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. The steps necessary to conduct the soil survey investigation include the following:

Step 1. Obtain Clearance and Locates: When required, provide for necessary landowner permission to trespass and utility clearance or locates prior to the start of work.

Step 2. Determine Sampling Locations and Methods: Test holes can be drilled, dug by hand, power augered, back hoed, or completed by any other practical method. The method used should ensure the attainment of representative, uncontaminated samples. Sampling and testing procedures should conform to the following requirements:

- **Determine Horizontal and Vertical Test Hole Locations** for virgin alignment. Test holes should be no farther apart than approximately 500 feet in continuous cut sections and no farther than approximately 1,000 linear feet in other sections. In addition, test holes should be drilled whenever there is a variation in soil or geological conditions. Sampling locations and depths should be coordinated with the Region Materials Engineer (RME) in order to obtain a sufficient number of test holes and materials to outline subsurface complexities.

- **Determine Water Table Depth** by drilling at least one boring a minimum of ten feet in depth. If the water table is not encountered within the ten feet, additional depth should be coordinated with the RME. Borings drilled to determine water table depth should occur at a minimum distance of 1,000 linear feet, if a variation in geologic or geomorphic conditions occurs, or as determined by the RME.

- **Determine Coring Locations for Pavement Rehabilitation** of existing roadways and coordinate with the Region Materials Engineer. Coring should be spaced to provide sufficient data of pavement thickness and condition to perform pavement design. Researching as-constructed plans will help in determine coring locations. Cores should be retained for further evaluation in the laboratory.

- **Collect Subgrade Soil Samples and Test**:

  - **Classification**: per AASHTO M 145 and U.S. Army Corps of Engineers
  - **Soil Moisture Density**: per AASHTO T 99
  - **Resistance Value:** Colorado Procedure L 3101 and L 3102
  - **Swell Consolidation Test**: ASTM D 4546 at 200 psf surcharge
  - **Sulfate Ion Content** in Water or Water Soluble Sulfate Ion Content in Soil: Colorado Procedure L 2103. Refer to Chapter 200 of the *Field Materials Manual*.

Step 3. Provide Documentation of sample locations and other details required in CDOT Forms #554 (Soil Survey Field Report) and #555 (Preliminary Soil Survey). More information on the preliminary soil survey can be found in Chapter 200 of the *Field Materials Manual*.

The engineering properties of the subgrade are obtained from the soil subgrade investigation. The designer should assemble all information gathered during the soil investigation survey. This information will form the basis for characterizing subgrade properties for design.

4.3 Subgrade and Embankment

Subgrade can be categorized as shown in **Figure 4.1 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.

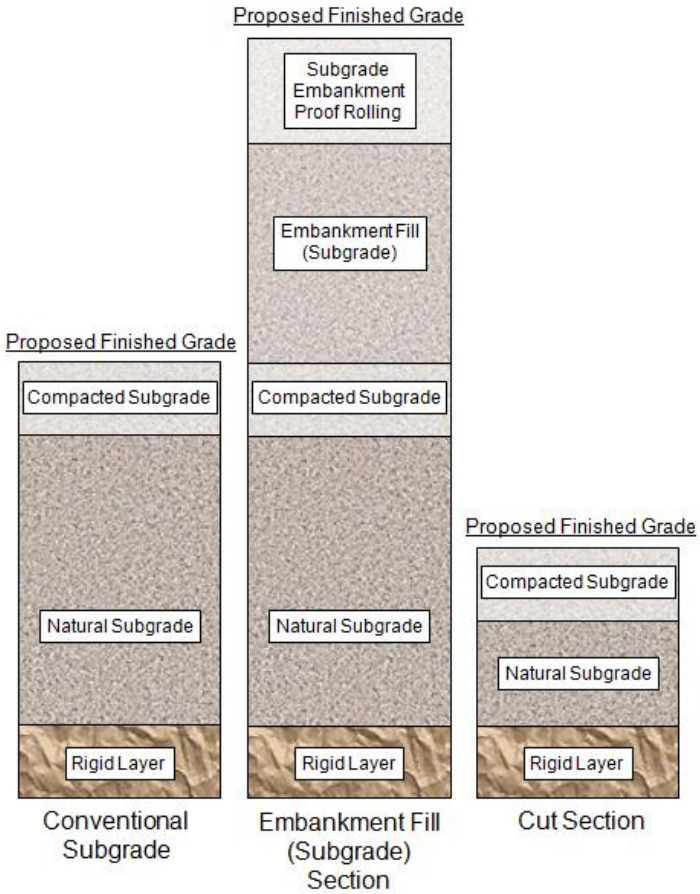


Figure 4.1 Subgrade Preparation

- **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 4.2 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 4.3 Subgrade Material Properties in M-E Design** and **Figure 4.4 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.

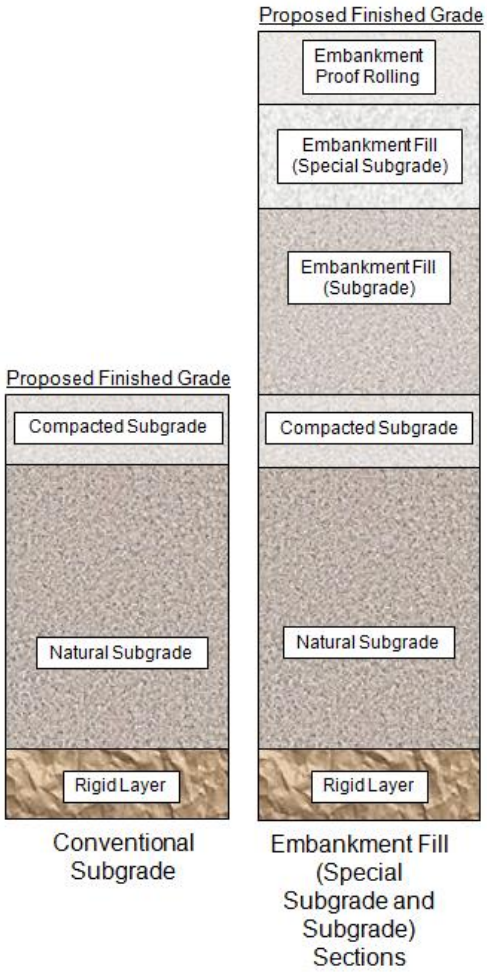


Figure 4.2 Special Cases of Embankment Fill

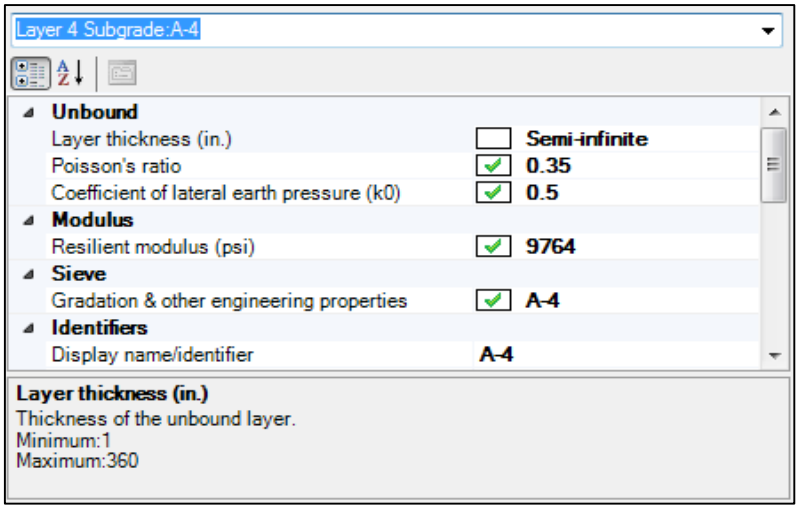
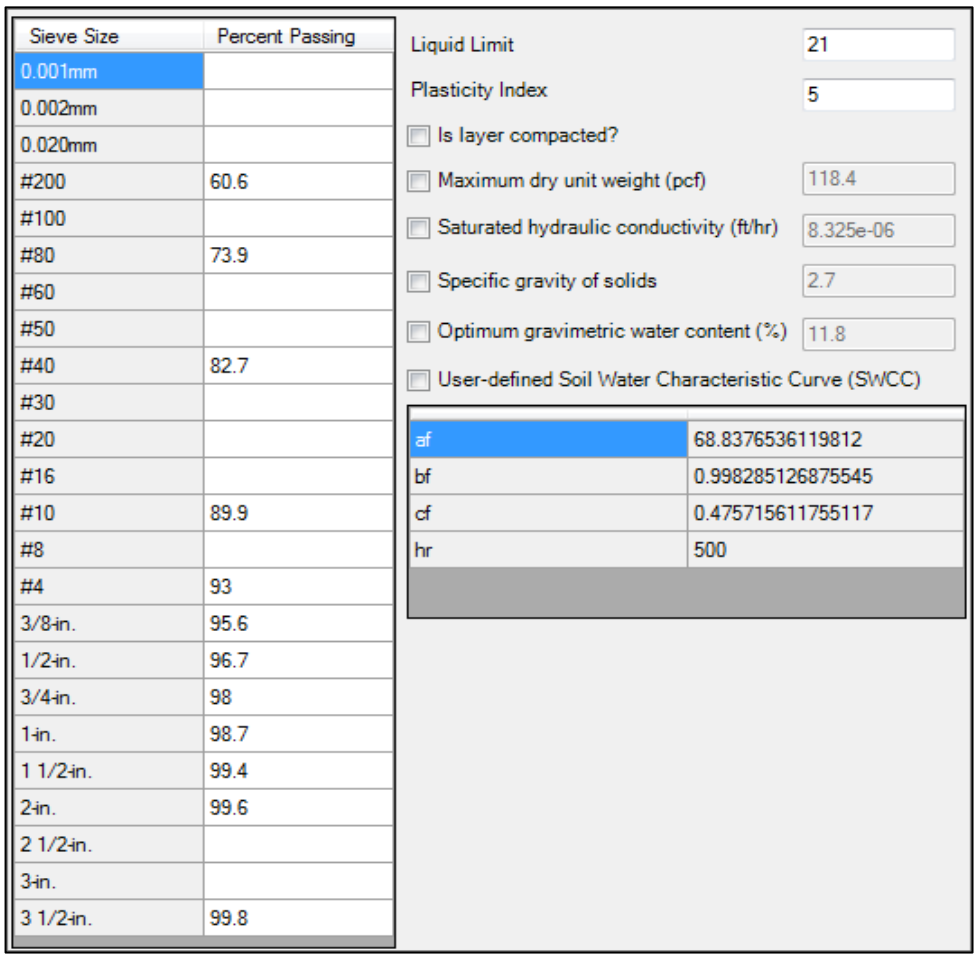


Figure 4.3 Subgrade Material Properties in the M-E Design



Sieve Size	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	60.6
#100	
#80	73.9
#60	
#50	
#40	82.7
#30	
#20	
#16	
#10	89.9
#8	
#4	93
3/8-in.	95.6
1/2-in.	96.7
3/4-in.	98
1-in.	98.7
1 1/2-in.	99.4
2-in.	99.6
2 1/2-in.	
3-in.	
3 1/2-in.	99.8

Liquid Limit	21
Plasticity Index	5
Is layer compacted?	
Maximum dry unit weight (pcf)	118.4
Saturated hydraulic conductivity (ft/hr)	8.325e-06
Specific gravity of solids	2.7
Optimum gravimetric water content (%)	11.8
User-defined Soil Water Characteristic Curve (SWCC)	

af	68.8376536119812
bf	0.998285126875545
cf	0.475715611755117
hr	500

Figure 4.4 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 4.2 Recommended Subgrade Inputs in New Flexible and JPCJ Designs**.
- HMA Overlays of Existing Flexible Pavement: **Table 4.3 Recommended Subgrade Inputs for HMA Overlays of Existing Flexible Pavemen**t.
- Overlays of Existing Rigid Pavement: **Table 4.4 Recommended Subgrade Inputs for Overlays of Existing Rigid Pavement**.

Table 4.2 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 4.3 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 4.4 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 4.5 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)

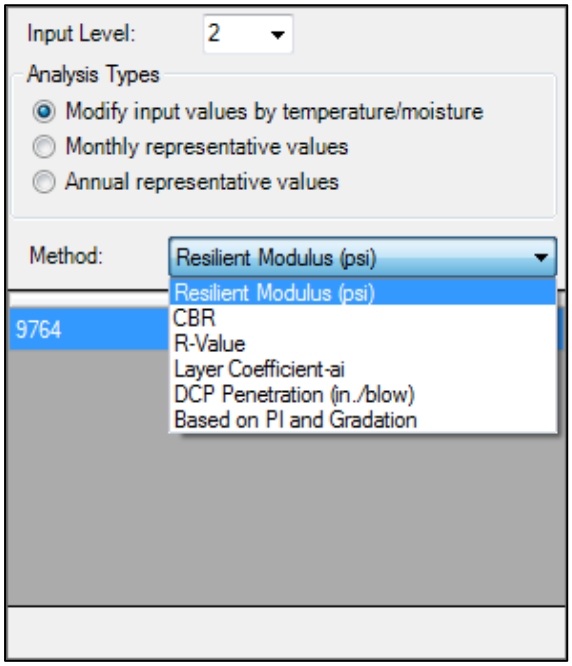


Figure 4.5 M-E Design Software Screenshot for Level 2 Resilient Modulus Input

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 **Table 4.2 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Level 3 Inputs

Typical M_r values for Level 3 inputs are presented in **Table 4.5 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 4.5 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 4.6 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 4.5 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.

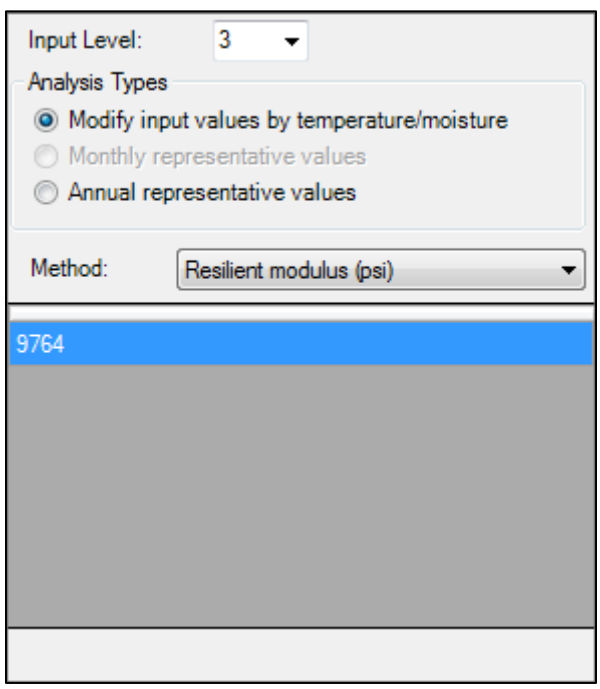


Figure 4.6 M-E Design Software Screenshot for Level 3 Resilient Modulus Input

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 4.6 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 **Table 4.2 Recommended Subgrade Inputs in New Flexible and JPCP Designs**.

Table 4.6 Average Backcalculated to Laboratory Determined Elastic Modulus Ratios

Layer Type	Location	Mean E_R/M_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 **Table 4.2 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 **Table 4.2 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 4.8 k-value vs. Soil Classification**. Greater detail can be found using **Table 4.7 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 4.7 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 4.7 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 4.9 k-values Versus Degree of Saturation for A-4 through A-7 Soils**. Each line represents the middle range of reasonable values $\pm$ 40 psi/in.

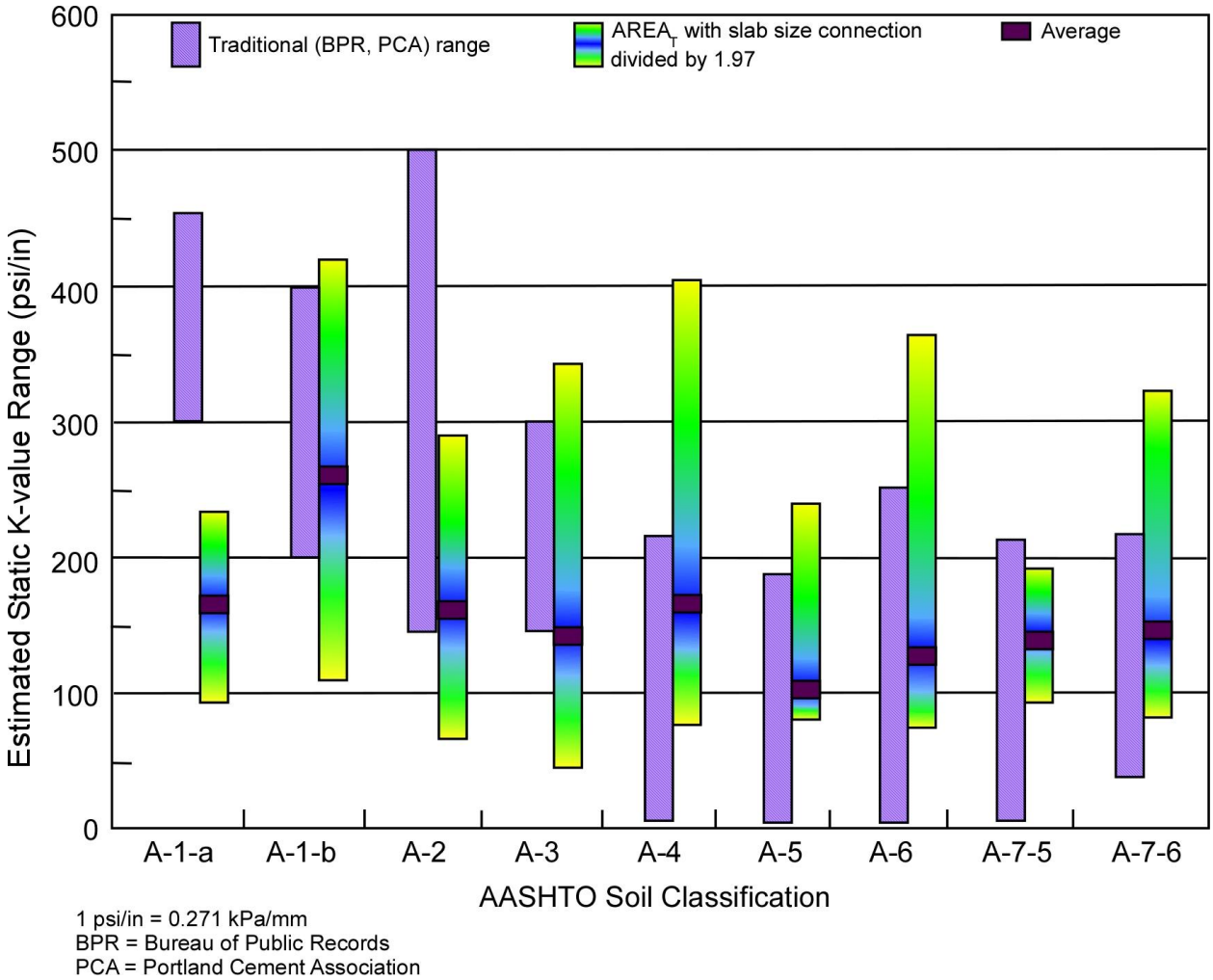


Figure 4.7 k-value vs. Soil Classification

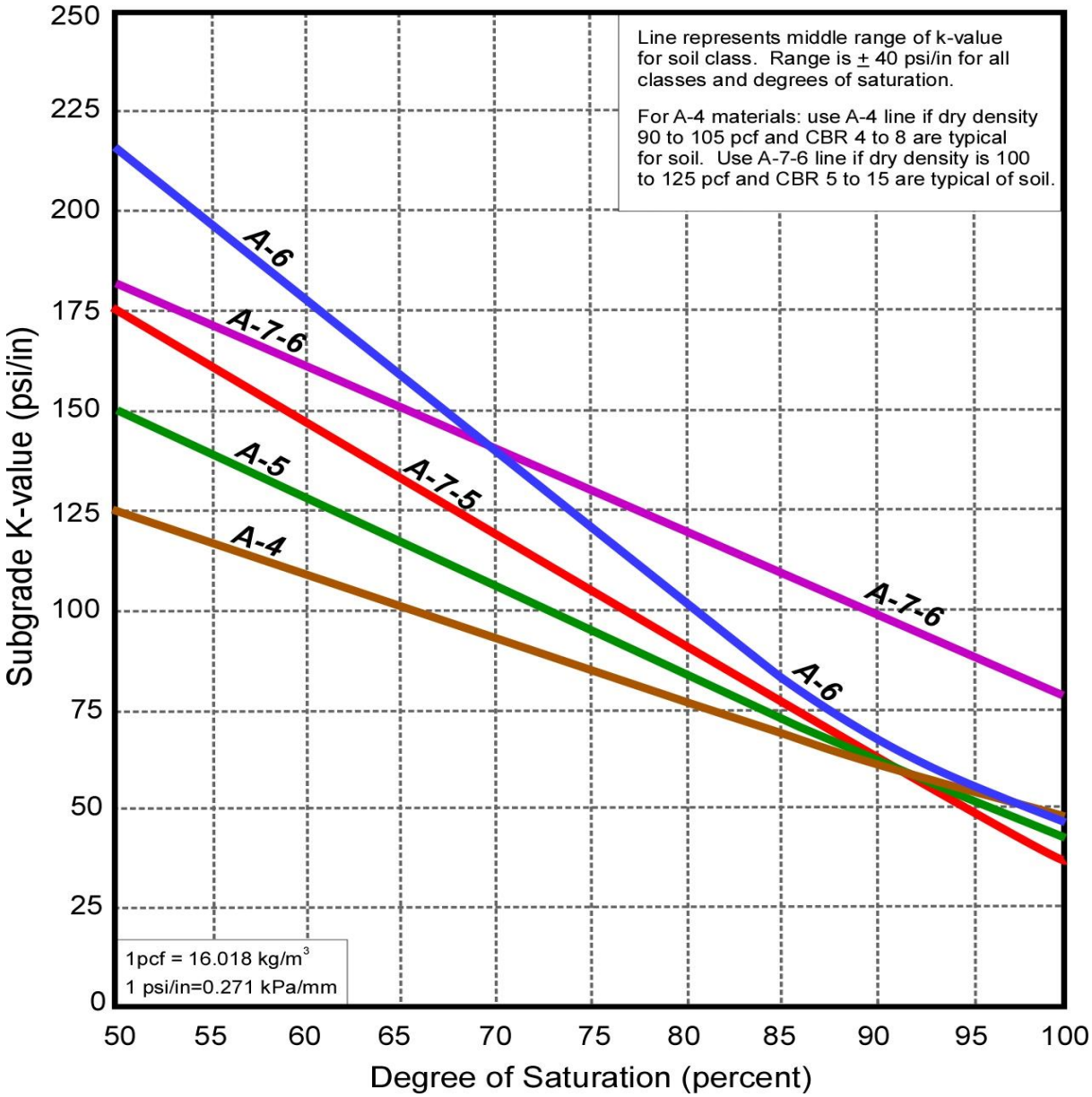


Figure 4.8 k-value vs. Degree of Saturation for A-4 Through A-7 Soils

Table 4.7 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 4.9 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.

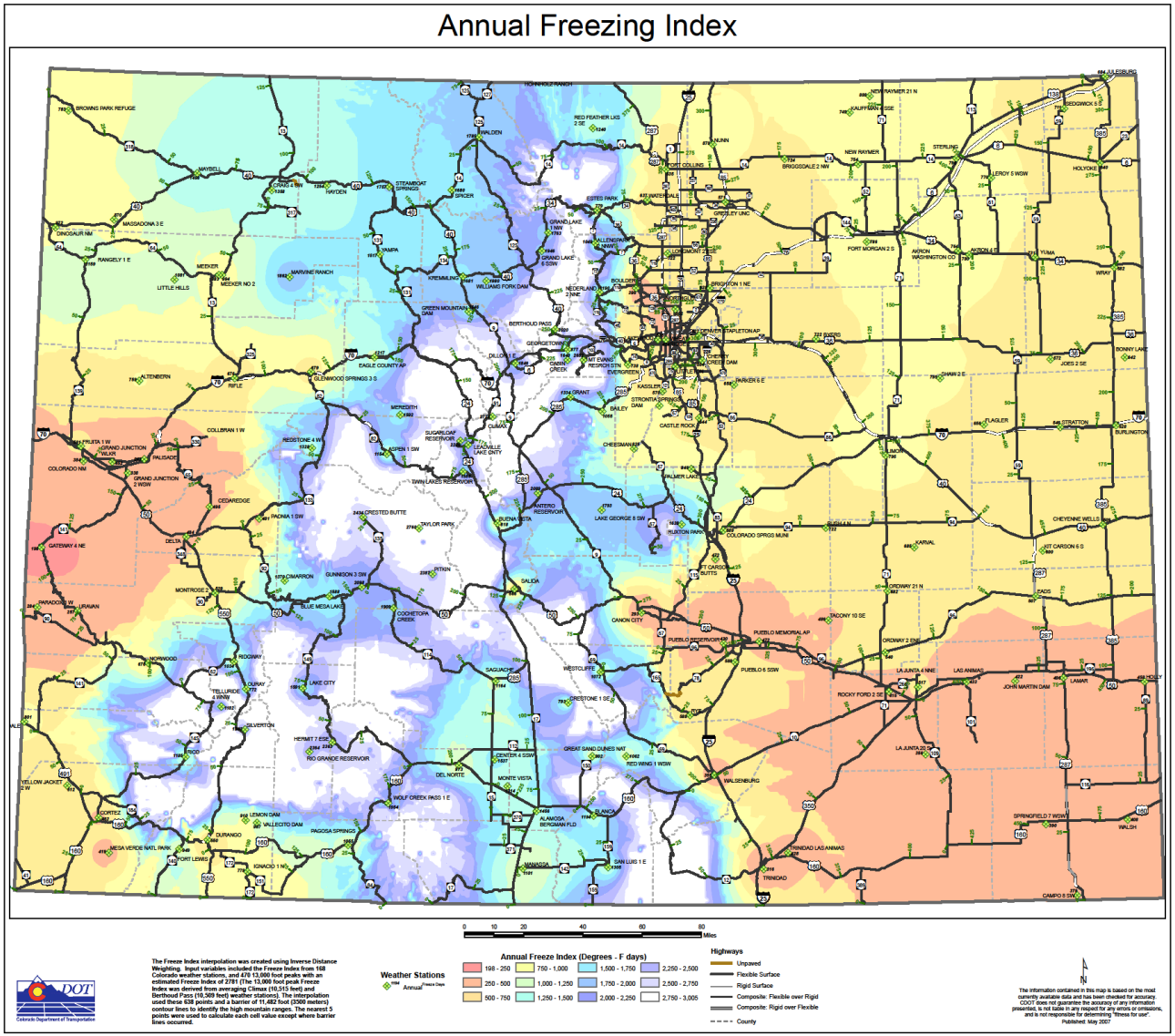


Figure 4.9 Colorado Annual Freezing Index (Degrees-Fahrenheit Days)

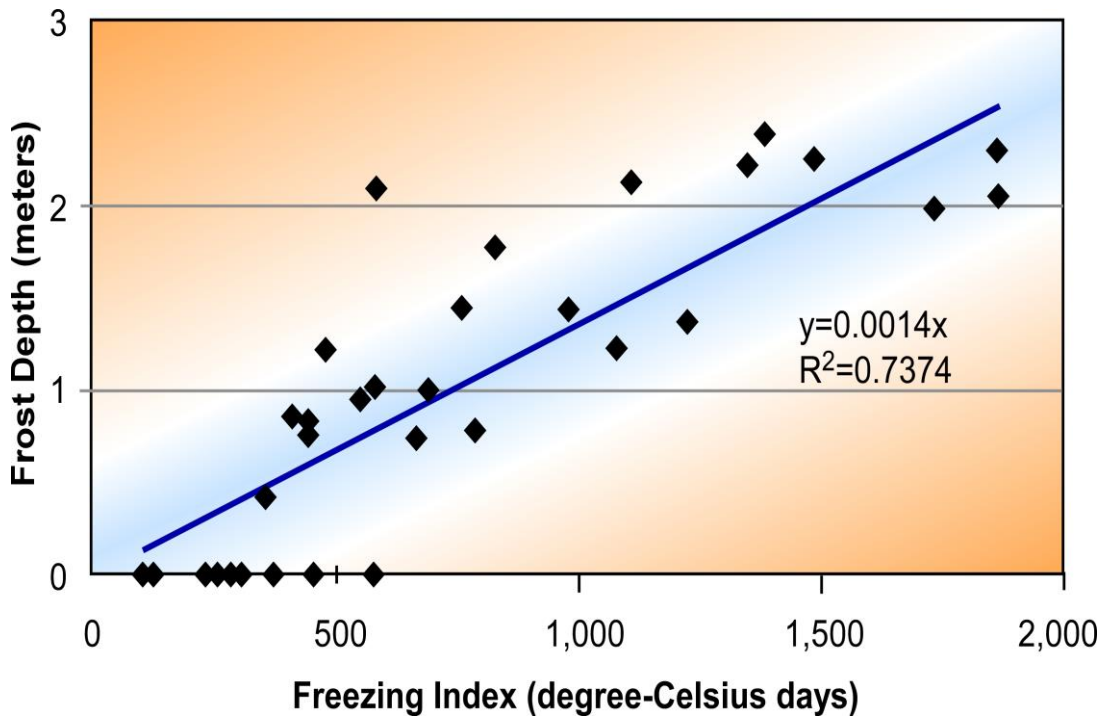


Figure 4.10 Frost Depth to Annual Freezing Index

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 4.10 Frost Depth to Annual Freezing Index**). The data scatter is influenced by local site conditions. Refer to **Figure 4.11 Frost Susceptible Soil Classifications** for possible scatter.

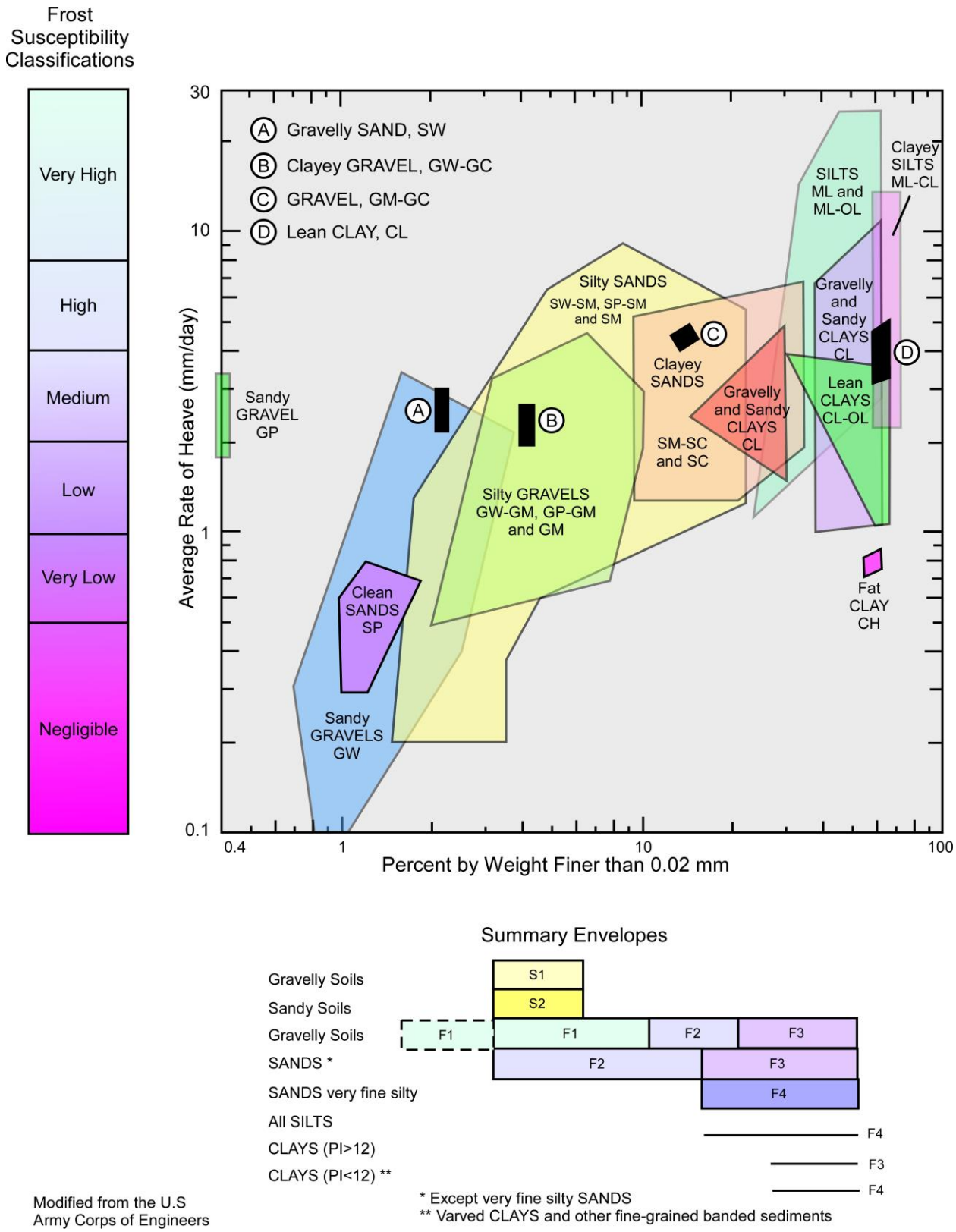


Figure 4.11 Frost Susceptible Soil Classifications

Figure 4.11 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 4.8 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 4.12 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 4.8 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

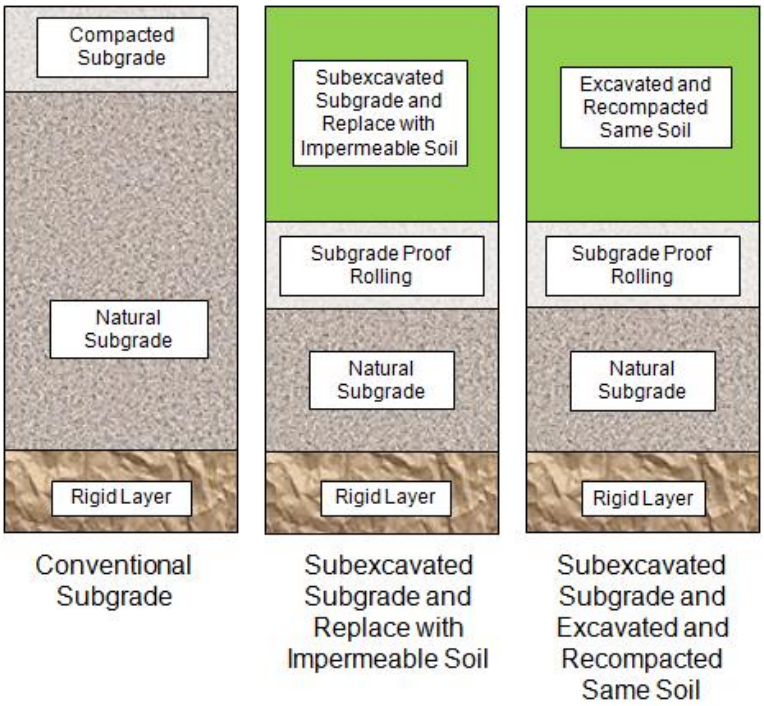


Figure 4.12 Subexcavated Subgrade Layers

The risk of swell potential is always a concern to the designer. The categories of the "swell damage risk" is shown in **Table 4.9 Probable Swell Damage Risk**. The designer should use **Table 4.9 Probable Swell Damage Risk** and **Table 4.8 Treatment of Expansive Soils** to decide the risk.

Table 4.9 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 4.12 Effective Depth of Moisture Treatment** and **Figure 4.23 Recommended Depth of Moisture Treatment** graph the concept of slope of the bump and depth of recommended moisture treatment**. Table 4.9, Figure 4.13 Effective Depth of Moisture Treatment** and **Figure 4.14 Recommended Depth of Moisture Treatment** use the percent swell to determine the depth of subgrade treatment. **Table 4.8 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.

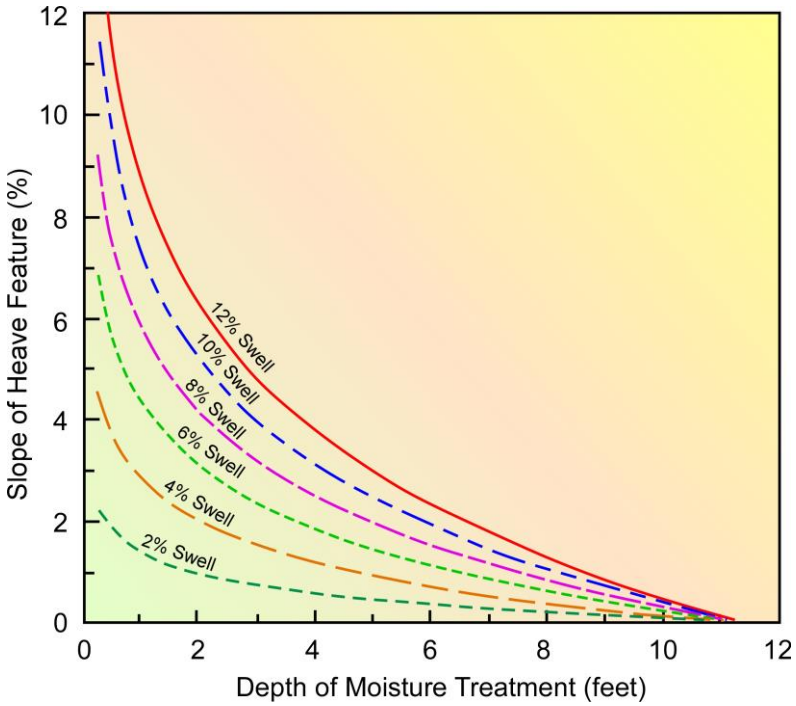


Figure 4.13 Effective Depth of Moisture Treatment

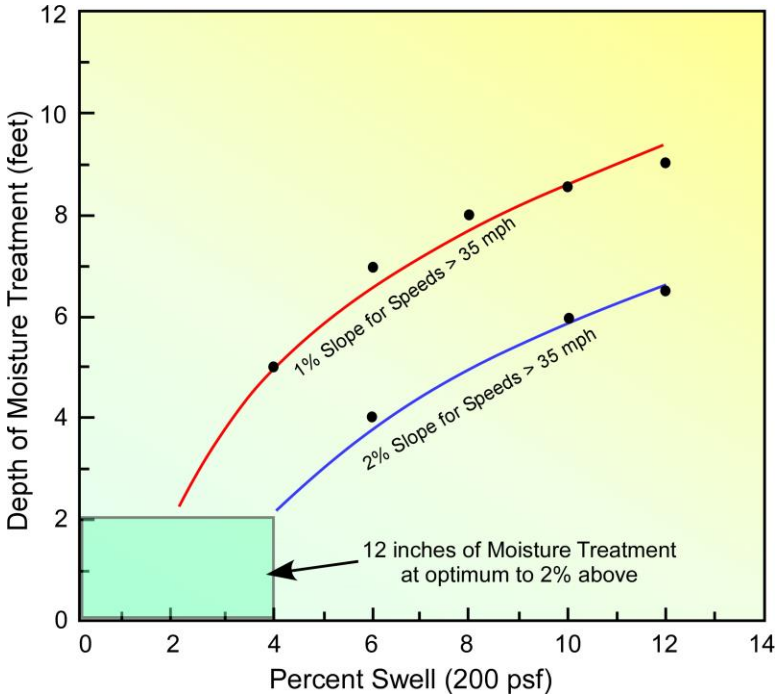


Figure 4.14 Recommended Depth of Moisture Treatment

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 4.15 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.

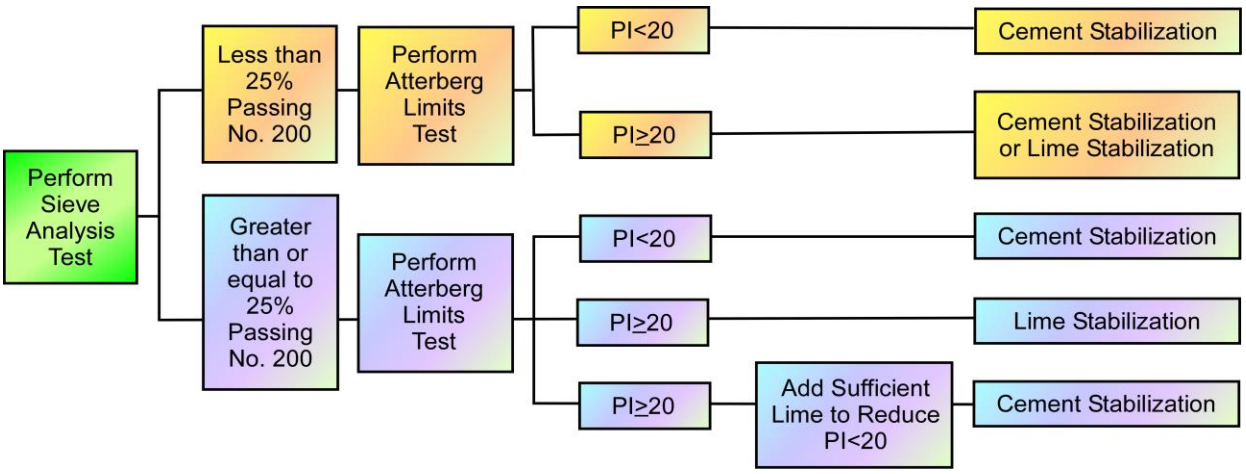


Figure 4.15 Lime/Cement-Stabilization Flow Chart

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 4.16 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 4.8 Treatment of Expansive Soils.** The recompaction shall be at 2 ± 1 percent above optimum moisture control, see **Figure 4.16 Lime Treated Structural Subgrade Layer. Figure 4.17 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.

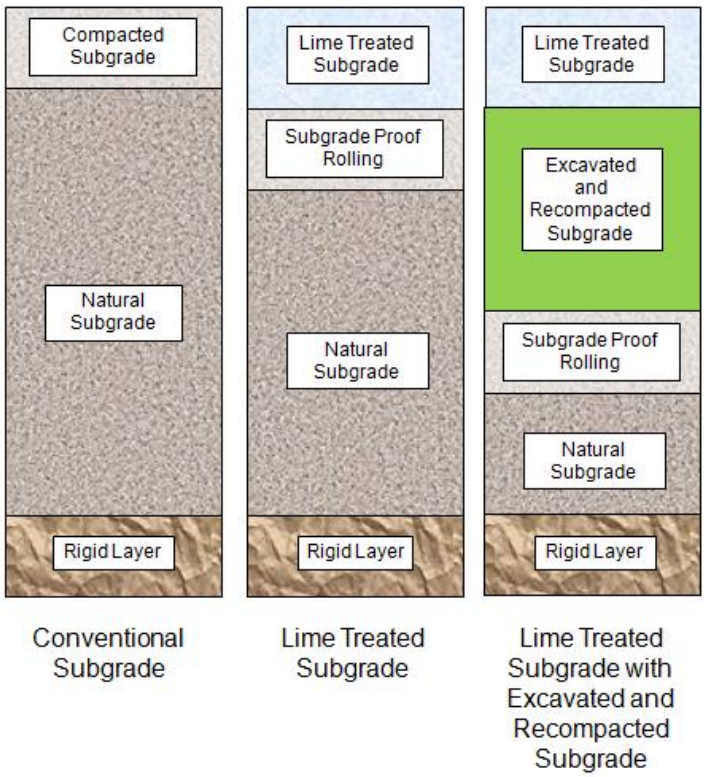


Figure 4.16 Lime Treated Structural Subgrade Layer

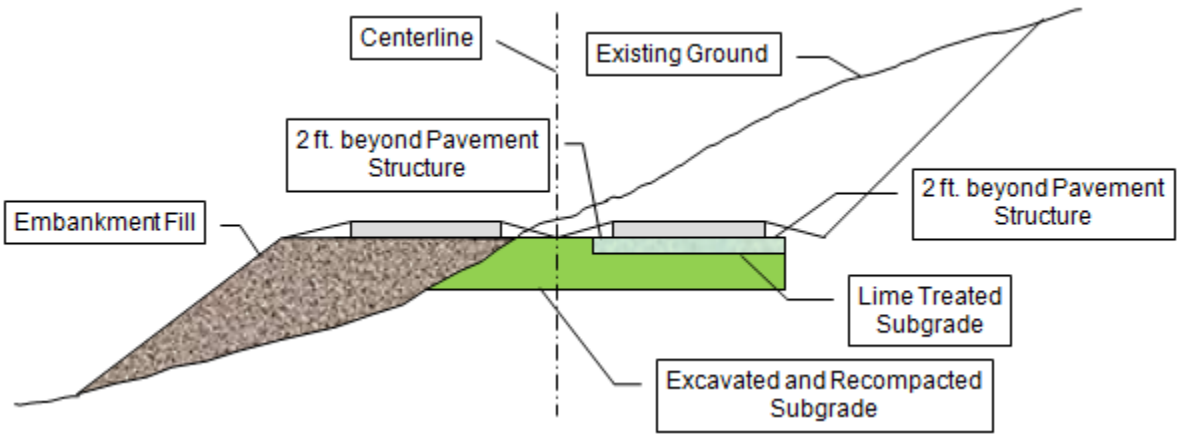


Figure 4.17 Cross Section of Lime Treated Cut Section Subgrade

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 4.18 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.

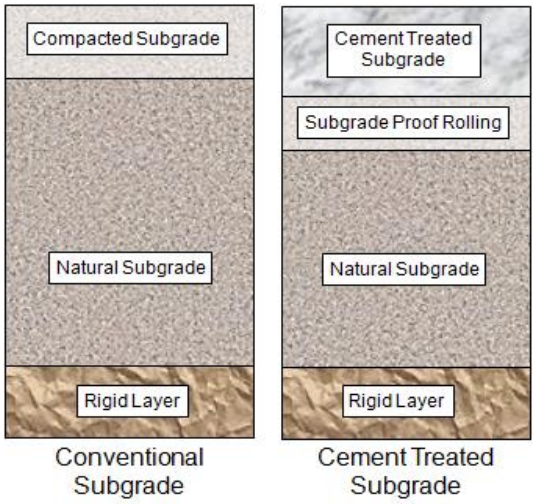


Figure 4.18 Cement Treated Structural Subgrade Layer

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 4.19 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.

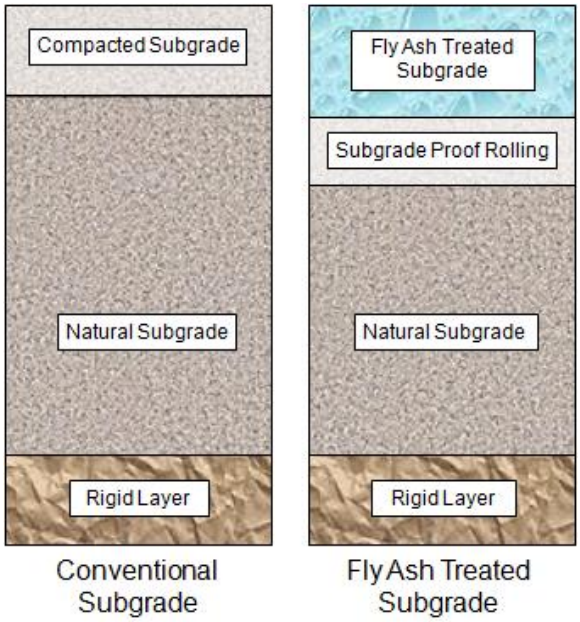


Figure 4.19 Fly Ash Treated Structural Subgrade Layer

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 4.10 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 = $\tau_f = c_u < 2{,}000$ 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
  - $M_r \approx < 4{,}500$ psi (30 MPa)

- High water table

- High sensitivity: dynamic disturbance results in viscous flow.

Table 4.10 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 4.10 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 ≤ c_u ≤ 5,000 psf (90 kPa ≤ c_u ≤ 240 kPa) 3 ≤ CBR ≤ 8 4,500 psi ≤ M_r ≥ 11,600 psi (30 MPa ≤ M_r ≥ 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 ≤ c_u ≤ 5,000 psf (30 kPa ≤ c_u ≤ 240 kPa) 3 ≤ CBR ≤ 8 1,500 psi ≤ M_r ≥ 11,600 psi (10 MPa ≤ M_r ≥ 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 underlaying 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.

$$D_{15B} \leq 5 \times D_{85S} \quad \textbf{Eq. 4-5}$$

$$D_{50B} \leq 25 \times D_{50S} \quad \textbf{Eq. 4-6}$$

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.