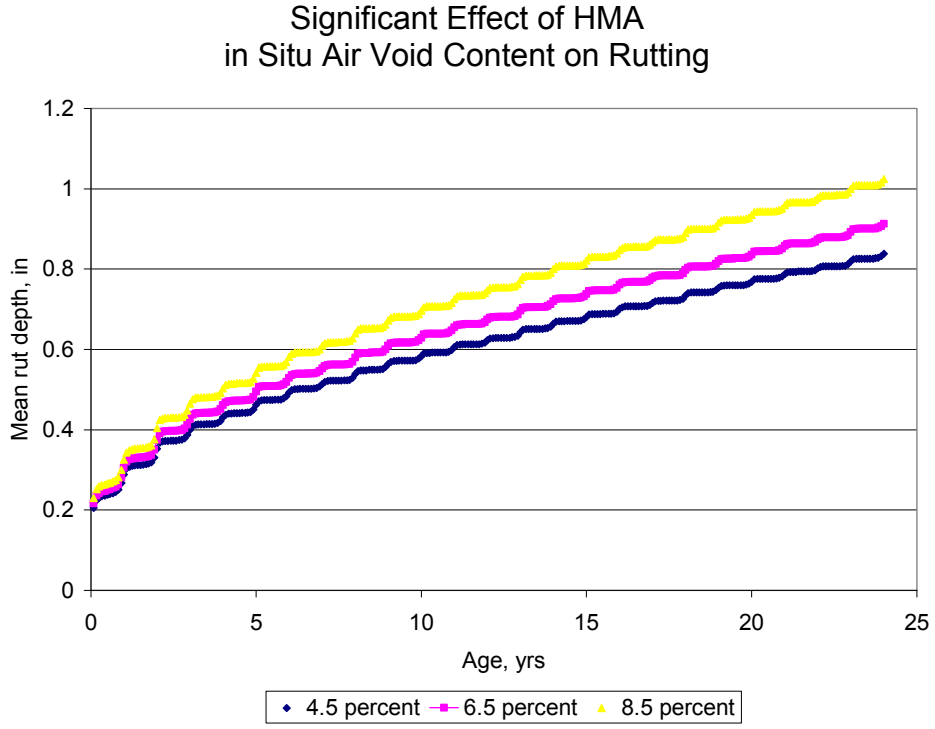

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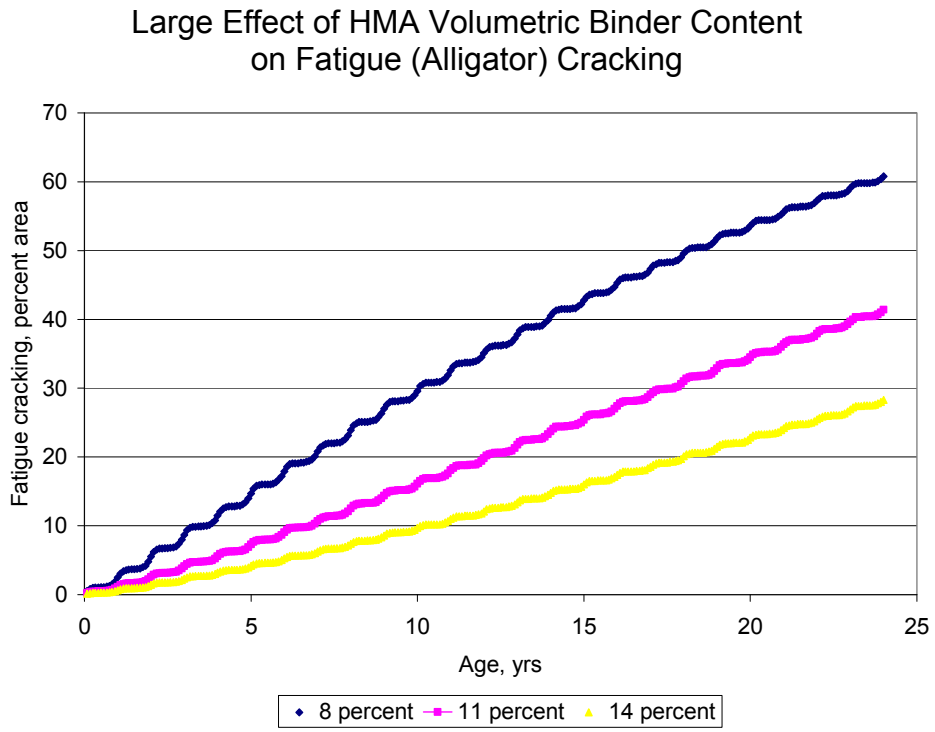

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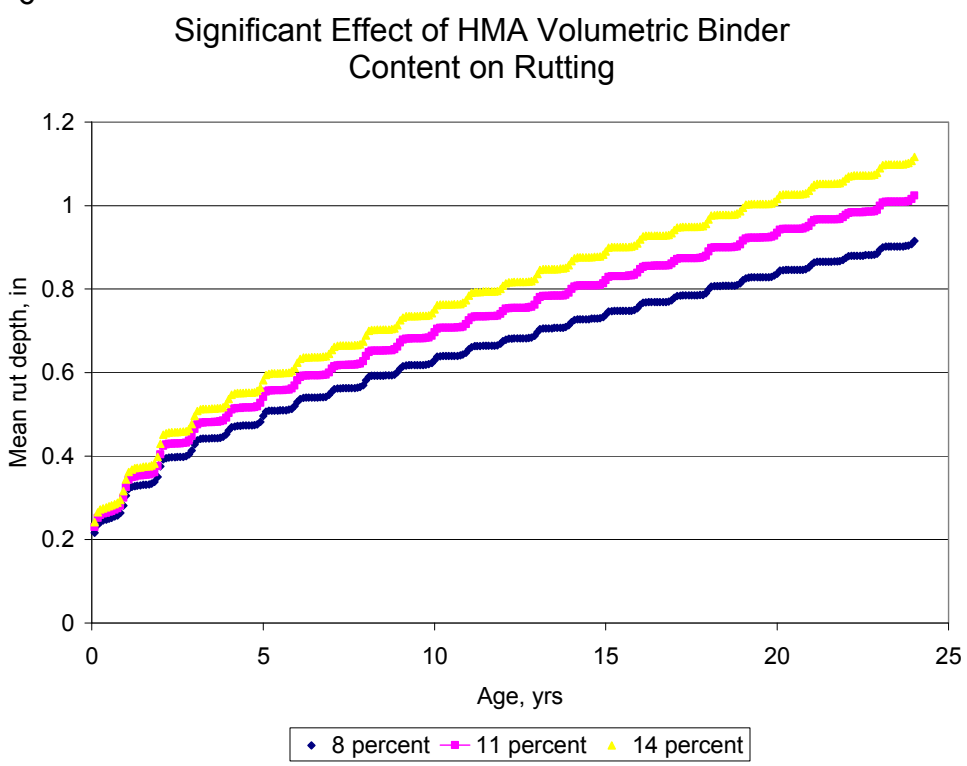

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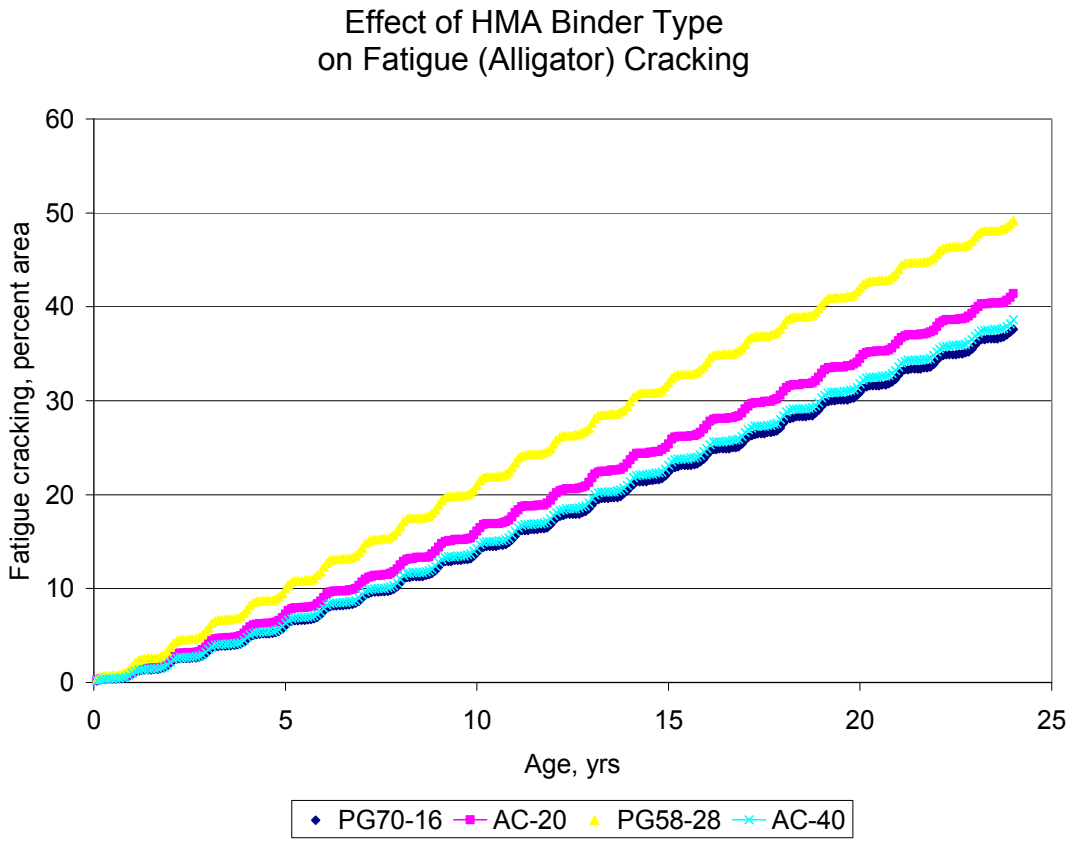

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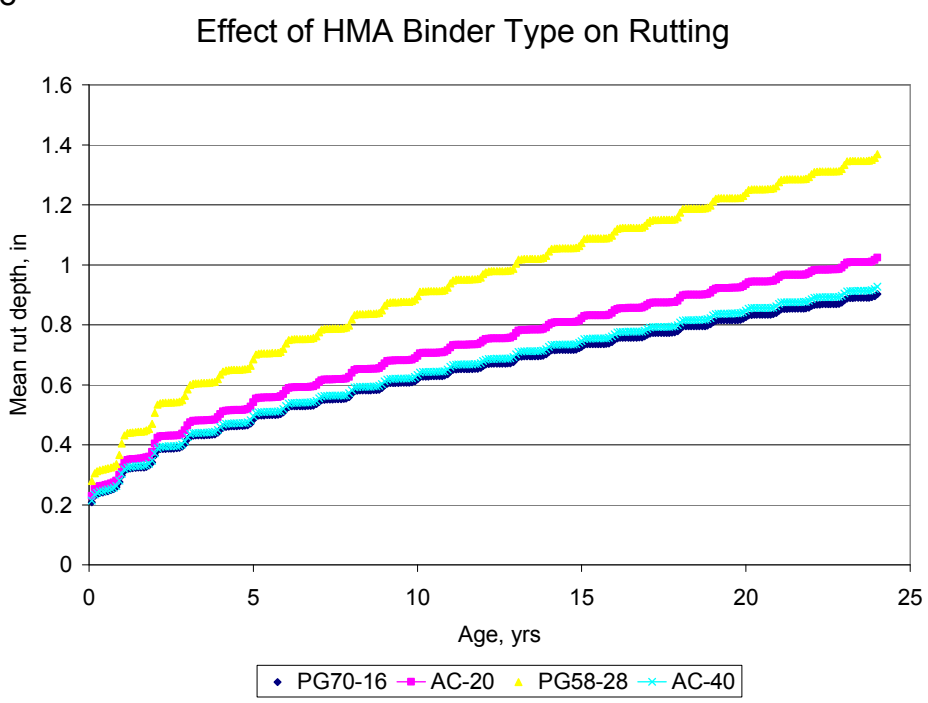


Table 15 – 9

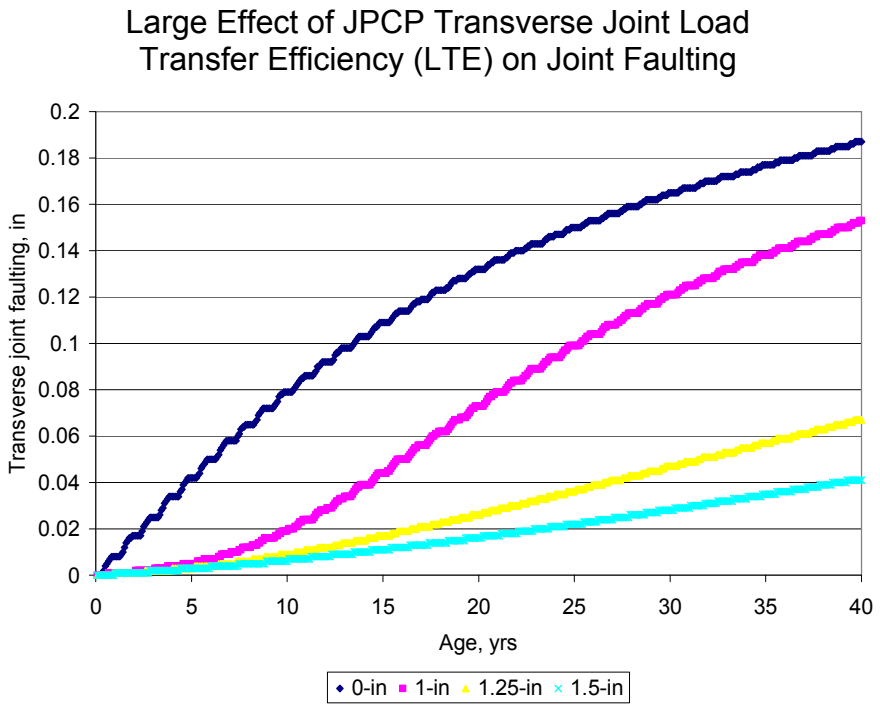
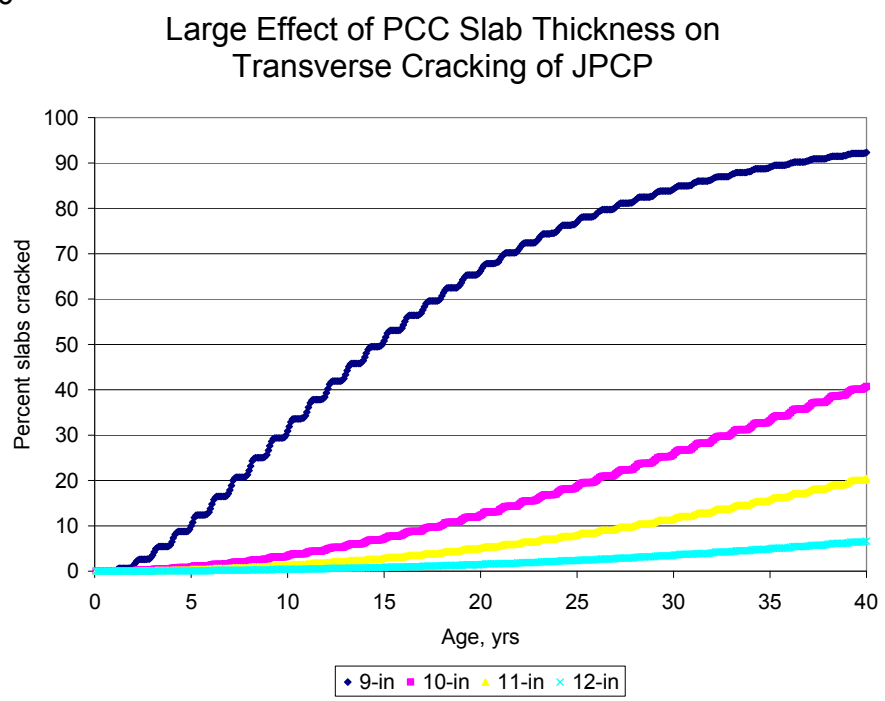


Table 15 – 10



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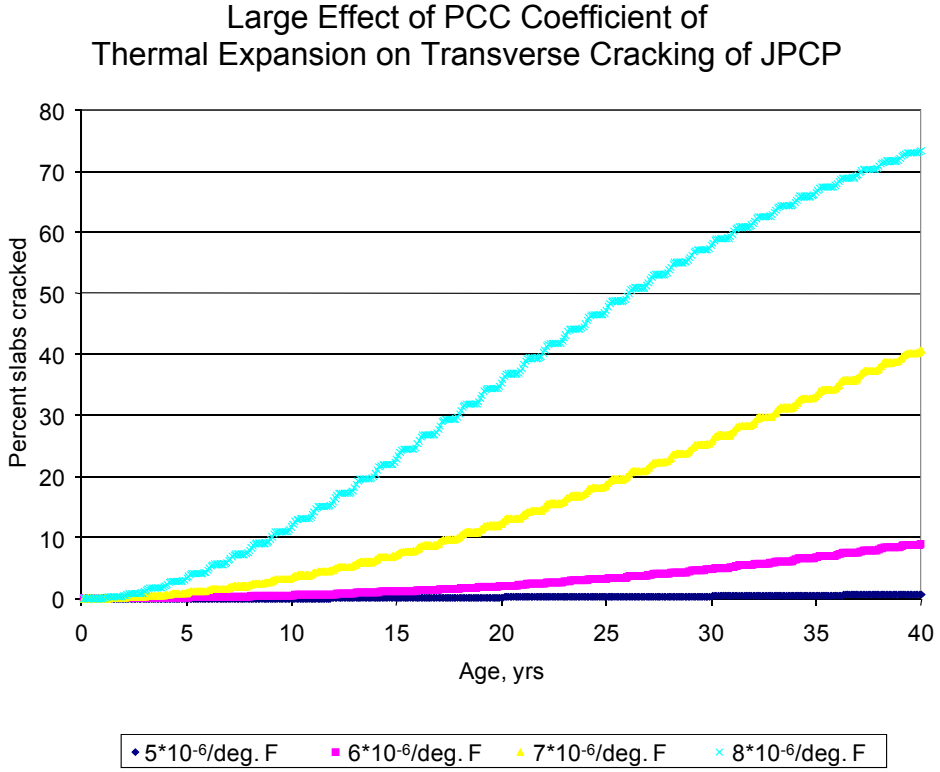

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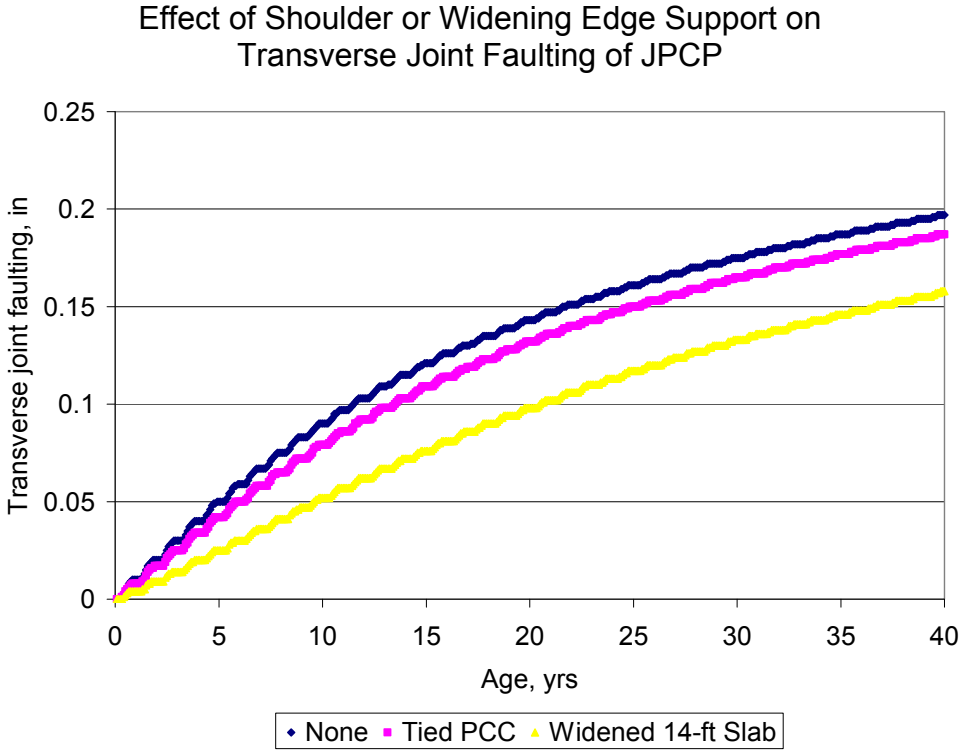


Table 15 – 13

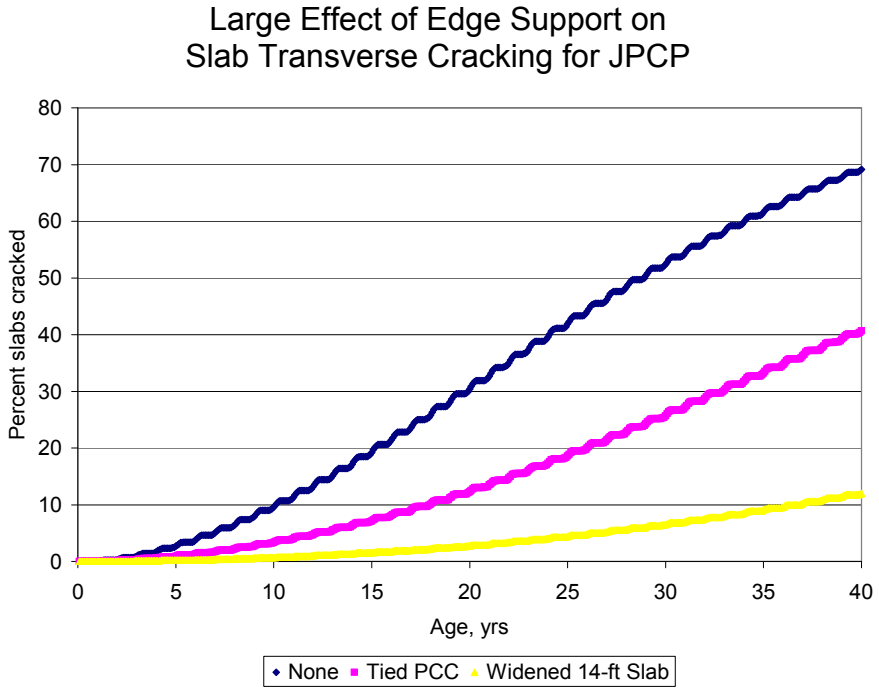
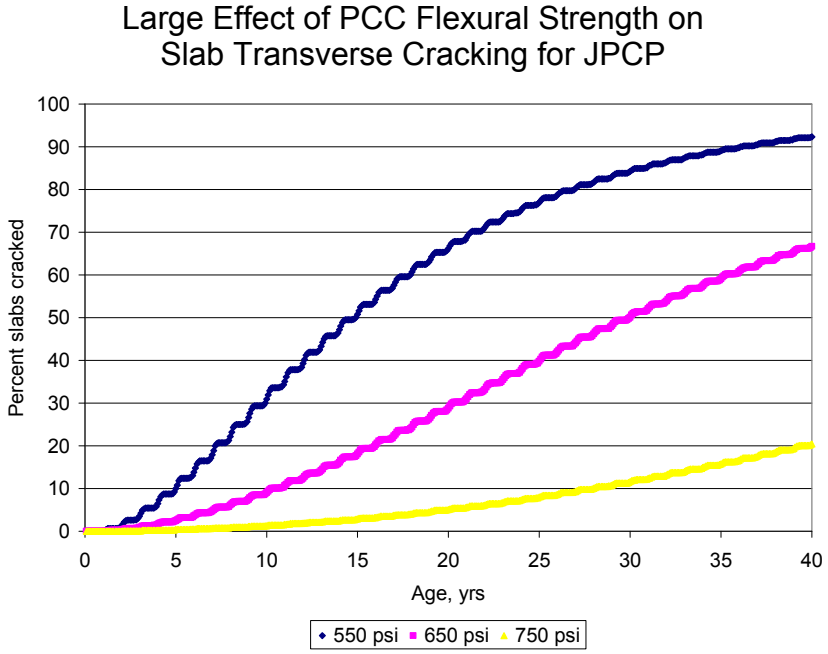


Table 15 – 14



Section 16
JPCP Design Features

JPCP design features have a significant impact on predicted JPCP performance. Designers can optimize JPCP design to produce the most cost effective pavement solution by selecting these inputs carefully. General guidance on selection of JPCP design inputs are provided in this section.

Summary of ME Design Sensitivity Results for Utah New/Reconstructed JPCP

JPCP Design Parameter	Recommended Inputs for JPCP Optimization
Slab thickness	Range: 6 to 16-in
Permanent curl/warp effective temperature difference (ºF)	-10 **[Critical]** **Do not change this input.**
Joint spacing (ft)	15 **[Critical] Do not exceed 15 ft.** **Shorter joint spacing may be used.**
Sealant type	Liquid
Doweled transverse joints	Yes, for most projects, except low volume roads **[Critical]**
Dowel diameter (inch)	Reference Standard Drawings for dowel bar diameter. If the dowel bar size obtained from Standard Drawings does not pass the performance criteria, then it may be necessary to increase the design size. The ME Design software will indicate joint faulting as not passing if the chosen bar is too small. **[Critical]** Note that these guidelines are only for transverse joints.
Dowel bar spacing (inch)	12 (Use 12 inches between dowels where there are five dowels per wheelpath). Note this is for transverse joints only.
Edge Support, Tied PCC shoulder, Long-term LTE (%)	• 40 for tied shoulders separately placed • 50 for tied shoulders monolithically placed
Base type	Actual specified
PCC-Base Interface Friction	The following lengths of time for full contact friction between the PCC slab and base course are recommended (means and range obtained from calibration): • Asphalt stabilized base: use full design analysis period. • Cement stabilized or lean concrete base: use 136 months (range of 0 to 360 mo.). • Unbound material base: use full design analysis period. • Unbonded overlay (with HMA separation layer): default set by ME Design.
Erodibility index of Base 1 Extremely erosion resistant 2 Very erosion resistant 3 Erosion resistant 4 Fairly erodible 5 Very erodible	Recommendations: • Permeable Base-extremely erosion resistant, Use 1. • Asphalt concrete-extremely erosion resistant, Use 1 if granular subbase placed below; otherwise 2 or 3. • Lean concrete (Ec > 2,000,000 psi- extremely erosion resistant), Use 1 if granular subbase placed below; otherwise 3. • Untreated dense graded aggregate- fairly erodible, Use 4. • Subgrade soil- very erodible, Use 5.

Section 17
Rehabilitation Inputs

Rehabilitation design is very similar to new or reconstructed design. The recommendations to these similar inputs will not be repeated. Rehabilitation design does require a few new inputs and some modifications of other inputs that are related to the existing pavement. The existing pavement has typically deteriorated from its original as-constructed condition through fracture, distortion, or material disintegration. Some of the material properties may also have aged and changed over time such as the oxidation of asphalt and the hardening of concrete. ME Design can account for these effects through modifying various design inputs and through a few new inputs related to the condition of the existing pavement. These modifications are basically used to adjust the various modulus of the existing pavement.

This section covers the modifications required of previously described inputs and the new inputs required for rehabilitation design. These inputs vary depending on the existing pavement and on the type of rehabilitation. Input recommendations are given for the following combinations of existing pavement and rehabilitation type:

1. HMA or JPCP overlay of existing HMA pavement – Table 17 – 1
2. JPCP unbonded overlay of existing JPCP – Table 17 – 2
3. HMA overlay of existing JPCP intact slab and fractured slab – Table 17– 3
4. CPR diamond grinding existing JPCP – Table 17 – 3

Table 17 – 1

Characterize Existing HMA or Existing HMA Overlaid HMA Pavement for HMA Overlay or JPCP Overlay (conventional white topping) Design		
Existing Pavement	**Rehabilitation Action**	**Rehabilitation Design Inputs Existing Pavement**
HMA or HMA overlaid HMA	HMA Overlay or JPCP Overlay	Dynamic modulus of existing HMA (Level 3): • Condition survey of alligator fatigue cracking in wheelpaths. • Compute percent area of traffic lane with alligator cracking, all levels of severity. • Select Pavement Rating Excellent <3% Good 4-5% Fair 6-10% Poor 11-20% Very Poor >20%
		Base course resilient modulus: • Back calculate from FWD testing and adjust for unusual conditions. • Use default values from Section 14 • Limit resilient modulus of unbound base to 2-3 times that of subgrade to avoid decompaction.

		Subgrade resilient modulus (Level 2 FWD back-calculation):
		• Determine AASHTO Soil Class from county soil maps* for the predominant soil. This also provides gradations and Atterberg limits. See also project geotechnical report.
		• Conduct FWD testing along the project in the outer wheelpath at regular intervals.
		• Back calculate subgrade field (elastic solid) Es at in situ moisture from FWD deflections using an appropriate elastic layered model (including bedrock if needed) or use the AASHTO 93 model outer sensor approach (if no bedrock). Clean data by removing unusual points.
		• Adjust each back calculated Es elastic modulus from a "field" elastic half space to a "lab" value and from an "in situ" moisture content to optimum moisture through the following multiplier adjustment:
		  o Coarse Grained Soils use 0.67
		  o Fine Grained Soils use 0.55.
		• This is the "lab adjusted M_r at optimum moisture content." Use for the ME Design input subgrade resilient modulus.
		This approach provides for a M_r that can be used as a direct input for the subgrade of either a reconstruct or for an overlay design using the FWD to obtain the subgrade M_r. This is the same approach used in the 2007 national calibration and the results should agree reasonably with Level 3 results in Section 14.
		The subgrade design input resilient modulus can also be roughly estimated using CBR. The ME Design software provides the following equation to estimate the resilient modulus.
		$\mathbf{M_r = 2555\ (CBR)^{0.64}}$ [CBR < 30%]
		
		An evaluation of this equation indicated that the resulting resilient modulus (M_r) for a CBR <30 are similar to those in Section 14 at optimum moisture content. For example, an A-4 soil with a CBR = 15 results in an estimated M_r = 14,457 psi (at optimum). The mean value for an A-4 soil is 16,429 psi (flexible) and 15,000 psi (rigid). However, this approach can result in serious errors in the estimation of subgrade support and is recommended only for preliminary estimation of subgrade M_r. FWD testing and backcalculation is the recommended procedure.

* The USDA-NRCS soil survey database.

Table 17 – 2

Characterizing Existing JPCP for Unbonded JPCP Overlay Design		
Existing Pavement	**Rehabilitation Action**	**Rehabilitation Inputs for Existing Pavement**
JPCP	Unbonded JPCP overlay	Effective elastic modulus of intact concrete slab: • Determine percent slab cracking of existing JPCP. • Select condition for input (transverse cracks all severities): Good 10% slabs cracked Moderate 20% slabs cracked Severe 50% slabs cracked • Determine EBASE/DESIGN = CBD * ETEST Where: EBASE/DESIGN = Design modulus of elasticity of existing concrete slab, psi CBD = Coefficient reduction factor: o 0.42 to 0.75 existing pavement “Good” condition. o 0.22 to 0.42 existing pavement “Moderate” condition. o 0.042 to 0.22 existing pavement “Severe” condition. ETEST = Elastic modulus of the existing uncracked concrete, psi. (estimate by testing of cores by ASTM C469 or using 28-day modulus and multiplying by 1.2 for approximate long term modulus)
		Modulus of Fractured JPCP: • Crack and seat JPCP: 125,,000 psi • Rubblized JPCP: 60,000 psi
		Unbound base course modulus: • Use default values from Section 14 • Limit resilient modulus of unbound base to 2-3 times that of subgrade to avoid decompaction of the base.
		Stabilized base course modulus: • Estimate cement stabilized E from Section 14 • Estimate asphalt stabilized dynamic modulus through volumetric and gradation inputs (Level 3).

		Subgrade resilient modulus from FWD testing and backcalculation: • Determine AASHTO Soil Class from county soil maps* for the predominant soil. This also provides gradations and Atterberg limits. See also the project geotechnical report. • Conduct FWD testing along the project in the center of the slab at regular intervals. • Back calculate "field" subgrade k value at in situ moisture content from FWD deflections on top of the slab. • Run the ME Design program with default INPUT M_r for the subgrade based on AASHTO Classification. • The ME Design OUTPUT k-values for given months must be compared to the back calculated k-values for same months. The input subgrade M_r must be adjusted until the FWD back calculated k-value matched that k-value in the ME Design output for the month of FWD testing. This approach is exactly what was done in the original national ME Design work. It ensures that the M_r and k-value used to compute stresses and deflections were reasonable and generally matched the field. This approach was applied to all of the Utah LTPP and PMS sections and found to produce M_r values that were similar. Subgrade resilient modulus from CBR testing: The subgrade design input resilient modulus can also be roughly estimated using CBR. The ME Design software provides the following equation to estimate the resilient modulus. $\mathbf{M_r = 2555\ (CBR)^{0.64}}$ [CBR < 30%] An evaluation of this equation indicated that the resulting resilient modulus (M_r) for a CBR <30 are similar to those in Section 14 at optimum moisture content. For example, an A-4 soil with a CBR = 15 results in an estimated M_r = 14,457 psi (at optimum). The mean value for an A-4 soil is 15,000 psi (rigid). However, this approach can result in serious errors in the estimation of subgrade support and is recommended only for preliminary estimation of subgrade M_r. FWD testing and backcalculation is the recommended procedure.

* The USDA-NRCS soil survey database.

Table 17 – 3

Characterizing Existing JPCP for HMA Overlay Design or CPR		
Existing Pavement	**Rehabilitation Action**	**Rehabilitation Inputs for Existing Pavement**
JPCP	HMA overlay or CPR	Elastic modulus of concrete slab: • Determine percent slab cracking of existing JPCP. • Input this percentage into ME Design Rehabilitation window. • Determine what percentage of cracked slabs will be replaced prior to HMA overlay or CPR and enter this into Rehabilitation window. • Estimate elastic modulus of existing slab by testing of cores using ASTM C469, or estimate using 28-day modulus and multiplying by 1.2 for approximate long term modulus.
		Modulus of Fractured JPCP (for HMA overlay): • Crack and seat JPCP: 125,000 psi • Rubblized JPCP: 60,000 psi • Unbound base course modulus: • Use default values from Section 14. • Limit resilient modulus of unbound base to 2-3 times that of subgrade.
		Stabilized base course modulus: • Estimate cement stabilized E from Section 14. • Estimate asphalt stabilized dynamic modulus through volumetric and gradation inputs (Level 3).
		Subgrade resilient modulus: • Determine AASHTO Soil Class from county soil maps* for the predominant soil. This also provides gradations and Atterberg limits. See also the project geotechnical report. • Use procedure described for JPCP overlay for FWD testing and back calculation.

* The USDA-NRCS soil survey database.

Section 18
Performing New or Reconstruct Pavement and Rehabilitation Designs

Designing a new or reconstructed HMA or JPCP pavement, CPR, and HMA and JPCP overlays require the following major steps:

1. **Select a trial design** – Use the current UTOT/AASHTO procedure or the experience of the designer as a starting point.
2. **Select the appropriate performance criteria and design reliability level for the project at hand.**
3. **Obtain all inputs for the pavement under consideration.** These inputs can be obtained using three different levels of effort as previously described. Run a small sensitivity to see how much it affects the design if a given input is unknown. Note that the ME Design software allows users to directly import most of the traffic electronic files. Traffic data can be obtained from UDOT Traffic Statistics office.
4. **Run the ME Design software**. Examine the inputs and outputs.
5. **Examine carefully the input summary.** Ensure the inputs are correct and what the designer intended.
6. **Examine all of the layer material moduli outputs.** Do this month by month over time to determine their reasonableness.
7. **Assess the trial design.** Has it met each of the performance criteria at the design reliability level?
8. **Criteria not met.** Determine how this design deficiency can be remedied by altering the materials used, layering, or other design details such as layer thickness, asphalt grade, and dowel bar diameter.
9. **Revise trial design as needed.** Revised the inputs/trial design and rerun the program if the trial design has input errors, material output problems, other potential problems, or has exceeded the performance criteria at the given level of reliability. Iterate until the performance criteria have been met. This design is a feasible design for further consideration in the pavement selection process when this criteria has been met. Refer to sensitivity analysis tables for HMA and PCC to determine which inputs have the biggest impact on the distresses and adjust these inputs to achieve suitable design.

Section 19
ME Design Outputs Used For Performance Assessment

The ME Design software analyzes a given trial design that is input and predicts its performance in terms of key distress types and smoothness. The ME Design software also has the capability of limited optimization on layer thickness. The designer must input a range of thickness of HMA or PCC and base course and the program will iterate until it identifies the minimum thicknesses that still meet the design reliability and performance criteria.

Materials properties and other factors are output on a month-by-month basis over the design period. Each pavement type and rehabilitation type has its own specific output tables and charts. The designer examines the output materials properties and other factors to see if reasonable results are being obtained. Occasionally a weather station may contain erroneous temperatures, precipitation, and other values that cause major problems with the layer moduli output from ME Design. These must be reviewed and new weather stations used to provide reasonable moduli.

The output provides the HMA Dynamic Modulus (E*) and the resilient modulus (M_r) for unbound layers of asphalt pavements for each month over the design period. Vehicle speed and temperature affect the HMA material E* greatly. Moisture content and frost condition affects the unbound materials M_r greatly. The designer can observe these and assess their reasonableness.

The output provides the PCC modulus of rupture and modulus of elasticity for each month over the design period for concrete pavements. The back calculated subgrade k-value is also output monthly. Load transfer efficiency LTE at joints is also output. Use a larger dowel if the LTE drops below 70 percent. Moisture content and frost condition affects the unbound materials M_r and k-value greatly. The designer can observe these and assess their reasonableness.

The designer must examine the key distress type outputs and smoothness to see if they are meeting the performance criteria. The first two years of key distress output is shown for an HMA pavement below. The distress and IRI are output at the end of each month over the design period. The number of cumulative Heavy Trucks (Class 4 and above) are also shown in the design traffic lane. Examples of ME Design output tables and plots for new HMA pavement and new JPCP analysis are presented in Tables 19 – 1 through 19 – 14

The red horizontal line for all distress/IRI plots represents the limiting performance criteria at a given level of reliability. The design is acceptable if distress/IRI at the specified reliability is less than the red line over the entire design period. Another method for assessing design adequacy is to review the Reliability Output (Table 19 – 2).

The Distress Target and its corresponding Reliability Target are the first right hand columns listed followed by the Distress Predicted and the Reliability Predicted. The pavement passes if the Reliability Predicted is greater than the Reliability Target. The pavement fails if the reverse is true. The designer must alter the trial design to correct the problem if any key distress fails. This "trial and error" process allows the pavement designer to essentially build the pavement in his or her computer prior to building it in the field to see if it will perform. Problems with the design and materials for the given subgrade, climate, and traffic, can be corrected and an early failure avoided. This is the power of the ME design methodology.

New HMA and Rehabilitation with HMA

MS Excel output has many different Tabs that include tables and graphical results about the trial project design.

- **Grand Summary:** Summary of performance and reliability results, traffic, , and design
- **Climate:** Summary of all computed climate related data such as rainfall, freezing index, temperature profiles within the pavement, and so on for the given project.
- **Several Distress Summaries:** Summary of all predicted distress at 50 percent reliability (provided IRI predictions at specified reliability level) including bottom up alligator fatigue cracking, transverse low temperature cracking, permanent deformation or rutting, and IRI.
- **Layer Modulus:** Various tabular and graphical summaries of layer moduli for all layers at various depth (presented for the entire analysis period).
- **HMA Modulus:** Plot of HMA layers modulus over the analysis period.
- **Materials Summaries:** Graphical and tabular summaries of many concrete, base, and subgrade material properties over time.
- **Traffic Summaries:** Graphical and tabular summaries of many traffic related outputs over time.
- subgrade) versus age (at 50 and specified reliability levels).
- **IRI:** Plot of predicted IRI versus age (at 50 and specified reliability levels).

The designer can also use the output information to optimize the HMA and JPCP designs. This means that by observing what is the key type of distress occurring, one or more design inputs can be modified to achieve a more cost effective optimum design.

For example, if a JPCP is faulting and creating a lower reliability, then the dowel bar diameter can be increased to prevent faulting. The slab thickness should not be increased to try to fix a faulting problem.

For example, if a HMA pavement is rutting creating a lower reliability, the layer in which the most rutting is occurring should be located and steps taken to reduce that faulting. An upgrade in binder type should be tried for example if the majority of rutting is in the HMA layers.

New JPCP and Rehabilitation with JPCP (including CPR)

MS Excel output has many different Tabs that include tables and graphical results about the trial project design.:

- **Grand Summary:** Summary of all inputs such as traffic, climate, design, reliability and performance..
- **Climate:** Summary of all computed climate related data such as rainfall, freezing index, temperature profiles within the pavement, and so on for the given project.
- **Several Distress Summaries and graphics:** Summary of all predicted distress at 50 percent reliability provided IRI predictions at specified reliability level.
- **Materials Summaries:** Graphical and tabular summaries of many concrete, base, and subgrade material properties over time.
- **Traffic Summaries:** Graphical and tabular summaries of many traffic related outputs over time.

Table 19 – 1

Distress Summary Output Worksheet for New HMA

Month	Pavement Age (years)	Heavy Trucks (cum.)	Crack Spacing (ft)	Mean Predicted Distress				Predicted Distress @ Reliability			
				IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal fracture (ft/mile)	Total Cracking (Reflective + Alligator) (%)	IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal fracture (ft/mile)	Total Cracking (Reflective + Alligator) (%)
10/2012	0.08	26,192	500	70.7	0.018	0.00	0.00	96.6	0.038	27.17	-
11/2012	0.17	49,556	500	70.8	0.019	0.00	0.00	96.7	0.040	27.17	-
12/2012	0.25	73,584	500	70.8	0.019	0.00	0.00	96.8	0.041	27.17	-
1/2013	0.33	97,541	500	70.9	0.020	0.00	0.00	96.8	0.042	27.17	-
2/2013	0.42	122,177	500	70.9	0.020	0.00	0.00	96.9	0.043	27.17	-
3/2013	0.50	148,071	500	71.0	0.021	0.00	0.00	97.0	0.044	27.17	-
4/2013	0.58	176,918	500	71.0	0.023	0.00	0.00	97.1	0.045	27.17	-
5/2013	0.67	208,979	500	71.3	0.028	0.00	0.01	97.4	0.051	27.17	-
6/2013	0.75	242,349	500	71.8	0.041	0.00	0.02	98.2	0.066	27.17	-
7/2013	0.83	270,405	500	72.5	0.056	0.00	0.03	99.0	0.086	27.17	-
8/2013	0.92	301,282	500	72.8	0.064	0.00	0.04	99.6	0.097	27.17	-
9/2013	1.00	328,757	500	73.0	0.066	0.00	0.05	99.7	0.100	27.17	-
10/2013	1.08	355,683	500	73.0	0.066	0.00	0.05	99.8	0.100	27.17	-
11/2013	1.17	379,700	500	73.0	0.067	0.00	0.05	99.9	0.101	27.17	-
12/2013	1.25	404,401	500	73.1	0.067	0.00	0.05	99.9	0.101	27.17	-
1/2014	1.33	429,029	500	73.1	0.067	0.00	0.05	100.0	0.101	27.17	-
2/2014	1.42	454,355	500	73.2	0.067	0.00	0.05	100.0	0.101	27.17	-
3/2014	1.50	480,974	500	73.2	0.067	0.00	0.05	100.1	0.101	27.17	-
4/2014	1.58	510,629	500	73.3	0.067	0.00	0.05	100.2	0.101	27.17	-
5/2014	1.67	543,587	500	73.4	0.069	0.00	0.06	100.4	0.104	27.17	-
6/2014	1.75	577,891	500	73.7	0.076	0.00	0.07	100.8	0.114	27.17	-
7/2014	1.83	606,733	500	74.2	0.086	0.00	0.09	101.5	0.127	27.17	-
8/2014	1.92	638,475	500	74.5	0.093	0.00	0.11	101.9	0.137	27.17	-
9/2014	2.00	666,719	500	74.6	0.095	0.00	0.12	102.1	0.139	27.17	-
Etc.	over	20 years									
7/2032	19.83	8,557,670	500	107.0	0.283	0.00	1.88	145.7	0.389	27.17	-
8/2032	19.92	8,609,850	500	107.3	0.285	0.00	1.91	146.0	0.391	27.17	-
9/2032	20.00	8,656,280	500	107.4	0.285	0.00	1.92	146.2	0.392	27.17	-

Table 19 – 2

Reliability Summary for New HMA

Distress Prediction Summary					
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	172.00	141.79	90.00	98.97	Pass
Permanent deformation - total pavement (in.)	0.75	0.35	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.70	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	27.17	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	1033.83	90.00	99.36	Pass
Permanent deformation - AC only (in.)	0.25	0.09	90.00	100.00	Pass

Table 19 – 3

Plot of Computed HMA Dynamic Modulus for New HMA

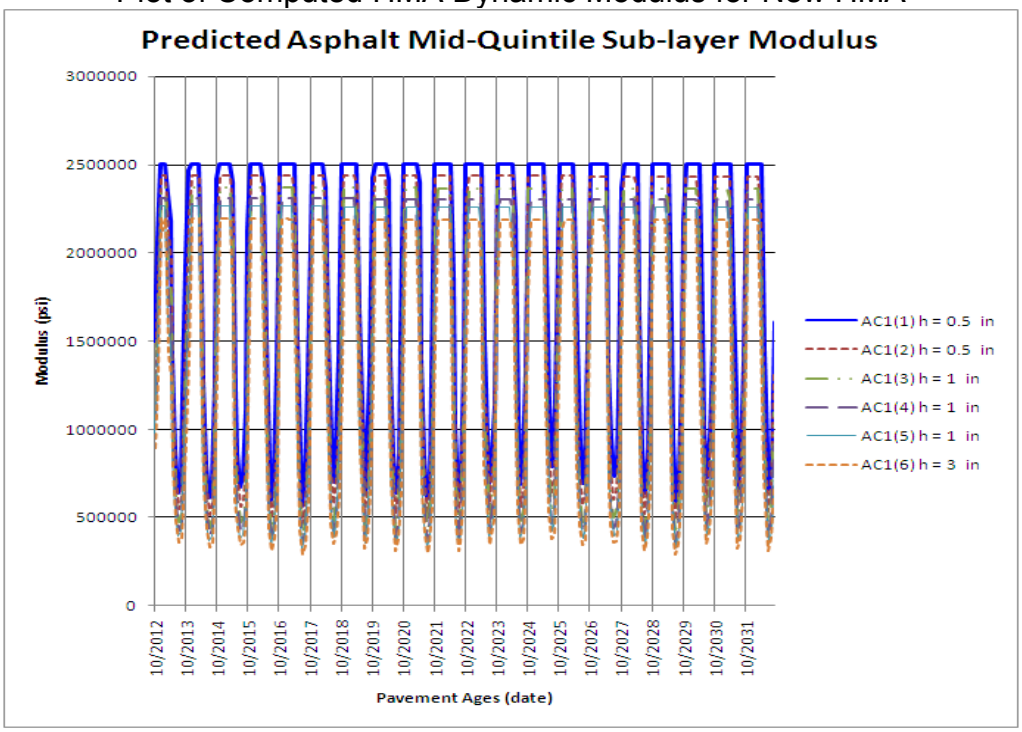


Table 19 – 4

Plot of Bottom-Up Damage
Over Analysis Period for New HMA

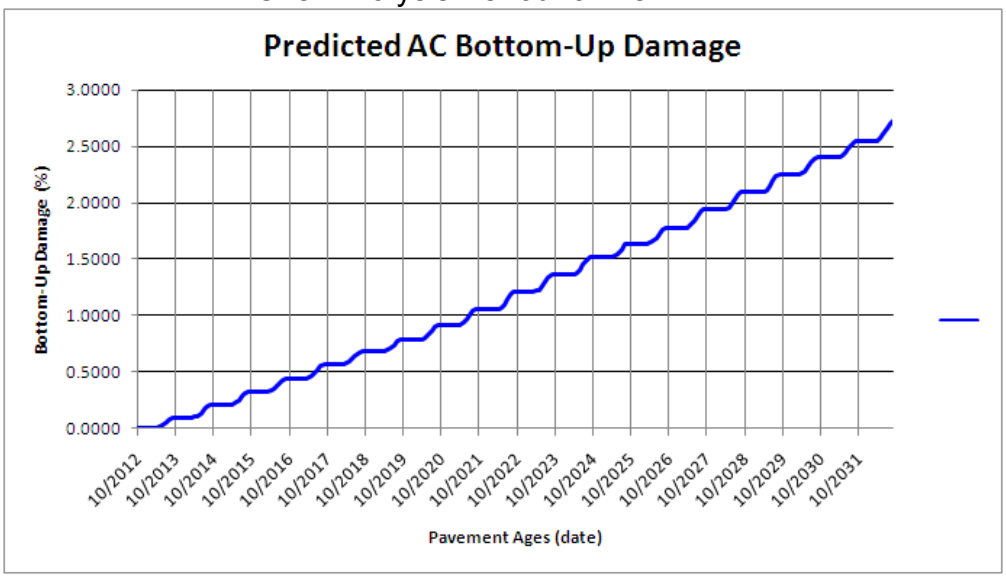


Table 19 – 5

Plot of Bottom-Up Cracking
Over Analysis Period for New HMA

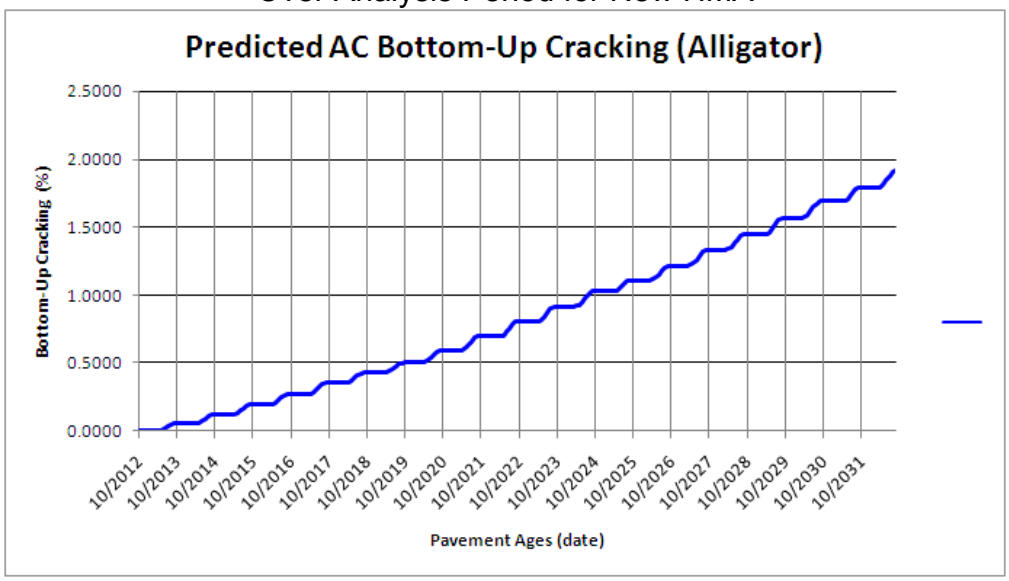


Table 19 – 6

Plot of Thermal Cracking
Over Analysis Period for New HMA

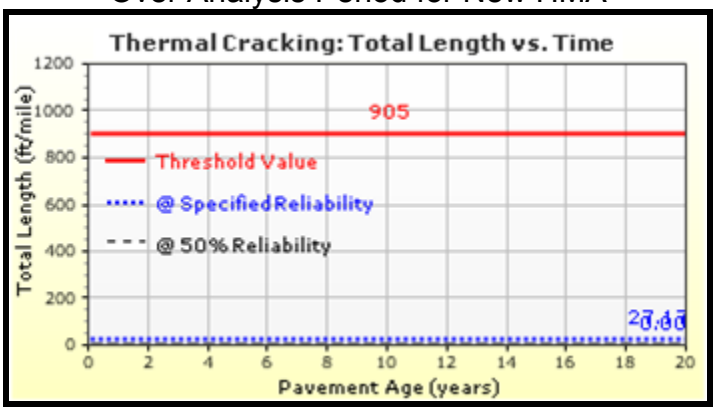


Table 19 – 7

Plot of Rutting Over Analysis Period
for New HMA

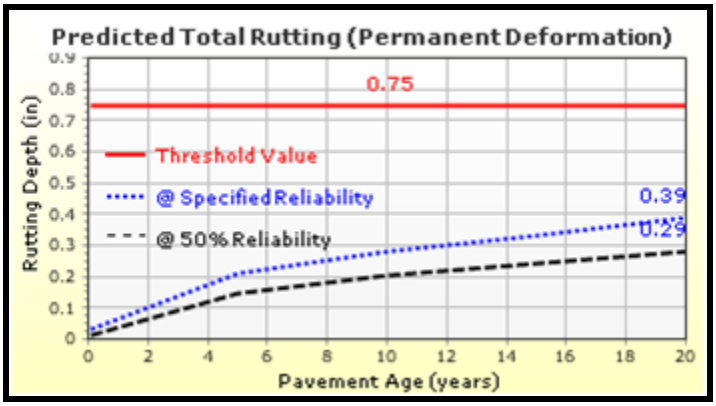


Table 19 – 8

Plot of IRI
Over Analysis Period for New HMA

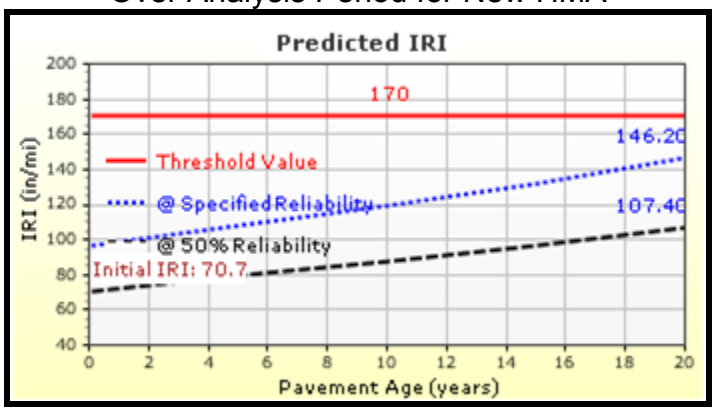


Table 19 – 9

Distress Summary Output Worksheet for New JPCP

Month	Pavement Age (years)	Heavy Trucks (cum.)	Mean Predicted Distress			Predicted Distress @ Reliability		
			IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent slabs)	IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent slabs)
6/2012	0.08	51,457	70.0	0.000	0.00	105.1	0.023	4.92
7/2012	0.17	105,549	70.1	0.000	0.01	105.6	0.024	5.45
8/2012	0.25	150,483	70.1	0.000	0.01	105.6	0.024	5.45
9/2012	0.33	187,744	70.2	0.000	0.01	105.8	0.025	5.45
10/2012	0.42	225,565	70.2	0.000	0.01	105.8	0.025	5.45
11/2012	0.50	259,016	70.2	0.000	0.01	105.8	0.025	5.45
12/2012	0.58	288,746	70.2	0.000	0.01	106.0	0.026	5.45
1/2013	0.67	324,207	70.2	0.000	0.01	106.0	0.026	5.45
2/2013	0.75	361,520	70.2	0.000	0.01	106.0	0.026	5.45
3/2013	0.83	403,108	70.3	0.000	0.01	106.2	0.026	5.45
4/2013	0.92	446,129	70.3	0.000	0.01	106.2	0.026	5.45
5/2013	1.00	493,092	70.4	0.001	0.01	106.3	0.026	5.45
6/2013	1.08	546,608	70.4	0.001	0.02	106.3	0.026	5.62
7/2013	1.17	602,863	70.4	0.001	0.02	106.5	0.027	5.62
8/2013	1.25	649,595	70.4	0.001	0.03	106.5	0.027	5.75
9/2013	1.33	688,346	70.4	0.001	0.03	106.5	0.027	5.75
10/2013	1.42	727,680	70.5	0.001	0.03	106.7	0.027	5.75
11/2013	1.50	762,468	70.5	0.001	0.03	106.7	0.027	5.75
12/2013	1.58	793,388	70.6	0.001	0.03	106.8	0.028	5.75
1/2014	1.67	830,267	70.6	0.001	0.03	106.8	0.028	5.75
2/2014	1.75	869,073	70.6	0.001	0.03	106.8	0.028	5.75
3/2014	1.83	912,324	70.6	0.001	0.03	107.0	0.028	5.75
4/2014	1.92	957,066	70.6	0.001	0.03	107.0	0.028	5.75
5/2014	2.00	1,005,910	70.7	0.001	0.03	107.1	0.028	5.75
Etc. over entire 30 year design life								
5/2041	29.00	22,307,500	90.3	0.027	1.18	141.7	0.073	9.47
6/2041	29.08	22,418,600	90.4	0.027	1.20	141.8	0.073	9.51
7/2041	29.17	22,535,500	90.5	0.027	1.23	141.9	0.073	9.57
8/2041	29.25	22,632,500	90.5	0.027	1.24	141.9	0.073	9.59
9/2041	29.33	22,713,000	90.6	0.027	1.25	142.0	0.074	9.61
10/2041	29.42	22,794,700	90.7	0.028	1.25	142.2	0.074	9.61
11/2041	29.50	22,867,000	90.7	0.028	1.25	142.3	0.074	9.61
12/2041	29.58	22,931,200	90.7	0.028	1.25	142.3	0.074	9.61
1/2042	29.67	23,007,800	90.8	0.028	1.25	142.4	0.074	9.61
2/2042	29.75	23,088,400	90.8	0.028	1.25	142.5	0.074	9.61
3/2042	29.83	23,178,200	90.9	0.028	1.25	142.6	0.074	9.61
4/2042	29.92	23,271,100	91.0	0.028	1.25	142.7	0.075	9.61
5/2042	30.00	23,372,600	91.0	0.028	1.26	142.8	0.075	9.63

Table 19 – 10

Plot of Faulting
Over Analysis Period for New JPCP

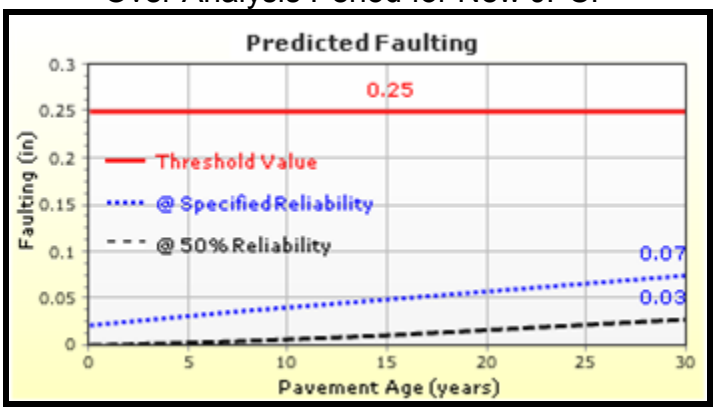


Table 19 – 11

Plot of LTE
Over Analysis Period for New JPCP Without Dowels

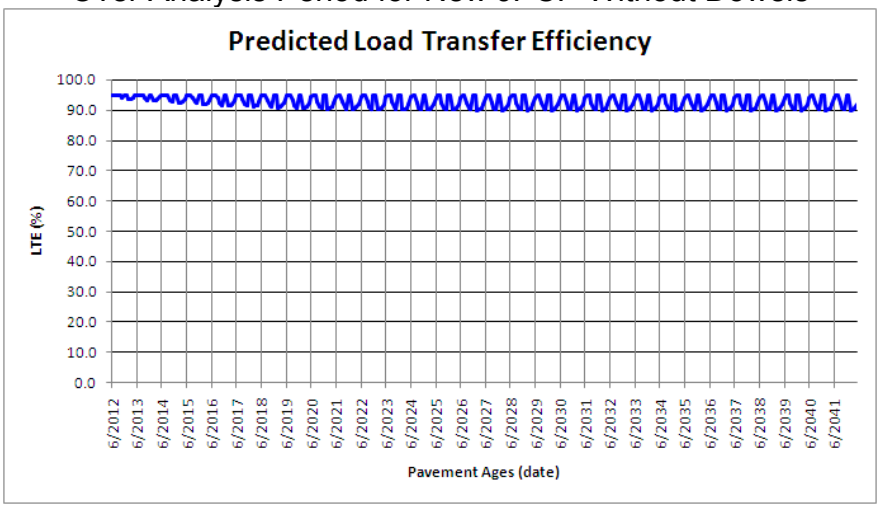


Table 19 – 12

Plot of Cumulative Damage (Cracking)
Over Analysis Period for New JPCP

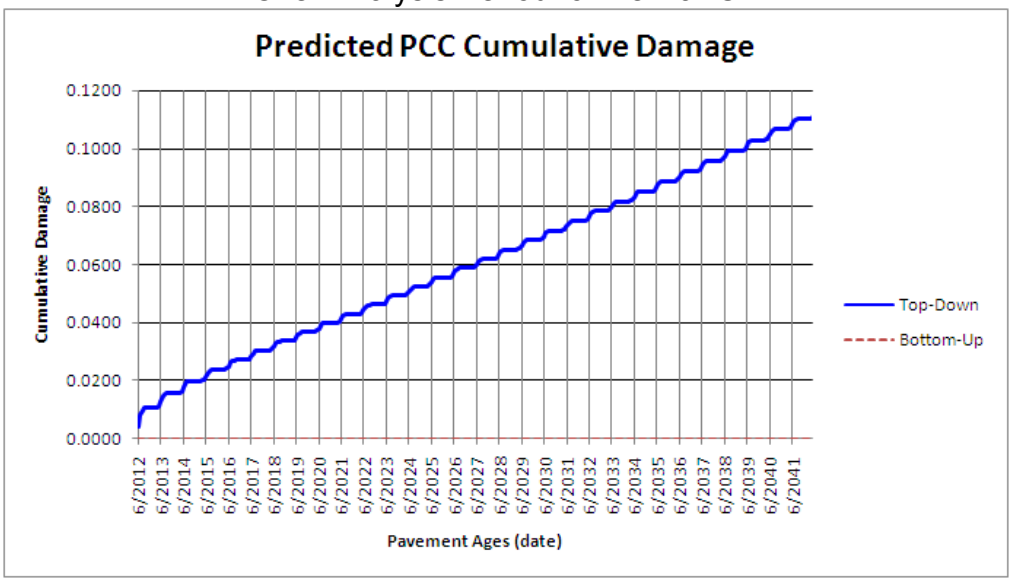


Table 19 – 13

Plot of Cracking
Over Analysis Period for New JPCP

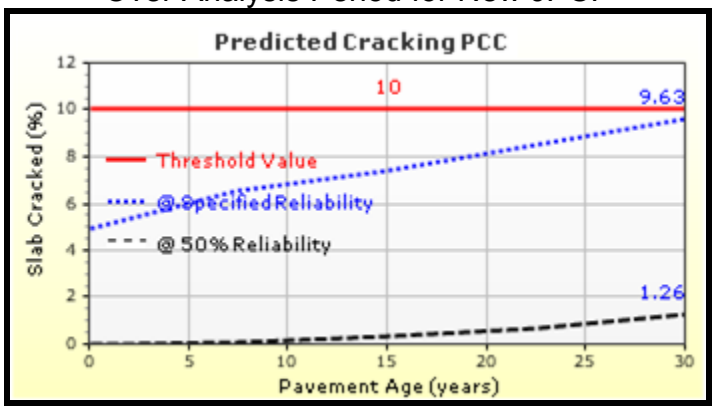
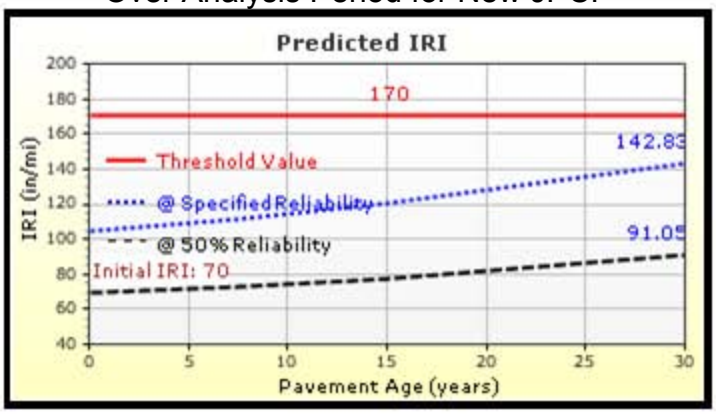


Table 19 – 14

Plot of IRI
Over Analysis Period for New JPCP



APPENDIX A
List of Region Designated Pavement Corridors

UDOT Designated Pavement Corridors

Region 1 – Concrete Corridors
I-84 except for Weber Canyon
I-15
SR-26
SR-204
SR-68
SR-39 From SR-126 to SR-203
SR-104
US-89 Davis and Weber County
US-91 From SR-165 to 2500 North
1000 West Cache County

Region 1 – Asphalt Corridors
I-84 Weber canyon MP 87.6 to MP 94.8
US-89 Logan Canyon
US-91 Brigham and Wellsville canyons
SR-167 Trappers Loop
SR-226 Snowbasin Road
SR-158 Powder Mountain Road
SR-39 Ogden Canyon and Monti Cristo
SR-243 Beaver Mountain Road
SR-66 Morgan to East Canyon
SR-67 Big Mountain

Region 2 – Concrete Corridors
I-15
I-80 Grantsville to to Wyoming Line
SR-68 SR-201 to Bangerter Highway
I-84 Echo to Morgan County Line
SR-201
Bangerter HWY
I-215
5600 West 201 to I-80
Mt. View Corridor
US 40

Region 2 – Asphalt Corridors
US-89
I-80 Wendover to Grantsvillle and Silver Creek to Echo – Lower Trucks and ADT SR-36
SR-171
SR – 186 47th South
SR-210

SR-209
SR…….32,35,36,48,58,65,71,73,86,89,112,138,140,150,151,152,171,172,173,181,184,186,190,196,199,202,209,210,224,248,266,269,270,280,2,302

Region 3 – Concrete Corridors
I-15 – Spanish Fork to Utah/SL Co. Line
SR-40 From Summit/Wasatch Co. Line to Heber
SR-52
SR-73 from I-15 to US-89
SR-75
SR-77
US-89 – University Ave to I-15 Lehi
SR-145
SR-180
US-189 – 1-15 to SR-52
SR-265

Region 3 – Asphalt Corridors
All other routes

Region 4 – Asphalt and Concrete Corridors
General
The following determinations were made by the Region 4 Engineering Team. The purpose is to identify preferred pavement types for select routes within Region 4. Potential deterrents and justifications are also identified. **Roadways and roadway segments not identified are suitable candidates for either flexible or rigid pavement types.**

Interstates
<u>I-15</u>

MP 59-68
Preference Flexible
Why Unstable sub-grade
MP 138-144
Preference Flexible
Why Unstable sub-grade

<u>I-70</u>

MP 47-231.67
Preference Flexible
Why Unstable sub-grades, Lack of suitable pcc aggregates, Canyons, steep grades, High elevations (snow removal issues)

NHS

US-6

MP 202-300
Preference Flexible
Why Unstable sub-grade, Lack of suitable pcc aggregates, Canyons, Steep grades, High elevations (snow removal issues)

US-9
MP 47.77 - 57.07
Preference Flexible
Why Unstable sub-grade, High elevations (snow removal issues), Remote

US-50
MP 60-75
Preference Flexible
Why Unstable sub-grade

US-89
MP 0-226
Preference Flexible
Why Unstable sub-grade in areas Lack of suitable pcc aggregates Remote

US-191
MP 26-37
Preference Flexible
Why Unstable sub-grade

MP 128-157
Preference Flexible
Why Unstable sub-grade

State
SR-10
ALL
Preference Flexible
Why Unstable sub-grade

SR-12
ALL
Preference Flexible
Why High elevations (snow removal issues), Canyons, Steep grades, Remote

SR-24

ALL
Preference Flexible
Why High elevations (snow removal issues), Canyons, Steep grades, Remote

SR-95
ALL
Preference Flexible
Why Unstable sub-grade in areas, Lack of suitable pcc aggregates, Remote

SR-163
ALL
Preference Flexible
Why Unstable sub-grade in areas, Lack of suitable pcc aggregates, Remote 97

SR-261
ALL
Preference Flexible
Why Unstable sub-grade in areas, Lack of suitable pcc aggregates, Remote

SR-262
ALL
Preference Flexible
Why Unstable sub-grade in areas, Lack of suitable pcc aggregates, Remote

SR-276
ALL
Preference Flexible
Why Unstable sub-grade in areas, Lack of suitable pcc aggregates, Remote

APPENDIX B
MEPDG Design Review Checklist

Use the following steps to review a submitted cone file for conformance with UDOT ME Design practices:

1. Error Checking – *MEPDG Function*
   a. Review submitted design for any errors or non-typical items that have been included in the design.
   b. Errors are for numbers that are completely inappropriate for design analysis such as an E* value for lower frequency that is higher than the value for the adjacent frequency. Frequency and modulus are proportional and as the frequency goes up, the modulus should go up.
2. Database Comparison – *MEPDG Function*
   a. This is similar to the error checking but focuses more on the abnormal inputs.
   b. Some of the inputs may be appropriate but will need explanation or justification such as if CIR values are used for an asphalt layer the values used may not meet the default limits
3. Capacity Limits – *User Function*
   a. Check the traffic information to see if it is appropriate or exceeding the limits of the roadway. Traffic that exceeds the limits of the roadway will need to be justified.
4. Cone File Comparison – *MEPDG Function*
   a. Use if you wish to compare a cone file of your own to the submitted cone file.
   b. This requires you to have a cone file of your own that may take some time to develop.
5. Sensitivity Analysis – *MEPDG Function*
   a. Each of the above comparisons will give a list of deviations. Errors should be corrected, but the remainder can be reviewed for sensitivity to design such as changing the unit weight of HMA may not affect the design at all but changing the in-place air voids will have a significant impact. Sensitive items include:
      1) Binder grade
      2) HMA Gradations and Volumetrics
      3) PCC CTE
      4) PCC Strengths
      5) Base Modulus values
6. Other spot check options – *User Functions*:
   a. Review base modulus values to verify that values are rising and falling in conjunction with typical dry and wet periods throughout the year.
   b. Verify binder grade is appropriate for application
   c. Review HMA gradation values

d. Review PCC Flexural and M_r strengths

Defaults That Should Not Change

1. Default local calibration constants
2. Traffic
   a. All traffic inputs other than AADT, % trucks, growth and lane configuration
   b. Capping of AADT for service based on Highway Capacity Manual
3. Performance Criteria
   a. Design Life
   b. Performance Criteria, including reliability and standard deviation
   c. Initial IRI – Within Range – Refer to Table 9 – 7.
4. Weather Stations – Selected by Region Pavement Management Engineer including the water table depth.
5. Pavement Structure
   a. Curl/Warp temperature difference
   b. Sealant type – "other"(not preformed)
   c. Dowel size and spacing – in accordance with PV Standard Drawings
6. Materials Layer Inputs
   a. HMA
      1) NCHRP 1-37A Calibration Constants (National Calibration)
      2) Fatigue Endurance – none
      3) HMA Property Reference Temperature
      4) HMA Creep Values – Use default calculations from other HMA inputs
      5) Poisson's Ratio
      6) Asphalt General Thermal Properties
      7) Thermal Cracking values – use defaults calculated from other HMA inputs
   b. PCC
      1) Curing method to be "Curing Compound"
      2) PCC Shrinkage values – Use default calculation from other PCC inputs
      3) PCC Thermal properties – Use defaults
      4) Poisson's ratio
   c. Base Courses
      1) Poisson's ratio
      2) Coefficient of lateral pressure

Inputs That Are Allowed to Change

1. Traffic
    a. AADT
    b. Truck Percentage
    c. Lane configurations
    d. Growth factor
2. Subgrade Layer Properties – Require multiple lab/field testing for justification (1 per 800 feet on reconstruct, 1 per mile when in conjunction with FWD)
    a. CBR/Moduli at optimum moisture content
        1) Saturated CBR*1.49 for fine grained
        2) Saturated CBR*1.81 for coarse grained (A-3 or better)
    b. soil classification
    c. Depth to bedrock
3. Base Layer Properties – Require multiple lab/field testing for justification
    a. CBR with cap at 100
    b. Modulus values based on lab test
    c. Soil classification
    d. Must be followed in the field according to Standard Specification Section 02056, Embankment, Borrow, and Backfill and Section 02721, Untreated Base Course (UBC).
        1) Must submit 3-4 samples to start with and one per every day in penalty after that.
4. HMA Surfacing
    a. Layer thickness
    b. Binder Grade or G* and Delta (lab values)
    c. Gradation, according to specifications of proposed mix
    d. Mix E*(with laboratory testing) capped at 700,000 psi
    e. Field values for voids, effective binder content, unit weight
5. PCC Surfacing
    a. Layer thickness
    b. 28 day strengths (based on mix designs or field values)
    c. Coefficient of Thermal Expansion (CTE) New and Existing Intact PCC Section 14.4. Also see Appendix H for Typical CTE values for material pits around.
    d. Flex strength, to a maximum of 750 psi
    e. PCC Modulus of Rupture, to a maximum of 4,500,000 psi
6. Pavement Structure
    a. Joint Spacing – maximum of 15 feet, minimum of 10 feet except for thin overlays, no random spacings
    b. Tied shoulder
    c. Widened slab
    d. Erodibility index

APPENDIX C
Unified Soil Classification System (USCS)

The adoption of the principles of soil mechanics by the engineering profession has inspired numerous attempts to devise a simple classification system that will tell the engineer the properties of a given soil. Many classifications have come into existence based on certain properties of soils such as texture, plasticity, strength, and other characteristics. A few classification systems have gained fairly wide acceptance but rarely has any system provided the complete information on a soil that the engineer needs. Nearly every engineer who practices soil mechanics will add judgment and personal experience as modifiers to whatever soil classification system he uses. A common basis of soil classification is necessary so when an engineer classifies a soil as a certain type, this classification will convey the proper characteristics and behavior of the material since sometimes designs and plans are reviewed by people entirely removed from a project. The classification should reflect those behavior characteristics of the soil that are pertinent to the project under consideration.

BASIS OF THE USCS

The USCS is based on identifying soils according to their textural and plasticity qualities and on their grouping with respect to behavior. Soils seldom exist in nature separately as sand, gravel, or any other single component. They are usually found as mixtures with varying proportions of particles of different sizes. Each component part contributes its characteristics to the soil mixture. The USCS is based on those characteristics of the soil that indicate how it will behave as an engineering construction material. The following properties have been found most useful for this purpose and form the basis of soil identification. They can be determined by simple tests and, with experience, can be estimated with some accuracy.

- Percentages of gravel, sand, and fines (fraction passing the No. 200 sieve).
- Shape of the grain-size-distribution curve.
- Plasticity and compressibility characteristics. In the USCS, the soil is given a descriptive name and a letter symbol indicating its principal characteristics.

PURPOSE AND SCOPE

It is the purpose of this appendix to describe the various soil groups in detail and to discuss the methods of identification so that a uniform classification procedure may be followed by all who use the system. Placement of the soils into their respective groups is accomplished by visual examination and laboratory tests as a means of basic identification. USCS in its present form may not prove entirely adequate in all cases. It is intended that the classification of soils according to this system have some degree of elasticity and that the system not be followed blindly nor regarded as completely rigid.

DEFINITIONS OF SOIL COMPONENTS

It is necessary to establish a basic terminology for the various soil components and to

define the terms used before soils can be classified properly in any system, including the one presented in this manual. The terms cobbles, gravel, sand, and fines (silt or clay) are used to designate the size ranges of soil particles in the USCS. The gravel and sand ranges are further subdivided into the groups as presented in Table C-1. The limiting boundaries between the various size ranges have been arbitrarily set at certain US standard sieve sizes. The terms silt and clay are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity in the finest soil component. The minus No. 200 sieve material is silt if the LL and PI plot below the "A" line on the plasticity chart and is clay if the LL and PI plot above the "A" line on the chart. All LL and PL tests are based on minus No. 40 sieve fraction of a soil. This definition holds for inorganic silts and clays and for organic silts but is not valid for organic clays since these soils plot below the "A" line. The names of the basic soil components can be used as nouns or adjectives when describing or classifying a soil.

THE CLASSIFICATION SYSTEM

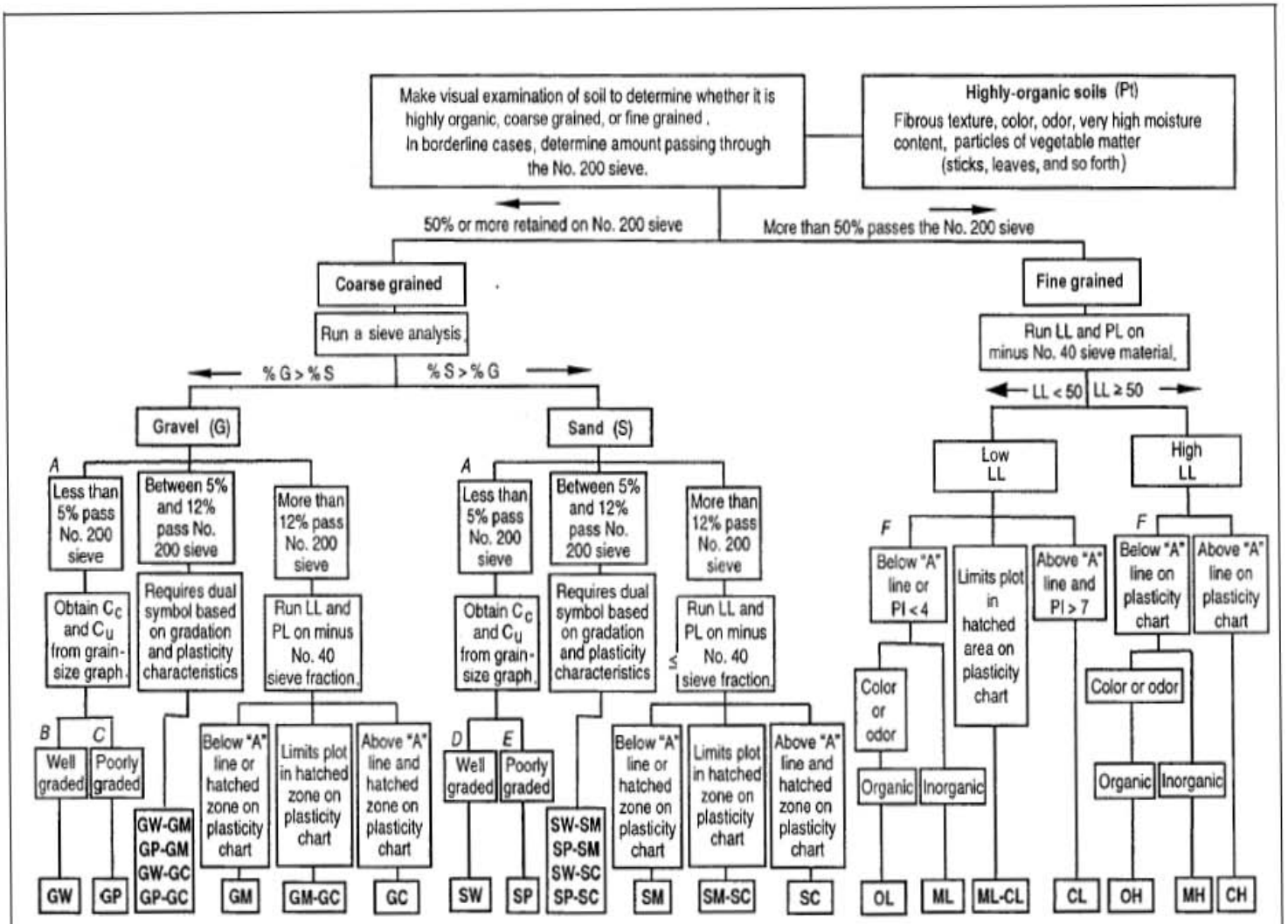
Table C-1 illustrates the process of the classification system.

Table C-1

Soil Particle Size Ranges	
Component	**Size Range**
Cobbles	Above 3 inch
Gravel	3 inch to No. 4 sieve
Coarse	3 inch to ¾ inch
Fine	¾ inch to No. 4 sieve
Sand	No. 4 to No. 200 Sieves
Coarse	No. 4 to No. 10 Sieves
Medium	No. 10 to No. 40 Sieves
Fine	No. 40 to No. 200 Sieves
Fines (clay or silt)	Below No. 200 sieve (no minimum size)

The procedures are designed to apply generally to the identification of soils regardless of the intended engineering uses. Table C-1 also assists in identifying the symbols and soil descriptions within this system. It shows the schematic method of classifying soils from the results of laboratory tests. Columns 1 through 5 of Table C-3, identify the three major divisions of the classification system and the group symbols that distinguish the individual soil types. Names of typical and representative soil types found in each group are shown in column 6.

Table C-2



A If fines interfere with free-draining properties, use a double symbol such as GW-GM.
B For well-graded gravel, the C_u must be > 4 and the C_c must be ≥ 1 and ≤ 3.
C For poorly graded gravel, the C_u must be ≤ 4 and/or the C_c is < 1 or > 3.
D For well graded sand, the C_u must be > 6 and the C_c must be ≥ 1 and ≤ 3.
E For poorly graded sand, the C_u must be ≤ 6 and/or the C_c is < 1 or > 3.
F In cases where organic material can't be determined by color or odor, a LL and PL test must be conducted on a sample of natural moisture content and a sample that has been oven-dried. Organic soils will show a radical drop in plasticity for the oven-dried sample compared to the retained-moisture sample. Inorganic soils generally fall within ± 1 or 2 percent of each other.

SOIL GROUPS AND GROUP SYMBOLS

Soils are primarily identified as coarse grained, fine grained, and highly organic. Textures are also grouped. Coarse-grained soils have 50 percent or more by weight of the overall soil sample retained on the No. 200 sieve. Fine-grained soils have more than 50 percent by weight passing the No. 200 sieve. Highly-organic soils are generally easy to identify by visual examination. The coarse-grained soils are subdivided into gravel and gravelly soils (G) and sands and sandy soils (S). Fine-grained soils are subdivided on the basis of their LL and plasticity properties. The symbol L is used for soils with LLs of 50 and less and the symbol H for soils with LLs in excess of 50. Peat and other highly organic soils are designated by the symbol Pt and are not subdivided.

There is generally no clear-cut boundary between gravelly soils and sandy soils and the exact point of division is relatively unimportant. Coarse-grained soils are classified as G for identification purposes if the greater percentage of the coarse fraction (that which is retained on the No. 200 sieve) is larger than the No. 4 sieve. They are classed as S if the greater portion of the coarse fraction is finer than the No. 4 sieve. Borderline cases may be classified as belonging to both groups. The G and S groups are each divided into four secondary groups:

1. Well-graded material with little or no fines – symbol W, groups GW and SW.
2. Poorly graded material with little or no fines – symbol P, groups GP and SP.
3. Coarse material with nonplastic fines or fines with low plasticity – symbol M, groups GM and SM.
4. Coarse material with plastic fines – symbol C, groups GC and SC.

The fine-grained soils are subdivided into groups based on whether they have a relatively low (L) or high (H) LL. These two groups are further subdivided as follows:

1. Inorganic silts and very fine sandy soils, silty or clayey fine sands, micaceous and diatomaceous soils, and elastic silts – symbol M, groups ML and MH.
2. Inorganic clays – symbol C, groups CL and CH.
3. Organic silts and clays – symbol O, groups OL and OH.

Coarse-Grained Soils

Soils of the GW, GP, SW, and SP groups are defined as having less than 5 percent passing the No. 200 sieve in the following paragraphs. Soils that have between 5 and 12 percent passing the No. 200 sieve are classed as borderline and will be discussed later in this appendix.

GW and SW Groups

These groups comprise well-graded gravelly and sandy soils having little or no nonplastic fines (less than 5 percent passing the No. 200 sieve). The presence of the fines must not noticeably change the strength characteristics of the coarse-grained fraction and must not interfere with its free-draining characteristics. Evaluate and discuss under “Laboratory Identification” if the material contains less than 5 percent

fines that exhibit plasticity. The material should not contain more than 3 percent of soil grains smaller than 0.02 millimeter in size in areas subject to frost action.

GP and SP Groups

Poorly-graded gravels and sands containing little or no non-plastic fines (less than 5 percent passing the No. 200 sieve) are classed in the GP and SP groups. The materials may be classed as uniform gravels, uniform sands, or nonuniform mixtures of very coarse material and very fine sand with intermediate sizes lacking (sometimes called skip graded, gap graded, or step graded). The latter group often results from borrow excavation in which gravel and sand layers are mixed. This information should be evaluated and the soil classified as discussed subsequently under “Laboratory Identification” if the fine fraction exhibits plasticity.

GM and SM Groups

The GM and SM groups comprise gravels or sands with fines (more than 12 percent passing the No. 200 sieve) with low or no plasticity. The PI and LL of soils in the group should plot below the “A” line on the plasticity chart. The gradation of the materials is not considered significant and both well and poorly graded materials are included. Some of the sands and gravels in this group will have a binder composed of natural cementing agents proportioned so the mixture shows negligible swelling or shrinkage. The dry strength of such materials is provided by a small amount of soil binder or by cementation of calcareous material or iron oxide. The fine fraction of other materials in the GM and SM groups may be composed of silts or rock-flour types having little or no plasticity and the mixture will exhibit no dry strength.

GC and SC Groups

The GC and SC groups comprise gravelly or sandy soils with fines (more than 12 percent passing the No. 200 sieve) with either low or high plasticity. The PI and LL of soils in the group should plot above the “A” line on the plasticity chart. The gradation of the materials is not considered significant and both well and poorly graded materials are included. The plasticity of the binder fraction has more influence on the behavior of the soils than does variation in gradation. The fine fraction is generally composed of clays.

Table C-3

Characteristics of Soil Groups Pertaining to Embankments and Foundations

Major Divisions (1)	(2)	Letter (3)	Symbols Hatching (4)	Symbols Color (5)	Name (6)	Value for Embankments (7)	Permeability cm per sec (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP			Poorly graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM		Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC			Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	Sand and Sandy Soils	SW		Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP			Poorly graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM		Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC			Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core or flood-control structures	$k = 10^{-6}$ to 10^{-8}
Fine-Grained Soils	Silts and Clays LL < 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL			Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	Silts and Clays LL ≥ 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic-fill dam, not desirable in rolled-fill construction	$k = 10^{-4}$ to 10^{-6}
		CH			Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH			Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Pt		Orange	Peat and other highly organic soils	Not used for construction	

NOTES: 1. Values in columns 7 and 11 are for guidance only. Design should be based on actual test results.
2. The equipment listed in column 9 will usually produce the desired densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.
3. The range of dry unit weights listed in column 10 are for compacted soil at OMC when using the Standard Proctor Test (ASTM 1557-91).

Table C-3 Continued

Compaction Characteristics (9)	Max Dry Unit Weight Std Proctor (pcf) (10)	Value for Foundations (11)	Requirements for Seepage Control (12)
Good; tractor, rubber-tired, or steel-wheeled roller	125 -135	Good bearing value	Positive cutoff
Good; tractor, rubber-tired, or steel-wheeled roller	115 -125	Good bearing value	Positive cutoff
Good; with close control; rubber-tired or sheepsfoot roller	120 -135	Good bearing value	Toe trench to none
Fair; rubber-tired or sheepsfoot roller	115 -130	Good bearing value	None
Good; tractor	110 -130	Good bearing value	Upstream blanket and toe drainage or wells
Good; tractor	100 -120	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
Good with close control; rubber-tired or sheepsfoot roller	110 -125	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
Fair; sheepsfoot or rubber-tired roller	105 -125	Good to poor bearing value	None
Good to poor; close control essential; rubber-tired or sheepsfoot roller	95 -120	Very poor, susceptible to liquefaction	Toe trench to none
Fair to poor; sheepsfoot or rubber-tired roller	95 -120	Good to poor bearing value	None
Fair to poor; sheepsfoot roller	80 -100	Fair to poor bearing value, may have excessive settlements	None
Poor to very poor; sheepsfoot roller	70 - 95	Poor bearing value	None
Fair to poor; sheepsfoot roller	75 -105	Fair to poor bearing value	None
Poor to very poor; sheepsfoot roller	65 - 100	Very poor bearing value	None
Compaction not practical		Remove from foundations	

Fine Grained Soils

The following paragraphs discuss fine-grained soils in their sub-groupings:

ML and MH Groups

The symbol M has been used to designate predominantly silty materials and micaceous or diatomaceous soils in these groups. The symbols L and H represent low and high LLs, respectively, and an arbitrary dividing line between the two is set at an LL of 50. The soils in the ML and MH groups are sandy silts, clayey silts, or inorganic silts with relatively low plasticity. Also included are loess-type soils and rock flours. Micaceous and diatomaceous soils generally fall within the MH group but may extend into the ML

group when their LL is less than 50. The same is true for certain types of kaolin clays and some elite clays having relatively low plasticity.

CL and CH Groups

The symbol C stands for clay, with L and H denoting low or high LL in these groups. These soils are primarily inorganic clays. Low-plasticity clays are classified as CL and are usually lean, sandy, or silty clays. The medium and high plasticity clays are classified as CH. These include the fat clays, gumbo clays, certain volcanic clays, and bentonite. The glacial clays of the northern US cover a wide band in the CL and CH groups.

OL and OH Groups

The soils in the OL and OH groups are characterized by the presence of organic matter, and represented with the symbol O. Organic silts and clays are classified in these groups. The materials have a plasticity range that corresponds with the ML and MH groups.

Highly Organic Soils

The highly organic soils usually are very compressible and have undesirable construction characteristics. They are classified into one group, designated by the symbol Pt. Peat, humus, and swamp soils with a highly-organic texture are typical soils of the group. Particles of leaves, grass, branches, or other fibrous vegetable matter are common components of these soils.

IDENTIFICATION OF SOIL GROUPS

The USCS is arranged so that most soils can be classified into at least the three primary groups (coarse grained, fine grained, and highly organic) by means of visual examination and simple field tests. Classification into the subdivisions can also be made by visual examination with some degree of success. More positive identification may be made through laboratory testing. Depending on the purposes for which the soils in question are to be used in many instances a tentative classification determined in the field is of great benefit and may be all the identification that is necessary. The general or field-identification methods as well as the individual laboratory test methods are all explained in great detail in Chapter 2. It is emphasized that the two methods of identification are never entirely separated. Certain characteristics can only be estimated by visual examination. It may be necessary to verify the classification by laboratory tests in borderline cases. Conversely, the field methods are entirely practical for preliminary laboratory identification and may be used to an advantage in grouping soils in such a manner that only a minimum number of laboratory tests need be run.

LABORATORY IDENTIFICATION

Identifying soils in the laboratory is done by determining the gradation and plasticity characteristics of the materials. The gradation is determined by sieve analysis, and a grain-size curve is usually plotted as percent finer (or passing) by weight against a logarithmic scale of grain size in millimeters. DD Form 1207 is typically used for this purpose. Plasticity characteristics are evaluated by means of the LL and PL tests on the

soil fraction finer than the No. 40 sieve. The laboratory test procedures for the LL and PL determination can be found in Section IV of Chapter 2.

MAJOR SOIL GROUPS

The first step in identifying a soil is to determine whether it is coarse grained, fine grained, or highly organic in the laboratory identification procedures shown in Table C-1. This may be done by visual examination in most cases. It may be necessary to screen a representative dry sample over a No. 200 sieve and determine the percentage passing in some borderline cases such as with very-fine sands or coarse silts. Fifty percent or less passing the No. 200 sieve identifies the soil as coarse grained and more than 50 percent identifies the soil as fine grained. The percentage limit of 50 has been selected arbitrarily for convenience in identification as it is obvious that a numerical difference of 1 or 2 in this percentage will make no significant change in the soil's behavior. The identification procedure is continued according to the proper headings in Table C-1 after the major group is established.

Coarse-Grained Soils

A complete sieve analysis must be run on coarse-grained soils and a gradation curve plotted on a grain-size chart. It may be desirable to supplement the sieve analysis with a hydrometer analysis to define the gradation curve for particle sizes smaller than the No. 200 sieve size for some soils containing a substantial amount of fines. Preliminary identification is made by determining the percentage of material in the gravel (above No. 4 sieve) and sand (No. 4 to No. 200 sieve) sizes. The material is classes as G if there is a greater percentage of gravel than sand. The material is classes as S if there is a greater percentage of sand than gravel. The distinction between these groups is purely arbitrary for convenience in following the system. The next step is to determine the amount of material passing the No. 200 sieve. Since the subgroups are the same for gravels and sands, they will be discussed jointly in the following paragraphs.

GW, SW, GP, and SP Groups

These groups comprise nonplastic soils having less than 5 percent passing the No. 200 sieve and in which the fine fraction does not interfere with the soil's free-draining properties. An examination is made of the shape of the grain-size curve if the above criteria are met. Materials that are well graded are classified as GW or SW. Poorly graded materials are classified as GP or SP.

A soil's gradation curve and curve data should meet the following qualifications to be classed as well graded:

- The grain-size distributions of well-graded materials generally plot as smooth and regular concave curves with no sizes lacking or no excess of material in any size range.
- The coefficient of uniformity (Cu) of well-graded gravels is greater than 4 and of well-graded sands is greater than 6. The Cu is determined by dividing the grain-size diameter passing at 60 percent by the grain-size diameter passing at 10 percent.

- The coefficient of curvature (Cc) must be between 1 and 3. The Cc is determined by the following formula:

$$\frac{(D_{30})^2}{D_{60} \times D_{10}} = \textit{between 1 and 3}$$

Where–
D30 = grain diameter at 30 percent passing
D60 = grain diameter at 60 percent passing
D10 = grain diameter at 10 percent passing

The Cc ensures that the grading curve will have a concave curvature within relatively narrow limits for a given D60 and D10 combination. All gradations not meeting the foregoing criteria are classed as poorly graded. Poorly graded soils (GP and SP) are those having nearly straight-line gradations, convex gradations, nearly vertical gradations, and "hump" gradations typical of skip-graded materials.

NOTE: In the preceding paragraph, soils of the GW, GP, SW, and SP groups were defined as having less than a 5 percent fraction passing the No. 200 sieve. Soils having between 5 and 12 percent passing the No. 200 sieve are classed as borderline and are discussed later. ***GM, SM, GC and SC Groups***

The soils in these groups are composed of those materials having more than a 12 percent fraction passing the No. 200 sieve. They may or may not exhibit plasticity. The LL and PL tests are required on the fraction finer than the No. 40 sieve for identification. The tests should be run on representative samples of moist material—not on air- or oven-dried soils. This precaution is desirable as drying affects the limits values to some extent, as will be explained further in the discussion of fine-grained soils. Materials in which the LL and PI plot below the "A" line on the plasticity chart are classed as GM or SM. Gravels and sands in which the LL and PI plot above the "A" line on the plasticity chart are classed as GC or SC. It is considered that in the identification of materials in these groups, the plasticity characteristics overshadow the gradation characteristics; therefore, no distinction is made between well- and poorly graded materials.

Borderline Soils
Coarse grained soils containing between 5 and 12 percent material passing the No. 200 sieve are classed as borderline and carry a dual symbol (for example, GW-GM). Coarse grained soils with less than 5 percent passing the No. 200 sieve but that are not free draining or wherein the fine fraction exhibits plasticity are also classed as borderline and are given a dual symbol.

Fine-Grained Soils
Once the identity of a fine grained soil has been established, further identification is accomplished principally by the LL and PL tests in conjunction with the plasticity chart. The plasticity chart is a plot of LL versus PI on which is imposed a diagonal line called the "A" line and a vertical line at a LL of 50. The "A" line is defined by the equation PI =

0.73 (LL-20). The “A” line above a liquid limit of about 29 represents an important empirical boundary between typical inorganic clays (CL and CH), which are generally located above the line and plastic soils containing organic colloids (OL and OH) or inorganic silty soils (ML and MH). The vertical line at an LL of 50 separates silts and clays of low LL (L) from those of high LL (H). There is considerable overlapping of the properties of the clayey and silty soil types in the low part of the chart below an LL of about 29 and in the range of PI from 4 to 7. The separation between CL and OL or ML soil types in this region is accomplished by a cross-hatched zone on the plasticity chart between 4 and 7 PI and above the “A” line. The CL soils in this region are those having a PI above 7 while OL or ML soils are those having a PI below 4. Soils plotting within the cross-hatched zone should be classed as borderline.

The various soil groups are shown in their respective positions on the plasticity chart. Experience has shown that compressibility is about proportional to the LL and that soils having the same LL possess about equal compressibility (assuming that other factors are essentially the same). On comparing the physical characteristics of soils having the same LL, you find that with increasing the PI, the cohesive characteristics increase and the permeability decreases. From plots of the results of limits tests on a number of samples from the same fine-grained deposit, it is found that for most soils these points lie on a straight line or in a narrow band that is almost parallel to the “A” line. The identification of the various groups of fine-grained soils is discussed in the following paragraphs.

ML, CL, and OL Groups

A soil having an LL of less than 50 falls into the low LL (L) group. A plot of the LL and PI on the plasticity chart will show whether the soil falls above or below the “A” line and cross-hatched zone. Soils plotting above the “A” line and cross-hatched zone are classed as CL and are usually typical inorganic clays. Soils plotting below the “A” line or cross-hatched zone are inorganic silts or very fine sandy silts (ML) or organic silts or organic silt-clays of low plasticity (OL). Since two groups fall below the “A” line or cross-hatched zone, further identification is necessary. The distinguishing factor between the ML and OL groups is the absence or presence of organic matter. This is usually identified by color and odor. A comparison may be made between the LL and PL of a moist sample and one that has been oven-dried. An organic soil will show a radical drop in plasticity after oven or air drying. An inorganic soil will generally show a change in the limits values of only 1 or 2 percent, which may be either an increase or a decrease. For the foregoing reasons, the classification should be based on the plot of limits values determined before drying. Soils containing organic matter generally have lower specific gravities and may have decidedly higher water contents than inorganic soils; therefore, these properties may be of assistance in identifying organic soils. In special cases, determining the organic content may be made by chemical methods, but the procedures just described are usually sufficient.

MH, CH, and OH Groups

Soils with an LL greater than 50 are classed in group H. The LL and PI values are plotted on the plasticity chart to identify the soils. The soil classifies as CH if the points fall above the “A” line. A determination is made as to whether or not organic material is present if they fall below the “A” line (as described in the preceding paragraph). Inorganic materials are classed as MH and organic materials are classed as OH.

Highly-Organic Soils

Little more can be said as to the laboratory identification of highly-organic soils (Pt) than has been identified in the field-identification procedures. These soils are usually identified readily on the basis of color, texture, and odor. Moisture determinations usually show a natural water content of several hundred percent, which is far in excess of that found for most soils. Specific gravities of the solids in these soils may be quite low. Some peaty soils can be remolded and tested for the LLs and PLs. Such materials usually have an LL of several hundred percent and fall well below the “A” line on the plasticity chart.

Borderline Classifications

It is inevitable in the use of the classification system that soils will be encountered that fall close to the boundaries established between the various groups. Boundary zones for the amount of material passing the No. 200 sieve and for the lower part of the plasticity chart have been incorporated as a part of the system, as discussed subsequently. The accepted rule in classifying borderline soils is to use a double symbol (for example, GWGM). It is possible, in rare instances, for a soil to fall into more than one borderline zone and, if appropriate symbols were used for each possible classification, the result should be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated and it is considered best to use only a double symbol in these cases, selecting the two that are believed most representative of the probable behavior of the soil. The symbols representing the poorer of the possible groupings should be used in cases of doubt.

Coarse-Grained Soils

The coarse-grained soils were classified in the GW, GP, SW, and SP groups if they contained less than 5 percent of material passing the No. 200 sieve in previous discussions. Similarly, soils were classified in the GM, GC, SM, and SC groups if they had more than 12 percent passing the No. 200 sieve. The range between 5 and 12 percent passing the No. 200 sieve is designated as borderline. Soils falling within it are assigned a double symbol depending on both the gradation characteristics of the coarse fraction and the plasticity characteristics of the minus No. 40 sieve fraction. A well-graded sandy soil with 8 percent passing the No. 200 sieve, a LL of 28, and a PI of 9 would be designated as SM-SC. Another type of borderline classification occurs for those soils containing appreciable amounts of fines (groups GM, GC, SM, and SC) and whose LL and PL values plot in the lower portion of the plasticity chart. The method of classifying these soils is the same as for fine grained soils plotting in the same region, as presented in the following paragraph.

Fine-Grained Soils

Discussion has been presented of a zone on the plasticity chart below a LL of about 29 and ranging between PI values of 4 and 7. Several soil types exhibiting low plasticity plot in this general region on the plasticity chart and no definite boundary between silty and clayey soils exists. Use a double symbol (such as ML-CL) if a finegrained soil, groups CL and ML, or the minus No. 40 sieve fraction of a coarsegrained soil (groups GM, GC, SM, and SC) plots within the cross-hatched zone on the plasticity chart. Note that in the descriptive name of the soil type as indicated on Table C-3 silty and clayey may be used to describe silt or clay soils. It is considered advisable to discuss the connotations used in this system since the definitions of these terms are now somewhat different from those used by many soils engineers. The terms silt and clay are used to describe those soils with LLs and PLs plotting respectively below and above the “A” line and cross-hatched zone on the plasticity chart in the USCS. The terms silty and clayey may be used as adjectives in the soil names when the limits values plot close to the “A” line as a logical extension of this concept. For example, a clay soil with an LL of 40 and a PI of 16 may be called a silty clay. The adjective silty is not applied to clay soils having an LL in excess of about 60.

Expansion of Classification

It may be necessary to expand the USCS by subdividing existing groups to classify soils for a particular use. The indiscriminate use of subdivisions is discouraged and careful study should be given to any soil group before adopting such a step. All subdivisions should be designated preferably by a suffix to an existing group symbol. The suffix should be selected carefully so there will be no confusion with existing letters that already have meanings in the classification system. The basis and criteria for the subdivision in each case where an existing group is subdivided should be explained so that anyone unfamiliar with it may understand the subdivision properly.

Descriptive Soil Classification

At many stages in the soils investigation of a project—from the preliminary boring log to the final report—the engineer finds it convenient to give the soils he is working with a name rather than an impersonal classification symbol (such as GC). This results primarily from the fact that he is accustomed to talking in terms of gravels, sands, silts, and clays and finds it only logical to use these same names in presenting the data. The soil names have been associated with certain grain sizes in the textural classification as shown on the grain-size chart. Such a division is generally feasible for the coarse grained soils. The use of such terms as silt and clay may be entirely misleading on a textural basis. The terms silt and clay have been defined on a plasticity basis, as discussed previously. The use of a name classification based on texture within a given region of the country is often feasible since the general behavior of similar soils is consistent over the area. The same classification may be entirely inadequate in another area. The descriptive classification, if used intelligently, has a rightful place in soil mechanics but its use should be carefully evaluated by all concerned.

Description From Classification Sheet

Column 6 of Table C-3 lists typical names given to the soil types usually found within the various classification groups. By following either the field or laboratory investigation procedure and determining the proper classification group in which the soil belongs, it is usually an easy matter to select an appropriate name from the classification sheet. Some soils may be readily identified and properly named by only visual inspection. A word of caution is considered appropriate on the use of the classification system for certain soils (such as marls, calyces, coral, and shale) where the grain size can vary widely depending on the amount of mechanical breakdown of soil particles. The group symbol and textural name for these soils have little significance and the locally used name may be important.

Other Descriptive Terms

Records of field explorations in the form of boring logs can be of great benefit to the engineer if they include adequate information. The general characteristics of the soils as to plasticity, strength, moisture, and so forth provide information essential to a proper analysis of a particular problem in addition to the group symbol and the name of the soil. Locally accepted soil names should also be used to clarify the data to local bidders and to protect the government against later legal claims. The size of particles, mineralogical composition, shape of grains, and character of the binder are relevant features for coarse grained soils. Strength, moisture, and plasticity characteristics are important for fine-grained soils. Characteristics such as stratification, structure, consistency in the undisturbed and remolded states, cementation, and drainage are pertinent to the descriptive classification when describing undisturbed soils, such. Pertinent items used in describing soils are shown Table C-3. Use the Terzaghi classification based on unconfined compressive strength as a tentative standard to achieve uniformity in estimating the consistency of soils. Several examples of descriptive classifications are:

- SP Uniform, fine, clean sand with rounded grains
- SM Well-graded gravelly silty sand, angular chert gravel, ½ inch maximum size, silty binder with low plasticity, well-compacted and moist
- ML Light brown, fine, sandy silt. Very low plasticity. Saturated and soft in the undisturbed state
- CH Dark gray, fat clay. Stiff in the undisturbed state. Soft and sticky when remolded

CHARACTERISTICS OF SOIL GROUPS PERTAINING TO EMBANKMENTS AND FOUNDATIONS

The major properties of a soil proposed for use in an embankment or foundation that are of concern to the design or construction engineer are its strength, permeability, and consolidation and compaction characteristics. Other features may be investigated for a specific problem but in general some or all of the properties mentioned are of primary importance in an earth embankment or foundation project of any magnitude. It is common practice to evaluate the properties of the soils in question by means of laboratory or field tests and to use the results of such tests as a basis for design and construction. The factors that influence strength, consolidation, and other characteristics are numerous and some of them are not completely understood. It is

impractical to evaluate these features by means of a general soils classification. The soil groups in a given classification do have reasonably similar behavior characteristics. This information is not sufficient for design purposes but it will give the engineer an indication of the soil behavior when used as a component in construction. This is especially true in the preliminary examination for a project when neither time nor money for a detailed soils testing program is available. Keep in mind that only generalized characteristics of the soil groups are included and they should be used primarily as a guide and not as the complete answer to a problem. For example, it is possible to design and construct an earth embankment of almost any type of soil and on practically any foundation. Certain soils may be better-suited to the job than others when a choice of materials is possible. It is on this basis that the behavior characteristics of soils are presented in the following paragraphs and on the classification sheet. A structure's use is often the principal deciding factor in selecting soil types as well as the type of protective measures that will be used. It is impossible to cover all possible considerations in the brief description of pertinent soil characteristics contained in this appendix since each structure is a special problem within itself.

Features On The Soils-Classification Sheet

General characteristics of the soil groups pertinent to embankments and foundations are presented in Table C-3. Columns 1 through 5 show major soil divisions, group symbols, and the hatching and color symbols. The names of soil types are given in column 6. The basic features are the same as those presented previously in soils classification. The table also shows the suitability of the materials for use in embankments (strength and permeability characteristics), the minimum or range of permeability values to be expected for the soil groups, general compaction characteristics, the suitability of the soils for foundations (strength and consolidation), and the requirements for seepage control, especially when the soils are encountered in the foundation for earth embankments (permeability). Brief discussions of these features are presented in the following paragraphs.

Table C-4

Major Divisions (1)	(2)	Letter (3)	Symbols: Hatching (4)	Symbols: Color (5)	Name (6)	Value As Subgrade When not Subject to Frost Action (7)	Value As Subbase When not Subject to Frost Action (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent
		GP			Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good
		GM u				Good	Fair
		GC			Clayey gravels, gravel-sand-clay mixtures	Good	Fair
	Sand and Sandy Soils	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good
		SP			Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair
		SM d		Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good
		SM u				Fair	Poor to fair
		SC			Clayey sands, sand-silt mixtures	Poor to fair	Poor
Fine-Grained Soils	Silts and Clays LL < 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable
		CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable
		OL			Organic silts and organic silt-clays of low plasticity	Poor	Not suitable
	Silts and Clays LL ≥ 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable
		CH			Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable
		OH			Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable
Highly Organic Soils		Pt		Orange	Peat and other highly-organic soils	Not suitable	Not suitable

NOTES: 1. Divisions of the GM and SM groups (column 3) into subdivisions of d and u are applicable to roads and airfields only. Subdivision is based on the LL and PI; suffix d (for example, GMd) will be used when the LL is 25 or less and the PI is 5 or less; the suffix u will be used otherwise.

Table C-4 Continued

Value As Base When not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Dry Unit Weight (pcf) (14)	Typical Design Values	
						CBR (15)	Subgrade Modulus k (lb per cu in) (16)
Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125 -140	40 - 80	300 - 500
Fair to Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110 -140	30 - 60	300 - 500
Fair to Good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125 -145	40 - 60	300 - 500
Poor to not suitable	Slight to medium	Slight	Poor to practi-cally impervious	Rubber-tired roller, sheepsfoot roller	115 -135	20 - 30	200 - 500
Poor to not suitable	Slight to medium	Slight	Poor to practi-cally impervious	Rubber-tired roller, sheepsfoot roller	130 -145	20 - 40	200 - 500
Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110 -130	20 - 40	200 - 400
Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	105 -135	10 - 40	150 - 400
Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120 -135	15 - 40	150 - 400
Not suitable	Slight to high	Slight to medium	Poor to practi-cally impervious	Rubber-tired roller, sheepsfoot roller	100 -130	10 - 20	100 - 300
Not suitable	Slight to high	Slight to medium	Poor to practi-cally impervious	Rubber-tired roller, sheepsfoot roller	100 -135	5 - 20	100 - 300
Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90 -130	15 or less	100 - 200
Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90 -130	15 or less	50 - 150
Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90 -105	5 or less	50 - 100
Not suitable	Medium to very high	High	Fair to poor	Rubber-tired roller, sheepsfoot roller	80 -105	10 or less	50 - 100
Not suitable	Medium	High	Practically impervious	Rubber-tired roller, sheepsfoot roller	90 -115	15 or less	50 - 150
Not suitable	Medium	High	Practically impervious	Rubber-tired roller, sheepsfoot roller	80 -110	5 or less	25 - 100
Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

NOTES (continued):

2. The equipment listed in column 13 will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.

a. Processed base materials and other angular material. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.

b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.

c. Equipment Size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:

- **Crawler-type tractor—total weight in excess of 30,000 pounds.**
- **Rubber-tired equipment—wheel load in excess of 15,000 pounds; wheel loads as high as 40,000 pounds may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).**
- **Sheepsfoot roller—unit pressure (on 6- to 12-square-inch foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the feet should be at least 5 percent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.**

3. The range of dry unit weights listed in column 14 are for compacted soil at OMC when using the Standard Proctor Test (ASTM 1557-91).

4. The maximum CBR values (column 15) that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

Table C-5

Terzaghi Classification	
Unconfined Compressive Strength (Tons/Sq/Ft)	**Consistency**
< 0.25	Very Soft
0.25 to 0.50	Soft
0.50 to 1.00	Medium
1.00 to 2.00	Stiff
2.00 to 4.00	Very Stiff
> 4.00	Hard

Suitability of Soils for Embankments

Three major factors that influence the suitability of soils for use in embankments are permeability, strength, and ease of compaction. The gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) are stable, pervious, and able to attain good compaction with crawler-type tractors and rubber-tired rollers. The poorly graded materials may not be quite as desirable as those that are well graded, but all of the materials are suitable for use in the pervious sections of earth embankments. Poorly graded sands (SP) may be more difficult to use and in general should have flatter embankment slopes than the SW soils. The gravels and sands with fines (groups GM, GC, SM, and SC) have variable characteristics depending on the nature of the fine fraction and the gradation of the entire sample. These materials are often sufficiently impervious and stable to be used for impervious sections of embankments. The soils in these groups should be carefully examined to ensure that they are properly zoned with relation to other materials in an embankment. The CL group is best adapted for embankment construction of the fine-grained soils. These soils are impervious, fairly stable, and give fair to good compaction with sheepsfoot or rubber-tired rollers. The MH soils, while not desirable for rolled-fill construction, may be used in the core of hydraulic-fill structures. Soils of the ML group may or may not have good compaction characteristics and, in general, must be closely controlled in the field to secure the desired strength. CH soils have fair stability when used on flat slopes but have detrimental shrinkage characteristics that may necessitate blanketing them or incorporating them in thin interior cores of embankments. Soils containing organic matter (groups OL, OH, and Pt) are not commonly used for embankment construction because of the detrimental effects of the organic matter present. Such materials may often be used to advantage in blankets and stability berms where strength is not important.

Permeability and Seepage Control

Since the permeability and requirements for seepage control are essentially functions of the same property of a soil, they will be discussed jointly. The subject of seepage in relation to embankments and foundations may be roughly divided into three categories:

1. Seepage through embankments
2. Seepage through foundations
3. Control of uplift pressures

These are discussed in relation to the soil groups in the following paragraphs.

Seepage Through Embankments

It is the relative permeability of adjacent materials rather than the actual permeability of such soils in the control of seepage through embankments that governs their use in a given location. An earth embankment is not watertight and the allowable quantity of seepage through it is largely governed by the use to which the structure is put. For example considerable seepage may be allowed in a flood-control project and the structure will still fulfill the storage requirements. Much less seepage is allowable for an irrigation project because pool levels must be maintained. The more impervious soils (GM, GC, SM, SC, CL, MH, and CH) may be used in core sections or in homogeneous embankments to retard the flow of water. It is important that seepage not emerge on the downstream slope or the possibility of drawdown exists on upstream slopes so more pervious materials are usually placed on the outer slopes. The coarse-grained, free-draining soils (GW, GP, SW, SP) are best-suited for this purpose. Material is usually graded from least pervious to more pervious from the center of the embankment outward where a variety of material is available. Use care in the arrangement of materials in the embankment to prevent piping within the section. The foregoing statements do not preclude the use of other arrangements of materials in embankments. Dams have been constructed successfully entirely of sand (SW, SP, and SM) or of silt (ML) with the section made large enough to reduce seepage to an allowable value without the use of an impervious core. Coarse grained soils are often used in drains and toe sections to collect seepage water in downstream sections of embankments. The soils used will depend largely on the material that they drain. Free draining sands (SW and SP) or gravels (GW and GP) are preferred but a silty sand (SM) may effectively drain a clay (CL and CH) and be entirely satisfactory.

Seepage Through Foundations

The use of the structure involved often determines the amount of seepage control necessary in foundations just as it does in the case of embankments. Cases could be cited where the flow of water through a pervious foundation would not constitute an excessive water loss and no seepage control measures would be necessary if adequate provisions were made against piping in critical areas.

Take necessary measures to use pervious soils if seepage control is desired. Free draining gravels (GW and GP) are capable of carrying considerable quantities of water and some positive control such as a cutoff trench may be necessary. Clean sands (SW and SP) may be controlled by a cutoff or by an upstream impervious blanket. A drainage trench at the downstream toe or a line of relief wells will not reduce the amount of seepage but either will serve to control seepage and route the flow into collector systems where it can be led away harmlessly. Slightly less pervious material such as silty gravels (GM), silty sands (SM), or silts (ML) may require a minor amount of seepage control such as a toe trench or if they are sufficiently impervious, no control may be necessary. The relatively impervious soils (GC, SC, CL, OL, MH, CH, and OH) usually pass such a small volume of water that seepage control measures are not necessary.

Control of Uplift Pressures

The problem of control of uplift pressures is directly associated with pervious foundation soils. Uplift pressures may be reduced by lengthening the path of seepage (by a cutoff or upstream blanket) or by measures for pressure relief in the form of wells, drainage trenches, drainage blankets, or pervious downstream shells. Free-draining gravels (GW and GP) may be treated by any of the aforementioned procedures; however, to obtain the desired pressure relief, the use of a positive cutoff may be preferred, as blanket, well, or trench installations would probably have to be too extensive for economical accomplishment of the desired results. Free-draining sands (SW and SP) are generally less permeable than the gravels and, consequently, the volume of water that must be controlled for pressure relief is usually less. Therefore a positive cutoff may not be required and an upstream blanket, wells, or a toe trench may be entirely effective. In some cases a combination of blanket and trench or wells may be desirable.

Silty soils (silty gravels [GM], silty sands [SM], and silts [ML]) usually do not require extensive treatment; a toe drainage trench or well system may be sufficient to reduce uplift pressures. The more impervious silty materials may not be permeable enough to permit dangerous uplift pressures to develop, and in such cases, no treatment is indicated. In general, the more impervious soils (GC, SC, CL, OL, MH, CH, and OH) require no treatment for control of uplift pressures. However, they do assume importance when they occur as a relatively thin top stratum over more pervious materials. In such cases, uplift pressures in the lower layers acting on the base of the impervious top stratum can cause heaving and formation of boils; treatment of the lower layer by some of the methods mentioned above is usually indicated in these cases. It is emphasized that control of uplift pressures should not be applied indiscriminately just because certain types of soils are encountered. Rather, the use of control measures should be based on a careful evaluation of conditions that do or can exist, and an economical solution should be reached that will accomplish the desired results.

Compaction Characteristics

Table C-3 shows the general compaction characteristics of the various soil groups. The evaluations given and the equipment listed are based on average field conditions where proper moisture control and thickness of lift are attained and a reasonable number of passes of the compaction equipment are required to secure the desired density. The sheepsfoot and rubber-tired rollers are commonly used pieces of equipment for lift construction of embankments. Some advantages may be claimed for the sheepsfoot roller in that it leaves a rough surface that affords better bond between lifts and it kneads the soil which allows better moisture distribution.

Rubber-tired equipment referred to in the table is considered to be heavily loaded compactors or earthmoving equipment with a minimum wheel load of 15,000 pounds. The thickness of a compacted lift is usually reduced to about 2 inches if ordinary wobble-wheel rollers are used for compaction. Granular soils with little or no fines generally show good compaction characteristics with the well-graded materials (GW and SW) usually furnishing better results than the poorly graded soils (GP and SP).

The sandy soils, in most cases, are best compacted by crawler-type tractors. Rubber-tired equipment and sometimes steel-wheel rollers are effective on gravelly materials. Coarse-grained soils with fines of low plasticity (groups GM and SM) show good compaction characteristics with either sheepsfoot rollers or rubber-tired equipment. The range of moisture contents for effective compaction may be very narrow and close moisture control is desirable. This is also true of the silty soils in the ML group. Soils of the ML group may be compacted with rubber-tired equipment or with sheepsfoot rollers. Gravels and sands with plastic fines (groups GC and SC) show fair compaction characteristics. This quality may vary somewhat with the character and amount of fines. Rubber-tired or sheepsfoot rollers may be used. Sheepsfoot rollers are generally used for compacting fine-grained soils. The compaction characteristics of such materials are variable. Lean clays and sandy clays (CL) are the best, fat clays and lean organic clays or silts (OL and CH) fair to poor, and organic or micaceous soils (MH and OH) usually poor.

It is highly desirable to investigate the compaction characteristics of the soil by means of a field test section for most construction projects of any magnitude. Column 10 shows the ranges of unit dry weight for soils compacted according to the compaction test method as described in ASTM 1557-91. It is emphasized that these values are for guidance only. Design or construction control should be based on laboratory test results.

Suitability of Soils for Foundations

Suitability of soils for foundations of embankments or structures depends primarily on the strength and consolidation characteristics of the subsoils. The type of structure and its use will largely govern the adaptability of a soil as a satisfactory foundation. Large settlements may be allowed and compensated for by overbuilding for embankments. The allowable settlement of structures such as control towers may be small to prevent overstressing the concrete or steel on which they are built or because of the necessity for adhering to established grades. A soil may be entirely satisfactory for one type of construction but may require special treatment for other types.

Strength and settlement characteristics of soils depend on a number of variables such as structure, in-place density, moisture content, and cycles of loading in their geologic history that are not readily evaluated by a classification system such as used here. Only very general statements can be made as to the suitability of the various soil types as foundations for these reasons. This is especially true for fine-grained soils.

Generally, gravels and gravelly soils (GW, GP, GM, and GC) have good bearing capacity and undergo little consolidation under load. Well-graded sands (SW) usually have a good bearing value. Poorly graded sands and silty sands (SP and SM) may exhibit variable bearing capacity depending on their density. This is true to some extent for all coarse-grained soils but is especially critical for uniformly graded soils of the SP and SM groups. Such soils, when saturated, may become “quick” and present an additional construction problem. Soils of the ML group may be subject to liquefaction and may have poor bearing capacities, particularly where heavy structure loads are

involved. The CL group is probably the best from a foundation standpoint of the fine-grained soils, but in some cases the soils may be soft and wet and exhibit poor bearing capacity and fairly large settlements under load. Soils of the MH groups and normally consolidated CH soils may show poor bearing capacity and large settlements. Organic soils (OL and OH) have poor bearing capacity and usually exhibit large settlement under load. The type of structure foundation selected is governed by such factors as the bearing capacity of the soil and the magnitude of the load for most of the fine-grained soils discussed above. It is possible that simple spread footings might be adequate to carry the load without excessive settlement in many cases. Use alternate methods if the soils are poor and structure loads are relatively heavy. Pile foundations may be necessary in some cases and in special instances, particularly in the case of some CH and OH soils. It may be desirable and economically feasible to remove such soils from the foundation. Highly organic soils are generally very poor foundation materials. These may be capable of carrying very light loads but, in general, are unsuited for most construction purposes. Highly organic soils in the foundation may be removed if limited in extent, they may be displaced by dumping firmer soils on top, or piling may be driven through them to a stronger layer. Proper treatment will depend on the structure involved.

Graphical Presentation Of Soils Data

It is customary to present the results of soils explorations on drawings or plans as schematic representations of the borings or test pits with the soils encountered using various symbols. Commonly used hatching symbols are small, irregular round symbols for gravel, dots for sand, vertical lines for silts, and diagonal lines for clays.

Combinations of these symbols represent the various combinations of materials found in the explorations. This system has been adapted to the various soil groups in the USCS and the appropriate symbols are shown in Table C-3. Hatching symbols may be omitted and the appropriate group letter symbol written in the boring log. The effective size of coarse grained soils and the natural water content of fine grained soils should be shown by the side of the log in addition to the symbols on boring logs. Other descriptive abbreviations may be used as deemed appropriate. The use of color to delineate soil types on maps and drawings is desirable in certain instances. A suggested color scheme to show the major soil groups is described in column 5 of Table C-3.

Characteristics Of Soil Groups Pertaining To Roads And Airfields

The properties desired in soils for foundations under roads and airfields and for base courses under flexible pavements are adequate strength, good compaction characteristics, adequate drainage, resistance to frost action in areas where frost is a factor, and acceptable compression and expansion characteristics. Some of these properties, if inadequate in the soils available, may be supplied by proper construction methods. For instance, materials having good drainage characteristics are desirable but if such materials are not available locally, adequate drainage may be obtained by installing a properly designed water-collecting system. Strength requirements for base course materials used immediately under the pavement of a flexible pavement structure are high and only good quality materials are acceptable. Low strengths in subgrade

materials may be compensated for in many cases by increasing the thickness of overlying concrete pavement or of base materials in flexible pavement construction. It may be seen that the proper design of roads and airfield pavements requires the evaluation of soil properties in more detail than is possible by using the general soils classification system. The grouping of soils in the classification system allows for a general indication of their behavior in road and airfield construction.

Features On The Soils-Classification Sheet

General characteristics of the soil groups pertinent to roads and airfields are presented in Table C-4. Columns 1 through 5 show major soil divisions, group symbols, hatching and color symbols. Column 6 gives names of soil types. Column 7 evaluates the performance (strength) of the soil groups when used as subgrade materials that will not be subject to frost action. Columns 8 and 9 make a similar evaluation for the soils when used as subbase and base materials. Column 10 shows potential frost action. Column 11 shows compressibility and expansion characteristics. Column 12 presents drainage characteristics. Column 13 shows types of compaction equipment that perform satisfactorily on the various soil groups. Column 14 shows ranges of unit dry weight for compacted soils. Column 15 gives ranges of typical CBR values. Column 16 gives ranges of modulus of subgrade reaction (k). The various features presented are discussed in the following paragraphs.

Subdivision of Coarse-Grained Soil Groups

Note that in column 3 the basic soil groups (GM and SM) have each been subdivided into two groups designated by the suffixes d and u which have been chosen to represent desirable and less desirable (undesirable) base materials, respectively. This subdivision applies to roads and airfields only and is based on field observation and laboratory tests on the behavior of the soils in these groups. Basis for the subdivision is the LL and PI of the fraction of the soil passing the No. 40 sieve. The suffix d is used when the LL is 25 or less and the PI is 5 or less; otherwise, the suffix u is used. Typical symbols for soils in these groups are GMd and SMu.

Values of Soils as Subgrade, Subbase, or Base Materials

The descriptions in columns 7 through 9 give a general indication of the suitability of the soil groups for use as subgrades, subbase, or base materials, provided they are not subject to frost action. In areas where frost heaving is a problem, the value of materials as subgrades or subbases will be reduced, depending on the potential frost action of the material as shown in column 10. Proper design procedures should be used in situations where this is a problem.

The coarse-grained soils, in general, are the best subgrade, subbase, and base materials. The GW group has excellent qualities as a subgrade and subbase and is good as base material. Note that “excellent” is not used for any of the soils for base courses. Excellent should be used in reference to a high-quality processed crushed stone. Poorly graded gravels and some silty gravels (groups GP and GMd) are usually only slightly less desirable as subgrade or subbase materials and under favorable conditions may be used as base materials for certain conditions. Poor gradation and

other factors sometimes reduce the value of such soils to the extent that they offer only moderate strength and their value as a base material is less. The GMu, GC, and SW groups are reasonably good subgrade materials but are generally poor to not suitable as bases. The SP and SMd soils are usually considered fair to good subgrade and subbase materials but in general are poor to not suitable for base materials. The SMu and SC soils are fair to poor subgrade and subbase materials and are not suitable for base materials. The finegrained soils range from fair to very poor subgrade materials as follows:

• Silts and lean clays	ML and CL	fair to poor
• Organic silts, lean organic clays, and micaceous or diatomaceous soils	OL and MH	poor
• Fat clays and fat organic clays	CH and OH	poor to very poor

These qualities are compensated for in flexible pavement design by increasing the thickness of overlying base material and in rigid pavement design by increasing the pavement thickness or by adding a base course layer. None of the fine grained soils are suitable as subbase or base materials. The fibrous organic soils (group Pt) are very poor subgrade materials and should be removed wherever possible; otherwise, special construction measures should be adopted. They are not suitable as subbase and base materials. The CBR values shown in column 15 give a relative indication of the strength of the various soil groups as used in flexible pavement design. Similarly, values of subgrade modulus (k) in column 16 are relative indications of strengths from plate-bearing tests as used in rigid pavement design. Actual test values should be used for this purpose instead of the approximate values shown in the tabulation as these tests are used for pavement design.

Sand-clay-gravel mixtures (GC) are generally considered the most satisfactory for wearing surfaces on unsurfaced roads. They should not contain too large a percentage of fines and the PI should be in the range of 5 to about 15.

Potential Frost Action

The relative effects of frost action on the various soil groups are shown in column 10. Two conditions must be present simultaneously before frost action will be a major consideration regardless of the frost susceptibility of the various soil groups. A source of water during the freezing period and a sufficient period for the freezing temperature to penetrate the ground. Water necessary for the formation of ice lenses may become available from a high groundwater table or a capillary supply, within the soil voids, or through infiltration. The degree of ice formation that will occur in any given case is influenced by environmental factors such as topographic position, stratification of the parent soil, transitions into cut sections, lateral flow of water from side cuts, localized pockets of perched groundwater, and drainage conditions. The silts and fine silty sands are generally the worst offenders as far as frost is concerned. Coarse-grained materials with little or no fines are affected only slightly if at all. Clays (CL and CH) are subject to frost action, but the loss of strength of such materials may not be as great as for silty soils. Inorganic soils containing less than three percent of grains finer than 0.02 millimeter in diameter by weight are generally not frost susceptible. Two acceptable

methods of design of pavements are available where frost-susceptible soils are encountered in subgrades and frost is a definite problem. Either a sufficient depth of acceptable granular material is placed over the soils to prevent freezing in the subgrade and thereby prevent the detrimental effects of frost action or a reduced depth of granular material is used, thereby allowing freezing in the subgrade and the design is based on the reduced strength of the subgrade during the frost-melting period. Appropriate drainage measures to prevent the accumulation of water in the soil pores will help to diminish ice segregation in the subgrade and subbase in many cases.

Compressibility and Expansion

Two types of soil characteristics are applicable to road and runway design. The first is the relatively long-term compression or consolidation under the dead weight of the structure. The second is the short-term compression and rebound under moving wheel loads. The long-term consolidation of soils becomes a factor in design primarily when heavy fills are made on compressible soils. It will have little influence on the pavement's load carrying capacity if adequate provision is made for this type of settlement during construction. Provide adequate protection when elastic soils subject to compression and rebound under wheel load are encountered as even small movements of this soil may be detrimental to the base and wearing course of pavements. It is fortunate that the free-draining, coarse-grained soils (GW, GP, SW, and SP), which in general make the best subgrade and subbase materials, exhibit almost no tendency toward high compressibility or expansion. The compressibility of soils increases with an increasing LL. The foregoing is not completely true, as compressibility is also influenced by soil structure, grain shape, previous loading history, and other factors that are not evaluated in the classification system. Undesirable compressibility or expansion characteristics may be reduced by distributing the load through a greater thickness of overlying material. This is adequately handled by the CBR method of design for flexible pavements. Rigid pavements may require the addition of an acceptable base course under the pavement.

Drainage Characteristics

The drainage characteristics of soils are a direct reflection of their permeability. The evaluation of drainage characteristics for use in roads and runways is shown in column 12. The presence of moisture in base, subbase, and subgrade materials, except for free-draining, coarse-grained soils may cause the development of pore water pressures and loss of strength. The moisture may come from infiltration of rainwater or by capillary rise from an underlying water table. Free-draining materials permit rapid draining of water but they also permit rapid ingress of water. These materials may serve as reservoirs to saturate the less-pervious materials if such materials are adjacent to less-pervious materials and have free access to water. It is obvious that in most instances adequate drainage systems should be provided. The gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) have excellent drainage characteristics. The GMd and SMd groups have fair-to-poor drainage characteristics, whereas the GMu, GC, SMu, and SC groups may be practically impervious. Soils of the ML, MH, and Pt groups have fair-to-poor drainage characteristics. All of the other groups have poor drainage characteristics or are practically impervious.

Compaction Equipment

The compaction of soils for roads and runways, especially for the latter, requires that a high degree of density be attained at the time of construction so that detrimental consolidation will not take place under traffic. The detrimental effects of water are lessened in cases where saturation or near saturation takes place. Processed materials, such as crushed rock, are often used as base course and such materials require special treatment in compaction. Types of compaction equipment that will usually produce the desired densities are shown in column 13. Several types of equipment are listed for some of the soil groups this is because variations in soil type within a given group may require the use of different equipment.

More than one type of equipment may be necessary to produce the desired densities in some cases. Steel-wheeled rollers are recommended for angular materials with limited amounts of fines, crawler-type tractors or rubber-tired rollers for gravels and sands, and sheepsfoot rollers for coarse-grained or fine-grained soils having some cohesive qualities. Rubber-tired rollers are also recommended for final compaction operations for most soils except those with a high LL. Suggested minimum weights of the various types of equipment are shown in note 2 of Table C-4. Column 14 shows ranges of unit dry weight for soils compacted according to the compaction test method as described in ASTM 1557-91. These values are included primarily for guidance. Base the design or control of construction on actual test results.

APPENDIX D
Utah New/Reconstruction HMA Pavement Design Example

Reconstruction Project Design

This project is being designed as a reconstruction for a section of Utah State Route 36 from Mills Junction to I-80, Tooele County, Utah, west of Salt Lake City near the Great Salt Lake. The old existing pavement will be removed and a new HMA structure constructed.

Design Life and Construction

The HMA pavement has a 20-year design life. The UTBC base will be constructed in August, the HMA in September, and the project opened to traffic in October 2016.

Construction Requirements

Assuming a good quality construction with stringent ride specifications, the pavement is expected to have an initial IRI of approximately 50 inch/mile.

Performance and Reliability Criteria

The performance criteria were selected using Table 9-7 as a guide for a primary highway. The pavement will have no more than 15 percent alligator cracking at 90 percent reliability level and no more than 0.75 inch total rutting (mean of inner and outer wheelpath) at a reliability level of 90 percent at the end of the 20-year design life. The smoothness is designed to maintain an IRI of less than 170 inch/mile at a reliability level of 90 percent. Transverse cracking is limited to 1267 ft/mile which corresponds to a 50-ft transverse crack spacing at 90 percent reliability. These criteria are all entered into the Performance Criteria window. Other distress types (e.g., top down cracking) shown are not used by UDOT for design.

Traffic

WIM and ATR data were obtained for sites that were representative of the project site and used to develop project specific traffic inputs as follows:

- Volume adjustment factors. The initial two-way average annual daily truck traffic (AADTT) on this highway is estimated to be 2,000 trucks (Classes 4 through 13) during the first year of its service. This value will be adjusted using direction and lane adjustment factors. Specific inputs are as follows:
    - Initial two-way AADTT: 2,000
    - Directional distribution: 50 percent trucks in each direction.
    - Lane distribution: 90 percent trucks in outer design lane (2 lanes in design direction).
    - Operational speed: 55 mph
- Monthly volume adjustments are typically truck volume differences

throughout the year. Monthly variations in truck traffic were measured and are included in the input screen shown in Table D-1.

Table D-1

Monthly Adjustment Factors for New HMA Design Example

Tabular Representation of Traffic Inputs

Volume Monthly Adjustment Factors Level 3: Default MAF

Month	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
January	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.7	0.7
February	0.8	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.7	0.7
March	0.9	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.8	0.7
April	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
May	1.2	1.1	1.2	1.1	1.2	1.1	1.1	1.0	1.3	1.4
June	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.3	1.3
July	1.2	1.1	1.2	1.1	1.1	1.1	1.1	1.1	1.3	1.3
August	1.2	1.1	1.1	1.1	1.1	1.0	1.1	1.0	1.2	1.2
September	1.1	1.1	1.1	1.0	1.1	1.0	1.0	1.0	1.1	1.1
October	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.1	1.0	1.0
November	0.9	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.9	0.9
December	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.7	0.7

- Hourly truck distribution: The hourly truck distribution is only used for concrete pavements and not for asphalt pavements.
- Vehicle class distribution is the percent of each vehicle class in the traffic stream is an important input. Vehicle class distribution for this project was measured at an ATR site near the project. Specific inputs are shown in Table D-2. This distribution is unusual in that the Class 5 single unit truck is the most common type of vehicle on this rural highway.
- Truck traffic growth is projected using a linear or compound model. Truck traffic has grown from 1 to over 10 percent on Utah highways over the years. A compound growth rate of 2.8 percent was determined for this project after plotting past truck growth over time.

Table D-2

Vehicle Class Distribution for New HMA Design Example

Distributions by Vehicle Class

Vehicle Class	AADTT Distribution (%) (Level 3)	Growth Factor	
		Rate (%)	Function
Class 4	6.9%	2.8%	Compound
Class 5	54.48%	2.8%	Compound
Class 6	6.41%	2.8%	Compound
Class 7	1.17%	2.8%	Compound
Class 8	12.36%	2.8%	Compound
Class 9	5.38%	2.8%	Compound
Class 10	2.95%	2.8%	Compound
Class 11	3.68%	2.8%	Compound
Class 12	1.59%	2.8%	Compound
Class 13	5.08%	2.8%	Compound

- Axle load distribution. The axle load distribution is the most important traffic input. Damage is caused by the heavy single, tandem, tridem, and quad axle loads. The distributions were obtained from WIM equipment that is representative of this highway. A portion of the tandem axle distribution is shown in Table D-3. The highest loads in these distributions appear to cause the majority of fatigue damage and permanent deformation to HMA pavement.

Table D-3

Portion of Tandem Axle Load Distribution for New HMA Design Example
(Note: numbers in cells are percent of tandem axle loads
for each month, truck class, and tandem axle load).

	Vehicle	Total	Tandem Axle Load, Lbs.									
Month	Class	Percent	32000	34000	36000	38000	40000	42000	44000	46000	48000	50000
January	4	100.02	2.78	2.5	2.83	2.36	3.2	3.96	2.5	5.18	3.77	5.84
January	5	0	0	0	0	0	0	0	0	0	0	0
January	6	100.02	3.52	2.57	2.78	2.04	2.53	2.16	1.69	1.9	1.56	1.74
January	7	100	2.7	1.98	2.43	1.97	2.78	3.04	3.09	4.68	4.08	5.96
January	8	100.01	5.68	3.73	3.98	2.69	3.3	3.62	2.61	3.47	2.37	2.63
January	9	100.01	3.38	2.71	3.6	2.8	3.5	3.6	2.96	3.63	2.84	3.65
January	10	100	7.32	4.26	3.56	2.37	2.52	2.54	1.95	2.27	1.62	2.11
January	11	100.04	4.58	4.98	6.27	4.49	5.6	5.24	5.69	5.07	4.36	4.18
January	12	99.99	3.25	3.09	5.79	5.47	7.84	7.1	5.02	5.96	3.9	3.92
January	13	100	3.36	2.89	4.71	3.98	5.01	5.11	3.82	4.72	3.89	4.92

- General traffic inputs. These consist of lateral truck/wheel wander and number of axles per truck, axle configuration, and wheel base.
  - Axles per truck: Mean number of axles per vehicle class (Table D-4).
  - Lateral truck/wheel wander – three inputs are required here:
    1. Mean wheel location or the distance from outer edge of truck wheel to lane marking (paint stripe: 18-inch standard used in calibration).
    2. Standard deviation of lateral truck wander: 10-inch standard used in calibration.
    3. Design lane width: This distance is paint stripe to paint stripe. This is 12 ft.

Table D-4

Number of Axles per Truck
for New HMA Design Example

Number of Axles per Truck				
Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.58	0.42	0	0
Class 5	2	0	0	0
Class 6	0.84	1	0	0
Class 7	0.49	0.91	0.27	0.26
Class 8	2.09	0.82	0.01	0
Class 9	1.3	1.84	0.005	0
Class 10	1.02	0.97	0.98	0.04
Class 11	3.52	0.28	0.32	0
Class 12	2.69	1.07	0.14	0.19
Class 13	2.19	1.28	0.89	0.04

- Axle configuration: Axle width, spacing, and pressure (Table D-5).
  - Actual axle width (edge to edge of tire) outside dimensions: 8.5-ft, typical
  - Dual tire spacing: 12-in (typical used in calibration). Wide tires can only be considered in the Special Traffic Analysis routine in the Design ME for HMA pavements.
  - Tire pressure: 120 psi, hot rolling pressure used in calibration.

Table D-5

Axle Configuration for New HMA Design Example

Axle Configuration

Traffic Wander	
Mean wheel location (in.)	18
Traffic wander standard deviation (in.)	10
Design lane width (ft)	12

Axle Configuration	
Average axle width (ft)	8.5
Dual tire spacing (in.)	12
Tire pressure (psi)	120

Average Axle Spacing	
Tandem axle spacing (in.)	51.6
Tridem axle spacing (in.)	49.2
Quad axle spacing (in.)	49.2

Wheelbase does not apply

Climate Inputs

The project site is in the vicinity of Lake Point, Utah, close to the southern shore of the Great Salt Lake. The latitude and longitude of this site is as follows, obtained from various sources such as GPS units or Google Earth:

- Latitude: 40.6562 degrees (decimal)
- Longitude: -112.2891 degrees (decimal)

The designer enters the latitude and longitude and elevation into the Design ME and uses the Interpolate Climate Data for Given Location button. The estimated depth of water table, in this case 25 ft, must also be entered before generating a climatic file for the project (Table A-6). The Utah Division of Water Rights has a well drilling database and geologic well logs available at http://www.waterrights.utah.gov/wellinfo/default.asp that provide information on water depth or elevation observed in wells throughout the State. This resource, along with project geotechnical reports or the USDA-NRCS soil survey database can also be used for estimating depth of water table for a pavement project site.

The closest weather station for this example is Salt Lake City International Airport, 19 miles away and the next closest is Ogden at 36 miles. Since this weather station is so close to the project it will not be combined with other weather stations to create a virtual station. If there is any question, the designer can simply run the design with different combinations of weather stations and observe the results.

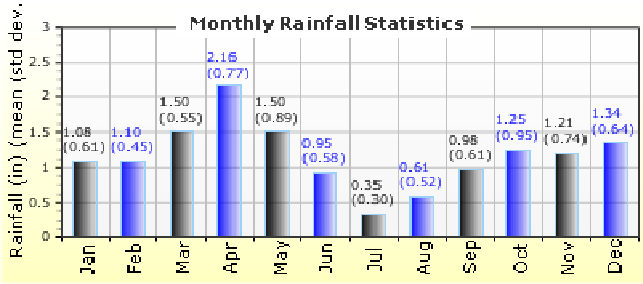
Table D-6

Climate Inputs for New HMA Design Example

Climate Inputs

Climate Data Sources:

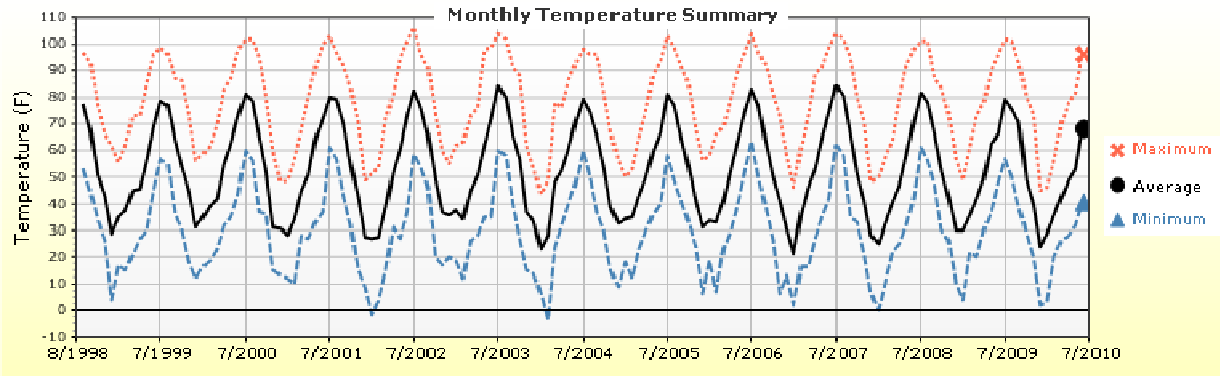
Climate Station Cities:	Location (lat lon elevation(ft))
SALT LAKE CITY, UT	40.78700 -111.96800 4220

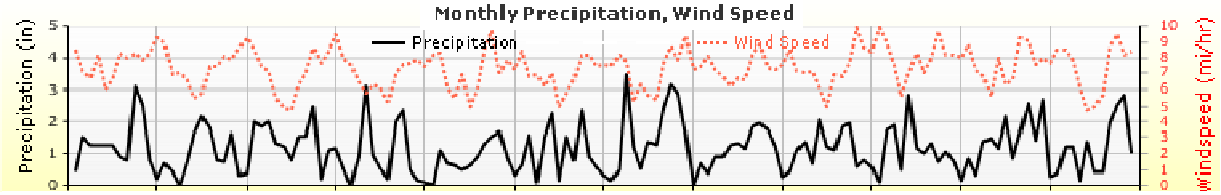


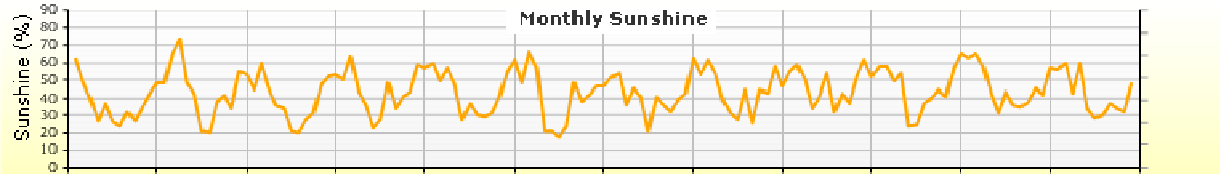
Annual Statistics:

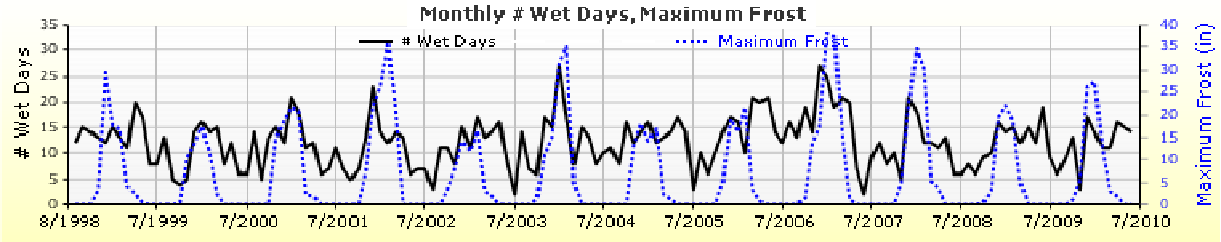
Mean annual air temperature (ºF)	53.47
Mean annual precipitation (in.)	14.00
Freezing index (ºF - days)	382.68
Average annual number of freeze/thaw cycles:	75.98
Water table depth(ft)	20.00

Monthly Climate Summary:









HMA Design Properties

The HMA dynamic modulus (E*) can be predicted by two different methods as shown on the input screen shown as Table D-7. It is recommended to always use the NCHRP 1-37A Viscosity based model which was nationally calibrated. Always select True for the NCHRP 1-37A HMA Rutting Model Coefficients button. The Interface Friction values should always be 1.00 to indicate no slippage between layers.

Table D-7

Illustration of HMA Design Inputs

Design Properties

HMA Design Properties

Use Multilayer Rutting Model	False
Using G* based model (not nationally calibrated)	False
Is NCHRP 1-37A HMA Rutting Model Coefficients	True
Endurance Limit	-
Use Reflective Cracking	True

Structure - ICM Properties	
AC surface shortwave absorptivity	0.85

Layer Name	**Layer Type**	**Interface Friction**
Layer 1 Flexible : Default asphalt concrete	Flexible (1)	1.00
Layer 2 Flexible : Default asphalt concrete	Flexible (1)	1.00
Layer 3 Non-stabilized Base : A-1-a	Non-stabilized Base (4)	1.00
Layer 4 Non-stabilized Base : A-1-a	Non-stabilized Base (4)	1.00
Layer 5 Subgrade : A-4	Subgrade (5)	-

Local Calibration Factors

The local calibration in Utah established that the flexible pavement rutting model was biased or over predicted actual rutting in the field most of the time so new State calibration coefficients were established. These new values are entered into the ME Design Explorer window in the upper left hand of the main screen. Note that there are two ways to enter a new calibration coefficient.

1. Current Project Specific Calibration Factors: Incorporating the new Utah rutting local calibration factors here will only work for the current project.
2. ME Design Calibration Factors (Recommended): Incorporating the new Utah rutting local calibration factors here will work for all future projects. Be sure to add them for both New Flexible design and Rehabilitation Flexible overlay design.

The recommended Utah local calibration factors are shown in Table D-8. This information is output in the detailed Excel file.

- New Flexible Pavement
  - AC Rutting
    - BR1 = 0.58 instead of 1.0
    - Standard Deviation = 0.0694*Pow(RUT,0.2439)+0.001
  - Base (UTBC) Rutting
    - Granular “Subgrade” Rutting BS1 = 0.71 instead of 1.00

        - Standard Deviation = 0.1439*Pow(BASERUT,0.6711)+0.001
          (Note: these are actually the base course (UTBC) only.)
    - Subgrade Rutting
        - Fine Subgrade Rutting BS1 = 0.28 instead of 1.00
        - Standard Deviation = 0.0531*Pow(SUBRUT,0.5012)+0.001
        - (Note: these are actually the subgrade (coarse or fine grained) only

Be sure to verify that the local calibration factors are used for each run by checking the ME Design output PDF file or the Excel file (Calibration tab).

Table D-8
Illustration of ME Design Calibration Factors
Note: Utah specific local calibration factors are included for the rutting model AC Br1=0.58, Granular base/subgrade Bs1=0.71, Fine grain subgrade=0.28 plus the standard deviations of these models.

Calibration Coefficients

AC Fatigue

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_\perp}\right)^{k_2 \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 \beta_{f3}}$$

$$C = 10^M$$

$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

k1: 0.007566
k2: 3.9492
k3: 1.281
Bf1: 1
Bf2: 1
Bf3: 1

AC Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$

$$C_1 = -0.1039 * H_a^2 + 2.4868 * H_a - 17.342$$

$$C_2 = 0.0172 * H_a^2 - 1.7331 * H_a + 27.428$$

Where:

H_{ac} = *total AC thickness(in)*

ε_p = *plastic strain* (in/in)
ε_r = *resilient strain* (in/in)
T = *layer temperature* $(^\circ F)$
N = *number of load repetitions*

AC Rutting Standard Deviation	0.0694*Pow(RUT,0.2439)+0.001	
AC Layer	K1:-3.35412 K2:1.5606 K3:0.4791	Br1:0.58 Br2:1 Br3:1

Thermal Fracture

$$C_f = 400 * N\left(\frac{\log C / h_{ac}}{\sigma}\right)$$

$$\Delta C = (k * \beta t)^{n+1} * A * \Delta K^n$$

$$A = 10^{(4.389 - 2.52 * \log(E * \sigma_m * n))}$$

C_f = *observed amount of thermal cracking(ft/500ft)*
k = *refression coefficient determined through field calibration*
$N()$ = *standard normal distribution evaluated at()*
σ = *standard deviation of the log of the depth of cracks in the pavments*
C = *crack depth(in)*
h_{ac} = *thickness of asphalt layer(in)*
ΔC = *Change in the crack depth due to a cooling cycle*
ΔK = *Change in the stress intensity factor due to a cooling cycle*
A, n = *Fracture parameters for the asphalt mixture*
E = *mixture stiffness*
σ_M = *Undamaged mixture tensile strength*
β_t = *Calibration parameter*

Level 1 K: 1.5	Level 1 Standard Deviation: 0.1468 * THERMAL + 65.027
Level 2 K: 0.5	Level 2 Standard Deviation: 0.2841 *THERMAL + 55.462
Level 3 K: 1.5	Level 3 Standard Deviation: 0.3972 * THERMAL + 20.422

CSM Fatigue

$$N_f = 10^{\left(\frac{k_1 \beta_{C1} \left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{C2}}\right)}$$

N_f = *number of repetitions to fatigue cracking*
σ_s = *Tensile stress(psi)*
M_r = *modulus of rupture(psi)*

k1: 1	k2: 1	Bc1: 1	Bc2:1

Subgrade Rutting

$$\delta_a(N) - \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^\beta} \right|$$

δ_a = *permanent deformation for the layer*
N = *number of repetitions*
ε_v = *average veritcal strain(in/in)*
$\varepsilon_0, \beta, \rho$ = *material properties*
ε_r = *resilient strain(in/in)*

Granular		**Fine**	
k1: 2.03	Bs1: 0.71	k1: 1.35	Bs1: 0.28
Standard Deviation (BASERUT) 0.1439*Pow(BASERUT,0.6711)+0.001		Standard Deviation (BASERUT) 0.0531*Pow(SUBRUT,0.5012)+0.001	

AC Cracking

AC Top Down Cracking

$$FC_{top} = \left(\frac{C_4}{1 + e^{(C_1 - C_2 * \log_{10}(Damage))}}\right) * 10.56$$

c1: 7	c2: 3.5	c3: 0	c4: 1000
AC Cracking Top Standard Deviation			
200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))			

AC Bottom Up Cracking

$$FC = \left(\frac{6000}{1 + e^{(C_1 + C_1' + C_2 + C_2' * \log_{10}(D + 100))}}\right) * \left(\frac{1}{60}\right)$$

$$C_2' = -2.40874 - 39.748 * (1 + h_{ac})^{-2.856}$$

$$C_1' = -2 * C_2'$$

c1: 1	c2: 1	c3: 6000
AC Cracking Bottom Standard Deviation		
1.13+13/(1+exp(7.57-15.5*LOG10(BOTTOM+0.0001)))		

CSM Cracking

$$FC_{ctb} = C_1 + \frac{C_2}{1 + e^{C_3 - C_4(Damage)}}$$

C1: 1	C2: 1	C3: 0	C4: 1000
CSM Standard Deviation			
CTB*1			

IRI Flexible Pavements

C1 - Rutting
C2 - Fatigue Crack
C3 - Transverse Crack
C4 - Site Factors

C1: 40	C2: 0.4	C3: 0.008	C4: 0.015

Structure and Layer Materials Inputs

A set of structural and materials inputs are now selected forming an initial design that will be evaluated for its performance. The procedure can be conducted in two ways:

1. Iterative procedure where the user develops a trial design and makes several modifications to it before a feasible and economic or final design is achieved. The trial design can be obtained using another design procedure such as the AASHTO 1993 or an alternative of interest.
2. The Optimization routine in ME Design is used to determine the layer thicknesses that meet all of the design criteria. Table D-9 shows the results for an optimized design.

The thickness of only the HMA layer was varied from 5.5 to 7.5 inches to obtain a design that passed all of the distress and IRI criteria for this example. Note that the HMA can be divided into several unique layers for detailed design as desired.

- 1.5 inch SMA layer
- 5.5 to 7.5 inch HMA layer: total thickness of various sub-layers
- 4 inch UTBC unbound granular base course, A-1-a
- 12 inch GB unbound granular borrow, A-1-a
- Semi-infinite uncompacted natural subgrade layer A-4 soil

Table D-9

Design Structure Output for New HMA Design Example

Design Inputs

Design Life:	20 years	Base construction:	August, 2016	Climate Data Sources	40.787, -111.968
Design Type:	Flexible Pavement	Pavement construction:	September, 2016		
		Traffic opening:	October, 2016		

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	Default asphalt concrete	1.5
Flexible	Default asphalt concrete	6.5
NonStabilized	A-1-a	4.0
NonStabilized	A-1-a	12.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	14.5
Air voids (%)	6.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2016 (initial)	2,000
2026 (10 years)	3,735,140
2036 (20 years)	8,658,240

Surface Shortwave Absorptivity HMA Surface

Use the calibration standard of 0.85. This controls the flow of heat through the HMA. This value has been found to provide accurate temperature measurements through the HMA after it ages and the color turns gray.

Layer Materials Properties

SMA (Surface Layer)

Inputs required for the SMA layer is shown in the appropriate ME Design input screens as shown in Table D-10. Note that the effective binder content is specifically for as-built volumetric conditions (not lab) thus much higher than the gravimetric percentage normally used in practice. The air voids are in situ or as-built and are higher than the mix design air voids.

Table D-10
SMA Layer Gradation, Binder Type Selection, and Mix Volumetric and Temperature Properties for New HMA Design Example

Layer 1 Flexible : Default asphalt concrete

Asphalt		
Thickness (in.)	1.5	
Unit weight (pcf)	150.0	
Poisson's ratio	Is Calculated?	True
	Ratio	-
	Parameter A	-1.63
	Parameter B	3.84E-06

Asphalt Dynamic Modulus (Input Level: 3)

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	77
No.4 sieve	60
No.200 sieve	6

Asphalt Binder

Parameter	Value
Grade	Superpave Performance Grade
Binder Type	70-28
A	9.715
VTS	-3.217

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	14.5
Air voids (%)	6.5
Thermal conductivity (BTU/hr-ft-ºF)	0.67
Heat capacity (BTU/lb-ºF)	0.23

Identifiers

Field	Value
Display name/identifier	Default asphalt concrete
Description of object	
Author	
Date Created	10/29/2010 11:00:00 PM
Approver	
Date approved	10/29/2010 11:00:00 PM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

HMA Layer

This is the layer whose thickness will be varied to meet the structural design requirements. Note that normally a thick HMA layer can be divided into several layers during the ME DESIGN analysis. Inputs for this layer are given in Table D-11.

D-11

HMA Layer Gradation, Binder Type Selection, and Mix Volumetric and Temperature Properties for New HMA Design Example

Layer 2 Flexible : Default asphalt concrete

Asphalt		
Thickness (in.)	6.5	
Unit weight (pcf)	150.0	
Poisson's ratio	Is Calculated?	True
	Ratio	-
	Parameter A	-1.63
	Parameter B	3.84E-06

Asphalt Dynamic Modulus (Input Level: 3)

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	77
No.4 sieve	60
No.200 sieve	6

Asphalt Binder

Parameter	Value
Grade	Superpave Performance Grade
Binder Type	70-28
A	9.715
VTS	-3.217

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	11.6
Air voids (%)	7
Thermal conductivity (BTU/hr-ft-ºF)	0.67
Heat capacity (BTU/lb-ºF)	0.23

Identifiers

Field	Value
Display name/identifier	Default asphalt concrete
Description of object	
Author	
Date Created	10/29/2010 11:00:00 PM
Approver	
Date approved	10/29/2010 11:00:00 PM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Base and Subbase (Unbound Granular Material) Layers

The untreated base course (UTBC) and granular borrow (GB) layers consist of unbound granular materials corresponding to an A-1-a AASHTO classification. Inputs for the UTBC layer are shown in Table D-12. The UTBC modulus is selected as 40,000 psi at optimum moisture and density.

Table D-12

UTBC (Base) Properties for New HMA Pavement Design Example

Layer 3 Non-stabilized Base : A-1-a

Unbound	
Layer thickness (in.)	4.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
40000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-1-a
Description of object	Default material
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	6.0
Plasticity Index	1.0
Is layer compacted?	True

	Is User Defined?	Value
Maximum dry unit weight (pcf)	False	127.7
Saturated hydraulic conductivity (ft/hr)	False	5.054e-02
Specific gravity of solids	False	2.7
Optimum gravimetric water content (%)	False	7.4

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	7.2555
bf	1.3328
cf	0.8242
hr	117.4000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	
#50	
#40	20.0
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8-in.	57.2
1/2-in.	63.1
3/4-in.	72.7
1-in.	78.8
1 1/2-in.	85.8
2-in.	91.6
2 1/2-in.	
3-in.	
3 1/2-in.	97.6

Inputs for the GB layer are shown in Table D-13. The resilient modulus of the GB is limited to 2 to 3 times the subgrade layer to avoid an unrealistic result. The subgrade input as shown in the next section is 12,383 psi, thus the unbound layer above this should be in the order of 2 to 3 times greater and was estimated at 20,000 psi. ME Design defaults were used for the gradations for both UTBC and GB.

Table D-13

Granular Borrow (Subbase) Properties for
New HMA Pavement Design Example

Layer 4 Non-stabilized Base : A-1-a

Unbound	
Layer thickness (in.)	12.0
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
20000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	**Value**
Display name/identifier	A-1-a
Description of object	Default material
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	6.0
Plasticity Index	1.0
Is layer compacted?	False

	Is User Defined?	**Value**
Maximum dry unit weight (pcf)	False	127.2
Saturated hydraulic conductivity (ft/hr)	False	5.054e-02
Specific gravity of solids	False	2.7
Optimum gravimetric water content (%)	False	7.4

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	7.2555
bf	1.3328
cf	0.8242
hr	117.4000

Sieve Size	**% Passing**
0.001mm	
0.002mm	
0.020mm	
#200	8.7
#100	
#80	12.9
#60	
#50	
#40	20.0
#30	
#20	
#16	
#10	33.8
#8	
#4	44.7
3/8-in.	57.2
1/2-in.	63.1
3/4-in.	72.7
1-in.	78.8
1 1/2-in.	85.8
2-in.	91.6
2 1/2-in.	
3-in.	
3 1/2-in.	97.6

Subgrade (Soil) Layer

The subgrade AASHTO classification, the percent passing the #200 sieve, and the laboratory resilient modulus (M_r) at optimum moisture content and density are the critical required inputs for the ME Design. It is important to note that the required input for M_r is at optimum moisture and density. The program then varies the M_r from month to month that may result in higher or lower values over time. The M_r is important because it is used to calculate the stresses and deflections with the pavement structure. These are then used to compute fatigue damage and fatigue cracking and permanent deformation or rutting. The inputs for the subgrade soil are provided in Table D-14.

There are three approaches to estimate the input subgrade resilient modulus. These include the following from most accurate to least accurate.

1. (Level 2 Recommended and most accurate) Measure FWD deflections along the project or nearby projects if needed with similar soils and backcalculate the subgrade elastic modulus. Then adjust the elastic modulus to a "lab optimum moisture and density" by multiplying by the proper coefficient (either 0.55 fine grain or 0.67 coarse grain).

   FWD testing was accomplished along the existing pavement. A deflection profile was created and examined and significant outliers removed. The backcalculated elastic subgrade modulus, Es, was computed point by point along the project using either the AASHTO procedure with the outer sensor (at 60 in) or a backcalculation software program such as ModComp. Here is an example calculation of the subgrade elastic modulus for the mean point along the project:

   $$Es = 0.24\ P / d\ S = 0.24 * 9850 / 0.00175 * 60 = 22{,}514 \text{ psi}$$

   Where: Es = Elastic modulus of the subgrade, psi
   P = FWD load in lbs.
   d = Deflection at spacing S from the loading plate, in
   S = Spacing to outer sensor, in

   The subgrade was a fine grained A-4 soil and thus the adjustment factor to reduce this elastic modulus from a "field value at in situ moisture" to a "lab value at optimum moisture content is 0.55 as explained in Section 17. The subgrade resilient modulus is calculated as follows:

   M_r (lab value at optimum moisture) = 0.55 * 22,514 = **12,383 psi**

2. (Level 2 Approximate only) Establish the CBR of the subgrade material from the borings. The mean CBR along the project was 10 (do not use the lowest CBR). Use the correlation provided in the software to estimate an approximate Mr of the subgrade soil from the following equation:

Table D-14

Subgrade Soil Strength and Other Properties for New HMA Design Example

Layer 5 Subgrade : A-4

Unbound	
Layer thickness (in.)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 2)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
12383.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-4
Description of object	Default material
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	21.0
Plasticity Index	5.0
Is layer compacted?	False

	Is User Defined?	Value
Maximum dry unit weight (pcf)	True	106.5
Saturated hydraulic conductivity (ft/hr)	False	5.09e-05
Specific gravity of solids	False	2.7
Optimum gravimetric water content (%)	True	14.9

User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	68.8377
bf	0.9983
cf	0.4757
hr	500.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	60.6
#100	
#80	73.9
#60	
#50	
#40	82.7
#30	
#20	
#16	
#10	89.9
#8	
#4	93.0
3/8-in.	95.6
1/2-in.	96.7
3/4-in.	98.0
1-in.	98.7
1 1/2-in.	99.4
2-in.	99.6
2 1/2-in.	
3-in.	
3 1/2-in.	99.8

$\mathbf{M_r = 2555\ (CBR)^{0.64}}$ [CBR < 30%]

A CBR = 10 would result in an estimated **Mr = 11,153 psi** at optimum moisture and density.

3. (Level 3 Approximate only) Establish the AASHTO soil classification of the top several feet of soil either through soil borings along the project or soil maps. Use the soil class to estimate the Mr at optimum moisture and density based on default averages provided.

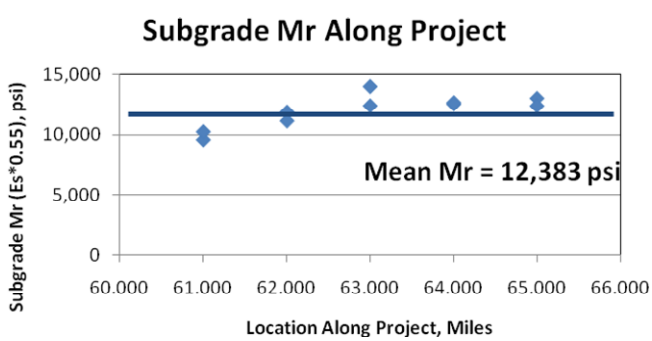
   Information obtained from the county soil report (USDA-NRCS soil survey database) indicated an A-4 soil classification along much of this project. A mean resilient modulus for an A-4 soil based on national correlations is provided in Section 14.6 at optimum moisture and density of **16,500 psi**.

Table D-15 shows a plot of the final input M_r (lab value at optimum moisture) along the project. The most accurate Level 2 procedure of FWD testing and backcalculation resulted in a mean M_r along the project of 12,383 psi. Note that if Level 3 default had been assumed, a value of 16,500 psi would have been obtained. This could make a difference in the design. For the other Level 2 approach using the CBR to estimate the resilient modus a value of 11,153 psi was obtained.

Level 2 FWD testing and backcalculation and adjustment of subgrade M_r along the project of 12,383 psi and this will be used in this design.

Table D-15

Table D-14



Thermal (Transverse) Cracking

Low temperature transverse cracking is predicted by ME Design using data shown in Table D-16. All of the inputs are estimated from various models for level 3.

Table D-16
HMA thermal cracking properties including creep compliance, indirect tensile strength, coefficient of thermal contraction for new HMA design example.

Thermal Cracking (Input Level: 3)

Indirect tensile strength at 14 °F (psi)	437.75
Thermal Contraction	
Is thermal contraction calculated?	True
Mix coefficient of thermal contraction (in./in./°F)	-
Aggregate coefficient of thermal contraction (in./in./°F)	5.0e-006
Voids in Mineral Aggregate (%)	19.5

	Creep Compliance (1/psi)		
Loading time (sec)	-4 °F	14 °F	32 °F
1	3.19e-007	5.46e-007	7.80e-007
2	3.55e-007	6.46e-007	1.02e-006
5	4.10e-007	8.07e-007	1.46e-006
10	4.56e-007	9.55e-007	1.92e-006
20	5.08e-007	1.13e-006	2.51e-006
50	5.86e-007	1.41e-006	3.60e-006
100	6.53e-007	1.67e-006	4.72e-006

Run ME Design to Predict Trial Design Performance over Design Period

The ME Design software can begin the analysis process to predict the performance of the trial design over the 20 year design life of the pavement after all design inputs are provided and all inputs are colored green as shown below. Click on “Run Analysis”. The program runs the Traffic, Climate, Thermal Cracking, HMA Analysis, and Summary/IRI models and reports the analysis status on the upper right hand corner of the screen (see Table D-17). The program creates a summary file and other output files in the project directory at the end of the analysis C:\MyDARWin\Projects. The summary file is in a PDF file (short summary) and a MS Excel file output (longer more details). A description of the summary file content is presented later in this section.

Table D-17

ME Design Program Output Screen After Completing A Run

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	170.00	131.95	90.00	99.63	Pass
Permanent deformation - total pavement (in.)	0.75	0.50	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	15.00	14.84	90.00	90.28	Pass
AC thermal cracking (ft/mile)	905.00	27.17	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	10000.00	1140.22	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.75	0.25	90.00	100.00	Pass

Layer Moduli

The modulus of each layer month by month is extremely important to review to see if they are reasonable. Table D-17 was extracted from the Design ME output to illustrate a few of the output layer moduli. It is very important for the designer to understand the typical range of HMA, UTBC, GB, and subgrade modulus values. For example, the following is summarized for each material during the month of May:

- HMA E*: The dynamic modulus varies from 683,036 psi during warmer temperature 20 percent hours to 1,875,350 psi during coldest temperatures 20 percent hours in the month.
- UTBC Mr: The input Mr at optimum moisture was 40,000 psi. However, the program estimates moisture content each month and adjusts the Mr from month to month. The untreated base course averaged 54,480 psi in May but ranges from about 27,000 to 61,000 psi over the year. During a later cold winter, the material may become frozen during one or more months with a frozen modulus of about 900,000 psi.
- GB Mr: The 12-in GB is divided into three 4-in sublayers by the software. The granular borrow is 30,040 psi in May (middle layer) but varies from month to month and between the three 4-in sublayers of GB.
- Subgrade Mr: The subgrade is also sublayered. The upper portion of the A-4 subgrade is 6,303 psi in May and appears to be steadily decreasing over the first year as moisture content increases with time. The modulus eventually levels off about the first year when moisture stabilizes.

Table D-18

Summary of layer moduli values as output for this project for the first year.

	HMA E* Temperature Quintiles, psi					UTBC	Granular Borrow GB			A-4 Subgrade
Month	1 Coldest	2	3	4	5 Warmest	Mr, psi	Mr, psi			Mr, psi
Oct	1,514,460	1,339,040	1,231,850	1,120,600	960,926	61,440	30,480	30,640	30,740	8,581
Nov	2,349,250	1,988,530	1,752,730	1,553,630	1,353,610	61,400	30,420	30,540	30,600	7,863
Dec	2,860,710	2,860,710	2,860,710	2,435,520	1,399,850	58,320	30,560	30,560	30,580	7,455
Jan	2,860,440	2,860,440	2,822,570	2,615,420	2,312,910	27,880	13,780	17,100	30,440	7,170
Feb	2,860,310	2,860,310	2,860,310	2,612,490	2,097,430	27,800	18,480	21,440	30,320	6,860
Mar	2,860,050	2,336,550	1,898,340	1,546,330	1,237,800	35,680	23,800	25,780	30,180	6,575
Apr	2,859,780	2,158,960	1,731,990	1,434,710	1,109,450	45,120	28,340	29,020	30,060	6,414
May	1,875,350	1,348,270	1,068,210	867,881	683,036	54,480	30,140	30,040	29,960	6,303
Jun	1,146,800	845,450	674,117	574,252	496,025	60,800	30,100	29,980	29,900	6,229
July	649,925	570,285	517,602	470,510	414,301	61,280	30,080	29,940	29,840	6,179
Aug	672,620	590,635	543,838	499,843	441,517	60,680	29,780	29,660	29,560	6,130
Sep	1,213,890	901,620	807,162	706,762	610,186	60,560	29,660	29,520	29,420	6,068

Distress Summary

The Distress Summary sheet in the output file provides a month by month overall summary of the HMA trial design for the project including critical material properties, traffic, and distress data. Detailed data for each distress type is provided on separate sheets as noted above.

The distress summary sheet shown in Table D-18 indicates that this pavement carried 8.656 million heavy trucks in the design lane over the design period (the first month showed 26,192 trucks) and this provides an overall idea of the traffic loading on the pavement. Total cracking (alligator bottom up fatigue cracking), AC thermal transverse cracking, permanent deformation (total rutting), and IRI are shown in the table for the first half year and the final half year. They all increase over time and traffic over the 20 year design life. All of these values look reasonable over the 20 year period. The predicted distress at the specified reliability is also presented.

Table D-19

Distress Summary Output for New HMA Pavement Design for the 1.5-in SMA / 6.5-in HMA Trial Design

Month	Pavement Age (years)	Heavy Trucks (cum.)	Crack Spacing (ft)	Mean Predicted Distress					Predicted Distress @ Reliability			
				IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal cracking (ft/mile)	Total Cracking (%)	Reflective Cracking (%)	IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal cracking (ft/mile)	Bottom-Up Cracking (%)
10/2016	0.08	29,131	500	53.3	0.083	0.00	0.004	0.000	69.7	0.119	27.17	1.452
11/2016	0.17	54,417	500	53.6	0.090	0.00	0.006	0.000	70.1	0.127	27.17	1.454
12/2016	0.25	76,863	500	53.8	0.093	0.00	0.007	0.000	70.3	0.130	27.17	1.455
1/2017	0.33	98,941	500	54.0	0.097	0.00	0.009	0.000	70.6	0.135	27.17	1.457
2/2017	0.42	122,707	500	54.1	0.101	0.00	0.010	0.000	70.9	0.139	27.17	1.459
3/2017	0.50	147,094	500	54.4	0.106	0.00	0.014	0.000	71.2	0.145	27.17	1.462

(Intermediate years not included)

3/2036	19.50	8,351,110	500	95.5	0.410	0.00	2.380	0.000	129.9	0.488	27.17	13.149
4/2036	19.58	8,398,880	500	95.7	0.410	0.00	2.380	0.000	130.1	0.488	27.17	13.235
5/2036	19.67	8,452,110	500	95.9	0.411	0.00	2.410	0.000	130.4	0.488	27.17	13.521
6/2036	19.75	8,504,060	500	96.2	0.412	0.00	2.440	0.000	130.8	0.490	27.17	13.885
7/2036	19.83	8,557,060	500	96.5	0.416	0.00	2.480	0.000	131.3	0.494	27.17	14.329
8/2036	19.92	8,609,250	500	96.8	0.418	0.00	2.510	0.000	131.7	0.496	27.17	14.671
9/2036	20.00	8,658,240	500	97.0	0.418	0.00	2.530	0.000	131.9	0.496	27.17	14.843

Note the following definitions for selected columns in Table D-18 for the Mean Predicted Distress columns.

- Month – Month since opening to traffic.
- Pavement Age – time since opening to traffic in years.
- Heavy Trucks – design lane, Class 4 to 13, cumulative over time.
- Crack Spacing – Not a useful output.
- IRI – mean of both wheelpaths.
- Permanent deformation – mean total rutting in all layers (average of both wheel paths (wire line definition)).
- AC thermal fracture – ft/mile (divide by 12 to determine no. cracks/mile), all severities.
- Total Cracking – percent area of lane, all severities. Note that this is Bottom Up Fatigue Cracking or Alligator Cracking.
- Reflective Cracking -- this column is only for HMA overlays, not new design.

The Predicted Distress at Reliability is computed at the design reliability. These values are higher than the mean since they represent the upper reliability level used to compare to the design criteria for pass/fail purposes.

Design Reliability (Pass/Fail)

The Reliability Summary tab (see Table D-19) of the ME Design output shows the information below. Each of the following are compared to their design criteria for the given trial design: terminal IRI, HMA bottom up fatigue cracking, HMA thermal fracture, and permanent deformation both for HMA only and total pavement. AC top-down fatigue cracking is not considered in design currently. The IRI, rutting, bottom-up fatigue cracking (alligator), and thermal cracking all meet the design criteria at the 90 percent level of design reliability.

Table D-20

Summary of Design Reliability for Example HMA Design

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	170.00	131.95	90.00	99.63	Pass
Permanent deformation - total pavement (in.)	0.75	0.50	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	15.00	14.84	90.00	90.28	Pass
AC thermal cracking (ft/mile)	905.00	27.17	90.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	10000.00	1140.22	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.75	0.25	90.00	100.00	Pass

Plots showing alligator cracking, rutting, and IRI are also given in the ME Design output (Tables D-20 through D-23). The top curve is distress or IRI predicted at the specified reliability level of 90 percent for cracking, rutting, and IRI. The 50 percentile or mean model prediction is the curve directly under the top curve. These two curves represent total rutting and not rutting in the individual pavement layers.

Table D-20

Plot showing alligator cracking, percent lane area over time. The horizontal red line is the design criteria of 10 percent, the black lower curve is mean 50 percent prediction, and the blue curve is the 90 percent reliability level.)

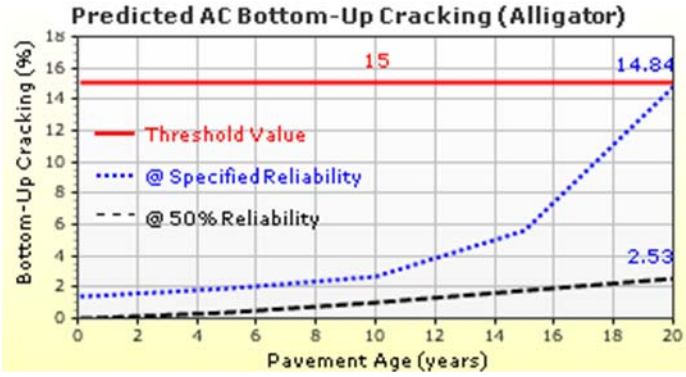


Table D-21
Permanent deformation or rutting over time. The horizontal red line is the design criteria of 0.50-in, top blue curve is the 90 percent reliability level, black curve down is mean 50 percent total rutting prediction.

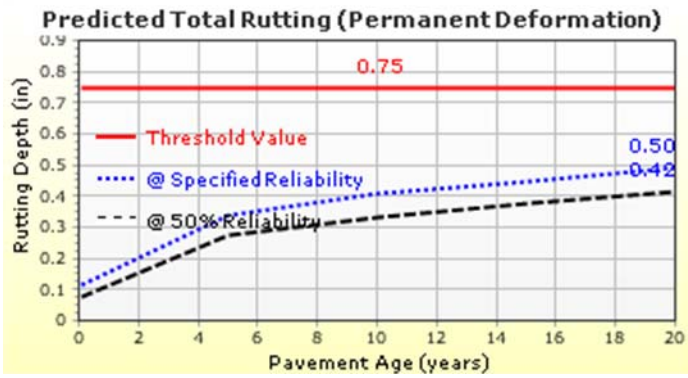


Table D-22
Total rutting over time showing components of HMA, aggregate base, and subgrade rutting. Most of rutting is in HMA layer.

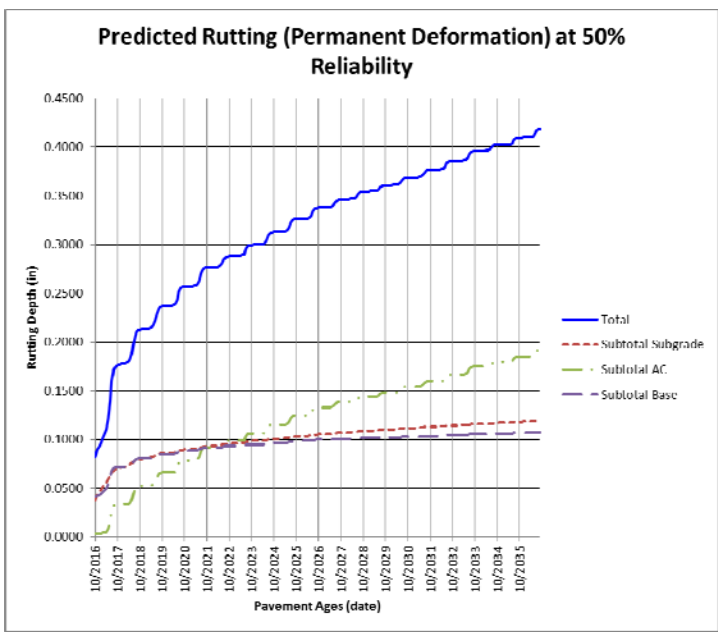
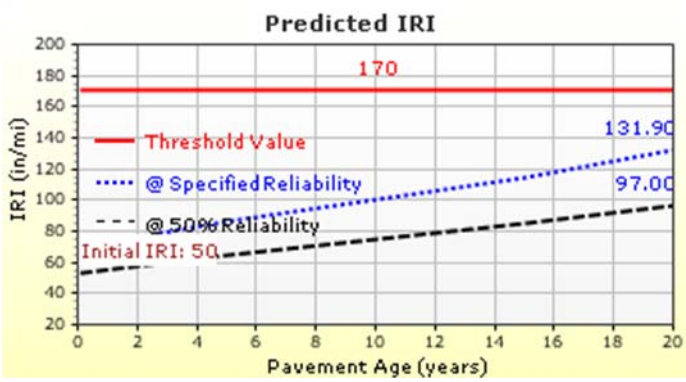


Table D-23
Plot showing predicted IRI over the design life of the final HMA pavement. The design criterion of 170 in/miles is met by the top curve at 90 percent reliability. Note that top curve is the 90 percentile and the lowest curve is the mean 50 percentile prediction curve.



Transverse low temperature cracking did not show any distress for this new design project.

Optimized Design

The designer usually needs to try several trial designs to arrive at an acceptable or optimal design. The surface SMA will be held at 1.5-in and the trial HMA thickness values of 5.5, 6.0, 6.5, and 7.5 inches were run for this design. These trial designs showed the following results.

- 5.5-in HMA Failed
- 6.0-in HMA Failed
- 6.5-in HMA Passed all criteria (OPTIMUM DESIGN)
- 7.0-in HMA Passed all criteria, overdesign

Through the modification of layer thickness and material properties various other alternative designs can be evaluated. Design inputs that could be modified to obtain an optimum design:

- Different layers of HMA to make up the thickness
- Aggregate base thickness
- Binder grade
- Subgrade stabilization

Once a design passes all of the criteria it can be considered a feasible alternative that can be subjected to a cost analysis and compared to other alternatives.

APPENDIX E
Utah New JPCP Design Example

This project is being designed as a reconstruction for a section of Interstate 15 near Nephi in Juab County, Utah. The deteriorated existing JPCP pavement will be removed and a new JPCP structure constructed. Poor subgrade soils are known to exist at this location.

Design Life

The jointed plain concrete pavement (JPCP) has a 40-year design life and it is anticipated that construction (PCC placement) will begin in June 2015 and this section will be opened to traffic one month later in September 2015.

Construction Requirements

The pavement is expected to have an initial IRI of approximately 60 in/mile assuming a good quality of construction with stringent ride specifications.

Analysis Parameters

It is expected that at the end of the 40 year design life at 95 percent confidence, the pavement will have no more than 10 percent slabs with transverse cracking and no more than 0.15 inch joint faulting. The smoothness should be maintained at an IRI of less than 170 in/mi at a reliability level of 95 percent at least over the first 20 years when a diamond grinding may be required. These performance criteria are all entered into the Performance Criteria window of the ME Design software.

JPCP Local Calibration Factors

The concrete pavement coefficient of thermal expansion (CTE) measured by FHWA/LTPP in previous years was found to be in error (approximately $0.8*^{10-6}$/F too high). This error was corrected and a new national calibration was completed in 2011 under NCHRP 20-07. The new national calibration coefficients were recently validated for Utah concrete pavement transverse cracking and joint faulting and found to be unbiased. Thus, these calibration coefficients that utilize the correct CTE values measured in the UDOT lab can now be used in Utah.

Fortunately, UDOT has recently sponsored an extensive lab testing program of the CTE of concrete produced from about 20 pits across the State and these properly measured CTE values are lower as described above. The validation of the new national calibration coefficients was conducted using Utah data and the correct (lower) CTE values was used for each Utah JPCP section. The validation study showed that NCHRP 20-07 models for JPCP transverse cracking and transverse joint faulting are unbiased and adequate for Utah conditions. This makes it possible to use a UDOT lab measured CTE (AASHTO T 336, Coefficient of Thermal Expansion of Hydraulic Cement

Concrete) directly in the design of JPCP without modification.

The Utah validated NCHRP 20-07 calibration factors are shown below:

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP Transverse Cracking Model	C4	0.6
	C5	-2.05
	PCC Cracking Standard Deviation	Pow(57.08*CRACK,0.33) +1.5

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP Transverse Joint Faulting Model	C1	0.5104
	C2	0.00838
	C3	0.00147
	C4	0.008345
	C5	5999
	C6	0.8404
	C7	5.9293
	C8	400
	PCC Faulting Standard Deviation	0.0831*Pow(FAULT,0.3426)+0.00521

The IRI model indicated no bias and was not changed from the original calibration coefficients.

Model or Submodel Type	Model Coefficients	Calibration Factors
JPCP IRI Model	J1 (for Cracking)	0.8203
	J2 (for Spalling)	0.4417
	J3 (for Faulting)	1.4929
	J4 (for Site Factor)	25.24
	PCC IRI Standard Deviation	5.4

The Utah local calibration coefficients are shown below for cracking and faulting of JPCP for this project. These should be checked after each run of the program.

Calibration Coefficients

PCC Faulting

$$C_{12} = C_1 + (C_2 * FR^{0.25})$$
$$C_{34} = C_3 + (C_4 * FR^{0.25})$$
$$FaultMax_0 = C_{12} * \delta_{curling} * \left[\log(1 + C_5 * 5.0^{EROD}) * \log\left(P_{200} * \frac{WetDays}{p_s}\right)\right]^{C_6}$$
$$FaultMax_i = FaultMax_0 + C_7 * \sum_{j=1}^{m} DE_j * \log(1 + C_5 * 5.0^{EROD})^{C_6}$$
$$\Delta Fault_i = C_{34} * (FaultMax_{i-1} - Fault_{i-1})^2 * DE_i$$
$$C_8 = DowelDeterioration$$

C1: 0.5104	C2: 0.00838	C3: 0.00147	C4: 0.008345
C5: 5999	C6: 0.8404	C7: 5.9293	C8: 400
PCC Reliability Faulting Standard Deviation			
0.0831*Pow(FAULT,0.3426)+0.00521			

IRI-jpcp

C1 - Cracking	C1: 0.8203	C2: 0.4417
C2 - Spalling	C3: 1.4929	C4: 25.24
C3 - Faulting	**Reliability Standard Deviation**	
C4 - Site Factor	5.4	

PCC Cracking

$$\log(N) = C1 \cdot \left(\frac{MR}{\sigma}\right)^{C2}$$
$$CRK = \frac{100}{1 + C4\, FD^{C5}}$$

Fatigue Coefficients		**Cracking Coefficients**	
C1: 2	C2: 1.22	C4: 0.6	C5: -2.05
PCC Reliability Cracking Standard Deviation			
Pow(57.08*CRACK,0.33) + 1.5			

Traffic

There exists a WIM site near this project so Level 1 traffic inputs were readily available.

<u>Truck Volume and Axle Volume Estimates</u>

Volume Adjustment Factors – The initial two-way average annual daily truck traffic (AADTT) on this highway is estimated to be 4,000 trucks (Classes 4 through 13) during the first year of its service. This value will be adjusted through several volume adjustment factors. These include:

1. **Directional distribution –** 50 percent trucks in each direction
2. **Lane distribution –** 90 percent trucks in outer design lane
3. **Monthly volume adjustments –** There are typically truck volume differences throughout the year. Monthly variations in truck traffic are summarized below. These slight variations typically do not typically have a significant effect on pavement performance except for unusual situations of recreational highways and farm to market roads. Values used are shown below and were obtained from the nearby WIM site (Table E-1).

Table E-1

Monthly Adjustment Factors for New JPCP Trial Design.

Volume Monthly Adjustment Factors Level 3: Default MAF

Month	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
January	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.7	0.7
February	0.8	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.7	0.7
March	0.9	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.8	0.7
April	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
May	1.2	1.1	1.2	1.1	1.2	1.1	1.1	1.0	1.3	1.4
June	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.3	1.3
July	1.2	1.1	1.2	1.1	1.1	1.1	1.1	1.1	1.3	1.3
August	1.2	1.1	1.1	1.1	1.1	1.0	1.1	1.0	1.2	1.2
September	1.1	1.1	1.1	1.0	1.1	1.0	1.0	1.0	1.1	1.1
October	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.1	1.0	1.0
November	0.9	0.9	0.9	0.9	0.9	1.0	0.9	1.0	0.9	0.9
December	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.7	0.7

4. **Hourly truck distribution –** Truck volume varies over a 24-hour period. The following values were used in the analysis as shown in Table E-2 which affects PCC pavement design and performance.

Table E-2

Hourly Distribution Factors for New JPCP Trial Design

Truck Distribution by Hour

Hour	Distribution (%)	Hour	Distribution (%)
12 AM	2.3%	12 PM	5.9%
1 AM	2.3%	1 PM	5.9%
2 AM	2.3%	2 PM	5.9%
3 AM	2.3%	3 PM	5.9%
4 AM	2.3%	4 PM	4.6%
5 AM	2.3%	5 PM	4.6%
6 AM	5%	6 PM	4.6%
7 AM	5%	7 PM	4.6%
8 AM	5%	8 PM	3.1%
9 AM	5%	9 PM	3.1%
10 AM	5.9%	10 PM	3.1%
11 AM	5.9%	11 PM	3.1%
		Total	100%

5. **Vehicle class distribution –** The percent of each vehicle class in the traffic stream is an important input. These are shown in Table E-3 for this project as measured at the WIM site near the project. The Class 9 truck is the most common type of vehicle on this rural highway.

Table E-3

Vehicle Class Distribution Factors for New JPCP Trial Design

Distributions by Vehicle Class

Vehicle Class	AADTT Distribution (%) (Level 3)	Growth Factor	
		Rate (%)	Function
Class 4	2.39%	2.8%	Linear
Class 5	17.47%	2.8%	Linear
Class 6	1.61%	2.8%	Linear
Class 7	0.08%	2.8%	Linear
Class 8	14.62%	2.8%	Linear
Class 9	54.06%	2.8%	Linear
Class 10	2.28%	2.8%	Linear
Class 11	1.04%	2.8%	Linear
Class 12	0.26%	2.8%	Linear
Class 13	6.19%	2.8%	Linear

6. **Truck traffic growth –** Truck traffic has grown from 1 to over 10 percent on Utah highways over the years. A linear growth rate of 2.8 percent was determined after plotting past truck growth over time for this project.
7. **Axle load distribution –** The axle load distribution is the most important traffic input. Damage is caused by the heavy single, tandem, tridem, and quad axle loads. The distributions used were measured at the nearby WIM stations in both directions.

Table E-4

Illustration of Partial Axle Weight Distribution table.

	Truck	Tandem Axle Loads, pounds									
Month	Class	32000	34000	36000	38000	40000	42000	44000	46000	48000	50000
January	4	14.9	6.09	4.53	4.5	4.11	0.87	2.91	2.91	0	2.21
January	5	0	0	0	0	0	0	0	0	0	0
January	6	3.78	3.93	2.78	2.9	1.5	1.23	0.86	0.86	0.38	0.45
January	7	0	0	0	0	0	0	0	0	0	0
January	8	1.02	1.38	0.77	0.88	0.72	0.9	0.59	0.59	0.36	0.45
January	9	6.63	6.53	5.88	4.82	3.93	2.7	2.18	2.18	1.7	0.79
January	10	2.35	2.33	1.92	0.88	0.5	0.2	0.49	0.49	0.44	0.08
January	11	0	0	0	0	0	0	0	0	0	0
January	12	0	0	0	0	0	0	0	0	0	0
January	13	1.03	0.67	0.7	0.52	0.39	0.28	0.47	0.47	0.29	0.1

General Traffic Inputs

These consist of lateral truck/wheel wander, number of axles per truck, axle configuration, and wheel base.

Lateral truck/wheel wander – Three inputs are required here as shown in Table E-5.

1. Mean wheel location – Distance from outer edge of truck wheel to lane marking or paint stripe – 18 inch standard used in calibration.
2. Standard deviation of lateral truck wander – 10 inch standard used in calibration.
3. Design lane width – This distance is paint stripe to paint stripe. It is not slab width as measured from longitudinal joint to longitudinal joint. This is 12 feet.

Axle configuration – Axle width, spacing, and tire pressure are required as shown in Table E-5.

1. Actual axle width (edge to edge of tire) outside dimensions – 8.5 ft, typical
2. Dual tire spacing – 12 inch typical used in calibration.
3. Tire pressure – 120 psi, hot rolling pressure used in calibration.

Table E-5

Axle Configuration and Truck Wheel Base for New JPCP Design Example

Axle Configuration

Traffic Wander	
Mean wheel location (in.)	18
Traffic wander standard deviation (in.)	10
Design lane width (ft)	12

Axle Configuration	
Average axle width (ft)	8.5
Dual tire spacing (in.)	12
Tire pressure (psi)	120

Average Axle Spacing	
Tandem axle spacing (in.)	51.6
Tridem axle spacing (in.)	49.2
Quad axle spacing (in.)	49.2

Wheelbase				
Value Type	Axle Type	Short	Medium	Long
Average spacing of axles (ft)		12	15	18
Percent of Trucks (%)		17	22	61

Wheel base – This dimension refers to the distance from the steering axle and the next axle of the truck tractor and ranges from less than 12 to more than 20 ft. Three typical spacings are used:

1. short 11 to 13.5 ft
2. medium 13.5 to 16.5 ft
3. long 16.5 to 20 ft see Table B-7

The percentages of truck tractors that fall into each of these ranges are shown in Table

E-5. These data were compiled from national traffic studies. Wheel base distribution affects top down cracking of JPCP.

Axles per truck – Mean number of axles per vehicle/truck class (Table E-6).

Table E-6

Number of Axles Per Truck for New JPCP Trial Design

Number of Axles per Truck				
Vehicle Class	**Single Axle**	**Tandem Axle**	**Tridem Axle**	**Quad Axle**
Class 4	1.58	0.42	0	0
Class 5	2	0	0	0
Class 6	0.84	1	0	0
Class 7	0.49	0.91	0.27	0.26
Class 8	2.09	0.82	0.01	0
Class 9	1.3	1.84	0.005	0
Class 10	1.02	0.97	0.98	0.04
Class 11	3.52	0.28	0.32	0
Class 12	2.69	1.07	0.14	0.19
Class 13	2.19	1.28	0.89	0.04

Climate Inputs

The project site is in the vicinity of Nephi, Utah. The latitude and longitude of this site is as follows obtained from various sources such as GPS units or Google Earth:

Latitude 39.7172 degrees (decimal)
Longitude -111.828 degrees (decimal)

The designer enters the project site latitude and longitude and elevation (5128-ft) into the ME Design and selects the most appropriate weather station or creates a virtual weather station. One well site located near I-15 indicated a water table depth of 27-ft. http://geology.utah.gov/geothermal/interactive/index.html However, given the softness of the subgrade soil the water table may be much higher in some areas.

The closest weather station is Provo at 35 miles north and the next closest was Price at 58 miles. The elevation of Provo at 4497-ft and Price at 5830-ft includes the 5128-ft of the project site near Nephi. All other weather stations were further away and not appropriate. These two were selected for a virtual weather station for the Nephi project site. This virtual weather station contains the sunrise time, sunset time and radiation for each day of the design life period. The temperature, rainfall, air speed, sunshine, and depth of ground water table are included for each 24 hour period in each day of the design life.

Table E-7 provides the output from the climatic data analysis for the project site.

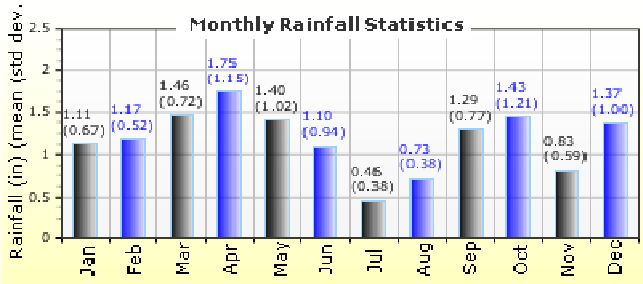
Table E-7

Climate Inputs for New JPCP Design Example

Climate Inputs

Climate Data Sources:

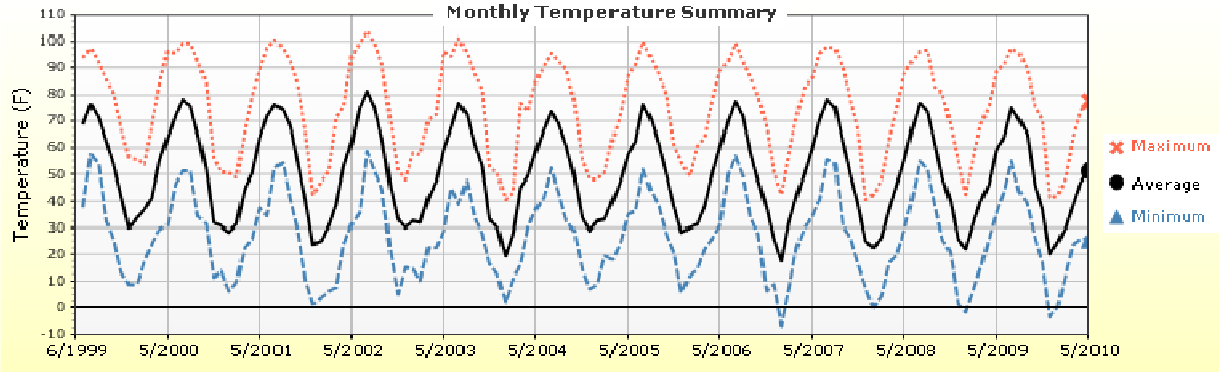
Climate Station Cities:	Location (lat lon elevation(ft))
PROVO, UT	40.21900 -111.71700 4497
PRICE, UT	39.54500 -110.75000 5830

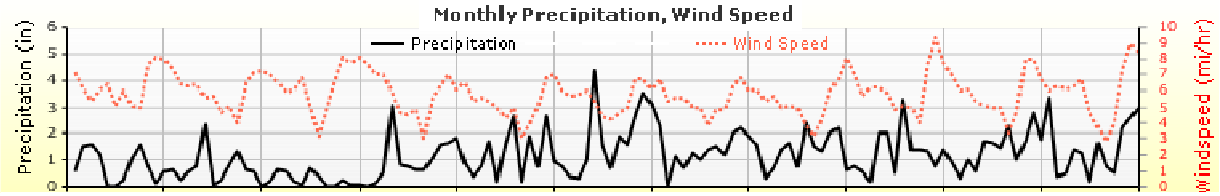


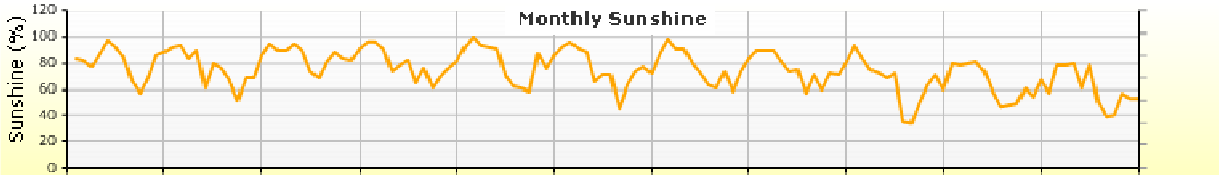
Annual Statistics:

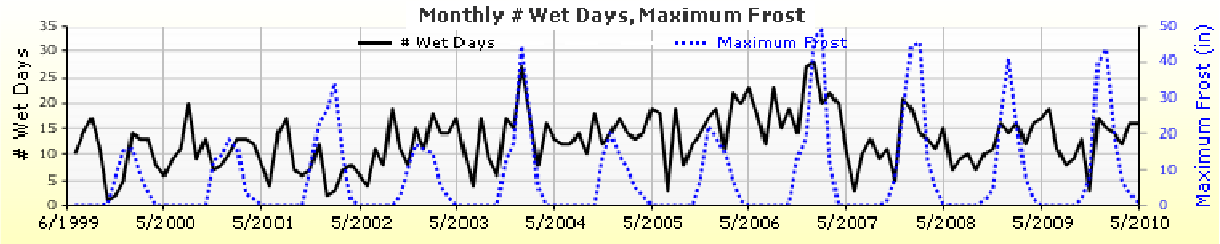
Mean annual air temperature (ºF)	50.19
Mean annual precipitation (in.)	14.11
Freezing index (ºF - days)	619.03
Average annual number of freeze/thaw cycles:	117.33
water table depth(ft)	10.00

Monthly Climate Summary:









Structure and Layer Material Definition

Select a set of structural and materials inputs to form the trial design that will be evaluated for its performance. The procedure is an iterative procedure and the designer will have to develop a trial design and make several modifications to it, before a feasible and economic or final design is achieved. The trial design can be obtained using another design procedure such as the AASHTO 1993 or an alternative of interest. The following trial design structure and layer material types were selected for this example given the poor subgrade and heavy traffic (see Table E-8):

- 12.0 inch JPCP layer.
- Dowels at all transverse joints.
- Dowel diameter required to control joint faulting increases with the number of trucks in the design lane and other slab (e.g., joint spacing) and support properties. Trial value of 1.5-in diameter will be evaluated.
- Joints will be formed with a single saw cut and include a liquid filler.
- Slab width is 12 ft wide and length is 15 ft perpendicular joint spacing.
- Tied PCC shoulders with 50 percent load transfer efficiency.
- 3-in HMA base layer to provide increased support and minimize faulting potential. The base erodibility factor will be 1.0. There will be full friction between the PCC slab and HMA base over the 480 month design period.
- 6-in untreated base course (UTBC).
- Subgrade: Due to the poor subgrade soil, it may be necessary to stabilize the top few inches to provide more long term stability to the heavily loaded pavement.

The calibration standard of 0.85 was used for Surface Shortwave Absorptivity PCC Surface. This controls the flow of heat through the slab. This value has been found to provide accurate temperature measurements through the slab as the white slabs change into gray with aging.

Table E-8

Design Features Inputs for New JPCP Design Example

Design Properties

JPCP Design Properties

Structure - ICM Properties	
PCC surface shortwave absorptivity	0.85

Doweled Joints	
Is joint doweled ?	True
Dowel diameter (in.)	1.50
Dowel spacing (in.)	12.00

Tied Shoulders	
Tied shoulders	True
Load transfer efficiency (%)	50.00

PCC joint spacing (ft)	
Is joint spacing random ?	False
Joint spacing (ft)	15.00

Widened Slab	
Is slab widened ?	False
Slab width (ft)	12.00

PCC-Base Contact Friction	
PCC-Base full friction contact	True
Months until friction loss	480.00

Sealant type	Other(Including No Sealant... Liquid... Silicone)

Erodibility index	1

Permanent curl/warp effective temperature difference (ºF)	-10.00

Pavement Layers Material Properties

PCC (Surface Layer)

Inputs for the General and Thermal, Mix, and Strength Properties of the PCC slab are presented in Tables E-9, E-10 and E-11. The most important input in Table E-9 is the PCC coefficient of thermal expansion (CTE). Several Utah typical CTE values of concrete can be seen in Appendix H. This value depends mainly upon the type of coarse aggregate used in the PCC mix since it has the greatest volume in the mix. The CTE for this project was selected based on the assumption that the coarse aggregate would be quartzite. One pit near Nephi shows CTE = 5.13 / ^{o}F for concrete with quartzite as coarse aggregate. The Statewide mean is 5.6 / ^{o}F for concrete with quartzite and this will be used for this design as shown in Section 14.4 in this MOI.

Table E-9

PCC General and Thermal Properties Inputs

PCC	
Thickness (in.)	12.0
Unit weight (pcf)	145.0
Poisson's ratio	0.2
Thermal	
PCC coefficient of thermal expansion (in./in./ºF x 10^-6)	5.6
PCC thermal conductivity (BTU/hr-ft-ºF)	1.25
PCC heat capacity (BTU/lb-ºF)	0.28

Table E-10

PCC Mix Properties Inputs for New JPCP Design Example

Mix		
Cement type		Type II (2)
Cementitious material content (lb/yd^3)		564
Water to cement ratio		0.443
Aggregate type		Quartzite (0)
PCC zero-stress temperature (ºF)	Calculated Internally?	True
	User Value	-
	Calculated Value	98.4
Ultimate shrinkage (microstrain)	Calculated Internally?	True
	User Value	-
	Calculated Value	526.2
Reversible shrinkage (%)		50
Time to develop 50% of ultimate shrinkage (days)		35
Curing method		Curing Compound

Inputs for concrete strength and modulus are critical. These values must be selected as

means and not specification limits. The variation of strength and modulus is included in the design reliability considered through the model error residuals. Do not reduce strength below the mean regularly achieved in the field. A mean 28-day modulus of rupture of 723 psi is used as recommended for Level 3.

Table E-11

PCC Strength Properties Inputs for New JPCP Design Example

PCC strength and modulus (Input Level: 3)	
28-Day PCC modulus of rupture (psi)	723.0
28-Day PCC elastic modulus (psi)	-

Base (HMA) and Subbase (UTBC) Layers

The base and subbase layers consist of a 3-in HMA dense graded layer over a 6-in UTBC corresponding to an A-1-a AASHTO classification. Inputs for the HMA base layer are given in Table E-12 based on recommendations in Section 14.3.

The resilient moduli of the UTBC were selected based on recommendations in Section 14.6 Resilient Modulus Table (25,000 psi for UTBC or A-1-a materials given in Table E-13.

Table E-12

HMA Base Course
Properties

Layer 2 Flexible : Default asphalt concrete

Asphalt		
Thickness (in.)	3.0	
Unit weight (pcf)	150.0	
Poisson's ratio	Is Calculated?	False
	Ratio	0.35
	Parameter A	-
	Parameter B	-

Asphalt Dynamic Modulus (Input Level: 3)

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	77
No.4 sieve	60
No.200 sieve	6

Asphalt Binder

Parameter	Value
Grade	Superpave Performance Grade
Binder Type	64-22
A	10.98
VTS	-3.68

General Info

Name	Value
Reference temperature (ºF)	70
Effective binder content (%)	11.6
Air voids (%)	7
Thermal conductivity (BTU/hr-ft-ºF)	0.67
Heat capacity (BTU/lb-ºF)	0.23

Identifiers

Field	Value
Display name/identifier	Default asphalt concrete
Description of object	
Author	
Date Created	10/29/2010 11:00:00 PM
Approver	
Date approved	10/29/2010 11:00:00 PM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Table E-13

Untreated Base Course (UTBC) Properties for New JPCP Design Example

Layer 3 Sandwich/Fractured : Sandwich Granular

Chemically Stabilized	
Layer thickness (in.)	6
Poisson's ratio	0.2
Unit weight (pcf)	145

Strength	
Elastic/resilient modulus (psi)	25000

Thermal	
Heat capacity (BTU/lb-°F).	0.28
Thermal conductivity (BTU/hr-ft-°F)	1.25

Identifiers

Field	Value
Display name/identifier	Sandwich Granular
Description of object	Default
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Stabilized Subgrade Layer

The subgrade is classified as an A-4 silty soil and as described in the next section on subgrade characterization, it is a very soft soil. Designs prepared without stabilizing the subgrade were not practical solutions due to the increased IRI or roughness over the 40 year design period. Therefore, it was decided to stabilize the top 8-in of the subgrade. Lab tests are required to establish the best a proper amount of stabilizer. It is assumed that the stabilized material will have a minimum resilient modulus of 60,000 psi over time. The properties of this stabilized layer are summarized in Table E-14.

Note that the structure formed with the chemically stabilized soil is called a “sandwich” layer in that the UTBC is sandwiched between the stabilized soil and the HMA base. This type of design must be entered as a “Sandwich” layer and the Mr value assigned to the UTBC will remain constant over the design period since the subdrainage model will not apply to this layer.

Table E-14

Inputs for Chemically Stabilized Subgrade Layer

Layer 4 Chemically Stabilized : Lime stabilized	
Chemically Stabilized	
Layer thickness (in.)	8
Poisson's ratio	0.2
Unit weight (pcf)	125
Strength	
Elastic/resilient modulus (psi)	60000
Thermal	
Heat capacity (BTU/lb-°F).	0.28
Thermal conductivity (BTU/hr-ft-°F)	1.25

Identifiers	
Field	**Value**
Display name/identifier	Lime stabilized
Description of object	Default material
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Subgrade Soil Layer

The subgrade AASHTO classification and the laboratory resilient modulus (M_r) at optimum moisture content is the required input for the ME Design. This input along with the modulus of each layer is used in the program to calculate the stresses and deflections, incremental fatigue damage, joint load transfer efficiency, and incremental cracking and joint faulting and finally IRI.

Information obtained from the county soil report (USDA-NRCS soil survey database) indicated an A-4 silty soil classification along much of this project. Data from the suggested gradation and dry unit weight were entered into the ME Design window as shown in Table E-15.

ME Design requires an input subgrade resilient modulus at optimum moisture and density. There are three ways to estimate this subgrade input resilient modulus:

1. **Level 2 Backcalculation Modulus (FWD Testing):** The recommended Level 2 approach is to measure deflections on top of the existing pavement using an FWD and convert that into an input subgrade resilient modulus. A deflection profile along a portion of the old existing JPCP project at center slab was obtained for a representative 500-segment of this project. The FWD loading ranged from 9,000 to 16,000 lbs. (a constant 9,000 lbs. is adequate) and a deflection basin was measured on the slab surface. The in situ subgrade dynamic k-value was backcalculated at all FWD testing points along the project using the procedure in AASHTO 1993 (Part III). The three equations for calculation of the subgrade K-value are as follows (AREA, Radius of Relative Stiffness, and K-Value):

AREA(5 sensors) = 3+6*D18/D12+9*D24/d12+18*D36/d12+12*D60/D12

Where: AREA5 = Area calculated for the deflection basis using five sensors.
D12, D18, D24, D36, D60 = Deflection at 12, 18, 24, 36, and 60-in from center of plate, in.

Radius Relative Stiffness = ((LN((48-AREA5)/158.4))/-0.476)^2.22

K-Value = Load/(D12/1000)/RadiusRelStiff^2)*0.12188*EXP(-0.79432*

EXP(-0.07074*RadRelStiff))

Where: Load = Load on the FWD plate, Lbs.
AREA = Deflection basin area based on 5 sensors.
RadRelStiff = Radius of Relative Stiffness, in
K-Value = Backcalculated dynamic subgrade K-Value, psi/in

These equations are programmed into an EXCEL spreadsheet called "Subgrade Mr K UDOT02" which can be used for the backcalculation. An example is provided:

Example for one point on existing JPCP pavement:

D12 = 2.28 (0.001 in)
D18 = 2.20
D24 = 2.13
D36 = 1.93
D60 = 1.61
FWD Load = 9,578 lbs.

AREA5 = 40.86 in^2

Radius of Relative Stiffness = 64.0 in

Dynamic K-value = 124 psi/in

Table E-15

Subgrade A-4 Soil Properties for New JPCP Design Example

Layer 6 Subgrade : A-4

Unbound	
Layer thickness (in.)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5

Modulus (Input Level: 3)

Analysis Type:	Modify input values by temperature/moisture
Method:	Resilient Modulus (psi)

Resilient Modulus (psi)
8000.0

Use Correction factor for NDT modulus?	-
NDT Correction Factor:	-

Identifiers

Field	Value
Display name/identifier	A-4
Description of object	Default material
Author	AASHTO
Date Created	1/1/2011 12:00:00 AM
Approver	
Date approved	1/1/2011 12:00:00 AM
State	
District	
County	
Highway	
Direction of Travel	
From station (miles)	
To station (miles)	
Province	
User defined field 2	
User defined field 3	
Revision Number	0

Sieve

Liquid Limit	21.0
Plasticity Index	5.0
Is layer compacted?	False

	Is User Defined?	Value
Maximum dry unit weight (pcf)	False	121.9
Saturated hydraulic conductivity (ft/hr)	False	4.904e-06
Specific gravity of solids	False	2.7
Optimum gravimetric water content (%)	False	10.1

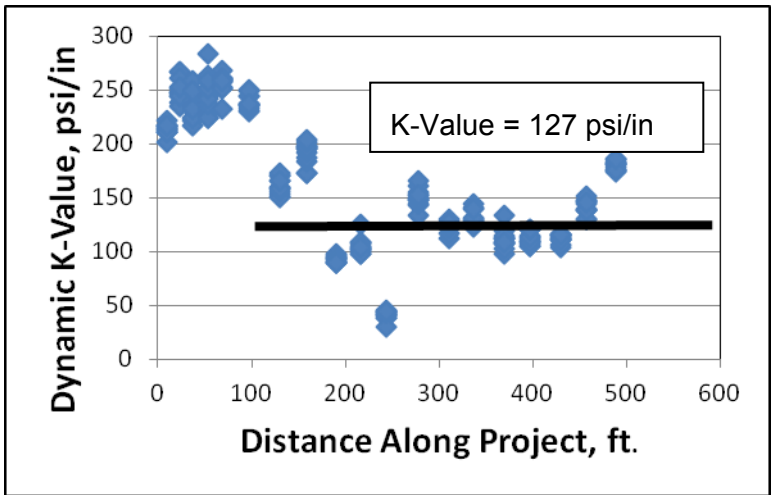
User-defined Soil Water Characteristic Curve (SWCC)	
Is User Defined?	False
af	47.4983
bf	1.1361
cf	0.6143
hr	460.0000

Sieve Size	% Passing
0.001mm	
0.002mm	
0.020mm	
#200	36.0
#100	
#80	73.9
#60	
#50	
#40	82.7
#30	
#20	
#16	
#10	89.9
#8	
#4	93.0
3/8-in.	95.6
1/2-in.	96.7
3/4-in.	98.0
1-in.	98.7
1 1/2-in.	99.4
2-in.	99.6
2 1/2-in.	
3-in.	
3 1/2-in.	99.8

A plot of a representative 500-ft section of the project is shown in Table E-16. The deflection profile shows that most of the length has a very soft and poor subgrade modulus with an average dynamic K-Value = 127 psi/in. This value will be used in design of the project since it represents a significant proportion of project length.

Table E-16

Backcalculated Dynamic k-value Using Data From
FWD Center Slab Loading on the existing JPCP project October 1999.



For new or reconstruction, the subgrade K-Value cannot be entered directly into the ME Design program, only the subgrade resilient modulus, Mr. Thus, the ME Design software must be used iteratively to obtain the proper input Mr of the subgrade. The program was first run with the default resilient modulus from MOI Section 14.6 for the A-4 soil (15,000 psi) and the output k-value in October (when FWD was tested) obtained from the output. If the output k-value is significantly different than 127 psi/in for October when FWD testing was performed, then the input resilient modulus is adjusted up or down until the output k-value is approximately 127 psi/in. Iterations were performed and it was determined that an input Mr of 8,000 psi resulted in a subgrade dynamic k-value close to 127 psi/in in October conditions. The Mr = 8,000 psi was then used in the subgrade design input for the reconstructed JPCP. This is a very low subgrade Mr value indicating significant subgrade problems could develop.

2. **CBR Correlation with Mr:** The MEPDG provides a correlation between CBR and Mr for subgrade soils (CBR < 30). This approach is explained in the MOI. No CBR was available for this project.

3. **Level 3 Subgrade Soil Classification:** Utah verified subgrade resilient modulus default values as summarized in Table E-14. The A-4 soil classification default design modulus is 15,000 psi from Table E-14. This value is nearly twice higher than the 8,000 psi obtained from FWD backcalculation. This project shows the value in FWD testing of a project in that the actual soil support may be very

different than the average soil.

Given the very low value of subgrade Mr and K-value, it is recommended that the subgrade be appropriately stabilized. In this case, an A-4 silty soil would most likely require cement to increase the modulus from 8,000 psi to about 60,000 psi. A layer of about 8-in is needed to provide improved uniform and lasting support for the pavement.

Run ME Design to Predict Material Properties And Performance Over The Design Period

The initial trial design section is now developed. Given the poor subgrade and heavy truck traffic, it will be difficult to obtain a passing design with an untreated base course. Thus, an HMA base is proposed along with subgrade stabilization. The initial trial design is as follows:

- 12-in JPCP slab, 15-ft joint spacing, tied PCC shoulder, 1.5-in diameter dowels.
- 3-in HMA base to provide good stability, minimize erosion and faulting, and provide high friction/bonding with the slab which results in a reduction in slab thickness.
- 6-in UTBC unbound (sub)base course.
- 8-in Stabilized (probably with cement) subgrade layer (60,000 psi).
- A-4 Silty very soft subgrade.

Note that the program requires two identical subgrade layers below the stabilized subgrade to run the drainage model.

After all inputs have been obtained and entered the ME Design software can begin the analysis and design process. All design inputs need to be colored green as show in Table E-17. Appropriately name the design file and click on *Save* and then *Run*. The program runs the Traffic, Climate, Materials Modulus, Load Transfer Efficiency, Joint Faulting, Slab Cracking, and IRI modules and reports the analysis status on the upper right hand corner of the screen.

The program creates a PDF summary file and also an MS Excel file with more detailed information. Both tabular and graphical information are obtained. The detailed summary file is in a MS Excel format which portions can be copied and pasted into a design report. The PDF and Excel files contain a summary of all inputs as well as outputs such as climate, reliability, distress and IRI, joint load transfer efficiency (LTE), joint faulting, slab transverse cracking, and IRI. Distress and IRI outputs are provided for both mean and specified reliability level in tabular and graphical formats.

Table E-17

ME Design Program Example Layout Screen After Completing All Inputs.

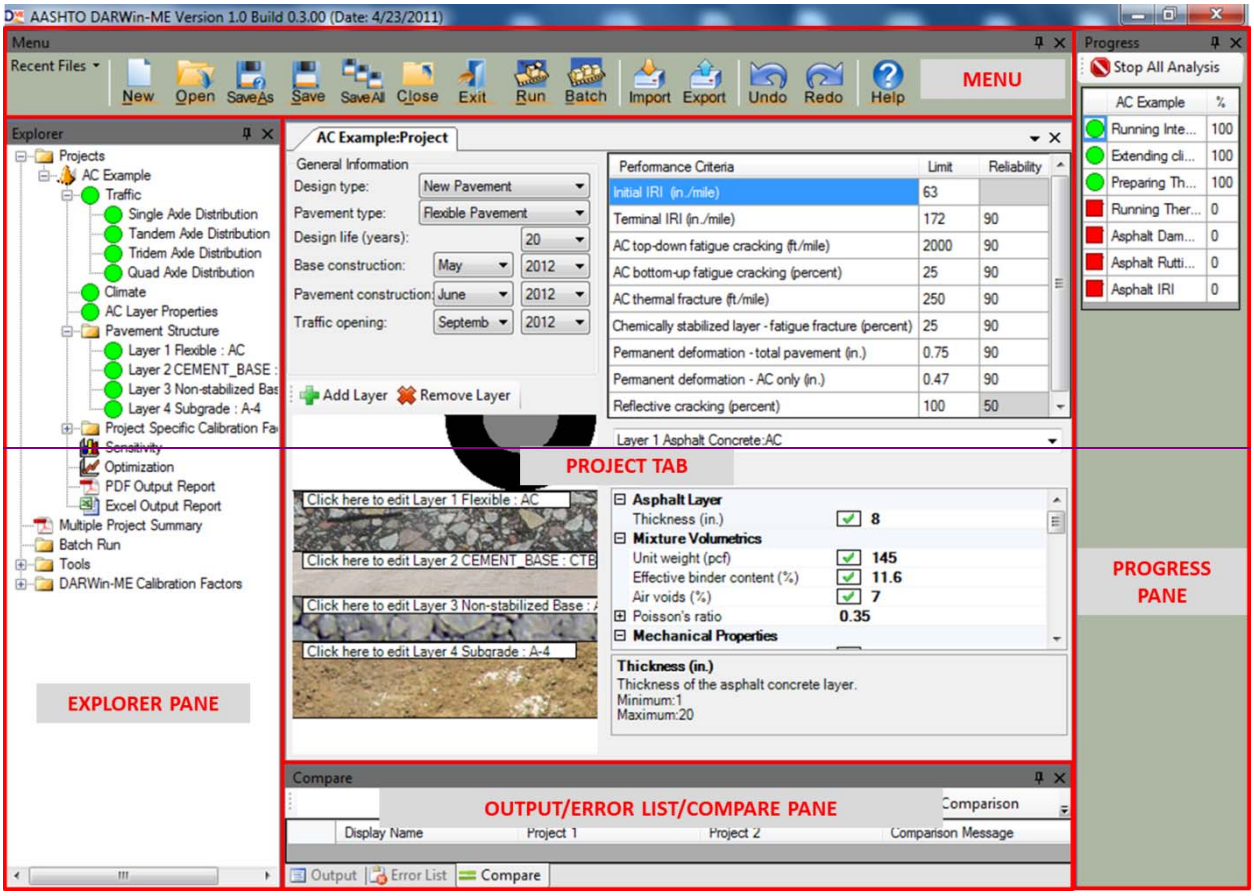


Table E-18 for example, shows some JPCP properties monthly over the 40 year design life. During the first month of traffic, September, the modulus of elasticity of the PCC slab was 4.66 million psi, the PCC modulus of rupture was 771 psi, the dynamic E* modulus of the HMA base was 594,460 psi, the dynamic subgrade k-value was 164 psi/in. Designers should review these data to see if they all look reasonable as these do over the 40 year period.

Table E-18

Output Tab: PCC Strength and Modulus, HMA Base E*, and Subgrade K-Value Outputs During First Year.

Predicted, * PCC Modulus * PCC Flexural Strength * Unbound base/Subbase Modulus * Unbound Material Dynamic-k Value

Month	Pavement Age (years)	Heavy Trucks (cumulative)	PCC Modulus (E) (Mpsi)	PCC Strength (MR) (psi)	Base Modulus(E) (Kpsi)	Dynamic k (psi/in)
9/2014	0.08	56,569	4.66	771.00	594.46	164
10/2014	0.17	114,554	4.69	779.00	984.25	164
11/2014	0.25	165,967	4.71	786.00	1561.98	163
12/2014	0.33	213,362	4.74	791.00	2248.01	162
1/2015	0.42	260,185	4.76	795.00	2339.03	162
2/2015	0.50	310,837	4.77	799.00	2195.1	162
3/2015	0.58	361,344	4.79	802.00	1687.13	162
4/2015	0.67	416,623	4.80	805.00	1204.93	163
5/2015	0.75	477,254	4.81	808.00	821.82	164
6/2015	0.83	537,401	4.83	810.00	524.51	165
7/2015	0.92	597,817	4.84	812.00	366.42	165
8/2015	1.00	657,273	4.85	814.00	416.56	165

The Distress Summary sheet in the output file provides a month by month overall summary of the JPCP trial design for the project including critical material properties, traffic, and distress data. Detailed data for each distress type is provided on separate sheets. The distress summary sheet shown in Table E-19 indicates that this pavement carried 40.6 million heavy trucks in the design lane over the design period.

The first month showed 56,569 trucks and this provides an overall idea of the traffic loading on the pavement. Over 40 years, about 40 million trucks will use this traffic lane. The first month shows the IRI to be 60 in/mile which was assumed to be the initial value after construction. Other distresses were zero. Then month by month, the accumulation of damage and distress occurs until the end of the 40 year design life.

The yellow cells show the predictions at 50 percent (or mean) and the turquois colored cells show the prediction at 95 percent reliability.

Table E-19

Summary output distress for the JPCP example trial design showing first and last year of 40 year design period.

Predicted Distress								
			Mean Predicted Distress			Predicted Distress @ Reliability		
Month	Pavement Age (years)	Heavy Trucks (cum.)	IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent slabs)	IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent slabs)
9/2014	0.08	56,569	60.1	0.000	0.00	86.8	0.014	2.47
10/2014	0.17	114,554	60.2	0.000	0.00	87.3	0.016	2.47
11/2014	0.25	165,967	60.4	0.000	0.00	88.0	0.018	2.47
12/2014	0.33	213,362	60.4	0.000	0.00	88.1	0.018	2.47
1/2015	0.42	260,185	60.6	0.001	0.00	88.4	0.019	2.47
2/2015	0.50	310,837	60.7	0.001	0.00	88.7	0.020	2.47
3/2015	0.58	361,344	60.8	0.001	0.00	89.0	0.021	2.47
4/2015	0.67	416,623	60.9	0.001	0.00	89.3	0.021	2.47
5/2015	0.75	477,254	61.1	0.001	0.00	89.8	0.022	2.47
6/2015	0.83	537,401	61.2	0.001	0.00	90.1	0.023	2.47
7/2015	0.92	597,817	61.3	0.001	0.00	90.4	0.023	2.47
8/2015	1.00	657,273	61.4	0.001	0.00	90.6	0.024	2.47

Last Year of Design Period

9/2053	39.08	39,389,100	127.7	0.067	0.01	197.3	0.130	3.84
10/2053	39.17	39,510,400	127.9	0.068	0.01	197.5	0.130	3.84
11/2053	39.25	39,617,900	128.0	0.068	0.01	197.8	0.131	3.84
12/2053	39.33	39,717,100	128.2	0.068	0.01	197.9	0.131	3.84
1/2054	39.42	39,815,100	128.3	0.068	0.01	198.1	0.131	3.84
2/2054	39.50	39,921,000	128.4	0.068	0.01	198.2	0.131	3.84
3/2054	39.58	40,026,700	128.5	0.068	0.01	198.4	0.131	3.84
4/2054	39.67	40,142,300	128.7	0.069	0.01	198.7	0.132	3.84
5/2054	39.75	40,269,200	128.8	0.069	0.01	198.8	0.132	3.84
6/2054	39.83	40,395,000	128.9	0.069	0.01	199.0	0.132	3.84
7/2054	39.92	40,521,400	129.0	0.069	0.01	199.1	0.132	3.84
8/2054	40.00	40,645,800	129.2	0.069	0.01	199.3	0.132	3.84

The Distress Prediction Summary of the ME Design output shows the key Reliability information as presented in Figure E-20. This table shows whether or not the trial design passes all performance criteria at the specified level of reliability. For this case, cracking and faulting passed 95 percent but IRI failed at 27 years with about 83 percent reliability. The MOI provides for this situation in that diamond grinding is typically planned for 20 years or more and the IRI passes at 25 years. Thus, the pavement design is acceptable structurally (e.g., fatigue cracking and joint faulting) but may need a retexturing after 25 or more years. A diamond grinding would remove any roughness and lower the IRI to a value near its initial construction value.

Plots showing faulting, cracking, and IRI are given in the ME Design output and are shown in Tables E-21, E-22, and E-23. The blue (top) curve is the 95 percentile prediction. The bottom black curve is the mean or 50 percentile prediction. The red line is the design criteria at the specified reliability level of 95 percent. All of the reliability

curves are below the critical line except IRI as previously discussed.

Table E-20

Summary of Output Design Reliability for Example JPCP Design.

Distress Prediction Summary					
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	170.00	199.30	95.00	83.09	Fail
Mean joint faulting (in.)	0.15	0.13	95.00	98.26	Pass
JPCP transverse cracking (percent slabs)	10.00	3.84	95.00	100.00	Pass

Table E-21

Plot of Predicted Faulting Versus Pavement Age.

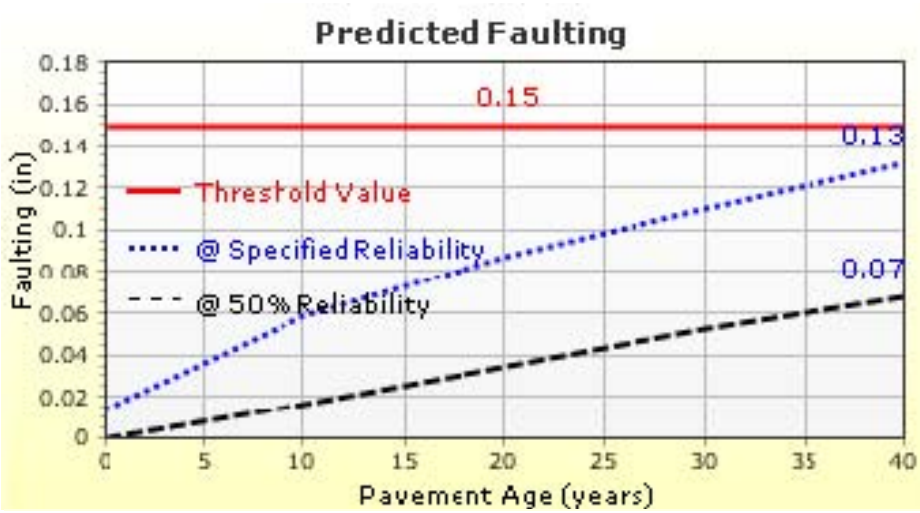


Table E-22

Plot of Predicted Transverse Cracking Versus Pavement Age

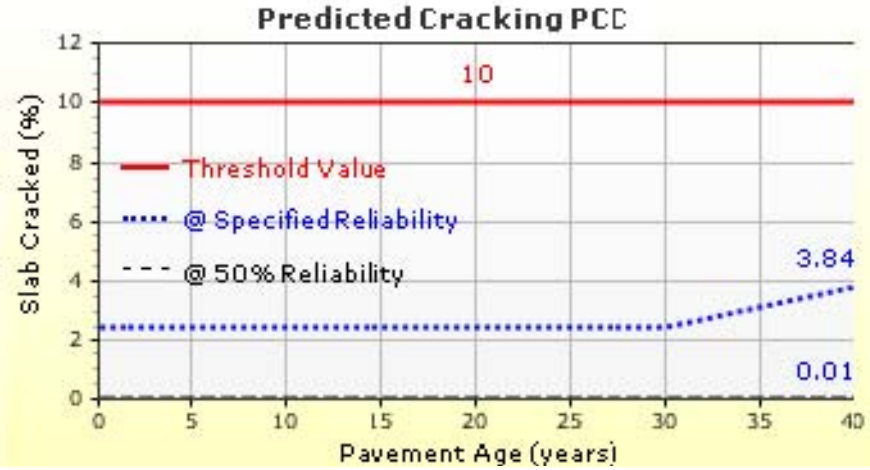
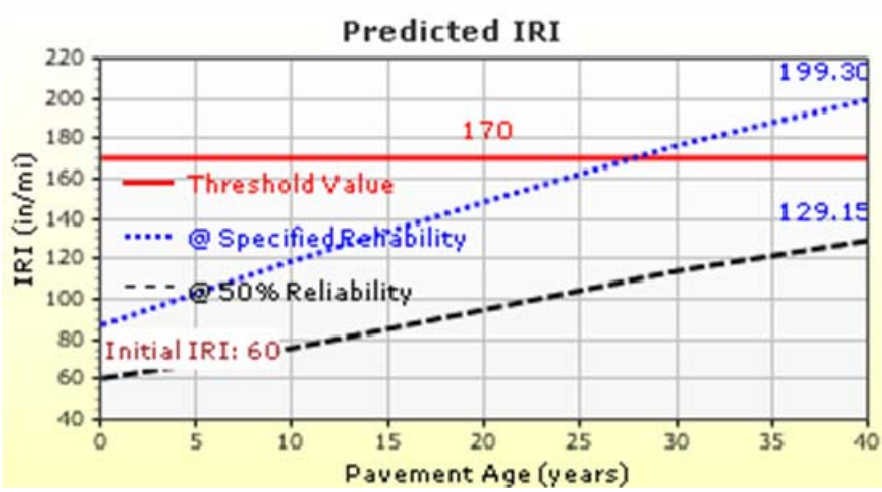


Table E-23

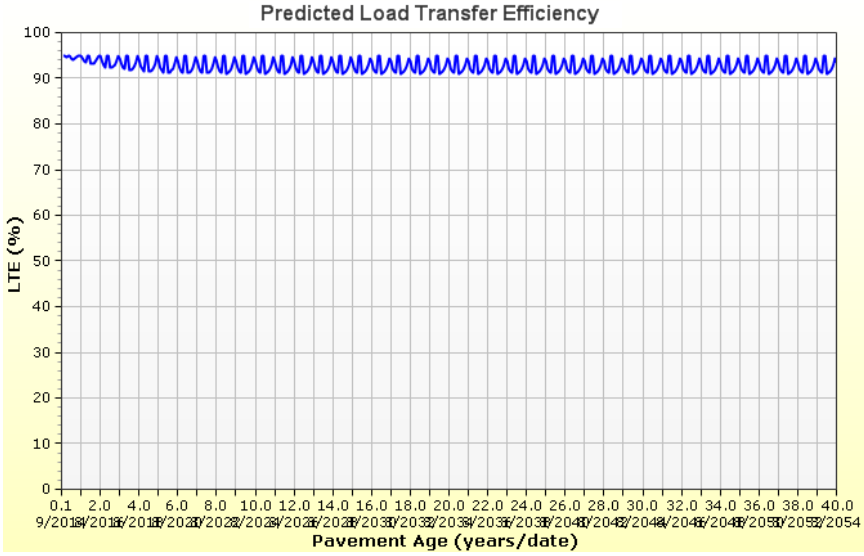
Plot of Predicted IRI Versus Pavement Age



The Table E-23 shows the last remaining data of interest which is the transverse joint load transfer over the 40 year design life. There the value remains above 90 percent throughout the entire period as shown. This means that there is no significant loosening of the dowels in the holes due to repeated load bearing stresses. This means that the joint cannot fault significantly.

Table E-24

Prediction of transverse joint load transfer efficiency over 40 year design life.



Design Optimization

The ME Design software has the capability to iterate on slab thickness to achieve a thickness that meets the design criteria at the required level of reliability. The Optimization routine requires the designer to specify the minimum and maximum thickness of the slab and the program iterates between these two limits until it finds the thinnest slab that meets the criteria.

The only critical aspect of this process is that dowel diameter required to control joint faulting must be keyed to the slab thickness. The software can do this automatically by adding the following rules in the software Optimization input screen.

Use	Property	Rule	Criteria
TRUE	Dowel Diameter (in.)	1	[THICK]>=8
TRUE	Dowel Diameter (in.)	1.125	[THICK]>=9
TRUE	Dowel Diameter (in.)	1.25	[THICK]>=10
TRUE	Dowel Diameter (in.)	1.375	[THICK]>=11
TRUE	Dowel Diameter (in.)	1.5	[THICK]>=12
TRUE	Dowel Diameter (in.)	1.625	[THICK]>=13
TRUE	Dowel Diameter (in.)	1.75	[THICK]>=14

Therefore, as the program iterates on slab thickness, a slab of 12 in would be analyzed with a 1.5 inch dowel bar.

Modifying the Trial Design

The user may have to modify the trial design if any of the performance criteria are not met. They are all met for this trial design. The following suggestions are provided for reducing slab cracking, joint faulting, and smoothness IRI if they are critical.

The following actions can be taken in a revised trial design if, for example, cracking did not pass the criteria at the desired reliability level:

1. Increase slab thickness
2. Decrease joint spacing
3. Provide a stabilized base course
4. Provide a tied concrete shoulder
5. Ensure that the slab has full friction with the base course over the entire design life (this occurs naturally for HMA and UTBC but not for CTB/LCB

The following actions can be taken in a revised trial design if joint faulting did not pass the criteria at the desired reliability level:

1. Increase the diameter of the dowel bar across the transverse joint.
2. Provide a stabilized base course with low erosion factor.

3. Provide a tied shoulder.
4. Do not increase slab thickness to reduce faulting, use other more effective means suggested above including increased diameter of dowel bars.

The following actions can be taken in a revised trial design if IRI did not pass the criteria at the desired reliability level:

1. Take any actions above to reduce slab cracking and joint faulting
2. Reduce the initial IRI (as constructed IRI) by adjusting smoothness specifications.
3. If the subgrade has a high passing #200 sieve content, stabilized the top few inches of the subgrade and input a reduced percent passing.

A design that passes all of the criteria can be considered a feasible alternative that can be subjected to a cost analysis and compared to other alternatives.

APPENDIX F (Must be rerun with new software and calibration)

Utah Thin PCC Overlay of HMA Design Example

Thin PCC Overlay (6 inch) of HMA Project Design using MEPDG

This project is being designed as a thin (6 inch) PCC overlay over existing HMA for a section of Utah State Route 6 south of Price, Utah. The existing HMA will be milled to remove rutting and oxidized surfacing and a new thin PCC overlay placed.

Design Life

The concept of this thin PCC overlay is designed to apply a typical 6 x 6 x 6 PCC concept to extend existing pavement life rather than provide a specific design life. The 6 x 6 x 6 concept includes a 6 inch thick PCC layer with joints at 6 foot spacings in both directions, placed over a milled HMA surface to provide bonding between the layers. This application is typically expected to last 10 years on a medium volume road. Analysis of thin PCC application should be to determine if the application is appropriate for the road rather than determining the thickness of PCC overlay necessary. Use of a thicker PCC overlay should be analyzed using the typical PCC overlay practices. The PCC overlay will be placed in September, 2011, and opened to traffic in October, 2011.

Construction Requirements

The pavement is expected to have an initial IRI of approximately 70 inch/mile assuming a good quality construction with incentive ride specifications and both milling and paving operations.

Analysis Parameters

The performance criteria were selected for a primary, non-interstate highway, at 90 percent reliability level.

- 0.20 inch mean joint faulting
- 15 percent transverse slab cracking
- IRI of 223 inch/mile

Traffic

WIM and ATR data were obtained from UDOT for sites that were representative of the project site and used to develop project specific traffic inputs as follows:

- Initial two-way average annual daily truck traffic (AADTT) is 1,685.
- Volume adjustment factors.
  - Directional distribution: 49.5 percent trucks in each direction.
  - Lane distribution: 100 percent trucks in design lane (1 lane in design direction).
  - Operational speed: 65 mph

- The following inputs were based on UDOT measured traffic data:
  - Hourly truck distribution
  - Vehicle class distribution
  - Axle load distribution
- Truck traffic growth is projected using a linear or compound model. Truck traffic has grown from 1 to over 10 percent on Utah highways over the years. A compound growth rate of 3 percent was used for this project based on the 10 year design life.
- General traffic inputs. These consist of lateral truck/wheel wander and number of axles per truck, axle configuration, and wheel base.
  - Lateral truck/wheel wander – three inputs are required here:
    1. Mean wheel location or the distance from outer edge of truck wheel to lane marking (paint stripe: 18 inch standard used in calibration
    2. Standard deviation of lateral truck wander: 10 inch standard used in calibration
    3. Design lane width: This distance is paint stripe to paint stripe. This is 12 ft.

- Axles per truck: Mean number of axles per truck/vehicle class.
- Axle configuration: Axle width, spacing, and pressure.
  - Actual axle width (edge to edge of tire) outside dimensions: 8.5 ft, typical
  - Dual tire spacing: 12 inch (typical used in calibration). Wide tires can only be considered in the Special Traffic Analysis routine in the MEPDG for HMA pavements.
  - Tire pressure: 120 psi, hot rolling pressure used in calibration.

Climate Inputs

The project site is in the vicinity of Emery, Utah. The latitude and longitude of this site is as follows, obtained from various sources such as GPS units or Google Earth:

- Latitude: 38.55 degrees.minutes
- Longitude: -111.15 degrees.minutes

The estimated depth of water table is in excess of 30 ft based on well log information from the Utah Division of Water Rights.

The selected weather station for this example is Price, Utah (10 miles) to create a virtual weather station.

Structure and Layer Materials Definition

The typical set of structural and materials inputs associated with a thin PCC overlay are now entered creating the design that will be evaluated for its performance. Based on the typical 6 x 6 x 6 rehabilitation approach, the following inputs should be used

Joint Design	
Joint spacing (ft):	6
Sealant type:	None
Dowel diameter (in):	None
Dowel bar spacing (in):	None
Edge Support	None
Long-term LTE(%):	n/a
Widened Slab (ft):	n/a
Base Properties	
Base type:	Asphalt treated
Erodibility index:	Extremely Resistant (1)
PCC-Base Interface	Full friction contact
Loss of full friction (age in months):	360

A thin PCC overlay is only effective if it is bonded to the surface below. Using an unbounded thin PCC overlay will typically result in significant cracking failure early within the pavement's life.

Surface Shortwave Absorptivity

Use the calibration standard of 0.85. This controls the flow of heat through the surface.

Layer Materials Properties

Layer 1 - PCC (Surface Layer)

The following inputs required for the PCC surface layer are entered in the appropriate MEPDG input screens. The mix properties and strength properties are based on a typical UDOT specified AA(AE) mix used for paving purposes. The mix design for a thin PCC overlay will typical include fibers as the joints are intended to be undoweled and unsealed and the fibers will aid in keeping the joints tighter.

The CTE value is a critical one for cracking and should be based on information on local aggregates. This information can be obtained from the UDOT Central Materials Division.

General Properties

PCC material	JPCP
Layer thickness (in):	6
Unit weight (pcf):	150
Poisson's ratio	0.2

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	5.5
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	620
Water/cement ratio:	0.42
Aggregate type:	Granite
PCC zero-stress temperature (F°)	Derived
Ultimate shrinkage at 40% R.H (microstrain)	Derived
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35
Curing method:	Curing compound

Strength Properties

Input level:	Level 3
28-day PCC modulus of rupture (psi):	720
28-day PCC compressive strength (psi):	n/a

Layer 2 – Existing HMA

The existing HMA pavement surface needs to be in good condition to allow for good performance of a thin PCC overlay section. Inputs pertaining to this layer should be representative of the in-situ pavement and not the original construction criteria. This is primarily applicable to the default binder properties. In this example, the original PG 64-28 was bumped to a PG 70-22 to account for the aging and oxidation of the pavement. Layer thicknesses less than 4 inches or in “fair” or “poor” condition will likely result in failing designs.

The As-Built Volumetric properties are based on the original construction information, with the binder bumped.

Material type:	Asphalt concrete (existing)
Layer thickness (in):	6

General Properties

General

Reference temperature (F°):	70

Volumetric Properties as Built

Effective binder content (%):	11
Air voids (%):	6.5
Total unit weight (pcf):	145

Poisson's ratio:	0.35 (user entered)

Thermal Properties

Thermal conductivity asphalt (BTU/hr-ft-F°):	0.67
Heat capacity asphalt (BTU/lb-F°):	0.23

Asphalt Mix

Cumulative % Retained 3/4 inch sieve:	0
Cumulative % Retained 3/8 inch sieve:	15
Cumulative % Retained #4 sieve:	40
% Passing #200 sieve:	8

Layer 3 - Base and Subbase (Granular Material) Layers

Base course properties are based on the original construction or field investigation data.

Unbound Material:	A-1-a
Thickness(in):	20

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	38000

Layer 4 - Subgrade (Soil) Layer

The subgrade AASHTO classification and the laboratory resilient modulus (M_r) at **optimum moisture content and density** is the required input for the MEPDG. It is important to note that this input for M_r is at optimum moisture because the value is higher than it is wet of optimum (which is the proper input for the 1993 AASHTO Guide).

Unbound Material:	A-7-6
Thickness(in):	Semi-infinite

Strength Properties

Input Level:	Level 3
Analysis Type:	Representative value (User Input Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	6000
Moisture Content(%):	-9999

A more accurate subgrade modulus can be obtained if the project can be tested with the FWD. The input subgrade resilient modulus can be estimated as follows:

Evaluate the 6 x 6 x 6 application for a 10 year Design

All MEPDG inputs have been obtained and the MEPDG can be run with varying HMA overlay thicknesses. The performance criteria previously selected were used to establish if the design Passed or Failed. Thickness of 6 inches was run and the following results obtained.

- IRI Failed to achieve 90 percent reliability
- Mean Joint Faulting Passed
- Percent Cracked Slabs Passed

In this case, the IRI for the pavement is expected to be high, but the structural integrity of the road is intact. Some engineering judgment is likely required on the actual field performance expectation based on actual past projects and their IRI performance as this is a pavement system that has not been used extensively in the calibration of the MEPDG.

Project: 6 by 6 by 6 trial.dgp

Reliability Summary

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	223	90	259	33.45	Fail
Transverse Cracking (% slabs cracked)	15	90	0	99.999	Pass
Mean Joint Faulting (in)	0.2	90	0.139	90.37	Pass

APPENDIX G (Must be rerun with new software and calibration)

Utah HMA Overlay of Fractured PCC Design Example

Fractured PCC and HMA Overlay Project Design

This project is being designed as a fractured PCC and HMA overlay for a section of Utah I-15 near Nephi, Juab County, Utah, south of Salt Lake City. The existing JPCP will be fractured (rubblized for this example) and a new HMA overlay placed.

Design Life

The HMA overlay has a 20 year traffic design life. The JPCP will be fractured in June 2011, the HMA placed in July 2011, and opened to traffic in August 2011.

Construction Requirements

The pavement is expected to have an initial IRI of approximately 70 inch/mile assuming a good quality construction with incentive ride specifications.

Analysis Parameters

The performance criteria were selected for an Interstate highway.

- 10 percent alligator cracking at 95 percent reliability level.
- 0.50 inch total rutting (mean of inner and outer wheelpath) at a reliability level of 95 percent at the end of the 20 year design life.
- 905 ft/mile or 70 ft average spacing transverse low temperature cracking.
- IRI of less than 170 inch/mile at a reliability level of 95 percent.

Traffic

WIM and ATR data were obtained for sites that were representative of the project site and used to develop project specific traffic inputs as follows:

- Initial two-way average annual daily traffic (AADT) is 20,000. Percent trucks is 25 percent.
- Volume adjustment factors.
  - o Directional distribution: 50 percent trucks in each direction.
  - o Lane distribution: 90 percent trucks in outer design lane (2 lanes in design direction).
  - o Operational speed: 60 mph
- Monthly volume adjustments are typically truck volume differences throughout the year. Monthly variations in truck traffic were measured and are included.
- Hourly truck distribution: The hourly truck distribution is only used for concrete pavements and not for asphalt pavements.
- Vehicle class distribution is the percent of each vehicle class in the traffic stream is an important input. Vehicle class distribution for this project was

measured at an ATR site near the project. This distribution is unusual in that the Class 5 single unit truck is the most common type of vehicle on this rural highway.

- Truck traffic growth is projected using a linear or compound model. Truck traffic has grown from 1 to over 10 percent on Utah highways over the years. A compound growth rate of 4 percent was used for this project after plotting past truck growth over time.
- Axle load distribution. The axle load distribution is the most important traffic input. Damage is caused by the heavy single, tandem, tridem, and quad axle loads. The distributions were obtained from WIM equipment that is representative of this highway. The highest loads in these distributions appear to cause the majority of fatigue damage and permanent deformation to HMA pavement.
- General traffic inputs. These consist of lateral truck/wheel wander and number of axles per truck, axle configuration, and wheel base.
    - Lateral truck/wheel wander – three inputs are required here:
        1. Mean wheel location or the distance from outer edge of truck wheel to lane marking (paint stripe: 18 inch standard used in calibration
        2. Standard deviation of lateral truck wander: 10 inch standard used in calibration
        3. Design lane width: This distance is paint stripe to paint stripe. This is 12 ft.
- Axles per truck: Mean number of axles per truck/vehicle class.

- Axle configuration: Axle width, spacing, and pressure.
    - Actual axle width (edge to edge of tire) outside dimensions: 8.5 ft, typical
    - Dual tire spacing: 12 inch (typical used in calibration). Wide tires can only be considered in the Special Traffic Analysis routine in the MEPDG for HMA pavements.
    - Tire pressure: 120 psi, hot rolling pressure used in calibration.

Climate Inputs

The project site is in the vicinity of Lake Point, Utah, close to the southern shore of the Great Salt Lake. The latitude and longitude of this site is as follows, obtained from various sources such as GPS units or Google Earth:

- Latitude: 38.26 degrees.minutes
- Longitude: -111.8 degrees.minutes

The designer enters the latitude and longitude and elevation into the MEPDG and uses the Interpolate Climate Data for Given Location button.

The estimated depth of water table, in this case 20 ft must also be entered before generating a climatic file for the project (Table D-6). The Utah Division of Water Rights has a well drilling database and geologic well logs available at

http://www.waterrights.utah.gov/wellinfo/default.asp
that provide information on water depth or elevation observed in wells throughout the State. This resource, along with project geotechnical reports or the USDA-NRCS soil survey database can also be used for estimating depth of water table for a pavement project site.

The selected weather stations for this example is Salt Lake City International (54 miles), Price (73 miles) and Milford (122 miles) to create a virtual weather station.

HMA Design Properties

The HMA dynamic modulus (E*) can be predicted by two different methods. It is recommended to always use the NCHRP 1-37A Viscosity based model which was nationally calibrated.

Always check the HMA Rutting Model Coefficients button. The local calibration in Utah established that the rutting model was biased or over predicted and new State calibration coefficients were established. These new values are entered into the Tools pull down bar. Be sure to verify that the local calibration factors are used for each run by checking the MEPDG output Excel file "Inputs Summary" tab.

Illustration of Utah Tools Tabs
for Providing Local Calibration Factors

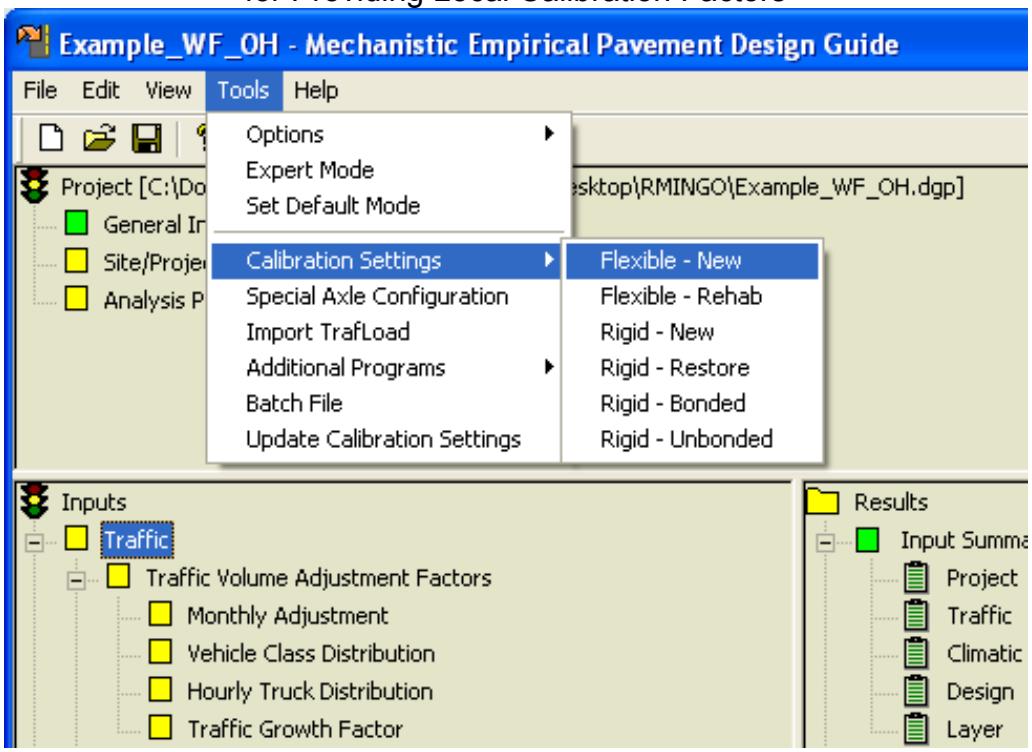
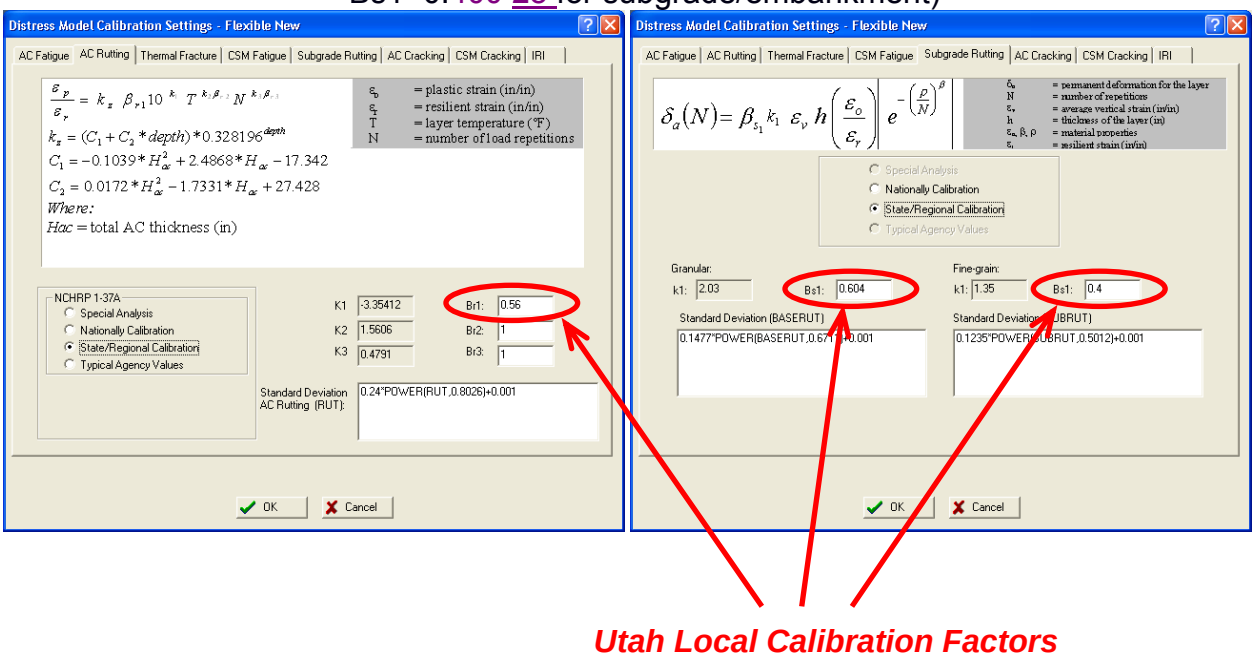


Illustration of Utah Rutting Model Local Calibration Factors in AC Rutting and Subgrade Rutting tabs (Br1=0.~~560~~ 58 for HMA, Granular Bs1=0.~~604~~ 71 for UTBC, and Fine-grain Bs1=0.~~400~~ 28 for subgrade/embankment)



Utah Local Calibration Factors

Structure and Layer Materials Definition

A set of structural and materials inputs are now selected forming the trial design that will be evaluated for its performance. The procedure is an iterative procedure and the user will have to develop a trial design and make several modifications to it before a feasible and economic or final design is achieved. The trial design can be obtained using another design procedure such as the AASHTO 1993 or an alternative of interest.

The thickness of HMA overlay over the 10 in rubblized JPCP was varied from 5 to 7 inches to obtain a design that passed all of the distress and IRI criteria. Note that the column labeled Interface has a 1, indicated full friction (see Table D-7).

- 5 to 7 inch HMA layer (total thickness of various HMA layers)
- 10 inch Rubblized PCC slab
- 4 inch Lean Concrete baseslip-formed lean concrete
- 3.2 inch GB (unbound granular borrow, A-1-a)
- Semi-infinite uncompacted natural subgrade layer A-4 soil

<u>Surface Shortwave Absorptivity HMA Surface</u>

Use the calibration standard of 0.85. This controls the flow of heat through the HMA. This value has been found to provide accurate temperature measurements through the HMA after it ages and the color turns gray.

Layer Materials Properties

HMA (Surface Layer)

Inputs required for the HMA layer are entered in the appropriate MEPDG input screens.

- Note that the effective binder content is specifically for as-built volumetric conditions thus much higher than the gravimetric percentage normally used in practice – 13 percent was used.
- The air voids are in situ or as-built and are higher than the mix design air voids. Note that normally a thick layer will be divided into several layers and the MEPDG can handle this easily. Only one HMA layer is included for this example – 6.5 percent was used.
- Unit weight was 148 pcf.
- Gradation of the HMA layer was input as follows:
  - Retained on ¾ inch sieve 0 percent
  - Retained on ⅜ inch sieve 24 percent
  - Retained on ¼ inch sieve 52 percent
  - Passing the #200 seive 6 percent
- Binder Grade: PG 64-34

Fractured/Rubblized Concrete Slab

The rubblized concrete slab modulus depends on the degree of fracture. Backcalculation has shown it to vary widely along a project. Over time it typically increases in value as further cementation occurs. Modulus values ranging from 40,000 to 150,000 psi have been backcalculated. The HMA overlay becomes too thin if too high a modulus is chosen. A value of 40,000 psi is selected for this example. A sensitivity analysis could be performed to determine its effect.

Base and Subbase (Granular Material) Layers

The 4 inch lean concrete base (LCB) was good quality material and normally has a modulus of about 2 million psi. The rubblizing of the JPCP above may damage this layer and thus the modulus was reduced to 1 million psi.

The 3.2 inch granular borrow (GB) layer consists of unbound granular materials corresponding to an A-1-a AASHTO classification and a resilient modulus of 35,000 psi. MEPDG default was used for the gradation.

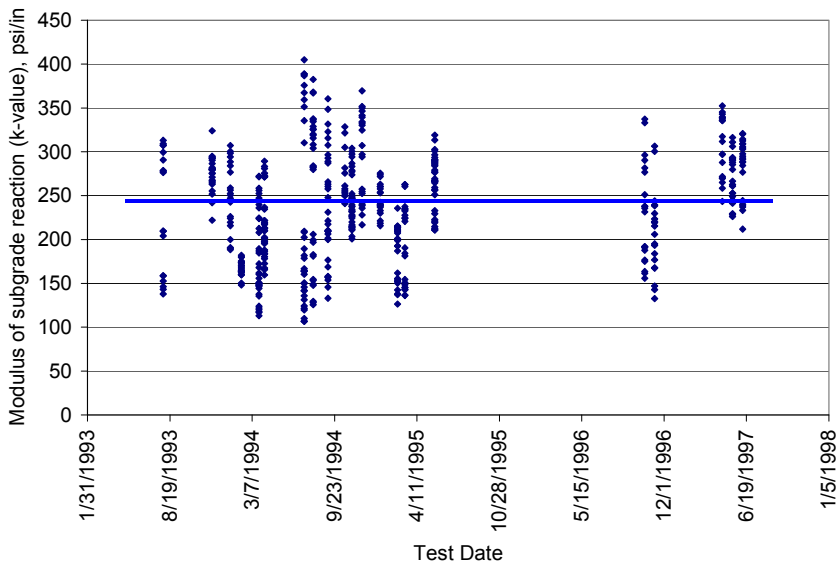
Subgrade (Soil) Layer

The subgrade AASHTO classification and the laboratory resilient modulus (M_r) at **optimum moisture content and density** is the required input for the MEPDG. It is important to note that this input for M_r is at optimum moisture because the value is higher than it is wet of optimum (which is the proper input for the 1993 AASHTO Guide).

Information obtained from the county soil report (USDA-NRCS soil survey database) indicated an A-4 soil classification along much of this project. The MEPDG default resilient modulus at optimum moisture and density is 15,000 psi. Enter tis value into MEPDG if no deflection data is available.

A more accurate subgrade modulus can be obtained if the project can be tested with the FWD. The input subgrade resilient modulus can be estimated as follows:

- Test the JPCP with the FWD along the project before rubblization at slab centers every 500-1000 ft. The section had been tested a several time periods and the dynamic k-value has been backcalculated below to be approximately 250 psi/inch for this example.
-



- A better estimated of the subgrade resilient modulus can be obtained by the following process:
  - Enter the default M_r into the MEPDG and run the program for a couple of years and look at the dynamic k-value output.
  - If this value is approximately 250 psi/inch (the backcalculated value) then 15,000 psi is the proper M_r input.
  - If this value is significantly higher or lower than 250 psi/inch then you can iterate by running the MEPDG with a higher or lower M_r subgrade input until you obtain a dynamic k value of 250 psi/inch.
  - A dynamic k value of about 250 psi/in was obtained meaning that 15,000 psi is the proper input M_r for the subgrade for this example.

Determine HMA Overlay Thickness for 20 year Design

All MEPDG inputs have been obtained and the MEPDG can be run with varying HMA overlay thicknesses. The performance criteria previously selected were used to establish if the design Passed or Failed. Thickness of 5 inches was run and the following results obtained.

- IRI Failed to achieve 95 percent reliability
- AC bottom up fatigue cracking Passed
- AC thermal fracture Passed
- Total permanent deformation Passed

Project: Utah I15 Nephi
Rubble 03
Reliability Summary **5 in HMA Overlay**

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	95	120.9	93.09	Fail
AC Bottom Up Cracking (Alligator Cracking) (%):	10	95	1.3	99.98	Pass
AC Thermal Fracture (Transverse Cracking) (ft/mi):	905	95	1	99.999	Pass
Permanent Deformation (AC Only) (in):	0.5	95	0.41	78.22	Fail
Permanent Deformation (Total Pavement) (in):	0.75	95	0.52	96.84	Pass

Given these results the HMA overlay thickness was increased to 7 inches and the MEPDG was rerun. Some criteria were still not met.

The final run included HMA overlay thickness of 7 inches and PG 70-34 and all performance criteria passed at 95 percent reliability. This design was considered adequate.

Project: Utah I15 Nephi Rubble 04
Reliability Summary 7in HMA Overlay & PG 70-34

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	170	95	116.5	95.21	Pass
AC Bottom Up Cracking (Alligator Cracking) (%):	10	95	0	99.999	Pass
AC Thermal Fracture (Transverse Cracking) (ft/mi):	905	95	1	99.999	Pass
Permanent Deformation (AC Only) (in):	0.5	95	0.33	95.53	Pass
Permanent Deformation (Total Pavement) (in):	0.75	95	0.43	99.87	Pass

APPENDIX H (Must be rerun with new software and calibration)

CTE Comparisons

Table H-1

CTE Comparisions with actual reference specimen CTE as 16.0 x 10-6 in/in/°C									
Pit Location	**Concrete Supplier**	**Cylinder #**	**CTE Test #1 (x 10-6 in/in/ °C)**	**CTE Test #2 (x 10-6 in/in/ °C)**	**Avg. CTE (x 10-6 in/in/ °C)**	**Avg. CTE (x 10-6 in/in/ °F)**	**Min. CTE**	**Max. CTE**	**Primary Aggregate Classification**
Moab	LeGrand Johnson ?	1	7.95	8.01					
		2	7.82	7.81	7.96	4.42	7.81	8.08	Granite/Quartzite
		3	8.07	8.08					
Monticello	Sonderegger Inc.	1	9.74	9.83					
		2	9.6	9.42	9.60	5.33	9.39	9.83	Quartzite
		3	9.63	9.39					
Cedar City	Sunroc Corp.	1	7.88	7.66					
		2	7.9	7.76	7.80	4.33	7.66	7.90	?
		3	7.85	7.73					
Hurricane	Interstate Rock Products	1	7.84	8.05					
		2	7.61	7.62	7.69	4.27	7.51	8.05	Volcanic
		3	7.53	7.51					
St. George	Sunroc Corp.	1	8.5	8.21					
		2	8.33	8.05	8.33	4.63	8.05	8.50	Limestone
		3	8.49	8.38					
Tooele	Harper Ready Mix	1	10.63	10.9					
		2	10.92	10.83	10.73	5.96	10.47	10.92	Quartzite
		3	10.47	10.64					
Pt. of Mountain	Geneva	1	10.31	10.43					
		2	10.44	10.5	10.42	5.79	10.31	10.50	Quartzite
		3	10.34	10.49					
Mouth of Big	Binggeli Rock Products	1	9.52	9.53					
Cottonwood Canyon		2	9.24	9.25	9.43	5.24	9.24	9.53	Quartzite
(Walker Pit)		3	9.53	9.49					
Heber City	Binggeli Rock Products	1	10.82	10.81					
		2	10.9	10.75	10.84	6.02	10.75	10.90	Quartzite
		3	10.87	10.89					
Brigham City - (7.8 Bag mix	JBP	1	10.8	10.94					
		2	10.89	10.98	10.95	6.08	10.80	11.09	Quartzite
		3	11	11.09					
Highland	Westroc	1	8.22	8.14					
		2	8.38	8.21	8.27	4.60	8.14	8.40	Limestone/Dolomite
		3	8.4	8.28					
Vernal	Binggeli Rock Products	1	10.03	9.9					
		2	9.54	9.57	9.84	5.47	9.54	10.06	Quartzite
		3	10.06	9.95					
Randlett	Tri-County Concrete	1	10.72	10.92					
		2	11.13	11.06	10.94	6.08	10.72	11.13	Quartzite
		3	10.86	10.93					
South Weber	Geneva Rock	1	11.07	11.16					
		2	11	11.08	11.09	6.16	11.00	11.16	Quartzite
		3	11.06	11.15					
Nephi	Staker Parsons	1	9.37	9.17					
		2	9.23	9.15	9.23	5.13	9.15	9.37	Quartzite
		3	9.24	9.23					
Brigham City - (6.5 bag mix	JBP	1	11.09	11.01					
		2	10.42	10.45	10.84	6.02	10.42	11.18	Quartzite
		3	11.18	10.89					
Elsinore	Western Rock	1	8.32	8.29					
		2	8.54	8.42	8.35	4.64	8.23	8.54	Volcanic
		3	8.32	8.23					
Nibley	LeGrand Johnson ?	1	9.33	9.11					
		2	9.26	9.45	9.26	5.15	9.11	9.45	Limestone
		3	9.25	9.17					
Fruitland	Cross Roads Concrete	1	10.75	10.69					
		2	10.63	10.53	10.67	5.93	10.53	10.75	Quartzite
		3	10.72	10.69					
			Maximum CTE	Minimum CTE					
			11.18	7.51					

ACPT

THE ADVANCED CONCRETE PAVEMENT TECHNOLOGY (ACPT) Products Program is an integrated, national effort to improve the long-term performance and cost-effectiveness of the Nation's concrete highways. Managed by the Federal Highway Administration through partnerships with State highway agencies, industry, and academia, the goals of the ACPT Products Program are to reduce congestion, improve safety, lower costs, improve performance, and foster innovation.

The ACPT Products Program identifies, refines, and delivers for implementation available technologies from all sources that can enhance the design, construction, repair, and rehabilitation of concrete highway pavements. The ACPT Marketing Plan enables technology transfer, deployment, and delivery activities to ensure that agencies, academia, and industry partners can derive maximum benefit from promising ACPT products in the quest for long-lasting concrete pavements that provide a safe, smooth, and quiet ride.

www.fhwa.dot.gov/pavement/concrete

U.S. Department of Transportation
Federal Highway Administration

TechBrief
OCTOBER 2011 | FHWA-HIF-09-015

Coefficient of Thermal Expansion in Concrete Pavement Design

This TechBrief describes the coefficient of thermal expansion (CTE) of concrete, its role in the behavior of concrete pavements, and recommendations for how to determine its value for concrete pavement design and analysis purposes. The sensitivity of concrete pavement performance prediction models in the Mechanistic-Empirical Pavement Design Guide to the CTE is discussed. Laboratory tests and other methods for determining or estimating the CTE are described, and the results of CTE laboratory tests on cores from Long-Term Pavement Performance pavement sections are summarized. Practical guidelines are provided for determining or estimating CTE and for taking into account the effect of CTE on a concrete slab's response to temperature changes when designing and constructing concrete pavements.

Introduction

Concrete expands when its temperature increases and contracts when its temperature decreases. The measure of how concrete changes in volume in response to temperature change is called the coefficient of thermal expansion (CTE) of concrete, defined as the change in unit length per degree of temperature change. The CTE of a concrete paving mixture depends on the aggregate type and degree of saturation.

Since coarse aggregate makes up the bulk of the volume of concrete, the most influential factor in the CTE of the concrete is the CTE of the coarse aggregate. Quartz has the highest CTE of the coarse aggregate types commonly used in concrete pavement construction, and the CTEs of other commonly used coarse aggregate types depend largely on their quartz content. Typical values for the CTE of concrete depending on the type of aggregate used are shown in table 1.

Coarse aggregate has the most effect on the CTE value, but fine aggregate is also a factor. Natural sands are typically high in silica (high CTE), and manufactured crushed limestone fine aggregates are lower in CTE.

The CTE of cement paste is quite sensitive to moisture content, but the CTE of concrete is less so, due to the mitigating influence of the coarse aggregate

Table 1. Coefficient of Thermal Expansion (CTE) of Concrete by Aggregate Type (LTPP Standard Date Release 25.0)

Primary Aggregate Class	Average CTE (/°F x 10^{-6})	Standard Deviation (s) (/°F x 10^{-6})	Average CTE (/°C x 10^{-6})	Standard Deviation (s) (/°C x 10^{-6})	Sample Count[1]
Andesite	4.32	0.42	7.78	0.75	52
Basalt	4.33	0.43	7.80	0.77	141
Chert	6.01	0.42	10.83	0.75	106
Diabase	4.64	0.52	8.35	0.94	91
Dolomite	4.95	0.40	8.92	0.73	433
Gabbro	4.44	0.42	8.00	0.75	8
Gneiss	4.87	0.08	8.77	0.15	3
Granite	4.72	0.40	8.50	0.71	331
Limestone	4.34	0.52	7.80	0.94	813
Quartzite	5.19	0.50	9.34	0.90	131
Rhyolite	3.84	0.82	6.91	1.47	7
Sandstone	5.32	0.52	9.58	0.94	84
Schist	4.43	0.39	7.98	0.70	30
Siltstone	5.02	0.31	9.03	0.56	21
				Total Sample Count	2,251

1. A total of 2,991 CTE values are available in LTPP Standard Data Release 25.0 (January 2011); 628 CTE values were not used due to aggregate class not defined or only one sample available for the primary aggregate type, and 112 CTE outlier values were also not included in the table.

(Powers and Brownyard 1947; Yeon et al. 2009). The CTE of concrete is highest at a relative humidity of about 70 percent (U.S. Army COE 1981) and 20 to 25 percent lower when the concrete is fully saturated.

How CTE Influences Concrete Pavement Behavior

Changes in concrete volume in response to temperature change are responsible for several aspects of concrete pavement behavior. Daily and seasonal cycles of temperature change in a concrete slab cause cyclic opening and closing of joints and cracks. To minimize transverse cracking, a jointed pavement constructed with a concrete with a high CTE may need a shorter joint spacing than a pavement constructed with a concrete with a lower CTE, which would increase the initial construction cost.

During daytime, when the top of a concrete slab warms up more than the bottom of the slab, the concrete will expand at the top of the slab more than at the bottom. If this differential deformation is not restrained (by dowels at the transverse joints, tie bars at the longitudinal joints, or both, and the slab's own weight), the slab will curl downward. If, on the other hand, daytime downward curling of the slab is restrained along the slab's edges, the result will be higher bearing stresses between the concrete and the dowels.

Similarly, during nighttime, when the top of a concrete slab cools down more than the bottom of the slab, the concrete will contract at the top of the slab more than at the bottom. If this differential deformation is not restrained (by dowels at the transverse joints, tie bars at the longitudinal joints, or both), the slab will curl upward. If, on the other hand, nighttime upward curling of the slab is restrained along the slab's edges, the result will be higher bearing stresses between the concrete and the dowels.

If the base layer below the slab is sufficiently soft that the slab can curl upward or downward and still remain in full contact with the base layer in the middle of the slab and along its edges, the stress induced in the slab by a traffic load will not be much different than if the slab were flat and in full contact with the base layer. However, if the base layer below the slab is sufficiently stiff that when the slab curls upward or downward in response to a temperature gradient through its depth, a portion of the slab curls out of contact with the base, the stress induced in the slab by a traffic load will be greater than if the slab were flat and in full contact with the base. This is particularly a concern with nighttime (upward) curling, when reduced support at slab edges and corners will result in increased edge and corner stresses under traffic loads.

The CTE of concrete also has an influence on the performance of continuously reinforced concrete pavement (CRCP). The steel content of CRCP is designed to achieve a crack spacing that is fairly uniform and within the range of about 3 to 6 ft (1 to 2 m). Too short a crack spacing may increase the likelihood of punchouts, and too long a crack spacing may increase the likelihood of steel ruptures. If the CTE of the concrete is higher than is assumed (or implicit) in the design of the steel, the desired crack spacing and uniformity may not be achieved. It is important to determine the concrete CTE (based on past experience or new testing) during the design phase, to adjust the design to achieve the desired level of performance, and to require that the CTE value be verified during construction.

Test Methods for Determining CTE

The AASHTO test method for determining the CTE of concrete is T 336-11. This laboratory test involves measuring the change in length of a saturated concrete core or cylinder, 4 in. (10 mm) in diameter, while it is subjected to an increase in temperature from 50 °F to 122 °F (10 °C to 50 °C) and then a decrease in temperature back to 50 °F. The concrete sample and test apparatus are completely submerged in a water bath to maintain saturation of the concrete during the test. Although the CTE of concrete at 100 percent saturation is not as high as at a somewhat lower moisture content, the laboratory test is run on saturated samples so that the moisture content is controlled. CTE testing equipment from two vendors and a concrete specimen mounted in the CTE test apparatus are shown in figure 1.

The measurements during the expansion (heating) and contraction (cooling) segments of the test are adjusted to account for the effect of the temperature changes on the test apparatus itself, and the CTE of the concrete is calculated for each of the two test segments as the change in the length of the sample per degree of temperature change, divided by the sample length. The testing sequence is repeated if necessary until the CTE values from the expansion and contraction segments of the test are within 0.20 millionths per °F (0.3 millionths per °C) of each other. The CTE of the concrete is then calculated as the average of the two consecutive CTE values obtained, one from the expansion segment of the test and one from the contraction segment of the test.

The U.S. Army Corps of Engineers has a similar test method for determining the CTE of concrete (U.S. Army COE 1981). This test method, CRD-C 39-81, directs that the test be conducted over a temperature range of 40 to 140 °F (5 to 60 °C). The Corps of Engineers test method directs that when the length change in the concrete test specimen is measured between only two temperatures, a single value of the CTE should be reported, but that when length change measurements are made at various temperatures, the curve of CTE versus temperature should be presented and the calculated CTE values for the different temperature intervals should be stated.

Mechanistic–Empirical Pavement Design Guide Recommendations for Determining CTE

For Level 1 design—the level that requires the greatest accuracy in inputs and is considered appropriate for the most important projects—the Mechanistic–Empirical Pavement Design Guide (MEPDG) recommends laboratory testing of concrete samples to determine the CTE (AASHTO 2008).

Many States have begun to characterize their typical portland cement concrete mixtures using their

Figure 1. Measuring the CTE of concrete. Testing equipment in use at the FHWA concrete laboratory and, at left, a concrete specimen mounted for testing.

typical aggregates and storing these CTE values in a database. They will use these values, based on the project location, as a CTE input. By definition, these values are not a Level 1 input, but they are a more realistic input than a Level 2 or 3 input.

For Level 2 design—the level that is considered appropriate for routine, real-world projects—the MEPDG recommends that the concrete CTE be estimated as the average of the CTE values of the aggregate and cement paste, weighted with respect to their volumetric proportions in the mix.

For Level 3 design—the level that requires the least accuracy in inputs—the MEPDG permits the use of a typical value of CTE. The value to be used should be the typical value for the concrete made with the type of aggregates to be used in the project. Table 1 provides the range of concrete CTE values obtained from laboratory tests of cores from the Long-Term Pavement Performance (LTPP) program. It should be noted that these values are based on aggregates from across the United States and Canada. These CTE values may vary significantly across regions, depending on the mineralogy.

Information on typical concrete CTE ranges for different aggregate types is also available in the MEPDG (ARA-ERES 2004) based on the uncorrected LTPP CTE data and from other sources (Mindess and Young 1981; Kosmatka et al. 2002; Jahangirnejad et al. 2008).

How CTE Influences Performance Prediction With the MEPDG

The MEPDG identifies the CTE as one of the concrete material inputs required for critical response computations. The value used for the CTE of concrete has a significant effect on the prediction of slab cracking and, to a lesser extent, joint faulting in the MEPDG (Malella et al. 2005). Both of these distresses play a role in the MEPDG's prediction of pavement roughness. Higher CTE values correspond to

CTE Testing and MEPDG Distress Models

The new Mechanistic-Empirical Pavement Design Guide (MEPDG) models for JCPs were developed using the LTPP database. One of the LTPP data parameters used was the concrete CTE. The concrete CTE data used for the original concrete pavement distress model development were found to be in error (Crawford et al. 2010) due to an error in the test procedure used to determine the CTE data. The test procedure used was the AASHTO TP 60-00 (AASHTO 2005) test method, and its use resulted in determination of higher CTE values. The TP 60 test method recommends a value of 17.3 x 10^{-6}/°C for the 304 stainless steel specimen used to calibrate the CTE test frame, but the CTE of the 304 stainless steel specimen determined according to ASTM E 228 is 15.0 x 10^{-6}/°C, use of which results in a lower CTE for concrete by the same proportion.

The method for determining the CTE of the stainless steel specimen used to calibrate the CTE frame has been addressed in the new AASHTO T 336 test method (AASHTO 2011; Tanesi et al. 2010). Use of the new test method results in lower CTE values than those determined using the TP 60-00 test method. The CTE values in the LTPP Standard Data Release 24.0 and later have been corrected to conform to the T 336 test method and are the ones reported in table 1.

As of August 2011, the concrete pavement distress models incorporated in the recently released (July 2011) version of the DARWin-ME™ software (incorporating the MEPDG Version 1.1 distress models) are based on the CTE values determined using the TP 60-00 test method. As a result, Darwin ME users are advised to use the uncorrected CTE values, as listed in table 11-5 of the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (Interim Edition) published by AASHTO in 2008, or to use CTE data determined using the TP 60-00 test method. If the available CTE data were determined using the T 336 procedure, then the CTE values should be adjusted for use with DARWin-ME by adding the difference between the assumed CTE of the calibration bar, 17.3 x 10^{-6}/°C, and the ASTM E 228 CTE value for the 304 stainless steel calibration specimen. The difference should be approximately 1.5 x 10^{-6}/°C.

greater predicted amounts of slab cracking, greater joint faulting, and greater pavement roughness.

Recommendations

The MEPDG provides the opportunity to quantify the effect of concrete CTE on the predicted performance of JCP and CRCP. The MEPDG's prediction of JCP slab cracking is sensitive to the input CTE, and to a lesser extent, so is the MEPDG's prediction of joint faulting. Both of these distresses play a role in the MEPDG's prediction of pavement roughness.

Given the sensitivity of several of the MEPDG's concrete pavement distress models to the concrete CTE input, for Level 1 design, the CTE should be determined (using the AASHTO T 336-11 test method) by conducting tests on cylinders with the same aggregate type and mixture design as will be used in the construction of the pavement.

For Level 3 design, the data provided in table 1 should be used. These are the average CTE values obtained from laboratory testing of hundreds of cores from LTPP concrete pavement sections and are also the typical midrange values of CTE of concrete reported in several sources.

As noted in the boxed text, it is important that if the DARWin-ME™ software (incorporating the MEPDG Version 1.1 distress models) is used, then adjustments to the CTE values should be made if these values were determined using the AASHTO T 336 method or if CTE values from table 1 are used.

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UTAH DEPARTMENT OF TRANSPORTATION

TECHNICAL BULLETIN MT- 02.02
November 6, 2002

Tack/Prime Coats

Prime coats are a cutback asphalt applied to a prepared sub-grade or untreated base course. Prime coats provide a membrane of bitumen to seal road bases, sub-bases, and sub-grades and prevent entrance of water or water loss by evaporation. Prime coats help maintain the balance of moisture and reduce the loss of fines from the surface by heavy rainfall and also help provide good bonding of a bituminous overlay.

Tack Coats are an emulsified asphalt applied to the existing surface or new pavement surface and intermediate lifts. Tack coats are an application of liquid asphalt material to ensure a bond between the surface being paved and the next course.

PROPER APPLICATION

Apply prime coats to a compacted surface free from ruts, corrugations, and all other irregularities. For tack coats, clean the surface of all material that prevent the tack coat from bonding to the surface such as mud, dirt and leaves. Cover all tacked surfaces with surfacing materials the same day the tack coat is applied. For prime and tack coats, protect all structures such as guardrails and guideposts from spattering and use a pressure distributor to apply the asphalt in a uniform and continuous spread. Higher application rates may be necessary on milled surfaces. Apply as much tack as possible without leaving significant puddling on the road surface. Strive for 90% surface coverage.

Do not apply to wet surfaces, or when the surface temperature is below 50 EF. Do not apply tack or prime coats when weather conditions will prevent proper adherence.

SPECIFICATIONS/DETAILS

- For prime coats, MC 70 (medium curing) and MC 250 are typical.
- The most common emulsion types for tack coats are diluted CSS-1h (cationic, slow-setting, hard base asphalt), CSS-1, SS-1, and SS-1h. Diluted coats allow a higher application rate and require a longer cure time.
- Keep viscosity between 50 and 100 centistokes. (AASHTO T 201)
- For prime coats use blotter material following AASHTO T 27 if prime coat fails to penetrate.
- See UDOT Standard Specification Section 02748 PRIME COAT/TACK COAT

COST INFORMATION

A 1-1 diluted CSS solution unit cost is approximately $100/ton.
MC type asphalts are approximately $235/ton.
Costs for these asphalts are not fixed and should be referred to as an approximation only.

FURTHER INFORMATION

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UTAH DEPARTMENT OF TRANSPORTATION

TECHNICAL BULLETIN MT- 05.08

May 24, 2005

ASPHALT SURFACING MATERIALS TYPICAL APPLICATION RATES

A designer should have a basic knowledge of how the various paving materials are placed in construction so they can correctly determine the correct paving quantities. The information provided herein is approximate and for estimating purposes only. Typical values have been assumed and any values may change considerably for a specific location. You should verify the unit weights with the Region Materials Engineer.

Surface Treatments

1. Open-Graded Surface Course
   Weight: 135 lb/ft^3
   Asphalt: PG Binder, 6.25%
2. Chip Seal Coat
   Emulsion: CRS-2, CRS-2P, LMCRS-2, HFMS-2, HFMS-2P
   0.35 gal/yd^2 (New Pavement)
   0.45 gal/yd^2 (Milled Pavement)
3. Prime Coat
   Cut-Back: MC-70 or MC-250
   0.5 gal/yd^2
4. Tack Coat
   Emulsion: CSS-1, CSS-1H, SS-1, SS-1H (diluted 2 parts emulsion to1 part water)
   0.10 gal/yd^2 (New Pavement)
   0.15 gal/yd^2 (Milled Pavement)
5. Flush Coat
   Emulsion: CSS-1, CSS-1H, SS-1, SS-1H, HFMS-2P (diluted 2 parts emulsion to1 part water)
   0.12 gal/yd^2

Surface Courses

1. Hot Mix Asphalt
   Weight: 152 lb/ft^3
   Asphalt: 5.20% of total wet weight

Base Courses

1. Lean Concrete Base: bid by square yard, @ 4 inch thick
2. Untreated Base Course
   Weight: 138 lb/ft^3
3. Granular Borrow
   Weight: 133 lb/ft^3

FURTHER INFORMATION

UDOT Standard Specification 02745, Bituminous Materials
UDOT Materials Technical Bulletin, MT-05.06, “Selecting and Using Asphalt Emulsions”
Asphalt Institute Manual Series No. 19, “A Basic Emulsion Manual”
Asphalt Institute Manual Serials No. 4, “The Asphalt Handbook”

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UTAH DEPARTMENT OF TRANSPORTATION

TECHNICAL BULLETIN MT- 03.01

March 4, 2003

Tire Rubber Modified Hot-Applied Chip Seal Coat

A hot applied application of tire rubber modified (TRM) PG asphalt binder is applied to the pavement, followed by an application of precoated, uniformly graded aggregate. The 5-6% tire rubber, dissolved in an elastomer modified asphalt binder (PG70-28), enhances adhesion for better chip retention. High application rates can be achieved with no appreciable bleeding. The dissolved tire rubber contributes carbon black, which decreases oxidation and UV degradation rates. Once the TRM PG asphalt binder cools, the mat can be swept and opened to traffic. The process is a rational method of utilizing waste tire rubber.

PROPER APPLICATION

This surface application is suitable for chip sealing high volume, interstate and primary routes and is particularly suitable where fast curing times and minimal traffic disruptions are desired. This process can also be used on city/county streets where bleeding and tracking must be avoided. Application details include: (1) Placing between May 15 and August 31, placing on dry pavements, and when pavement temperatures are 70 °F in the shade and rising.

SPECIFICATIONS/DETAILS

A special provision 02785 is available. It includes a generic specification for the Tire Rubber Modified PG Binder (PG70-28TRM), pre-coated cover aggregate, and other construction details. As with any Chip-Seal application, basic “state-of-the-art” construction practices must be addressed. The TRM Binder should be sprayed at temperatures from 350 to 360 °F. Cover aggregate must be pre-coated in a pug-mill with approximately 0.1 % PG58-22 and stockpiled. The pre-coated aggregate must be placed immediately after spraying the binder. Minimums of two pneumatic-tired rollers are used after the cover material has been spread. The mat can be swept soon after rolling, when the TRM binder reaches an ambient temperature. No fog sealing is required. Complete requirements can be found in the Special Provision. A successful TRM Hot-Applied Chip Seal was successfully placed on I-80, Echo Jct. to Castle Rock. Granite Construction Company was the prime contractor.

COST INFORMATION

TRM Modified Binder is typically sprayed around 0.38 GSY resulting in an effective coverage of approximately 0.35 GSY when cooled, whereas a CRS-2P is sprayed about 0.45 GSY. The higher CRS-2P rate accounts for approximately 35% water and results in an effective residual coverage of approximately 0.30 GSY. The TRM Hot-Applied process requires pre-coated chips: whereas a CRS-2P process involves fog-sealing the completed mat with CSS emulsion. Normal traffic flow is allowed back on the TRM mat after sufficient rolling and sweeping (generally within 1 to 2 hours). Emulsions require longer cure times and additional traffic handling. Based on these factors and a 10-year usage in Texas, a typical cost for the hot-applied system is $0.809 per square yard,

versus $0.917 per square yard for the CRS-2P application.

RELATED APPLICATIONS

Surface applications ranked to anticipated performance for severe service conditions:

1. Open-Graded Friction Coarse - design life: 8-9 years.
2. Ultra Thin Bonded Wearing Courses (a high polymer modified asphalt emulsion applied to a pavement surface followed immediately by a thin lift of hot mix asphalt). Design life: 7-8 years.
3. Chip Seals using pre-coated chips and a hot applied tire rubber modified PG binder (PG70-28TRM). Design life: 6-8 years.
4. Macro Surfacing (A tightly controlled chip-seal operation using a specially designed, chemical setting emulsion. Design life: 5-7 years.
5. Chip seal using a polymer modified emulsified asphalt (CRS-2P, HMFS-2P, HFRS-2P, LMCRS-2). Design life: 4-5 years.

FURTHER INFORMATION

Contact Howard Anderson, at (801) 633-8770, handerson@utah.gov
"A Basic Emulsion Manual", Asphalt Institute Manual Series No. 19-second edition
"Chip Seal Best Practices" by Larry Gay, Region 4 Materials Engineer, lgay@utah.gov