COLORADO
Department of Transportation

M-E Pavement Design Manual

2021

INTRODUCTION

Purpose of Manual

The purpose of this Pavement Design Manual is to provide the Colorado Department of Transportation (CDOT) and consultant pavement designers with a uniform and detailed procedure for designing pavements on CDOT projects. This manuals should be used after July 1, 2021.

Organization of the Manual

The manual is organized in a manner that affords the users with simple and methodical steps in the design of pavements for the Colorado state highway system. The contents are arranged carefully to provide users with sufficient flexibility in selecting and focusing on the appropriate topics and chapters that will suit their specific pavement design needs. There are four major pavement design categories presented in this manual; new construction/reconstruction, rehabilitation with overlays, rehabilitation without overlays, and intersection designs. Each category contains CDOT's current procedures utilized in the design of flexible and rigid pavements. Also included are relevant and required input data including pavement design information, subgrade and base materials, pavement type selection, life cycle cost analysis, pavement justification report (PJR), and appendices. These chapters are provided to support and document the entire pavement design process. The Introduction Pavement Design Manual Organization Flow Chart depicts a general overview of how this manual is organized.

Importance of Pavement Design

CDOT spends more than 30 percent of its annual construction and maintenance budget on pavements. Therefore, pavements need to be properly designed using an analytical process with accurate design inputs. A pavement design needs to be performed during the early phase of project development. This step ensures that pavement design is used to estimate and establish the project cost rather than the project cost dictating the pavement design.

Training

This manual provides general and detailed information about pavement design processes and procedures applicable to various locations in the State of Colorado. Information on more comprehensive training courses entitled Pavement Design and Life Cycle Cost Analysis and other materials related training classes is available through the CDOT Materials and Geotechnical Branch, Pavement Management and Design Program.

Approved Pavement Design Methods

The AASHTO Mechanistic-Empirical (M-E) design procedure using AASHTOWare Pavement M-E Design software (formerly DARWin-ME™) is the recommended method to determine pavement design thickness. The CDOT strongly recommends using the AASHTO Interim Mechanistic Empirical Pavement Design Guide (MEPDG) Manual of Practice along with the latest CDOT Pavement Design Manual.

Coordinating Designs with Other Agencies

Other agencies should contact either the Region Materials Engineer (RME) or the Pavement Design Program Manager (PDMP) concerning CDOT and Region policies relating to pavement issues.

Data Collection

The data collected for new construction and rehabilitation projects are somewhat different. The pavement rehabilitation project will take the largest data collection effort. In many instances, it may be necessary to design for both pavement reconstruction and pavement rehabilitation. The final selection between the two will involve a study of costs, traffic handling, and other related items.

Pavement Justification Report (PJR) and Other Documentation

A PJR is a formal engineering document that presents all analysis, data, and other considerations used to design a pavement. Guidelines for the information that needs to be included in a pavement design report are contained in this manual. For the special cases identified below that do not require a pavement design report, the documentation should include a brief description of the criteria, engineering considerations, and or Region policy used in the decision process. For other reporting requirements, contact the RME for guidance. The PJR shall be sent to the CDOT Region Materials Engineer. A copy of the PJR on all surface treatment projects and all new or reconstruction projects with Hot Mix Asphalt (HMA or Portland Cement Concrete Pavement (PCCP) material costs greater than $3,000,000 will be sent to the PDPM. Access and local agency project PJR's will not be required to be submitted to the PDPM.

Projects Needing a Pavement Justification Report

HMA overlays less than 2 inches are considered a preventive maintenance treatment, and therefore a PJR may not be required. Nevertheless, considering the significant investment thin overlays represent, these treatments should be considered in an overall pavement preservation program. For design categories not covered above, contact the RME or the PDPM for guidance about recommended design procedures and documentation requirements.

Responsibility, Approval, and Signature Authority

Pavement design and documentation is primarily the responsibility of the engineer of record and must be reviewed and approved by the RME. In the event that the RME position is vacant, the pavement designs shall be forwarded to the CDOT Materials and Geotechnical Branch Manager. For the pavement design work prepared by a consultant, the PJR shall be stamped, signed, and dated by the consultant and shall include his/her Professional Engineer's License number. The development of pavement design in CDOT is done in English units, which is the standard.

Electronic Documentation

CDOT is transitioning toward accepting all submittals, forms, project records and supporting documents in electronic format. This Manual reflects technology as of (date). Users should work in partnership with CDOT staff to continue to advance this effort in between Manual updates.

Adobe Sign

Adobe Sign is the electronic signature and professional seal software selected by CDOT and required for use on Project Records including Change Modification Orders (CMO), which facilitates automated workflows including the ability to route Project Records for acknowledgements, electronically sealing and/or signing. Adobe Sign is not the eSignature program selected for use on document requiring a CDOT Controller or State Controller signature (contracts).

Deliverables Management

CDOT uses a series of tools in the Bentley suite for design, construction and engineering documents. One of them is ProjectWise Deliverables Management. This is a cloud-based service that streamlines how a project team works with transmittals, submittals, and Requests for Information (RFI). It provides improved visibility into these processes and also retains confidentiality when legally required.

ProjectWise Deliverables Management is utilized to ensure that documents are submitted, completed and processed on schedule. Functions include: ensuring delivery to correct parties, enabling faster reviews and responses, automating an audit trail thereby increasing accountability with detailed recordkeeping, connecting entire supply chain through a secure cloud platform and leveraging project dashboards to monitor workflows and evaluate project performance.

ProjectWise Deliverables Management is capable of handling reference files used in design.

Project Share

The Cloud-based software tool hosted in the Bentley / Microsoft Azure Cloud used for document collaboration. Project Share connects to and synchronizes with ProjectWise Explorer, such that files placed in a Project Share folder, which is synchronized with ProjectWise Explorer, are automatically copied to the same folder in ProjectWise Explorer. Note that Project Share is not used for DGN reference files in design.

ProjectWise Explorer

Bentley ProjectWise Explorer is the Electronic Document Management System (EDMS) for archiving all electronic Project Records set forth in the CDOT Record File Plans.

Definitions

Adobe Acrobat DC. The software selected by CDOT and required for use in order to create and/or modify a PDF (portable document format) Project Record, to retain a record in an ISO Compliant format. By using Adobe Acrobat DC tools, the software “Smart Scans” Project Records to meet state and federal legal requirements prior to archiving in ProjectWise Explorer.

Adobe Sign. The electronic signature and professional seal software selected by CDOT and required for use on Project Records including Change Modification Orders (CMO), which facilitates automated workflows including the ability to route Project Records for acknowledgements, electronically sealing and/or signing.

Electronic Document Management System. ("EDMS") ProjectWise Explorer which has been selected by CDOT as the EDMS for CDOT Project Records.

Form 950 "Project Closure". This CDOT form provides notice of financial closure of the project. It includes notification to the FHWA that the project is closed and includes an electronically generated Project Record retention date.

ISO Compliant. A Record retained in a format approved by the International Organization for Standardization, a worldwide federation of national standards which refers to the ISO 19005 series of standards with PDF/A-1 approved as a minimum. Archiving an electronic Record in an ISO Compliant format ensures that it can be read in one hundred years, regardless of the hardware or software used to create the record. An ISO Compliant Record replaces microfilm as a method of archiving.

Naming Convention. A thread of acronyms that allows the CDOT Project Record to be correctly named and located in the ProjectWise Explorer locally-hosted or cloud-based EDMS.

Project Records. Engineering, Design, Specialty Group, and Construction Records pertaining to CDOT projects, including change modification orders (CMO). See § 24-80-101(1), C.R.S. "Record" shall also mean information that is inscribed on a tangible medium or that is stored in an electronic or other medium. See § 24-71.3-102(13), C.R.S. For further clarification, see relevant CDOT Records File Plans pertaining to Project Records.

Project Share. The Cloud-based software tool hosted in the Bentley / Microsoft Azure Cloud used for document collaboration. Project Share connects to and synchronizes with ProjectWise Explorer, such that files placed in a Project Share folder, which is synchronized with ProjectWise Explorer, are automatically copied to the same folder in ProjectWise Explorer.

ProjectWise Explorer The Bentley software system utilized by the Department for archiving Project Records.

Record File Plan. CDOT's internal governing document developed by each division, program, or unit which contain the state and federal legal retention requirements for CDOT Records pertaining to the specific Records. Record File Plans include the correct location in ProjectWise Explorer for each Project Record.

Smart Scanning. The term CDOT uses to meet state and federal retention requirements for CDOT Project Records by utilizing Adobe Acrobat to make Project Records searchable, page aligned, and compressed. It also means archived in an ISO Compliant format. Note that some mediums, such as video files and image files cannot be archived in an ISO Compliant format. In this case, the files shall be retained in their original format.

CDOT Legal Requirements Regarding Record Retention

CDOT's legal requirements to retain project records extend not only to CDOT employees but also the consultants, contractors and local agencies who work on CDOT project records. As a public agency, CDOT is legally required under state and federal law to retain certain Project Records for specified time periods. These time periods are set forth in the CDOT Record File Plans.

Compliance with Procedural Directive 21.1 "Requirements for the Retention of Records for Specified Design, Construction, Engineering, and Specialty Groups (Paper and Electronic)" effective June 20, 2019.

<u>General Reference to PD:</u>
CDOT's requirements for Project Records are set forth in Procedural Directive ("PD") 21.1 "Requirements for the Retention of Records for Specified Design, Construction, Engineering, and Specialty Groups (Paper and Electronic)" effective June 20, 2019. *The requirements of Procedural Directive (PD) 21.1 apply to CDOT employees and to contractors, consultants and local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records. It applies to all CDOT projects including local agency, P3, Innovative, Design-Build and CMGC projects.*

<u>Applicability</u>
Procedural Directive 21.1 shall apply to all divisions, offices, and regions of CDOT engineers and project staff who develop, handle, or receive records. It also applies to all projects, including but not limited to capital engineering projects, local agency, P3, Innovative, Design-Build (DB) and Construction Management General Contracting projects (CMGC). It applies to all consultants, contractors and local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records.

<u>Archiving Project Records in ProjectWise.</u>
All active and future Project Records shall be archived in Project Share / ProjectWise Explorer Electronic Document Management System on an ongoing basis rather than at the conclusion of the project.

Phases or milestones from scoping to project closure shall be established for archiving purposes. Record File Plans indicate the correct archive location for these records. They are located in the Governing Documents folder under "5 – Record File Plans". For external users, a link to this file is included in all project share sites.

<u>CDOT's EDMS for Project Records</u>
Bentley ProjectWise Explorer is the Electronic Document Management System (EDMS) for archiving all electronic Project Records set forth in the CDOT Record File Plans.

If project consultants are using Aconex, the PM and CDOT Resident Engineer must develop a phased approach to migrate records into ProjectWise Explorer on an ongoing basis within 45 days of the project final acceptance.

Record Retention Schedules for Project Records

CDOT's Record File Plans contain a list of the public records that are required to be retained, as well as the electronic folder in ProjectWise Explorer where they will be archived. A link to the CDOT Record File Plans is made available in each Bentley Project Share site. This link will provide access for consultants, contractors and local agencies to CDOT Record File Plans.

CDOT's project records are created and retained in electronic format unless the record has a retention period of 3.5 years or less from the Form 950 closure date. If the retention period is shorter, the Project Engineer along with the Region Finals Administrator shall make the determination to retain documents in paper form.

Project Records that are subject to the following categories must be retained for seven years from the Form 950 close date (may be longer if FEMA requirements apply):

- Major project (CMGC, DB, P3 or other innovative contract projects)
- Subject of internal or external audit
- Litigation hold
- Emergency funded

Project Records must be archived according to milestones established by the project engineer on an ongoing basis rather than at the conclusion of the project.

Smart Scanning (ISO Compliant Requirement)

Properly archiving Project Records means that they will be preserved in digital PDF format so that they can be read with original fidelity in one hundred years regardless of the hardware or software used to create them. This ensures that CDOT's most critical records with long-term or permanent retention requirements may be retained in digital form rather than paper or microfilm.

Project Records with retention periods longer than 3.5 years must be "Smart Scanned" prior to archiving. Training on Smart Scanning is available by registering through the Transportation Engineering Training Program ("TETP") website located here: https://www.codot.gov/programs/tetp Smart Scanning makes the Project Record searchable, compressed, page aligned, and in compliance with International Standard Organization's ("ISO") standard PDF/A-1b. Project Records which do not need to be Smart Scanned are the following:

1. Project Records approved by the Project Engineer and CDOT Finals Administrator to be submitted in paper form. The CDOT Finals Administrator and Project Engineer may determine that Project Records with a retention period of 3.5 years or less from the CDOT Form 950 closure date can be provided to CDOT in paper form.
2. Videos, photos, image files, and other media formats which cannot be converted to PDF. Certain files are unable to be Smart Scanned and must be placed in ProjectWise Explorer in their original formats.

Paper Record Retention

If paper Project Records have a retention period of 3.5 years or less from the Form 950 project closure date, they may be scanned and retained electronically or retained in paper format until they have met their retention period. A Destruction Form shall then be completed. Once approved, the records may then be shredded or disposed of.

Project Records in paper form are now retained by the Regions for archiving until the Records meet their retention period. Headquarters no longer receives a copy.

Naming Conventions for File Names

Use standard naming conventions (PD 21.1 Appendix "A") and as noted in Record File Plans. For questions on naming conventions, ask CDOT Finals Administrators.

Adobe Sign: CDOT's Electronic Signature Software for Project Records.

Unless otherwise notified by the Chief Engineer, Adobe Sign is CDOT's approved electronic workflow signature software for "Project Records." This includes the use of Adobe Sign for sealing with the professional engineer seal (see Procedural Directive 508.1 below, which sets forth requirements for sealing). Adobe Sign may not be utilized for any document which requires a signature from the CDOT Controller or State Controller.

For all Project Records that do not require a CDOT Controller/State Controller signature, Adobe Sign shall be used for both eSignatures and eSeals on Project Records. Note that Adobe Sign is permissible for use on contract modification orders ("CMO") given that CMOs do not require a signature by the Office of the State Controller. Adobe Sign work flows for Project Records will significantly cut down time routing paper records for signature, and will automatically archive the signed Project Record in ProjectWise.

Local Agency Records

On Local Agency projects with CDOT oversight, Local Agencies follow their own record retention schedules that adhere to the Inter-Governmental Agreement with CDOT. However, specific documents in the CDOT Record File Plans are required to be retained by CDOT and must be provided to the CDOT Local Agency Coordinator by the local agency or its representative. CDOT uses Bentley Project Share for this purpose so that the Local Agency can transmit the project record to the CDOT Local Agency Coordinator using the project-specific Project Share site. The Local Agency Coordinator will then archive the project record utilizing the synchronization function in Project Share, and the document will automatically be archived in the correct ProjectWise Explorer folder.

CDOT Responsibilities:

- Resident Engineers:
  - Must ensure that their staff are trained to properly archive records in the correct location and format.
  - Include a provision requiring compliance with PD 21.1 in all task orders.
  - Provide a copy of PD 21.1 with the Notice to Proceed.

- Project Managers:
  - Must fill out all attribute fields known at the time of project creation and thereafter when modifications occur. Attribute fields are filled out in SAP CJ20N (and, when launched, On Track).
- Finals Administrators:
  - Responsible for creating three electronic plan sets in PWZ Explorer: Award Set with watermark, Record Set with watermark, As-Constructed Plan with watermark.
- Records Coordinators
  - Records Coordinators are selected by their Appointing Authority to handle Project Records. Their responsibilities are set forth in PD 51.1 and in the Overview of Records Management and Records Coordinator Certification available through SAP/My Learning.
- Engineering Contracts:
  - Must include in contracts that PWZ Explorer is CDOT's EDMS for Project Records.
  - Standards and Specifications Unit must include relevant requirements of PD 21.1 in project special provisions by January 2020 (deadline extended to July 30, 2020).

Procedural Directive (PD) 508.1 "Requirements for the Use of the Professional Engineer's Seal"

General Description

PD 508.1 defines the procedures for the use of the Professional Engineer seal by CDOT employees, consultants, contractors and local agencies who perform engineering work for CDOT.

All CDOT, local agency and consulting Engineers must utilize electronic sealing (rather than mechanical sealing on paper) by January 2020 unless an exception request and approval is granted by the Chief Engineer.

Beginning January 2021, no exemptions will be granted to the electronic sealing requirements.

Applicability

The requirements of PD 508.1 apply to CDOT employees and to contractors, consultants and local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records. It applies to all CDOT projects including local agency, P3, Innovative, Design-Build and CMGC projects. PD 508.1 must be read together with PD 21.1. Sealed Project Records must be retained in ProjectWise Explorer in conformance with the CDOT Record File Plans.

Engineering designs, Record Sets and Contract Modification Orders, contract drawings and specifications for CDOT projects prepared by COOT employees or by contractors or consultants who perform work for CDOT, or by local agencies who perform work for projects with COOT oversight and/or funding or federal funding passed through CDOT, shall be Sealed in accordance with Procedural Directive 508.1.

Legal Requirements for Sealing

CDOT's Sealing requirements are dictated by and adhere to the Sealing requirements for licensed engineers set forth in the AES Board Rules, 4 CCR 730-1, which have the effect of law. The AES Board Rules dictate which documents require a Seal. The AES Board Rules have the effect of law. These include Record Sets, Contract Modification Orders, VECP's M&S Standards and changes thereto. To limit the scope of responsibility to one or more disciplines, a statement must be included adjacent to the Seal which limits responsibility to those portions of work done, such as: "My responsibility with respect to this standard plan revision is limited to----------" Transmittal and storage of all CDOT project records shall adhere to the requirements of Procedural Directive 21.1and CDOT's Record File Plans. The Sealed Record Set is required to be deposited in CDOT's ProjectWise Explorer. This will constitute the official record and will be retained permanently.

Responsibilities

- Engineer in Responsible Charge:
    - Must seal respective documents for work within their scope of work, including local agencies. Must ensure that all seals are obtained on the record set. This includes the limitation of scope for each seal.
    - The Engineer in Responsible Charge on a local agency project with COOT oversight is required to Seal all documents within the scope of their work. They shall be responsible for depositing the Seal Record Set into ProjectWise within 45 days of the award.
- CDOT Resident Engineer:
    - Is responsible for ensuring that all documents requiring Seals are obtained within 45 days of award of the construction project and archived in the correct PWZ Explorer folder.

Exclusions from Sealing Requirements

Manufactured Components

Engineers may specify manufactured components (e.g., impact attenuators, products on the Approved Product List ("APL")), which are exempted by statute as part of design documents. Manufactured components for the purposes of this Procedural Directive shall consist of such items as a pump, motor, steel beam or other types of items that are manufactured in multiple units for selection and use in projects which must be designed by Engineers. Systems of manufactured components which are specific to a particular use or application must also be designed by an Engineer. The Engineer may show the manufactured component on the drawing or document and is responsible for the correct selection and specification of the manufactured component but is not responsible for the proper design and manufacture of the manufactured component.

Stormwater Management Plans

- Stormwater Management Plans (SWMPs) and Erosion/Sediment Control Plans are excluded from the Seal requirement. Stormwater Management Plan sheets that do not contain engineering information (e.g. hydrology, hydraulics) are not considered "engineering drawings"; therefore, Sealing by a professional engineer is not required.
- Engineering features (e.g., ditches, storm sewer and permanent water quality facilities) required for the management of stormwater on the project shall be included in the plans on separate sheets as details with the associated information which would require Sealing in accordance with this Directive.

SUMMARY OF MANUAL REVISIONS FROM 2020

SECTION	MAJOR REVISIONS
Introduction, Acronyms and Definitions	▪ Updated contact list ▪ New Section – Electronic Documentation
Chapter 1	▪ Added link to M-E Design templates.
Chapter 2	
Chapter 3	▪ New Figure 3.7 Vehicle Length by Axle Classification. ▪ Section 3.13 OTIS Traffic: guidance for calculating traffic if OTIS data does not match the construction year.
Chapter 4	
Chapter 5	▪ Sections 5.2 and 5.3: Removed CP L 3101 and replaced with T-190.
Chapter 6	
Chapter 7	▪ Section 7.14 Widened lanes shall not be used to reduce pavement thickness.
Chapter 8	▪ Section 8.15.2 Chip Seals: Guidance for using Type I or II chip seals depending on bicycle traffic volume.
Chapter 9	
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Chapter 12	
Chapter 13	▪ Section 13.2.3 Included an example for triangular distribution. ▪ Section 13.2.6 Guidance for triangular distribution using $\pm$ 3% reliability; including example. ▪ Section 13.4 Discount Rate: updated to 1.38% with a standard deviation of 0.54% ▪ Section 13.5.1 Initial construction costs: add 5% to bottom mat and include safety edge. ▪ Section 13.5.2 AC Cost Adjustment update; includes process for calculating. ▪ Section 13.7.13 Real Cost: annual maintenance costs used in RealCost be multiplied by the entire length of the project. ▪ Section 13.10 Alternate Bid: re-running a LCCA if bid quantities change. ▪ Table 13.2 Combine full and partial depth repairs to 1.6%; and change cross stitching bars from 190 to 12 per lane mile.
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ACKNOWLEDGEMENTS

The Materials and Geotechnical Branch of the Colorado Department of Transportation thanks the following individuals who contributed their expertise, knowledge, and time in reviewing the CDOT Pavement Design Manual.

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ACRONYMS COMMON TO CDOT

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
ABC	Aggregate Base Course
ACI	American Concrete Institute
ACPA	American Concrete Pavement Association
ADT	Average Daily Traffic
AMC	Annual Maintenance Cost
ARA	Asphalt Rejuvenating Agent
ASR	Alkali Silica Reactivity
CAPA	Colorado Asphalt Pavement Association
CBR	California Bearing Ratio
CDOT	Colorado Department of Transportation
CFR	Code of Federal Regulations
CIR	Cold In-Place Recycling
CP	Colorado Procedure
CTB	Cement Treated Base
CPPP	Concrete and Physical Properties Program
DARWin™	Design Analysis and Rehabilitation Program for Windows
DTD	Division of Transportation Development
EATB	Emulsified Asphalt Treated Base
ESAL	Equivalent Single Axle Load
FASB	Foamed Asphalt Stabilized Base

FDR	Full Depth Reclamation
FHWA	Federal Highway Administration
FIR	Field Inspection Review
FOR	Final Office Review
FMM	Field Materials Manual
FWD	Falling Weight Deflectometer
HBP	Hot Bituminous Pavement
HIR	Hot In-Place Recycling
HMA	Hot Mix Asphalt
HMAP	Hot Mix Asphalt Pavement
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
LCCA	Life Cycle Cost Analysis
LL	Liquid Limit
LS	Loss of Support
LTB	Lime Treated Base
LTPP	Long Term Pavement Performance
MMS	Maintenance Management System
MGPEC	Metropolitan Government Pavement Engineering Council
M_r	Resilient Modulus
MR	Modulus of Rupture
MUTCD	Manual on Uniform Traffic Control Devices
NMAS	Nominal Maximum Aggregate Size
N_{DES}	Recommended SuperPave™ Gyratory Design Revolution

NDT	Nondestructive Testing
NLPM	Network Level Pavement Manager
PCCP	Portland Cement Concrete Pavement
PDM	Pavement Design Manual
PG	Performance Grade
PI	Plasticity Index
PJR	Pavement Justification Report
PMA	Polymer Modified Asphalt
PMBB	Plant Mix Bituminous Base
PMBP	Plant Mix Bituminous Pavement
PDPM	Pavement Design Program Manager
PM	Pavement Manager
PMS	Pavement Management System
PMSC	Plant Mix Seal Coat
PTSC	Pavement Type Selection Committee
PSI	Present Serviceability Index
PWF	Present Worth Factor
RAP	Reclaimed Asphalt Pavement
RCP	Reclaimed Concrete Pavement
RCC	Roller Compacted Concrete
RIC	Research Implementation Council
RME	Region Materials Engineer
RSL	Remaining Service Life
SHRP	Strategic Highway Research Program
SMA	Stone Matrix Asphalt

SN	Structural Number
TCP	Traffic Control Plan
VFA	Voids Filled with Asphalt
VMA	Voids in the Mineral Aggregate
WIMS	Weigh-In-Motion Station
WSN	Weighted Structural Number
WWF	Welded Wire Fabric

DESIGN OF PAVEMENT STRUCTURES DEFINITIONS

ADT (Current Year)
The average two-way daily traffic (ADT), in the number of vehicles, for the current year. The average 24-hour volume, being the total number during a stated period, divided by the number of days in that period. Unless otherwise stated, the period is a year.

ADT (Design Year)
The average two-way daily traffic for the future year used as a target in design.

AADT
The annual average two-way daily traffic volume. It represents the total traffic on a section of roadway for the year, divided by 365. It includes both weekday and weekend traffic volumes.

Analysis Period
The period of time for which the economic analysis is to be made. Ordinarily, the period will include at least one rehabilitation activity.

Approach Slab
Section of pavement just prior to joint, crack, or other significant roadway feature relative to the direction of traffic.

Arterial Highway
A highway primarily for through traffic, usually on a continuous route.

Asphalt Mix Design
The process and documentation of proportions of asphalt, cement, and mineral aggregate with the percentages of each component and size of particle that will result in a homogeneous mix and can be compacted into asphaltic concrete.

Asphalt Rejuvenating Agent (ARA)
A bituminous emulsion sprayed on new asphalt pavements to seal them from the adverse environmental effect of air and water. ARA is also used on dry, weathered asphalt pavement to give them new vitality and plasticity.

Asphalt Overlay
One or more courses of asphalt construction on an existing pavement. The overlay may include a leveling course, to correct the contour of the old pavement, followed by uniform course or courses to provide needed thickness.

At-Grade Intersection
An intersection where all roadways join or cross at the same level.

Axle Load
The total load transmitted by all wheels on a single axle extending across the full width of the vehicle. Tandem axles 40 inches or less apart will be considered as a single axle.

Base Course
The layer or layers of specified or selected material of designed thickness placed on a subbase or subgrade to support a surface course.

Bituminous
A term used to designate materials that are derived from petroleum, coal, tar, etc.

Bituminous Surface Treatment
Alternate layers of bituminous binder material and stone chips.

Binder
Asphalt cement used to hold stones together for paving.

Bleeding
A type of asphalt pavement distress identified by a film of bituminous material on the pavement surface that creates a shiny, glass-like, reflective surface that may be tacky to the touch in warm weather.

Block Cracking
The occurrence of cracks that divide the asphalt surface into approximately rectangular pieces, typically one square foot or more in size.

Blowup
The result of localized upward movement or shattering of a slab along a transverse joint or crack.

California Bearing Ratio (CBR) Test
An empirical measure of bearing capacity used for evaluation of bases, subbases, and subgrades for pavement thickness design.

Cement Treated Base
A base consisting of a mixture of either mineral aggregate or granular soil and portland cement mixed, and spread on a prepared subgrade to support a surface course.

Centerline
The painted line separating opposing traffic lanes.

Channels
A ditch or canal adjacent to the roadway.

Chipping
Breaking or cutting off small pieces from the surface.

Chip Seal
A seal coat consisting of the application of asphalt followed by a cover aggregate.

Cohesive Failure
The loss of a material’s ability to bond to itself resulting in the material splitting or tearing apart from itself (i.e. joint sealant splitting).

Cold In-Place Recycled Pavement
A pavement structure composed of an asphalt concrete wearing surface and portland cement concrete slab. An asphalt concrete overlay on a Portland Cement Concrete (PCC) slab is also called a composite pavement.

Control of Access
The condition where the right of owners or occupants of abutting land or other persons to access light, air, or view in connection with a highway is controlled by a public authority.

Collector
A road of the intermediate functional category that collects traffic from the local roads to arterials or distributes traffic to local roads from arterials.

Concrete Overlay (Whitetopping)
The procedure for placing Portland Cement Concrete (PCC) overlays over existing Hot Mix Asphalt (HMA) pavements. Concrete overlay may be either conventional, thin, or ultra-thin depending on the required thickness of the PCC overlay. In general, conventional Concrete overlay uses 8 inches or greater.

- Thin concrete overlay uses greater than 4 but less than 8 inches.
- Ultra-thin concrete overlay uses 4 inches or less thickness of PCC overlay.

Constant Dollars
Un-inflated dollars that represent the prevailing prices for all elements at the base year for the analysis.

Corner Break
A portion of a jointed concrete pavement separated from the slab by a diagonal crack intersecting the transverse and longitudinal joint, which extends down through the slab allowing the corner to move independently from the rest of the slab.

Corrective Maintenance
Corrective maintenance could be a planned or unplanned strategy that restores the existing roadway to the intended design life. Typically, this process occurs within the first five years after construction.

Corridor
A grouping of project segments that are on the same highway facility that have some or all of the following characteristics: logical termini (i.e. begin/end point), similar roadway cross section, geologic and materials conditions, and future traffic. The projects in a corridor are advanced through preconstruction project development together to approximately 30 percent design in an effort to identify ROW, Utility and other resource impacts.

Cross-Stitching
A repair technique for longitudinal cracks and joints that are in reasonably good condition. The purpose of cross-stitching is to maintain aggregate interlock and provide added reinforcement and strength to the crack or joint. The technique uses deformed tie bars inserted into holes drilled across a crack at angles of 35 to 45 degrees depending on slab thickness.

DARWin^{TM}
A software program that performs the complex calculations for design and analysis of pavement structures. DARWin^{TM} is an acronym for Design, Analysis, and Rehabilitation for Windows.

Deflection Analysis
The procedure used to establish pavement strength indices based on pavement deflections induced by a force.

Deformed Bar
A reinforcing bar for rigid slabs. Most often used to tie slabs together in the longitudinal direction across lane lines including tying travel lanes and shoulders.

Design Period
The number of years from initial construction or rehabilitation until terminal service life. This term should not be confused with pavement life or analysis period. By adding asphalt overlays as required, pavement life may be extended indefinitely or until geometric considerations or other factors make the pavement obsolete. The initial design period is the number of years for which the volume and type of traffic and the resultant wheel or axle load application are forecast, and on which pavement designs are calculated.

Design Traffic (18k ESAL)
The design traffic is the total number of equivalent 18,000-lb single axle load (18k ESAL) applications expected during the design period. This can be calculated or obtained from CDOT personnel at the Traffic Analysis Unit of the Division of Transportation Development.

Deterministic Life Cycle Cost Analysis
A traditional cost comparison process where each item of interest is assigned a fixed discrete value, usually a value most likely to occur based on historical data and user judgement. This value includes all costs over the life of the project, such as construction, maintenance, and rehabilitation adjusted to a present value.

Diamond Grinding
A process of improving a pavement's ride by creating a smooth, uniform profile by removing faulting, slab warping, studded tire wear, and patching unevenness.

Discount Rate
A value in percent used for comparing the alternative uses of funds over a period of time. The discount rate may be defined as the difference between the market interest rate and inflation rate using constant dollars over the analysis period.

Dowel
A load transfer device in a rigid slab usually consisting of a plain, epoxy coated, round steel bar. Most often used to provide load transfer between slabs in the transverse direction that are within the same lane.

Drainage Coefficients
Factors used to modify structural layer coefficients in flexible pavements, or stresses in rigid pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration.

Durability Cracking
The breakup of concrete due to freeze-thaw expansive pressures within certain aggregates. Also called D-Cracking.

Economic Analysis
A justification of the expenditure required and the comparative worth of a proposed improvement as compared to other alternate plans.

Economic Life
Economic life is the total useful life of a pavement structure including the extended service life gained when the initial pavement is supplemented by the addition of structural layers. It also defines the period of time beyond which further use is not economical.

Edge Cracking
Fracture and material loss in pavements without paved shoulders which occurs along the pavement perimeter. Caused by soil movement beneath the pavement.

Embankment (Embankment Soil)
The prepared or natural soil underlying the pavement structure.

Emulsified Asphalt Treated Base
A base consisting of a mixture of mineral aggregate and emulsified asphalt spread on a subgrade to support a surface course.

Equivalence Factor
A numerical factor that expresses the relationship of a given axle load in terms of their effect on the serviceability of a pavement structure. All axle loads are equated in terms of the equivalent number of repetitions of an 18,000-poound single axle.

Equivalent Single Axle Loads (ESALs)
The effect on pavement performance of any combination of axle loads of varying magnitude expressed in terms of the number of 18,000-lb single-axle loads required to produce an equivalent effect. This is calculated by summing the equivalent 18,000-pound single axle loads (18k ESALs) used to combine mixed traffic to design traffic for the design period. The value of 18k ESALs is obtained as an accumulative total from the beginning of use until and including the design year. The 18k ESAL is calculated by multiplying the annual design traffic volume by the Traffic Equivalence Factor (e) at a given Terminal Serviceability Index (P_t).

Expansion Factor
A factor expressing the expected traffic growth trend on a particular section of highway.

Expressway
A divided arterial highway for through traffic with full or partial control of access and generally with grade separations at major intersections.

Fatigue Cracking
A series of small, jagged, interconnecting cracks caused by failure of the asphalt concrete surface under repeated traffic loading (also called alligator cracking).

Fault
Difference in elevation between opposing sides of a joint or crack.

Flexible Pavement
A pavement structure of which the surface course is made of asphaltic concrete, that maintains intimate contact with and distributes loads to the subbase or subgrade and depends upon aggregate interlock, particle friction, and cohesion for stability.

Foamed Asphalt Stabilized Base
A base consisting of mixed wet unheated aggregates and asphalt cement while the asphalt cement is in a foamed state.

Fog Seal
A seal coat consisting of an application of diluted asphalt emulsion without an aggregate cover.

Free Edge
Pavement border that is able to move freely.

Freeway
An expressway with full control of access and all at-grade intersections eliminated.

Full Depth Asphalt
A asphaltic concrete pavement structure consisting of one and only one layer. There is no base, subbase, or intermediary layer of gravel between the asphaltic concrete layer and subgrade.

Full Depth Reclamation
A rehabilitation technique in which the full thickness of asphalt pavement and a predetermined portion of the underlying materials (base, subbase and/or subgrade) is uniformly pulverized and blended to provide an upgraded, homogeneous base material. This new stabilized base course may be used for an asphalt or concrete wearing surface.

Functional Deficiency
Any condition that adversely affects the roadway user. These include poor surface friction and texture, hydroplaning and splash from wheel path rutting, and excess surface distortion.

Functional Maintenance
A planned strategy of low cost treatments that are meant to sustain the roadway and its appurtenances in a manner that delivers a condition in order to keep traffic moving.

Grade Separation
A crossing of two highways, or a highway and a railroad, at different levels.

Granular Base
A base consisting of mineral aggregate laid and compacted on a subbase or subgrade to support a surface course.

Grooving

Grooving restores skid resistance to concrete pavements. It increases the surface friction and surface drainage capabilities of a pavement by creating small longitudinal or transverse channels that drain water from underneath the tire, reducing the potential for hydroplaning.

Hairline Crack
A fracture that is very narrow in width, less than 0.125 inches (3 mm).

Hinged Joint
A joint between two rigid pavement slabs in which flexure is permitted but separation and vertical displacement of abutting rigid slabs are prevented by metal ties and mechanical or aggregate interlock.

Hot Bituminous Pavement
A combination of mineral aggregate and bituminous material, mixed in a central plant, laid and compacted while hot, to act as a surface course and carry traffic. Hot Bituminous Pavement is an older design usage. Also known as Plant Mixed Bituminous Pavement, see Hot Mix Asphalt for current designation.

Hot In-Place Recycled Pavement: Heater Remixing
A pavement rehabilitation process that consists of reworking the existing pavement with a heating device, reshaping, and compaction. This operation may be performed with or without the addition of a rejuvenating agent, aggregates, or new asphalt mix.

Hot In-Place Recycled Pavement: Heater Repaving
A pavement rehabilitation process that consists of reworking the existing pavement with a heating device. During the lay down process of the old rejuvenated material, a virgin lift will be added reshaped and compacted.

Hot In-Place Recycled Pavement: Heater Scarifying
A pavement rehabilitation process that consists of reworking the existing pavement with a heating device. A rejuvenating agent will be added to the old mix reshaped and compacted.

Hveem Stabilometer
A device for the measurement of the lateral pressure transmitted by a soil or aggregate being subjected to a vertical load. The pressure obtained is used to compute the R-value, which is the internal resistance or the internal friction property of a bituminous pavement or a base. The data obtained is used to compute the relative stability.

Hydroplaning
To skid on wet pavement because water on the pavement causes the tires to lose contact with it.

Joint Seal Damage
Any distress associated with the joint sealant, or lack of joint sealant.

Keyway
A groove on either vertical or horizontal face of a concrete slab. A keyway is often molded in concrete structures. A keyway molded on a vertical face of a concrete slab will provide interlock and load transfer to an adjacent slab. A keyway molded on a horizontal face of a concrete structure will provide interlock and resist horizontal movement of a concrete structure molded over the keyway.

Lane Factor
Factors used to convert total truck traffic to Design Lane Truck Traffic given the number of lanes.

Lanes to Shoulder Drop-off
The difference in elevation between the traffic lane and the shoulder.

Lane to Shoulder Separation
Widening of the joint between the traffic lane and the shoulder.

Lime-Treated Base
A base consisting of a mixture of soil, hydrated lime, and water usually mixed in place and placed to support a pavement structure, or the components thereof.

Load Transfer Device
A mechanical means designed to carry loads across a joint in a rigid slab.

Local Street or Local Road
A street or road primarily for access to residence, business, or other abutting property.

Longitudinal
Parallel to the pavement centerline.

Low Volume Road
A road with a two-directional Average Annual Daily Traffic (AADT) of less than 100 trucks per day and less than 1,000 cars per day.

Maintenance
The preservation of the entire roadway, including surface, shoulders, roadsides, structures, and such traffic control devices as are necessary for its safe and efficient utilization.

Major Rehabilitation
Pavement treatments that consist of structural enhancements that extend the serviceable life of an existing pavement and improve its load-carrying capability.

Map Cracking
A series of interconnected hairline cracks in portland cement concrete pavements that extend only into the upper surface of the concrete. It includes cracking typically associated with Alkali-Silica Reactivity (ASR).

Mechanistic-Empirical Pavement Design Guide
The guide and its accompanying software that provides a uniform basis for the design of flexible, rigid, and composite pavements, using mechanistic-empirical approaches which are more realistically characterize in-service pavements and improve the reliability of designs.

Micro-Surfacing
A seal coat consisting of the application of polymer modified emulsion followed by a cover of aggregates selected for properties of hardness and angularity.

Minor Rehabilitation
Pavement treatments consisting of functional or structural enhancements made to the existing pavement sections to improve pavement performance or extend serviceable life.

Modulus of Elasticity (E)
A measure of the rigidity of a material and its ability to distribute loads defined by the ratio of strain to stress in a portland cement concrete pavement slab.

Modulus of Subgrade Reaction (k-value)
Westergard's modulus of subgrade reaction for use in rigid pavement design (the load in pounds per square inch on a loaded area of the roadbed soil or subbase divided by the deflection, in inches, of the roadbed soil or subbase), psi/in. The modulus of subgrade reaction is the supporting capability of a soil measured by its ability to resist penetration of a series of loaded stacked plates.

Modulus of Rupture (S'_c)
An index of the flexural strength of the portland cement concrete pavement. It is a measure of the extreme fiber stress developing under slab bending, the mode in which most concrete pavements are loaded. The modulus of ruptured required by the design procedure is the mean value determined after 28 days using third-point-loading (AASHTO T97).

Nominal Maximum Aggregate Size
One sieve size larger than the first sieve to retain more than 10 percent of the material (Roberts et al., 1996).

Overlays

- **Leveling Course:** The layer of material placed on an existing paved surface to eliminate irregularities prior to placing an overlay or a surface course. Milling procedures are to be considered the primary method to address rutting and are to be used instead of a leveling course to remove ruts whenever possible.

- **Overlay Course:** Surfacing course, either plant mixed or road mixed, placed over an existing pavement structure after placement of a leveling course, as appropriate.

Partial Depth Reclamation
A rehabilitation technique in which a portion of the asphalt pavement is pulverized, mixed with a stabilizing agent, and placed back on the remaining pavement surface. Partial depth reclamation is limited to correcting only those distresses that are surface problems in the asphalt layer.

Patch
An area where the pavement has been removed and replaced with a new material.

Patch Deterioration
Distress occurring within a previously repaired area.

Pavement
The part of roadway having a constructed surface for the facilitation of vehicular movement.

Pavement Design (Design, Structure Design)
The specifications for materials and thickness of the pavement components.

Pavement Joints
The designed vertical planes of separation or weakness. Complete details of concrete pavement joints are given a standard specifications in CDOT's *Standard Plans M & S Standards*.

Joints Used in Portland Cement Concrete Pavement

- **Construction Joints:** Joints made necessary by a prolonged interruption in placing concrete. They are formed by placing concrete up to one side of a planned joint and allowing it to set before the concrete is placed on the other side of a joint. They may be either longitudinal or transverse.

- **Contraction Joints:** Joints placed either transversely at recurrent intervals or longitudinally between traffic lanes to control cracking.

- **Expansion Joints:** Transverse joints located to provide for expansion without damage to themselves, adjacent slabs, or structures.

- **Weakened Plane Joints (Longitudinal and Transverse**): Weakened plane joints are placed both longitudinally and transversely in PCCP. CDOT specifies using a saw to cut the weakened planes at $^{T}/_{3}$ in PCCP.

Pavement Maintenance
Typically, these treatments are preventive in nature and are intended to keep the pavement in serviceable condition. They may be classified as corrective, preventive, reactive, or functional.

Pavement Management
Pavement management is the evaluation, documentation, and analysis of the amount, quality, and type of pavement under the responsibility of any given owner or agency. It is also the planning and budgeting for the upkeep and replacement of paved assets.

Pavement Performance
The trend of serviceability with load applications.

Pavement Rehabilitation
Work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing material and/or completing any other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy. This could include the complete removal and replacement of the pavement structure.

Pavement Structure
The combination of subbase, base course, and surface course placed on a prepared subgrade to support the traffic load and distribute it to the roadbed.

Pavement Section
A layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Most soils can be adequately represented for pavement design purposes by means of the soil support value for flexible pavements and a modulus of subgrade reaction for rigid pavements

Performance Period
The period of time that the initially constructed or rehabilitated pavement structure will last (perform) before reaching its terminal serviceability. This is also called the design period.

Permeability
The property of soils which permits the passage of any fluid. Permeability depends on grain size, void ratio, shape, and arrangement of pores.

Plant Mixed Bituminous Base
A base consisting of mineral aggregate and bituminous materials mixed in a central plant, and laid and compacted while hot on a subbase or a subgrade to support a surface course.

Plant Mixed Bituminous Pavement
A combination of mineral aggregate and bituminous material mixed in a central plant, laid and compacted while hot to act as a surface course and carry traffic. Plant Mixed Bituminous Pavement is an older designation usage. Also known as Hot Bituminous Pavement, see Hot Mix Asphalt for current designation.

Plant Mixed Seal Coat
A seal coat consisting of a combination of mineral aggregate and bituminous material mixed in a central plant, laid, and compacted while hot.

Polished Aggregate
Surface mortar and texturing warn away to expose coarse aggregate in the concrete.

Popouts
Small pieces of pavement broken loose from the surface.

Pothole
A bowl-shaped depression in the pavement surface.

Prepared Roadbed
In place roadbed soils compacted or stabilized according to provisions of applicable specifications.

Present Serviceability Index (PSI)
A number derived by a formula for estimating the serviceability rating calculated from measurements of certain physical features of the pavement.

Preventive Maintenance
Preventive maintenance is a planned strategy of cost-effective treatments performed on an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without significantly increasing the structural capacity.

Prime Coat
Bituminous materials used on aggregate base courses to provide good adhesion to the hot mix asphalt layer placed above.

Probabilistic Life Cycle Cost Analysis

A process where probabilistic LCCA inputs are described by probability functions that convey both the range of likely inputs and the likelihood of their occurrence. Probabilistic LCCA also allows for the simultaneous computation of differing assumptions for many different variables. Probabilistic LCCA allow the value of individual data inputs to be defined by a frequency (probability) distribution.

Pumping

The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab.

Punchout

A localized area of a continuously reinforced concrete pavement bounded by two transverse cracks and a longitudinal crack. Aggregate interlock decreases over time and is eventually lost which leads to steel rupture and allows the pieces to be punched down into the subbase and subgrade.

Raveling

The wearing away of the pavement surface caused by the dislodging of aggregate particles.

Reactive Maintenance

Reactive maintenance is an unplanned, therefore, unscheduled; sometimes immediate treatments performed on an existing roadway system and its appurtenances that is necessary to avoid serious consequences.

Reconstruction

Treatments requiring full removal and replacement and or improvement of the existing pavement structure which includes subbase, base course, and surface course due to pavement condition and structural capabilities. A LCCA is required. Typical AASHTO criteria are addressed and designed to current standards.

Reflection Cracking

The fracture of asphalt concrete above joints in the underlying pavement layer(s).

Reinforcement

Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.

Reliability

The probability, expressed as a percentage that a pavement structure will carry the traffic for which it is designed over the design or analysis period.

Remaining Service Life (RSL)

The number of years a pavement is expected to last until maintenance and rehabilitation treatments no longer improve or maintain the surface condition.

Resilient Modulus (M_r)

A measure of the modulus of elasticity of roadbed soil or other pavement material. In M-E Design, the subgrade resilient modulus M_r is measured at optimum moisture content and density. This M_r is different than the AASHTO 1993 empirical design procedure which was basically a “wet of optimum” M_r. The input M_r is then internally adjusted to field conditions by the M-E Design software on a month to month basis based on water table depth, precipitation, temperature, soil suction, and other factors.

Rigid Pavement
A pavement structure of which the surface course is made of portland cement concrete.

Rigid Slab
A section of portland cement concrete pavement bounded by joints and edges designed for continuity of flexural stress.

Roadbed
The graded portion of a highway within top and side slopes prepared as a foundation for the pavement structure and shoulder.

Roadbed Material
The material below the pavement structure in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.

Roadway
The portion of a highway including shoulders, for vehicular use.

Roundabout
A circular intersection with yield control of all entering traffic, channelized approaches, counter-clockwise circulation, and appropriate geometric curvature to ensure travel speeds on the circulatory roadway are typically less than 30 miles per hour.

Rutting
Longitudinal surface depressions in the wheel paths.

Sand Seal
A seal coat consisting of the application of asphalt emulsion followed by a sand cover aggregate.

Scaling
The deterioration of the upper 0.125 to 0.5 inches of the concrete surface, resulting in the loss of surface mortar.

Seal Coat
A thin treatment consisting of bituminous material, usually with cover aggregate, applied to a surface as an armor coat or for delineation. The term includes but is not limited to sand seal, chip seal, slurry seal, and fog seal.

Service Life
The service life is the number of years a pavement is expected to last from completion of construction until pavement failure.

Serviceability
The ability, at the time of observation, of a pavement to serve traffic using the facility. Also, serviceability is a pavement's ability to provide adequate support and a satisfactory ride at any specific time.

Serviceability Index
A number that is indicative of the pavement's ability to serve traffic at any specific time in its life.

Shelf Project
A project that has been advanced through preconstruction process and completed the Pavement Type Selection and Life Cycle Cost Analysis. A final pavement type has been identified, is developed using Alternate Pavement Type Bidding methodology, or has gone through the Pavement Type Selection Committee and the Chief Engineer has recommended a preferred alternative.

Shoving
Permanent, longitudinal displacement of a localized area of the pavement surface caused by traffic pushing against the pavement.

Single Axle Load
The total load transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 40 inches apart and extending across the full width of the vehicle.

Skid Hazard
Any condition that might contribute to making a pavement slippery when wet.

Slot Stitching
A technique for repairing longitudinal cracks or joints. It is an extension of dowel bar retrofit, which is used to add dowel bars to existing transverse joints. The purpose of slot-stitching is to provide positive mechanical interconnection between two slabs or segments. The deformed bars placed in the slots hold the segments together serving to maintain aggregate interlock and provide added reinforcement and strength to the crack or joint. The bars also prevent the crack or joint from vertical and horizontal movement or widening. Larger diameter bars (> 25mm, > 1 inch) also serve to provide long-term load transfer capabilities.

Slurry Seal
A seal coat consisting of a semi fluid mixture of asphaltic emulsion and fine aggregate. This type of seal is usually placed in a very thin course of $^{1}/_{8}$ to $^{1}/_{4}$ inches.

Soil Support Value
A number that expresses the relative ability of a soil or aggregate mixture to support traffic loads through the pavement structure.

Spalling
Cracking, breaking, chipping, or fraying of the concrete slab surface within 2 feet of a joint or crack.

Squeegee Seal
A seal coat similar to a sand seal, consisting of the application of asphalt emulsion and sand. The application of a squeegee seal differs from that of a sand seal in that a surface drag is used to spread the emulsion to seal cracks

Stabilometer R-Value
A numerical value expressing the measure of a soil's or aggregate's ability to resist the transmission of vertical load in a lateral or horizontal direction. A test for evaluating bases, subbases, and subgrades for pavement thickness design. It is measured with a stabilometer.

Standard Normal Deviate (Z_R)
The standard normal deviate is a statistical value identical to the Z-scale value used in the standard normal distribution. It is a measure of the deviation of any observations from the mean of all observations expressed in terms of the number of standard deviations. Each calculated Z value corresponds to a certain level of significance, confidence interval, certainty, or reliability value in a standard normal distribution curve. The standard normal deviate (Z) can be calculated from the equation:

$$Z = \frac{\text{(Observed Value - Mean of all Observed Values)}}{\text{Standard Deviation of all Observations}}$$

Stone Matrix Asphalt (SMA)
A mixture of crushed coarse aggregate, crushed fine aggregate, mineral filler, asphalt cement, and stabilizing agent typically used as a wearing course. A stabilizing agent is used to prevent drain down of the asphalt cement and typically consists of fibers, polymers, or limestone dust (powder).

Structural Deficiency
Any condition that adversely affects the load carrying capability of the pavement structure. These include inadequate thickness, cracking, distortion, and disintegration. Several types of distress (i.e., caused by poor construction techniques, low temperature cracking) are not initially caused by traffic loads, but do become more severe under traffic to the point that they also detract from the load carrying capability of the pavement.

Structural Layer Coefficient (a_1, a_2, a_3)
The empirical relationship between structural number (SN) and layer thickness that expresses the relative ability of a material to function as a structural component of the pavement and express the relative strength of a layer in a pavement structure.

Structural Number (SN)
An index derived from an analysis of traffic, roadbed soil conditions, and environment that may be converted to thickness of flexible pavement layers by using suitable structural layer coefficients related to the type of materials being used in each layer of the pavement structure.

Subbase
The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course. Subgrade treated with lime, fly ash, cement kiln dust, or combination of stabilization will be considered subbase.

Subgrade
The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

Surface Course
The uppermost component of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer is also sometimes called the wearing course.

Surface Life
A period of time where treatments can be performed on a pavement that maintain or improve the surface condition.

Tack Coat
A light application of emulsified asphalt applied to an existing asphalt or portland cement concrete pavement surface. It is used to ensure a bond between the surface being paved and the overlaying course. Typically 0.10 gal/yd^2 of diluted CSS1h.

Tandem Axle Load
The total load transmitted to the road by two consecutive axles whose centers may be included between parallel vertical planes spaced more than 40 inches but not more than 96 inches apart, extending across the full width of the vehicle.

Tie Bar
A deformed steel bar or connector embedded across a longitudinal joint for a rigid slab to prevent separation of abutting slabs.

Tining
A process by which it is achieved by a mechanical device equipped with a tining head (metal rake) that moves laterally across the width of the paving surface.

Treated Base
A layer of base material stabilized with asphalt, portland cement, or other suitable stabilizers.

Traffic Equivalence Factor (e)
A numerical factor that expresses the relationship of a given axle load to another axle load in terms of their effect on the serviceability of a pavement structure.

Transverse
Perpendicular to the pavement centerline.

Triple Axle Load
The total load transmitted to the road by three consecutive axles whose centers may be included between parallel planes spaced more than 40 inches but no more than 96 inches apart, extending across the full width of the vehicle.

Water Bleeding
Seepage of water from joints or cracks.

Weathering
The wearing away of the pavement surface caused by the loss of asphalt binder.

Weigh-In-Motion (WIM) Station
The process of measuring the dynamic tire forces of a moving vehicle and estimating the corresponding tire loads of the static vehicle.

Welded Wire Fabric (WWR)
A two-way reinforcement system for rigid slabs, fabricated from cold-drawn steel wire and having parallel longitudinal wires welded at regular intervals to parallel transverse wires. The wires may be either smooth or deformed.

Whitetopping (old definition)
The procedure for placing portland cement concrete overlays over existing hot mix asphalt pavements. Whitetopping may be either conventional, thin, or ultra-thin depending on the required thickness of the PCC overlay. In general, conventional whitetopping uses 8 inches or greater:

- **Thin whitetopping** uses greater than 4 but less than 8 inches.
- **Ultra-thin whitetopping** uses 4 inches or less thickness of a PCC overlay.

MECHANISTIC-EMPIRICAL (M-E) PAVEMENT DESIGN BASIC DEFINITIONS

These definitions may be slightly different from the definition in the previous section. These basic definition as are to agree with the usage as in the Mechanistic-Empirical (M-E) Pavement Design Guide. Some have been modified to clarify this manual's notation.

Basic Definitions of the Roadway

Base

The layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to support surface course. The layer directly beneath the PCC slab is called the base layer.

- **Aggregate Base:** A base course consisting of compacted aggregates which includes granular base and unbound granular base.

- **Asphalt Concrete Base:** Asphalt concrete used as a base course. This may include asphalt base course, hot-mixed asphalt base, asphalt-treated base, bituminous aggregate base, bituminous base, bituminous concrete base, and plant mix bituminous base.

- **Cold Mix Asphalt:** Asphalt concrete mixtures composed of aggregate and/or asphalt emulsions or cutback asphalts, which do not require heating during mixing. This may include emulsified asphalt treated base.

- **Permeable Aggregate Base:** A crushed mineral aggregate base, treated or untreated, having a particle size distribution such that when compacted the interstices will provide enhanced drainage properties. It may include a granular drainable layer, untreated permeable base, free-draining base, and stabilized treated permeable base.

- **Asphalt Treated Permeable Base:** A permeable base containing a small percentage of asphalt cement to enhance stability. This may include asphalt-treated open-graded base or asphalt treat base; permeable.

- **Cement Treated Base:** A base course consisting of mineral aggregates blended in place or through a pugmill with a small percentage of portland cement to provide cementitious properties and strengthening. This may also include aggregate cement, cement-stabilized graded aggregate, and cement-stabilized base.

- **Lean Concrete Base:** A base course constructed of plant mixed mineral aggregates with a sufficient quantity of portland cement to provide a strong platform for additional pavement layers and placed with a paver.

- **Lime-Fly Ash Base:** A blend of mineral aggregate, lime, fly ash, and water combined in proper proportions and producing a dense mass when compacted.

- **Cement Treated Permeable Base:** An open-graded aggregate base treated with portland cement to provide enhanced base strength and reduce erosion potential.

Fabric Layers

- **Geosynthetics:** A planar material manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related materials. It serves six primary functions: filtration, drainage, separation, reinforcement, fluid blockage, and protection. Typical geosynthetics include geotextiles, geomembranes, and geogrids.

- **Geotextiles:** Permeable fabric made of textile materials used as filters to prevent soil migration, separators to prevent soil mixing, and reinforcement to add shear strength to a soil.

- **Geomembranes:** Impermeable polymer sheeting used as fluid barriers to prevent migration of liquid pollutants in the soil.

- **Geogrids:** Polymeric grid material having relatively high tensile strength and a uniformly distributed array of large apertures (openings). The apertures allow soil particles on either side to come in direct contact, thereby increasing the interaction between the geogrid and surrounding soils. Geogrids are used primarily for reinforcement.

Roadbed
The graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

Subbase
The layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. This may include granular subbase and unbound granular subbase. **Note:** The layer directly below the PCC slab is now called a base layer, not a subbase layer.

Subgrade
The top surface of a roadbed upon which the pavement structure and shoulders are constructed.

- **Select Material:** A suitable native material obtained from a specified source, such as a particular roadway cut or borrow area, having specified characteristics to be used for a specific purpose.

- **Soil Cement:** A mechanically compacted mixture of soil, portland cement, and water used as a layer in a pavement system to reinforce and protect the subgrade or subbase. It may also include cement-treated subgrade.

- **Lime Stabilized Subgrade:** A prepared and mechanically compacted mixture of hydrated lime, water, and soil supporting the pavement system that has been engineered to provide structural support.

Surface Course
One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called the “wearing” course.

ESTIMATING FORMULAS, CALCULATIONS, AND CONVERSION FACTORS

Estimating Formulas

- **Diluted Emulsified Asphalt**: 0.10 gal./sq. yd. (diluted - slow setting)
- **Bituminous Pavement**: 110 lbs./sq. yd./1" thickness
- **Aggregate Base Course (Class 2 and 6):** 133 lbs./cu. ft.
- **Filter Material**: 110 lbs./cu. ft.
- **Hydrated Lime**: 26.4 lbs./sq. yd./ 8 in. depth at 4% lime
- 39.6 lbs./sq. yd./12 in. depth at 4% lime
- 59.4 lbs./sq. yd./12 in. depth at 6% lime
- **Asphalt Rejuvenating Agent**: 0.15 gal./sq. yd. (diluted)
- **Asphalt Rejuvenating Agent:** 0.15 gal./sq. yd. (non-diluted asphalt rejuvenating agent for use with Item 404 - Heater and Scarifying Treatment)
- **Micro-Surfacing Seal Coat**: 35 lbs./sq. yd. (based on an average rut depth of ¾ inches)
- **Crack Sealant:** quantities of crack sealant were estimated based on the level of cracking and the following ratios. The quantities shown here are for information only.
    - Heavy: 2 tons per lane mile
    - Medium: 1 ton per lane mile
    - Light: 0.5 ton per lane mile
    - Very Light: 0.25 ton per lane mile

Conversion Factors

- 1 ton = 0.90718 metric ton
- 1 lb/cu. ft = 16.018 kg/cu. meter
- 1 psi/in. = 0.271 kpa/mm
- 0.10 gal./sq. yd. = 0.453 L/sq. meter
- 0.15 gal./sq. yd. = 0.70 L/sq. meter
- 110 lbs/sq. yd./one inch = 2.34 kg/sq. meter/25.4 millimeter
- 110 lbs/cu. ft. = 1762 kg./cu. meter
- 133 lbs/cu. ft. = 2130 kg/cu. meter
- inches = 50.8 mm or 50 mm (rounded for pavement design)
- inches = 101.6 mm or 100 mm (rounded for pavement design)
- $^1/_2$ inch = 12.7 mm or 12.5 (rounded for pavement design)
- A U.S. gallon (determined by fluid volume at 72°F, at sea level) of fresh water weighs exactly 8.3452641 lbs.

Incentive/Disincentive Calculations

$I/DP = (PF - 1) \times (QR) \times (UP) \times (W \div 100)$

Where: I/DP = Incentive/disincentive payment
PF = Pay factor
QR = Quantity in tons of HMA represented by the process
UP = Unit bid price of asphalt mix
W = Element factor from Table 105-2 of *CDOT's Standards and Specifications*

When AC is Paid for Separately UP Shall Be:

$UP = [(Ton_{HMA}) \times (UP_{HMA}) + (Ton_{AC}) \times (UP_{AC})] \div Ton_{HMA}$

Where: Ton_{HMA} = Tons of asphalt mix
UP_{HMA} = Unit bid price of asphalt mix
Ton_{AC} = Tons of asphalt cement
UP_{AC} = Unit bid price of asphalt cement

For the Joint Density Element:

$UP = UP_{HMA}$

Where: UP_{HMA} = Unit Bid Price of Asphalt Mix

When AC is Paid for Separately UP Shall Be:

$UP = [(BTon_{HMA}) \times (BUP_{HMA}) + (BTon_{AC}) \times (BUP_{AC})] \div BTon_{HMA}$

Where: $BTon_{HMA}$ = Bid tons of asphalt mix
BUP_{HMA} = Unit bid price of asphalt mix
$BTon_{AC}$ = Bid tons of asphalt cement
BUP_{AC} = Unit bid Price of asphalt cement

CHAPTER 1
INTRODUCTION

1.1 Introduction

The Colorado Department of Transportation (CDOT) has adopted the *AASHTO Interim Mechanistic-Empirical Pavement Design Guide (MEPDG) Manual of Practice* for pavement design and analysis along with the AASHTOWare Pavement M-E Design software, otherwise called the M-E Design software. The M-E Design software uses the methodology and pavement design models described in the *AASHTO Interim MEPDG Manual of Practice*. The pavement design models in the M-E Design software were calibrated and validated using extensive Colorado pavement performance data.

This manual presents the following information to assist CDOT pavement design engineers to perform pavement designs using the *AASHTO Interim MEPDG Manual of Practice* and the M-E Design software. CDOT has updated to M-E Design Version 2.3.1 for 2020.

- An overview of the AASHTO Pavement M-E Design procedure
- An overview of the M-E Design software
- Guidelines for obtaining all needed inputs for design/analysis
- Guidance to perform pavement design/analysis using the software
- Examples of pavement design using the Design Guide software

This guidance will assure adequate strength and durability to carry the predicted traffic loads for the design life of each project. Alternative designs (flexible and rigid) should be considered for each project and specific project conditions. The final design should be based on a thorough investigation of projected traffic, specific project conditions, life-cycle economics, and the performance of comparable projects with similar structural sections and conditions.

1.1.1 Electronic Documentation

CDOT is transitioning toward accepting all submittals, forms, project records and supporting documents in electronic format. This Manual reflects technology as of (date). Users should work in partnership with CDOT staff to continue to advance this effort in between Manual updates.

<u>Adobe Sign</u>
Adobe Sign is the electronic signature and professional seal software selected by CDOT and required for use on Project Records including Change Modification Orders (CMO), which facilitates automated workflows including the ability to route Project Records for acknowledgements, electronically sealing and/or signing. Adobe Sign is not the eSignature program selected for use on document requiring a CDOT Controller or State Controller signature (contracts).

<u>Deliverables Management</u>
CDOT uses a series of tools in the Bentley suite for design, construction and engineering documents. One of them is ProjectWise Deliverables Management. This is a cloud-based service that streamlines how a project team works with transmittals, submittals, and Requests

for Information (RFI). It provides improved visibility into these processes and also retains confidentiality when legally required.

ProjectWise Deliverables Management is utilized to ensure that documents are submitted, completed and processed on schedule. Functions include: ensuring delivery to correct parties, enabling faster reviews and responses, automating an audit trail thereby increasing accountability with detailed recordkeeping, connecting entire supply chain through a secure cloud platform and leveraging project dashboards to monitor workflows and evaluate project performance.

ProjectWise Deliverables Management is capable of handling reference files used in design.

Project Share

The Cloud-based software tool hosted in the Bentley / Microsoft Azure Cloud used for document collaboration. Project Share connects to and synchronizes with ProjectWise Explorer, such that files placed in a Project Share folder, which is synchronized with ProjectWise Explorer, are automatically copied to the same folder in ProjectWise Explorer. Note that Project Share is not used for DGN reference files in design.

ProjectWise Explorer

Bentley ProjectWise Explorer is the Electronic Document Management System (EDMS) for archiving all electronic Project Records set forth in the CDOT Record File Plans.

Definitions

Adobe Acrobat DC. The software selected by CDOT and required for use in order to create and/or modify a PDF (portable document format) Project Record, to retain a record in an ISO Compliant format. By using Adobe Acrobat DC tools, the software "Smart Scans" Project Records to meet state and federal legal requirements prior to archiving in ProjectWise Explorer.

Adobe Sign. The electronic signature and professional seal software selected by CDOT and required for use on Project Records including Change Modification Orders (CMO), which facilitates automated workflows including the ability to route Project Records for acknowledgements, electronically sealing and/or signing.

Electronic Document Management System. ("EDMS") ProjectWise Explorer which has been selected by CDOT as the EDMS for CDOT Project Records.

Form 950 "Project Closure". This CDOT form provides notice of financial closure of the project. It includes notification to the FHWA that the project is closed and includes an electronically generated Project Record retention date.

ISO Compliant. A Record retained in a format approved by the International Organization for Standardization, a worldwide federation of national standards which refers to the ISO 19005 series of standards with PDF/A-1 approved as a minimum. Archiving an electronic Record in an ISO Compliant format ensures that it can be read in one hundred years, regardless of the hardware or software used to create the record. An ISO Compliant Record replaces microfilm as a method of archiving.

Naming Convention. A thread of acronyms that allows the CDOT Project Record to be correctly named and located in the ProjectWise Explorer locally-hosted or cloud-based EDMS.

Project Records. Engineering, Design, Specialty Group, and Construction Records pertaining to CDOT projects, including change modification orders (CMO). See § 24-80-101(1), C.R.S. "Record" shall also mean information that is inscribed on a tangible medium or that is stored in an electronic or other medium. See § 24-71.3-102(13), C.R.S. For further clarification, see relevant CDOT Records File Plans pertaining to Project Records.

Project Share. The Cloud-based software tool hosted in the Bentley / Microsoft Azure Cloud used for document collaboration. Project Share connects to and synchronizes with ProjectWise Explorer, such that files placed in a Project Share folder, which is synchronized with ProjectWise Explorer, are automatically copied to the same folder in ProjectWise Explorer.

ProjectWise Explorer The Bentley software system utilized by the Department for archiving Project Records.

Record File Plan. CDOT's internal governing document developed by each division, program, or unit which contain the state and federal legal retention requirements for CDOT Records pertaining to the specific Records. Record File Plans include the correct location in ProjectWise Explorer for each Project Record.

Smart Scanning. The term CDOT uses to meet state and federal retention requirements for CDOT Project Records by utilizing Adobe Acrobat to make Project Records searchable, page aligned, and compressed. It also means archived in an ISO Compliant format. Note that some mediums, such as video files and image files cannot be archived in an ISO Compliant format. In this case, the files shall be retained in their original format.

CDOT Legal Requirements Regarding Record Retention

CDOT's legal requirements to retain project records extend not only to CDOT employees but also the consultants, contractors and local agencies who work on CDOT project records. As a public agency, CDOT is legally required under state and federal law to retain certain Project Records for specified time periods. These time periods are set forth in the CDOT Record File Plans.

Compliance with Procedural Directive 21.1 "Requirements for the Retention of Records for Specified Design, Construction, Engineering, and Specialty Groups (Paper and Electronic)" effective June 20, 2019.

General Reference to PD

CDOT's requirements for Project Records are set forth in Procedural Directive ("PD") 21.1 "Requirements for the Retention of Records for Specified Design, Construction, Engineering, and Specialty Groups (Paper and Electronic)" effective June 20, 2019. *The requirements of Procedural Directive (PD) 21.1 apply to CDOT employees and to contractors, consultants and* *local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records. It applies to all CDOT projects including local agency, P3, Innovative, Design-Build and CMGC projects.*

Applicability

Procedural Directive 21.1 shall apply to all divisions, offices, and regions of CDOT engineers and project staff who develop, handle, or receive records. It also applies to all projects, including but not limited to capital engineering projects, local agency, P3, Innovative, Design-Build (DB) and Construction Management General Contracting projects (CMGC). It applies to all consultants, contractors and local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records.

Archiving Project Records in ProjectWise.

All active and future Project Records shall be archived in Project Share / ProjectWise Explorer Electronic Document Management System on an ongoing basis rather than at the conclusion of the project.

Phases or milestones from scoping to project closure shall be established for archiving purposes. Record File Plans indicate the correct archive location for these records. They are located in the Governing Documents folder under "5 – Record File Plans". For external users, a link to this file is included in all project share sites.

CDOT's EDMS for Project Records

Bentley ProjectWise Explorer is the Electronic Document Management System (EDMS) for archiving all electronic Project Records set forth in the CDOT Record File Plans.

If project consultants are using Aconex, the PM and CDOT Resident Engineer must develop a phased approach to migrate records into ProjectWise Explorer on an ongoing basis within 45 days of the project final acceptance.

Record Retention Schedules for Project Records

CDOT's Record File Plans contain a list of the public records that are required to be retained, as well as the electronic folder in ProjectWise Explorer where they will be archived. A link to the CDOT Record File Plans is made available in each Bentley Project Share site. This link will provide access for consultants, contractors and local agencies to CDOT Record File Plans.

CDOT's project records are created and retained in electronic format unless the record has a retention period of 3.5 years or less from the Form 950 closure date. If the retention period is shorter, the Project Engineer along with the Region Finals Administrator shall make the determination to retain documents in paper form.

Project Records that are subject to the following categories must be retained for seven years from the Form 950 close date (may be longer if FEMA requirements apply):

- Major project (CMGC, DB, P3 or other innovative contract projects)
- Subject of internal or external audit
- Litigation hold
- Emergency funded

Project Records must be archived according to milestones established by the project engineer on an ongoing basis rather than at the conclusion of the project.

Smart Scanning (ISO Compliant Requirement)

Properly archiving Project Records means that they will be preserved in digital PDF format so that they can be read with original fidelity in one hundred years regardless of the hardware or software used to create them. This ensures that CDOT's most critical records with long-term or permanent retention requirements may be retained in digital form rather than paper or microfilm.

Project Records with retention periods longer than 3.5 years must be "Smart Scanned" prior to archiving. Training on Smart Scanning is available by registering through the Transportation Engineering Training Program ("TETP") website located here: https://www.codot.gov/programs/tetp Smart Scanning makes the Project Record searchable, compressed, page aligned, and in compliance with International Standard Organization's ("ISO") standard PDF/A-1b. Project Records which do not need to be Smart Scanned are the following:

1) Project Records approved by the Project Engineer and CDOT Finals Administrator to be submitted in paper form. The CDOT Finals Administrator and Project Engineer may determine that Project Records with a retention period of 3.5 years or less from the CDOT Form 950 closure date can be provided to CDOT in paper form.
2) Videos, photos, image files, and other media formats which cannot be converted to PDF. Certain files are unable to be Smart Scanned and must be placed in ProjectWise Explorer in their original formats.

Paper Record Retention

If paper Project Records have a retention period of 3.5 years or less from the Form 950 project closure date, they may be scanned and retained electronically or retained in paper format until they have met their retention period. A Destruction Form shall then be completed. Once approved, the records may then be shredded or disposed of.

Project Records in paper form are now retained by the Regions for archiving until the Records meet their retention period. Headquarters no longer receives a copy.

Naming Conventions for File Names

Use standard naming conventions (PD 21.1 Appendix "A") and as noted in Record File Plans. For questions on naming conventions, ask CDOT Finals Administrators.

Adobe Sign: CDOT's Electronic Signature Software for Project Records

Unless otherwise notified by the Chief Engineer, Adobe Sign is CDOT's approved electronic workflow signature software for "Project Records." This includes the use of Adobe Sign for sealing with the professional engineer seal (see Procedural Directive 508.1 below, which sets forth requirements for sealing). Adobe Sign may not be utilized for any document which requires a signature from the CDOT Controller or State Controller.

For all Project Records that do not require a CDOT Controller/State Controller signature, Adobe Sign shall be used for both eSignatures and eSeals on Project Records. Note that Adobe Sign is permissible for use on contract modification orders ("CMO") given that CMOs do not require a signature by the Office of the State Controller. Adobe Sign work flows for Project Records will significantly cut down time routing paper records for signature, and will automatically archive the signed Project Record in ProjectWise.

Local Agency Records

On Local Agency projects with CDOT oversight, Local Agencies follow their own record retention schedules that adhere to the Inter-Governmental Agreement with CDOT. However, specific documents in the CDOT Record File Plans are required to be retained by CDOT and must be provided to the CDOT Local Agency Coordinator by the local agency or its representative. CDOT uses Bentley Project Share for this purpose so that the Local Agency can transmit the project record to the CDOT Local Agency Coordinator using the project-specific Project Share site. The Local Agency Coordinator will then archive the project record utilizing the synchronization function in Project Share, and the document will automatically be archived in the correct ProjectWise Explorer folder.

CDOT Responsibilities

- Resident Engineers:
    - Must ensure that their staff are trained to properly archive records in the correct location and format.
    - Include a provision requiring compliance with PD 21.1 in all task orders.
    - Provide a copy of PD 21.1 with the Notice to Proceed.
- Project Managers:
    - Must fill out all attribute fields known at the time of project creation and thereafter when modifications occur. Attribute fields are filled out in SAP CJ20N (and, when launched, On Track).
- Finals Administrators:
    - Responsible for creating three electronic plan sets in PWZ Explorer: Award Set with watermark, Record Set with watermark, As-Constructed Plan with watermark.
- Records Coordinators
    - Records Coordinators are selected by their Appointing Authority to handle Project Records. Their responsibilities are set forth in PD 51.1 and in the Overview of Records Management and Records Coordinator Certification available through SAP/My Learning.
- Engineering Contracts:
    - Must include in contracts that PWZ Explorer is CDOT's EDMS for Project Records.
    - Standards and Specifications Unit must include relevant requirements of PD 21.1 in project special provisions by January 2020 (deadline extended to July 30, 2020).

Procedural Directive (PD) 508.1 "Requirements for the Use of the Professional Engineer's Seal"

General Description

PD 508.1 defines the procedures for the use of the Professional Engineer seal by CDOT employees, consultants, contractors and local agencies who perform engineering work for CDOT.

All CDOT, local agency and consulting Engineers must utilize electronic sealing (rather than mechanical sealing on paper) by January 2020 unless an exception request and approval is granted by the Chief Engineer.
Beginning January 2021, no exemptions will be granted to the electronic sealing requirements.

Applicability

The requirements of PD 508.1 apply to CDOT employees and to contractors, consultants and local agencies who develop, transfer, augment, or are in any way involved with or responsible for CDOT records. It applies to all CDOT projects including local agency, P3, Innovative, Design-Build and CMGC projects. PD 508.1 must be read together with PD 21.1. Sealed Project Records must be retained in ProjectWise Explorer in conformance with the CDOT Record File Plans.

Engineering designs, Record Sets and Contract Modification Orders, contract drawings and specifications for CDOT projects prepared by COOT employees or by contractors or consultants who perform work for CDOT, or by local agencies who perform work for projects with COOT oversight and/or funding or federal funding passed through CDOT, shall be Sealed in accordance with Procedural Directive 508.1.

Legal Requirements for Sealing

CDOT's Sealing requirements are dictated by and adhere to the Sealing requirements for licensed engineers set forth in the AES Board Rules, 4 CCR 730-1, which have the effect of law. The AES Board Rules dictate which documents require a Seal. The AES Board Rules have the effect of law. These include Record Sets, Contract Modification Orders, VECP's M&S Standards and changes thereto. To limit the scope of responsibility to one or more disciplines, a statement must be included adjacent to the Seal which limits responsibility to those portions of work done, such as: "My responsibility with respect to this standard plan revision is limited to-----------" Transmittal and storage of all CDOT project records shall adhere to the requirements of Procedural Directive 21.1and CDOT's Record File Plans. The Sealed Record Set is required to be deposited in CDOT's ProjectWise Explorer. This will constitute the official record and will be retained permanently.

Responsibilities

- Engineer in Responsible Charge:
  - Must seal respective documents for work within their scope of work, including local agencies. Must ensure that all seals are obtained on the record set. This includes the limitation of scope for each seal.
  - The Engineer in Responsible Charge on a local agency project with COOT oversight is required to Seal all documents within the scope of their work. They shall be responsible for depositing the Seal Record Set into ProjectWise within 45 days of the award.

- CDOT Resident Engineer:
  - Is responsible for ensuring that all documents requiring Seals are obtained within 45 days of award of the construction project and archived in the correct PWZ Explorer folder.

Exclusions from Sealing Requirements

Manufactured Components

Engineers may specify manufactured components (e.g., impact attenuators, products on the Approved Product List ("APL")), which are exempted by statute as part of design documents. Manufactured components for the purposes of this Procedural Directive shall consist of such items as a pump, motor, steel beam or other types of items that are manufactured in multiple units for selection and use in projects which must be designed by Engineers. Systems of manufactured components which are specific to a particular use or application must also be designed by an Engineer. The Engineer may show the manufactured component on the drawing or document and is responsible for the correct selection and specification of the manufactured component but is not responsible for the proper design and manufacture of the manufactured component.

Stormwater Management Plans

- Stormwater Management Plans (SWMPs) and Erosion/Sediment Control Plans are excluded from the Seal requirement. Stormwater Management Plan sheets that do not contain engineering information (e.g. hydrology, hydraulics) are not considered "engineering drawings"; therefore, Sealing by a professional engineer is not required.
- Engineering features (e.g., ditches, storm sewer and permanent water quality facilities) required for the management of stormwater on the project shall be included in the plans on separate sheets as details with the associated information which would require Sealing in accordance with this Directive.

1.2 Scope and Limitations

1.2.1 Limitations

Design of the pavement structure includes the termination of the thickness of subbases, bases, and surfacing to be placed over subgrade soils. An important aspect of this design is the selection of available materials that are most suited to the intended use. Their grouping in horizontal layers under the pavement, from poorer layers on the bottom to better layers on the top, should be such that the most benefit will be derived from the inherent qualities of each material. In establishing the depth of each layer, the objective is to provide a minimum thickness of overlying material that will reduce the unit stress on the next lower layer and commensurate with the load-carrying capacity of the material within that layer.

The design of the roadbed cross-section is not an exact science. With many variables to be correlated, reducing the problem to exact mathematical terms applied to structures is extremely difficult. Present practice, as discussed herein, stems from mechanistic procedures and empirical relationships developed from test tracks and other pavement experiments, as well as, the observation of pavements under service throughout the state. Research continues on this subject and current design methods may be subject to frequent modification.

1.2.2 Scope

Pavement structure sections, except for experimental construction for research, are to be designed using methods or standards described in **Table 1.1 Recommended Pavement Design Procedures.** Although M-E Design allows pavement design and analysis of seventeen pavement types, not all of these pavement types have been calibrated for Colorado conditions. Furthermore, this design procedure does not include performance prediction models for thin and ultra-thin concrete overlay designs. Designers are advised as much as possible to follow recommendations presented in **Table 1.1 Recommended Pavement Design Procedures** for selecting appropriate pavement design/analysis methodology for a given pavement type.

Table 1.1 Recommended Pavement Design Procedures

Pavement Type	Design Methodology	
	CDOT 2018 Pavement M-E Design Manual	CDOT 2014 Pavement Design Manual (18k ESAL Design)
New HMA	✓	
Flexible Overlays of Existing HMA	✓	
Flexible Overlays of Existing Rigid	✓	
New Rigid	✓	
PCC Overlays of Existing Rigid	✓	
Thin and Ultrathin Concrete Overlay		✓
Concrete Pavement Restoration	✓	
Flexible Pavement for Intersections	✓	
Rigid Pavement for Intersections	✓	

1.3 Overview of AASHTO Pavement Mechanistic-Empirical Design Procedure

The AASHTO Pavement M-E Design Procedure is based on mechanistic-empirical design concepts. This means the design procedure calculates pavement responses such as stresses, strains, and deflections under axle loads and climatic conditions, and accumulates the damage over the design analysis period. The procedure empirically relates calculated damage over time to pavement distresses and smoothness based on performance of actual projects in Colorado. More details are found in the following documents.

- AASHTO, *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice*, July 2008, Interim Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

- AASHTO, *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide*, November 2010, American Association of State Highway and Transportation Officials, Washington, DC, 2010.

- NCHRP, 1-37A Project. *2002 Design Guide: Design of New and Rehabilitated Pavement Structures*, National Cooperative Highway Research Program, National Academy of Sciences, DC, 2004.

The pavement design computations using the M-E Design procedure and software are an iterative process as shown in the flowchart in **Figure 1.1 M-E Design Process**. The software provides:

- A user interface to input design variables
- Computational models for month by month analysis and performance prediction
- Results and outputs from the analysis for decision making
- Outputs in both pdf and Microsoft Excel formats suitable for use in design reports

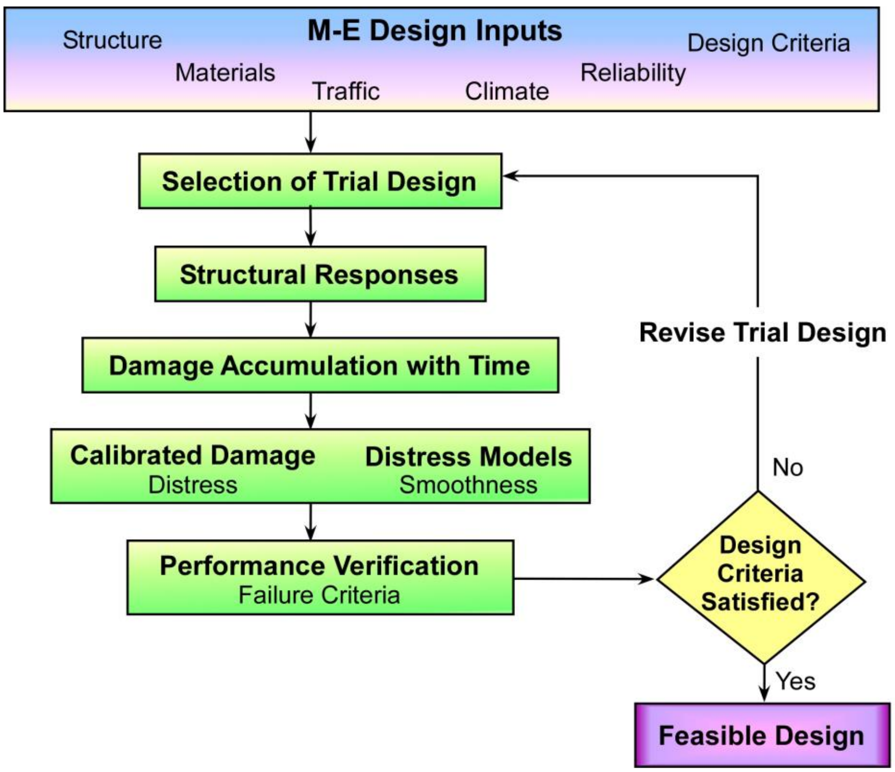


Figure 1.1 M-E Design Process

The design iterative process with the M-E Design procedure involves the following key steps:

1. The designer develops a trial design and obtains all inputs.

2. The software computes the traffic, climate, damage, key distresses (fatigue cracking, rutting, joint faulting, etc.), and International Roughness Index (IRI) over the design life on a month by month basis for concrete and a two week basis for HMA pavement.

3. The predicted performance (distress and IRI) over the design life is compared to the design performance criteria at a desired level of design reliability. Does the design pass or fail to meet the design reliability for each distress and IRI?

4. The design may be modified as needed to meet performance and reliability requirements.

1.4 Overview of AASHTOWare Pavement M-E Design Software

The AASHTOWare Pavement M-E Design software is a production-ready software tool for performing pavement designs using the methodology described in the AASHTO *MEPDG Manual of Practice*. The M-E Design software performs a wide range of analysis and calculations in a rapid, easy to use format. With its many customized features, the M-E Design software will help simplify the pavement design process and result in improved, cost-effective designs. The following subsections provide a brief overview of the process involved in installing, uninstalling, and running the M-E Design software.

A very detailed and comprehensive user manual for the M-E Design software is available with the software. Since the details of this process are likely to change over time, they are not repeated here. The HELP document can be easily obtained in two ways:

- From the Windows Start menu, click '*All Programs'* and then select the '*AASHTO DARWin-ME'* folder, refer to **Figure 1.2 Location of M-E Design Software HELP Document**.

- Press the '*F1 key'* after opening the software, see **Figure 1.3 M-E Design Software Default Window** and **Figure 1.4 M-E Design Software HELP Document**.

1.4.1 Installing M-E Design Software

For more information on installing the M-E Design software files, minimum software requirements, and licensing agreements, contact the CDOT IT System Administrator or refer to the M-E Design software HELP document.

1.4.2 Uninstalling M-E Design Software

Never just delete the various files of the M-E design software. Always uninstall the software using the procedure outlined in the M-E Design software HELP document. For more information of uninstalling the M-E Design software files, contact the CDOT IT System Administrator or refer to the M-E Design software help document.

Note: This process does not remove the :hcd weather station files under the folder. This folder must be manually deleted if desired. If existing old MEPDG weather station files exist, it is recommended to remove all of the files and then download the new weather station files.

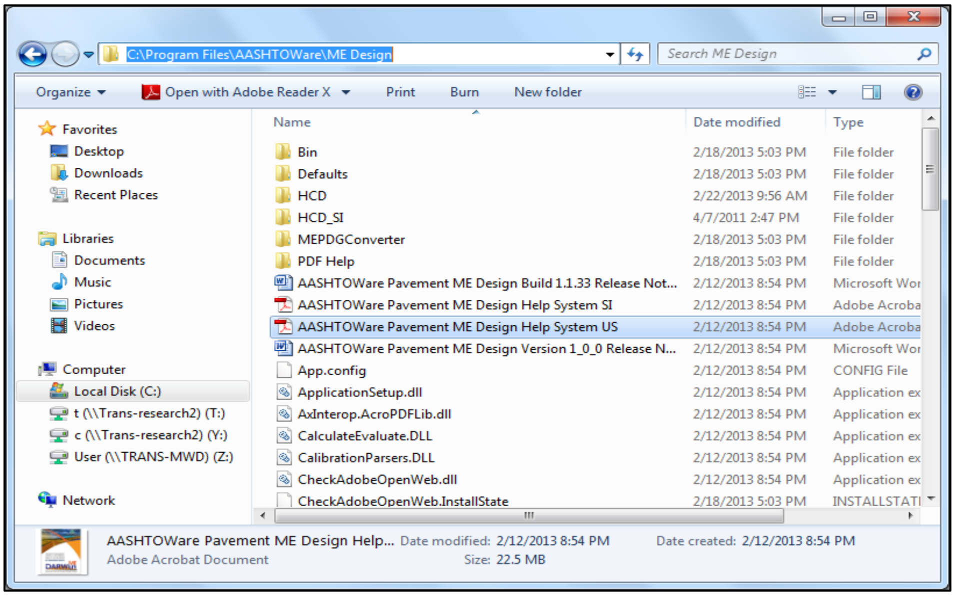


Figure 1.2 Location of M-E Design Software HELP Document

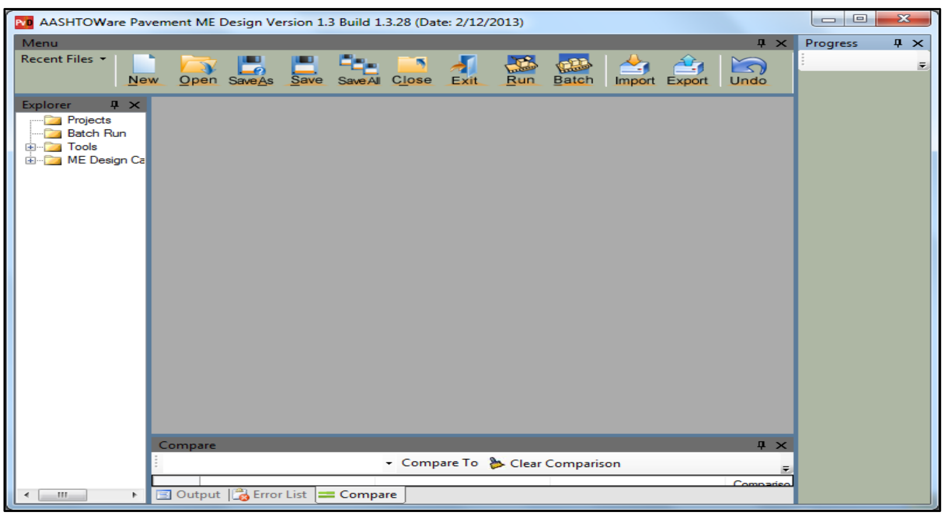


Figure 1.3 M-E Design Software Default Window

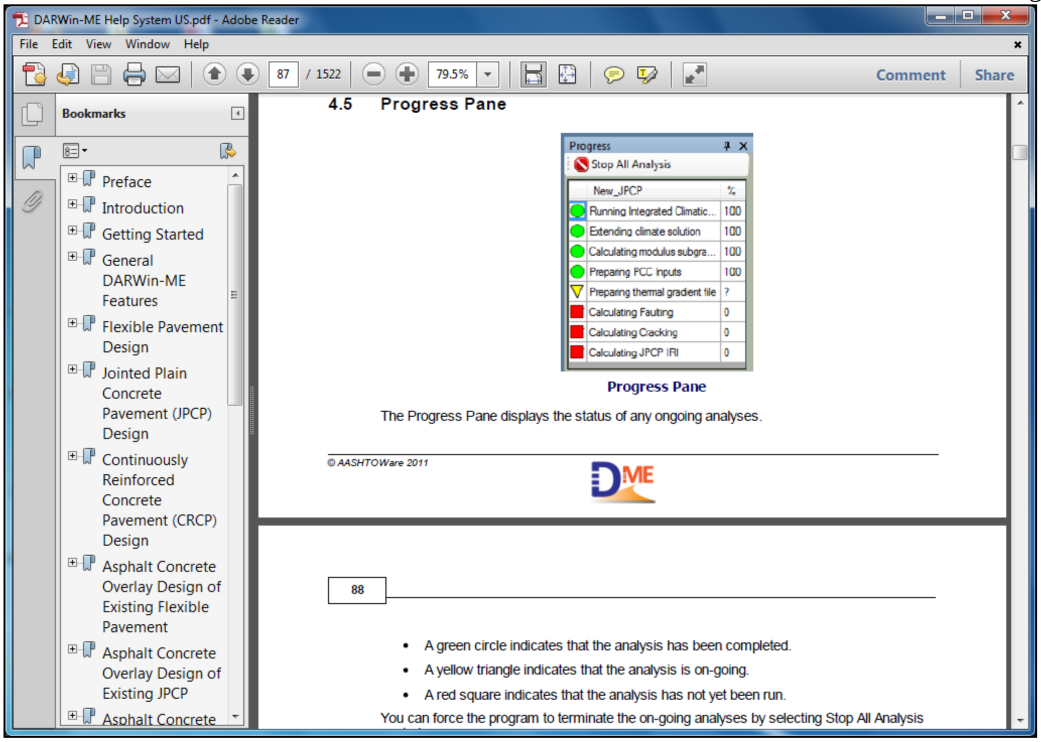


Figure 1.4 M-E Design Software HELP Document

1.4.3 Running M-E Design Software

The M-E Design program will be added to your Windows Start menu during installation and an icon will be added to the PC desktop.

Click the '*Start'* button in the bottom left corner of your screen to find the M-E Design software.

1. Go to the '*Programs'* option to see a list of folders and programs.
2. Select the '*DARWin-ME'* folder and click on the design guide icon.

The program can also be run by double-clicking the '*M-E Design'* icon on the desktop. The software opens with a splash screen shown in **Figure 1.5 M-E Design Software Splash Screen**. A new file must be opened for each new project, much like opening a new file for each document on a word processor or other standard Windows applications. A maximum of ten projects can be opened together by clicking the '*Open Menu'* in M-E Design (see **Figure 1.6 Open M-E Design Projects**). Select '*New'* from the menu on the tool bar to open a new project. A typical layout of the program is shown in **Figure 1.7 M-E Design Software Main Window** and **Figure 1.8 M-E Design Software Project Tab.** As of January 2020 all M-E Design templates and database files (structural/design, traffic, climate, HMA, PCCP, etc) are available in on CDOT's website at https://www.codot.gov/business/designsupport/materials-and-geotechnical/manuals/pdm.

The user first provides the general project information and the inputs for three main categories: traffic, climate, and structure. All inputs for the software program are color coded as shown in **Figure 1.9 M-E Design Software Color-Coded Inputs to Assist User Input Accuracy.**

Input screens that require user entry of data are coded red. Those that have default values but not yet verified and accepted by the user are coded yellow. Default inputs that have been verified and accepted by the user or when the user enters design-specific inputs are coded green. The program will not run until all input screens are either yellow or green.

The user may choose to run the analysis by clicking on the '*Run'* button after all inputs are provided for the trial design. The software will execute the damage analysis and the performance prediction engines for the trial design's input. The user can view input and output summaries created by the program when the execution of the run is complete. The program creates a summary of all inputs and provides an output summary of the distress and performance prediction in both tabular and graphical formats. All charts are plotted in both pdf and Microsoft Excel formats and may be incorporated into electronic documents and reports.

Figure 1.5 M-E Design Software Splash Screen

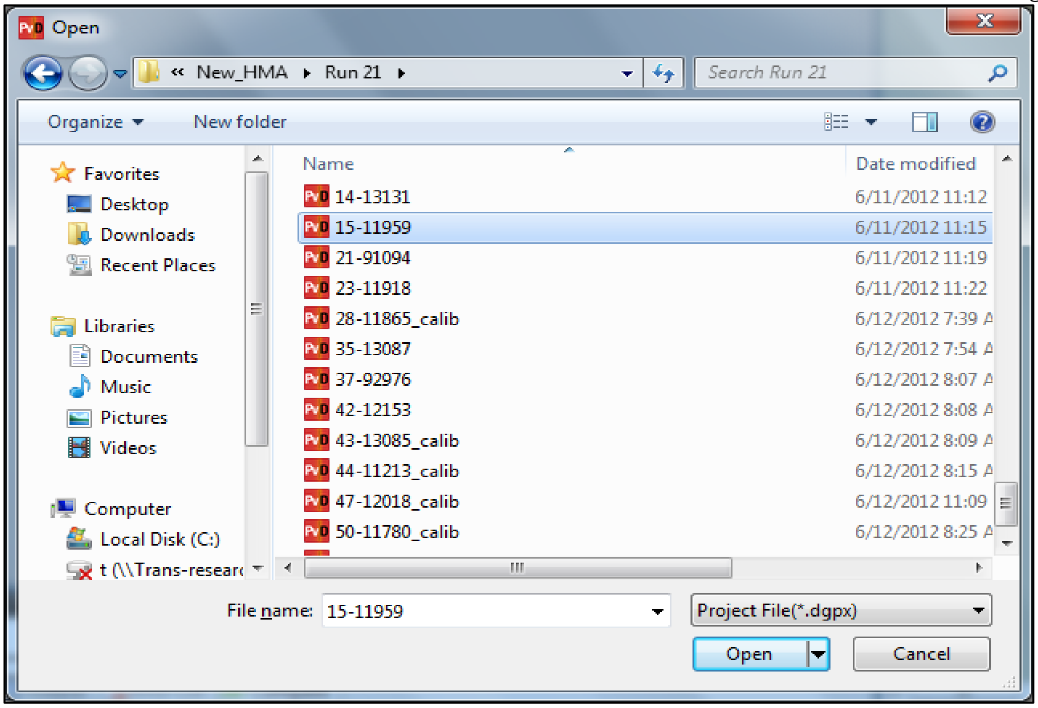


Figure 1.6 Open M-E Design Projects

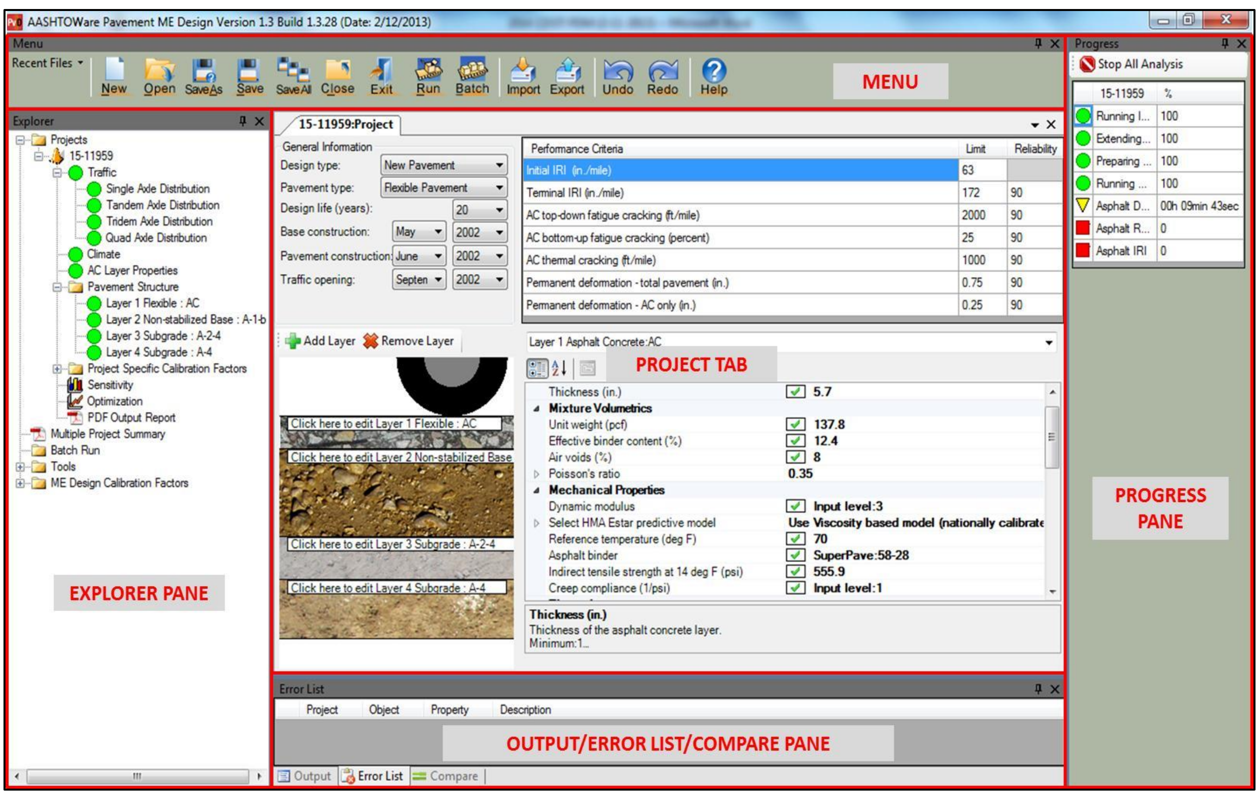


Figure 1.7 M-E Design Software Main Window

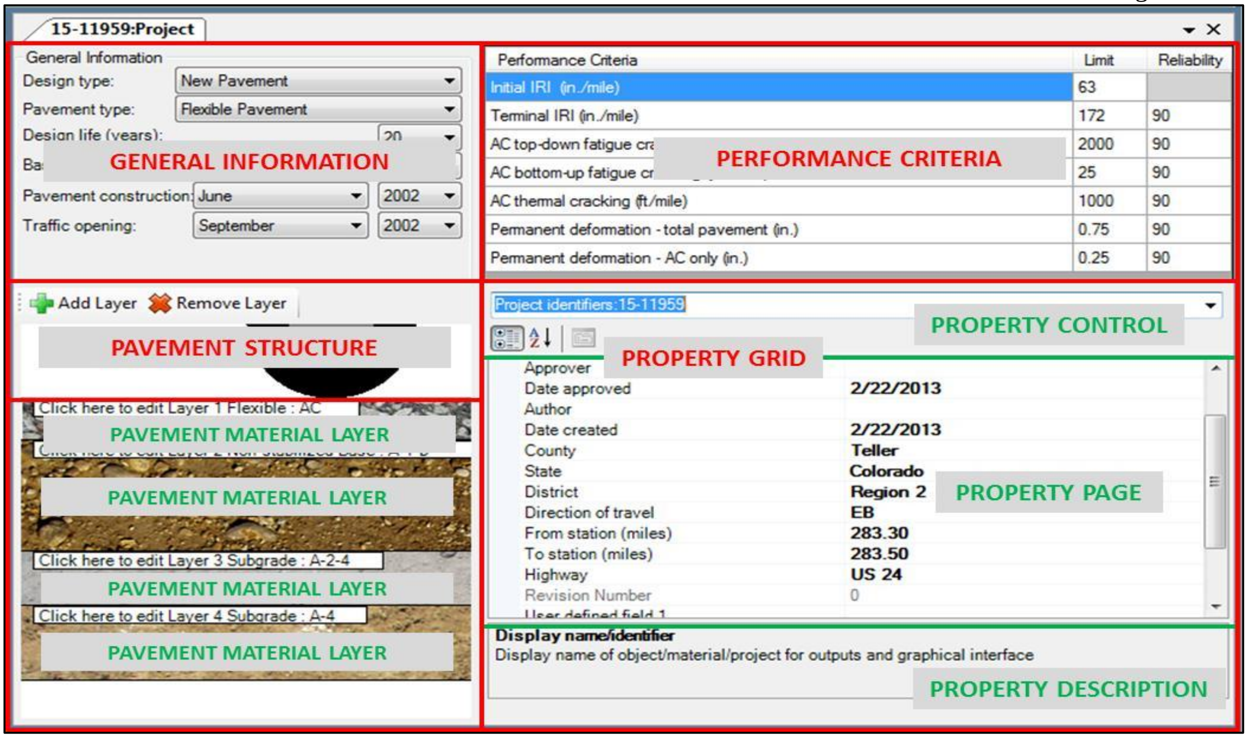


Figure 1.8 M-E Design Software Project Tab

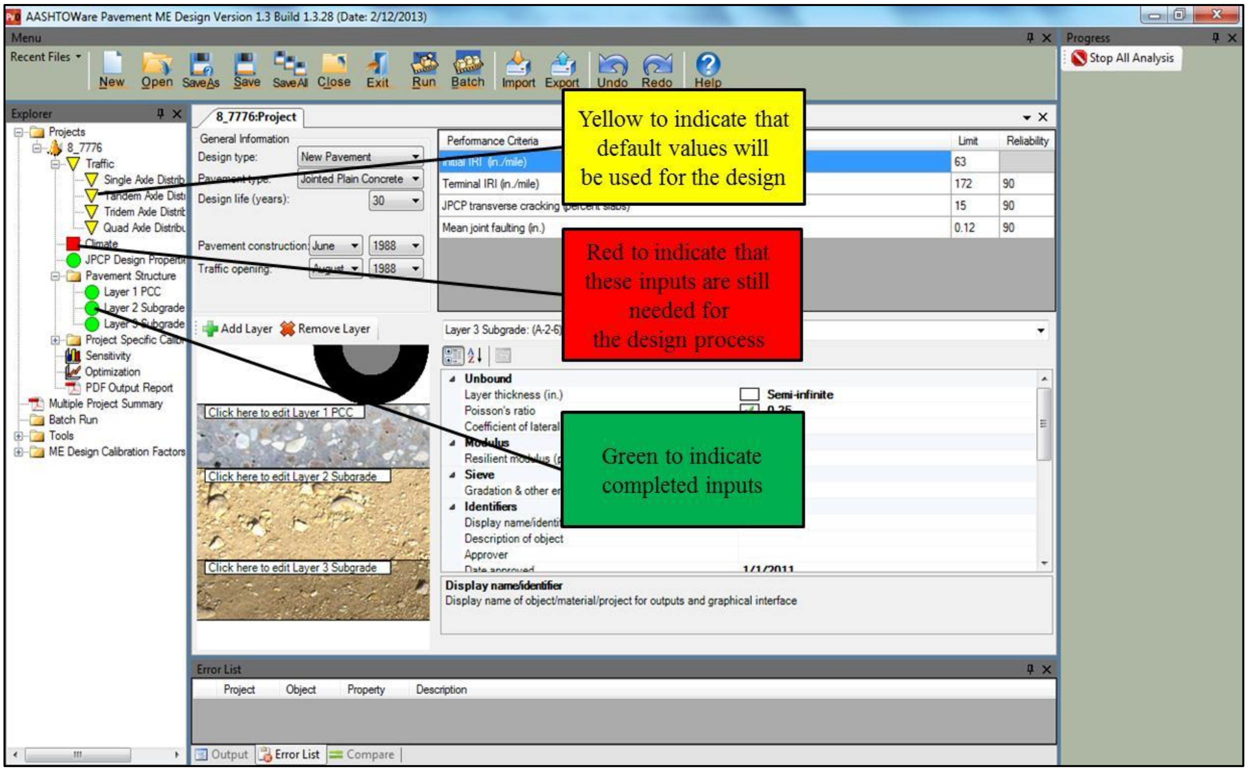


Figure 1.9 M-E Design Software Color-Coded Inputs to Assist User Input Accuracy

1.5 Working with the M-E Design Database

M-E 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 to data from individual projects such as pavement layers, materials, traffic, climate, backcalculation, etc. This section briefly describes how to set-up a M-E Design database in both MS SQL and ORACLE environments.

Download and Access Instructions
Blank M-E Design databases for MS SQL and ORACLE can be found in the Database Resource Documents section at http://www.me-design.com/. The user must have a valid user name and password to access the website. The login credentials will be supplied by AASHTO at the time of software purchase.

Database Installation
The following sections describe the installation process for creating a blank M-E Design database.

Installation Requirements
The requirements for installing and creating a blank M-E Design database are as follows:

- A user with administrative privileges on the target machine will be required to set up the M-E Design database.
- The maximum size of the M-E Design database shall be no greater than 10 GB.
- ORACLE 10g Release 2 or ORACLE Client 10g Release 2 or greater (contains the ORACLE Provider for OLEDB)
- Microsoft SQL Server 2005 or Express (and later versions)

Once the database is installed, the user can open the M-E Design software and select '*Open M-E Design'* with a data base connection check box (see **Figure 1.10 M-E Design Software Splash Screen Showing Database Login Location.**)

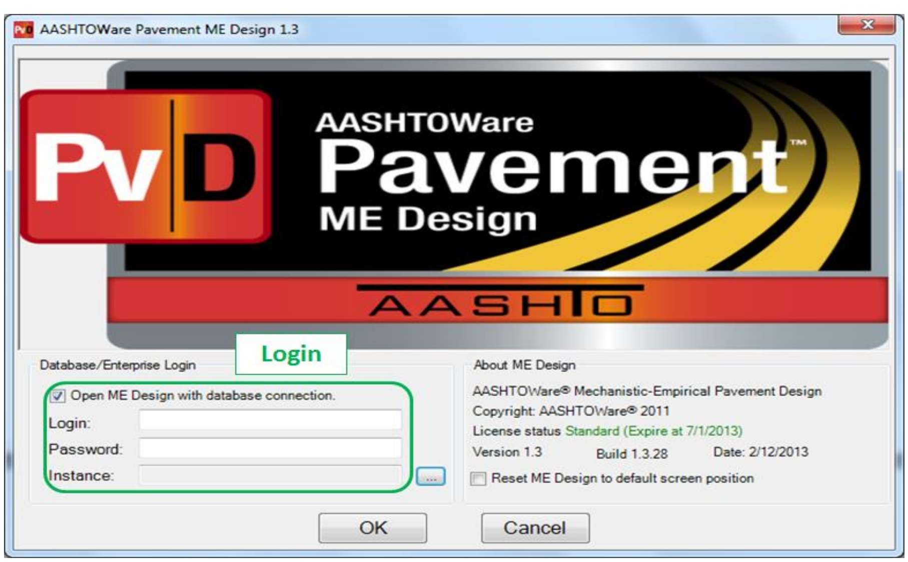


Figure 1.10 M-E Design Software Splash Screen Showing Database Login Location

Enter the Login name and Password supplied by the CDOT IT Department to access the M-E Design database, see **Figure 1.11 M-E Design Software Splash Screen Showing Database Login Information**.

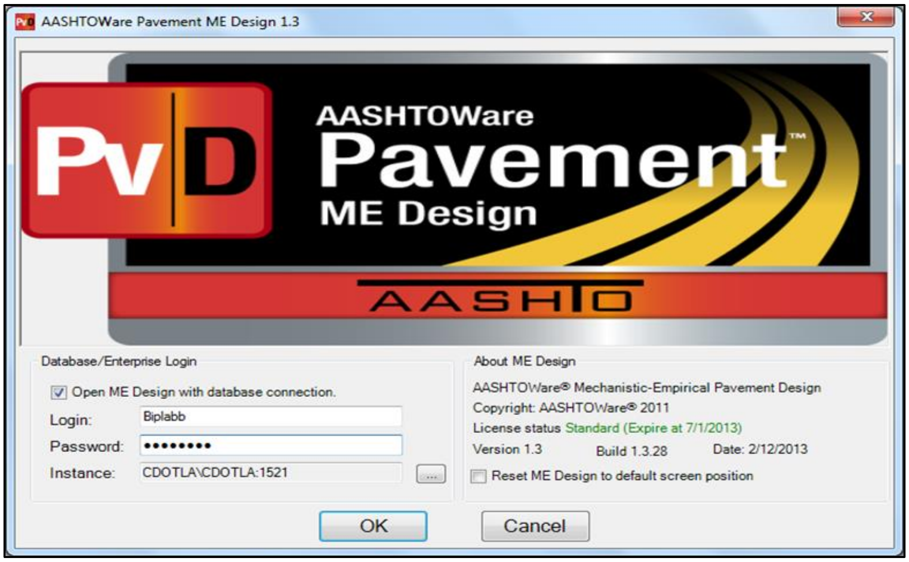


Figure 1.11 M-E Software Splash Screen Showing Database Login Information

1.5.1 Saving to M-E Design Database

This section will discuss how to save M-E Design elements to the database. It will also highlight the differences in how the elements are saved on each screen and supply screenshots for each example. **Note:** In order for the '*Save to Database*' option to be available, the user must connect to a M-E Design database during the login process.

Saving Projects

When a user saves a project, all elements of the project are saved in the database. If any of the project elements have an error, the user will be informed of the error with a message box and asked to correct the error before continuing. There are two ways to save a project to the database:

1. Right click on the project name under the '*Projects*' node and select '*Save to Database*' (see **Figure 1.12 Saving and Entire Project to M-E Design Database (Option 1)**).

2. Click to highlight the project name under the '*Projects*' node and click the '*Insert*' icon on the menu bar across the top of the application (see **Figure 1.13 Saving an Entire Project to the M-E Design (Option 2)**).

If the project contains no errors in the message, '*Project Inserted Successfully*' will pop up (see **Figure 1.14 Window Showing Successful Project Save**). Click '*OK*' to close the message box.

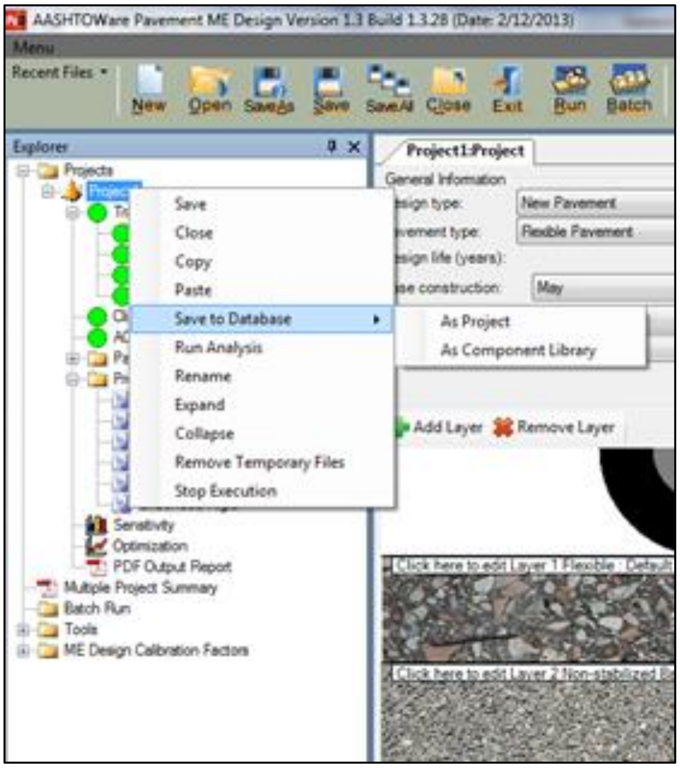


Figure 1.12 Saving an Entire Project to the M-E Design Database (Option 1)

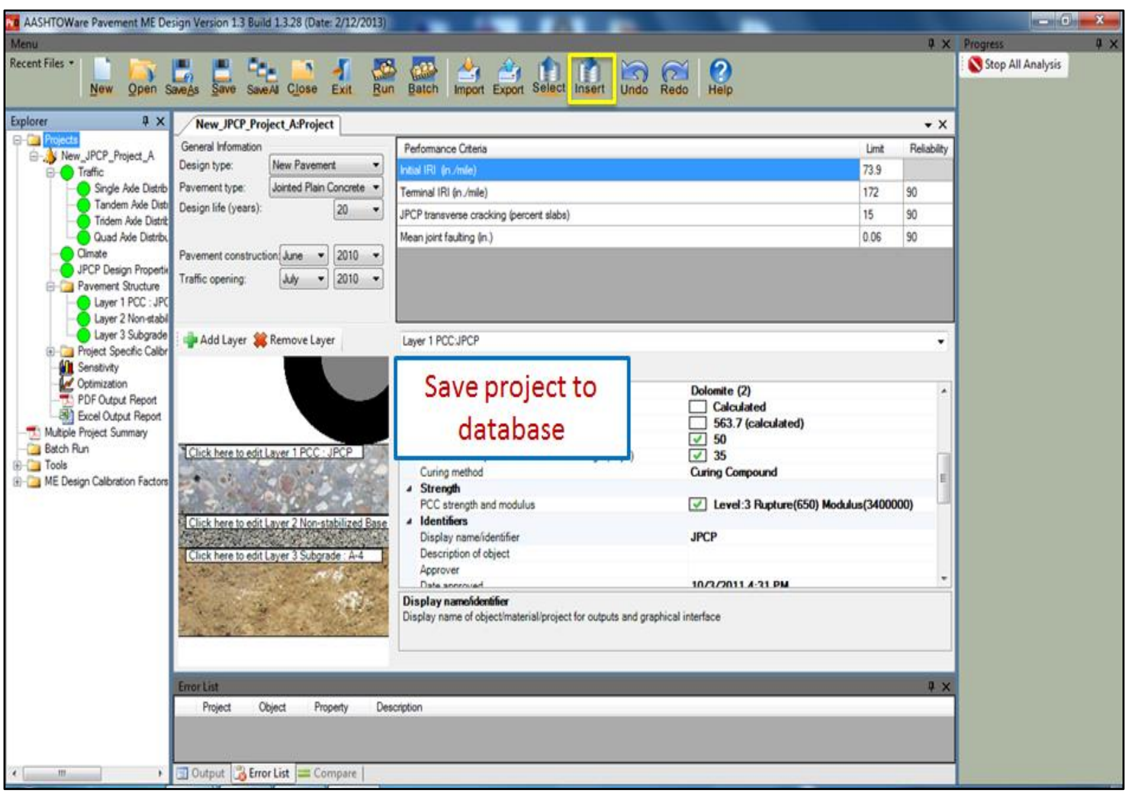


Figure 1.13 Saving an Entire Project to the M-E Design Database (Option 2)

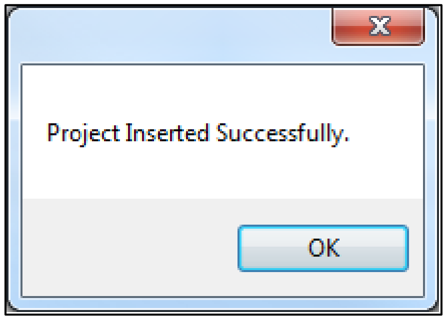


Figure 1.14 Window Showing Successful Project Save

Once a project is saved to the database, the project cannot be saved again under the same name. Only project elements can be "saved over" or updated once they exist in the database. To change the '*Display name/Identifier*' of your project, right click on the project title in the Explorer pane and select '*Rename*' (see **Figure 1.15 Changing the Project Display Name/Identifier**). Chose a new name for your project and then right click on the project in the Explorer and select '*Save to Database*'. The project will now save with a new name.

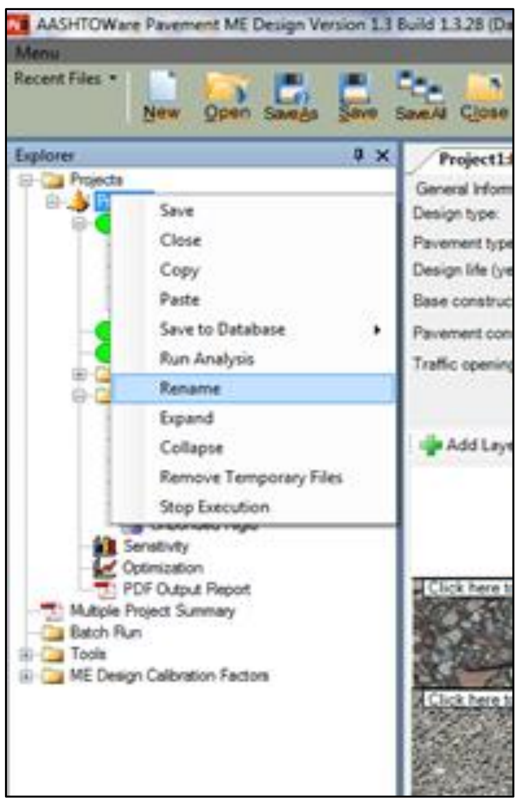


Figure 1.15 Changing the Project Display Name/Identifier

Saving Project Elements

Saving project elements is similar to the steps described in the section titled Saving Projects. Project elements include but are not limited to the following:

- Traffic
    - Single axle distribution
    - Tandem axle distribution
    - Tridem axle distribution
    - Quad axle distribution
- Climate
- Any layers added under "*Pavement Material Layers*" node
- Backcalculation

There is one primary difference between saving an entire project and saving elements within the project. Unlike projects, project elements can be saved over and over again without having to modify any element identifiers. This means if the user wants to save a project element such as '*Traffic*', make changes to it, and save it again, the program will update the project with the new traffic information instead of creating a new one.

All the elements described above have a '*Save to Database'* method associated with them, with a few special cases for traffic and its associated elements. The traffic element provides two unique saving methodologies.

1. Right clicking on the '*Traffic'* node and selecting '*Save to Database'* will save information under the '*Traffic'* node only (see **Figure 1.16 Saving Traffic Data**).

2. The user may also elect to double click on the '*Traffic'* node which will open the traffic interface. The user can then right click on any of the views within the interface including vehicle class distribution and growth, monthly adjustment, or axles per truck; and select '*Save to Database'* to save the applicable traffic element to the database (see **Figure 1.17 Saving Specific Traffic Elements**).

Note: This is the only way to save these particular traffic elements independently as they do not appear in the Explorer tree.

In contrast, saving any one of the axle load elements automatically saves all the others as well. **Figure 1.18 Saving Axle Load Distribution Elements** shows how to save axle load distribution elements in the M-E Design database. If the axle load distribution contains no errors, the message '*Axle Load Inserted Successfully'* will pop up (see **Figure 1.19 Window Showing Successful Axle Load Distribution Save**). Click '*OK*' to close the message box.

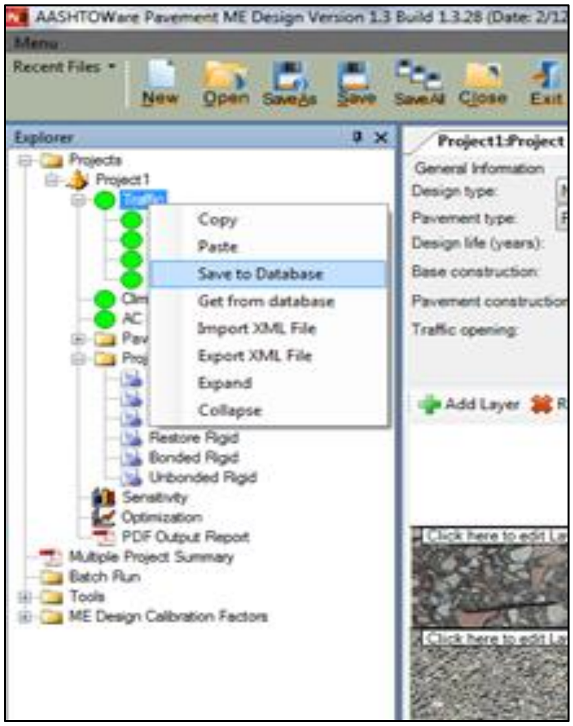


Figure 1.16 Saving Traffic Data

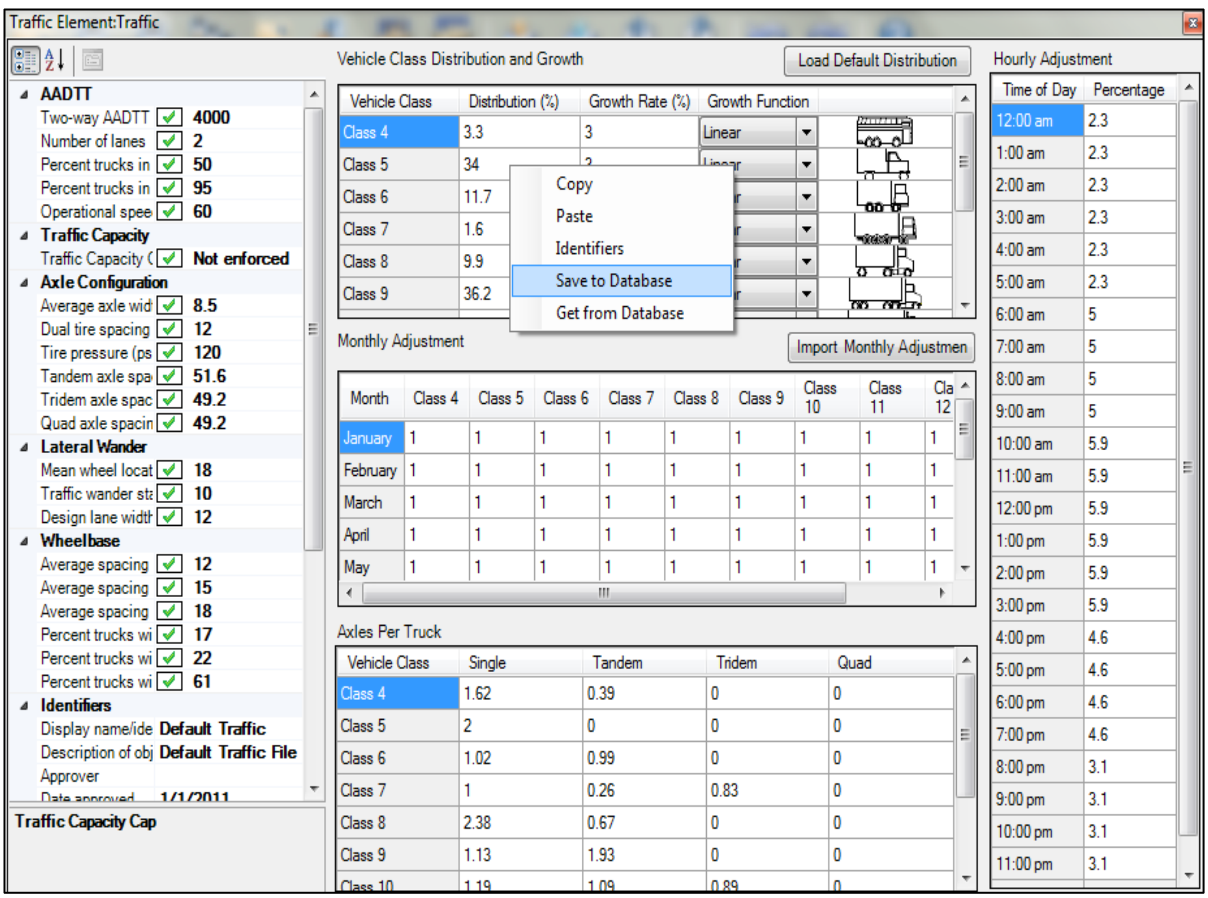


Figure 1.17 Saving Specific Traffic Elements

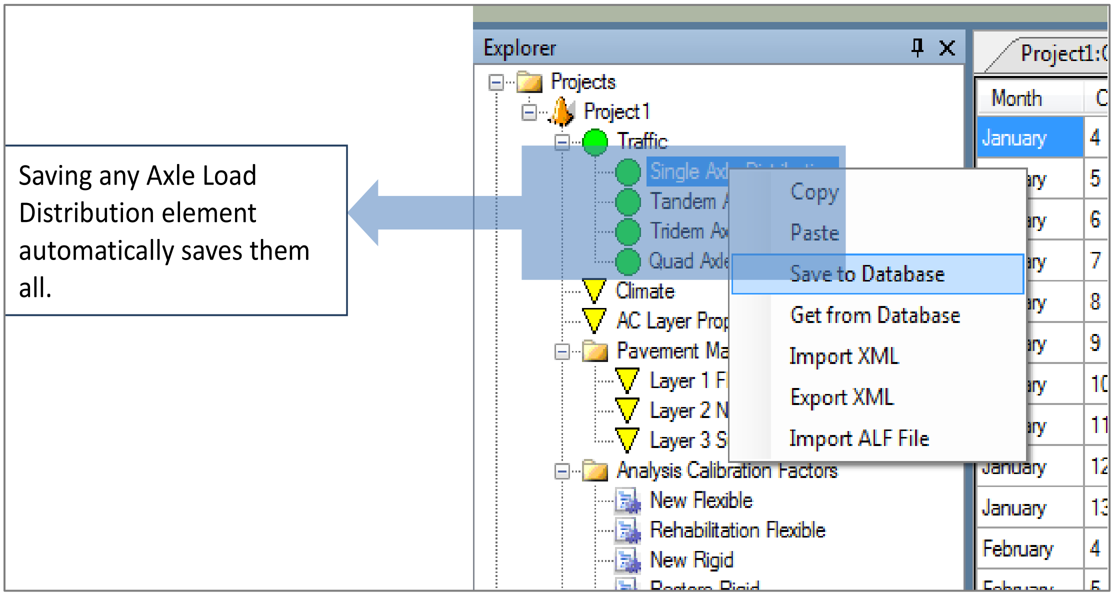


Figure 1.18 Saving Axle Load Distribution Elements

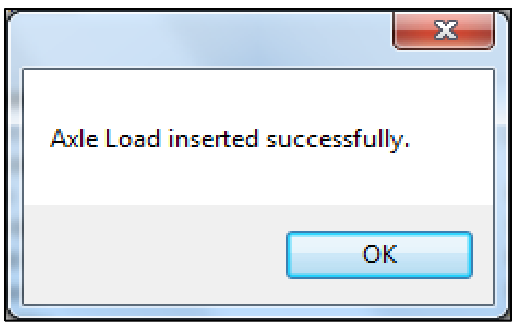


Figure 1.19 Window Showing Successful Axle Load Distribution Save

If the user receives the following error message shown in **Figure 1.20 Error Saving Axle Load Distribution** while saving the project element, then either the user needs to change the existing name of the element/project they are trying to save, or fill in the '*Display Name/Identifier'* field for the element.

This means the user needs to open the axle load distribution interface, right click, and select '*Identifiers'* (see **Figure 1.21 Defining Identifiers for Axle Load Distribution**). The user can fill in the '*Display Name/Identifier'* field shown in **Figure 1.22 Editing Display Name/Identifiers for Axle Load Distribution** and '*Close'* the window. Now the axle load distribution element is saved to the database.

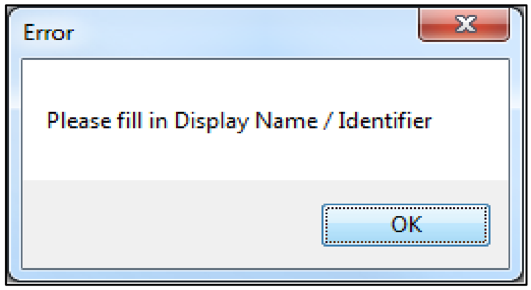


Figure 1.20 Error Saving Axle Load Distribution

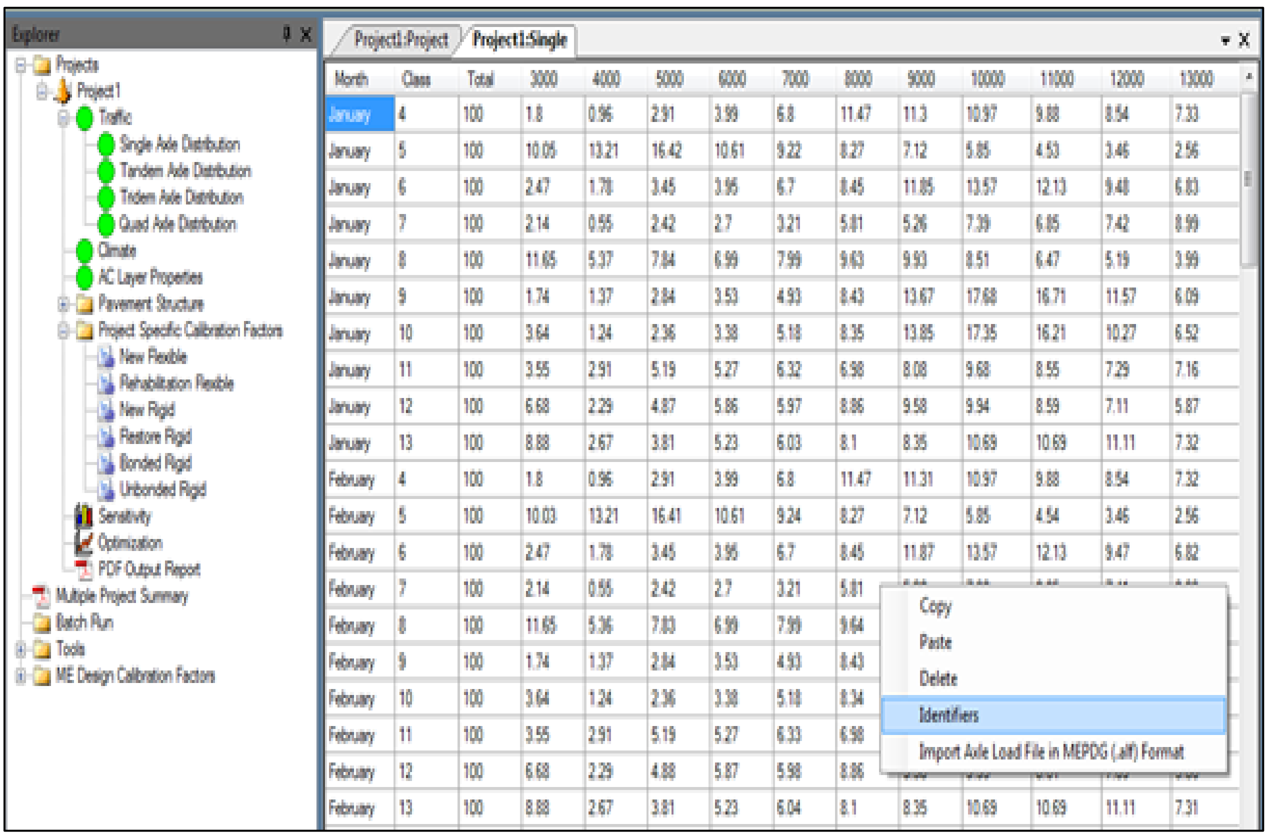


Figure 1.21 Defining Identifiers for Axle Load Distribution

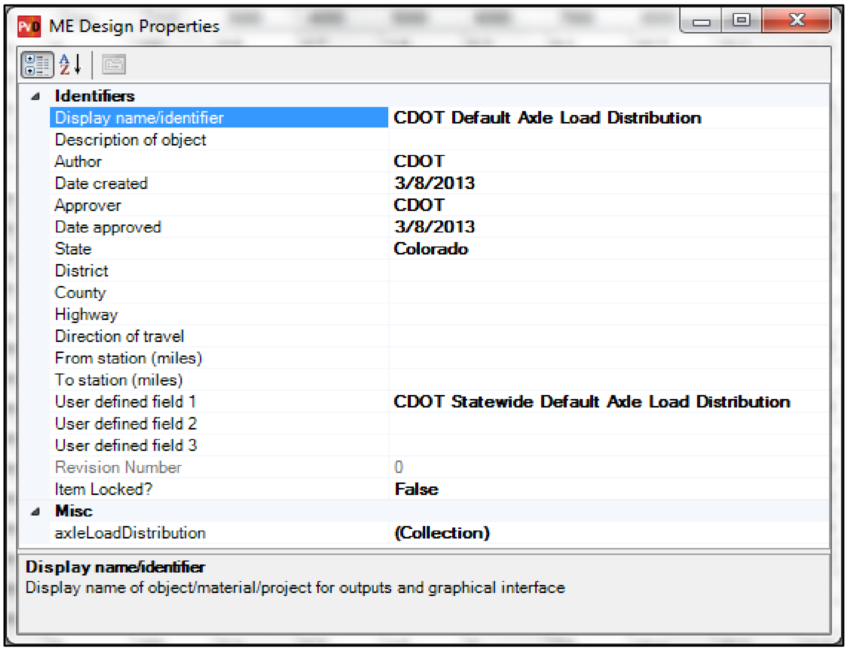


Figure 1.22 Editing Display Name/Identifiers for Axle Load Distribution

1.5.2 Retrieving or Importing from M-E Design Database

The data import process works similar to the save process in which the user should right click on the project or element they wish to import and select '*Get from Database'*. This will load the database information into the appropriate project.

Importing a Project

There are two ways to import an entire project from the database:

1. Right click on the project name under the '*Projects'* node and select '*Get from Database'*.

2. Click to highlight the project name under the '*Projects'* node and click the '*Select'* icon on the menu bar across the top of the application (see **Figure 1.23 Importing an Entire Project from M-E Design Database**).

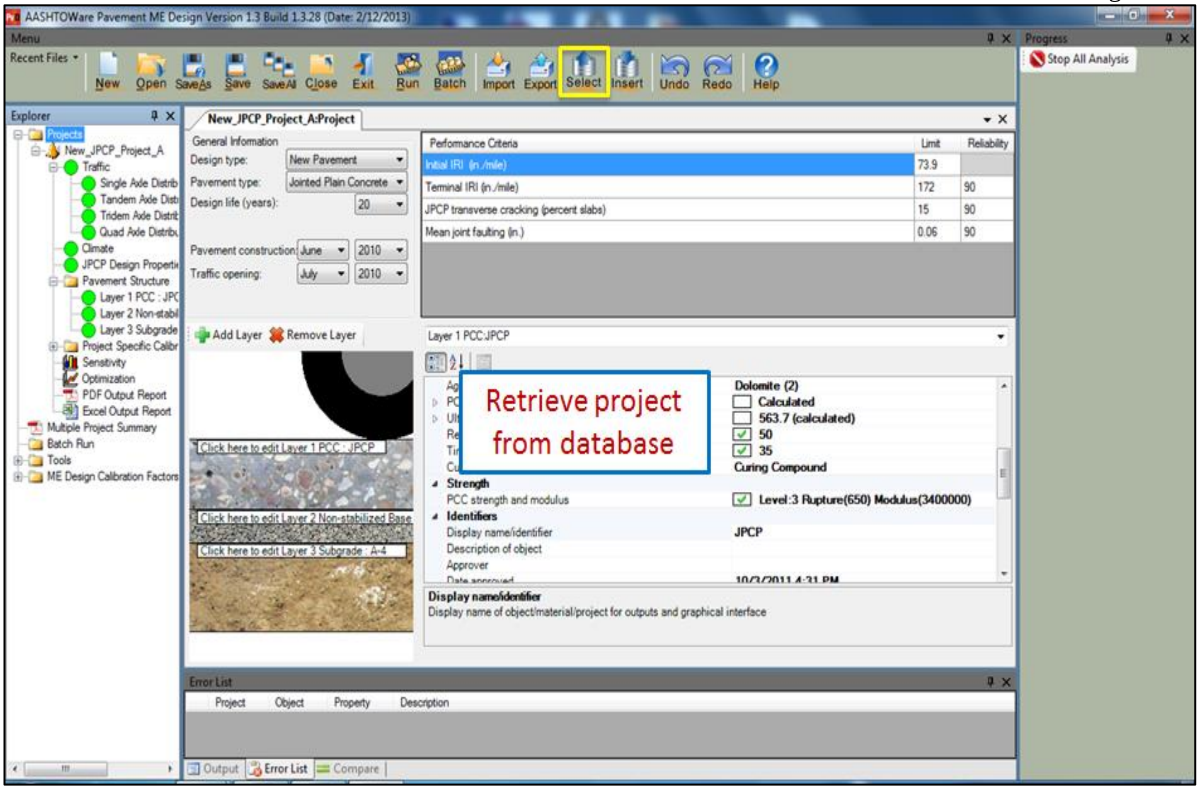


Figure 1.23 Importing an Entire Project from the M-E Design Database

This will open the search tool in M-E Design and will allow users to search for the database objects they wish to pull into the current projects, or they may load an existing project from memory. **Note:** If a user selects an element, but has no active projects in the explorer, a new project will be created. One of the projects from the list can then be selected and loaded into the user interface. Click '*OK*' to import a project or project element from the database. (see **Figure 1.24 Selecting a Project to Import from M-E Design Database**). Once the statement has been generated, the user clicks on the '*Search*' button and is presented with the following screen.

Importing Elements into a Project

To import project elements, right click on the element you wish to import, and click '*Get from Database*'. This will bring up a window asking the user to select the element they wish to retrieve from the database. For example, to load climate data from the database, the user should right click on '*Climate*' and select '*Get from Database*' (see **Figure 1.25 Getting an Element from the M-E Design Database**). The M-E Design element is then loaded into the current project.

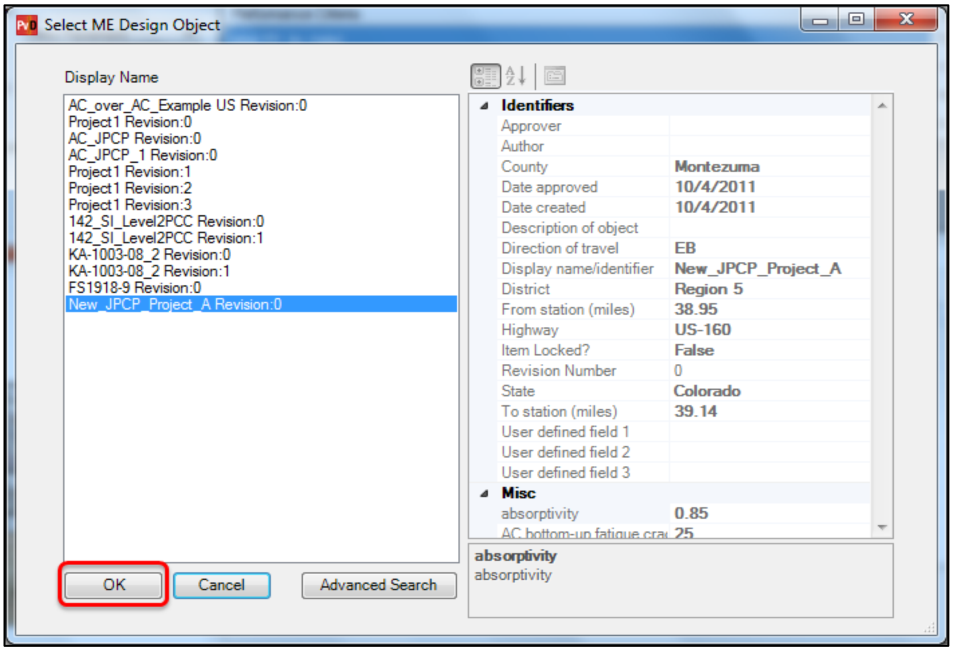


Figure 1.24 Selecting a Project to Import from the M-E Design Database

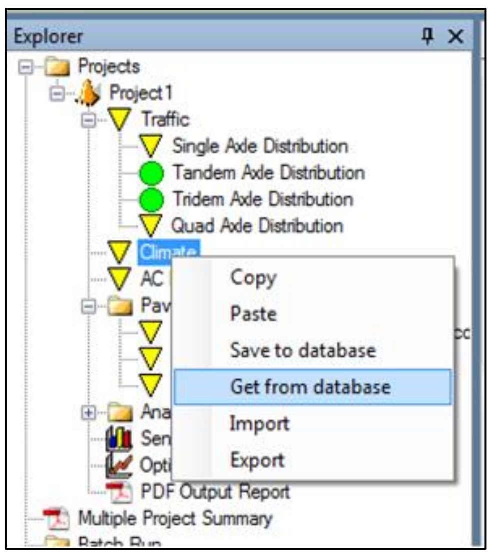


Figure 1.25 Getting an Element from the M-E Design Database

<u>Using Advanced Search Tool</u>

After opening the search tool in M-E Design, click the '*Advanced Search'* option. This will open an advanced search tool which allows a user to form queries to search for the database objects they wish to pull into the current project or to load an existing project from memory. **Note:** If a user selects an element, but has no active projects in the explorer, a new project will be created. Projects and project elements can be queried to find data which matches specific M-E Design criteria. In the example below, the user has selected the project and the variable(s) they wish to use a search. **Figure 1.26 Advanced Search Blank Window in the M-E Design Database** shows the advanced search window.

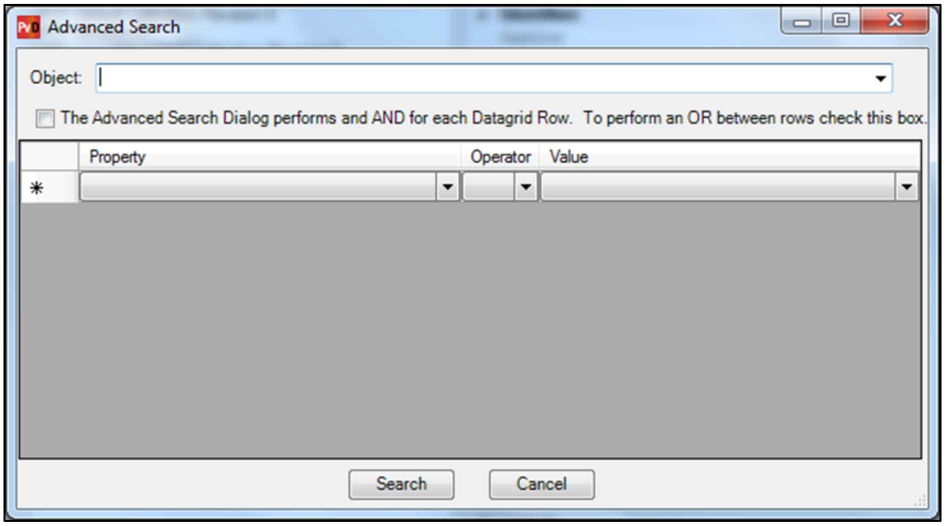


Figure 1.26 Advanced Search Blank Window in the M-E Design Database

First, the user selects the 'O*bject Type'* (in this case '*Project'*) they wish to filter. Next, they select a property associated with the object type (in this case '*Display Name'*). Finally, the user selects a value to match with the property (in this case '*HMA over HMA'*). The user then selects which type of operator to apply to the statement (in this case'='). Refer to **Figure 1.27 Advanced Search Window with Information**.

Pressing the search button runs the filter and produces a list of projects or project elements for users to select. In this case, the entire statement is generated and shown in **Figure 1.28 Selecting a Project Using Advanced Search Tool**, where '*Display Name = HMA'* place the arrow over HMA, and press '*OK'* to import the project or project element in the M-E Design interface.

<u>A Special Note on Traffic</u>

As previously mentioned, the traffic element works slightly different from the other M-E Design elements. All of the traffic elements for retrieving data from the database mirror the functionality of the save operation (i.e. retrieving a single axle distribution element will import tandem, tridem, and quad axle distribution elements).

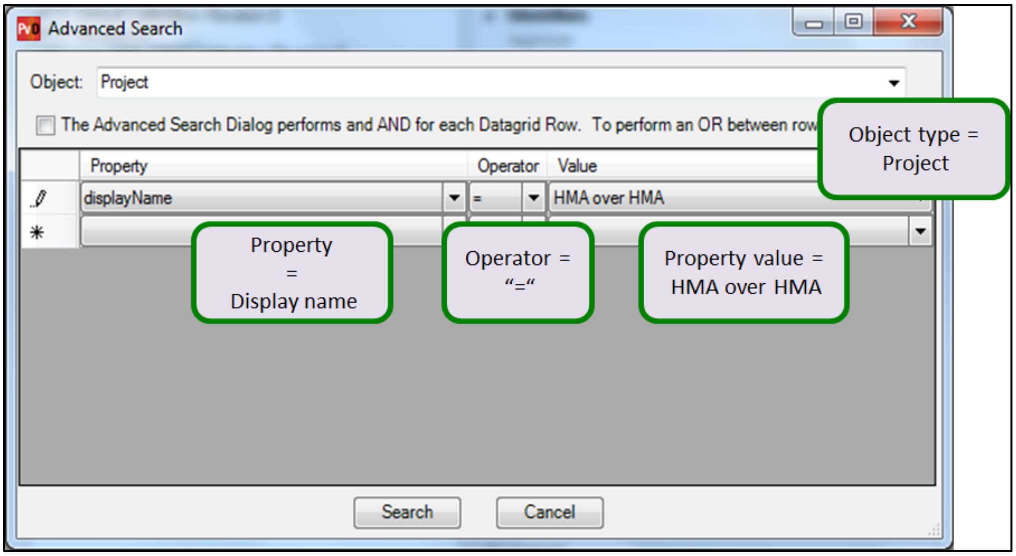


Figure 1.27 Advanced Search Window with Information

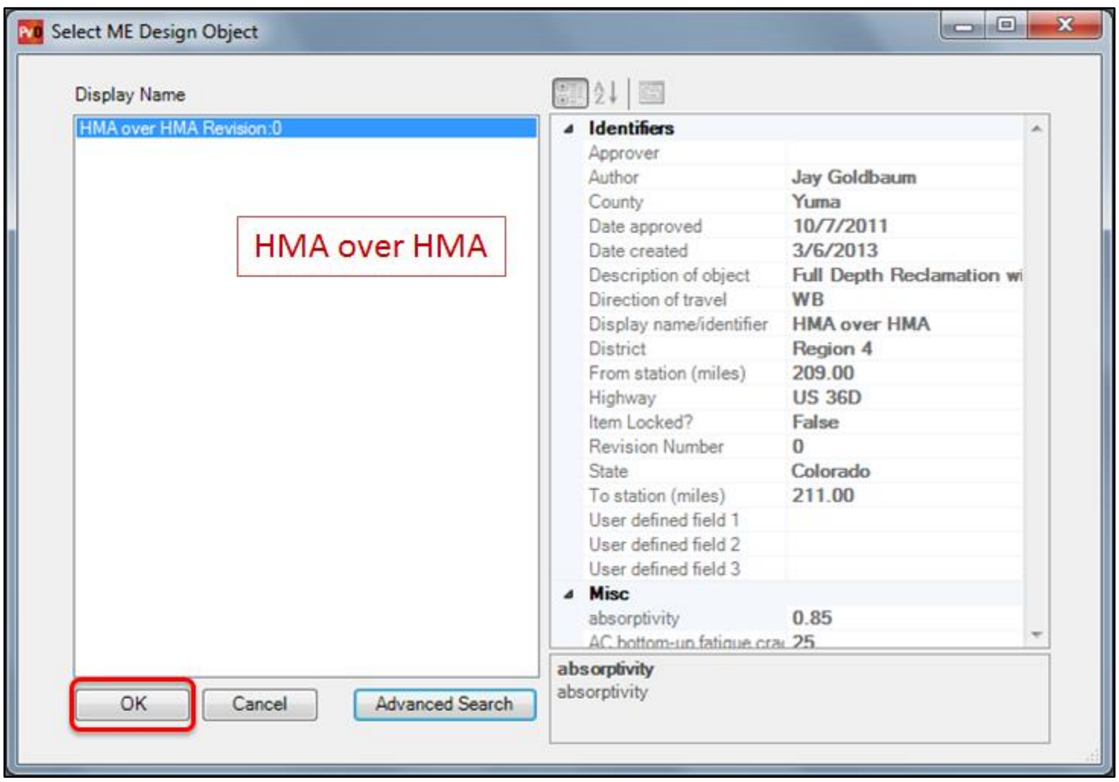


Figure 1.28 Selecting a Project Using Advanced Search Tool

1.5.3 Optimization Function

The M-E Design Program has a built in tool that allows the designer to find the minimal thickness of a strata layer while maintaining a constant thickness of all other pavement layers. The tool will be referred to as the 'Optimization' function, or optimization for short. The function allows a designer to input a maximum and minimum thickness value for a strata and the program will run the designs changing the thickness of that strata until the thinnest, passing thickness is determined. For example, if the design requires 6 inches of concrete, the designer may use the optimization feature for the aggregate base course layer and choose between 6 and 20 inches. The program will run a design using 6 inches; if it passes the program will produce the standard pdf report for 6 inches. If the design fails at 6 inches the program will run a design using 20 inches, if it passes then it will choose the middle value, in this scenario 13 inches and run a design. The program will incrementally change the aggregate base thickness until the thinnest layer thickness that passes is determined. Steps for using the optimization feature are shown below:

Step 1. Click on the Optimization Function, **Figure 1.29 The Optimization Function**.

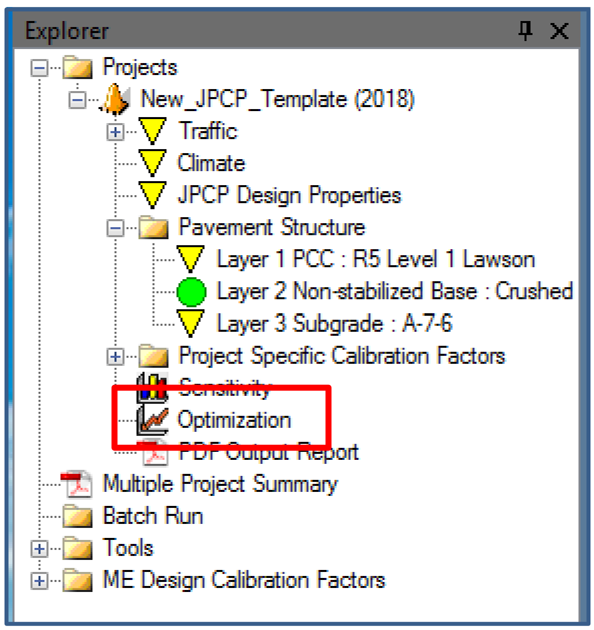


Figure 1.29 The Optimization Function

Step 2. Check the box of the layer you want to optimize**, Figure 1.30 Selecting the Layer for Optimization**

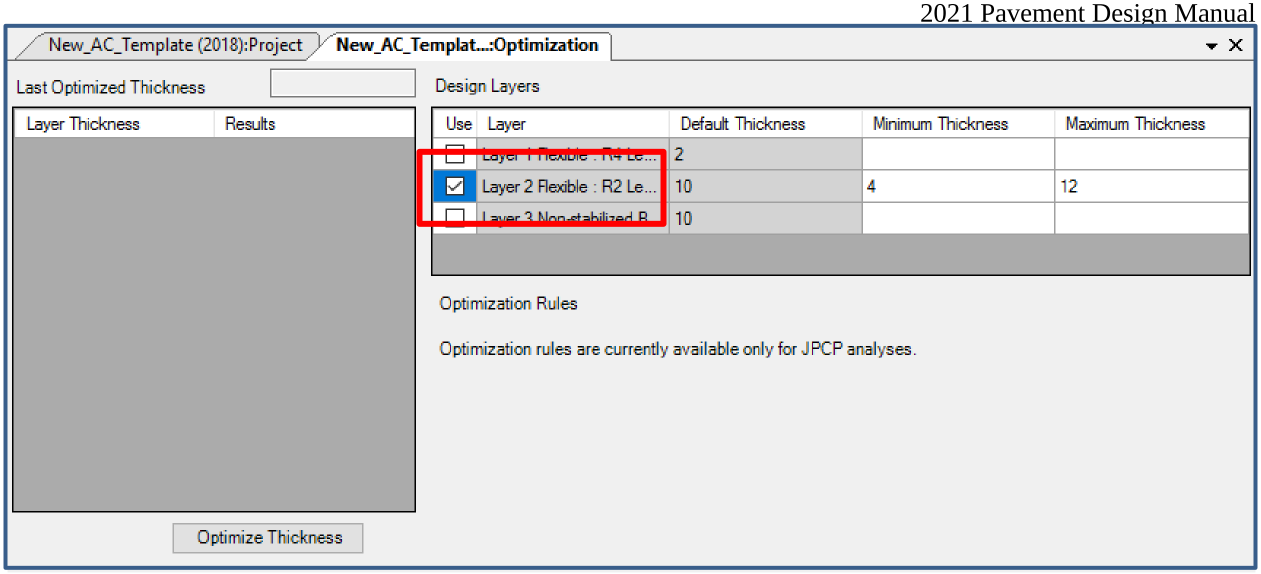


Figure 1.30 Selecting the Layer for Optimization

Step 3. Input the minimum and maximum thickness values for the strata, **Figure 1.31 Optimization Input Values.**

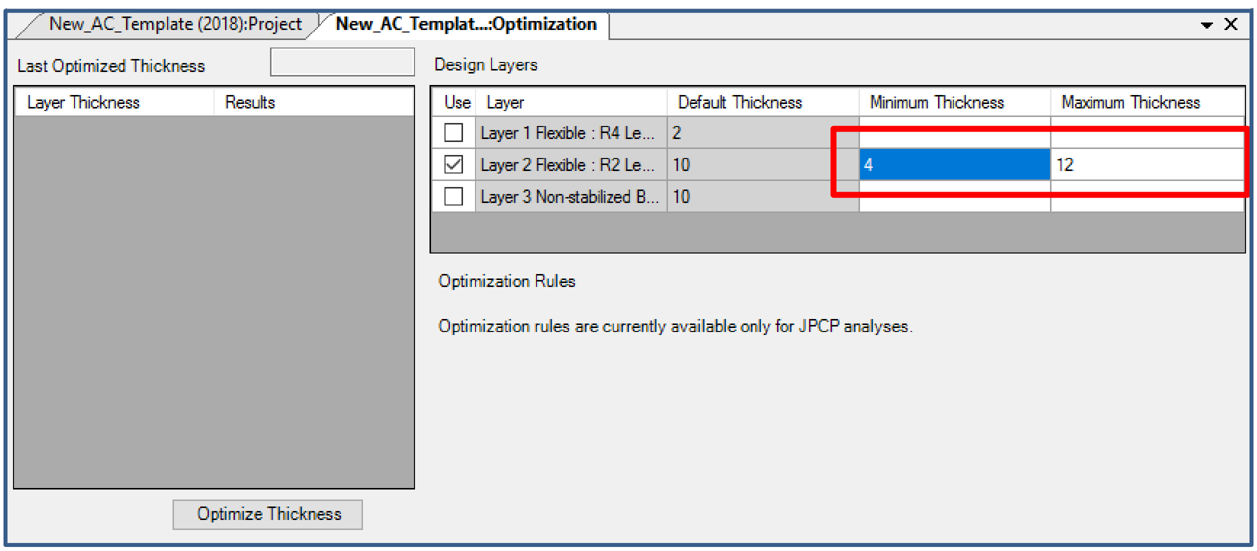


Figure 1.31 Optimization Input Values

Step 4. Click on the '*Optimize Thickness'* button, **Figure 1.32 Optimize Thickness Button.**

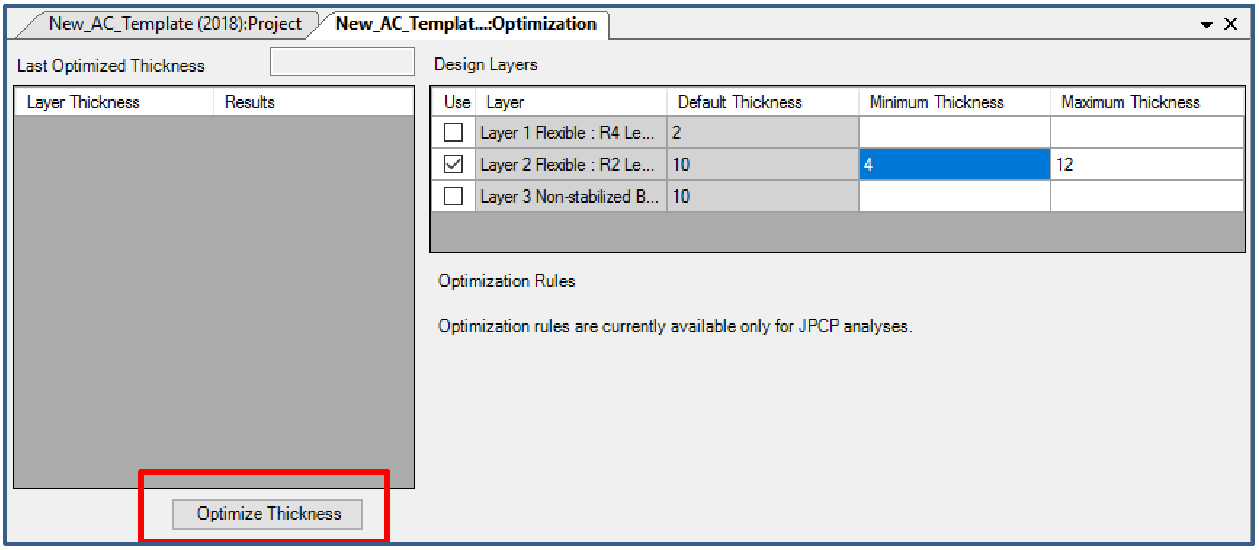


Figure 1.32 Optimize Thickness Button

The program will run until the thinnest passing layer is determined**, Figure 1.33 Optimization Results Screenshot.** A pdf. for this design will be provided. The pdf. will show the word '*optimized*' adjacent to the thickness of the strata used in the optimization**.**

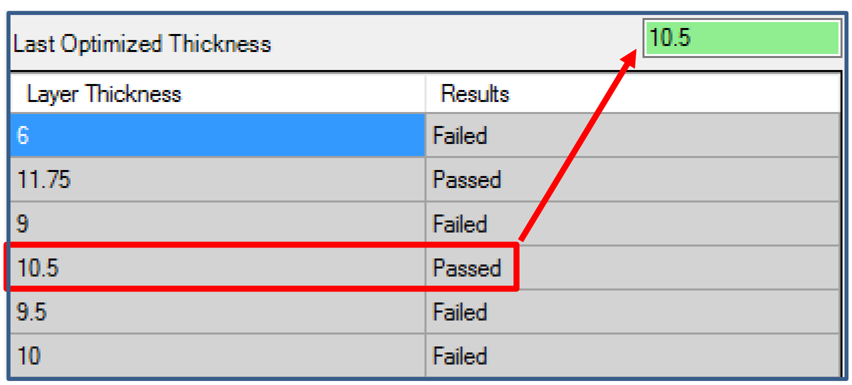


Figure 1.33 Optimization Results Screenshot

References

1. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

2. *AASHTO Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide*, November 2010, American Association of State Highway and Transportation Officials, Washington, DC, 2010.

3. CDOT Final_Calibration_June_12_2012.

CHAPTER 2
PAVEMENT DESIGN INFORMATION

2.1 Introduction

This chapter provides pavement designers the general information required for conducting pavement design and analysis using the M-E Design software. This section does not include traffic, climate, and material related inputs.

2.2 Site and Project Identification

Site/project location information is used to identify the project under design. This input has no bearing on design but is very helpful in documenting a design for review and record keeping. The M-E Design software provides the ability to enter site or project identification information such as the location of the project, jurisdiction, identification numbers, beginning and ending milepost, direction of traffic, date created, and date approved.

2.3 Project Files/Records Collection and Review

2.3.1 Project Data Collection

Information gathered should include such items as "As Built" plans from previous projects, pavement design data, materials and soil properties, climate conditions, determination of traffic inputs, and any information relevant to major maintenance.

2.3.2 Field Survey

A pavement evaluation should be conducted to determine the cause of the pavement deterioration. Information gathered in this survey includes review items such as distress, drainage conditions, roughness, traffic control options, safety considerations, any other overall project conditions, and assessments including an estimate of drivable life. For new alignments, the soil survey investigation records are reviewed.

2.3.3 Initial Selection

Preliminary alternate designs are developed to repair the existing distress and prevent future problems. Based on an evaluation of various candidate alternatives, the first cuts are made at this time, as is a determination if additional data is needed.

2.3.4 Physical Testing

Testing includes collection of additional information such as coring, deflection testing, resilient modulus, permeability, moisture content, etc.

2.3.5 Evaluation and Selection

The selection of new construction and rehabilitation techniques includes identifying the various constraints associated with the project, such as:

- Funding (first cost consideration)
- Traffic control
- Design period
- Geometric problems
- Right of way
- Utilities
- Vertical clearance problems (i.e. overhead clearance)

2.3.6 Historic M-E Design Software Files

Pavement design/analysis projects created in M-E Design software are saved as .dgpx files. After a design/analysis run has been successfully completed, the application will generate a pdf file and Microsoft Excel spreadsheet containing input summary and output results of the trial design. There are several project or CDOT specific input files for traffic, climate, and material characterization associated with the pavement design/analysis projects.

The M-E Design software includes a database option that facilitate enterprise level data management for archiving and searching projects, comparing inputs of any two projects, and creating input data libraries. Each object (i.e. any discrete item such as pavement material layer data, axle load distribution factors, climate and design features, or the project itself) has a unique informational tag called identifiers. The designers can use identifiers to identify, search, filter, save, and retrieve information in a database environment.

The designer should review the data files available with the software system and the database. Project records including the project files, input files, calibration factors, and the output records should be stored in the appropriate data storage systems specified by CDOT. For reasons of software update and input changes, the designer should keep track of the software version, project time stamps, and input modifications using the identifiers of M-E Design software objects.

2.3.7 Records Review

Review of historic and current project files and/or records is an important aspect of pavement design/analysis. A review of these records may reveal key details of interest and significance to the pavement designer. Reviewing the project files and/or records will be the most beneficial to the pavement designer who has not been with the project since its original construction. In reviewing the project files and/or records, the pavement designer should be on the alert for any information relating to pavement design and construction. The Regions should keep copies of the information in the original report for 5 to 8 years.

Records review typically comprises of the following activities:

- Review construction and maintenance files.
- Review previous distress surveys and pavement management records. If possible, establish performance trends and deterioration rates.
- Review previous Falling Weight Deflectometer (FWD) deflection test data.
- Review previous pavement borings and laboratory test results of pavement materials and subgrade soils.
- For existing pavements, perform a windshield survey or an initial surveillance of the roadway's surface, drainage features, and other related items.
- Identify roadway segments with similar or different surface and subsurface features. As much as possible, isolate each unique factor that will influence pavement performance.
- Identify the field testing/material sampling requirements for each segment and the associated traffic control requirements.
- Determine if the pavement performed better or worse than similar designs.

The information gathered in records review can be used to divide a new alignment or existing pavement into units with similar site conditions. Existing pavements may be further divided into units with similar design features and performance characteristics.

2.4 Site Investigation

It may be advantageous to visit the proposed project site a few times during the development of a pavement or rehabilitation design. The pavement designer may find it desirable to make a brief visit to the project site as the first step in the scoping process. As the investigation proceeds, events may develop which will make it desirable to revisit the project site. The following are some of the items that should be determined during visits to the project site.

2.4.1 Abutting Land Usage

The abutting land usage will have an effect on the selection of a pavement type or rehabilitation design procedure. If the abutting land is rural, then a note should be made of its use such as farming, ranching, or other with descriptive details as needed. If the property is urban, a record of usage in terms of residential or commercial is helpful. Additional details on type of residences or commercial usage are also helpful.

2.4.2 Existing/Proposed Project Geometrics

Notes should be made as to the type and typical section including the vertical and horizontal alignment characteristics. Data concerning the typical section should indicate the average and maximum 'cut and fill' heights and extent over the project. Items such as the number of travel lanes, shoulders, type and extent of curb and gutter, and vertical clearances at structures should be recorded.

2.4.3 Pavement Condition Survey

Pavement condition is a key input required for the determination of feasible rehabilitation alternatives. The CDOT Pavement Management System (PMS) provides network-level pavement condition data for use in the preliminary evaluation of the project. If there is no PMS data for a roadway section of interest, one should conduct a manual distress survey of the project to assess the pavement condition and establish the causes of distresses/failure.

2.4.4 Drainage Characteristics

Drainage characteristics should be noted during the visit to the project. Items such as the general terrain drainage, existing pavement drainage, and bridge drainage structures need to be noted. The number of bridges, how the existing pavement terminates at the bridge ends, and if the bridges have bridge approach slabs is important to note. The condition of the bridge end/approach slab and the approach slab/pavement interface conditions are of special interest when concrete pavement exists.

Distresses can be related to particular moisture properties of the materials in the pavement. If the existence of these properties is not recognized and corrected where possible, the rehabilitation work will be wasted by allowing the same type of moisture-related distress to reoccur. The recognition of the amount, severity, and cause of moisture damage also plays an important role in the selection of the rehabilitation scheme to be utilized on the pavement. This information will help in the structural evaluation of the pavement.

Moisture-related distresses develop from external and internal factors that influence the moisture condition in a pavement. An example of external factors are the climatic factors in an area that regulate the supply of moisture to the pavement. Internal factors are pavement material properties whose interaction with moisture influences pavement performance.

The recognition of each distress and the mechanism causing that distress are necessary if the correct rehabilitation procedures are to be selected. Each distress type that develops within a pavement will be load, environment-related, or a combination of the two. Moisture will serve to accelerate this deterioration when it is environment-related. Moisture problems must be recognized and corrected to prevent future deterioration.

The fact that moisture problems may appear in any layer emphasizes the necessity of having a logical procedure for examining the pavement in order to determine the cause of the problem. Non-Destructive Testing (NDT) will indicate the overall structural level of the pavement. However, NDT alone cannot identify which component of the pavement is responsible for the strength loss. Distress analysis must be utilized in conjunction with the NDT analysis to identify potential moisture-related problems. If the subgrade has moisture problems that caused the distress, it may do no good to overlay, recycle, rework the pavement, or stabilize the base without also addressing the subgrade. If the base or subbase has moisture problems one may need to rework or stabilize the base and/or rework the drainage of the granular layer. **Table 2.1 Moisture-Related Distress in Flexible Pavements** and **Table 2.2 Moisture Related Distress in Rigid**

Pavements contains a breakdown of the more common moisture-related distresses for flexible and rigid pavements.

2.5 Construction and Maintenance Experience

On any given project, there are always construction and maintenance experiences with pavement structures that were never entered into the permanent records relating to the project. Usually, it was not realized that information such as this would be useful in the future. The Program Engineers, Resident Engineers, Project Engineers, Construction Inspectors, and other personnel involved with the project may have useful information if interviewed. The Region Maintenance Superintendent and other maintenance personnel may have pavement performance data that do not appear elsewhere in the records. Frequently, maintenance forces have repaired substantial sections of the project and this information is not always readily available in the records.

2.6 Pavement Management System (PMS) Condition Data

The PMS provides network-level pavement condition information for planning and programming purposes. PMS data are used to help select reconstruction, rehabilitation, preventive maintenance projects, and evaluate performance trends. It also provides pavement condition information useful for performing a preliminary evaluation of a project.

For M-E Design, site-specific or project-specific past performance data is used to characterize the existing pavement's condition for use in rehabilitation design. The specifics of how PMS condition data is used is presented in **Chapter 8 Principles of Design for Pavement Rehabilitation with Flexible Overlays** and **Chapter 9 Principles of Design for Pavement Rehabilitation with Rigid Overlays** for rehabilitation designs using flexible and rigid overlays.

CDOT collects and reports pavement performance data on a tenth mile basis, in only one direction of all two-lane highways. CDOT PMS data of relevance to the M-E Design are the following:

- International Roughness Index (IRI)
- Rutting
- Faulting
- Cracking distress

For more information about PMS data, contact the PMS unit or the Region Pavement Manager.

Table 2.1 Moisture-Related Distress in Flexible Pavements

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Materials Problem	Load Associated	Structural Defect Begins In		
						Asphalt	Base	Subgrade
Surface Defect	Bleeding	No	Accentuated by high temp	Bitumen	No	Yes	No	No
	Raveling	No	No	Aggregate	Slightly	Yes	No	No
	Weathering	No	Humidity and light dried bitumen	Bitumen	No	Yes	No	No
Surface Deformation	Bump or Distortion	Excess moisture	Frost Heave	Strength moisture	Yes	No	Yes	Yes
	Corrugation or Rippling	Slight	Climatic and suction relations	Unstable mix	Yes	Yes	Yes	Yes
	Shoving	No	-	Unstable mix loss of bond	Yes	Yes	No	No
	Rutting	Excess in granular layers	Suction and materials	Compaction properties	Yes	Yes	Yes	Yes
	Depression	Excess	Suction and materials	Settlement fill material	Yes	No	No	Yes
	Potholes	Excess	Frost heave	Strength moisture	Yes	No	Yes	Yes
Cracking	Longitudinal	Yes	Spring thaw strength loss	-	Yes	Faulty construction	Yes	Yes
	Alligator	Yes drainage	-	Possible mix problems	Yes	Yes, Mix	Yes	Yes
	Transverse	Yes	Low-temp. freeze thaw cycles	Thermal properties	No	Yes temperature susceptible	Yes	Yes

Table 2.2 Moisture-Related Distress in Rigid Pavements

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Materials Problem	Load Associated	Structural Defect Begins In		
						Asphalt	Base	Subgrade
Surface Defect	Spalling	Possible	No	-	No	Yes	No	No
	Crazing	No	No	Rich mortar	No	Yes weak surface	No	No
Surface Deformation	Blow-up	No	Temp.	Thermal properties	No	Yes	No	No
	Pumping	Yes	Moisture	Fines in base moisture sensitive	Yes	No	Yes	Yes
	Faulting	Yes	Moisture suction	Settlement deformation	Yes	No	Yes	Yes
	Curling	Possible	Moisture and temp.	-	No	Yes	No	No
Cracking	Corner	Yes	Yes	Follows pumping	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Possible	Cracking follows moisture build-up	Yes	No	Yes	Yes
	Punch-out	Yes	Yes	Deformation following cracking	Yes	No	Yes	Yes
	Joint	Produces damage later	Possible	Proper filler and clean joints	No	Joint	No	No

2.7 Design Performance Criteria and Reliability (Risk)

Performance verification is the basis of the acceptance or rejection of a trial design evaluated using the M-E Design software. A successful design is one where all the selected performance threshold limits are satisfied at their chosen levels of reliability at the end of the design life.

M-E Design requires the designer to specify the critical levels or threshold values of pavement distresses and smoothness to judge the adequacy of a design. The type of distresses used in performance verification is specific to the pavement type (flexible or rigid) and design (rehabilitation or new design). Additionally, design reliability levels are required to account for uncertainty and variability that is expected to exist in pavement design and construction, as well as, in the application of traffic loads and climatic factors over the design life. The threshold and reliability levels for distresses and smoothness significantly impact construction costs and performance. The designer must set realistic numerical limits or threshold values for each performance criterion and reasonable reliability levels for a given design life.

Limits on the various performance criteria should be considered along with design reliability and design period. Both performance criteria and reliability factors are determined based on the functional classification of the roadway and whether it is in an urban or a rural location. Once selected, the limits should be used consistently throughout the pavement type selection and design calculations. **Consultation of the mix design(s) with the RME shall occur**.

Recommended Range for Reliability

The reliability is a factor of safety to account for the inherent variations in construction, materials, traffic, climate, and other design inputs. **Table 2.3 Reliability (Risk)** provides the recommended values for the pavement structure to survive the design period traffic. Reliability values recommended for use in previous editions of the AASHTO Design Guide should not be used with M-E Design. Reliability is not dependent on either type of pavement or type of project.

Table 2.3 Reliability (Risk)

Functional Classification	Value for Reliability
Interstate	80-95
Principal Arterials (freeways and expressways)	75-95
Principal Arterials (other)	75-95
Minor Arterial	70-95
Major Collectors	70-90
Minor Collectors	50-90
Local	50-80

Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction of Flexible Pavement Projects, Table 2.5 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction Projects of Rigid Pavement, Table 2.6 Recommended Threshold Values of Performance Criteria for Rehabilitation Projects of Flexible Pavements and **Table 2.7 Recommended Threshold Values of Performance Criteria for Rehabilitation Projects of Rigid Pavements** provide the threshold values recommended in M-E Design for pavements. M-E Design also requires the designer to enter the expected initial smoothness (IRI) at the time of construction. **It is recommended to use an initial IRI value of 61 inches/mile for all HMA projects and 78 inches/mile for all PCC projects** as they reflect targets that are documented using smoothness data from flexible and rigid pavements constructed between 2011 and 2016. **It is recommended the same reliability value be used for all distresses; any changes should have Region Materials and Staff Materials approval.**

Figure 2.1 Performance Criteria and Reliability in the M-E Design Software for a Sample Flexible Pavement Design presents the M-E Design software screenshot showing performance criteria and the corresponding design reliability values selected for the design/analysis of a sample flexible pavement design.

Figure 2.2 Performance Criteria and Reliability in the M-E Design Software for a Sample JCPC Design presents the M-E Design software screenshot showing performance criteria and the corresponding design reliability values selected for the design/analysis of a sample rigid pavement design.

Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction of Flexible Pavement

Flexible Pavement		
Performance Criteria	**Maximum Value at End of the Design Life**	**Determines the Years to First Rehabilitation (Minimum Age Shall be 14 Years)**
Terminal IRI (inches per mile)		Interstate – 160 Principal Arterial – 200 Minor Arterial – 200 Major Collector – 200 Minor Collector – 200* Local Roadway – 200*
AC Top-Down Fatigue Cracking (feet per mile)		Interstate – 2,000 Principal Arterial – 2,500 Minor Arterial – 3,000 Major Collector – 3,000 Minor Collector – 3,000* Local Roadway – 3,000*
AC Bottom-Up Fatigue Cracking (percent lane area)	Interstate – 10 Principal Arterial – 25 Minor Arterial – 25 Major Collector – 25 Minor Collector – 25* Local Roadway – 25*	
AC Thermal Cracking (feet per mile)	Interstate – 1,500 Principal Arterial – 1,500 Minor Arterial – 1,500 Major Collector – 1,500 Minor Collector – 1,500* Local Roadway – 1,500*	
Permanent Deformation (total inches)		Interstate – 0.55 Principal Arterial – 0.65 Minor Arterial – 0.80 Major Collector – 0.80 Minor Collector – 0.80* Local Roadway – 080*
Permanent Deformation AC Only (inches)		Interstate – 0.40 Principal Arterial – 0.50 Minor Arterial – 0.65 Major Collector – 0.65 Minor Collector – 0.65* Local Roadway – 0.65*
Additional Thresholds for Chemically Stabilized Layer		
Fatigue Fracture (percent lane area) (For semi-rigid base layer)		Interstate – 10 Principal Arterial – 25 Minor Arterial – 25 Major Collector – 25 Minor Collector – 25* Local Roadway – 25*
AC Total Fatigue Cracking Bottom Up + Reflective (percent lane area) (For semi-rigid base layer)		Interstate – 10 Principal Arterial – 25 Minor Arterial – 25 Major Collector – 25 Minor Collector – 25* Local Roadway – 25*
AC Total Transverse Cracking Thermal + Reflective (feet per mile) (For semi-rigid base layer)		Interstate – 1,500 Principal Arterial – 1,500 Minor Arterial – 1,500 Major Collector – 1,500 Minor Collector –1,500* Local Roadway – 1,500*

Note: * M-E Design has not been calibrated for minor collectors or local roadways. Exceptions to the threshold values may be approved by the RME.

Table 2.5 Recommended Threshold Values of Performance Criteria for Rehabilitation of Flexible Pavement Projects

Flexible Pavement		
Performance Criteria	**Maximum Value at End of the Design Life (Minimum Age Shall Be 10 Years)**	
Terminal IRI (inches per mile)	Interstate – 160 Principal Arterial – 200 Minor Arterial – 200 Major Collector – 200 Minor Collector – 200* Local Roadway – 200*	
AC Top-Down Fatigue Cracking (feet per mile)	Interstate – 2,000 Principal Arterial – 2,500 Minor Arterial – 3,000 Major Collector – 3,000 Minor Collector – 3,000* Local Roadway – 3,000*	
AC Bottom-Up Fatigue Cracking (percent lane area)	Interstate – 10 Principal Arterial – 25 Minor Arterial – 25 Major Collector – 25 Minor Collector – 25* Local Roadway – 25*	
AC Thermal Cracking (feet per mile)	Interstate – 1,500 Principal Arterial – 1,500 Minor Arterial – 1,500 Major Collector – 1,500 Minor Collector – 1,500* Local Roadway – 1,500*	
Permanent Deformation (total inches)	Interstate – 0.55 Principal Arterial – 0.65 Minor Arterial – 0.80 Major Collector – 0.80 Minor Collector – 0.80* Local Roadway – 0.80*	
Permanent Deformation AC Only (inches)	Interstate – 0.40 Principal Arterial – 0.50 Minor Arterial – 0.65 Major Collector – 0.65 Minor Collector – 0.65* Local Roadway – 0.65*	
AC Total Fatigue Cracking Bottom-Up + Reflective (percent lane area)	Interstate – 20 Principal Arterial – 35 Minor Arterial – 35 Major Collector – 35 Minor Collector – 35* Local Roadway – 35*	Use 50% Reliability
AC Total Transverse Cracking Thermal + Reflective (feet per mile)	Interstate – 2,500 Principal Arterial – 2,500 Minor Arterial – 2,500 Major Collector – 2,500 Minor Collector – 2,500* Local Roadway – 2,500*	
Additional Thresholds for Chemically Stabilized Layer		
Fatigue Fracture (percent lane area) (For semi-rigid base layer)	Interstate – 20 Principal Arterial – 35 Minor Arterial – 35 Major Collector – 35 Minor Collector – 35* Local Roadway – 35*	
Note: * M-E Design has not been calibrated for minor collectors or local roadways. Exceptions to the threshold values may be approved by the RME.		

Table 2.6 Recommended Threshold Values of Performance Criteria for New Construction of Rigid Pavement

Rigid Pavement (JPCP)		
Performance Criteria	**Maximum Value at End of the Design Life**	**Determines the Year to First Rehabilitation (Minimum Age Shall Be 27 Years)**
Terminal IRI (inches per mile)		Interstate – 160 Principal Arterial – 200 Minor Arterial – 200 Major Collector – 200 Minor Collector – 200* Local Roadway – 200*
Transverse Slab Cracking (percent)		Interstate – 7.0 Principal Arterial – 7.0 Minor Arterial – 7.0 Major Collector – 7.0 Minor Collector – 7.0* Local Roadway – 7.0*
Mean Joint Faulting (inches)	Interstate – 0.12 Principal Arterial – 0.14 Minor Arterial – 0.20 Major Collector – 0.20 Minor Collector – 0.20* Local Roadway – 0.20*	
Note: * M-E Design has not been calibrated for minor collectors or local roadways. Exceptions to the threshold values may be approved by the RME.		

Table 2.7 Recommended Threshold Values of Performance Criteria for Rehabilitation of Rigid Pavement Projects

Rigid Pavement (JPCP)	
Performance Criteria	**Maximum Value at End of the Design Life (Minimum Age Shall Be 20 Years)**
Terminal IRI (inches per mile)	Interstate – 160 Principal Arterial – 200 Minor Arterial – 200 Major Collector – 200 Minor Collector – 200* Local Roadway – 200*
Transverse Slab Cracking (percent)	Interstate – 7.0 Principal Arterial – 7.0 Minor Arterial – 7.0 Major Collector – 7.0 Minor Collector – 7.0* Local Roadway – 7.0*
Mean Joint Faulting (inches)	Interstate – 0.12 Principal Arterial – 0.14 Minor Arterial – 0.20 Major Collector – 0.20 Minor Collector – 0.20* Local Roadway – 0.20*
Note: * M-E Design has not been calibrated for minor collectors or local roadways. Exceptions to the threshold values may be approved by the RME.	

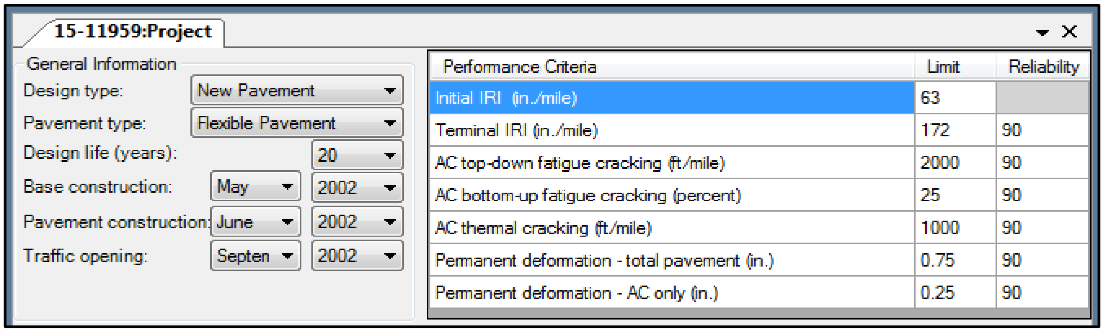


Figure 2.1 Performance Criteria and Reliability in the M-E Design Software for a Sample Flexible Pavement Design

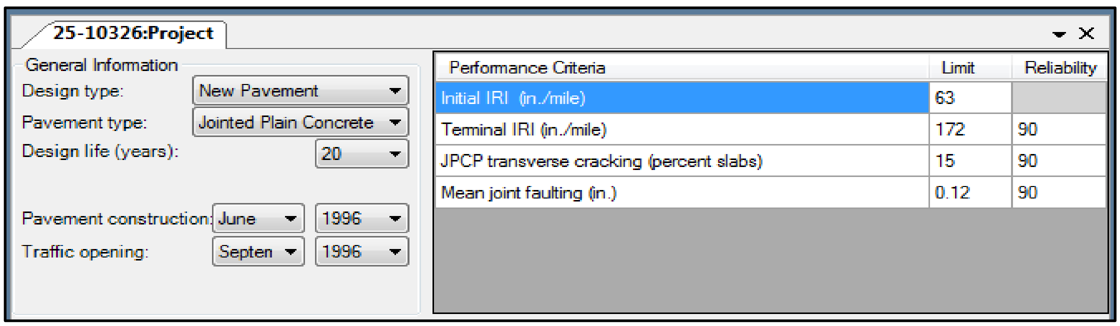


Figure 2.2 Performance Criteria and Reliability in the M-E Design Software for a Sample JPCP Design

The appropriate functional classification for a certain roadway can be determined from the information on CDOT Form #463: Design Data, completed for the specific highway project being designed. A blank CDOT Form #463 is shown in the Appendix of the *CDOT Project Development Manual* and **APPENDIX B: FORMS** of this manual. As an example, CDOT Form #463 identifies a segment of State Highway 83 as a principal arterial; the reliability for this roadway can be obtained from **Table 2.3 Reliability (Risk).** As the table shows, the reliability for this road may range from 75 to 95 percent. This is a high profile road, so the reliability is set at 95 percent.

CDOT has a map available designating highway functional classifications, see **Figure 2.3 Functional Classification Map**. The map may be downloaded from the following website: http://alphainternal.dot.state.co.us/App_DTD_DataAccess/Downloads/StatewideMaps/func_class_pdf.pdf

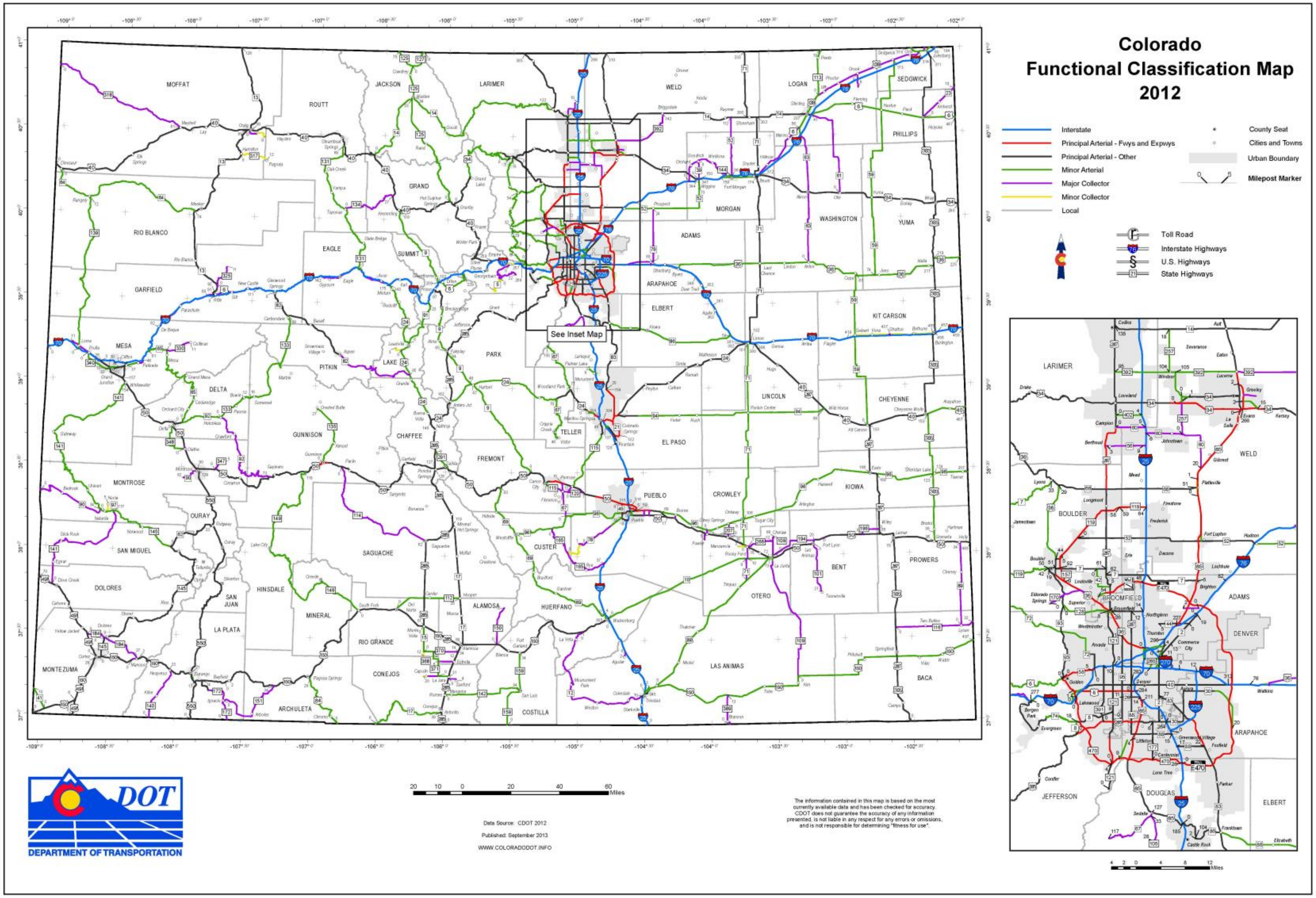


Figure 2.3 Function Classification Map

2.8 Defining Input Hierarchy

M-E Design employs a hierarchical approach to input parameters with regard to traffic, material, and condition of existing pavement. This approach provides the designer with a lot of flexibility in obtaining the design inputs for a project based on the criticality of the project and available resources.

For many of the design inputs, M-E Design allows the designer to select any of three levels of inputs:

- **Level 1:** Project-specific or site-specific inputs are obtained from direct testing or measurements. Obtaining Level 1 inputs requires more resources and time than other levels. Level 1 input would typically be used for designing heavily trafficked pavements or wherever there is dire safety or economic consequences of early failure. Examples include measuring dynamic modulus of hot mix asphalt (HMA) using laboratory testing, measuring PCC elastic moduli using laboratory testing, or using on-site measured traffic classification data.

- **Level 2:** Inputs are estimated from correlations or regression equations derived from a limited testing program or obtained from the agency database. This level could be used when resources or testing equipment are not available for tests required for Level 1. Examples include estimating resilient modulus of unbound materials and subgrade from R-values, estimating PCC elastic moduli from compressive strength tests, or using traffic classification data based on the functional class of the roadway.

- **Level 3:** Inputs are based on "best-estimated" or typical values for the local region. This level might be used for design where there are minimal consequences of early failure (i.e. lower volume roads). Examples include using default resilient modulus values for unbound materials, estimating PCC elastic moduli from 28-day compressive or flexural strength tests, or using default traffic classification data.

The designer can also select a mix of input levels for a given project. For instance, the designer can select the HMA creep compliance at Level 1, subgrade resilient modulus at Level 2, and traffic load spectra at Level 3 for analyzing a flexible pavement trial design. The computational algorithms, procedures, and performance models for predicting distress and smoothness are exactly the same irrespective of the input level used in the design; however, the accuracy of the inputs as defined by the input level may affect the accuracy of performance prediction results.

The input hierarchy provides a powerful tool to show the advantages of good engineering design (using Level 1 inputs) in improving the reliability of the design, and the possibility to reduce pavement construction and rehabilitation costs. It is recommended the designer obtain the inputs that are appropriate and practical for the magnitude of the project under design. Larger, more significant projects require more accurate design inputs.

The selection of the hierarchical level for a specific input depends on several factors, including:

- Sensitivity of the pavement performance to a given input
- The criticality of the project
- The information available at the time of design
- The resources and time available to the designer to obtain the inputs

The designer should consider the above mentioned factors and select a predominant level of input hierarchy based on the recommendations presented in **Table 2.8 Selection of Input Hierarchical Level. Note:** The term "Predominant Input Hierarchy" implies the designer should, as much as possible, provide inputs at the selected input level.

Table 2.8 Selection of Input Hierarchical Level

Criticality/Sensitivity of Design	Description	Predominant Input Hierarchy
Very Critical	High volume interstates, urban freeways, and expressways	Level 1
Critical	Principal arterials, rural interstates, heavy haul (i.e. mining, logging routes)	Level 1 or Level 2
Some What Critical	Minor arterial and collectors	Level 2 or Level 3
Not Critical	Local roads	Level 3

2.9 Drainage

Water is a fundamental variable in most problems associated with pavement performance and is directly or indirectly responsible for many of the distresses found in pavement systems. A well-drained pavement section is required to maintain the strength coefficients assigned to individual components of a hot mix asphalt pavement section. Edge drains, cross drains, and drainage layers all must tie into a collection system or some other means to carry collected water away from intersections and the pavement section. Installing drainage systems that collect and impound water rather than diverting it away from the pavement section should never be allowed.

The M-E Design procedure does not consider the effects of drainage directly in pavement design/analysis methodology. Drainage effects are considered indirectly through seasonal adjustments of unbound material, subgrade moisture, and related impacts on the strength/modulus.

As good drainage is a prerequisite to any good design, designers must always consider strategies for combating the effects of water in a pavement system such as:

- Preventing water from entering the pavement
- Providing drainage to remove excess water quickly
- Building the pavement strong enough to resist the combined effect of load and water

It is preferable to exclude water from the pavement and provide for rapid drainage. The cost of improving the drainage should be compared to the cost of building a stronger pavement. It is more likely drainage improvements will outperform a stronger pavement. To obtain adequate pavement drainage, the designer should consider providing three types of drainage systems that may include surface, groundwater, and structural drainage.

It is important to understand the roadway geometry, particularly the drainage gradients in the roadway prism, when selecting the type of base. As long as the base will be able to carry drainage away from the pavement structure, a gravel base will perform adequately. It is also important to note that these values apply only to the effects of drainage on untreated base and subbase layers.

2.9.1 Subdrainage Design

Subdrainage is an important consideration in new construction or reconstruction and in the resurfacing, restoration, and rehabilitation of pavement systems. Detailed procedures for pavement subsurface design are provided in several publications, including:

- *CDOT Drainage Design Manual*
- *Guidelines for the Design of Highway Internal Drainage System,* AASHTO 1986 Guide's Appendix AA
- FHWA's DRIP software
- *MEPDG 2004 Design Documents Part 3*, Chapter 1

If necessary, the pavement designer should coordinate with the respective Region Hydraulics Engineer and/or Staff Hydraulics Engineer where a pavement drainage problem is anticipated. The pavement designer may consult the references provided above.

References

1. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

2. DRIP 2.0 Software Program < http://isddc.dot.gov/OLPFiles/FHWA/010942.pdf >

CHAPTER 3
TRAFFIC AND CLIMATE

Traffic and climate related inputs required for conducting pavement design and analysis using M-E Design software are discussed in this chapter.

3.1 Traffic

Prior to M-E Design, the number of 18,000-pound Equivalent Single Axle Loads (18-kip ESAL) represented the amount of traffic and its characteristics. However, M-E Design traffic input requirements are more detailed and can be categorized as follows, refer to **Figure 3.1 Traffic Inputs in the M-E Design Software**:

- Base year traffic information
  - Analysis period or pavement design life
  - Date newly constructed or rehabilitated pavement is opened to traffic
  - Two-way average annual daily truck traffic (AADTT)
  - Number of lanes in design direction
  - Truck direction distribution factor
  - Lane distribution factor
  - Operational speed

- Traffic adjustment factors
  - Monthly adjustment factors
  - Vehicle class distribution
  - Truck hourly distribution
  - Growth rate and type
  - Number of axles per truck
  - Axle load distribution factors

- General traffic inputs
  - Lateral wander of axle loads
  - Axle configuration
  - Wheelbase
  - Tire pressure

This section primarily deals with traffic input requirements for pavement designs using M-E Design software. The 18-kip ESALs are still required for asphalt binder selection, see **Section 6.12.3 Binder Selection** and pavement designs using the CDOT thin and ultra-thin Concrete Overlay design procedures. Refer to the *CDOT 2012 Pavement Design Manual* for information on traffic characterization using the ESAL methodology.

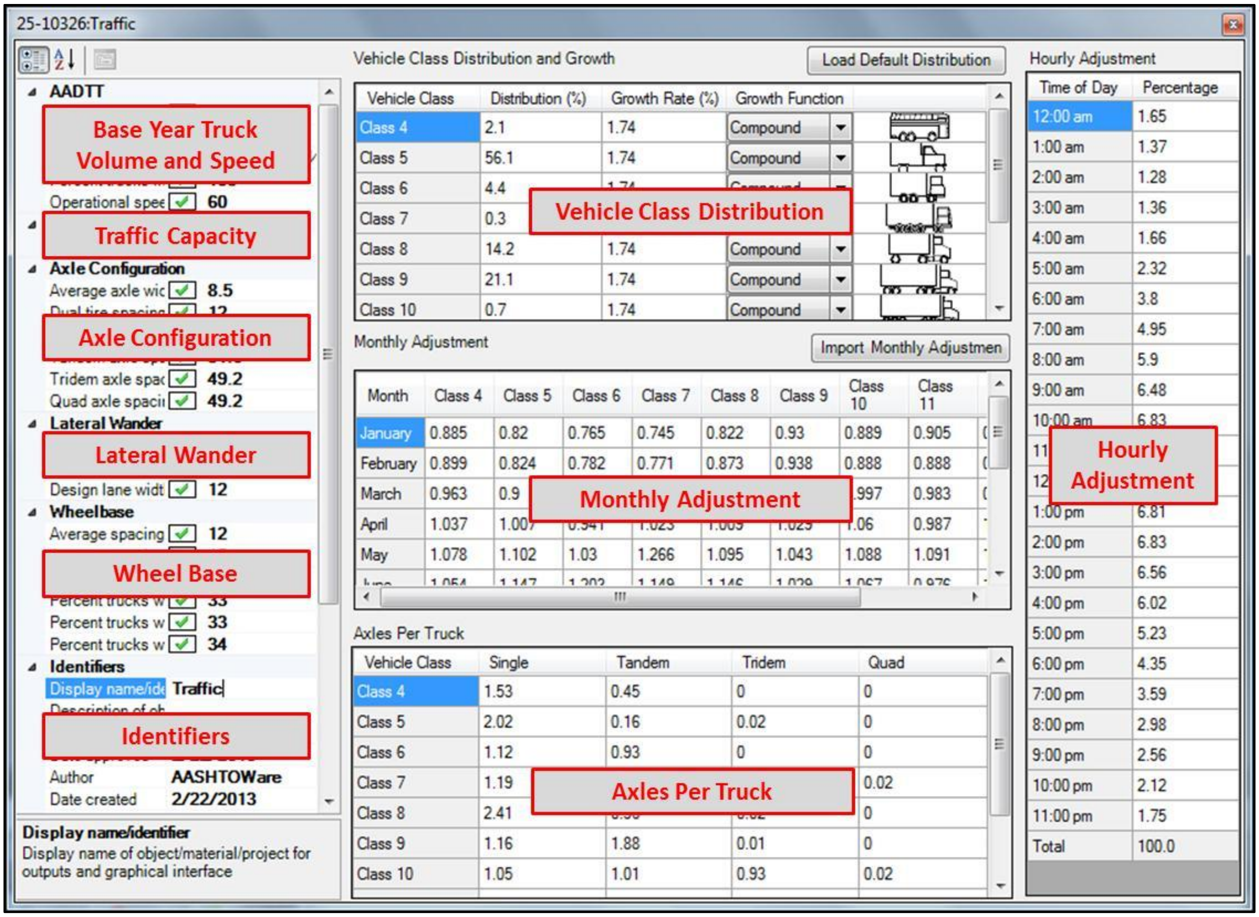


Figure 3.1 Traffic Inputs in the M-E Design Software

3.1.1 CDOT Traffic Data

The Department has various sites on the highway system where instruments have been placed in the roadway to measure axle loads as a vehicle passes over the site. These stations, called Weigh-in-Motion (WIM) sites, can provide accurate information of the existing traffic load. An estimate of growth over the design period will be needed to calculate the traffic load during the design period. The link http://dtdapps.coloradodot.info/Otis/TrafficData is used to access traffic load information.

The Division of Transportation Development (DTD) Traffic Analysis Unit supplies traffic analysis for pavement structure design. All vehicular traffic on the design roadway is projected for the design year in the categories of passenger cars, single unit trucks, and combination trucks with various axle configurations. The DTD Traffic Analysis Unit will make adjustments for directional distribution and lane distribution.

The DTD provides traffic projections of Average Annual Daily Traffic (AADT) and ESALs. The designer must request 10, 20, and 30-year traffic projections for flexible pavements and 20 and 30-year traffic projections for rigid pavements from the Traffic Section of DTD. Requests for traffic projections should be coordinated with the appropriate personnel of DTD. The pavement designer can help ensure accurate traffic projections are provided by documenting local conditions and planned economic development that may affect future traffic loads and volumes. The DTD should be notified of special traffic situations when traffic data are requested. Some special situations may include:

- A street that is or will be a major arterial route for city buses.
- A roadway that will carry truck traffic to and from heavily used distribution or freight centers.
- A highway that will experience an increase in traffic from a connection to a major, high-traffic area.
- A highway that will be constructed in the near future.
- A roadway that will experience a decrease in traffic due to the future opening of a parallel roadway facility.

3.1.2 Traffic Inputs Hierarchy

The M-E Design methodology defines three levels of traffic data inputs based on how well the pavement designer can estimate future truck traffic for the roadway being designed. **Table 3.1 Hierarchy of Traffic Inputs** presents the hierarchy description of traffic inputs and common data sources. Refer to **Table 2.8 Selection of Input Hierarchical Level** for selection of an appropriate hierarchical level for traffic inputs. **Table 3.2 Recommendations of Traffic Inputs at Each Hierarchical Level** presents the traffic input requirements of the M-E Design method and the recommendations for obtaining these inputs at each hierarchical input level.

Table 3.1 Hierarchy of Traffic Inputs

Input Hierarchy	Description
Level 1	Site-specific traffic data determined from site-specific measurements of weigh-in-motion data • Volume counts • Traffic adjustment factors • Axle load distribution
Level 2	Site-specific traffic volume counts • CDOT averages of traffic adjustment factors and axle load data • Derived averages from CDOT weigh-in-motion • Automatic vehicle classification historical data
Level 3	Site-specific traffic volume counts and national averages of traffic adjustment factors and axle load data (M-E Design software defaults)

Table 3.2 Recommendations of Traffic Inputs at Each Hierarchical Level

Input	Level 1	Level 2	Level 3
AADT	Use project specific historical traffic volume data **Section 3.1.3 Volume Counts**		
Traffic Growth Rate Distribution Factor	Use project specific historical traffic volume data **Section 3.1.5 Growth Factors for Trucks**		
Lane and Directional Distribution Factor	Use project specific values	**Section 3.1.4 Lane and Directional Distributions**	
Vehicle Class Distribution	Use project specific values	Use CDOT averages **Table 3.5 Level 2 Vehicle Class Distribution Factors**	Use M-E Design software defaults
Monthly Adjustment Factor	Use project specific values	Use CDOT averages **Table 3.7 Level 2 Monthly Adjustment Factors**	
Hourly Distribution Factor	Use project specific values	Use CDOT averages **Table 3.8 Hourly Distribution Factors**	
Axle Load Distribution	Use project specific values	Use CDOT averages **Section 3.1.10 Axle Load Distribution**	
Operational Speed	Use posted or design speed (Levels 1 and 2 not available)		
Number of Axles Per Truck	Use project specific values	Use CDOT averages **Table 3.6 Level 2 Number of Axles Per Truck**	
Lateral Traffic Wander	Use M-E Design software defaults (Levels 1 and 2 not available) **Section 3.1.12 Lateral Wander of Axle Load**		
Axle Configuration	Use M-E Design software defaults (Levels 1 and 2 not available) **Section 3.1.13 Axle Configuration and Wheelbase**		
Wheelbase	Use project specific values	Use national defaults **Section 3.1.13 Axle Configuration and Wheelbase**	
Tire Pressure	Use M-E Design software defaults (Levels 1 and 2 not available) **Section 3.1.14 Tire Pressure**		

3.1.3 Volume Counts

M-E Design characterizes traffic volume as the Annual Average Daily Truck Traffic (AADTT) (see **Figure 3.2 OTIS Screenshot**). AADTT is a product of Annual Average Daily Traffic (AADT) and percent trucks (FHWA vehicle Classes 4 through 13). Project specific AADTT for the base year is required for pavement design/analysis of all hierarchical input levels. CDOT reports both AADT and AADTT, thus historical AADT and/or AADTT estimates for a specific project segment can be accessed from the link:
http://dtdapps.coloradodot.info/Otis/TrafficData.

Stations | AADT | Future Traffic | ESAL

Found **27** Short Duration stations and **1** Continuous Count stations. Click the magnifying glass icon in front of a station to see count data below.

Projection Year: 2038

Export to Excel

	Station ID	Route	Start	End	AADT	Year	Single Trucks	Combined Trucks	% Trucks	DHV	Projected AADT	Projected Single Trucks	Projected Combined Trucks
	100994	025A	199.397	200.132	239,000	2016	5,500	6,500	5	8	323,128	7,436	8,788
	100995	025A	200.132	201.592	253,000	2016	6,600	6,100	5	8	339,273	8,851	8,180
	100996	025A	201.592	202.688	232,000	2016	6,300	5,600	5.1	8	321,320	8,726	7,756
	100997	025A	202.688	203.537	224,000	2016	6,000	5,600	5.2	8	305,312	8,178	7,633
	100998	025A	203.537	204.037	217,000	2016	6,700	6,500	6.1	8	286,223	8,837	8,574
	100999	025A	204.037	205.057	221,000	2016	7,100	6,900	6.3	8	291,499	9,365	9,101
	101000	025A	205.057	205.919	213,000	2016	6,800	6,600	6.3	8	285,633	9,119	8,851
	101001	025A	205.919	206.149	211,000	2016	6,800	6,800	6.4	8	282,951	9,119	9,119
	101002	025A	206.149	206.335	206,000	2016	6,600	6,600	6.4	8	280,778	8,996	8,996
	101003	025A	206.335	206.991	203,000	2016	6,300	6,300	6.2	8	263,291	8,171	8,171
	101004	025A	206.991	207.581	195,000	2016	6,200	6,200	6.4	8	255,060	8,110	8,110
	101005	025A	207.581	207.99	227,000	2016	6,100	6,800	5.7	8	276,940	7,442	8,296

Figure 3.2 OTIS Screenshot

The designer needs to be vigilant when using OTIS's traffic counts since the date of data collection/calculation may not match with the that of the project. When this occurs the designer needs to use OTIS to calculate the projected traffic for when the project/roadway will be completed. The projected traffic for the year the project is completed and open to the traveling public should be used for the initial traffic volume in M-E Design. An example is as follows:

> Example: A project on State Highway 76 between mile markers 0.0 and 6.0 is planned to be completed and open to the public in 2022. The OTIS data, **Figure 3.3 OTIS Projected Traffic for I-76** shows the data was calculated in 2018.

Stations | AADT | Future Traffic | ESAL

Found **5** Short Duration stations and **0** Continuous Count stations. Click the magnifying glass icon in front of a station to see count data below.

Projection Year: 2040

Export to Excel

	Station ID	Route	Start	End	AADT	Year	Single Trucks	Combined Trucks	% Trucks	DHV	Projected AADT	Projected Single Trucks	Projected Combined Trucks
	103379	076A	0	1.768	86,00	2018	4000	5,400	10.9	8	115,326	5,364	7,241
	103380	076A	1.768	3.223	89,00	2018	3000	4,600	8.6	8	124,244	4,188	6,422
	103381	076A	3.223	4.217	86,00	2018	3400	4,400	9	8	121,948	4,821	6,239
	103382	076A	4.217	5.777	86,00	2018	4900	6,600	13.4	8	123,840	7,056	9,504
	103383	076A	5.777	6.803	84,00	2018	4000	5,000	10.7	8	111,720	5,320	6,650

Figure 3.3 OTIS Projected Traffic for I-76

> Step 1. Change the '*Projection Year*' to 2022, see red box on **Figure 3.4 OTIS 2022 Projected Traffic for I-76**. Once the year has been changed OTIS will automatically calculate the projected traffic volumes.
>
> Step 2. Use the station with the highest traffic volume, in this example the highest volume is between mile markers 4.217 and 5.777, see the purple box on **Figure 3.4 OTIS 2022 Projected Traffic for I-76**. If the designer needs to analyze smaller sections within the project for multiple designs, then they should use the traffic counts of the station closest to the design segment.

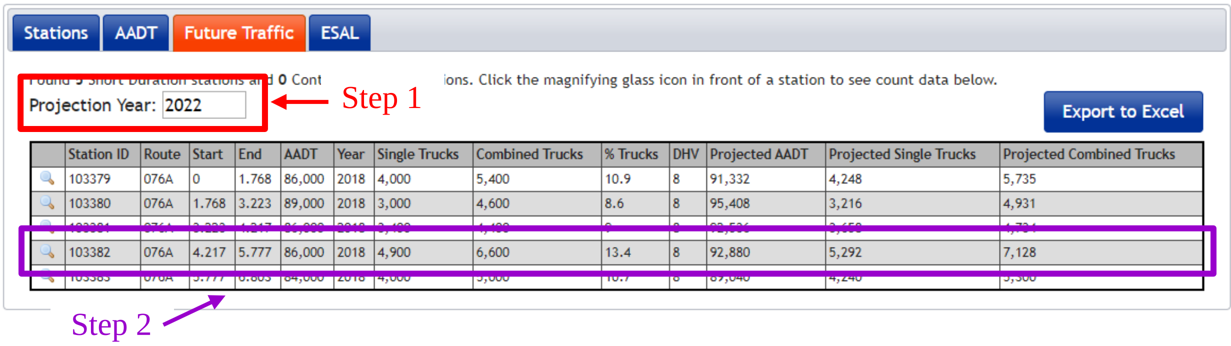


Figure 3.4 OTIS 2022 Projected Traffic for I-76

3.1.4 Lane and Directional Distributions

The most heavily used lane is referred to as the design lane. Generally, the outside lanes are the design lanes. Traffic analysis determines a percent of all trucks traveling on the facility for the design lanes. This is also referred to as a lane distribution factor.

The percent of trucks in the design direction is applied to the two directional AADT to account for any differences to truck volumes by the direction. The percent trucks in the design direction is referred to as the directional distribution factor. Generally, the directional distribution factor is a 50/50 percent split. If the number of lanes and volumes are not the same for each direction, it may be appropriate to design a different pavement structure for each direction of travel, **Figure 3.6 Diagram of Lanes for M-E Design and LCCA.**

CDOT uses a design lane factor to account for the lane and directional distribution which are combined into one factor, the design lane factor. **Table 3.3 Design Lane Factor** shows the relationship of the design lane factor versus the lane and directional distributions. **Figure 3.5 M-E Design Software Screenshot of AADTT** presents the M-E Design software screenshot of lane and directional distribution factors.

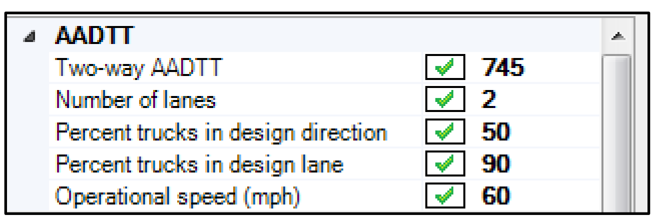


Figure 3.5 M-E Design Software Screenshot of AADT

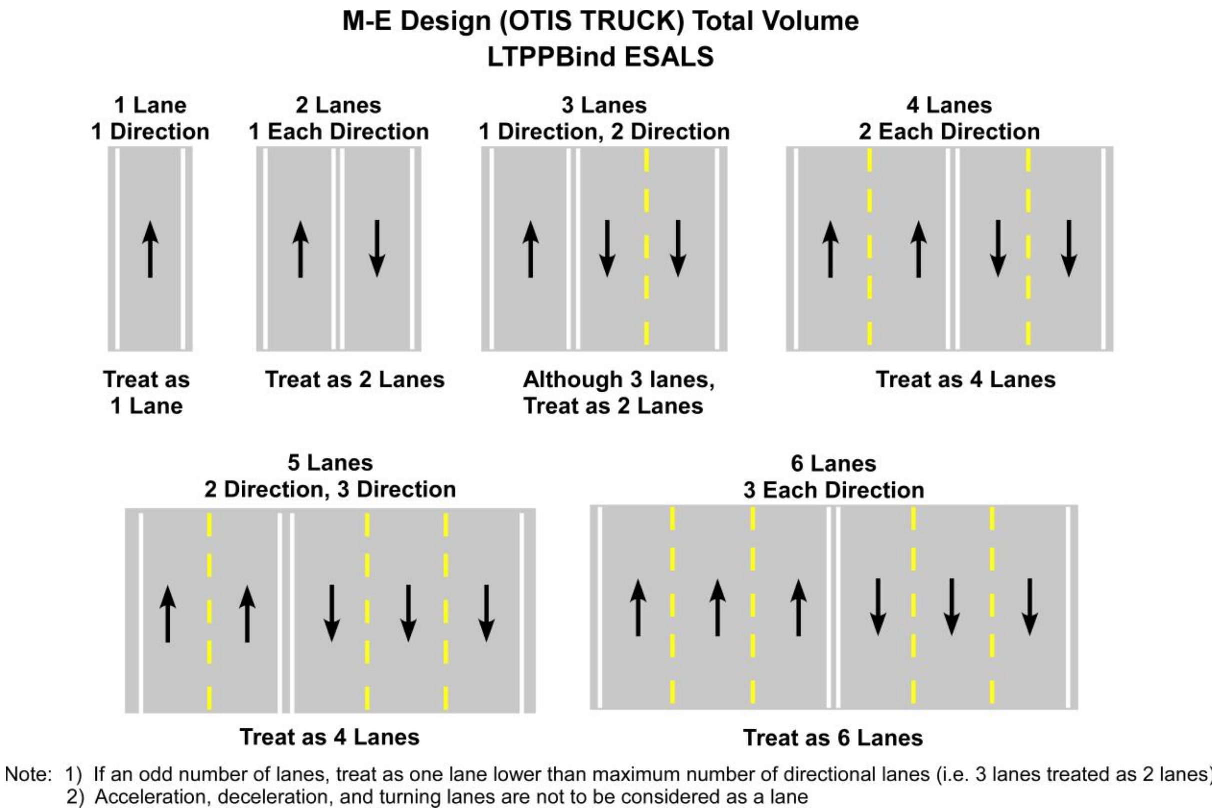


Figure 3.6 Diagram of Lanes for M-E Design and LCCA

Table 3.3 Design Lane Factor

Type of Facility	Number of Lanes in Design Direction	Design Lane Factor	Percent of Total Trucks in the Design Lane (Outside Lane)	Directional Split (Design Direction/ Non-design Direction)
One Way	1	1.00	100	NA
2-Lanes	1	0.60	100	60/40
4-Lanes	2	0.45	90	50/50
6-Lanes	3	0.309	60	50/50
8-Lanes	4	0.25	50	50/50
Note: The *Highway Capacity Manual*, 2000 (Exhibit 12-13) recommends using a default value for directional split of 60/40 on a two-lane highway may it be rural or urban (3).				

3.1.5 Growth Factors for Trucks

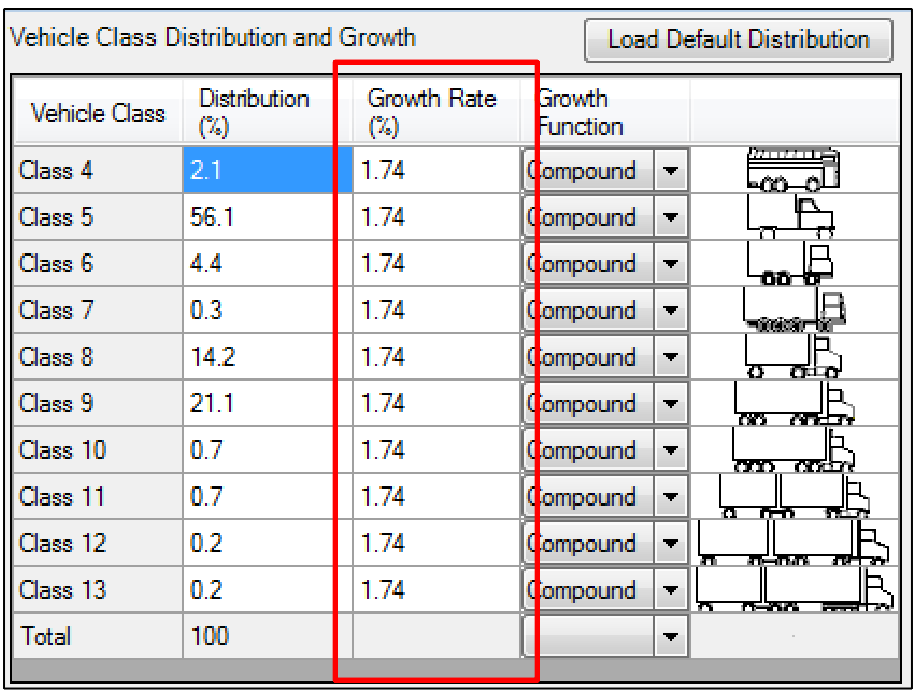
The number of vehicles using a pavement tends to increase with time. A simple growth rate assumes the AADT is increased by the same amount each year. A compound growth rate assumes the AADT percent growth rate for any given year is applied to the volume during the preceding year. CDOT a **compound** growth rate. Use equation **Eq. 3-1** or **Table 3.4 Growth Rate Determined Using OTIS 20-Year Growth Factor**.

$$\mathbf{T_f = (1+r)^{\mathit{n}}} \qquad \textbf{Eq. 3-1}$$

Where:

T_f = growth factor
r = rate if growth expressed as a fraction
n = number of years

The CDOT traffic analysis unit may be consulted to estimate the increase in truck traffic over time (using the M-E Design approach). The M-E Design software has the capability to use different growth rates for different truck classes, but assumes the growth rate remains the same throughout the analysis period, see **Figure 3.7 M-E Design Software Screenshot of Growth Rate**. Additionally, the estimated traffic volumes to be used in the pavement design can be subjected to roadway capacity limits. Project specific growth rates are required for pavement design/analysis for all hierarchical input levels. An estimate of truck volume growth over the design period can be accessed from the link http://dtdapps.coloradodot.info/Otis/TrafficData.



Vehicle Class Distribution and Growth

Load Default Distribution

Vehicle Class	Distribution (%)	Growth Rate (%)	Growth Function
Class 4	2.1	1.74	Compound
Class 5	56.1	1.74	Compound
Class 6	4.4	1.74	Compound
Class 7	0.3	1.74	Compound
Class 8	14.2	1.74	Compound
Class 9	21.1	1.74	Compound
Class 10	0.7	1.74	Compound
Class 11	0.7	1.74	Compound
Class 12	0.2	1.74	Compound
Class 13	0.2	1.74	Compound
Total	100		

Figure 3.7 M-E Design Software Screenshot of Growth Rate

Table 3.4 Growth Rate Determined Using OTIS 20-Year Growth Factor

20 Year Growth Factor (OTIS)	r (%)
1.00	0.000
1.05	0.245
1.10	0.478
1.15	0.703
1.20	0.916
1.25	1.122
1.30	1.320
1.35	1.512
1.40	1.697
1.45	1.877
1.50	2.048
1.55	2.196
1.60	2.378
1.65	2.535
1.70	2.689
1.75	2.840
1.80	2.983
1.85	3.123
1.90	3.261
1.95	3.393
2.00	3.526
2.05	3.655
2.10	3.784
2.15	3.902
2.20	4.021
2.25	4.139

20 Year Growth Factor (OTIS)	r (%)
2.30	4.256
2.35	4.364
2.40	4.475
2.45	4.584
2.50	4.690
2.55	4.793
2.60	4.894
2.65	4.995
2.70	5.092
2.75	5.179
2.80	5.283
2.85	5.377
2.90	5.464
2.95	5.559
3.00	5.647
3.05	5.834
3.10	5.820
3.15	5.905
3.20	5.988
3.25	6.070
3.30	6.149
3.35	6.232
3.40	6.310
3.45	6.386
3.50	6.465

3.1.6 Vehicle Classification

M-E Design requires a vehicle class distribution which represents the percentage of each truck class (Classes 4 through 13) within the truck traffic distribution as part of the AADTT for the base year. The sum of the percent AADTT of all truck classes should equal 100. This normalized distribution is determined from an analysis of AVC data and represents data collected over multiple years. CDOT uses a classification scheme of categorizing vehicles into three bins. These vehicle classifications types are (1):

- Passenger vehicles: Classes 1-3 are 0-20 feet
- Single unit trucks: Classes 4-7 are 20-40 feet
- Combination trucks: Classes 8-13 and greater than 40 feet long

These bins are further broken down into 13 classes, **Figure 3.8 CDOT Vehicle Classification**. The 13 classification scheme follows FHWA vehicle type classification. For some situations, a fourth bin containing all unclassified vehicles is used. Additional classes, Class 14 and 15, may also be included in the fourth bin. CDOT vehicle classes are presented in **Figure 2.3 Functional Classification Map**. FHWA vehicle classes with definitions are presented as follows (2). **Note:** The M-E Design method does not include vehicle Classes 1 to 3 (i.e. light weight vehicles) and Classes 14 and 15 (i.e. unclassified vehicles).

Class 1: Motorcycles: All two or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handlebars rather than steering wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles. This vehicle type may be reported at the option of the State.

Class 2: Passenger Cars: All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers, including passenger cars pulling recreational or other light trailers.

Class 3: Other Two-Axle, Four-Tire Single Unit Vehicles: All two-axle, four-tire, vehicles other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, carryalls, and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification. Because automatic vehicle classifiers have difficulty distinguishing Class 3 from Class 2, these two classes may be combined into Class 2.

Class 4: Buses: All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires, or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. Modified buses should be considered a truck and should be appropriately classified.

Class 5: Two-Axle, Six-Tire, Single-Unit Trucks: All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with two axles and dual rear wheels.

Class 6: Three-Axle Single-Unit Trucks: All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with three axles.

Class 7: Four or More Axle Single-Unit Trucks: All trucks on a single frame with four or more axles.

Class 8: Four or Fewer Axle Single-Trailer Trucks: All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.

Class 9: Five-Axle Single-Trailer Trucks: All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.

Class 10: Six or More Axle Single-Trailer Trucks: All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.

Class 11: Five or fewer Axle Multi-Trailer Trucks: All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.

Class 12: Six-Axle Multi-Trailer Trucks: All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.

Class 13: Seven or More Axle Multi-Trailer Trucks: All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

Note: In reporting information on trucks the following criteria should be used:

- Truck tractor units traveling without a trailer will be considered single-unit trucks.
- A truck tractor unit pulling other such units in a "saddle mount" configuration will be considered one single-unit truck and defined only by the axles on the pulling unit.
- Vehicles are defined by the number of axles in contact with the road, therefore, "floating" axles are counted only when in the down position.
- The term "trailer" includes both semi and full trailers.

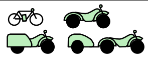
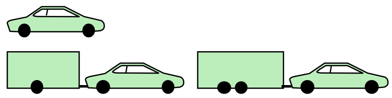
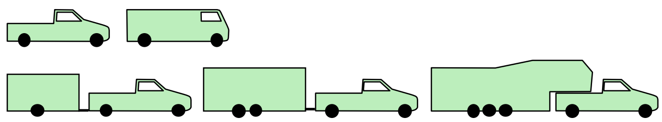
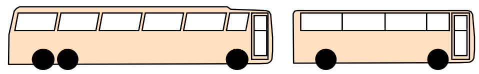
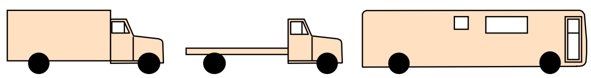
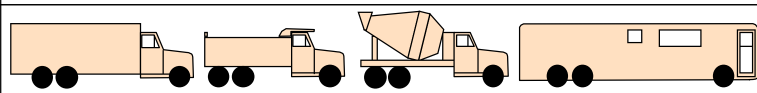
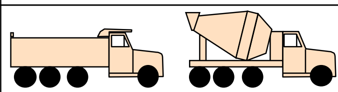
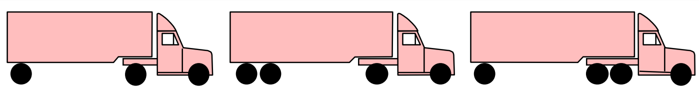
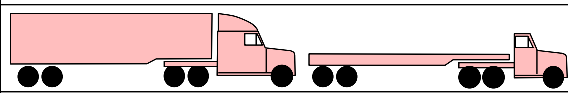
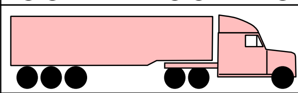
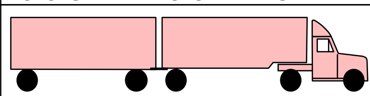
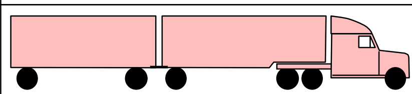
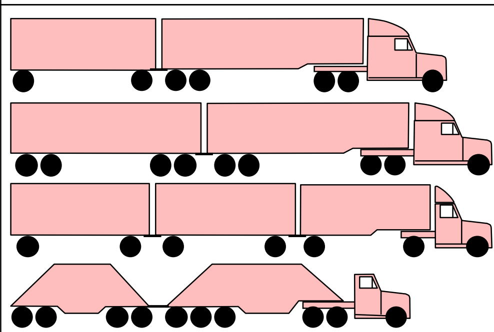
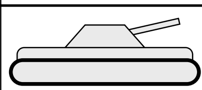
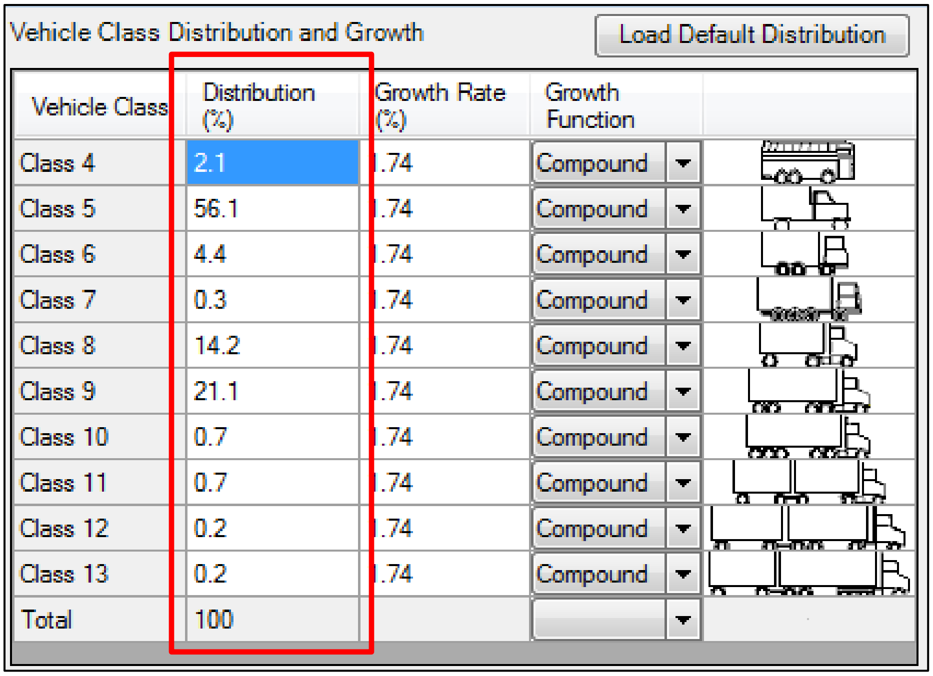
	Class	Schema	Description
Light-weight Vehicles	1		all motorcycles plus two wheel axles
	2		all cars plus one/two axle trailers
	3		all pickups and vans single/dual wheels plus one/two/three axle trailers
Single Unit Vehicles	4		buses single/dual wheels
	5		two axle, single unit single/dual wheels
	6		three axle, single unit
	7		four axle, single unit
Combination Unit Vehicles	8		four or less axles, single trailers
	9		five axles, single trailers
	10		six or more axles, single trailers
	11		five or less axles, multi-trailers
	12		six axles, multi-trailers
	13		seven or more axles, multi-trailers
Unclassified Vehicles	14		Unclassifiable vehicle
	15		Not used

Figure 3.8 CDOT Vehicle Classifications

For M-E Design, the vehicle class distribution inputs can be defined at three hierarchical input levels. See **Figure 3.9 M-E Design Software Screenshot of Vehicle Class Distribution**. The three input levels are described in the following sections.



Vehicle Class Distribution and Growth

Load Default Distribution

Vehicle Class	Distribution (%)	Growth Rate (%)	Growth Function
Class 4	2.1	.74	Compound
Class 5	56.1	.74	Compound
Class 6	4.4	.74	Compound
Class 7	0.3	.74	Compound
Class 8	14.2	.74	Compound
Class 9	21.1	.74	Compound
Class 10	0.7	.74	Compound
Class 11	0.7	.74	Compound
Class 12	0.2	.74	Compound
Class 13	0.2	.74	Compound
Total	100		

Figure 3.9 M-E Design Software Screenshot of Vehicle Class Distribution

Figure 3.10 Vehicle Classification by Axle Length is based off a study of 4,245,260 records of vehicle data and show the length ranges observed in the LTPP data for each of the 16 axle classes. Each end of the bar and histogram were truncated at the point where the histogram was one standard deviation of the average length per class. This figure illustrates the overlapping nature of attempts to map axle-based classification to length bins.

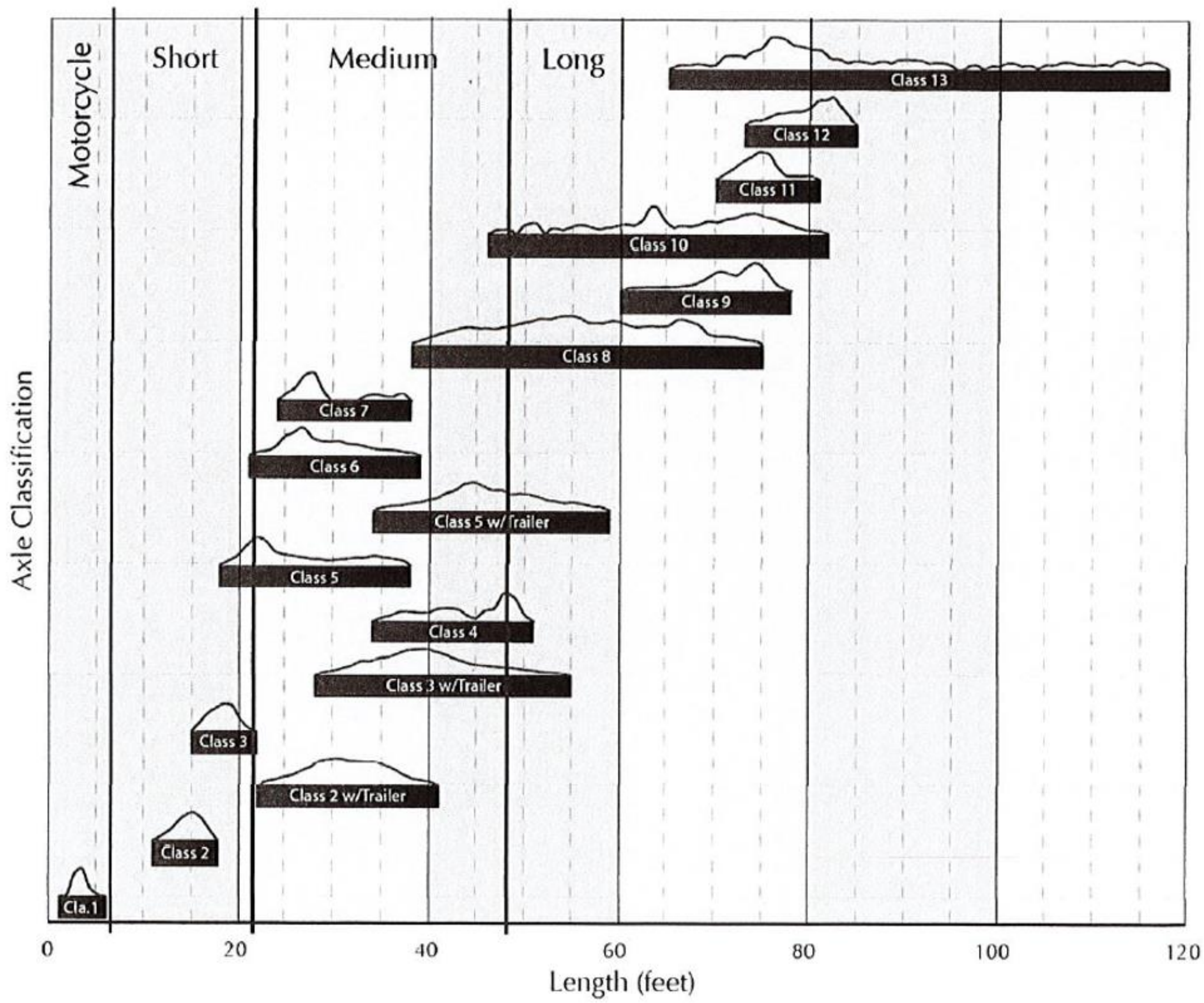


Figure 3.10 Vehicle Classification and Axle Length

3.1.6.1 Level 1 Vehicle Class Inputs

Level 1 inputs are the actual measured site data (over 24-hours) and must be used for highways with heavy seasonal and atypical traffic. This data can be obtained from the CDOT DTD.

3.1.6.2 Level 2 Vehicle Class Inputs

Level 2 inputs are the regional average values determined from traffic analyses of data from various WIM and AVC sites in Colorado. The traffic data analyses indicated three vehicle class distribution clusters defined according to location and highway functional class. The descriptions of vehicle class clusters are presented as follows, refer to **Table 3.5 Class 5 and Class 9 Distribution per Cluster Type**:

- **Cluster 1**: This distribution had one large primary peak for Class 5 vehicles with percentage ranging from 40 to 75. There was a secondary peak for Class 8 and 9 trucks with percentage ranging from 10 to 30 percent. The main highway functional class was 4-lane rural principal arterials (non-interstate, US highways and state routes), and a few sections of urban freeways.

- **Cluster 2**: This distribution had two distinct peaks for Class 5 and 9 vehicles. The percentage of Class 5 ranged from 5 to 35 and the percentage of Class 9 ranged from 40 to 80. The main highway functional class was 4-lane rural principal arterial, interstate, and highways.

- **Cluster 3**: This distribution had two distinct peaks for Class 5 and 9 vehicles with percentages of each class ranging from 15 to 50, with Class 9 trucks having a slightly higher percentage than other truck types. The main highway functional classes were 2-lane rural principal arterials (other), 2-lane rural major collectors, and 4-lane urban principal arterials.

Table 3.5 Class 5 and Class 9 Distribution Per Cluster Type

Cluster	Class 5 Distribution (%)	Class 9 Distribution (%)	Most Common Highway Functional Class
Cluster 1	40-75	10-30	• 4-lane rural principal arterial (non-interstate) • A few urban freeways
Cluster 2	5-35	40-80	• 4-lane rural principal arterial (other) • Interstate highways
Cluster 3	15-50	15-50	• 2-lane rural principal arterial (other) • 2-lane rural major collector • 4-lane urban principal arterial

As a minimum, selection of the appropriate cluster type must be based on project location as shown in **Table 3.6 Level 2 Vehicle Class Distribution Factors** and **Figure 3.11 Vehicle Class Distribution Factors for CDOT Clusters**. Designers must choose the default vehicle class distribution for the cluster that most closely describes the design traffic stream for the roadway under design.

3.1.6.3 Level 3 Vehicle Class Inputs

For situations, where CDOT clusters are not suitable and Level 1 data is not available, designers may use an appropriate default Truck Traffic Class (TTC) group in the M-E Design software. TTC factors were developed using traffic data from over a 100 WIM and AVC sites located nationwide. The data was obtained from FHWA LTPP program data.

Designers may select the most appropriate from seventeen TTC groups that best describe the truck traffic mix of a given project. **Figure 3.12 Truck Traffic Classification Groups** presents a screenshot of the seventeen TTC groups and their descriptions in the M-E Design software.

Table 3.6 Level 2 CDOT Vehicle Class Distribution Factors

Vehicle Class	Cluster 1 (Predominately Class 5)	Cluster 2 (Predominately Class 9)	Cluster 3 (Predominately Class 5 and 9)
	4-Lane Rural Principal Arterial (Non-Interstate)	4-Lane Rural Principal Arterial (Interstates and Highways)	2-Lane Rural Principal Arterial (other) 2-Lane Rural Major Collector 4-Lane Urban Principal Arterial
4	2.1	2.7	5.1
5	56.1	19.3	32.3
6	4.4	4.5	18
7	0.3	0.3	0.3
8	14.2	4.6	4.9
9	21.1	61.9	36.8
10	0.7	1.6	1.2
11	0.7	2.7	0.7
12	0.2	1.3	0.5
13	0.2	1.1	0.2

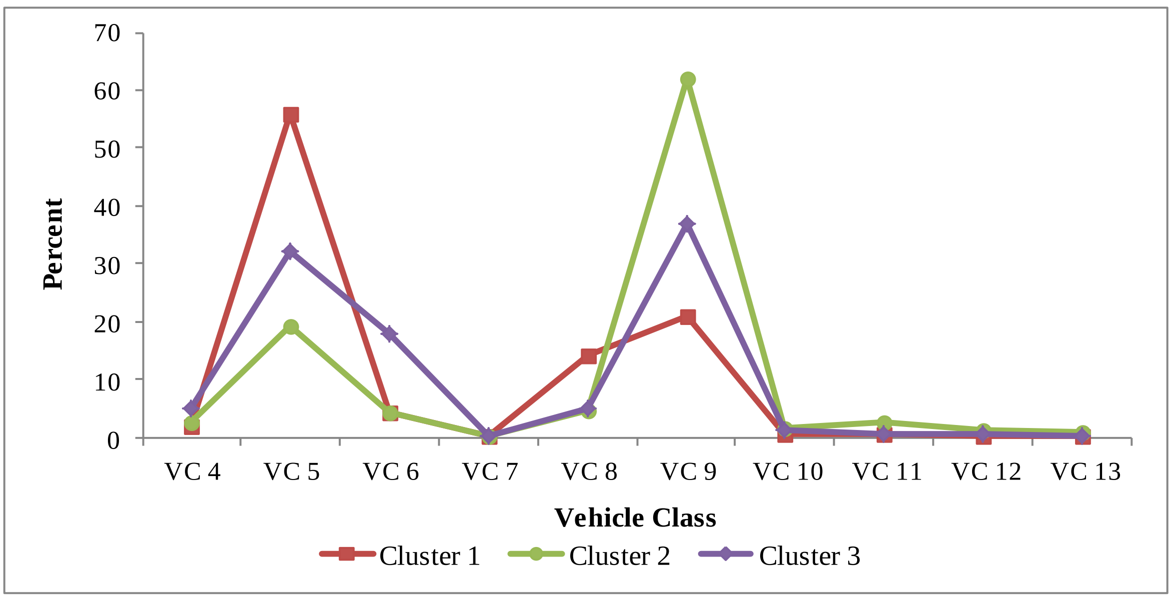


Figure 3.11 Vehicle Class Distribution Factors for CDOT Clusters

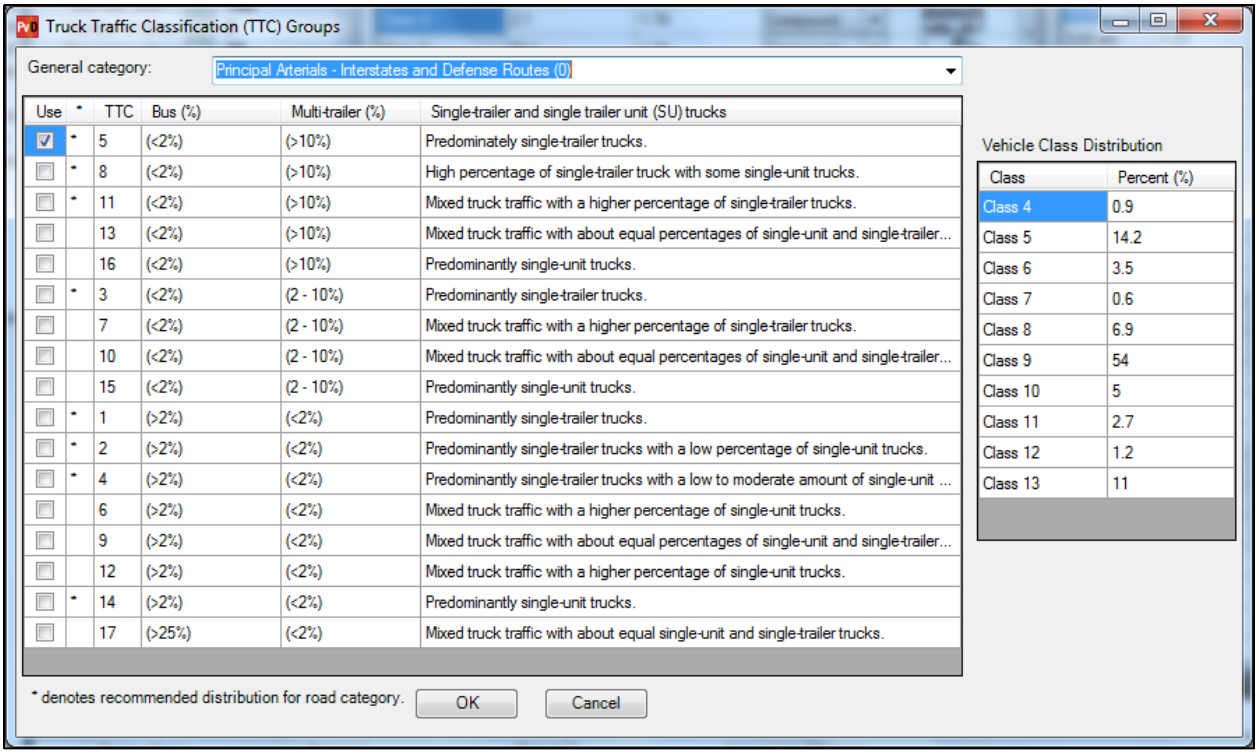


Figure 3.12 Truck Traffic Classification Groups

3.1.7 Number of Axles per Truck

This input represents the average number of axles for each truck class (FHWA vehicle Class 4 to 13) and each axle type (single, tandem, tridem, and quad). For the M-E Design, the number of axles per truck can be defined at three hierarchical input levels. **Figure 3.13 M-E Design Screenshot of Number of Axles Per Truck** presents the M-E Design software screenshot for the number of axles per truck. Three input levels are described in the following sections.

3.1.7.1 Level 1 Number of Axles Per Truck

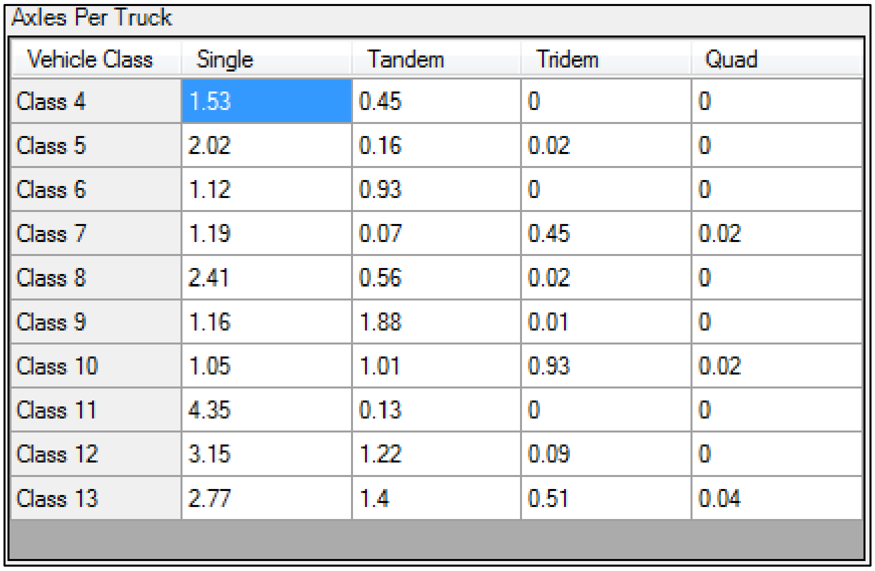
Level 1 inputs are the actual measured site data and must be used for highways with heavy seasonal and atypical traffic. This data can be obtained from the CDOT DTD.

3.1.7.2 Level 2 Number of Axles Per Truck

Level 2 inputs are the statewide average values determined from traffic analyses of data from various WIM and AVC sites in Colorado. Refer to **Table 3.7 Level 2 Number of Axles Per Truck** for CDOT statewide averages.

3.1.7.3 Level 3 Number of Axles Per Truck

Level 3 inputs are the M-E Design software defaults. This level is <u>not</u> recommended.



Axles Per Truck

Vehicle Class	Single	Tandem	Tridem	Quad
Class 4	1.53	0.45	0	0
Class 5	2.02	0.16	0.02	0
Class 6	1.12	0.93	0	0
Class 7	1.19	0.07	0.45	0.02
Class 8	2.41	0.56	0.02	0
Class 9	1.16	1.88	0.01	0
Class 10	1.05	1.01	0.93	0.02
Class 11	4.35	0.13	0	0
Class 12	3.15	1.22	0.09	0
Class 13	2.77	1.4	0.51	0.04

Figure 3.13 M-E Design Screenshot of Number of Axles Per Truck

Table 3.7 Level 2 Number of Axles Per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
4	1.53	0.45	0.00	0.00
5	2.02	0.16	0.02	0.00
6	1.12	0.94	0.00	0.00
7	1.19	0.07	0.45	0.02
8	2.41	0.56	0.02	0.00
9	1.16	1.90	0.01	0.00
10	1.15	1.01	0.93	0.02
11	4.35	0.29	0.02	0.00
12	3.27	1.22	0.09	0.00
13	2.77	1.40	0.51	0.04

3.1.8 Monthly Adjustment Factors (Trucks)

Truck traffic monthly adjustment factors represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month. The sum of monthly factors for all months for each vehicle class must equal 12. These monthly distribution factors may be determined from WIM, AVC, or manual truck traffic counts. Axle data shall come from CDOT's data base.

For the M-E Design, the monthly adjustment factors can be defined at three hierarchical input levels, see **Figure 3.9 M-E Design Screenshot of Monthly Adjustment Factors**. The input levels are described in the following sections.

3.1.8.1 Level 1 Monthly Adjustment Factors

Level 1 inputs are the actual measured site data and must be used for highways with heavy seasonal and atypical traffic. This data can be obtained from the CDOT DTD, **Figure 3.14 M-E Design Screenshot of Monthly Adjustment Factors.**

3.1.8.2 Level 2 Monthly Adjustment Factors

Level 2 inputs are the statewide average values determined from traffic analyses of data from various WIM and AVC sites in Colorado. Refer to **Table 3.8 Level 2 Monthly Adjustment Factors** for Level 2 averages. The axle data and clusters shall come from CDOT's data base.

3.1.8.3 Level 3 Monthly Adjustment Factors

Level 3 inputs are the M-E Design software defaults. This level is not recommended for use on CDOT projects

Monthly Adjustment

Import Monthly Adjustmen

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	0.885	0.82	0.765	0.745	0.822	0.93	0.889	0.905	0.918	0.862
February	0.899	0.824	0.782	0.771	0.873	0.938	0.888	0.888	0.976	0.83
March	0.963	0.9	0.843	1.066	0.993	0.99	0.997	0.983	0.919	0.925
April	1.037	1.007	0.941	1.023	1.009	1.029	1.06	0.987	1.031	1.05
May	1.078	1.102	1.03	1.266	1.095	1.043	1.088	1.091	1.123	0.999
June	1.054	1.147	1.203	1.149	1.146	1.029	1.067	0.976	1.083	1.035
July	1.103	1.209	1.467	1.279	1.175	0.995	1.09	1.057	1.082	1.255
August	1.117	1.158	1.275	1.034	1.148	1.049	1.089	1.101	1.055	0.968
Septem...	1.064	1.114	1.116	1.032	1.05	1.041	1.066	1.07	0.976	1.081
October	1.029	1.011	0.966	0.979	0.985	1.043	1.017	1.031	0.944	1.103
Novem...	0.912	0.906	0.857	0.862	0.879	1.004	0.951	0.998	1.001	1.031
Decem...	0.859	0.802	0.755	0.794	0.825	0.909	0.798	0.913	0.892	0.861

Figure 3.14 M-E Design Screenshot of Monthly Adjustment Factors

Table 3.8 Level 2 Monthly Adjustment Factors

Month	Vehicle/Truck Class									
	4	5	6	7	8	9	10	11	12	13
Jan	0.885	0.820	0.765	0.745	0.822	0.930	0.889	0.905	0.918	0.862
Feb	0.899	0.824	0.782	0.771	0.873	0.938	0.888	0.888	0.976	0.830
Mar	0.963	0.900	0.843	1.066	0.993	0.990	0.997	0.983	0.919	0.925
Apr	1.037	1.007	0.941	1.023	1.009	1.029	1.060	0.987	1.031	1.050
May	1.078	1.102	1.030	1.266	1.095	1.043	1.088	1.091	1.123	0.999
Jun	1.054	1.147	1.203	1.149	1.146	1.029	1.067	0.976	1.083	1.035
Jul	1.103	1.209	1.467	1.279	1.175	0.995	1.090	1.057	1.082	1.255
Aug	1.117	1.158	1.275	1.034	1.148	1.049	1.089	1.101	1.055	0.968
Sep	1.064	1.114	1.116	1.032	1.050	1.041	1.066	1.070	0.976	1.081
Oct	1.029	1.011	0.966	0.979	0.985	1.043	1.017	1.031	0.944	1.103
Nov	0.912	0.906	0.857	0.862	0.879	1.004	0.951	0.998	1.001	1.031
Dec	0.859	0.802	0.755	0.794	0.825	0.909	0.798	0.913	0.892	0.861

3.1.9 Hourly Distribution Factors (Trucks)

The hourly distribution factors represent the percentage of the total truck traffic within each hour of the day and are required for the analysis of only rigid pavements. Site-specific hourly distribution factors may be estimated from WIM, AVC, or manual truck traffic counts.

For the M-E Design, the hourly distribution factors can be defined at three hierarchical input levels. The three input levels are described in the following sections.

3.1.9.1 Level 1 Hourly Distribution Factors

Level 1 inputs are the actual measured site data and must be used for highways with heavy seasonal and atypical traffic. This data can be obtained from the CDOT DTD.

3.1.9.2 Level 2 Hourly Distribution Factors

Level 2 inputs are the statewide average values determined from traffic analyses of data from various WIM and AVC sites in Colorado. Refer to **Table 3.9 Hourly Distribution Factors** and **Figure 3.16 Level 2 Hourly Distribution Factors**. The axle data and clusters shall come from CDOT's database.

3.1.9.3 Level 3 Hourly Distribution Factors

Level 3 inputs are the M-E Design software defaults. This level is not recommended.

Table 3.9 Hourly Distribution Factors

Time Period	Distribution, percent	Time Period	Distribution, percent
12:00 a.m. - 1:00 a.m.	1.65	12:00 p.m. - 1:00 p.m.	6.75
1:00 a.m. - 2:00 a.m.	1.37	1:00 p.m. - 2:00 p.m.	6.81
2:00 a.m. - 3:00 a.m.	1.28	2:00 p.m. - 3:00 p.m.	6.83
3:00 a.m. - 4:00 a.m.	1.36	3:00 p.m. - 4:00 p.m.	6.56
4:00 a.m. - 5:00 a.m.	1.66	4:00 p.m. - 5:00 p.m.	6.02
5:00 a.m. - 6:00 a.m.	2.32	5:00 p.m. - 6:00 p.m.	5.23
6:00 a.m. - 7:00 a.m.	3.80	6:00 p.m. - 7:00 p.m.	4.35
7:00 a.m. - 8:00 a.m.	4.95	7:00 p.m. - 8:00 p.m.	3.59
8:00 a.m. - 9:00 a.m.	5.90	8:00 p.m. - 9:00 p.m.	2.98
9:00 a.m. - 10:00 a.m.	6.48	9:00 p.m. - 10:00 p.m.	2.56
10:00 a.m. – 11:00 a.m.	6.83	10:00 p.m. - 11:00 p.m.	2.12
11:00 a.m. – 12:00 p.m.	6.85	11:00 p.m. - 12:00 a.m.	1.75

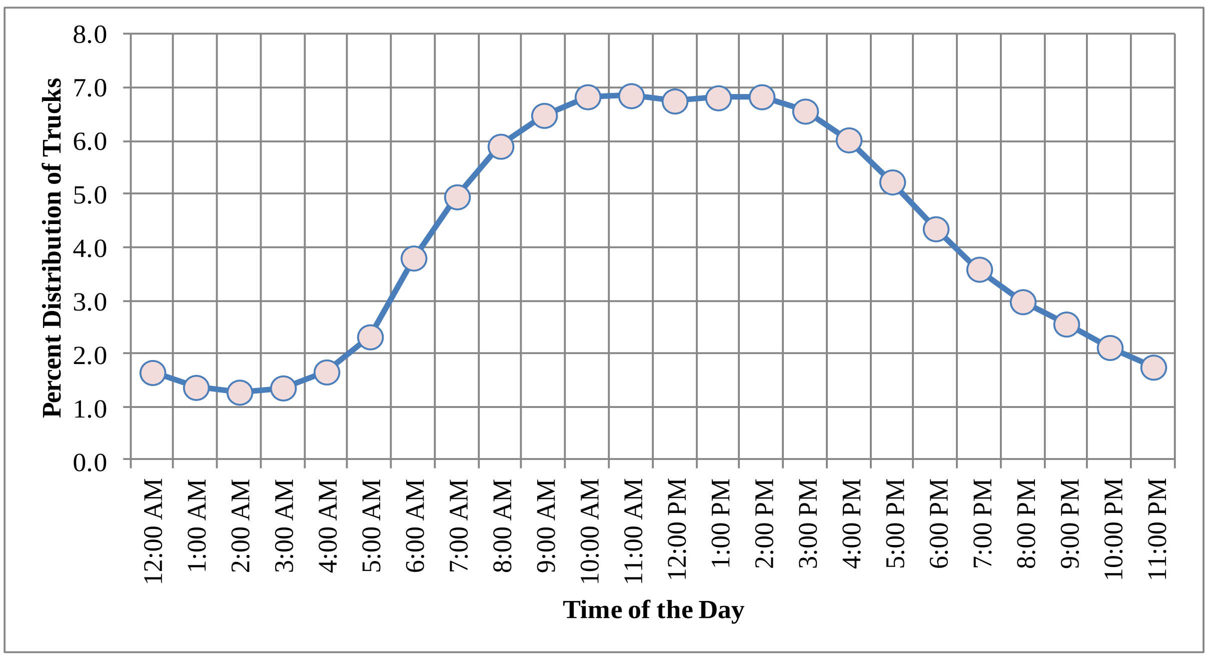


Figure 3.15 Level 2 Hourly Distribution Factors

3.1.10 Axle Load Distribution

The axle load distribution factors represent the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (Classes 4 through 13). A definition of load intervals for each axle type is provided below:

- **Single Axles:** 3,000 lb. to 40,000 lb. at 1,000-lb. intervals
- **Tandem Axles**: 6,000 lb. to 80,000 lb. at 2,000-lb. intervals
- **Tridem and Quad Axles**: 12,000 lb. to 102,000 lb. at 3,000-lb. intervals. Developing site-specific axle load distribution factors involves the processing of a massive amount of WIM data. The processing should be completed external to the M-E Design software using traffic loading analysis software.

For M-E Design, the axle load distribution factors can be defined at three hierarchical input levels. See **Figure 3.17 Single Axle Distribution in the M-E Design Software** for a screenshot of axle load distribution factors in the M-E Design software. The input levels are described in the following sections.

3.1.10.1 Level 1 Axle Load Distribution Factors

Level 1 inputs are the actual measured site data and must be used for highways with unique traffic characteristics and heavy haul routes (i.e. mining, lumber, and agricultural routes). This data can be obtained from the CDOT DTD.

3.1.10.2 Level 2 Axle Load Distribution Factors

Level 2 inputs are the statewide average values determined from traffic analyses of data from various WIM and AVC sites in Colorado. **Table 3.10 Level 2 Axle Load Distribution Factors (Percentages)** through **Table 3.13 Level 2 Quad Axle Load Distribution Factors (Percentages),** presents the CDOT averages of axle load distribution factors for single, tandem, tridem and quad axles for each truck class, respectively. The axle data and clusters shall come from CDOT's data base.

Figure 3.18 CDOT Averages of Single Axle Load Distribution (Classes 5 and 9 only) presents the load distributions of single axles for vehicle Classes 5 and 9. **Figure 3.19 CDOT Averages of Tandem Axle Load Distribution (Classes 5 and 9 only)** presents the load distributions of tandem axles for vehicle Classes 5 and 9. Electronic versions of the Level 2 axle load distributions factors can be obtained from the CDOT Pavement Design office.

3.1.10.3 Level 3 Axle Load Distribution Factors

Level 3 inputs are the M-E Design software defaults. This level is not recommended for use on CDOT projects.

25-10326:Single

Month	Class	Total	3000	4000	5000	6000	7000	8000	9000	10000	11000	12000	13000	140
January	4	99.97	0.28	0.73	1.77	5.18	8.12	12.73	10.08	11.45	9.11	9.81	6.59	7.11
January	5	100.01	3.69	9.71	14.2	17.72	12.56	11.97	7.19	6.2	3.63	3.19	1.71	1.93
January	6	100.02	2.73	2.86	3.85	4.77	4.61	7.48	9.46	14.63	11.36	10.35	5.42	5.76
January	7	100.01	3.78	3.45	2.25	2.75	3.07	3.85	3.32	6.38	7.33	8.54	6.63	6.61
January	8	100.01	7.61	6.63	7.1	8.63	8.44	11.24	9.57	10.41	6.57	5.67	3.15	3.56
January	9	100.02	1.42	2.5	2.93	3.43	3.39	5.89	9.34	18.41	17.14	14.29	5.72	4.49
January	10	99.99	0.92	1.23	1.93	3.3	3.66	6.43	9.17	16.61	15.03	13.75	6.74	6.86
January	11	100.02	1.69	2.17	3.87	6.46	6.14	7.89	8.72	12.31	9.15	8.6	5.12	6.85
January	12	99.98	2.2	3.48	5.08	7.98	7.27	10.22	11.02	14	9.32	8.24	4.47	5
January	13	100.01	3.13	3.18	3.4	6.19	5.25	7.45	7.88	9.91	7.39	8.07	5.02	7.52
February	4	99.99	0.23	0.81	1.74	5.33	8.49	12.67	10.35	11.55	9.15	9.72	6.51	7.1
February	5	100.02	3.98	10.45	15.97	19.7	13.19	11.09	6.08	5.2	2.96	2.58	1.39	1.6
February	6	100.01	2.73	2.85	3.84	4.81	5.06	7.94	9.89	14.79	11.42	10.11	5.22	5.35
February	7	100.03	4.9	1.93	2.27	1.32	4.43	3.99	2.65	4.74	8.66	9.29	7.32	8.44
February	8	99.97	7.33	6.45	7.07	8.51	8.59	11.42	9.48	10.35	6.54	5.62	3.22	3.59
February	9	100	1.47	2.53	2.94	3.44	3.43	6.13	9.36	18.27	16.86	14.31	5.79	4.65
February	10	100	0.97	1.18	1.98	3.36	3.84	6.83	9.38	16.42	14.83	14.36	7.49	6.52
February	11	100.01	0.95	2.18	3.59	6.29	5.7	7.97	8.71	12.13	8.99	8.68	5.57	7.37
February	12	99.99	1.56	2.55	4.06	9.1	7.09	9.61	11.67	12.98	9.67	8.91	4.65	6.85
February	13	99.97	3	3.05	3.24	5.78	4.5	8.1	9.66	9.37	7.94	7.68	5.86	8.8
March	4	100	0.27	0.74	1.61	5.05	7.8	12.35	10.28	11.6	9.29	9.89	6.47	7.27
March	5	100	4.73	10.98	15.79	19.04	12.61	10.96	6.15	5.26	3.02	2.66	1.44	1.63

Figure 3.16 Single Axle Distribution in the M-E Design Software

Table 3.10 Level 2 Single Axle Load Distribution Factors (Percentages)

Mean Axle Load (lbs.)	Vehicle/Truck Class									
	4	5	6	7	8	9	10	11	12	13
3,000	0.24	4.71	2.19	3.49	8.44	1.39	0.76	1.85	1.51	2.59
4,000	0.78	11.26	2.75	3.13	7.28	2.51	1.41	2.11	2.97	3.03
5,000	1.77	16.33	3.98	2.56	7.40	3.00	2.30	3.59	4.66	3.27
6,000	5.24	18.85	5.03	2.64	8.36	3.54	3.49	6.44	8.65	5.20
7,000	8.19	12.49	4.79	2.86	8.10	3.41	3.73	6.09	7.66	4.89
8,000	12.87	10.93	7.67	3.92	10.75	5.87	6.41	8.41	10.14	7.37
9,000	10.32	6.13	9.77	3.87	9.17	9.19	9.18	9.19	11.54	8.06
10,000	11.46	5.22	15.52	5.65	10.06	18.64	17.04	12.53	14.27	10.20
11,000	9.21	2.97	12.24	6.04	6.37	17.62	15.60	9.05	9.77	8.25
12,000	9.87	2.56	10.78	7.46	5.59	14.63	14.47	8.87	8.93	8.60
13,000	6.45	1.39	5.47	6.33	3.07	5.65	7.00	5.49	4.75	5.97
14,000	7.05	1.62	5.52	8.39	3.56	4.26	6.33	6.88	5.34	8.08
15,000	4.78	1.15	3.54	7.22	2.55	2.32	3.63	5.22	3.41	6.20
16,000	2.68	0.69	2.06	5.82	1.55	1.50	1.92	3.20	1.74	3.64
17,000	2.53	0.79	2.15	7.44	1.76	1.64	1.80	3.50	1.70	3.88
18,000	1.56	0.52	1.42	4.57	1.18	1.23	1.05	2.15	0.76	2.19
19,000	1.35	0.51	1.28	4.82	1.15	1.11	0.80	1.84	0.63	1.96
20,000	0.83	0.33	0.79	3.63	0.73	0.68	0.54	1.01	0.35	1.20
21,000	0.76	0.32	0.67	2.78	0.65	0.51	0.51	0.82	0.26	0.94
22,000	0.47	0.21	0.42	1.79	0.38	0.30	0.31	0.40	0.20	0.58
23,000	0.41	0.22	0.36	1.46	0.34	0.23	0.26	0.29	0.18	0.51
24,000	0.23	0.15	0.23	0.76	0.20	0.16	0.22	0.26	0.09	0.42
25,000	0.20	0.16	0.21	0.62	0.19	0.14	0.20	0.14	0.08	0.45
26,000	0.13	0.12	0.15	0.53	0.13	0.09	0.14	0.08	0.05	0.47
27,000	0.11	0.08	0.13	0.60	0.12	0.08	0.13	0.08	0.07	0.29
28,000	0.06	0.03	0.08	0.33	0.07	0.05	0.08	0.06	0.04	0.12
29,000	0.07	0.03	0.08	0.31	0.07	0.04	0.08	0.06	0.04	0.17
30,000	0.06	0.02	0.06	0.30	0.05	0.03	0.06	0.05	0.03	0.10
31,000	0.03	0.02	0.04	0.09	0.04	0.02	0.04	0.03	0.01	0.07
32,000	0.03	0.02	0.04	0.16	0.04	0.02	0.04	0.03	0.01	0.08
33,000	0.02	0.01	0.03	0.11	0.03	0.01	0.03	0.02	0.01	0.06
34,000	0.02	0.01	0.03	0.05	0.03	0.01	0.05	0.02	0.00	0.09
35,000	0.01	0.01	0.02	0.02	0.02	0.01	0.01	0.01	0.00	0.03
36,000	0.01	0.01	0.02	0.01	0.02	0.01	0.03	0.01	0.01	0.05
37,000	0.01	0.00	0.02	0.04	0.02	0.00	0.01	0.01	0.00	0.03
38,000	0.01	0.01	0.02	0.02	0.02	0.00	0.01	0.02	0.00	0.05
39,000	0.00	0.00	0.01	0.01	0.01	0.00	0.02	0.00	0.00	0.03
40,000	0.00	0.00	0.01	0.03	0.02	0.00	0.01	0.00	0.00	0.02
41,000	0.14	0.14	0.42	0.16	0.45	0.09	0.31	0.18	0.11	0.89

Table 3.11 Level 2 Tandem Axle Load Distribution Factors (Percentages)

Mean Axle Load, lbs.	Vehicle/Truck Class									
	4	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
6,000	0.41	38.29	2.94	12.80	18.36	3.21	0.90	4.34	2.19	3.22
8,000	1.51	24.51	7.75	2.15	9.01	5.20	1.57	1.62	3.19	3.76
10,000	2.68	16.41	12.42	3.45	9.79	7.57	3.08	3.78	4.89	5.06
12,000	4.17	8.75	12.11	3.65	10.51	8.61	5.30	6.50	9.15	7.11
14,000	4.46	4.66	9.72	3.15	10.15	8.29	7.08	13.11	10.75	8.50
16,000	4.82	2.61	7.83	0.70	8.39	7.24	8.17	8.03	11.61	8.73
18,000	6.53	1.60	6.30	2.20	6.65	6.08	8.73	8.03	12.58	8.04
20,000	8.19	1.03	5.26	0.65	5.50	5.21	8.66	8.31	12.86	7.51
22,000	9.39	0.71	4.49	3.40	4.33	4.74	8.02	9.39	10.78	7.33
24,000	10.04	0.49	3.86	4.00	3.33	4.50	7.08	9.00	8.14	6.27
26,000	9.41	0.31	3.47	6.15	2.41	4.53	6.35	8.10	5.33	5.05
28,000	8.81	0.21	3.20	2.10	1.83	4.77	6.00	6.46	3.37	4.19
30,000	8.53	0.14	3.32	4.35	1.60	5.41	5.67	4.88	2.06	4.46
32,000	6.48	0.08	2.94	3.15	1.19	5.40	4.73	2.95	0.97	3.34
34,000	4.95	0.05	2.71	5.85	1.08	5.48	4.21	2.16	0.55	2.91
36,000	3.51	0.03	2.48	5.85	0.97	4.66	3.51	1.02	0.33	2.83
38,000	2.10	0.02	2.15	7.55	0.88	3.28	2.54	0.61	0.34	2.16
40,000	1.29	0.02	1.74	6.05	0.74	2.01	1.99	0.44	0.27	2.17
42,000	0.78	0.01	1.39	4.00	0.60	1.20	1.64	0.32	0.15	1.34
44,000	0.52	0.01	1.05	2.50	0.50	0.77	1.10	0.19	0.09	0.83
46,000	0.37	0.01	0.75	3.85	0.39	0.52	0.81	0.09	0.04	0.84
48,000	0.26	0.00	0.52	1.20	0.30	0.36	0.70	0.09	0.12	0.93
50,000	0.19	0.00	0.37	1.60	0.23	0.26	0.53	0.08	0.03	0.62
52,000	0.13	0.02	0.34	4.15	0.19	0.19	0.37	0.05	0.02	0.87
54,000	0.11	0.01	0.24	1.15	0.15	0.14	0.26	0.04	0.02	0.31
56,000	0.08	0.01	0.18	1.40	0.13	0.10	0.20	0.05	0.04	0.28
58,000	0.05	0.00	0.12	0.15	0.11	0.07	0.16	0.03	0.01	0.23
60,000	0.04	0.00	0.08	1.00	0.08	0.05	0.15	0.03	0.02	0.15
62,000	0.03	0.00	0.06	0.75	0.07	0.04	0.11	0.07	0.01	0.12
64,000	0.02	0.00	0.05	0.60	0.05	0.03	0.07	0.02	0.00	0.22
66,000	0.01	0.00	0.03	0.00	0.05	0.02	0.05	0.01	0.00	0.09
68,000	0.01	0.00	0.03	0.00	0.03	0.02	0.10	0.01	0.00	0.11
70,000	0.01	0.00	0.01	0.00	0.03	0.01	0.03	0.01	0.00	0.04
72,000	0.00	0.00	0.02	0.40	0.02	0.01	0.03	0.01	0.00	0.05
74,000	0.01	0.00	0.01	0.00	0.03	0.01	0.01	0.01	0.00	0.05
76,000	0.00	0.00	0.01	0.00	0.02	0.00	0.02	0.01	0.00	0.03
78,000	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.01	0.00	0.02
80,000	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.01	0.00	0.02
82,000	0.05	0.00	0.05	0.00	0.23	0.04	0.06	0.16	0.05	0.25

Table 3.12 Level 2 Tridem Axle Load Distribution Factors (Percentages)

Mean Axle Load, lbs.	Vehicle/Truck Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	65.36	0.00	4.82	11.33	38.87	15.53	0.00	19.21	3.20
15,000	0.00	17.43	0.00	3.96	7.69	11.93	10.88	0.00	6.55	4.21
18,000	0.00	8.73	0.00	3.78	9.59	8.99	9.05	0.00	6.99	4.87
21,000	0.00	4.26	0.00	6.28	9.32	5.50	7.23	0.00	14.85	3.31
24,000	0.00	1.65	0.00	3.79	7.83	3.82	6.03	0.00	3.22	2.59
27,000	0.00	0.98	0.00	5.04	7.42	3.24	6.05	0.00	0.63	3.11
30,000	0.00	0.48	0.00	4.84	7.77	2.90	5.79	0.00	3.41	3.75
33,000	0.00	0.24	0.00	5.82	5.88	2.90	5.78	0.00	6.59	4.29
36,000	0.00	0.34	0.00	8.30	5.45	2.93	6.49	0.00	6.02	5.24
39,000	0.00	0.12	0.00	8.19	4.74	2.65	5.87	0.00	5.54	6.88
42,000	0.00	0.11	0.00	9.17	4.17	2.76	5.58	0.00	6.16	7.31
45,000	0.00	0.06	0.00	8.36	3.60	2.52	4.06	0.00	2.33	6.91
48,000	0.00	0.06	0.00	7.35	3.02	2.14	2.71	0.00	5.15	6.34
51,000	0.00	0.06	0.00	4.93	2.75	2.12	2.23	0.00	4.50	6.75
54,000	0.00	0.03	0.00	3.28	1.49	1.67	1.68	0.00	2.97	7.60
57,000	0.00	0.04	0.00	3.77	1.64	1.46	1.36	0.00	2.37	5.84
60,000	0.00	0.01	0.00	1.22	1.32	0.98	1.05	0.00	0.00	5.41
63,000	0.00	0.01	0.00	2.88	0.62	0.60	0.69	0.00	3.23	4.18
66,000	0.00	0.00	0.00	0.86	0.47	0.46	0.53	0.00	0.10	2.55
69,000	0.00	0.00	0.00	0.55	0.49	0.35	0.40	0.00	0.16	1.56
72,000	0.00	0.00	0.00	0.50	0.36	0.25	0.26	0.00	0.00	1.08
75,000	0.00	0.00	0.00	0.46	0.38	0.21	0.22	0.00	0.00	0.78
78,000	0.00	0.00	0.00	0.43	0.57	0.15	0.13	0.00	0.00	0.57
81,000	0.00	0.00	0.00	0.25	0.36	0.13	0.10	0.00	0.00	0.43
84,000	0.00	0.01	0.00	0.42	0.24	0.08	0.08	0.00	0.00	0.34
87,000	0.00	0.00	0.00	0.09	0.12	0.07	0.05	0.00	0.00	0.33
90,000	0.00	0.00	0.00	0.53	0.24	0.06	0.03	0.00	0.00	0.22
93,000	0.00	0.00	0.00	0.01	0.09	0.04	0.02	0.00	0.00	0.11
96,000	0.00	0.00	0.00	0.02	0.09	0.03	0.02	0.00	0.00	0.03
99,000	0.00	0.00	0.00	0.01	0.03	0.01	0.01	0.00	0.00	0.04
102,000	0.00	0.01	0.00	0.10	0.90	0.17	0.06	0.00	0.00	0.18

Table 3.13 Level 2 Quad Axle Load Distribution Factors (Percentages)

Mean Axle Load, lbs.	Vehicle/Truck Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	0.00	0.00	41.50	39.41	0.00	0.00	13.63
15,000	0.00	0.00	0.00	3.73	0.00	0.00	6.08	0.00	0.00	3.04
18,000	0.00	0.00	0.00	0.00	0.00	0.00	5.50	0.00	0.00	4.15
21,000	0.00	0.00	0.00	16.67	0.00	0.15	16.55	0.00	0.00	4.46
24,000	0.00	0.00	0.00	0.17	0.00	0.00	0.60	0.00	0.00	19.83
27,000	0.00	0.00	0.00	0.00	0.00	0.00	1.10	0.00	0.00	1.99
30,000	0.00	0.00	0.00	0.00	0.00	0.00	0.78	0.00	47.75	1.84
33,000	0.00	0.00	0.00	0.00	0.00	8.35	1.16	0.00	14.70	5.11
36,000	0.00	0.00	0.00	0.00	0.00	50.00	2.23	0.00	19.35	1.89
39,000	0.00	0.00	0.00	0.00	0.00	0.00	1.60	0.00	13.80	4.63
42,000	0.00	0.00	0.00	0.00	0.00	0.00	0.96	0.00	0.00	5.71
45,000	0.00	0.00	0.00	0.00	0.00	0.00	3.04	0.00	0.00	1.21
48,000	0.00	0.00	0.00	15.00	0.00	0.00	2.14	0.00	1.90	3.81
51,000	0.00	0.00	0.00	0.00	0.00	0.00	1.34	0.00	0.00	3.76
54,000	0.00	0.00	0.00	0.00	0.00	0.00	1.39	0.00	0.00	4.01
57,000	0.00	0.00	0.00	0.00	0.00	0.00	1.95	0.00	2.45	1.80
60,000	0.00	0.00	0.00	33.33	0.00	0.00	5.33	0.00	0.00	3.31
63,000	0.00	0.00	0.00	0.00	0.00	0.00	2.20	0.00	0.00	2.49
66,000	0.00	0.00	0.00	14.47	0.00	0.00	3.08	0.00	0.00	3.46
69,000	0.00	0.00	0.00	16.67	0.00	0.00	0.88	0.00	0.00	2.80
72,000	0.00	0.00	0.00	0.00	0.00	0.00	0.46	0.00	0.00	1.38
75,000	0.00	0.00	0.00	0.00	0.00	0.00	0.14	0.00	0.00	2.04
78,000	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.45
81,000	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.00	0.00	0.28
84,000	0.00	0.00	0.00	0.00	0.00	0.00	0.19	0.00	0.00	1.60
87,000	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.03
90,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.71
93,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
96,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
99,000	0.00	0.00	0.00	0.00	0.00	0.00	1.61	0.00	0.00	0.00
102,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.56

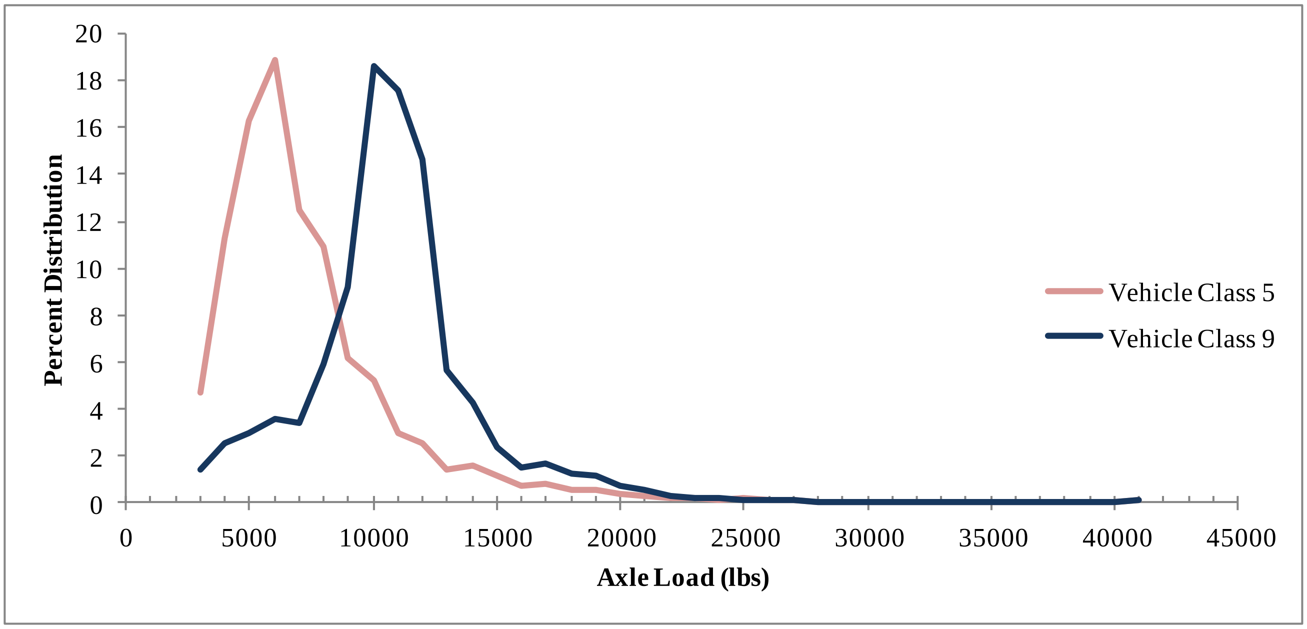


Figure 3.17 CDOT Averages of Single Axle Load Distribution (Classes 5 and 9 only)

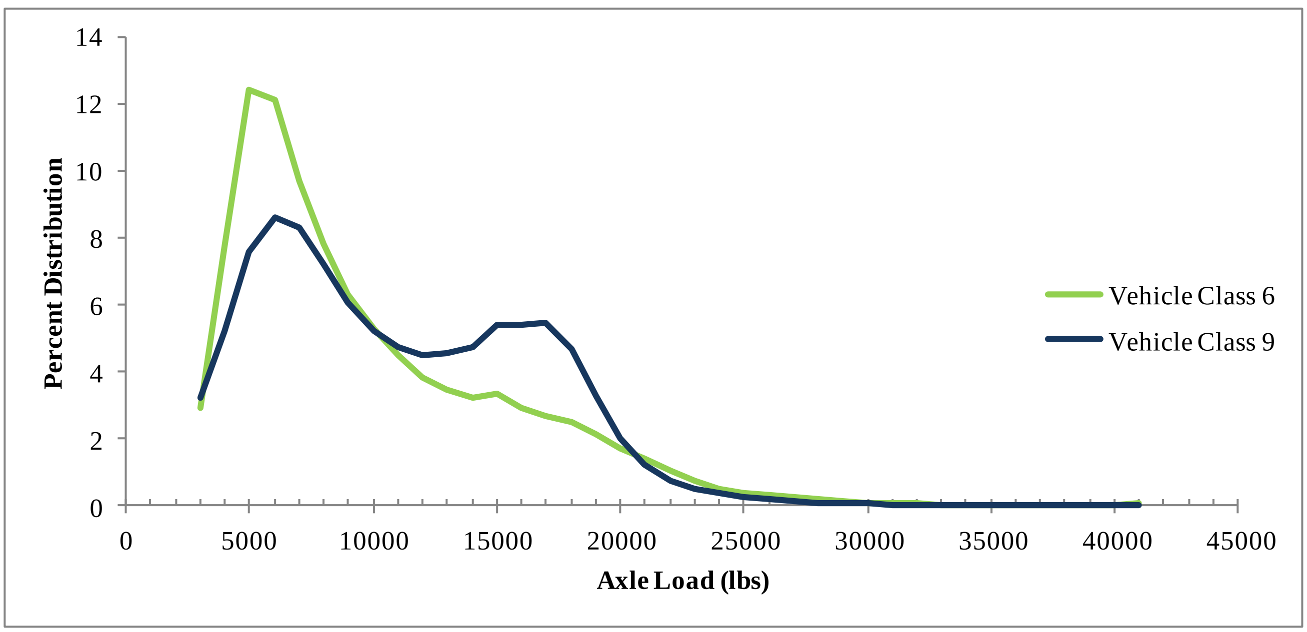


Figure 3.18 CDOT Averages of Tandem Axle Load Distribution (Classes 5 and 9 only)

3.1.11 Vehicle Operational Speed (Trucks)

The vehicle operational speed of trucks or the average travel speed generally depends on many factors, including the roadway facility type (interstate or otherwise), terrain, percentage of trucks in the traffic stream, and so on. Truck speed has a significant impact on the HMA dynamic modulus (E*) and the predicted performance. Lower speeds resulting higher incremental damage, i.e. more fatigue cracking or deeper ruts or faulting. The posted truck speed limit is suggested unless local site conditions, such as a steep upgrade or bus stop, require a lower speed.

3.1.11.1 Lateral Wander of Axle Loads

The inputs required for characterizing lateral wander (see **Figure 3.20 M-E Design Software Screenshot of Traffic Lateral Wander** include the following:

- **Mean Wheel Location:** This is the distance from the outer edge of the wheel to the pavement marking (see **Figure 3.21 Schematic of Mean Wheel Location**). The M-E Design software provides a default value of 18 inches which is recommended unless a measure value is available.

- **Traffic Wander Standard Deviation:** This is the standard deviation of the lateral traffic wander. The wander is used to predict distress and performance by determining the number of axle load applications over a specified point. For standard lane widths, a standard deviation value of 10 inches is suggested unless a measured value is available. A lower or higher lateral wander value is suggested for narrower or wider lanes, respectively.

- **Design Lane Width:** This is the distance between the lane markings on either side of the design lane (see **Figure 3.22 Schematic of Design Lane Width**).

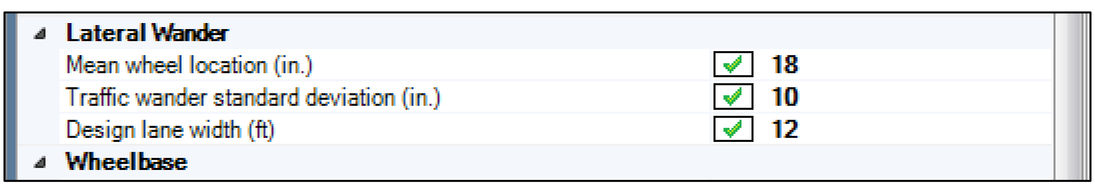


Figure 3.19 M-E Design Software Screenshot of Traffic Lateral Wander

Figure 3.20 Schematic of Mean Wheel Location

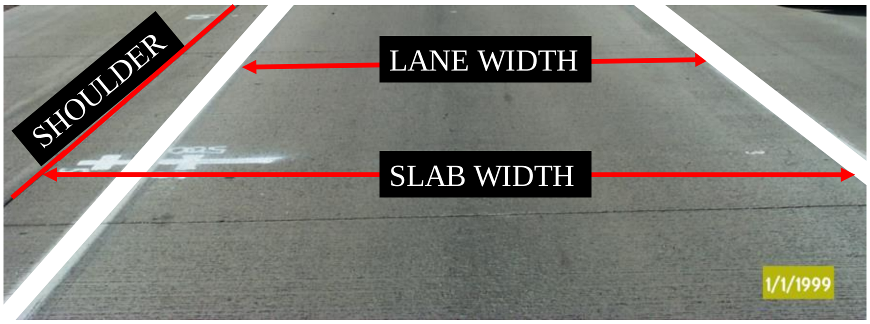


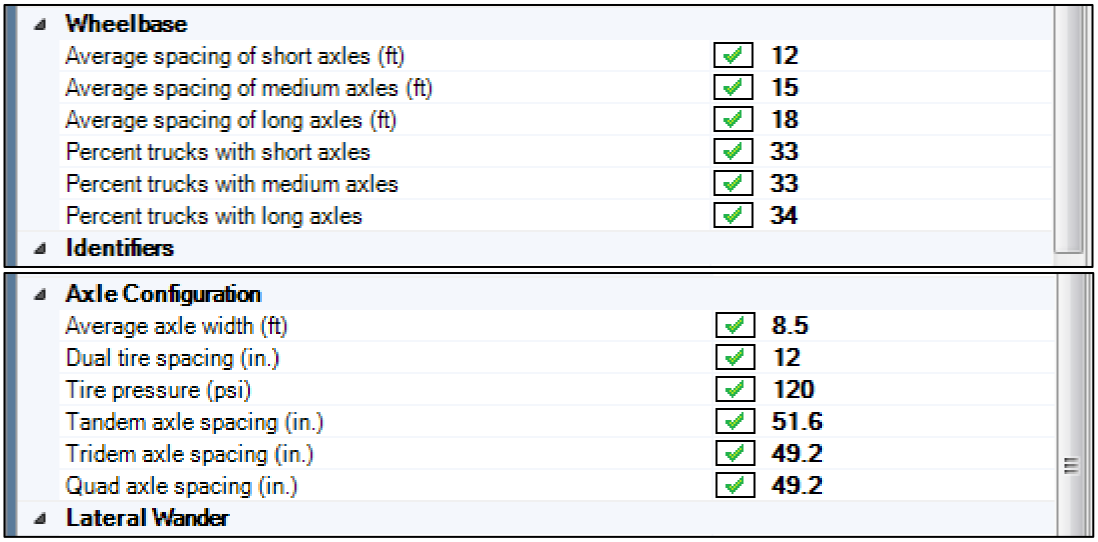
Figure 3.21 Schematic of Design Lane Width

3.1.12 Axle Configuration and Wheelbase

The inputs needed to describe the configurations of the typical tire and axle loads (see **Figure 3.23 Axle Configuration and Wheelbase in the M-E Design Software** and **Figure 3.24 Schematic of Axle Configuration and Wheel Base**) include:

- **Average Axle Width:** This input is the distance between two outside edges of an axle. The recommended value of axle width for trucks is 8.5 feet.

- **Dual Tire Spacing:** This input is the distance between centers of a dual tire. The recommended value of dual tire spacing for trucks is 12 inches.



Wheelbase	
Average spacing of short axles (ft)	12
Average spacing of medium axles (ft)	15
Average spacing of long axles (ft)	18
Percent trucks with short axles	33
Percent trucks with medium axles	33
Percent trucks with long axles	34
Identifiers	
Axle Configuration	
Average axle width (ft)	8.5
Dual tire spacing (in.)	12
Tire pressure (psi)	120
Tandem axle spacing (in.)	51.6
Tridem axle spacing (in.)	49.2
Quad axle spacing (in.)	49.2
Lateral Wander	

Figure 3.22 Axle Configuration and Wheelbase in the M-E Design Software

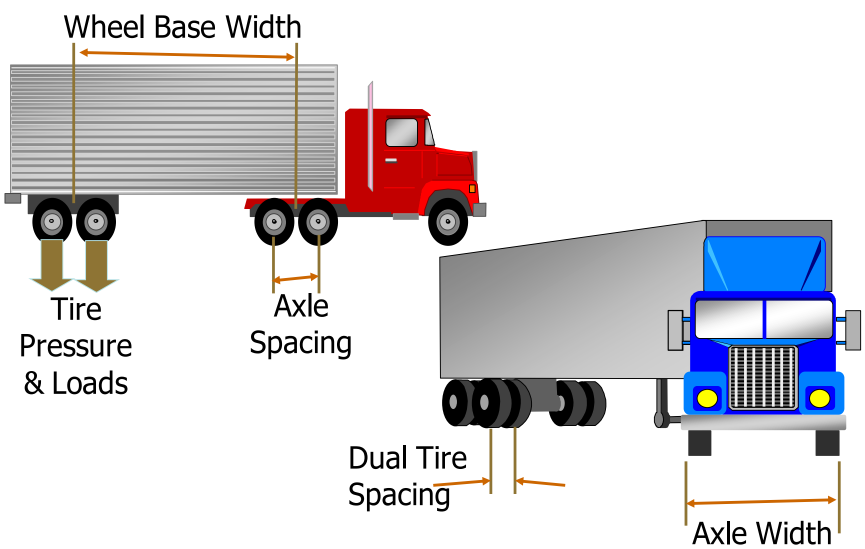


Figure 3.23 Schematic of Axle Configuration and Wheelbase

- **Axle Spacing:** This input is the distance between the two consecutive axles of a tandem, tridem, or quad truck. It is used in determining the number of load applications for JPCP top-down cracking. The spacing of the axles is recorded in the WIM database. These values have been found to be relatively constant for the standard truck classes. The following values are suggested for use unless the predominant truck class has different axle spacing.
  - Tandem axle spacing: 51.6 inches
  - Tridem axle spacing: 49.2 inches
  - Quad axle spacing: 49.2 inches

- **Wheelbase:** This input is the distance between the centers of the front and rear axles. It is used in determining the number of load applications for JPCP top-down cracking. The wheelbase is recorded in the WIM database. The following national averages are suggested for use, unless site-specific wheelbase values are available.

    - Trucks with short spacing (10-13.5 feet): 17.5%
    - Trucks with medium spacing (13.5 to 16.5 feet): 21.6%
    - Trucks with long spacing (16.5 to 20.0 feet): 60.9%

3.1.13 Tire Pressure

Tire pressure may vary with the tire type. A constant value of hot inflation tire pressure representing the average operating conditions should be used. The hot inflation pressure is typically about 10 to 15 percent greater than the cold inflation pressure. A hot inflation tire pressure value of 120 psi is suggested for use unless a special loading condition is simulated.

3.1.14 Traffic Files in Electronic Format for the M-E Design Software

Designers can create their own traffic input files in electronic formats by directly inputting the data using the traffic input interface of the M-E Design software. This is not recommended for most of the required inputs with exceptions for simple inputs such as AADTT, growth rate, etc.

For more complex input types such as the axle load distribution or axles per truck, the designers can add Level 1 and 2 inputs in electronic format from the CDOT DTD. Level 3 input data can be retrieved directly from the M-E Design software.

3.2 Climate

Climate data for the M-E Design software is obtained from weather stations located throughout the state. Information from these stations (temperature, precipitation, wind speed, percent sunshine, and relative humidity) are used to predict the temperature and moisture profiles within the pavement structure during the design life. In addition, the M-E Design software requires the depth to groundwater table (GWT) as an input. The GWT is an important input in the program. Based on calculations from the climate data, the water level may change with seasonal precipitation creating thicker pavement structures. The closer to the surface the GWT is the more likely a thicker pavement structure will be needed. If the designer does not know the GWT then 10 feet, should be used. However, if the designer suspects the GWT is less than 10 feet but does not have boring data to confirm, a boring or other method should be used to determine the GWT. **Note:** The GWT depth value entered in the M-E Design software is the depth below the final pavement surface.

For critical designs, the GWT data can be obtained from Colorado Division of Water Resources database, United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database, or project-specific soil borings. For non-critical designs, one should guestimate the GWT depth based on designer’s experience.

3.2.1 Creating Project Specific Climate Input Files

The M-E Design software will identify the six closest weather stations for a given project location based on its geographic coordinates. Designers can select one or more weather stations based on the proximity to the project location. A single weather station can be selected when the project is within reasonable proximity. It is recommended that a virtual station may be made by selecting at least three surrounding weather stations. Proximity is defined in terms of longitude, latitude, and elevation. The designer should select the stations that are closest to the project in elevation and distance. Given Colorado's mountainous terrain, caution should be used if the elevations are significantly different even if the stations are relatively close to the project. The recommendations for selecting climatic inputs are presented in **Table 3.14 Recommendations for Climatic Inputs**. A screenshot of the climate tab from the M-E Design software is presented in **Figure 3.25 Climate Tab in the M-E Design Software**.

Climate data is currently available for 42 weather stations in Colorado, see **Figures 3.26** though **Figure 3.31**. Weather stations located near the border of neighboring states (Utah, Wyoming, Nebraska, New Mexico, Oklahoma, Kansas and Arizona) may also be used. **Table 3.15 Geographic Coordinates and Data Availability of Colorado Weather Stations** presents the geographic coordinates of Colorado weather stations, including start and end dates of available hourly weather records.

Climate data shown in **Table 3.15 Geographic Coordinates and Data Availability of Colorado Weather Stations** range from 3.9 to 55.9 years. M-E Design calculates the effects of the climate data using the following analysis.

3.2.1.1 Design Life Greater Than Climate Data

If a project has a design life that exceeds the climate data, the program will repeat the climate data until the design life is met.

- **Example:** If a project has a 30 year design life and the station used is (23036) Buckley (also known as Aurora) which only has 10.3 years of data, the program will repeat the 10.3 years until it reaches 30 years. Thus, the data will be repeated 2.91 times.

- **Note:** If an anomaly year of significant freezing or heating exists within the data set, the effect of that year may be exaggerated since it may be used multiple times during a calculation. If this occurs, it is suggested that the designer use a virtual weather station if another station is in reasonable distance and elevation. Using two or more weather stations may help balance the data from the anomaly.

3.2.1.2 Design Life Less Than the Climate Data

Some weather stations have an exceptional amount of climate data. The M-E Design Program uses the first years of data until the design life is met. This means the most current climate data is not used, rather the oldest which may be as indicative of current weather patterns. As such the designer should manually select the most current weather data from the 'Hourly Climate Data' tab shown on **Figure 3.25 Climate Tab in the M-E Design Software**. This is done by scrolling down month and year bars and clicking on the '*Verify Weather*' button.

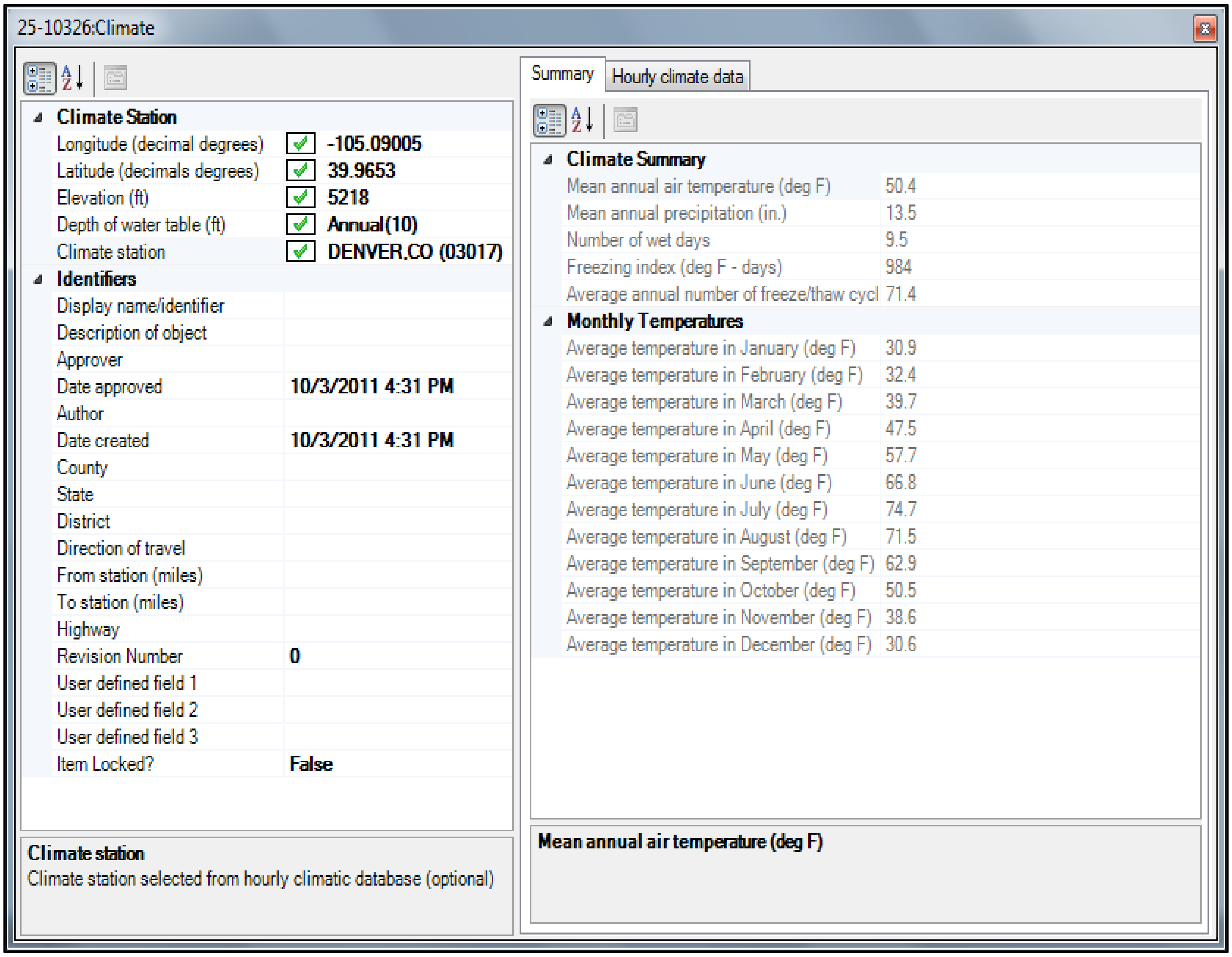
- **Example:** A project has a 20 year design life and is using the (03065) Broomfield weather station, 25.6 years of climate data is available from September 1984 to March 2010. The designer needs to click on the 'Hourly Climate Data' tab and customize the weather data the program will use to be from 1990 to 2010.

- **Note:** During the mid and late 1970's the United States experienced unusually cold, arctic periods for extended times. During this time areas became frozen and remained frozen, thus resulting in an increase in the freezing index and decrease in freeze/thaw cycles. Also, it appears that most locations in Colorado has had increased precipitation since the 1970's, thus caution should be taken if data from the 1970 is used.

Table 3.14 Recommendations for Climatic Inputs

Climate Inputs	Recommendations
Weather Station ≤ 50 Miles and Elevation Difference ≤ 500 feet	Import specific weather station
Weather Station > 50 Miles Elevation Difference > 500 feet	Create a virtual weather station that includes two or more surrounding weather stations
Depth of Water Table (feet)	Actual depth may be found in County Soil Reports[1], project geotechnical reports, or an estimate based on the area. The depth of the water table typically ranges from 3 to 100 feet. If the water table is encountered within the upper 10 feet the designer should investigate dewatering methods and/or drains to lower the water table's elevation. Separate designs should be made for areas that do not have a high water table versus those that do. If dewatering is not an option, the design will likely result in a thick pavement.

Note:
[1] The United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database. Another available resource for estimating depth of water table for a project site is the Colorado Division of Water Resources database and geologic well logs available online at http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/survey/geo/.



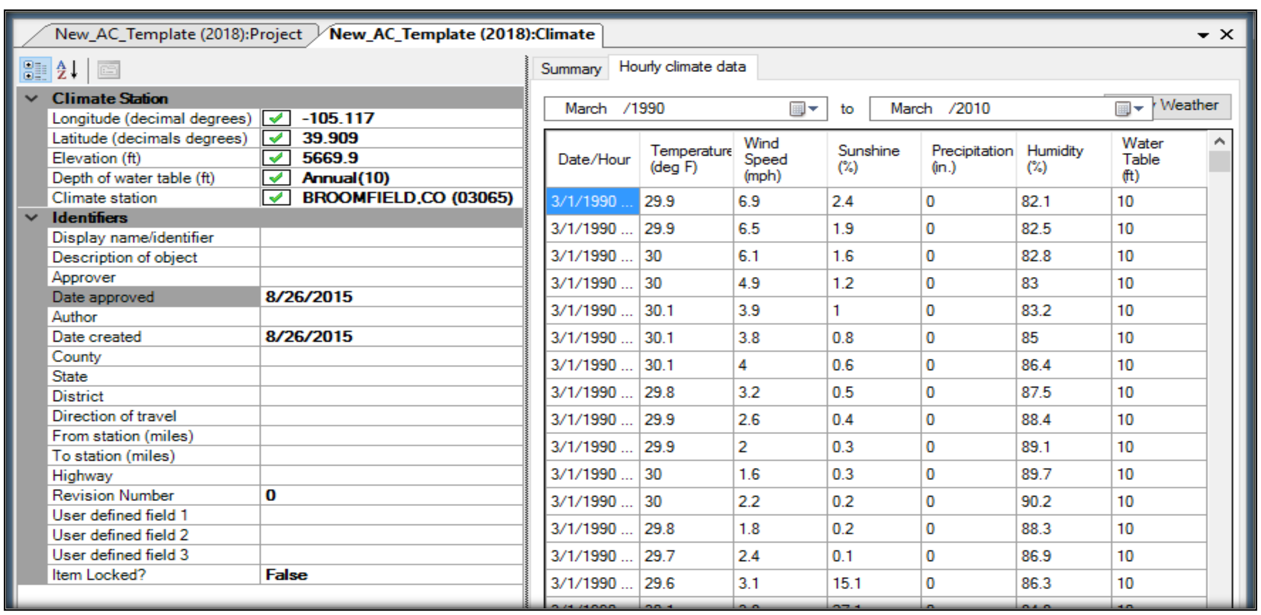


Figure 3.24 Climate Tab in the M-E Design Software

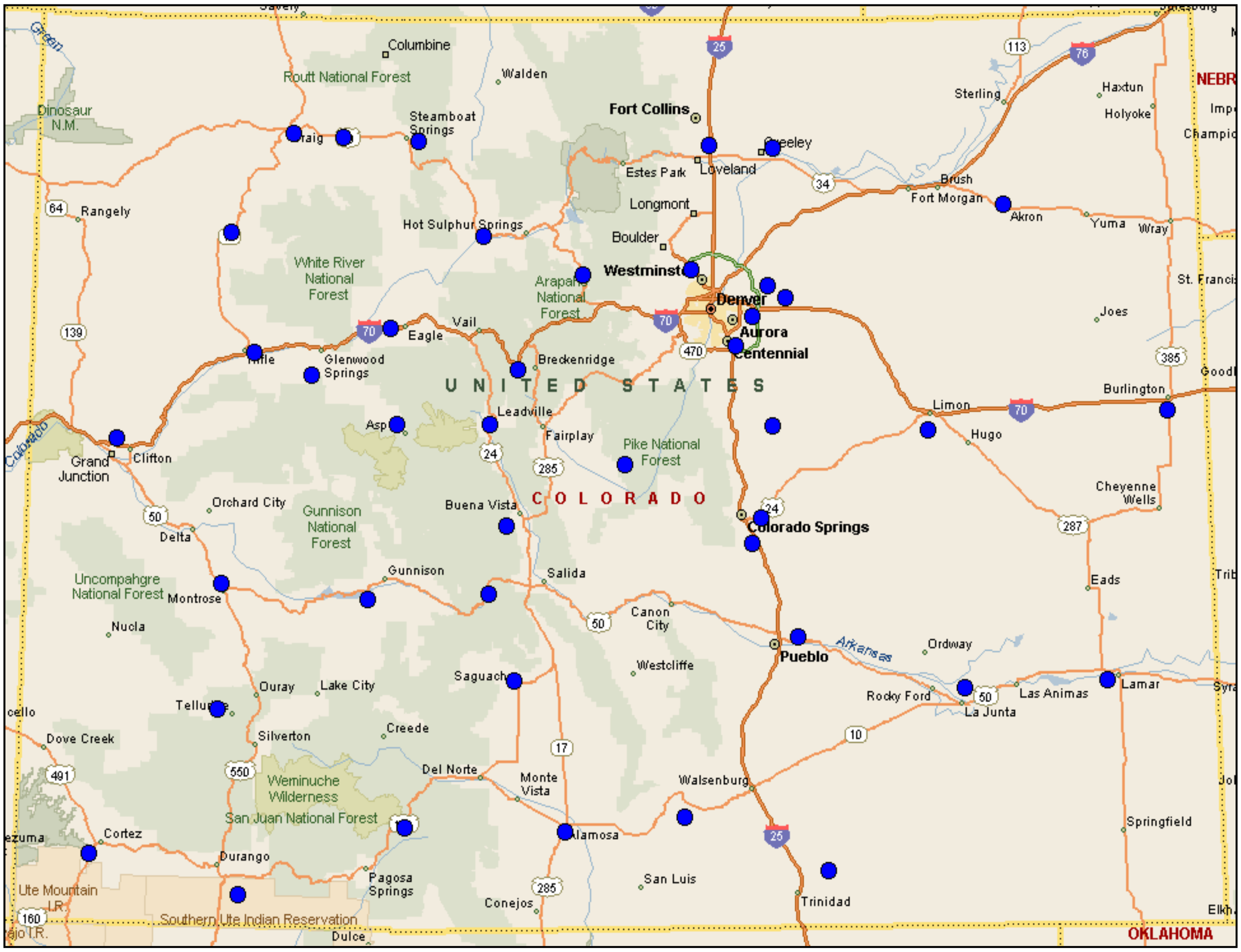


Figure 3.25 Location of Colorado Weather Stations

Table 3.15 Geographic Coordinates and Data Availability of Colorado Weather Stations

Station Number	Station	Latitude	Longitude	Elevation	Start Date	End Date	Years of Data
24015	Akron/Washington County	40.172	-103.232	4621	6/1/1973	3/31/2010	36.9
23061	Alamosa Muni(AWOS)	37.436	-105.866	7540.9	1/1/1973	3/31/2010	37.3
93073	Aspen Pitkin County SAR	39.223	-106.868	7742	1/1/1973	3/31/2010	37.3
23036	Aurora (Buckley AFB)	39.702	-104.752	5662	1/1/2000	3/31/2010	10.3
03065	Broomfield/Jefferson County	39.909	-105.117	5669.9	9/1/1984	3/31/2010	25.6
03026	Burlington	39.245	-102.284	4216.8	2/1/1999	3/31/2010	11.2
93067	Centennial	39.57	-104.849	5828	10/1/1983	3/31/2010	26.5
93037	Colorado Springs Municipal AP	38.812	-104.711	6169.9	1/1/1973	3/31/2010	37.3
03038	Copper Mountain Resort	39.467	-106.15	12074	8/1/2004	3/31/2010	5.7
93069	Cortez/Montezuma County	37.303	-108.628	5914	1/1/1973	3/31/2010	37.3
12341	Cottonwood Pass	38.783	-106.217	9826	7/1/2005	3/31/2010	4.8
24046	Craig-Moffat	40.495	-107.521	6192.8	9/1/1996	3/31/2010	13.6
03017	Denver (DIA) 03017	39.833	-104.658	5431	1/1/1995	3/31/2010	15.3
12342	Denver Nexrad 12342	39.783	-104.55	5606.9	5/1/2006	3/31/2010	3.9
93005	Durango/La Plata Airport	37.143	-107.76	6685	1/1/1973	3/31/2010	37.3
23063	Eagle County Airport	39.643	-106.918	6535	1/1/1973	3/31/2010	37.3
03040	Elbert County Airport	39.217	-104.633	7060	6/1/2004	3/31/2010	5.8
94015	Fort Carson/Butts	38.7	-104.767	5869.4	1/1/1969	3/31/2010	41.3
94062	Fort Collins Airport	40.452	-105.001	5016	5/1/1986	3/31/2010	23.9
12344	Glenwood Springs	39.433	-107.383	10603.5	7/1/2005	3/31/2010	4.7
23066	Grand Junction Airport	39.134	-108.538	4838.8	1/1/1973	3/31/2010	37.3
24051	Greeley/Weld County Airport	40.436	-104.618	4648.9	8/1/1991	3/31/2010	18.7
93007	Gunnison County Airport	38.452	-107.034	7673.8	4/1/1976	3/31/2010	34.0
94025	Hayden/Yampa (AWOS)	40.481	-107.217	6602	1/1/1973	5/31/2010	37.4
94076	Kremmling Airport	40.054	-106.368	7411	6/1/2004	3/31/2010	5.8
23067	La Junta Muni Airport	38.051	-103.527	4214.8	1/1/1961	3/31/2010	49.3
03042	La Veta Pass	37.5	-105.167	10216.7	7/1/2004	3/31/2010	5.8
03013	Lamar Muni Airport	38.07	-102.688	3070	1/1/1980	3/31/2010	30.3
93009	Leadville/Lake County Airport	39.228	-106.316	9926.7	7/1/1987	3/31/2010	22.8
93010	Limon Muni Airport	39.189	-103.716	5365.1	1/1/2004	3/31/2010	6.2
94050	Meeker	40.049	-107.885	6390	12/1/1978	3/31/2010	31.4
93013	Montrose Regional Airport	38.505	-107.898	5758.8	1/1/1973	3/31/2010	37.3
03039	Pagosa Springs	37.45	-106.8	11790.9	6/1/2004	3/31/2010	5.8
93058	Pueblo Airport	38.29	-104.498	4720.1	6/1/1954	3/31/2010	55.9
03016	Rifle/Garfield Airport	39.526	-107.726	5543.9	7/1/1987	3/31/2010	22.8
03069	Saguache Muni Airport	38.097	-106.169	7826	10/1/2004	3/31/2010	5.5
03041	Salida/Monarch Pass	38.483	-106.317	12030.7	6/1/2004	3/31/2010	5.8
12343	Steamboat Springs	40.467	-106.767	10633.1	4/1/2005	5/31/2010	5.2
03011	Telluride Regional Airport	37.954	-107.901	9078	12/1/2000	3/31/2010	9.3
23070	Trinidad/Animas County AP	37.259	-104.341	5743	1/1/1973	3/31/2010	37.3

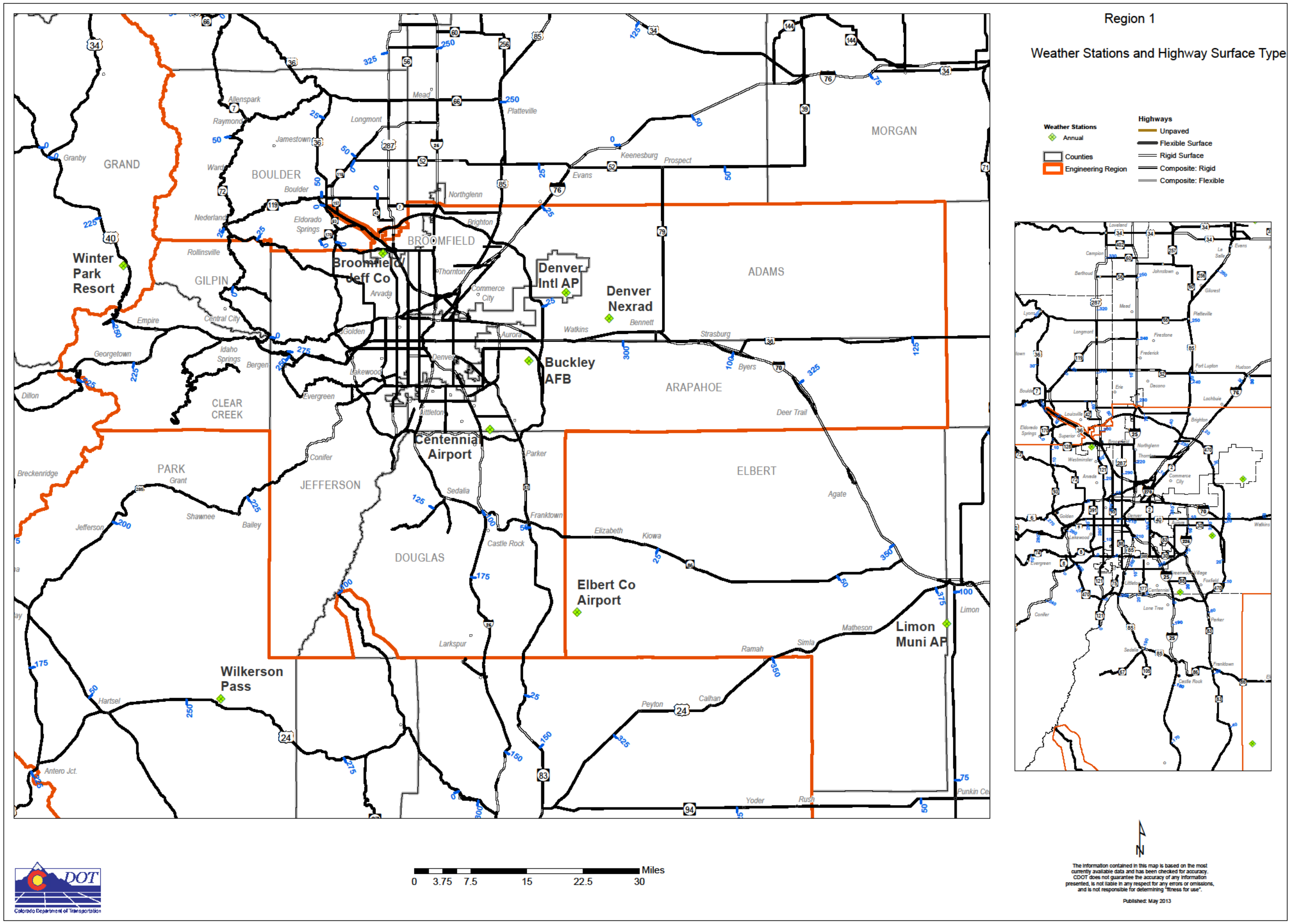


Figure 3.26 Region 1 Weather Stations and Highway Surface Type Map

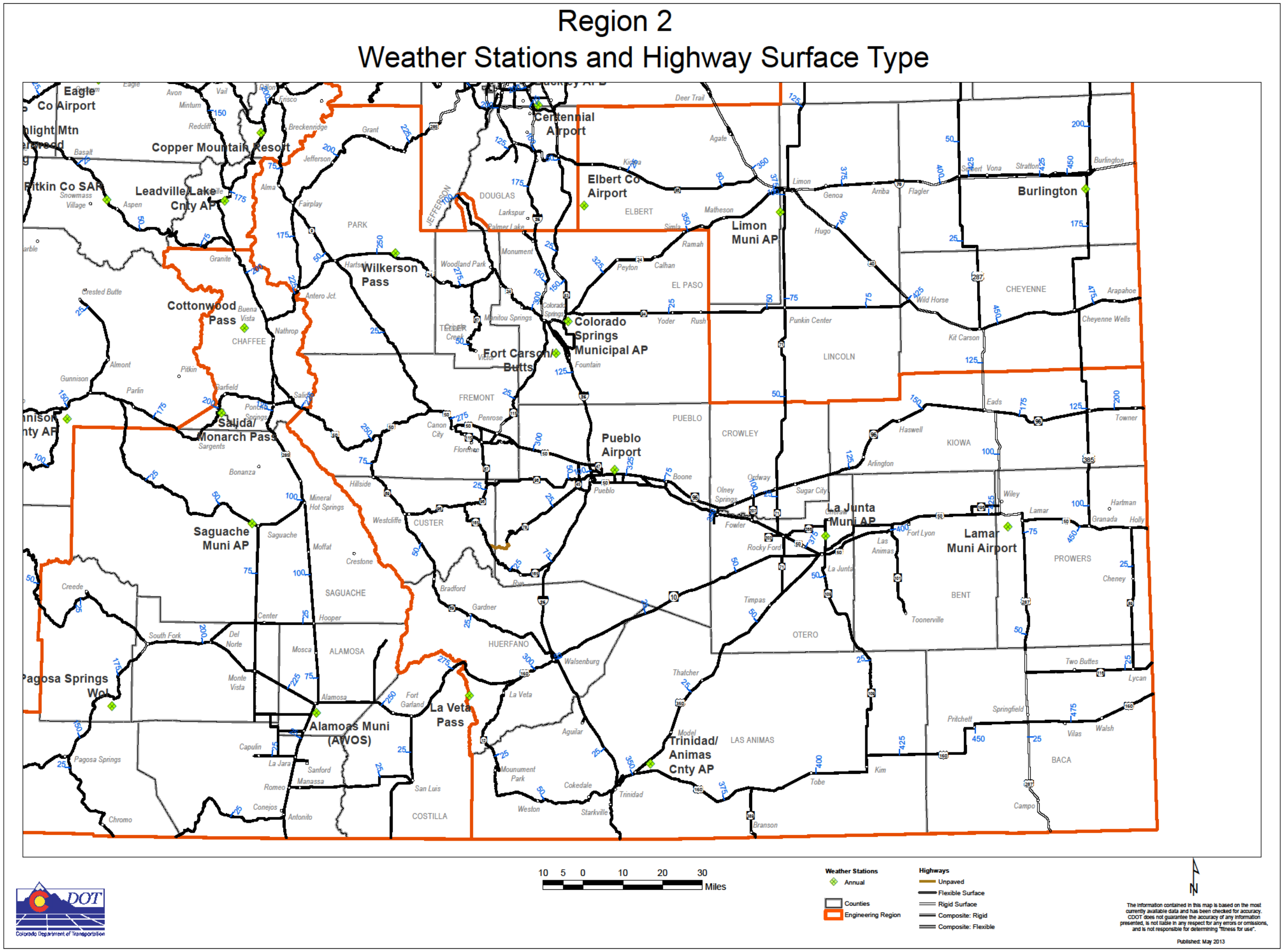


Figure 3.27 Region 2 Weather Stations and Highway Surface Type Map

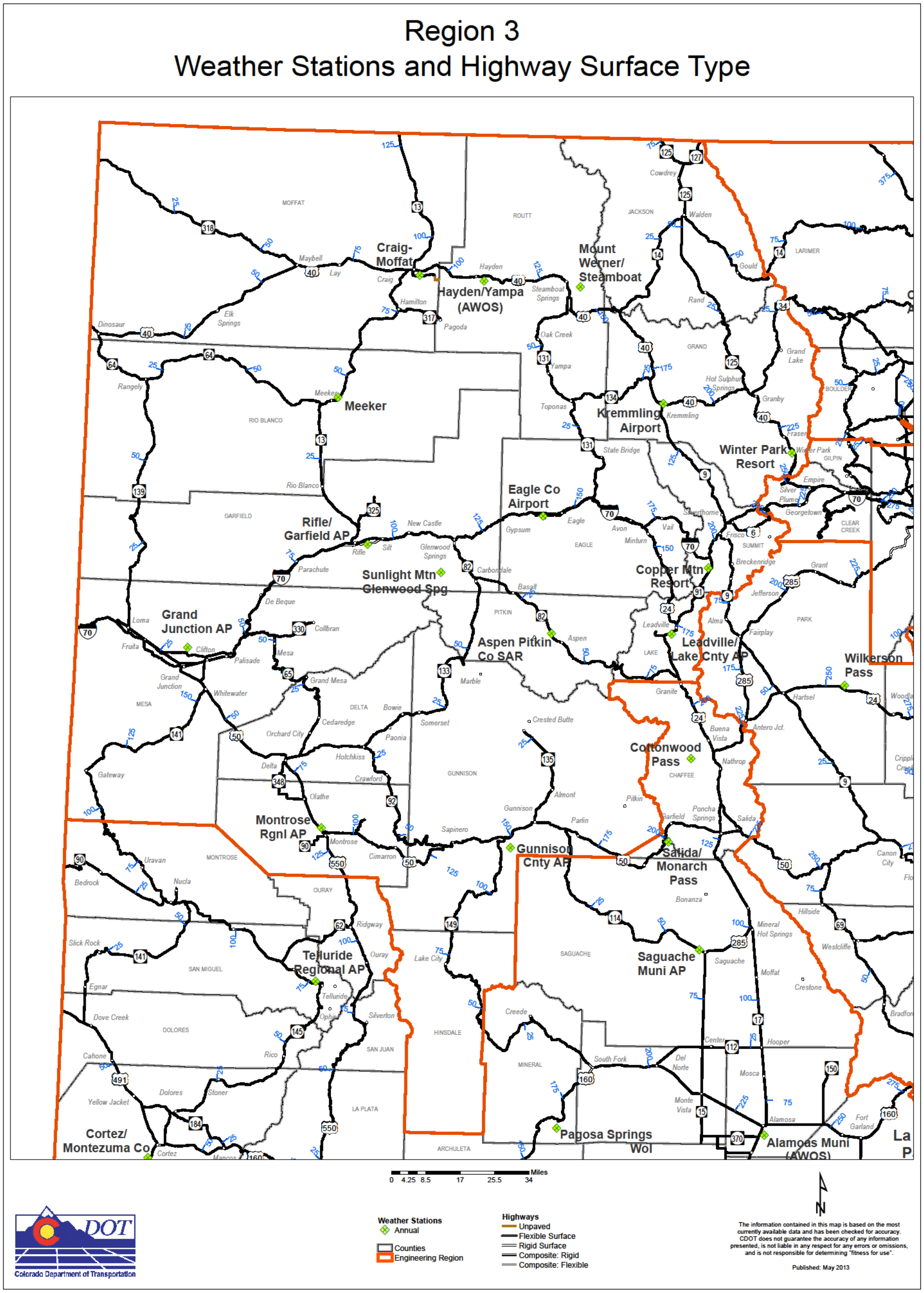


Figure 3.28 Region 3 Weather Stations and Highway Surface Type Map

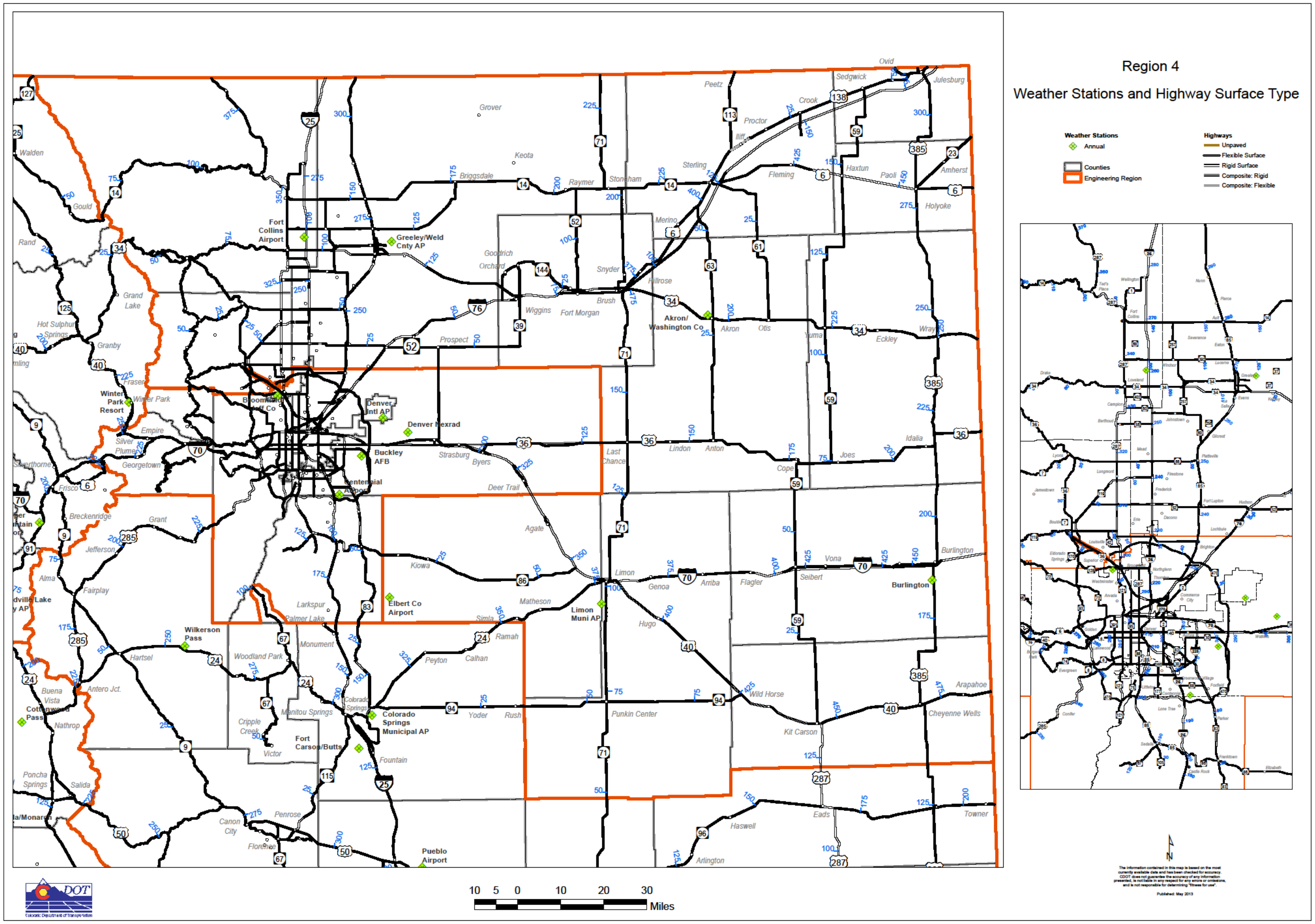


Figure 3.29 Region 4 Weather Stations and Highway Surface Type Map

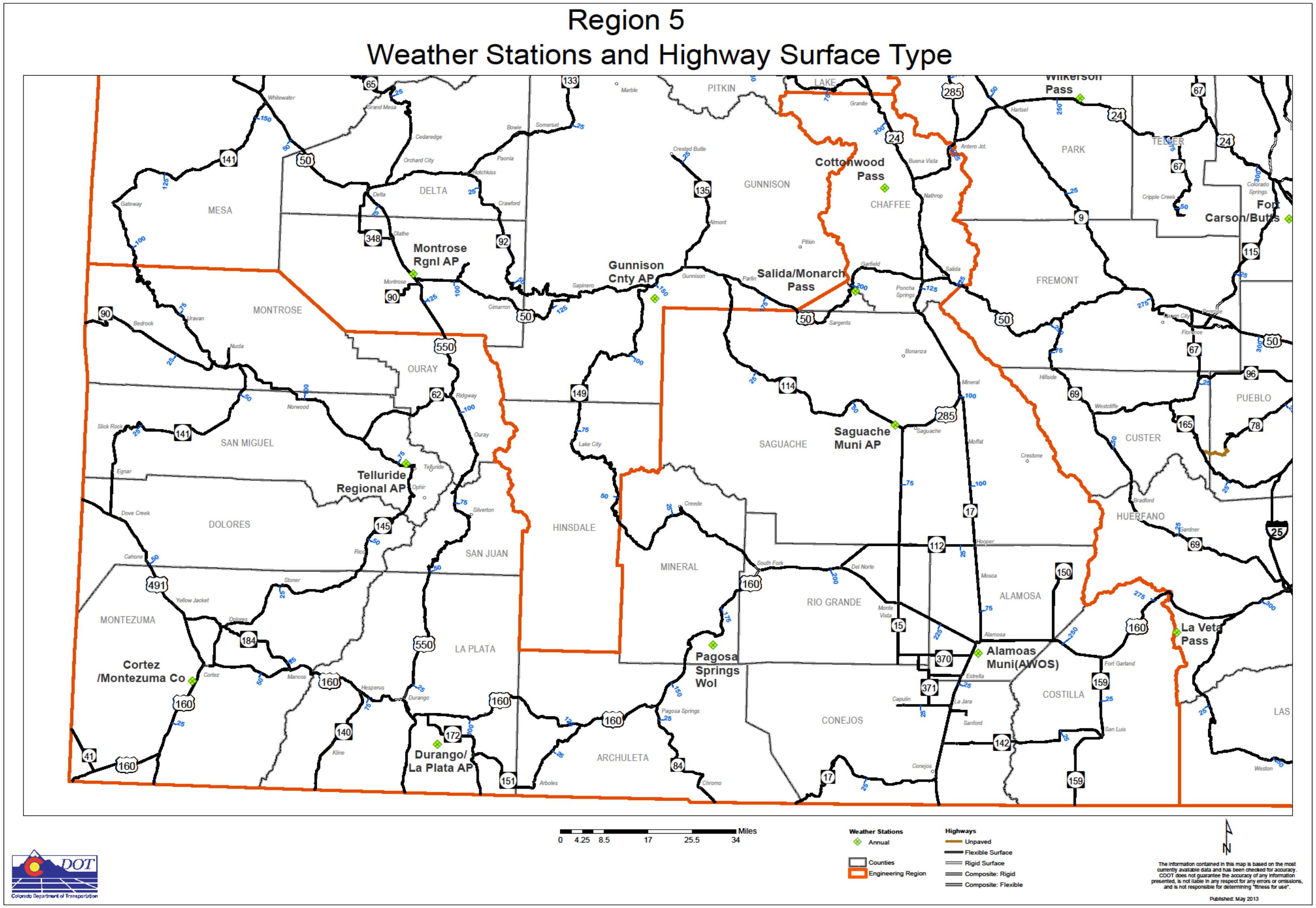


Figure 3.30 Region 5 Weather Stations and Highway Surface Type Map

References

1. *Heavy Vehicle Travel Information System*, Field Manual, FHWA publication PDF version, May 2001 (revised), obtained at website, http://www.fhwa.dot.gov/ohim/tvtw/hvtis.htm

2. *Highway Capacity Manual*, Transportation Research Board, National Research Council, Washington, D.C., 2000.

3. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

CHAPTER 4
SUBGRADE

4.1 Introduction

Subgrade is the top surface of a roadbed upon which the pavement structure and shoulders are constructed. The subgrade can be further subdivided and described as imported soil or a man-made compacted layer of the same soil as beneath it (natural subgrade). This chapter provides procedures and recommended guidelines for determining the design parameters of the subgrade soils or foundation for use in new and rehabilitated pavement designs. The subgrade support is a key fundamental input in pavement design as the selection of overlying layer types, thicknesses, and properties. Regardless of the pavement type, the subgrade is characterized in a similar manner. The M-E Design procedure categorizes major subgrade types as shown in **Table 4.1 M-E Design Major Subgrade Categories**.

Table 4.1 M-E Design Major Subgrade Categories

Material Category	Sub-Category
Rigid Foundation	Solid, Massive and Continuous Highly Fractured, Weathered
Subgrade Soils	Gravelly Soils (A-1; A-2) Sandy Soils • Loose Sands (A-3) • Dense Sands (A-3) • Silty Sands (A-2-4; A-2-5) • Clayey Sands (A-2-6; A-2-7) Silty Soils (A-4; A-5) Clayey Soils • Low Plasticity Clays (A-6) ♦ Dry-Hard ♦ Moist Stiff ♦ Wet/Saturated-Soft • High Plasticity Clays (A-7) ♦ Dry-Hard ♦ Moist Stiff ♦ Wet/Saturated-Soft

4.2 Soil Survey Investigation

The M-E Design process begins with a preliminary soil survey. Geotechnical investigations are typically required for new construction and reconstruction projects. Contact the Regional Materials Engineer or CDOT Materials and Geotechnical Branch to request a geotechnical investigation, and refer to Chapter 200 of the Field Material Manual for further information.

4.2.1 Soil Surveys and Preliminary Soil Profile

Procedure Overview

This set of guidelines generally follows the current practices CDOT personnel use for obtaining soil profiles. It is intended to establish standardized procedures for use by Region Materials personnel in the performance of uniform and adequate soils investigations. Soil surveys are conducted prior to new alignments and most road widening projects as part of the pavement design process. The purpose of soil surveys are to locate the various soil types within the existing and proposed roadway at elevations above and below the profile grade. The extent of each soil type is noted and each type is identified by the AASHTO classification method. The condition of sub-soils upon which embankments will be constructed is evaluated to avoid problems such as:

- Pavement design
- Slope design
- Slope appearance
- Cost
- Landslides
- Embankment subsidence and settlement
- Excavation characteristics
- Expansive materials
- Drainage
- Compaction characteristics

All of these problems are directly related to:

- The character and distribution of soil and rock bodies, both inside and outside of the right-of-way.
- The influence of surface and sub-surface water on these materials.

With the proper amount and type of samples and field information, the designers are provided with data denoting the types of materials to be encountered, the vertical and horizontal boundaries of the changes in these materials, and their strength and deformation characteristics. Adequate preliminary investigation will help prevent uneconomical over-design and unforeseen failure resulting from under-design.

Proper investigations to achieve these goals cannot be dictated by a rigidly prescribed set of procedures, although certain basic requirements must be satisfied in each investigation. Both the detail and extent of the investigation will vary depending on the individual problem, the nature of the project under consideration, and the allowable risk of failure.

Investigations may sometimes need to go beyond the minimum soil profile recommendation presented within this document. Projects in special problem areas or in areas of rough terrain are the most likely to require more extensive investigations. Such studies are especially recommended for high-speed, multi-lane facilities in rough terrain. The Soils & Geotechnical Program (S&GP) of the Central Laboratory or outside consultants will conduct these studies.

4.2.2 Soil Survey Classification

Soil surveys may be classified as reconnaissance or preliminary, depending upon the type of information developed and the stage of project development during which each is performed.

4.2.2.1 Reconnaissance Soil Surveys

Reconnaissance surveys are general in nature and are performed during Phase II (Corridor Location study) of project development under the CDOT Action Plan. The survey, including sample locations and methods, may be performed either by Region Materials Personnel, Soils & Geotechnical Program (S&GP), or outside consultants as determined by mutual agreement. The information developed during these surveys is used in preparation of Environmental Impact Statements for proposed projects. These surveys are performed only if the necessary information cannot be obtained from existing data, such as soil maps, test reports from previous projects in the area, etc. Information required from reconnaissance surveys:

a) AASHTO classification of all major soil types present in the corridor.
b) Identification of landforms or geologic formations with which each is associated.
c) Description of specific engineering problems associated with each.

This information will be included in the soils and geology reconnaissance report prepared for each project and should be developed through joint effort of Region Materials Personnel and the Engineering Geologist or Geotechnical Engineer assigned to the project. The field survey, if required, will consist only of identifying the major soils present and obtaining representative bulk samples of each. Usually, no line will have been established at this point in the project development and sample locations may be selected without regard for line and grade. Samples may be taken by the most convenient method available that insures the samples are representative of the major soil types and large enough to permit accurate laboratory classification. Sampling methods are discussed in more detail in **Section 4.2.3.4 Sampling Methods.**

4.2.2.2 Preliminary Soil Surveys

Preliminary soil surveys are performed during Phase III (Preliminary Design) of project development under the CDOT Action Plan. The information developed during these surveys is used in project design and preparation of cost estimates and must therefore be as accurate as possible. These surveys are performed on all new alignments and most widening projects. Two checklists; **Region Preliminary Soil Survey Sampling Checklist and Region Soil Survey Drilling Checklist** are provided in **Section 4.2.14.1 Field Observations and Sampling.** This is not meant to be all inclusive, rather a reference guide that may be used.

One of the most important items to be determined during the survey is the relationship between soil boundaries and the line and grade of the proposed project. If soil survey personnel do not know the location of line and grade at the time of the investigation, they cannot be certain the soil conditions encountered in the borings represent conditions to be encountered during construction. In particular, they cannot be sure the soil conditions have been sampled to proper depth below finished grade if they do not know where finished grade will be located.

It is important to identify the presence of sulfates in soils at project locations. This can be determined by visiting the following website: http://websoilsurvey.nrcs.usda.gov/app/

This website can provide soil engineering properties as well as approximate location, depth, and concentrations of sulfates. If the presence of sulfates on project locations is suspected, the preliminary soils survey needs to sample and test the soil layers. During the preliminary soil survey, one sample, per soil type, will be tested approximately 1 per 1,000 linear feet of two-lane roadway or fraction thereof. Frequency may be reduced or increased per the RME's recommendation. The boring depth for the preliminary soils survey will be a minimum of 3 feet below the proposed finished grade. The sample size will be a minimum of 5 lbs. per soil type. Where water is present at drainages, a minimum of 1 pint sample will be taken of the water. CP-L 2103 will be used in the testing of sulfates in water or soil and can be performed in the field or by the Region Lab if adequate facilities and equipment are available.

4.2.3 Soil Surveys

Soil and rock materials encountered in borings or surface outcrops should be identified and described per Sections 12 and 13 of this guidance. Accurate descriptions of soil or rock encountered in the field are important to the economic planning of the project design. Avoid complicated descriptions (not relevant to design or construction problems). Consultation with the Regional Materials Engineer and Project Team in order to collaboratively determine actual sample locations, frequencies, and depths is highly recommended prior to sampling and testing.

Approved traffic control shall be use as required based on the boring locations specified for the project. Borings can be drilled or dug by hand, power auger, power rotary drill, backhoe, or any other practical method. In any case, it is of the utmost importance to use the method that will insure the attainment of representative, uncontaminated samples whether bulk samples, undisturbed samples, core samples, drill cutting samples, or split-spoon samples. Care should be taken to make sure that loose, sloughed soil or rock in the bottom of the borings is not mixed in with samples representing the given depth. Where uncertainty exists as to the reliability of a sample, the sample shall be discarded and a new sample shall be collected. In the following paragraphs, the term "drilled" is used to mean any appropriate method for advancing a boring.

4.2.3.1 Horizontal Distribution of Borings

Borings will be spaced no farther apart than 500 feet in continuous cut and fill sections, and no farther apart than one mile under any circumstance. In addition, borings should be drilled wherever there is any variation in soil or geological conditions, base gravels, and/or pavement thicknesses. Time should be taken to obtain a sufficient number of borings to outline current pavement conditions and sub-surface complexities. During the design phase of the project, if it is determined that additional data or samples are needed, such will be obtained and a supplemental report submitted.

Since there is, at times, considerable lag between the time of the preliminary soil profile and actual construction, borings drilled through the existing pavement should be held to a minimum as directed by the Regional Materials Engineer. Such borings present maintenance problems, and excessive drilling in the traffic flow presents needless hazards.

When taking soil surveys on proposed widening jobs, attention should be given to areas where CMP, RCP, or box culverts may be extended, replaced, or added. Quite often these areas will require excavation of unsuitable materials such as organic rich material or unsuitable material.

Such requirement for excavation should be reported with respect to stationing, distance from survey line, and approximate depth. If it is not practical to drill borings in the unsuitable material, it may be possible to get a rough estimate of depth by probing with a bar or rod.

4.2.3.2 Proposed Ne Line and/or Grade

For cut sections or where differing soil conditions are anticipated, borings should be spaced as shown in **Figure 4.1 Recommended Location of Borings in the Cut Section**. At locations 1 and 3, borings should be drilled on proposed outside shoulder line (edge of pavement) at the daylight line between cut and fill. An additional boring should be drilled at location 2 (highest elevation of terrain on center line).

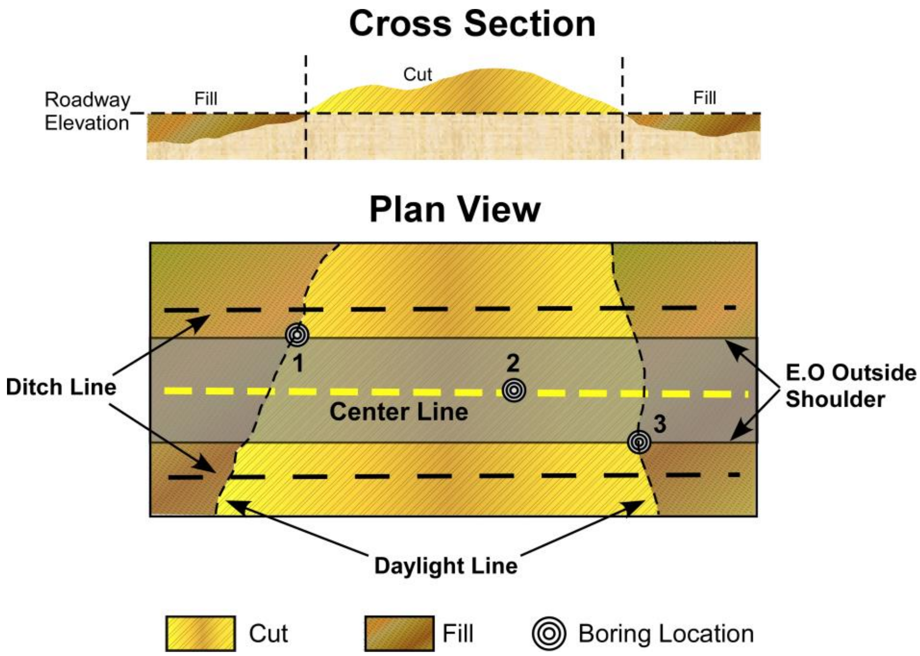


Figure 4.1 Recommended Location of Borings in the Cut Section

For fills/embankments, boring(s) should be drilled on centerline as shown in **Figure 4.2 Recommended Location of Borings in a Fill Section**. One of the borings shall be located at the point of the thickest proposed fill. Additional borings may be required for global stability analysis.

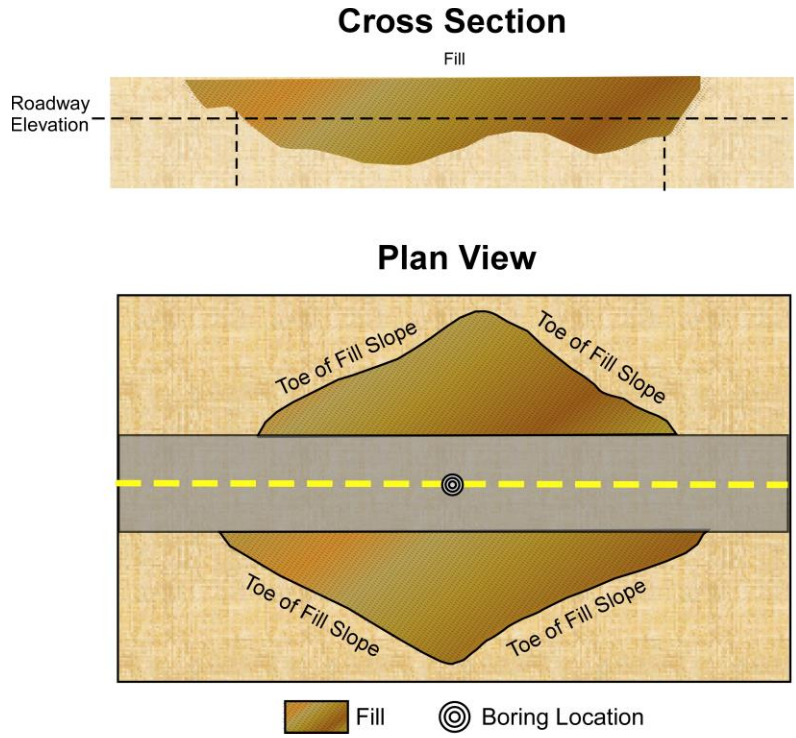


Figure 4.2 Recommended Location of Borings in a Fill Section

4.2.3.3 Boring Depth

For existing grades or cuts, borings shall extend at least 10 feet below finished grade. For new grades requiring embankments greater than 5 feet, borings shall extend at least 2 times the total height of the proposed fill below the base elevation or 5 feet into hard substratum (N>30). If that depth is greater than the depth capability of the equipment available to Region personnel, the S&GP or commercial drilling contractors will be requested to provide drilling services. Such services would be performed under supervision of Region personnel, assisted by S&GP if desired.

For proposed cut sections the depths of borings and sampling requirements should be as shown in **Figure 4.3 Recommended Depth of Borings in Cut Sections**. As per boring location 2, **Figure 4.3 Recommended Depth of Borings in Cut Sections**, soil and/or rock layers A, B, C, and D should be separately sampled or similarized.

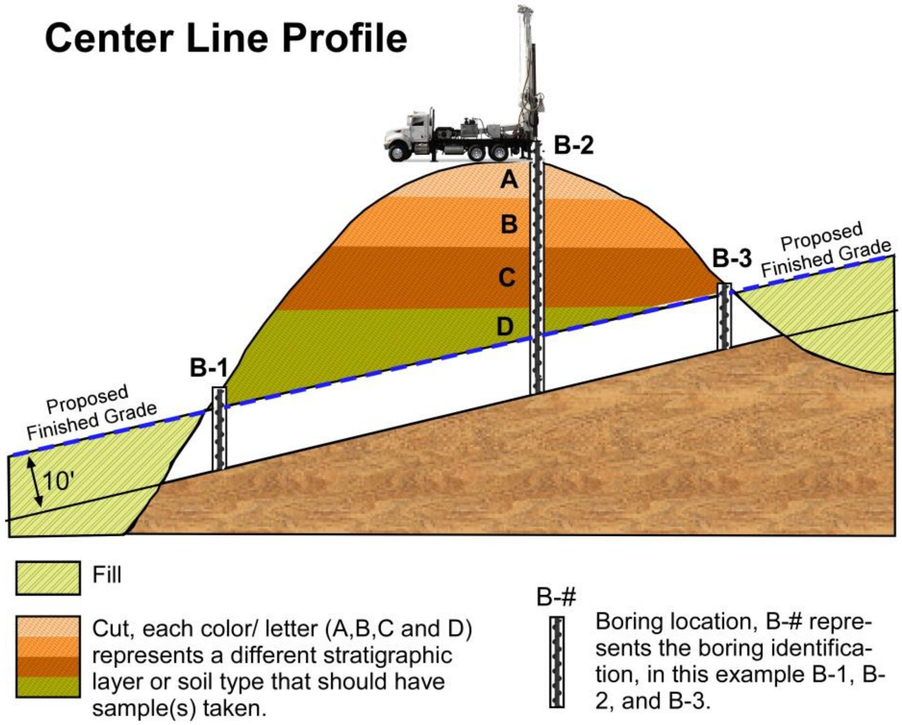


Figure 4.3 Recommended Depth of Borings in Cut Sections

For fills whose proposed maximum height is more than 5 feet, the depths of borings and the sampling recommendations should be as shown in **Figure 4.4 Recommended Depths of Borings in Fill**. Unless the bedrock or firm substratum is too hard for the drilling method being employed, all borings (such as Location #1, **Figure 4.4 Recommended Depths of Borings in Fill)** should penetrate at least 5 feet into the hard substratum. However, in such cases the desirability of drilling to hard bedrock should be considered in at least one boring.

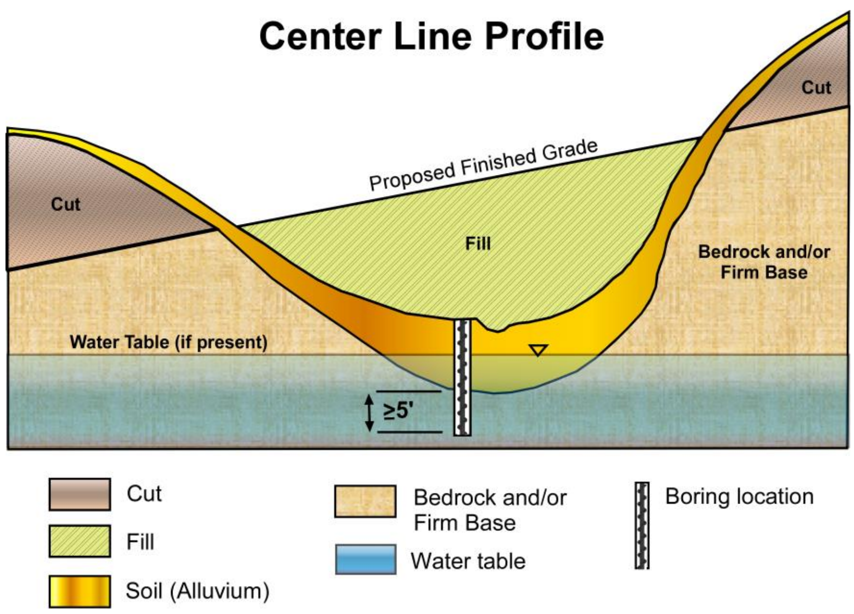


Figure 4.4 Recommended Depths of Borings in Fill

Where alluvial soils as shown in **Figure 4.4 Recommended Depths of Borings in Fill** are composed of soft, compressible, fine grained materials, samples should be collected for consolidation testing. For at-grade sections all borings shall extend at least 10 feet below existing ground. All soils shall be sampled in bulk or similarized.

4.2.3.4 Sampling Methods

A sample should be taken for each soil encountered except for the material which might be used as topsoil. If topsoil is going to be required on the project, the lateral extent and depth of material, which could be utilized for topsoil, should be noted on CDOT Form #554. If the same soil is found in more than one boring, it may be similarized to a soil already sampled. Similarization is the process of combining or eliminating samples from nearby locations that exhibit similar physical properties such as color, grain size, gradation, plasticity, roundness, etc. This increases productivity and efficiency while reducing cost for sample shipment and laboratory analysis. Care should be exercised in similarizing soils and additional samples should be taken where doubt exists. Similarization will be limited to one mile. Soil samples taken in each boring will be visually classified and similarized in the Region by certified inspectors and testers prior to submittal for laboratory analysis.

Borings should be numbered consecutively from Boring #1, starting from project station 0+00. Mile posts can be substituted for stationing if project stationing has not been developed at the time of the soil survey. Each soil layer encountered in the boring shall be identified by the boring number followed by letter A, B, C, etc. In Boring #1,the first layer would be 1 A, the second 1 B, etc. Each layer shall be sampled in bulk or similarized. A bulk sample should be composed of at least one full sack and weigh at least 33 lbs. Pavement cores shall be collected and photograph (2MB file size) of the asphalt or concrete pavement. Core sample diameters will be 4 inch minimum for HMA and the size necessary to drill the boring for PCCP. Alternate sampling methods may be requested by the Regional Materials Engineer or the Designers. These may include, but are not limited to:

- Standard Penetration Tests per ASTM D1586 - Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.
- Colorado Procedure – Laboratory 3201-10, Standard Method of Test for Continuous Penetration Test.
- Geophysical Survey per FHWA-HRT-05-028, Geophysical Methods for Transportation Applications.

4.2.4 Hydraulic Conditions

The distribution and mode of occurrence of surface and sub-surface water should be noted and included as part of all reports. Where free water is encountered in any boring, the water level is to be checked and noted on CDOT Form #555 along with the date and hour of the observation. In cases where a high water table is suspected, it is recommended that the boring be drilled or dug at least to two feet below the elevation of the water table. Where possible, the boring is to be left open for a period of at least 24 hours and the water level, date, and hour recorded.

The location of all springs should be determined both horizontally and vertically with respect to centerline and grade line. The location of lakes, ponds, swampy areas, and reservoirs should be noted. Notes should especially be taken if the water is expected to influence the stability of pavements, cut slopes, or embankments. The normal annual precipitation at the project site should be determined from the most recent isohyetal map.

4.2.5 Piping

Piping is the mechanical movement of particles due to seepage. Areas requiring culverts, foundations, and ditch linings should be investigated to determine whether the soil is subject to piping. Piping often occurs in silts, fine sands, and loosely compacted material. Concentration of seepage into a few channels may cause piping. If the preliminary investigation indicates conditions and soils that could cause piping, the Staff Hydraulics Unit should be requested to make a thorough investigation.

4.2.6 Condition of Existing Pavements

The condition of existing concrete or asphalt pavements should be taken into account for stabilization and may be noted on a station-to-station basis on CDOT Form #903. This information is used for assignment of strength coefficients. Report the type and thickness of existing pavement and any previous pavement stabilization.

4.2.7 Frost

In areas of severe frost action, the soil should be checked for frost susceptibility. If necessary, recommendation should be made for the removal and replacement of frost susceptible soil with non-frost heaving material. Non-frost heaving material should be replaced to a depth of ⅓ to ½ the estimated frost penetration. The ground water table (perched tables or aquifers included) should be checked on all projects and in areas of severe frost action. The bottom of ditch linings should be kept at least three feet above the water table (unless the foundation materials are free draining sands or gravels).

4.2.8 Adjacent Terrain

This information is used primarily by the CDOT Staff Hydraulics Unit in determining rainfall runoff factors in the design of drainage structures. Rather than noting conditions on a station-to-station basis, a general statement relative to the project as a boring should be made. If there are distinct breaks over the length of the project, each type of terrain should be noted. Such designations as rolling grassland, steep timbered slopes, paved commercial, etc. are appropriate.

4.2.9 Excavation Characteristics

During the investigation, notes should be kept concerning the estimated excavation characteristics of all soil or rock materials encountered. Materials should be classified as:

- Common excavation,
- Ripping required, or
- Pre-blasting required.

It is often necessary to construct shallow embankments from cuts or borrow pits containing boulders too large to be buried in the fills. The disposal of such boulders can be a problem on each project where this condition occurs. If such oversized material is encountered during the investigation, it should be noted on CDOT Form #555 so the Project Manager can include a NOTE in the plans that this material will usually become the property of the Prime Contractor, and is required that they dispose of material per local laws and applicable State regulations.

4.2.10 Embankment Foundations

The construction of highways over weak, compressible soils presents some of the more difficult problems in soil mechanics. If embankments are constructed over foundation soils having insufficient strength to support the added load, shear failure or slip-outs may occur, or the underlying soft material may displace by outward plastic flow.

If the foundation soil is highly compressible, excessive settlement of the embankment may occur, resulting in damage or destruction of the pavement, damage to structures, or hazards to traffic due to distortion of the profile and cross section of the roadbed. Such settlement may occur even if the strength of the foundation is high enough to preclude shear failure.

For the above reasons, it is recommended that Region personnel request a foundation investigation be performed by the S&GP where embankments more than 5 feet in height will be constructed on soft foundation soils.

4.2.11 Swelling Soils

Swelling soils are common in Colorado and are frequently encountered during highway construction. To minimize damage to roadways from swelling action, it is necessary that these soils be recognized when encountered in the field and the soil boundaries along the project be determined during the preliminary soil survey.

A detailed map showing boundaries of swelling soil areas classified by the amount of swell potential has been published by the Colorado Land Use Commission and has been distributed to all CDOT Regions. This map should be consulted prior to commencing any soils survey, whether reconnaissance or preliminary.

It is sometimes difficult to identify swelling soils visually, but the following criteria are often helpful:

- **Texture** - When dry, the natural surface exposures of swelling soils usually exhibit an irregular or pebbly texture resembling popcorn.
- **Plasticity** - All swelling soils are plastic and most are highly plastic. The presence of plasticity can be determined in the field by moistening a sample and attempting to roll a thread in the palm of the hand.
- **Bentonite Clay** - A common clay causing swell in soils is bentonite, which usually occurs in shales, either as fine particles invisible to the naked eye or as thin, light colored bands which contrast with the darker color of the shale and are oriented parallel to the bedding. The bands range in color from light tan to light greenish gray and may range in thickness from a fraction of an inch to as much as two or three inches. Pieces of this material will adhere to a moistened finger and will break down in a matter of minutes if dropped into water.

If any of these characteristics are noted during the soil survey (particularly in those areas indicated on the map) or if the possibility of swell is suspected for any other reason, notation to this effect should be made on CDOT Form #554.

Even though a soil contains expansive clays, it may not swell if the in-place moisture is high enough. It is therefore important to know the actual moisture content of the soil in order to assess the possibility of problems due to swell. For this reason, if swelling soils are identified or suspected during the soil survey, moisture samples should be taken at or slightly below the elevation of the proposed grade line in those areas where the soils are present.

Problems due to expansive soils usually occur in cut areas and in transitions from cut to fill areas. They could also occur in fill areas where moderate to high swelling soils are used for fill. These soils are usually identified by:

- Liquid limit
- Plasticity index
- Expansion pressure
- Swell-consolidation

The liquid limit and plasticity index usually correlate with swell potential in the laboratory. However, they may not be related to the swell potential in the field because of moisture content, density, and chemicals in the in-situ soil.

Many potential high swelling soils in areas of high ground water have taken on enough moisture that additional swelling is unlikely to occur. But certain dry, dense and often un-weathered soils must be treated to lesson swell potential. If a treatment is determined to be necessary, then the type of treatment shall be determined by the Region Materials Engineer or it may be advisable to request additional analysis by the CDOT Soils & Geotechnical Program.

4.2.12 Soil Identification and Description

For engineering purposes, soil is defined as any naturally occurring unconsolidated material composed of mineral grains with gases or liquids occupying the inter-granular spaces. A complete soil identification for engineering purposes shall follow *ASTM D2488 – Standard Practice for Description of Soils* and include:

Angularity - Describe the angularity of the sand (coarse sizes only), gravel, cobbles, and boulders, as angular, subangular, subrounded, or rounded. A range of angularity may be stated, such as: subrounded to rounded.

Shape - Describe the shape of the gravel, cobbles, and boulders as flat, elongated, or flat and elongated if applicable; otherwise, do not mention the shape. Indicate the fraction of the particles that have the shape.

Color - Describe the color. Color is an important property in identifying organic soils, and within a given locality it may also be useful in identifying materials of similar geologic origin. If the sample contains layers or patches of varying colors, this shall be noted and all representative colors shall be described. The color shall be described for moist samples. If the color represents a dry condition, this shall be stated in the description.

Odor - Describe the odor if organic or unusual. Soils containing a significant amount of organic material usually have a distinctive odor of decaying vegetation. This is especially apparent in fresh samples. If the samples are dried, the odor may often be revived by moistening the sample and slightly heating it. Odors from petroleum products, chemicals or other substances shall be described. Some fumes emitting from soil samples, especially of a chemical nature, may pose a health risk. Proper safety protocols which may include the use of personal protective equipment must be followed in these instances. It is the responsibility of the user to determine the extent of the health risk and the correct protocols to follow.

Moisture Condition - Describe the moisture condition as dry, moist, or wet.

HCl Reaction (if available) - Describe the reaction with HCl as none, weak, or strong. Since calcium carbonate is a common cementing agent, a comment of its presence on the basis of the reaction with dilute hydrochloric acid is important.

Consistency - For intact fine-grained soil, describe the consistency as very soft, soft, firm, hard, or very hard. This observation is inappropriate for soils with significant amounts of gravel.

Cementation - Describe the cementation of intact coarse-grained soils as weak, moderate, or strong.

Structure - Describe the structure of intact soils as stratified, laminated, fissured, slickensided, blocky, lensed, or homogeneous.

Range of Particle Sizes - For gravel and sand components, describe the range of particle sizes that are retained on the No. 4 sieve (gravel) and No. 200 sieve (sand). For example, about 20 % fine to coarse gravel, about 40 % fine to coarse sand.

Maximum Particle Size - Describe the maximum particle size found in the sample. For example, gravel up to 2 inches in diameter.

Additional Comments – Any additional information shall be noted, such as the presence of roots or root borings, difficulty in drilling or augering the boring, caving of the trench or boring, presence of mica, trash or other man made materials, etc.

4.2.13 Rock Identification and Description

For engineering purposes rock is defined as a naturally occurring mineralogical aggregate, which in an intact, unfractured sample will yield a laboratory unconfined compressive strength greater than or equal to 200 psi. A complete rock description for engineering purposes includes:

Classification - Reference is made to **Table 4.2 Rock Classification**. This is a relatively simple but practical system which can be used by the field person, whether geologist, engineer, or technician.

Color - As for soils (**See Section 4.2.12 Soil Identification and Description**)

Hardness and Degree of Cementation:

- Soft - Can be scratched with a fingernail.
- Moderately Hard - Can be scratched easily with a knife but cannot be scratched with a fingernail.
- Hard - Difficult to scratch with a knife.
- Very Hard **-** Cannot be scratched with a knife

Partings in the Rock - Including fractures, faults, and joints:

- Intact - No partings.
- Widely Fractured - Partings more than 10 feet apart.
- Closely Fractured - Partings less than 10 feet apart but more than 6 inches apart.
- Brecciated Partings - Less than 6 inches apart.

Moisture Content - Moisture content in rock cannot be determined by simple tests such as those used for soil, but should be estimated visually. As with soils, the terms dry, moist, and wet are adequate for field description.

4.2.14 Determination of Need for Culvert Protection

The best time to observe, sample, or report conditions indicating the need for corrosion protection of culverts is on the preliminary soil survey as shown on CDOT Form #554, see **Figure 2.5 CDOT Form #554**. However, completed soil surveys should be reviewed where it seems necessary. If additional samples are required, submit on a CDOT Form #157, see **Figure 2.8 CDOT Form #157**. The class of pipe required to resist abrasion and corrosion shall be determined using the *CDOT Pipe Material Selection Policy*.

4.2.14.1 Field Observations and Sampling

Past performance of culvert material is the best source of information. The local Maintenance Foreman can provide a history of culvert performance in the area. Observation of culverts on projects in adjacent areas of similar soil conditions will also provide useful information. Uncoated galvanized pipe, which shows no corrosion after at least two years of service, does not require soil or water sampling. However, a coated pipe, which shows no corrosion, may be in an environment that would attack an uncoated pipe. Samples of both the soil in contact with the pipe and the water going through the pipe would provide this information.

The condition of the interior of a culvert tells only part of the story. In most cases, the corrosive substances are in the soil in contact with the pipe, rather than in the water. Therefore, to truly appraise the amount of corrosive attack, it is necessary to expose and examine some of the pipe exterior. The presence of extensive rust spots would indicate a serious condition. A soil sample should be taken near any observered corrosion to determine if it is due to a high or low pH, or corrosive salts. The extent and location of the corrosion would be noted on CDOT Form #554, see **Figure 2.5 CDOT Form #554**.

Crystals, encrustations and alkali deposits in the streambed near the waterline, are signs of a possible corrosive water. Stains on the rocks are usually associated with minerals, therefore a tailing dump or mine drainage should be looked for upstream. If found, it should be noted on CDOT Form #554.

Table 4.2 Rock Classification

Sedimentary Rocks	Coarse-grained[1]	Conglomerate	Dominant grain size is boulders or gravel.
		Sandstone	Dominant grain size is sand.
	Fine-grained[2]	Shale	Thin-bedded. Dominant grain size is clay and silt.
		Limestone	Usually light-colored, composed of calcite and/or dolomite (will usually effervesce with dilute HCl).
Igneous and Metamorphic Rocks	Coarse-grained[1]	Gneiss	Composed of alternating bands of different colored minerals.
		Schist	Major component is mica layered structure.
		Marble	Coarse grained limestone.
		Granite	Granular, ranging in color from light to medium gray to salmon pink.
		Diorite	Contains approximately equal proportions of dark and light colored minerals.
		Gabbro	Granular dark gray to black.
	Fine-grained[2]	Rhyolite	Nearly white to light gray.
		Quartzite	Composed entirely of quartz.
		Andesite	Medium gray.
		Basalt	Dark gray to black (sometimes porous or vesicular).

Notes: [1] Individual crystals or fragments, which compose the rock, *can* be seen with the unaided eye.
[2] Individual crystals or fragments, which compose the rock, *cannot* be seen with the unaided eye.

Water that seeps out of the ground or from some layer in an embankment will probably have variations in the amount of dissolved salts from season to season, depending on the volume of water moving through the soil and the amount and availability of soluble mineral matter. It may be necessary to sample water seeps in spring, summer, and fall verify the water's chemistry.

Alkali deposits on the soil (such as from Mancos and Pierre Shales) and fine silty soils should be tested.

The Central Laboratory recommends all suspected soils and water be sampled. The accompanying CDOT Form #554 or #157 should mention the conditions that prompted the sampling, and the exact location in reference to the proposed or existing culvert.

Soil and water samples will be run in the laboratory to determine pH, hardness, alkali content, etc. Recommendations from the laboratory concerning required protective action may be based on evaluation of one or several of these test results and their interactions.

Unusual stains, encrustations of salt, alkali, or unpleasant odors should be mentioned on CDOT Form #554 or #157, as these are indicative of conditions which may cause culvert corrosion. The possible existence of an abrasive condition should also be noted. A serious problem should be discussed with the Hydraulics Unit for a possible solution.

A water sample shall be a minimum of 1 pint in volume. The water sample shall be collected and stored in a clean, unreactive and leak proof container. The soil sample should weigh at least a pound and be sent in a plastic bag.

On the basis of field observations and laboratory tests (where deemed necessary) the Region shall recommend to the Staff Design Engineer the types of culvert to be used and their location.

Region Preliminary Soil Survey Sampling Checklist – 2018

This checklist is provided as support for field personnel in conducting the soil survey. It is not intended to be a guidance document nor is it a sign of work fulfillment if completed. Communication with the Region Materials Engineer is required to insure all required sampling is conducted to meet the project specifications.

Sampling of Boring Materials

1. Take one sample per soil type containing at least 33 lbs. (15 kg) of minus No. 4 screen materials for ***Classification.***
2. A minimum of one boring per 1,000 linear feet of roadway will be done.
3. Minimum depth of 3 feet below proposed finished subgrade is required.
4. At least one boring shall be drilled to a depth of at least 10 feet in order to determine the presence of water and bedrock.
5. Soil samples taken in each boring will be **visually classified and similarized** in the Region.
6. Soil samples will be logged on Form #555 by Region personnel.
7. Borings will be logged individually in numerical order following the convention noted in the Soil Survey / Preliminary Soil Profile, Subsection 6.4.
8. Samples that are similar will be logged and similarize (as applicable) after the initially encountered soil type(s).
9. There will not be more than 1 mile between similarized soil samples.
10. Soil samples for ***Sulfate*** tests will be collected for **each** soil type in **each** boring.
11. Soil and water (if available) samples for Corrosion tests for pipe selection will be collected at inlet or outlet loccations where water or soil contact water transport structure (pipe, culvert, etc.)
12. A minimum of 5 lbs. of soil will be sampled for ***Sulfate*** and ***Corrosion*** tests.
13. A minimum of 1/2 quart (500 ml) of water will be sampled for ***Corrosion*** tests.
14. **Sulfate** and **Corrosion** samples will be sealed in a container or bag, marked with the Test No. and logged on Form #555 by placing an “S” for sulfate testing only and a “C” for corrosion tests in the **Sulfate/Corrosion** column. A copy of Form #157 and Form #555 will be included in the **Sulfate/Corrosion** submittal to be sent to the Central Laboratory **Chemical Unit**.
15. Corrosion tests include Sulfate, Chloride, pH, and Soil Resistivity for pipe material type selection.

Materials Ownership and Forms

1. The soil samples will be logged on the most current Preliminary Soil Survey Form #555.
2. Form #157 will be completed with specified soil tests by Region personnel.
3. Form #157 and Form #555 will be included in the sample bag with the tag (Form #633) marked appropriately.
4. Electronic Form #555 shall be e-mailed to the Central Lab Soils Program lab manager.
5. Soil samples will be sent to the Region Materials Lab for analysis. The Central Lab Soil Program lab manager can be contacted if assistance it required for sample analysis..
6. Samples for ***Sulfate*** and ***Corrosion*** tests will be tagged (Form #633) and sent to the Region Materials Lab or Central Lab’s Chemical Unit and submitted through Site Manager Materials.

<u>**Materials Ownership and Documentation**</u>

1. ***Field*** or ***Region Lab*** will use CP 20, CP 21, and the Form #564 to complete the soil classification.
2. ***Field* or *Region*** will follow CP 24 and mathematically scalp the gradation on the appropriate sieve and determine if there are significant variations in the material from the preliminary soil survey.
3. **If there are significant variations from the preliminary soil survey,** all +3/8, +#4, and - #4 materials will be separated and retained in separate bags.
4. The sample material with a Form #157 requesting an R-value will be sent to the Region Lab (*****) or Central Lab.
5. The soil classification on Form #564 will also be sent to the Region Lab or Central Lab.
6. If ***no*** significant variations are found, record on the Form #219 for project documentation.

<u>**Borrow Pits**</u> (refer to Standard Specifications for Road & Bridge Construction for details)

Contractor Source: The cost of complying with Section 106.02, (b) *Contractor Source* requirements, including sampling, testing, and corrective action by the Contractor, shall be included in the work.

CDOT reserves the right to verify the contractor's source.

<u>**Materials Ownership, Sampling, and Forms (FMM QA Schedule)**</u>

1. If embankment will support concrete pavement or be chemically stabilized, during production one soil sample per 2,000 yds^3 or fraction thereof will be tested for sulfate from the designated source by CDOT project or Region personnel.
2. Results will be documented on Forms #157 and #323.
3. During qualification of a borrow source, one 5 lb. sample of soil, per soil type, will be submitted to the Chemical Unit of the Central Laboratory for sulfate content.

<u>**Notes:**</u>

1. Region Lab/Soils Program will perform classification of soils.
2. Chemical Unit will perform chemical analysis of soil samples for sulfates.
3. Chemical Unit will provide the Project with the chemical analysis on qualification of borrow sources.
4. For the preliminary soil survey, the Chemical Unit will provide the Region Materials Program with the chemical analysis reports and forward the results to the Soils Program.
5. The Soils Program will input the chemical results onto the electronic Form #555, and forward the completed preliminary soil survey to the Region Materials Program.
6. Chemical Unit will perform chemical analysis of soil samples for corrosion tests and will provide test results to the Region for pipe material type selection.
7. ***** If the Region Lab has the ability to perform T 190 then no sample needs to be sent to the Central Lab.

Region Soil Survey Drilling Checklist

Reconnaissance of Drill Site

	Yes	No	N/A
1. Was a reconnaissance survey of the area to be drilled performed?	☐	☐	☐
2. Have landowner clearances and locates been obtained?	☐	☐	☐
3. Have temporary easements been obtained?	☐	☐	☐
4. Have drilling methods been determined?	☐	☐	☐
5. Have roadway condition and type of pavement been noted?	☐	☐	☐
6. Have rock outcrops been noted?	☐	☐	☐
7. Have survey cross sections or profiles been performed?	☐	☐	☐
8. Is there drilling for existing roadway?	☐	☐	☐
9. Is there drilling for new or extension of roadway surface?	☐	☐	☐
10. Have structures and culverts been identified?	☐	☐	☐
11. Has the Soil Survey Field Report, Form # 554 been completed?	☐	☐	☐
12. Have sulfate/corrosion resistance samples been taken?	☐	☐	☐

Preliminary Soil Survey

	Yes	No	N/A
General			
1. Preliminary Soil Survey, Form #555 worksheet available and used?	☐	☐	☐
2. Borings drilled in roadway?	☐	☐	☐
3. Borings drilling in shoulder?	☐	☐	☐
4. Boring drilled in R.O.W.?	☐	☐	☐
5. 1 boring per 1,000 linear feet of 2-lane roadway minimum?	☐	☐	☐
6. 1 boring per 500 linear feet of 2-lane roadway in cut/fill areas minimum?	☐	☐	☐
7. 1 boring to a depth of at least 10 feet?			
8. Is the finished grade known?	☐	☐	☐
9. Depth of boring minimum of 3-10 feet below finished roadway grade?	☐	☐	☐
10. Is the finished grade unknown?	☐	☐	☐
11. Depth of boring minimum of 3-10 feet into subgrade material?	☐	☐	☐
12. Pavement cores collected, labeled, and photographed?	☐	☐	☐
13. Additional drilling performed after the finished grade is known?	☐	☐	☐
14. Water table encountered and depth noted?	☐	☐	☐
15. Drilling adjacent to Wetlands?	☐	☐	☐
16. Ground water wells established?	☐	☐	☐
17. In-situ samples taken?	☐	☐	☐
18. Have sulfate/corrosion resistance samples been taken?	☐	☐	☐
Cut Areas			
1. Boring location similar to **Figure SS-1**?	☐	☐	☐
2. Boring depth similar to **Figure SS-3**?	☐	☐	☐
3. Depth of boring minimum of 3 feet below finished roadway grade?	☐	☐	☐
4. Additional drilling performed in cut sections needed?	☐	☐	☐
Fill Areas			
1. If proposed fill is greater than 5 feet, were borings 2 x H?	☐	☐	☐
2. Boring location similar to **Figure SS-2**?	☐	☐	☐
3. Boring depth 5 feet into hard substratum?	☐	☐	☐
4. Boring depth similar to **Figure SS-4**?	☐	☐	☐

* If suspicious material is encountered during drilling

- Stop Drilling
- Do not move the drill rig
- Secure area and provide traffic control if necessary
- Contact Region Environmental and/or Region Safety Coordinator

Mathematically Scalping a Gradation
(Instructions for when a Preliminary Soil Survey has been performed.)

When less than 75 percent is passing the 3/4 inch sieve, divide the 3/8 inch sieve percent by the 1 inch sieve percent and multiply the quotient by 100. The result will yield the "as run" gradation reported on CDOT Form #555. Perform this calculation on each successive sieve. When more than 75 percent is passing the 3/4 inch sieve, use the 3/4 inch sieve percent as a divisor and then perform the same calculation on each successive sieve.

			< 75%					
Sieve	3	1	3/4	3/8	#4	#10	#40	#200
% Passing	100	66	61	50	45	41	28	16
As Run		100	100	76	68	62	42	24

Scalp
(50 / 66) * 100 = 76

			> 75%					
Sieve	3	1	3/4	3/8	#4	#10	#40	#200
% Passing	100	99	98	95	90	80	57	21
As Run		100	100	97	92	82	58	21

Scalp
(95 / 98) * 100 = 97

Cumulative Setup for a R-Value

			< 75%					
Sieve	3	1	3/4	3/8	#4	#10	#40	#200
% Passing	100	66	61	50	45	41	28	16
As Run		100	100	76	68	62	42	24

Scalp
(50 / 66) * 100 = 76

	R-value Setup		
100	76	68	
	X	X	
	12	12	
+ 3/8	288		(100-76) * 12 = 288
+ #4	384		(100-68) * 12 = 384
- #4	1200		

			> 75%					
Sieve	3	1	3/4	3/8	#4	#10	#40	#200
% Passing	100	99	98	95	90	80	57	21
As Run		100	100	97	92	82	58	21

Scalp
(95 / 98) * 100 = 97

	R-value Setup		
100	97	92	
	X	X	
	11	11	
+ 3/8	33		(100-97) * 11 = 33
+ #4	88		(100-92) * 11 = 88
- #4	1100		

CDOT Forms #554, #555, and #157; Examples and Instructions

CDOT Form #554, see **Figure 2.5 CDOT Form #554** shall be used as the first sheet on each Soil Survey.

Full distribution, as indicated on the form, will be made at the time samples are transmitted to the Central Laboratory.

The report number from CDOT Form #554 (**Figure 2.5 CDOT Form #554)**shall be placed on all of CDOT Form #555 sheets included in the Soil Survey.

CDOT Form #555, see **Figure 4.6 CDOT Form #555, as Submitted by the Region** and **Figure 4.7 CDOT Form #555, as Completed by the Central Laborator**y, may be used in place of the field notebook. However, the electronic Form #555 shall be e-mailed to the Soils Program Laboratory Manager when samples have been submitted to the Central laboratory.

The Region office may elect to type the information from the field notebook or original CDOT Form #555 onto another Form #555. A hard copy of CDOT Form #554 and #555 shall accompany samples submitted to the Central Laboratory.

A copy of CDOT Form #555 may be made for Region Materials Laboratory files. No other distribution of the partially completed Form #555 is necessary.

When samples have been processed in the Central Laboratory, CDOT Form #555 will be completed and distributed.

Distribution of photocopies will be made as indicated on CDOT Form #554.

Serial #1267

COLORADO DEPARTMENT OF TRANSPORTATION **SOIL SURVEY FIELD REPORT**		Report 000023	
		Project # IM 0253-151	
		Location I-25, SH 7 to WCR 16	
Function 3200	Part. P	Project code (SA#) 11925	Region 4 · Date 5/5/02

Begin station 189+00	End station 569+00	Length 5.3 KM. → MI.
Equations (stations) 212+00 Bk =212+10 Ah		
Structures (stations) 240+00, E-12-B, Crow Creek; 312+00,E-17-A, Deer Creek; 640+00,E-18-F, Dry Wash		
Type of construction New Alignment		Compaction type: T99
No. of test holes 25	No. of samples 17	Proposed pavement type Flexible
Adjacent terrain data Rolling Hills		
Perform tests for swelling soil Yes	Water sample 1	
Are old uncoated culverts corroding? Yes	If yes, or area does not contain uncoated pipe, either descriptive documentation, samples or both are required per "Soil Survey Procedure" in the Design Manual.	
Record number and type of samples submitted for corrosion analysis. If submitted on separate CDOT Form #157, give report No.	1	Water
	2	Soil
Type of drilling equipment used 4" Auger	Resident Engineer Dave Forsyth	

Comments

Swampy area between Sta. 345+50 - 348+25.
Existing landslide on hillside @ Sta. 350+00 30' Lt.

Centerline located adjacent to pond between
Sta. 410+25 - 410+00.

All excavation will be common except rock outcrop between
Sta. 470+20 & 472+50 which will require blasting.

Large boulders (2'-3') embedded in grade @ Sta. 514+00

Sampled by Fidel Gonzales	Title E/PS Tech III	Supervisor (Proj./Res./Matls.) signature Corey Stewart / P.E. I
White - Staff Materials & Geotechnical Yellow - Resident Engineer's Office (Project file) Pink - Region Materials office		Address 1050 Lee Hill Rd. Boulder, Co. 80302

CDOT Form #554 1/01

Figure 4.5 CDOT Form #554

COLORADO DEPARTMENT OF TRANSPORTATION
PRELIMINARY SOIL SURVEY

Form #157 No. 351633	Form #554 No. 25687	Date Submitted: 04/17/2015
Region: 1	Contract ID: C18180	
Project No. FBR 0404-050		
Project Location: US 40 Over Sand Creek		

User ID MAYHEWT

Note 1: If samples are submitted leave sieve analysis section blank
Note 2: Comments should be placed in the discription column of the form
Note 3: Sulfate content expressed as a percent (Dry soil), or ppm in water.

Sample ID/ Station & Log	Test No.	Description	Sulfate Content (SO_4)	Percent passing 3"	1"	3/4"	3/8"	#4	#10	#40	#200	Liquid limit	Plastic Index	Classification & Group Index	Mois-ture %	R-Val	M_R P.S.I.
MP 97 to 103.3																	
MP 97+20 8' LT																	
0" to 5"	1A	HMA															
5" to18"	1B	ABC-sample													0.7		
18" to 40"(refusal)	1C	Red, Gravelly, silt-sm	0.02												0.8		
MP 98+00 6' RT																	
0" to 5"	2A	HMA															
5" to16"	2B	ABC, similar to 1B															
16" to 30"(refusal)	2C	Brown,gravelly,silt-sm	0.00												1.1		
MP 99+00 8' RT																	
0" to 8"	3A	HMA															
8" to12"	3B	ABC, similar to 1B															
12" to 28"(refusal)	3C	Similar to 2C	0.00														

☐ CDOT Central Lab
☐ Region Materials Engineer
☐ Resident Engineer

Previous editions are obsolete and may not be used.

CDOT Form #555 5/14

Figure 4.6 CDOT Form #555, as Submitted by the Region

COLORADO DEPARTMENT OF TRANSPORTATION
PRELIMINARY SOIL SURVEY

Form #157 No. 351633 | Form #554 No. 25687 | Date Submitted: 04/17/2015

Region: 1 | Contract ID: C18180

Project No. FBR 0404-050

Project Location: US 40 Over Sand Creek

User ID MAYHEWT

Note 1: If samples are submitted leave sieve analysis section blank
Note 2: Comments should be placed in the discription column of the form
Note 3: Sulfate content expressed as a percent (Dry soil), or ppm in water.

Sample ID/ Station & Log	Test No.	Description	Sulfate Content (SO_4)	Percent passing								Liquid limit	Plastic Index	Classification & Group Index	Mois-ture %	R-Val	M_R P.S.I.
				3"	1"	3/4"	3/8"	#4	#10	#40	#200						
MP 97 to 103.3																	
MP 97+20 8' LT																	
0" to 5"	1A	HMA															
5" to18"	1B	ABC-sample				100	87	64	49	29	19.5	28	8	A-2-4(0)	0.7	73	28853
18" to 40"(refusal)	1C	Red, Gravelly, silt-sm	0.02			100	86	74	62	48	33.1	26	11	A-2-6(0)	0.8	50	25317
MP 98+00 6' RT																	
0" to 5"	2A	HMA															
5" to16"	2B	ABC, similar to 1B												A-2-4(0)		73	28853
16" to 30"(refusal)	2C	Brown,gravelly,silt-sm	0.00			100	76	59	50	36	22.8	28	11	A-2-6(0)	1.1	55	19492
MP 99+00 8' RT																	
0" to 8"	3A	HMA															
8" to12"	3B	ABC, similar to 1B												A-2-4(0)			28853
12" to 28"(refusal)	3C	Similar to 2C	0.00											A-2-6(0)			19492

☐ CDOT Central Lab
☐ Region Materials Engineer
☐ Resident Engineer

Previous editions are obsolete and may not be used.

CDOT Form #555 5/14

Figure 4.7 CDOT Form #555, as Completed by the Central Laboratory

COLORADO DEPARTMENT OF TRANSPORTATION **FIELD REPORT FOR SAMPLE IDENTIFICATION OR MATERIALS DOCUMENTATION** **Metric units** ☐ **yes** ☑ **no**	Region 1	Field sheet # 120227
	Contract ID C18180	Date Submitted 03/16/2015
	Project No. FBR 0404-050	
	Project Location US 40 Over Sand Creek	

Material Type Embankment, Soil			Field Lab phone 719-555-2525	Cell Phone 719-555-5353
Material Code (LIMS) 203.03.01.01	Item 203	Class	Grading	Special Provisions ☑ yes
Previously used on Project No.:		Previous CDOT Form #157 F/S No.(s):		☑ CDOT Form #633 (sack) ☐ CDOT Form #634 (can)

- Sample Identification: Quantity & Unit of material submitted, describe tests required, precise location sample removed from (stationing), etc.
- Materials Documentation: Field inspected (describe appearance, weight/dimensions, model/serial number), COC &/or CTR provided , etc.

Submitting (6) canvas bags of soil for preliminary soil survey.

Please complete the following tests: T89, T90, and M145 CP-L3101 (Min 50)

Soil Survey enclosed in bag #1

User ID KOCHISL

Sample ID (#1)	Sample ID (#2)	Sample ID (#3)
153G113625	153G3738	153G114101
Sample ID (#4)	**Sample ID (#5)**	**Sample ID (#6)**
153G114523	153G115236	153G120559

APL/QML Acceptance: APL Ref. No.	Product name:	Date checked:
APL/QML Acceptance: APL Ref. No.	Product name:	Date checked:

Preliminary ☑	Construction ☐	Maintenance ☐	Emergency ☐	Date needed 04/01/2015

Contractor	Supplier
Sampled from (Pit, roadway, windrow, stock, etc.)	Pit name or owner

Quantity represented 1/LANE MILE, MIN	Previous quantity	Total quantity to date

Sample submitted: ☑ Yes ☐ No	Shipped specified quantity to: 6 ☑ Central lab ____ ☐ Region lab	Via CDOT	Date 03/17/2015

Sampled or inspected by (print name) LESLIE KOCHIS	Title EPST III	E-mail leslie.kochis@dot.state.co.us
Supervisor (Pro./Res./Matls. Engr./Maint. Supt.) (print name) KARL LARSON	Title CEPM I	Residency LIMON

Distribution: White copy - CDOT Central Laboratory
(submit white copy only if sample or information is directed to Staff Materials)
Canary copy - Region Materials Engineer
Pink copy - Resident Engineer

Previous editions are obsolete and may not be used.

CDOT Form #157 4/14

Figure 4.8 CDOT Form #157

4.3 Subgrade and Embankment

Subgrade can be categorized as shown in **Figure 4.9 Subgrade Preparation**.

- **Conventional**: Man-made compacted layer (typically 12 inches) of the subgrade soil over the uncompacted natural soil material. Conventional subgrade involves the pre-reconditioning of the natural subgrade material into a compacted layer. Pre-reconditioning typically involves proof rolling usually before placement of other engineered layers.

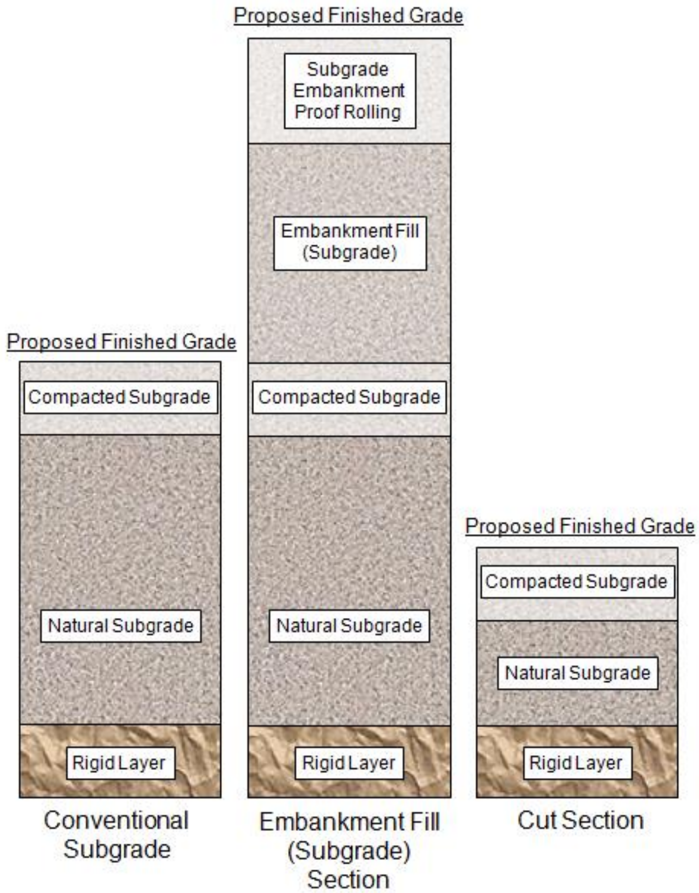


Figure 4.9 Subgrade Preparation

- **Embankment Fill:** Placement of a thick layer of imported soil or rock material over the uncompacted natural soil, typically located in a fill section. The typical soil or rock embankment material has a maximum dry density of not less than 90 pounds per cubic foot. Other properties such as resilient modulus (M_r) must be as specified in the contract plans, specifications, and as presented below:

  - **Soil Embankment:** Shall consist predominantly of materials smaller than 4.75mm (No. 4) sieve in diameter. Soil embankment is constructed with moisture density control in accordance with the requirements of Subsection 203.07 - Construction of Embankment and Treatment of Cut Areas with

Moisture and Density Control of *CDOT Standard Specification for Road and Bridge Construction*.

- **Rock Embankment:** Shall consist of materials with 50 percent or more by weight, at field moisture content, of particles with least dimension diameters larger than 4.75 mm (No. 4) sieve and smaller than 6 inches. Rock embankment is constructed without moisture density control in accordance with the requirements of Subsection 203.08 - Construction of Embankments without Moisture and Density Control of *CDOT Standard Specification for Road and Bridge Construction*.

- **Cut Section:** The finished subgrade cut section scarified to a depth of 6 inches with moisture applied or removed as necessary and compacted to a specified relative compaction.

The designer needs to be aware of a few fill embankment requirements. Claystone or soil-like nondurable shale, as defined by Colorado Procedure CP 26, shall not be treated as sound rock and shall be pulverized, placed, and compacted as soil embankment. Claystone or soil-like non-durable shale particles greater than 12 inches in diameter shall not be placed in the embankment (17).

A special case of compacted subgrade is a fill section where the fill is comprised of two layers of subgrades with different engineering properties. The lower fill may comprise of a lesser resilient modulus than the upper layer. For illustration purposes, the upper embankment fill layer is shown here as special subgrade. The upper layer may require engineered material with a higher resilient modulus than the lower layer such as a M_r value of 25,000 psi in the top 2 feet of subgrade, and the lower layer may have a M_r value of 10,000 psi (see **Figure 4.10 Special Cases of Embankment Fill**).

4.4 Subgrade Characterization for the M-E Design

4.4.1 General Characterization

The subgrade characterization procedure for M-E Design is dependent on pavement type and design (new or rehabilitation). The inputs required are the resilient modulus, soil classification, moisture content, dry density, saturated conductivity, and other physical/engineering properties (see **Figure 4.11 Subgrade Material Properties in M-E Design** and **Figure 4.12 M-E Design Software Screenshot for Other Engineering/Physical Properties of Subgrade**).

Note: In M-E Design, the subgrade resilient modulus M_r is measured at optimum moisture content and density. This M_r is different than the AASHTO 1993 empirical design procedure which was basically a "wet of optimum" M_r. The input M_r is then internally adjusted to field conditions by the M-E Design software on a month to month basis based on water table depth, precipitation, temperature, soil suction, and other factors. Select the software option *Modify Input Values by Temperature/Moisture* to allow the software to seasonally adjust the input M_r to field conditions.

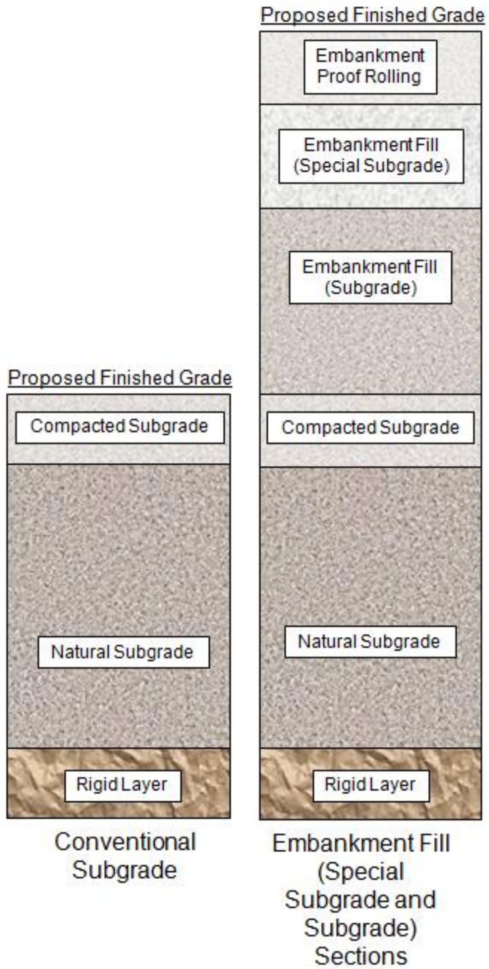


Figure 4.10 Special Cases of Embankment Fill

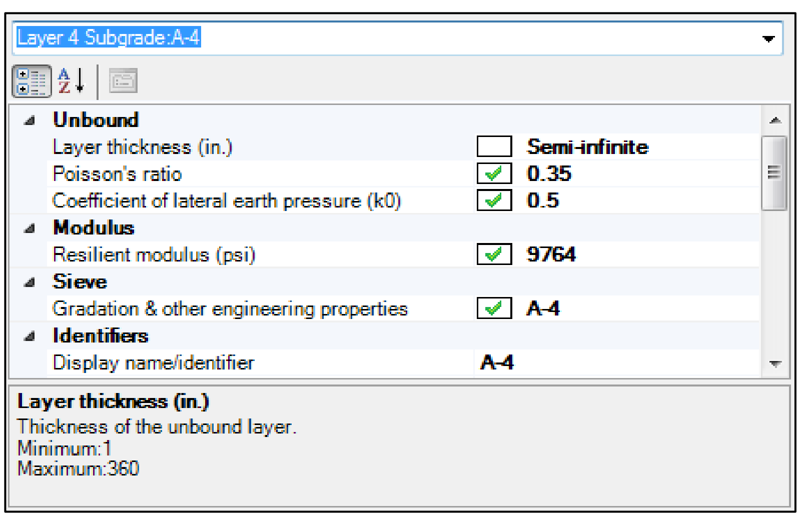


Figure 4.11 Subgrade Material Properties in the M-E Design

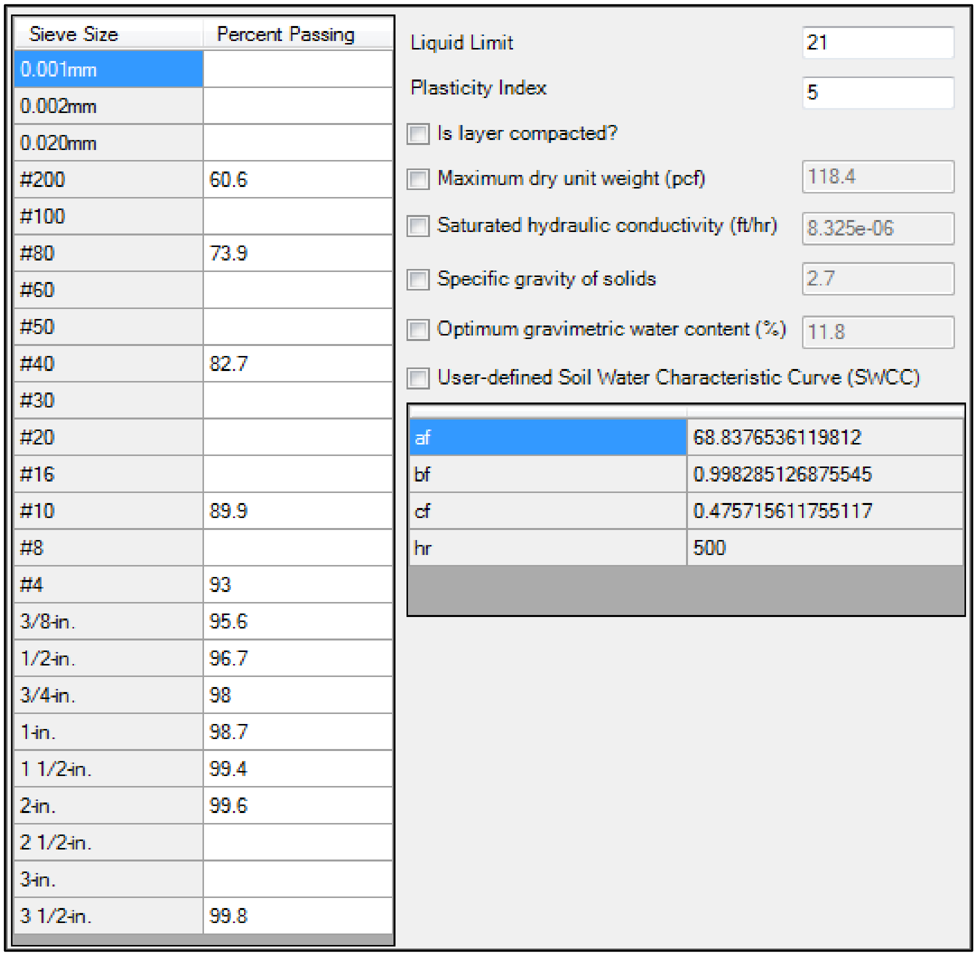


Figure 4.12 M-E Design Software Screenshot for Other Engineering/Physical Properties of Subgrade

The input requirements for subgrade characterization are presented by pavement type and design:

- New Flexible and New JPCP: **Table 4.3 Recommended Subgrade Inputs in New Flexible and JPCJ Designs**.
- HMA Overlays of Existing Flexible Pavement: **Table 4.4 Recommended Subgrade Inputs for HMA Overlays of Existing Flexible Pavemen**t.
- Overlays of Existing Rigid Pavement: **Table 4.5 Recommended Subgrade Inputs for Overlays of Existing Rigid Pavement**.

Table 4.3 Recommended Subgrade Inputs for New Flexible and JPCP Designs

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	**Level 2**	**Level 3**
New Flexible and JPCP	Resilient modulus	Not available	CDOT lab testing	AASHTO Soil Classification
	Gradation	Not available	Colorado Procedure 21-08	Use CDOT defaults
	Atterberg limit[1]	Not available	AASHTO T 195	Use CDOT defaults
	Poisson's ratio	Not available	Use M-E Design software defaults	Use M-E Design software default of 0.4
	Coefficient of lateral pressure	Not available	Use M-E Design software defaults	Use M-E Design software default of 0.5
	Maximum dry density	Not available	AASHTO T 180	Estimate internally using gradation, plasticity index, and liquid limit.[2]
	Optimum moisture content	Not available	AASHTO T 180	
	Specific gravity	Not available	AASHTO T 100	
	Saturated hydraulic conductivity	Not available	AASHTO T 215	
	Soil water characteristic curve parameters	Not available	Not applicable	

Note:
[1] For drainage reasons if non-plastic use PI = 1
[2] The M-E Design software internally computes the values of the following properties based on the inputs for gradation, liquid limit, plasticity index, and if the layer is compacted. If the designer chooses, they may modify the internally computed default values. The software updates the default values to user-defined values once the user clicks outside the software's input screen.

Table 4.4 Recommended Subgrade Inputs for HMA Overlays of Existing Flexible Pavement

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	Level 2	Level 3
HMA Overlays of Existing Flexible Pavement	Resilient modulus	FWD deflection testing and backcalculated resilient modulus	CDOT lab testing	AASHTO soil classification
	Gradation	Colorado Procedure 21-08		Use CDOT defaults
	Atterberg limit[1]	AASHTO T 195		Use CDOT defaults
	Poisson's ratio	Use software defaults		Use M-E Design software default of 0.4
	Coefficient of lateral pressure	Use software defaults		Use M-E Design software default of 0.5
	Maximum dry density	AASHTO T 180		Estimate internally using gradation, plasticity index, and liquid limit.[2]
	Optimum moisture content	AASHTO T 180		
	Specific gravity	AASHTO T 100		
	Saturated hydraulic conductivity	AASHTO T 215		
	Soil water characteristic curve parameters	Not applicable		

Note:
[1] For drainage reasons if non-plastic use PI = 1
[2] The M-E Design software internally computes the values of the following properties based on the inputs for gradation, liquid limit, plasticity index, and if the layer is compacted. If the designer chooses, they may modify the internally computed default values. The software updates the default values to user-defined values once the user clicks outside the software's input screen.

Table 4.5 Recommended Subgrade Inputs for Overlays of Existing Rigid Pavement

Pavement and Design Type	Material Property	Input Hierarchy		
		Level 1	Level 2	Level 3
Overlays of Rigid Pavement	Resilient Modulus	FWD deflection testing and backcalculated dynamic k-value[3]	CDOT lab testing	AASHTO soil classification
	Gradation	Colorado Procedure 21-08		Use CDOT defaults
	Atterberg Limit [1]	AASHTO T 195		Use CDOT defaults
	Poisson's ratio	Use software defaults		Use M-E Design software default of 0.4
	Coefficient of lateral pressure	Use software defaults		Use M-E Design software default of 0.5
	Maximum dry density	AASHTO T 180		Estimate internally using gradation, plasticity index, and liquid limit.[2]
	Optimum moisture content	AASHTO T 180		
	Specific gravity	AASHTO T 100		
	Saturated hydraulic conductivity	AASHTO T 215		
	Soil water characteristic curve parameters	Not applicable		

Note:
[1] For drainage reasons if non-plastic use PI = 1
[2] The M-E Design software internally computes the values of the following properties based on the inputs for gradation, liquid limit, plasticity index, and if the layer is compacted. If the designer chooses, they may modify the internally computed default values. The software updates the default values to user-defined values once the user clicks outside the software's input screen.
[3] The k-value represents the subgrade layer, as well as, unbound layers including granular aggregate base and subbase layers.

4.4.2 Modeling Subgrade Layers in M-E Design Software

The M-E Design software divides the pavement structure, including subgrade, into sublayers for analysis purposes. The software divides the top 8 feet of a pavement structure and subgrade into a maximum of 19 sublayers. The remaining subgrade is treated as a semi-infinite layer. The designer should consider the following to properly characterize subgrade in M-E Design:

- **Modeling Embankments**
    - When a full-depth flexible or semi-rigid pavement is placed directly on a thick embankment fill, the top 10 inches is modeled as an Aggregate Base Layer, while the remaining embankment is modeled as the Subgrade Layer 1. The M_r and other physical/engineering properties remain the same for both layers. The natural subgrade below the embankment fill is modeled as Subgrade Layer 2.

- **Modeling Thick Aggregate Bases**

    - When a thick granular aggregate base (more than 10 inches) is used, the top 10 inches is modeled as an aggregate base layer, while the remaining aggregate is modeled as the Subgrade Layer 1. The M_r and other physical/engineering properties remain the same for both layers. The compacted or natural subgrade below the thick aggregate base is modeled as lower subgrade layers.

- **Modeling Compacted Subgrade**

    - The compacted and natural subgrade are modeled as separate subgrade layers.

- **Need for Improvement**

    - The designer should establish the need for improving or strengthening the existing subgrade based on subsurface investigation results. Typically, if the subgrade has a M_r less than 10,000 psi, subgrade improvement could be considered.

- **Effects of Frost Susceptible/Active Soils**

    - The M-E Design software does not directly predict the increase in distresses caused by expansive, frost susceptible, and collapsible soils. Treatments to such problematic soils could be considered (outside the M-E Design analysis) as a part of the design strategy.

- **Modeling Bedrock**

    - Bedrock or any hard layer encountered more than about 20 feet below the pavement will have an insignificant effect on the calculated pavement responses and predicted distresses/IRI. Inclusion of bedrock in the pavement structure below 20 feet is **<u>not</u>** recommended.

- **Modeling Geosynthetics**

    - Filter fabrics, geotextiles, and geogrids cannot be directly included in the pavement structure.

4.4.3 Recommended Inputs for Subgrade/Embankment Materials

4.4.3.1 Inputs for New HMA and JPCP

<u>Level 1 Inputs</u>

Level 1 inputs are not available for new HMA and JPCP designs in this manual since they are project specific values.

Level 2 Inputs

The designer must input a single value of design M_r. Two approaches are available for Level 2 design subgrade M_r:

- **Laboratory Resilient Modulus**: The design M_r may be obtained through laboratory resilient modulus tests conducted in accordance with AASHTO T 307, Determining the Resilient Modulus of Soils and Aggregate Materials. Subgrade design M_r should reflect the range of stress states likely to be developed beneath flexible or rigid pavements subjected to moving wheel loads. Therefore, the laboratory measured M_r should be adjusted for the expected in-place stress state for use in M-E Design software. Stress state is determined based on the depth at which the material will be located within the pavement system (i.e., the stress states for specimens to be used as base or subbase or subgrade may differ considerably).

- **CDOT Resilient Modulus, R-value Correlation:** The design M_r may be obtained through correlations with other laboratory tested soil properties such as the R-value. Equation **Eq. 4-1** gives an approximate correlation of resistance value (R-value) to M_r. This equation is valid only for AASHTO T 190 procedure. If the R-value of the existing subgrade or embankment material is estimated to be greater than 50, a FWD analysis or resilient modulus by AASHTO T 307 should be performed. CDOT uses Hveem stabilometer equipment to measure strength properties of soils and bases. This equipment yields an index value called the R-value. The R-value is considered a static value and the M_r value is considered a dynamic value.

$$\mathbf{Mr = 3438.6 * R^{0.2753}} \qquad \textbf{Eq. 4-1}$$

Where:

M_r = resilient modulus (psi)
R = R-value obtained from the Hveem stabilometer

This equation **should be** used for R-values of 50 or less. Research is currently being done for soils with R-values greater than 50. The Hveem equipment does not directly provide resilient modulus values, rather, it provides the R-value which is then used to obtain an approximation of resilient modulus from correlation formulas.

The M-E Design software allows the designer to estimate M_r using other soil properties (see **Figure 4.13 M-E Design Software Screenshot for Level 2 Resilient Modulus Input**).

- California Bearing Ration (CBR)
- R-value
- Layer coefficient (a_i)
- Dynamic Cone Penetrometer (DCP) Penetration
- Plasticity Index (PI) and gradation (i.e., percent passing No. 200 sieve)

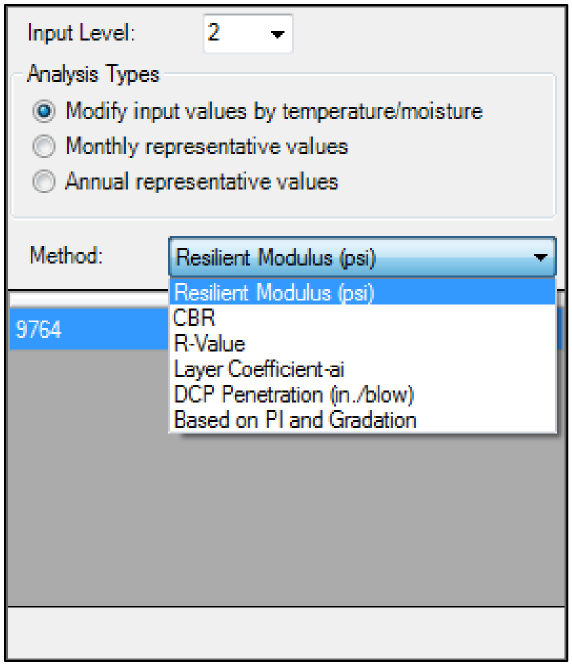


Figure 4.13 M-E Design Software Screenshot for Level 2 Resilient Modulus Input

The mathematical relationship between the M_r value and the above mentioned soil properties are hard coded in the M-E Design software, and the estimation is done internally. The M_r to R-value correlation in the software follows the relationship provided in the *AASHTO 1993 Pavement Design Guide*. Other engineering properties may be obtained as recommended in **Tables 4.4 and 4.5 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Level 3 Inputs

Typical M_r values for Level 3 inputs are presented in **Table 4.6 Level 3 Resilient Modulus For Embankments and Subgrade**. **Note:** The M_r values presented in this table are at optimum moisture content and maximum dry density. **Table 4.6 should only be used for a preliminary pavement design when a resilient modulus or R-value is unavailable. The final pavement design shall use Level 1 or 2 resilient modulus value(s) specific to the project that is/are obtained either in a laboratory or via Equation 4-1**. **Figure 4.14 M-E Design Software Screenshot for Level 3 Resilient Modulus Input** presents the screenshot showing the Level M_r input in the M-E Design software which uses predictive equations based on soil class, gradation, plasticity index, liquid limit, and internally calculates other engineering properties.

Table 4.6 Level 3 Resilient Modulus For Embankments and Subgrade
(Only Use For A Preliminary Design)

AASHTO Soil Classification	Resilient Modulus (M_r) at Optimum Moisture (psi)	
	Flexible Pavements	Rigid Pavements
A-1-a	19,700	14,900
A-1-b	16,500	14,900
A-2-4	15,200	13,800
A-2-5	15,200	13,800
A-2-6	15,200	13,800
A-2-7	15,200	13,800
A-3	15,000	13,000
A-4	14,400	18,200
A-5	14,000	11,000
A-6	17,400	12,900
A-7-5	13,000	10,000
A-7-6	12,800	12,000

Note: This table is only to be used during a preliminary design when there is minimum knowledge of the subgrade properties. Levels 1 and 2 values **must** be used for all final designs.

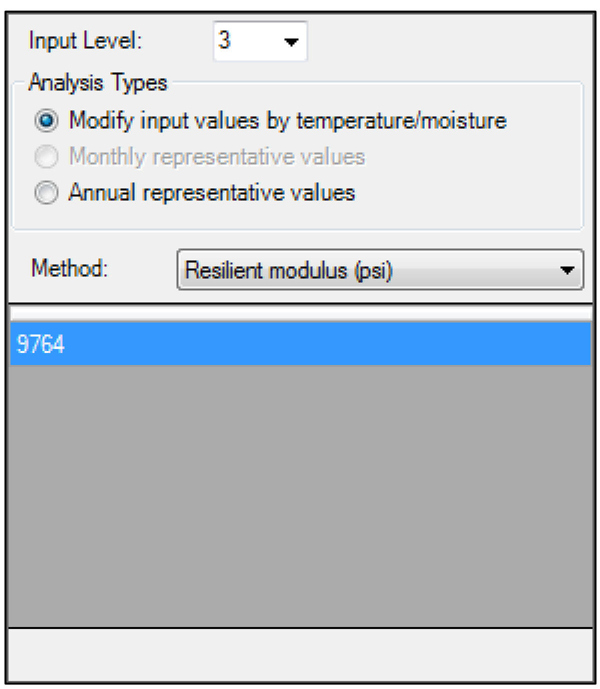


Figure 4.14 M-E Design Software Screenshot for Level 3 Resilient Modulus Input

4.4.3.2 Inputs for HMA Overlay of Existing Flexible Pavements

Level 1 Inputs

Level 1 design subgrade M_r (at in-situ moisture content) for overlays of existing pavement designs, is obtained through FWD testing and backcalculation of pavement deflection data. **APPENDIX C: Deflection Testing and Backcalulation Method** contains detailed information on how to perform FWD testing and process pavement deflection data to obtain backcalculated elastic moduli.

The subgrade elastic modulus (E_R) values obtained from backcalculation of FWD deflection data do not match with the resilient modulus values measured in the laboratory. The FWD backcalculated elastic modulus values represent field conditions under dynamic loading and require an adjustment to laboratory test conditions. The adjustment factors to convert FWD backcalculated elastic modulus to laboratory resilient modulus values are presented in **Table 4.7 Average Backcalculated to Laboratory Determined Elastic Modulus Ratios**. In the M-E Design software, the backcalculated in-situ subgrade M_r should be entered in conjunction with the in-situ subgrade moisture content. Average moisture content measured at the time of FWD testing is recommended for use. Other engineering properties may be obtained as recommended in **Tables 4.4** and **4.5 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Table 4.7 Average Backcalculated to Laboratory Determined Elastic Modulus Ratios

Layer Type	Location	Mean M_r /E_R Ratio
Unbound Granular Base and Subbase Layers	Granular base/subbase between two stabilized layers (cementitiuos or asphalt stabilized materials)	1.43
	Granular base/subbase under a PCC layer	1.32
	Granular base/subbase under an HMA surface or base layer	0.62
Embankment and Subgrade Soils	Embankment or subgrade soil below a stabilized subbase layer or stabilized soil	0.75
	Embankment or subgrade soil below a flexible or rigid pavement without a granular base/subbase layer	0.52
	Embankment or subgrade soil below a flexible or rigid pavement with a granular base or subbase layer	0.35

Note:
E_R = Elastic modulus backcalculated from deflection basin measurements.
M_r = Elastic modulus of the in-place materials determined from laboratory repeated load resilient modulus test.

Level 2 Inputs

Follow the guidelines presented in **Level 2 Inputs for New HMA and JPCP**.

Level 3 Inputs

Follow the guidelines presented in **Level 3 Inputs for New HMA and JPCP**.

4.4.3.3 Inputs for Overlays of Existing Rigid Pavements

Level 1 Inputs

The modulus of subgrade reaction (k-value) is a required input for rigid rehabilitation designs, unbonded concrete overlays, HMA over existing JPCP, and JPCP over AC designs. M-E Design also requires the month FWD testing was performed for seasonal adjustments.

The "effective" dynamic k-value represents the compressibility of underlying layers (i.e. unbound base, subbase, and subgrade layers) upon which the upper bound layers and existing HMA or PCC layer is constructed. The dynamic k-value is obtained through FWD testing and backcalculation of pavement deflection data. **APPENDIX C – Deflection Testing and Backcalculation Method** contains detailed information on how to perform FES testing and process pavement deflection data to obtain the dynamic modulus of subgrade reaction. The designer should only use Level 1 inputs because this level will show the pavement's response. **Note**: The k-value used in the *1998 AASHTO Supplement* is a static elastic k-value, while M-E Design uses the dynamic k-value. Other engineering properties may be obtained as recommended in **Tables 4.4** and **4.5 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Level 2 Inputs

Level 2 subgrade M_r is obtained from field testing such as R-value tests. Follow the guidelines presented in **Level 2 Inputs for New HMA and JPCP**.

M-E Design software will internally convert the M_r input to an effective, single dynamic k-value as a part of input processing. This conversion is performed internally for each month of the design analysis period and utilized directly to compute critical stresses and deflections in the incremental damage accumulation over the analysis period. Other engineering properties may be obtained as recommended in **Tables 4.4** and **4.5 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Level 3 Inputs

Follow the guidelines presented for **Level 3 Inputs for New HMA and JPCP**.

- **Estimating or Measuring the k-value:** The *1998 AASHTO Supplement* outlines three procedures to estimate or measure the k-value. There is no direct laboratory procedure for determining the initial k-value, however, there are three procedures for estimating the initial k-value. One of the procedures has three methods of correlations to determine the initial k-value. The procedures and methods are:
  - Correlations with soil type and other soil properties or tests
  - Correlation using soil classification
  - Correlation to California Bearing Ratio
  - Correlation by Dynamic Cone Penetrometer Plate bearing tests
  - Deflection testing and backcalculation (recommended)

A procedure not described in the 1998 AASHTO Supplement is using an R-value correlated to the dynamic M_r and a simplified, older AASHTO relationship equation to obtain a k-value.

After selecting which procedure to use, the designer continues to adjust the initial k-value. Two adjustment steps follow. The first step is to adjust the initial k-value to a seasonal effective k-value for the effects of a shallow rigid layer and/or an embankment above the natural subgrade.

- **Correlations of Initial k-value Using Soil Classifications:** Initial k-values may be correlated to the soil type and basic physical properties. In general, the static k-value can be determined using a simplified graphical depiction of soil classification in **Figure 4.15 k-value vs. Soil Classification**. Greater detail can be found using **Table 4.8 k-value Ranges for Various Soil Types.**

- **Cohesionless Soils (A-1 and A-3):** Recommended k-value ranges for insensitive to moisture variation A-1 and A-3 soils are summarized in Table 11 of the *1998 AASHTO Supplement* as shown in **Table 4.8 k-value Ranges for Various Soil Types** which has typical ranges of dry density and CBR for each soil type.

- **Granular Materials (A-2):** Recommended k-values for granular materials that fall between A-1 and A-3 soils are summarized in Table 11 of the *1998 AASHTO Supplement* as shown in **Table 4.8 k-value Ranges for Various Soil Types** which has typical ranges of dry density and CBR for each soil type.

- **Cohesive Soils (A-4 through A-7):** Recommended k-values for AASHTO classification of fine-grained A-4 through A-7 soils as a function of saturation are shown in the *1998 AASHTO Supplement* and in **Figure 4.16 k-values Versus Degree of Saturation for A-4 through A-7 Soils**. Each line represents the middle range of reasonable values ± 40 psi/in.

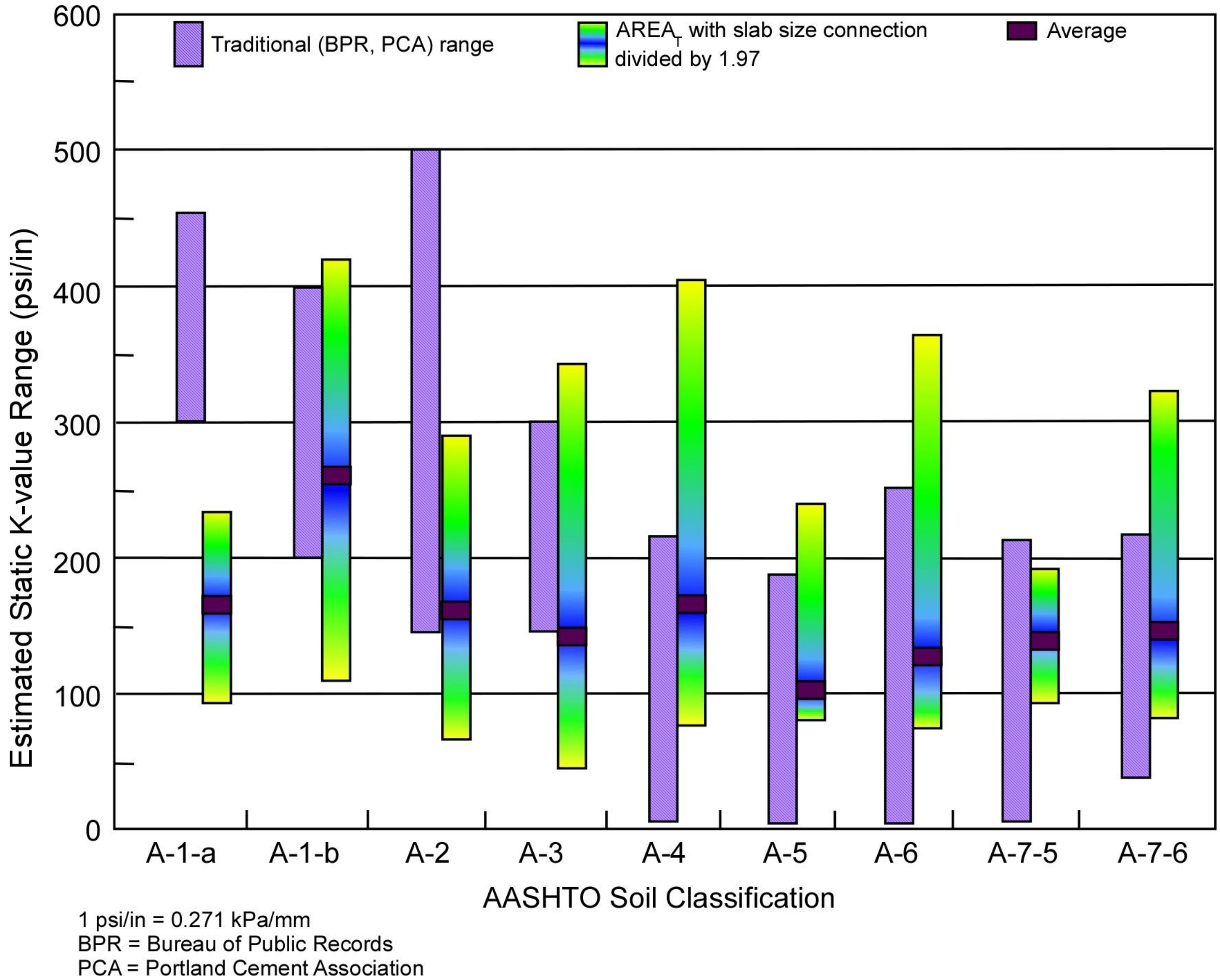


Figure 4.15 k-value vs. Soil Classification

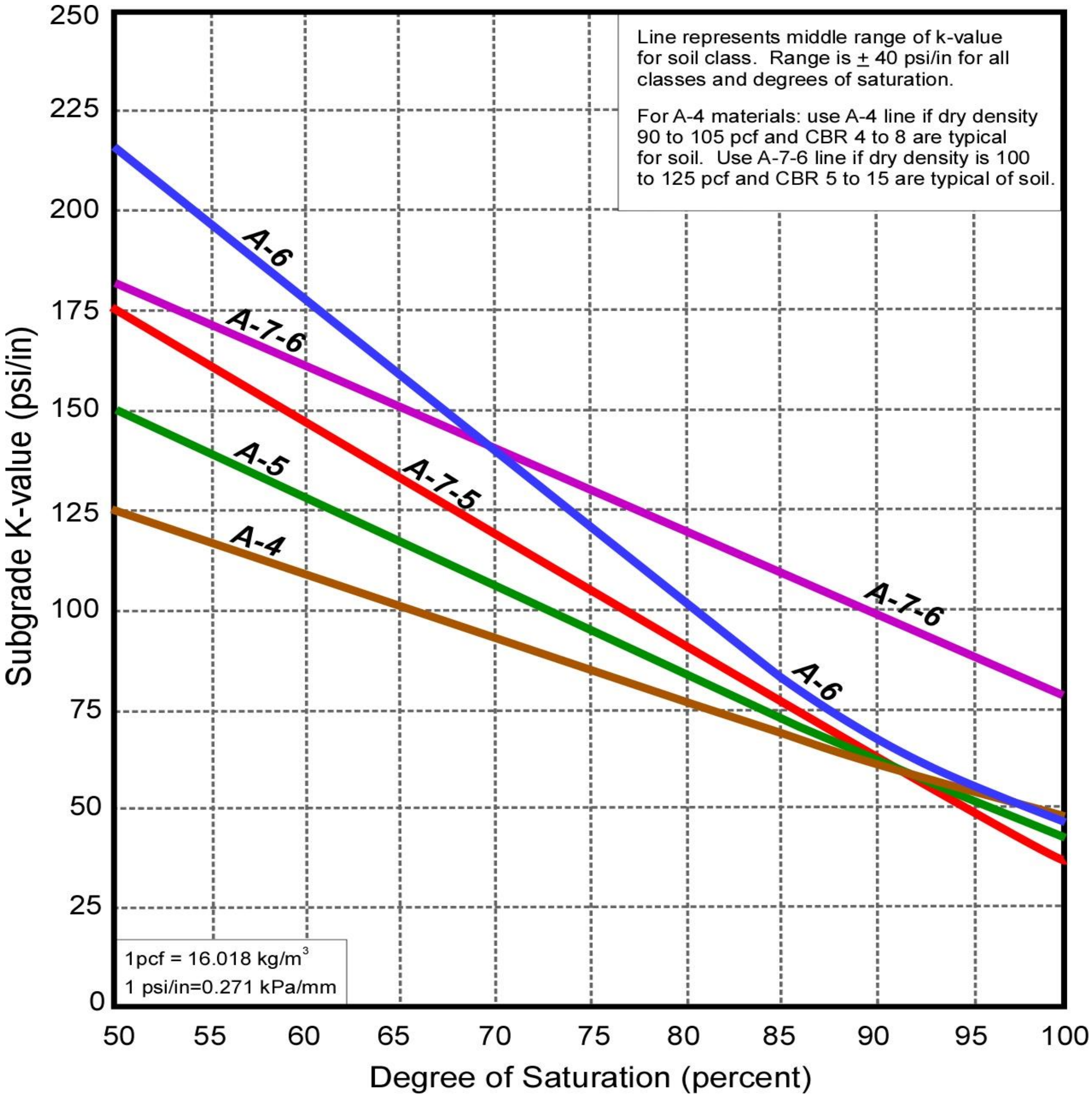


Figure 4.16 k-value vs. Degree of Saturation for A-4 Through A-7 Soils

Table 4.8 k-value Ranges for Various Soil Types

AASHTO Classification	Description	Unified Class	Dry Density (lb/ft^3)	CBR (percent)	k-value (psi/in)
Coarse-grained soils					
A-1-a, well graded	Gravel	GW, GP	125 - 140	60 - 80	300 - 450
A-1-a, poorly graded			120 - 130	35 - 60	300 - 400
A-1-b	Coarse sand	SW	110 - 130	20 - 40	200 - 400
A-3	Fine sand	SP	105 - 120	15 - 25	150 - 300
A-2 soils (granular materials with high fines)					
A-2-4, gravelly	Silty gravel	GM	130 - 145	40 – 80	300 - 500
A-2-5, gravelly	Silty sandy gravel				
A-2-4, sandy	Silty sand	SM	120 – 135	20 – 40	300 – 400
A-2-4, sandy	Silty gravelly sand				
A-2-6, gravelly	Clayey gravel	GC	120 – 140	20 – 40	200 – 450
A-2-7, gravelly	Clayey sandy gravel				
A-2-6, sandy	Clayey sand	SC	105 - 130	10 - 20	150 – 350
A-2-7, sandy	Clayey gravelly sand				
Fine-grained soils					
A-4	Silt	ML, OL	90 – 105	4 – 8	25 – 165*
	Silt/sand/gravel mixture		100 – 125	5 – 15	40 – 220 *
A-5	Poorly graded silt	MH	80 – 100	4 – 8	25 – 190*
A-6	Plastic clay	CL	100 – 125	5 – 15	25 – 255*
A-7-5	Moderately plastic elastic clay	CL, OL	90 – 125	4 – 15	25 – 215*
A-7-6	Highly plastic elastic clay	CH, OH	80 - 110	3 – 5	40 – 220*

Note: * k-value of fine grained soil is highly dependent on the degree of saturation. See Figure 40. These recommended k-value ranges apply to a homogeneous soil layer at least 10 ft. (3 m) thick. If an embankment layer less than 10 ft. (3 m) thick exists over a softer subgrade, the k-value for the underlying soil should be estimated from this table and adjusted for the type and thickness of embankment material using Figure 43. If a layer of bedrock exists within 10 ft. (3 m) of the top of the soil, the k should be adjusted using Figure 43. (These notes refer to figures in the *1998 AASHTO Supplement*).

4.5 Rigid Layer

A rigid layer is defined as the lower soil stratum with a high resilient or elastic modulus (greater than 100,000 psi). A rock layer may consist of bedrock, severely weathered bedrock, igneous, metamorphic, sedimentary material, or combinations of each, which cannot be excavated without blasting or the use of large mechanical equipment used for ripping bedrock, or over-consolidated clays. For example, a thick shale or claystone layer would be considered a rigid layer.

In M-E Design, the presence of a rigid layer within 10 feet of the pavement surface may have an influence on the structural responses of pavement layers. The designer may need to use multiple subgrade layers especially when the depth to the rigid layer exceeds 100 inches. **Note: The thickness of the last subgrade layer is limited to 100 inches when a rigid layer is added to the pavement structure in M-E Design. The rigid layer can be ignored for pavement design when the depth exceeds 20 feet.**

The M-E Design software recommended default elastic modulus values are 750,000 psi for solid, massive and continuous bedrock and 500,000 psi for highly fractured and weathered bedrock. The suggested default value for Poisson's ratio is 0.15.

4.6 Rock Fill

In pavement design, a rock fill would be a rigid layer and is defined in Subsection 203.03 - Embankment of *CDOT Standard Specification for Road and Bridge Construction*, 2011 (34). Rock fill shall consist of sound, durable stones, boulders, or broken rock not less than six inches in the smallest dimension. At least 50 percent of the rock used shall have a volume of 2 cubic feet or more, as determined by physical or visual measurement.

4.7 Frost Susceptible Soils

In areas subject to frost, soils may be removed and replaced with selected, nonsusceptible material. Where such soils are too extensive for economical removal, they may be covered with a sufficient depth of suitable material to overcome the detrimental effects of freezing and thawing. The need for such measures and the type and thickness of material required must be determined on the basis of local experience and types of materials (20). Frost heaving may be caused by crystallization of ice lenses in voids of soils containing fine particles. Bearing capacity may be reduced substantially during thawing periods. Frost heaving can be more severe during freeze-thaw periods because water is more readily available. Several cycles of freeze and thaw may occur during a winter season and cause more damage than one long period of freezing in more northerly areas of the state.

To compute the monthly or annual freezing index and estimate frost heave depth, the following equation is used:

$$FI = \sum_{i=1}^{n} (0 - T_i)$$

Eq. 4-2

Where:

FI = freezing index, degrees Celsius (°C) degree-days
T_i = average daily air temperature on day i, °C
n = days in the specified period when average daily temperature is below freezing
i = number of days below freezing

When using this equation, only the days where the average daily temperature is below freezing are used. Therefore, the freezing index is the negative of the sum of all average daily temperatures below 0 °C within the given period (29).

See **Figure 4.17 Colorado Annual Freezing Index (Degrees-Fahrenheit Days)** for a map of Colorado showing isopieth lines for the annual freezing index. The isopieth lines are in units of degree-Fahrenheit days. The highest Freezing Index values are in the mountains, Berthoud Pass, Taylor Park, and Climax. The lowest values are on the western side of the state, Gateway, Uravan, and Palisade. **Note:** The Freezing Index values do not necessarily follow elevations.

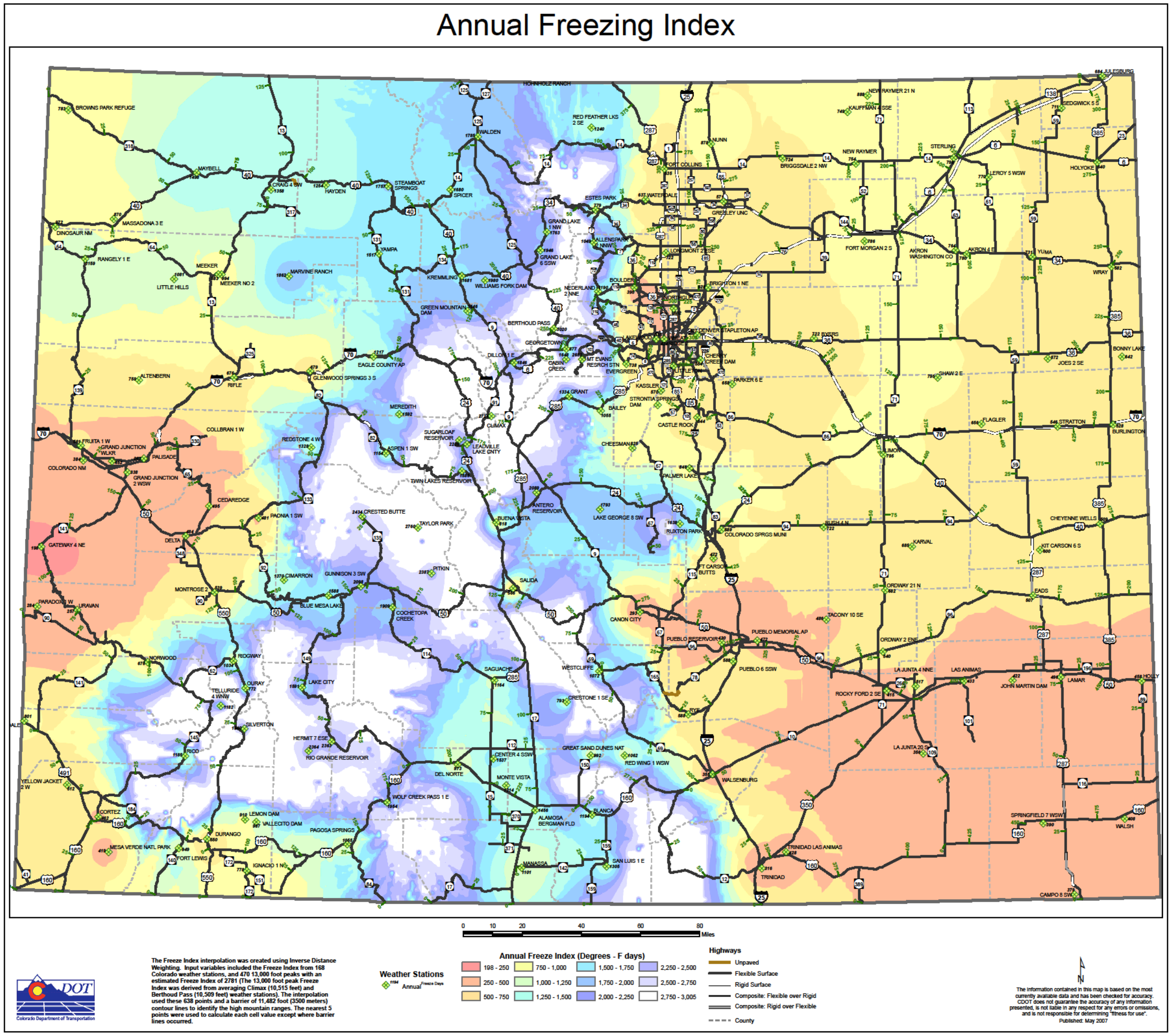


Figure 4.17 Colorado Annual Freezing Index (Degrees-Fahrenheit Days)

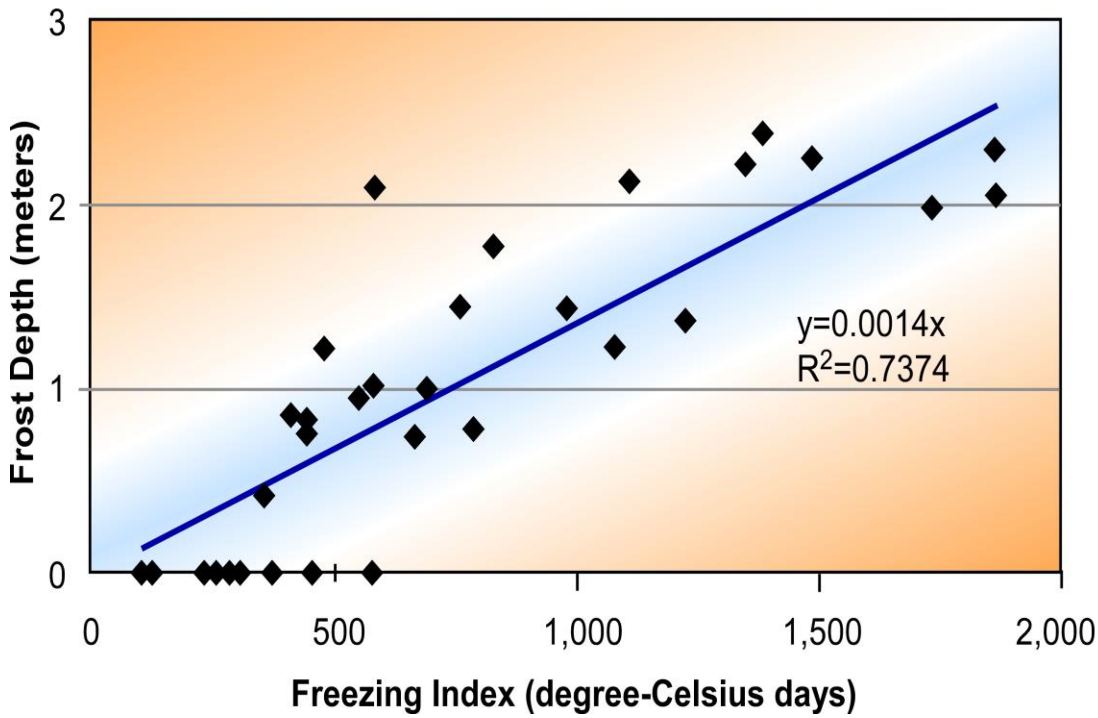


Figure 4.18 Frost Depth to Annual Freezing Index

To convert the Annual Freezing Index (degrees-Fahrenheit days) to (degrees-Celsius days) use equation **Eq. 4-3**. The conventional conversion formula has the term 32 °F and is accounted for in the number of days below freezing.

FI = Annual Freezing Index (°C days) **Eq. 4-3**
= ($^5/_9$) Annual Freezing Index (°F days)

There is a relationship between the Annual Freezing Index (FI) and frost depth. The seasonal monitoring program with FHWA Long-Term Pavement Performance sites analyzed this relationship (see equation **Eq. 4-4**) (30).

Frost Depth = 0.0014 x FI **Eq. 4-4**

Where:

Frost depth is in meters
FI is the annual freezing index (°Celsius days)

A graph was developed to show the relationship of frost depth versus freezing index, (see **Figure 4.18 Frost Depth to Annual Freezing Index**). The data scatter is influenced by local site conditions. Refer to **Figure 4.19 Frost Susceptible Soil Classifications** for possible scatter.

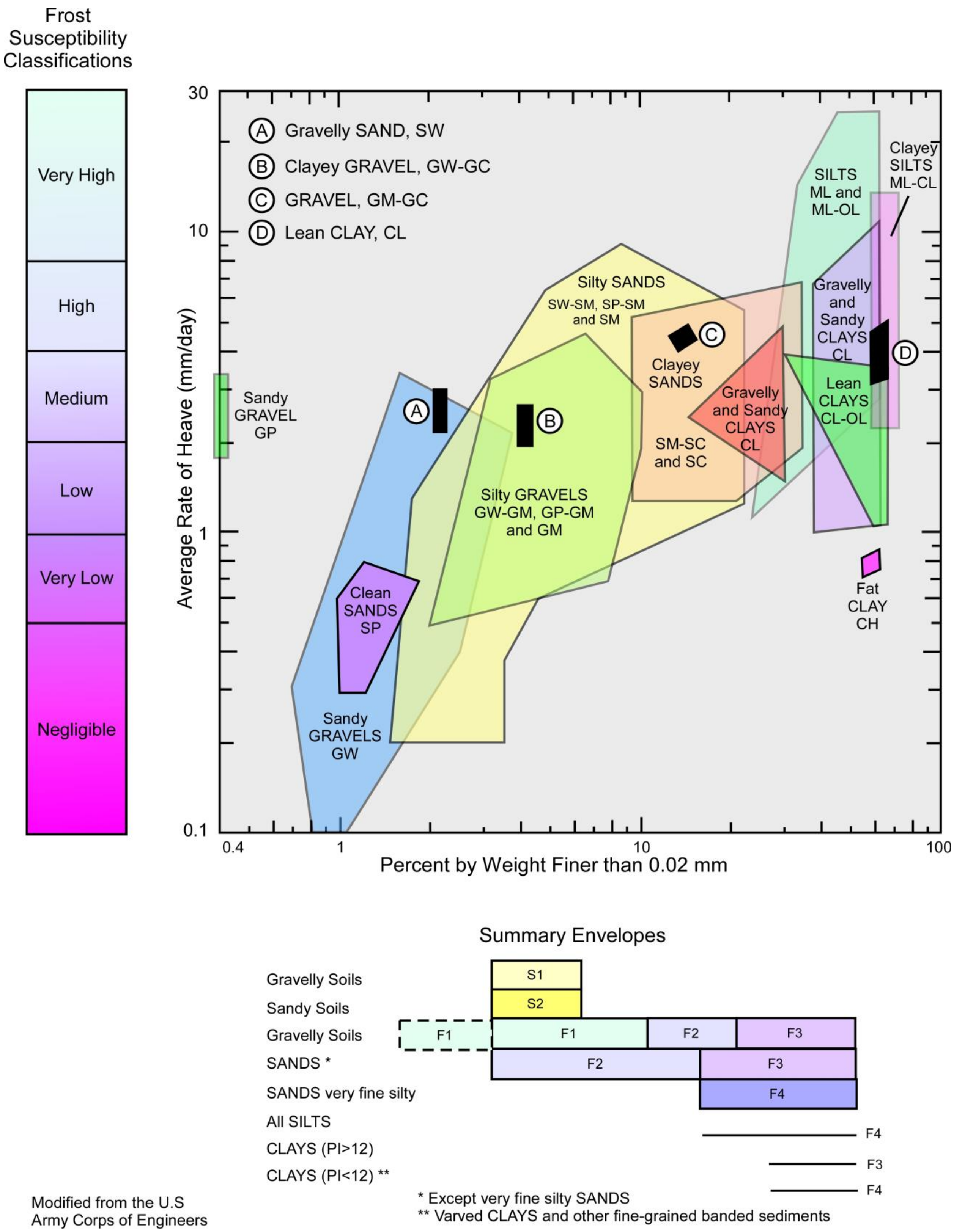


Figure 4.19 Frost Susceptible Soil Classifications

Figure 4.19 Frost Susceptible Soil Classifications shows frost susceptibility for various soil classifications (31). The figure shows rates of heave in laboratory freezing tests on remolded soils. Because of the severity of the remolded laboratory test, the rates of heave shown in the figure are generally greater than may be expected under normal field conditions.

Frost susceptible soils have been classified into general groups (16):

- Gravels, crushed rock, sands, and similar materials exhibit little or no frost action when clean and free draining under normal freezing conditions.

- Silts are highly frost susceptible. The relatively small voids, high capillary potential/action, and relatively good permeability accounts for this characteristic.

- Clays are cohesive and, although their potential capillary action is high, their capillary rate is low. Although frost heaving can occur in clay soils, it is not as severe as for silts since the impervious nature of clays makes passage of water slow. The supporting capacity of clays must be reduced greatly during thaws, although significant heave has not occurred.

- Muck is an unsuitable material with a minimum of 15 percent organic material, in either natural subgrade, fill embankment, or cut sections and should be removed. Muck may be soil formed from decaying plant materials. Problems with highly organic soils are related to their extremely compressible nature. Those of relatively shallow depth, are often most economically excavated and replaced with suitable select material. Deeper deposits have been alleviated by placing surcharge embankments for preconsolidation with provisions on removal of water (20).

In using the pavement design procedures, it is understood to use the final material properties of the soils in construction as inputs for the design analysis. Therefore, the calculation of depth of frost penetration and suitable low frost susceptible soils must be performed prior to pavement design.

4.8 Sulfate Subgrade Soils

Sulfate induced problems in soils stabilized using calcium-based stabilizing agents such as lime and portland cement has been documented since the late 1950’s in the United States. A number of highly qualified cement chemists have studied the mechanism in an effort to understand and control sulfate attack on portland cement concrete structures. It is very important for the designers to understand the fundamentals of sulfate-induced distress and the risk levels when sulfate soils are stabilized with lime or with other calcium-based stabilizing agents.

Sulfates typically are concentrated closer to the surface in the drier, western regions. Moving eastward into wetter and more humid climates, the general rule is that if sulfates are present they tend to concentrate at deeper depths. For preliminary soil information, two valuable tools can be used to assess the presence and significance of sulfates within an area. These are the *United States*

Department of Agriculture's County Soils Report, and the "Web Soil Survey" developed by the Natural Resources Conservation Service (NRCS) of the United States Department of Agriculture. The "Web Soil Survey" is located at http://websoilsurvey.nrcs.usda.gov/app/ and allows the user to locate the construction job site, identify where sulfates typically occur, and determine the depth to expect significant concentrations.

The *County Soils Report* provides agricultural and engineering data for each soil. It is conveniently tabulated and generally shows the presence of gypsum and other sulfate salts, as well as, the depth of significant concentrations if any exist. This is an extremely valuable reconnaissance tool. It is very important not only to identify the presence of sulfates but also the depth of occurrence. For example, a soil may be essentially sulfate free in the upper 2 or 3 feet but have sulfate concentrations at a depth of 6 feet. In this case, sulfates would not be of concern during normal surface stabilization operations but could be of concern in cut and fill areas.

If sulfates are present and identified in the soils report, a field testing plan should be established with the Geotechnical Engineer. The frequency of testing depends on the level of sulfates present and the geological information for the region. If initial testing confirms the presence of sulfates in concentrations that may present problems, additional testing for the concentration of water-soluble sulfates may be warranted prior to recommending lime stabilization of the subgrade. Refer to *Chapter 200 of CDOT Field Materials Manual* for more information on sulfates.

4.9 Expansive Subgrade Soils

Soils that are excessively expansive should receive special consideration. One solution is to cover these soils with a sufficient depth of select material to overcome the detrimental effects of expansion. Expansive soils may often be improved by compaction at water contents over the optimum. In other cases, it may be more economical to treat expansive soils by stabilizing with a suitable stabilizing agent, such as lime (20).

One treatment of expansive soils is by performing the following subexcavation method. Subexcavate the expansive soil (dry dense unweathered shales and dry dense clays) and backfill with impermeable soil at 95 percent of maximum dry density at or above optimum moisture, in accordance with AASHTO Designation T 99. This treatment should carry through the cut area and transition from cut to fill until the depth of fill is approximately equal to the depth of treatment.

Table 4.9 Treatment of Expansive Soils is to be used as a guide to determine the depth of treatment as revised from Colorado Department of Highways Memo #323 (Construction) Swelling Soils, 1/5/1966. Projects on the interstate and National Highway System will require treatment of expansive soils. Treatment may take the form of subexcavation and replacing with impermeable soil, or subexcavate and recompact with moisture control of the same soil, see **Figure 4.20 Subexcavated Subgrade Layers**. Granular soils should not be used as backfill for subexcavation or replacement of expansive subgrade soils without a filter separator layer and edge drains to collect and divert the water from the pavement structure (26).

Table 4.9 Treatment of Expansive Soils

Plasticity Index	Depth of Treatment Below Normal Subgrade Elevation
10 – 20	2 feet
20 – 30	3 feet
30 – 40	4 feet
40 – 50	5 feet
More than 50	Placed in the bottom of the fills of less than 50 feet, or greater than 6 feet in height, or wasted.

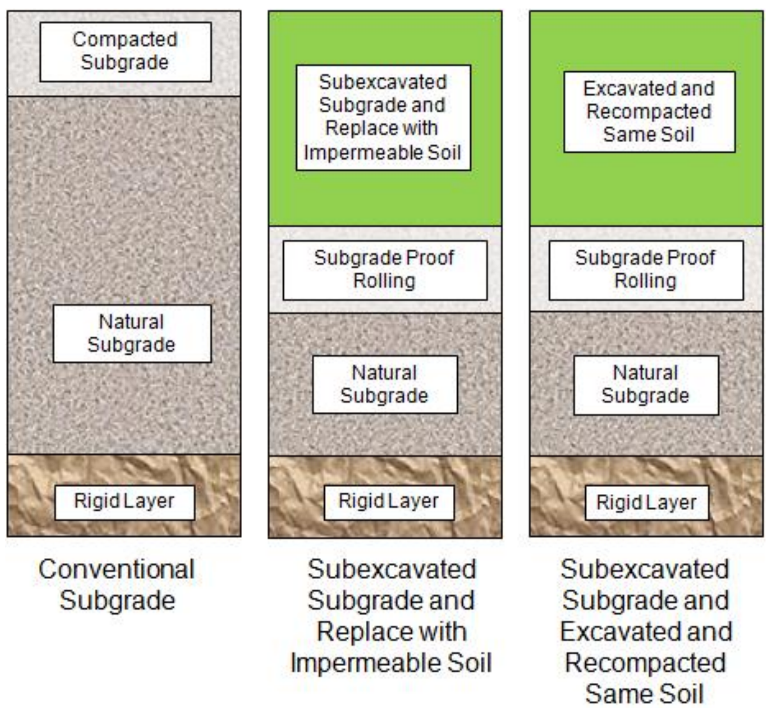


Figure 4.20 Subexcavated Subgrade Layers

The risk of swell potential is always a concern to the designer. The categories of the "swell damage risk" is shown in **Table 4.10 Probable Swell Damage Risk**. The designer should use **Table 4.10 Probable Swell Damage Risk** and **Table 4.9 Treatment of Expansive Soils** to decide the risk.

Table 4.10 Probable Swell Damage Risk

Swell (%)	Swell Pressure (psf, at 200 psf surcharge)	Probable Swell Damage Risk
0	0	None
0 - 1	0 - 1,000	Low
1 - 5	1,000 - 5,000	Medium
5 - 20	5,000 - 10,000	High
Over 20	Over 10,000	Very High

The Metropolitan Government Pavement Engineers Council (MGPEC) has published potential swell risk characterized by the driver's perception. Under the Section - Swelling Soils of the publication *Development of Pavement Design Concepts*, April 1998 (24) it documents the driver's perception concept. A driver's perception of a bump is directly related to the slope of the bump and perception of pavement roughness is related to the vehicle speed. A design criteria separation of below and above 35 mph was found to be an appropriate separation. Slopes representing the maximum allowable movement before causing discomfort to the driving public have been analyzed relating to vehicle speed. Streets with speeds less than 35 mph have a discomfort level of a 2 percent change. Higher speed streets and highways have a discomfort level of a 1 percent change. The slope of the heave is also related to the depth of the moisture treatment (subexcavation by means of excavate and recompact). **Figure 4.21 Effective Depth of Moisture Treatment** and **Figure 4.22 Recommended Depth of Moisture Treatment** graph the concept of slope of the bump and depth of recommended moisture treatment**. Figure 4.21 Effective Depth of Moisture Treatment** and **Figure 4.22 Recommended Depth of Moisture Treatment** use the percent swell to determine the depth of subgrade treatment. **Table 4.9 Treatment of Expansive Soils** uses the plasticity index to determine the depth of subgrade treatment. The designer should consider each method and know the field conditions to make a reasonable decision.

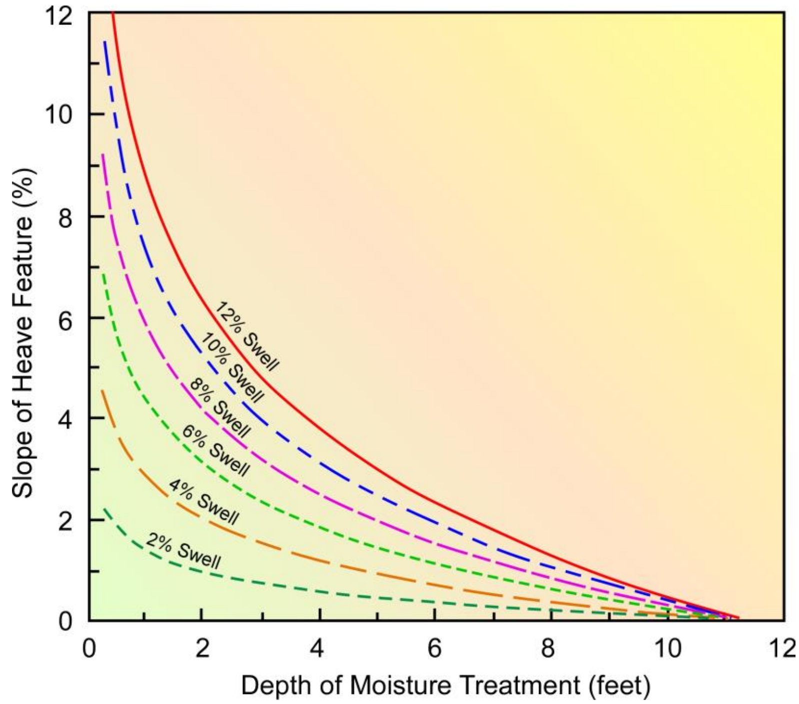


Figure 4.21 Effective Depth of Moisture Treatment

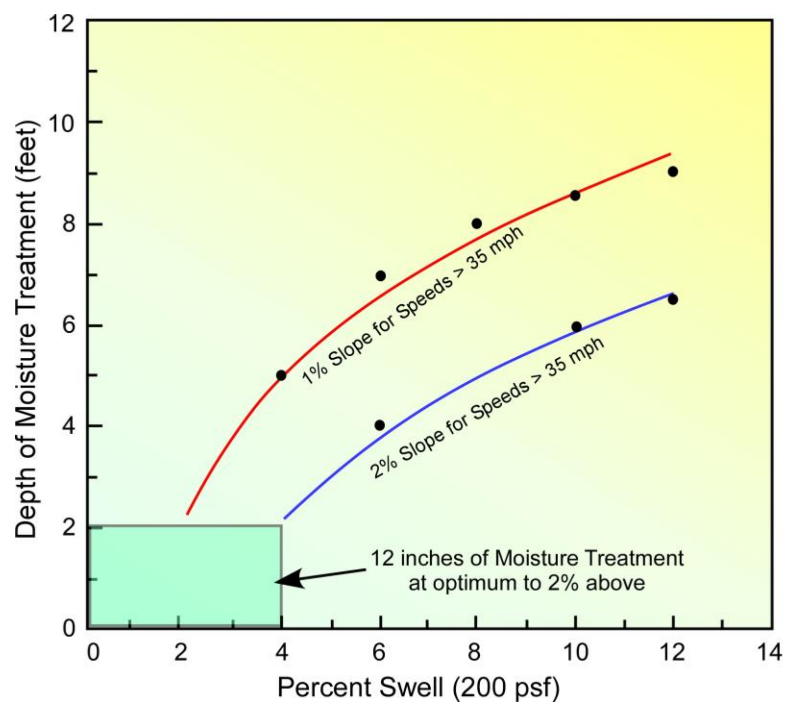


Figure 4.22 Recommended Depth of Moisture Treatment

4.10 Stabilizing Agents

The strength and stability of all subgrade soils improve with compaction. For certain subgrade soils, the strength gained even after compaction may not be adequate. Similarly, silty and clayey subgrade soils may be collapsible or expansive in nature, and thus not suitable for pavement construction. Stabilization of soils is an effective method for improving the properties of soil and pavement system performance. Mechanical stabilization is the process in which the properties of subgrade soils are improved by blending and compacting the soils without the use of admixtures or stabilizing agents. Unstable and expansive subgrade soils may be stabilized through chemical stabilization; many stabilizing agents may be effective by improving the in-lace soil properties rather than removing and replacing material or increasing base thickness. The objective of stabilizing agents is to increase the strength and stiffness, improve workability and constructability, and reduce the plasticity index (PI) and swell potential for expansive clays. Availability or financial considerations may be the determining factor in which a stabilizing agent is used.

Approved stabilizing agents are asphalt, lime, lime/fly ash, fly ash, portland cement, and approved chemical stabilizers. Other agents may be used with prior approval of CDOT. The approved stabilizing agents are combined with selected aggregate or soils, or with native materials to improve their stability and strength as load carrying elements of structural sections. The type and amount of stabilizing agent should be developed from tests of available materials, followed by cost comparisons against untreated materials.

Lime generally performs better on fine-grained materials, cement on coarse-grained soils, and fly ash performs well mostly on silty sands. Cement also provides highly effective clay stabilization, usually with the added benefit of higher strength gain, but quality control may be difficult. The following chart, **Figure 4.23 Lime/Cement Stabilization Flow Chart**, provides a good estimate of the lime and cement for a certain soil type dependent upon gradation and plasticity index.

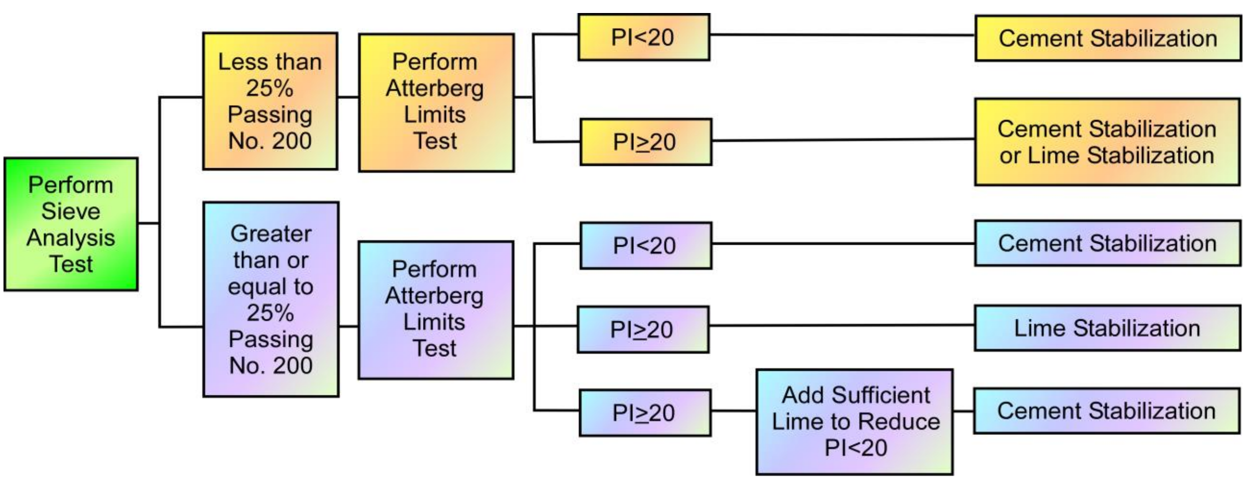


Figure 4.23 Lime/Cement-Stabilization Flow Chart

4.10.1 Lime Treated Subgrade

When swell potential as determined by ASTM D 4546 is found to be greater than 0.5 percent using a 200 psf surcharge, stabilization should be used per *CDOT Standard Specification for Road and Bridge Construction, 2011* specification book, Table 307-1. If the R-value of the subgrade soil is greater than 40, the use of a base layer is not recommended in the structural layering of a potential swelling soil. Soil with a plasticity index of more than 50 should be placed in the bottom of the fills of less than 50 feet in height, or wasted. The backfill soil should be uniform and all lenses or pockets of very high swelling soil should be removed and replaced with the predominant type of soil that has a plasticity index less than 50. If removal is not practical or subgrade soils were determined to have a plasticity index greater than 10, in-place treatment such as a lime-treated subgrade is recommended. A subgrade proposed for lime treatment should be investigated for sulfates. In some cases, such as construction over a rocky subgrade or when having to maintain traffic over a widened section, an aggregate base may be desirable.

Lime treated subgrade consists of blending the existing subgrade material with a minimum of 3 percent lime by weight per design, to the specified depth and compaction (see **Figure 4.24 Lime Treated Structural Subgrade Layer)**. Lime may be either quicklime or hydrated lime, shall conform to the requirements of ASTM C 977 along with a rate of slaking test for quicklime in accordance with ASTM C 110, and shall be the product of a high-calcium limestone as defined by ASTM C 51. The use of dolomitic or magnesia quicklime with magnesium oxide contents in excess of 4 percent, carbonated hydrated lime, and lime kiln dust or cement kiln dust shall not be allowed unless approved by the RME.

Some soils, when treated with lime, will form cementitious compounds resulting in a relatively high strength material. Lime reduces the ability of clays to absorb water, thus increasing internal friction and shear strength. Lime provides greater workability by changing the clays into friable sand-like material and reduces the plasticity index and swell potential.

The designer should test the soil for the concentration of water-soluble sulfates prior to recommending lime stabilization of the subgrade. Water-soluble sulfate content should be less than 0.2 percent by mass. Sulfate content greater than 0.2 percent can cause an adverse reaction among the lime, soil, sulfate ions, and water. This can lead to loss of stability and cause swelling or heave. Additionally, excessive lime in the subgrade can create leaching of calcium into the ground water. For more information, see *Chapter 200 of the CDOT Field Materials Manual*.

Additional treatment of the natural subgrade may be needed. If lime treatment depth seems to be too thick to be practical, the swell potential subgrade may need to be excavated and recompacted to a depth as shown in **Table 4.9 Treatment of Expansive Soils.** The recompaction shall be at 2 ± 1 percent above optimum moisture control, see **Figure 4.24 Lime Treated Structural Subgrade Layer. Figure 4.25 Cross Section of Lime Treated Cut Section Subgrade** shows the extent of the subexcavation excavated and recompacted treatment, or moisture treatment in cross sectional view.

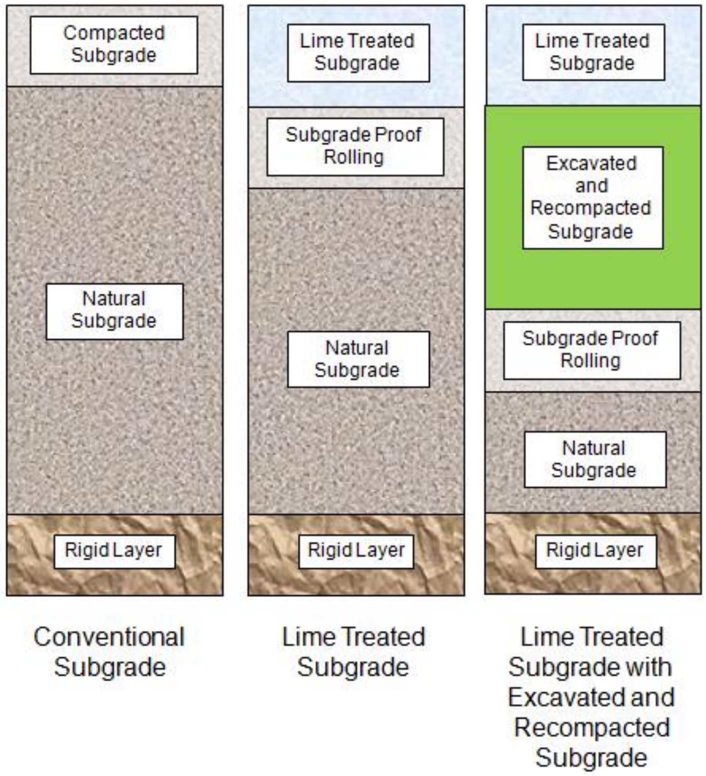


Figure 4.24 Lime Treated Structural Subgrade Layer

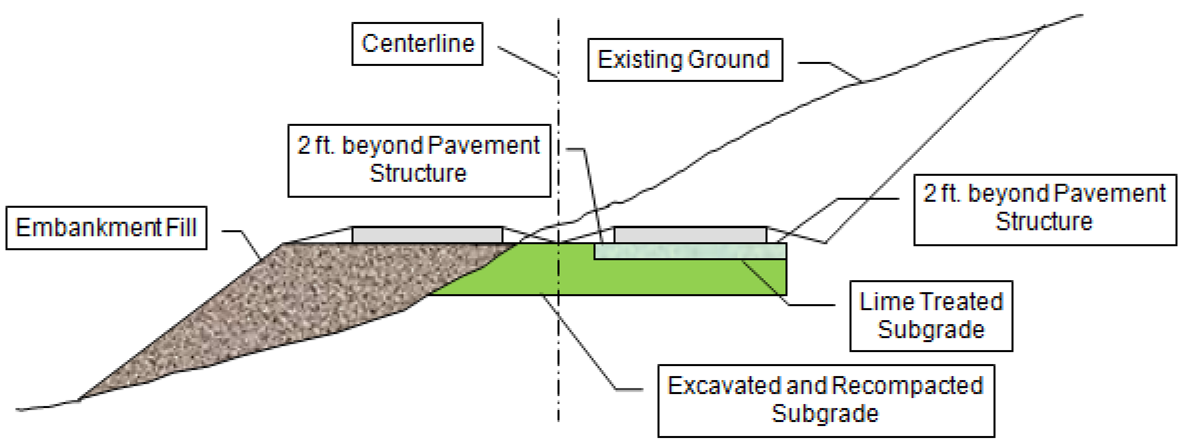


Figure 4.25 Cross Section of Lime Treated Cut Section Subgrade

4.10.2 Cement Treated Subgrade

Cement is typically used to stabilize fine and coarse grained sands and low plastic index clays where the plasticity index is less than 20, see **Figure 4.26 Cement Treated Structural Subgrade Layer**. Cement treated subgrade will have higher unconfined strength, reduced permeability will inhibit leaching, and can rapid set within two hours of the subgrade being treated. Normal percentages used in cement treated subgrade are from 2 to 15 percent by weight.

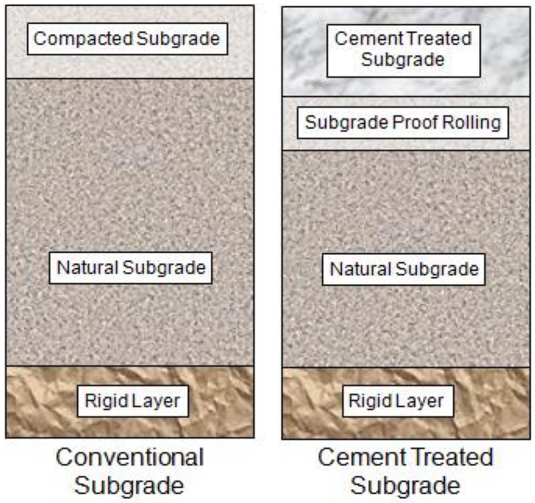


Figure 4.26 Cement Treated Structural Subgrade Layer

4.10.3 Fly Ash and Lime/Fly Ash Treated Subgrade

CDOT recommends the use of Class C fly ash as a stabilizing agent due to its calcium content. It can be used in sands and clays with low plasticity indices and at percentages of up to 25 percent. Fly ash percentages in the subgrade of greater than 25 percent can lead to a decrease in density and durability issues. Fly ash treated subgrade will typically experience increased unconfined compressive strengths similar to lime, as well as, increased sand maximum densities (see **Figure 4.27 Fly Ash Treated Structural Subgrade Layer)**.

When used, the typical lime/fly ash content of a mixture ranges from 12 to 30 percent with lime to fly ash ratios of 1:3 to 1:4 being common. Class C fly ash is recommended for these mixtures, however, the designer may use high carbon Class C fly ash for soil stabilization.

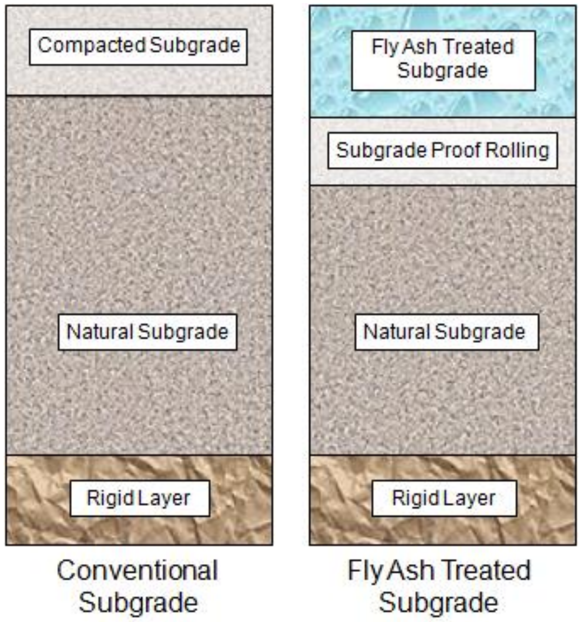


Figure 4.27 Fly Ash Treated Structural Subgrade Layer

4.11 Geosynthetic Fabrics and Mats

4.11.1 Introduction

Geosynthetic fabrics and mats can be used as reinforcement in a variety of ways within and below the pavement section. Anytime poor or marginally acceptable in-situ soils are encountered, geosynthetic fabrics and mats should be considered. CDOT Soils and Rockfall Program personnel are available to help in the selection of the most appropriate product. Technical representatives for individual brand materials are also available.

Listed below are conditions for an in-situ subgrade where a geosynthetic may be used as a viable alternative. The listing and **Table 4.11 Application and Associated Functions of Geosynthetics in Roadway Systems** are from the publication *FHWA-NHI-07-092, Geosynthetic Design & Construction Guidelines Reference Manual, August 2008, Chapter 5.0* (33).

- Poor soils
  - USCS: SC, CL, CH, ML,MH, OL, OH and PT
  - AASHTO: A-5, A-6, A-7-5 and A-7-6

- Low undrained shear strength
  - Shear strength = τ_f = c_u < 2,000 psf (90 kPa), c_u is the undrained strength
  - CBR < 3 (**Note**: soaked saturated CBR as determined with ASTM D 4429)
  - R-value (California) ≈ < 20
  - M_r ≈ < 4,500 psi (30 MPa)

- High water table
- High sensitivity: dynamic disturbance results in viscous flow.

Table 4.11 Application and Associated Functions of Geosynthetics in Roadway Systems shows additional guidance of when and how geosynthetics can be used as a separator, stabilizer, base reinforcement, or drainage material. For additional information on material use and approved products, the CDOT Materials Bulletin dated January 25, 2008, clarifies the terminology and application of geosynthetics (32).

Table 4.11 Application and Associated Functions of Geosynthetics in Roadway Systems
(Table 5.1 *FHWA-NHI-07-092, Geosynthetic Design & Construction Guidelines Reference Manual*, August 2008)

Application	Function(s)	Subgrade Strength	Qualifier
Separator	Separation Secondary: filtration[1]	2,000 psf $\leq c_u \leq$ 5,000 psf (90 kPa $\leq c_u \leq$ 240 kPa) 3 $\leq$ CBR $\leq$ 8 4,500 psi $\leq M_r \geq$ 11,600 psi (30 MPa $\leq M_r \geq$ 80 MPa)	Soils containing high fines A-1-b, A-2-4, A-2-5, A-2-6, A-4, A-5, A-6, A-7-5, A-7-6
Stabilization	Separation, filtration and some reinforcement (especially CBR < 1) Secondary: separation	c_u < 2,000 psf (90) kPa CBR < 3 M_r < 4,500 psi (30 MPa)	Wet, saturated fine grained soils (i.e. silt, clay, and organic soils)
Base Reinforcement	Reinforcement Secondary: separation	600 psf $\leq c_u \leq$ 5,000 psf (30 kPa $\leq c_u \leq$ 240 kPa) 3 $\leq$ CBR $\leq$ 8 1,500 psi $\leq M_r \geq$ 11,600 psi (10 MPa $\leq M_r \geq$ 80 MPa)	All subgrade conditions, reinforcement located within 6 to 12 inches of pavement
Drainage	Transmission and filtration Secondary: separation	Not applicable	Poorly drained subgrade
Note: [1] Always evaluate filtration requirements.			

4.11.2 Separator Layer

If coarse, open-graded base or subbase courses are used, it may be necessary to provide a means for preventing the intrusion of the underlying fine-grained roadbed soils. Historically preventive measures usually consist of providing a layer of suitable material to act as a barrier between the roadbed soils and the susceptible subbase or base. An engineered aggregate layer serves this purpose. To ensure the gradation of the separator layer will prevent subgrade fines from migrating up, the following criteria are imposed (20, 22). Equation **Eq. 4.5** may be referred to as the piping ratio.

$$\mathbf{D_{15B} \leq 5 \times D_{85S}} \qquad \textbf{Eq. 4-5}$$

$$\mathbf{D_{50B} \leq 25 \times D_{50S}} \qquad \textbf{Eq. 4-6}$$

Where:

D_{15B} = particle size wherein 15 percent of the base or subbase course particles are smaller than this size

D_{85S} = particle size wherein 85 percent of the roadbed soil particles are smaller than this size

D_{50B} = particle size wherein 50 percent of the base or subbase course particles are smaller than this size

D_{50S} = particle size wherein 85 percent of the roadbed soil particles are smaller than this size

Separation fabrics used to separate fine grain silts and clays from open-graded drainage mats and subbase/base materials are an especially valuable and cost-effective application. Without them, a soft subgrade could inundate the open void spaces of drainage mats and base courses, thereby decreasing their strength and ability to drain.

4.12 Material Sampling and Testing

Sampling involves coring the existing pavement to determine layer thicknesses, permit visual inspection of the subsurface condition, and obtain material samples of unbound layers for further testing. For an existing pavement, the types of tests performed on the extracted materials should depend on the type of distress(s) observed. Contact the Region Materials Engineer and see Chapter 200 of the *Field Materials Manual* for information on recommended sampling intervals and further guidance on available material test methods.

References

1. *Guidelines for Stabilization of Soils Containing Sulfates*, Technical Memorandum, by Austin White Lime, Chemical Lime, Texas Lime, distributed by National Lime Association, Arlington, VA, August 2000.

2. Hunter, D., *Lime-Induced Heave in Sulfate-Bearing Clay Soil*, ASCE Journal of Geotechnical Engineering Vol. 114 Issue 2, 1988 pp. 150-167.

3. Little, D. N., *Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime*, National Lime Association, Kendall/Hunt Publishing Company, Dubuque, Iowa, 1995.

4. Hopkins, T. C., T. L. Beckham, L. Sun, B. Ni and B. Butcher, *Long-Term Benefits of Stabilizing Soil Subgrades*, Kentucky Transportation Center, College of Engineering, University of Kentucky, Lexington, Kentucky, 2002.

5. Copas, T. L. and H. A. Pennock, *Lime- Fly Ash-Stabilized Bases and Subbases*, NCHRP Synthesis of Highway Practice 37, TRB, National Research Council, Washington, D.C., 1976.

6. McManis, K. L. and A. Arman, *Class C Fly Ash as a Full or Partial Replacement for Portland Cement or Lime*, Transportation Research Record 1219, TRB, National Research Council, Washington, D.C., 1989, pp. 68-81.

7. Nicholson, P. G., V. Kashyap, and C.F. Fujii, *Lime and Fly Ash Admixture Improvement of Tropical Hawaiian Soils*, Transportation Research Record 1440, TRB, National Research Council, Washington, D.C., 1994, pp. 71-78.

8. Petry, T. M., *Performance-Based Testing of Chemical Stabilizers*, Transportation Research Record 1589, TRB, National Research Council, Washington, D.C., 1997, pp. 36-41.

9. Prusinski, J. R., and S. Bhattacharja, *Effectiveness of Portland Cement and Lime in Stabilizing Clay Soils*, Transportation Research Record 1652, TRB, National Research Council, Washington, D.C., 1999, pp. 215-227.

10. Parsons, R. L., and J. P. Milburn, *Engineering Behavior of Stabilized Soils*, Transportation Research Record 1837, TRB, National Research Council, Washington, D.C., 2003, pp. 20-29.

11. *1972 Roadway Design Manual*, Colorado Department of Transportation.

12. *1980 Roadway Design Manual*, Colorado Department of Transportation.

13. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2005.

14. Yeh, Shan-Tai, and Su, Cheng-Kuang, *Resilient Properties of Colorado Soils*, Final Report CDOH-DH-SM-89-9, Colorado Department of Highways, December 1989.

15. Chang, Nien-Yin, Chiang, Hsien-Hsiang, and Jiang, Lieu-Ching, *Resilient Modulus of Granular Soils with Fines Contents*, Final Research Report CDOT 91-379, Colorado Department of Transportation, May 1994.

16. *Guide for Mechanistic-Empirical Design*, Final Report, NCHRP Project 1-37A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Submitted by ARA, INC., ERES Consultants Division, Champaign, IL, March 2004.

17. *CDOT Field Materials Manual*, Colorado Department of Transportation, 2004.

18. *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC, 1993.

19. Supplement to the AASHTO Guide for Design of Pavement Structures, Part II, - Rigid Pavement Design & Rigid Pavement Joint Design, American Association of State Highway and Transportation Officials, Washington, DC, 1998.

20. *AASHTO Interim Guide for Design of Pavement Structures 1972*, published by American Association of State Highway and Transportation Officials, Washington, DC, 1972.

21. DRIP 2.0 Software Program <http://www.fhwa.dot.gov/pavement/software.cfm>

22. Mallela, Jagannath, Larson, Gregg, Wyatt, Tim, Hall, Jim, and Barker, Walter, *User's Guide for Drainage Requirement In Pavements- Drip 2.0 Microcomputer Program*, ERES Consultants, Division of Applied Research Associates, Inc., 505 W. University Avenue, Champaign, IL 61820 and prepared for Federal Highway Administration, 400 Seventh Street, SW, Washington, DC 20590, July 2002.

23. *LTPP Data Analysis Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*, Publication FHWA-RD-96-198, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, January 1997.

24. *Development of Pavement Design Concepts*, prepared by CTL/Thompson, Inc., Consulting Engineers, 1971 West 12th Ave, Denver, CO 80204 and prepared for the Metropolitan Government Pavement Engineers Council, April 1998.

25. *FHWA Reclaimed Asphalt Concrete - User Guideline - Granular Base*, <http://www.fhwa.dot.gov/research> (1/25/2006).

26. Ardani, Ahmad, *Expansive Soil Treatment Methods in Colorado*, Final Report CDOT-DTD-R-92-2, Colorado Department of Transportation, March 1992.

27. Colorado Department of Highways Memo #323, (Construction) Swelling Soils, 1/5/1966.

28. *Basic Asphalt Recycling Manual*, Federal Highway Administration, 400 Seventh Street, SW, Washington, DC 20590 and Asphalt Recycling and Reclaiming Association, #3 Church Circle, PMB 250, Annapolis, MD 21401, 2001.

29. *Long-Term Pavement Performance Information Management System: Pavement Performance Database User Reference Guide*, Publication No. FHWA-RD-03-088 (Interim Update), Federal Highway Administration, Research, Development and Technology, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, September 2005.

30. Long-Term Pavement Performance (LTPP) Data Analysis Support: National Pooled Fund Study TPF-5(013), Effects of Multiple Freeze Cycles and Deep Frost Penetration on Pavement Performance and Cost, Publication No. FHWA-HRT-06-121, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, November 2006.

31. *Engineering and Design Pavement Criteria for Seasonal Frost Conditions, Mobilization Construction*, Engineering Manual EM1110-3-138, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, Washington, D.C. 20314, April 9, 1984.

32. CDOT Materials Bulletin, 2008 Number 1, Geosynthetics and Geotextiles, Project Development Branch (for Materials and Geotechnical Branch), January 25, 2008.

33. *FHWA-NHI-07-092, Geosynthetic Design & Construction Guidelines Reference Manual*, NHI Course No. 132013, National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., August 2008.

34. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2011. http://www.coloradodot.info/business/designsupport/construction-specifications/2011-Specs

35. *AASHTO Mechanistic-Empirical Pavemnt Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

CHAPTER 5
GRANULAR AND TREATED BASE MATERIALS

5.1 Bases

A base course is a layer of material beneath the pavement's surface course. The design and construction of a pavement structure may include one or more base courses and is constructed on the subbase course, or, if no subbase is used, directly on the natural subgrade. It may be used in various combinations to design the most economical structural section for a specific project. Bases should be non-erodible, especially under rigid pavements, and may be constructed of gravels, mixtures of soil and aggregate, mixtures of asphalt and aggregate, mixtures of cement and aggregate or soil, or other innovative materials. Bases may be made of unbound materials, such as gravel, or bound materials, such as lime treated subgrade (17).

5.2 Sampling Base Materials During a Soil Survey Investigation

Base and subbase material samples are collected for information and testing during the soil survey investigation. The purpose of material sampling is to gather information for the design of pavement rehabilitation and/or new pavement structure. Follow the steps described in **Section 4.2 Soil Survey Investigation** for conducting soil survey investigations.

During the investigation, collect base and subbase samples for the following information and testing:

- **Thickness**
- **Gradation:** CP 21, PI and LL (AASHTO T 89 and T 90)
- **Resistance Value:** T 180 and L 3102
- **Fill All Sample Holes:** provide and place patching material similar to the existing surface.
- **Combine:** similar soil and aggregate types encountered; note locations and depths.

5.3 Aggregate Base Course (ABC)

Aggregate base is normally specified as the lowest element of any structural section because it generally results in the most economical design. It may consist of more than one layer, see **Figure 5.1 Unbound Aggregate Base Course Layers**.

Aggregate base courses under flexible pavements provide a significant increase in structural capacity. Pavement design of flexible pavement depends on the wheel loads being distributed over a greater area as the depth of the pavement structure increases. Thick granular layers aim to improve the natural soil subgrade foundation of weak, fine-grained subgrades and are generally greater than 18 inches thick (16). Added benefits include improved drainage by preventing the accumulation of free water, protection against frost damage, preventing intrusion of fine-grained roadbed soils in base layers, providing a uniform underlying surface course support, and providing a construction platform.

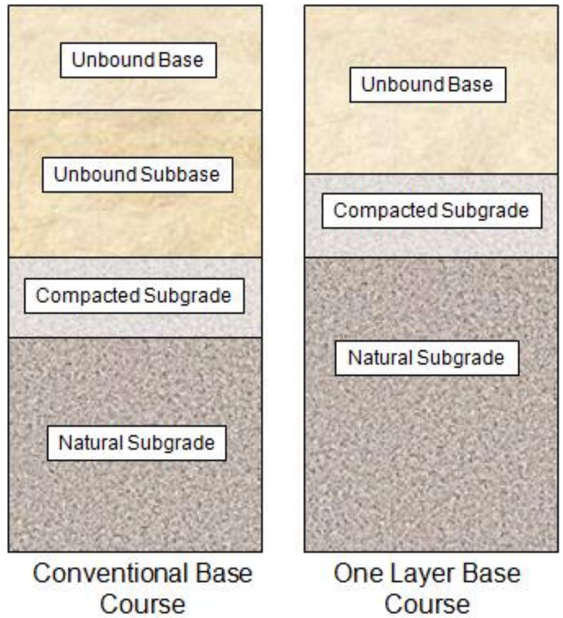


Figure 5.1 Unbound Aggregate Base Course Layers

Subbase layers are usually distinguished from the base course layers by less stringent specification requirements for strength, plasticity, and gradation. Because the subbase course must be of significantly better quality than the roadbed soil, the subbase is often omitted if roadbed soils are of high quality. When the roadbed soils are of relatively poor quality and the design procedure indicates the requirement for substantial thickness of pavement, alternate designs should be prepared for structural sections with and without a subbase. A selection may be made based on availability and relative costs for a base and subbase (20). Unbound subbase layers may be pit-run gravels comprised of rounded rock, sand, and soil mixture. Typically, sand or granular materials, or course grained materials with limited fines, corresponding to AASHTO A-1 and A-2 soils may be used. California Bearing Ratio (CBR) and/or resilient modulus testing may measure strength and stiffness of the subbase. Subbases having strengths and stiffness of CBR values 6 percent or greater, corresponding resilient moduli (M_r) of approximately 8,000 psi, R-value of 50, or structural coefficient (a_3) of 0.06 would be designated as an aggregate subbase material.

A CDOT base's M_r may range from 20,000 to 48,675 psi. Slight differences of the suggested values can be found in charts, graphs, and correlation tables of other publications. CDOT Aggregate Base Course Class 1, 2 or 3 would be classified as a subbase. Class 1 and 2 are more restrictive because of the sieve sizing than Class 3 (pit-run). Aggregate base courses Class 4 and Class 6 limit the fines from 3 to 12 percent passing the No. 200 sieve. When the gradation approaches the 12 percent passing, the base becomes impermeable, and as such, when the gradation approaches the 3 percent limit they tend to be more permeable.

Aggregate base courses under rigid pavements provide a drainage layer, protection against frost damage, uniform, stable, permanent support, and support for the heavy equipment used during rigid pavement placement, and reduce pumping. There is some increase in structural capacity when a base is placed under a rigid pavement, but typically not a significant amount (17). Bases provide uniform support of rigid pavements across the joints and under the entire slab. A nonerodable base is most desirable. To limit pumping of fines through the joints, a good base course gradation such as an Aggregate Base Course (Class 6) limits the fines from 3 to 12 percent passing the No. 200 sieve. The base course is considered a structural layer of the pavement along with the concrete slab, thus its thickness and modulus are important design values (19).

Aggregates for bases should be crushed stone, crushed slag, crushed gravel, natural gravel, or crushed reclaimed concrete or asphalt material and shall conform to the requirements of Section 703.03 of *CDOT Standard Specifications for Road and Bridge Construction* and **Table 5.1 CDOT Classification for Aggregate Base Course** for reclaimed asphalt pavement and quality requirements of AASHTO M 147. Placement and compaction of each lift layer shall continue until a density of not less than 95 percent of the maximum density determined in accordance with AASHTO T 180 has been achieved (13). FHWA also recommends using only crushed aggregates in the unbound base layer to maintain good mechanical interlock. The design thickness should be rounded up to the next 1.0 inch increment.

Table 5.1 CDOT Classification for Aggregate Base Course

Sieve Size	Mass Percent Passing Square Mesh Sieves						
	LL Not Greater Than 35			LL Not Greater Than 30			
	Class 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7
6"			100				
4" (100 mm)		100					
3" (75 mm)		95-100					
2 1/2" (60 mm)	100						
2" (50 mm)	95-100			100			
1 1/2" (37.5 mm)				90-100	100		
1" (25 mm)					95-100		100
3/4" (19 mm)				50-90		100	
#4 (4.75 mm)	30-65			30-50	30-70	30-65	
#8 (2.36 mm)						25-55	20-85
#200 (75 µm)	3-15	3-15	20 max.	3-12	3-15	3-12	5-15

NOTE: Class 3 material shall consist of bank or pit-run material.

5.3.1 Unbound Layer Characterization in M-E Design

The unbound layer characterization in M-E Design is similar to that of subgrade characterization. The inputs required for unbound layer characterization are the resilient modulus and other physical/engineering properties such as soil classification, moisture content, dry density, saturated hydraulic conductivity, etc., and follows the same guidelines used in subgrade material characterization. **Note:** M-E Design prefers to have a minimum of three unbound layers for a successful design.

- New Flexible an JPCP: **Table 4.2 Recommended Subgrade Inputs in New Flexible and JPCP Designs.**
- HMA Overlays of Existing Flexible Pavement: **Table 4.3 Recommended Subgrade Inputs for HMA Overlays of Existing Flexible Pavement.**
- Overlays of Existing Rigid Pavement: **Table 4.4 Recommended Subgrade Inputs for Overlays of Existing Rigid Pavement.**

The design M_r of the aggregate base and subbase layers must be adjusted for limiting modulus criteria and modified accordingly. This check is necessary to avoid decompaction and build-up of tensile stresses in the unbound layers.

The M_r of the unbound material in each layer is a function of the layer thickness and the modulus of the next underlying layer (including subgrade layers). **Note:** The unbound materials are stress-dependent; the M_r value decreases with increasing depth as the induced stresses attenuate. Therefore, to avoid decompaction, the M_r of the aggregate base and subbase layers should not exceed the limiting modulus criteria determined using **Figure 5.2 Limiting Modulus Criteria of Unbound Aggregate Base Layers** and **Figure 5.3 Limiting Modulus Criteria of Unbound Subbase Layers**. The *AASHTO Interim MEPDG Manual of Practice* recommends the design M_r value of the unbound material be capped at the corresponding limiting modulus.

Using **Figure 5.2 Limiting Modulus Criteria of Unbound Aggregate Base Layers** and **Figure 5.3 Limiting Modulus Criteria of Unbound Subbase Layers** involves entering the graph with a known value of the modulus of the lower layer and the thickness of the next overlying layer. The figures limit the maximum values of 100,000 psi and 40,000 psi for base and subbase course materials, respectively.

> Example**:** If the M_r of the underlying subgrade layer is 10,000 psi and the thickness of the overlying subbase layer is 8 inches, the M_r of the overlying layer is limited to approximately 28,500 psi.

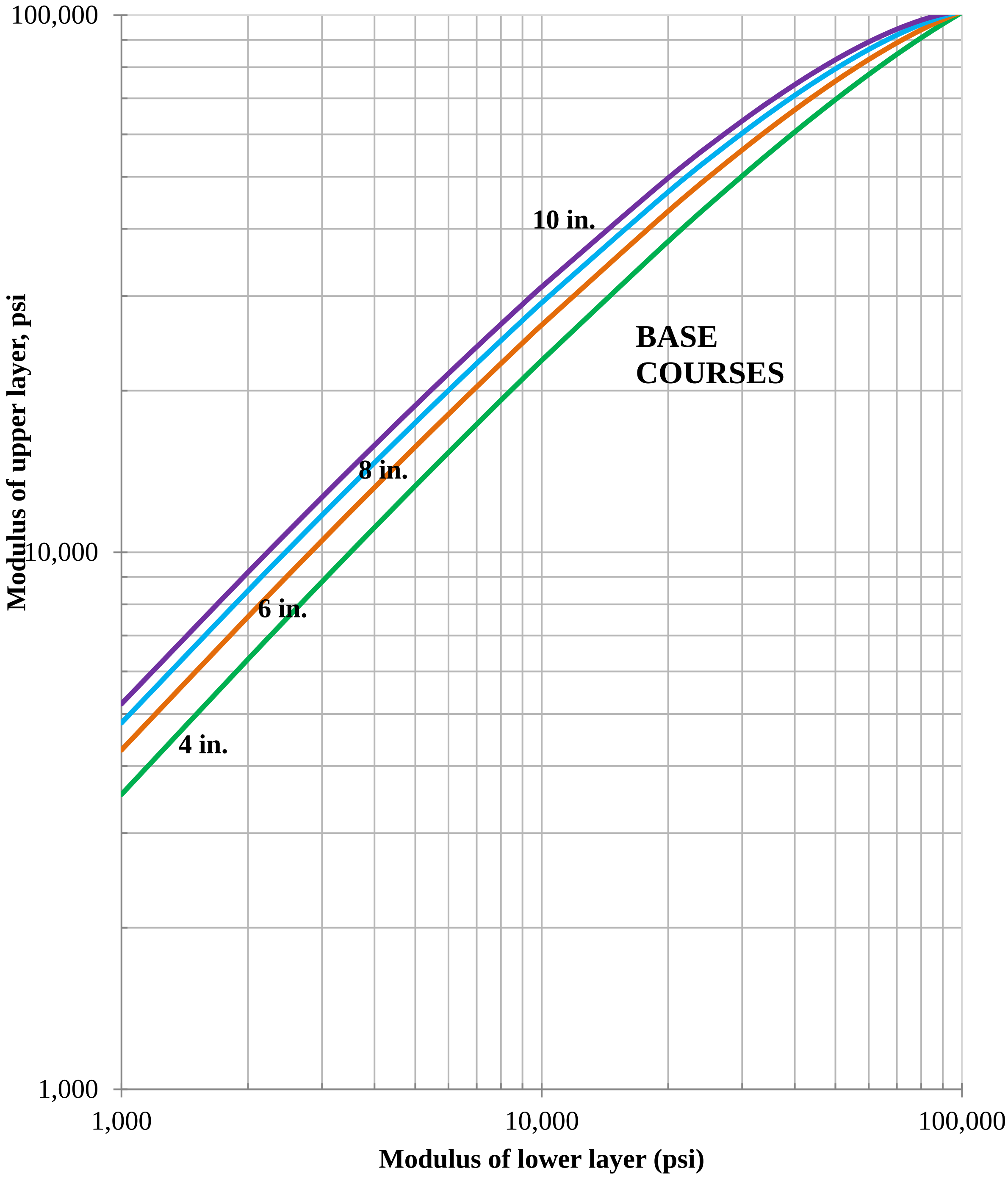


Figure 5.2 Limiting Modulus Criteria of Unbound Aggregate Base Layers

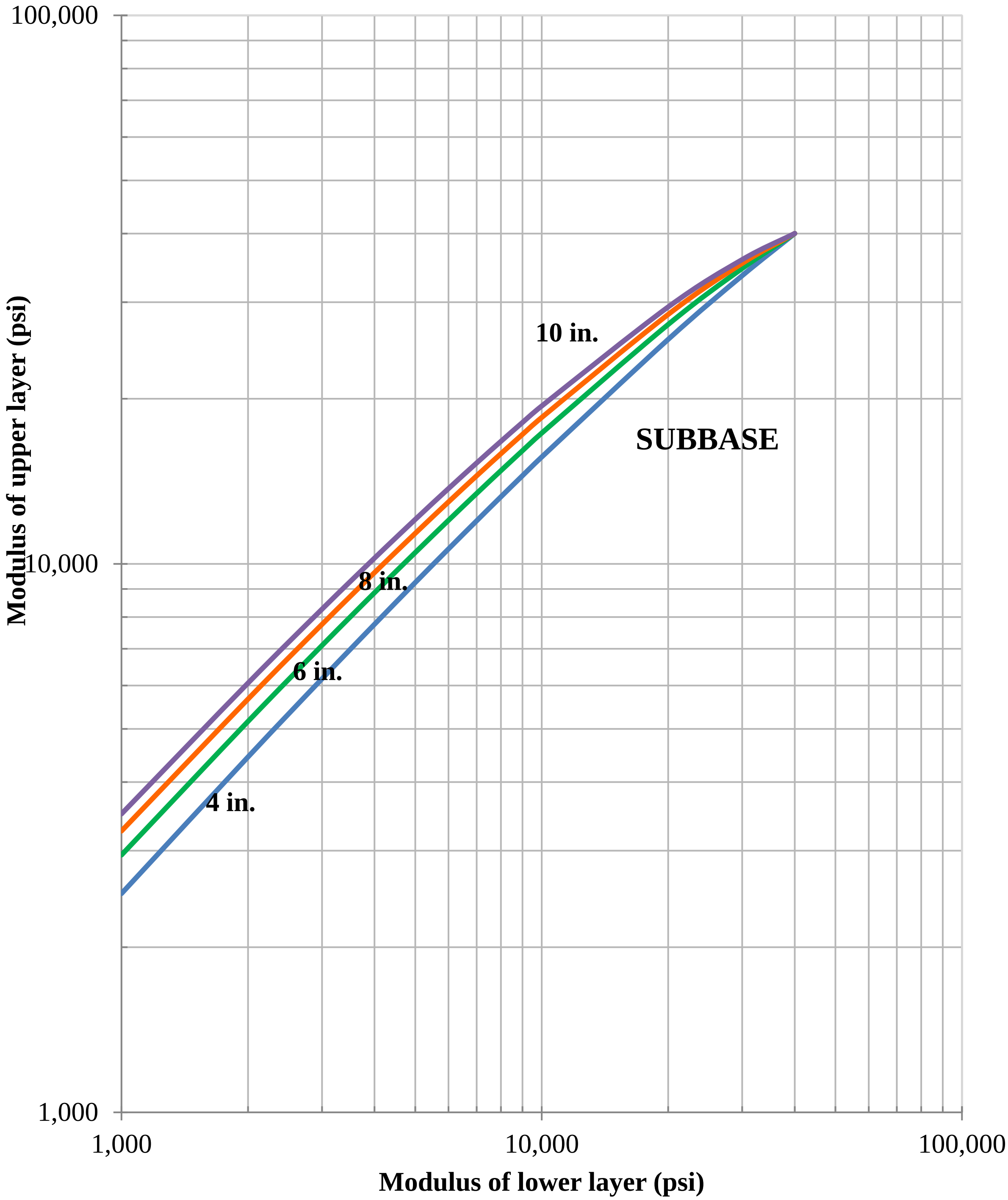


Figure 5.3 Limiting Modulus Criteria of Unbound Subbase Layers

5.3.2 Modeling Unbound Aggregate Base Layers in M-E Design Software

To properly characterize unbound layers for M-E Design, the designer should consider the following:

- **Modeling Thick Aggregate Bases**

  - When a thick granular aggregate base (more than 12 inches) is used, the top 8 or 10 inches is modeled as an aggregate base layer, while the remaining aggregate is modeled as Subgrade Layer 1. The M_r and other physical/ engineering properties remain the same for both layers. The compacted or natural subgrade below the thick aggregate base is modeled as lower subgrade layers as appropriate.

- **Modeling Thin Aggregate Bases**

  - If a thin aggregate base layer is used between two thick unbound materials, the thin layer should be combined with the weaker or lower layer.

  - When similar aggregate base and subbase materials are combined, the material properties of the combined layer should be those from the thicker layer.

    - Averaging the material properties is <u>not</u> recommended.

    - When similar materials have about the same thickness, the material with the lower modulus value should be used.

- **Limiting Modulus Criteria**

  - The designer must make sure the M_r of the unbound layer does not exceed the limiting modulus determined using **Figure 5.2 Limiting Modulus Criteria of Unbound Aggregate Base Layers** and **Figure 5.3 Limiting Modulus Criteria of Unbound Subbase Layers**.

- **Stabilized Layer**

  - Granular base materials treated with a small amount of stabilizers, such as asphalt, emulsion, cement, lime, or other pozzolanic materials for constructability reasons should be defined as an unbound layer or combined with other unbound layers, as necessary.

  - Per Applied Research Associates, Inc., since Colorado has no calibration coefficient, one **<u>should not</u>** use a stabilized layer. Rather the designer should treat the layer as a high quality subbase or base course with a constant modulus.

- **Soil Aggregate Materials**
    - Sand and other soil-aggregate materials should be defined separately from crushed stone or crushed aggregate base materials.

5.4 Treated Base Course

The use of bases in the design of rigid pavements is a function of the pavement material's structural quality characterized by the modulus of rupture and elastic modulus. In comparison to the strength of the concrete slab, the structural contributions of the underlying layers are relatively small. Treated or untreated bases can be used under rigid pavements, but is not mandatory. **Figure 5.4 Stabilized Treated Structural Base Layers** shows several materials historically used by CDOT as bases.

- **Treated Bases** under flexible pavements are similar to rigid pavements, as such the structural capacity is increased while decreasing the flexible pavement's thickness. These bases are used to strengthen a weak subgrade and are another design tool in the layering system where lower quality materials are in the bottom courses.

- **Plant Mix Bituminous Base (PMBB)** is composed of a mixture of aggregate, filler (if required), hydrated lime, and bituminous material. The aggregate and bituminous materials are mixed at a central batch plant. Several aggregate fractions are sized, uniformly graded, and combined in such proportions that the resulting composite blend meets the job-mix formula. PMBB is a very good non-erodible base.

- **Emulsified Asphalt Treated Base (EATB)** is composed of a mixture of aggregate, water (if required), and emulsified asphalt. The aggregate and emulsified asphalt is mixed at a central batch plant and the aggregates are specified per the classification of an aggregate base course. In certain instances subgrades may be used if they are sandy and do not have an excessive amount of material finer than the No. 200 sieve. Placement and spreading is by approved spreading devices capable of achieving specified surface tolerances and a compaction not less than 95 percent of AASHTO T 180.

- **Cement Treated Base (CTB)** is a mixture of aggregate and portland cement. The aggregate is obtained from scarifying the existing roadway and shall meet specified gradation. Mixing is accomplished by means of a mixer that will thoroughly blend the aggregate with the cement. The mixer is equipped with a metering device that will introduce the required quantity of water during the mixing cycle. Another option is to have the aggregate proportioned and mixed with cement and water at a central batch plant. Compaction is not less than 95 percent of AASHTO T 134.

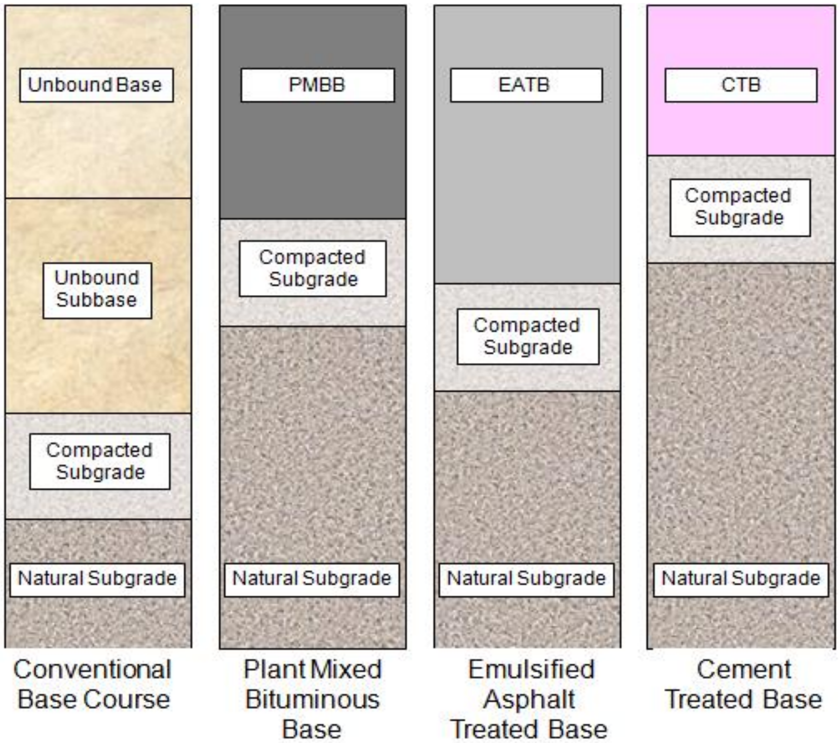


Figure 5.4 Stabilized Treated Structural Base Layers

5.4.1 Characterization of Treated Base in M-E Design

Treated base materials include lean concrete, cement stabilized, open-graded cement stabilized, soil cement, lime-cement-fly ash, and lime treated materials should be considered as a bound layer. Materials with chemical stabilizers engineered to provide long-term strength and durability should be considered a chemically stabilized layer (i.e. cement treated, lean concrete, pozzolonic treated). Lime and/or lime-fly ash stabilized soils engineered to provide structural support can also be considered a chemically stabilized layer. These mixtures have a sufficient amount of stabilizer mixed in with the soil, as such these types of layers are placed directly under the PCC or lowest asphalt layer. **Figure 5.5 M-E Software Screenshot for Treated Base Inputs** presents a screenshot of treated base materials. **Note:** M-E Design has a stratigraphic layer called Sandwich Granular. This layer should only be used when the designer has a layer of untreated base placed 'sandwiched' between a chemically stabilized subgrade HMA layer.

Aggregate or granular base materials lightly treated with small amounts of chemical stabilizers to enhance constructability or expedite construction (i.e. lower the plasticity index, improve the strength) **should not** be considered a chemically stabilized layer. Typically, lightly stabilized materials are placed deeper in the pavement structure. **Note**: Currently Colorado does not have a calibration coefficient for a stabilized layer, therefore one should treat the layer as a high quality subbase or base course with a constant modulus. The material inputs required for characterizing treated base layers in M-E Design are presented in **Tables 5.2** through **Table 5.6**.

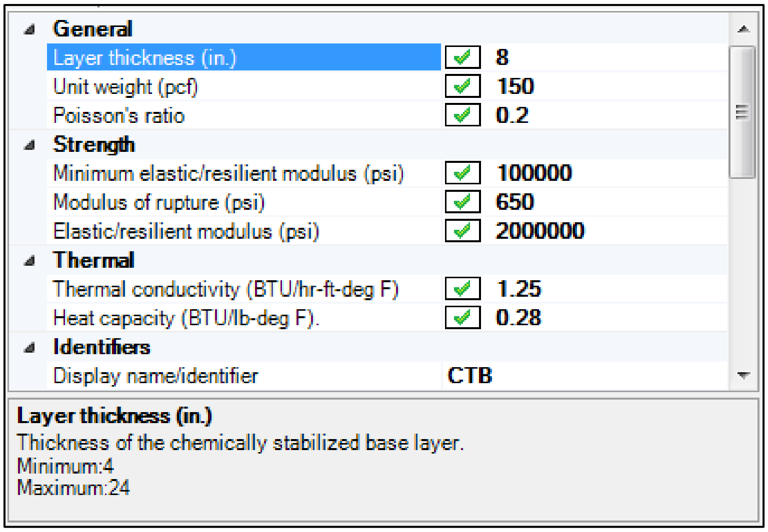


Figure 5.5 M-E Design Software Screenshot for Treated Base Inputs

Table 5.2 Characterization of Treated Bases in M-E Design

<table>
<tr><th>Input Property</th><th>Level 1</th><th>Level 2</th><th>Level 3</th></tr>
<tr><td>Elastic/Resilient Modulus</td><td rowspan="2">Table 5.3 Level 1 Input Requirement and Corresponding Testing Protocols for Characterization of Treated Bases in M-E Design</td><td>Table 5.4 Level 2 Correlations for Elastic Modulus of Treated Bases</td><td rowspan="2">Table 5.6 Level 3 Default Elastic Modulus and Flexural Strength of Treated Bases</td></tr>
<tr><td>Modulus of Rupture (flexible pavements)</td><td>Table 5.4 Level 2 Correlations for Flexural Strength of Treated Bases</td></tr>
<tr><td>Minimum Elastic / Resilient Modulus (flexible pavements)</td><td colspan="3">Use the following values:
• Lean concrete: 300,000 psi
• Cement stabilized aggregate: 100,000 psi
• Open graded cement stabilized: 50,000 psi
• Soil cement: 25,000 psi
• Lime-cement-fly ash: 40,000 psi
• Lime stabilized soils: 15,000 psi</td></tr>
<tr><td>Poisson’s Ratio</td><td colspan="3">Use typical values:
• Lean concrete & cement stabilized aggregate: 0.10 to 0.20
• Soil cement: 0.15 to 0.35
• Lime-fly ash materials: 0.10 to 0.15
• Lime stabilized soil: 0.15 to 0.20</td></tr>
<tr><td>Thermal Conductivity</td><td colspan="3">Use the M-E Design software default value of 1.25 BTU/hr-ft-°F</td></tr>
<tr><td>Heat Capacity</td><td colspan="3">Use the M-E Design software default value of 0.28 BTU/lb-°F</td></tr>
<tr><td>Total Unit Weight</td><td colspan="3">Use the M-E Design software default value of 150 lb/ft^3</td></tr>
</table>

Table 5.3 Level 1 Input Requirement and Corresponding Testing Protocols for Characterization of Treated Bases in M-E Design

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	✓		ASTM C 469
		Flexural strength	✓		AASHTO T 97
	Lime-cement-fly Ash	Resilient modulus		✓	No test protocols available. Estimate using Levels 2 and 3
	Soil Cement	Resilient modulus	✓		Mixture Design and Testing Protocol (MDTP) in conjunction with AASHTO T 307
	Lime Stabilized Soil	Resilient modulus		✓	No test protocols available. Estimate using Levels 2 and 3
Existing	Lean Concrete & Cement-Treated Aggregate	FWD backcalculated modulus	✓		ASTM D4694
	Lime-Cement-Fly Ash				
	Soil Cement				
	Lime Stabilized Soil				

Table 5.4 Level 2 Correlations for Elastic Modulus of Treated Bases

Material Type	Recommended Correlations
Lean Concrete[1]	$E = 57{,}000 \times \sqrt{f'_c}$ f'_c = compressive strength, psi (AASHTO T 22) (18)
Cement Treated Aggregate[1]	
Open Graded Cement Stabilized	No correlations are available
Soil Cement[2]	$E = 1200 \times q_U$ q_u = unconfined compressive strength, psi (ASTM D 1633) (18)
Lime-Cement-Fly Ash[2]	$E = 500 + q_U$ q_u = unconfined compressive strength, psi (ASTM C 593) (19)
Lime Stabilized Soils[2]	$M_r = 0.124 \times q_u + 9.98$ q_u = unconfined compressive strength, psi. (ASTM D 5102) (17)

Note: E is the modulus of elasticity in psi and M_r = resilient modulus in ksi.
[1] Compressive strength f_c can be determined using AASHTO T22.
[2] Unconfined compressive strength q_u can be determined using the MDTP.

Table 5.5 Level 2 Correlations for Flexural Strength of Treated Base

Material Type	Test Protocol	Typical M_r (psi)
Lean Concrete	AASHTO T 22	$M_r \approx 20\%$ of q_u (conservative estimate)
Cement Treated Aggregate		
Soil Cement	ASTM D 1633	
Lime-Cement-Fly Ash	ASTM C 593	
Lime Stabilized Soils	ASTM D 5102	
Open Graded Cement Stabilized Aggregate	Not available	Not available

Note: q_u = unconfined compressive strength

Table 5.6 Level 3 Default Elastic Modulus and Flexural Strength of Treated Bases

Material Type	E or M_r Range (psi)	E or M_r Typical (psi)	Flexural Strength (psi)
Lean Concrete	1,500,000 to 2,500,000	2,000,000	450
Cement Stabilized Aggregate	700,000 to 1,500,000	1,000,000	200
Soil Cement	50,000 to 1,000,000	500,000	100
Lime-Cement-Fly Ash	500,000 to 2,000,000	1,500,000	150
Lime Stabilized Soils[1]	30,000 to 60,000	45,000	25
Open Graded Cement Stabilized Aggregate	—	750,000	200

Note: [1] For reactive soils within 25 percent passing No. 200 sieve and plasticity index of at least 10.

5.4.2 Modeling Treated Base in M-E Design

To properly model a treated base or a stabilized subgrade in M-E Design, the designer should consider the following:

- **Plant Mix Bituminous Base:** This layer is produced at a central batch plant in a similar manner conventional asphalt mixtures are produced and should be considered either as or combined with a HMA base layer.

- **Emulsified Asphalt Treated Base:** This layer is composed of crushed stone base materials and emulsified asphalt. It should be combined with the crushed stone base materials or considered as an unbound aggregate mixture.

- **Cement Treated Base:** Cement treated and other pozzolanic stabilized materials that are engineered to provide structural support should be treated as a separate layer. Where a small portion of cement and/or other pozzolanic materials are added to granular base materials for constructability issues, such layers should be considered as an unbound material and combined with those unbound layers if necessary.

- **Lime and/or Lime-Fly Ash Stabilized Soils:** These soils may be considered a stabilized material if the layer is engineered to provide structural support; otherwise, they could be considered an unbound layer that is insensitive to moisture and the resilient modulus (stiffness) of the layer can be held constant over time.

5.5 Permeable Bases

Open-graded aggregate bases are becoming popular. Permeable bases may be unstabilized or stabilized and should be placed in a layer at least 4 inches thick. Care must be taken when designing with permeable bases as they are subject to freeze-thaw cycles.

- **Unstabilized** permeable bases contain smaller size aggregates to provide interlock, however this creates a lower permeability. Typically, the coefficient of permeability is 1,000 to 3,000 feet/day. Unstabilized bases are difficult to compact and density is difficult to measure. CDOT does not recommend using an unstabilized permeable base.

- **Stabilized** permeable bases are open-graded aggregates that have been stabilized with asphalt or portland cement. Stabilization of the base does not appreciably affect the permeability of the material and provides a very stable base during the construction phase. The coefficient of permeability is greater than 3,000 feet/day. Stabilized bases provide a stable working platform for construction equipment.

- **Asphalt** stabilized permeable bases contain 2 to 2.5 percent asphalt by weight. Care must be used in construction to prevent over rolling which can lead to degradation of the aggregate and loss of permeability. The base should be laid at a temperature of 200°F to 250°F and compacted between 100°F and 150°F.

- **Cement** stabilized bases have 2 to 3 bags of portland cement/cubic yard. This provides a very strong base that is easily compacted with a vibratory screed and plate. Curing can occur by covering the base with polyethylene sheeting for 3 to 5 days or with a fine water mist sprayed several times the day after the base is placed.

The designer is suggested to use FHWA's DRIP 2.0 software. The software has capabilities to perform roadway geometry calculations for the drainage path, sieve analysis calculations, inflow calculations, permeable base design, separator design (geotextile or aggregate layer), and edgedrain design (see **Figure 5.6 Structural Permeable aggregate Base Course Layers)**. The software may be obtained from the website: http://www.fhwa.dot.gov/pavement/desi.cfm.

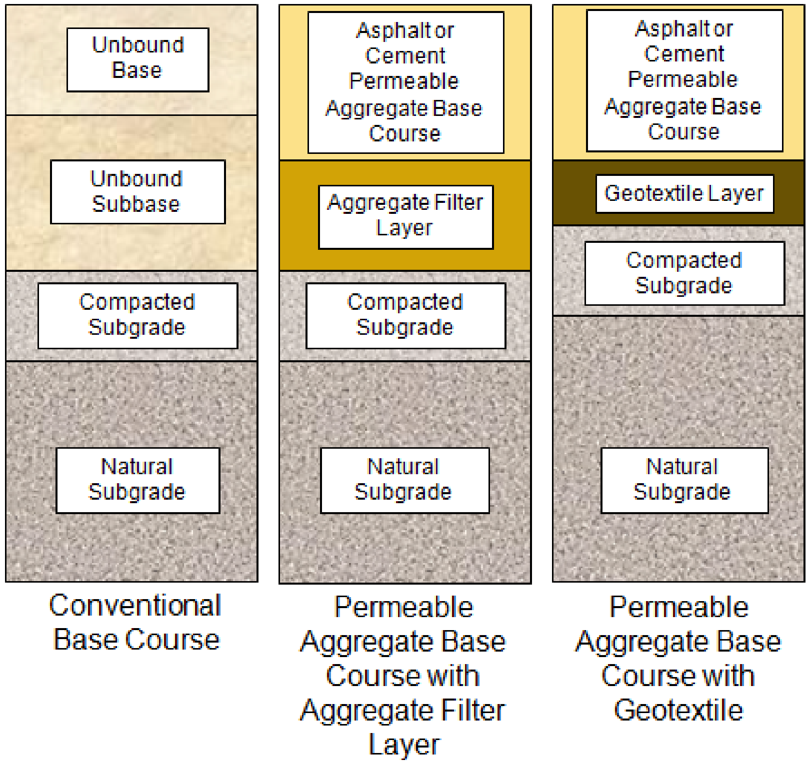


Figure 5.6 Structural Permeable Aggregate Base Course Layers

Drainage is particularly important where heavy flows of water are encountered (i.e., springs or seeps), where detrimental frost conditions are present, or where soils are particularly susceptible to expansion with increase in water content. Special subsurface drainage may include provisions of a permeable material beneath the pavement for interception and collection of water, and/or pipe drains for collection and transmission of water. Special surface drainage may require facilities like dikes, paved ditches, and catch basins (20).

5.6 Reclaimed Asphalt and Concrete Pavement

Refer to **Figure 5.7 Reclaimed Asphalt and Concrete Pavement Base Layers** for using reclaimed asphalt or concrete for a base layer.

5.6.1 Reclaimed Asphalt Pavement Base

Recycled asphalt pavement may be used as a granular base or subbase provided it meets gradation and minimum R-values specified in contract documents. Recycled asphalt used as an aggregate base is discussed in this section as a cold recycling process compared to a hot process. The cold recycling process of asphalt consists of recovered, crushed, screened, and blended material with conventional aggregates, and is placed as a conventional granular material.

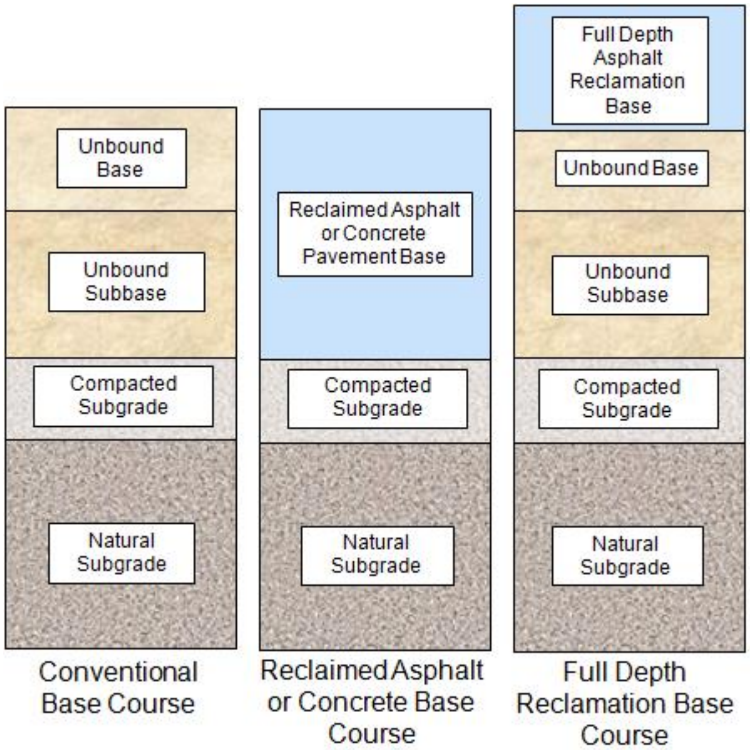


Figure 5.7 Reclaimed Asphalt and Concrete Pavement Base Layers

5.6.1.1 Reclaimed Asphalt Pavement (RAP) Base

Aggregate for Reclaimed Asphalt Pavement (RAP) base shall meet the grading requirements of **Table 5.7 CDOT Classification for Reclaimed Asphalt Program Aggregate Base Course**. The aggregate shall have a liquid limit of non-viscous (NV), plasticity index of non-plastic (NP), and a Los Angeles percentage of wear of 45 or less. Placement and compaction of each lift layer shall continue until a density of at least 100 percent on the maximum wet density as determined in accordance with Colorado Procedure CP-53 has been achieved (13), see **Figure 5.8 Photos of Reclaimed Asphalt Pavement Base.**

Source: http://www.pavementinteractive.org and http://www.wjgraves.com

Figure 5.8 Photos of Reclaimed Asphalt Pavement Base

5.6.1.2 Full Depth Asphalt Reclaimed Base (FDR)

A full depth asphalt reclaimed base is an in-place process that pulverizes the existing pavement and thoroughly mixes the individual surface and granular base course layers into a relatively homogeneous mixture. It is then recompacted as a granular base (25), see **Figure 5.9 Photos of Full Depth Asphalt Reclaimed Base**. Stabilizing agents may be added with a laboratory mix design to optimize the quantity of stabilizing agent and other properties of the reclaim mix. Pavement distresses that can be treated by full depth asphalt reclamation are as follows (28):

- Cracking from age, fatigue, slippage, edge, block, longitudinal, reflection, and discontinuity.
- Reduced ride quality due to swell, bumps, sags, and depressions, which are not contributed to swelling soils.
- Permanent deformations in the form of rutting, corrugation, and shoving
- Loss of bonding between layers and stripping
- Loss of surface integrity due to raveling, potholes, and bleeding
- Inadequate structural capacity

Table 5.7 CDOT Classification for Reclaimed Asphalt Pavement Aggregate Base Course

Sieve Size	Mass Percent Passing Square Mesh Sieves	
	ABC (RAP)	
	Lower Limit	Upper Limit
2" (50 mm)	100	-
1 1/2" (37.5 mm)	-	-
1" (25 mm)	85	100
3/4" (19 mm)	75	100
1/2" (12.5 mm)	55	90
3/8" (9.5 mm)	45	80
#4 (4.75 mm)	25	55
#8 (2.36 mm)	-	-
#16 (1.18 mm)	5	25
#30 (600 µm)	-	-
#50 (300 µm)	-	-
#100 (150 µm)	-	-
#200 (75 µm)	0	5

Source: http://west-cansealcoating.com and http://www.rocksolidstabilization.com

Figure 5.9 Photos of Full Depth Asphalt Reclaimed Base

5.6.2 Reclaimed Concrete Pavement Base (RCP)

Reclaimed Concrete Pavement (RCP) may be used as a granular base or subbase, similar to recycled asphalt. RCP is the recycling of recovered, crushed, and screened concrete pavement that is placed as a conventional granular material. RCP shall meet all conventional granular material requirements and have all steel removed in the recovering process.

5.7 Base Layer Made of Rubblized Rigid Pavement

Rubblization is a fracturing of existing rigid pavement creating a high-density granular material. The rough, hard particles provide an internal friction to resist rutting while the lack of tension prevents cracking in the surface layer. The reasoning for this is the more concrete available for expansion and contraction during temperature changes, the greater the movement of the slab, thus, the greater the opening of joints and cracks. Rubblization reduces the size of concrete pieces so the expansion and contraction has minimum movement. The space between the fractured pieces moves less so cracks are not reflected through the surface course. An edge drain system needs to be installed to remove water captured between the fractured concrete slabs. The fractured concrete pavement has been found to be more permeable than a dense graded compacted base layer (see **Figure 5.10 Rubblized Base Course** and **Figure 5.11 Photo of Rigid Pavement Being Rubblized**).

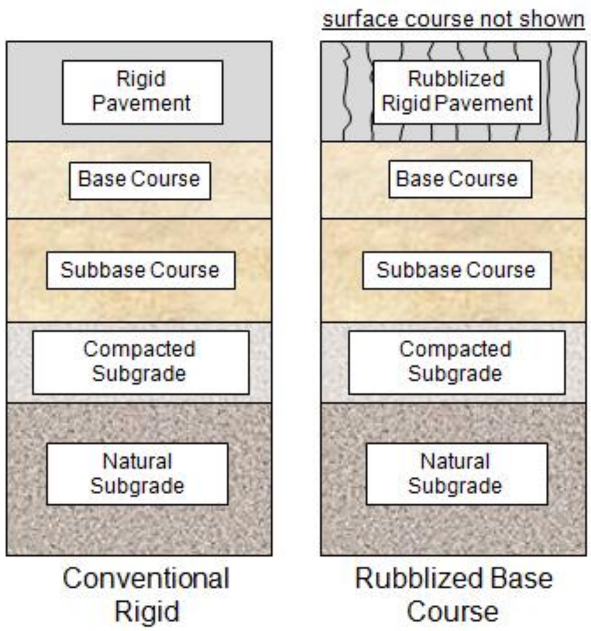
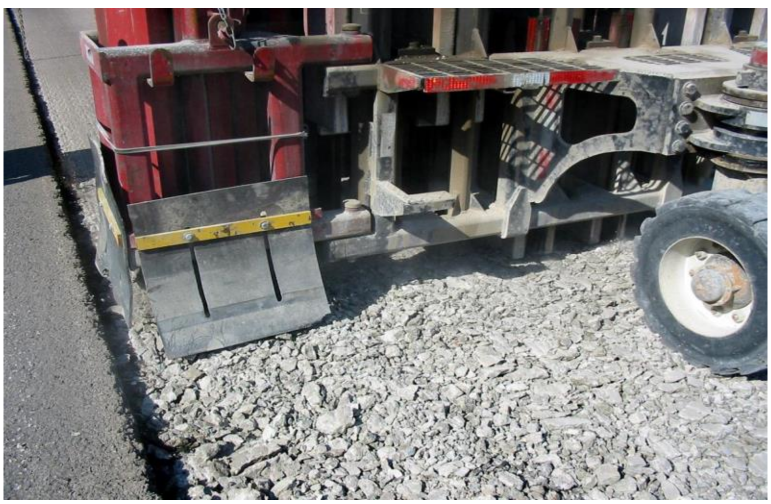


Figure 5.10 Rubblized Base Course



Source: http://www.antigoconstruction.com

Figure 5.11 Photo of Rigid Pavement Being Rubblized

5.8 Material Sampling and Testing

Sampling involves coring the existing pavement to determine layer thicknesses, make a visual inspection of the subsurface condition, and obtain material samples of unbound layers for further testing. For an existing pavement, the types of tests performed on the extracted materials should depend on the type of distress observed. Contact the Region Materials Engineer and see Chapter 200 of the *Field Materials Manual* for information on recommended sampling intervals and further guidance on available material test methods.

References

1. *Guidelines for Stabilization of Soils Containing Sulfates*, Technical Memorandum, by Austin White Lime, Chemical Lime, Texas Lime, distributed by National Lime Association, Arlington, VA, August 2000.

2. Hunter, D., *Lime-Induced Heave in Sulfate-Bearing Clay Soil*, ASCE Journal of Geotechnical Engineering Vol. 114 Issue 2, 1988 pp. 150-167.

3. Little, D. N., *Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime*, National Lime Association, Kendall/Hunt Publishing Company, Dubuque, Iowa, 1995.

4. Hopkins, T. C., T. L. Beckham, L. Sun, B. Ni and B. Butcher, *Long-Term Benefits of Stabilizing Soil Subgrades*, Kentucky Transportation Center, College of Engineering, University of Kentucky, Lexington, Kentucky, 2002.

5. Copas, T. L. and H. A. Pennock, *Lime- Fly Ash-Stabilized Bases and Subbases*, NCHRP Synthesis of Highway Practice 37, TRB, National Research Council, Washington, D.C., 1976.

6. McManis, K. L. and A. Arman, *Class C Fly Ash as a Full or Partial Replacement for Portland Cement or Lime*, Transportation Research Record 1219, TRB, National Research Council, Washington, D.C., 1989, pp. 68-81.

7. Nicholson, P. G., V. Kashyap, and C.F. Fujii, *Lime and Fly Ash Admixture Improvement of Tropical Hawaiian Soils*, Transportation Research Record 1440, TRB, National Research Council, Washington, D.C., 1994, pp. 71-78.

8. Petry, T. M., *Performance-Based Testing of Chemical Stabilizers*, Transportation Research Record 1589, TRB, National Research Council, Washington, D.C., 1997, pp. 36-41.

9. Prusinski, J. R., and S. Bhattacharja, *Effectiveness of Portland Cement and Lime in Stabilizing Clay Soils*, Transportation Research Record 1652, TRB, National Research Council, Washington, D.C., 1999, pp. 215-227.

10. Parsons, R. L., and J. P. Milburn, *Engineering Behavior of Stabilized Soils*, Transportation Research Record 1837, TRB, National Research Council, Washington, D.C., 2003, pp. 20-29.

11. *1972 Roadway Design Manual*, Colorado Department of Transportation.

12. *1980 Roadway Design Manual*, Colorado Department of Transportation.

13. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2005.

14. Yeh, Shan-Tai, and Su, Cheng-Kuang, *Resilient Properties of Colorado Soils*, Final Report CDOH-DH-SM-89-9, Colorado Department of Highways, December 1989.

15. Chang, Nien-Yin, Chiang, Hsien-Hsiang, and Jiang, Lieu-Ching, *Resilient Modulus of Granular Soils with Fines Contents*, Final Research Report CDOT 91-379, Colorado Department of Transportation, May 1994.

16. *Guide for Mechanistic-Empirical Design*, Final Report, NCHRP Project 1-37A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Submitted by ARA, INC., ERES Consultants Division, Champaign, IL, March 2004.

17. *CDOT Field Materials Manual*, Colorado Department of Transportation, 2004.

18. *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC, 1993.

19. Supplement to the AASHTO Guide for Design of Pavement Structures, Part II, - Rigid Pavement Design & Rigid Pavement Joint Design, American Association of State Highway and Transportation Officials, Washington, DC, 1998.

20. *AASHTO Interim Guide for Design of Pavement Structures 1972*, published by American Association of State Highway and Transportation Officials, Washington, DC, 1972.

21. DRIP 2.0 Software Program < http://www.fhwa.dot.gov/pavement/desi.cfm>

22. Mallela, Jagannath, Larson, Gregg, Wyatt, Tim, Hall, Jim, and Barker, Walter, *User's Guide for Drainage Requirement In Pavements- Drip 2.0 Microcomputer Program*, ERES Consultants, Division of Applied Research Associates, Inc., 505 W. University Avenue, Champaign, IL 61820 and prepared for Federal Highway Administration, 400 Seventh Street, SW, Washington, DC 20590, July 2002.

23. *LTPP Data Analysis Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*, Publication FHWA-RD-96-198, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, January 1997.

24. *Development of Pavement Design Concepts*, prepared by CTL/Thompson, Inc., Consulting Engineers, 1971 West 12th Ave, Denver, CO 80204 and prepared for the Metropolitan Government Pavement Engineers Council, April 1998.

25. *FHWA Reclaimed Asphalt Concrete - User Guideline - Granular Base*, <http://www.fhwa.dot.gov/research> (1/25/2006).

26. Ardani, Ahmad, *Expansive Soil Treatment Methods in Colorado*, Final Report CDOT-DTD-R-92-2, Colorado Department of Transportation, March 1992.

27. Colorado Department of Highways Memo #323, (Construction) Swelling Soils, 1/5/1966.

28. *Basic Asphalt Recycling Manual*, Federal Highway Administration, 400 Seventh Street, SW, Washington, DC 20590 and Asphalt Recycling and Reclaiming Association, #3 Church Circle, PMB 250, Annapolis, MD 21401, 2001.

29. *Long-Term Pavement Performance Information Management System: Pavement Performance Database User Reference Guide*, Publication No. FHWA-RD-03-088 (Interim Update), Federal Highway Administration, Research, Development and Technology, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, September 2005.

30. Long-Term Pavement Performance (LTPP) Data Analysis Support: National Pooled Fund Study TPF-5(013), Effects of Multiple Freeze Cycles and Deep Frost Penetration on Pavement Performance and Cost, Publication No. FHWA-HRT-06-121, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, November 2006.

31. *Engineering and Design Pavement Criteria for Seasonal Frost Conditions, Mobilization Construction*, Engineering Manual EM1110-3-138, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, Washington, D.C. 20314, April 9, 1984.

32. CDOT Materials Bulletin, 2008 Number 1, Geosynthetics and Geotextiles, Project Development Branch (for Materials and Geotechnical Branch), January 25, 2008.

33. *FHWA-NHI-07-092, Geosynthetic Design & Construction Guidelines Reference Manual*, NHI Course No. 132013, National Highway Institute, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., August 2008.

34. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2011.
http://www.coloradodot.info/business/designsupport/construction-specifications/2011-Specs

35. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

CHAPTER 6
PRINCIPLES OF DESIGN FOR FLEXIBLE PAVEMENT

6.1 Introduction

Design of flexible pavement structures involves the consideration of numerous factors, the most important are truck volume, weight and distribution of axle loads, HMA, underlying material properties, and the supporting capacity of the subgrade soils. **Typical reconstruction projects should have a design life of 20 years for reconstructions and 10 years for rehabilitations** unless mitigating circumstances exist.

Methods are presented in this section for the design of the flexible pavement structure with respect to thickness of the subbase, base, surface courses, and the quality and strength of the materials in place. Interaction between pavement materials and climate is evaluated as part of the M-E Design process.

6.2 M-E Design Methodology for Flexible Pavement

M-E Design uses an iterative process. The key steps in the design process include the following:

1. **Select a Trial Design Strategy**

2. **Select Appropriate Performance Indicator Criteria for the Project:** Establish criteria for acceptable pavement performance (i.e. distress/IRI) at the end of the design period. Performance criteria were established to reflect different magnitudes of key pavement distresses which trigger major rehabilitation or reconstruction. CDOT criteria for acceptable performance is based on highway functional class and location.

3. **Select Appropriate Reliability Level for the Project:** The reliability is in essence a factor of safety that accounts for inherent variations in construction, materials, traffic, climate, and other design inputs. The level of reliability selected should be based on the criticality of the design and selected for each individual performance indicator. CDOT criteria for a desired reliability is based on highway functional class and location.

4. **Assemble All Inputs for the Pavement Trial Design Under Consideration:** Define subgrade support, asphalt concrete and other paving material properties, traffic loads, climate, pavement type and design, and construction features. The inputs required to run the M-E Design program may be obtained using one of three hierarchical levels and need not be consistent for all inputs in a given design. The hierarchical level for a given input is selected based on the importance of the project, input, and resources at the disposal of the user.

5. **Run the M-E Design Software:** The software calculates changes in layer properties, damage, key distresses, and IRI over the design life. The key steps include:

a) Processing input to obtain monthly values of traffic, seasonal variations of material, and climatic inputs needed in design evaluations for the entire design period.
b) Computing structural responses (stresses and strains) using multilayer elastic theory or finite element based pavement response models for each axle type and load and each damage-calculation increment throughout the design period.
c) Calculating accumulated distress at the end of each analysis period for the entire design period.
d) Predicting key distresses (rutting, bottom-up/top-down fatigue cracking, and thermal cracking) at the end of each analysis period throughout the design life using calibrated mechanistic-empirical performance models.
e) Predicting IRI as a function of initial IRI, distresses accumulating over time, and site factors at the end of each analysis increment.

6. **Evaluate Adequacy of the Trial Design:** The trial design is considered "adequate" if none of the predicted distresses/IRI exceed the performance indicator criteria at the design reliability level chosen for the project. If any criteria has been exceeded, one must determine how the deficiency can be remedied by altering material types, properties, layer thicknesses, or other design features.

7. **Revise the Trial Design, as Needed:** If the trial design is deemed "inadequate", one must revise the inputs and re-run the program until all performance criteria have been met. Once met, the trial design becomes a feasible design alternative.

Design alternatives that satisfy all performance criteria are considered feasible from a structural and functional viewpoint and may be considered for other evaluations, such as life cycle cost analysis. Consultation of the mix design(s) with the RME **shall** occur. A detailed description of the design process is presented in the interim edition of the *AASHTO Mechanistic-Empirical Pavement Design Guide Manual of Practice,* AASHTO 2008.

6.3 Select a Trial Design Strategy

6.3.1 Flexible Pavement Design Types

Figure 6.1 Asphalt Concrete Pavement Layer Systems illustrates well known CDOT combinations of asphalt concrete structural pavement layers. Designers can select from among several flexible pavement options as shown below:

- **Conventional Flexible Pavements:** Flexible pavements consisting of a relatively thin asphalt concrete layer placed over an unbound aggregate base layer and subgrade.

- **Deep-Strength AC Pavements:** Flexible pavements consisting of a relatively thick asphalt concrete layer placed over an unbound aggregate base layer and subgrade.

- **Full-Depth AC Pavements:** Asphalt concrete layers placed directly over the subgrade.

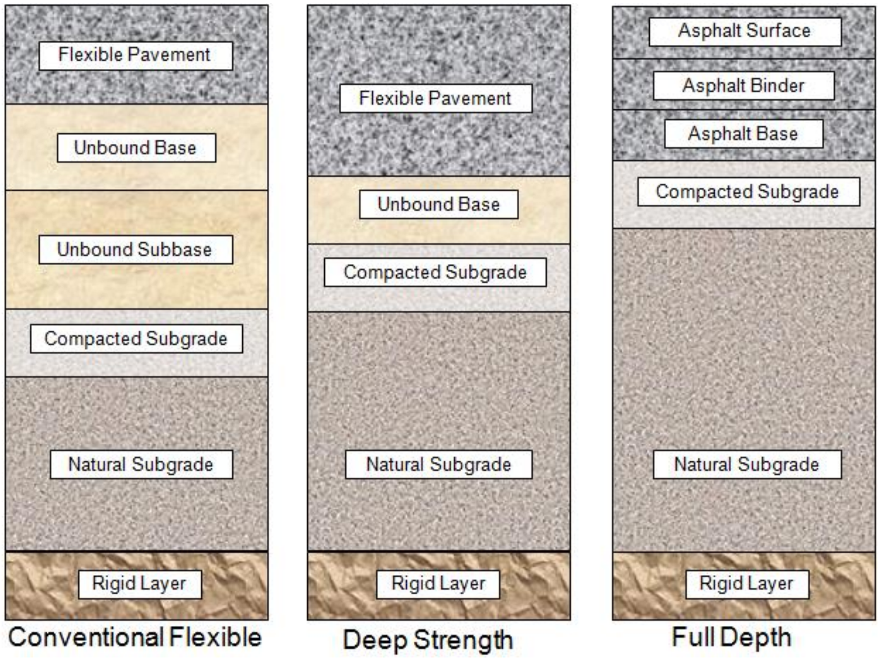


Figure 6.1 Asphalt Concrete Pavement Layer Systems

The asphalt concrete layer in **Figure 6.1 Asphalt Concrete Pavement Layer Systems** may be comprised of several layers of asphalt concrete courses to include a surface course, intermediate or binder course, and a base course (see **Figure 6.2 Structural Layers**). The surface, binder, and base courses are typically different in composition and are placed in separate construction operations (3).

- **Surface Course:** The surface course normally contains the highest quality materials. It provides characteristics such as friction, smoothness, noise control, rut and shoving resistance, and drainage. It also serves to prevent the entrance of excessive quantities of surface water into the underlying HMA courses, bases, and subgrade.

- **Intermediate/Binder Course**: The intermediate course, sometimes called binder course, consists of one or more lifts of structural HMA placed below the surface course. Its purpose is to distribute traffic loads so stresses transmitted to the pavement foundation will not result in permanent deformation to the course. It also facilitates the construction of the surface course.

- **Base Course:** The base course consists of one or more HMA lifts located at the bottom of the structural HMA course. Its major function is to provide the principal support of the pavement structure. The base course should contain durable aggregates that will not be damaged by moisture or frost action.

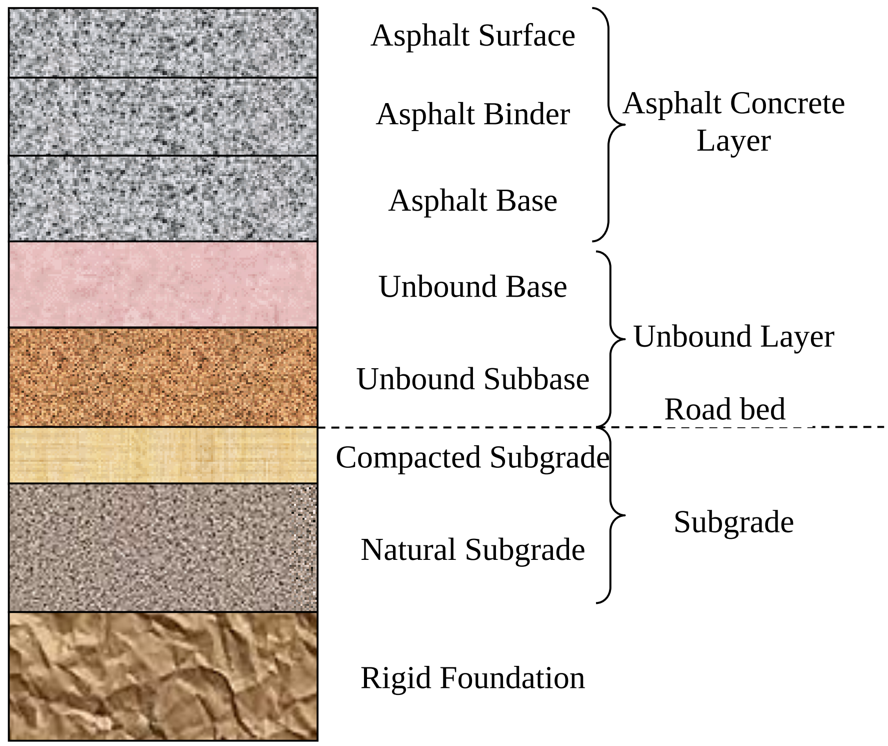


Figure 6.2 Structural Layers

6.3.2 Concept of Perpetual Pavements

A perpetual pavement is defined as an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement (6). Full depth and deep-strength asphalt pavement structures have been constructed since the 1960s. Full-depth pavements are constructed directly on subgrade soils and deep-strength sections are placed on relatively thin (4 to 6 inches) granular base courses. A 20-year traffic design period is to be used for the traffic loading. One of the chief advantages of these pavements is that the overall section of the pavement is thinner than those employing thick granular base courses. Such pavements have the added advantage of significantly reducing the potential for fatigue cracking by minimizing the tensile strains at the bottom of the asphalt layer (7) (see **Figure 6.1 Asphalt Concrete Pavement Layer Systems)**. An asphalt perpetual pavement structure is designed with a durable, rut and wear resistant top layer with a rut resistant intermediate layer and a fatigue resistant base layer (see **Figure 6.3 Perpetual Pavement Design Concept**).

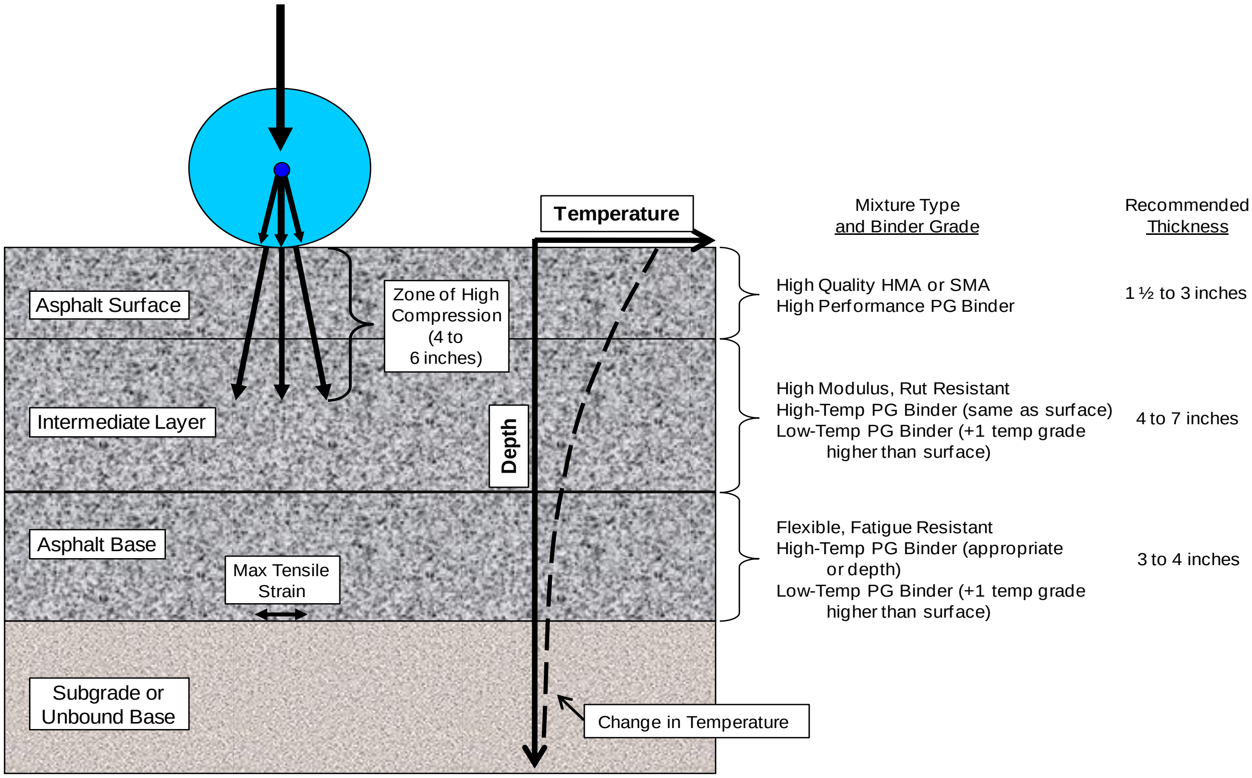


Figure 6.3 Perpetual Pavement Design Concept

This concept may be used in conventional, deep strength, or full depth asphalt structural layering. In mechanistic design, the principles of physics are used to determine a pavement's reaction to loading. Knowing the critical points in the pavement structure, one can design against certain types of failure or distress by choosing the right materials and layer thicknesses (7). Therefore, the uppermost structural layer resists rutting, weathering, thermal cracking, and wear. SMAs or dense-graded SuperPave mixtures provide these qualities. The intermediate layer provides rutting resistance through stone-on-stone contact and durability is imparted by the proper selection of materials. Resistance to bottom-up fatigue cracking is provided by the lowest asphalt layer having a higher binder content or by the total thickness of pavement reducing the tensile strains in this layer to an insignificant level (6).

6.3.3 Establish Trial Design Structure

The designer must establish a trial design structure (combination of material types and thicknesses). This is done by first selecting the pavement type of interest (see **Figure 6.4 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)**). M-E Design automatically provides the top layers of the selected pavement type. The designer may add or remove pavement structural layers and/or modify the layer material type and thickness as appropriate. **Figure 6.5 M-E Design Software Screenshot of Flexible Pavement Trial Design Structure** shows an example of flexible pavement trial design with pavement layer configuration on the left and layer properties of the AC surface course on the right.

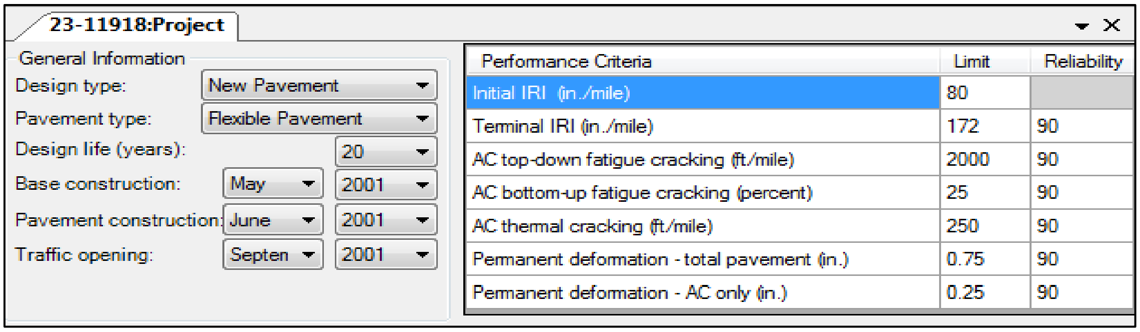


Figure 6.4 M-E Design Software Screenshot Showing General Information (left), Performance Criteria Reliability (right)

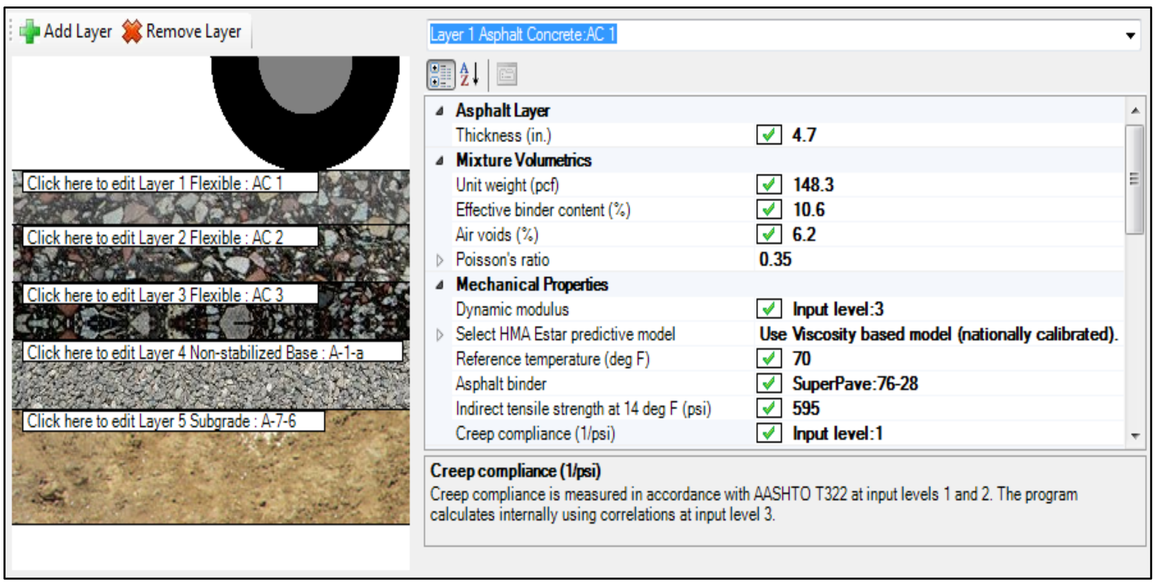


Figure 6.5 M-E Design Software Screenshot of Flexible Pavement Trial Design Structure

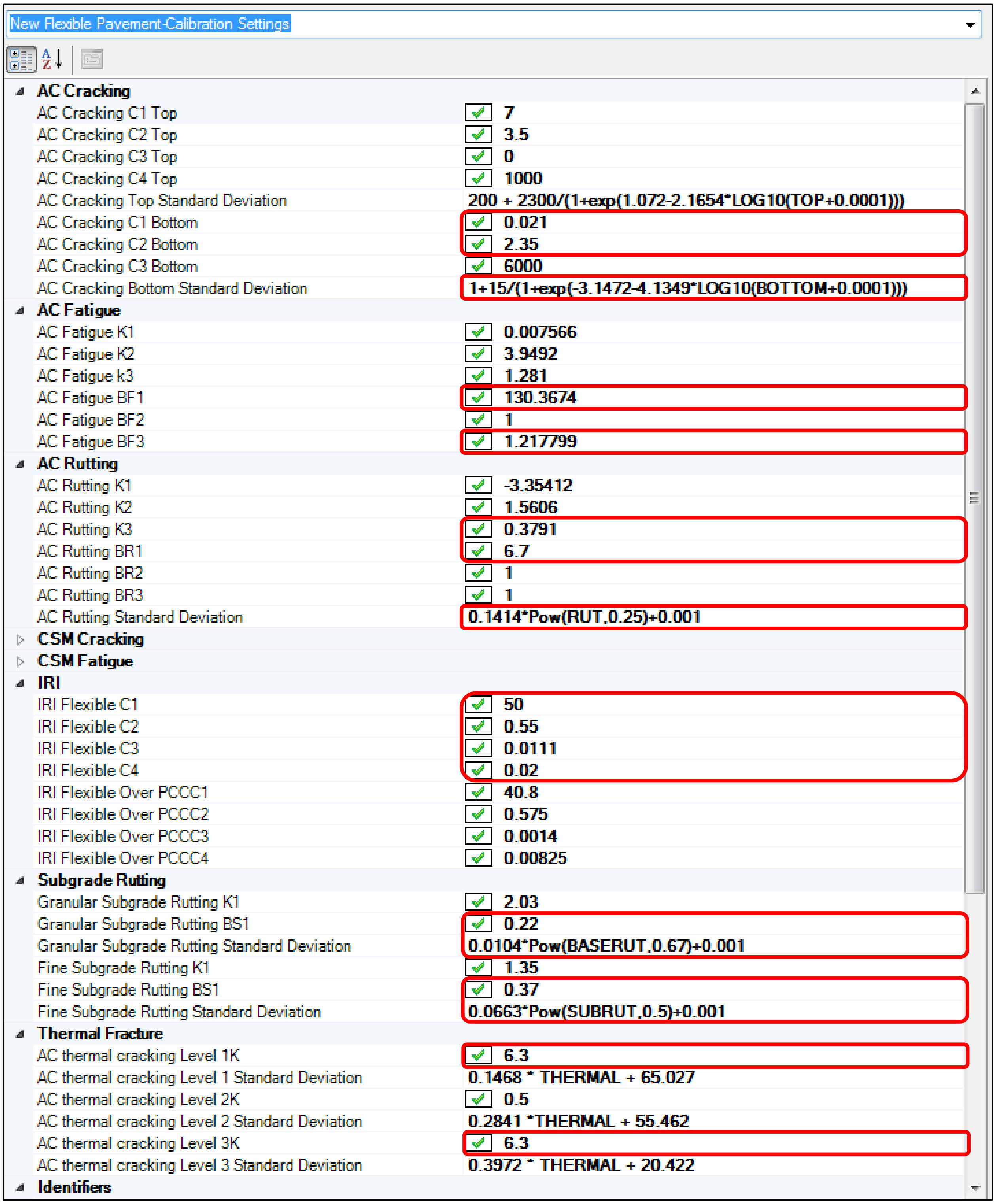
6.4 Select the Appropriate Performance Indicator Criteria for the Project

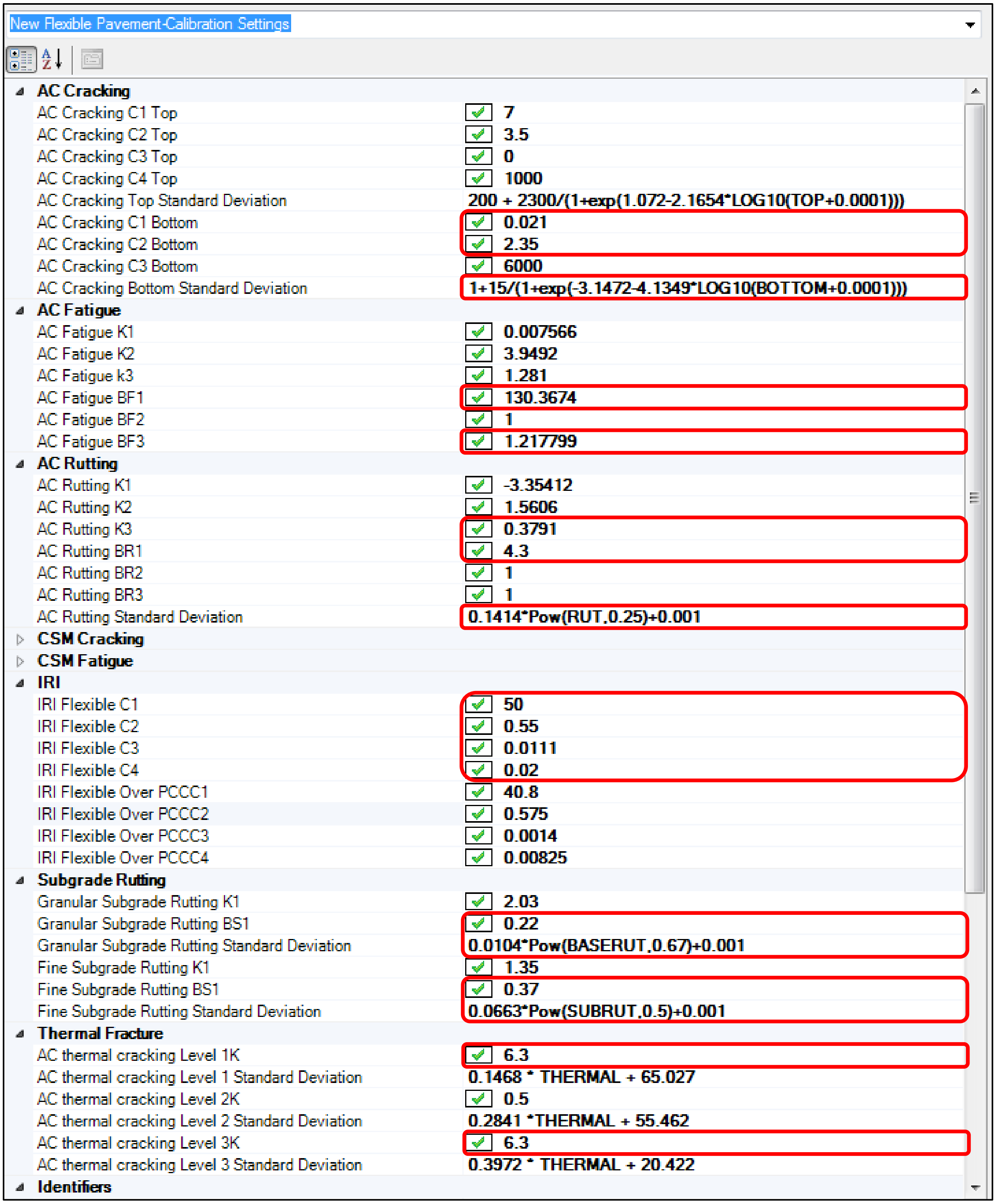
Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction Projects presents recommended performance criteria for flexible pavement design. The designer should enter the appropriate performance criteria based on functional class. An appropriate initial smoothness (IRI) is also required, **<u>For new flexible pavements, the recommended initial IRI is 61 inches/mile</u>**. **Figure 6.4 M-E Design Software Screenshot Showing General Information (left) Performance Criteria and Reliability (right)** shows performance criteria for a sample flexible pavement trial design. The coefficients of performance prediction models considered in the design of a new flexible pavement are shown in **Figure 6.6 Performance Prediction Model Coefficients for Flexible Pavement Designs (Marshall Mix)** through **Figure 6.8 Performance Prediction Model Coefficients for Flexible Pavement Designs (PMA Mix)**. The value of AC rutting coefficient (BR1) is based on the type of HMA.

New Flexible Pavement-Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.021
AC Cracking C2 Bottom	2.35
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	130.3674
AC Fatigue BF2	1
AC Fatigue BF3	1.217799
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.3791
AC Rutting BR1	7.6742
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.1414*Pow(RUT,0.25)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	50
IRI Flexible C2	0.55
IRI Flexible C3	0.0111
IRI Flexible C4	0.02
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.22
Granular Subgrade Rutting Standard Deviation	0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.37
Fine Subgrade Rutting Standard Deviation	0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	6.3
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	6.3
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 6.6 Performance Prediction Model Coefficients for Flexible Pavement Designs (Marshall Mix)


New Flexible Pavement-Calibration Settings
AC Cracking
AC Cracking C1 Top 7
AC Cracking C2 Top 3.5
AC Cracking C3 Top 0
AC Cracking C4 Top 1000
AC Cracking Top Standard Deviation 200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom 0.021
AC Cracking C2 Bottom 2.35
AC Cracking C3 Bottom 6000
AC Cracking Bottom Standard Deviation 1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue
AC Fatigue K1 0.007566
AC Fatigue K2 3.9492
AC Fatigue k3 1.281
AC Fatigue BF1 130.3674
AC Fatigue BF2 1
AC Fatigue BF3 1.217799
AC Rutting
AC Rutting K1 -3.35412
AC Rutting K2 1.5606
AC Rutting K3 0.3791
AC Rutting BR1 6.7
AC Rutting BR2 1
AC Rutting BR3 1
AC Rutting Standard Deviation 0.1414*Pow(RUT,0.25)+0.001
CSM Cracking
CSM Fatigue
IRI
IRI Flexible C1 50
IRI Flexible C2 0.55
IRI Flexible C3 0.0111
IRI Flexible C4 0.02
IRI Flexible Over PCCC1 40.8
IRI Flexible Over PCCC2 0.575
IRI Flexible Over PCCC3 0.0014
IRI Flexible Over PCCC4 0.00825
Subgrade Rutting
Granular Subgrade Rutting K1 2.03
Granular Subgrade Rutting BS1 0.22
Granular Subgrade Rutting Standard Deviation 0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1 1.35
Fine Subgrade Rutting BS1 0.37
Fine Subgrade Rutting Standard Deviation 0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture
AC thermal cracking Level 1K 6.3
AC thermal cracking Level 1 Standard Deviation 0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K 0.5
AC thermal cracking Level 2 Standard Deviation 0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K 6.3
AC thermal cracking Level 3 Standard Deviation 0.3972 * THERMAL + 20.422
Identifiers



New Flexible Pavement-Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.021
AC Cracking C2 Bottom	2.35
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	130.3674
AC Fatigue BF2	1
AC Fatigue BF3	1.217799
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.3791
AC Rutting BR1	4.3
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.1414*Pow(RUT,0.25)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	50
IRI Flexible C2	0.55
IRI Flexible C3	0.0111
IRI Flexible C4	0.02
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.22
Granular Subgrade Rutting Standard Deviation	0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.37
Fine Subgrade Rutting Standard Deviation	0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	6.3
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	6.3
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 6.7 Performance Prediction Model Coefficients for Flexible Pavement Designs (Polymer Modified Superpave Mix)

6.5 Select the Appropriate Reliability Level for the Project

Recommended reliability levels for flexible pavement designs are located in **Table 2.3 Reliability (Risk)**. The designer should select an appropriate reliability level based on highway functional class and location. **Figure 6.4 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)** shows design reliability values for a sample flexible pavement trial design.

6.6 Assemble M-E Design Software Inputs

6.6.1 General Information

6.6.1.1 Design Period

The design period for new flexible pavement construction and reconstruction is at least 20 years. For special designs, the designer may use a different design period as appropriate.

6.6.1.2 Construction Dates and Timeline

The following inputs are required to specify the construction dates and timeline (see **Figure 6.5 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)**):

- Base/subbase construction month and year
- Pavement construction month and year
- Traffic open month and year

The designer may select the most likely month and year for construction completion of the key activities listed above. Selection is based on the designer's experience, agency practices, or estimated from the planned construction schedule. For large projects that extend into different paving seasons, it is suggested each paving season be evaluated separately and the designer judge the acceptability of the trial design based on the more conservative situation. The M-E Design software does not consider staged construction events, nor does it consider the impact of construction traffic on damage computations.

Note: The pavement performance predictions begin from the month the pavement is open to traffic. The changes to pavement material properties due to time and environmental conditions are calculated beginning from the month and year the material was placed.

6.6.1.3 Identifiers

Identifiers are helpful in documenting the project location and recordkeeping. M-E Design allows designers to enter site or project identification information such as the location of the project (route signage, jurisdiction, etc.), identification numbers, beginning and ending milepost, direction of traffic, and date.

6.6.2 Traffic

Several inputs are required for characterizing traffic for the M-E Design software and are described in detail in **Section 3.1 Traffic**.

6.6.3 Climate

The climate input requirements for the M-E Design software are described in detail in **Section 3.2 Climate**.

6.6.4 Pavement Layer Characterization

As shown in **Figure 6.2 Structural Layers**, a typical flexible pavement design comprises of the following pavement layers: asphalt concrete, unbound aggregate base layers, and subgrade. The inputs required by M-E Design for characterizing these layers are described in the following sections.

6.6.4.1 Asphalt Concrete Characterization

Asphalt concrete types used in Colorado include:

- **Hot Mix Asphalt (HMA)**: Composed of aggregates with an asphalt binder and certain anti-stripping additives.

- **Stone Matrix Asphalt (SMA)**: Gap-graded HMA that maximizes rutting resistance and durability with a stable stone-on-stone skeleton held together by a rich mixture of AC, filler, and stabilizing agents.

The designers should apply the following guidelines when defining an asphalt concrete layer:

- As much as possible and as appropriate, the asphalt concrete layers must be combined into three layers: surface, intermediate and base. Asphalt layers with similar HMA mixtures may be combined into a single layer.

- When multiple layers are combined, the properties of the combined layer should be the weighted average of the individual layers.

- The M-E Design software does not consider very thin layers (thickness less than 1.5 inches).

- Weakly stabilized asphalt materials (i.e. sand-asphalt) should not be considered an asphalt concrete layer.

- M-E Design models layer by layer rutting. **Table 6.1 Layered Rut Distribution** shows the percentages used for calculating the final rutting in Colorado.

Table 6.1 Layered Rut Distribution

Layer	Colorado Percent Distribution	Global Percent Distribution
Hot Mix Asphalt	60	80
Aggregate Base Course	10	5
Subgrade	30	15

Designers are required to input volumetric properties such as air voids, effective asphalt content by volume, aggregate gradation, mix density, and asphalt binder grade (see **Figure 6.8 Asphalt Concrete Layer and Material Properties in M-E Design**). The designers are also required to input the engineering properties such as the dynamic modulus, creep compliance, indirect tensile strength of HMA materials, and the viscosity versus temperature properties of rolling thin film oven (RTFO) aged asphalt binders. These inputs can be obtained following the input hierarchy levels depending on the criticality of the project. The volumetric properties entered into the program need to be representative of the in-place asphalt concrete mixture. The project-specific in-place mix properties will not be available at the design stage. The designer should use typical values available from previous construction records or target values from the project specifications.

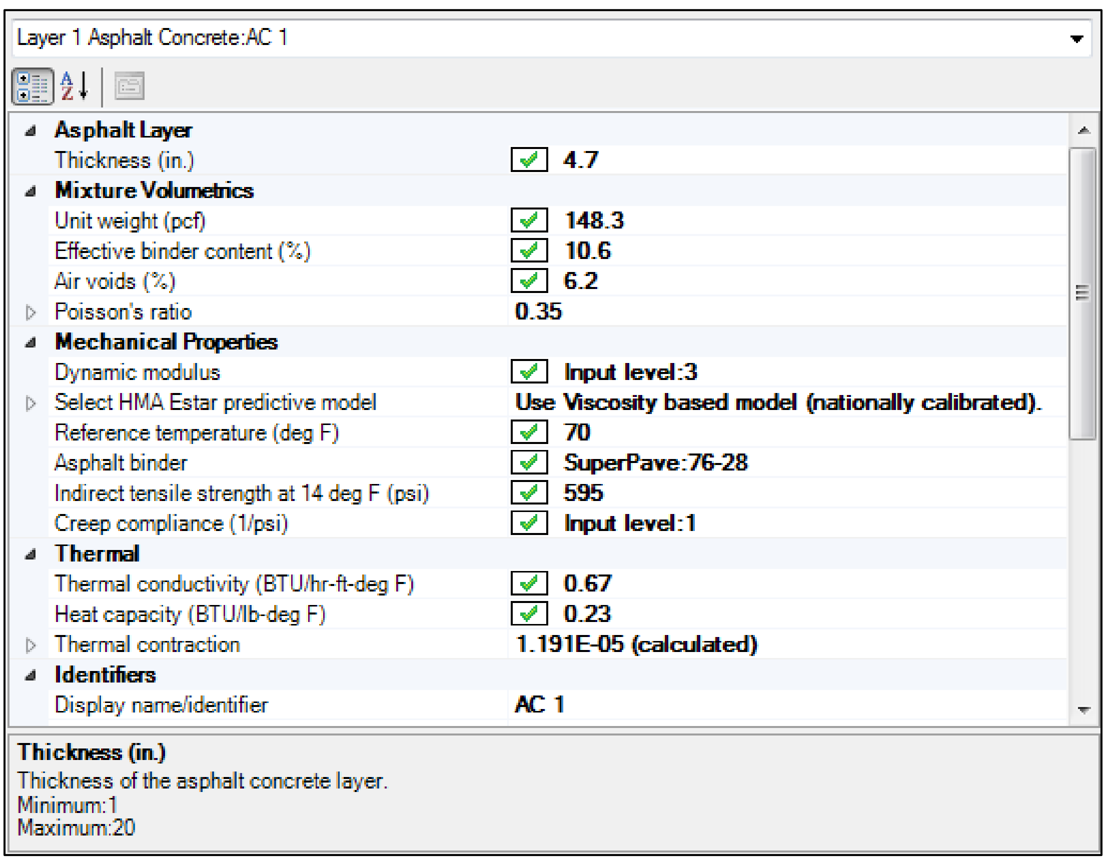


Figure 6.8 Asphalt Concrete Layer and Material Properties in M-E Design

Table 6.2 Input Properties and Recommendations for HMA Material Characterization presents the HMA input requirements of the M-E Design Method and recommendations for obtaining inputs at each hierarchical input level. The designer may use Level 1 inputs of typical CDOT HMA mixtures for Level 2 and 3 inputs. See **APPENDIX F** and **Table 2.6 Selection of Input Hierarchical Level** for selection of an appropriate hierarchical level for HMA characterization. For new construction (i.e. new HMA) the designer **<u>should always click "True" for the Possion's Ratio</u>** (currently the default value is "False").

Table 6.2 Input Properties and Recommendations for HMA Material Characterization

Input Property	Level 1	Level 2	Level 3
Dynamic Modulus (E*)	Mix specific E* and/or AASHTO TP62 test results	Gradation (**APPENDIX E**)	
Asphalt Binder Properties	Binder properties from laboratory testing of HMA using AASHTO T315		Binder grade (**APPENDIX E**)
Tensile Strength [1] at 14 ºF	AASHTO T322 test results	Use tensile strength and creep compliance (**APPENDIX E**)	
Creep Compliance			
Poisson's Ratio	M-E Design software option (*Is Poisson's ratio calculated?*)		Use 0.35
Air Voids	Use air voids (**APPENDIX E**)		
Volumetric Asphalt Content	Use volumetric asphalt content (**APPENDIX E**)		
Total Unit Weight	Use total unit weight (**APPENDIX E**)		
Surface Shortwave Absorptivity	Use 0.85		
Coefficient of Thermal Contraction of the Mix	1.3E-05 in./in./°F (mix CTE) and 5.0 E-06 in./in./°F (aggregate CTE)		
Thermal Conductivity	0.67 Btu/(ft)(hr)(ºF)		
Heat Capacity	0.23 BTU/lb.- ˚F		
Reference Temperature	70 ˚F		

Note: [1] The designer should use Level 1 Inputs. The Level 3 Inputs for tensile strength are much smaller which will cause more thermal cracking and greater creep compliance.

6.6.4.2 Unbound Layers and Subgrade Characterization

Refer to **Section 5.3.1 Unbound Layer Characterization in M-E Design** for unbound aggregate base layer characterization. Refer to **Section 4.4 Subgrade Characterization for M-E Design** for subgrade characterization.

6.7 Run M-E Design Software

Designers should examine all inputs for accuracy and reasonableness prior to running the M-E Design software. Next, one should run the software to obtain outputs required to determine if the trial design is adequate. After a trial run has been successfully completed, M-E Design will generate a report in form of a PDF and/or Microsoft Excel file, refer to **Figure 6.9 Sample Flexible Pavement Trial Design PDF Output Report**. The output report has input information, reliability of design, material properties, and predicted performance. It also includes the month to month estimates of material properties over the entire design period in either tabular or graphical form. For a flexible pavement trial design, the report provides the following:

- Monthly estimates of HMA dynamic modulus for each sublayer
- Monthly estimates of resilient modulus of unbound layers and subgrade
- Monthly estimates of AADTT
- Monthly estimates of climate parameters
- Cumulative trucks (FHWA Class 4 through 13) over the design period
- Cumulative ESALs over the design period (an intermediate file in the project folder)

After the trial run is complete, the designer should re-examine all inputs and outputs for accuracy and reasonableness before accepting a trial design as complete.

23-11918

File Name: C:\Users\bbhattacharya.RESTRICT\Desktop\COLORADO MEPDG\FINAL REPORT\23-11918.dgpx

Design Inputs

Design Life: 20 years
Design Type: Flexible Pavement

Base construction: May, 2001
Pavement construction: June, 2001
Traffic opening: September, 2001

Climate Data Sources (Lat/Lon): 39.909, -105.117; 39.702, -104.752; 39.833, -104.658; 39.57, -104.849; 39.783, -104.55; 40.452, -105.001; 39.217, -104.633; 40.436, -104.618

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	AC 1	4.7
Flexible	AC 2	5.0
Flexible	AC 3	5.0
NonStabilized	A-1-a	6.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.6
Air voids (%)	6.2

Traffic

Age (year)	Heavy Trucks (cumulative)
2001 (initial)	1,936
2011 (10 years)	8,349,420
2021 (20 years)	20,287,400

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	172.00	170.82	90.00	90.59	Pass
Permanent deformation - total pavement (in.)	0.75	0.50	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.53	90.00	100.00	Pass
AC thermal cracking (ft/mile)	250.00	84.34	90.00	99.99	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	256.64	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.25	0.28	90.00	80.53	Fail

Figure 6.9 Sample Flexible Pavement Trial Design PDF Output Report

6.8 Evaluate the Adequacy of the Trial Design

The output report of a flexible pavement trial design includes the monthly accumulation of the following key distress types at their mean values and chosen reliability for the entire design period:

- **Alligator Fatigue Cracking:** Traditional wheel path cracking that initiates at the bottom of the HMA layer and propagates to the surface under repeated load applications. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures. Fatigue cracking is highly dependent on the effective asphalt content by volume and air voids.

- **Transverse Cracking:** Thermal cracks typically appear as transverse cracks on the pavement surface due to low temperatures, hardening of the asphalt, and/or daily temperature cycles. Excessive transverse cracking may adversely affect ride quality.

The designer should examine the results to evaluate if the performance criteria for each of the above-mentioned indicators are met at the desired reliability. **If alligator fatigue cracking or transverse cracking criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.**

The output report also includes the monthly accumulation of the following secondary distress types and smoothness indicators at their mean values and chosen reliability for the entire design period:

- **Permanent Deformation**: The report includes HMA rutting and total permanent deformation (includes rutting on unbound layers and subgrade). Excessive rutting may cause safety concerns.

- **Surface-Initiated Fatigue Cracking or Longitudinal Cracking**: These load-related cracks appear at the HMA surface and propagate downwards. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures.

- **IRI**: The roughness index represents the profile of the pavement in the wheel paths. Higher IRI indicates unacceptable ride quality.

The designer should examine the results to evaluate if the performance criteria for permanent deformation, surface-initiated fatigue cracking or longitudinal cracking, and IRI meet the minimum of 14 years at the desired reliability. If any of the criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.

Another important output is the reliability level of each performance indicator at the end of the design period. If the reliability value predicted for the given performance indicator is greater than the target/desired value, the trial design passes for that indicator. If the reverse is true, then the trial design fails to provide the desired confidence and the performance indicator will not reach the critical value during the pavement's design life. In such an event, the designer needs to alter the trial design to correct the problem.

The strategies for modifying a trial design are discussed in **Section 6.9 Modifying Trial Designs**. The designer can use a range of thicknesses to optimize the thickness of the trial design to make it more acceptable. In addition, the software allows the designer to perform a sensitivity analysis on the key inputs. The results of the sensitivity analysis can be used to further optimize the trial design if modifying AC thickness alone does not produce a feasible design alternative. A detail description of thickness optimization procedure and sensitivity analysis is provided in the *Software HELP Manual*.

6.9 Modifying Trial Designs

An unsuccessful trial design may require revisions to ensure all performance criteria are satisfied. The trial design is modified by systematically revising the design inputs. In addition to layer thickness, many other design factors influence performance predictions. The design acceptance is distress-specific; in other words, the designer needs to first identify the performance indicator that failed to meet the performance target and modify one or more design inputs that has a significant impact on the given performance indicator. The impact of design inputs on performance indicators is typically obtained by performing a sensitivity analysis. Strategies used to produce a satisfactory design by modifying design inputs can be broadly categorized into to following:

- Pavement layer considerations
- Increasing layer thickness
- Modifying layer type and layer arrangement
- Foundation improvements (i.e. stabilize the upper subgrade soils)
- Pavement material improvements:
    - Use of higher quality materials (i.e. use of polymer modified asphalt, crushed stones)
    - Material design modifications (i.e. increase asphalt content, reduce amount of fines, modify gradations etc.)
    - Construction quality (i.e. reduce HMA air voids, increase compaction density, decrease as-constructed pavement smoothness)

Once again, when modifying the design inputs, the designer needs to be aware of the sensitivity of these inputs to various distress types. Changing a single input to reduce one distress may result in an increase in another distress. For example, the designer may consider using a harder asphalt to reduce HMA rutting, but that will likely increase the predicted transverse cracking. **Table 6.3 Modifying Flexible Pavement Trial Designs** presents a summary of inputs that may be modified to optimize trial designs and produce a feasible design alternative.

Table 6.3 Modifying Flexible Pavement Trial Designs

Distress/IRI	Design Inputs that Impact
AC Rutting	• Use a polymer modified asphalt for the HMA surface layer • Increase the dynamic modulus of the HMA mixture(s) • Reduce the asphalt content in the HMA mixture(s) • Increase the amount of crushed aggregate • Increase the amount of manufactured fines in the HMA mixture

Transverse Cracking	• Decrease the stiffness of the AC surface mix
	  ▪ Use a softer asphalt
	  ▪ Increase asphalt binder
	  ▪ Increase indirect tensile strength
	  ▪ Reduce creep compliance
	• Increase AC layer thickness

Alligator Cracking	• Increase HMA layer thickness
	• Increase HMA dynamic modulus for HMA layers thicker than 5 inches and decrease HMA dynamic modulus for HMA layers thinner than 3 inches
	• Revise the mixture design of the HMA base layer
	  ▪ Increase asphalt binder content
	  ▪ Achieve higher density and lower air voids during compaction
	  ▪ Use harder asphalt/polymer modified asphalt but ensure good compaction is achieved
	  ▪ Increase percent manufactured fines, and/or percent crushed aggregates
	• Reduce stiffness gradients between upper and lower layers
	  ▪ Using a higher quality/stiffer HMA layer on top of poor quality/low resilient modulus granular base or foundation tends to increase fatigue cracking
	• Increase the thickness or stiffness of a high quality unbound base layer and/or use a stabilized layer

Unbound Base Rutting	• Increase the resilient modulus of the aggregate base • Increase the density of the aggregate base • Stabilize the upper foundation layer for weak, frost susceptible, or swelling soils • Place a layer of select embankment material with adequate compaction • Increase the HMA or granular layer thickness • Address drainage related issues to protect from the detrimental effects of moisture

Subgrade Rutting	• Increase the layer stiffness and layer thickness of any layers above the subgrade layers:
	  ▪ Increase HMA and/or unbound layer thickness or stiffness
	  ▪ Include a stabilized drainable base

Distress/IRI	Design Inputs that Impact
	• Improve the engineering properties of the subgrade material: ▪ Increase the stiffness (modulus) of the subgrade layer(s) itself through the use of lime stabilized subgrade ▪ Effective use of subsurface drainage systems, geotextile fabrics, and impenetrable moisture barrier wraps to protect from the detrimental effects of moisture ▪ Increase the grade elevation to increase the distance between the subgrade surface and ground water table
IRI	• Reduce initial IRI (achieving smoother as-constructed pavement surface through more stringent smoothness criteria) • Improve roadbed foundation (replace frost susceptible or expansive subgrade with non-frost susceptible or stabilized subgrade materials) • Place subsurface drainage system to remove ground water

Figure 6.10 Sensitivity of HMA Alligator Cracking to Truck Volume through **Figure 6.31 Sensitivity of HMA IRI to Base Thickness**. **Figure 6.29 Sensitivity of HMA IRA to AC Thickness** presents sensitivity plots of a sample flexible pavement trial design showing the effects of key inputs, such as traffic volume, asphalt binder content, asphalt binder grade, air voids, base type, base thickness, and climate on key distresses/IRI. **Note:** The plots do not exhaustively cover the effects of all key factors on flexible pavement performance; other significant factors are not shown herein.

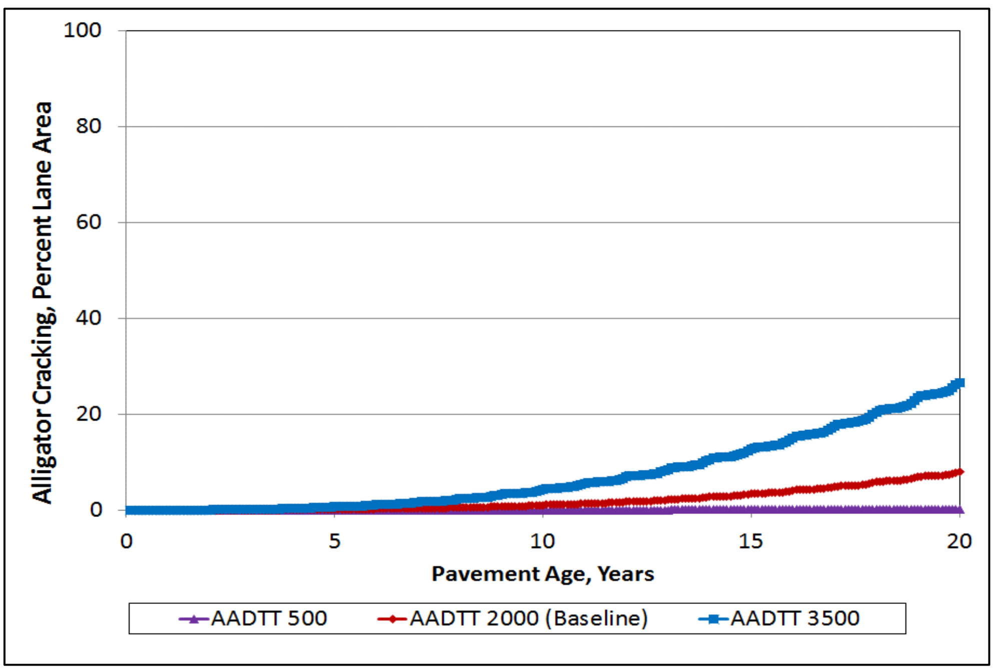


Figure 6.10 Sensitivity of HMA Alligator Cracking to Truck Volume

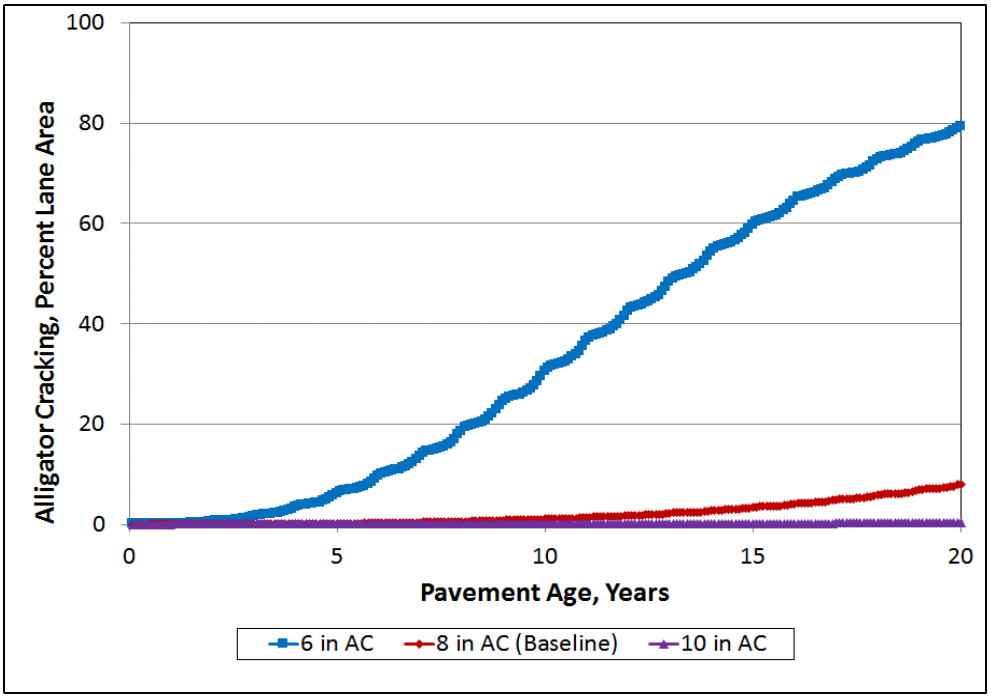


Figure 6.11 Sensitivity of HMA Alligator Cracking to AC Thickness

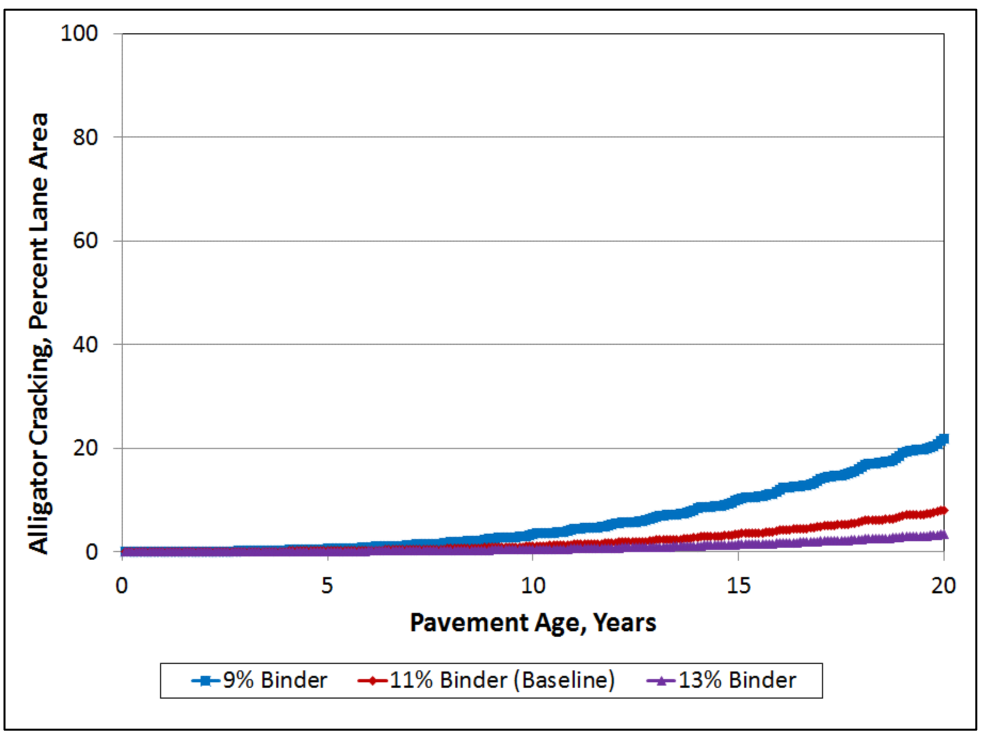


Figure 6.12 Sensitivity of HMA Alligator Cracking to Asphalt Binder Content

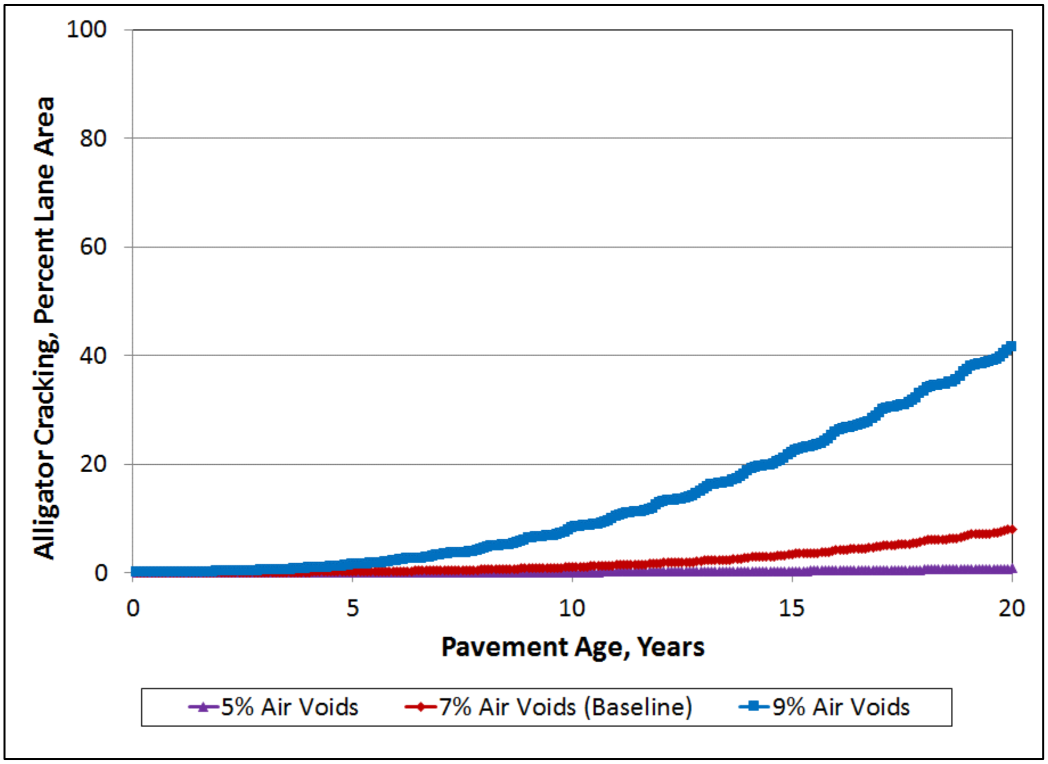


Figure 6.13 Sensitivity of HMA Alligator Cracking to Air Voids

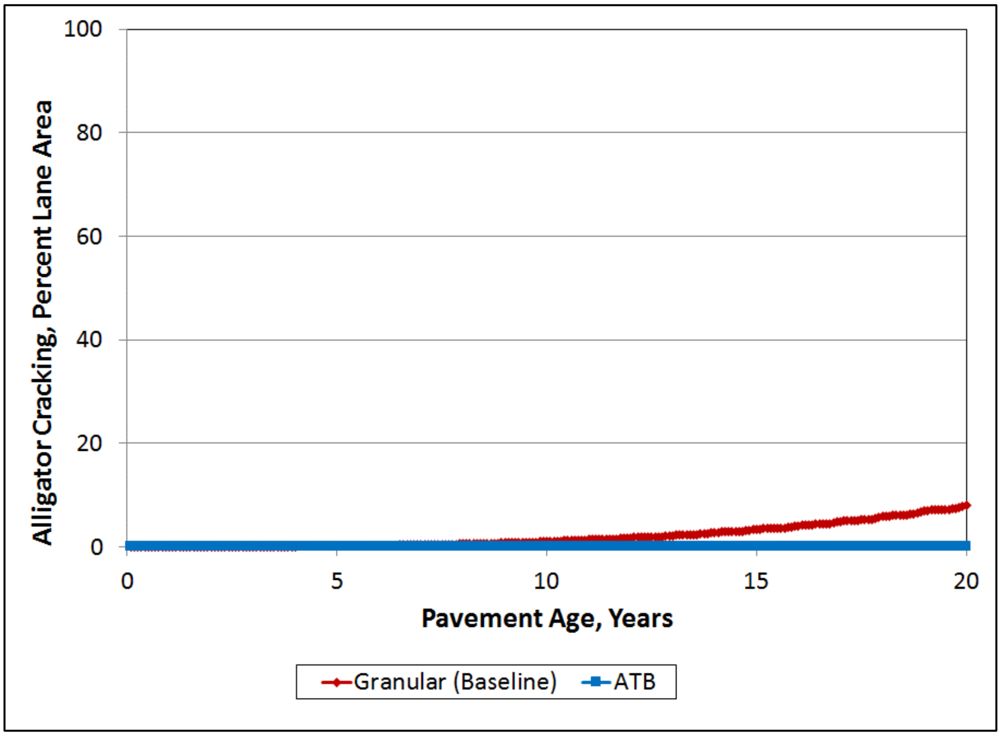


Figure 6.14 Sensitivity to HMA Alligator Cracking to Base Type

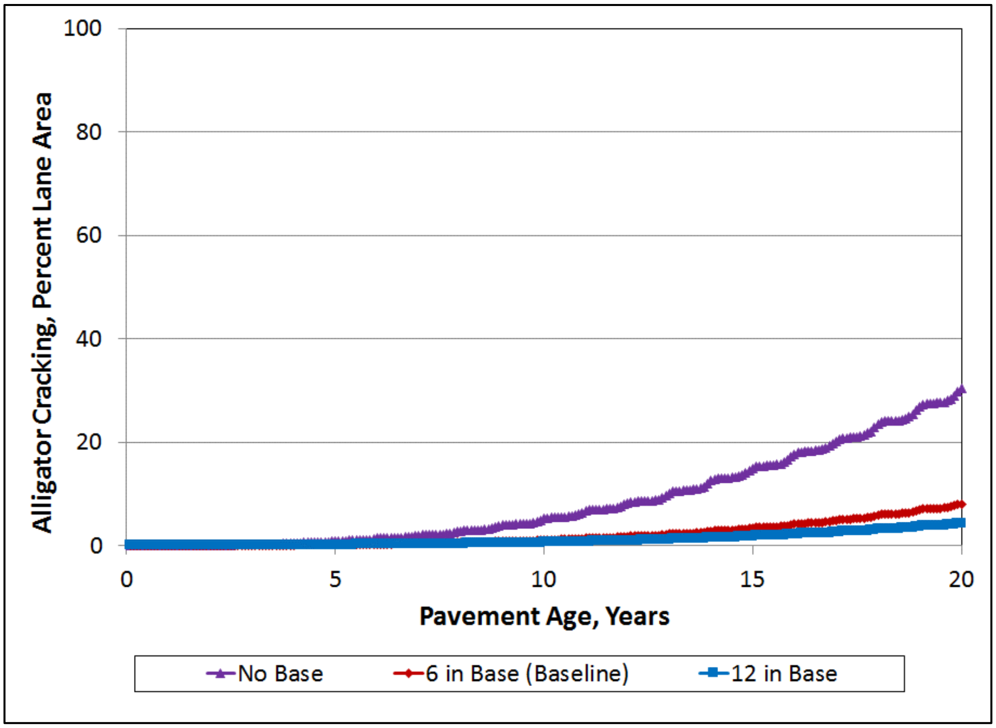


Figure 6.15 Sensitivity of HMA Alligator Cracking to Base Thickness

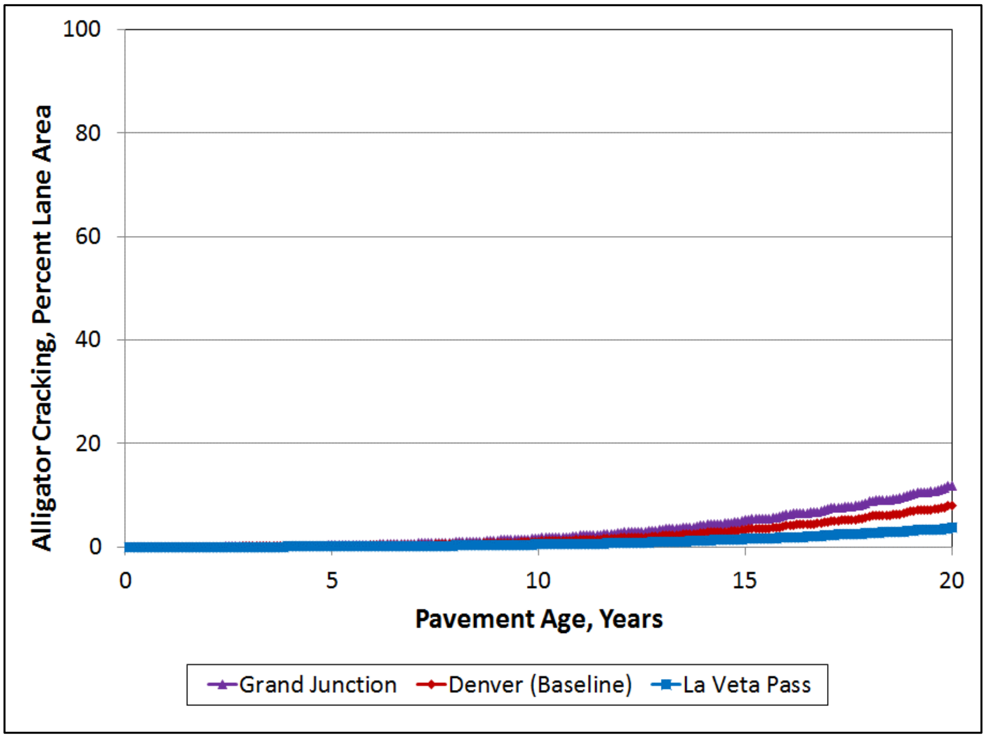


Figure 6.16 Sensitivity of HMA Alligator Cracking to Climate

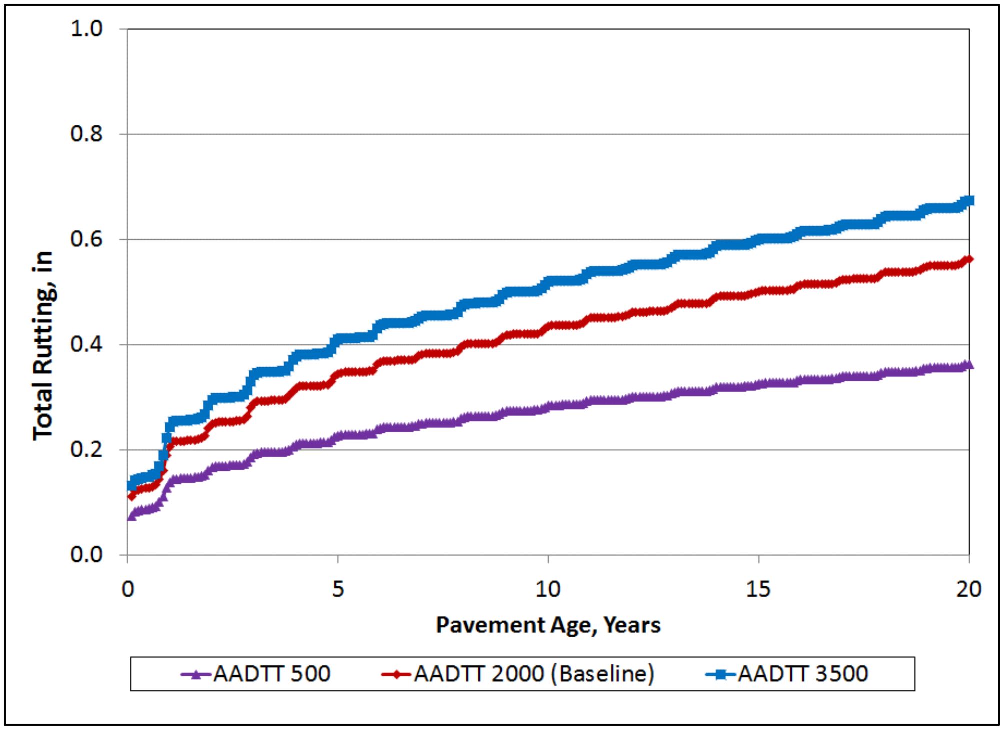


Figure 6.17 Sensitivity of Total Rutting to Truck Volume

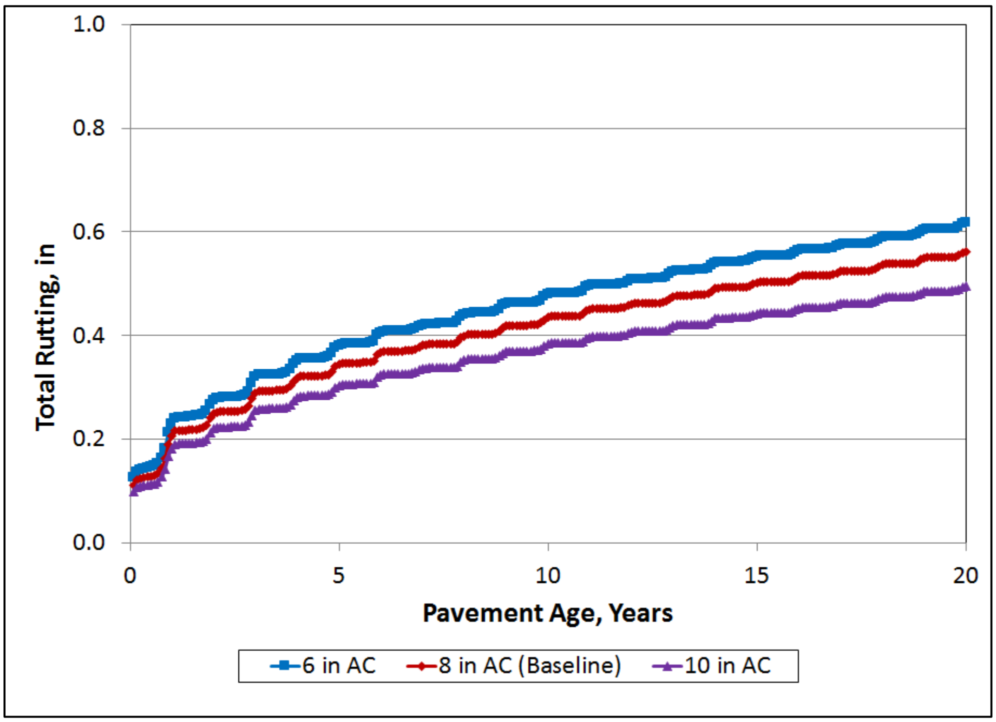


Figure 6.18 Sensitivity of Total Rutting to AC Thickness

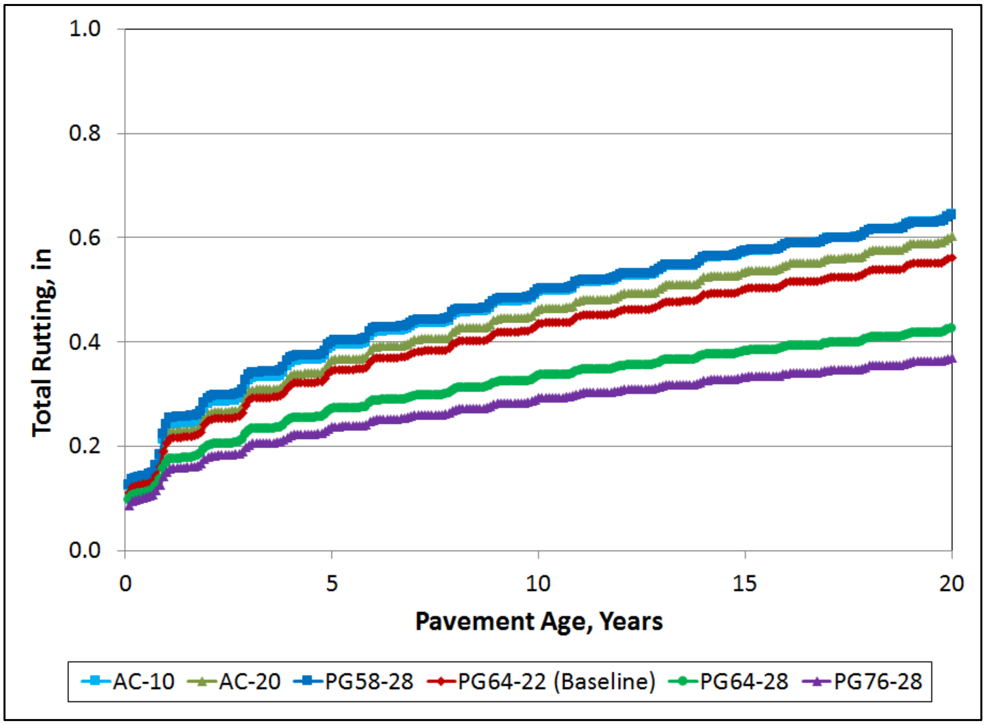


Figure 6.19 Sensitivity of Total Rutting to Asphalt Binder Grade

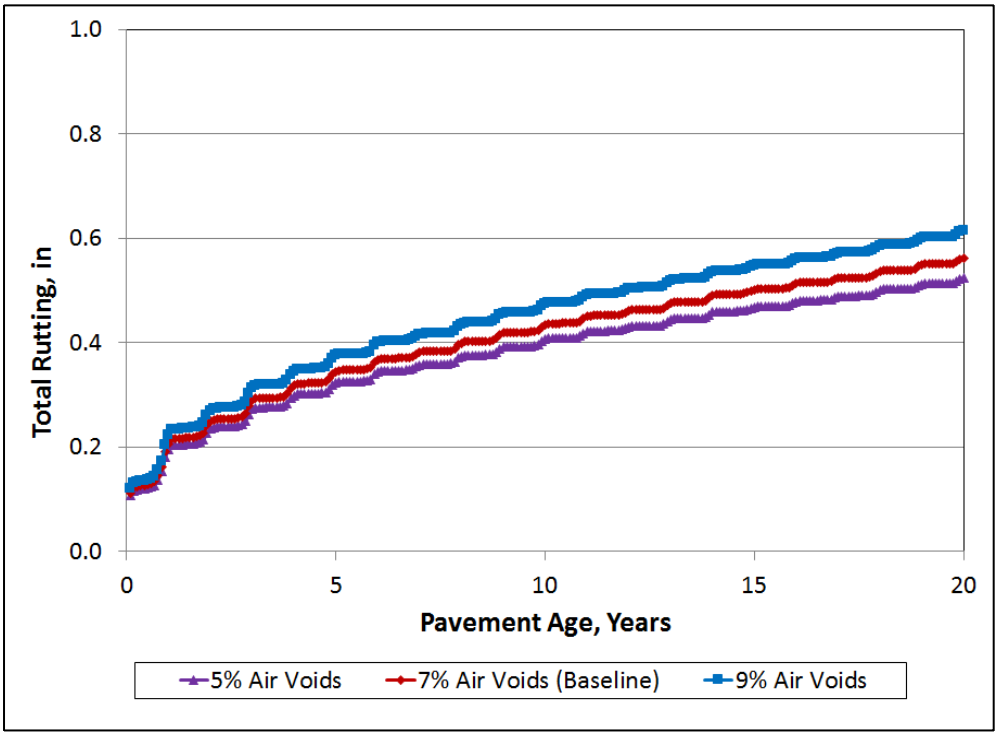


Figure 6.20 Sensitivity of Total Rutting to Air Voids

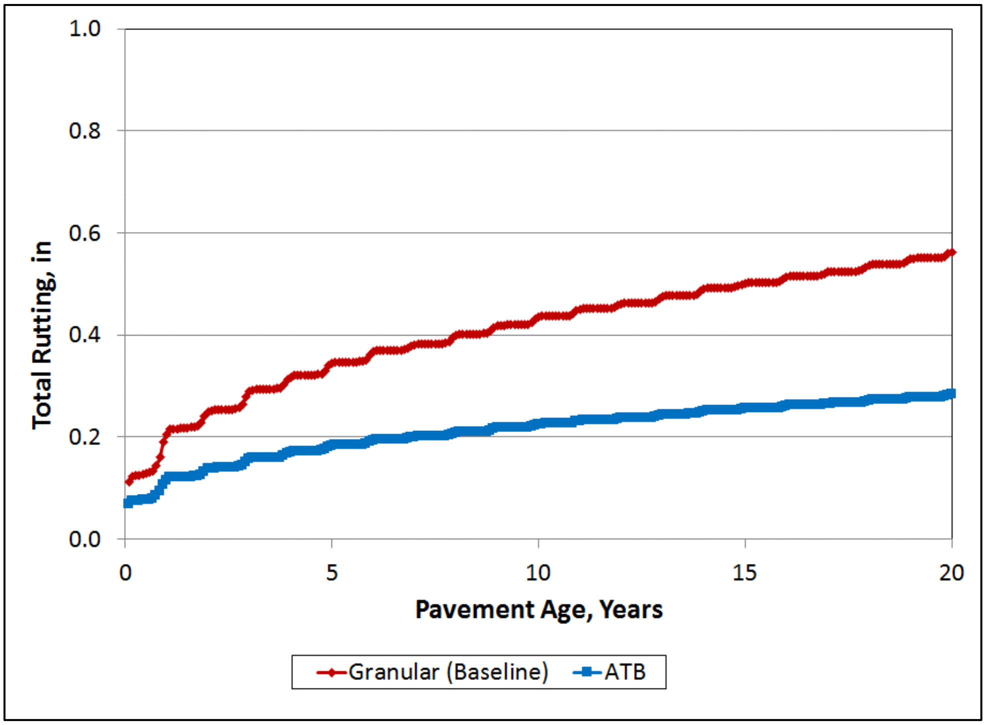


Figure 6.21 Sensitivity of Total Rutting to Base Type

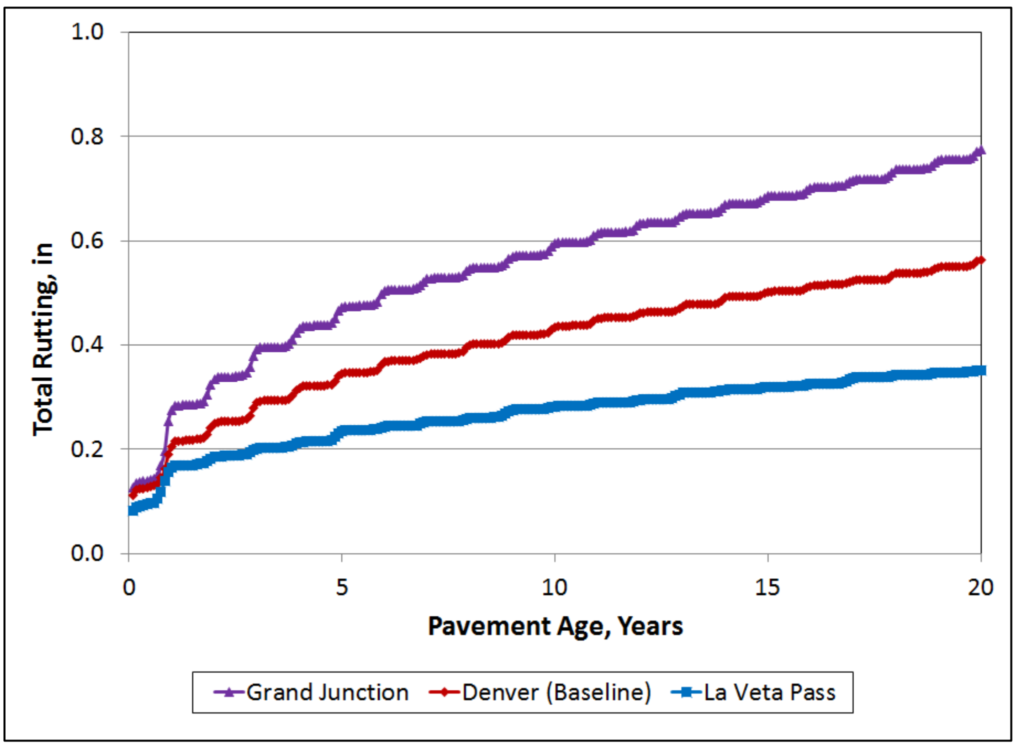


Figure 6.22 Sensitivity of Total Rutting to Climate

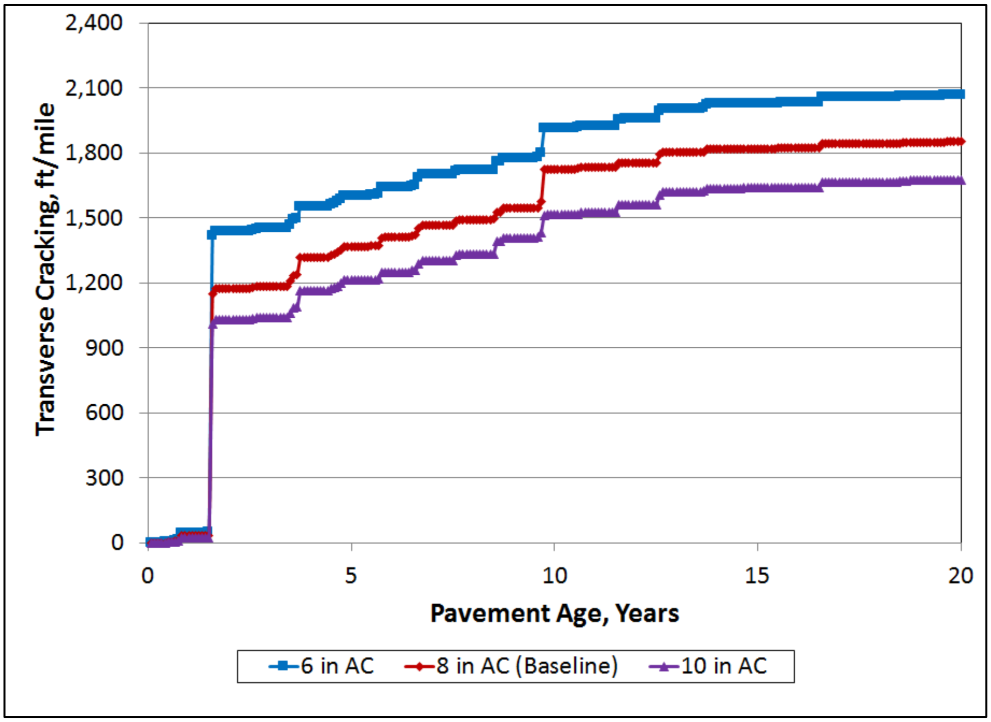


Figure 6.23 Sensitivity of HMA Transverse Cracking to Thickness

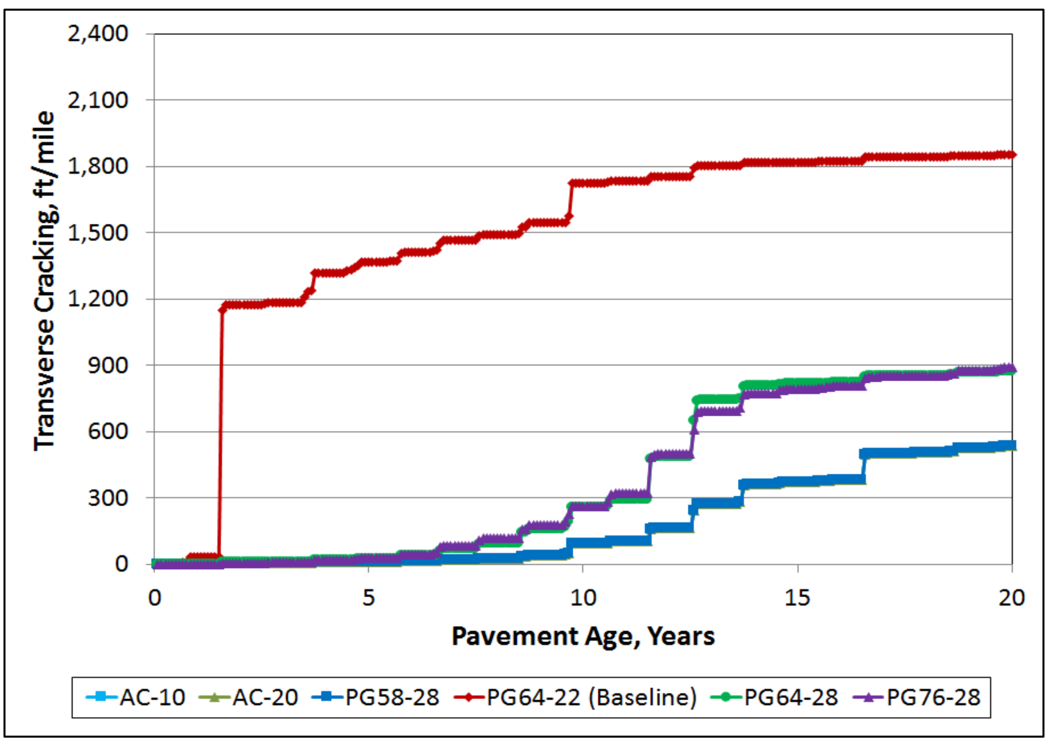


Figure 6.24 Sensitivity of HMA Transverse Cracking to Asphalt Binder Grade

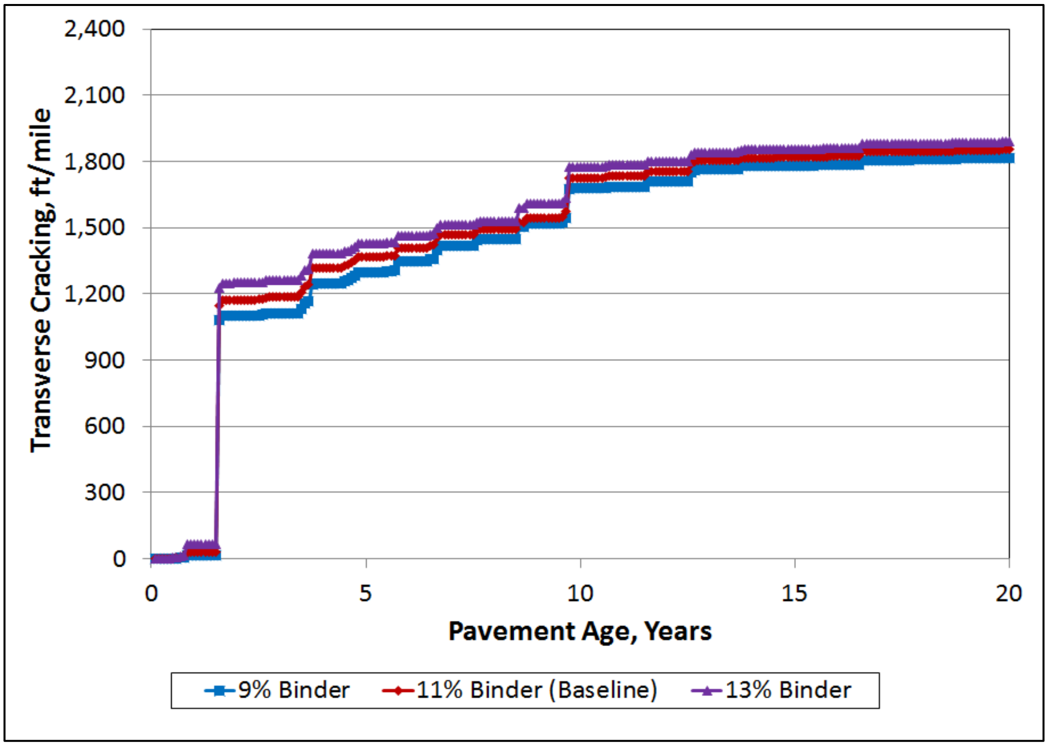


Figure 6.25 Sensitivity of HMA Transverse Cracking to Asphalt Binder Content

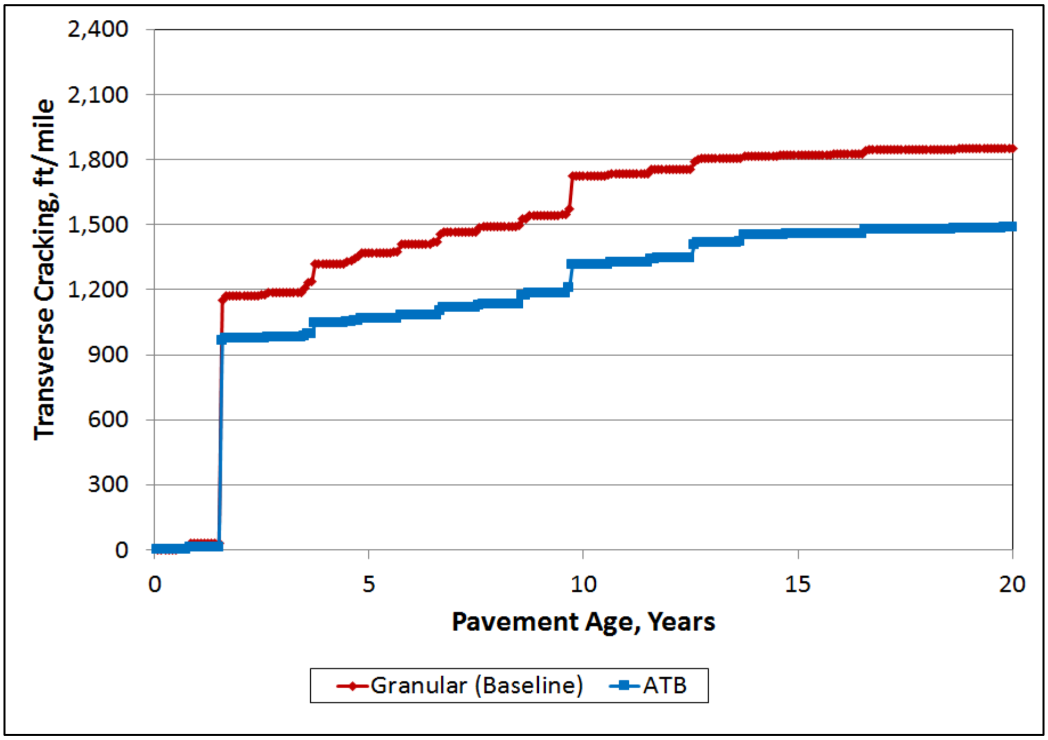


Figure 6.26 Sensitivity of HMA Transverse Cracking to Base Type

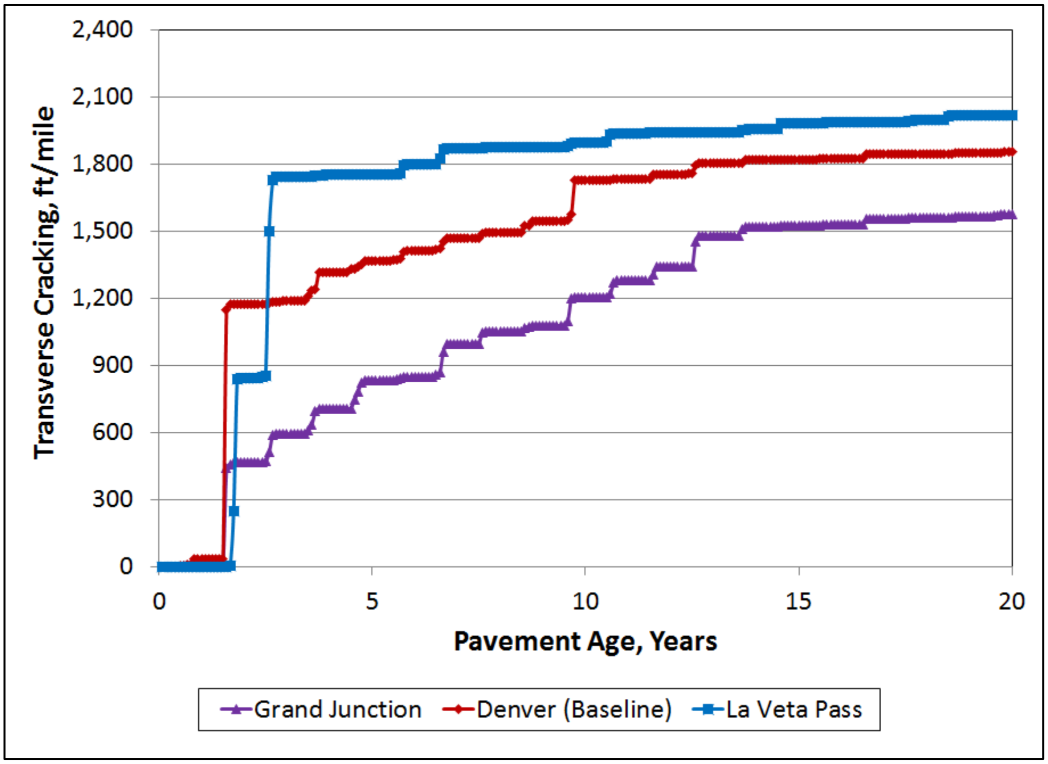


Figure 6.27 Sensitivity of HMA Transverse Cracking to Climate

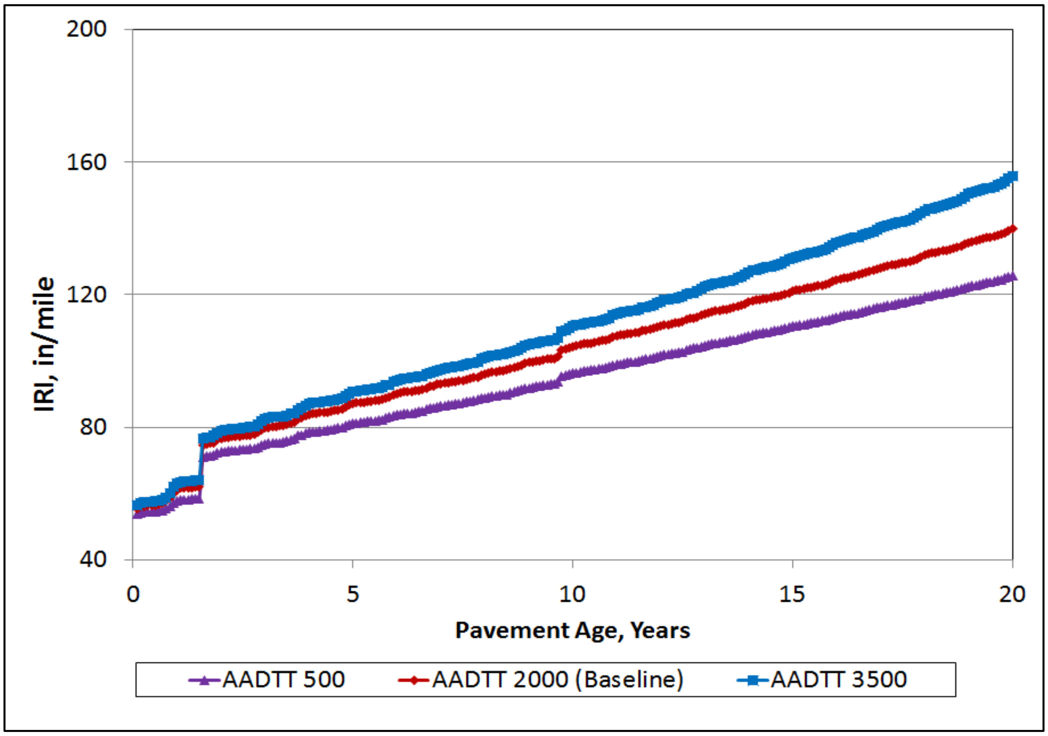


Figure 6.28 Sensitivity of HMA IRI to Truck Volume

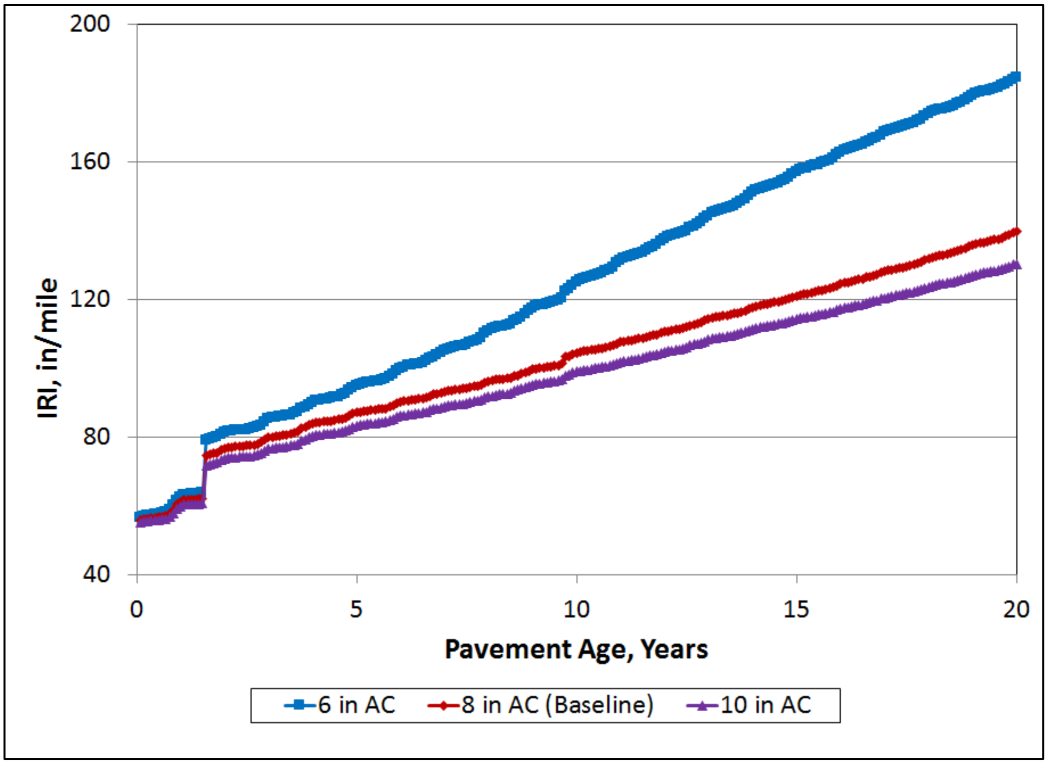


Figure 6.29 Sensitivity of HMA IRI to AC Thickness

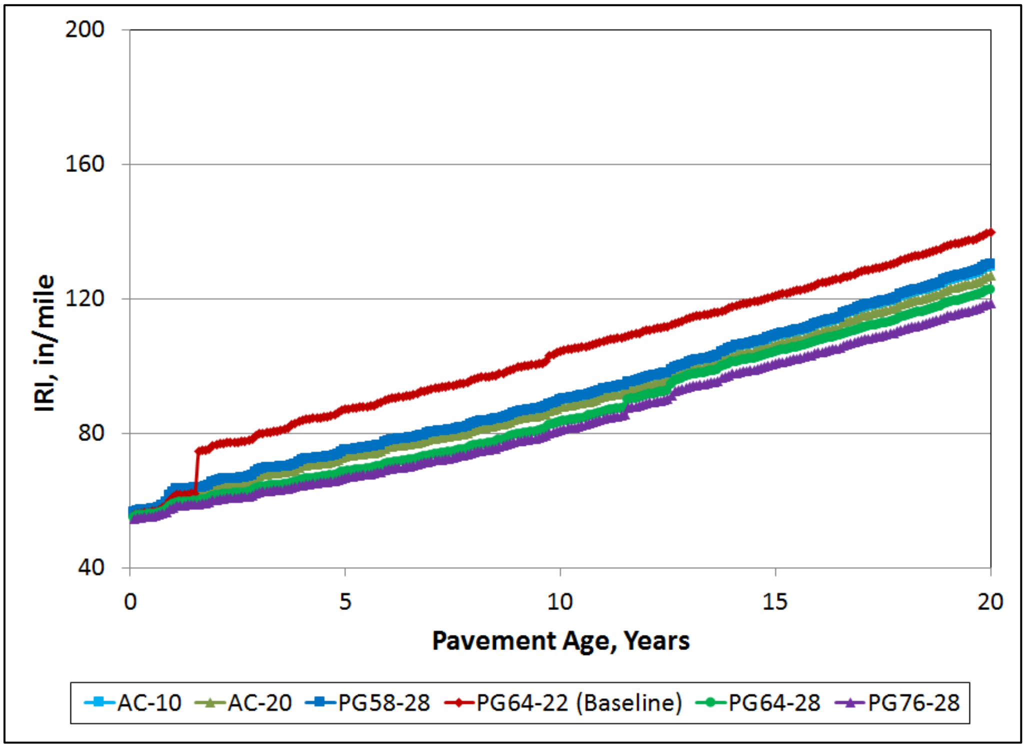


Figure 6.30 Sensitivity of HMA IRI to Asphalt Binder Grade

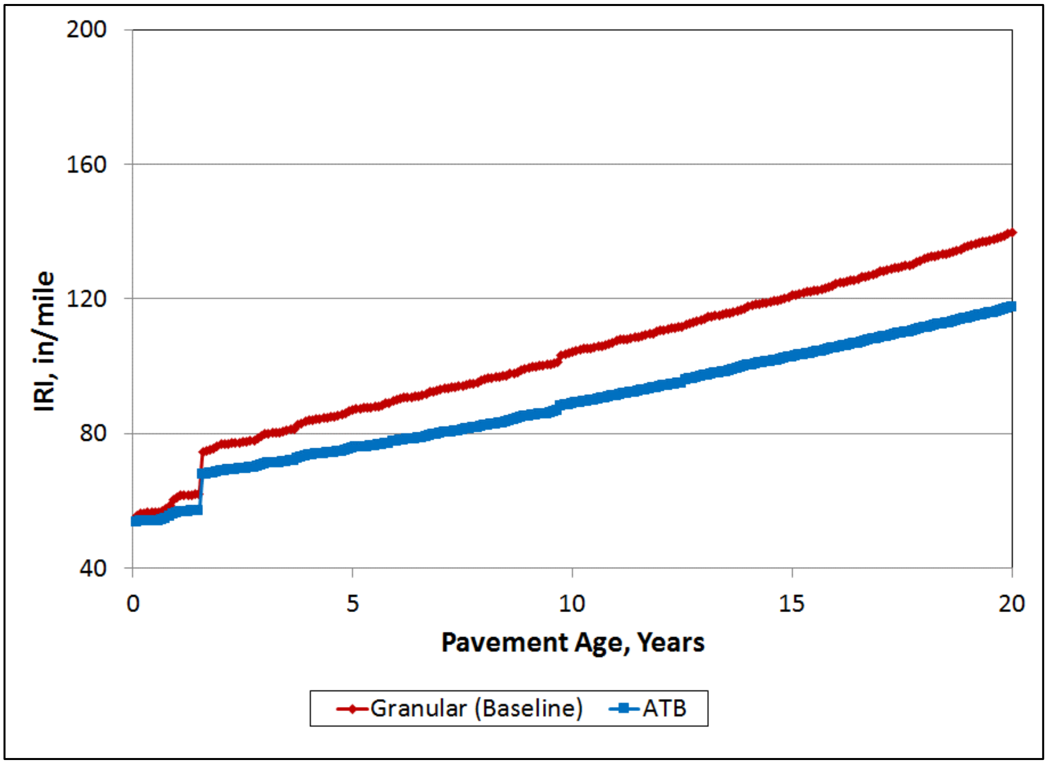


Figure 6.31 Sensitivity of HMA IRI to Base Thickness

6.10 HMA Thickness with ABC

As a minimum, the designer should include 4 inches of ABC for any thickness of HMA when the design truck traffic is less than 500 trucks per day. Six inches of ABC should be used for any thickness of HMA when the design truck traffic is greater than 500 trucks per day.

6.11 Required Minimum Thickness of Pavement Layer

Compaction of a hot mix asphalt pavement during its construction is the single most important factor affecting the ultimate performance of the pavement. Achieving adequate compaction increases pavement performance by decreasing rutting, reducing damage due to moisture and oxidation, and increasing the stability of the mix. Factors affecting the cooling rate of the mat include the layer thickness, the temperature of the mix when placed, ambient temperature, temperature of the base, and wind conditions. Layer thickness is the single most important variable in the cooling rate of an asphalt mat, especially for thin layers. This is especially true in cool weather because thin layers of an asphalt mat have less capacity to retain heat than thicker lifts of pavement. The thicker layers of an asphalt mat help to maintain the temperature at a workable level, thus increasing the time available for compaction. Because of the increased difficulty in achieving density and the importance of achieving compaction, a minimum layer thickness for construction of HMA pavement is two inches. A designer of special mixes, such as stone matrix

The suggested thicknesses for bicycle paths is shown on **Table 6.4 Minimum Thickness for Bicycle Paths.**

Table 6.4 Minimum Thicknesses for Bicycle Paths

Design Truck Traffic	Hot Mix Asphalt Pavement (inches)	Aggregate Base Course (inches)
Multi-use sidewalks [1]	4.0	6.0
Sidewalks [2]	3.0	6.0 [3]

Notes:
[1] Maintenance vehicles may include light duty trucks.
[2] Pedestrian and bicycle only, typical snow removal equipment would be a snow blower.
[3] May be reduced to 3.0 inches in thickness if suitable subgrade exists and approved by the RME.

asphalt or thin lift HMA should look at minimum thickness requirements of the particular product. The minimum thickness of these special mixes is likely to be a dimension other than two inches.

6.12 Asphalt Materials Selection

6.12.1 Aggregate Gradation

Definitions of Aggregate Size:

- **Nominal Maximum Aggregate Size (NMAS):** The size of aggregate of the smallest sieve opening through which the entire amount of aggregate is permitted to pass.
  - **Note:** For Item 403 - HMA and SMA, the Nominal Maximum Size is defined as one sieve size larger than the first sieve to retain more than ten percent of the aggregate.
- **Maximum Aggregate Size** is defined as one size larger than nominal maximum size. The flexible pavement usually consists of ¾ inch nominal maximum aggregate size (NMAS) in the lower layers, with a hot mix asphalt (HMA) Grading S. The top surface layer, should be either stone matrix asphalt (SMA) or a Grading SX. SMA mixes are often used in areas expected to experience extreme traffic loading. When low to high traffic loads are expected, a ½ inch NMAS, Grading SX should be used.

CDOT uses the No. 30 sieve as one of the job-mix formula tolerance sieves. **Table 6.5 Master Range Table for Stone Mix Asphalt**. is based (with some exceptions) on NCHRP No. 4 and ³/₈ inch and AASHTO ½ inch and ¾ inch SMA gradations ranges, where the No. 30 sieve range is included in the ½ inch and ¾ inch gradations.

SMA Gradation Nomenclature Example:
The ¾ inch (19.0 mm) gradation is named the ¾ inch Nominal Maximum Aggregate Size gradation because the first sieve that retains more than 10 percent is the ½ inch sieve, and the next sieve larger is the ¾ inch sieve, refer to **Table 6.5 Master Range Table for Stone Mix Asphalt**.

A CDOT study (1) found less thermal segregation in the top lift when Grading SX mixes were used. HMA Grading SX can also be used where layers are very thin or where the pavement must taper into an existing pavement. A study from Auburn University (2) found little difference in the stability or rutting of ¾ inch and ½ inch NMAS mixes. CDOT cost data for 2005 showed a slight increase in the cost per ton of Grading SX mixes as compared to Grading S mixes with the same bid quantities.

HMA with a 1-inch NMAS, Grading SG, should not be used in the surface layer. Although Grading SG mixes have been used in specialized situations, they are not currently used or accepted on a regular basis for pavement mixes. CDOT has found that the production and placement of Grading SG mixes are prone to segregation and the use should be discouraged.

Table 6.5 Master Range Table for Stone Matrix Asphalt

Sieve Size	**Percent by Weight Passing Square Mesh Sieves**			
	#4 (4.75 mm) Nominal Maximum	**3/8" (9.5 mm) Nominal Maximum**	**1/2" (12.5 mm) Nominal Maximum**	**3/4" (19.0 mm) Nominal Maximum**
1 " (25 mm)	-	-	-	100
¾" (19.0 mm)	-	-	100	90-100
½" (12.5 mm)	100	100	90-100	50-88
3/8" (9.5 mm)	100	90-100	50-80	25-60
#4 (4.75 mm)	90-100	26-60	20-35	20-28
#8 (2.36 mm)	28-65	20-28	16-24	16-24
#16 (1.18mm)	22-36	-	-	-
#30 (600 µm)	18-28	12-18	12-18	12-18
#50 (300 µm)	15-22	10-15	-	-
#100 (150 µm)	-	-	-	-
#200 (75 µm)	12-15	8-12	8-11	8-11

For structural overlays, the minimum allowed layer thickness will be 2 inches. For functional overlays used in preventive maintenance or other treatments, thinner lifts are allowed.

Table 6.6 HMA Grading Size and Location Application and **Table 6.7 HMA Grading Size and Layer Thickness** gives guidance for mix selection and recommended layer thicknesses for various layers and nominal maximum aggregate sizes.

Table 6.6 HMA Grading Size and Location Application

CDOT HMA Grade	Nominal Maximum Aggregate Size (NMAS)	Application
SF	No. 4 sieve	Leveling course, rut filling, scratch course, etc.
ST	3/8 inch	Thin lifts and patching
SX	½ inch	Top layer (preferred)
S	¾ inch	Top layer, layers below the surface, patching
SG	1 inch	Layers below the surface, deep patching

Table 6.7 HMA Grading Size and Layer Thickness

CDOT HMA Grade	Nominal Maximum Aggregate Size (NMAS)	Overlay Layer Thickness (inches)	
		Minimum	Maximum
SX	½ inch	1.50	3.00
S	¾ inch	2.25	3.50
SG	1 inch	3.00	4.00
SF	No. 4 sieve	0.75 [1]	1.50
ST	3/8 inch	1.125	2.50
Note: [1] Layers of SF mixes may go below 1 inch as needed to taper thin lift to site conditioning (i.e. rut filling).			

6.12.2 Selection of SuperPave™ Gyratory Design

To choose the appropriate number of revolutions of a SuperPave™ gyratory asphalt mix design on a particular project, determining the design 18k ESALs and the high temperature environment for the project is necessary. The following steps should be followed to determine the proper SuperPave™ gyratory design revolutions for a given project:

Step 1. Determine 18k ESALs: In order to obtain the correct SuperPave™ gyratory compaction effort (revolutions), the 18k ESALs **must** be a 20-year cumulative 18k ESAL of the design lane in one direction. The compaction effort simulates the construction compaction roller to obtain the correct voids properties to resist the intended traffic in the design lane. The department's traffic analysis unit of the Division of Transportation Development (DTD) automatically provides an ESAL calculator. One must use a 20-year design, appropriate number of lanes, and a specified flexible pavement. Even a 10-year asphalt overlay must use a 20-year cumulative 18k ESAL number for the design lane.

Step 2. Reliability for the 7-Day Average Maximum Air Temperature: The next decision is to determine the type of project being designed. For new construction or reconstruction, asphalt cement with 98 percent reliability for low and high temperature properties is recommended. For overlays, asphalt cement with 98 percent reliability for high temperature properties (rutting resistance) and 50 percent reliability for low temperature properties (cracking resistance) is recommended. Asphalt cements with lower than 98 percent reliability against rut resistance should not be specified. In the SuperPave™ system, anything between 50 percent and 98 percent reliability is considered 50 percent reliability for the purpose of binder selection. The low temperatures are specified at a lower reliability for overlays because of reflection cracking.

Step 3. Determine Weather Data for the Project: Obtain the highest 7-day average maximum air temperature, based on weather data in the project area from the computer program LTPPBind 3.1 (beta). Refer to **Section 6.12.3 Binder Selection** for a further explanation of LTPPBind 3.1 (beta). From the appropriate high temperature, find the environmental category for the project from **Table 6.8 Environmental Categories**. The Environmental Categories are from CDOT Pavement Management Program's Environmental Zones. The Environmental Zones (Categories) are one of four pavement groupings used to group pavements into families that have similar characteristics.

Table 6.8 Environmental Categories

Highest 7-Day Average Air Temperature	High Temperature Category
> 97°F (> 36°C)	Hot (southeast and west)
> 88° to 97°F (> 31° to 36°C)	Moderate (Denver, plains and west)
81° to 88°F (27° to 31°C)	Cool (mountains)
< 81°F (< 27°C)	Very Cool (high mountains)

Step 4. Selection of the Number of Design Gyrations (N_{DES}): Select the N_{DES} from **Table 6.9 Recommended SuperPave™ Gyratory Design Revolution (N_{DES})**. For example, Table 6.7 shows that for 5,000,000 18k ESALs and a high temperature category of "Cool", the design revolutions should be 75.

Table 6.9 Recommended SuperPave™ Gyratory Design Revolution (N_{DES})

CDOT Pavement Management System Traffic Classification (20 Year Design ESAL)	20 Year Total 18k ESAL in the Design Lane	High Temperature Category			
		Very Cool	Cool	Moderate	Hot
Low	< 100,000	50	50	50	50
	100,000 to < 300,000	50	75	75	75
Medium	300,000 to < 1,000,000	75	75	75	75
	1,000,000 to < 3,000,000	75	75	75	100
High	3,000,000 to < 10,000,000	75	75	100	100
Very High	10,000,000 to < 30,000,000	---	---	100	---
Very Very High	≥ 30,000,000	---	---	125	---

Note: Based on *Standard Practice for SuperPave™ Volumetric Design for Hot-Mix Asphalt (HMA)*, AASHTO Designation R 35-04.

6.12.3 Binder Selection

Performance graded (PG) binders have two numbers in their designation, such as PG 58-34. Both numbers describe the pavement temperatures in degrees Celsius at which the pavement must perform. The first number (58 in the example) is the high temperature standard grade for the pavement, and the second number (minus 34 in the example) is the low temperature standard grade. PG 64-28 (rubberized) or PG 76-28 (polymerized) or bituminous mixtures should only be placed directly on an existing pavement or milled surface that does not show signs of stripping or severe raveling. Cores should be taken to determine if stripping is present. Because of a limited number of tanks, Colorado local suppliers only have the capacity to supply a limited number of asphalt cement grades. **Table 6.10 Available Asphalt Cement Grades in Colorado** shows available grades that maybe used and/or available on CDOT projects.

Table 6.10 Available Asphalt Cement Grades in Colorado

Polymer Modified	Unmodified
PG 76-28 PG 70-28 PG 64-28 PG 58-34	PG 64-22 PG 58-28

Note: The Region Materials Engineer may select a different gyratory design revolution for the lower HMA lifts.

LTPPBind 3.1 (beta) is a working version, dated September 15, 2005. Beta only means it is going through the 508-compliance process for the visually disabled users as required by the Federal Government. The computer program may be obtained from the following web address:
http://www.fhwa.dot.gov/pavement/ltpp/ltppbind.cfm

The program allows the user to select the asphalt binder grade for the appropriate project site conditions. In the *Preferences* under the *File* menu, use 12.5mm (½ inch) for the CDOT target rut depth default value. The computer program has a help menu to assist the user and supporting technical information regarding the computation of design temperatures required for the selection of the asphalt binder grade as provided in the *Climatic Data* and *Algorithms* sections. The algorithms are broken down under four subsections. Each algorithm equation is shown and briefly explained for high temperature, low temperature, PG with depth, and PG grade bumping.

- **High Temperature:** The high temperature is based on a rutting damage model. The LTPP high temperature model was not used in this version since it provided very similar results to the SHRP Model at 98 percent reliability. Initially, the user must select a preference for a target rut depth, but they have the option to change the target rut depth. The default is 12.5 mm (½ inches).

- **Low Temperature:** The low temperature is based on LTPP climatic data using air temperature, latitude, and depth to surface.

- **PG with Depth:** LTPP pavement temperature algorithms were used to adjust the PG for a depth into the pavement. The LTPP algorithms are empirical models developed from seasonal monitoring data.

- **PG Grade Bumping:** PG grade bumping was based on the rutting damage concept for high temperature adjustments. Adjustments were developed as the difference between PG for standard traffic conditions (ESAL of 3 million and high speed) and site conditions. 187 sites throughout the U.S. for five target rut depths were analyzed. The PG adjustments were then averaged by various ESAL ranges, traffic speeds, and Base PG.

The following steps should be followed to determine the proper SuperPave™ asphalt cement grade for a given project:

Step 1. Determine Proper Reliability to Satisfy Pavement Temperature Property Requirements: The first step is to determine what type of project is being designed.

- For new construction or reconstruction, asphalt cement with 98 percent reliability for both low and high pavement temperature properties is recommended.

- For overlays, asphalt cement with 98 percent reliability for high pavement temperature properties (rutting resistance) and 50 percent reliability for low pavement temperature properties (cracking resistance) are recommended.

- Asphalt cements with lower than 98 percent reliability against rut resistance should not be specified.

- In the SuperPave™ system, anything between 50 and 98 percent reliability is considered 50 percent reliability for the purpose of binder selection.

- The low pavement temperatures are specified at a lower reliability for overlays because of reflection cracking.

- Refer to **Figure 6.32 PG Binder Grades** for a graphical representation of reliability.

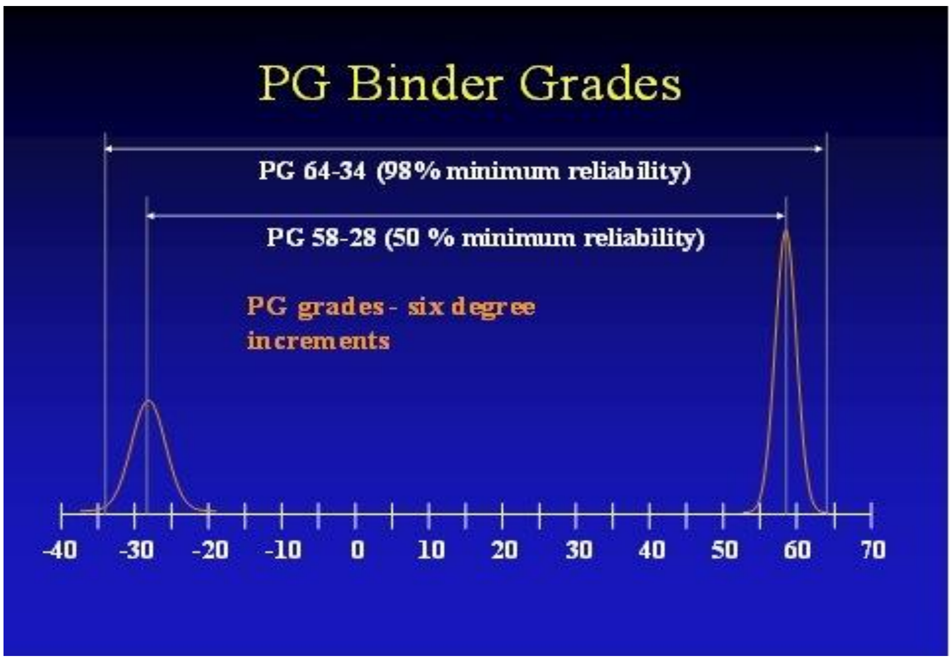


Figure 6.32 PG Binder Grades

Step 2. Determine Weather Data for the Project: Obtain the SuperPave™ recommended asphalt cement grade, based on weather data and traffic in the project area. Recommendations on 98 percent reliability high and low pavement temperature weather stations are found in **Figure 6.33 Colorado 98 Percent Reliability LTPP High Pavement Temperature Weather Station Models** and **Figure 6.34 Colorado 98 Percent Reliability LTPP Low Pavement Temperature Weather Station Models**, neither of which accounts for grade bumping. The program also calculates the reliability of various asphalt cements for a given location. This source will yield the 98 and 50 percent reliability asphalt cement for a project area with a free flowing traffic condition, which is described in Step 3. For example, when the recommendations call for a PG 58-22 for a given project, due to the available binder grades in Colorado, a PG 64-22 would be specified. This selection provides for rut resistance while preserving the same level of resistance to cracking. Because of the danger of rutting, in no case should the recommended high temperature requirements be lowered based on availability. Each RME has a copy of this program.

Step 3. Select Location of Roadway: Place the cross hair on the location of area of interest in the weather data program LTPP Bind. The program selects five weather stations surrounding the area of interest. The designer has the option to use any number of weather stations representative of the climate at the area of interest.

Step 4. Adjust HMA Performance Grade Binder to Meet Layer Depth, Traffic Flow and Loading Requirements: SuperPave™ high temperature reliability factors are based on historical weather data and algorithms to predict pavement temperature. At a depth layer of 1 inch or more below the surface, high temperature recommendations are changed because of the depth and temperatures at that depth.

For pavements with multiple layers a lesser grade may be specified for lower layers based on the amount of material needed and other economical design decisions. In many cases, the requirements for lower layers might be obtained with an unmodified or more economical grade of asphalt cement. It is recommended that at least 10,000 tons of mix in the lower layer is needed before a separate asphalt cement is specified for the lower layer.

Adjustments can be made to the base high temperature binder through the '*PG Binder Selection'* screen. Adjustments to reliability, depth of layer, traffic loading, and traffic speed (fast and slow) will be required. These adjustments are called grade bumping. Additional grade bumping may be performed for stop and go traffic characteristics such as intersections. This extra grade bump may be applied, but is suggested the designer have prior regional experience on doing such.

6.12.3.1 Example

Example: A new roadway project will be constructed near Sugarloaf Reservoir. It will have two lanes per direction and a traffic characteristic of slow moving because it is a winding mountain road. Find the appropriate binder grade. N_{DES} for the surface layer is obtained in the same manner as the previous example and has a design revolution of 75.

Step 1. Determine 18k ESAL: Design Lane ESALs = 4,504,504 from DTD web site (20 year 18k ESAL in the design lane).

Step 2. Use LTPP Software Database: Use LTPPBind software database to obtain the data from the nearest weather station, Sugarloaf Reservoir. Appropriate weather stations can be determined from information on state, county, coordinates, location, and/or station ID. **Figure 6.34 LTPP Interface Form for Weather Station Selection (Version 3.1)** is where the cross hair is placed for the new roadway project. **Figure 6.35 LTPP Weather Station Output Data (Version 3.1)** shows the data at the weather station Sugarloaf Reservoir.

Step 3. Select the Desired Weather Stations: The LTPPBind software gives the option to select the weather stations that provide the best weather data at the project location (see the upper table in **Figure 6.36 LTPP PG Binder Selection at 98 Percent Reliability**). Check the first three weather stations. Uncheck the two weather stations furthest from the project, these stations are too far from the site and not representative of site conditions.

Step 4. Select the Temperature Adjustments: Because this is a principal arterial and a new construction project, 98 percent reliability is chosen with a layer depth of zero (0) for the surface layer (see **Figure 6.36 LTPP PG Binder Selection at 98 Percent Reliability)**.

Step 5. Select the Traffic Adjustments for High Temperature: Select the appropriate traffic loading and traffic speed. The design lane ESALs are 4,504,504 and the traffic speed is slow. Grade bumping is automatic and is demonstrated by toggling in appropriate cells. The following data summarized in **Table 6.11 SuperPave™ Weather Data Summary** are obtained from Steps 1 through 5.

Step 6. Select Final Binder: Table 6.9 Available Asphalt Cement Grades in Colorado lists the binder grades available in Colorado. A PG 58-28 (unmodified) is available, but it does not meet the low temperature requirement. The lowest temperature binders available in Colorado can meet is -34° C. This is available in PG 58-34 (polymer modified). Therefore, at 98 percent reliability use PG 58-34.

Step 7. Find the Temperature that Falls into the Environmental Category: Use **Table 6.11 Environmental Categories (restated)** to obtain the highest 7-day average air temperature, 24.3°C. **Table 6.12 Environmental Categories (restated)** shows the temperature falls into the category 'Very Cool' (high mountains).

Step 8 Select the Gyratory Design Revolution (N_{DES}): Table 6.13 Recommended SuperPave™ Gyratory Design Revolution (N_{DES}) shows at 4,504,504 18k ESAL and a high temperature category of "Very Cool" the design revolutions should be 75.

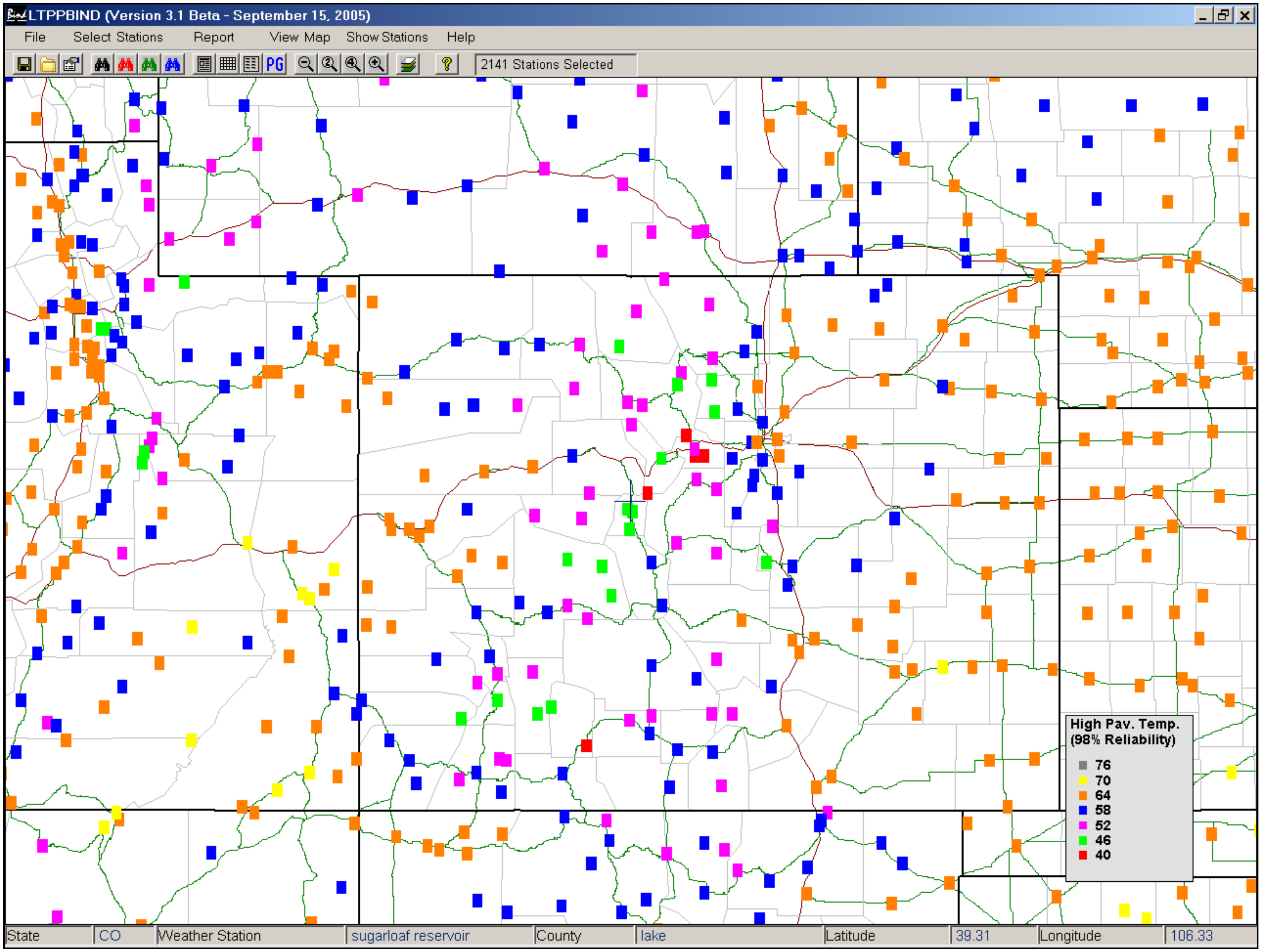


Figure 6.33 Colorado 98 Percent Reliability LTPP High Pavement Temperature Weather Station Models

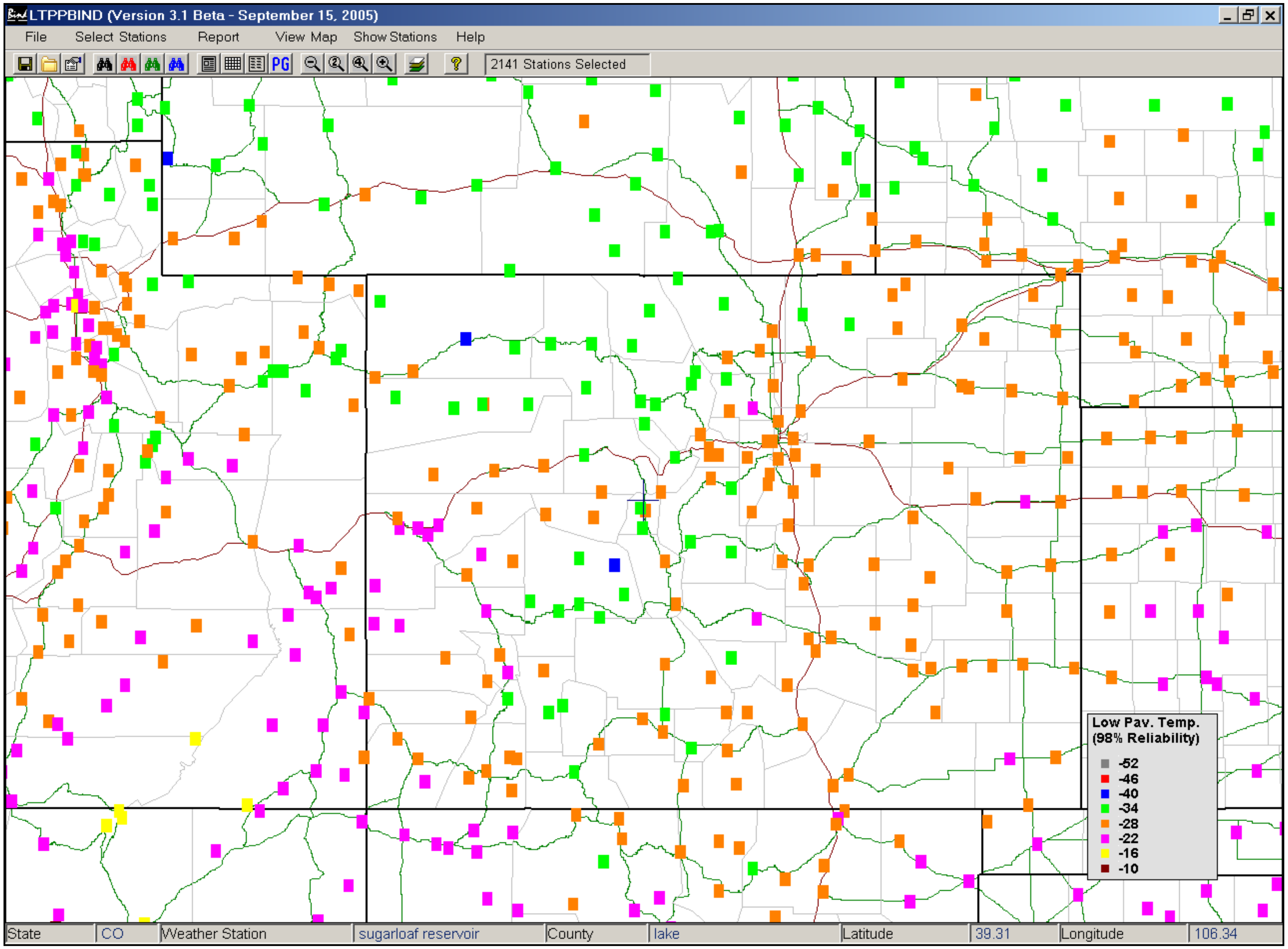


Figure 6.34 Colorado 98 Percent Reliability LTPP Low Pavement Temperature Weather Station Models

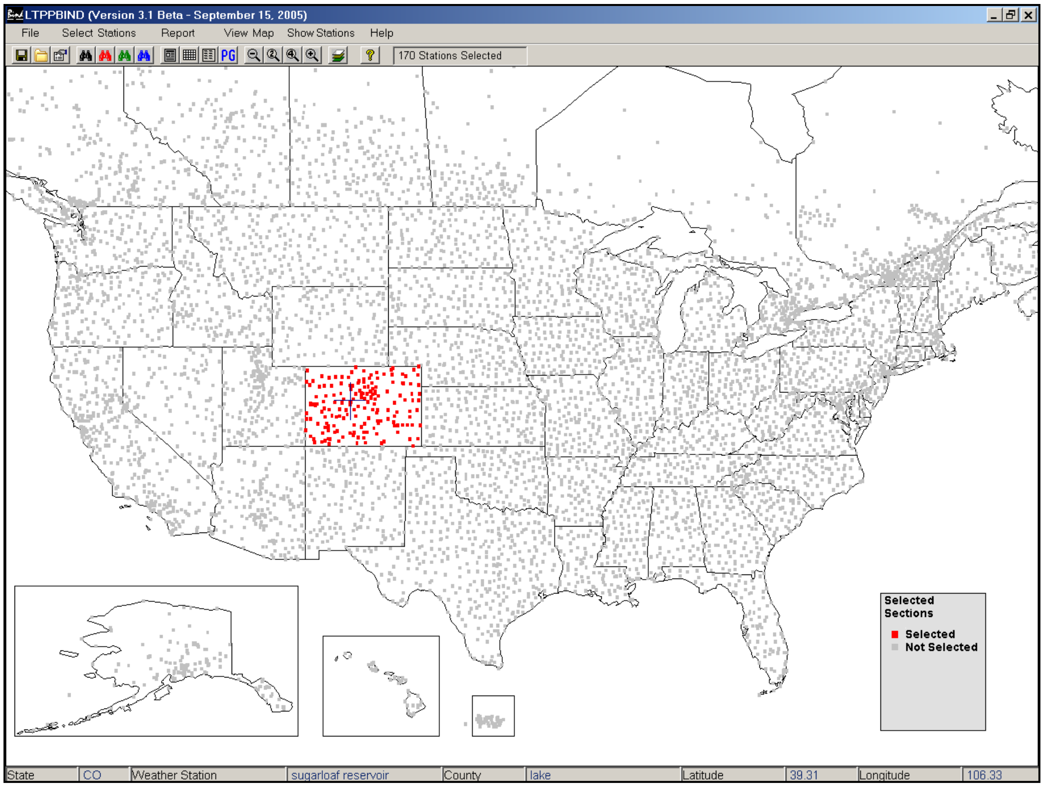
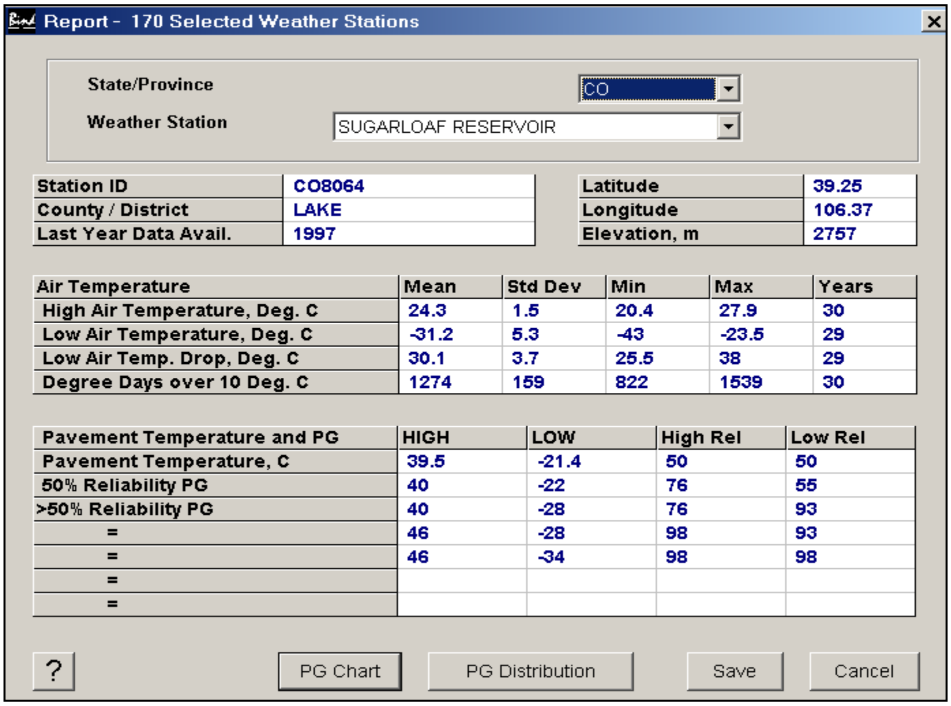


Figure 6.35 LTPP Interface Form for Weather Station Selection (Version 3.1)



Report - 170 Selected Weather Stations

State/Province: CO

Weather Station: SUGARLOAF RESERVOIR

Station ID	CO8064	Latitude	39.25
County / District	LAKE	Longitude	106.37
Last Year Data Avail.	1997	Elevation, m	2757

Air Temperature	Mean	Std Dev	Min	Max	Years
High Air Temperature, Deg. C	24.3	1.5	20.4	27.9	30
Low Air Temperature, Deg. C	-31.2	5.3	-43	-23.5	29
Low Air Temp. Drop, Deg. C	30.1	3.7	25.5	38	29
Degree Days over 10 Deg. C	1274	159	822	1539	30

Pavement Temperature and PG	HIGH	LOW	High Rel	Low Rel
Pavement Temperature, C	39.5	-21.4	50	50
50% Reliability PG	40	-22	76	55
>50% Reliability PG	40	-28	76	93
=	46	-28	98	93
=	46	-34	98	98
=				
=				

? PG Chart PG Distribution Save Cancel

Figure 6.36 LTPP Weather Station Output Data (Version 3.1)

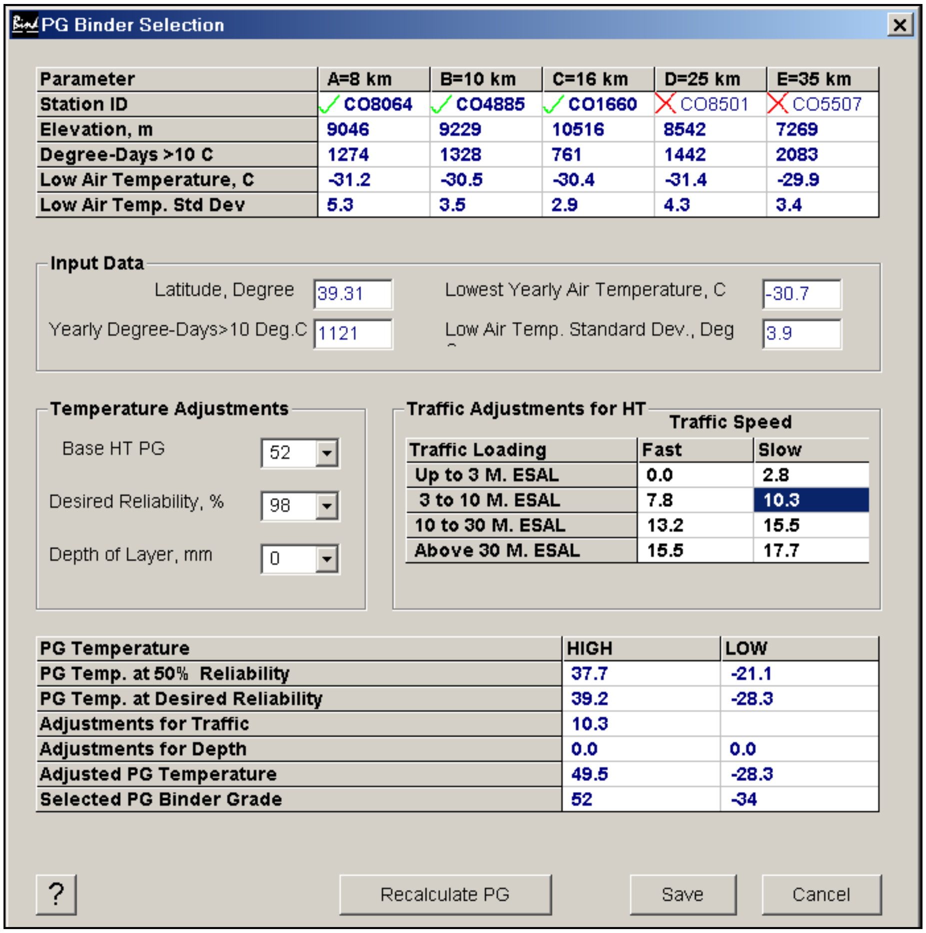


Figure 6.37 LTPP Binder Selection at 98 Percent Reliability

Table 6.11 SuperPave™ Weather Data Summary

98 Percent Reliability		
Depth of Layer	0 mm	
Traffic Loading and Speed Adjustment	10.3°C (slow)	
PG Binder Grade	52	-34

Table 6.12 Environmental Categories (restated)

Highest 7-Day Average Air Temperature	High Temperature Category
> 97°F (> 36°C)	Hot (southeast and west)
> 88° to 97°F (> 31° to 36°C)	Moderate (Denver, plains and west)
81° to 88°F (27° to 31°C)	Cool (mountains)
< 81°F (< 27°C)	Very Cool (high mountains)

Table 6.13 Recommended SuperPave™ Gyratory Design Revolution (N_{DES}) (restated)

CDOT Pavement Management System Traffic Classification (20 Year Design ESAL)	20 Year Total 18k ESAL in the Design Lane	High Temperature Category			
		Very Cool	Cool	Moderate	Hot
Low	< 100,000	50	50	50	50
	100,000 to < 300,000	50	75	75	75
Medium	300,000 to < 1,000,000	75	75	75	75
	1,000,000 to < 3,000,000	75	75	75	100
High	3,000,000 to < 10,000,000	75	75	100	100
Very High	10,000,000 to < 30,000,000	---	---	100	---
Very Very High	≥ 30,000,000	---	---	125	---
Note: Based on *Standard Practice for SuperPave™ Volumetric Design for Hot-Mix Asphalt (HMA*), AASHTO Designation R 35-04.					

6.12.4 Asphalt Binder Characterization for M-E Design

For flexible pavement design using M-E Design, the viscosity of the asphalt binder is a critical input parameter to incorporate the viscoelastic response (i.e. time-temperature dependency) of asphalt concrete mixtures. The asphalt binder viscosity is used in the calculations of dynamic modulus values of asphalt mixtures for aged and unaged conditions. The key input parameters that define the viscosity temperature relationship are the slope (A) and intercept (VTS) resulting from a regression of the asphalt binder viscosity values measured or estimated at different temperatures.

Laboratory testing of asphalt binders is required to develop viscosity temperature relationships at the Level 1 input hierarchy. For performance grade binders, the asphalt binder viscosity values can be estimated from the dynamic shear rheometer test data conducted in accordance with AASHTO T 315, Determining the Rheological of Asphalt Binder Using a Dynamic Shear Rheometer (DSR). Alternatively, for conventional grade binders (i.e. penetration grade or viscosity grade), the asphalt binder viscosity values can be obtained from a series of conventional tests, including absolute and kinematic viscosities, specific gravity, softening point, and penetrations. At the hierarchical input Level 3, the default values of A-VTS parameters included in M-E Design are based on the asphalt binder grade selection.

For flexible pavement rehabilitation designs, the age-hardened binder properties can be determined using asphalt binder extracted from field cores of asphalt pavement layers that will remain in place after rehabilitation. For projects where asphalt is not extracted, historical information and data may be used. **Table 6.14 Recommended Sources of Inputs for Asphalt Binder Characterization** presents recommended sources for asphalt binder characterization at different hierarchical input levels. Refer to the *AASHTO Intrim MEPDG Manual of Practice* and MEPDG Documentation for more information.

Table 6.14 Recommended Sources of Inputs for Asphalt Binder Characterization

Materials Category	**Measured Property**	**Recommended Test Protocol**	**Hierarchical Input Level**		
			3	2	1
Asphalt Binder	Asphalt binder complex shear modulus (G*) and phase angle (δ); at 3 test temperatures, **or**	AASHTO T 315			
	Conventional binder test data: Penetration, **or**	AASHTO T 49			
	Ring and ball softening point Absolute viscosity Kinematic viscosity Specific gravity, **or**	AASHTO T 53 AASHTO T 202 AASHTO T 201 AASHTO T 228		✓	✓
	Brookfield viscosity	AASHTO T 316			
	Asphalt binder grade: PG grade, **or**	AASHTO M 320			
	Viscosity grade, **or**	AASHTO M 226	✓		
	Penetration grade	AASHTO M 20			
	Rolling thin film oven aging	AASHTO T 315		✓	✓

6.13 Asphalt Mix Design Criteria

6.13.1 Fractured Face Criteria

CDOT's aggregate fractured face criteria requires the aggregate retained on the No. 4 sieve must have at least two mechanically induced fractured faces (2) (see **Table 6.15 Fracture Face Criteria**).

Table 6.15 Fractured Face Criteria

Percent Fractured Faces of 20 Year 18k ESAL in Design Lane	SF	ST	SX	S	SG	SMA
Non-Interstate Highways or Pavements with < 10,000,000 Total 18K ESALs	60%	60%	60%	60%	90%	90%
Interstate Highways or Pavements with > 10,000,000 Total 18K ESALs	70%	70%	70%	70%	90%	90%

6.13.2 Air Void Criteria

A design air void range of 3.5 to 4.5 percent with a target of 4.0 percent will be used on all SX, S, SG, and ST mixes. A design air void range of 4.0 to 5.0 percent with a target of 4.5 percent will be used on all SF Mixes. Refer to **Table 6.16 Minimum VMA Requirements** for design air voids and minimum VMA requirements and criteria for voids at N_{DES}. The air void criteria will be applied to the approved design mix. The nominal maximum size is defined as one size larger than the first sieve to retain more that 10 percent. The designer should interpolate specified VMA values for design air voids between those listed in the table. All mix designs shall be run with a gyratory compactor angle of 1.25 degrees. CDOT Form #43 will establish construction targets for asphalt cement and all mix properties at air voids up to 1.0 percent below the mix design optimum. The designer should extrapolate VMA values for production (CDOT Form 43) air voids beyond those listed in **Table 6.16 Minimum VMA Requirements.**

Table 6.16 Minimum VMA Requirements

Nominal Maximum Size[1] mm (in)	Design Air Voids [2, 3]			
	3.5%	4.0%	4.5%	5.0%
37.5 (1 1/2")	11.6	11.7	11.8	N/A
25.0 (1")	12.6	12.7	12.8	
19.0 (3/4")	13.6	13.7	13.8	
12.5 (1/2")	14.6	14.7	14.8	
9.5 (3/8")	15.6	15.7	15.8	16.9

Note:
[1] The nominal maximum size defined as one size larger than the first sieve to retain more than 10%.
[2] Interpolate specified VMA values for design air voids between those listed.
[3] Extrapolate specified VMA values for production air voids between those listed.

6.13.3 Criteria for Stability

Criteria for stability and voids filled with asphalt (VFA) are shown in **Table 6.17 Criteria for Stability and Voids Filled with Asphalt (VFA)**.

Table 6.17 Criteria for Stability and Voids Filled with Asphalt (VFA)

SuperPave™ Gyratory Revolutions (N_{DES})	Hveem Minimum Stability*	VFA (%)
125	30	65-75
100	30	65-75
75	28	65-80
50	**	70-80

Note: 1-inch mix (CDOT Grade SG) has no stability requirements.
* Hveem Stability criteria for mix design approval and for field verification.
** Hveem Stability is not a criterion for mixes with a N_{DES} of 50.

6.13.4 Moisture Damage Criteria

Moisture damage criteria are shown in **Table 6.18 Moisture Damage Criteria.**

Table 6.18 Moisture Damage Criteria

Characteristic	Value
Minimum dry split tensile strength, (psi)	30
Minimum tensile strength ratio, CP-L 5109, (%)	80
Minimum tensile strength ratio, CP-L 5109, SMA, (%)	70

6.13.5 Warm Mix Asphalt (WMA)

Warm mix asphalt provides a means to reduce the carbon footprint of both highway agencies and the asphalt industry. Warm Mix Asphalt (WMA) refers to a group of technologies that allows asphalt mixtures to be produced and placed at lower temperatures. Through the application of chemical and/or mechanical means, liquid asphalt is made more workable at reduced temperatures. This reduces the amount of fuel used to produce the mixtures as well as the emissions originating from production and placement. The different process and products to manufacture WMA are broken into three categories: chemical additives, plant foaming devices, and material foaming processes.

Chemical Additives: Chemical additives can be divided into two classes: organic (long-chain waxes) and surfactants. Long-chain waxes lower the viscosity of the asphalt binder at working temperatures and then harden at service temperatures. Surfactants lower the surface tension of the liquid binder, improving its ability to coat and compact at lower temperatures.

Plant Foaming: Plant foaming devices are systems that can be mounted on batch and continuous plants. These devices inject a small amount of water (1-3 percent by weight) into the asphalt binder before it is introduced to the aggregate. As water comes into contact with the hot asphalt binder in an expansion chamber, it vaporizes and expands to about 1,700 times its liquid binder and allows more thorough coating of the aggregate particles. Production temperatures for mechanical foaming systems range from 250-275 °F.

Material Foaming: Material foaming processes use either moist sand or zeolite (a water bearing material) to foam the asphalt binder as it is mixed. In this process, hot asphalt and coarse aggregate are combined before a sand fraction containing a carefully controlled amount of moisture is added, causing expansion of the binder. A coating additive is employed in the wet sand method. For zeolite, the mineral contains a small amount of water in its crystalline structure that is released at high temperatures. The wet sand method requires plant modifications as well as the use of an additive, while the zeolite method requires plant modifications to introduce the material and zeolite (9).

Benefits of using WMA include the following:

- **Low Temperature Paving:** WMA may be placed at lower temperatures than traditional HMA. The additives used allow for better aggregate coating and workability. Reduction in cure time has been observed, thus cooler pavement temperatures allows traffic onto the new mat sooner.
- **Energy and Fuel Savings**: Reduction in production temperature results in reduction in energy consumption. Traditional HMA requires temperatures in excess of 300°F at the production plant, while WMA production may require temperatures in the low 200's°F.
- **Environmental Benefits:** Decreased production temperature allows for better air quality and reduced emissions at the plant and paver. Thus, worker exposure to fumes and odors and general atmospheric emissions are reduced. (10)

- **Paving Time:** WMA has an extended paving window by allowing the contractor to work at night, extending the paving season, and paving during sudden weather changes.
- **Haul Distance Improvements:** The lower temperatures associated with WMA allows the contractor to haul materials over a longer distance with less heat loss to the mix.

6.14 Effective Binder Content (By Volume)

Effective binder content (P_{be}) is the amount of binder not absorbed by the aggregate, i.e. the amount of binder that effectively forms a bonding film on the aggregate surfaces. Effective binder content is what the service performance is based on and is calculated based on the aggregate bulk specific gravity (G_{sb}) and the aggregate effective specific gravity (G_{se}). The higher the aggregate absorption, the greater the difference between G_{se} and G_{sb}. The effective binder content by volume is the effective binder content (P_{be}) times the ratio of the bulk specific gravity of the mix (G_{mm}) and the specific gravity of the binder (G_b). The formula is:

$$\mathbf{P_{be}\ (by\ volume) = P_{be} * (G_{mm} / G_b)}$$

Where

P_{be} = effective asphalt content, percent by total weight of mixture
G_{mm} = bulk specific gravity of the mix
G_b = specific gravity of asphalt (usually 1.010)

P_{be} is determined as follows:

$$\mathbf{P_{be} = P_b - (P_{ba}/100) * P_s}$$

Where

P_b = asphalt, percent by total weight of mixture
P_{ba} = absorbed asphalt, percent by total weight of aggregate
P_s = aggregate, percent by total weight of mixture

P_{ba} is determined as follows:

$$\mathbf{P_{ba} = 100\ ((G_{se} - G_{sb}) / (G_{sb} * G_{se})) * G_b}$$

Where

P_{ba} = absorbed asphalt, percent by total weight of aggregate
G_{se} = effective specific gravity of aggregate
G_{sb} = bulk specific gravity of aggregate

6.15 Rumble Strips

When Rumble Strips are installed, they shall be of the style and location as shown on CDOT's *Standard Plans*, *M & S Standards*, July 2012 Plan Sheet No. M-614-1, Rumble Strips.

References

1. Gilbert, K., *Thermal Segregation*, 2005, CDOT Report No. CDOT-DTD-R-2005-16. http://www.coloradodot.info/programs/research/pdfs/2005/thermalsegregation.pdf..

2. Brown, E.R. and Bassett, C.E. *Effects of Maximum Aggregate Size on Rutting Potential and Other Properties of Asphalt-Aggregate Mixtures 1990*. Transportation Research Record 1259, TRB, National Research Council, Washington, D.C., 1990, pp. 107-119.

3. *HMA Pavement Mix Type Selection Guide*, FHWA Information Series 128, Federal Highway Administration, 400 7th Street S.W., Washington, DC 20590 and National Asphalt Pavement Association (NAPA), 5100 Forbes Blvd, Lanham, MD 20706, January 2001.

4. *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Submitted by ARA, INC., ERES Consultants Division, Champaign, IL, March 2004.

5. Locander, Robert, *Analysis of Using Reclaimed Asphalt Pavement (RAP) as a Base Material*, Final Report, CDOT Report No. CDOT-2009-5, Colorado Department of Transportation, February 2009.

6. *Perpetual Pavements - A Synthesis.* Asphalt Pavement Alliance, 5100 Forbes Boulevard, Lanham, MD 20706 : Asphalt Pavement Alliance, 1/2002. Synthesis.

7. *Perpetual Bituminous Pavements.* Transportation Research Board (TRB). 2101 Constitution Avenue, NW, Washington, DC 20418 : Transportation Research Board, National Research Center, December 2001. Research Circular. TRB Committee on General Issues in Asphalt Technology (A2D05). Number 503, ISSN 0097-8515.

8. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

9. *Warm Mix Asphalt – We All Win!, David E Newcomb, P.E., Ph.D.,* article on www.co-asphalt.com, March 2013.

10. *Warm Mix Asphalt: The Product and Its Availability, Colorado Asphalt Pavement Association*, Colorado asphalt Pavement Association, Denver, Colorado December 14, 2007.

CHAPTER 7
PRINCIPLES OF DESIGN FOR RIGID PAVEMENT

7.1 Introduction

Rigid pavement design is based on the mechanistic-empirical (M-E) design concepts. The design procedure utilizes distress and smoothness prediction models developed and calibrated locally. The *MEPDG Design Guide* and the *AASHTO Interim MEPDG Manual of Practice* documents provide a detailed description of the M-E concepts for rigid pavement designs.

The design procedures described in this chapter can be used for design of new or reconstructed rigid pavements. There are no fundamental differences in the pavement design procedure for new alignment and reconstruction, however, the potential reuse of the materials from the existing pavement structure can be an important issue. Refer to **CHAPTER 9: Principles of Design for Pavement Rehabilitation with Rigid Overlay** when rehabilitation designs are necessary with rigid overlays or restoration projects.

<u>The design life for typical thin white topping should be 10 to 20 years for rehabilitations and 30 years for reconstruction.</u> An overview of the proven concrete pavement practices the Colorado Department of Transportation (CDOT) has implemented over the last several years is documented in the Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8).

7.2 M-E Design Methodology for Rigid Pavement

The M-E Design of rigid pavements is an iterative process. The key steps in the design process include the following:

- **Select a Trial Design Strategy**

- **Select the Appropriate Performance Indicator Criteria for the Project:** Establish criteria for acceptable pavement performance (i.e. distress/IRI) at the end of the design period. CDOT criteria for acceptable performance is based on highway functional class and location. The performance criteria is established to reflect magnitudes of key pavement distresses and smoothness that trigger major rehabilitation or reconstruction.

- **Select the Appropriate Reliability Level for the Project:** The reliability is a factor of safety to account for inherent variations in construction, materials, traffic, climate, and other design inputs. The level of reliability selected should be based on the criticality of the design. CDOT criteria for desired reliability is based on highway functional class and location. The desired level of reliability is selected for each individual performance indicator.

- **Assemble All Inputs for the Pavement Trial Design Under Consideration:** Define subgrade support, PCC and other paving material properties, traffic loads, climate, pavement type, and design/construction features. The inputs required to run M-E Design may be obtained using one of three hierarchical levels of effort and need not be consistent for all of the inputs in a given design. A hierarchical level for a given input is selected based on the importance of the project and input, and the resources at the disposal of the designer.

- **Run the M-E Design Software:** The software calculates changes in layer properties, damage, key distresses, and IRI over the design life. The key steps include:

  - **Processing Input** to obtain monthly values of traffic inputs and seasonal variations of material and climatic inputs needed in the design evaluations for the entire design period.

  - **Computing Structural Responses** (stresses and strains) using finite element based pavement response models for each axle type and load and damage-calculation increment throughout the design period.

  - **Calculating Accumulated Distress** and/or damage at the end of each analysis period for the entire design period.

  - **Predicting Key Distresses** (JPCP transverse cracking and joint faulting) at the end of each analysis period throughout the design life using the calibrated mechanistic-empirical performance models.

  - **Predicting Smoothness** as a function of initial IRI, distresses that accumulate over time, and site factors at the end of each analysis increment.

- **Evaluate the Adequacy of the Trial Design:** The trial design is considered "adequate" if none of the predicted distresses/IRI exceed the performance indicator criteria at the design reliability level chosen for the project. If any of the criteria has been exceeded, determine how this deficiency can be remedied by altering material types and properties, layer thicknesses, or other design features.

- **Revise the Trial Design, as Needed:** If the trial design is deemed "inadequate", revise the inputs/trial design and re-run the program. Iterate until all the performance criteria have been met. Once they have been met, the trial design becomes a feasible design alternative.

The design alternatives that satisfy all performance criteria are considered feasible from a structural and functional viewpoint and can be further considered for other evaluations, such as life cycle cost analysis. A detailed description of the design process is presented in the interim edition of the AASHTO *Mechanistic-Empirical Pavement Design Guide Manual of Practice,* AASHTO, 2008.

7.3 Select Trial Design Strategy

7.3.1 Rigid Pavement Layers

Figure 7.1 Rigid Pavement Layers shows a conventional rigid layered system. The PCC slab may be placed over base, subbase, or directly on a prepared subgrade. The base (layer directly beneath the PCC slab) and subbase layers (layer placed below the base layer) may include unbound aggregates, asphalt stabilized granular, cement stabilized, lean concrete, crushed concrete, lime stabilized, recycled asphalt pavement (RAP), and other materials. Base/subbase layers may be dense graded or permeable drainage layers.

Transverse joints are closely spaced in JPCP, typically between 10 and 20 feet, to minimize transverse cracking from temperature and moisture gradients. JPCP may have tied or untied longitudinal joints.

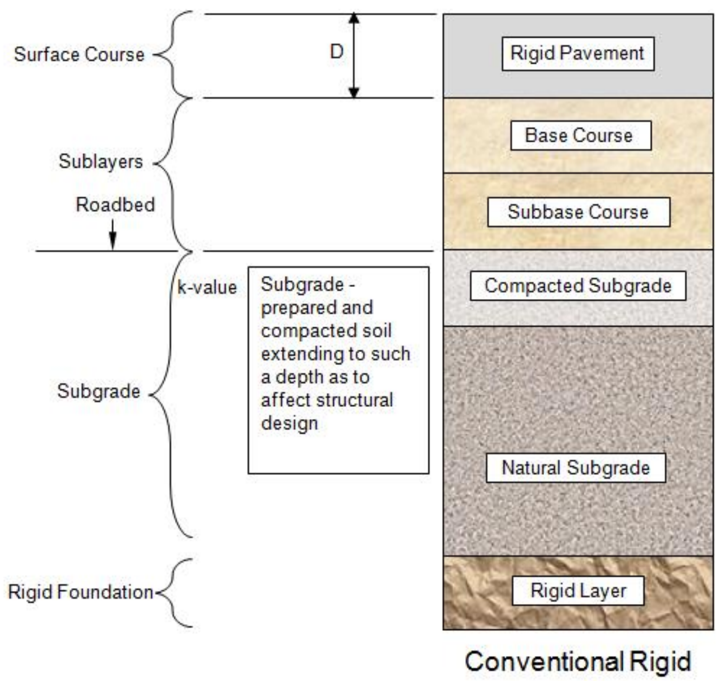


Figure 7.1 Rigid Pavement Layers

7.3.2 Establish Trial Design Structure

The designer must establish a trial design structure (combination of material types and thicknesses). This is done by first selecting the pavement type (see **Figure 7.2 M-E Design Screenshot Showing General Information Performance Criteria and Reliability**). M-E Design automatically provides the top layers of the selected pavement type. The designer may add or remove pavement structural layers and modify layer material type and thickness as appropriate. **Figure 7.3 M-E Design Screenshot of Rigid Pavement Trial Design Structure** shows the pavement layer configuration of a sample rigid pavement and trial design on the left and layer properties of the PCC slab on the right.

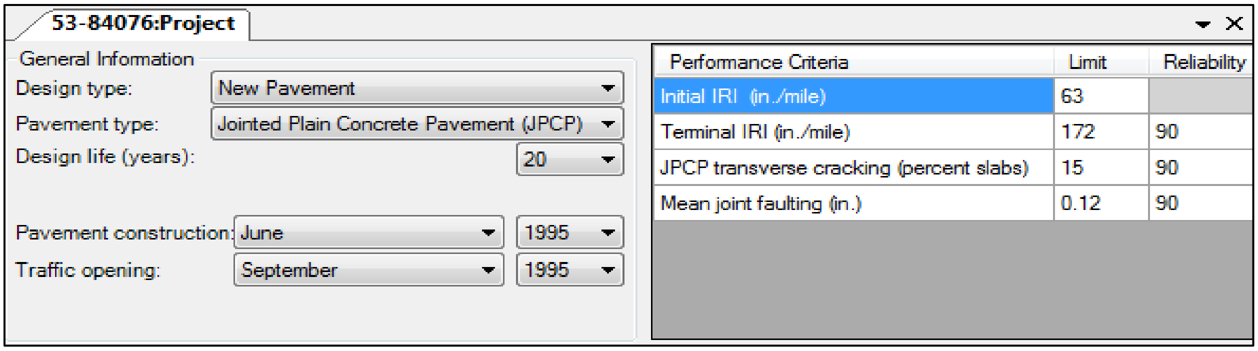


Figure 7.2 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability

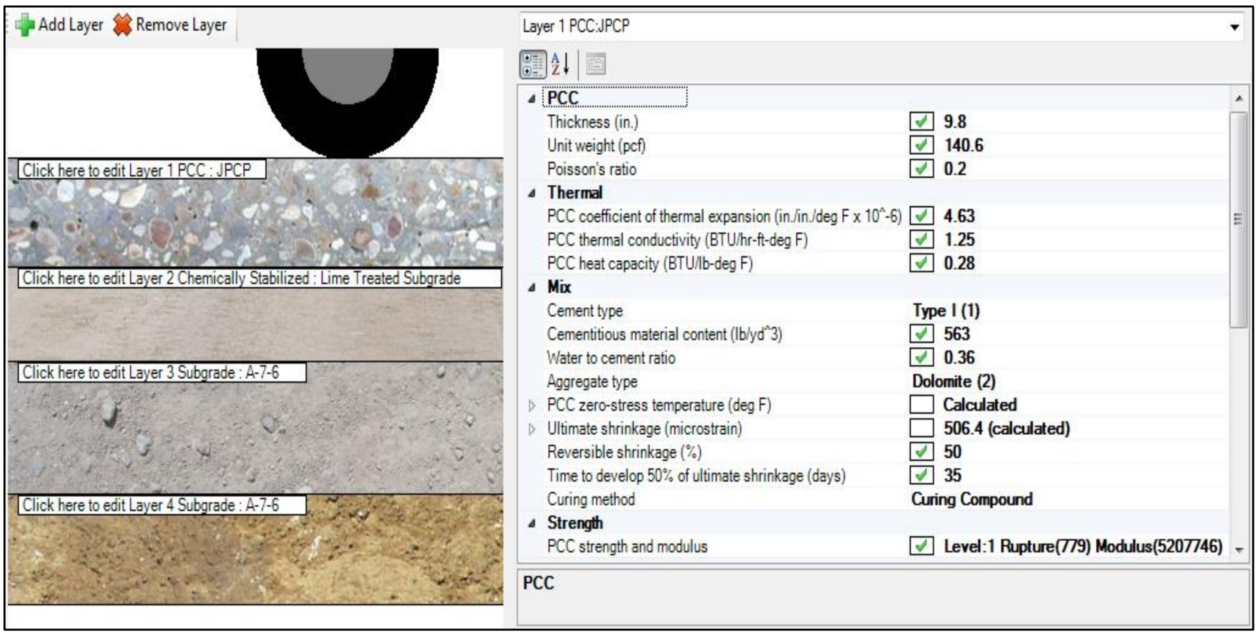


Figure 7.3 M-E Design Screenshot of Rigid Pavement Trial Design Structure

7.4 Select the Appropriate Performance Indicator Criteria for the Project

Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction Projects presents recommended performance criteria for a rigid pavement design. The designer should enter the appropriate performance criteria based on functional class. An appropriate initial smoothness (IRI) is also required. **For new rigid pavements, the recommended initial IRI is 78 inches/mile.** This recommendation is for regular paving projects and projects with incentive-based smoothness acceptance; the designer may modify this value as needed. **Figure 7.2 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability** shows performance criteria for a sample rigid pavement trial design. The coefficients of performance prediction models considered in the design of a rigid pavement are shown in **Figure 7.4 Performance Prediction Model Coefficients for Rigid Pavement Designs.**

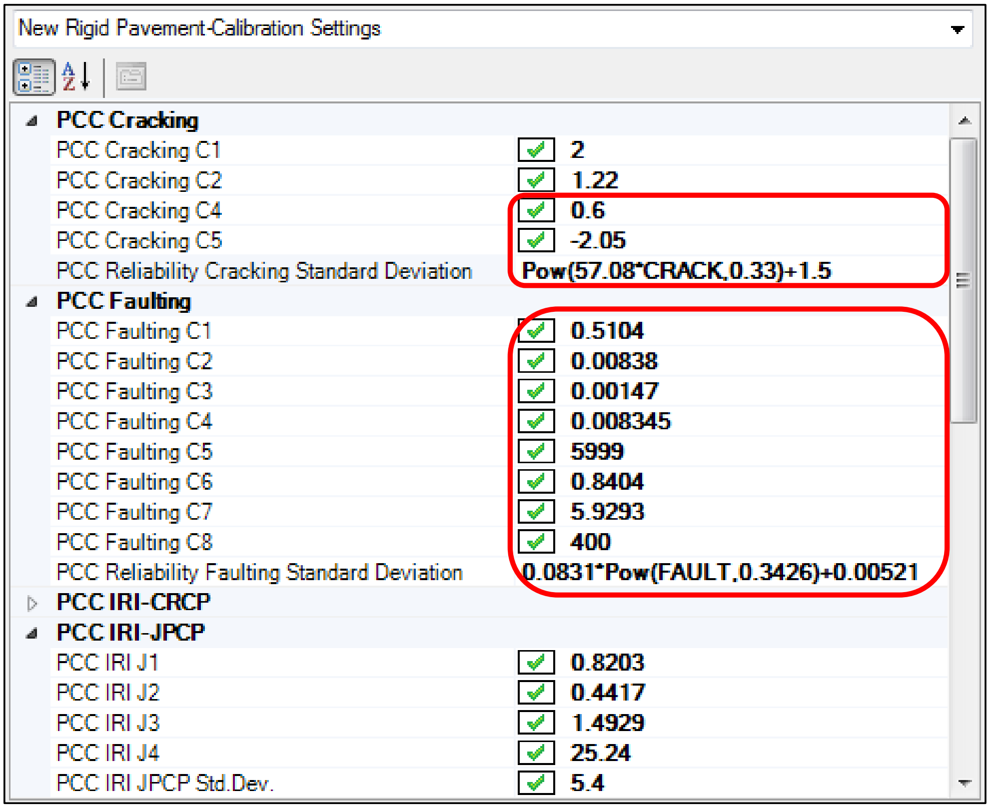


Figure 7.4 Performance Prediction Model Coefficients for Rigid Pavement Designs

7.5 Select the Appropriate Reliability Level for the Project

Table 2.3 Reliability (Risk) presents recommended reliability levels for rigid pavement designs. The designer should select an appropriate reliability level based on highway functional class and location (see **Figure 7.2 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability**).

7.6 Assemble the M-E Design Inputs

7.6.1 General Information

7.6.1.1 Design Period

The design period for new rigid pavement construction and reconstruction is 20 or 30 years. It is recommended a 30-year design period be used for rigid pavements. Selection of a design period other than 10, 20, or 30 years needs to be supported by a LCCA or other overriding considerations.

7.6.1.2 Project Timeline

The following inputs are required to specify the project timeline in the design (see **Figure 7.2 M-E Design Screenshot Showing General Information, Performance Criteria and Reliability**).

- Pavement construction month and year
- Traffic open month and year

The designer may select the most likely month and year when the PCC surface layer is scheduled to be placed, and when the pavement section is scheduled to be opened to traffic. Changes to the surface layer material properties due to time and environmental conditions are considered beginning from the construction date. **Due to warping, curling and other factors, if the actual month(s) of construction is unknown then the month of May <u>should be</u> used**.

7.6.1.3 Identifiers

Identifiers are helpful in documenting the project location and recordkeeping. M-E Design allows designers to enter site or project identification information, such as the location of the project (route signage, jurisdiction, etc.), identification numbers, beginning and ending milepost, direction of traffic, and date.

7.6.1.4 Traffic

Several inputs are required for characterizing traffic for M-E Design and are described in detail in **Section 3.1 Traffic.**

7.6.1.5 Climate

The climate input requirements for M-E design are described in detail in **Section 3.2 Climate**.

7.6.1.6 Pavement Layer Characterization

As shown in **Figure 7.1 Rigid Pavement Layers**, a typical rigid pavement design comprises of the following pavement layers: PCC, treated and/or unbound aggregate base, and subgrade. The inputs required by the M-E Design software for characterizing these layers are described in the following sections.

7.6.1.7 Portland Cement Concrete

The inputs required for PCC layer characterization are divided into three categories (see **Figure 7.5 PCC Layer and Material Properties in M-E Design**).

- **General and Thermal Properties:** This category includes layer thickness, Poisson's ratio, Coefficient of Thermal Expansion (CTE), thermal conductivity, and heat capacity.

- **PCC Mix-Related Properties**: This category includes cement type (Types I, II, or III), cement content, water/cement (or w/c) ratio, aggregate type, PCC zero-stress temperature, ultimate shrinkage at 40 percent relative humidity, reversible shrinkage, and curing method.

- **Strength and Stiffness Properties**: This category includes modulus of rupture (flexural strength), static modulus of elasticity, and/or compressive strength.

These inputs are required for predicting pavement responses to applied loads, long-term strength and elastic modulus, and effect of climate (temperature, moisture, and humidity) on PCC expansion and contraction. **Table 7.1 PCC Material Inputs and Recommendations for New JPCP Designs** presents recommendations for inputs used in PCC material characterization for a new JPCP design. Level 1 inputs of typical CDOT PCC mixtures may be used for Levels 2 and 3 (see **APPENDIX G**). Refer to **Table 2.6 Selection of Input Hierarchical Level** for selection of an appropriate hierarchical level for material inputs.

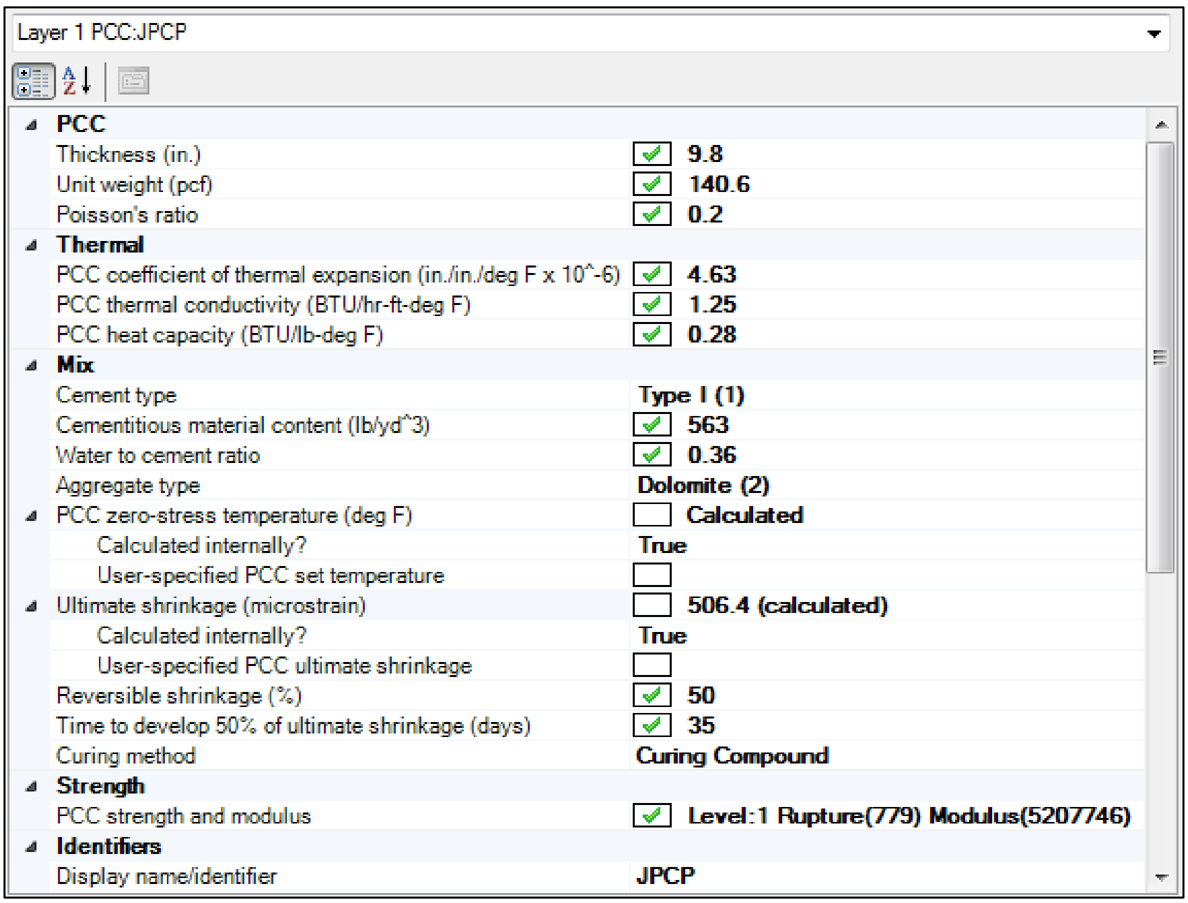


Figure 7.5 PCC Layer and Material Properties in M-E Design

Table 7.1 PCC Material Inputs and Recommendations for New JPCP Design

Input Property (Strength)	Input Hierarchy		
	Level 1	**Level 2**	**Level 3**
Elastic Modulus	Mix specific values (ASTM C 469)		Use typical values from **APPENDIX G**. Select a mix that is closest to the project. Use a default ratio of 1.20 for 20-year / 28-day strength gain of elastic modulus and flexural strength.
Flexural Strength	Mix specific values (AASHTO T 97)		
Compressive Strength		Mix specific values (AASHTO T 22)	
Unit Weight	Mix specific values (AASHTO T 121)	**APPENDIX G**	
Poisson's Ratio	Mix specific values (ASTM C 469)	**APPENDIX G**	
Coefficient of Thermal Expansion	Mix specific values (AASHTO TP 60)	**APPENDIX G**	
Surface Shortwave Absorptivity	0.85		
Thermal Conductivity	1.25		
Heat Capacity	0.28		
Cement Type	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Cementitious Material Content	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Water to Cement Ratio	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Curing Method	Select an appropriate method based on Section 412.14 of *CDOT Standard Specifications for Road and Bridge Construction*		
PCC Zero-stress Temperature	Internally calculated		
Ultimate Shrinkage	Internally calculated		
Reversible Shrinkage	50 percent		
Time to Develop 50 Percent of Ultimate Shrinkage	35 days		

7.6.1.8 Asphalt Treated Base Characterization

The asphalt treated base layer is modeled as a HMA layer. The material input requirements are identical to those of a conventional HMA layer as described in **Section 6.6.4.1 Asphalt Concrete**

Characterization, with an exception to indirect tensile strength and creep compliance values. For JPCP designs, no sub-layering is done within the asphalt treated base layer.

7.6.1.9 Chemically Stabilized Base Characterization

Refer to **Section 5.4.1 Characterization of Treated Base in M-E Design** for treated base characterization.

7.6.1.10 Unbound Material Layers and Subgrade Characterization

Refer to **Section 5.3.1 Unbound Layer Characterization in M-E Design** for unbound aggregate base layer characterization; and refer to **Section 4.4 Subgrade Characterization for M-E Design** for subgrade characterization.

7.6.2 JPCP Design Features

JPCP design features and construction practices influence long-term performance. The common design features considered in M-E Design (see **Figure 7.6 M-E Design Screenshot of JPCP Design Features**) include the following:

- Surface shortwave absorptivity: Refer to **Table 7.1 PCC Material Inputs and Recommendations for New JPCP Design**
- Joint spacing: Refer to **Section 7.10 Joint Spacing (L**)
- PCC-base contact friction: Refer to **Section 7.11 Slab/Base Friction**
- Permanent curl/warp effective temperature difference: Refer to **Section 7.12 Effective Temperature Differential (°F)**
- Widened slab: Refer to **Section 7.14 Lane Edge Support** Condition
- Dowel bars: Refer to **Section 7.13 Dowel Bars (Load Transfer Devices) and Tie Bars**
- Tied shoulders: Refer to **Section 7.13 Dowel Bars (Load Transfer Devices) and Tie Bars** and **Section 7.14 Lane Edge Support Condition**
- Base type and erodibility index: Refer to **Section 7.15 Base Erodibility**
- Sealant type: Refer to **Section 7.16 Sealant Type**

7.7 Run M-E Design

Designers should examine all inputs for accuracy and reasonableness prior to running M-E Design. The designer will run the software to obtain outputs required for evaluating whether the trial design is adequate. After a trial run has been successfully completed, M-E Design will generate a report in form of a PDF and/or Microsoft Excel file, see **Figure 7.7 Sample Rigid Pavement Design PDF Output Report**. The report contains the following information: inputs, reliability of design, materials and other properties, and predicted performance.

After the trial run is complete, the designer should examine all inputs and outputs for accuracy and reasonableness. The output report also includes the estimates of material properties and other properties on a month-by-month basis over the entire design period in either tabular or graphical form. For a JPCP pavement trial design, the report provides the following:

- PCC flexural strength/modulus of rupture
- PCC elastic modulus
- Unbound material resilient modulus
- Subgrade k-value
- Cumulative trucks (FHWA Class 4 through 13) over the design period

Once again, the designer should examine the above mentioned parameters to assess their reasonableness before accepting a trial design as complete.

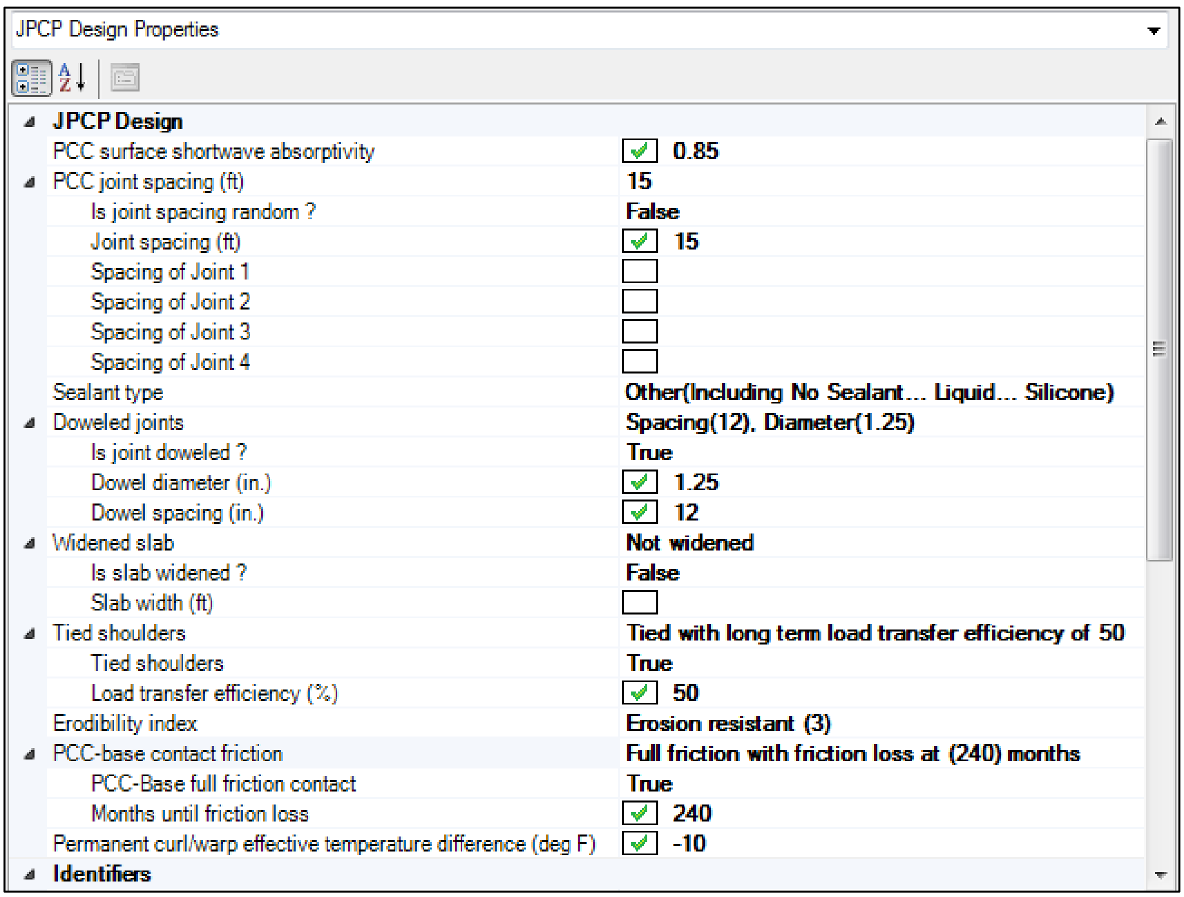


Figure 7.6 M-E Design Screenshot of JPCP Design Features

53-84076

File Name: C:\Users\bbhattacharya.RESTRICT\Desktop\COLORADO MEPDG\FINAL REPORT\53-84076.dgpx

Design Inputs

Design Life:	20 years	Existing construction:	-	Climate Data Sources (Lat/Lon)	39.909, -105.117
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	June, 1995		39.833, -104.658
		Traffic opening:	September, 1995		39.702, -104.752
					39.57, -104.849
					39.783, -104.55
					40.452, -105.001
					40.436, -104.618

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP	9.8
Cement_Base	Lime Treated Subgrade	8.5
Subgrade	A-7-6	12.0
Subgrade	A-7-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
1995 (initial)	619
2005 (10 years)	2,625,540
2015 (20 years)	6,251,140

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	172.00	95.59	90.00	100.00	Pass
Mean joint faulting (in.)	0.12	0.03	90.00	100.00	Pass
JPCP transverse cracking (percent slabs)	15.00	1.92	90.00	100.00	Pass

Figure 7.7 Sample Rigid Pavement Design PDF Output Report

7.8 Evaluate the Adequacy of the Trial Design

The output report of a rigid pavement trial design includes the monthly accumulation of the following key distress types at their mean values and chosen reliability for the entire design period:

- **Joint Faulting**: This is an indicator of erosion of sublayers and the effectiveness of joint LTE. A critical value is reached when joint faulting results in excess roughness, which is unacceptable to drivers and difficult to remove by re-texturing.
    - The designer should examine the results to evaluate if the performance criteria for joint faulting are met at the desired reliability. If joint faulting has not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.
    - The output report also includes the monthly accumulation of the following secondary distress types and smoothness indicators at their mean values and chosen reliability values for the entire design period.
- **Percent Slabs Cracked**: This is the mean predicted transverse cracks that form from fatigue damage at the top and bottom of the slab. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures.

- **IRI**: This is a function of joint faulting and slab cracking along with climate and subgrade factors. A high IRI indicates unacceptable ride quality.

  - The designer should examine the results to evaluate if the performance criteria for percent slabs cracked and IRI meet the **minimum of 27 years** at the desired reliability.

  - If any of the criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.

Another important output is the reliability levels of each performance indicator at the end of the design period. If the reliability value predicted for the given performance indicator is greater than the target/desired value, the trial design passes for that indicator. If the reverse is true, then the trial design fails to provide the desired confidence and performance indicator will not reach the critical value during the pavement's design life. In such an event, the designer needs to alter the trial design to correct the problem.

The strategies for modifying a trial design are discussed in **Section 7.9 Modifying Trial Designs**. The designer can use a range of thicknesses to optimize the trial design and make it more acceptable. Additionally, the software allows the designer to perform a sensitivity analysis for key inputs. The results of the sensitivity analysis can be used to further optimize the trial design if modifying PCC thickness alone does not produce a feasible design alternative. A detail description of the thickness optimization procedure and sensitivity analysis is provided in the *Software HELP Manual*.

7.9 Modifying Trial Designs

An unsuccessful trial design may require revisions to ensure all performance criteria are satisfied. The trial design is revised by systematically modifying the design inputs. The design acceptance in M-E Design is distress-specific; in other words, the designer needs to first identify the performance indicator that failed to meet the performance targets and modify one or more design inputs that has a significant impact on a given performance indicator accordingly. The impact of design inputs on performance indicators is typically obtained by performing a sensitivity analysis.

The strategies to produce a satisfactory design by modifying design inputs can be broadly categorized into:

- Pavement layer considerations:
  - Increasing layer thickness
  - Modifying layer type and layer arrangement
  - Foundation improvements

- Pavement material improvements:
  - Use of higher quality materials
  - Material design modifications
  - Construction quality

Remember, when modifying the design inputs, the designer needs to be aware of input sensitivity to various distress types. Changing a single input to reduce one distress may result in an increase in another distress. **Table 7.2 Modifying Rigid Pavement Trial Designs** presents a summary of inputs that may be modified to optimize trial designs and produce a feasible design alternative.

Table 7.2 Modifying Rigid Pavement Trial Designs

Distress/IRI	Design Inputs that Impact
Transverse Cracking	• Increase slab thickness • Increase PCC strength • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient • PCC tied shoulder (separate placement or monolithic placement). • Widened slab (1 to 2 feet) • Use PCC with a lower coefficient of thermal expansion
Joint Faulting	• Increase slab thickness • Reduce joint width over analysis period • Increase erosion resistance of base (specific recommendations for each type of base) • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient • PCC tied shoulder • Widened slab (1 to 2 feet)
IRI	• Require more stringent smoothness criteria and greater incentives • Increase slab thickness • Ensure PCC has proper entrained air content • Decrease joint spacing • Widen the traffic lane slab by 2 feet • Use a treated base (if nonstabilized dense graded aggregate was specified) • Increase diameter of dowels

Figure 7.8 Sensitivity of JPCP Transverse Cracking to PCC Thickness through **Figure 7.19 Sensitivity of JPCP IRI to Design Reliability** presents sensitivity plots of a sample rigid pavement trial design showing the effects of key inputs, such as traffic volume, PCC thickness, PCC coefficient of thermal expansion, and design reliability on key distresses. **Note:** The plots do not cover the effects of all key factors on rigid pavement performance; other significant factors are not shown herein.

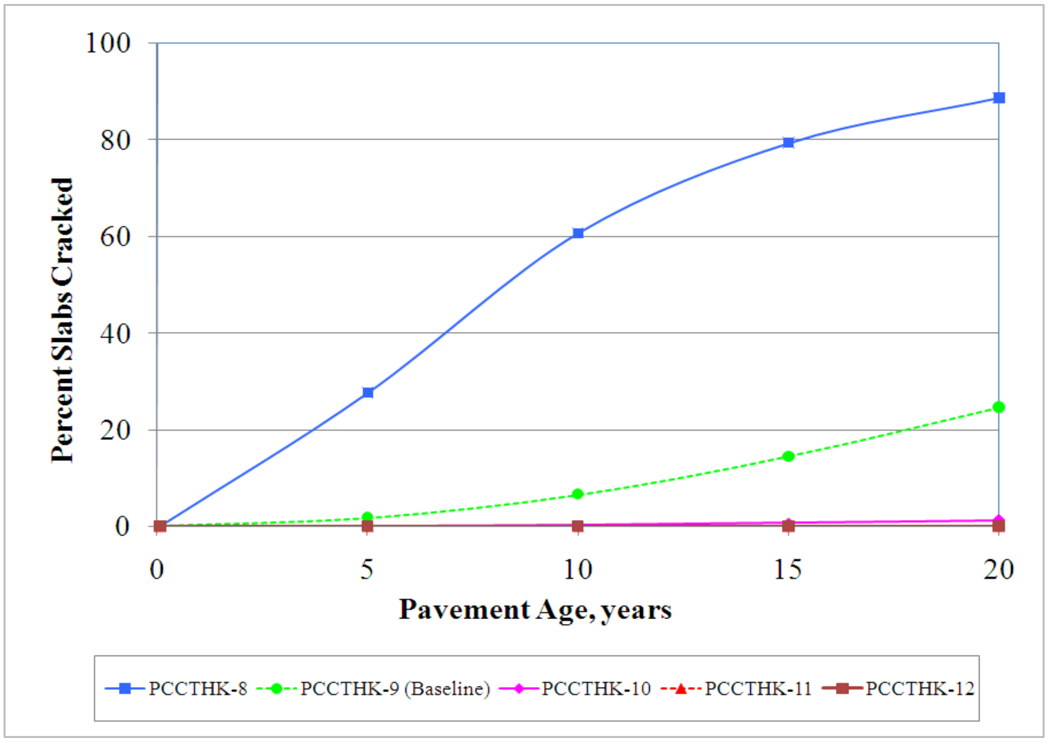


Figure 7.8 Sensitivity of JPCP Transverse Cracking to PCC Thickness

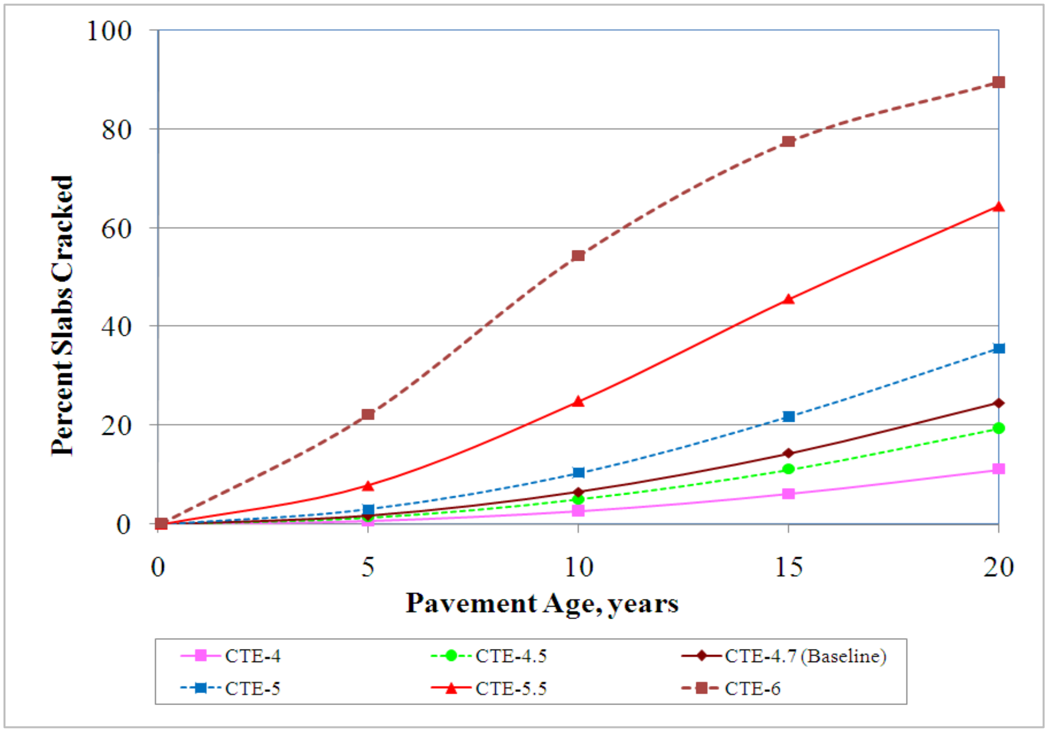


Figure 7.9 Sensitivity of JPCP Transverse Crackling to PCC Coefficient of Thermal Expansion

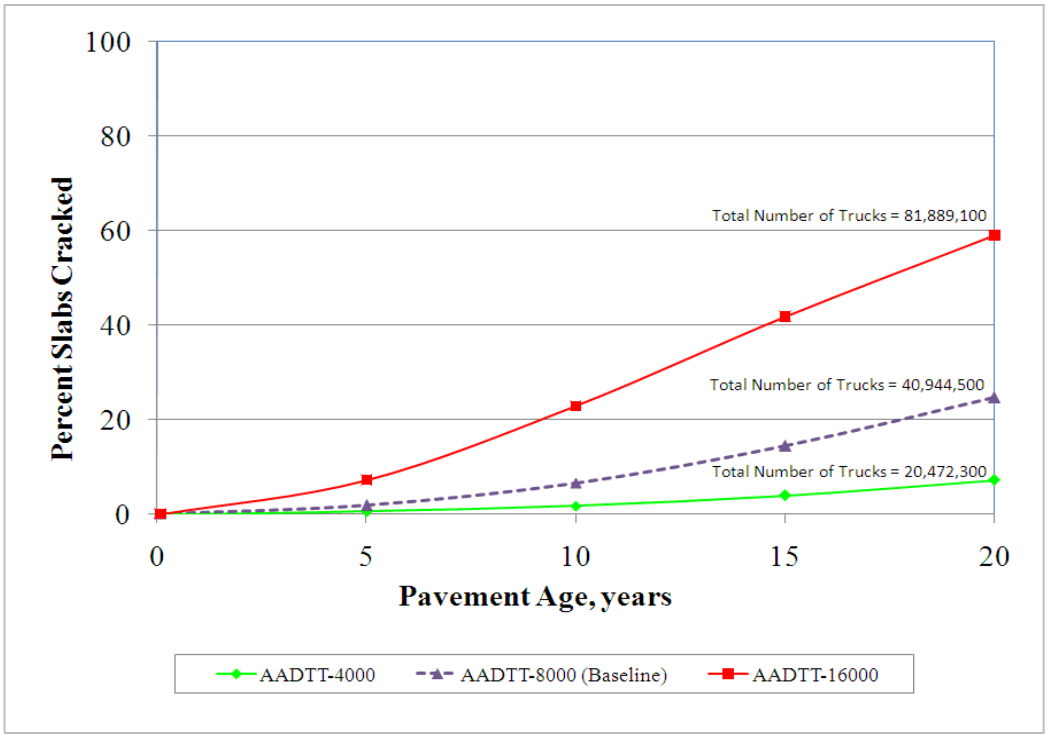


Figure 7.10 Sensitivity of JPCP Transverse Cracking to Traffic Volume

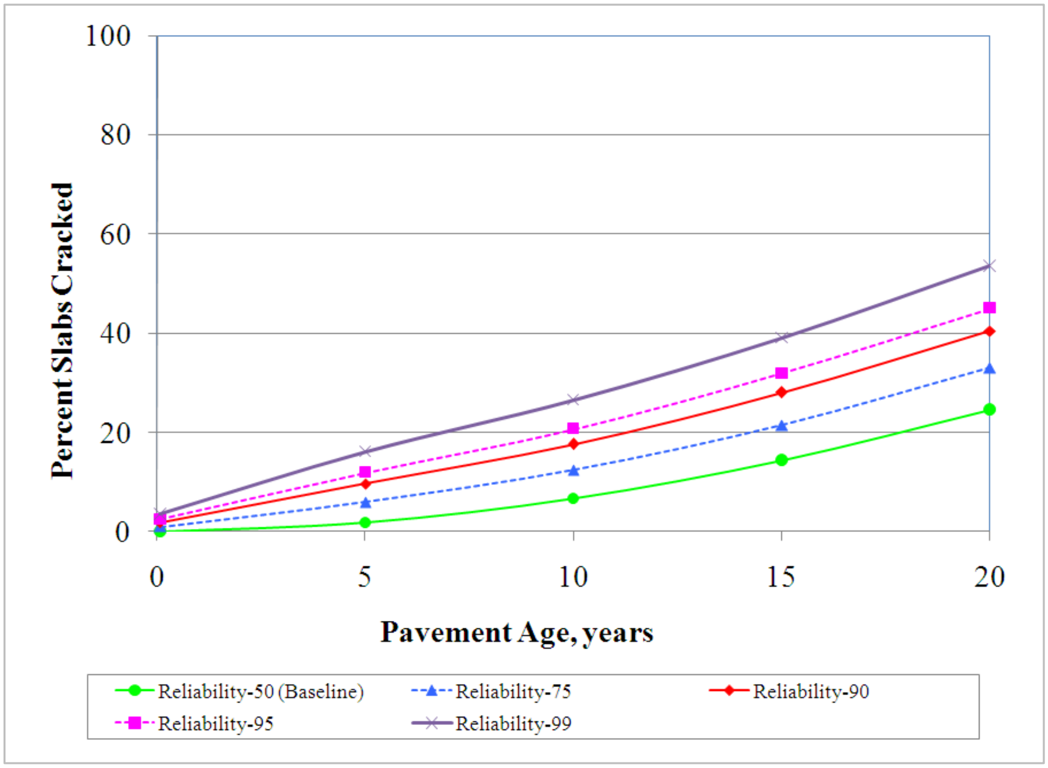


Figure 7.11 Sensitivity of JPCP Transverse Cracking to Design Reliability

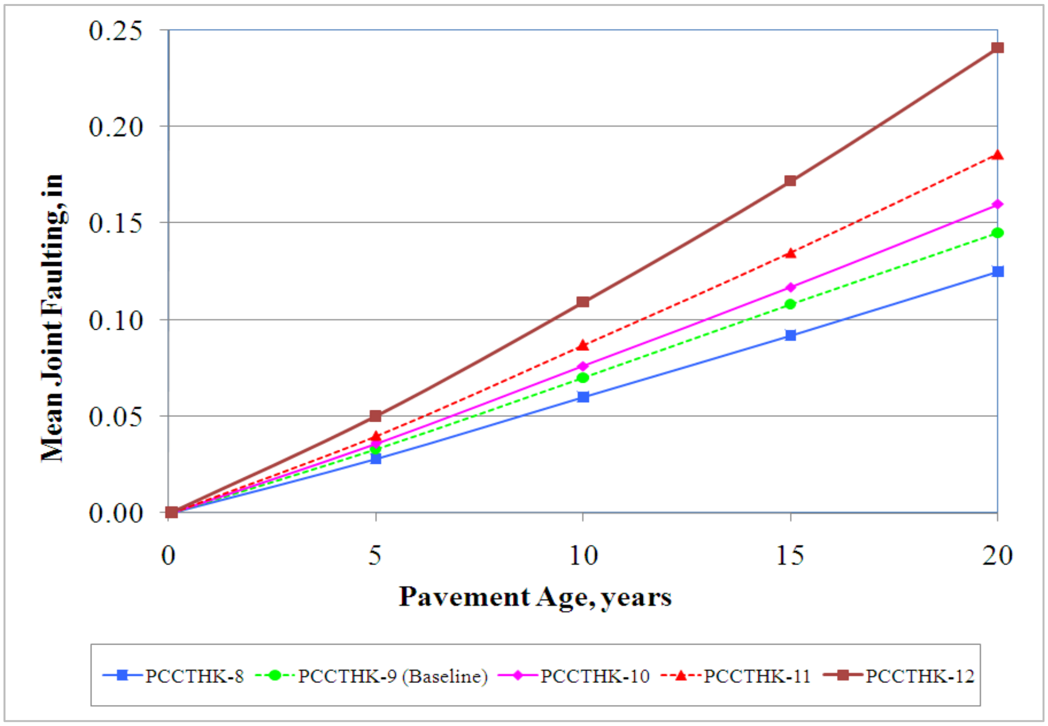


Figure 7.12 Sensitivity of JPCP Faulting to PCC Thickness

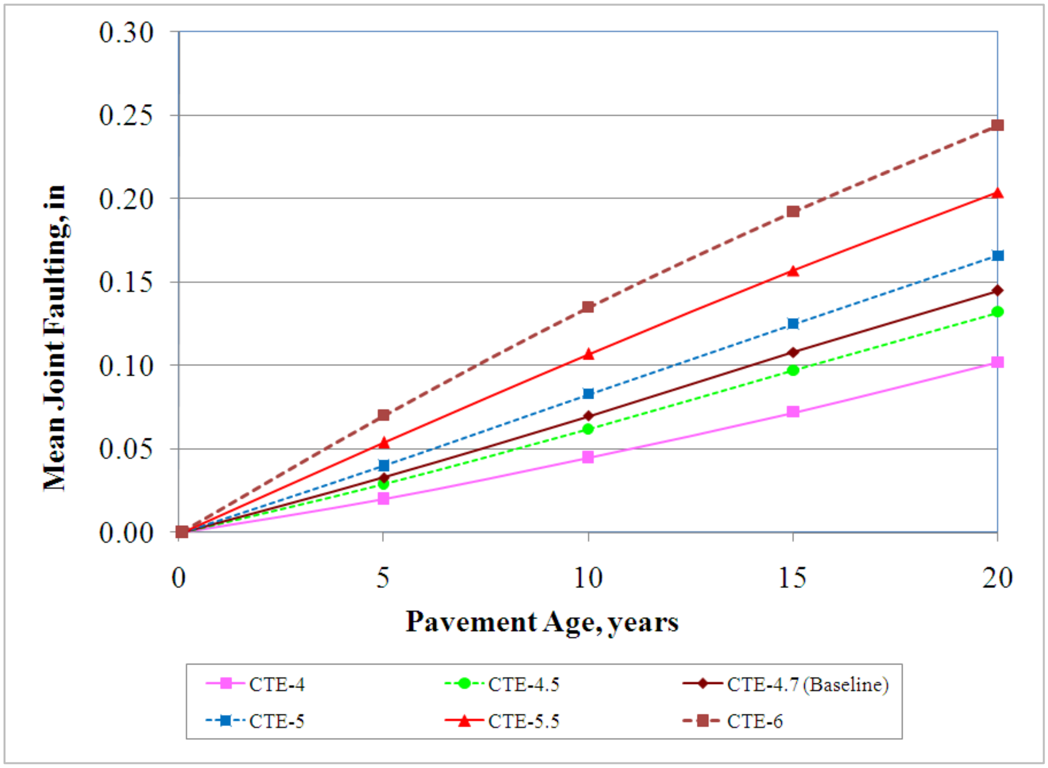


Figure 7.13 Sensitivity of JPCP Faulting to PCC Coefficient of Thermal Expansion

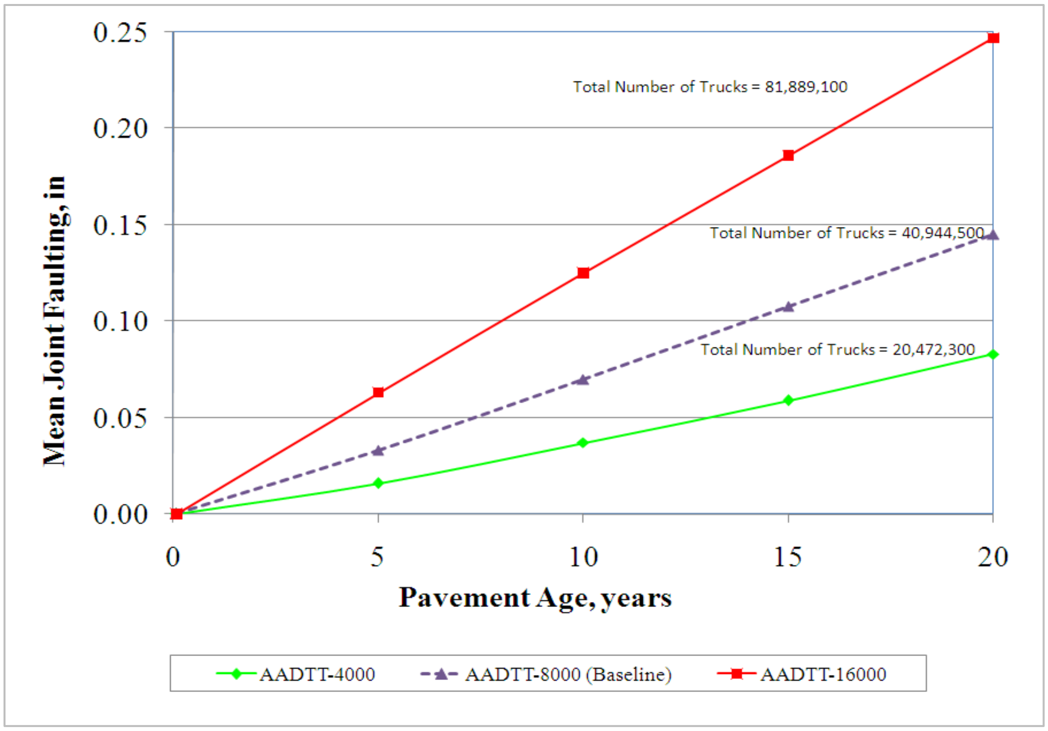


Figure 7.14 Sensitivity of JPCP Faulting to Traffic Volume

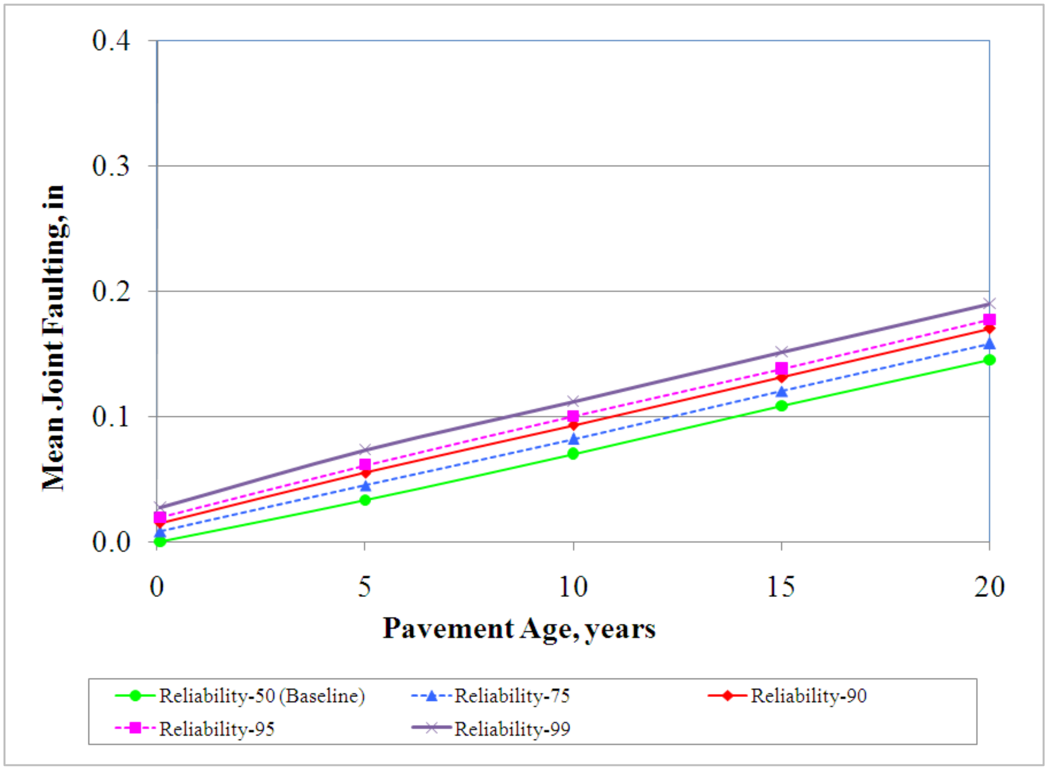


Figure 7.15 Sensitivity of JPCP Faulting to Design Reliability

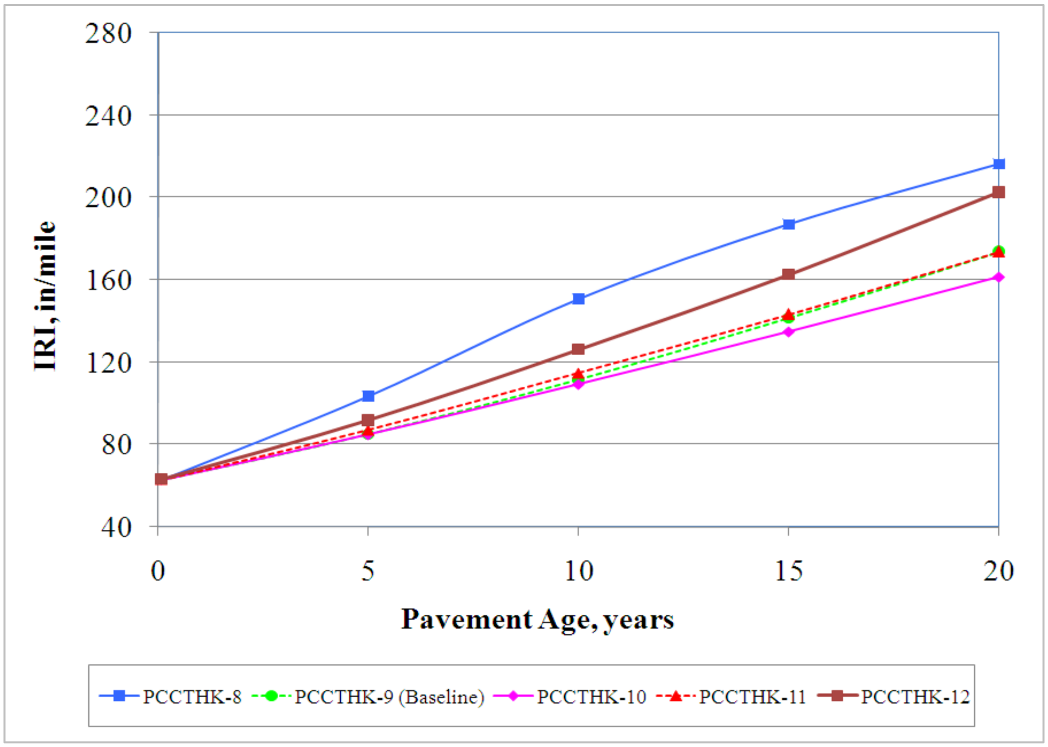


Figure 7.16 Sensitivity of JPCP IRI to PCC Thickness

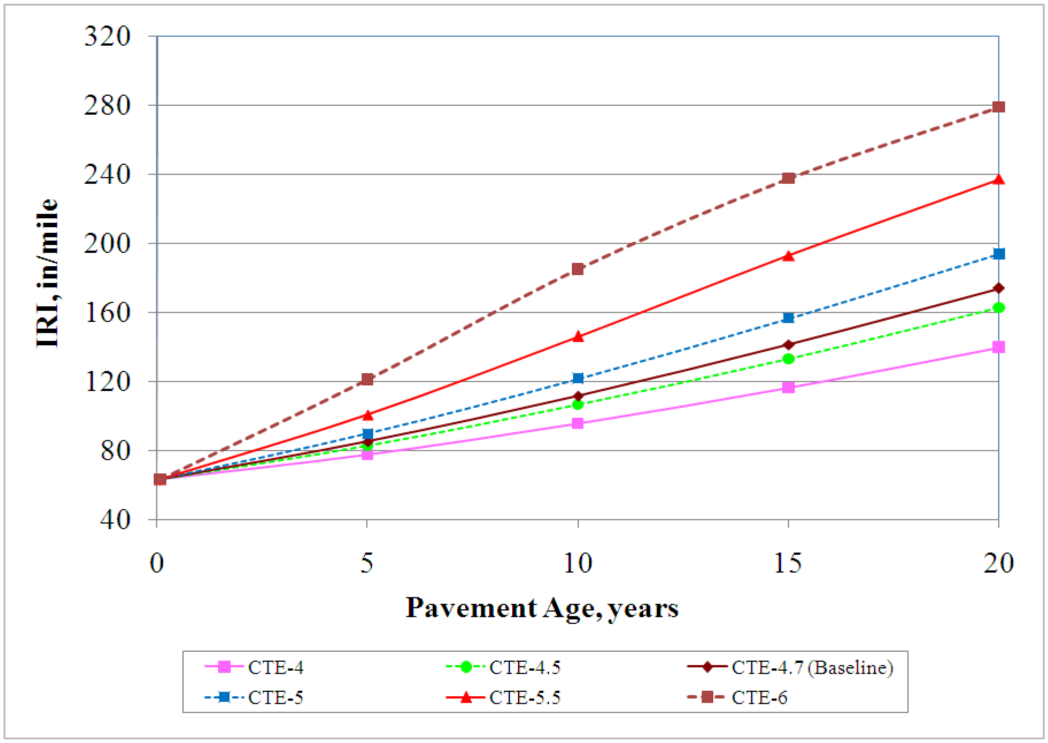


Figure 7.17 Sensitivity of JPCP Faulting to PCC Coefficient of Thermal Expansion

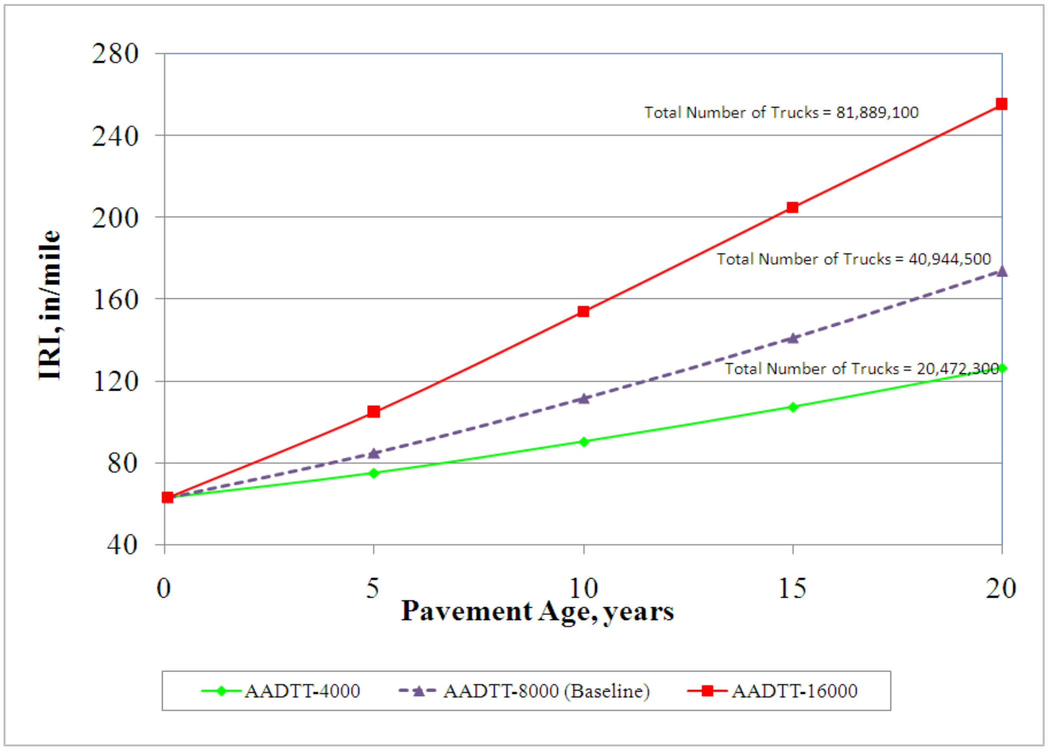


Figure 7.18 Sensitivity of JPCP IRI to Traffic Volume

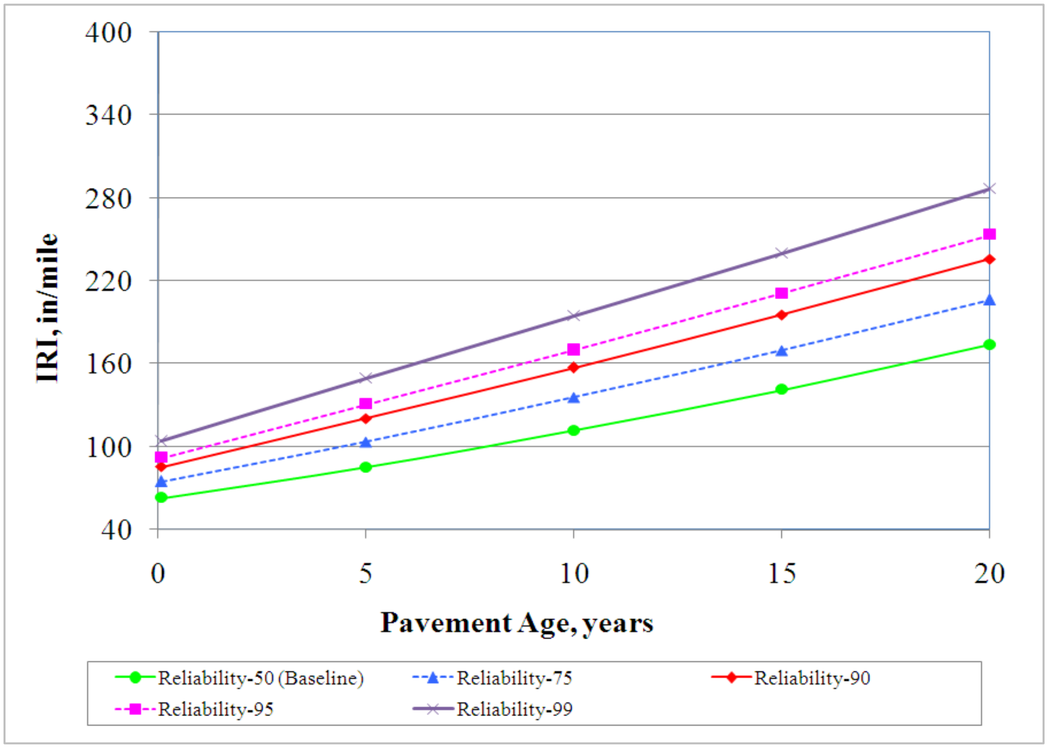


Figure 7.19 Sensitivity of JPCP IRI to Design Reliability

7.10 Joint Spacing (L)

In general, the spacing of both transverse and longitudinal contraction joints depends on local conditions of materials and environment, whereas expansion and contraction joints are primarily dependent on layout and construction capabilities. For contraction joints, when a positive temperature gradient, or base frictional resistance increases; the spacing increases as the concrete tensile strength increases. Spacing is also related to the slab thickness and joint sealant capabilities.

Determination of the required slab thickness includes an input for joint spacing. As joint spacing increases, stresses due to thermal curling and moisture warping increase. CDOT designs their PCCP using the Jointed Plain Concrete Pavement (JPCP) method. For a detailed illustration, see CDOT's current Standard Plan Sheet M-412-1. **CDOT uses a joint spacing of 15 feet maximum for concrete pavement thicknesses over 6 inches, 12 feet maximum for concrete thicknesses of 6 inches or less, and a minimum of 8 feet for any full depth pavement**.

7.11 Slab/Base Friction

The time over which full contact friction exists between the PCC slab and the underlying layer (usually the base course) is an input in M-E Design. This factor indicates (1) whether or not the PCC slab/base interface has full friction at construction, and (2) how long full friction will be available at the interface if present after construction. This factor is a significant input in JPCP cracking predictions since a monolithic slab/base structure is obtained when full friction exists at the interface.

Global calibration of JPCP performance prediction models show full contact friction exists over the life of the pavements for all base types, with the exception of cement treated or lean concrete base. Therefore, it is recommended the designer set the "months to full contact friction" between the JPCP and the base course equal to the design life of the pavement for unbound aggregate, asphalt stabilized, and cementitious stabilized base courses.

For cement treated or lean concrete base, the months of full contact friction may be reduced if attempts are made to debond the base from the PCC slab. The age at which debonding occurs can be confirmed through construction specifications and/or historical records. If no efforts are made to debond the interface, the designer is recommended to use 10 years of full interface friction.

The inputs required for M-E Design software are as follows:

- Presence or absence of PCC-Base full-friction contact
- Months until friction loss
    - Use the design life (in months) for asphalt treated and aggregate base types
    - Use 120 months for lean concrete and cement treated base

7.12 Effective Temperature Differential (°F)

An effective temperature differential includes the effects of temperature, precipitation, and wind. Wind is considered because if moist, it has an influence on the surface. Wind may be drier at the surface of the slab creating a larger differential. The same concept may be applied to temperature differences.

Curling is slab curvature produced by a temperature gradient throughout the depth of the slab. Warping is moisture-induced slab curvature. As shown in **Figure 7.20 Curling and Warping**, a positive gradient occurs when temperature and/or moisture levels at the top of a PCC slab are higher than at the bottom of the PCC slab, resulting in downward curvature. In contrast, negative gradients occur when the temperature and moisture in the slab are greater at the bottom, resulting in upward slab curvature. Curling and warping actions may offset or augment each other. During summer days, curling may be counteracted by warping. During summer nights, the curling and warping actions may compound each other. Gradients, as shown in **Figure 7.20 Curling and Warping** are primarily non-linear in nature (5).

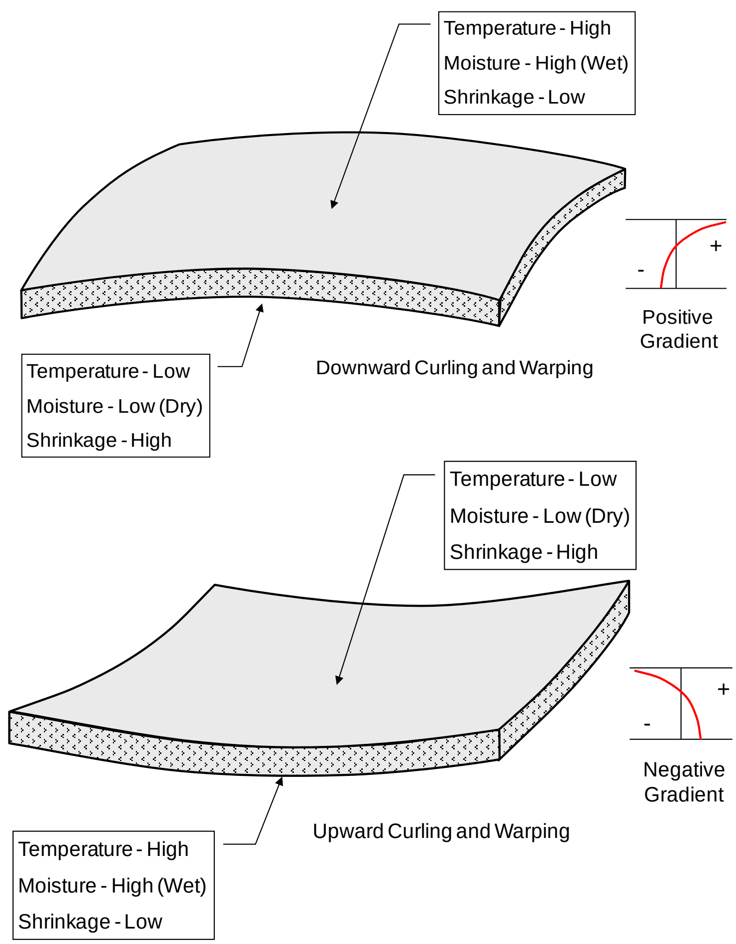


Figure 7.20 Curling and Warping

The magnitude of thermal and moisture gradients within a pavement are influenced by factors of daily temperature and relative humidity conditions, base layer type, slab geometry with constraints, shrinkage characteristics, and concrete mixture characteristics. The key characteristics of concrete mixtures that influence pavement response to thermal gradients are the coefficient of thermal expansion, thermal conductivity, and specific heat (5).

Paving operations are often performed during the morning and daytime of hot sunny days, a condition that tends to expose the newly paved slabs to a high temperature differential from the intense solar radiation and heat of hydration. Depending on the exposure conditions, a significant amount of positive temperature gradient may be present at the time of hardening. On the other hand, shrinkage occurs when the surface drying and bottom moisture wicken into the base/subbase. This resultant condition has been termed the "zero-stress temperature gradient" and is permanently locked into the slab at the time of construction. The permanent components of curling and warping are considered together and are indistinguishable. Creep occurs over time and negates the effects of the permanent curvature, but only a portion of the permanent curling and warping actually affects the long term pavement response (7). Refer to CDOT Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8) for additional discussion on curling.

M-E Design's <u>recommended value for permanent curl/warp is -10°F</u> (obtained through optimization) for all new and reconstructed rigid pavements in all climatic regions. This is an equivalent linear temperature difference from top to bottom of the slab.

7.13 Dowel Bars (Load Transfer Devices) and Tie Bars

Load transfer is used to account for the ability of a concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks. Load transfer devices, aggregate interlock, and the presence of tied longitudinal joints along with tied shoulders all have an effect. All new rigid pavements, new construction and reconstruction, including ramps, auxiliary lanes, acceleration/deceleration lanes, and urban streets will require epoxy coated smooth dowel bars in the transverse joints for load transfer. Smooth dowel bars aid the transfer of load across joints and allow thermal contraction in the PCCP. Since these transverse joints must be allowed to expand and contract, deformed tie bars <u>should never</u> be used as load transfer devices in the transverse direction. Most pavements should be dowelled.

If the pavement has shoulders, the shoulders must be portland cement concrete and tied to the travel lanes. Two major advantages of using tied portland cement concrete shoulders is the reduction of slab stress and increased service life. Concrete shoulders of three feet or greater may be considered a tied shoulder. Pavements with monolithic or tied curb and gutter that provide additional stiffness and keep traffic away from the edge may be treated as a tied shoulder. Studies have shown that on interstate projects, increasing the outside slab an additional two feet is equivalent to a tied shoulder. In a typical situation with 12-foot lane widths, the paint stripe is placed at 12 feet and the longitudinal joint is sawed and tied at 14 feet. Requiring the longitudinal joint to coincide with the lane line is recommended in urban locations. 14-foot longitudinal joints may not be appropriate for ramps, since ramps are usually much thinner in comparison to the main line pavement.

Dowel bar diameter and tie bar size versus thickness of concrete pavement and type of base is tabulated and noted in *CDOT Standard Plans, M & S Standard Drawing*, July 2012, M-412-1, Sheet 5, Reinforcing Size Table (9). The table is reproduced in **Table 7.3 Reinforcing Size Table (20-Year or Greater Design Life)**.

Table 7.3 Reinforcing Size Table (20-Year or Greater Design Life)

Pavement Thickness (T) (inches)	Dowel Bar Diameter (inches)	Minimum Concrete Cover (inches)
7 < T < 8	1	2.5
8 ≤ T ≤ 10	1.25	3
10 < T ≤ 15	1.50	3

- Tie bars for longitudinal joints shall conform to AASHTO M 284 and shall be Grade 60, epoxy-coated, and deformed.
- Tie bar length is to be 30 inches and spaced at 36 inches on center.
- Tie bar size is No. 5 when pavement is placed on unbound bases.
- Tie bar size is No. 6 when pavement is placed on lime treated soil, asphalt treated, cement treated, milled asphalt, or recycled asphalt pavement bases.

Dowel bars for transverse joints shall conform to AASHTO M 254 for the coating and to ASTM A 615, Grade 60 for the core material and shall be epoxy-coated, smooth, and lightly greased, pre-coated with wax or asphalt emulsion, or sprayed with an approved material for their full length.

Details illustrating dowel placement tolerances are shown on *CDOT Standard Plans, M & S Standard Drawing*, July 2012, M-412-1, Sheet 1. Dowel bar placement is at $^{T}/_{2}$ depth and saw cuts of $^{T}/_{4}$ (see **Figure 7.21 Details of Dowel Bar Placement)** for roadways 8 inches or greater in thickness.

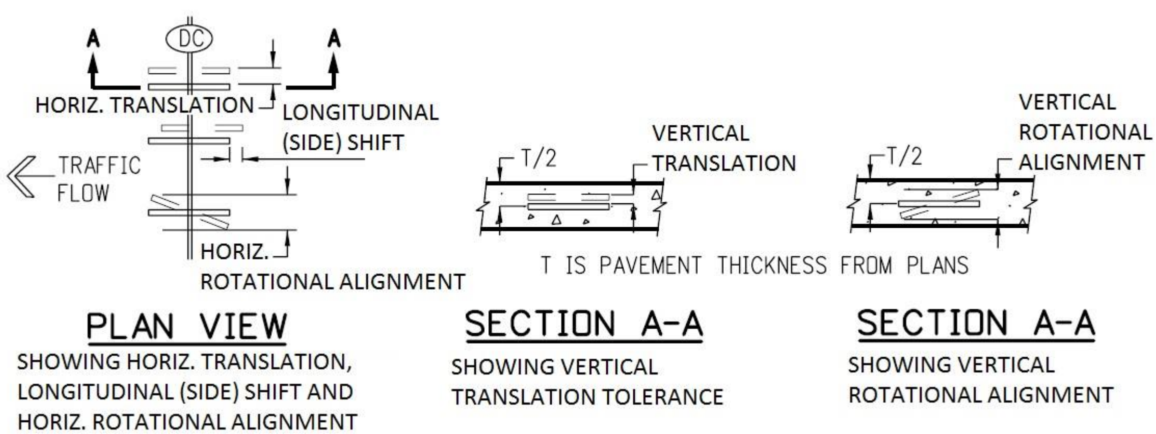


Figure 7.21 Details of Dowel Bar Placement

The tolerances are referenced in Subsection 412.13 of the CDOT *Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised. The tolerance table is reproduced in **Table 7.4 Dowel Bar Target Placement Tolerances**. Tolerances are based on *NCHRP Report 637, Guidelines for Dowel Alignment in Concrete Pavements* (22).

Table 7.4 Dowel Bar Target Placement Tolerances

Position	Tolerance (inches)
Horizontal and Vertical Translation	1
Longitudinal (Side) Shift Translation	3
Horizontal and Vertical Rotational Alignment	1.5

For tied concrete shoulders, M-E Design requires the input of the long-term or terminal deflection load transfer efficiency (LTE) between the lane (PCC outer lane slab) and shoulder's longitudinal joint. The LTE is defined as the ratio of deflections of the unloaded and loaded slabs. The higher the LTE, the greater the support provided by the shoulder to reduce critical responses of the mainline slabs. Typical long-term deflection LTE are:

- 50 to 70 percent for a monolithically constructed and tied PCC shoulder
- 30 to 50 percent for a separately constructed tied PCC shoulder
- Untied concrete shoulders or other shoulder types that do not provide significant support, therefore a low LTE value should be used.

7.14 Widened Lanes

Widened lanes may be used for some PCC designs, however the roadway's geometry and curb and gutter may dictate weather widened lanes are practical. Chapter 4 of CDOT's Design Guide shown typical sections for a standard 12 foot wide lane and is located at the following website: https://www.codot.gov/business/designsupport/bulletins_manuals/cdot-roadway-design-guide-2018/dg18-ch04/view. Widened lanes should only be used in the design lane, **Figure 7.22 Diagram of a Widened Lane**. The maximum width of the design lane is 13 feet**.** The following guidelines should also be used.

- If the thin concrete overlay is equal to or greater than 7 inches, the designer may use widened lanes if determined feasible.
- Widened lanes shall not be used to reduce the pavement thickness. Widened lanes allows the stresses to be moved from the slab edge reducing the possibility of edge and corner cracking along the wheel path. Caution should be used by the designer as the wider the slab the more likely longitudinal cracking down the center of the slab is to occur.

If a designer chooses to use widened lanes then they must submit two designs to confirm the slab thickness is not being reduced; 1) the original design using 12 foot wide lanes, and 2) the design using widened lanes. When comparing the widened lane/slab to the non-widened (12 foot wide) slab designs, all other parameters in the designs shall be identical. Only the width of the slab shall be modified to provide accurate comparison. The RME has the final decision if a widened lane design is allowed.

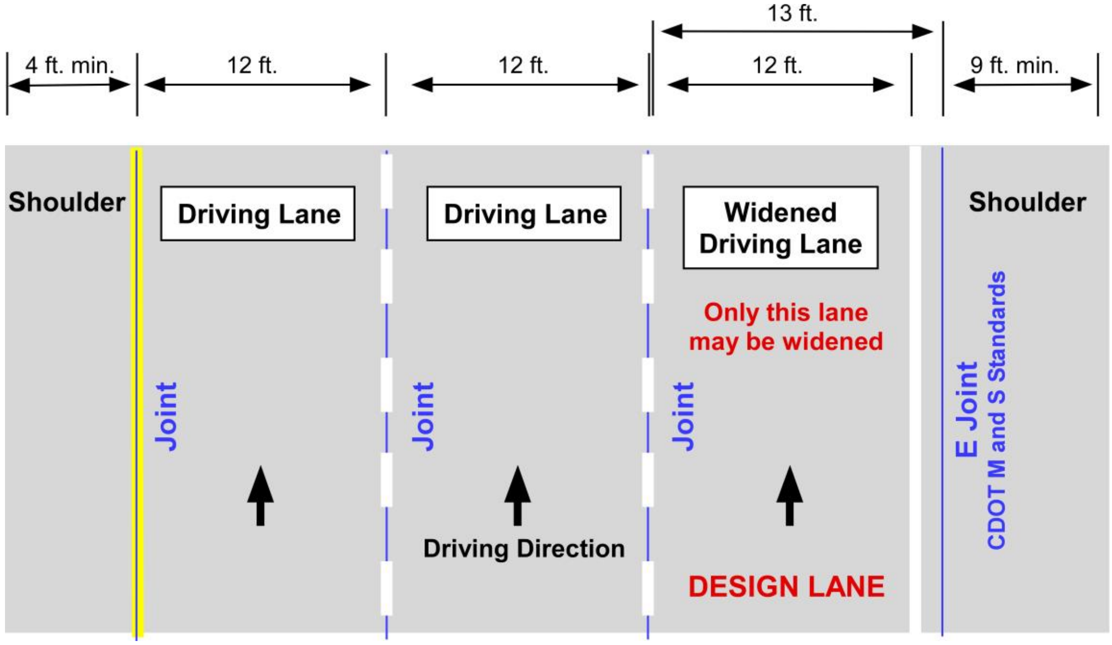


Figure 7.22 Diagram of a Widened Lane

7.15 Lane Edge Support Condition (E)

- Conventional lane width (12 feet) with free edge
- Conventional lane width (12 feet) with tied concrete shoulder
- Wide slab (i.e. 13 feet) with conventional traffic lane width (12 feet)

Refer to CDOT Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8), and *Evaluation of Premature PCCP Longitudinal Cracking in Colorado*, Final Research Report CDOT-DTD-R-2003-1, dated January 2003 (11) for additional discussion on widen slabs.

7.16 Base Erodibility

The erodibility index allows the designer to select the base's resistance to erosion. The potential for base or subbase erosion (layer directly beneath the PCC layer) has a significant impact on the initiation and propagation of pavement distress. Different base types are classified based on long-term erodibility behavior as follows:

- Class 1: Extremely erosion resistant materials
- Class 2: Very erosion resistant materials

- Class 3: Erosion resistant materials
- Class 4: Fairly erodible materials
- Class 5: Very erodible materials

Rigorous definitions of the material types that qualify under these various categories are presented in **Table 7.5 Material Types and Erodibility Class**.

Table 7.5 Material Types and Erodibility Class

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Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8 percent cement; or with long-term compressive strength > 2,500 psi. (> 2,000 psi. at 28-days), and a granular subbase layer or a stabilized soil layer, or a geotextile fabric placed between the treated base and subgrade, otherwise Class 2. (b) Hot mixed asphalt concrete with 6 percent asphalt cement that passes appropriate stripping tests (see **Figure 2.2.8**) and aggregate tests; and a granular subbase layer or a stabilized soil layer, otherwise Class 2. (c) Permeable drainage layer; asphalt treated aggregate (see **Figure 2.2.8** and **Table 2.2.57** for guidance) or cement treated aggregate (see **Table 2.2.58** for guidance) and an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement treated granular material with 5 percent cement manufactured in plant, or long-term compressive strength 2,000 to 2,500 psi (1,500 to 2,000 psi at 28-days) and a granular subbase layer or a stabilized soil layer, or a geotextile fabric placed between the treated base and subgrade; otherwise Class 3. (b) Asphalt treated granular material with 4 percent asphalt cement that passes the appropriate stripping test and a granular subbase layer or a treated soil layer, or a geotextile fabric placed between the treated base and subgrade; otherwise Class 3.
3	(a) Cement-treated granular material with 3.5 percent cement manufactured in plant, or long-term compressive strength 1,000 to 2,000 psi (750 psi to 1,500 at 28-days). (b) Asphalt treated granular material with 3 percent asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade)

7.17 Sealant Type

Sealant type applied for transverse joints is a key input used in a joint spalling model which is used for predicting JPCP smoothness. The sealant options are liquid, silicone, and preformed, however, for M-E Design the designer should use a silicone sealant.

7.18 Concrete Pavement Minimum Thickness

The minimum thickness requirement may be changed on a project to project bases depending on traffic, soil conditions, bases, etc. (see **Table 7.6 Minimum Thickness for Highways, Roadways and Bicycle Paths**).

Table 7.6 Minimum Thicknesses for Highways, Roadways, and Bicycle Paths

Design Truck Traffic	Portland Cement Concrete Pavement (inches)	Aggregate Base Course (inches)
Greater than 1,000,000 ESALs (equivalent to 250 AADTT[5] for 30 year designs)	7.0	6.0
Less than or equal to 1,000,000 ESALs for driveways	6.0	6.0
Multi-use sidewalks [1]	6.0	6.0 [2]
Sidewalks [3, 4]	4.0	6.0 [2]

Notes:
[1] Maintenance vehicles may include light duty trucks.
[2] May be reduced to 4.0 inches in thickness if approved by the RME.
[3] Pedestrian and bicycle only, typical snow removal equipment would be a snow blower.
[4] Per Standard Plan No. M-609-1, Curb, Gutters and Sidewalks of CDOT's *M&S Standards*, July 2012.
[5] AADTT is the average annual daily truck traffic.

7.19 Concrete Pavement Texturing, Stationing, and Rumble Strips

- **Texture:** Final surface of the pavement shall be uniformly textured with a broom, burlap drag, artificial turf, or diamond ground to obtain a specified average texture depth of the panel being greater than 0.05 inches. Refer to CDOT Final Research Report CDOT-2012-10, *Assessment of Concrete Pavement Texturing Methodologies in Colorado*, dated October 2012 (25), and CDOT Final Research Report CDOT-DTD-R-2005-22, *PCCP Texturing Methods*, dated January 2005 (12).

- **Stationing**: Stationing shall be stamped into the outside edge of the pavement at 500-foot intervals on each outside mainline shoulder as shown on *CDOT Standard Plans, M & S Standard Drawing*, July 2012, Standard Plan No. M-412-1, Concrete Pavement Joints.

- **Rumble Strips:** When rumble strips are installed, they shall be of the style and location as shown on *CDOT Standard Plans, M & S Standard Drawing*, July 2012, Standard Plan Sheet No. M-614-1, Rumble Strips.

7.20 Concrete Pavement Materials Selection

Concrete pavement is a construction paving material that consists of cement (commonly portland cement), other cementitious materials (fly ash), aggregate (gravel and sand), water, and chemical admixtures. The concrete solidifies and hardens after mixing and placement due to a chemical process known as hydration. The water reacts with cement, which bonds the other components together, eventually creating a hard stone-like material.

CDOT designates a concrete pavement mix as a Class P. **Table 7.7 Concrete Classification** shows the specified mix properties. Class E is a fast track mix that may be substituted for Class P. Class P and E are defined in Section 601 Structural Concrete and 701 Hydraulic Cement of *CDOT Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised.

Table 7.7 Concrete Classification

Concrete Class	Required Field Compressive Strength (psi)	Minimum Cementitious Content (lbs/yd^3)	Air Content Percent Range (Total)	Maximum Water Cement Ratio
P	4,500 at 28 days	520	4-8	0.44
E	4,500 at 28 days	520	4-8	0.44
Note: Table taken from Standard Special Provision: *Revision of Sections 105, 106, 412, 601 and 709 Conformity to the Contract of Portland Cement Concrete Pavement and Dowel Bars and Tie Bars for Joints*, dated April 30, 2015				

7.20.1 Understanding pH in Concrete Mixes

A brief explanation of pH is presented in **Section S.1.4.2 pH Scale** in the **SUPPLEMENT** chapter. When applied to pavement design, freshly poured concrete can have a pH of 11 to 13 making it very alkaline. This high initial alkalinity helps resist corrosion, but as concrete ages, the pH can drop to around 8 increasing the degradation of steel reinforcement and load transfer devices. The high alkalinity of concrete can also affect the performance of fresh and hardened concrete when admixtures are used.

7.20.2 Alkali Aggregate Reactivity

The high alkalinity of concrete can cause serious problems when interacting with different parts of the mix, namely alkali-silica and alkali-carbonate reactions. Alkali-silica reactivity (ASR) is the process in which certain minerals in the aggregate along with the presence of moisture are broken down by the highly alkaline environment of concrete. This process produces a gel-like substance that expands adding tensile forces to the concrete matrix, which then leads to external cracking of the concrete slab (13). The cracking allows more water to infiltrate creating more gel and more expansion. Ultimately, the concrete destroys itself. The ASR chemical reaction is expressed in equation **Eq. 7-1** (15).

$$\mathbf{SiO_2 + 2NaOH + H_2O \rightarrow NA_2SiO_3 + 2H_20} \qquad \textbf{Eq. 7-1}$$
$$\text{silica} + \text{alkali} + \text{water} \rightarrow \text{alkali-silica gel}$$

Alkali-carbonate reactivity (ACR) is much less common than ASR, but it does have similar expansive properties that occur within the aggregate and deteriorate concrete pavement. The ACR reaction is dependent on certain types of clay rich, or impure, dolomitic limestones rarely used in concrete because of their inherently weak structure (14). The ACR chemical reaction known as dedolomitization is represented in equation **Eq. 7-2** (15). The cracking pattern is shown in **Figure 7.23 Idealized Sketch of Cracking Pattern in Concrete Mass Caused By Internal Expansion**.

$$\mathbf{CaMg(CO_3)_2 + 2NaOH \rightarrow Mg(OH_2) + CaCO_3 + NaCO_3} \qquad \textbf{Eq. 7-2}$$
$$\text{Dolomite} + \text{Alkali} \rightarrow \text{Brucite} + \text{Calcite} + \text{Sodium Carbonate}$$

"Sandgravel" aggregates in parts of Kansas, Nebraska, Colorado, and Wyoming, especially those from the Platte, Republican, and Laramie Rivers, have been involved in the deterioration of concrete (17). In 1983 a team was formed to evaluate the concrete pavement condition in Colorado and to recommend rehabilitation methods for these pavements. This team identified that one-third of the pavements inspected suffered from ASR (19). A follow up study conducted in 1987 focused on the cause of ASR in Colorado. The study concluded that aggregates in the Denver Metro area showed no signs of ASR reaction, but aggregate from the Three Bells pit near Windsor demonstrated rapid signs of expansion. This study led CDOT to modify its specifications and require low alkali cement for all concrete pavement, it also identified the need for Class F fly ash in areas were reactive aggregates have been a problem (20).

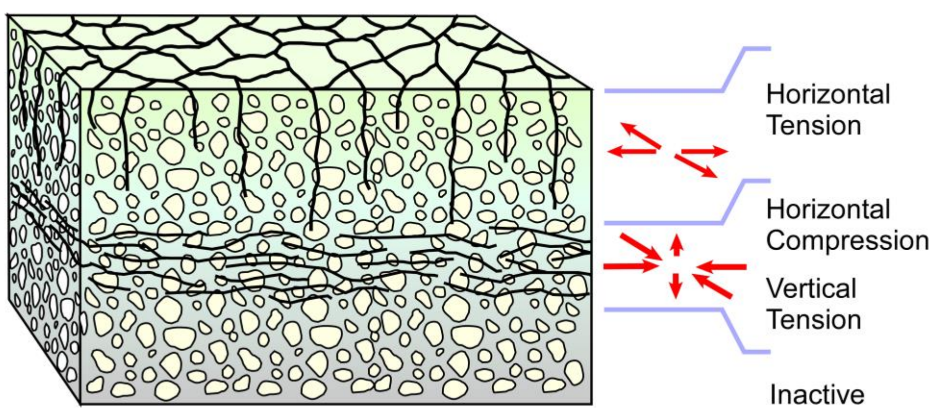

Figure 7.23 Idealized Sketch of Cracking Pattern in Concrete Mass Caused by Internal Expansion

(Figure 93, *Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual*, July 2006)

7.20.3 Sulfate Resistant Concrete Pavement

Sulfates may be found in soil and water and are referred to as "alkali". The sulfates in soils and water are the main source of external sulfate attack on concrete pavement. Although the mechanism of sulfate attack is complex, it is primarily thought to be caused by two chemical reactions: 1) the formation of gypsum through the combination of sulfate and calcium ions, and/or 2) the formation of ettringite through the combination of sulfate ions and hydrated calcium aluminate (18). Ettringite ($Ca_6[Al(OH)_6]^2(SO_4)_3 \cdot 26H_2O_2$) is a high-sulfate, calcium sulfo-aluminate mineral which naturally occurs in curing concrete. The problem appears when ettringite forms after the concrete has set, this is known as Delayed Ettringite Formation (DEF). This process is extremely harmful, because as ettringite crystals form they expand and create internal tensile stresses in the cement matrix (21). These stresses will cause the concrete to crack, but may not be apparent for 3-10 years (18).

Sulfate attack is a chemical reaction between sulfates and the calcium aluminate (C_3A) in cement, resulting in surface softening (22) (see **Figure 7.24 Sulfate Attack**). Steps taken to prevent the development of distress due to external sulfate attack include minimizing the tri-calcium aluminate content in the cement or reducing the quantity of calcium hydroxide in the hydrated cement paste though the use of pozzolanic materials. It is also recommended that a w/c ratio less than 0.45 be used to help mitigate external sulfate attack (18).

Severity levels of potential exposure to sulfate attack have been developed. **Table 7.8 Requirements to Protect Against Damage to Concrete by Sulfate Attack from External Sources of Sulfates** shows the classification levels of potential exposure. Concrete pavement mix designs must provide protection against sulfate attack, thus cementitious material requirements are modified. As the severity of potential exposure increases, the cementitious material requirements become more stringent and the water cement ratio becomes less stringent. Refer to Section 601 Structural Concrete of CDOT *Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised for additional cementitious material requirements.

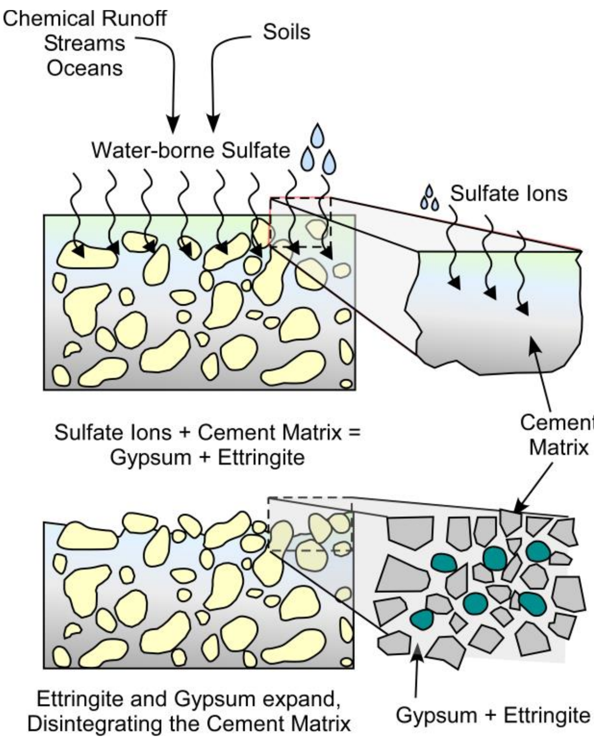

Figure 7.24 Sulfate Attack
*(*Figure 5-18*, Integrated Materials and Construction Practices for Concrete Pavement: State-of-the-33 Practice Manual)*

Table 7.8 Requirements to Protect Against Damage to Concrete by Sulfate Attack from External Sources of Sulfates

Severity of Potential Exposure	Water-soluble Sulfate (SO_4), Percent Dry Soil	Sulfate (SO_4) in Water (ppm)	Maximum Water Cement Ratio	Cementitious Material Requirements
Class 0	0.00 to 0.10	0 to 150	0.50	Class 0
Class 1	0.11 to 0.20	150 to 1,500	0.50	Class 1
Class 2	0.21 to 2.00	1,501 to 10,000	0.45	Class 2
Class 3	2.01 or greater	10,001 or greater	0.40	Class 3

References

1. *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, DC, 1993.

2. *Supplement AASHTO Guide for Design of Pavement Structures, Part II, - Rigid Pavement Design & Rigid Pavement Joint Design*, American Association of State Highway and Transportation Officials, Washington, DC, 1998.

3. *Techniques for Pavement Rehabilitation*, Nichols Consulting Engineers, Chtd., U.S. Department of Transportation, Federal Highway Administration and National Highway Institute, Arlington, Virginia, Sixth Edition, January 1998. http://www.fhwa.dot.gov/pavement/pub_details.cfm?id=375

4. *LTPP Data Analysis Phase I: Validation of Guidelines for k-Value Selection and Concrete Pavement Performance Prediction*, Publication FHWA-RD-96-198, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, VA 22101-2296, January 1997.

5. Wells, Steven A., *Early Age Response of Jointed Plain Concrete Pavements to Environmental Loads*, thesis was presented and defended on July 27, 2005, University of Pittsburgh, School of Engineering, 2005.

6. *Computer-Based Guidelines for Concrete Pavements, Volume II: Design and Construction Guidelines and HIPERPAV® II User's Manual*, Publication No. FHWA-HRT-04-122, U.S. Department of Transportation, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, Mclean VA 22101-2296, February 2005.

7. *Guide for Mechanistic-Empirical Design*, Final Report, NCHRP Project 1-37A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Submitted by ARA, INC., ERES Consultants Division, Champaign, IL, March 2004.

8. Ardani, Ahmad, *Implementation of Proven PCCP Practices in Colorado*, Final Report, Report No. CDOT-DTD-R-2006-9, Colorado Department of Transportation, Research Branch, April 2006.

9. CDOT Standard Plans, Concrete Pavement Joints M-412-1, July 2006.

10. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2005.

11. Ardani, Ahmad, Hussain, Shamshad, and LaForce, Robert, *Evaluation of Premature PCCP Longitudinal Cracking in Colorado*, Final Report, Report No. CDOT-DTD-R-2003-1, Colorado Department of Transportation, Research Branch, January 2003.

12. Ardani, Ahmad and Outcalt, William (Skip), *PCCP Texturing Methods*, Final Report, Report No. CDOT-DTD-R-2005-22, Colorado Department of Transportation, Research Branch, January 2005.

13. *Alkali Silica Reactivity*, Canadian Strategic Highway Research Program, Transportation Association of Canada, Technical Brief #13, April 1996, 1 October 2007. http://www.cshrp.org/products/csbf-e13.pdf

14. *Alkali Aggregate Reaction*, Concrete Technology, Portland Cement Association, 2007, 2 October 2007. http://www.cement.org/tech/cct_dur_AAR.asp

15. *Alkali-Aggregate Reaction in Roads & Bridges*, West, Graham, London: Thomas Telford Ltd. 1996 pp. 10-12.

16. *Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual*, Publication No. FHWA-HRT-04-150, 1401 East Broad Street, Richmond, VA 23219 and Federal Highway Administration, Office of Infrastructure R&D, 6300 Georgetown Pike, McLean, VA 22101, July 2006.

17. *Standard Practice for Concrete for Civil Works Structures*, Publication No. EM 1110-2-2000, Department of the Army, US Army Corps of Engineers, Washington, DC 20314-1000, 1 February 1994.

18. *Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements - Volume 1: Final Report*, Report No. FHWA-RD-01-163, U.S. Department of Transportation, Federal Highway Administration, Research, Development and Technology, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, Mclean VA 22101-2296, August 2002.

19. *Rehabilitation of Concrete Pavements*, Report No. CDOH-83-1, Colorado Department of Highways, U.S. Department of Transportation, Federal Highway Administration, January 1983.

20. *Colorado Reactive Aggregate*, Report No. CDOH-DH-DML-87-5, Colorado Department of Highways, U.S. Department of Transportation, Federal Highway Administration, June 1987.

21. *Integrated Materials and Construction Practices for Concrete Pavement: State-of-the-Practice Manual*, FHWA Publication No. HIF-07-004, U.S. Department of Transportation, Federal Highway Administration, October 2007.

22. *Guidelines for Dowel Alignment in Concrete Pavements*, NCHRP Report 637, National Cooperative highway Research Program, Transportation Research Board, Washington, D.C., 2009.

23. *Standard Specifications for Road and Bridge Construction*, Colorado Department of Transportation, Denver, Co, 2011. http://www.coloradodot.info/business/designsupport/construction-specifications/2011-Specs

24. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.

25. Rasmussen, Robert Otto and Sohaney, Richard C., *Assessment of Concrete Pavement Texturing Methodologies in Colorado*, CDOT Final Research Report CDOT-2012-10, dated October 2012 (24).