SUPPLEMENT
MATERIAL PROPERTIES OF SUBGRADE, SUBBASE, BASE, FLEXIBLE AND RIGID LAYERS

S.1 Introduction

The designer needs to have a basic knowledge of soil properties to include soil consistency, sieve analysis, unit weight, water content, specific gravity, elastic modulus, Poisson's ratio, unconfined compression strength, modulus of rupture, and indirect tensile strength. Resilient modulus and R-value needs to be understood. The *Mechanistic-Empirical (M-E) Pavement Design Guide* (24) will aggressively use these properties in the design of pavements.

The Resilient Modulus (M_r) was selected to replace the soil support value used in previous editions as noted when it first appeared in the *AASHTO Guide for Design of Pavement Structures 1986* (2). The AASHTO guide for the design of pavement structures, which was proposed in 1961 and then revised in 1972 (1), characterized the subgrade in terms of soil support value (SSV). SSV has a scale ranging from 1 to 10, with a value of 3 representing the natural soil at the Road Test. AASHTO Test Method T 274 determined the M_r referenced in the 1986 AASHTO Guide. The compacted layer of the roadbed soil was to be characterized by the M_r using correlations suitable to obtain a M_R value. Procedures for assigning appropriate unbound granular base and subbase layer coefficients based on expected M_r values were also given in the 1986 AASHTO Guide. The *1993 AASHTO Guide for Design of Pavement Structures* (3): Appendix L, lists four different approaches to determine a design resilient modulus value. These are laboratory testing Non-Destructive Testing (NDT) backcalculation, estimating resilient modulus from correlations with other properties, and original design and construction data (4).

S.1.1 Soil Consistency

Soil consistency is defined as the amount of effort required to deform a soil. This level of effort allows the soil to be classified as either soft, firm, or hard. The forces that resist the deformation and rupture of soil are cohesion and adhesion. Cohesion is a water-to-water molecular bond, and adhesion is a water-to-solid bond (17). These bonds depend on water, so consistency directly relates to moisture content, which provides a further classification of soil as dry consistence, moist consistence, and wet consistence.

The Atterberg Limits takes this concept a step further, by labeling the different physical states of soil based on its water content as liquid, plastic, semi-solid, and solid. The boundaries that define these states are known as the liquid limit (LL), plastic limit (PL), shrinkage limit (SL), and dry limit (DL). The liquid limit is the moisture content at which soil begins to behave like a liquid and flow. The plastic limit is the moisture content where soil begins to demonstrate plastic properties, such as rolling a small mass of soil into a long thin thread. The plasticity index (PI) measures the range between LL and PL where soil is in a plastic state. The shrinkage limit is defined as the moisture content at which no further volume change occurs as the moisture content is continually reduced (18). The dry limit occurs when moisture no longer exists within the soil.

The Atterberg limits are typically used to differentiate between clays and silts. The test method for determining LL of soils is AASHTO T 89-02. AASHTO T 90-00 presents the standard test method for determining PL and PI.

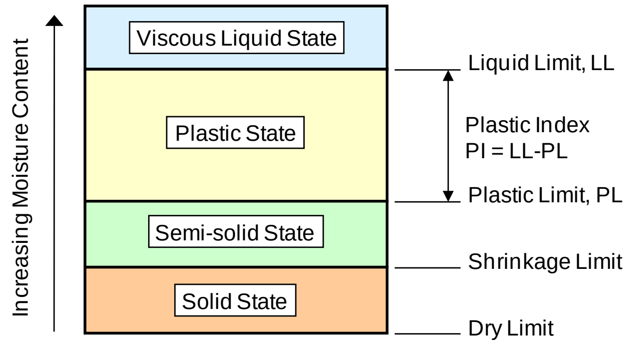


Figure S.1 Atterberg Limits

S.1.2 Sieve Analysis

The sieve analysis is performed to determine the particle size distribution of unbound granular and subgrade materials. In the M-E Design program, the required size distribution are the percentage of material passing the No. 4 sieve (P_4) and No. 200 sieve (P_{200}). D_{60} represents a grain diameter in inches for which 60% of the sample will be finer and passes through that sieve size. In other words, 60% of the sample by weight is smaller than diameter D_{60}. D_{60} = 0.1097 inches.

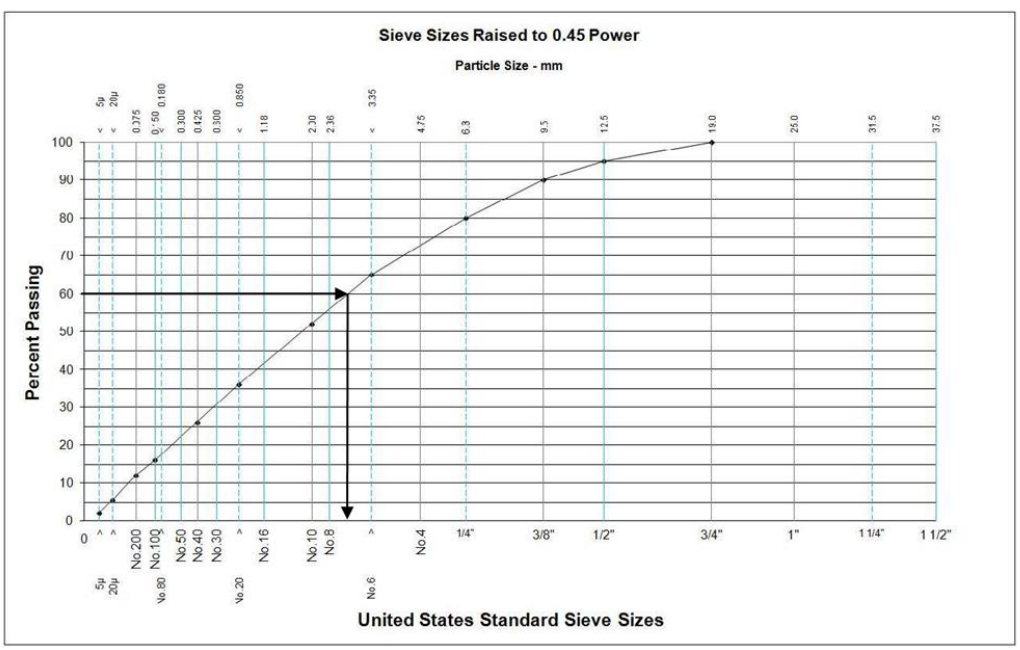


Figure S.2 Gradation Plot

Table S.1 Nominal Dimensions of Common Sieves

US Nominal Sieve Size	Size (mm)	US Nominal Sieve Size	Size (mm)
2"	50.0	No. 8	2.36
1 1/2"	37.5	No. 10	2.00
1 1/4"	31.5	No. 16	1.18
1"	25.0	No. 20	850 µm
3/4"	19.0	No. 30	600 µm
1/2"	12.5	No. 40	425 µm
3/8"	9.5	No. 50	300 µm
1/4"	6.3	No. 80	180 µm
No. 4	4.75	No. 100	150 µm
No. 6	3.35	No. 200	75 µm

S.1.3 Unit Weight, Water Content, and Specific Gravity

Maximum dry density ($\gamma_{dry\ max}$) and optimum gravimetric moisture content (w_{opt}) of the compacted unbound material is measured using AASHTO T 180 for bases or AASHTO T 99 for other layers. Specific gravity (G_s) is a direct measurement using AASHTO T 100 (performed in conjunction with consolidation tests - AASHTO T 180 for unbound bases or AASHTO T 99 for other unbound layers).

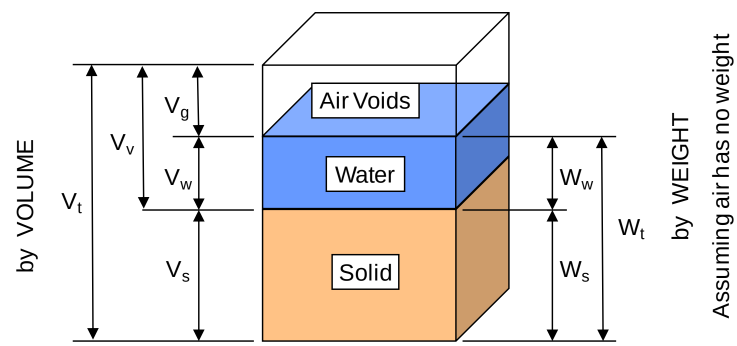


Figure S.3 Soil Sample Constituents

Unit Weight:

$$\gamma = \frac{W_t}{V_t} = \frac{W_w + W_s}{V_g + V_w + V_s} \quad \textbf{Eq. S.1}$$

Dry Density (mass):

$$\gamma_{dry} = \frac{W_s}{V_t} = \frac{W_s}{V_g + V_w + V_s} \qquad \textbf{Eq. S.2}$$

In the consolidation (compaction) test the dry density cannot be measured directly, what are measured are the bulk density and the moisture content for a given effort of compaction.

Bulk Density or oven-dry unit mas:

$$\gamma_{dry} = \frac{W_s + W_w}{V_t} = \frac{W_t}{V_t\,(1+w)} = \frac{\gamma}{1 + w} = \frac{(W_t / V_t)}{(1 + (W_w / W_s))} \qquad \textbf{Eq. S.2}$$

Specific Gravity:

$$Gs = \frac{\gamma_s}{\gamma_w} = \frac{(W_s / V_s)}{\gamma_w} = \frac{\gamma_s}{62.4} \qquad \textbf{Eq. S.4}$$

Where:

γ = Unit weight (density), pcf
γ_{dry} = Dry density, pcf
γ_{bulk} = Bulk density, pcf
$\gamma_{dry\ max}$ = Maximum dry unit weight, pcf
G_s = Specific gravity (oven dry)
W_t = total weight
W_w = weight of water
W_s = weight of solids
V_t = total volume
V_v = volume of voids
V_g = volume of air (gas)
V_w = volume of water
V_s = volume of solids
w = water content
w_{opt} = optimum water content
γ_s = density of solid constituents
γ_w = 62.4 pcf at 4 °C

The maximum dry unit weight and optimum water content are obtained by graphing as shown in **Figure S.4 Plot of Maximum Dry Unit Weight and Optimum Water Content**.

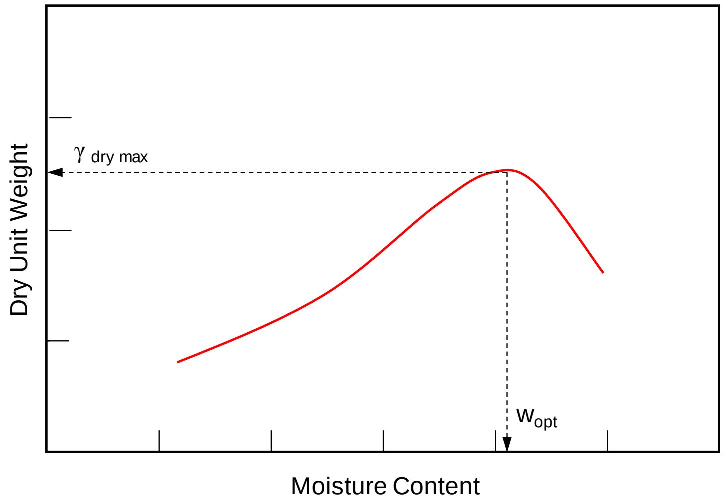


Figure S.4 Plot of Maximum Dry Unit Weight and Optimum Water Content

S.1.4 Pavement Materials Chemistry

Periodic Table

The periodic table is a tabular method of displaying the 118 chemical elements, refer to **Figure S.5 Periodic Table**. Elements are listed from left to right as the atomic number increases. The atomic number identifies the number of protons in the nucleus of each element. Elements are grouped in columns, because they tend to show patterns in their atomic radius, ionization energy, and electronegativity. As you move down a group the atomic radii increases, because the additional electrons per element fill the energy levels and move farther from the nucleus. The increasing distance decreases the ionization energy, the energy required to remove an electron from the atom, as well as decreases the atom's electronegativity, which is the force exerted on the electrons by the nucleus. Elements in the same period or row show trends in atomic radius, ionization energy, electron affinity, and electronegativity. Within a period moving to the right, the atomic radii usually decreases, because each successive element adds a proton and electron, which creates a greater force drawing the electron closer to the nucleus. This decrease in atomic radius also causes the ionization energy and electronegativity to increase the more tightly bound an element becomes.

pH Scale

Water (H_2O) is a substance that can share hydrogen ions. The cohesive force that holds water together can also cause the exchange of hydrogen ions between molecules. The water molecule acts like a magnet with a positive and negative side, this charge can prove to be greater than the hydrogen bond between the oxygen and hydrogen atom causing the hydrogen to join the adjacent molecule (19). This process can be seen molecularly **Figure S.6 Dissociation of Water** and is expressed chemically in **Equation S.5**.

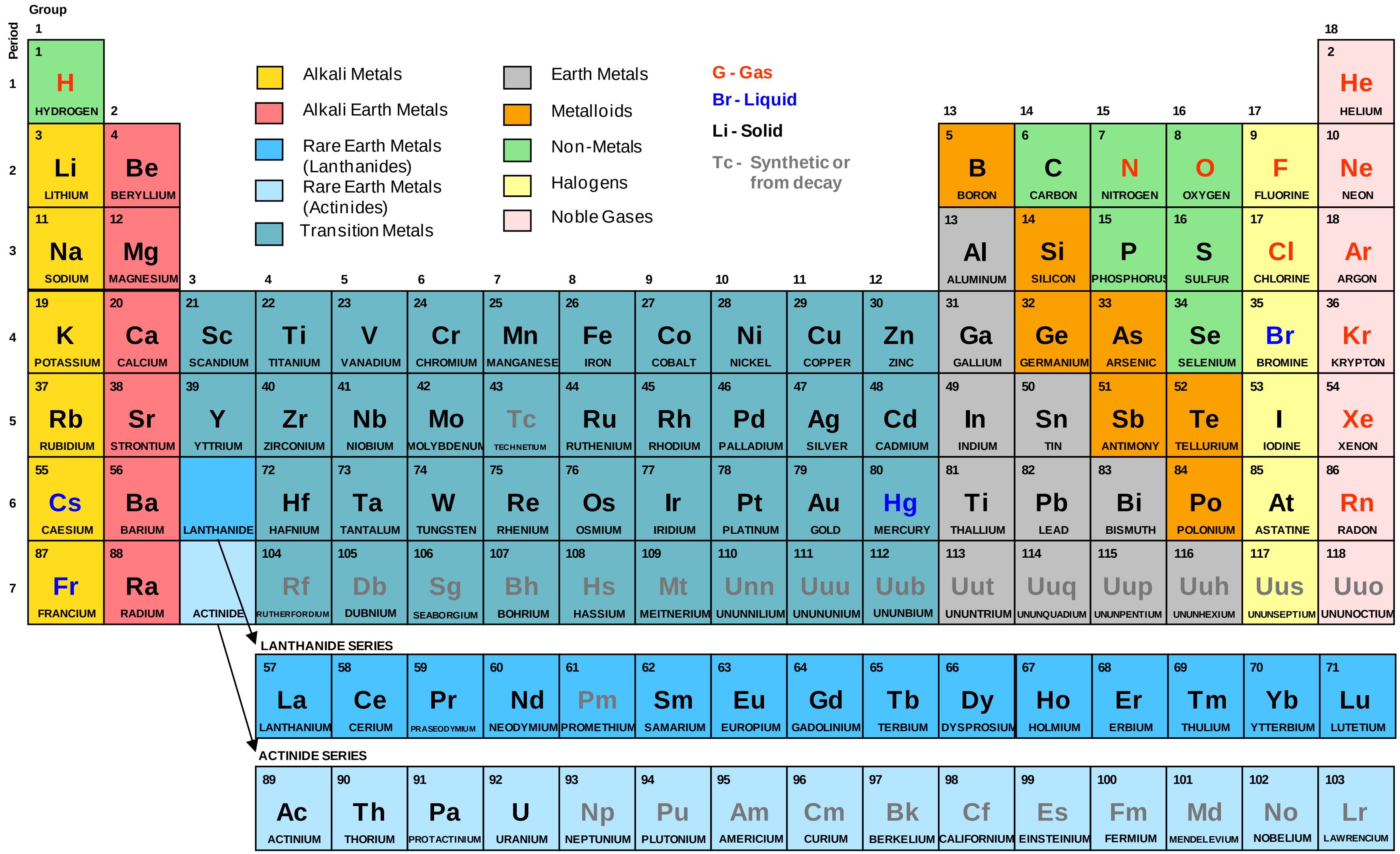


Figure S.5 Periodic Table

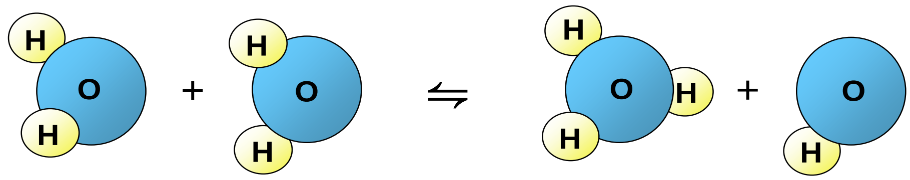


Figure S.6 Dissociation of Water

$$\mathbf{2H_2O = H_2O + (aq) + OH - (aq)} \qquad \textbf{Eq. S.5}$$

The pH of a solution is the negative logarithmic expression of the number of H^+ ions in a solution. When this is applied to water with equal amounts of H^+ and OH^- ions the concentration of H^+ will be 0.00000001, the pH is then expressed as $-\log 10^{-7} = 7$. From the neutral water solution of 7 the pH scale ranges from 0 to 14, zero is the most acidic value and 14 is the most basic or alkaline, refer to **Figure S.7 pH Scale**.

An acid can be defined as a proton donor, a chemical that increases the concentration of hydronium ions $[H_3O^+]$ or $[H^+]$ in an aqueous solution. Conversely, we can define a base as a proton acceptor, a chemical that reduces the concentration of hydronium ions and increases the concentration of hydroxide ions $[OH^-]$ (18).

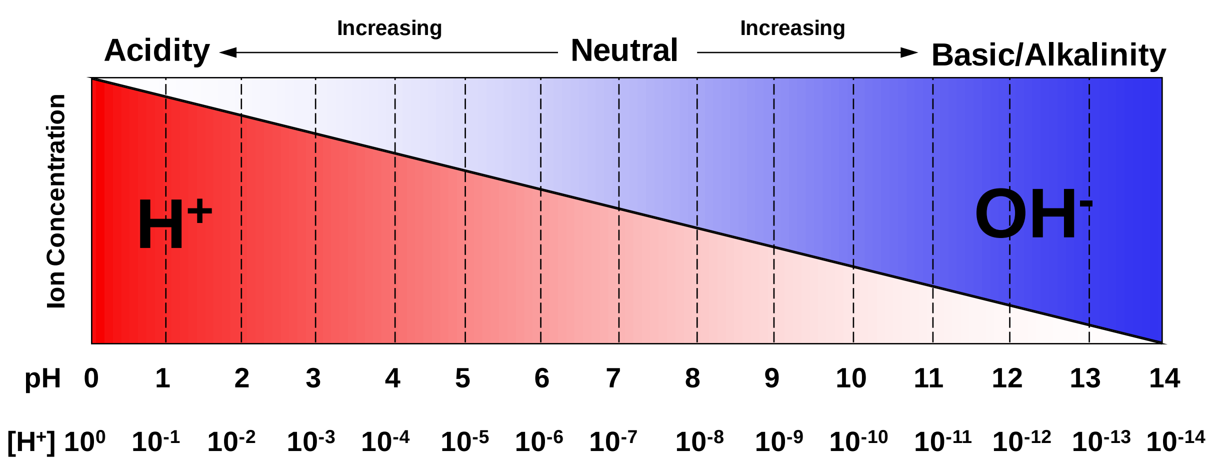


Figure S.7 pH Scale

S.1.5 Elastic Modulus

Elastic Modulus (E):

$$\mathbf{E = \frac{\sigma}{\varepsilon}} \qquad \textbf{Eq. S.6}$$

Where:

$$\textbf{Stress} = \sigma = \textbf{Load/Area} = \textbf{P/A} \qquad \textbf{Eq. S.7}$$

$$\textbf{Strain} = \varepsilon = \frac{\textbf{Change in length}}{\textbf{Initial length}} = \frac{\Delta L}{L_o} \qquad \textbf{Eq. S.8}$$

A material is elastic if it is able to return to its original shape or size immediately after being stretched or squeezed. Almost all materials are elastic to some degree as long as the applied load does not cause it to deform permanently. The modulus of elasticity for a material is basically the slope of its stress-strain plot within the elastic range.

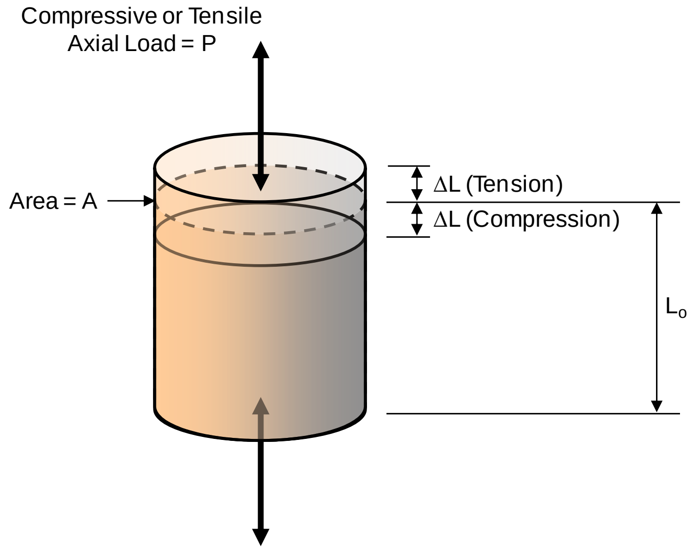


Figure S.8 Elastic Modulus

Concrete Modulus of Elasticity

The static Modulus of Elasticity (E_c) of concrete in compression is determined by ASTM C 469. The chord modulus is the slope of the chord drawn between any two specified points on the stress-strain curve below the elastic limit of the material.

$$E_c = \frac{(\sigma_2 - \sigma_1)}{(\varepsilon_2 - 0.000050)} \qquad \textbf{Eq. S.9}$$

Where:

E_c = Chord modulus of elasticity, psi

σ_1 = Stress corresponding to 40% of ultimate load

σ_2 = Stress corresponding to a longitudinal strain; ε_1 = 50 millionths, psi

ε_2 = Longitudinal strain produced by stress σ_2

Asphalt Dynamic Modulus |E*|

The complex Dynamic Modulus (|E*|) of asphalt is a time-temperature dependent function. The |E*| properties are known to be a function of temperature, rate of loading, age, and mixture characteristics such as binder stiffness, aggregate gradation, binder content, and air voids. To account for temperature and rate of loading, the analysis levels will be determined from a master curve constructed at a reference temperature of 20°C (70°F) (5). The description below is for developing the master curve and shift factors of the original condition without introducing aged binder viscosity and additional calculated shift factors using appropriate viscosity.

|E*| is the absolute value of the complex modulus calculated by dividing by the maximum (peak to peak) stress by the recoverable (peak to peak) axial strain for a material subjected to a sinusoidal loading.

A sinusoidal (Haversine) axial compressive stress is applied to a specimen of asphalt concrete at a given temperature and loading frequency. The applied stress and the resulting recoverable axial strain response of the specimen is measured and used to calculate the |E*| and phase angle. See **Equation S.10** for |E*| general equation and **Equation S.11** for phase angle equation. Dynamic modulus values are measured over a range of temperatures and load frequencies at each temperature. Refer to **Table S.2 Recommended Testing Temperatures and Loading Frequencies**. Each test specimen is individually tested for each of the combinations. The table shows a reduced temperature and loading frequency as recommended. See **Figure S.9 Dynamic Modulus Stress-Strain Cycles** for time lag response. See **Figure S.10 |E*| vs. Log Loading Time Plot at Each Temperature**. To compare test results of various mixes, it is important to normalize one of these variables. 20°C (70°F) is the variable that is normalized. Test values for each test condition at different temperatures are plotted and shifted relative to the time of loading. See **Figure S.11 Shifting of Various Mixture Plots**. These shifted plots of various mixture curves can be aligned to form a single master curve (26). See **Figure S.12 Dynamic Modulus |E*| Master Curve**. The |E*| in determined by AASHTO PP 61-09 and PP 62-09 test methods (27-28).

Table S.2 Recommended Testing Temperatures and Loading Frequencies

PG 58-XX and Softer		PG 64-XX and PG 70-XX		PG 76-XX and Stiffer	
Temp. (°C)	Loading Freq. (Hz)	Temp. (°C)	Loading Freq. (Hz)	Temp. (°C)	Loading Freq. (Hz)
4	10, 1, 0.1	4	10, 1, 0.1	4	10, 1, 0.1
20	10, 1, 0.1	20	10, 1, 0.1	20	10, 1, 0.1
35	10, 1,0.1,0.01	40	10, 1,0.1,0.01	45	10, 1,0.1,0.01

$$|E^*| = \sigma_0 / \varepsilon_0 \qquad \textbf{Eq. S.10}$$

Where:

$|E^*|$ = Dynamic modulus

σ_0 = Average peak-to-peak stress amplitude, psi

ε_0 = Average peak-to-peak strain amplitude, coincides with time lag (phase angle)

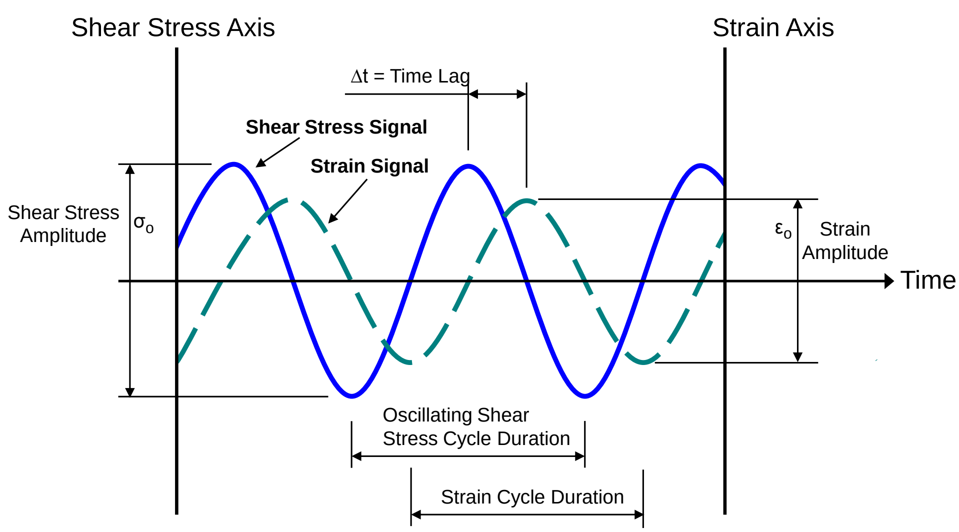


Figure S.9 Dynamic Modulus Stress-Strain Cycles

The phase angle θ is calculated for each test condition and is:

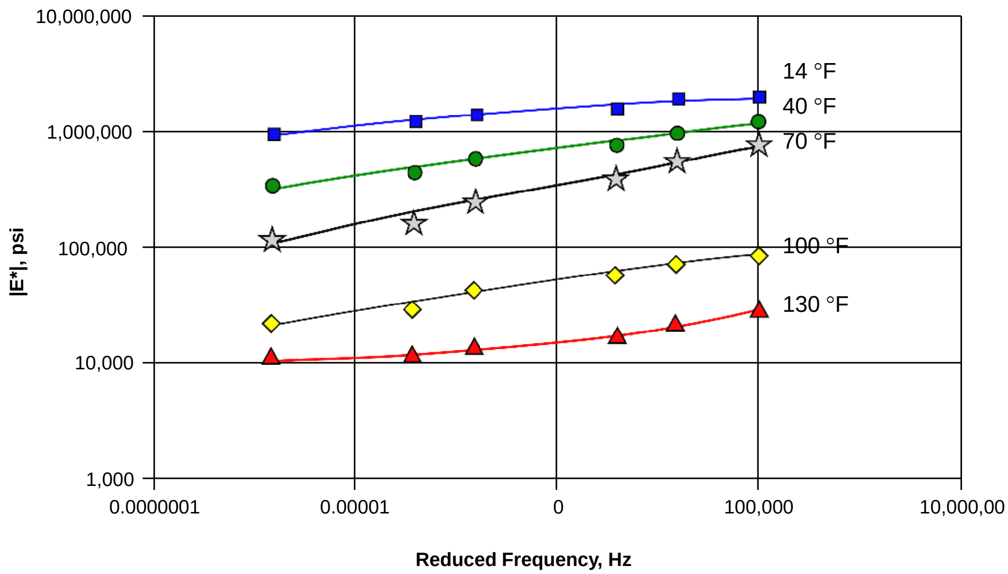
$$\theta = 2\pi f \Delta t \qquad \textbf{Eq. S.11}$$

Where:

θ = phase angle, radian

f = frequency, Hz

Δt = time lag between stress and strain, seconds



The |E*| master curve can be represented by a sigmoidal function as shown (27):

$$\log|E^*| = \delta + \frac{(\mathrm{Max} - \delta)}{1 + e^{\beta + \gamma\left\{\log f + \frac{\Delta E_{\alpha}}{19.14714}\left[\left(\frac{1}{T}\right) - \left(\frac{1}{T_r}\right)\right]\right\}}}$$ **Eq. S.12**

Where:

|E*| = Dynamic modulus, psi
Δ, β and γ = fitting parameters
Max = limiting maximum modulus, psi
f = loading frequency at the test temperature, Hz
E_σ = energy (treated as a fitting parameter)
T = test temperature, °K
T_f = reference temperature, °K

Fitting parameters δ and α depend on aggregate gradation, binder content, and air void content. Fitting parameters β and γ depend on the characteristics of the asphalt binder and the magnitude of δ and α. The sigmoidal function describes the time dependency of the modulus at the reference temperature.

The maximum limiting modulus is estimated from HMA volumetric properties and limiting binder modulus.

$$|E^*|_{max} = P_c\left[4{,}200{,}000\left(1-\frac{VMA}{100}\right)+435{,}000\left(\frac{VFA \times VMA}{10{,}000}\right)+\frac{1-P_c}{\dfrac{1-\dfrac{VMA}{100}}{4{,}200{,}000}+\dfrac{VMA}{435{,}000(VFA)}}\right]$$ **Eq. 13**

Where:

$$P_c = \frac{\left[20+\dfrac{435{,}000(VFA)}{VMA}\right]^{0.58}}{650+\left[\dfrac{435{,}000(VFA)}{VMA}\right]^{0.58}}$$ **Eq. S.14**

$|E^*|_{max}$ = limiting maximum HMA dynamic modulus, psi
VMA = voids in the mineral aggregate, percent
VFA = voids filled with asphalt, percent

The shift factors describe the temperature dependency of the modulus.

Shift factors to align the various mixture curves to the master curve are shown in the general form as (27):

$$\mathbf{Log\ [\alpha_{(T)}] = \frac{\Delta E_\alpha}{19.14714}\ [\ (1\ /\ T)\ \backslash\ (1\ /\ T_r)\ ]}$$ **Eq. S.15**

Where:

$\alpha_{(T)}$ = shift factor at temperature (T)
ΔE_α = activation energy (treating as a fitting parameter)
T = test temperature, °K
T_r = reference temperate, °K

A shift factor plot as a function of temperature for the mixtures is shown in **Figure S.13 Shift Factor Plot**.

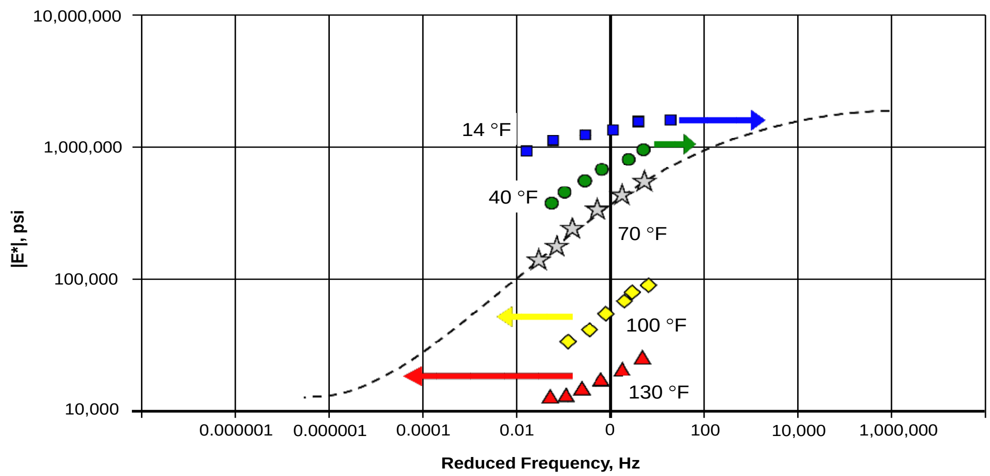


Figure S.10 Shifting of Various Mixture Plots

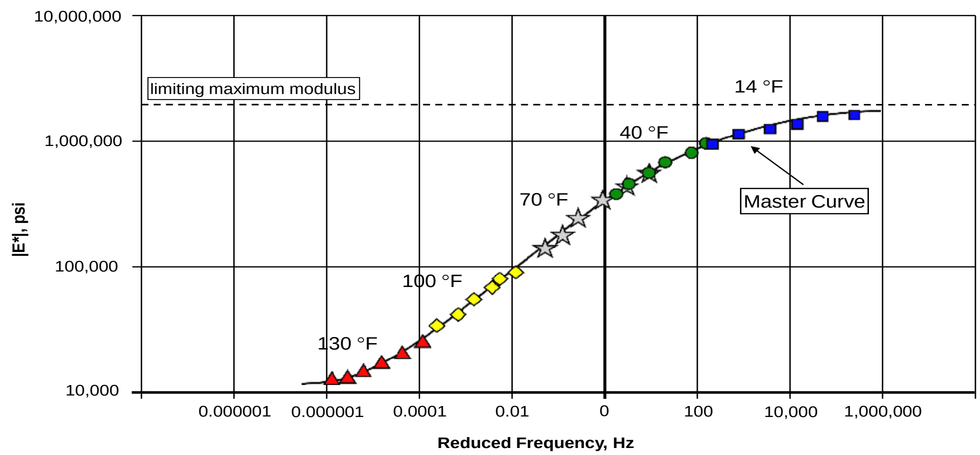


Figure S.11 Dynamic Modulus |E*| Master Curve

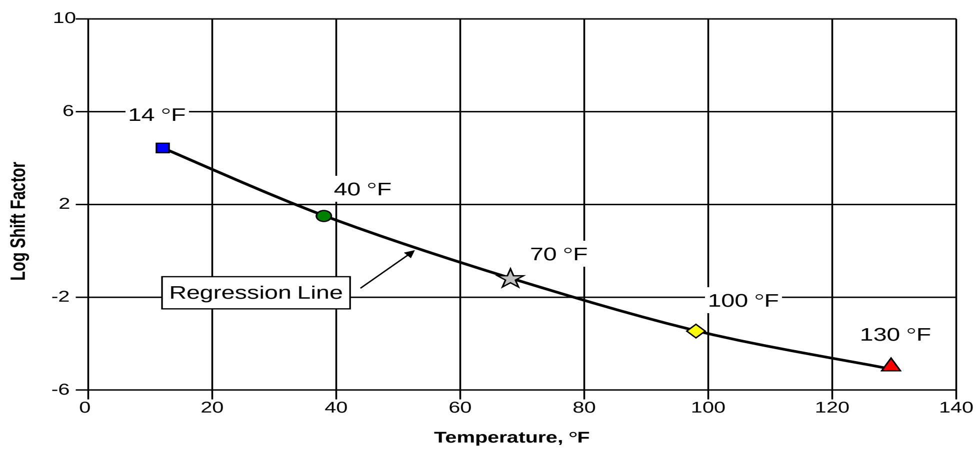


Figure S.12 Shift Factor Plot

S.1.6 Binder Complex Shear Modulus

The complex shear modulus, G* is the ratio of peak shear stress to peak shear strain in dynamic (oscillatory) shear loading between a oscillating plate a fixed parallel plate. The test uses a sinusoidal waveform that operates at one cycle and is set at 10 radians/second or 1.59 Hz. The oscillating loading motion is a back and forth twisting motion with increasing and decreasing loading. Stress or strain imposed limits control the loading. The one cycle loading is a representative loading due to 55 mph traffic. If the material is elastic, then the phase lag is zero. G' represents this condition and is said to be the storage modulus. If the material is wholly viscous, then the phase lag is 90° out of phase. G" represents the viscous modulus. G* is the vector sum of G' and G". Various artificially aged specimens and/or in a series of temperature increments may be tested. The DSR test method is applicable to a temperature range of 40°F and above.

$$G^* = \tau_{max} / \gamma_{max} \qquad \textbf{Eq. S.16}$$

$$\tau_{max} = \frac{2T_{max}}{\pi r^3} \qquad \textbf{Eq. S.17}$$

$$\gamma_{max} = \theta_{max}\,(r) / h \qquad \textbf{Eq. S.18}$$

Where:

G^* = binder complex shear modulus
τ_{max} = maximum shear stress
γ_{max} = maximum shear strain
T_{max} = maximum applied torque
r = radius of specimen
θ_{max} = maximum rotation angle, radians
h = height of specimen

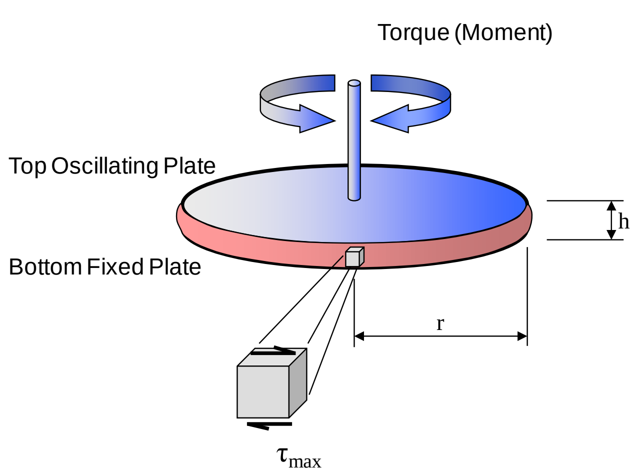


Figure S.13 Binder Complex Shear Modulus Specimen Loading

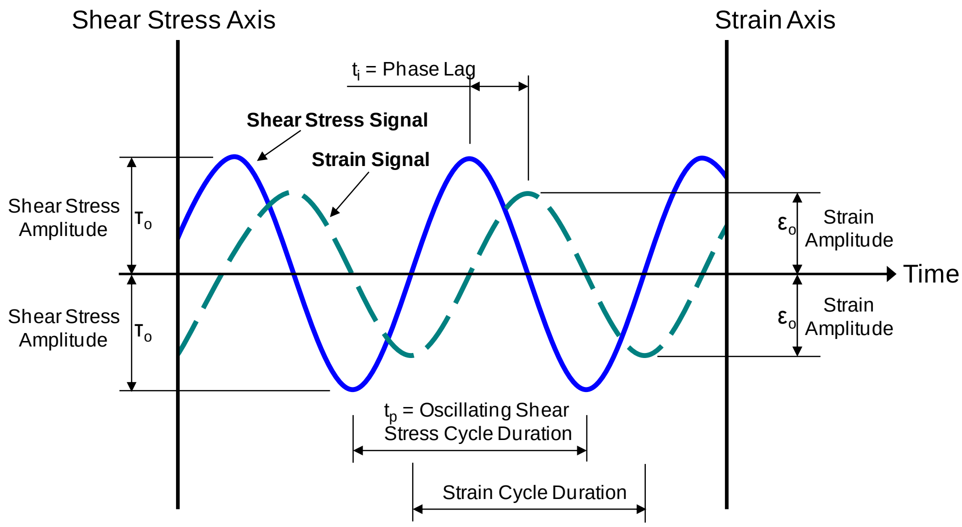


Figure S.14 Binder Complex Shear Modulus Shear-Strain Cycles

A relationship between binder viscosity and binder complex shear modulus (with binder phase angle) at each temperature increment of 40, 55, 70 (reference temperature), 85, 100, 115 and 130°F are obtained by:

$$\eta = \frac{G^*}{10}(1 / \sin \delta) \times 4.8628 \qquad \textbf{Eq. S.19}$$

Where:

η = viscosity
G^* = binder complex shear modulus
δ = binder phase angle

The regression parameters are found by using Equation S.20 by linear regression after log-log transformation of the viscosity data and log transformation of the temperature data:

$$\text{Log}(\log \eta) = A = VTS \times \log T_R \qquad \textbf{Eq. S.20}$$

Where:

η = binder viscosity
A, VTS = regression parameters
T_R = temperature, degrees Rankin

S.1.7 Poisson's Ratio

The ratio of the lateral strain to the axial strain is known as Poisson's ratio, μ:

$$\mu = \varepsilon_{lateral} / \varepsilon_{axial} \qquad \textbf{Eq. S.21}$$

Where:

μ = Poisson's ratio

$\varepsilon_{lateral}$ = strain width or diameter

= change in diameter/origin diameter

$$= \Delta D / D_0 \qquad \textbf{Eq. S.22}$$

ε_{axial} = strain in length

= change in length/original length

$$= \Delta L / L_0 \qquad \textbf{Eq. S.23}$$

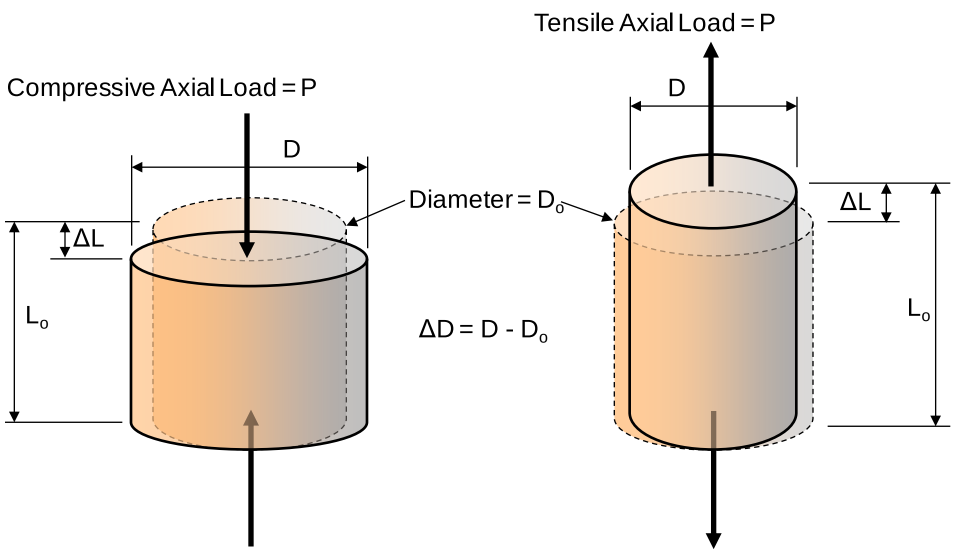


Figure S.15 Poisson's Ratio

S.1.8 Coefficient of Lateral Pressure

The coefficient of lateral pressure (k_0) is the term used to express the ratio of the lateral earth pressue to the vertical earth pressure:

Cohesionless Materials:

$$\mathbf{k_0 = \mu / (1- \mu)} \qquad \textbf{Eq. S.24}$$

Cohesive Materials:

$$\mathbf{k_0 = 1 - \sin \theta} \qquad \textbf{Eq. S.25}$$

Where:

k_0 = coefficient of lateral pressure

μ = Poisson's ratio

θ = effective angle of internal friction

S.1.9 Unconfined Compressive Strength

Unconfined compressive strength (f'_c) is shown in Equation **Eq. S.26**. The compressive strength of soil cement is determined by ASTM D 1633. The compressive strength for lean concrete and cement treated aggregate is determined by AASHTO T 22, lime stabilized soils are determined by ASTM D 5102, and lime-cement-fly ash is determined by ASTM C 593.

$$\mathbf{f'_c = P / A} \qquad \textbf{Eq. S.26}$$

Where:

f'_c = unconfined compressive strength, psi
P = maximum load
A = cross sectional area

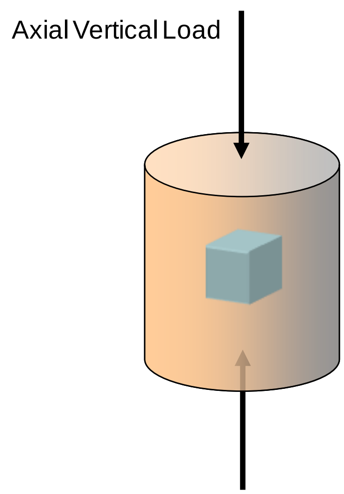


Figure S.16 Unconfined Compressive Strength

S.1.10 Modulus of Rupture

The Modulus of Rupture (M_r) is maximum bending tensile stress at the surface of a rectangular beam at the instant of failure using a simply supported beam loaded at the third points. The M_r is a test conducted solely on portland cement concrete and similar chemically stabilized materials. The rupture point of a concrete beam is at the bottom. The classical formula is shown in Equation **Eq. S.27**. The M_r for lean concrete, cement treated aggregate, and lime-cement-fly ash are determined by AASHTO T 97. Soil cement is determined by ASTM D 1635.

$$\boldsymbol{\sigma_{b,max} = (M_{max}c) / I_c} \qquad \textbf{Eq. S.27}$$

Where:

M_{max} = maximum moment
c = distance from neutral axis to the extreme fiber
I_c = centroidal area moment of inertia

If the fracture occurs within the middle third of the span length the M_r is calculated by:

$$S'_c = (PL) / (bd^2) \qquad \textbf{Eq. S.28}$$

If the fracture occurs outside the middle third of the span length by not more than 5% of the span length the M_r is calculated by:

$$S'_c = (3Pa) / (bd^2) \qquad \textbf{Eq. S.29}$$

Where:

S'_c = modulus of rupture, psi
P = maximum applied load
L = span length
b = average width of specimen
d = average depth pf specimen
a = average distance between line of fracture and the nearest support on the tension surface of the beam

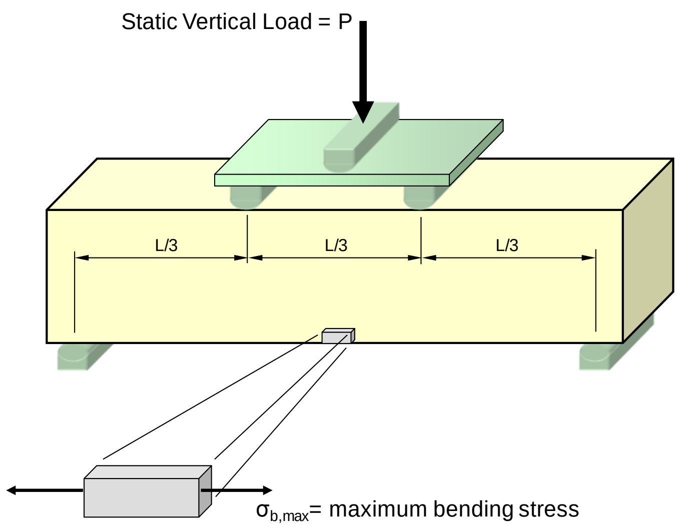


Figure S.17 Three-Point Beam Loading for Flexural Strength

S.1.11 Tensile Creep and Strength for Hot Mix Asphalt

The tensile creep is determined by applying a static load along the diametral axis of a specimen. The horizontal and vertical deformations measured near the center of the specimen are used to calculate tensile creep compliance as a function of time. The Creep Compliance, D(t) is a time-dependent strain divided by an applied stress. The Tensile Strength, S_t is determined immediately after the tensile creep (or separately) by applying a constant rate of vertical deformation (loading movement) to failure. AASHTO T 322 - Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device, using 6 inch diameter by 2 inch height molds, determines Creep Compliance and Tensile Strength. CDOT uses CP-L 5109 - *Resistance* *of Compacted Bituminous Mixture to Moisture Induced Damage* to determine the tensile strength using 4 inch diameter by 2.5 inch height molds for normal aggregate mixtures.

Creep Compliance

$$D_{(t)} = \varepsilon_t / \sigma \qquad \text{Eq. S.30}$$

Where:

$D_{(t)}$ = creep compliance at time, t

ε_t = time-dependent strain

σ = applied stress

Tensile Strength

$$S_t = 2P / (\pi t D) \qquad \text{Eq. S.31}$$

Where:

S_t = tensile strength, psi
P = maximum load
T = specimen height
D = specimen diameter

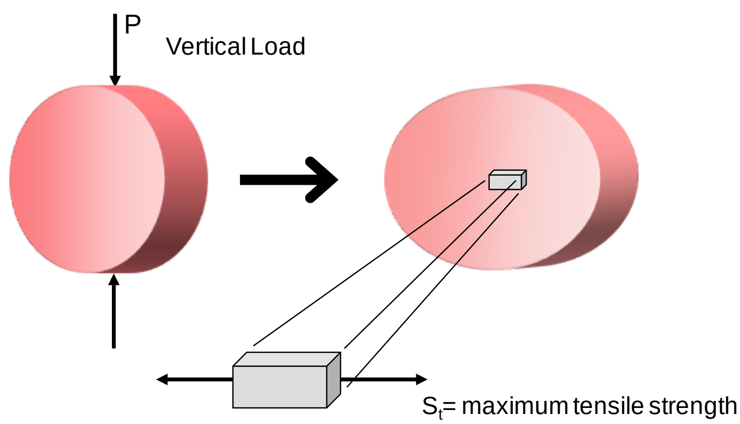


Figure S.18 Indirect Tensile Strength

S.2 Resilient Modulus of Conventional Unbound Aggregate Base, Subbase, Subgrade, and Rigid Layer

The subgrade resilient modulus is used for the support of pavement structure in flexible pavements. The graphical representation (see **Figure S.21 Distribution of Wheel Load to subgrade Soil (M_r)**) is the traditional way to explain the interaction of subgrade reaction to a moving wheel load. As the wheel load moves toward an area of concern, the subgrade reacts with a larger reaction.

When the wheel loading moves away the subgrade reaction i is less. That variable reaction is the engineering property Resilient Modulus. Critical locations in the layers have been defined for the Mechanistic-Empirical Design. Refer to **Figure S.22 Critical Stress/Strain Locations for Bases, Subbases, Subgrade, and Rigid Layer**. CDOT has historically used the empirical design methodology using structural coefficients of base (a_2) and subbase (a_3) layers. The rigid layer was only accounted for when it was close to the pavement structure.

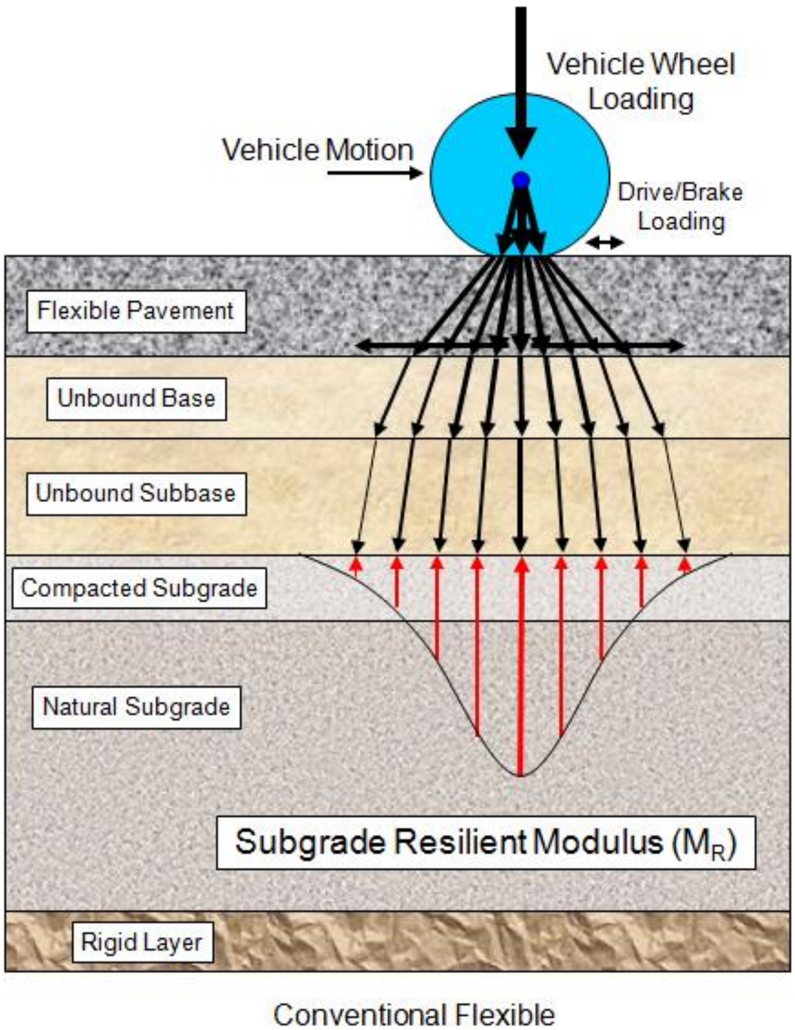


Figure S.19 Distribution of Wheel Load of Subgrade Soil (M_r)

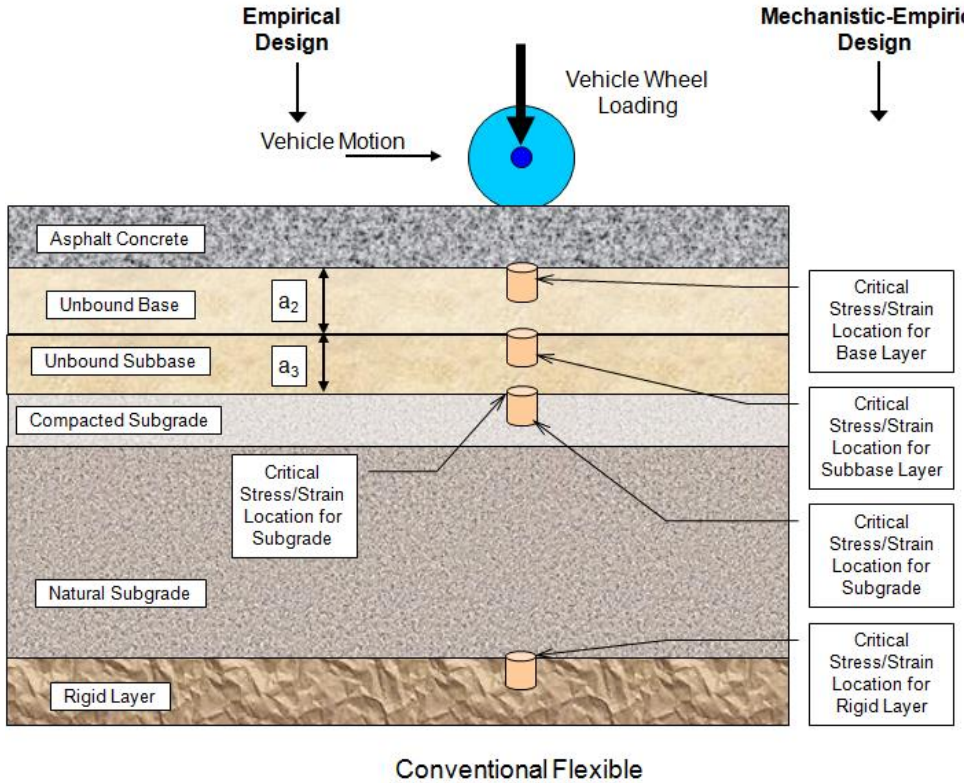


Figure S.20 Critical Stress/Strain Locations for Bases, Subbases, Subgrade, and Rigid Layer

S.2.1 Laboratory M_r Testing

The critical location for the subgrade is at the interface of the subbase and subgrade. The material subgrade element has the greatest loads at this location when the wheel loadings are directly above. Refer to **Figure S.31 Critical Stress Locations for Stabilized Subgrade**.

While the modulus of elasticity is stress divided by strain for a slowly applied load, resilient modulus is stress divided by strain for rapidly applied loads, such as those experienced by pavements.

Resilient modulus is defined as the ratio of the amplitude of the repeated cyclical (resultant) axial stress to the amplitude of resultant (recoverable) axial strain.

$$M_r = \sigma_d / \varepsilon_r \qquad \textbf{Eq. S.32}$$

Where:

M_r = resilient modulus

σ_d = repeated wheel load stress (deviator stress) = applied load/cross sectional area

ε_r = recoverable strain = $\Delta L/L$ = recoverable deformation / gauge length

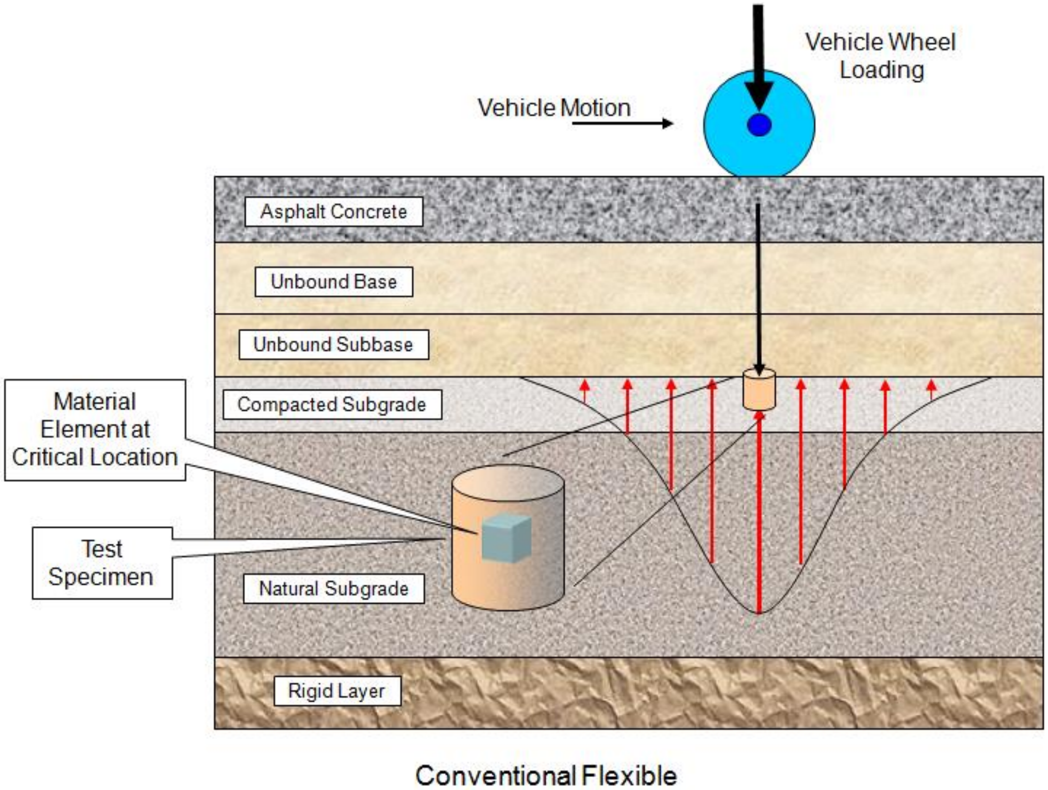


Figure S.21 Subgrade Material Element at Critical Location

The test is similar to the standard triaxial compression test, except the vertical stress is cycled at several levels to model wheel load intensity and duration typically encountered in pavements under a moving load. The confining pressure is also varied and sequenced through in conjunction with the varied axial loading to specified axial stresses. The purpose of this test procedure is to determine the elastic modulus value (stress-sensitive modulus) and by recognizing certain nonlinear characteristics for subgrade soils, untreated base and subbases, and rigid foundation materials. The stress levels used are based on type of material within the pavement structure. The test specimen should be prepared to approximate the in-situ density and moisture condition at or after construction (5). The test is to be performed in accordance with the latest version of AASHTO T 307. **Figure S.24 Resilient Modulus Test Specimen Stress State** and **Figure S.25 Resilient Modulus Test Specimen Loading** are graphical representations of applied stresses and concept of cyclical deformation applied deviator loading.

Traditionally, the stress parameter used for sandy and gravelly materials, such as base courses, is the bulk stress.

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \qquad \textbf{Eq. S.33}$$

For cohesive subgrade materials, the deviatoric stress is used.

$$\sigma_d = \sigma_1 - \sigma_3 \qquad \textbf{Eq. S.34}$$

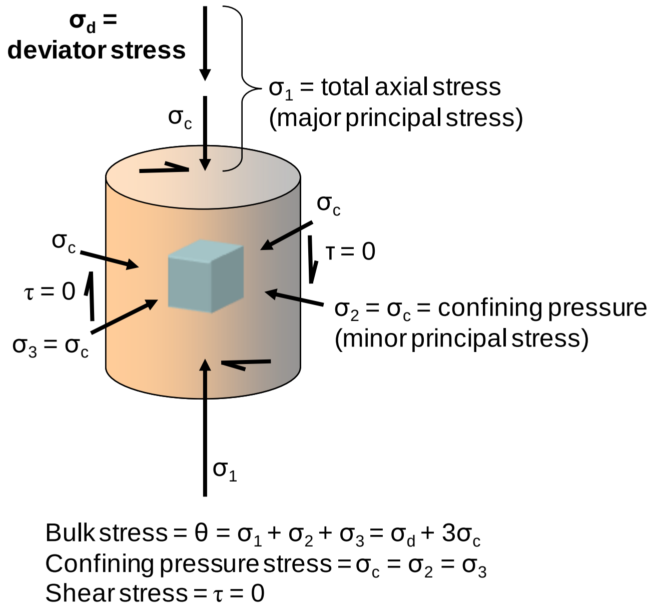


Figure S.22 Resilient Modulus Test Specimen Stress State

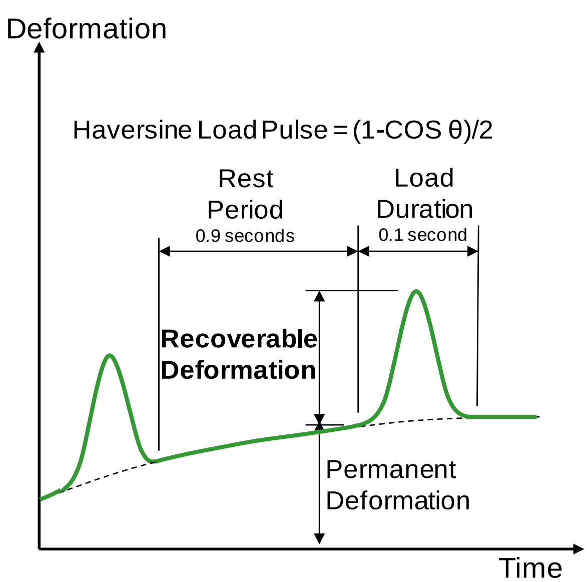


Figure S.23 Resilient Modulus Test Specimen Loading

In recent years, the octahedral shear stress, which is a scalar invariant (it is essentially the root-mean-square deviatoric stress), has been used for cohesive materials instead of the deviatoric stress.

$$\boldsymbol{\tau_{oct} = \tfrac{1}{3} * \sqrt{[(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_2)^2]}} \qquad \textbf{Eq. S.35}$$

The major material characteristics associated with unbound materials are related to the fact that moduli of these materials may be highly influenced by the stress state (non-linear) and in-situ moisture content. As a general rule, coarse-grained materials have higher moduli as the state of confining stress is increased. In contrast, clayey materials tend to have a reduction in modulus as the deviatoric or octahedral stress component is increased. Thus, while both categories of unbound materials are stress dependent (non-linear), each behaves in an opposite direction as stress states are increased (5).

S.2.2 Field M_r Testing

An alternate procedure to determine the M_r value is to obtain a field value. Determination of an in-situ value is to backcalculate the M_r from deflection basins measured on the pavement's surface. The most widely used deflection testing devices are impulse loading devices. CDOT uses the Falling Weight Deflectometer (FWD) as a Nondestructive Test (NDT) method to obtain deflection measurements. The FWD device measures the pavement surface deflection and deflection basin of the loaded pavement, making it possible to obtain the pavement's response to load and the resulting curvature under load. A backcalculation software program analyzes the pavements response from the FWD data. Unfortunately, layered elastic moduli backcalculated from deflection basins and laboratory measured resilient modulus are <u>not</u> equal for a variety of reasons. The more important reason is that the uniform confining pressures and repeated vertical stresses used in the laboratory do not really simulate the actual confinement and stress state variation that occurs in a pavement layer under the FWD test load or wheel loading (9). Additional information on NDT is provided in **APPENDIX C**.

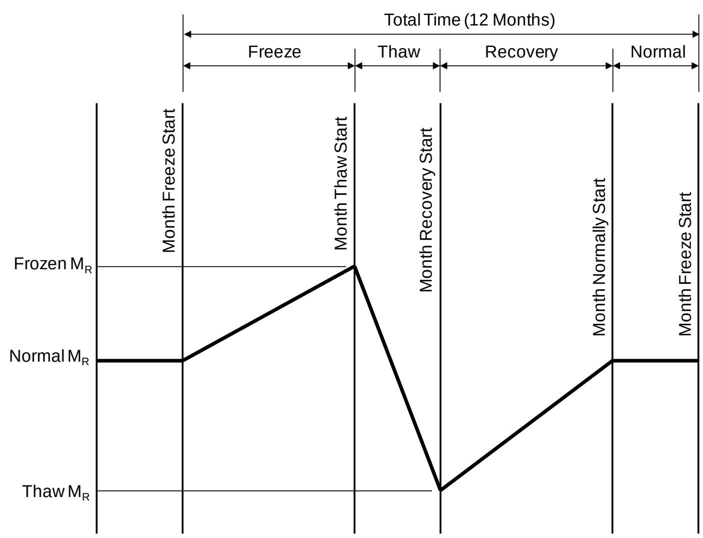


Figure S.24 Resilient Modulus Seasonal Variation

S.3 Resistance Value (R-value)

The Resistance Value (R-value) test is a material stiffness test. The test procedure expresses a material's resistance to deformation as a function of the ratio of transmitted lateral pressure to applied vertical pressure. The R-value is calculated from the ratio of the applied vertical pressure to the developed lateral pressure and is essentially a measure of the material's resistance to plastic flow. Another way the R-value may be expressed is it is a parameter representing the resistance to the horizontal deformation of a soil under compression at a given density and moisture content. The R-value test, while being time and cost effective, does not have a sound theoretical base and it does not reflect the dynamic behavior and properties of soils. The R-value test is static in nature and irrespective of the dynamic load repetition under actual traffic.

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 to be used is determined in accordance with Colorado Procedure - Laboratory 3102, Determination of Resistance Value at Equilibrium, a modification of AASHTO T 190, *Resistance Value and Expansion Pressure of Compacted Soils*.

The inability of the stabilometer R-value to realistically reflect the engineering properties of granular soils with less than 30 percent fines has contributed to its poor functional relationship to M_r in that range (7).

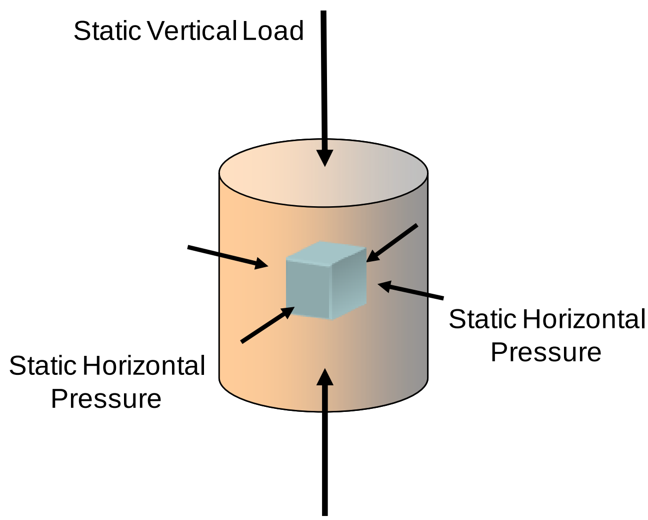


Figure S.25 Resistance R-value Test Specimen Loading State

A number of correlation equations have been developed. The Asphalt Institute (8) has related M_r to R-value repeated in the 1986 AASHTO Guide and expressed as follows (2)(5)(6):

$$\mathbf{M_r = A + B \times (R\text{-}value)} \qquad \textbf{Eq. S.36}$$

Where:

M_r = units of psi
A = a value between 772 and 1,155
B = a value between 396 and 555

CDOT uses the correlation combining two equations:

S_1 = [(R-5) / 11.29] + 3 **Eq. S.37**

M_r = 10 [(S_1 + 18.72) / 6.24] **Eq. S.38**

Where:

M_r = resilient modulus, psi.
S_1 = soil support value
R = R-value obtained from the Hveem stabilometer

Figure S.28 Correlation Plot Between Resilient Modulus and R-value plots the correlations of roadbed soils. In the **Figure S.29 Correlation Plot Betweeen Resilient Modulus and R-value**, the CDOH/CDOT current design curve and the referenced 1986 AASHTO equations were based on the AASHTO Test Method T 274 to determine the M_r value. The plot is to show the relative relationship of each equation to each other.

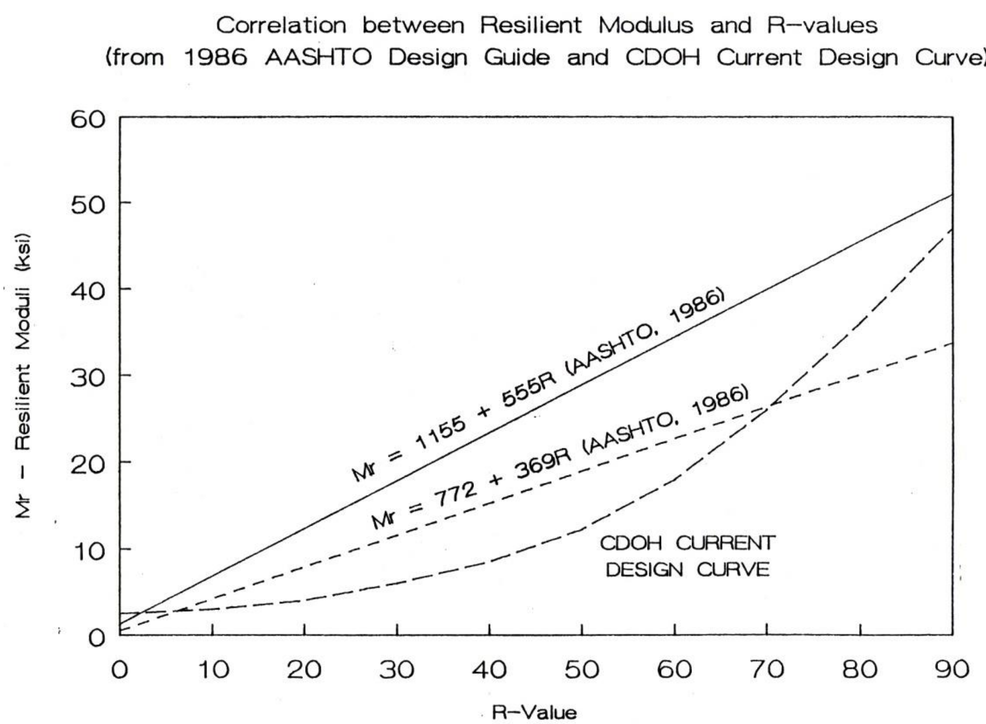


Figure S.26 Correlation Plot between Resilient Modulus and R-value
(*Resilient Properties of Colorado Soils, pg 15, FiguRe 2.10,* 1989 (6))

Table S.3 Comparisons of M_r Suggested NCHRP 1-40D and Colorado Soils with R-values is a comparison of M_r values. The test procedure was in accordance to AASHTO 307, Type 2 Material with a loading sequence in accordance with SHRP TP 46, Type 2 Material. Additional testing of Colorado soils with 2 and 4 percent above optimum moisture were conducted to simulate greater moisture contents if the in-situ soils have an increase in moisture. Generally, the strengths decreased, but not always. Colorado soils exhibit a lower M_r than the recommended values from publication NCHRP 1-37A, Table 2.2.51.

S.4 Modulus of Subgrade Reaction (k-value)

The k-value is used for the support of rigid pavements structures. The graphical representation (**Figure S.28 Distribution of Wheel Load to Subgrade Reaction (k-value))** is the traditional way to explain the interaction of subgrade reaction to a moving wheel load. As the wheel load moves toward an area of concern, the subgrade reacts with a slightly larger reaction and when the wheel loading moves away the subgrade reaction it is less. That variable reaction is the engineering property k-value. As an historical note, in the 1920's, Westergaard's work led to the concept of the modulus of subgrade reaction (k-value). Like elastic modulus, the k-value of a subgrade is an elastic constant which defines the material's stiffness or resistance to deformation. The value k actually represents the stiffness of an elastic spring.

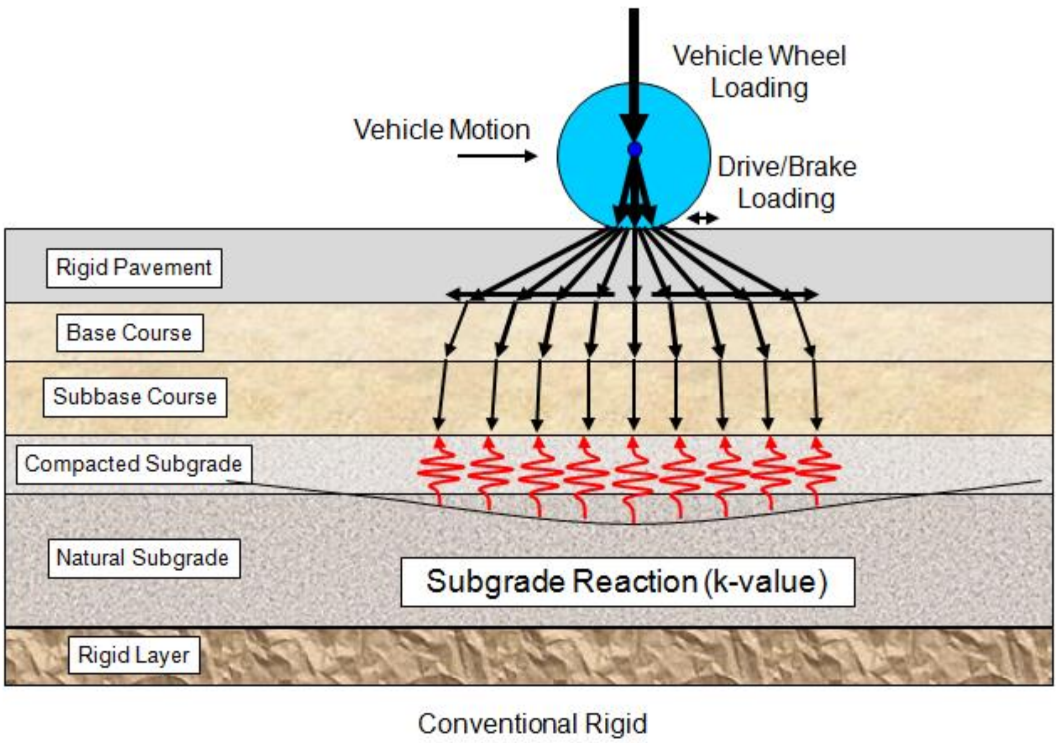


Figure S.27 Distribution of Wheel Load to Subgrade Reaction (k-value)

Table S.3 Comparisons of M_r Suggested NCHRP 1-40D and Colorado Soils with R-values

Research Results Digest of NCHRP Project 1-40D (July 2006)				Soil Classification	Colorado Soils (Unpublished Data 7/12/2002)			
Flexible Subgrades		Rigid Subgrades			R-value	Optimum M_r	2% Over Optimum M_r	4% Over Optimum M_r
Opt. M_r (mean)	Opt. M_r (std dev)	Opt. M_r (mean)	Opt. M_r (std dev)					
29,650	15,315	13,228	3,083	A-1-a	yt	-	-	-
26,646	12,953	14,760	8,817	A-1-b	32	10,181	9,235	-
21,344	13,206	14,002	5,730	A-2-4	50	7,842	5,161	3,917
					37	11,532	5,811	4,706
					40	10,750	7,588	7,591
					38	7,801	7,671	-
-	-	-	-	A-2-5	-	-	-	-
20,556	12,297	16,610	6,620	A-2-6	35	8,024	4,664	4,343
					19	7,600	5,271	5,009
					45	8,405	5,954	5,495
					42	8,162	7,262	-
					37	7,814	5,561	4800*
					24	7,932	5,846	5210*
					49	10,425	9,698	8196*
16,250	4,598	-	-	A-2-7	13	7,972	4,702	3,511
					18	7,790	5,427	4,003
					29	8,193	5,558	5,221
					9	11,704	8,825	7,990
24,697	11,903	-	-	A-3	-	-	-	-
16,429	12,296	17,763	8,889	A-4	19	6,413	5,233	4,736
16,429 -	12,296 -	17,763 -	8,889 -	A-4 A-5	23	10,060	6,069	5,729
					49	7,583	7,087	6,311
					44	11,218	6,795	5794*
					-	-	-	-
14,508	9,106	14,109	5,935	A-6	21	7,463	3,428	2,665
14,508 13,004	9,106 13,065	14,109 7,984	5,935 3,132	A-6 A-7-5	8	5,481	3,434	2,732
					12	5,162	3,960	2,953
					14	4,608	3,200	2,964
					10	13,367	4,491	3,007
					19	6,638	3,842	3,456
					10	7,663	4,244	3,515
					15	5,636	3,839	3,551
					17	7,135	4,631	3,821
					21	6,858	5,488	4,010
					14	6,378	4,817	4,234
					8	5,778	5,243	4,934
					40	17,436	7,438	5,870
					27	7,381	5,491	-
					17	8,220	6,724	-
					26	11,229	9,406	5,238
11,666	7,868	13,218	322	A-7-6	6	4,256	2,730	1,785
11,666	7,868	13,218	322	A-7-6	8	4,012	2,283	1,909
					10	5,282	2,646	1,960
					11	4,848	3,159	2,157
					5	6,450	3,922	2,331
					6	5,009	2,846	2,410
					6	5,411	3,745	2,577
					11	4,909	3,340	2,795
					15	9,699	4,861	3,018
					16	6,842	4,984	3,216
					29	8,873	4,516	3,308
					14	4,211	3,799	3,380
					7	7,740	5,956	4,107
					23	8,154	6,233	4,734
					27	7,992	6,552	5,210

S.4.1 Static Elastic k-value

The gross k-value was used in previous AASHTO pavement design guides. It not only represented the elastic deformation of the subgrade under a loading plate, but also substantial permanent deformation. The static elastic portion of the k-value is used as an input in the 1998 AASHTO Supplement guide. The k-value can be determined by field plate bearing tests (AASHTO T 221 or T 222) or correlation with other tests. There is no direct laboratory test procedure for determining k-value. The k-value is measured or estimated on top of the finished roadbed soil or embankment upon which the base course and concrete slab is constructed. The classical equation for gross k-value is shown in **Equation S.39**.

$$\textbf{k-value} = \rho / \Delta \qquad \textbf{Eq. S.39}$$

Where:

k-value = modulus of subgrade reaction (spring constant)
ρ = applied pressure = area of 30" diameter plate
Δ = measured deflection

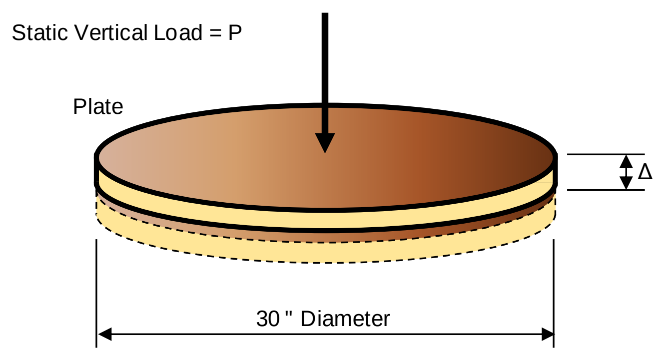


Figure S.28 Field Plate Load Test for k-value

S.4.2 Dynamic k-value

In the *AASHTO Guide for Mechanistic-Empirical Design, A Manual of Practice*, the effective k-value used is the effective <u>dynamic</u> k-value (24). Dynamic means a quick force is applied, such as a falling weight not an oscillating force. CDOT obtains the dynamic k-value from the Falling Weight Deflectometer (FWD) testing with a backcalculation procedure. There is an approximate relationship between static and dynamic k-value. The dynamic k-value may be converted to the initial static value by dividing the mean dynamic k-value by two to estimate the mean static k-value. CDOT uses this conversion because it does not perform the static plate bearing test.

FWD testing is normally performed on an existing surface course. In the M-E Design Guide software the dynamic k-value is used as an input for rehabilitation projects only. The dynamic k-value is not used as an input for new construction or reconstruction. One k-value is entered as an input in the rehabilitation calculation. The one k-value is the arithmetic mean of like backcalculated values and is used as a foundation support value. The software also needs the month the FWD is performed. The software uses an integrated climatic model to make seasonal adjustments to the support value. The software will backcalculate an effective single dynamic k-value for each month of the design analysis period for the existing unbound sublayers and subgrade soil. The effective dynamic k-value is essentially 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 entered k-value will remain as an effective dynamic k-value for that month throughout the analysis period, but the effective dynamic k-value for other months will vary according to moisture movement and frost depth in the pavement (24).

S.5 Bedrock

Table S.4 Poisson's Ratio for Bedrock
(Modified from Table 2.2.55 and Table 2.2.52, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Description	μ (Range)	μ (Typical)
Solid, Massive, Continuous	0.10 to 0.25	0.15
Highly Fractured, Weathered	0.25 to 0.40	0.30
Rock Fill	0.10 to 0.40	0.25

Table S.5 Elastic Modulus for Bedrock
(Modified from Table 2.2.54, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Description	E (Range)	E (Typical)
Solid, Massive, Continuous	750,000 to 2,000,000	1,000,000
Highly Fractured, Weathered	250,000 to 1,000,000	50,000
Rock Fill	Not available	Not available

S.6 Unbound Subgrade, Granular, and Subbase Materials

Table S.6 Poisson's Ratios for Subgrade, Unbound Granular and Subbase Materials
(Modified from Table 2.2.52, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Description	μ (Range)	μ (Typical)
Clay (saturated)	0.40 to 0.50	0.45
Clay (unsaturated)	0.10 to 0.30	0.20
Sandy Clay	0.20 to 0.30	0.25
Silt	0.30 to 0.35	0.325
Dense Sand	0.20 to 0.40	0.30
Course-Grained Sand	0.15	0.15
Fine-Grained Sand	0.25	0.25
Clean Gravel, Gravel-Sand Mixtures	0.354 to 0.365	0.36

Table S.7 Coefficient of Lateral Pressure
(Modified from Table 2.2.53, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Description	Angle of Internal Friction, ϕ	Coefficient of Lateral Pressure, k_o
Clean Sound Bedrock	35	0.495
Clean Gravel, Gravel-Sand Mixtures, and Coarse Sand	29 to 31	0.548 to 0.575
Clean Fine to Medium Sand, Silty Medium to Coarse Sand, Silty or Clayey Gravel	24 to 29	0.575 to 0.645
Clean Fine Sand, Silty or Clayey Fine to Medium Sand	19 to 24	0.645 to 0.717
Fine Sandy Silt, Non-Plastic Silt	17 to 19	0.717 to 0.746
Very Stiff and Hard Residual Clay	22 to 26	0.617 to 0.673
Medium Stiff and Stiff Clay and Silty Clay	19 to 19	0.717

S.7 Chemically Stabilized Subgrades and Bases

Critical locations in the layers have been defined for the M-E Design, refer to **Figure S.31 Critical Stress Locations for Stabilized Subgrade** and **Figure S.32 Critical Stress/Strain Locations for Stabilized Bases**. CDOT has historically used the empirical design methodology using structural coefficients of stabilized subgrade and base layers and assigned a_2 for the structural coefficient.

Lightly stabilized materials for construction expediency are not included. They could be considered as unbound materials for design purposes (5).

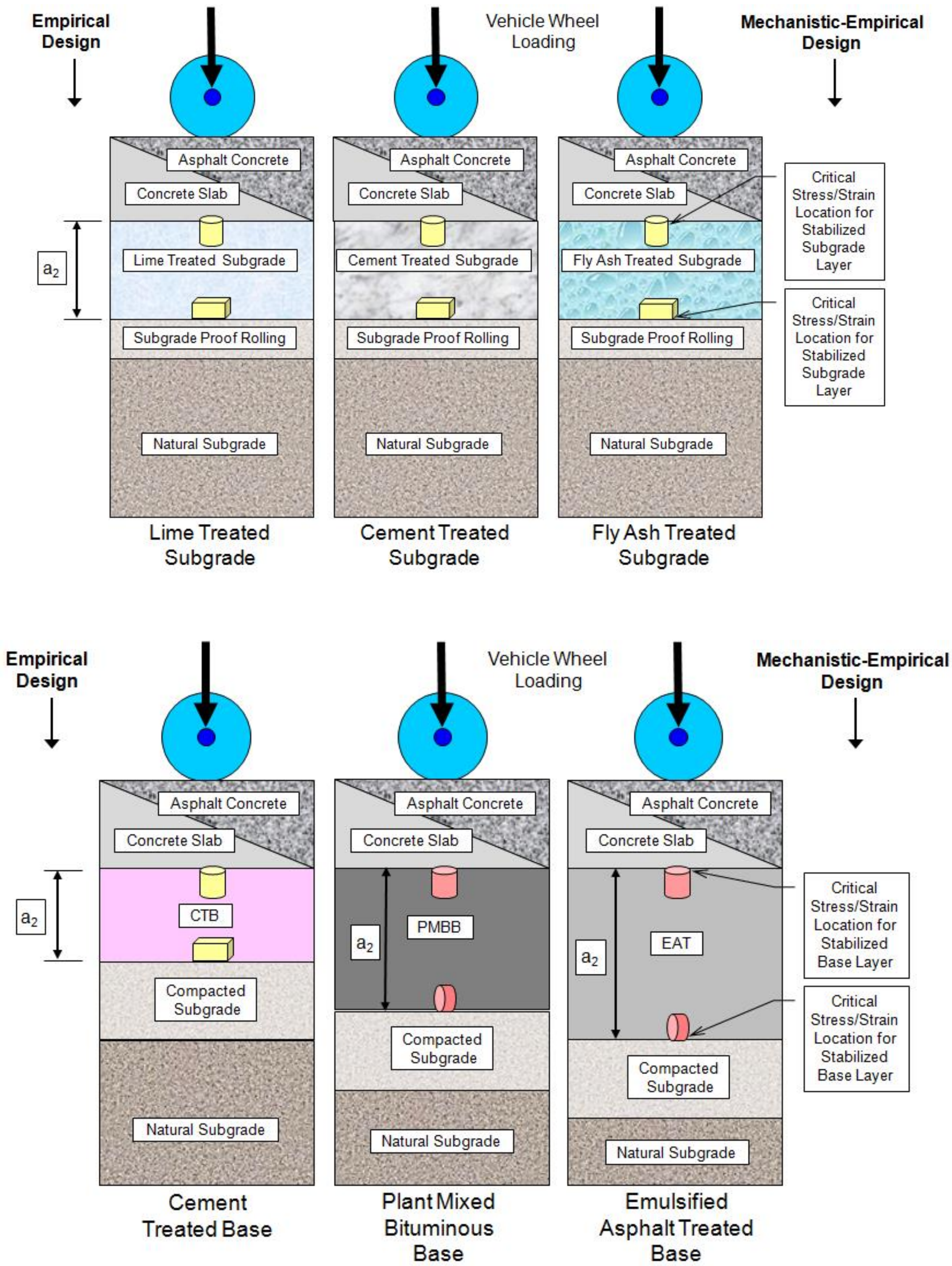


Figure S.29 Critical Stress Locations for Stabilized Subgrade

Table S.8 Poisson's Ratios for Chemically Stabilized Materials
(Table 2.2.48, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Chemically Stabilized Materials	Poisson's ratio, μ
Cement Stabilized Aggregate (Lean Concrete, Cement Treated, and Permeable Base)	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

Table S.9 Poisson's Ratios for Asphalt Treated Permeable Base
(Table 2.2.16 and Table 2.2.17, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Temperature, °F	μ (Range)	μ (Typical)
< 40 °F	0.30 to 0.40	0.35
40 °F to 100 °F	0.35 to 0.40	0.40
> 100 °F	0.40 to 0.48	0.45

Table S.10 Poisson's Ratios for Cold Mixed asphalt and Cold Mixed Recycled Asphalt Materials
(Table 2.2.18 and Table 2.2.19, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Temperature, °F	μ (Range)	μ (Typical)
< 40 °F	0.20 to 0.35	0.30
40 °F to 100 °F	0.30 to 0.45	0.35
> 100 °F	0.40 to 0.48	0.45

The critical location of vertical loads for stabilized subgrades are at the interface of the surface course and stabilized subgrade or top of the stabilized subgrade. The material stabilized subgrade element has the greatest loads at this location when the wheel loadings are directly above. Strength testing may be performed to determine compressive strength (f'_c), unconfined compressive strength (q_u), modulus of elasticity (E), time-temperature dependent dynamic modulus (E*), and resilient modulus (M_r).

The critical locations for flexural loading of stabilized subgrades are at the interface of the stabilized subgrade and non-stabilized subgrade or bottom of the stabilized subgrade. The material stabilized subgrade element has the greatest flexural loads at this location when the wheel loadings are directly above. Flexural testing may be performed to determine flexural strength (MR).

S.7.1 Top of Layer Properties for Stabilized Materials

Chemically stabilized materials are generally required to have a minimum compressive strength. Refer to **Table S.11 Minimum Unconfined Compressive Strengths for Stabilized Layers** for suggested minimum unconfined compressive strengths. 28-day values are used conservatively in design.

E, E*, and M_r testing should be conducted on stabilized materials containing the target stabilizer content, molded, and conditioned at optimum moisture and maximum density. Curing must also be as specified by the test protocol and reflect field conditions (5). **Table S.13 Typical M_r Values for Deteriorated Stabilized Materials** presents deteriorated semi-rigid materials stabilized showing the deterioration or damage of applied traffic loads and frequency of loading. The table values are required for HMA pavement design only.

Table S.11 Minimum Unconfined Compressive Strengths for Stabilized Layers
(Modified from Table 2.2.40, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Stabilized Layer	Minimum Unconfined Compressive Strength, psi [1, 2]	
	Rigid Pavement	Flexible Pavement
Subgrade, Subbase, or Select Material	200	250
Base Course	500	750
Asphalt Treated Base	Not available	Not available
Plant Mix Bituminous Base	Not available	Not available
Cement Treated Base	Not available	Not available

Note:
[1] Compressive strength determined at 7-days for cement stabilization and 28-days for lime and lime cement fly ash stabilization.
[2] These values shown should be modified as needed for specific site conditions.

Table S.12 Typical E, E*, or M_r Values for Stabilized Materials
(Modified from Table 2.2.43, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Stabilized Material	E or M_r (Range), psi	E or M_r (Typical), psi
Soil Cement (E)	50,000 to 1,000,000	500,000
Cement Stabilized Aggregate (E)	700,000 to 1,500,000	1,000,000
Lean Concrete (E)	1,500,000 to 2,500,000	2,000,000
Lime Stabilized Soils (M_r[1])	30,000 to 60,000	45,000
Lime-Cement-Fly Ash (E)	500,000 to 2,000,000	1,500,000
Permeable Asphalt Stabilized Aggregate (E*)	Not available	Not available
Permeable Cement Stabilized Aggregate (E)	Not available	750,000
Cold Mixed Asphalt Materials (E*)	Not available	Not available
Hot Mixed Asphalt Materials (E*)	Not available	Not available
Note: [1] For reactive soils within 25% passing No. 200 sieve and PI of at least 10.		

Table S.13 Typical M_r Values for Deteriorated Stabilized Materials
(Modified from Table 2.2.44, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Stabilized Material	Typical Deteriorated M_r (psi)
Soil Cement	25,000
Cement Stabilized Aggregate	100,000
Lean Concrete	300,000
Lime Stabilized Soils	15,000
Lime-Cement-Fly Ash	40,000
Permeable Asphalt Stabilized Aggregate	Not available
Permeable Cement Stabilized Aggregate	50,000
Cold Mixed Asphalt Materials	Not available
Hot Mixed Asphalt Materials	Not available

S.7.2 Bottom of Layer Properties for Stabilized Materials

Flexural Strengths or Modulus of Rupture (M_r) should be estimated from laboratory testing of beam specimens of stabilized materials. M_r values may also be estimated from unconfined (q_u) testing of cured stabilized material samples. **Table S.14 Typical Modulus of Rupture (M_r) Values for Stabilized Materials** shows typical values. The table values are required for HMA pavement design only

Table S.14 Typical Modulus of Rupture (M_r) Values for Stabilized Materials
(Modified from Table 2.2.47, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Stabilized Material	Typical Modulus of Rupture M_r (psi)
Soil Cement	100
Cement Stabilized Aggregate	200
Lean Concrete	450
Lime Stabilized Soils	25
Lime-Cement-Fly Ash	150
Permeable Asphalt Stabilized Aggregate	None
Permeable Cement Stabilized Aggregate	200
Cold Mixed Asphalt Materials	None
Hot Mixed Asphalt Materials	Not available

Tensile strength for hot mix asphalt is determined by actual laboratory testing in accordance with CDOT CP-L 5109 or AASHTO T 322 at 14 °F. Creep compliance is the time dependent strain divided by the applied stress and is determined by actual laboratory testing in accordance with AASHTO T 332.

S.7.3 Other Properties of Stabilized Layers

S.7.3.1 Coefficient of Thermal Expansion of Aggregates

Thermal expansion is the characteristic property of a material to expand when heated and contract when cooled. The coefficient of thermal expansion is the factor that quantifies the effective change one degree will have on the given volume of a material. The type of course aggregate exerts the most significant influence on the thermal expansion of portland cement concrete (3). National recommended values for the coefficient of thermal expansion in PCC are shown in **Table S.15 Recommended Values of PCC Coefficient of Thermal Expansion**.

Table S.15 Recommended Values of PCC Coefficient of Thermal Expansion
(Table 2.10, *AASHTO Guide for Design of Pavement Structures*, 1993)

Type of Course Aggregate	Concrete Thermal Coefficient (10^{-6} inch/inch/°F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8

Limestone	3.8

The Long-Term Pavement Performance (LTPP) database shows a coefficient of thermal expansion of siliceous gravels in Colorado. Siliceous gravels are a group of sedimentary "sand gravel" aggregates that consist largely of silicon dioxide (SiO_2) makeup. Quartz a common mineral of the silicon dioxide, may be classified as such, and is a major constituent of most beach and river sands.

Table S.16 Unbound Compacted Material Dry Thermal Conductivity and Heat Capacity
(Modified from Table 2.3.5, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Property	Soil Type	Range of μ	Typical μ
Dry Thermal Conductivity, K (Btu/hr-ft-°F)	A-1-a	0.22 to 0.44	0.30
	A-1-b	0.22 to 0.44	0.27
	A-2-4	0.22 to 0.24	0.23
	A-2-5	0.22 to 0.24	0.23
	A-2-6	0.20 to 0.23	0.22
	A-2-7	0.16 to 0.23	0.20
	A-3	0.25 to 0.40	0.30
	A-4	0.17 to 0.23	0.22
	A-5	0.17 to 0.23	0.19
	A-6	0.16 to 0.22	0.18
	A-7-5	0.09 to 0.17	0.13
	A-7-6	0.09 to 0.17	0.12
Dry Heat Capacity, Q (Btu/lb-°F)	All soil types	0.17 to 0.20	Not available

Table S.17 Chemically Stabilized Material Dry Thermal Conductivity and Heat Capacity
(Modified from Table 2.2.49, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004)

Material Property	Chemically Stabilized Material	Range of μ	Typical μ
Dry Thermal Conductivity, K (Btu/hr-ft-°F)	Lime	1.0 to 1.5	1.25
Dry Heat Capacity, Q (Btu/lb-°F)	Lime	0.2 to 0.4	0.28

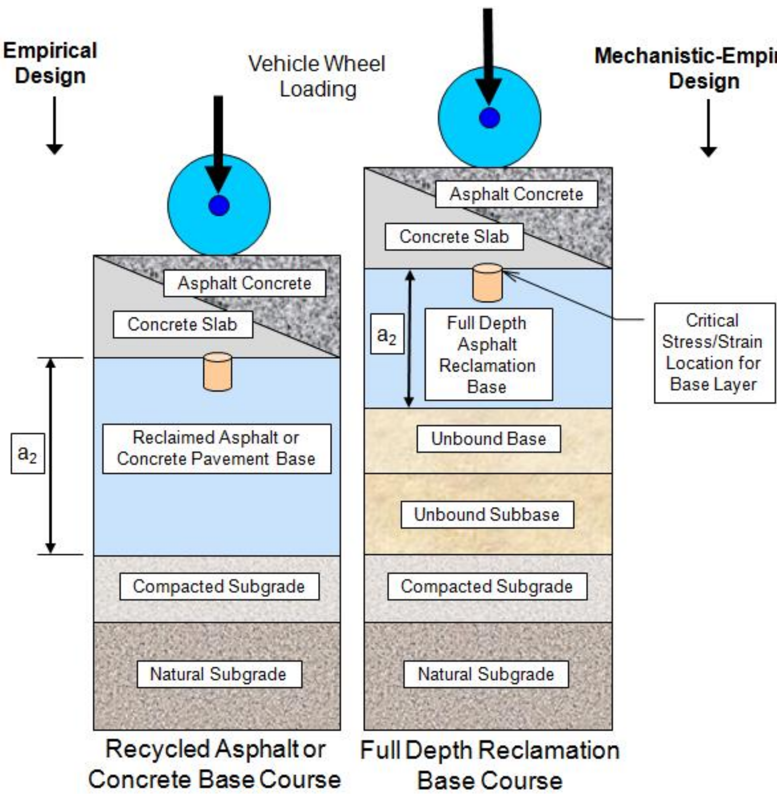


Figure S.30 Critical Stress Locations for Recycled Pavement Bases

Table S.18 Asphalt Concrete and PCC Dry Thermal Conductivity and Heat Capacity
(Modified from Table 2.2.21 and Table 2.2.39, *Guide for Mechanistic-Empirical Design*, Final Report, NCHRP Project 1-37A, March 2004)

Material Property	Chemically Stabilized Material	Range of μ	Typical μ
Dry Thermal Conductivity, K (Btu/hr-ft-°F)	Asphalt concrete	Not available	0.44 to 0.81
	PCC	1.0 to 1.5	1.25
Dry Heat Capacity, Q (Btu/lb-°F)	Asphalt concrete	Not available	0.22 to 0.40
	PCC	0.20 to 0.28	0.28

S.7.3.2 Saturated Hydraulic Conductivity

Saturated Hydraulic Conductivity (k_{sat}) is required to determine the transient moisture profiles in compacted unbound materials. Saturated hydraulic conductivity may be measured direct by using a permeability test AASHTO T 215.

S.8 Reclaimed Asphalt and Recycled Concrete Base Layer

The critical location vertical loads for reclaimed asphalt or recycled concrete bases are at the interface of the surface course and top of the recycled pavement. The recycled pavement element has the greatest loads at this location when the wheel loadings are directly above. Strength testing may be performed to determine modulus of elasticity (E) and/or resilient modulus (M_r). These bases are considered as unbound materials for design purposes. If the reclaimed asphalt base is stabilized and if an indirect tension (S_t) test can be performed then these bases may be considered as bound layers.

Table S.19 Cold Mixed Asphalt and Cold Mixed Recycled Asphalt Poisson's Ratios
(Table 2.2.18 and Table 2.2.19, *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004) [A Restatement of **Table S.10]**

Temperature (°F)	Range of μ	Typical μ
< 40	0.20 to 0.35	0.30
40 to 100	0.30 to 0.45	0.35
> 100	0.40 to 0.48	0.45

Table S.20 Typical E, E*, or M_r Values for stabilized Materials
(Modified from Table 2.2.43., *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, March 2004) [A restatement of **Table S.12**]

Stabilized Material	Range of E or M_r (psi)	Typical E or M_r (psi)
Soil Cement (E)	50,000 to 1,000,000	500,000
Cement Stabilized Aggregate (E)	700,000 to 1,500,000	1,000,000
Lean Concrete (E)	1,500,000 to 2,500,000	2,000,000
Lime Stabilized Soils (M_r^1)	30,000 to 60,000	45,000
Lime-Cement-Fly Ash (E)	500,000 to 2,000,000	1,500,000
Permeable Asphalt Stabilized Aggregate (E*)	Not available	Not available
Permeable Cement Stabilized Aggregate E	Not available	750,000
Cold Mixed Asphalt Materials (E*)	Not available	Not available
Hot Mixed Asphalt Materials (E*)	Not available	Not available
Note: [1] For reactive soils within 25% passing No. 200 sieve and PI of at least 10.		

S.9 Fractured Rigid Pavement

Rubblization is a fracturing of existing rigid pavement to be used as a base. The rubblized concrete responds as a high-density granular layer.

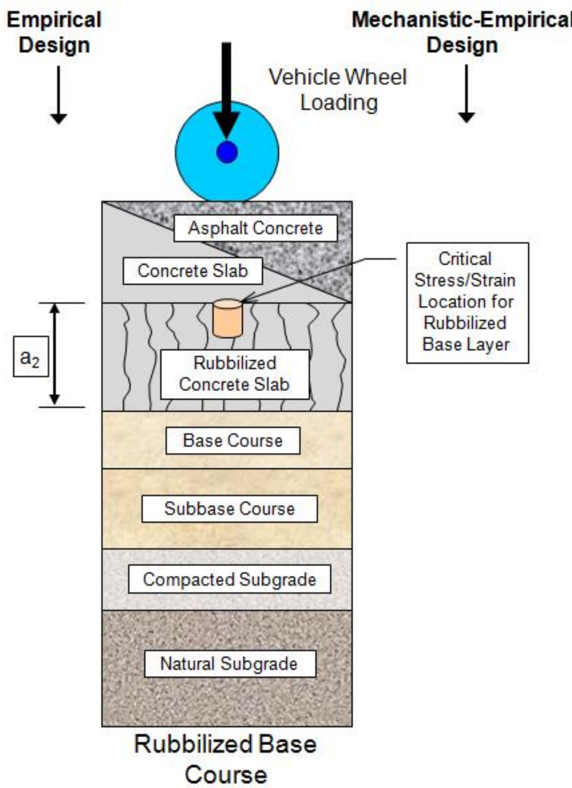


Figure S.31 Critical Stress Location for Rubblized Base

Table S.21 Poisson's Ratio for PCC Materials
(Table 2.2.29, *Guide for Mechanistic-Empirical Design, Final Report.*,
NCHRP Project 1-37A, Mar. 2004)

PCC Materials		Range of μ	Typical μ
PCC Slabs (newly constructed or existing)		0.15 to 0.25	0.20 (use 0.15 for CDOT)
Fractured Slab	Crack/seat	0.15 to 0.25	0.20
	Break/seat	0.15 to 0.25	0.20
	Rubblized	0.25 to 0.40	0.30

Table S.22 Typical M_r Values for Fractured PCC Layers
(Table 2.2.28, *Guide for Mechanistic-Empirical Design, Final Rpt.*, NCHRP Project 1-37A, Mar. 2004)

Fractured PCC Layer Type	Ranges of M_r (psi)
Crack and Seat or Break and Seat	300,000 to 1,000,000
Rubblized	50,000 to 150,000

S.10 Pavement Deicers

S.10.1 Magnesium Chloride

Magnesium Chloride ($MgCl_2$) is a commonly used roadway anti-icing/deicing agent in conjunction with, or in place of salts and sands. The $MgCl_2$ solution can be applied to traffic surfaces prior to precipitation and freezing temperatures in an anti-icing effort. The $MgCl_2$ effectively decreases the freezing point of precipitation to about 16° F. If ice has already formed on a roadway, $MgCl_2$ can aid in the deicing process.

Magnesium chloride is a proven deicer that has done a great deal for improving safe driving conditions during inclement weather, but many recent tests have shown the magnesium may have a negative impact on the life of concrete pavement. Iowa State University performed as series of experiments testing the effects of different deicers on concrete. They determined that the use of magnesium and/or calcium deicers may have unintended consequences in accelerating concrete deterioration (20). $MgCl_2$ was mentioned to cause discoloration, random fracturing and crumbling (20).

In 1999, a study was performed to identify the environmental hazards of $MgCl_2$. This study concluded that it was highly unlikely the typical $MgCl_2$ deicer would have any environmental impact greater than 20 yards from the roadway. It is even possible that $MgCl_2$ may offer a positive net environmental impact if it limits the use of salts and sands. The study's critical finding was that any deicer must limit contaminates, as well as, the use of rust inhibiting additives like phosphorus (21).

The 1999 study led to additional environmental studies in 2001. One study concluded that $MgCl_2$ could increase the salinity in nearby soil and water, which is more toxic to vegetation than fish (22). Another study identified certain 30% $MgCl_2$ solutions deicers used in place of pure $MgCl_2$ had far higher levels of phosphorus and ammonia. These contaminates are both far more hazardous to aquatic life than $MgCl_2$ alone (23).

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