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Evaluation of Soil Water Characteristic Curves (SWCC) in Pavement ME for Nevada's Unbound Materials

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16. Abstract

The American Association of State Highway and Transportation Officials (AASHTO) adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) as an interim pavement design standard in 2008. In 2015, the Nevada Department of Transportation (NDOT) implemented the MEPDG for the structural design of new and rehabilitated flexible pavements. Resilient modulus of the unbound material layers plays a large role in pavement performance, and this parameter changes with seasonal variation. Currently, the AASHTOWare Pavement ME software internally calculates this seasonal variation using climatic data and estimated unbound material parameters, including soil water characteristic curves (SWCC) and saturated hydraulic conductivity.

This study evaluated the impact of SWCC and saturated hydraulic conductivity on Nevada Pavement ME Design. An extensive laboratory evaluation was conducted on 23 Nevada unbound materials, which included testing for gradation, Atterberg limits, maximum dry density, optimum water content, specific gravity of solids, SWCC, methylene blue value (MBV), percent fines content (PFC), R-value, and saturated hydraulic conductivity. The model used to fit the SWCCs, consistent with the MEPDG, was the Fredlund and Xing model, which fit the SWCC data well. A sensitivity analysis was conducted in AASHTOWare Pavement ME (ver. 2.3.1), where directly measured unbound material properties, estimated unbound material properties, and internally estimated unbound material properties were used. It was found that for District 1, the internally estimated properties underestimate the impact of SWCC and saturated hydraulic conductivity, which is seen in an under-prediction of AC bottom-up fatigue cracking. For District 2, the internally estimated properties overestimate the impact of SWCC and saturated hydraulic conductivity, which is seen in an over-prediction of AC bottom-up fatigue cracking. In District 3, there was little impact from SWCC and saturated hydraulic conductivity on AC bottom-up fatigue cracking.

Additionally, historical records were collected from recent NDOT pavement projects and summarized in an electronic format. Combined with the laboratory evaluation, a comprehensive database for Nevada unbound material properties was produced. This database was used to make recommendations for unbound material properties for use in Nevada Pavement ME designs.

17. Key Words MEDPG, modulus, unbound materials, Fredlund and Xing, saturated hydraulic conductivity	18. Distribution Statement No restrictions.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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CHAPTER 1 INTRODUCTION AND BACKGROUND

Since July 2015, the Nevada Department of Transportation (NDOT) has implemented the use of the Mechanistic-Empirical Pavement Design Guide (MEPDG) and the accompanying AASHTOWare Pavement ME software to design new and rehabilitated flexible pavements (1–3). Pavement ME incorporates the effects of traffic loading, climatic conditions, and material properties, while conducting advanced mechanistic analysis of pavement structures. When taking into consideration climatic conditions, Pavement ME analyzes the effects of seasonal variation on the unbound material layers. Specifically, soil water characteristic curve (SWCC) parameters and saturated hydraulic conductivity are used to predict the degree of saturation of the unbound material, which is used to predict the resilient modulus (M_r) of that layer.

SWCC can be measured directly in the laboratory; however, the time and cost associated with the procedure has limited its use as part of a regular laboratory testing program. Because of this, correlations are developed with grain size distribution and soil index properties to predict the SWCC parameters. This has been shown to provide reasonable estimations for degree of saturation in subgrade materials as a function of time and depth, provided that the appropriate inputs were used.

NDOT currently uses the national database of SWCC developed under the National Cooperative Highway Research Program (NCHRP) 9-23A project to estimate SWCC parameters of subgrade materials in Nevada (4). However, generic SWCC parameter values are being used for unbound base material layers. Therefore, the purpose of this project is to provide a comprehensive database of SWCC default input values for unbound materials used in Nevada.

PURPOSE AND SCOPE

The primary objective of this study was to develop an organized database of material gradation and other engineering properties for the common types of aggregate base, select borrow, and subgrades from Nevada. To meet this objective, the followings tasks were completed and are summarized in figure 1:

- Collect information and create a database on locally available unbound material in Nevada. Historical records from NDOT pavement projects over the last 15 years were collected and summarized in an electronic database in an effort to create a more comprehensive database of commonly used unbound materials in Nevada.

- Conduct laboratory evaluation of unbound materials to evaluate the following properties; gradation, Atterberg limits, maximum dry unit weight (γ_{dmax}), specific gravity of solids (Gs), matric suction and volumetric water content, methylene blue value (MBV), and percent fines content (PFC).

- Conduct a sensitivity analysis using the AASHTOWare® Pavement ME Design software and the locally calibrated performance models for Nevada in order to assess the influence of the developed database on the NDOT MEDPG designs

- Incorporate the developed database into the NDOT MEPDG manual for designing flexible pavements in Nevada using the AASHTOWare Pavement ME

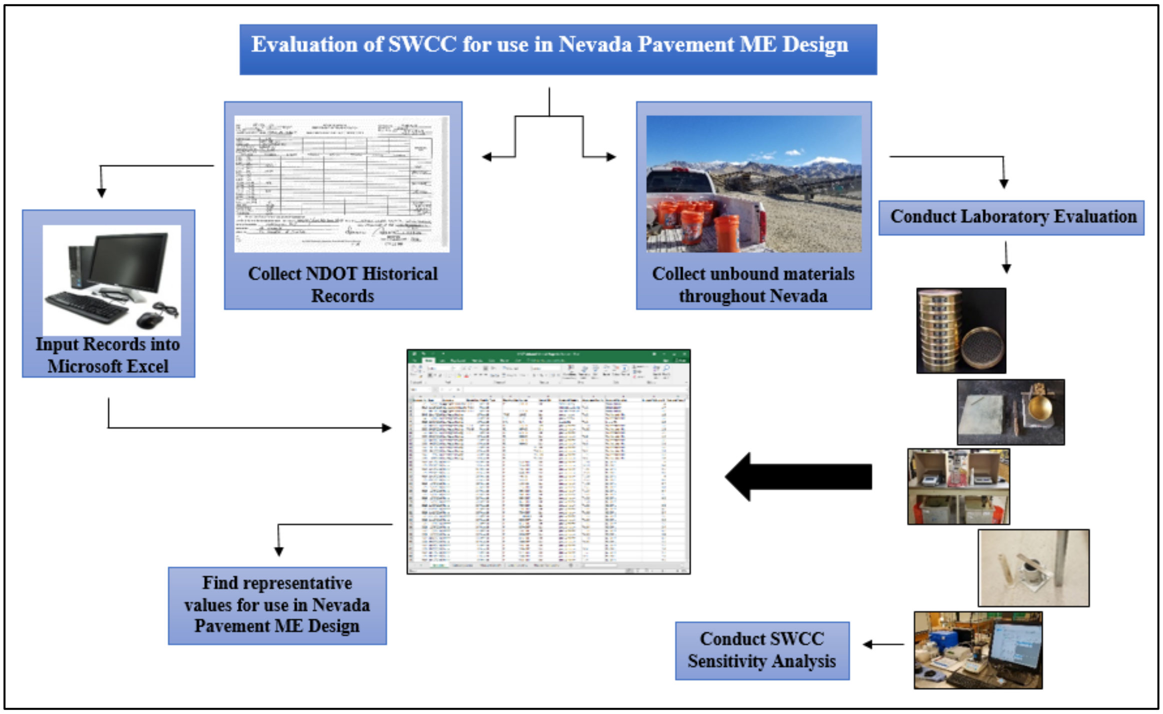

Figure 1. Overall flowchart of project approach.

To this end, base, borrow, and subgrade materials were sampled from all three NDOT Districts, which are defined by the boundaries shown in figure 2. Nine base materials, nine select borrow materials, and six subgrade materials were collected for testing.

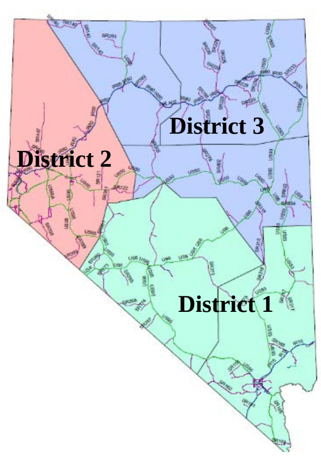

Figure 2. NDOT districts' boundaries.

LITERATURE REVIEW

This section provides a brief overview of the MEPDG, and discusses direct measurement and estimation methods for SWCC.

Overview of MEPDG

The development of the MEPDG was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration (FHWA), as part of the NCHRP project 1-37A (2). The purpose of this work was to provide a state-of-practice tool that can be used to design new and rehabilitated pavement, and it relies on mechanistic-empirical principles. This method is mechanistic in that pavement responses from loading are found through mathematical models, and it is empirical in that pavement responses are related to pavement performance. The mechanistic-empirical design method differs from previous methods by taking into consideration detailed traffic loading, climatic data, and material properties.

Several state highway agencies have implemented the MEPDG for their flexible pavement design. NDOT has recently adopted the MEPDG and its associated AASHTOWare Pavement ME software for designing new and rehabilitated flexible pavements in Nevada. Pavement ME requires five main categories of input parameters: traffic, climate, performance criteria targets and reliability, performance models, and material properties. Concerning traffic conditions, Pavement ME requires the input of average annual daily truck traffic (AADTT), the number of lanes per direction, percentage of trucks traveling in the design direction, percentage of trucks traveling in the design lane, operational speed, growth rate, and the distribution of each vehicle class. This information can be found using traffic data counts from NDOT.

To account for climate conditions needed for incremental damage accumulation, data from five weather-related parameters are required on an hourly basis over the duration of the design life. Those five parameters are air temperature, precipitation, wind speed, percentage of sunshine, and relative humidity. These data can be found by using a weather station near the project site. The AASHTOWare Pavement ME software contains a database of over 800 weather stations throughout the United States (US).

Performance criteria should also be defined. The performance criteria considered in Pavement ME for new flexible pavement designs are initial International Roughness Index (IRI), terminal IRI, AC top-down and bottom-up fatigue cracking, AC thermal cracking, total permanent deformation, and permanent deformation in only the AC layer. For each of the performance criteria, a limit and the level of reliability must be defined. Because this project deals specifically with materials in Nevada, the “Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement ME Design” was used to define the limits and reliability for each of the performance criteria (1).

Performance models must also be defined to be able to accurately predict the performance of the pavement design over the duration of the design life. Within Pavement ME, models for thermal cracking, fatigue cracking, and rutting can be defined. Using nationally calibrated models for a pavement design specifically in Nevada is not appropriate to use, as the pavement performance cannot be accurately predicted. Mixture specific models can be used, and these are appropriate to use when analyzing a specific mixture type. The "Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement ME Design" provides performance models that are specific to regions of Nevada, and these performance models were used for the sensitivity analysis portion of this report.

Input parameters for the material properties vary depending on the material being considered. The asphalt concrete (AC) layers require the input of several parameters, including: percentage of air voids at construction, layer thickness, effective binder content, dynamic shear modulus and phase angle, dynamic modulus, and thermal properties. When considering the unbound layers, the input parameters include SWCC, M_r, gradation, maximum dry density, optimum moisture content, saturated hydraulic conductivity, etc. The purpose of this study is to calibrate the input for SWCC and investigate the resulting impacts on new and rehabilitated pavement designs. Sensitivity analyses have been conducted to calibrate several of the input parameters in AASHTOWare Pavement ME, and this study focuses on SWCC.

Figure 3 summarizes the input parameters and how each of them are determined in accordance with the MEPDG Manual of Practice (5). Input parameters, such as *Gs*, optimum gravimetric water content (w_{opt}), γ_{dmax}, percent passing No. 200 (P_{200}), and the grain size at 50% passing (D_{60}) are all well-defined parameters. The procedures for each of these parameters are performed frequently, as they are relatively quick and simple tests for unbound materials. In contrast, SWCC testing is costly and time-consuming. Therefore, the SWCC input parameters are usually estimated using these well-defined parameters.

Three input levels are defined for the determination of the SWCC input parameters. Input level 1 involves direct measurement of matric suction and volumetric water content. By directly measuring w_{opt}, γ_{dmax}, and *Gs*, then the initial degree of saturation (S_{opt}), optimum volumetric water content (θ_{opt}), and the saturated volumetric water content (θ_{sat}) can be calculated using Equations 1–3. Non-linear regression analysis is then conducted to compute the SWCC model parameters. This procedure is summarized in figure 4.

$$\theta_{opt} = \frac{w_{opt}\gamma_{dmax}}{\gamma_{water}} \quad [1]$$

$$S_{opt} = \frac{\theta_{opt}}{1 - \frac{\gamma_{dmax}}{\gamma_{water} G_s}} \quad [2]$$

$$\theta_{sat} = \frac{\theta_{opt}}{S_{opt}} \quad [3]$$

Material Property	Input Level	Description
Specific Gravity (oven-dry), G_s	1	A direct measurement using AASHTO T100 (performed in conjunction with consolidation tests – AASHTO T180 for bases or AASHTO T 99 for other layers).
	2	Determined from P_{200}[1] and PI[2] of the layer as below: 1. Determine P_{200} and PI. 2. Calculate G_s (8): $G_s = 0.041(P_{200} * PI)^{0.29} + 2.65$
	3	Not applicable.
Optimum gravimetric water content, w_{opt}, and maximum dry unit weight of solids, γ_{dmax}	1	Typically, AASHTO T180 compaction test for base layers and AASHTO T99 compaction test for other layers.
	2	Determined from D_{60}[1], P_{200}[1] and PI[2] of the layer as illustrated below: 1. Read PI, P_{200}, and D_{60}. Identify the layer as a compacted base course, compacted subgrade, or natural in-situ subgrade. 2. Calculate S_{opt} (8): $S_{opt} = 6.752\ (P_{200} * PI)^{0.147} + 78$ 3. Compute w_{opt} (8): If $P_{200} * PI > 0$ $w_{opt} = 1.3\ (P_{200} * PI)^{0.73} + 11$ If $P_{200} * PI = 0$ $w_{opt\ (T99)} = 8.6425\ (D_{60})^{-0.1038}$ If layer is not a base course $w_{opt} = w_{opt\ (T99)}$ If layer is a base course $\Delta w_{opt} = 0.0156[w_{opt(T99)}]^2 - 0.1465 w_{opt(T99)} + 0.9$ $w_{opt} = w_{opt\ (T99)} - \Delta w_{opt}$ 4. To obtain G_s refer to the level 2 procedure for this input provided in this table above. 5. Compute $\gamma_{d\,max}$ for compacted materials, $\gamma_{d\,max\,comp}$ 6. $\gamma_{d\,max\,comp} = \dfrac{G_s \gamma_{water}}{1 + \dfrac{w_{opt} G_s}{S_{opt}}}$ 7. Compute $\gamma_{d\,max}$ If layer is a compacted material $\gamma_{d\,max} = \gamma_{d\,max\,comp}$ If layer is a natural in-situ material $\gamma_d = 0.90 \gamma_{d\,max\,comp}$ 8. EICM uses γ_d for $\gamma_{d\,max}$
	3	Not applicable.

[1] P_{200} and D_{60} can be obtained from a grain-size distribution test (AASHTO T 27).
[2] PI can be determined from an Atterberg limit test (AASHTO T 90).

Figure 3. Unbound material property inputs in MEPDG (2).

Input Level	Procedure to Determine SWCC Parameters	Required Testing
1	1) Direct measurement of suction (h) in psi, and volumetric water content (θ_w) pairs of values. 2) Direct measurement of optimum gravimetric water content, w_{opt} and maximum dry unit weight, $\gamma_{d\,max}$. 3) Direct measurement of the specific gravity of the solids, G_s. 4) Compute θ_{opt} as shown in equation 2.3.1. 5) Compute the S_{opt} as shown in equation 2.3.2. 6) Compute θ_{sat} as shown in equation 2.3.3. 7) Based on a non-linear regression analysis, compute the SWCC model parameters a_f, b_f, c_f, and h_r using the equation proposed by Fredlund and Xing, and the (h, θ_w) pairs of values obtained in step 1. $$\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[\ln\left[EXP(1) + \left(\frac{h}{a_f}\right)^{b_f} \right] \right]^{c_f}} \right]$$ $$C(h) = \left[1 - \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right]$$ 8) Input a_f(psi), b_f, c_f, and h_r (psi) into the Design Guide software. 9) EICM will generate the function at any water content (SWCC).	Pressure plate, filter paper, and/or Tempe cell testing. AASHTO T180 or AASHTO T99 for $\gamma_{d\,max}$. AASHTO T100 for G_s.

Figure 4. Input level 1 for SWCC in MEPDG (2).

While input level 1 is considered the most accurate method; direct measurement of matric suction and volumetric water content are too costly and time consuming to conduct for each project. Accordingly, input level 2 utilizes an estimation method where gradation and Atterberg Limits are correlated with SWCC model parameters. Here, w_{opt}, γ_{dmax}, and Gs are directly measured, as well as P_{200}, D_{60}, and PI. P_{200} is multiplied by PI, and S_{opt}, θ_{opt}, and θ_{sat} are calculated. Then, using nonlinear regression analysis, the Enhanced Integrated Climatic Model (EICM) computes the SWCC model parameters. Input level 3 also uses gradation and Atterberg Limits, which are then used in the EICM to automatically generate SWCC parameters. The calculation procedures for input levels 2 and 3 are summarized in figure 5. Detailed procedures on how to calculate w_{opt}, γ_{dmax}, and Gs are summarized in figure 3. For all three of the input levels used in the MEPDG Manual of Practice, the Fredlund and Xing model is utilized (6).

Input Level	Procedure to Determine SWCC Parameters	Required Testing
2	1) Direct measurement of optimum gravimetric water content, w_{opt} and maximum dry unit weight, $\gamma_{d\,max}$. 2) Direct measurement of the specific gravity of the solids, G_s. 3) Direct measurement of P_{200}, D_{60}, and PI. 3) The EICM will then internally do the following: a) Calculate $P_{200} * PI$. b) Calculate θ_{opt}, S_{opt}, and θ_{sat} as described for level 1. c) Based on a non-linear regression analysis, the EICM will compute the SWCC model parameters a_f, b_f, c_f, and h_r by using correlations with $P_{200}*PI$ and D_{60} *(13)*. i. If $P_{200}PI > 0$ $a_f = \frac{0.00364(P_{200}PI)^{3.35} + 4(P_{200}PI) + 11}{6.895}$, psi $\frac{b_f}{c_f} = -2.313(P_{200}PI)^{0.14} + 5$ $c_f = 0.0514(P_{200}PI)^{0.465} + 0.5$ $\frac{h_r}{a_f} = 32.44e^{0.0186(P_{200}PI)}$ ii. If $P_{200}PI = 0$ $a_f = \frac{0.8627(D_{60})^{-0.751}}{6.895}$, psi $\bar{b}_f = 7.5$ $c_f = 0.1772\ln(D_{60}) + 0.7734$ $\frac{h_r}{a_f} = \frac{1}{D_{60} + 9.7e^{-4}}$ d) The SWCC will then be established internally using the Fredlund and Xing equation as shown for Level 1.	AASHTO T180 or AASHTO T99 for $\gamma_{d\,max}$. AASHTO T100 for G_s. AASHTO T27 for P_{200} and D_{60}. AASHTO T90 for PI.
3	Direct measurement and input of P_{200}, PI, and D_{60}, after which EICM uses correlations with $P_{200}PI$ and D_{60} to automatically generate the SWCC parameters for each soil, as follows: 1) Identify the layer as a base course or other layer 2) Compute G_s as outlined in table 2.3.3 for Level 2. 3) Compute $P_{200} * PI$ 4) Compute S_{opt}, w_{opt}, and $\gamma_{d\,max}$ as shown for level 2. 6) Based on a non-linear regression analysis, the EICM will compute the SWCC model parameters a_f, b_f, c_f, and h_r by using correlations with $P_{200}PI$ and D_{60}, as shown for Level 2. 7) The SWCC will then be internally established using the Fredlund and Xing equation as shown for Level 1.	T27 for P_{200} and D_{60}. T90 for PI.

Figure 5. Input levels 2 and 3 for SWCC in MEPDG (2).

Soil moisture, suction, and temperature all play key roles in evaluating the environmental effects on M_r in the unbound layers. When all other factors are held constant, generally when the moisture content increases, the modulus decreases. When water in soil freezes, however, modulus values can greatly increase. This increase is followed by a reduced strength in the pavement structure when thawing occurs in the spring months. Therefore, defining SWCC input parameters for the unbound layers specific to the project location is critical in analyzing the performance of the pavement structure over time. The EICM considers the moisture, suction, and temperature as a function of time to determine a composite factor, F_{env}, that is used to adjust M_r with respect to these environmental factors. The optimum M_r (M_{ropt}), which is the M_r at maximum dry density and optimum water content, is multiplied by F_{env} to find the M_r adjusted for environmental effects, as shown by Equation 4.

$$M_r = F_{env} \times M_{ropt} \tag{4}$$

It is possible within each layer of the pavement structure, there are points that are frozen material, recovering material that is going back to its state before freezing happened, and unfrozen/fully recovered/normal material. The composite factor F_{env} represents a weighted average of adjustment factors associated with the material in each of these states. It is appropriate to use finite element analysis in order to determine F_{env}. The following steps are used by the EICM to determine the adjustment factor for frozen material:

1. P_{200}, PI, and D_{60} are user defined, and $P_{200}PI$ is calculated.

2. M_{ropt} is user defined.

3. Values for the frozen resilient modulus are assigned.

4. The frozen adjustment factor can then be calculated as a ratio of the frozen resilient modulus to M_{ropt}.

The adjustment factor for recovering materials is calculated at each node using finite element analysis, where freezing temperatures do not occur and the recovering ratio is less than 1. The EICM follows these steps to calculate the adjustment factor for recovering materials:

1. P_{200}, P_4, PI, and D_{60} are user defined. The estimated depth to groundwater table is user defined. $P_{200}PI$ is calculated.

2. The recovery ratio is calculated as a ratio of the number of hours elapsed since thawing started to the recovery period, which is a function of the material properties. Recovering material see an increase in modulus, which can be tracked by using the recovery ratio (RR), which ranges from 0 to 1.

3. S_{opt} is calculated.

4. From the SWCC, S_{equil} is calculated. Saturation can increase or decrease to an equilibrium value, which is S_{equil}. It can be calculated by using equations 5 and 6.

$$S_{equil} = C(h) \times \left[\frac{1}{\left[\ln\left[\exp(1) + \left(\frac{h}{a_f}\right)^{b_f} \right] \right]^{c_f}} \right] \quad [5]$$

$$C(h) = \left[1 - \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right] \quad [6]$$

5. Calculate R_{equil} value using the following equation:

$$\log R_{equil} = \log \frac{M_{Requil}}{M_{Ropt}} = a + \frac{b-a}{1+\exp[\ln\left(-\frac{b}{a}\right)+k_m(S_{equil}-S_{opt})]} \quad [7]$$

6. Compute the reduction factor as a function of P_{200}, P_4, and PI.

7. Compute the factor for recovering materials.

The following steps are used by the EICM to calculate the adjustment factor for unfrozen or fully recovered material:

1. Compute S_{opt}.

2. Calculate the adjustment factor for unfrozen or fully recovered material.

All of the above computations are made internally in the AASHTOWare Pavement ME software, so they are not computed by the user. The composite factor F_{env} is applied to the stress-dependent resilient modulus at each node of the finite element mesh, using the following equation:

$$M_r = F_{env} \times k_1 \times p_a \times \left(\frac{\theta}{P_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \quad [8]$$

where

M_r = resilient modulus (psi)
θ = bulk stress (psi)
τ_{oct} = octahedral shear stress (psi)
P_a = atmospheric pressure (psi)
k_1, k_2, k_3 = regression constants obtained by fitting M_r test data to equation

SWCC Direct Measurement Methodologies

Soil suction is commonly defined as a "free energy state of moisture within the soil, and it relates the moisture condition of the soil to its engineering behavior" (7). SWCC represents the relationship between the matric suction of the soil and its corresponding volumetric water content. Standard procedure ASTM D6836 describes five methods (Methods A through E) for measuring soil suction and volumetric water content (8).

Methods A, B, and C produce SWCCs in terms of matric suction. Method A, the hanging column method, can measure suctions ranging from 0 to 80 kPa. In this method, the application of matric suction is done by decreasing the pore water pressure and maintaining the pore gas pressure at atmospheric condition. Method B involves a pressure chamber and volumetric measurement, and it is suitable for suction measurements ranging from 0 to 1500 kPa. Similarly, Method C also uses a pressure chamber but the water content is measured gravimetrically instead, and it also measures suctions ranging from 0 to 1500 kPa. For both Methods B and C, pore water pressure is kept at atmospheric pressure. The pore gas pressure is then increased in order to apply suction. In methods A, B, and C, soil samples that have been saturated and are put into contact with a saturated porous plate or membrane, and suction is applied until water stops flowing from the sample from an attached capillary tube.

The chilled hygrometer method, Method D, using a dew point potentiometer is used to measure suction on the dry end of the SWCC. It measures at lower water contents and higher suctions (greater than 1000 kPa). This method measures suction in terms of total suction. it uses very small sample sizes and is generally used to measure points that will be combined with an SWCC that has been measured by another procedure.

Method E is the centrifuge method, which can measure suctions up to 120 kPa. This method is generally used for more coarse soils. Method E measures the suction in terms of matric suction. In this method, the specimen is subjected to centrifugal force. Suction is applied by changing the angular velocity. Both Method A and Method E are intended to measure the soil at lower suctions, where the sample is near saturation.

This study used an apparatus called the HYPROP, which is an automated measuring device that uses the simplified evaporation method. This method builds upon the work of Wind in 1968 (9). Five tensiometers were placed in a sample. As water evaporated from the sample, the weight was recorded, and an SWCC could be measured up to a suction of 50 kPa. In 1980, Schindler improved upon this initial evaporation method by reducing the number of tensiometers down to two, thus simplifying the evaluation of the SWCC (10).

The HYPROP relies on the above principles but has improved upon them, along with improvements in analysis made by Peters and Durner in 2008 (11). It can measure both the SWCC as well as the unsaturated hydraulic conductivity, at suctions ranging from saturation to nearly 1500 kPa. A diagram of the HYPROP apparatus is shown in figure 6 (12).

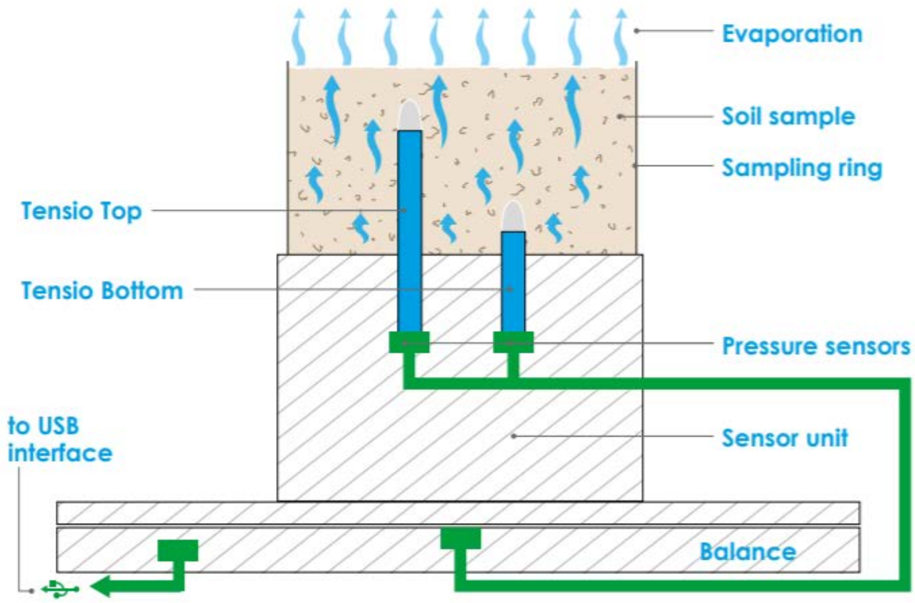

Figure 6. Diagram of HYPROP – after UMS (12).

As soil tension is measured, the optimal curve consists of four phases: regular measurement range, boiling delay, cavitation, and air entry. A diagram of the four phases is shown in figure 7. Regular measurement range is at the beginning of the test, where soil tension gradually increases but the curve has not flattened out. In the optimal curve, the boiling delay occurs where the tension has raised above the actual atmospheric air pressure. This occurs when the system is filled with no air. The boiling delay phase, however, is not necessary in evaluation of the SWCC. The cavitation phase is where the tension suddenly drops, as water vapor is formed in the tensiometer. After the sudden drop, the tension slowly decreases. The air entry phase is characterized by a second sudden drop in tension, but this time the tension drops to zero, as air enters the tensiometer. Typically, the boiling delay phase is never reached, but these curves can still be used for evaluation (12). A high level of repeatability has been shown for the HYPROP test results, although it has been shown that even when compacting samples to the same densities, there still is some level of variability (13).

Along with the evaporation method using the HYPROP, the chilled hygrometer method (Method D) from ASTM D 6836 using WP4C equipment was used in combination to measure suction values from saturation to the dry end of the SWCC. The HYPROP measures matric suction, which is the results of capillary forces. On the other hand, the chilled hygrometer measures total suction, which the combination of matric suction and osmotic suction. The osmotic suction is the suction attributed to dissolved salts in the pore fluid. The tension and volumetric water content measurements were then fitted using nonlinear regression and a least square error approach in Microsoft Excel, using the Fredlund and Xing model, represented by equations 9 and 10.

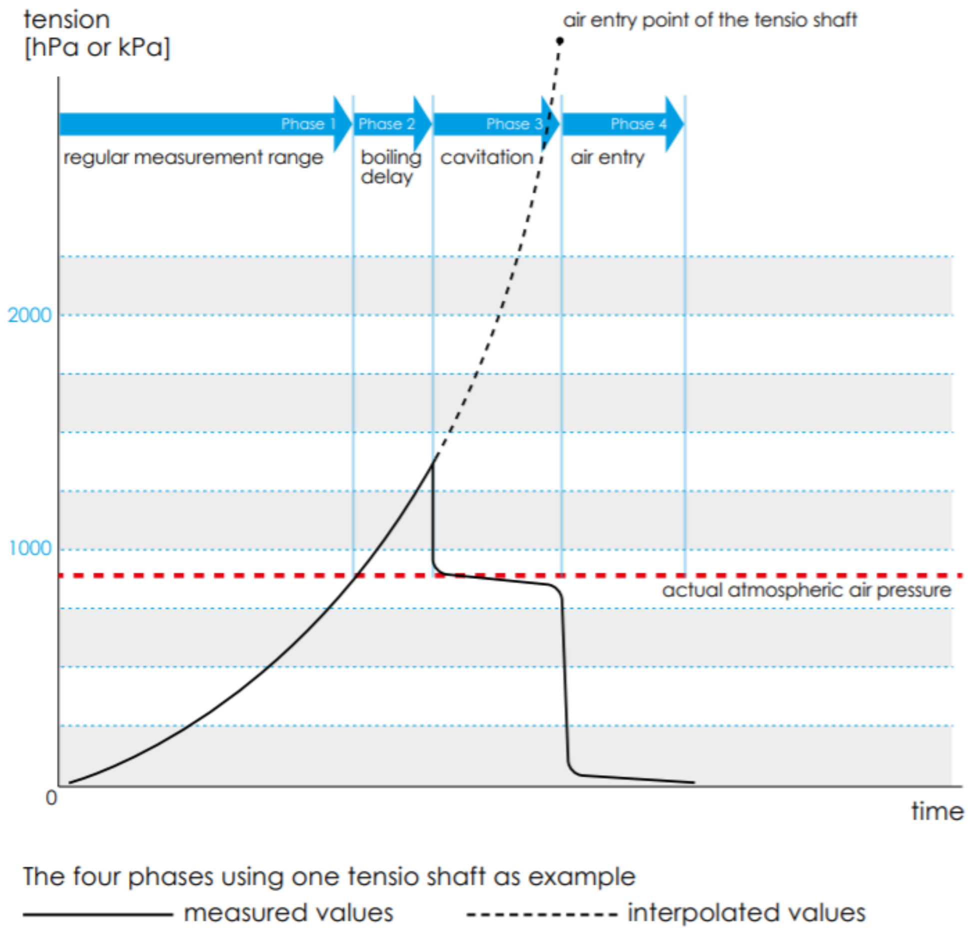

Figure 7. Phases of optimal tension measurement with HYPROP – after UMS (12).

$$\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[\ln\left[\exp(1) + \left(\frac{h}{a_f}\right)^{b_f} \right] \right]^{c_f}} \right] \qquad [9]$$

$$C(h) = \left[1 - \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \frac{1.45\times10^5}{h_r}\right)} \right] \qquad [10]$$

where

θ_w = volumetric water content
θ_{sat} = saturated volumetric water content
h = matric suction (psi).
$C(h)$ = correction factor
a_f, b_f, c_f, and h_r = fitting coefficients

The Fredlund and Xing model was developed based upon the assumption that the shape of the SWCC depends on the pore size distribution of the soil. The model has shown to be a good fit for several soil types over a range of suction values from 0 to 10^6 kPa. This is the complete range of the suction in soil from its fully saturated state to its dry state. It has been shown that when a soil is completely dry, the suction, h, reaches its maximum of about 10^6 kPa. This corresponds to 1.45×10^5 psi, which is used in the above equation. This creates an upper boundary for the SWCC equation. It can be found that the correction factor $C(h)$ is then equal to zero when h is equal to 1.45×10^5 psi, resulting in θ_w of zero. Figure 8 shows key components of an SWCC. The air-entry value corresponds to the suction in which the largest pores begin to fill with air. The residual water content is the point at which a large change in suction is required in order to reduce the water content. A more quantitative approach is taken to more clearly define the residual water content, where a tangent line is made at the inflection point. The plot shows two curves: adsorption curve and desorption curve. The curve considered for the development of the Fredlund and Xing model is the desorption curve. Nonlinear, least squares regression is used to determine a_f, b_f, c_f, and h_r. In the original Fredlund and Xing paper proposing the model, these fitting parameters are referred to as a, n, m, and ψ_r, respectively. The effects that varying each of these parameters has on the shape of the SWCC is represented in figure 9 to figure 11 (6).

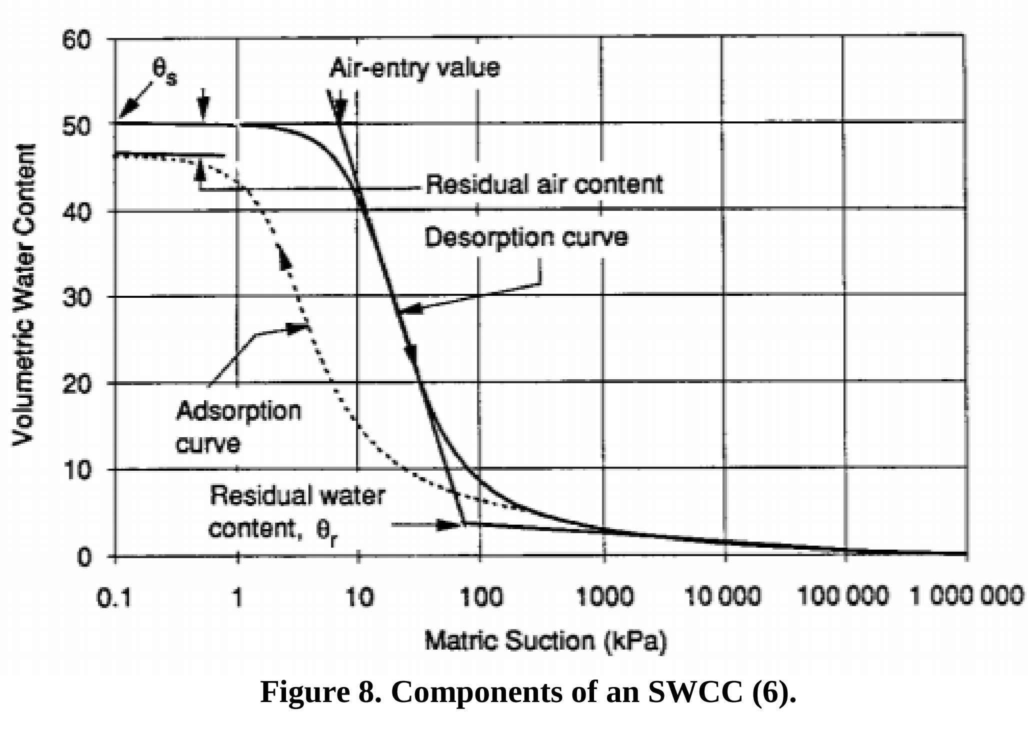

Figure 8. Components of an SWCC (6).

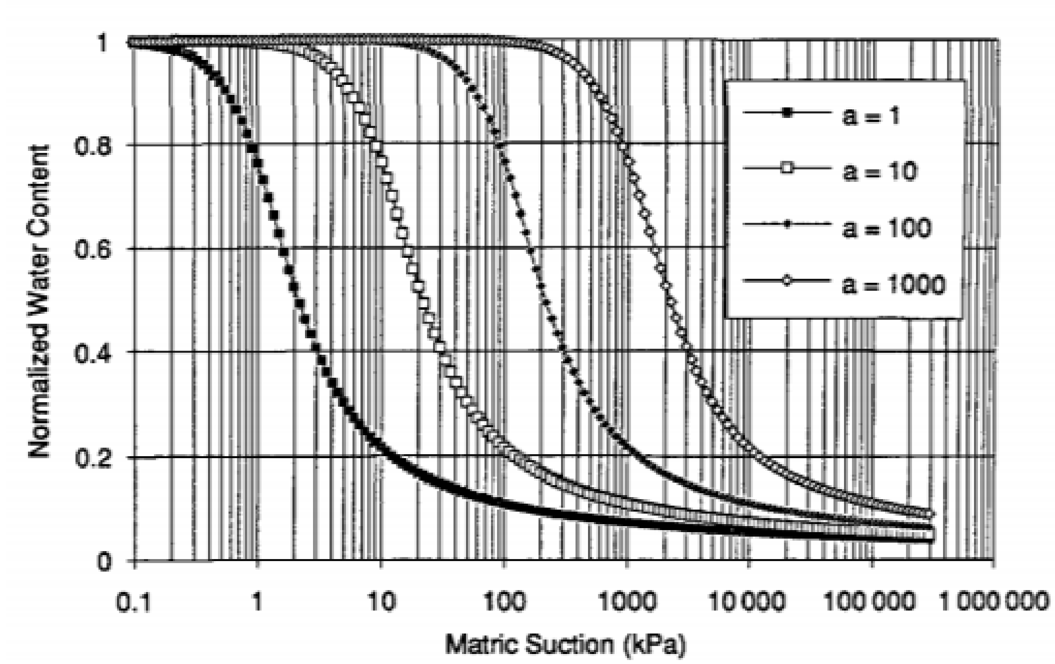

Figure 9. Effect that varying parameter "*a*" (i.e., a_f) has on SWCC shape (6).

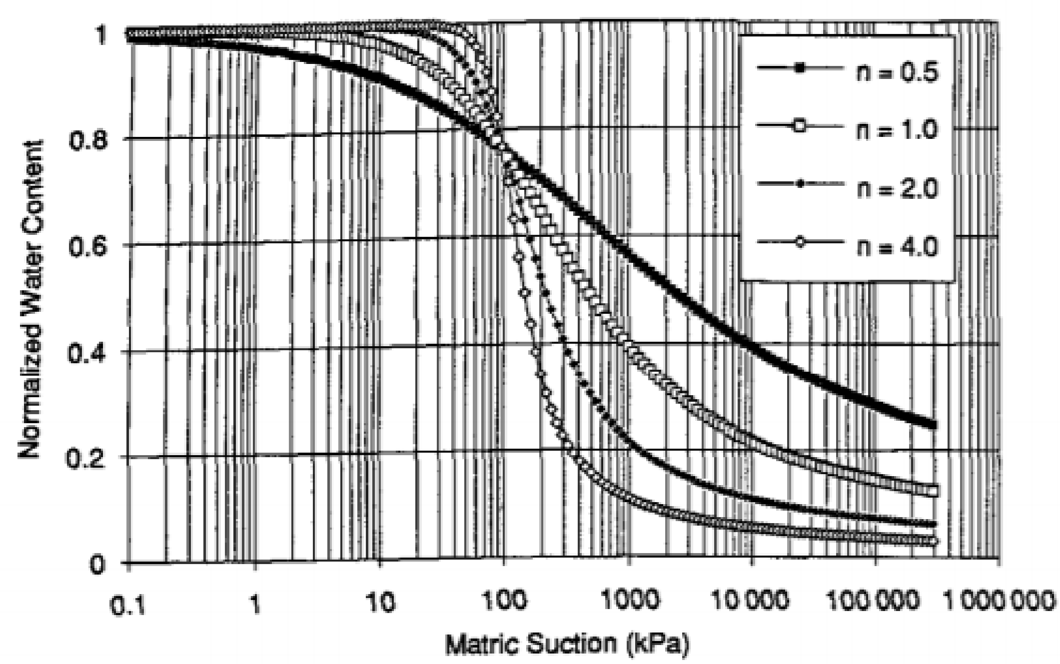

Figure 10. Effect that varying parameter "*n*" (i.e., b_f) has on SWCC shape (6).

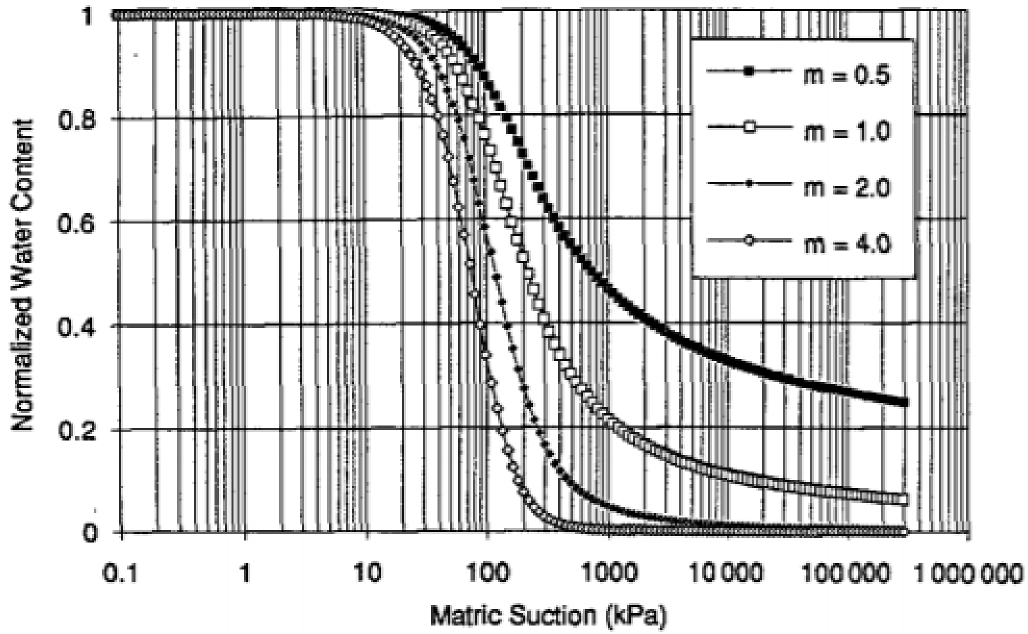

Figure 11. Effect that varying parameter "*m*" (i.e., c_f) has on SWCC shape (6).

SWCC Estimation Methodologies

The MEPDG utilizes the EICM as an engine that can handle the collection, input, characterization, and analysis of environmental and material properties of unbound materials. These properties influence the stiffness of the unbound layers, which in turn affects pavement performances. Climatic data and material properties of the unbound materials are needed for the EICM to predict environmental factors. A database which encompasses over 800 weather stations from the U.S. Weather Service is used to input climatic data. The material properties, however, require tests to be performed on the unbound materials.

Input level 1 for the SWCC input parameter requires direct measurement of matric suction and volumetric water content; however, this type of test is not regularly performed. The time and cost needed in order to perform SWCC testing have limited this type of testing as a part of a regular laboratory soil testing program. Other soil tests are regularly performed, as they take less time to conduct and require less expensive equipment. Therefore, the goal of the SWCC estimation techniques utilized in input levels 2 and 3 is to correlate SWCC model parameters with results from more common soil tests. Input level 2 involves the direct measurement of w_{opt}, γ_{dmax}, Gs, P_{200}, and D_{60}. The EICM then internally calculates $P_{200}*PI$, S_{opt}, θ_{opt}, and θ_{sat}. Based on nonlinear regression analysis, the EICM will internally calculate the SWCC model parameters, using the following equations. Finally, the SWCC is established using the Fredlund and Xing equation.

- Case 1: If $P_{200}PI>0$

$$\mathbf{a_f} = \frac{\mathbf{0.00364(P_{200}PI)^{3.35}+4(P_{200}PI)+11}}{\mathbf{6.895}} \text{ (psi)} \quad [11]$$

$$\frac{\mathbf{b_f}}{\mathbf{c_f}} = \mathbf{-2.313(P_{200}PI)^{0.14} + 5} \quad [12]$$

$$\mathbf{c_f = 0.0514(P_{200}PI)^{0.465} + 0.5} \quad [13]$$

$$\frac{\mathbf{h_r}}{\mathbf{a_f}} = \frac{\mathbf{1}}{\mathbf{D_{60}+9.7e^{-4}}} \quad [14]$$

- Case 2: If $P_{200}PI=0$

$$\mathbf{a_f} = \frac{\mathbf{0.8627(D_{60})^{-0.751}}}{\mathbf{6.895}} \text{ (psi)} \quad [15]$$

$$\bar{\boldsymbol{b}}_{\boldsymbol{f}} = \mathbf{7.5} \quad [16]$$

$$\mathbf{c_f = 0.1772\ln(D_{60}) + 0.7734} \quad [17]$$

$$\frac{\mathbf{h_r}}{\mathbf{a_f}} = \frac{\mathbf{1}}{\mathbf{D_{60}+9.7e^{-4}}} \quad [18]$$

Input level 3 relies on the results from only gradation and Atterberg Limits testing (P_{200}, D_{60}, and PI). The EICM then uses these parameters to first internally calculate G_s, using the correlation between specific gravity (oven dry basis) to $P_{200}*PI$. S_{opt}, w_{opt}, and γ_{dmax} are then internally calculated using additional correlation equations. Using nonlinear regression analysis, the EICM will then calculate the SWCC model parameters using the equations for input level 2. Finally, the SWCC is established using the Fredlund and Xing equation. It should be noted that AASHTOWare Pavement ME requires each of the parameters to be in pound-force per square inch.

SWCC Estimation from PFC and MBV

Part of this current project focused on determining if MBV could be used to reliably predict SWCC for Nevada's unbound materials. The MBV is used to estimate the PFC, which in turn, can be used to estimate the parameters (a_f, b_f, c_f, and h_r) of the SWCC for unbound aggregate materials. Improvements have been made on the traditional MBV test which have made it suitable for both the laboratory and field. As mentioned in the previous section, input levels 2 and 3 of the MEPDG estimate the parameters of the SWCC based on gradation and Atterberg Limits results. However, variability has been found in using these results to predict the SWCC. Therefore, research was conducted by Sahin et al. to estimate the SWCC parameters using MBV and PFC (7).

The study conducted by Sahin et al. started by sampling 20 types of unbound materials from 9 different quarries in Texas. To evaluate possible variability in production processes, two materials were sampled more than once from the same location at different times. SWCC direct measurements were made for each of the materials using the filter paper test, in accordance with standard procedure ASTM D5298. The test involves keeping a soil samples at specified water contents sealed in a container for a week with filter papers, and it measures both matric and total suctions. As time passes, the increase in the mass of the filter paper is measured, and the water content and suction are determined using a filter paper calibration curve. Using the SWCC results from filter plate testing, the fitting parameters (a_f, b_f, c_f, and h_r) were determined.

After an extensive testing program using over 100 samples, it was found that there is a correlation between MBV and PFC. Plotting PFC versus MBV results in a C-shaped curve, shown in figure 12. This curve can be divided into two zones, with the critical point being at an MBV of 7 mg/g. This point shows the separation between materials with plastic and non-plastic fines. Non-plastic soils resulted in an MBV below the critical point, and plastic soils resulted in an MBV above the critical point. This curve is similar to the specific energy diagram that is utilized in fluid dynamics, and the specific energy diagram came from the Bernoulli equation. For the study by Sahin et al., the specific energy diagram was applied to the correlation between MBV and PFC, with fitting parameters *a*, *n*, and *m* that control the shape of the PFC curve. This correlation is represented by equation 19.

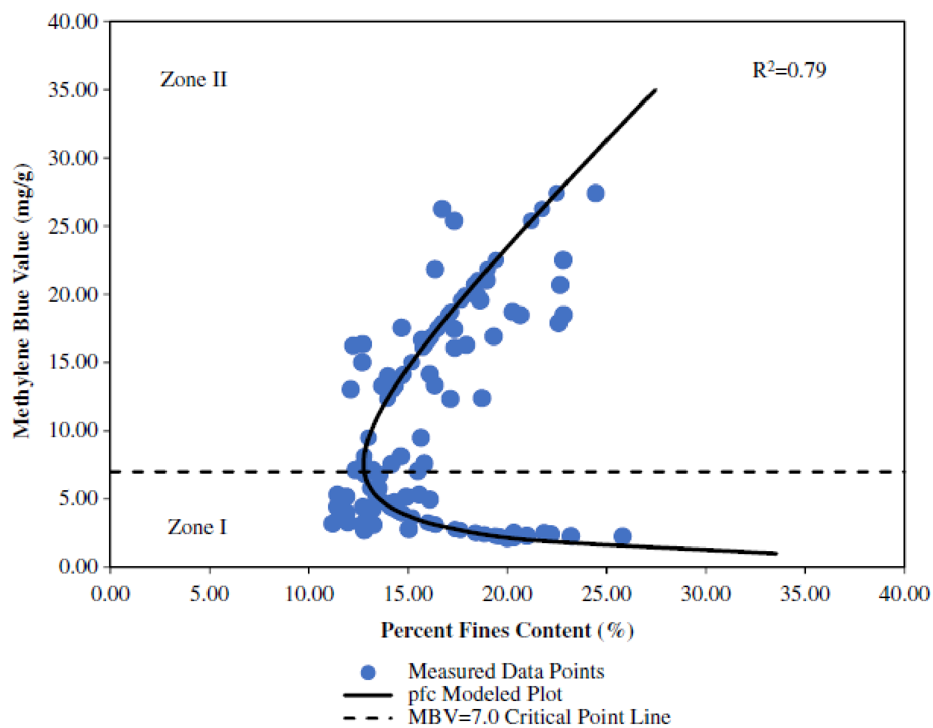

Figure 12. Relationship between MBV and PFC (7).

$$\mathrm{PFC} = \frac{\mathrm{a}}{(\mathrm{MBV})^{\mathrm{n}}} + \mathrm{m}(\mathrm{MBV}) \tag{19}$$

The fitting parameters of the Fredlund and Xing equation using MBV and PFC were found using regression analysis. The four parameters each have its own unique relationship to PFC. The parameter a_f represents the air-entry value, and its relationship to PFC is shown in equation 20. The parameter b_f represents the rate at which water is extracted from the soil, and it is represented by equation 21. The residual water content is represented by parameter c_f, shown in equation 22. Finally, the suction corresponding to the residual content is represented by parameter h_r, and is shown in equation 23.

$$\mathrm{a_f}(\mathrm{psi}) = 0.6384\mathrm{e}^{0.0369(\mathrm{PFC})} \tag{20}$$

$$\mathrm{b_f} = 11.748\mathrm{e}^{-0.037(\mathrm{PFC})} \tag{21}$$

$$\mathrm{c_f} = 0.126\mathrm{e}^{0.0211(\mathrm{PFC})} \tag{22}$$

$$\mathrm{h_r}(\mathrm{psi}) = -0.0018(\mathrm{PFC})^2 + 0.5206(\mathrm{PFC}) + 2.4305 \tag{23}$$

To verify the accuracy of these correlations, the predicted data was compared to the results from the filter paper testing. There were three known points on the SWCC curve. Point A is the volumetric water content at saturation. Point B is the measured suction and volumetric water content. Point C is the highest suction value, which corresponds to zero volumetric water content. These three points are shown by figure 13 from Sahin et al. (7). Through conducting multiple regression analysis, the fitting parameters could be determined. The curve generated by using the equations correlating the fitting parameters to PFC needed to pass through the measured point in order to verify accuracy, and a strong relationship between measured suction values and predicted suction values was found, as shown in figure 14. Figure 15 shows the SWCCs generated for Texas unbound materials using the correlations between the fitted parameters and PFC.

The research conducted by Sahin et al. showed that there is a relationship between MBV and SWCC. In comparison with other methods used to measure the SWCC, using the MBV test, the SWCC can be estimated more efficiently and with less quantity of material.

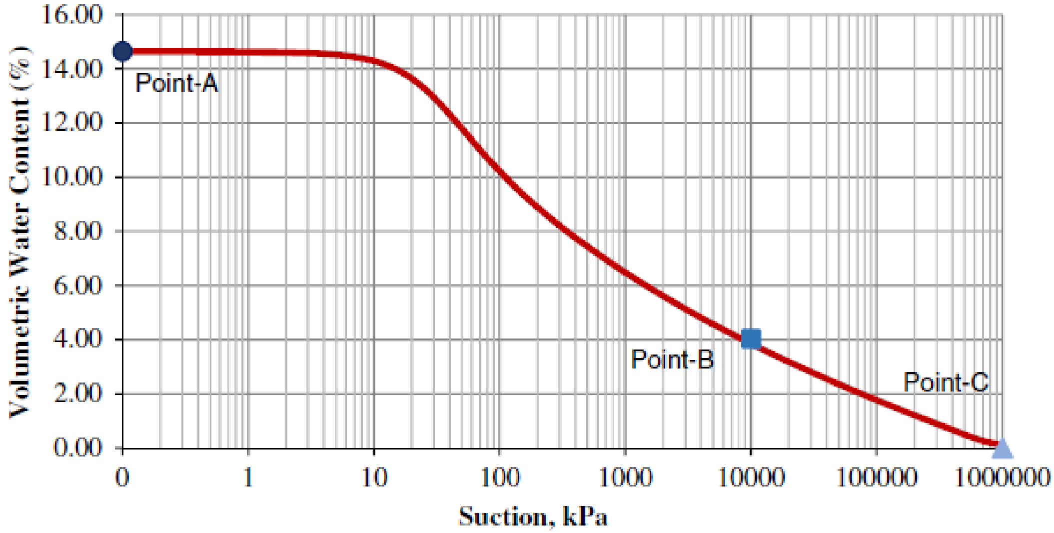

Figure 13. Illustration of method to find SWCC fitting parameters (7).

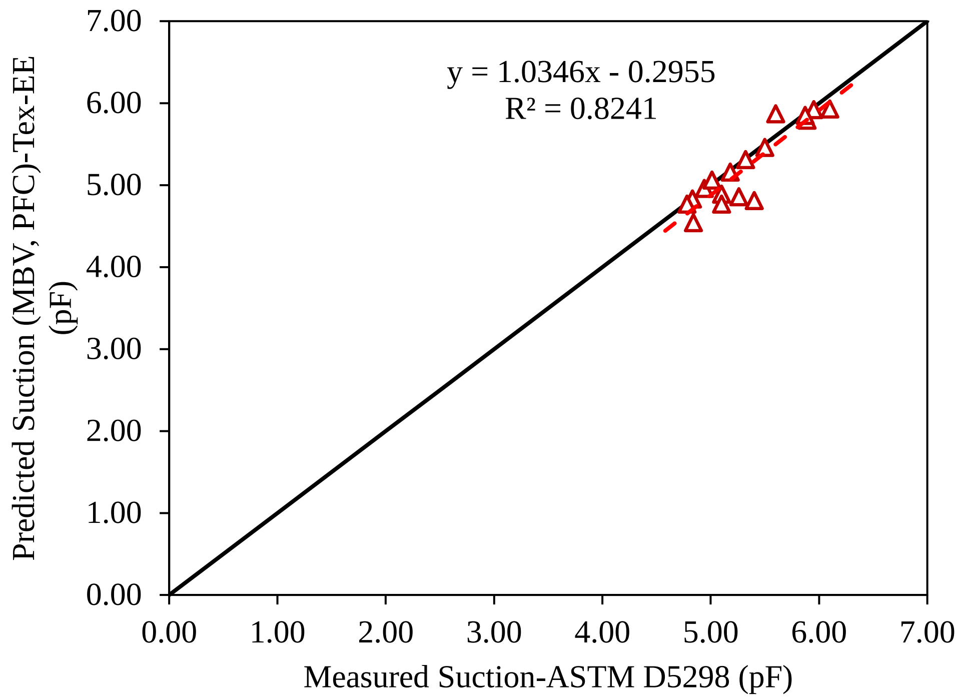

Figure 14. Relationship between measured and predicted suction for Texas' unbound materials (7).

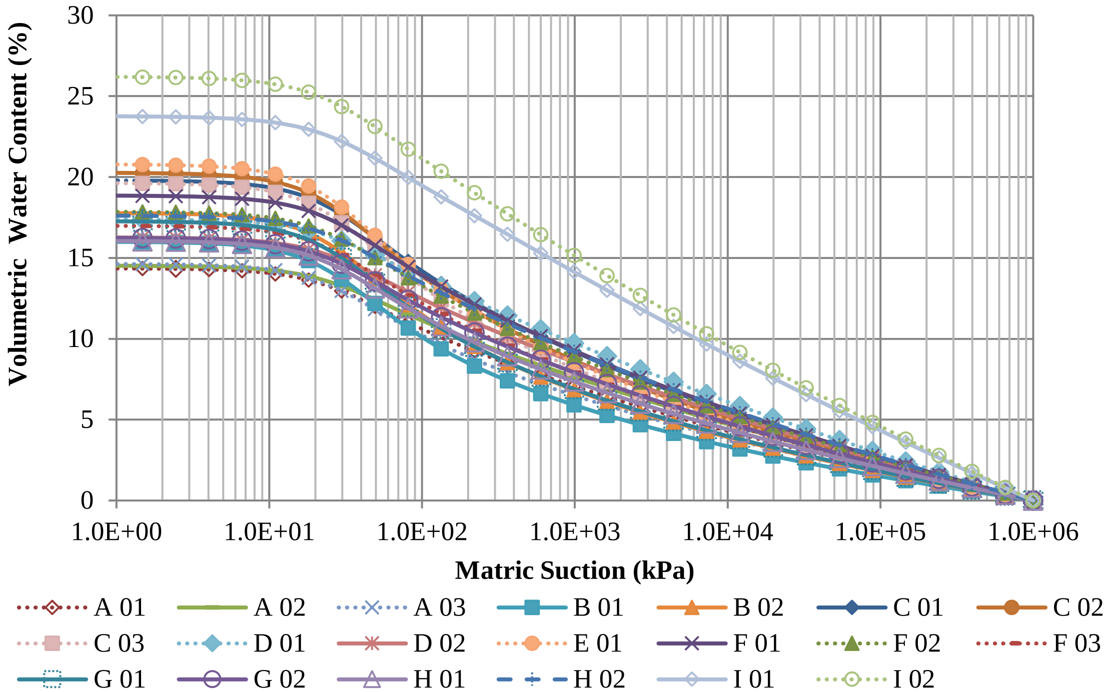

Figure 15. Generated SWCCs for Texas' unbound materials (7).

CHAPTER 2 RESEARCH APPROACH

This chapter describes the procedure followed for the collection of and the tests that were conducted on the sampled base, borrow, and subgrade materials from NDOT Districts 1, 2, and 3. These tests included soil classification, maximum dry unit weight and optimum water content, specific gravity of solids, mastic suction and volumetric water content, MBV, PFC, saturated hydraulic conductivity, and R-value.

UNBOUND MATERIALS DATABASE

One of the tasks of this project was to identify and collect necessary information on commonly used unbound material sources by NDOT. Whenever available, information on Atterberg Limits, gradation, moisture–density, specific gravity of solids, and R-value were collected. To accomplish this task, pavement projects from all three NDOT Districts over the past 15 years were identified and reviewed. Construction reports from the NDOT construction office in Carson City, as well as information from the national database developed under NCHRP 9-23A were collected (4).

NDOT Historical Data Collection

In addition to collecting commonly used base, borrow, and subgrade materials from all three NDOT districts, historical data were also collected from select past projects constructed within the past 15 years. First, to narrow down an extensive list of contracts, only projects that used both base and borrow materials were considered. From this list of projects, the largest projects were chosen. By collecting data from the larger projects, the intent was to collect data on the most commonly used unbound materials in Nevada.

Once these projects were identified, NDOT construction quality assurance test records from these projects were obtained. The data in these records included: gradation, Atterberg Limits, moisture–density, nuclear compaction, and *Gs*. A member of the UNR team traveled to the NDOT main office in Carson City to make copies of these records. These records were analyzed and summarized in an electronic format. A database of this information has been developed in Microsoft Excel.

An example of the gradation data collected is shown in figure 16 and figure 17. This example is from Contract 3585 in District 1. An example of Atterberg limits results are shown in figure 18. These results are for a Type 1 Class B base material from Contract 3585 in District 1.

Figure 19 and figure 20 show an example of moisture–density determination for a Type 1 Class B base material sample from Contract 3585 in District 1. Figure 21 shows the results for a nuclear compaction test. This test is on Type 1 Class B base material from Contract 3585 in District 1. Figure 22 is another moisture–density test record; however, this record also includes an apparent specific gravity measurement. Older moisture–density records had this measurement reported.

ate 8-26-16
age 1 of 3 Pages
eport No. 80
aterial Covered By Report TYPE IB BASE

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

DAILY REPORT OF TESTS MADE IN THE FIELD

Contract No. 3585
Contractor R.H.B.
Spec. Reference 704.03.03
704.02.02

oadway Line	CS							Specification Limit
tation	CS-136+00							
ourse (Lift)	1ST; 6'LT OF BFC							
ource of Sample	WINDROW							
cceptance Test No.	T-1-80							
formation Test No.	—							
SIEVE SIZES	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	
5 mm (3")								
3 mm (2½")								
0 mm (2")								
7.5 mm (1½")	100							100
1.5 mm (1¼")								
5 mm (1")	100							80-100
9 mm (¾")								
2.5 mm (½")								
.5 mm (⅜")								
.75 mm (No. 4)	44							30-65
.36 mm (No. 8)								
.00 mm (No. 10)								
.18 mm (No. 16)	24							15-40
00 µm (No. 30)								
25 µm (No. 40)								
50 µm (No. 100)								
5 µm (No. 200)	8							2-12
quid Limit	N/A							35 MAX
lasticity Index	NP							6 MAX
Fractured Face	99							15% MIN.
Absorption	N/A.							N/A

☐ *Please check box if sieve results are informational*

uantity of Material Represented By Each Test: ONE PER DAY / ONE PER 2000 TONS

ource of Materials: HAYWOOD PIT

ampled By: G. HOUGH

ested By: G. HOUGH & D. KIND

Remarks: TEST MET SPEC'S, INSPECTOR, CONTRACTER, & RE WERE NOTIFIED

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Resident Engineer: DAVID G. TITZEL OCT 12 2016

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BY: JDM

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Figure 16. Historical record gradation example first page.

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION
FIELD MATERIAL SIEVE SHEET

CONTRACT NO. 3585
MATERIAL TYP. IB BASE
DATE 8-26-16

Test No. T-1-
Size or Type TYPE IB BASE Mix %
Sampled by G. HOUGH
Tested by G. HOUGH, & D. KIND
Source WINDROW-CS-136+00, 6'LT. OF BFC 1ST LIFT

Weights-Moisture	Weights-Wash	Weights-Frac. Face
2645	2537	1423
2537	2338	16 NF
108	199	1407 FF

Moisture 4.3 % Frac. Face 99 %
LL N/A PI NP

Sieve Sizes	Weight Ret.	% Ret.	% Pass	Specs. Or % Pass
75 mm (3")				
63 mm (2½")				
50 mm (2")				
37.5 mm (1½")	0	0.0		100
31.5 mm (1¼")				
25 mm (1")	0	0.0	100.0	80-100
19 mm (¾")	29	1.1	98.9	
12.5 mm (½")	490	19.3	79.6	
9.5 mm (⅜")	355	14.0	65.6	
4.75 mm (No. 4)	547	21.5	44.1	30-65
2.36 mm (No. 8)	—	—	—	—
2.00 mm (No. 10)	373	14.7	29.4	
1.18 mm (No. 16)	139	5.5	23.9	15-40
600 µm (No. 30)	—	—	—	—
425 µm (No. 40)	177	7.0	16.9	
300 µm (No. 50)	50	2.0	14.9	
150 µm (No.100)	86	3.4	11.5	
75 µm (No. 200)	81	3.2	8.3	2-12
Pan	11	8.3		
Wash	199			

Test No.
Size or Type Mix %
Sampled by
Tested by
Source

Weights-Moisture	Weights-Wash	Weights-Frac. Face

Moisture % Frac. Face %
LL PI

Sieve Sizes	Weight Ret.	% Ret.	% Pass	Specs. Or % Pass
75 mm (3")				
63 mm (2½")				
50 mm (2")				
37.5 mm (1½")				
31.5 mm (1¼")				
25 mm (1")				
19 mm (¾")				
12.5 mm (½")				
9.5 mm (⅜")				
4.75 mm (No. 4)				
2.36 mm (No. 8)				
2.00 mm (No. 10)				
1.18 mm (No. 16)				
600 µm (No. 30)				
425 µm (No. 40)				
300 µm (No. 50)				
150 µm (No.100)				
75 µm (No. 200)				
Pan				
Wash				

Test No.
Size or Type Mix %
Sampled by
Tested by
Source

Weights-Moisture	Weights-Wash	Weights-Frac. Face

Moisture % Frac. Face %
LL PI

Sieve Sizes	Weight Ret.	% Ret.	% Pass	Specs. Or % Pass	Sum of %'s used	Specs. Or % Pass
75 mm (3")						
63 mm (2½")						
50 mm (2")						
37.5 mm (1½")						
31.5 mm (1¼")						
25 mm (1")						
19 mm (¾")						
12.5 mm (½")						
9.5 mm (⅜")						
4.75 mm (No. 4)						
2.36 mm (No. 8)						
2.00 mm (No. 10)						
1.18 mm (No. 16)						
600 µm (No. 30)						
425 µm (No. 40)						
300 µm (No. 50)						
150 µm (No.100)						
75 µm (No. 200)						
Pan						
Wash						

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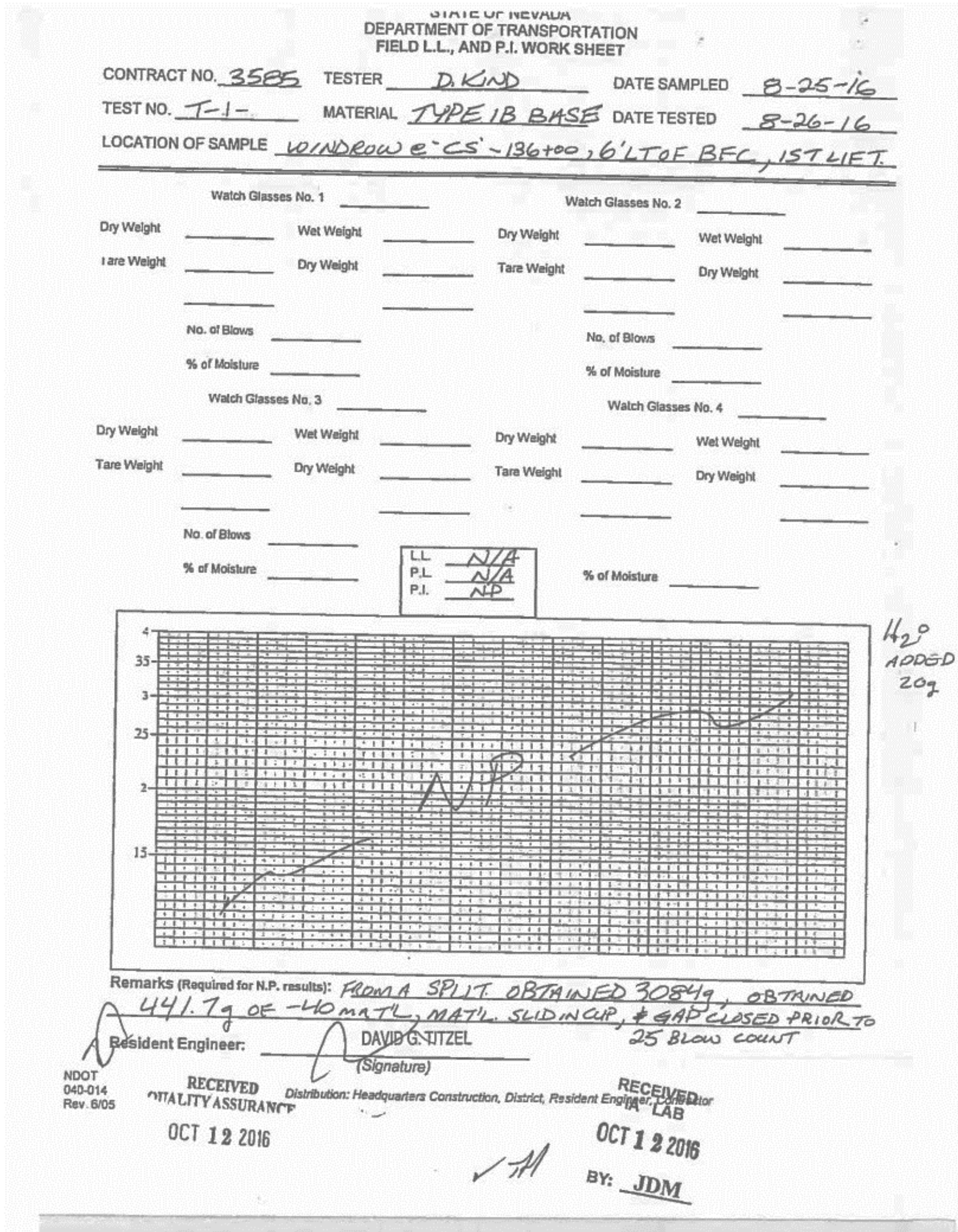
Figure 17. Historical record gradation example second page.


STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION
FIELD L.L., AND P.I. WORK SHEET

CONTRACT NO. 3585 TESTER D. KIND DATE SAMPLED 8-25-16

TEST NO. T-1- MATERIAL TYPE 1B BASE DATE TESTED 8-26-16

LOCATION OF SAMPLE WINDROW @ "CS" ~ 136+00, 6' LT OF BFC, 1ST LIFT.

Watch Glasses No. 1 Watch Glasses No. 2

Dry Weight Wet Weight Dry Weight Wet Weight

Tare Weight Dry Weight Tare Weight Dry Weight

No. of Blows No. of Blows

% of Moisture % of Moisture

Watch Glasses No. 3 Watch Glasses No. 4

Dry Weight Wet Weight Dry Weight Wet Weight

Tare Weight Dry Weight Tare Weight Dry Weight

No. of Blows

% of Moisture % of Moisture

L.L. N/A
P.L. N/A
P.I. NP

Remarks (Required for N.P. results): FROM A SPLIT OBTAINED 3084g, OBTAINED 441.7g OF -40 MAT'L, MAT'L SLID IN CUP, & GAP CLOSED PRIOR TO 25 BLOW COUNT

Resident Engineer: DAVID G. TITZEL (Signature)

NDOT 040-014 Rev. 6/05

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Figure 18. Historical record Atterberg limits example.

STATE OF NEVADA DEPARTMENT OF TRANSPORTATION
MOISTURE-DENSITY DETERMINATION, COMPACTION REPORT - PART 1

Contract No. 3585
Date 10-6-16

Test Hole No. (code)	424 1B-95		
Station	SI w27425		
Distance to Centerline	81' RT		
Embankment Depth	Finish Grade		
Type of Material	Type I Class B Base		
Compaction Equipment	Steel Drum Roller		

Sand Volume Data Nev. T102

1. Initial Wt. of Sand	kg (lbs)	50.0		
2. Wt. of Residue	kg (lbs)	15.5		
3. Wt. of Sand Used, 1 - 2	kg (lbs)	34.5		
4. Sand Density	Mg/m³ (lb/ft³)	81.6		
5. Vol. Hole + Cone + Plate, 3 / 4	m³ x 1000 (ft³)	.423		
6. Vol. of Cone + Plate	m³ x 1000 (ft³)	.167		
7. Vol. of Hole, 5 - 6	m³ x 1000 (ft³)	.256		

Excavated Sample Nev. T102

8. Total Excavated Wet Wt.	kg (lbs)	37.2		
9. Wet Wt.	grams	2297		
10. Dry Wt.	grams	2142		
11. Wt. of Water, 9 - 10	grams	155		
12. % Moisture, (11 / 10)100		7.2		
13. Field Wet Density, 8 / 7	Mg/m³ (lb/ft³)	145.3		
14. Field Dry Density, [13 / (100 + % Moisture)]100	Mg/m³ (lb/ft³)	135.5		

Data for Calculated Maximum Density Determination Nev. T108

Total Sample Wt. 2142
Wt. of + 3/4" 62
Wt. of + No. 4 ______

2.9 % retained on +3/4", ______ % retained on +No.4

(Method Selected) Method "A" ☐ Method "D" ☒

		Point 1	Point 2	Point 3	Point 4	Point 5
16. Wt. of Mold + Compacted Soil	grams	11213	11472	11492		
17. Wt. of Mold	grams	6429	6429	6429		
18. Wt. of Compacted Soil, 16 - 17	grams	4784	5043	5063		
19. Wet Density, (Line 18 / 453.59) x Factor	(lb/ft³)	140.6	148.2	148.8		
20. Dry Density, (Line 19 / (100 + % Moist.))100	(lb/ft³)	* 130.9	135.7	133.8		

Note: Mold Volume Factor, F = (1 / Volume)
For Method "A" (4" Mold) use 30.03
For Method "D" (6" Mold) use 13.33

Wet Density = (Wt. of Compacted Soil / 453.59) x Factor
Dry Density = (Wet Density / (100 + % Moisture)) x 100

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Moisture Content, Nev. T112 or T108 (Microwave Oven Procedure)

		Point 1	Point 2	Point 3	Point 4	Point 5
21. Wet Wt.	grams	1080.4	1035.8	779.2		
22. Dry Wt.	grams	1005.6	948.6	700.4		
23. Wt. of Water, 21 - 22	grams	74.8	87.2	78.3		
24. % Moisture, (23 / 22)100		7.4	9.2	11.2		

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* Oven-dry density points shall be plotted as ordinates (y-axis) and corresponding moisture contents as absicca (x-axis)

√DK

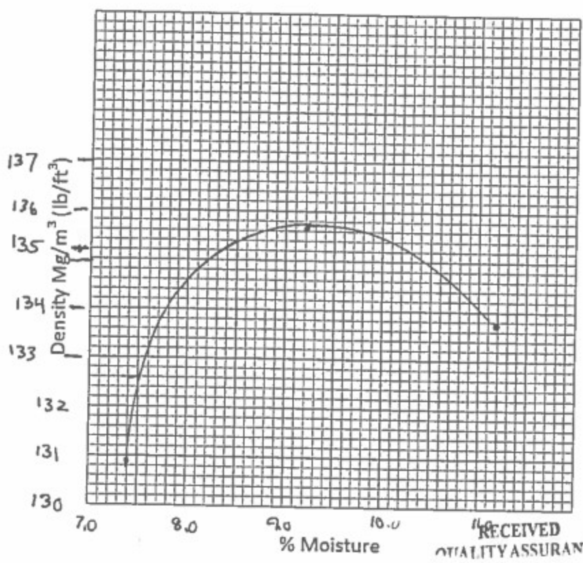
Figure 19. Historical record moisture–density example first page.

STATE OF NEVADA DEPARTMENT OF TRANSPORTATION
MOISTURE-DENSITY DETERMINATION, COMPACTION REPORT - PART 2

Oversized Aggregate Correction, Nev. T104

Apply to Method "A" if aggregate particles exceed 5% by mass retained on the 4.75 mm (No. 4) sieve
Apply to Method "D" if aggregate particles exceed 5% by mass retained on the 19.0 mm (3/4 in.) sieve

25. Wt. of Aggregate = A			
26. Wt. of Pycnometer = p			
27. (Wt. of Pyc. + Water) - p = M	M =	M =	M =
28. (Wt. of Pyc. + Water + Aggregate) - (p + A) = M_1	M_1 =	M_1 =	M_1 =
29. Wt. of Water Recovered from Pyc. = M_2	M_2 =	M_2 =	M_2 =
30. If $M_1 - M_2 \leq 14$ g, $G = [A / (M - M_1)] \times 62.4$			
31. If $M_1 - M_2 > 14$ g, $G = [A / [(M - M_1) + 0.5 (M_1 - M_2 - 14 g)]] \times 62.4$			
32. Decimal Equiv. of % + No.4 or + 3/4", (1 - P)	.029		
33. Decimal Equiv. of % - No. 4 or - 3/4", P	.971		
34. Maximum Dry Density = d (lb/ft³)	135.7		
35. Corrected Max. Dry Density $D = dG / [(d)(1 - P) + (G)(P)]$ (lb/ft³)	N/A		
36. % Compaction = (14 / 35)100	100		



As Plotted

Max. Dry Density (d)	135.7 Mg/m³ (lb/ft³)
Optimum Moisture	9.2 %

Corrected Max. Density and Opt. Moisture Utilizing the Coarse Aggregate Correction

Max. Dry Density (D)	N/A Mg/m³ (lb/ft³)
Optimum Moisture	%

Corrected Optimum Moisture Calculation:
[(1-P) x 2] + [P x "As Plotted" Opt. Moisture]

Note: For Metric, lb/ft³ / 62.4 = Mg/m³

Determine Decimal Equiv. of % ± No.4 or ± 3/4"
(Round to nearest 0.001)

37. Sieve Test No.	
38. Total Sample Wt.	2142
39. Wt. of + No.4 or + 3/4"	62
40. Dec. Equiv. (1-P), 39 / 38	.029
41. Wt. of - No.4 or - 3/4"	2080
42. Dec. Equiv. (P), 41 / 38	.971

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REMARKS: Test meets spec. Contractor, Inspector and RE notified of result

RECEIVED
IA LAB
NOV 02 2016

TESTED BY: G. Haugh

RESIDENT ENGINEER: DAVID G. TITZEL

NDOT
040-069
Rev. 02/13

Distribution: Headquarters Construction, District, Resident Engineer, Contractor

BY: JDM

Figure 20. Historical record moisture–density example second page.

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STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

NUCLEAR COMPACTION REPORT FOR SOILS AND AGGREGATES

Daily Standard Counts	
Density	Moisture
1812	1105

Contract No. 3585
Date 7-8-16
Nuclear Set No. BFC389
Correlation Date 3-29-16

Test No. (code)	290-1B-95		291-1B-95		292-1B-95	
Station	L~111+00		L~111+00		L~131+00	
Distance to Centerline	60' Rt		24' Rt		61' Rt	
Embankment Depth	19" above S.G.		19" above S.G.		19" above S.G.	
Type of Material	Type I class B Base		Type I class B Base		Type I class B Base	
Compaction Equipment	Dbl steel Drum		Dbl steel Drum		Dbl steel Drum	
WET DENSITY						
Probe Depth	8"		8"		8"	
Counts	2086 — 2093		1986 — 1938		2144 — 2090	
Wet Density #1 Mg/m³ (lb/ft³)	135.5		138.0		133.7	
Wet Density #2 Mg/m³ (lb/ft³)	135.4		138.6		134.7	
Wet Density #3 Mg/m³ (lb/ft³)						
Wet Density #4 Mg/m³ (lb/ft³)						
Average Wet Density Mg/m³ (lb/ft³)	135.5		138.3		134.2	
MOISTURE AND MOISTURE %						
Counts	153 — 153		120 — 126		125 — 132	
Moisture #1 Mg/m³ (lb/ft³) / M %	7.4	5.8	5.7	4.3	5.9	4.7
Moisture #2 Mg/m³ (lb/ft³) / M %	7.4	5.8	6.0	4.6	6.3	4.9
Moisture #3 Mg/m³ (lb/ft³) / M %						
Moisture #4 Mg/m³ (lb/ft³) / M %						
Avg. Moisture Mg/m³ (lb/ft³) / Avg. M %	7.4	5.8	5.9	4.5	6.1	4.8
CORRELATION DATA						
Moisture Offset (MCF)	1.88		1.88		1.88	
Wet Density Offset	2.1		2.1		2.1	
MODIFIED PROCTOR COMPACTION TEST RESULTS						
Test No.	285-1B-95		285-1B-95		285-1B-95	
Maximum Dry Density = (d) Mg/m³ (lb/ft³)	135.5		135.5		135.5	
Optimum Moisture %	8.1		8.1		8.1	
(if applicable)						
Corr. Max. Dry Density = (D) Mg/m³ (lb/ft³)	N/A		N/A		N/A	
Corr. Optimum Moisture %						
DRY DENSITY						
Dry Density #1 Mg/m³ (lb/ft³)	128.1		132.3		127.7	
Dry Density #2 Mg/m³ (lb/ft³)	128.0		132.5		128.4	
Dry Density #3 Mg/m³ (lb/ft³)						
Dry Density #4 Mg/m³ (lb/ft³)						
Average Dry Density Mg/m³ (lb/ft³)	128.1		132.4		128.1	
% Compaction Average	~~64.28.1~~ 95		98		95	
Accepted or Rejected	Accepted		Accepted		Accepted	

Remarks: Tests meet spec. contractor, Inspector and RE Notified of results

Tested By: Glenn Hough

Resident Engineer:

DAVID RECEIVED
IA LAB
AUG 0 9 2016
BY: JDM

VTH

NDOT
040-007
Rev. 07/14

Distribution: Headquarters Construction, District, Resident Engineer, Contractor

Figure 21. Historical record nuclear compaction example.

STATE OF NEVADA DEPARTMENT OF TRANSPORTATION

COMPACTION REPORT

Contract No. 3532

Date 10/2/13

Test Hole No. (code)	7-S-90		
Station	"L1" 883+25		
Distance to Centerline	45' Rt		
Embankment Depth	Subgrade		
Type of Material	Original Ground		
Compaction Equipment	Steel wheel		

OCT 24 2013

SAND VOLUME DATA

1. Initial Weight of Sand	kg (lbs)	40.00		
2. Weight of Residue	kg (lbs)	10.52		
3. Weight of Sand Used, 1 - 2	kg (lbs)	29.48		
4. Sand Density	Mg/m³ (pcf)	81.4		
5. Vol. Hole + Cone + Plate, 3 / 4	m³ x 1000 (ft³)	.362		
6. Vol. of Cone + Plate	m³ x 1000 (ft³)	.174		
7. Vol. of Hole, 5 - 6	m³ x 1000 (ft³)	.188		

EXCAVATED SAMPLE

		Part (g)	Total kg (lbs)	Part (g)	Total kg(lbs)	Part (g)	Total kg(lbs)
8. Wet Weight	g	2578	25.06				
9. Dry Weight	g	2506					
10. Weight of Water, 8 - 9	g	72					
11. % Moisture (10 / 9)100		2.9 %					
12. Dry Weight + 4.75 mm (+ #4)	g	1244					
13. Dry Weight - 4.75 mm (- #4)	g	1262					
14. FIELD DENSITY WET, 8 Total / 7	Mg/m³ (pcf)	133.3					
15. FIELD DENSITY DRY, [14 / (100 + % MOISTURE)]100	Mg/m³ (pcf)	129.5					

DATA FOR THE CALCULATED MAXIMUM DENSITY DETERMINATION NEV T101

Harvard Miniature Curve Test on - 4.75 mm (- #4)

Note: For Metric, grams / 62.4 = Mg/m³
For English, grams = pcf

	Wet	Dry	%M	Wet	Dry	%M	Wet	Dry	%M
1.	116.0	111.1	4.4 %						
2.	127.5	119.5	6.7 %						
3. *	140.0	128.8	8.7 %						
4.	138.1	124.1	11.3 %						
5.									

16. Total Sample, approx. Opt. Moisture [(1 - P) x 2] + [P x Test Opt. Moist.] — 5.4 %

Apparent Specific Gravity Test, G, Nev. T104

17. Wt. of Aggregate, (4.75 mm to 37.5 mm)(#4 to 1.5") = A	A = 500.0					
18. Wt. of Pycnometer = p	p = 441.5					
19. (Wt. of Pyc. + Water) - p = M	1444.0	M = 1002.5		M =		M =
20. (Wt. of Pyc., Water, Aggregate) - (p + A) = M_1	1755.3	M_1 = 813.8		M_1 =		M_1 =
21. Wt. of Water Recovered from Pyc. = M_2		M_2 = 801.7		M_2 =		M_2 =
22. If $M_1 - M_2 \le 14$ g, $G = A / (M - M_1)$		G = 2.65		G =		G =
23. If $M_1 - M_2 > 14$ g, $G = A / [(M - M_1) + 0.5 (M_1 - M_2 - 14)]$		G =		G =		G =

24. Constant, K, (.90 Metric) (56.16 English)		K = 56.16		
25. Decimal Equiv. of % + 4.75 mm (+ #4), (1 - P) = 12 / 9		1-P = .496		
26. Decimal Equiv. of % - 4.75 mm (- #4), P = 13 / 9		P = .504		
27. H.M.C.T., Dry = d	Mg/m³ (pcf)	d = 128.8		
28. CALCULATED MAX. DENSITY D = GK (1- P) + Pd (1.03)	Mg/m³ (pcf)	D = 140.7		
29. % COMPACTION = (15 / 28)100		92 %		

REMARKS: * H.M.C.T. failed to gain 1.4 grams & had excessive penetration; Test meets specs for minimum percent Compaction; Inspector notified

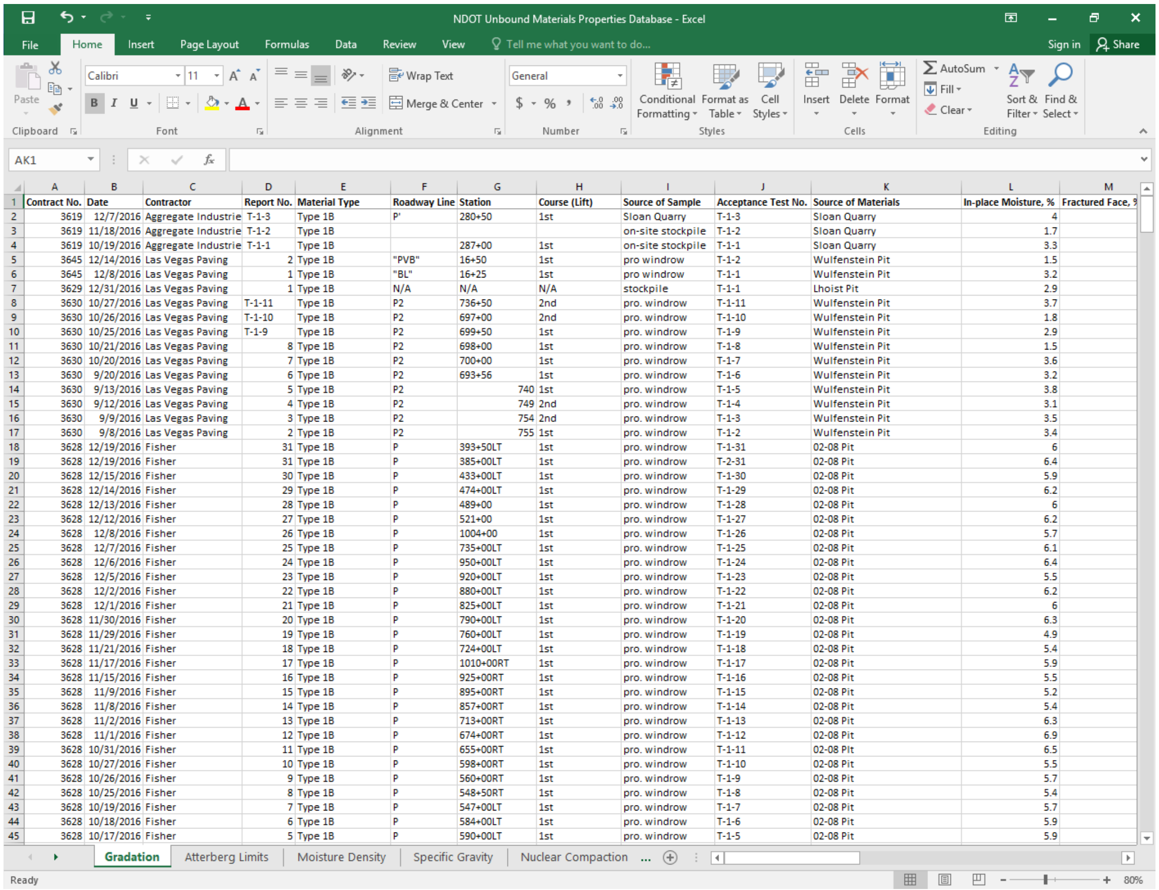
TESTED BY: Kalling / Parr / Lopez RESIDENT ENGINEER: [signature]

NDOT
040-004
Rev. 11/08

Distribution: Headquarters Construction, District, Resident Engineer, Contractor

Figure 22. Historical record apparent specific gravity example.

A screenshot of the comprehensive database for unbound materials in Nevada is shown in figure 23. Within the Excel file, there is a sheet for historical gradation, Atterberg limits, moisture–density, apparent specific gravity, and nuclear compaction data. In Chapter 6, a discussion on the use of this database to find representative values for each NDOT District is presented.



Contract No.	Date	Contractor	Report No.	Material Type	Roadway Line	Station	Course (Lift)	Source of Sample	Acceptance Test No.	Source of Materials	In-place Moisture, %	Fractured Face,
3619	12/7/2016	Aggregate Industrie	T-1-3	Type 1B	P'	280+50	1st	Sloan Quarry	T-1-3	Sloan Quarry	4	
3619	11/18/2016	Aggregate Industrie	T-1-2	Type 1B				on-site stockpile	T-1-2	Sloan Quarry	1.7	
3619	10/19/2016	Aggregate Industrie	T-1-1	Type 1B		287+00	1st	on-site stockpile	T-1-1	Sloan Quarry	3.3	
3645	12/14/2016	Las Vegas Paving	2	Type 1B	"PVB"	16+50	1st	pro windrow	T-1-2	Wulfenstein Pit	1.5	
3645	12/8/2016	Las Vegas Paving	1	Type 1B	"BL"	16+25	1st	pro windrow	T-1-1	Wulfenstein Pit	3.2	
3629	12/31/2016	Las Vegas Paving	1	Type 1B	N/A	N/A	N/A	stockpile	T-1-1	Lhoist Pit	2.9	
3630	10/27/2016	Las Vegas Paving	T-1-11	Type 1B	P2	736+50	2nd	pro. windrow	T-1-11	Wulfenstein Pit	3.7	
3630	10/26/2016	Las Vegas Paving	T-1-10	Type 1B	P2	697+00	2nd	pro. windrow	T-1-10	Wulfenstein Pit	1.8	
3630	10/25/2016	Las Vegas Paving	T-1-9	Type 1B	P2	699+50	1st	pro. windrow	T-1-9	Wulfenstein Pit	2.9	
3630	10/21/2016	Las Vegas Paving	8	Type 1B	P2	698+00	1st	pro. windrow	T-1-8	Wulfenstein Pit	1.5	
3630	10/20/2016	Las Vegas Paving	7	Type 1B	P2	700+00	1st	pro. windrow	T-1-7	Wulfenstein Pit	3.6	
3630	9/20/2016	Las Vegas Paving	6	Type 1B	P2	693+56	1st	pro. windrow	T-1-6	Wulfenstein Pit	3.2	
3630	9/13/2016	Las Vegas Paving	5	Type 1B	P2	740	1st	pro. windrow	T-1-5	Wulfenstein Pit	3.8	
3630	9/12/2016	Las Vegas Paving	4	Type 1B	P2	749	2nd	pro. windrow	T-1-4	Wulfenstein Pit	3.1	
3630	9/9/2016	Las Vegas Paving	3	Type 1B	P2	754	2nd	pro. windrow	T-1-3	Wulfenstein Pit	3.5	
3630	9/8/2016	Las Vegas Paving	2	Type 1B	P2	755	1st	pro. windrow	T-1-2	Wulfenstein Pit	3.4	
3628	12/19/2016	Fisher	31	Type 1B	P	393+50LT	1st	pro. windrow	T-1-31	02-08 Pit	6	
3628	12/19/2016	Fisher	31	Type 1B	P	385+00LT	1st	pro. windrow	T-2-31	02-08 Pit	6.4	
3628	12/15/2016	Fisher	30	Type 1B	P	433+00LT	1st	pro. windrow	T-1-30	02-08 Pit	5.9	
3628	12/14/2016	Fisher	29	Type 1B	P	474+00LT	1st	pro. windrow	T-1-29	02-08 Pit	6.2	
3628	12/13/2016	Fisher	28	Type 1B	P	489+00	1st	pro. windrow	T-1-28	02-08 Pit	6	
3628	12/12/2016	Fisher	27	Type 1B	P	521+00	1st	pro. windrow	T-1-27	02-08 Pit	6.2	
3628	12/8/2016	Fisher	26	Type 1B	P	1004+00	1st	pro. windrow	T-1-26	02-08 Pit	5.7	
3628	12/7/2016	Fisher	25	Type 1B	P	735+00LT	1st	pro. windrow	T-1-25	02-08 Pit	6.1	
3628	12/6/2016	Fisher	24	Type 1B	P	950+00LT	1st	pro. windrow	T-1-24	02-08 Pit	6.4	
3628	12/5/2016	Fisher	23	Type 1B	P	920+00LT	1st	pro. windrow	T-1-23	02-08 Pit	5.5	
3628	12/2/2016	Fisher	22	Type 1B	P	880+00LT	1st	pro. windrow	T-1-22	02-08 Pit	6.2	
3628	12/1/2016	Fisher	21	Type 1B	P	825+00LT	1st	pro. windrow	T-1-21	02-08 Pit	6	
3628	11/30/2016	Fisher	20	Type 1B	P	790+00LT	1st	pro. windrow	T-1-20	02-08 Pit	6.3	
3628	11/29/2016	Fisher	19	Type 1B	P	760+00LT	1st	pro. windrow	T-1-19	02-08 Pit	4.9	
3628	11/21/2016	Fisher	18	Type 1B	P	724+00LT	1st	pro. windrow	T-1-18	02-08 Pit	5.4	
3628	11/17/2016	Fisher	17	Type 1B	P	1010+00RT	1st	pro. windrow	T-1-17	02-08 Pit	5.9	
3628	11/15/2016	Fisher	16	Type 1B	P	925+00RT	1st	pro. windrow	T-1-16	02-08 Pit	5.5	
3628	11/9/2016	Fisher	15	Type 1B	P	895+00RT	1st	pro. windrow	T-1-15	02-08 Pit	5.2	
3628	11/8/2016	Fisher	14	Type 1B	P	857+00RT	1st	pro. windrow	T-1-14	02-08 Pit	5.4	
3628	11/2/2016	Fisher	13	Type 1B	P	713+00RT	1st	pro. windrow	T-1-13	02-08 Pit	6.3	
3628	11/1/2016	Fisher	12	Type 1B	P	674+00RT	1st	pro. windrow	T-1-12	02-08 Pit	6.9	
3628	10/31/2016	Fisher	11	Type 1B	P	655+00RT	1st	pro. windrow	T-1-11	02-08 Pit	6.5	
3628	10/27/2016	Fisher	10	Type 1B	P	598+00RT	1st	pro. windrow	T-1-10	02-08 Pit	5.5	
3628	10/26/2016	Fisher	9	Type 1B	P	560+00RT	1st	pro. windrow	T-1-9	02-08 Pit	5.7	
3628	10/25/2016	Fisher	8	Type 1B	P	548+50RT	1st	pro. windrow	T-1-8	02-08 Pit	5.4	
3628	10/19/2016	Fisher	7	Type 1B	P	547+00LT	1st	pro. windrow	T-1-7	02-08 Pit	5.7	
3628	10/18/2016	Fisher	6	Type 1B	P	584+00LT	1st	pro. windrow	T-1-6	02-08 Pit	5.9	
3628	10/17/2016	Fisher	5	Type 1B	P	590+00LT	1st	pro. windrow	T-1-5	02-08 Pit	5.9	

Figure 23. Screenshot of NDOT database for historical records.

NATIONAL CATALOG OF NATURAL SUBGRADE PROPERTIES

As part of the NCHRP 9-23A project, a map was developed by researchers at Arizona State University (4). This map shows varying soil types by location after inputting the latitude and longitude of the targeted location. Once the location is identified, several soil properties, varying with depth, are shown, including saturated hydraulic conductivity and SWCC parameters.

Figure 24 shows the map generated for Nevada in Google Earth. By zooming in on the section where the report is needed, the map identification code can be found. This code is then used as an input to run a query in the soil survey database, shown in figure 25. Once the identification information is filled in, a query can be run, which generates a report. A printable version of the generated report is shown in figure 26. To request this database visit: http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=3050.

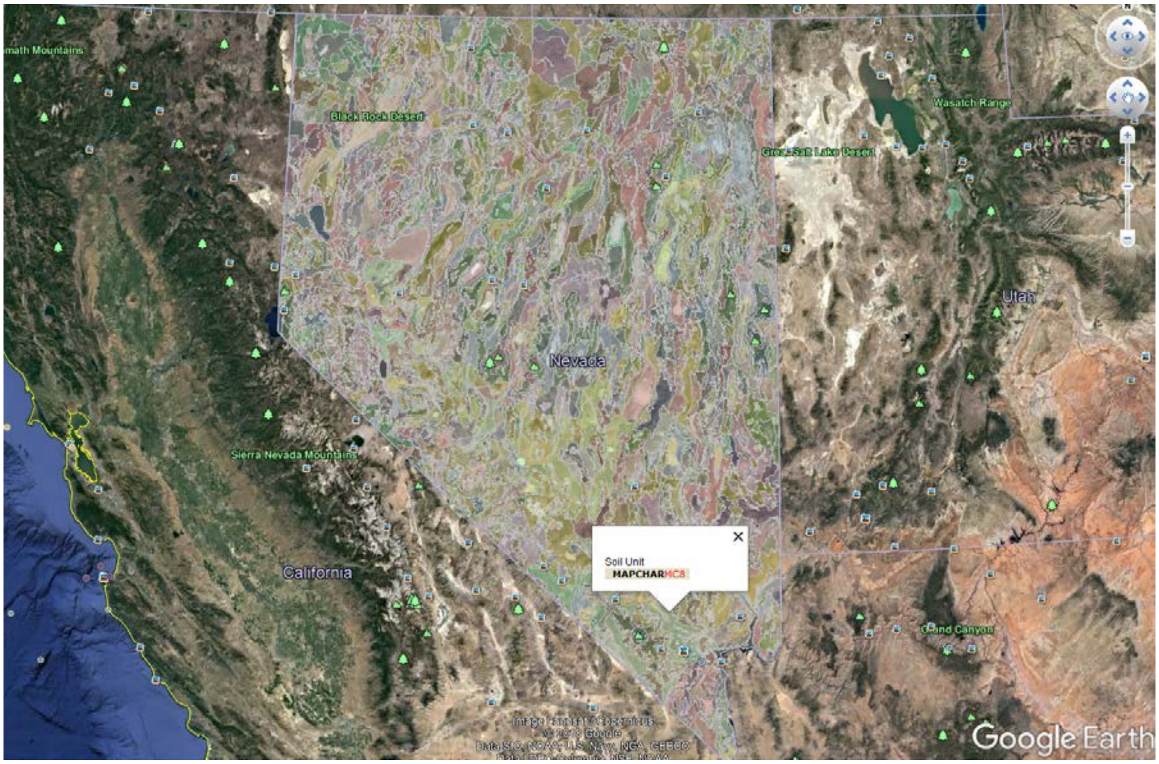


Figure 24. Soil map for Nevada shown in Google Earth.

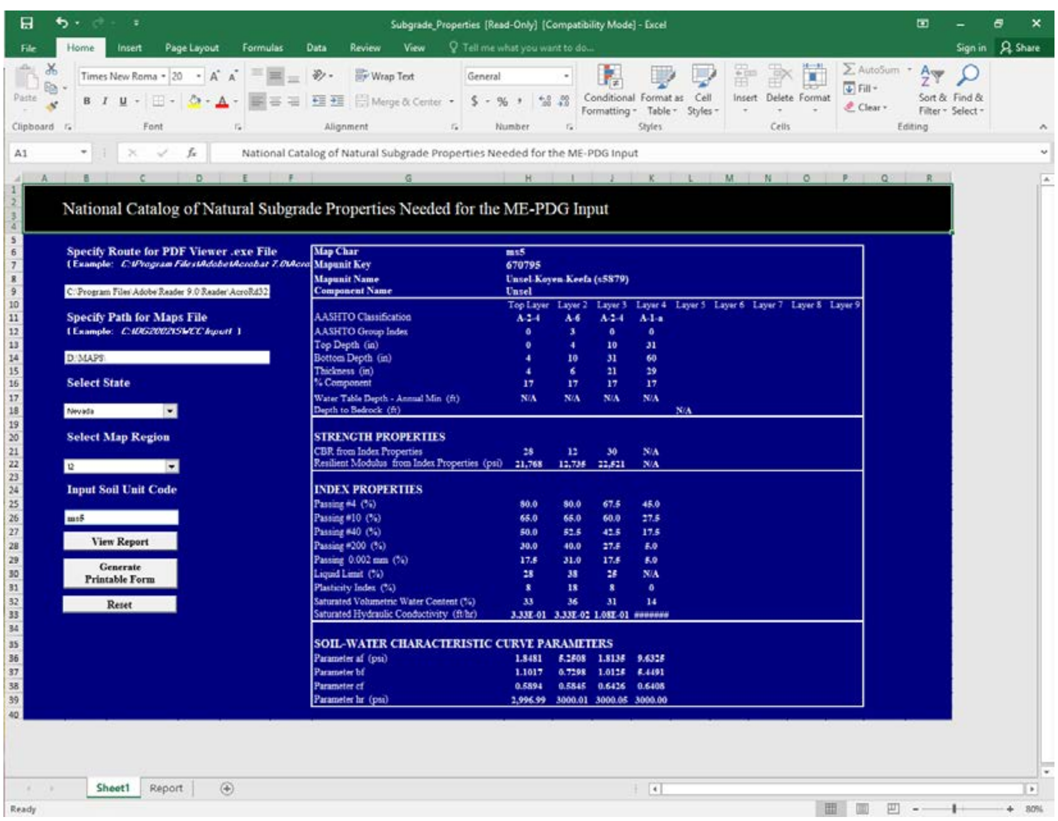


Figure 25. Soil survey database query.

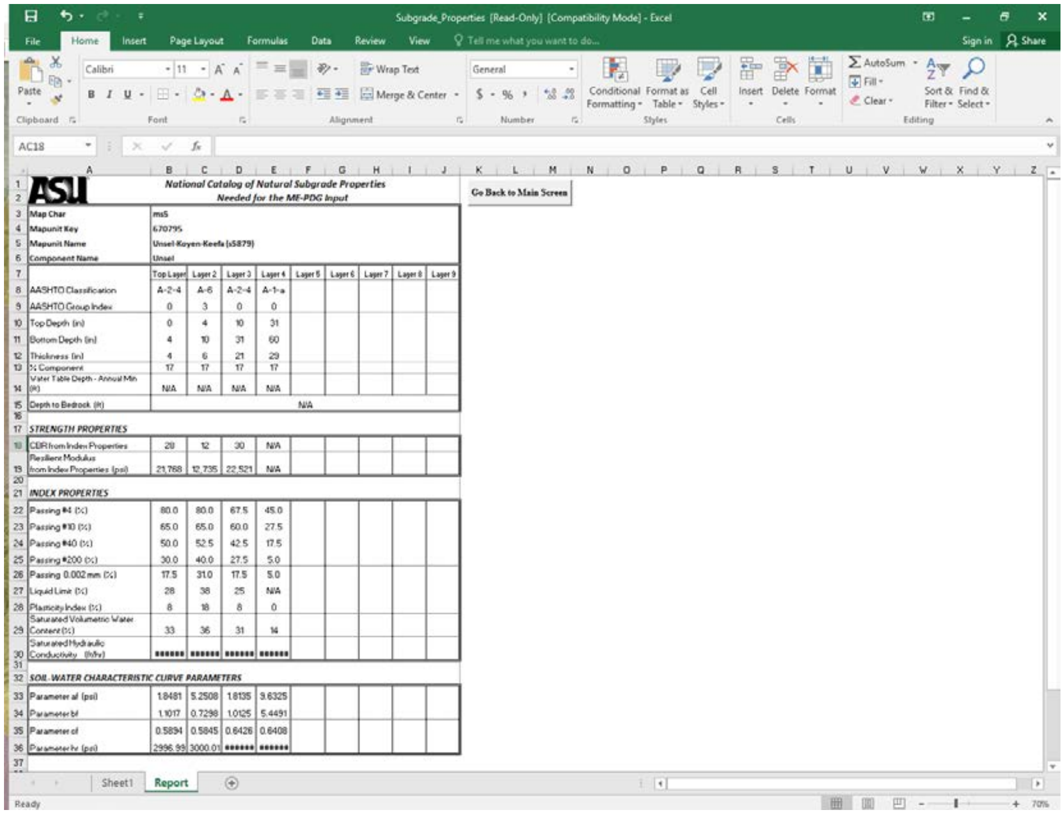


Figure 26. Generated report for Nevada.

The national catalog of natural subgrade properties is currently used in the "Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design," to determine the SWCC parameters for subgrade materials. It was used in the sensitivity analysis, which will be discussed further in Chapter 5.

MATERIAL COLLECTION

The materials tested in this project included base, borrow, and subgrade materials from all three NDOT districts. A total of nine base material types were collected—six from District 1, one from District 2, and two from District 3. Nine borrow material types were collected—five from District 1, three from District 2, and one from District 3. Six subgrade types were collected—four from District 1 and two from District 2. In total, 24 types of materials were sampled and tested.

Base and borrow materials were collected together whenever possible. Recent NDOT pavement construction projects were identified, and base and borrow materials were sampled from the pits used for these projects. Table 1 summarizes the base and borrow materials sampled from all three NDOT Districts. Figure 27 and figure 28 show the sampling locations for District 2 and District 3 base and borrow materials, respectively.

Table 1. Sampled Base and Borrow Materials.

ID	District	County	Pit	Borrow (No. of Buckets)	Type 1 Class B Base (No. of Buckets)
3605	1	Clark	Sloan Commercial Pit	–	20
3607	1	Esmeralda	Pit ES 03-08	10	20
3546	1	Clark	Apex Pit	10	20
3597	1	Clark	Lhoist Pit	10	20
3613	1	Clark	Material Pit 69-01	10	20
3583	1	Clark	LVP Lone Mountain Pit	10	20
Lockwood	2	Washoe	Lockwood Facility	15	15
SNC	2	Washoe	Sierra Nevada Construction Mustang Pit	30	–
Elko	3	Elko	Staker-Parson Pit	15	15
Hunnewill	3	Humboldt	Hunnewill Pit	–	15

–Material not collected.

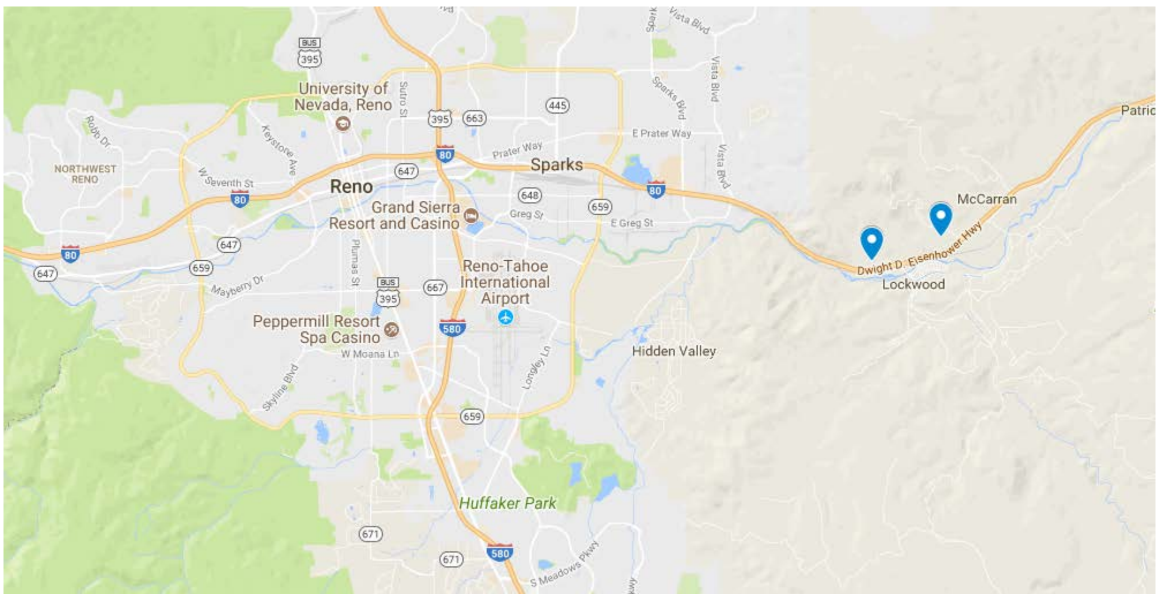


Figure 27. District 2 base and borrow sampling locations.

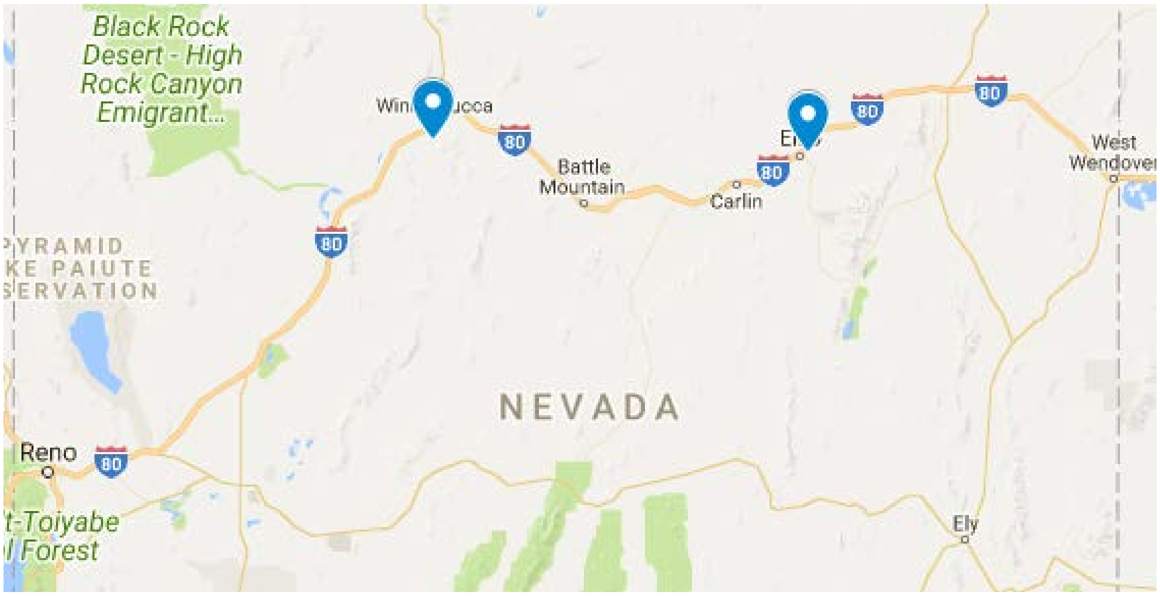


Figure 28. District 3 sampling locations.

Using the national catalog of natural subgrade properties, several types of subgrade materials were identified. Twelve locations throughout District 1 were identified. These proposed locations are shown in Figure 29 and table 2. The soil type as a function of depth was determined using the national catalog of natural subgrade properties. The AASHTO Soil Classifications A-1-a, A-1-b, A-2-4 and A-4 were found to be the most prominent soil types in District 1. Of the twelve proposed locations, six locations were sampled from. While the goal was to sample a wide variety of soil types, each of the subgrade types sampled from District 1 fell into AASHTO Soil Classification A-1-b or A-2-4; therefore, rather than naming each of the subgrade samples by their classification, for this report, they are labeled as "Sample 1," "Sample 2," etc.

Two locations in District 2 were identified for sampling. These locations were outside the Scrugham Engineering and Mines building (SEM) at UNR, where one subgrade was sampled, and Jacks Valley Road in Douglas County, where one subgrade was sampled. Figure 30, figure 31, and table 3 summarize the locations from where the materials were collected. Surface material outside of SEM at UNR was discarded, and the subgrade material was collected at a depth of two feet below the surface, as shown in figure 32.

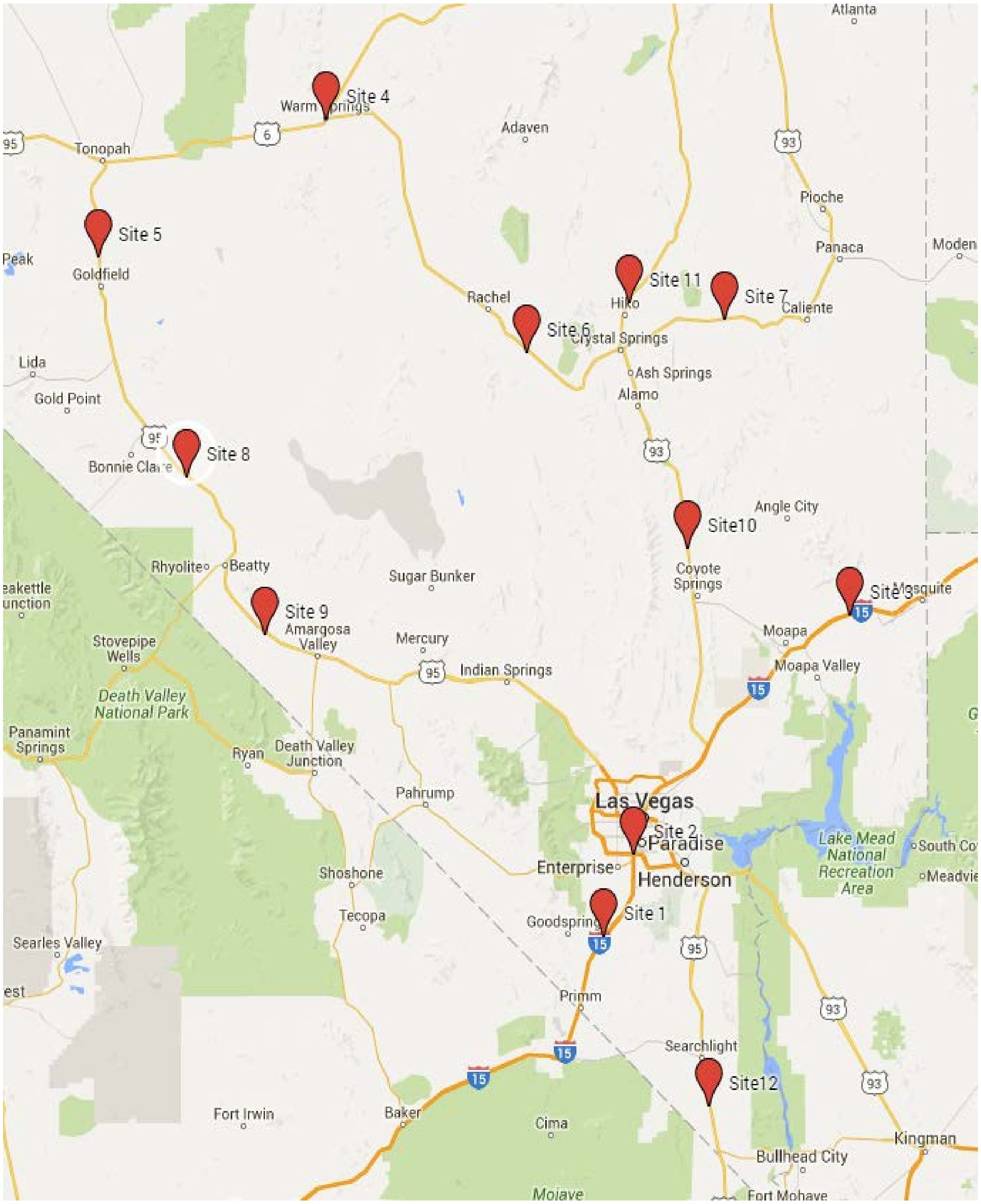


Figure 29. Proposed District 1 subgrade sampling location.

Table 2. Proposed District 1 Subgrade.

Site	Thickness (inch)	Soil Classification	Latitude (°)	Longitude (°)
1	2	A-4	35.8256	115.2970
	5.9	A-4		
2	9.1	A-2-4	36.0657	115.1806
3	2	A-2-4	36.7653	114.3457
	16.1	A-4		
	7.9	A-2-4		
4	1.2	A-4	38.1917	116.3685
	19.7	A-6		
	20.1	A-2-6		
	18.9	A-1-a		
5	5.1	A-1-a	37.7967	117.2461
	54.7	A-1-a		
6	9.1	A-2-4	37.4604	115.5078
7	2	A-4	37.6185	114.8291
	18.1	A-4		
8	5.9	A-1-b	37.1625	116.9055
	53.9	A-1-b		
9	7.9	A-1-a	36.7103	116.6061
	52	A-1-a		
10	2	A-4	36.9587	114.9719
	5.1	A-2-4		
11	3.9	A-1-b	37.6653	115.1998
	7.1	A-1-b		
	26.8	A-1-b		
12	7.9	A-5	35.3294	114.8962
	18.1	A-2-4		
	33.9	A-1-b		

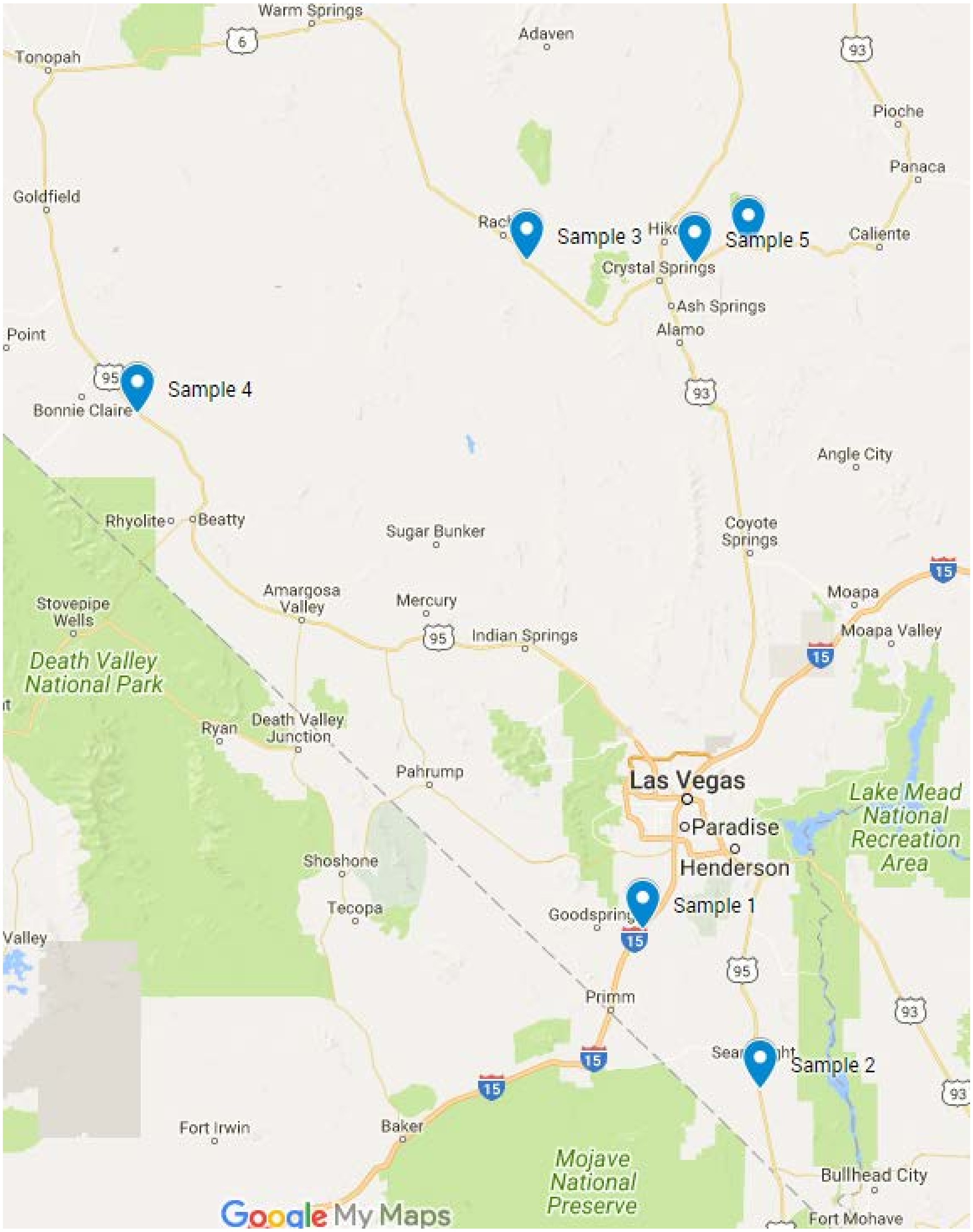


Figure 30. District 1 sampled subgrade locations.

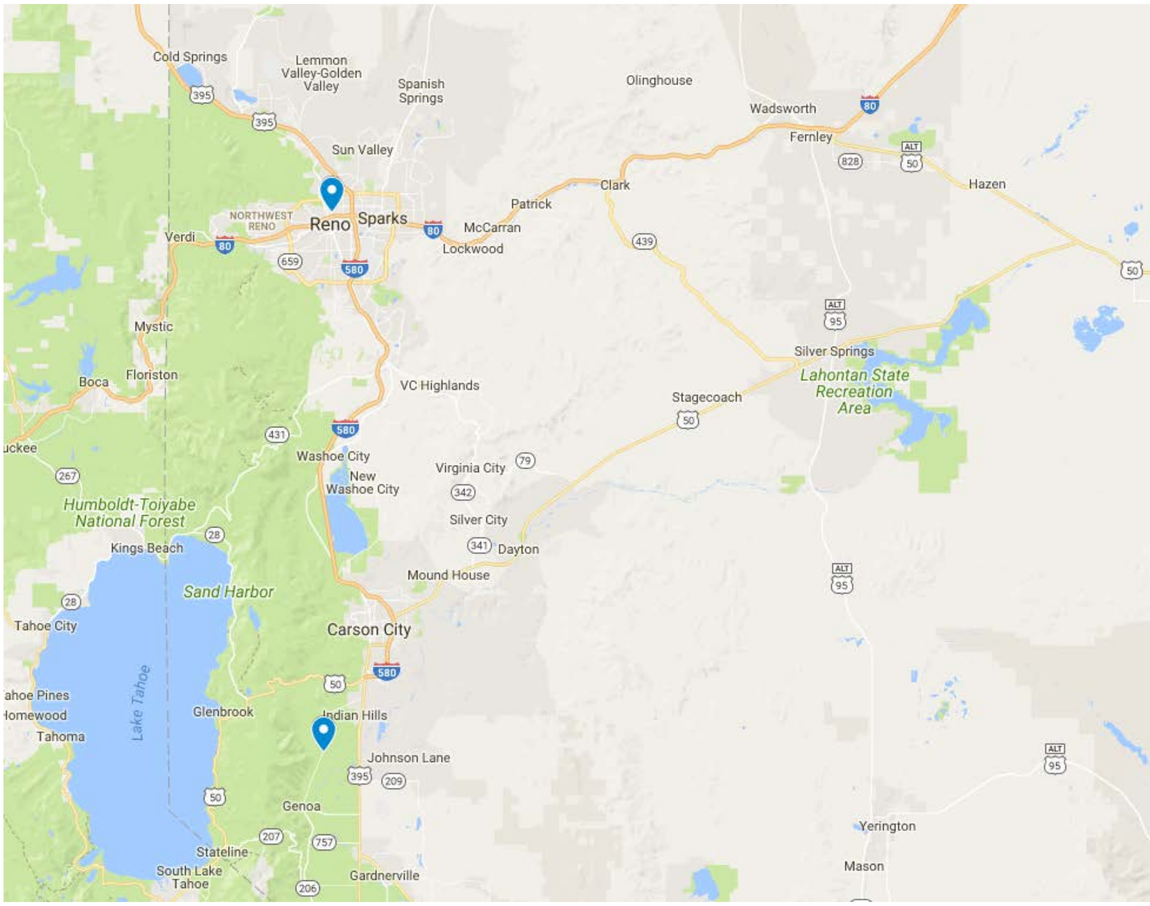


Figure 31 District 2 Base and borrow sampling locations.

Table 3. Sampled Subgrade Materials.

Subgrade	District	Location	Quantity (No. of Buckets)
Sample 1	1	I-15/Goodsprings	10
Sample 2	1	US-95/Searchlight	10
Sample 3	1	NV-375/Rachel	10
Sample 4	1	US-95/Bonnie Claire	10
Sample 5	1	US-93/Crystal Spring MP62	10
Sample 6	1	US-93/Crystal Spring MP67	10
Jacks Valley	2	Douglas County	10
SEM Soil	2	SEM Building at UNR	15

Figure 32. Sampling of SEM soil at a depth of two feet.

One type of drain rock material was sampled from District 2 at the Lockwood Facility as well; however, this material could not be tested. While gradation and coarse aggregate specific gravity testing could be conducted on the drain rock, all other testing including Atterberg limits, fine aggregate specific gravity, SWCC, MBV, and PFC testing could not be conducted. The drain rock is comprised of all coarse material, which is material retained on the No. 4 sieve that is too coarse of an aggregate blend to be able to conduct these tests.

LABORATORY EVALUATION

This section presents the laboratory testing program of the base, borrow, and subgrade materials that were sampled in this study. The materials were subjected to five groups of laboratory testing: soil classification, moisture–density relationship, specific gravity of solids, matric suction and volumetric water content, MBV, PFC, saturated hydraulic conductivity, and Resistance Value "R-Value." The following sections briefly describe the test methods.

Soil Classification Testing

The sampled materials were classified using particle size analysis and Atterberg limits following both AASHTO and Unified Soil Classification System (USCS) systems. The particle size analysis for the aggregate and soil materials was conducted in accordance with NDOT test method Nev. T206 and ASTM D 421 and D 422, respectively. NDOT test methods Nev. T210I, T211I, and T212I were used to determine the Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of the selected materials, respectively.

Particle Size Analysis

Aggregate from base and borrow materials were split into the sample size around 3000g and dried until to a constant weight at a temperature not exceeding 110°C. The dry aggregate was washed over sieve No. 10 and sieve No. 200. Retained materials on sieve No. 10, sieve No. 200, and washing vessel were transferred into a pan, dried at 110°C, and sieved through a set of sieves in a mechanical sieve shaker.

Materials from subgrade samples were split into the required sample size and dried at 60˚C. The dry material was pulverized by using a rubber head hammer. Washing was performed on sieve No. 10 and poured through sieve No. 200 until clear water appears. Retained materials on sieve No. 10 and sieve No. 200 were carefully transferred into a pan and dried at a temperature of 60°C. The dry material was pulverized again and sieve analysis was done in a mechanical sieve shaker.

Atterberg Limits

Liquid limit and plastic limit are often referred to as “Atterberg limits.” Based on its moisture content, soil can be in the state of; liquid, plastic, semi-solid, or solid. Liquid limit is the moisture content at which the soil transforms from plastic to liquid. Plastic limit is the moisture content at which the soil transforms from semi-solid to plastic. Liquid limit and plastic limit tests were conducted according to Nev. T210I and T211I, respectively.

A representative sample with minimum weight of 150g was obtained from passing sieve No. 40. Moisture was added and mixed until a uniform color is achieved. For the liquid limit test, the Casagrande apparatus was used to determine the number of blows to close the 13mm groove. The moisture content was changed in order to obtain three sets of number of blows in the range of; 25-35, 20-30, and 15-25. Around 8g of soil from the 25-35 was used for the plastic limit test. The sample was divided into 1.5-2g portion and rolled on a glass plate until it forms a 3mm thread. This process was continued until the thread crumbles at which the moisture content was obtained.

Figure 33 shows the apparatus and tools used for the liquid limit and plastic limit tests. The moisture content of the sample that gives 25 blows to close the groove by 13 mm is considered as the liquid limit.

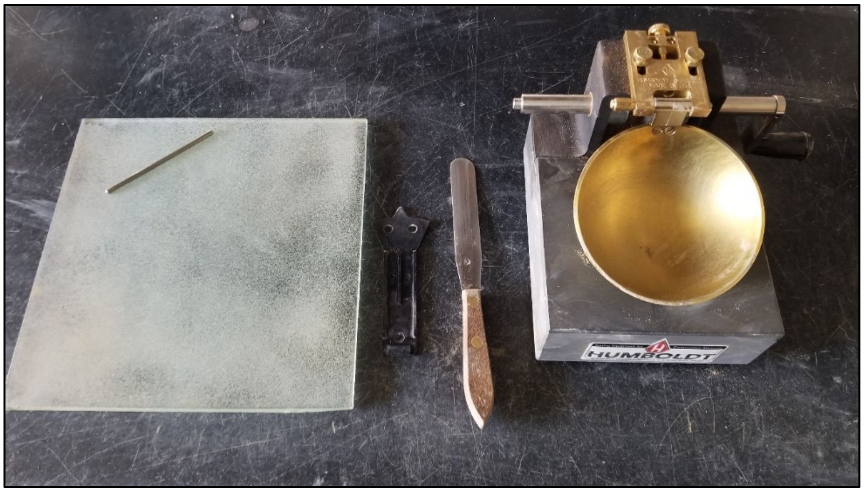


Figure 33. Atterberg limits testing equipment.

Moisture–Density Relationship

Compaction is the densification process of the material by applying mechanical energy. As the moisture content increases, water particles fill the air voids and increase the density of the material. This densification process occurs up to a certain moisture content, after which any additional water will displace the solid particles leading to reduction in the density. The corresponding moisture content at the maximum density is labeled as the optimum moisture content (OMC).

The moisture–density relationships for the various selected materials were established and OMC values corresponding to the maximum dry unit weight were identified in accordance with NDOT test method Nev. T108B. For method A, a 4-inch diameter sample was compacted in 5 equal lifts with 25 blows in each lift. For method B, a 6-inch diameter mold was compacted in 5 equal lifts with 54 blows in each lift. Both compaction methods used a 10 lb rammer with an 18 inch drop. Top lift was compacted with an extension collar and sample was trimmed to the mold surface level. Two moisture content samples were taken; one near top and one near bottom of compacted sample.

Specific Gravity of Solids

Specific gravity is defined as the ratio of the density of the material compared to the density of a standard material. In aggregate testing, water is used as the standard. Bulk specific gravity is used to calculate the volume that is occupied by the aggregates in a wide range of mixtures, including bituminous mixtures. Bulk specific gravity on a saturated surface dry (SSD) basis is to be used if the aggregate is wet, and bulk specific gravity on a dry basis is to be used if the aggregate is dry. Apparent specific gravity is not widely used in aggregate construction industry, but it is a measure of the relative density of the aggregate excluding the pore space within the aggregate that is accessible to water. Absorption is a measure of the change in mass of an aggregate after water has been absorbed. Specific gravity and absorption testing was performed on all base, borrow, and subgrade materials. It was conducted in accordance with AASHTO T 84, AASHTO T 85, and ASTM D854 on coarse aggregate, fine aggregate, and soils, respectively.

To prepare coarse aggregate samples for specific gravity and absorption testing, a sample of +No. 4 aggregate is washed in a washing vessel to remove any fine material. The sample is then submerged in water for a period of 15 to 19 hours. After the soaking period, the water is drained off of the sample, and the sample is rolled in a towel that has been lightly sprayed with water. This is done until the aggregate reaches SSD, where the surface of the aggregate shows no water. The aggregate is immediately weighed in a basket. It is then submerged in a water bath, which is at a temperature of 23±1.7°C. After recording the weight of the aggregate in water, the sample is taken out of the bath and placed in a pan to dry at 230°F until a constant mass is reached. The mass of the sample is recorded. The equipment for coarse aggregate specific gravity and absorption testing, including the basket, towel, water bath, and balance, are shown in figure 34.

Figure 34. Coarse aggreagte specific gravity and absorption testing equipment.

To prepare fine aggregate for specific gravity and absorption testing, a sample of –No. 4 aggregate is combined with at least 6% gravimetric water content and allowed to soak for a period of 15 to 19 hours. After the soaking period, the aggregate is dried to SSD. SSD condition of fine aggregate is determined by means of a slump test. The aggregate is scooped into a metal cone and then compacted using a tamper. After cleaning aggregate from around the cone using a brush, the cone is quickly lifted directly up. If the aggregate keeps the shape of the cone, it has not reached SSD. If a side of the aggregate pile slumps when the cone is removed, the aggregate has reached SSD. Once SSD is achieved, about 500 g of material is added to a pycnometer that is partially filled with water. The soil water mixture is agitated for 15 to 20 minutes to remove trapped air. This is done by rolling the pycnometer around and lightly shaking it. At the end of this agitation period, the pycnometer is filled to a specified volume with water and is weighed. The mixture is then poured into a pan and is dried at 230°F until a constant mass is reached. The dry weight is then recorded. The equipment used for fine aggregate testing, including the 500 mL pycnometer, cone, tamper, brush, funnel, spoon, and isopropyl alcohol, are shown in figure 35.

The procedure for testing specific gravity of soils, which was used to test subgrade materials, is very similar to the fine aggregate specific gravity procedure; however, it has two modifications. The sample size for this test is reduced. The agitation procedure is also slightly different. The soil water mixture is boiled rather than being agitated by hand. It is then cooled down to 23° and then weighed. The Bunsen burner and pycnometer setup for soil specific gravity testing is shown in figure 36.

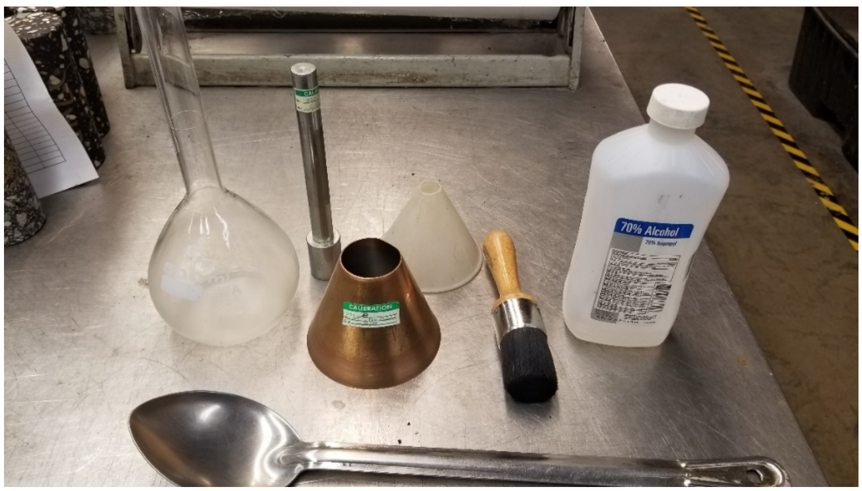


Figure 35. Fine aggreagte specific gravity and absorption testing equipment.

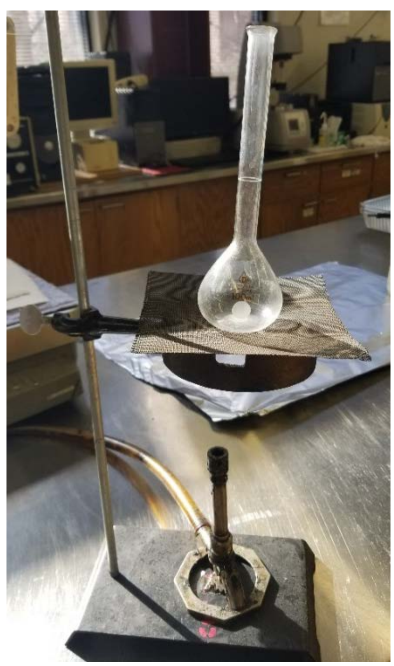


Figure 36. Soil specific gravity testing equipment.

Matric Suction and Volumetric Water Content

Matric suction is defined as the free energy state of soil. The SWCC produced from this testing is a plot of matric suction (or tension) versus volumetric water content. Two apparatuses were used by the research team to measure matric suction and volumetric water content. Different techniques are used depending on the range of matric suction that is being measured. The HYPROP device relies on the evaporation method. This method is used to measure matric suction ranges of about 0 to 100 kPa. After the peak matric suction point, air enters the sample and the matric suction starts to drop off back to 0 kPa. The WP4C device, which uses the chilled hygrometer method, was used to measure matric suction at lower moisture content and high matric suction ranges. This was done in accordance with ASTM D6836 testing proceddure.

The preparation procedure for HYPROP samples includes compaction, saturation, and de-airing water in the apparatus used for measuring theoretical maximum specific gravity of asphalt mixtures (G_{mm}), shown in figure 37. This test uses only material passing the No. 4 sieve. The specimen mold is a sample ring with a volume of 250 cubic centimeter. The soil is compacted into the sample mold using a small compaction mold that was fabricated for this project, shown in figure 38. It is compacted in 2 lifts, with 25 blows per lift from a standard proctor hammer. This compaction procedure was developed by trial and error to achieve the target compaction of 93.5% of maximum dry density at optimum water content. The SWCC input into the MEPDG is defined for the maximum dry density (100% of the maximum dry density); however, compacting the SWCC samples in the study to the maximum dry density proved to cause issues with testing. When compacted so close to maximum dry density, the tensiometers in the SWCC setup would push into the soil such that the surface of the SWCC sample would crack. Therefore, it was necessary to compact the samples at a lower compaction level of 93.5% of maximum dry density.

Figure 37. G_{mm} apparatus.

Figure 38. Compaction equipment for SWCC samples.

After compaction, the sample is then placed in deionized de-aired water to saturate for a minimum of 24 hours. The sample was considered fully saturated when there was free water on the surface of the sample. The water was de-aired using the G_{mm} apparatus. The way in which this was done was by filling the plastic container that is used in the saturation process with deionized water. This container is placed in the G_{mm} apparatus. Towels are placed around it so that the container does not move much throughout the vacuuming process. The G_{mm} apparatus is sealed and vacuum is applied. The vacuum is applied for 15 minutes, allowed to rest for 5 minutes, and then vacuum is applied for another 15 minutes. Afterward, the container is carefully removed from the G_{mm} apparatus, and the water is now de-aired. Twenty-four hours before planning to test the sample, de-ionized water in the tensiometers are de-aired. This is achieved using a system of vacuum syringes. The tensiometers are allowed to de-air overnight. Approximately three hours before planning to test the sample, the sensor unit water is de-aired. This is done using a vacuum syringe and ceramic de-airing apparatus. It should be noted that the de-ionized water used in both the tensiometer and sensor unit de-airing process is also de-aired beforehand by the G_{mm} apparatus. The tensiometers are then connected to the sensor unit, and the sensor unit is attached to the soil sample.

The sensor unit and soil sample are then placed on a balance and connected to the computer to measure matric suction and change in weight as water evaporates from the sample. The test runs for several days until the matric suction peaks and then drops down to near zero. Upon completion of the test, the soil sample is detached from the sensor unit and dried. The dry weight of the sample is then recorded in the HYPROP software. The data can then be exported to an Excel file for analysis. Pavement ME utilizes the Fredlund and Xing model, which uses equations 24 and 25, in order to calculate the volumetric water content across a range of matric suction values. The procedure for fitting the data with the Fredlund and Xing model is discussed more extensively in Chapter 2 and Appendix, but a nonlinear regression and least square error approach was used in Micorsoft Excel. The HYPROP setup is shown in figure 39.

$$\boldsymbol{\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[ln\left[exp(1) + \left(\frac{h}{a_f} \right)^{b_f} \right] \right]^{c_f}} \right]} \quad [24]$$

$$\boldsymbol{C(h) = \left[1 - \frac{ln\left(1 + \frac{h}{h_r}\right)}{ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right]} \quad [25]$$

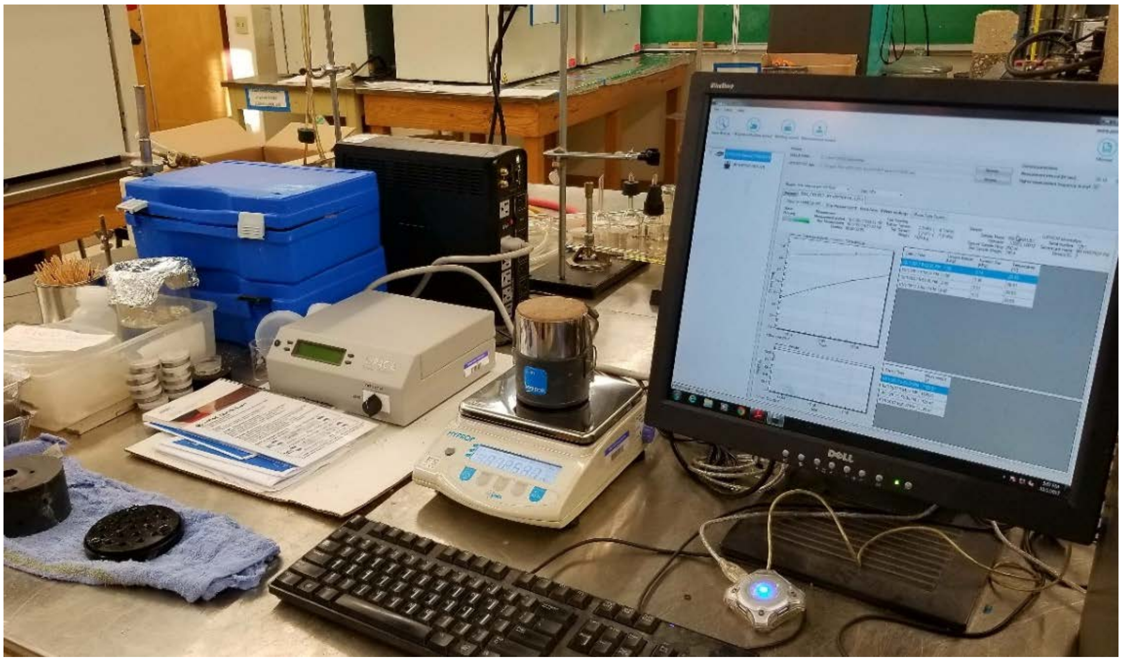


Figure 39. HYPROP setup.

The WP4C equipment is used in accordance with ASTM D6836, as well as the WP4C manual (14). Small sample cups are filled about halfway by volume with a representative sample of the soil at varying moisture contents. This is done to achieve a complete SWCC, with readings at high matric suction ranges. The soil is compacted into the sample cups using a small tamper, and the amount of material added to the sample cup and its volume should be noted so that volumetric water content can be calculated. The readings are manually added to the exported HYPROP Excel files to be added to the HYPROP readings. The WP4C equipment is shown in figure 40. The HYPROP measures matric suction, which is the suction induced by capillary forces. The WP4C measures total suction, which is the combination of matric suction and osmotic suction. The osmotic suction is the suction attributed to dissolved salts in the pore fluid.

Figure 40. WP4C equipment.

SWCC Equipment Calibration

The HYPROP device underwent an extensive calibration process in order to verify that the results produced by this equipment are valid. A borrow material sample from NDOT contract number 3607 was tested at UNR laboratory with extensive knowledge in SWCC measurement and analysis using the HYPROP device. The same borrow material was tested by the research team in the Pavement Engineering & Science (PES) laboratory with the new HYPROP equipment. The PES laboratory HYPROP device produced an SWCC that had a very similar shape to the SWCC produces by the other laboratory's HYPROP device. However, the SWCC produced by the PES laboratory was shifted up along the y-axis. After consulting with the other laboratory as well as the equipment manufacturer, along with repeating the test several times, it was found that specimen density plays a key role in the repeatability of SWCC testing. It was found that when the specimen is compacted close to the maximum dry density, that when attaching the tensiometers to the soil sample, the top of the sample cracks. Therefore, it was discovered that compacting the sample on the low end of the 95±1.5% of optimum was the most suitable compaction level for this project. Then, to help with the repeatability of this test, it was decided that all samples should be compacted very close to 93.5% of optimum. After this extensive equipment calibration process, regular SWCC testing was conducted.

In addition, the HYPROP and WP4C both need to be re-calibrated on a regular basis. After de-airing the water in the sensor unit, the zero point is checked for if re-calibration is required. This is achieved through the HYPROP View software. After attaching the tensiometers to the sensor unit, they also go through a re-calibration process to ensure the matric suction values they are reading are correct. The balance should also be re-calibrated before each test. The WP4C calibration process is less extensive. The equipment manufacturer provides 0.5 mol/kg KCl verification standards. A potassium chloride sample is poured into a WP4C sample cups and inserted into the device. The suction reading should be ±0.05 MPa of the correct reading of the KCl standard at that temperature. If the suction readings does not fit within that range, then the equipment should be adjusted. This calibration procedure should be done before each use.

Methylene Blue Value

MBV testing is conducted to measure the adsorption of methylene blue dye by clay. Methylene blue dye has a large polar organic molecule, $C_{16}H_{18}N_3S^+$, which is adsorbed onto negatively charged surfaces of clay materials. The adsorption of methylene blue changes the color of the methylene blue solution, such that the more methylene blue the clay adsorbs, the brighter the solution. The MBV is a quantification of this color change. The MBV is used in combination with the PFC as another method of SWCC estimation. This test was performed on NDOT District 1 borrow and subgrade materials by a member of the Texas A&M team. The test was performed in accordance with a new method, namely the W.R. Grace methylene blue test. This method is a modification of other traditional methods, such as ASTM C837 and AASHTO T 330-07 (15, 16). This method was used because it modifies the sampling and testing procedure such that a wider variety of soil types can be tested and a direct relationship between clay content and MBV can be established. The equipment used in this method include a colorimeter, a timer, a micropipette tip, a syringe, a 20 μm filter, an eyedropper, a 1.4 mL plastic tube, a portable balance, distilled water, and methylene blue solution. Twenty grams of material passing the No. 4 sieve is added to 30 mL of methylene blue solution. The slurry is shaken, and is then filtered through the filter paper with the syringe. The filtered solution is combined with distilled water until a total of 45 g is collected. This mixture is placed in a small glass tube and inserted into the colorimeter. The MBV reading is taken from the colorimeter, and a correction factor based on the concentration of the solution is applied to render the real MBV. The MBV equipment is shown in figure 41.

Figure 41. MBV equipment.

Percent Fines Content

The PFC test is performed to measure to total amount of specific surface area of the particle surfaces in fine materials. Clay materials are shown to have a larger specific surface area than other mineral particles. This test was also performed on NDOT District 1 borrow and subgrade materials by a member of the Texas A&M team. This test was conducted using a Horiba laser scattering particle size distribution analyzer. It has an optical light source as well as a large lens which measures the entire range of particle sizes down to 0.02 µm. The material used in testing is prepared in an ultrasonic bath and is mixed with distilled water, which produces a viscous solution. The PFC is calculated using equation 26 defined by Sahin et al. (7). The equipment used for PFC is shown in figure 42.

$$PFC = \frac{\%{-}No.2\ micron}{\%{-}No.200\ sieve} \qquad [26]$$

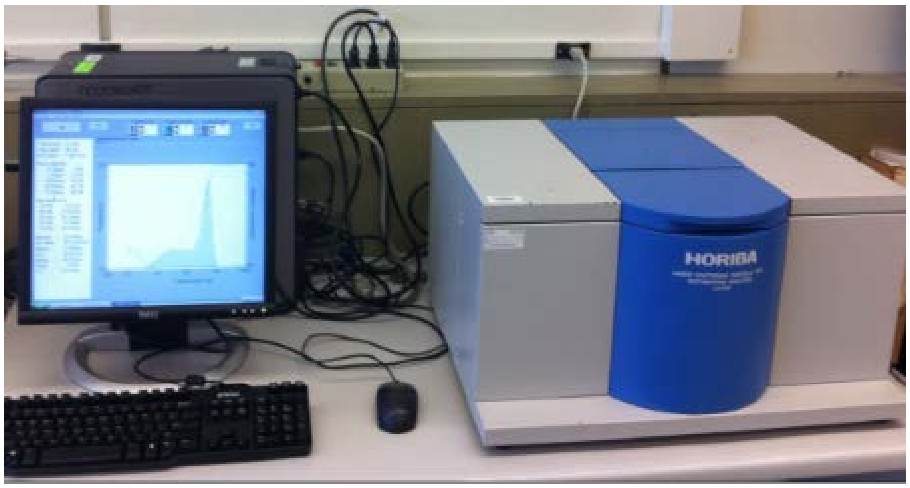


Figure 42. PFC equipment.

Saturated Hydraulic Conductivity

Saturated hydraulic conductivity is the ability of a saturated soil to conduct water while it is subjected to a hydraulic gradient. The constant-head method is typically used to measure saturated hydraulic conductivity in coarse soils, while the falling-head method is typically used for fine-grained soils. However, both methods are time consuming and were not used for this project. Instead, a member from Texas A&M measured the diffusivity of the samples. The saturated hydraulic conductivity is calculated from diffusivity and SWCC properties. This is achieved by taking the slope of the SWCC at 100% saturation and dividing it by the measured diffusivity. The diffusivity is measured by compacting two samples at optimum moisture content, which are then placed in a 100% relative humidity room at a temperature of 23°C. As time passes, the moisture content of the samples decreases. The decrease in moisture content is calculated. These samples were kept in the 100% relative humidity room until the moisture content is reduced to 2%. The loss in moisture content is used to find the diffusion rate of the sample.

This method of testing was conducted on District 1 borrow and subgrade samples. The HYPROP apparatus is able to measure hydraulic conductivity in addition to soil suction, so the saturated hydraulic conductivity testing for District 2 and 3 materials was conducted using the HYPROP.

Resistance R-Value

The R-value testing is an empirical measure of unbound materials strength and expansion potential which has been used in designing flexible pavements in Nevada. The R-value of the collected base, borrow, and subgrade materials were determined in accordance with NDOT test method Nev. T115D. Sample was split in to the required size and based on the gradation, four 1200g samples were batched for the R-value test. The initial moisture content was measured and different amount of water was added to get different moisture content. Steel mold with the diameter of 4 inch and height of 5 inch was used to prepare the sample. The mechanical kneading compactor was used to compact the sample as shown figure 43. For the compaction 100 tamps were applied to the specimen (using 200 psi foot pressure).

Figure 43. Kneading compactor.

The mold was placed on the exudation device shown in figure 44 after the compaction. A uniformly increasing load at a rate of 2,000 lb per minute was applied until exudation was achieved. The exudation pressure was calculated by taking the exudation load and dividing it by the area of the specimen. Then the sample was kept undisturbed for 16-20 hours with the addition of approximately 200 mL of water to calculate the expansion pressure as shown in figure 45. After the specimen is tested for expansion, it was forced into stabilometer as shown in figure 46. Horizontal pressure and displacement were obtained at vertical pressure of 160 psi.

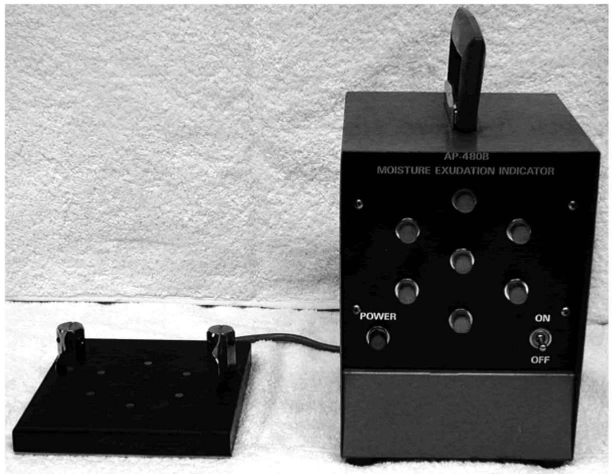


Figure 44. Exudation-indicator device.


Figure 45. Expansion pressure device.


Figure 46. R-value testing equipment.

The R-value was calculated from equation 27. The R-value is plotted against the exudation pressure. The final R-value was determined from the graph for the 300 psi exudation pressure.

$$R = 100 - \frac{100}{\left[\frac{2.5*(\mathrm{Pv}-1)}{\mathrm{D}*\mathrm{Ph}}+1\right]} \qquad [27]$$

Where,

R = R-value.
Pv = vertical pressure equal to 160 psi.
D = turns displacement reading.
Ph = Horizontal pressure (Stabilometer gauge reading for 160 psi vertical pressure).

CHAPTER 3 FINDINGS AND APPLICATIONS

This chapter discusses the results and findings from the laboratory evaluation described in Chapter 2. Additionally, a sensitivity analysis using AASHTOWare Pavement ME was conducted and is presented. Recommendations for unbound material properties to be used in Nevada Pavement ME design is also be discussed.

LABORATORY EVALUATION RESULTS

This section presents and discusses the results from the laboratory evaluation that was conducted on Nevada's unbound materials. Conformance with NDOT specifications is also discussed in this section.

Soil Classification Testing

Gradation and Atterberg Limits testing results are presented. Using these results, the material could be classified according to AASHTO and USCS soil classification systems.

Gradation

The gradation results for Districts 1, 2, and 3 base materials are shown in table 4 and table 5. The respective gradation curves for base materials are shown in figure 47 to figure 49. All the base materials collected are classified as Type 1 Class B base material, which is the most common base material used by NDOT. Each of the gradation tables contains a column listing the specification limits that the percent passing for that sieve must satisfy. The base materials collected for this study all meet the specification limits required for Type 1 Class B material in Nevada.

Table 4. District 1 Base Material Gradation.

Size (mm/inch)	Percent Passing						
	Specifi-cation	Contract No.					
		3546	3583	3597	3605	3613	3607
25.0 mm (1")	80-100	100	100	100	100	100	99.3
19.0 mm (3/4")	–	96.8	98.1	97.7	90.2	88.9	92.7
12.5 mm (1/2")	–	76.4	86.7	83.9	66.3	67.8	68.7
9.5 mm (3/8")	–	62.3	76.3	69.4	54.1	57.6	56.1
4.75 mm (No. 4)	30-65	40.8	45.6	43.4	35.3	38.6	45.4
2.36 mm (No. 8)	–	27.5	31.2	27.2	25.1	27.9	32.1
2.00 mm (No. 10)	–	25.2	29.1	24.7	23.3	26.1	28.9
1.18 mm (No. 16)	15-40	19.5	24.4	18.8	19.0	21.6	22.8
0.6 mm (No. 30)	–	14.9	20.4	14.1	15.0	18.3	17.8
0.425 mm (No. 40)	–	13.3	19.3	12.6	13.5	17.2	16.0
0.3 mm (No. 50)	–	12.0	17.0	11.4	12.1	15.8	14.5
0.15 mm (No. 100)	–	10.3	12.4	9.7	9.9	10.4	12.4
0.075 mm (No. 200)	2-12	8.8	8.7	8.3	7.7	5.3	10.0

–No specification.

Table 5. District 2 and District 3 Base Material Gradation.

Size (mm/inch)	Percent Passing			
	Specification	District 2 Lockwood	District 3 Elko	District 3 Hunnewill
25.0 mm (1”)	80-100	100	100	100
19.0 mm (3/4”)	–	96.7	99.7	98.1
12.5 mm (1/2”)	–	79.2	92.5	91.7
9.5 mm (3/8”)	–	68.5	83.1	81.0
4.75 mm (No. 4)	30-65	46.6	59.0	57.7
2.36 mm (No. 8)	–	33.6	43.3	43.7
2.00 mm (No. 10)	–	31.3	39.8	40.2
1.18 mm (No. 16)	15-40	25.2	31.6	31.6
0.6 mm (No. 30)	–	19.6	22.0	23.0
0.425 mm (No. 40)	–	16.6	17.7	19.4
0.3 mm (No. 50)	–	13.7	13.8	16.6
0.15 mm (No. 100)	–	10	9.7	12.9
0.075 mm (No. 200)	2-12	7.8	7.5	9.7

–No specification.

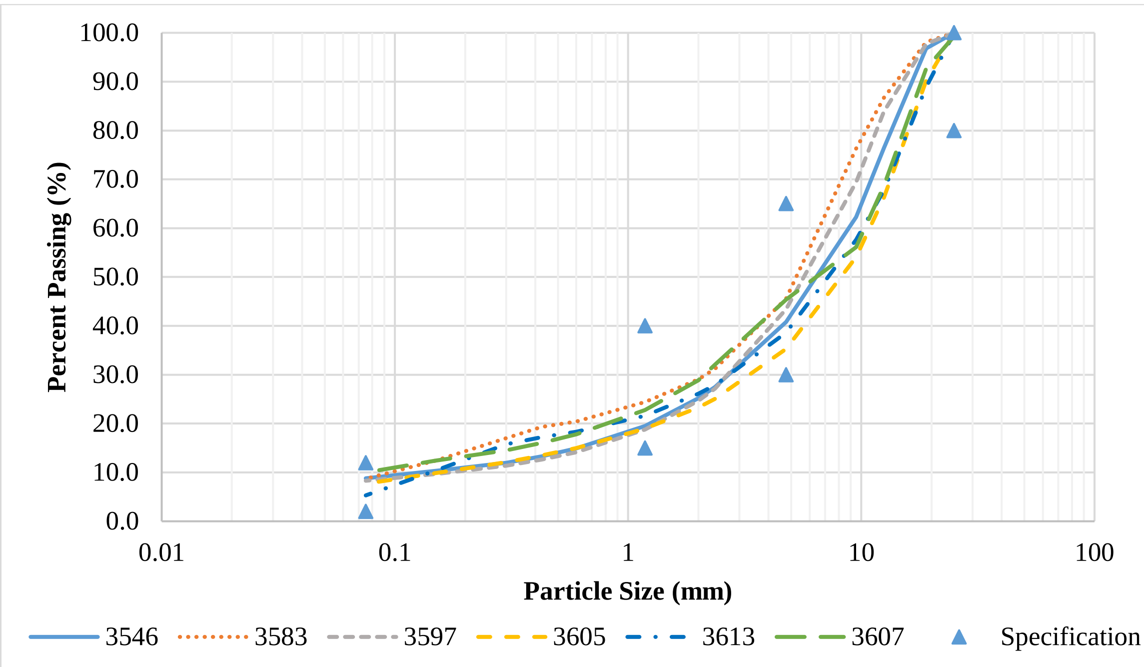


Figure 47. District 1 base material gradations.

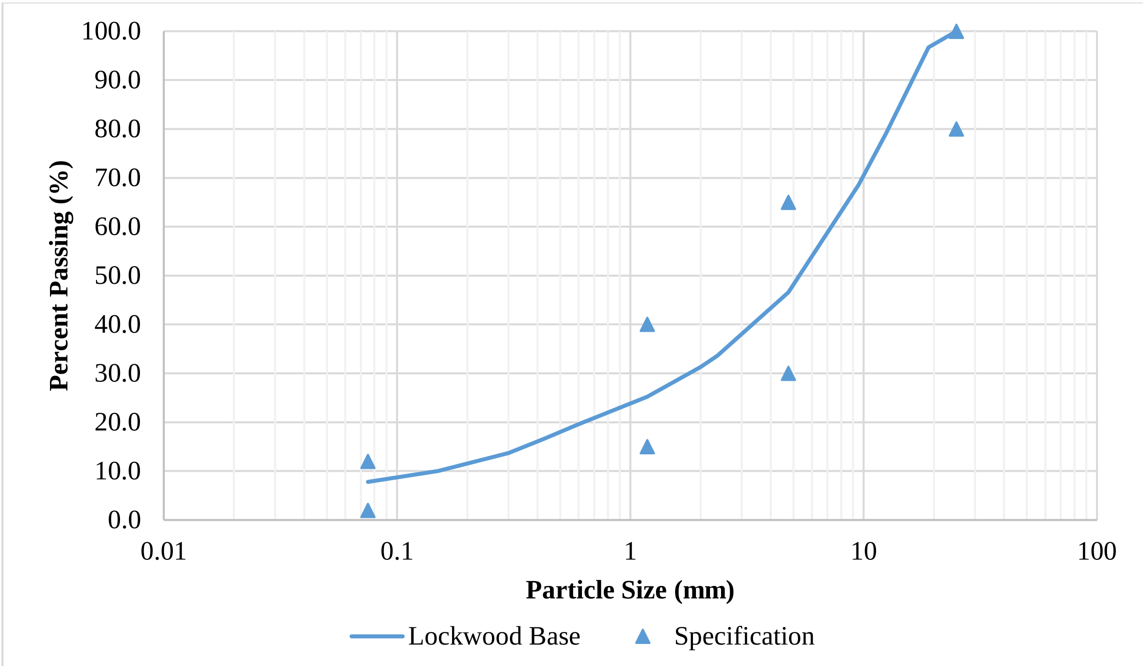


Figure 48. District 2 base material gradation.

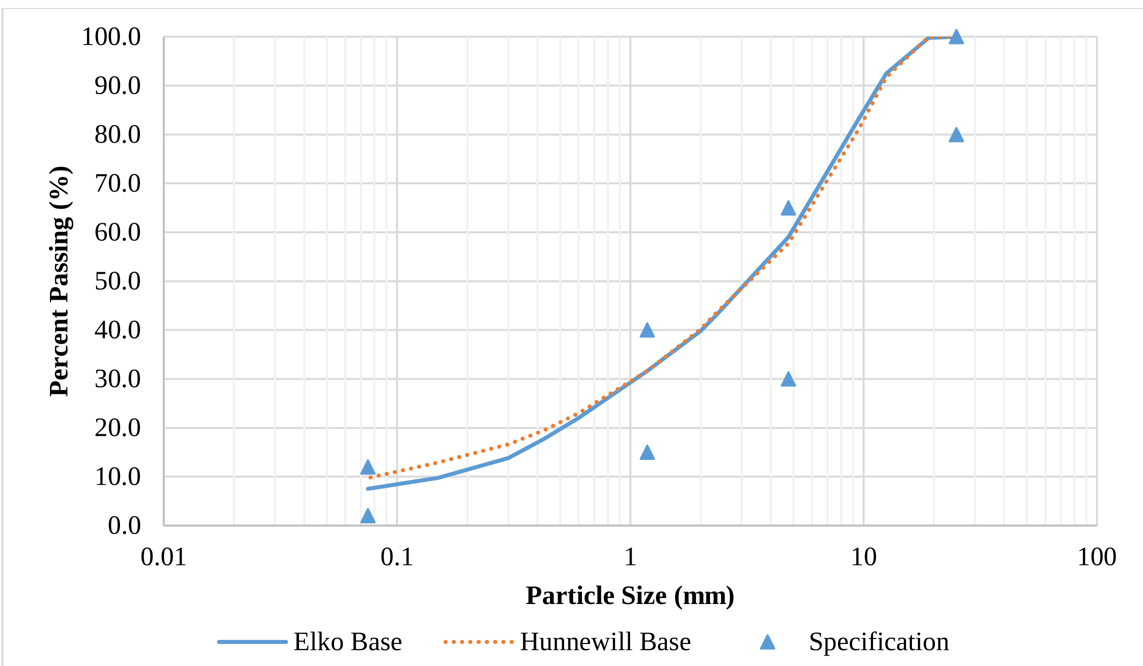


Figure 49 District 3 base material gradations.

The gradation results for Districts 1, 2, and 3 borrow materials are shown in table 6 and table 7. The gradation curves are shown in figure 50 to figure 52. According to NDOT specifications, the only criteria that borrow material gradations must meet is that 100% of the material must pass the 3-inch sieve. All the sampled borrow materials for this project satisfy this criterion. However, the gradations were highly variable, as evident in each of the gradation curve plots.

Table 6. District 1 Borrow Material Gradation.

Size (mm/inch)	Percent Passing					
	Specification	Contract No.				
		3546	3583	3597	3613	3607
75 mm (3”)	100	100	100	100	100	100
50 mm (2”)	–	100	100	100	100	100
37.5 mm (1.5”)	–	100	100	100	97.4	100
25.0 mm (1”)	–	100	99.1	97.7	89.9	98.0
19.0 mm (3/4”)	–	100	95.5	96.0	85.3	94.5
12.5 mm (1/2”)	–	100	92.9	90.2	76.8	89.9
9.5 mm (3/8”)	–	99.9	91.1	85.6	69.8	86.2
4.75 mm (No. 4)	–	79.9	88.1	71.7	53.3	75.9
2.36 mm (No. 8)	–	48.6	86.7	56.7	40.8	65.3
2.00 mm (No. 10)	–	43.0	86.4	53.3	38.1	62.6
1.18 mm (No. 16)	–	28.6	85.6	42.1	32.4	54.0
0.6 mm (No. 30)	–	18.4	84.6	32.4	27.9	43.0
0.425 mm (No. 40)	–	15.4	84.2	28.7	26.3	37.6
0.3 mm (No. 50)	–	13.3	83.5	25.7	24.0	32.0
0.15 mm (No. 100)	–	11.4	80.6	20.9	14.3	23.7
0.075 mm (No. 200)	–	10.5	66.9	16.4	7.3	16.4

–No specification.

Table 7. District 2 and District 3 Borrow Material Gradation.

Size (mm/inch)	% Passing				
	Specification	District 2	District 2	District 2	District 3
		Lockwood Borrow	SNC Primary	SNC Secondary	Elko Borrow
75 mm (3”)	100	100	100	100	100
50 mm (2”)	–	100	100	100	100
37.5 mm (1.5”)	–	100	100	100	100
25.0 mm (1”)	–	100	100	100	87.3
19.0 mm (3/4”)	–	98.8	100	100	82.0
12.5 mm (1/2”)	–	91.5	97.5	100	74.6
9.5 mm (3/8”)	–	82.9	91.7	100	68.9
4.75 mm (No. 4)	–	62.7	70.1	98.7	53.4
2.36 mm (No. 8)	–	48.1	54.1	69.9	40.9
2.00 mm (No. 10)	–	45.1	50.7	61.5	37.4
1.18 mm (No. 16)	–	37.5	41.8	40.7	29.3
0.6 mm (No. 30)	–	31.5	33.3	25.2	18.8
0.425 mm (No. 40)	–	29.1	30.1	20.6	13.8
0.3 mm (No. 50)	–	26.8	27.5	17.9	10.0
0.15 mm (No. 100)	–	22.6	23.6	14.6	6.5
0.075 mm (No. 200)	–	17.9	18.5	12.3	4.9

–No specification.

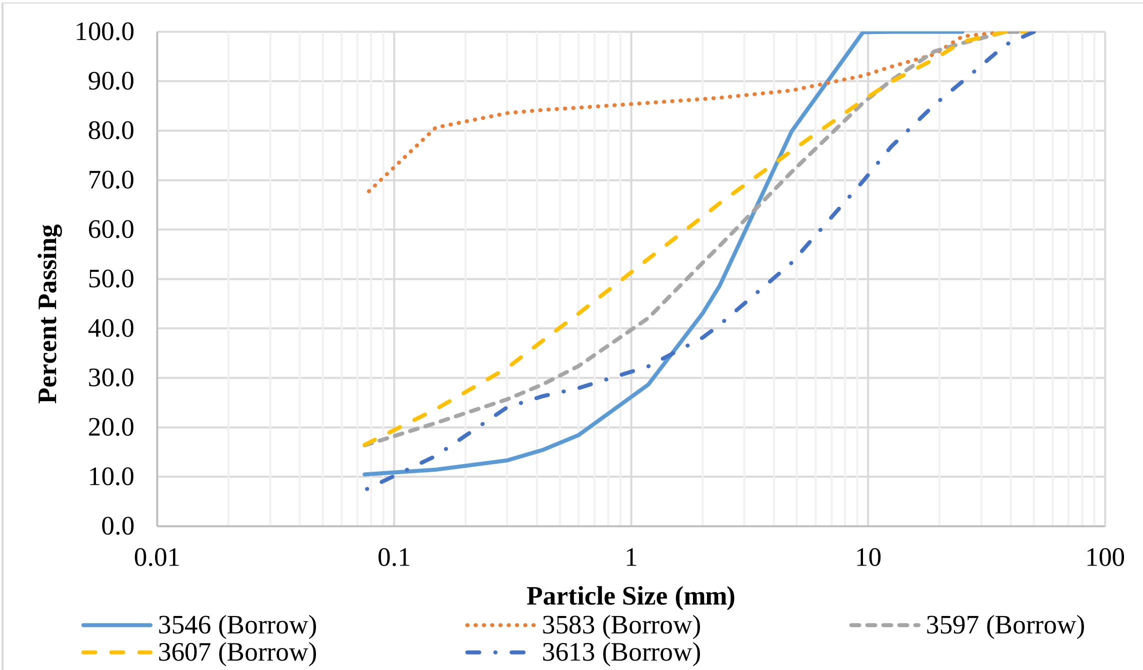


Figure 50. District 1 borrow material gradations.

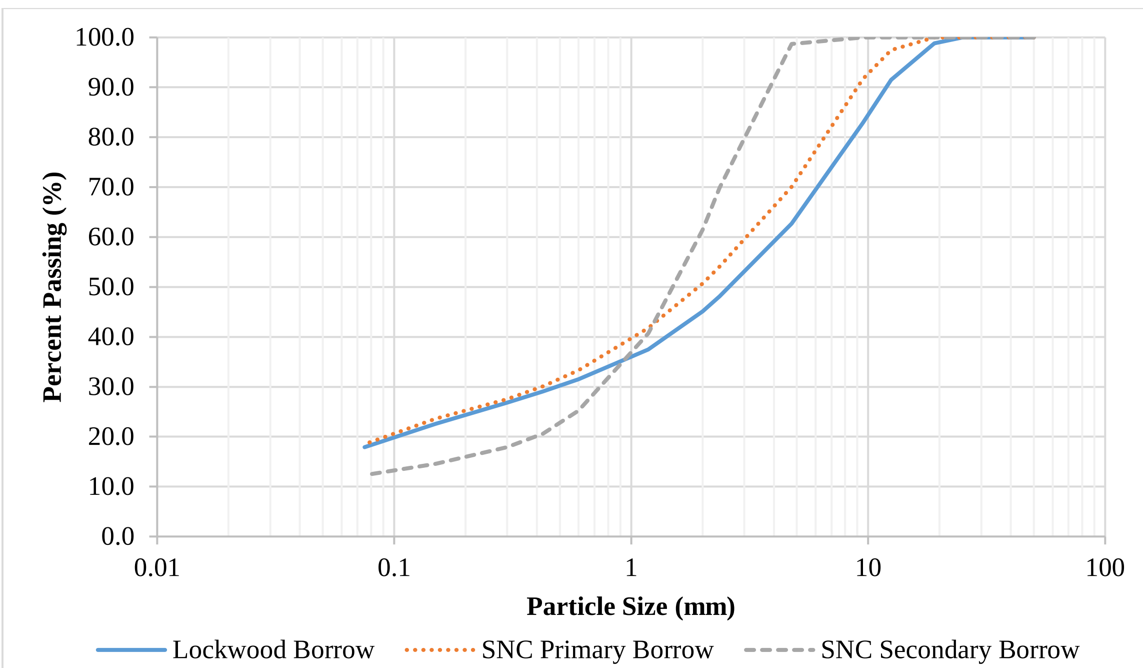


Figure 51. District 2 borrow material gradation.

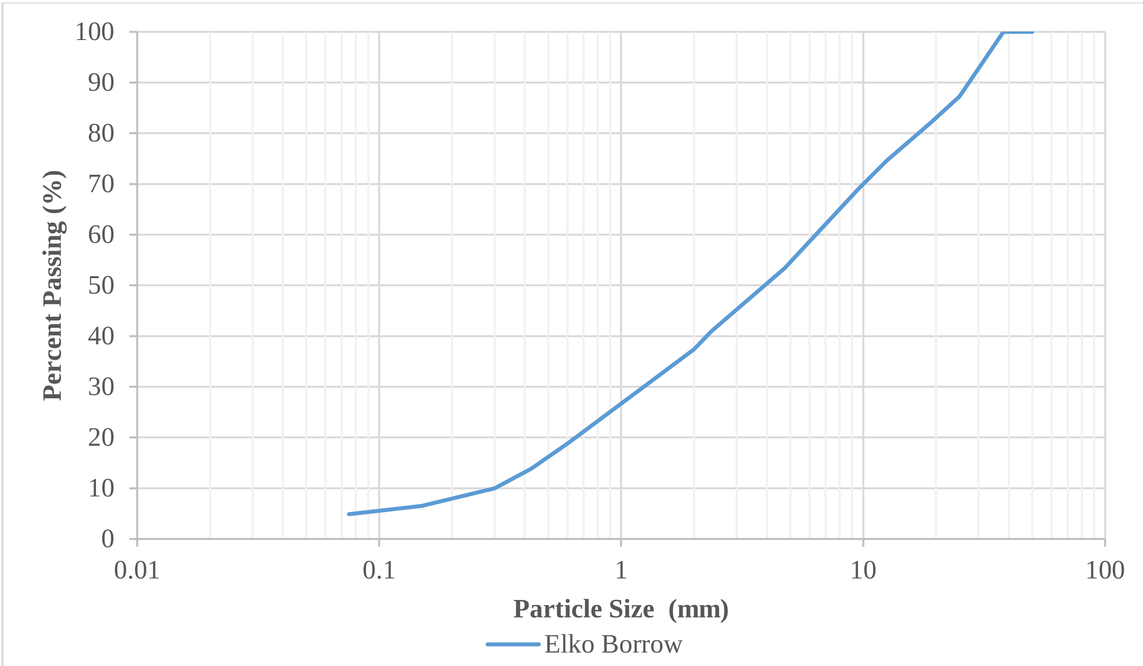


Figure 52. District 3 borrow material gradation.

The results for gradation of the subgrade materials are shown in table 8 and table 9. The curves are shown in figure 53 and figure 54. Subgrade material is the native material found at the project location.

Table 8. District 1 Subgrade Gradation.

Size (mm/inch)	Percent Passing					
	I-15/ Goodspring	US-95/ Search-light	NV-375/ Rachel	US-95/ Bonnie Claire	US-93/ Crystal Spring MP62	US-93/ Crystal Spring MP67
50.0 mm (2")	97.5	100	100	100	100	100
25.0 mm (1")	83.5	96.7	87.5	98.8	100	100
9.5 mm (3/8")	57.2	92.7	52.2	95.4	99.3	97.2
4.75 mm (No. 4)	43.4	87.8	33.5	92	95.6	89.3
2.00 mm (No. 10)	34.4	68.7	23.2	84.3	81.4	77.2
0.425 mm (No. 40)	28	43.9	15.2	37.6	44.5	52.6
0.3 mm (No. 50)	26.6	39.3	13.4	25.2	37.1	46.7
0.15 mm (No. 100)	22.6	31.5	9.6	11.7	25.5	35.7
0.075 mm (No. 200)	14.6	23.9	5.4	5.5	18.1	26

Table 9. District 2 Subgrade Gradation.

Size (mm/inch)	Percent Passing	
	Jacks Valley	UNR Soil at SEM
37.5 mm (1.5")	100	100
25.0 mm (1")	100	93.4
19.0 mm (3/4")	100	87.7
12.5 mm (1/2")	100	78.7
9.5 mm (3/8")	100	74.8
4.75 mm (No. 4)	99.7	66.4
2.36 mm (No. 8)	97.8	59.6
2.00 mm (No. 10)	96.8	57.4
1.18 mm (No. 16)	93	49.6
0.6 mm (No. 30)	81.8	36.9
0.425 mm (No. 40)	72.3	31.4
0.3 mm (No. 50)	61	27.2
0.15 mm (No. 100)	42.3	21.4
0.075 mm (No. 200)	26.1	16.2

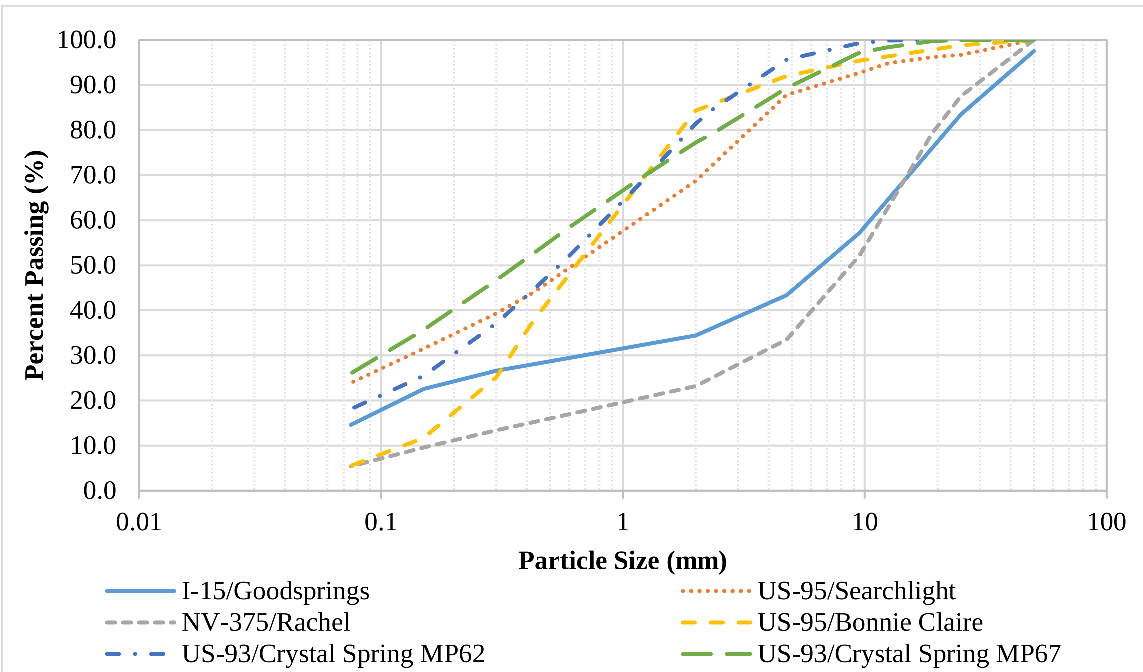


Figure 53. District 1 subgrade gradations.

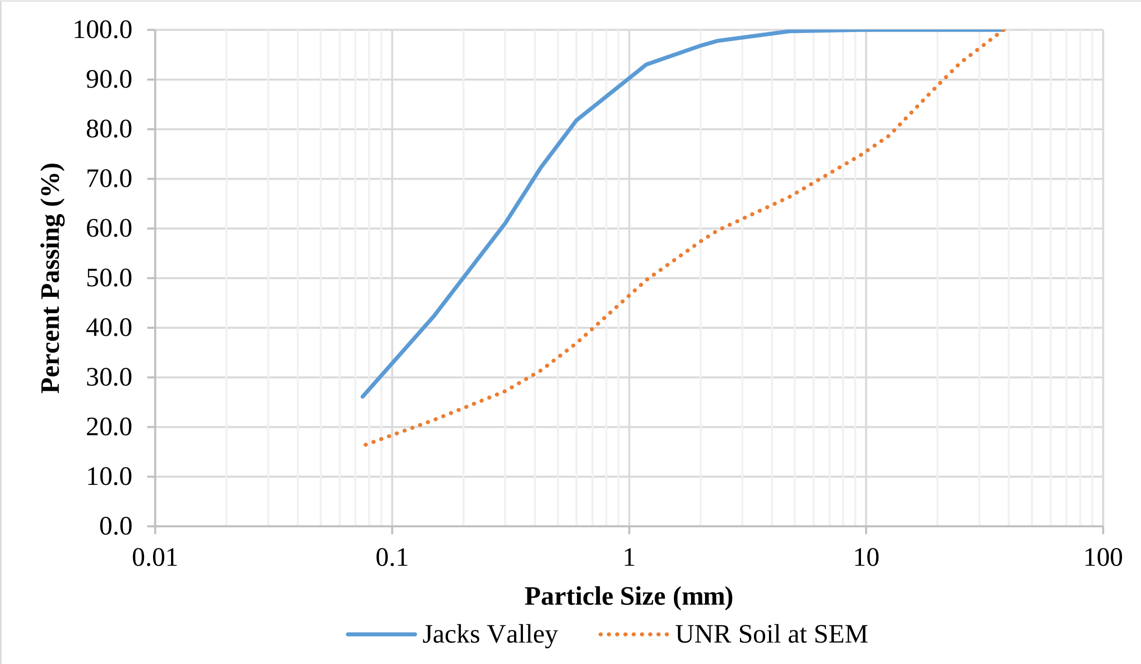


Figure 54. District 2 subgrade material gradations.

Atterberg Limits

All of the base materials from all three districts resulted as being non-plastic. The results of the Atterberg Limits testing for all borrow and subgrade materials are shown in table 10 and table 11, respectively. In some cases, non-plastic materials had issues being tested for resilient modulus, as there were not enough fine contents to hold the samples together for testing. This will be discussed further in the respective section.

Table 10. Borrow Material Atterberg Limits.

Source	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI
3546	16.5	14.5	2.0
3583	23.5	18.8	4.7
3597	22.2	18.9	3.3
3607	23.2	23.1	0.1
3613	N/A[1]	NP[2]	0.0
Lockwood Borrow	45.9	31.9	14.0
SNC Primary	39.1	24	15.1
SNC Secondary	N/A[1]	NP[2]	0.0
Elko Borrow	N/A[1]	NP[2]	0.0

[1]Not Applicable.
[2]Non-plastic.

Table 11. Subgrade Material Atterberg Limits.

Material	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI
I-15/Goodsprings	18.4	16.9	1.5
US-95/Searchlight	N/A[1]	NP[2]	0.0
NV-375/Rachel	30.9	26.6	4.3
US-95/Bonnie Claire	21.1	20.1	1.0
US-93/Crystal Spring MP62	19.6	17.7	1.9
US-93/Crystal Spring MP67	22.2	17.8	4.5
Jacks Valley	22.9	20.5	2.4
UNR Soil at SEM	24.0	20.4	3.6

[1]Not Applicable.
[2]Non-plastic.

Based on the data presented in table 10 and table 11, the following observations can be made:

- In the case of borrow materials, three of the evaluated materials were non-plastic (PI = 0), four of the materials were slightly plastic (PI < 7), and two of the materials were medium plastic (7 ≤ PI ≤ 17).
- In the case of subgrade, all evaluated materials were either non-plastic (PI = 0) or slightly plastic (PI < 7).

Soil Classification

After conducting sieve analysis and Atterberg Limits testing, the soil classification for each of the subgrade materials was determined. The most used classification systems are: AASHTO soil classification, and USCS. The AASHTO soil classification system is used mostly by highway agencies and is based on particle size distribution and soil plasticity. On the other hand, USCS is widely used by geotechnical engineers and is based on particle size distribution, liquid limit, soil plasticity, and organic matter concentrations.

Table 12 summarizes the AASHTO soil classification and USCS of all evaluated subgrade materials. The evaluated materials were mostly silt and clay-type materials with a general rating according to AASHTO M145 of excellent to good.

Table 12. Subgrade Material Soil Classifications.

Material	AASHTO Soil Classification (AASHTO M145)	USCS (ASTM D 2487)	
		Group Symbol	Group Name
I-15/Goodsprings	A-1-a	GM	Silty gravel
US-95/Searchlight	A-1-b	SM	Silty sand
NV-375/Rachel	A-1-a	GP-GM	Poorly graded gravel with silt
US-95/Bonnie Claire	A-1-b	SW-SM	Well-graded sand with silt
US-93/Crystal Spring MP62	A-1-b	SM	Silty sand
US-93/Crystal Spring MP67	A-2-4	SC	Clayey sand
Jacks Valley	A-2-4	SM-SC	Silty, clayey sand
UNR Soil at SEM	A-1-b	SM-SC	Silty, clayey sand

Moisture-Density Relationship

The results of the base, borrow, and subgrade material moisture density testing are shown in table 13 to table 15, respectively. If Method A was used, and if there was more than 5% material retained on the No. 4 sieve (from gradation), then a correction needed to be applied to the maximum dry density and the optimum water content. If Method D was used, and there was more than 5% material retained on the ¾ inch sieve, then a correction needed to be applied to the maximum dry unit weight and the optimum water content.

The base material exhibited the highest maximum dry density values, with an average of 143.5 pcf. It also had the lowest optimum moisture content values, with an average of 5.3%. In comparison, the borrow material had an overall average maximum dry density of 134.9 pcf and an average optimum moisture content of 7.4%. The subgrade material had an average maximum dry density lower than that of borrow material and equal to 129.9 pcf. It also had an average optimum moisture content higher than that of borrow material and equal to 8.2%. Figure 55 is a graphical representation of this information, showing the optimum moisture content and the maximum dry density for the three material types.

Table 13. Base Material Moisture Density Results.

Sample	Max Dry Density (pcf)	OMC (%)	Corrected Max Dry Density (pcf)	Corrected OMC (%)
3546	144.7	5.0	–	–
3583	147.3	5.6	–	–
3597	143.0	3.9	–	–
3605	147.5	5.0	149.7	4.7
3607	135.8	6.7	137.8	6.4
3613	141.6	3.5	144.4	3.3
Lockwood Base	138.2	8.0	–	–
Elko Base	129.7	8.4	141.1	5.8
Hunnewill base	132.8	7.2	145.5	5.0

–No correction.

Table 14. Borrow Material Moisture Density Results.

Sample	Max Dry Density (pcf)	OMC (%)	Corrected Max Dry Density (pcf)	Corrected OMC (%)
3546	136.9	7.2	144.8	5.7
3583	119.4	10.7	123.3	9.7
3597	133.8	6.2	142.9	5.0
3607	125.6	11.3	132.9	9.1
3613	143.2	5.4	–	–
Lockwood Borrow	125.4	9.3	137.0	6.6
SNC Primary	124.4	10.6	133.8	8.0
SNC Secondary	136.1	9.6	–	–
Elko Borrow	124.9	9.5	139.7	6.0

–No correction.

Table 15. Subgrade Material Moisture Density Results.

Sample	Max Dry Density (pcf)	OMC (%)
I-15/Goodsprings	134.9	6.3
US-95/Searchlight	133.3	6.6
NV-375/Rachel	139.2	6.1
US-95/Bonnie Claire	126.9	9.4
US-93/Crystal Spring MP62	122.4	9.8
US-93/Crystal Spring MP67	123.8	9.3
Jacks Valley	125.5	9.4
UNR Soil at SEM	132.8	8.5

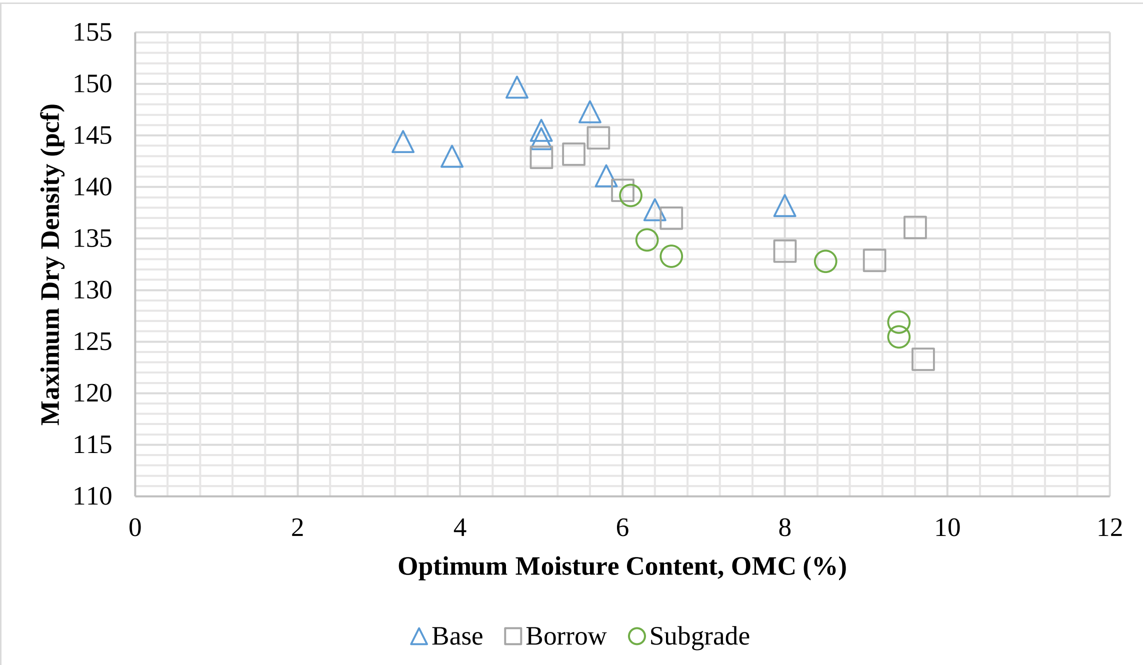


Figure 55. Moisture density summary of base, borrow and subgrade materials.

Specific Gravity of Solids

The results of the specific gravity testing are shown in table 16 to table 20. Fine aggregate bulk specific gravity values for the base materials ranged from 2.403 to 2.708, with an average of 2.594. Coarse aggregate bulk specific gravity values for the base materials ranged from 2.495 to 2.773, with an average of 2.706. Fine aggregate bulk specific gravity values for the borrow materials ranged from 2.293 to 2.684, with an average of 2.454. Coarse aggregate bulk specific gravity values for the borrow materials ranged from 2.333 to 2.696, with an average of 2.615. The specific gravity of the subgrade materials at 20°C ranged from 2.601 to 2.688, with an average of 2.645.

Table 16. Base Material Fine Aggregates Specific Gravity Results.

Sample	Bulk Specific Gravity	SSD Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
3546	2.684	2.701	2.728	0.60
3583	2.623	2.684	2.793	2.31
3597	2.708	2.715	2.727	0.26
3605	2.689	2.727	2.797	1.45
3607	2.427	2.532	2.709	4.29
3613	2.603	2.653	2.739	1.91
Lockwood Base	2.623	2.657	2.714	1.28
Elko Base	2.402	2.476	2.586	3.11
Hunnewill base	2.403	2.479	2.600	0.06

Table 17. Base Material Coarse Aggregates Specific Gravity Results.

Sample	Bulk Specific Gravity	SSD Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
3546	2.721	2.732	2.752	0.42
3583	2.773	2.794	2.833	0.76
3597	2.713	2.724	2.743	0.40
3605	2.735	2.753	2.787	0.69
3607	2.615	2.655	2.723	1.52
3613	2.679	2.704	2.749	0.95
Lockwood Base	2.773	2.821	2.904	1.57
Elko Base	2.495	2.540	2.612	1.80
Hunnewill base	2.562	2.606	2.680	1.73

Table 18. Borrow Material Fine Aggregates Specific Gravity Results.

Sample	Bulk Specific Gravity	SSD Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
3546	2.552	2.614	2.721	2.43
3583	2.336	2.378	2.439	1.80
3597	2.501	2.574	2.699	2.94
3607	2.293	2.435	2.673	6.20
3613	2.587	2.644	2.744	2.22
Lockwood Borrow	2.684	2.717	2.775	1.22
SNC Primary	2.606	2.682	2.821	1.92
SNC Secondary	2.650	2.712	2.824	2.32
Elko Borrow	2.554	2.567	2.589	0.52

Table 19. Borrow Material Coarse Aggregates Specific Gravity Results.

Sample	Bulk Specific Gravity	SSD Bulk Specific Gravity	Apparent Specific Gravity	Absorption (%)
3546	2.668	2.687	2.721	0.73
3583	2.696	2.658	2.727	1.54
3597	2.688	2.715	2.763	1.01
3607	2.333	2.437	2.605	4.47
3613	2.668	2.695	2.742	1.01
Lockwood Borrow	2.623	2.657	2.716	1.31
SNC Primary	2.634	2.669	2.713	1.02
SNC Secondary	–	–	–	–
Elko Borrow	2.444	2.505	2.603	2.43

–Not applicable.

Table 20. Subgrade Material Fine Aggregates Specific Gravity at 20°C.

Sample	Specific Gravity
I-15/Goodsprings	2.601
US-95/Searchlight	2.688
NV-375/Rachel	2.658
US-95/Bonnie Claire	2.651
US-93/Crystal Spring MP62	–
US-93/Crystal Spring MP67	–
Jacks Valley	2.604
UNR Soil at SEM	2.667

–Not data.

Matric Suction and Volumetric Water Content

The SWCC parameters resulting from the direct measurement of SWCCs are displayed in table 21 to table 23 for base, borrow, and subgrade materials, respectively. The corresponding plots for input level 1 SWCCs are shown in figure 56 to figure 58 for base, borrow, and subgrade material, respectively.

In the case of base material, the Elko base showed a very high a_f value when compared to all other base materials. It should be noted that the Elko base had a lower maximum dry density (130 pcf) than all the other base materials (133 to 147 pcf). For a higher density, the water being held in the lower suction ranges evaporates pretty quickly leading to the more tightly bound waters, which leads to a quicker dry down. Thus, air entry tends to occur sooner, leading to a lower air entry level (i.e., lower a_f). Accordingly, since the Elko Base has a lower maximum dry density, a higher a_f is expected in comparison to the other base materials.

Table 21. Base Material SWCC Parameters (Input Level 1).

Sample	a_f	b_f	c_f	h_r
3546	0.115	1.859	0.918	352.570
3583	0.647	1.582	0.553	404.77
3597	0.237	1.367	0.855	540.537
3605	1.650	0.996	2.968	6.665
3607	0.728	0.933	0.837	791.242
3613	1.847	2.527	4.521	1105.9
Lockwood Base	0.374	1.337	0.478	2.599
Elko Base*	39.468	0.648	12.727	105.385
Hunnewill base	0.316	1.844	0.598	0.717
Average	5.042	1.455	2.717	367.821
Standard Deviation	12.185	0.541	3.770	368.621

Table 22. Borrow Material SWCC Parameters (Input Level 1).

Sample	a_f	b_f	c_f	h_r
3546	0.365	1.511	3.531	23.933
3583	59.649	1.140	33.894	4.164
3597	2.134	0.119	3.814	613.120
3607	44.104	0.483	56.292	56.954
3613	1.016	5.323	1.911	9.087
Lockwood Borrow	1.979	2.055	0.555	5.623
SNC Primary	1.095	1.269	0.321	4.058
SNC Secondary	2.358	1.032	1.434	18.784
Elko Borrow	32.698	0.827	27.204	201.215
Average	16.155	1.529	14.328	104.104
Standard Deviation	21.704	1.442	18.983	189.497

Table 23. Subgrade Material SWCC Parameters (Input Level 1).

Sample	a_f	b_f	c_f	h_r
I-15/Goodsprings	0.513	3.332	0.256	1.387
US-95/Searchlight	2.324	1.612	1.121	59.311
NV-375/Rachel	2.119	2.760	2.619	50.235
US-95/Bonnie Claire	1.538	2.249	0.573	6.158
US-93/Crystal Spring MP62	–	–	–	–
US-93/Crystal Spring MP67	–	–	–	–
Jacks Valley	3.817	1.142	0.733	36.600
UNR Soil at SEM	1.952	1.198	0.747	45.542
Average	2.044	2.049	1.008	33.206
Standard Deviation	0.986	0.809	0.764	21.906

–No data.

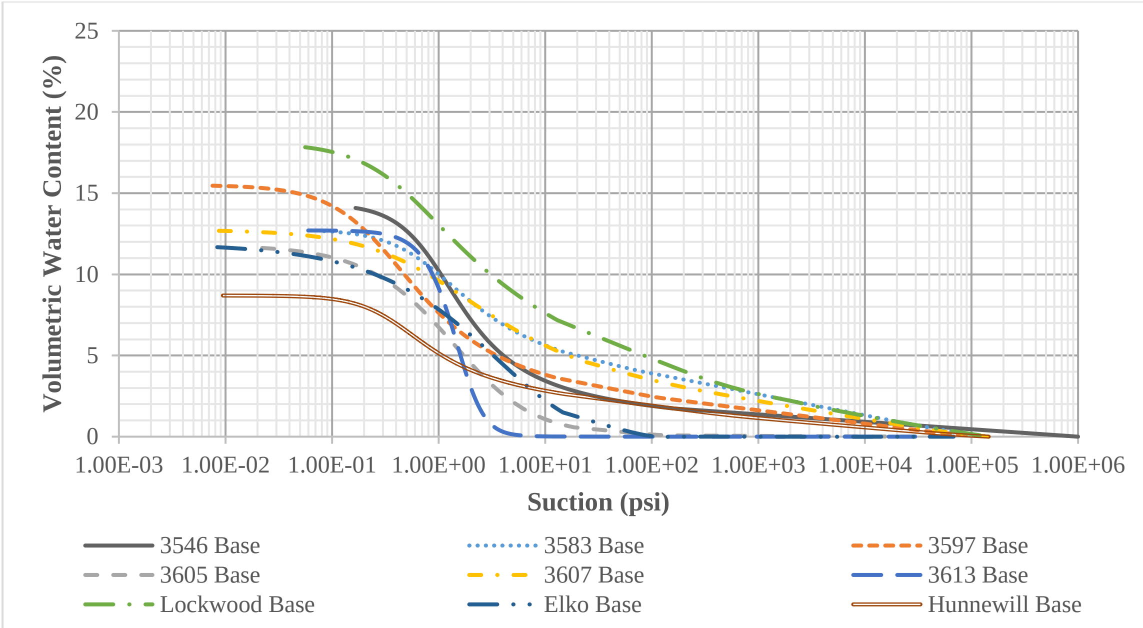


Figure 56. Base material SWCC (input level 1).

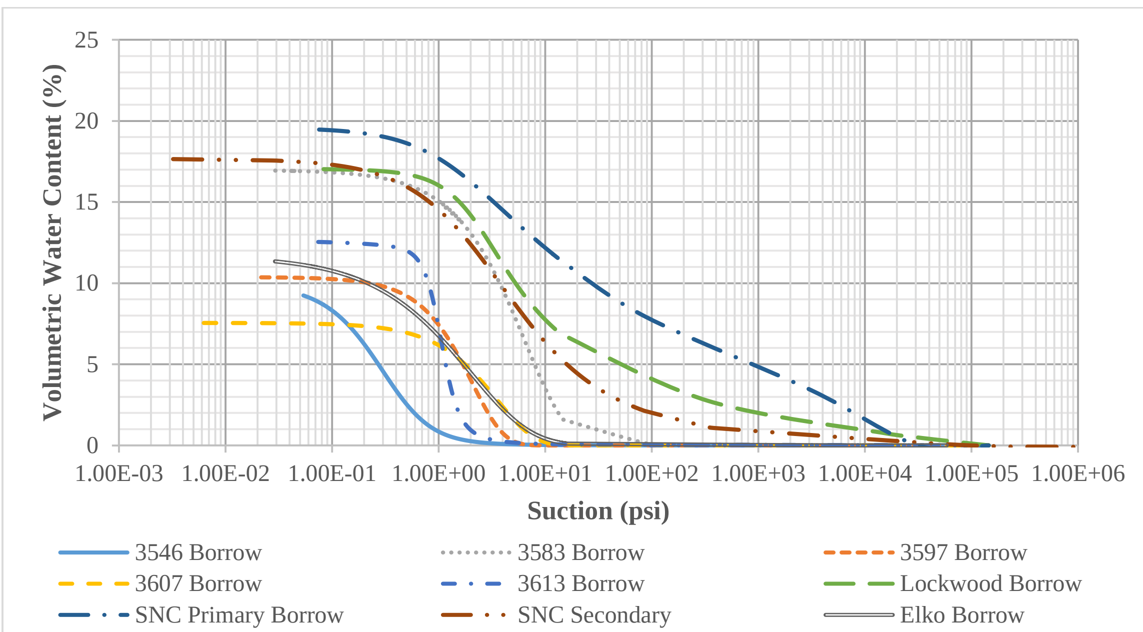


Figure 57. Borrow material SWCC (input level 1).

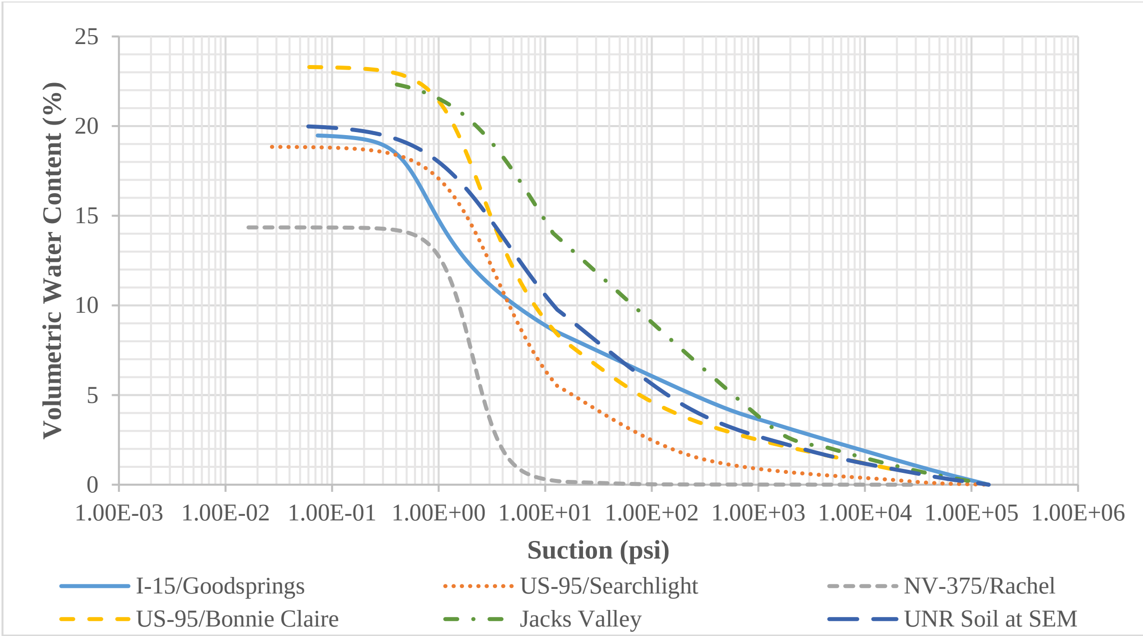


Figure 58. Subgrade material SWCC (input level 1).

The SWCC parameters were also found using input level 2, which is an estimation method relating SWCC to gradation and PI, as described in Chapter 2. The SWCC parameters resulting from input level 2 are shown in table 24 to table 26 for base, borrow, and subgrade material, respectively. The corresponding plots for input level 2 SWCCs are shown in figure 59 to figure 61 for base, borrow, and subgrade material, respectively.

Table 24. Base Material SWCC Parameters (Input Level 2).

Sample	a_f	b_f	c_f	h_r
3546	0.264	7.500	0.597	0.482
3583	0.348	7.500	0.532	0.803
3597	0.329	7.500	0.545	0.727
3605	0.250	7.500	0.610	0.435
3607	0.252	7.500	0.608	0.441
3613	0.252	7.500	0.608	0.441
Lockwood Base	0.319	7.500	0.552	0.686
Elko Base	0.424	7.500	0.485	1.132
Hunnewill base	0.340	7.500	0.537	0.771
Average	0.314	7.500	0.560	0.680
Standard Deviation	0.057	0.000	0.042	0.225

Table 25. Borrow Material SWCC Parameters (Input Level 2).

Sample	a_f	b_f	c_f	h_r
3546	27.969	1.038	0.711	3.750E+04
3583	1.230E+05	–0.217	1.245	1.383E+09
3597	3.713E+02	0.792	0.829	3.296E+04
3607	2.55	1.424	0.565	85.269
3613	1.595	5.000	0.500	51.753
Lockwood Borrow	5.758E+04	–0.0143	1.171	1.975E+08
SNC Primary	8.280E+04	–0.107	1.205	4.849E+08
SNC Secondary	0.285	7.500	0.579	0.555
Elko Borrow	0.622	7.500	0.395	2.104
Average	2.931E+04	3.876	0.800	2.295E+08
Standard Deviation	4.418E+04	2.919	0.310	4.364E+08

Table 26. Subgrade Material SWCC Parameters (Input Level 2).

Sample	a_f	b_f	c_f	h_r
I-15/Goodsprings	14.576	1.106	0.68	622.687
US-95/Searchlight	34.936	1.016	0.722	1.746E+03
NV-375/Rachel	6.127	1.228	0.628	227.045
US-95/Bonnie Claire	1.279	7.5	0.225	5.736
US-93/Crystal Spring MP62	–	–	–	–
US-93/Crystal Spring MP67	–	–	–	–
Jacks Valley	590.04	0.743	0.852	6.137E+04
UNR Soil at SEM	135.88	0.89	0.782	9.085E+03
Average	130.473	2.081	0.648	1.218E+04
Standard Deviation	210.532	2.429	0.202	2.222E+04

–No data.

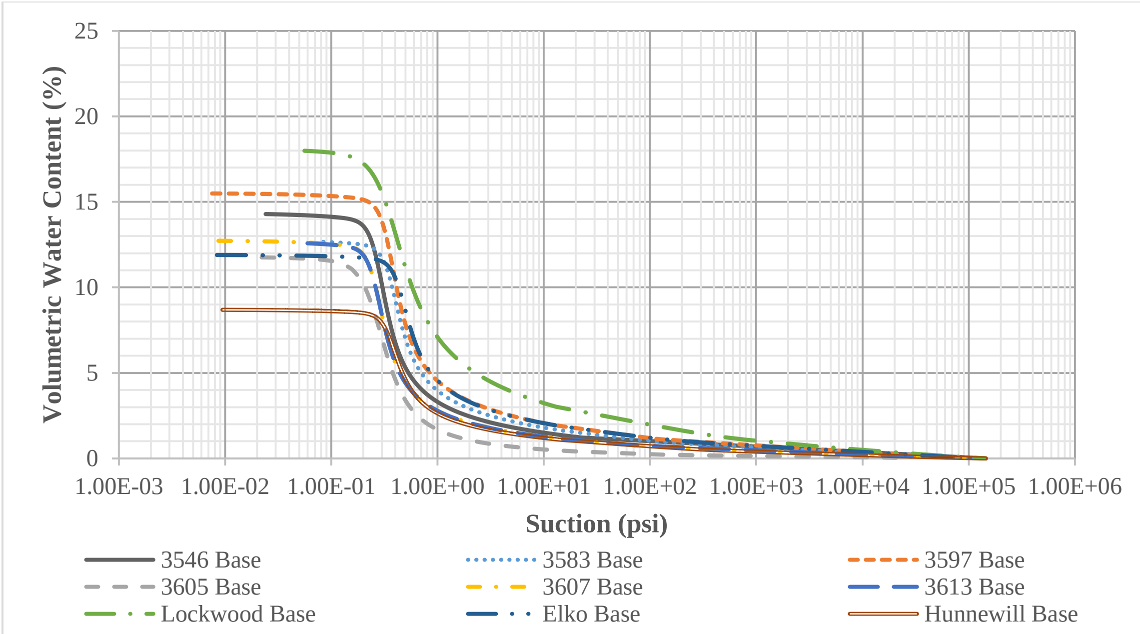


Figure 59. Base material SWCC (input level 2).

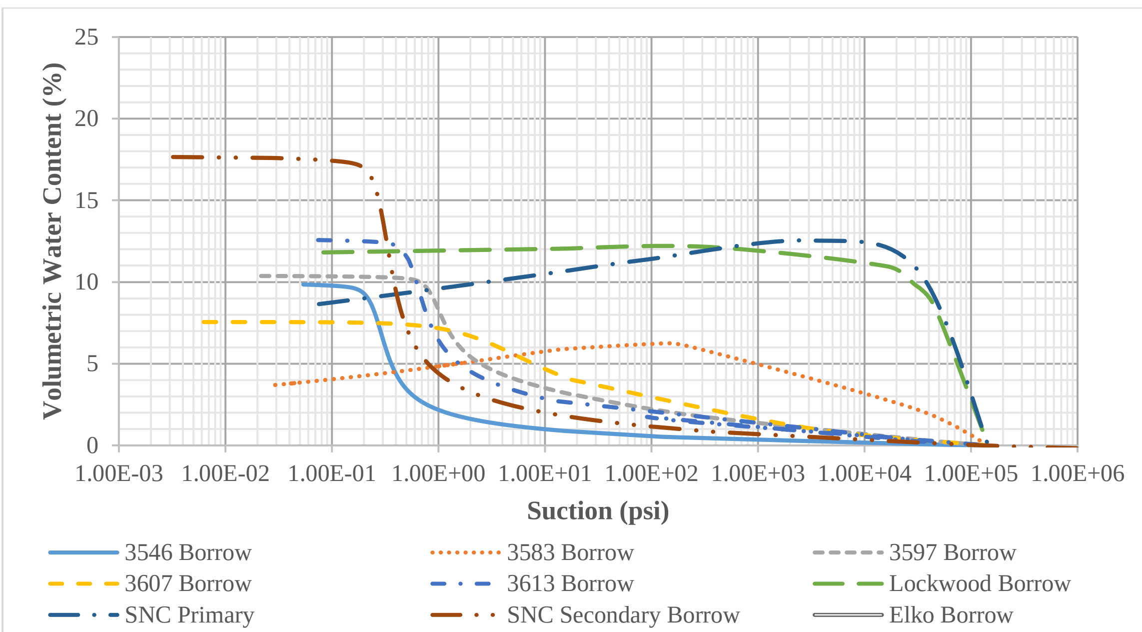


Figure 60. Borrow material SWCC (input level 2).

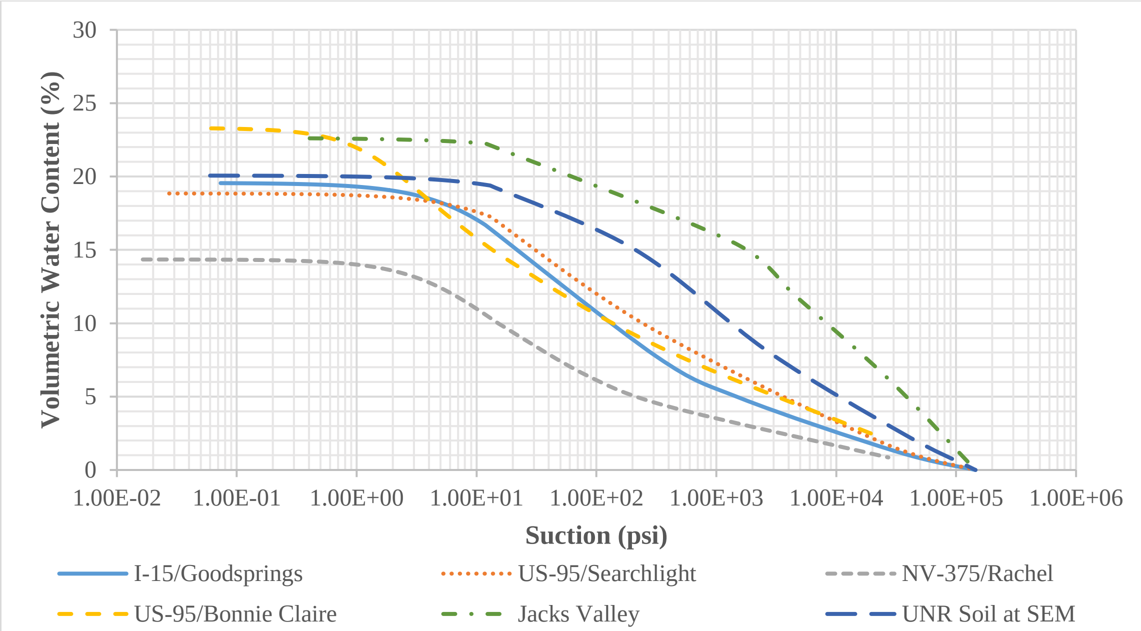


Figure 61. Subgrade material SWCC (input level 2).

MBV and PFC were also measured for District 1 borrow and subgrade materials. Using the Texas A&M estimation method, as described in Chapter 2, SWCC parameters could be determined. The resulting SWCC parameters are shown in table 27 and table 28 for District 1 borrow and subgrade materials, respectively. The SWCCs generated using these estimated parameters are shown in figure 62 and figure 63. The individual SWCCs for each material, including input level 1, input level 2, the Texas A&M Estimation, as well as the calibrated input level 2 that will be discussed later, can all be found in the Appendix.

Table 27. Borrow Material SWCC Parameters (Texas A&M Estimation Method).

Sample	a_f	b_f	c_f	h_r
3546	0.679	11.045	0.124	3.294
3583	0.703	10.667	0.127	3.777
3597	0.752	9.965	0.132	4.712
3607	0.638	11.748	0.120	2.431
3613	0.724	10.356	0.129	4.184
Average	0.699	10.756	0.126	3.680
Standard Deviation	0.039	0.610	0.004	0.779

Table 28. Subgrade Material SWCC Parameters (Texas A&M Estimation Method).

Sample	a_f	b_f	c_f	h_r
I-15/Goodsprings	0.704	10.657	0.127	3.789
US-95/Searchlight	0.638	11.748	0.120	2.431
NV-375/Rachel	0.740	10.132	0.131	4.484
US-95/Bonnie Claire	0.793	9.453	0.136	5.426
Average	0.719	10.498	0.129	4.033
Standard Deviation	0.056	0.839	0.006	1.092

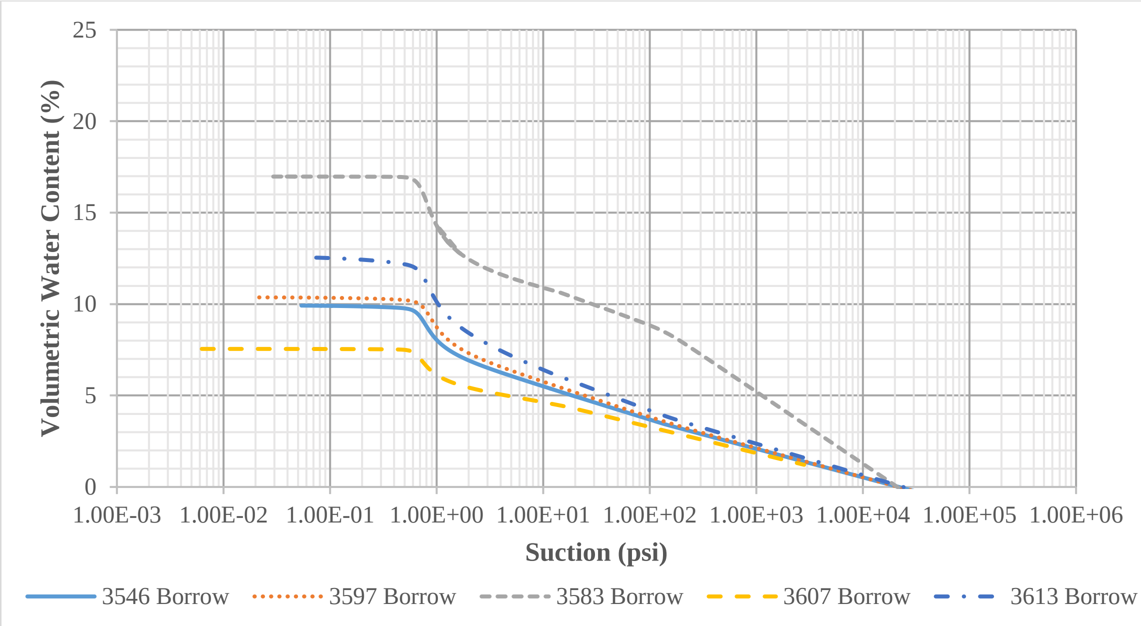


Figure 62. Borrow material SWCC (Texas A&M estimation method).

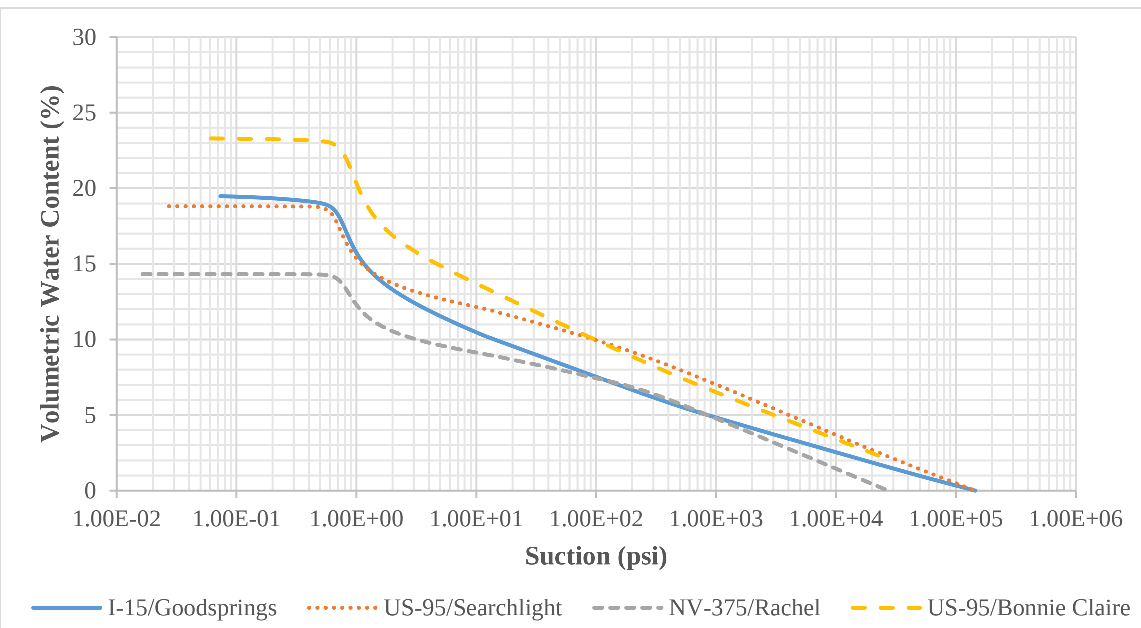


Figure 63. Subgrade material SWCC (Texas A&M estimation method).

As discussed in Chapter 2, the soil sample desaturates in three major stages. The first stage is the capillary zone which extends to the air entry value, where the largest pores in the sample begin to fill with air but the pore water saturates the sample and is held under suction from capillary forces. The a_f parameter is closely related to the air entry value. The second stage is known as the desaturation zone, which is between the air entry value and the residual water content. In this zone, there is a dramatic decrease in the volumetric water content and air increasingly fills the pores of the sample. The b_f and c_f control the slope of the curve in this stage. The slope describes the rate at which the moisture is lost from the sample. The h_r is the suction corresponding to the residual water content. This is the point as which a high change in suction is necessary to reduce the volumetric water content (17).

In comparing the different types of materials, the base material shows lower average a_f value than the borrow (a_f of 5.042 psi compared to 16.16 psi), but slightly higher than the subgrade materials (2.04 psi). Coarse grained materials tend to exhibit lower air entry values than finer grained materials (17). Essentially, due to higher maximum dry densities, water that is being held in the lower suction ranges can evaporate very quickly, leading to a lower air entry value.

The b_f and c_f describe the slope of the SWCC between the air entry value and residual water content. The base material had the lowest average b_f (1.455), followed by the borrow material (1.529), and then the subgrade (2.049). The subgrade material had the lowest average c_f (1.008), followed by the base material (2.717), and then the borrow material (14.328). When b_f is higher, this means that the rate at which moisture is lost from the sample is increased. Therefore, after the air entry value is reached, the subgrade material is experiencing the highest rate of moisture loss, followed by borrow material, then base material.

The parameter h_r is the suction that corresponds to the residual water content, where a great increase in suction is required to reduce the water content. The average h_r was highest for the base material (367.821 psi), followed by borrow material (104.104 psi), and finally by the subgrade material (33.206 psi). There is a high level of variation in h_r in comparison with the other parameters. As discussed, h_r is the suction corresponding to the residual volumetric water content; therefore, it is near the dry end of the SWCC. This is the portion of the SWCC where there is the transition between HYPROP and WP4C measurements. While the HYPROP measures matric suction, the WP4C measures total suction, which is matric suction plus osmotic suction. Osmotic suction is caused by salts or contamination in the sample. Because the WP4C measures total suction, it is sometimes necessary to correct these measurements, using pedotransfer functions (18). These are functions that predict the SWCC based on soil properties. Sometimes, the issue with the WP4C measurements can be that the SWCC prediction model can under predict the suction for the measurements. However, this ended up not being the case here.

As discussed in Chapter 2, the Fredlund and Xing equation is based on soil properties, specifically grain size distribution, and the saturated hydraulic conductivity is related directly to maximum dry density and specific gravity of solids (6, 19). Therefore, it was determined that pedotransfer functions were not needed to correct the WP4C measurements. However, it should be noted that the variation in h_r could partially be due to some presence of osmotic suction.

In comparing the input level 1 to input level 2 for base material, input level 2 generally underestimates parameter a_f, c_f, and h_r while it overestimates b_f. In comparing input level

1 to input level 2 for borrow material, input level 2 overestimates a_f. This difference is great for borrow materials with high P_{200}, such as Contract 3583 and the Lockwood Borrow. For borrow material, input level 2 tends to overestimate b_f in some cases and underestimate it in other cases. The underestimation cases are, again, where P_{200} is higher. Input level 2 underestimates c_f for borrow material and overestimates h_r. This is especially true for the materials where the PI is greater than 0. For subgrade materials, input level 2 generally overestimates a_f and h_r while it underestimates b_f and c_f. For both the District 1 borrow and subgrade materials that were tested for MBV and PFC, the Texas A&M estimation method tends to underestimate a_f, c_f, and h_r, and it overestimates b_f.

It is important to note that often a large-particle correction is applied to the SWCC. A common method is the Bouwer-Rice method (20). A simplified version of this method is presented by Bareither and Benson, where the SWCC is fitted using van Genuchten equation to define shape parameters α and n (21). The field application is found using the fitted α and n. This method works to correct the saturated volumetric water content for more coarse-grained material. While the calculations used in the MEPDG do not allow for this correction to be made, an effort to correct the saturated volumetric water content was made in this study by using the corrected maximum dry density and corrected optimum water content where applicable. A coarse aggregate correction is made in the moisture–density testing where appropriate, to ensure that the results reflect the entire aggregate blend. Further research should be conducted to assess if this procedure can accurately correct the SWCC for coarse aggregate.

Methylene Blue Value and Percent Fines Content

The results from the MBV testing conducted by Texas A&M are shown in table 29. These results were used to calculate PFC, which was used to estimate SWCCs. The PFC is calculated using equation 26. Results from PFC testing are shown in table 29.

Table 29. MBV Results for District 1 Borrow and Subgrade Materials.

Sample	MBV	PFC
3546 (Borrow)	0.00	1.668
3583 (Borrow)	4.32	2.610
3597 (Borrow)	0.00	4.450
3607 (Borrow)	–	–
3613 (Borrow)	0.00	3.408
I-15/Goodsprings (Subgrade)	9.34	2.634
US-95/Searchlight (Subgrade)	0.00	2.580
NV-375/Rachel (Subgrade)	7.36	4.000
US-95/Bonnie Claire (Subgrade)	7.17	5.874

–No data.

Saturated Hydraulic Conductivity

Saturated hydraulic conductivity is the ability of a saturated soil to conduct water while it is subjected to a hydraulic gradient. The Texas A&M team conducted saturated hydraulic conductivity testing by measuring diffusivity. This testing was performed on only District 1 borrow and subgrade materials. Due to unforeseen circumstances, District 2 and District 3 materials could not undergo this testing; therefore, the saturated hydraulic conductivity from the UNR SWCC testing was used for these materials, and the results are shown in table 30 to table 32 for base, borrow, and subgrade materials, respectively. The HYPROP apparatus is able to measure hydraulic conductivity in addition to soil suction, so the saturated hydraulic conductivity results presented are the hydraulic conductivity readings at full saturation.

Table 30. Base Material Saturated Hydraulic Conductivity.

Sample	Saturated Hydraulic Conductivity (ft/hr)
3546	9.68E-04
3583	6.32E-05
3597	8.57E-04
3605	5.32E-06
3607	1.68E-03
3613	2.86E-04
Lockwood Base	3.25E-04
Elko Base	5.95E-04
Hunnewill base	1.05E-03

Table 31. Borrow Material Saturated Hydraulic Conductivity.

Sample	Saturated Hydraulic Conductivity (ft/hr)
3546	5.44E-04
3583	4.13E-04
3597	5.20E-04
3607	4.32E-04
3613	3.94E-04
Lockwood Borrow	2.50E-04
SNC Primary	6.30E-05
SNC Secondary	4.03E-04
Elko Borrow	3.85E-04

Table 32. Subgrade Material Saturated Hydraulic Conductivity.

Sample	Saturated Hydraulic Conductivity (ft/hr)
I-15/Goodsprings	8.65E-04
US-95/Searchlight	3.20E-05
NV-375/Rachel	7.12E-04
US-95/Bonnie Claire	1.19E-04
Jacks Valley	6.95E-05
UNR Soil at SEM	4.62E-04

Resistance R-value

A summary of the R-value testing results for the evaluated materials are shown in Table 33 to Table 35. According to NDOT specifications, Type 1 Class B Base Materials must have a R-value of at least 70. All of the tested base materials meet this minimum specification. Borrow materials must have a R-value of 45. All of the tested borrow materials meet this minimum specification, except for Contract 3583 borrow from District 1.

Table 33. Resistance R-value Test Results for Base Materials (All Districts).

Material	Sam-ple No.	Density (pcf)	Moisture Content (%)	Exudation Pressure (psi)	R-value	R-value Corr.	R-value @300 psi Exudation Pressure
3583 (Base)	1	138.9	6.8	100	79	78	80
	2	138.1	5.8	333	81	80	
	3	140.2	5.5	518	83	82	
3597 (Base)	1	121.0	3.9	608	82	82	71
	2	125.6	4.5	478	77	75	
	3	127.3	4.8	204	73	71	
3605 (Base)	1	132.4	5.4	354	83	81	78
	2	135.3	5.2	540	86	86	
	3	134.0	6.0	275	77	77	
3607 (Base)	1	125.7	6.6	530	85	85	85
	2	124.3	7.6	298	85	85	
	3	122.9	7.2	175	84	84	
3613 (Base)	1	135.0	5.0	699	87	87	83
	2	138.7	5.9	204	84	82	
	3	136.3	5.5	388	85	84	
Lockwood Base	1	124.1	7.8	541	84	84	84
	2	130.2	8.4	340	86	84	
	3	129.2	8.9	228	83	83	
Elko Base	1	129.9	6.6	755	86	86	78
	2	128.6	7.5	444	79	79	
	3	125.2	8.2	100	76	76	
Hunnewill Base	1	129.0	6.8	723	82	82	73
	2	127.9	7.7	340	75	75	
	3	129.4	9.0	107	65	65	

Table 34. Resistance R-value Test Results for Borrow Materials (All Districts).

Material	Sam-ple No.	Density (pcf)	Moisture Content (%)	Exudation Pressure (psi)	R-value	R-value Corr.	R-value @300 psi Exudation Pressure
3546 (Borrow)	1	123.8	5.0	727	84	84	78
	2	123.4	6.5	441	82	82	
	3	124.2	6.9	287	79	78	
3583 (Borrow)	1	116.8	13.5	125	32	32	44
	2	119.0	11.8	734	70	70	
	3	118.6	12.6	355	47	47	
3597 (Borrow)	1	136.1	8.1	149	74	71	78
	2	134.4	7.2	731	85	85	
	3	137.0	7.8	411	83	82	
3607 (Borrow)	1	119.7	13.0	100	57	57	78
	2	119.3	12.2	271	76	76	
	3	120.1	11.1	587	81	81	
3613 (Borrow)	1	138.3	5.9	361	85	85	84
	2	139.6	6.7	227	83	83	
	3	141.5	5.5	566	85	85	
Lockwood (Borrow)	1	119.8	13.8	0.7	66	64	69
	2	117.5	15.4	0.45	53	53	
	3	119.1	13.3	1.88	80	79	
Elko (borrow)	1	–	–	–	–	–	74
	2	121.0	8.4	405	76	76	
	3	120.8	9.1	103	64	64	
SNC Primary Borrow	1	129.4	10.5	176	47	50	71
	2	128.2	9.2	639	84	84	
	3	127.6	10.0	340	77	77	
SNC Secondary Borrow	1	125.2	8.6	643	86	86	76
	2	123.2	9.4	406	76	76	
	3	128.7	10.4	124	81	81	

–No Data.

Table 35. Resistance R-value Test Results for Subgrade Materials (All Districts).

Material	Sample No.	Density (pcf)	Moisture Content (%)	Exudation Pressure (psi)	R-value	R-value Corr.	R-value @300 psi Exudation Pressure
I-15/Goodsprings	1	131.9	7.9	188	78	78	82
	2	129.5	7.2	468	82	82	
	3	130.8	7.5	268	81	81	
US-95/Searchlight	1	130.9	8.4	148	71	69	75
	2	130.1	7.9	682	80	80	
	3	130.7	8.2	254	74	74	
NV-375/Rachel	1	129.5	8.8	302	80	81	80
	2	130.7	9.5	171	76	76	
	3	130.3	8.1	663	85	85	
US-95/Bonnie Claire	1	121.8	11.4	172	72	71	74
	2	121.1	10.2	719	74	74	
	3	120.9	10.6	391	75	75	
US-93/Crystal Spring MP62	1	119.2	10.5	404	80	81	74
	2	119.8	10.9	225	66	68	
	3	119.5	9.9	694	78	78	
US-93/Crystal Spring MP67	1	120.5	11.3	231	51	51	71
	2	120.8	10.8	323	77	77	
	3	119.6	10.1	628	78	78	
Jacks Valley Subgrade	1	121.2	11.4	727	78	78	60
	2	121.3	13.6	366	68	68	
	3	115.6	14.4	172	40	40	
UNR Soil at SEM	1	132.2	9.0	365	77	75	65
	2	131.7	8.4	529	82	81	
	3	132.3	9.9	219	47	47	

Summary of Laboratory Evaluation Results

Each of the tests were conducted in accordance with the appropriate standard procedure. All materials met the specifications stated in the NDOT Specifications Book (22). Base materials have very tight specification limits, as compared to borrow materials, which is evident in the low level of variation in the gradation results. All base materials were non-plastic, and most borrow and subgrade materials were slightly plastic. The base material exhibited the highest maximum dry densities and lowest optimum water contents overall. The subgrade showed the lowest maximum dry densities and highest optimum water contents. The borrow materials were highly variable with maximum dry density and optimum water content. There were differences between the SWCC parameters found through direct measurement and estimation methods. The impact of these differences will be investigated further in the following section.

SENSITIVITY ANALYSIS

Using the AASHTOWare Pavement ME Design software (ver. 2.3.1), a sensitivity analysis was conducted. This was done to evaluate the influence of the developed database for SWCC and saturated hydraulic conductivity on NDOT MEPDG designs.

The sensitivity analysis was performed for a new flexible pavement design. This section discusses the material properties, traffic, and climate data that were used, as well as define the performance criteria. Input levels 1, 2, 3, and current NDOT SWCC recommendations were assessed.

Under input level 1 in the Pavement ME software, the properties defined for the unbound material layers are gradation, LL, PI, maximum dry unit weight, optimum moisture content, specific gravity of solids, saturated hydraulic conductivity, and SWCC fitting parameters. Under input level 2, gradation, LL, PI, maximum dry unit weight, optimum moisture content, and specific gravity of solids are defined by the user. The EICM then internally calculates saturated hydraulic conductivity and SWCC parameters. Under input level 3, which is the current recommendation used in the "Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design," only the gradation, LL, and PI are defined by the user (1). The EICM then internally calculates the maximum dry unit weight, optimum moisture content, specific gravity of solids, saturated hydraulic conductivity, and SWCC parameters.

General Information

Table 36 summarizes the inputs used for performance criteria in accordance with the "Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design." The limit for each performance criteria is defined, as well as its reliability. In comparison with the national performance criteria, lower limits are defined. This is true for initial IRI, terminal IRI, AC bottom-up fatigue cracking, permanent deformation in the total pavement structure, and permanent deformation in the AC layer. Depending on the expected traffic, reliability is also increased in Nevada in comparison to the national performance criteria reliability. This is true for terminal IRI, AC bottom-up fatigue cracking, and permanent deformation in the AC layer and in the total pavement structure. The reliability is lower in Nevada for AC top-down fatigue cracking and AC thermal cracking since these models are not calibrated to Nevada's conditions and are excluded from the design criteria.

Table 36. New Flexible Pavement Design Performance Criteria.

Performance Criteria	Limit	Reliability
Initial IRI (inch/mile)	60	–
Terminal IRI (inch/mile)	170	95
AC top-down fatigue cracking (feet/mile)	2,000	50
AC bottom-up fatigue cracking (%)	15	95
AC thermal cracking (feet/mile)	1,000	50
Permanent deformation–total pavement (inch)	0.5	95
Permanent deformation–AC only (inch)	0.15	95

–Not applicable.

The performance models were calibrated to account for local conditions for materials, climate, and traffic in Nevada. Using performance model calibration factors that are region specific is necessary, as the climate, materials, and traffic for each NDOT District differs. The local calibration was limited to the AC bottom-up fatigue cracking and the rutting models. Sensitivity analyses conducted on the thermal cracking model have shown some issues in its ability to correctly predict the thermal cracking measurements that have been made in the field. For that reason, it is recommended to not use the thermal cracking predictions when designing flexible pavements using Pavement ME in Nevada. Additionally, it has been shown that AC top-down fatigue cracking cannot be accurately predicted by fatigue mechanisms that are used to define cracking that starts at the bottom of the AC layer. Therefore, AC top-down fatigue cracking should also be excluded from the design and analysis. The two primary performance criteria considered in this analysis are for AC bottom-up fatigue cracking and permanent deformation.

Table 37 and table 38 summarize the traffic information used for each NDOT district. The user defined inputs include the two-directional AADTT, the percentage of trucks in each direction, the number of lanes per direction, the lane distribution factor, average annual growth rate, operational speed, and the distribution of truck traffic for each FHWA vehicle class (class 4 through 13). The traffic data used for District 1 came from US 95 near Las Vegas (23). For District 2, the traffic data came from US 395 near Carson City. The traffic data for District 3 came from I-80 near Elko and was extracted from NDOT “2016 Vehicle Class Distribution Report” (24).

Table 37. Traffic Data Used for Each District.

Property	District 1	District 2	District 3
Two-Directional average annual daily traffic (AADT)	2,350	1,594	1,496
Directional split, %	50	50	50
Number of lanes per direction	2	2	2
Lane distribution factor (80-100), %	90	90	90
Average annual growth rate, %	3	3	3
Operational Speed, mph	60	60	60

Table 38. FHWA Vehicle Class Distribution for Each District.

Districts	FHWA Vehicle Class (%)									
	4	5	6	7	8	9	10	11	12	13
1	4.59	11.73	1.79	0.01	8.29	57.53	1.4	2.55	1.4	10.71
2	16.13	40.63	5.4	0.01	12.04	19.51	0.12	3.85	0.65	1.66
3	16.97	17.38	0.86	0.86	2.81	6.96	2.41	0.54	0.27	50.94

The Pavement ME software contains a database of over 800 weather stations throughout the US. It is recommended to use a weather station within 100 miles of the project site. Therefore, for District 1, the Las Vegas weather station was used. The Reno weather station was used for District 2, and the Elko weather station was used for District 3. The depth to the water table should be defined by the user. The depth to the water table is defined as the distance from the water table to the project in feet. Specifically, it is determined to be the distance from the top of the subgrade to the water table level. Water table levels can be found using the United States Geological Survey (USGS) website (25).

The material properties defined for the AC layer are shown in table 39, with respect to each district. Each property was defined using the “Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design.” The representative mean dynamic modulus data in psi for the mixture used in each district is shown in table 40 to table 42. The representative mean shear modulus and phase angle data for each asphalt binder used in each district is shown in table 43 to table 45.

Table 39. AC Layer Properties for Pavement ME by District.

Parameter	District 1	District 2	District 3
Unit weight (pcf)	150	145	145
Effective binder content (%)	8.5	8.5	8.5
Air voids (%)	7	7	7
Poisson's ratio	0.35	0.35	0.35
Dynamic Modulus	Table 40	Table 41	Table 42
Reference Temperature (°F)	70	70	70
Asphalt Binder	Table 43	Table 44	Table 45
Indirect Tensile Strength at 14°F	Level 3	Level 3	Level 3
Creep Compliance	Level 3	Level 3	Level 3
Thermal Conductivity (BTU/hr-ft-°F)	0.67	0.67	0.67
Heat Capacity (BTU/lb-deg F)	0.23	0.23	0.23
Thermal Contraction (in/in/°F)	5 E-06	5 E-06	5 E-06

Table 40. Representative Dynamic Modulus Values in psi for District 1, PG 76-22NV Mixture.

Temperature (°F)	0.1 Hz	0.5 Hz	1 Hz	5 Hz	10 Hz	25 Hz
14	2,437,149	2,796,769	2,929,984	3,189,069	3,280,392	3,384,391
40	1,142,867	1,566,757	1,786,152	2208295	2,398,327	2,819,783
70	231,733	371,867	459,860	700905	841,850	1,041,907
100	49,451	79,212	99,621	174052	225,042	335,073
130	22,928	29,081	38,053	65800	77,131	107,196

Table 41. Representative Dynamic Modulus Values in psi for District 2, PG 64-28NV Mixture.

Temperature (°F)	0.1 Hz	0.5 Hz	1 Hz	5 Hz	10 Hz	25 Hz
14	1,631,380	2,008,344	2,164,343	2,500,790	2,632,250	2,792,318
40	628,946	885,602	1,008,706	1,324,511	1,472,121	1,685,424
70	122,675	212,544	264,370	436,082	526,218	678,018
100	25,282	41,756	52,208	97,192	126,317	183,386
130	12,340	17,689	23,032	34,827	44,416	71,565

Table 42. Representative Dynamic Modulus Values in psi for District 3, PG 64-28NV Mixture.

Temperature (°F)	0.1 Hz	0.5 Hz	1 Hz	5 Hz	10 Hz	25 Hz
14	1,727,052	2,107,737	2,263,477	2,595,567	2,723,831	2,878,790
40	661,937	934,530	1,066,170	1,385,400	1,528,233	1,751,700
70	124,687	213,457	266,323	442,423	538,683	706,700
100	34,902	54,718	67,373	118,600	151,013	222,847
130	14,977	20,178	23,423	39,520	50,332	74,025

Table 43. Representative Shear Modulus and Phase Angle Values for District 1, PG 72-22NV Asphalt Binder.

Temperature (°F)	Binder G* (Pa)	Phase angle (°)
147.2	7,355	58.9
158.0	4,638	58.4
168.8	2,873	60.0

Table 44. Representative Shear Modulus and Phase Angle Values for District 2, PG 64-28NV Asphalt Binder.

Temperature (°F)	Binder G* (Pa)	Phase angle (°)
136.4	5,880	64.0
147.2	3,281	65.0
158.0	1,882	66.0

Table 45. Representative Shear Modulus and Phase Angle Values for District 3, PG 62-28NV Asphalt Binder.

Temperature (°F)	Binder G* (Pa)	Phase angle (°)
136.4	5,719	65.9
147.2	3,145	67.1
158.0	1,783	68.6

The pavement design used coincided with the pavement design used in the recent NDOT study on resilient moduli. The same design was used for all districts, and is comprised of three layers: a surface AC layer, an aggregate base layer, and a subgrade layer. The subgrade layer is semi-infinite in thickness, the aggregate base layer is 16 inches in thickness, and the AC layer varied in thickness; 6 inch was used to begin with, and the AC layer thickness was then optimized through an iterative process in Pavement ME. For District 1, Pavement ME runs were conducted for AC thickness of 6.5, 7, and 7.5 inches. For District 2, the runs were conducted for AC thickness of 6, 6.5, and 7 inches. For District 3, the runs were performed for AC thickness of 7 and 7.5 inches. The thicknesses were chosen so that for each District, there would be one thickness that caused the pavement design to pass required performance criteria, and one that caused the pavement design to fail the AC bottom-up fatigue cracking criteria. In the case of District 1 and District 2, it was necessary to conduct Pavement ME runs at three AC layer thicknesses in order to simulate this wide range of performance. In the case of District 3, the 7.5 inch AC layer thickness run passed all performance criteria, and the 7 inch AC layer thickness run failed all AC bottom-up fatigue cracking criterion, so these two extremes were easily found. Figure 64 shows the pavement structural section used in sensitivity analysis effort.

Figure 64. Pavement design used for sensitivity analysis.

Pavement ME Analysis Using SWCC Input Level 1

The material properties used for each base material for input level 1 is summarized in table 46. For District 1, the base material from Contract 3605 was selected. The SWCC for the Contract 3605 base material was in the mid-range of all the SWCCs for District 1 base materials. The Lockwood Base was used from District 2, as it was the only base material sampled from District 2. The Elko Base was used for District 3. The only borrow material sampled from District 3 was the Elko Borrow, so the Elko Base was chosen because of the likelihood that these two materials would be used together on a project.

Poisson’s ratio, coefficient of lateral earth pressure, and resilient modulus values were taken from the “Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design” (1). When defining the resilient modulus, input level 2 was used. In order to evaluate the impact of SWCCs and saturated hydraulic conductivity on the pavement structure, it is critical to choose the option to vary the resilient modulus with temperature/climate. When choosing this option, this will allow the EICM to alter the resilient modulus by considering soil suction, temperature, and moisture through the composite factor, F_{env}, as previously discussed in Chapter 1. Gradation, Atterberg limits, maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum water content, and SWCC parameters were directly measured. The gradation data for each base material is shown in table 47. The SWCC parameters for each base material is shown in table 48 and the saturated hydraulic conductivity for each base material is shown in table 49.

Table 46. Base Material Properties for Input Level 1 (New Flexible Design).

Parameter	Contract 3605 (District 1)	Lockwood Base (District 2)	Elko Base (District 3)
Poisson's ratio	0.35	0.35	0.35
Coefficient of lateral earth pressure (k0)	0.5	0.5	0.5
Resilient Modulus (psi)	26,000	26,000	26,000
Gradation	Table 47	Table 47	Table 47
Plasticity Index	0	0	0
Liquid Limit	0	0	0
Maximum dry unit weight (pcf)	149.7	138.2	141.1
Saturated hydraulic conductivity (ft/hr)	Table 49	Table 49	Table 49
Specific gravity of solids	2.720	2.703	2.439
Optimum gravimetric water content (%)	4.7	8.0	5.8
SWCC	Table 48	Table 48	Table 48

Table 47. Base Material Gradations for All Input Levels.

Sieve Size	Contract 3605 (District 1)	Lockwood Base (District 2)	Elko Base (District 3)
25.0 mm (1 inch)	100.0	100.0	100.0
19.0 mm (3/4 inch)	90.2	96.7	99.7
12.5 mm (1/2 inch)	66.3	79.2	92.5
9.5 mm (3/8 inch)	54.1	68.5	83.1
4.75 mm (No. 4)	35.3	46.6	59.0
2.36 mm (No. 8)	25.1	33.6	43.3
2.00 mm (No. 10)	23.3	31.3	39.8
1.18 mm (No. 16)	19.0	25.2	31.6
0.6 mm (No. 30)	15.0	19.6	22.0
0.425 mm (No. 40)	13.5	16.6	17.7
0.3 mm (No. 50)	12.1	13.7	13.8
0.15 mm (No. 100)	9.9	10.0	9.7
0.075 mm (No. 200)	7.7	7.8	7.5

Table 48. Base Material SWCC Parameters for Input Level 1.

SWCC Parameter	Contract 3605 (District 1)	Lockwood Base (District 2)	Elko Base (District 3)
a_f	1.650	0.374	39.468
b_f	0.996	1.337	0.649
c_f	2.968	0.478	12.727
h_r	6.665	2.599	1,500

Table 49. Base Material Saturated Hydraulic Conductivity for Input Level 1.

SWCC Parameter	Contract 3605 (District 1)	Lockwood Base (District 2)	Elko Base (District 3)
Saturated Hydraulic Conductivity (ft/hr)	5.32E-04	3.25E-04	5.95E-04

The subgrade material properties for input level 1 are summarized in table 50. For District 1, the US-95/Searchlight subgrade was used in the analysis. This subgrade has an AASHTO Soil Classification of A-2-4, which is a common subgrade used in District 1. For District 2, the Jacks Valley subgrade was used. Two subgrades were sampled in District 2, including the Jacks Valley and the UNR soil at SEM. The Jacks Valley subgrade is more appropriate to use, as it was sampled near US 395, which is where the traffic data came from. No true subgrade was sampled from District 3, so the borrow material from Elko was used in this analysis.

Poisson’s ratio, coefficient of lateral earth pressure, and resilient modulus values were taken from the “Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design.” The first part of the input level 1 sensitivity analysis involved defining the resilient modulus at optimum moisture content and allowing the EICM to vary the modulus with temperature and climate conditions. The second part of the input level 1 sensitivity analysis involved defining the subgrade resilient modulus for each season using the seasonal coefficients defined in the “Manual for Designing Flexible Pavements in Nevada Using AASHTOWare Pavement-ME Design.” This was done so that the impact of the SWCC and saturated hydraulic conductivity could be evaluated for both the base and subgrade, and then that impact could also be evaluated just in the base material layer. Gradation, Atterberg limits, maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum water content, and SWCC parameters were directly measured.

Table 50. Subgrade Material Properties - Input Level 1.

Parameter	**US-95/ Searchlight (District 1)**	**Jacks Valley (District 2)**	**Elko Borrow Used as Subgrade (District 3)**
Poisson's ratio	0.4	0.4	0.4
Coefficient of lateral earth pressure (k0)	0.5	0.5	0.5
Resilient modulus (psi)	10,000	10,000	10,000
Gradation	Table 51	Table 51	Table 51
Plasticity Index	4.3	2.4	0.0
Liquid Limit	30.9	22.9	0.0
Maximum dry unit weight (pcf)	134.9	125.5	139.7
Saturated hydraulic conductivity (ft/hr)	Table 53	Table 53	Table 53
Specific gravity of solids	2.688	2.604	2.495
Optimum gravimetric water content (%)	6.3	9.4	6.0
SWCC	Table 52	Table 52	Table 52

Table 51. Subgrade Materials Gradations for All Input Levels.

Sieve Size	US-95/Searchlight (District 1)	Jacks Valley (District 2)	Elko Borrow Used as Subgrade (District 3)
37.5 mm (1.5 inch)	100.0	100.0	100.0
25.0 mm (1 inch)	87.5	100.0	87.3
19.0 mm (3/4 inch)	79.3	100.0	82.0
12.5 mm (1/2 inch)	–	100.0	74.6
9.5 mm (3/8 inch)	52.2	100.0	68.9
4.75 mm (No. 4)	33.5	99.7	53.4
2.36 mm (No. 8)	–	97.8	40.9
2.00 mm (No. 10)	23.2	96.8	37.4
1.18 mm (No. 16)	–	93.0	29.3
0.6 mm (No. 30)	–	81.8	18.8
0.425 mm (No. 40)	15.2	72.3	13.8
0.3 mm (No. 50)	13.4	61.0	10.0
0.15 mm (No. 100)	9.6	42.3	6.5
0.075 mm (No. 200)	5.4	26.1	4.9

–No data.

Table 52. Subgrade Materials SWCC Parameters for Input Level 1.

SWCC Parameter	US-95/Searchlight (District 1)	Jacks Valley (District 2)	Elko Borrow Used as Subgrade (District 3)
a_f	2.311	3.817	32.698
b_f	2.053	1.142	0.827
c_f	0.755	0.733	27.204
h_r	1.158	36.600	201.215

Table 53. Subgrade Materials Saturated Hydraulic Conductivity for Input Level 1.

SWCC Parameter	US-95/Searchlight (District 1)	Jacks Valley (District 2)	Elko Borrow Used as Subgrade (District 3)
Saturated Hydraulic Conductivity (ft/hr)	3.20E-05	6.95E-05	3.85E-04

The Pavement ME results from input level 1 are summarized in table 54. It is evident that the change in AC layer thickness resulted in a relatively low impact on terminal IRI and permanent deformation. However, a high impact on AC bottom-up fatigue cracking was observed. Bottom-up fatigue cracking develops as high tensile strains and stresses develop at the bottom of the AC layer under traffic loading. Cracking can initiate as a result of a reduction in support of the underlying layers. During the summer months, the resilient modulus of the unbound layers is at its peak. During the winter, the ground can freeze, greatly increasing the resilient modulus. In the spring months, the ground thaws, and the unbound layers are at their weakest. This is typically when bottom-up fatigue cracking is likely to occur. It is the most critical in District 1. It could be more critical in District 1, because of the difference in the calibration factors for the fatigue model in

District 1. The beta factor (laboratory to field factor) is much lower for District 1 than it is for Districts 2 and 3. This leads to a much lower number of cycles to failure, which would lead to a higher fatigue cracking prediction. Additionally, pavement with similar functional classification in District 1 experiences higher levels of traffic loading that Districts 2 and 3, which leads to a higher susceptibility to bottom-up fatigue cracking. In this sensitivity analysis, the traffic loading for District 1 reflects this, as it is higher than the other two districts.

Table 54. Pavement ME Results for Input Level 1.

Distress	Distress at reliability		Reliability (%)		Pass/Fail
Terminal IRI (inch/mile)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	170	126.04	95	99.97	Pass
District 1 - 7 inch	170	128.89	95	99.95	Pass
District 1 - 6.5 inch	170	131.78	95	99.91	Pass
District 2 - 7 inch	170	131.21	95	99.92	Pass
District 2 - 6.5 inch	170	132.12	95	99.91	Pass
District 2 - 6 inch	170	134.14	95	99.87	Pass
District 3 - 7.5 inch	170	129.10	95	99.94	Pass
District 3 - 7 inch	170	130.73	95	99.92	Pass
Permanent Deformation – Total Pavement (inch)	**Target**	**Predicted**	**Target**	**Achieved**	**Pass/Fail**
District 1 - 7.5 inch	0.5	0.34	95	100	Pass
District 1 - 7 inch	0.5	0.36	95	100	Pass
District 1 - 6.5 inch	0.5	0.38	95	99.98	Pass
District 2 - 7 inch	0.5	0.15	95	100	Pass
District 2 - 6.5 inch	0.5	0.16	95	100	Pass
District 2 - 6 inch	0.5	0.16	95	100	Pass
District 3 - 7.5 inch	0.5	0.28	95	99.97	Pass
District 3 - 7 inch	0.5	0.28	95	99.97	Pass
AC Bottom-Up Fatigue Cracking (% lane area)	**Target**	**Predicted**	**Target**	**Achieved**	**Pass/Fail**
District 1 - 7.5 inch	15	12.66	95	98.22	Pass
District 1 - 7 inch	15	27.46	95	75.36	Fail
District 1 - 6.5 inch	15	31.93	95	67.09	Fail
District 2 - 7 inch	15	4.61	95	100	Pass
District 2 - 6.5 inch	15	9.61	95	99.92	Pass
District 2 - 6 inch	15	26.37	95	77.2	Fail
District 3 - 7.5 inch	15	6.27	95	100	Pass
District 3 - 7 inch	15	19.61	95	87.11	Fail
Permanent Deformation – AC Only (inch)	**Target**	**Predicted**	**Target**	**Achieved**	**Pass/Fail**
District 1 - 7.5 inch	0.15	0.10	95	99.93	Pass
District 1 - 7 inch	0.15	0.11	95	99.87	Pass
District 1 - 6.5 inch	0.15	0.11	95	99.79	Pass
District 2 - 7 inch	0.15	0.07	95	100	Pass
District 2 - 6.5 inch	0.15	0.07	95	100	Pass
District 2 - 6 inch	0.15	0.07	95	100	Pass
District 3 - 7.5 inch	0.15	0.03	95	100	Pass
District 3 - 7 inch	0.15	0.03	95	100	Pass

Figure 65, Figure 66, and Figure 67 show the variation in the base layer modulus when measured SWCC and saturated hydraulic conductivity were inputted directly into the Pavement ME software. In District 1, it is shown that the modulus experiences an initial increase to about 40,000 psi, and then it decreases, and remains relatively constant. The modulus for District 2 and District 3 both see dramatic increases and decreases, but this can be attributed to the climatic data. In District 2, the pattern repeats for each 10 year cycle, where there is a great decrease in the modulus. For District 3, the pattern repeats for each 5 year cycle, where there is a dramatic increase in the modulus. These dramatic fluctuations are most likely caused by freeze-thaw cycles in the climatic data.

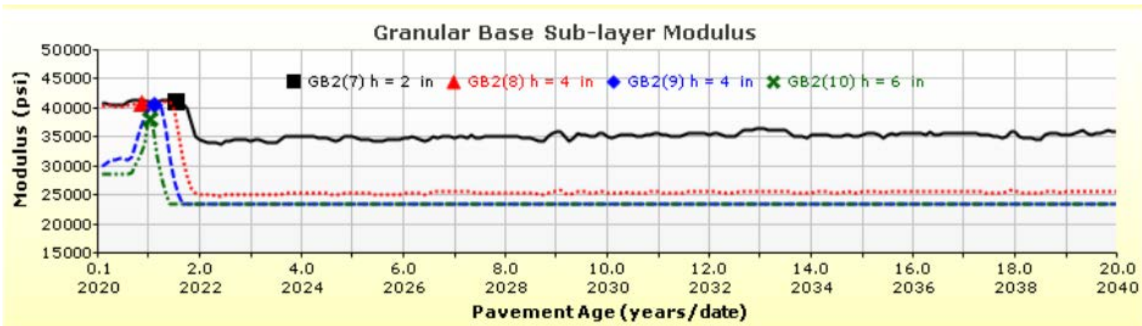


Figure 65. District 1 input level 1 variation in base material modulus for AC layer of 7.5 inch (Passing Design).

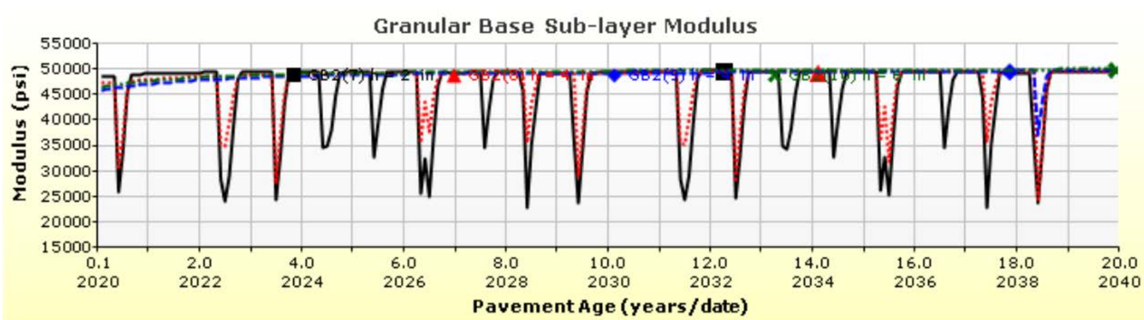


Figure 66. District 2 input level 1 variation in base material modulus for AC layer of 7 inch (Passing Design).

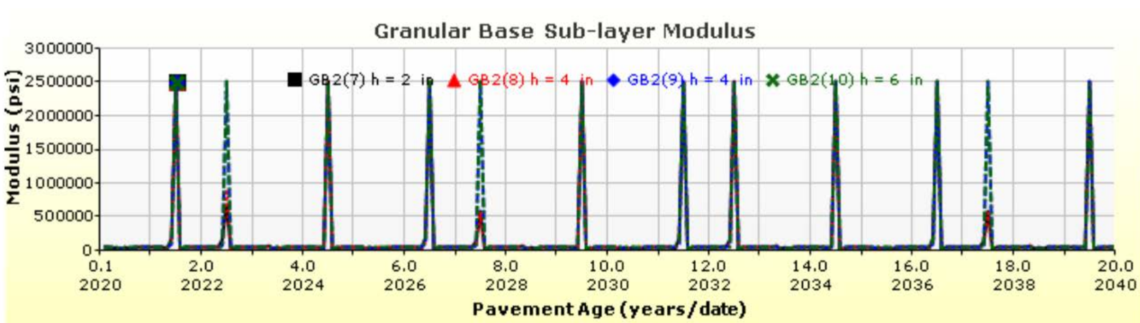


Figure 67. District 3 input level 1 variation in base material modulus for AC layer of 7.5 inch (Passing Design).

Pavement ME Analysis Using SWCC Input Level 2

The inputs for input level 2 in Pavement ME are similar to that of input level 1, except the SWCC parameters and the saturated hydraulic conductivity are not defined by the user. Instead, computations based on gradation and PI are performed internally by the EICM. Maximum dry unit weight, optimum water content, and specific gravity of solids are also defined by the user for input level 2.

Table 55 summarizes the Pavement ME results for input level 2. Between input level 1 and input level 2, a slight variation was observed in terminal IRI and permanent deformation (total and AC), especially in the case of District 1. On the other hand, the AC bottom-up fatigue cracking prediction showed the most variation between input level 1 and input level 2 as demonstrated for all three districts.

Rather than comparing the predicted values for each performance criteria, it is more useful to compare the change in reliability from input level 1 to input level 2. This is reflected in Table 56. The largest change is in District 1 for the 7 inch and 6.5 inch AC layers. For the District 1 and 7 inch AC design, there is a 32.7% relative increase in reliability for AC bottom-up fatigue cracking between input level 1 and input level 2. Similarly, for the District 1 and 6.5 inch AC design, there is a 32.4% relative decrease in reliability.

Figure 68, figure 69, and figure 70 show the variation in the base layer resilient modulus due to using the input level 2 for SWCC and saturated hydraulic conductivity. In the case of District 1, consistent fluctuations in the the modulus are observed. Again, in Districts 2 and 3, dramatic increases and decreases in the modulus can be attributed to climatic data.

Table 55. Pavement ME Results for Input Level 2.

Distress	Distress at specified reliability		Reliability (%)		Pass/Fail
Terminal IRI (inch/mile)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	170	121.99	95	99.99	Pass
District 1 - 7 inch	170	123.27	95	99.98	Pass
District 1 - 6.5 inch	170	125.27	95	99.97	Pass
District 2 - 7 inch	170	131.49	95	99.92	Pass
District 2 - 6.5 inch	170	132.41	95	99.91	Pass
District 2 - 6 inch	170	134.69	95	99.86	Pass
District 3 - 7.5 inch	170	129.23	95	99.94	Pass
District 3 - 7 inch	170	130.77	95	99.92	Pass
Permanent Deformation – Total Pavement (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.5	0.29	95	100	Pass
District 1 - 7 inch	0.5	0.30	95	100	Pass
District 1 - 6.5 inch	0.5	0.32	95	100	Pass
District 2 - 7 inch	0.5	0.16	95	100	Pass
District 2 - 6.5 inch	0.5	0.16	95	100	Pass
District 2 - 6 inch	0.5	0.17	95	100	Pass
District 3 - 7.5 inch	0.5	0.28	95	99.97	Pass
District 3 - 7 inch	0.5	0.29	95	99.96	Pass
AC Bottom-Up Fatigue Cracking (% lane area)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	15	4.12	95	100	Pass
District 1 - 7 inch	15	6.15	95	100	Pass
District 1 - 6.5 inch	15	18.66	95	88.8	Fail
District 2 - 7 inch	15	5.19	95	100	Pass
District 2 - 6.5 inch	15	12.22	95	98.61	Pass
District 2 - 6 inch	15	28.29	95	74.4	Fail
District 3 - 7.5 inch	15	5.94	95	100	Pass
District 3 - 7 inch	15	18.28	95	89.4	Fail
Permanent Deformation – AC only (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.15	0.11	95	99.79	Pass
District 1 - 7 inch	0.15	0.12	95	99.62	Pass
District 1 - 6.5 inch	0.15	0.12	95	99.47	Pass
District 2 - 7 inch	0.15	0.07	95	100	Pass
District 2 - 6.5 inch	0.15	0.07	95	100	Pass
District 2 - 6 inch	0.15	0.07	95	100	Pass
District 3 - 7.5 inch	0.15	0.03	95	100	Pass
District 3 - 7 inch	0.15	0.03	95	100	Pass

Table 56. Relative Change in Reliability from Input Level 2 to Input Level 1.

Terminal IRI (inch/mile)	From Level 1 to Level 2 (%)
District 1 - 7.5 inch	–0.020
District 1 - 7 inch	–0.030
District 1 - 6.5 inch	–0.060
District 2 - 7 inch	0.000
District 2 - 6.5 inch	0.000
District 2 - 6 inch	0.010
District 3 - 7.5 inch	0.000
District 3 - 7 inch	0.000
Permanent Deformation – Total Pavement (inch)	**From Level 1 to Level 2 (%)**
District 1 - 7.5 inch	0.000
District 1 - 7 inch	0.000
District 1 - 6.5 inch	–0.020
District 2 - 7 inch	0.000
District 2 - 6.5 inch	0.000
District 2 - 6 inch	0.000
District 3 - 7.5 inch	0.000
District 3 - 7 inch	0.010
AC Bottom-Up Fatigue Cracking (% lane area)	**From Level 1 to Level 2 (%)**
District 1 - 7.5 inch	–1.812
District 1 - 7 inch	–32.696
District 1 - 6.5 inch	–32.360
District 2 - 7 inch	0.000
District 2 - 6.5 inch	1.311
District 2 - 6 inch	3.627
District 3 - 7.5 inch	0.000
District 3 - 7 inch	–2.629
Permanent Deformation – AC only (inch)	**From Level 1 to Level 2 (%)**
District 1 - 7.5 inch	0.140
District 1 - 7 inch	0.250
District 1 - 6.5 inch	0.321
District 2 - 7 inch	0.000
District 2 - 6.5 inch	0.000
District 2 - 6 inch	0.000
District 3 - 7.5 inch	0.000

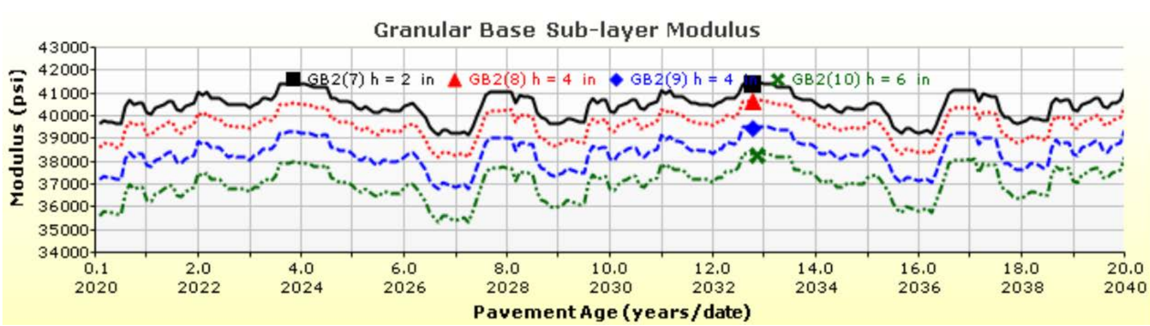


Figure 68. District 1 input level 2 variation in base material for AC layer of 7.5 inch (Passing Design).

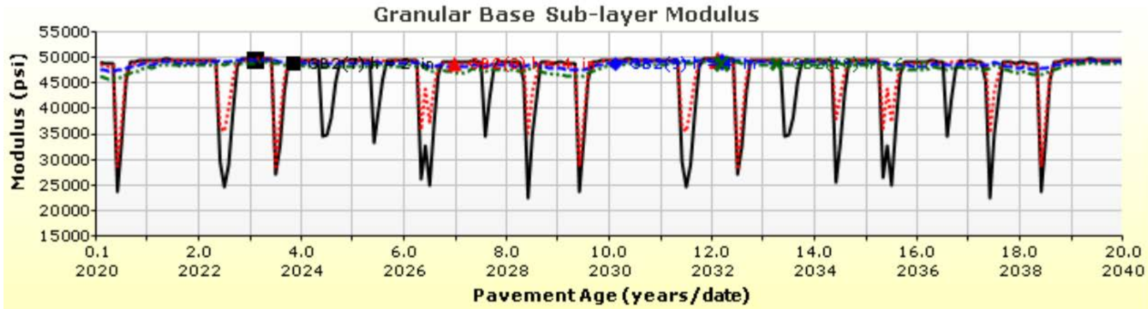


Figure 69. District 2 input level 2 variation in base material for AC layer of 7 inch (Passing Design).

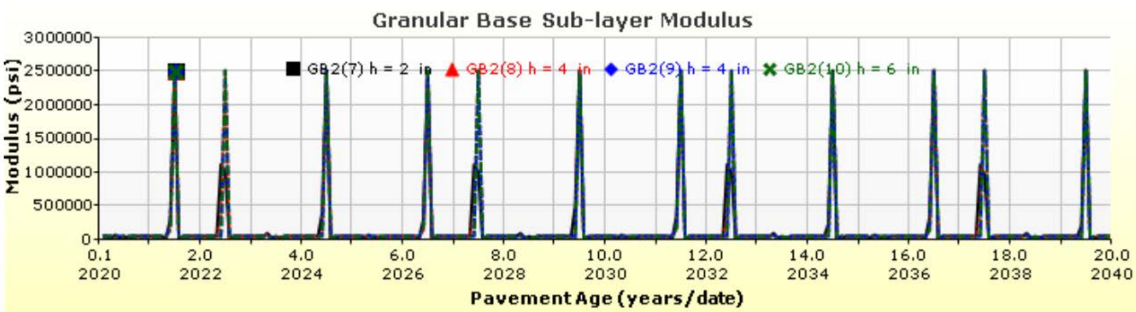


Figure 70. District 3 input level 2 variation in base material for AC layer of 7.5 inch (Passing Design).

Pavement ME Analysis Using SWCC Input Level 3

Most agencies use input level 3 as the current recommendation for SWCC and saturated hydraulic conductivity. In Pavement ME, input level 3 is run by only defining the gradation and Atterberg limits. Maximum dry unit weight, optimum moisture content, specific gravity of solids, saturated hydraulic water conductivity, and SWCC parameters are all computed internally based off the gradation and Atterberg limits inputs. Table 57 summarizes the Pavement ME results for input level 3.

Similar to input levels 1 and 2, all of the designs met the criteria required for terminal IRI and permanent deformation. The greater difference is in the AC bottom-up fatigue cracking. In comparison to input levels 1 and 2, the District 1 input level 3 resulted in a decrease in bottom-up fatigue cracking. Table 58 shows that from input level 1 to input level 3 there is a 49.1% relative change in the reliability of the predicted bottom-up fatigue cracking for the 7 inch AC design in District 1. This means that the estimations made in input level 3 underestimated the impact of the SWCC and saturated hydraulic conductivity on the pavement design in District 1. From input level 2 to input level 3, there is a 12.6% relative change, meaning that the predictions made by input level 2 are estimating more bottom-up fatigue cracking than input level 3, but less bottom-up fatigue cracking than input level 1.

Table 57. Pavement ME Results for Input Level 3.

Distress	Distress at specified reliability		Reliability (%)		Pass/Fail
Terminal IRI (inch/mile)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	170	120.82	95	99.99	Pass
District 1 - 7 inch	170	121.81	95	99.99	Pass
District 1 - 6.5 inch	170	123.06	95	99.98	Pass
District 2 - 7 inch	170	132.11	95	99.91	Pass
District 2 - 6.5 inch	170	133.62	95	99.88	Pass
District 2 - 6 inch	170	137.93	95	99.77	Pass
District 3 - 7.5 inch	170	129.1	95	99.94	Pass
District 3 - 7 inch	170	130.57	95	99.92	Pass
Permanent Deformation – Total Pavement (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.5	0.27	95	100	Pass
District 1 - 7 inch	0.5	0.28	95	100	Pass
District 1 - 6.5 inch	0.5	0.30	95	100	Pass
District 2 - 7 inch	0.5	0.17	95	100	Pass
District 2 - 6.5 inch	0.5	0.17	95	100	Pass
District 2 - 6 inch	0.5	0.16	95	100	Pass
District 3 - 7.5 inch	0.5	0.28	95	99.97	Pass
District 3 - 7 inch	0.5	0.28	95	99.97	Pass
AC Bottom-Up Fatigue Cracking (% lane area)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	15	3.57	95	100	Pass
District 1 - 7 inch	15	4.43	95	100	Pass
District 1 - 6.5 inch	15	8.06	95	100	Pass
District 2 - 7 inch	15	6.56	95	100	Pass
District 2 - 6.5 inch	15	20.52	95	85.71	Fail
District 2 - 6 inch	15	34.33	95	60.88	Fail
District 3 - 7.5 inch	15	5.72	95	100	Pass
District 3 - 7 inch	15	17.13	95	91.39	Fail
Permanent Deformation – AC Only (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.15	0.11	95	99.54	Pass
District 1 - 7 inch	0.15	0.12	95	99.09	Pass
District 1 - 6.5 inch	0.15	0.12	95	99.37	Pass
District 2 - 7 inch	0.15	0.07	95	100	Pass
District 2 - 6.5 inch	0.15	0.07	95	97.9	Pass
District 2 - 6 inch	0.15	0.06	95	100	Pass
District 3 - 7.5 inch	0.15	0.03	95	100	Pass
District 3 - 7 inch	0.15	0.03	95	100	Pass

In contrast, the input level 3 results for District 2 showed an increase in AC bottom-up fatigue cracking. In the case of District 2, the estimations made in input level 3 overestimated the impact of SWCC and saturated hydraulic conductivity. From input level 1 to input level 3 for the District 2 with the 6 inch AC layer thickness, there is a relative change of 21% in reliability for AC bottom-up fatigue cracking. From input level 2 to input level 3, there is also an increase in AC bottom-up fatigue cracking.

For District 3, the estimations made in input level 3 resulted in a slight decrease in AC bottom-up fatigue cracking in comparison to input level 1 and 2. This means that the estimations made in input level 3 slightly underestimate the impact of SWCC and saturated hydraulic conductivity.

Table 58. Change in Reliability from Input Level 1 to 3 and Input Level 2 to 3.

Terminal IRI (inch/mile)	**From Level 1 to Level 3 (%)**	**From Level 2 to Level 3 (%)**
District 1 - 7.5 inch	–0.020	0.000
District 1 - 7 inch	–0.040	–0.010
District 1 - 6.5 inch	–0.070	–0.010
District 2 - 7 inch	0.010	0.010
District 2 - 6.5 inch	0.030	0.030
District 2 - 6 inch	0.100	0.090
District 3 - 7.5 inch	0.000	0.000
District 3 - 7 inch	0.000	0.000
Permanent Deformation – Total Pavement (inch)	**From Level 1 to Level 3 (%)**	**From Level 2 to Level 3 (%)**
District 1 - 7.5 inch	0	0
District 1 - 7 inch	0.000	0.000
District 1 - 6.5 inch	–0.020	0.000
District 2 - 7 inch	0.000	0.000
District 2 - 6.5 inch	0.000	0.000
District 2 - 6 inch	0.000	0.000
District 3 - 7.5 inch	0.000	0.000
District 3 - 7 inch	0.000	–0.010
AC Bottom-Up Fatigue Cracking (% lane area)	**From Level 1 to Level 3 (%)**	**From Level 2 to Level 3 (%)**
District 1 - 7.5 inch	–1.812	0.000
District 1 - 7 inch	–32.696	0.000
District 1 - 6.5 inch	–49.054	–12.613
District 2 - 7 inch	0.000	0.000
District 2 - 6.5 inch	14.221	13.082
District 2 - 6 inch	21.140	18.172
District 3 - 7.5 inch	0.000	0.000
District 3 - 7 inch	–4.913	–2.226
Permanent Deformation – AC Only (inch)	**From Level 1 to Level 3 (%)**	**From Level 2 to Level 3 (%)**
District 1 - 7.5 inch	0.390	0.251
District 1 - 7 inch	0.781	0.532
District 1 - 6.5 inch	0.421	0.101
District 2 - 7 inch	0.000	0.000
District 2 - 6.5 inch	2.100	2.100
District 2 - 6 inch	0.000	0.000
District 3 - 7.5 inch	0.000	0.000
District 3 - 7 inch	0.000	0.000

Figure 71, figure 72, and figure 73 show the variation in the base layer resilient modulus using input level 3 for SWCC and saturated hydraulic conductivity. The pattern in the fluctuating modulus in District 1 looks similar to input level 2. However, the modulus in input level 2 has a range of about 35,000 to 41,000 psi. In input level 3, this range changes to about 48,000 to 50,000 psi. Thus, input level 3 for District 1 is estimating that the resilient modulus in the base material is going to be higher. District 2 shows the opposite trend. From input level 2, the range in the base modulus is about 25,000 to 50,000 psi. Input level 1 shows a modulus range of about 20,000 to 45,000 psi, indicating that input level 3 estimates the base modulus to be lower than in input level 1. The modulus for District 3 remains fairly constant across all Input Levels.

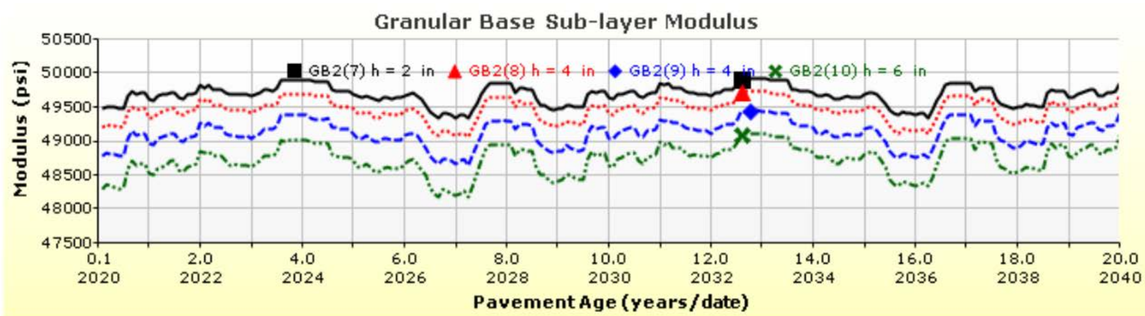


Figure 71. District 1 input level 3 variation in base material for AC layer of 7.5 inch (Passing Design).

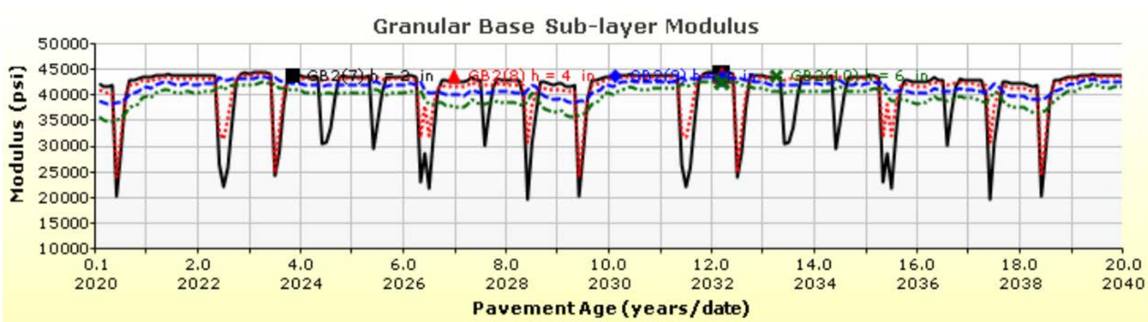


Figure 72. District 2 input level 3 variation in base material for AC layer of 7 inch (Passing Design).

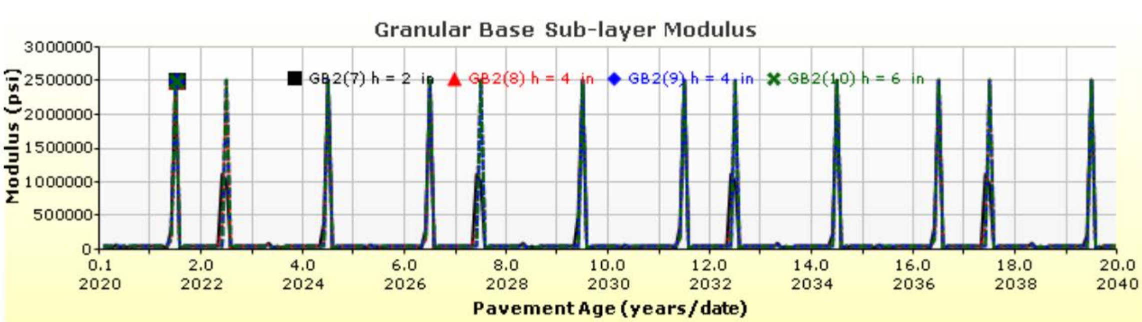


Figure 73. District 3 input level 3 variation in base material for AC layer of 7.5 inch (Passing Design).

Pavement ME Analysis Following Recommendations from Current NDOT Design Manual

The current recommendation by NDOT for SWCC and saturated hydraulic conductivity for use in Pavement ME is to use input level 3 for base material, and for subgrade materials to use values found from the national catalog of natural subgrade properties (4). To find the appropriate values to use, the location must be found on the map. Corresponding to that location, there is a map identification code. This code is then used to query the soil database and find several soil data, including gradation, saturated hydraulic conductivity, and SWCC parameters. This was a simple procedure for the District 1 subgrade, and Jacks Valley subgrade from District 2. The procedure was found to be more complex for the District 3 Elko Borrow, as this was not a true subgrade material found in that area. Instead, it is an imported subgrade material. The location of the Elko quarry was found, and several map units in that area were used to query the soil database, until a similar soil type was found. The SWCC parameters found using the ASU Soil Map, which were used in this analysis, are summarized in table 59.

Table 59. Subgrade SWCC Parameters Found Using the National Catalog of Natural Subgrade Properties.

SWCC Parameter	US-95/Searchlight (District 1)	Jacks Valley (District 2)	Elko Borrow Used as Subgrade (District 3)
a_f	2.263	0.947	2.136
b_f	1.023	1.090	1.017
c_f	0.696	0.434	0.890
h_r	2998.830	2994.910	3000.019

The Pavement ME results from using the current recommendations by NDOT are shown in table 60. The results for all three districts are comparable to the results from the input level 3 analysis. In comparison with the input level 1 results, specifically for AC bottom-up fatigue cracking, the predictions made using the current NDOT recommendations result in an underestimation of the impact of SWCC and saturated hydraulic conductivity in District 1, an overestimation in District 2, and a slight underestimation in District 3.

A summary of the relative changes in reliability between each input level and the current recommendations by NDOT are shown in table 61. The relative change in reliability for AC bottom-up fatigue cracking in District 1 from input level 1 to current recommendations is very similar to the relative change in reliability from input level 1 to input level 3. A change in reliability from District 1 input level 1 to current recommendations for AC bottom-up fatigue cracking for the 7 inch and 6.5 inch AC layers are 32.3% and 49.1%, respectively. In the same district, for the 6.5 inch AC layer, the relative change in reliability from input level 2 to the current recommendations is 12.6%.

District 2 also shows a change in reliability from the input level 3 AC bottom-up fatigue cracking in the 6 inch AC layer to the current recommendations. This relative change in reliability is 19.9%. In District 2, the current NDOT recommendations show a dramatic decrease in the predicted AC bottom-up fatigue cracking in comparison to input level 3.

Table 60. Pavement ME Results Using Current Recommendations by NDOT.

Distress	Distress at specified reliability		Reliability (%)		Pass/Fail
Terminal IRI (inch/mile)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	170	117.54	95	100	Pass
District 1 - 7 inch	170	118.56	95	99.99	Pass
District 1 - 6.5 inch	170	119.81	95	99.99	Pass
District 2 - 7 inch	170	133.17	95	99.89	Pass
District 2 - 6.5 inch	170	134.39	95	99.87	Pass
District 2 - 6 inch	170	136.58	95	99.81	Pass
District 3 - 7.5 inch	170	134.35	95	99.86	Pass
District 3 - 7 inch	170	136.13	95	99.81	Pass
Permanent Deformation – Total Pavement (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.5	0.26	95	100	Pass
District 1 - 7 inch	0.5	0.27	95	100	Pass
District 1 - 6.5 inch	0.5	0.29	95	100	Pass
District 2 - 7 inch	0.5	0.15	95	100	Pass
District 2 - 6.5 inch	0.5	0.16	95	100	Pass
District 2 - 6 inch	0.5	0.16	95	100	Pass
District 3 - 7.5 inch	0.5	0.29	95	99.86	Pass
District 3 - 7 inch	0.5	0.29	95	99.95	Pass
AC Bottom-Up Fatigue Cracking (% lane area)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	15	3.56	95	100	Pass
District 1 - 7 inch	15	4.41	95	100	Pass
District 1 - 6.5 inch	15	7.97	95	100	Pass
District 2 - 7 inch	15	5.58	95	100	Pass
District 2 - 6.5 inch	15	16.38	95	92.73	Fail
District 2 - 6 inch	15	29.17	95	72.97	Fail
District 3 - 7.5 inch	15	7.91	95	100	Pass
District 3 - 7 inch	15	23.67	95	88.18	Fail
Permanent Deformation – AC only (inch)	**Target**	**Predicted**	**Target**	**Achieved**	
District 1 - 7.5 inch	0.15	0.11	95	99.79	Pass
District 1 - 7 inch	0.15	0.12	95	99.62	Pass
District 1 - 6.5 inch	0.15	0.12	95	99.49	Pass
District 2 - 7 inch	0.15	0.07	95	100	Pass
District 2 - 6.5 inch	0.15	0.07	95	93.02	Pass
District 2 - 6 inch	0.15	0.07	95	100	Pass
District 3 - 7.5 inch	0.15	0.03	95	100	Pass
District 3 - 7 inch	0.15	0.03	95	100	Pass

Table 61. Relative Change in Reliability From Input Level 1, Input Level 2, and Input Level 3 to the Current Recommendations by NDOT.

Terminal IRI (inch/mile)	From Level 1 to Current (%)	From Level 2 to Current (%)	From Level 3 to Current (%)
District 1 - 7.5 inch	–0.030	–0.010	–0.010
District 1 - 7 inch	–0.040	–0.010	0.000
District 1 - 6.5 inch	–0.080	–0.020	–0.010
District 2 - 7 inch	0.030	0.030	0.020
District 2 - 6.5 inch	0.040	0.040	0.010
District 2 - 6 inch	0.060	0.050	–0.040
District 3 - 7.5 inch	0.080	0.080	0.080
District 3 - 7 inch	0.110	0.110	0.110
Permanent Deformation – Total Pavement (inch)	**From Level 1 to Current (%)**	**From Level 2 to Current (%)**	**From Level 3 to Current (%)**
District 1 - 7.5 inch	0.000	0.000	0.000
District 1 - 7 inch	0.000	0.000	0.000
District 1 - 6.5 inch	–0.020	0.000	0.000
District 2 - 7 inch	0.000	0.000	0.000
District 2 - 6.5 inch	0.000	0.000	0.000
District 2 - 6 inch	0.000	0.000	0.000
District 3 - 7.5 inch	0.110	0.110	0.110
District 3 - 7 inch	0.020	0.010	0.020
AC Bottom-Up Fatigue Cracking (% lane area)	**From Level 1 to Current (%)**	**From Level 2 to Current (%)**	**From Level 3 to Current (%)**
District 1 - 7.5 inch	–1.812	0.000	0.000
District 1 - 7 inch	–32.696	0.000	0.000
District 1 - 6.5 inch	–49.054	–12.613	0.000
District 2 - 7 inch	0.000	0.000	0.000
District 2 - 6.5 inch	7.196	5.963	–8.190
District 2 - 6 inch	5.479	1.922	–19.859
District 3 - 7.5 inch	0.000	0.000	0.000
District 3 - 7 inch	–1.228	1.365	3.512
Permanent Deformation – AC Only (inch)	**From Level 1 to Current (%)**	**From Level 2 to Current (%)**	**From Level 3 to Current (%)**
District 1 - 7.5 inch	0.140	0.000	–0.251
District 1 - 7 inch	0.250	0.000	–0.535
District 1 - 6.5 inch	0.301	–0.020	–0.121
District 2 - 7 inch	0.000	0.000	0.000
District 2 - 6.5 inch	6.980	6.980	4.985
District 2 - 6 inch	0.000	0.000	0.000
District 3 - 7.5 inch	0.000	0.000	0.000
District 3 - 7 inch	0.000	0.000	0.000

Figure 74, figure 75, and figure 76 show the variation in the base layer resilient modulus using the current NDOT recommendations for SWCC and saturated hydraulic conductivity. In District 1, the modulus varies from about 49,000 to 50,000 psi. In input level 3, the modulus varies from 48,000 to 50,000 psi; thus for District 1, input level 3 and the current NDOT recommendation for SWCC and saturated hydraulic conductivity resulted in similar moduli. For District 2, the modulus varies from about 49,400 to almost 50,000 psi. Input level 3 estimated a modulus range of 20,000 psi to 45,000 psi; thus, there is an increase in the modulus estimation. For District 3, the modulus is consistent with input levels 1, 2, and 3.

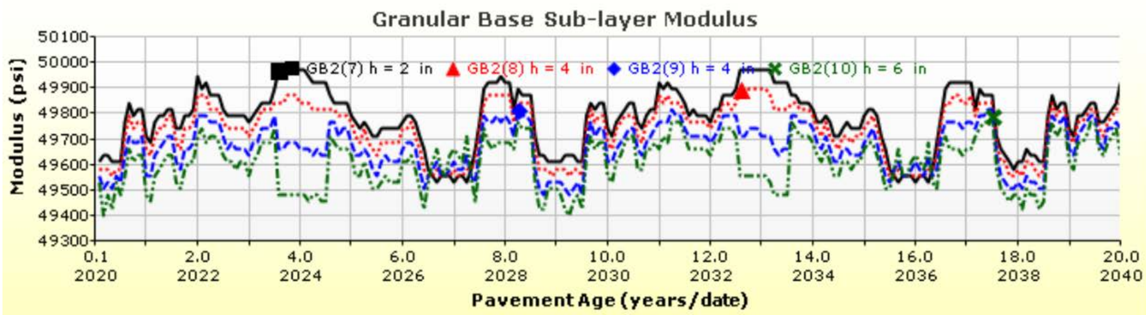


Figure 74. District 1 current NDOT recommendation, variation in base material for AC layer of 7.5 inches (Passing Design).

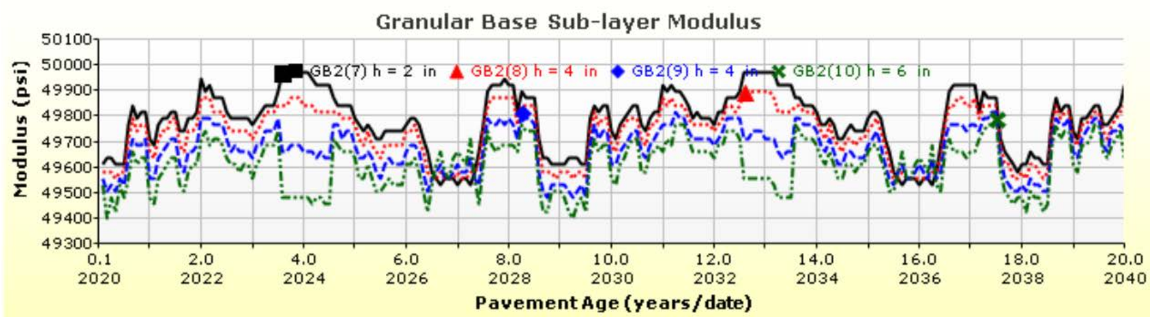


Figure 75. District 2 current NDOT recommendation, variation in base material for AC of 7 inches (Passing Design).

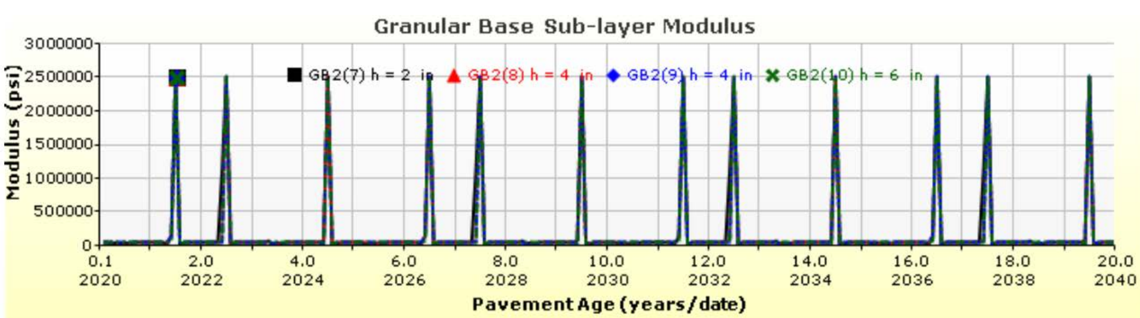


Figure 76. District 3 current NDOT recommendation, variation in base material for AC of 7.5 inches (Passing Design).

Overall Summary of Sensitivity Analysis

While the results for all three districts did not fit one trend, it was shown through the Pavement ME runs conducted that the input of SWCC and saturated hydraulic conductivity does have an impact on the predicted performance of the tested pavement designs. A summary of the difference in reliability levels for AC bottom-up fatigue cracking at the minimum AC thickness tested (6.5 inch for District 1, 6 inch for District 2, and 7 inch for District 3) is shown in figure 77. The reason the reliability levels were compared here at the minimum AC thickness tested was so that it would be less likely that a reliability level of 100% would be reached. When a reliability level of 100% is reached, this creates an upper boundary that makes comparison less accurate.

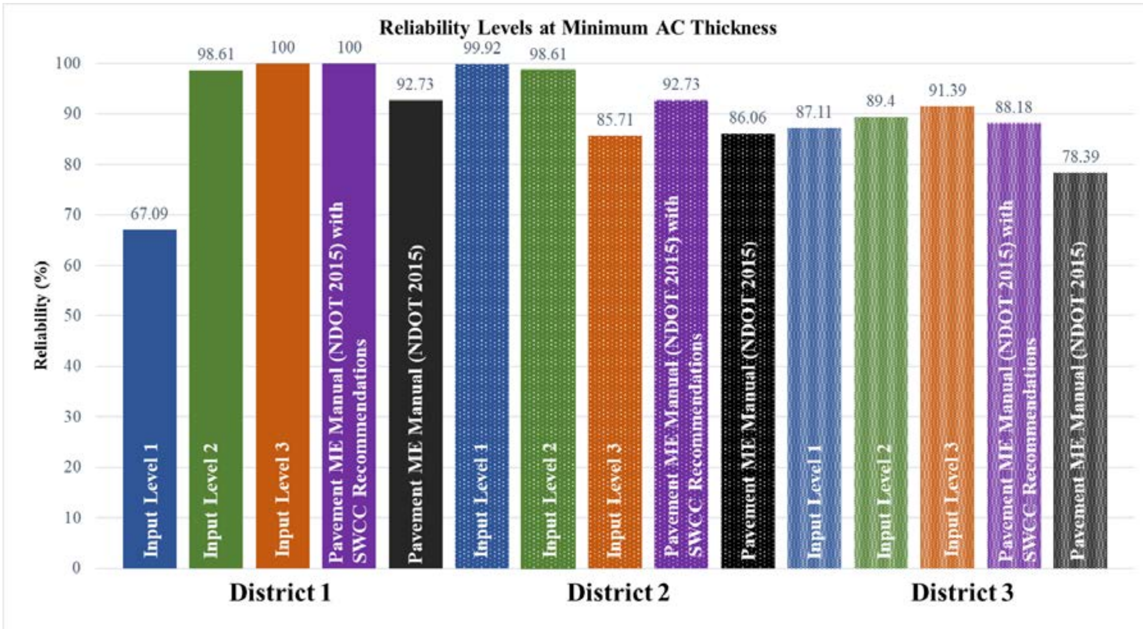


Figure 77. Comparison of reliability levels at the minimum passing design.

For District 1, input levels 2, 3, and the current NDOT recommendations greatly underestimate the impact of SWCC and saturated hydraulic conductivity. This is shown by an increase in reliability and a decrease in the predicted AC bottom-up fatigue cracking. This change indicates that the AC thickness would need to be increased in order to accommodate for the seasonal variation caused, in part, by the input of the directly measured SWCC parameters and saturated hydraulic conductivity. For District 1, using the current recommendations by NDOT, a passing pavement design uses a 6.5 inch AC layer. At input level 3, this design also passes all of the performance criteria. At input level 2 and input level 1, however, this design does not pass the criteria for AC bottom-up fatigue cracking. At input level 2, the minimum AC layer thickness that passes the AC bottom-up fatigue cracking performance criteria is a thickness of 7 inch. At input level 1, an AC layer thickness of 7 inch still does not pass the criteria for AC bottom-up fatigue cracking. The AC layer thickness that passes at input level 1 is 7.5 inch. This is a 1 inch increase in comparison to the current recommendations made by NDOT for SWCC and saturated hydraulic conductivity.

The District 2 results were in stark contrast to the trend found in District 1. For District 2, input levels 2, 3, and the current NDOT recommendations overestimate the impact of SWCC and saturated hydraulic conductivity. This is shown in the decrease in reliability and the increase in the percentage of AC bottom-up fatigue cracking. From these results, however, a change in AC thickness would not be recommended. AC bottom-up fatigue cracking is usually more of a concern in District 2, as this region experience more freeze-thaw cycles, leading to a more dramatic reduction in strength of the unbound layers. The impact of the SWCC and hydraulic conductivity direct measurement is not enough to change current recommendations.

The District 3 results showed that from direct measurement of SWCC and saturated hydraulic conductivity to input level 2, 3, and the current NDOT recommendations, there is a slight increase in reliability and a slight decrease in the predicted AC bottom-up fatigue cracking. The reliability level from input level 1 in comparison with the current NDOT recommendations is comparable at the minimum AC layer thickness. Although it can be seen that the direct measurement of SWCC and saturated hydraulic conductivity does have an impact on AC bottom-up fatigue cracking performance, this impact is not enough to recommend a change in the AC thickness. For all input levels, the pavement design that passed all performance criteria was the 7.5 inch AC thickness design.

As previously discussed, the pavement design in District 1 was more susceptible to AC bottom-up fatigue cracking than the designs for Districts 2 and 3. Generally, fatigue cracking is more of a concern in District 2 and 3. Permanent deformation is usually a more common concern in District 1. The fatigue model used for District 1 has a beta factor of 0.005. In comparison, the District 2 and 3 fatigue models use a beta factor of 50. Therefore, the number of cycles to fatigue failure is greatly reduced for District 1. Because of this, it was necessary to take out the influence of the beta factor for one set of Pavement ME runs to ensure that the changes in AC bottom-up fatigue cracking were caused by the impact of SWCC and saturated hydraulic conductivity, and was not being overpowered by the influence of the beta factor. This analysis showed the same trend in AC bottom-up fatigue cracking as the analysis with the beta factor added back in. Therefore, it was found that the influence of the beta factor for District 1 did not override the impact of the input of SWCC and saturated hydraulic conductivity.

Additionally, it was necessary to assess the impact of SWCC and saturated hydraulic conductivity on the base material while using the coefficients for seasonal variation for the resilient modulus in the subgrade layers. In doing so, the effect of the SWCC and saturated hydraulic conductivity is overridden by the seasonal coefficients, but only in the subgrade. The results were very similar to the results from the Pavement ME runs used for the bulk of this sensitivity analysis, where SWCC and hydraulic conductivity were inputted for both the base material and subgrade material layers. The results from these Pavement ME runs showed that the coefficients for seasonal variation in the subgrade layers may be adequate to use in place of the SWCC and saturated hydraulic conductivity inputs.

Figure 78 shows the AC thicknesses for the passing pavement design for each Input Level and each district. Because of the underestimation of the impact of SWCC and saturated hydraulic conductivity in District 1, an increase in AC layer thickness of 0.5 inch is recommended.

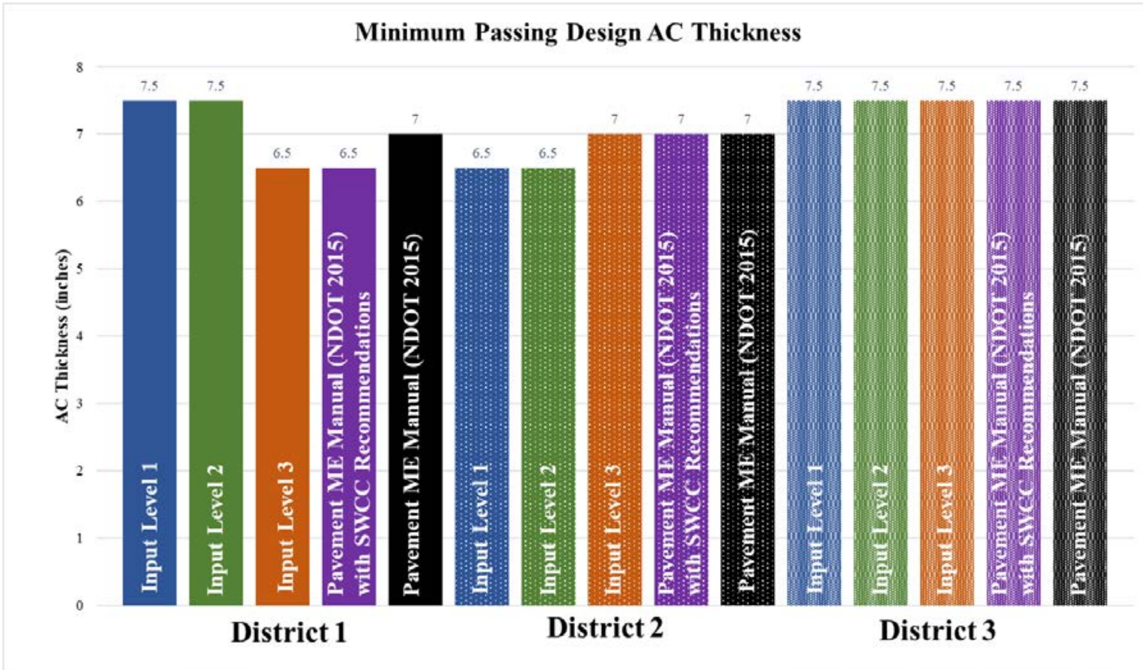


Figure 78. Minimum passing design AC thickness at each input level.

Within each district, there is some variation with climate; therefore, an additional investigation was conducted where the climate stations in Districts 1 and 2 were changed, while keeping all other parameters constant. For District 1, the weather station was changed from the Las Vegas station to the Mercury station. The town of Mercury is located about 80 miles northwest of the weather station at McCarran International Airport that was used in Las Vegas. Mercury is at a higher elevation and experiences an increased number of freeze-thaw cycles as compared to Las Vegas. For District 2, a virtual weather station was created at South Lake Tahoe. South Lake Tahoe is located about 60 miles southwest of Reno. It is at a much higher elevation than Reno and it experiences more precipitation and more freeze-thaw cycles than Reno. This additional analysis was conducted to further investigate the impact of SWCC and saturated hydraulic conductivity on AC bottom-up fatigue cracking.

Figure 79 shows a comparison of reliability levels between the different input levels. In order to stay consistent with the previous comparison in reliability levels from above, the same AC thickness designs were compared; thus 6.5 inch for District 1 and 6.0 inch for District 2 were used.

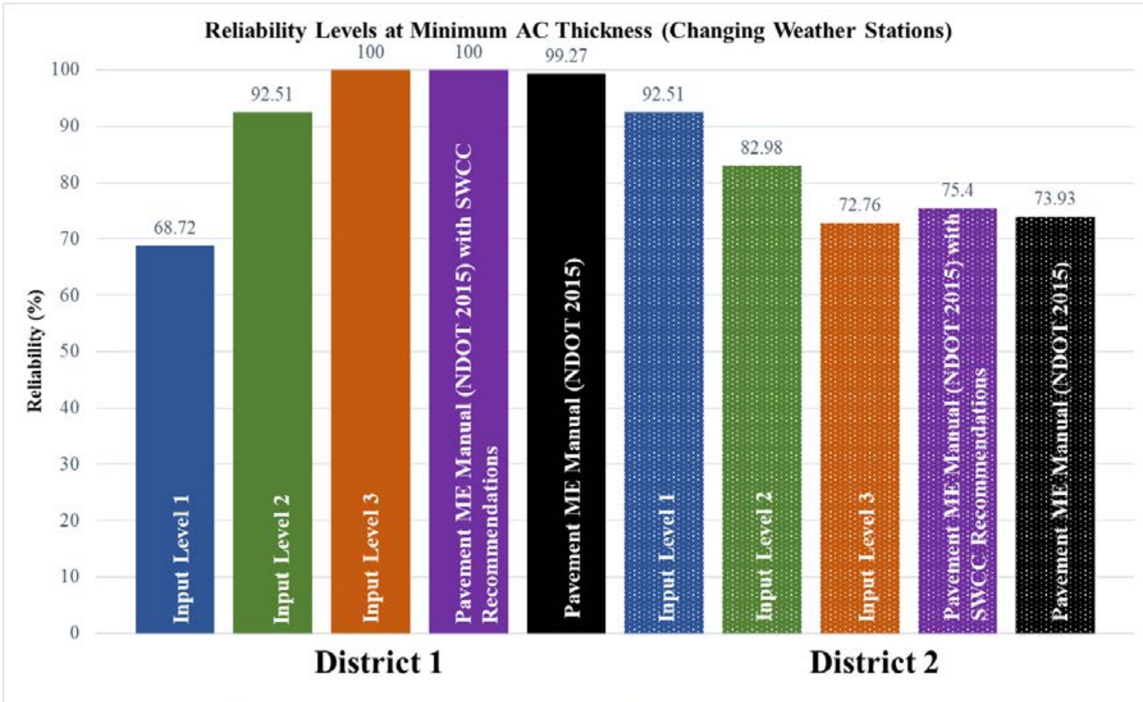


Figure 79. Comparison of reliability levels at the minimum passing design, changing weather stations while keeping all other inputs constant.

In District 1, changing the weather station had a low impact on the reliability levels for AC bottom-up fatigue cracking. While Mercury does have more freeze-thaw cycles than Las Vegas, the climate is still pretty similar to Las Vegas. A more useful comparison may be to use climate data farther north in District 1, such as in Tonopah. Tonopah is at a much higher elevation, and experiences much more snowfall than Las Vegas or Mercury. However, the MEPDG does not have a weather station near Tonopah, and a virtual weather station cannot currently be created for Tonopah within Pavement ME.

For District 2, changing the weather station to South Lake Tahoe had a great impact on bottom-up fatigue cracking. The reliability levels greatly decreased, in comparison to the analysis done using the Reno weather station. This is caused by South Lake Tahoe being at a greatly increased elevation and having many more freeze-thaw cycles than Reno. However, while the reliability levels were greatly decreased, a similar trend to the Reno analysis was found. From input level 1 to input level 2 to input level 3, the reliability level decreases. This means that input level 3 is overestimating the impact of SWCC and saturated hydraulic conductivity.

RECOMMENDATIONS FOR NDOT PAVEMENT ME DESIGN

The primary objective of this project is to provide NDOT with an organized database of material gradation and other engineering properties. This was completed through the collection of historical records from NDOT on recent pavement projects, as well as through an extensive laboratory evaluation for nine base materials, nine borrow materials, and six subgrade materials throughout Nevada. This section discusses how the findings from the historical data collection and laboratory evaluation can be incorporated into NDOT Pavement ME Design. This will be done by finding representative values to be used in the “Manual for Designing Flexible Pavements in Nevada using the AASHTOWare Pavement ME,” for unbound materials, either statewide or by region.

From the historical records, representative values for base material gradation, Atterberg limits, and moisture density could be found. Whenever possible, results from the historical data were used, since a much greater number of tests were performed on the unbound materials than in the laboratory evaluation alone. Because NDOT specifications require that 100% of the material needs to pass the 3-inch sieve for borrow material, the representative values for borrow material gradation and Atterberg limits must come from the laboratory evaluation. However, representative values for borrow material moisture and density could be found from historical construction quality assurance test records. For both base and borrow materials, the specific gravity of solids is tested in the laboratory.

Using the historical data, single factor Analysis of Variance (ANOVA) tests were conducted to determine if values for each engineering property could be used statewide or if the values for use in Pavement ME should be broken down by district instead. ANOVA allows for the testing of hypotheses about the average of a dependent variable across different groups, and in this case those groups are the three NDOT districts. ANOVA testing calculates an F-statistic. This is used to calculate the p-value. If the p-value is found to be less than 0.05, the null hypothesis is rejected. If the null hypothesis is rejected, then it is found that the average of the dependent variable is not the same for all groups. If the null hypothesis was rejected when comparing the dependent variable among all three districts, then other ANOVA analyses were performed to test if two districts could be coupled, such as Districts 1 and 2, Districts 2 and 3, or Districts 1 and 3. In these analyses, if the p-value was still found to be less than 0.05, then it was assumed that the value for that dependent variable must be used specifically for each district, rather than a representative value for the entire state to be used.

When evaluating all engineering properties across all three NDOT districts, it was found that all p-values found were less than 0.05. When comparing two NDOT districts at a time, it was, again, found that all p-values were less than 0.05. Therefore, representative values for each engineering property should be specific to the NDOT district, rather than using statewide representative values. An example of this is shown in table 62 and table 63. Table 62 shows the ANOVA test results when comparing Type 1 Class B base material maximum dry density values across all three NDOT Districts. Table 63 shows the ANOVA test results when comparing the maximum dry density values for just two NDOT Districts. From these results, and from the results from testing all unbound material property values, it could be determined that statewide representative values are not appropriate to use in NDOT Pavement ME design.

Table 62. ANOVA Test for Base Material Maximum Dry Density, from Historical Records.

Groups	Count	Sum	Average	Variance		
District 1	35	4880.8	139.4514	86.21728		
District 2	99	13317.8	134.5232	31.81139		
District 3	76	10664.8	140.3263	64.05983		
ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	1616.987875	2	808.4939	15.4199	5.72E-07	3.039508
Within Groups	10853.39136	207	52.43184			
Total	12470.37924	209				

Table 63. ANOVA Test for Base Material Maximum Dry Density, from Historical Records - Comparing Only Two Districts

Groups	Count	Sum	Average	Variance		
District 1	35	4880.8	139.4514	86.21728		
District 2	99	13317.8	134.5232	31.81139		
ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	628.0213789	1	628.0214	13.70477	0.000313	3.912875
Within Groups	6048.903994	132	45.82503			
Total	6676.925373	133				

Based on the statistical analyses performed, the representative values in the following tables for the base and borrow material properties should be used in NDOT Pavement ME design (table 64 to table 75). The number of samples used to find the representative values is also included and was dependent upon the material property. From the historical records, different material properties had data for varying numbers of samples. It should be noted that in cases where data from the laboratory evaluation alone had to be used, the values may not necessarily be as representative as data coming from the entire database using the historical records. It is recommended that further testing be done on more samples, especially Districts 2 and 3, in order to find more truly representative values. Figure 80 and figure 81 show representative gradations for base and borrow materials, respectively, for all three NDOT districts.

Table 64. Representative Base Material Gradations for Each District.

Sieve Size	Specifications	Percent Passing		
		District 1	District 2	District 3
31.5 mm (1.5 inch)		100	100	100
25.0 mm (1 inch)	80-100	98.1	98.6	99.6
19.0 mm (3/4 inch)		93.8	95.3	94.1
12.5 mm (1/2 inch)		80.7	76.1	80.7
9.5 mm (3/8 inch)		69.5	64.5	71.3
4.75 mm (No. 4)	30-65	52.1	43.0	53.0
2.36 mm (No. 8)		35.5	29.6	39.8
2.00 mm (No. 10)		33.6	28.0	36.4
1.18 mm (No. 16)	15-40	25.7	22.5	29.6
0.6 mm (No. 30)		22.3	18.5	24.9
0.425 mm (No. 40)		16.5	14.8	17.3
0.3 mm (No. 50)		14.6	13.0	17.0
0.15 mm (No. 100)		11.6	9.5	11.0
0.075 mm (No. 200)	2-12	8.7	6.6	8.1
Number of samples		188	285	124

Table 65. Representative Values for Base Material LL and PI.

District	LL	PI	Number of Samples
1	19.1	3.4	115
2	23.4	3.7	177
3	22.0	4.7	32

Table 66. Representative Values for Base Material Maximum Dry Density and Optimum Moisture Content.

District	Maximum Dry Density (pcf)	OMC (%)	Number of Samples
1	139.5	7.1	35
2	134.5	7.2	99
3	140.3	6.1	74

Table 67. Representative Values for Base Material Specific Gravity.

District	Bulk Specific Gravity	Number of Samples
1	2.670	5
2	2.703	1
3	2.461	2

Table 68. Representative Values for Base Material SWCC Parameters.

District	a_f	b_f	c_f	h_r	Number of Samples
1	1.6500	0.9959	2.9684	6.6648	5
2	0.3740	1.3374	0.4776	2.5991	1
3	39.4681	0.6486	12.7272	1499.9999	2

Table 69. Representative Values for Base Material Saturated Hydraulic Conductivity.

District	Saturated Hydraulic Conductivity (ft/hr)
1	5.32E-06
2	3.25E-04
3	5.95E-04

Table 70. Representative Values for Borrow Material Gradation.

Sieve Size	Specifications	Percent Passing		
		District 1	District 2	District 3
75 mm (3 inch)	100	100.0	100.0	100.0
50 mm (2 inch)		100.0	100.0	100.0
31.5 mm (1.5 inch)		99.5	100.0	100.0
25.0 mm (1 inch)		96.9	100.0	87.3
19.0 mm (3/4 inch)		94.3	99.6	82.0
12.5 mm (1/2 inch)		90.0	96.3	74.6
9.5 mm (3/8 inch)		86.5	91.5	68.9
4.75 mm (No. 4)		73.8	77.2	53.4
2.36 mm (No. 8)		59.6	57.4	40.9
2.00 mm (No. 10)		56.7	52.4	37.4
1.18 mm (No. 16)		48.5	40.0	29.3
0.6 mm (No. 30)		41.3	30.0	18.8
0.425 mm (No. 40)		38.4	26.6	13.8
0.3 mm (No. 50)		35.7	24.1	10.0
0.15 mm (No. 100)		30.2	20.3	6.5
0.075 mm (No. 200)		23.5	16.2	4.9
Number of samples		5	3	1

Table 71. Representative Values for Borrow Material LL and PI.

District	LL	PI	Number of Samples
1	21.4	2.5	4
2	42.5	14.6	2
3	–	–	–

–No data available. Assume representative values same as District 1 or 2 based on engineering judgment.

Table 72. Representative Values for Borrow Material Maximum Dry Density and Optimum Moisture Content.

District	Maximum Dry Density (pcf)	OMC (%)	Number of Samples
1	135.3	7.4	88
2	122.8	9.6	187
3	129.4	8.9	404

Table 73. Representative Values for Borrow Material Specific Gravity.

District	Bulk Specific Gravity	Number of Samples
1	2.487	5
2	2.648	3
3	2.503	1

Table 74. Representative Values for Borrow Material SWCC Parameters.

District	a_f	b_f	c_f	h_r	Number of Samples
1	59.649	1.140	33.894	4.164	5
2	1.9787	2.0551	0.5547	5.6230	3
3	32.6979	0.8272	27.2042	201.2146	1

Table 75. Representative Values for Borrow Material Saturated Hydraulic Conductivity.

District	Saturated Hydraulic Conductivity (ft/hr)
1	4.13E-04
2	2.50E-04
3	3.85E-04

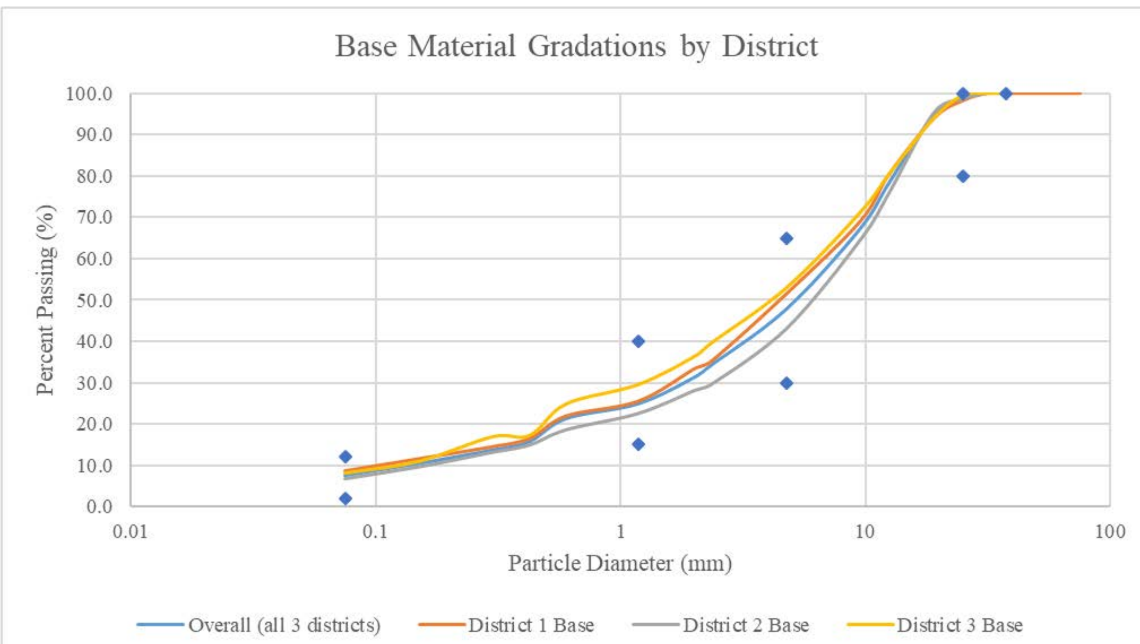


Figure 80. Representative base material gradations.

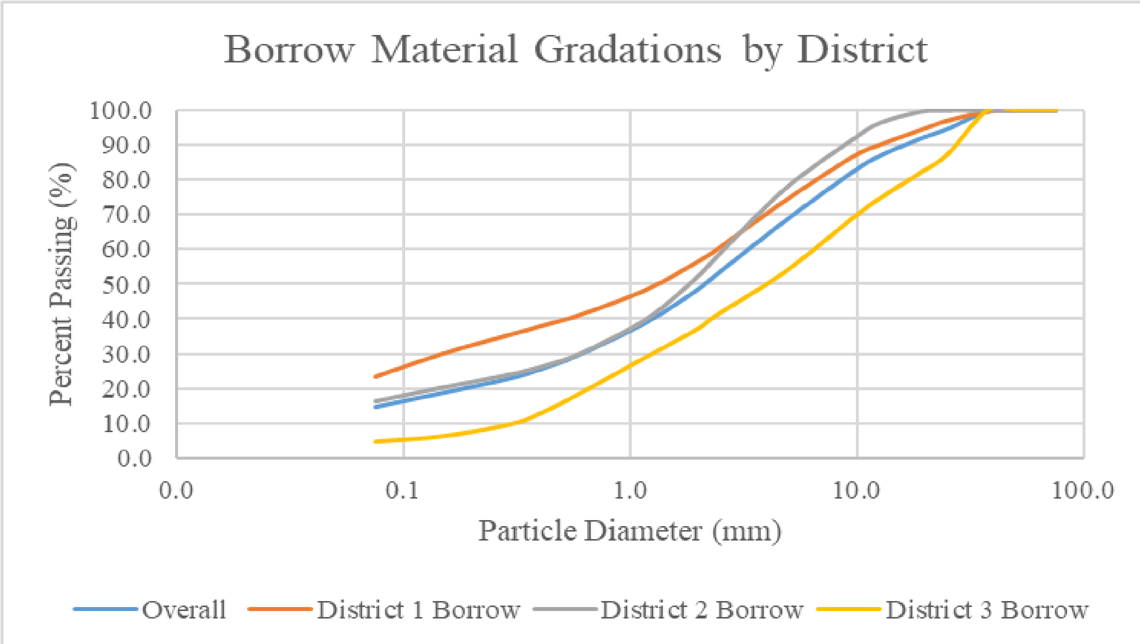


Figure 81. Representative borrow material gradations.

The SWCC parameters that have been recommended are from direct measurement and are the parameters for one base material and one borrow material in its respective district that is in the mid-range of all the SWCCs in its respective district. Additionally, non-linear regression analysis was performed to calibrate the current input level 2 models for SWCC. Equation 28 through equation 31 show the correlations for SWCC for materials where $P_{200}PI > 0$. Equation 32 through equation 35 show the correlations for SWCC for material where $P_{200}PI = 0$. The fitting parameters have been assigned a letter variable, and these parameters were determined using non-linear regression, fitting the input level 2 models to the SWCC parameters found using input level 1.

- Case 1: If $P_{200}PI > 0$

$$\boldsymbol{a_f = \frac{a(P_{200}PI)^b + c(P_{200}PI) + d}{e}, psi} \quad [28]$$

$$\boldsymbol{\frac{b_f}{c_f} = f(P_{200}PI)^g + h} \quad [29]$$

$$\boldsymbol{c_f = i(P_{200}PI)^j + k} \quad [30]$$

$$\boldsymbol{\frac{h_r}{a_f} = \frac{1}{D_{60} + le^m}} \quad [31]$$

- Case 2: If $P_{200}PI = 0$

$$\boldsymbol{a_f = \frac{a(D_{60})^b}{c}, psi} \quad [32]$$

$$\bar{b}_f = d \quad [33]$$

$$c_f = e\, ln(D_{60}) + f \quad [34]$$

$$\frac{h_r}{a_f} = \frac{1}{D_{60} + g e^{h}} \quad [35]$$

Table 76 to table 78 summarize the calibrated Input Level 2 SWCC parameters. Table 76 shows the calibrated parameters for base materials; however, these parameters are for use with base material that has $PI*P_{200} = 0$. This is because all the sampled base materials tested in the laboratory evaluation were all non-plastic materials. If the base material being considered has a $PI*P_{200} > 0$, then these calibrated models are not appropriate. Further testing should be done on base materials with $PI*P_{200} > 0$, in order to properly calibrate the SWCC input level 2 model for that case. Table 77 shows the calibrated parameters for use with borrow material that has $PI*P_{200} > 0$. Borrow material is highly variable material, and it is critical that the appropriate model be used for the district being considered. Table 78 shows the calibrated parameters for use with borrow material that has $PI*P_{200} = 0$. It should be noted that Pavement ME ver. 2.3.1 does not offer the user the flexibility to input the fitting parameters (*a* through *m*) shown in table 76 to table 78 for SWCC parameters. These fitting parameters can be valuable if user input option for SWCC model becomes available in the future versions of Pavement ME.

Table 76. Calibrated Base Material Input Level 2 Parameters for Nevada (Where PI*P200=0).

District	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*
1	0.227	4.452	13.440	1.516	5.396	7.790	5.6	-7.689
2	1.265	-0.075	3.383	1.337	0.182	0.704	9.6	-50
3	0.009	-0.751	0.034	1.246	0.002	0.773	9.7	-4

Table 77. Calibrated Borrow Material Input Level 2 Parameters for Nevada (Where PI*P200 > 0).

District	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*
1	0.003	0.350	0.405	11.000	3.103	0.714	0.139	0.140	0.051	0.465	25.900	35.7	1.86E-06
2	0.003	2.39E-04	0.037	11.049	13.825	0.152	0.579	0.000	0.039	0.000	0.399	3.274	2.5418 E-08
3	–	–	–	–	–	–	–	–	–	–	–	–	–

–Not available.
PI*P200 ranged between 2 and 314.

Table 78. Calibrated Borrow Material Input Level 2 Parameters for Nevada (Where PI*P200 = 0).

District	*a*	*b*	*c*	*d*	*e*	*f*	*g*	*h*
1	0.086	-1.593	2.167	-5.779	-0.591	0.775	9.7	-4
2	4.999	-1.159	7.584	-0.751	0.002	1.435	9.7	-4
3	20.246	-1.158	7.697	-0.751	0.177	25.366	9.7	-5.6

CHAPTER 4 CONCLUSIONS

The main goal of this project was to produce a database of parameters for unbound materials in Nevada. Historical records were collected from NDOT from recent pavement projects and summarized in an electronic format. Base, borrow, and subgrade materials were collected from all three NDOT Districts, and an extensive laboratory evaluation was conducted, which included testing for gradation, Atterberg limits, specific gravity of solids, maximum dry density, optimum water content, matric suction, MBV, PFC, saturated hydraulic conductivity, and resistance R-value.

The primary focus of this effort was to measure SWCCs for all unbound materials and to evaluate its impact on Nevada Pavement ME Design. This impact was found by conducting a sensitivity analysis using the AASHTOWare Pavement ME software (ver. 2.3.1), and evaluating the pavement performance while using input levels 1, 2, and 3, as well as current NDOT recommendations for SWCC and saturated hydraulic conductivity for the unbound materials. It was found that the input of SWCC and saturated hydraulic conductivity has an impact on AC bottom-up fatigue cracking and minimal impact on permanent deformation. The AC bottom-up fatigue cracking is most impactful in District 1, where traffic loading is increased in comparison to the other two districts.

It was found in District 1 that the current recommendations for SWCC and saturated hydraulic conductivity underestimate the impact from these parameters on pavement performance, specifically in AC bottom-up fatigue cracking. Input level 1, which involves the direct measurement of SWCCs and saturated hydraulic conductivity shows the need to design a pavement structure with an increased thickness in the AC layer by 0.5 inches (from 7 inch design passing current recommendations to 7.5 inch design passing input level 1). The impact of input level 1 SWCC and saturated hydraulic conductivity in District 2 showed the opposite trend, where according to input level 1, a 6.5 inch AC layer design passed the AC bottom-up fatigue cracking criteria, but only a 7 inch AC layer design could pass using the current recommendations. SWCC and saturated hydraulic conductivity inputs did not greatly impact AC bottom-up fatigue cracking in District 3.

Chapter 3 presents recommended values to be incorporated into Nevada Pavement ME Design. The recommended values are for base and borrow materials. It was found that altering the SWCC and hydraulic conductivity for only the subgrade material in Pavement ME does not impact the performance of the pavement structure; therefore, continued use of the national catalog of natural subgrade properties (4) could be appropriate to use for subgrade SWCC and saturated hydraulic conductivity input. However, the base and borrow material SWCC and saturated hydraulic conductivity should be specified, as the estimation made by the EICM in input level 3 has been shown to inaccurately predict SWCC and saturated hydraulic conductivity. Therefore, the representative values for gradation, Atterberg limits, maximum dry density, optimum water content, specific gravity of solids, SWCC, and saturated hydraulic conductivity for each district should be used instead.

Additionally, a calibration of the input level 2 models for base and borrow materials was performed, and these calibrated models are offered in Chapter 3. Before these models can be incorporated into Pavement ME Design, it is recommended that further testing be conducted on Nevada base material that has $PI*P_{200} > 0$, since not all base materials are non-plastic, as was the case for all base materials tested in this study. The District 2 base material calibrated input level 2 model should be used cautiously, as only one base material was collected from District 2, and this model may not be representative of all materials encountered in the district. The same can be said for the District 3 borrow material model.

While MBV and PFC testing was conducted on District 1 borrow and subgrade material, material from the other two districts could not be tested. Therefore, MBV and PFC testing should be performed on Districts 2 and 3 materials in order to better assess the use of MBV and PFC as parameters that be correlated to SWCC in Nevada's unbound materials.

Further research can be conducted to explore the use of other SWCC models for Pavement ME design. For example, the van Genuchten model is a commonly used approach in SWCC research (21). This may lead to improved SWCC model fitting, as computer programs such as RETC, which can be used to analyze SWCCs, can be used with the van Genuchten model.

An investigation should also be made in the MEPDG for correcting the SWCC for coarse aggregate. While the corrected maximum dry density and corrected optimum water content were used in this study, as applicable, this may not properly correct the SWCC. The Bouwer-Rice method is commonly used, and Bareither and Benson offer a simplified version of this method (21).

CHAPTER 5 REFERENCES

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CHAPTER 6 APPENDIX A

Laboratory test results are shown in this appendix, including moisture-density relationships and SWCC.

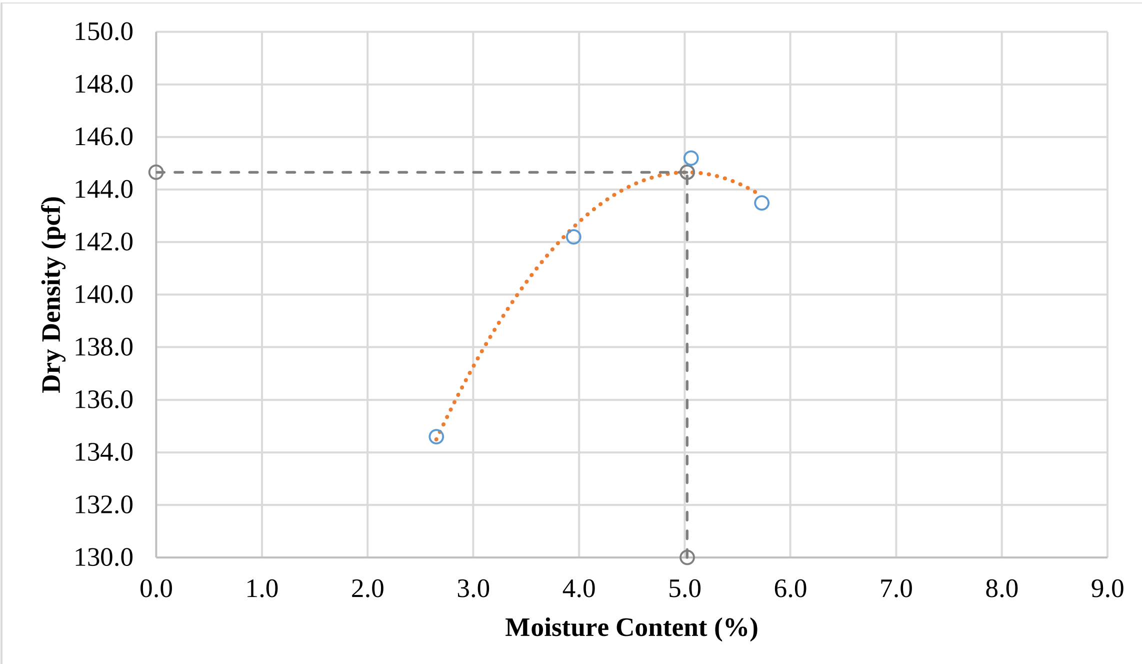

Figure 82. Moisture-density curve for base material (contract 3546).

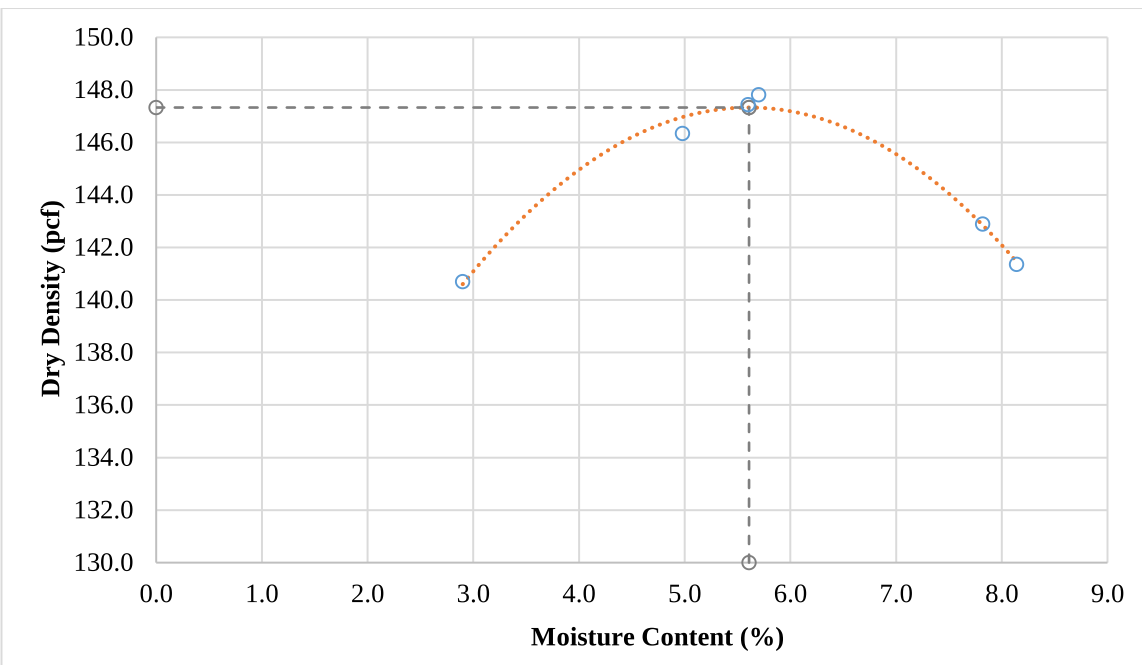

Figure 83. Moisture-density curve for base material (contract 3583).

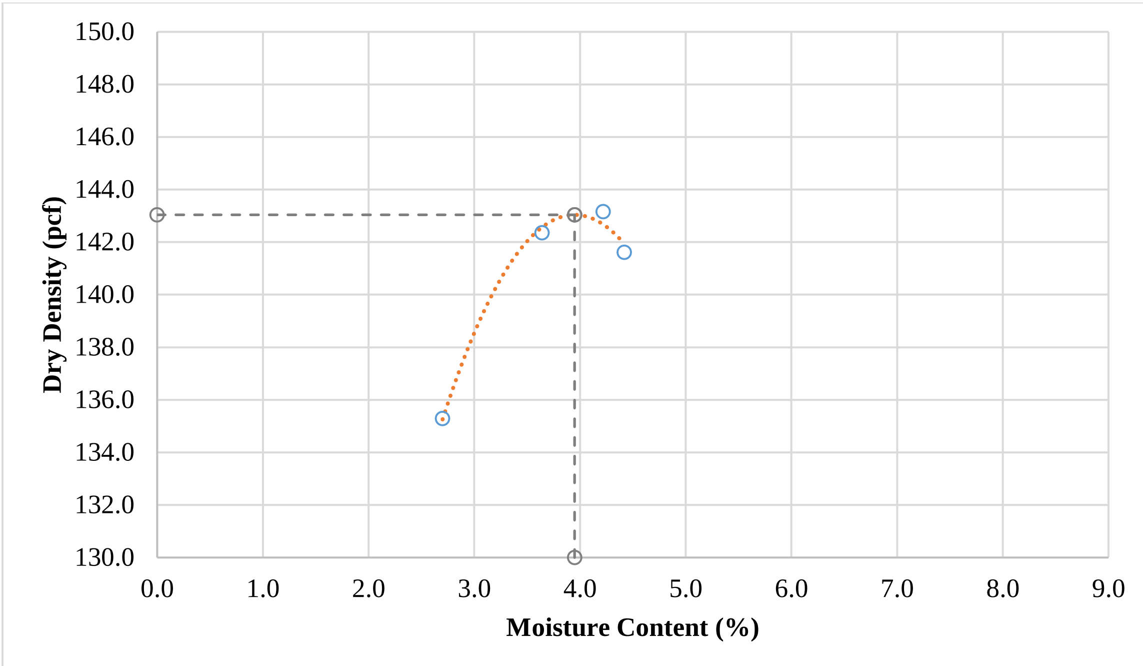

Figure 84. Moisture-density curve for base material (contract 3597).

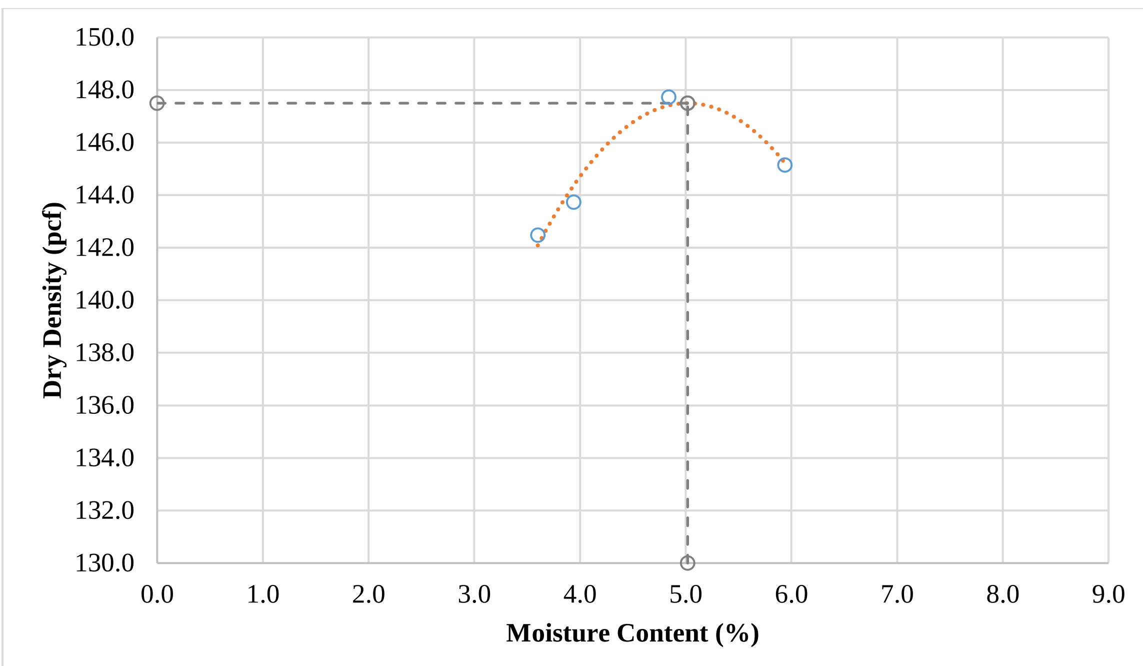

Figure 85. Moisture-density curve for base material (contract 3605).

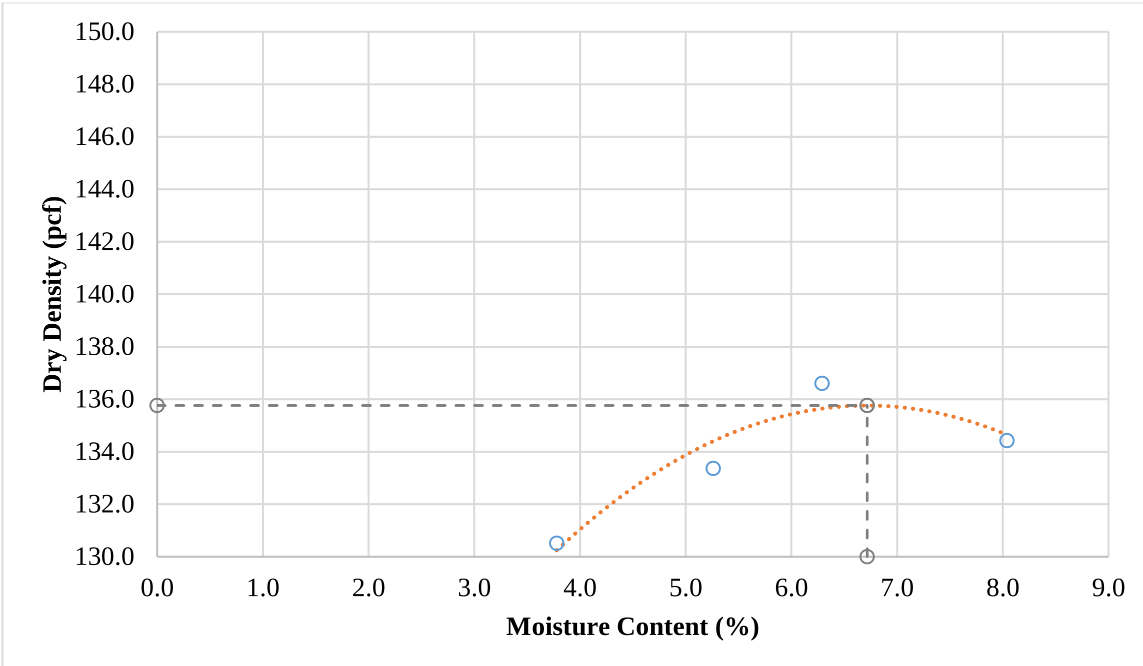

Figure 86. Moisture-density curve for base material (contract 3607).

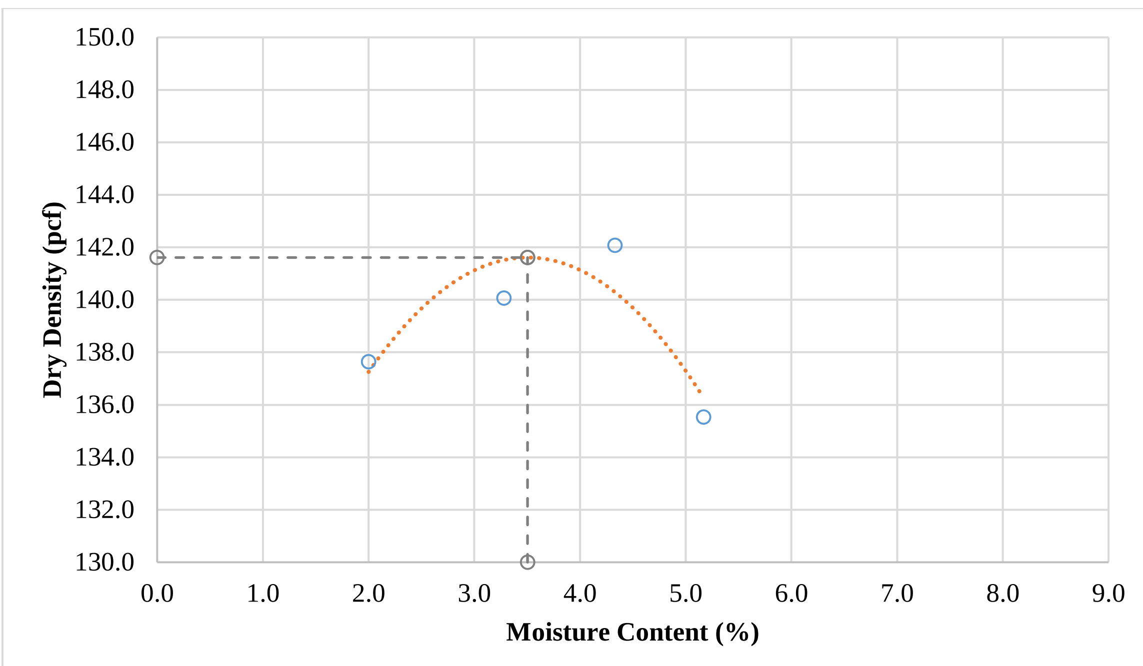

Figure 87. Moisture-density curve for base material (contract 3613).

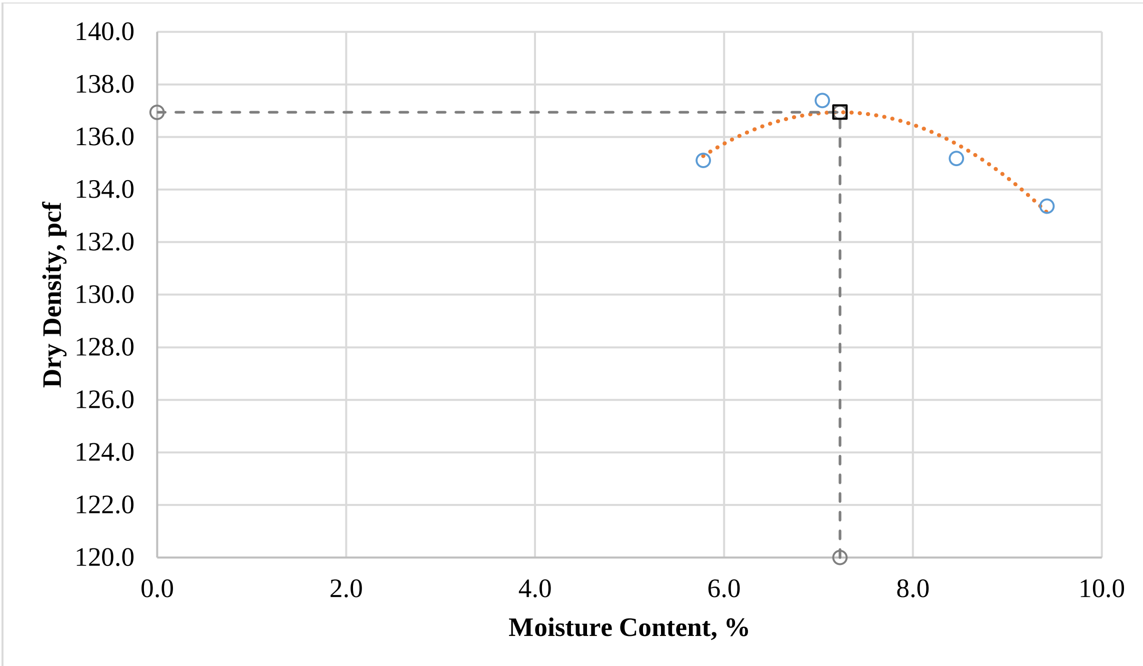

Figure 88. Moisture-density curve for borrow material (contract 3546).

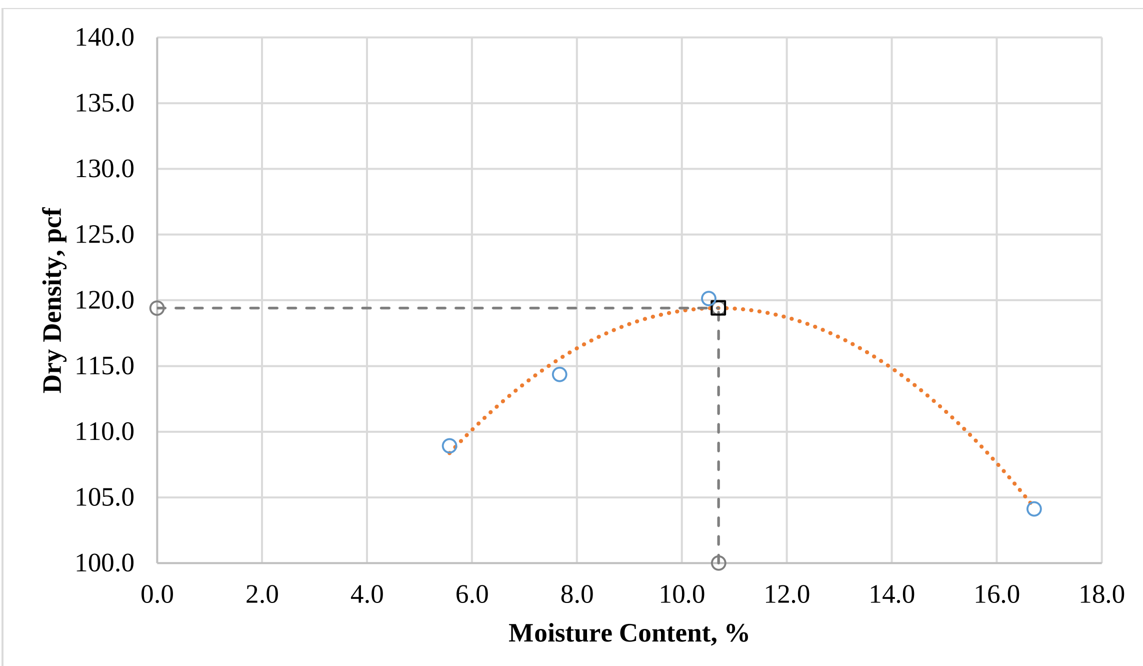

Figure 89. Moisture-density curve for borrow material (contract 3583).

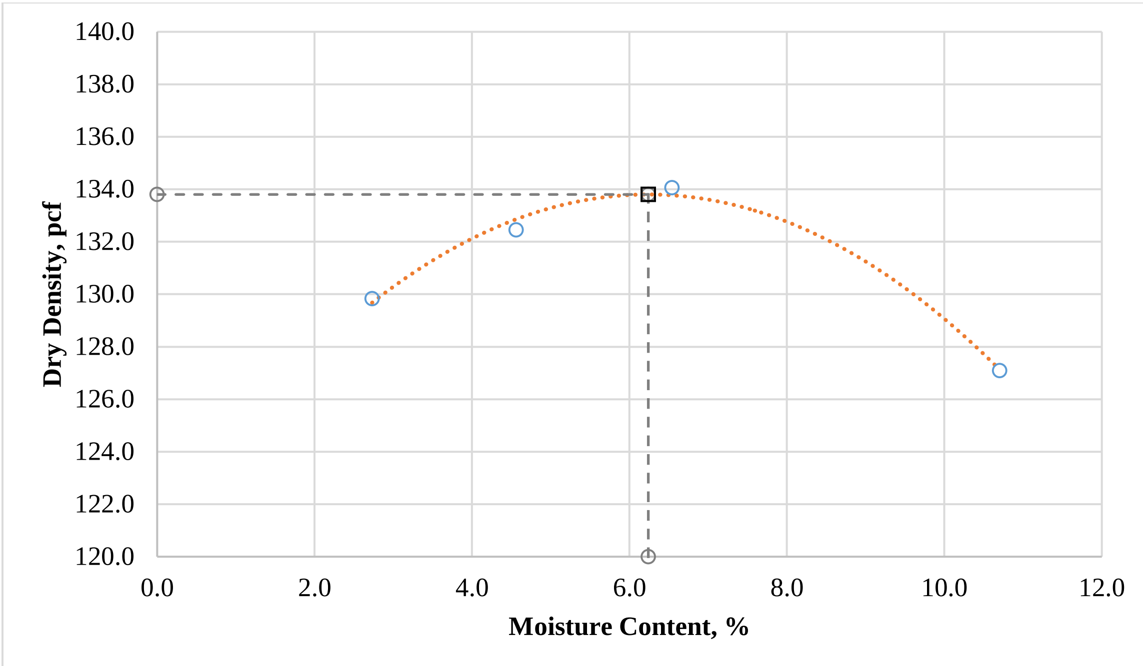

Figure 90. Moisture-density curve for borrow material (contract 3597).

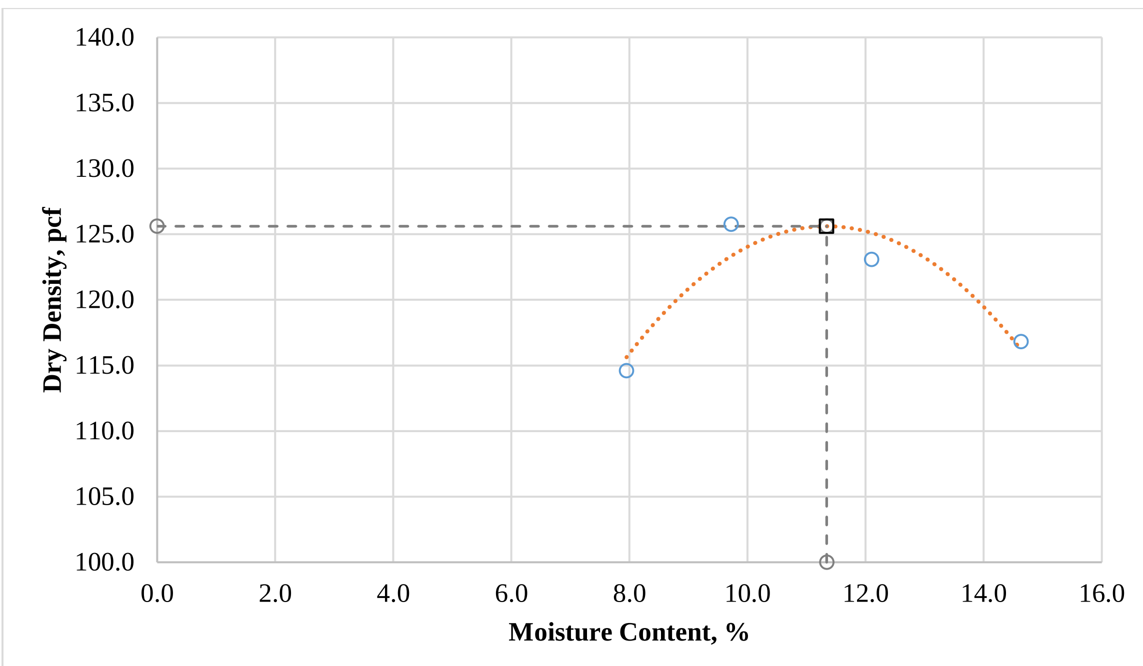

Figure 91. Moisture-density curve for borrow material (contract 3607).

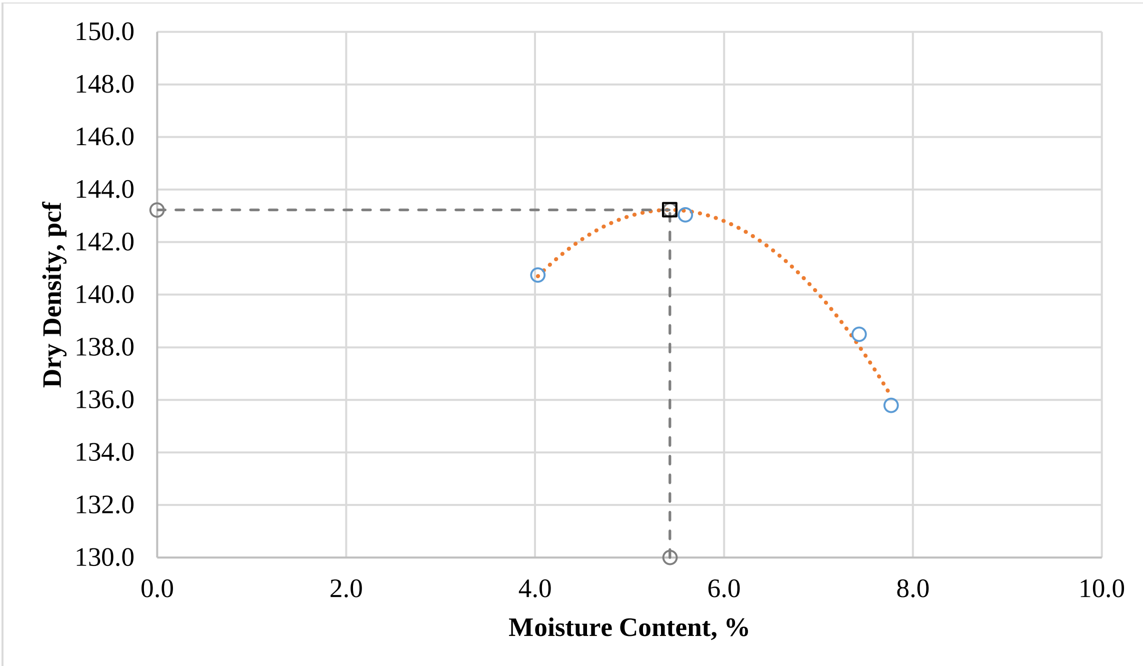

Figure 92. Moisture-density curve for borrow material (contract 3613).

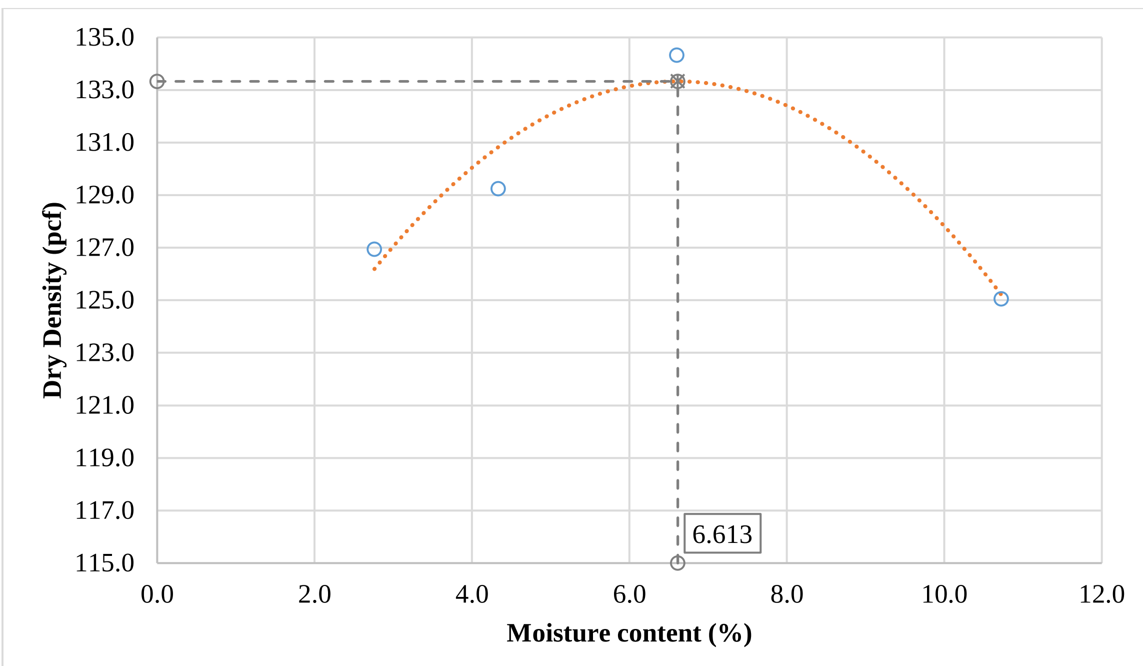

Figure 93. Moisture-density curve for subgrade material (US-95/Searchlight).

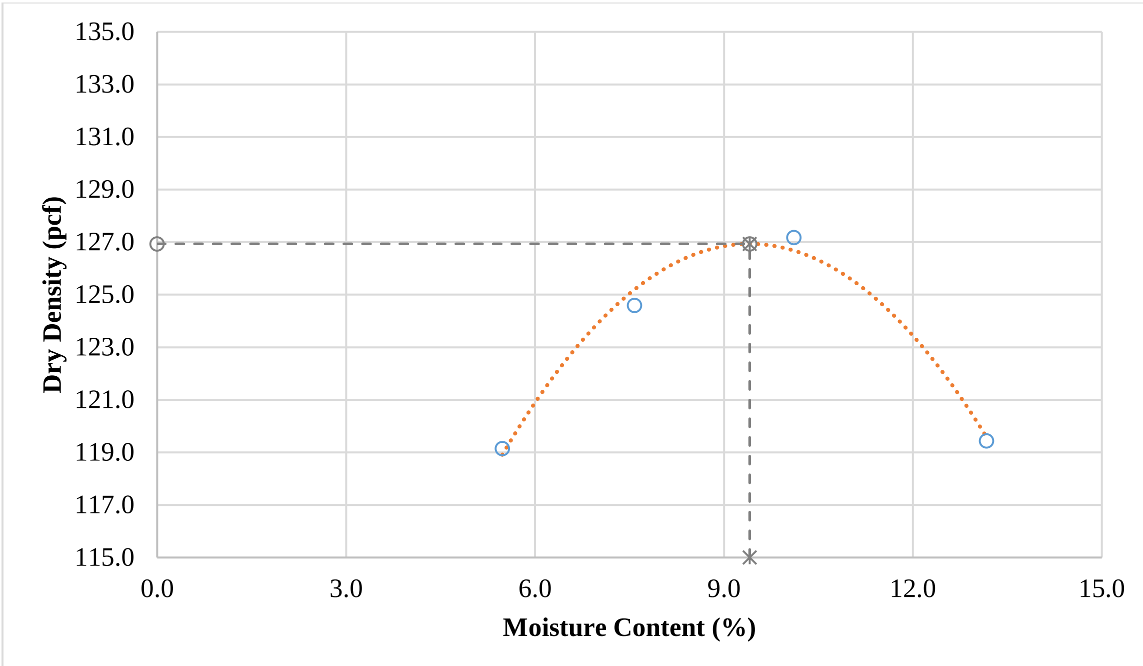

Figure 94. Moisture-density curve for subgrade material (US-95/Bonnie Claire).

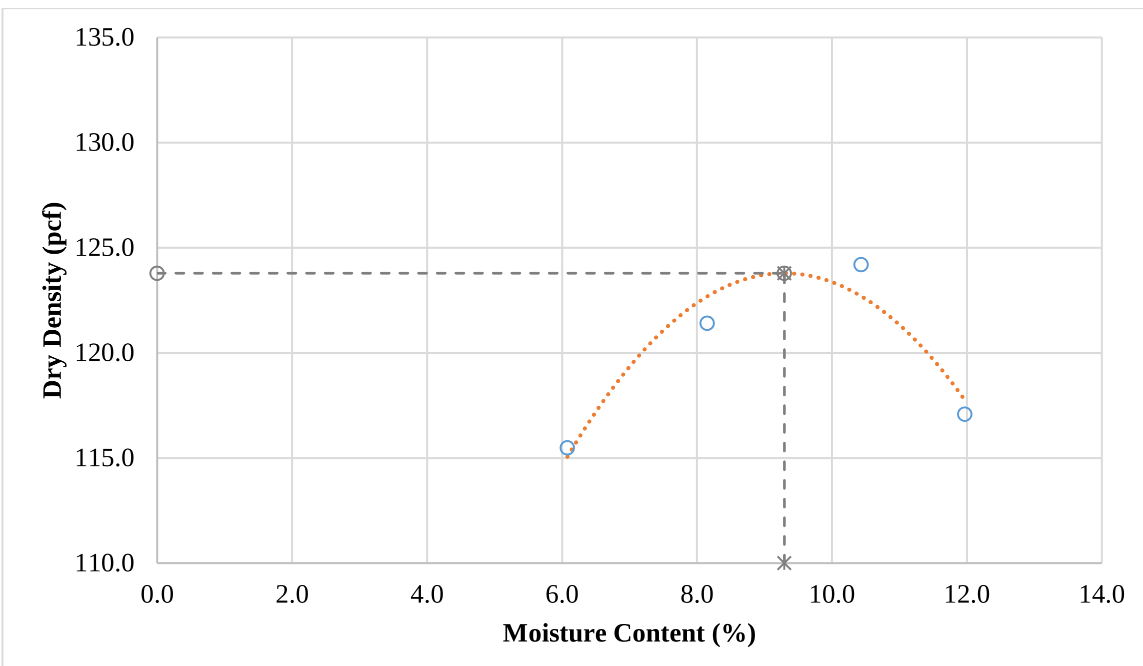

Figure 95. Moisture-density curve for subgrade material US-93/Crystal Spring MP67).

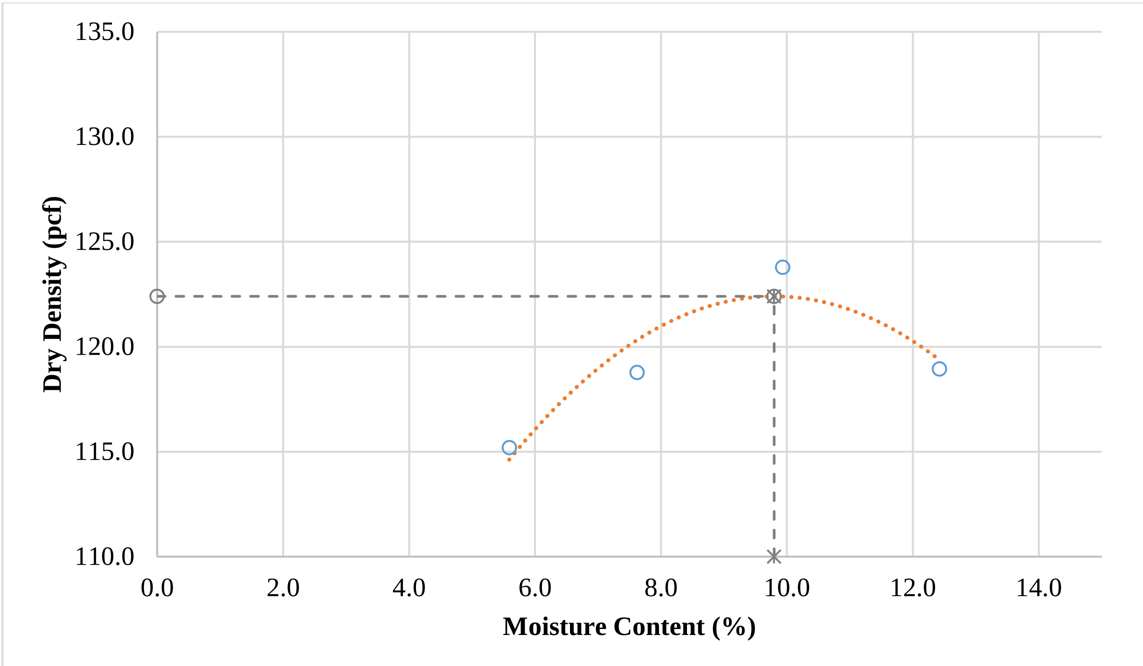

Figure 96. Moisture-density curve for subgrade material (US-93/Crystal Spring MP62).

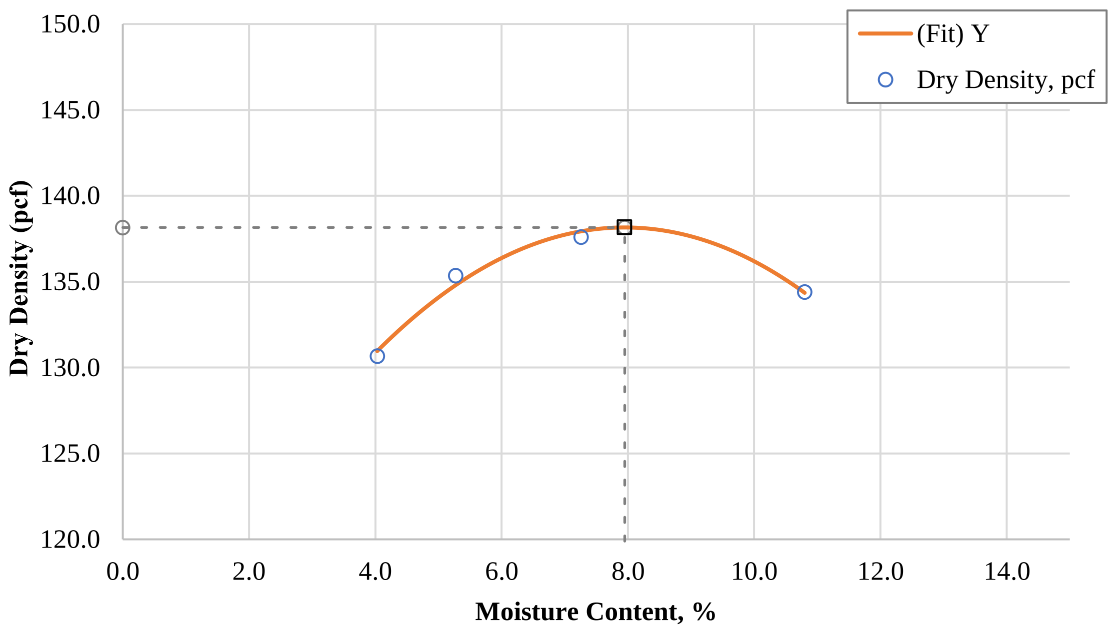

Figure 97. Moisture-density curve for Lockwood base.

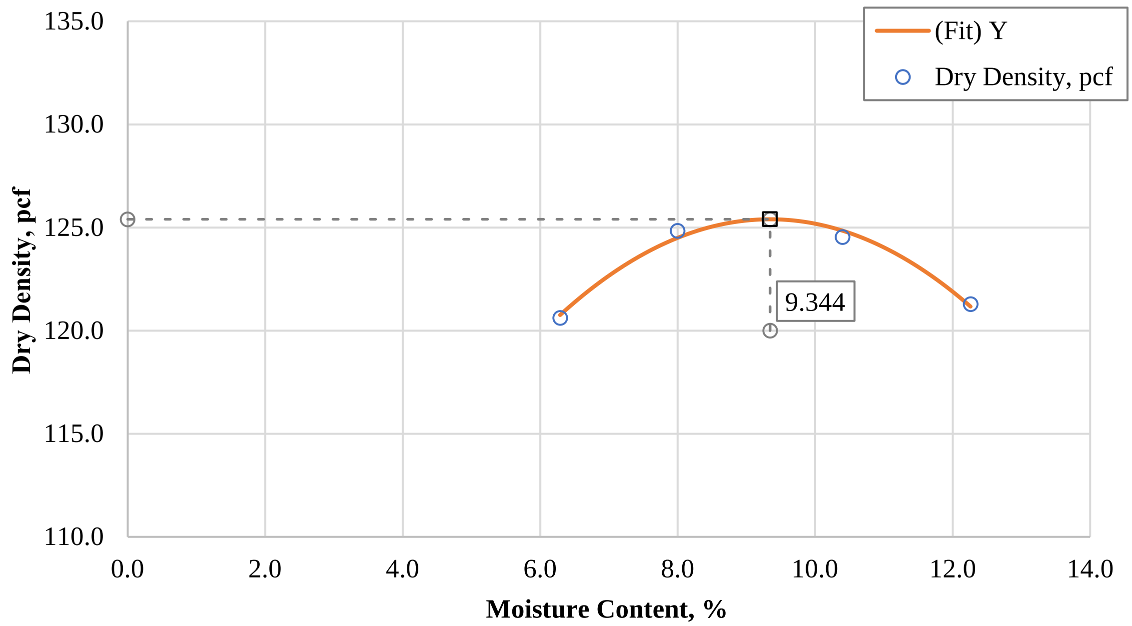

Figure 98. Moisture-density curve for Lockwood borrow.

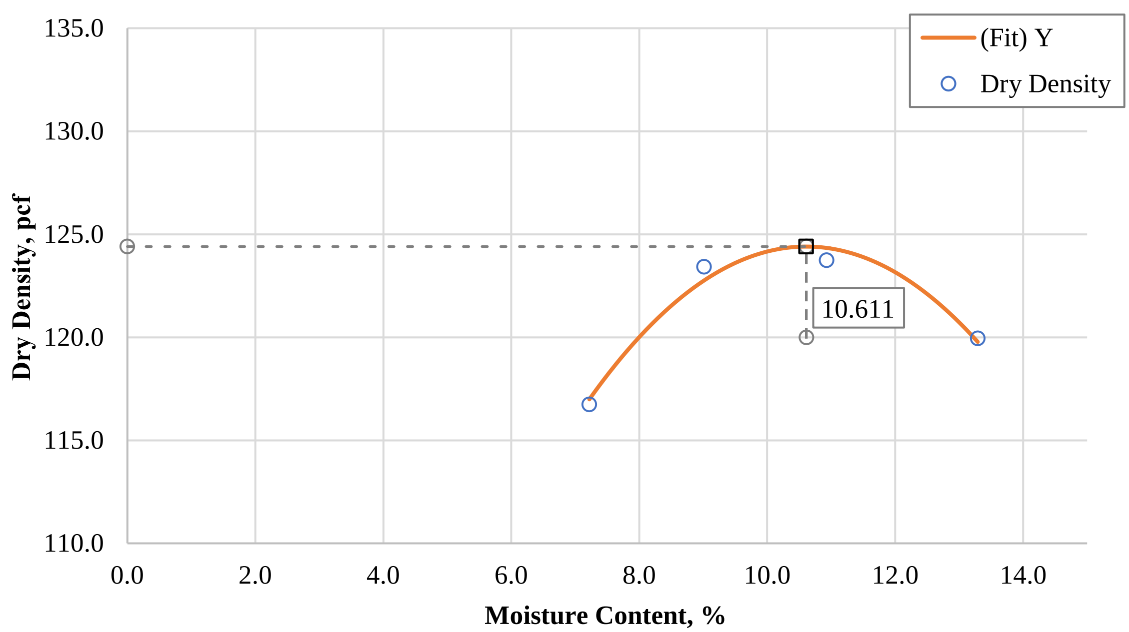

Figure 99. Moisture-density curve for SNC Primary borrow.

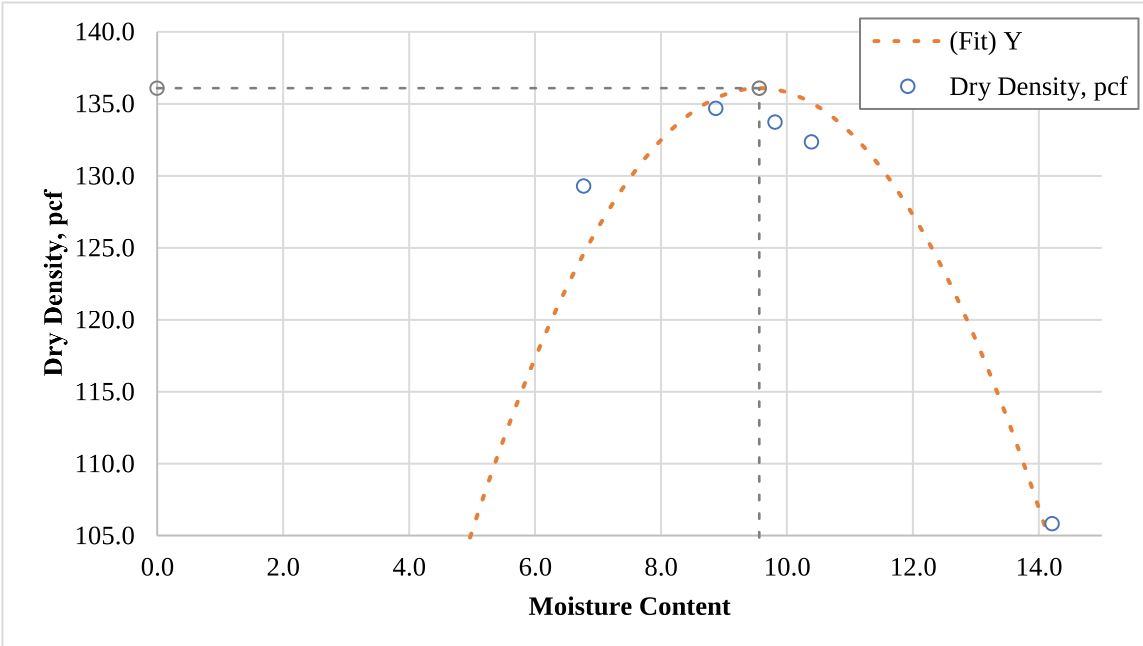

Figure 100. Moisture-density curve for SNC Secondary borrow.

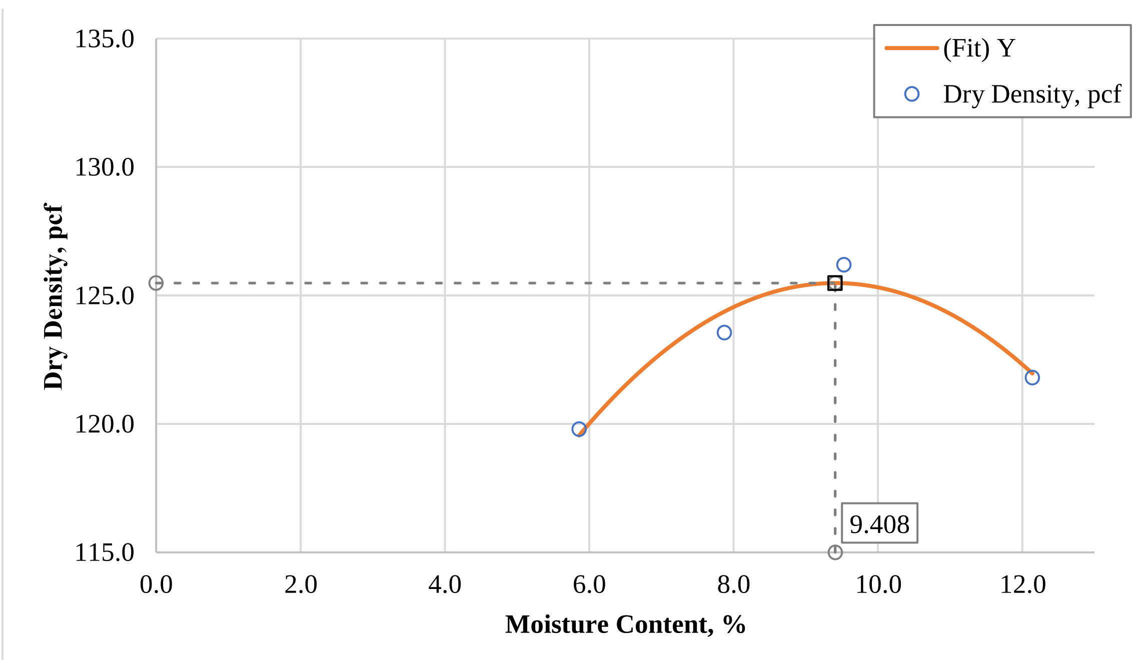

Figure 101. Moisture-density curve for Jacks Valley subgrade.

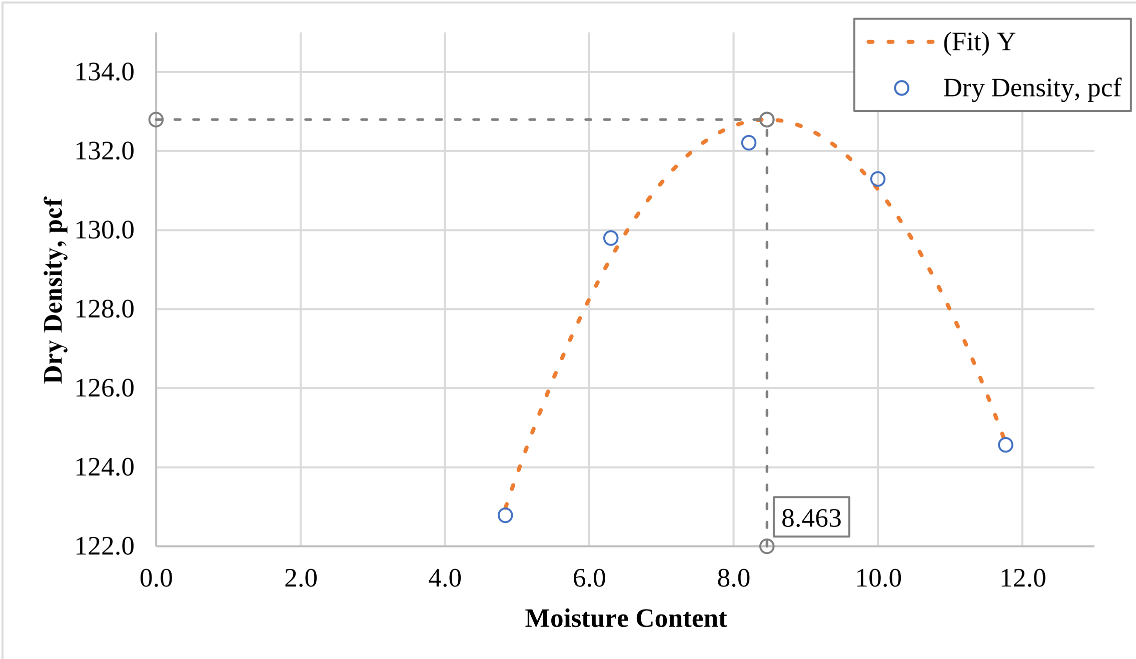

Figure 102. Moisture-density curve for SEM Soil at UNR.

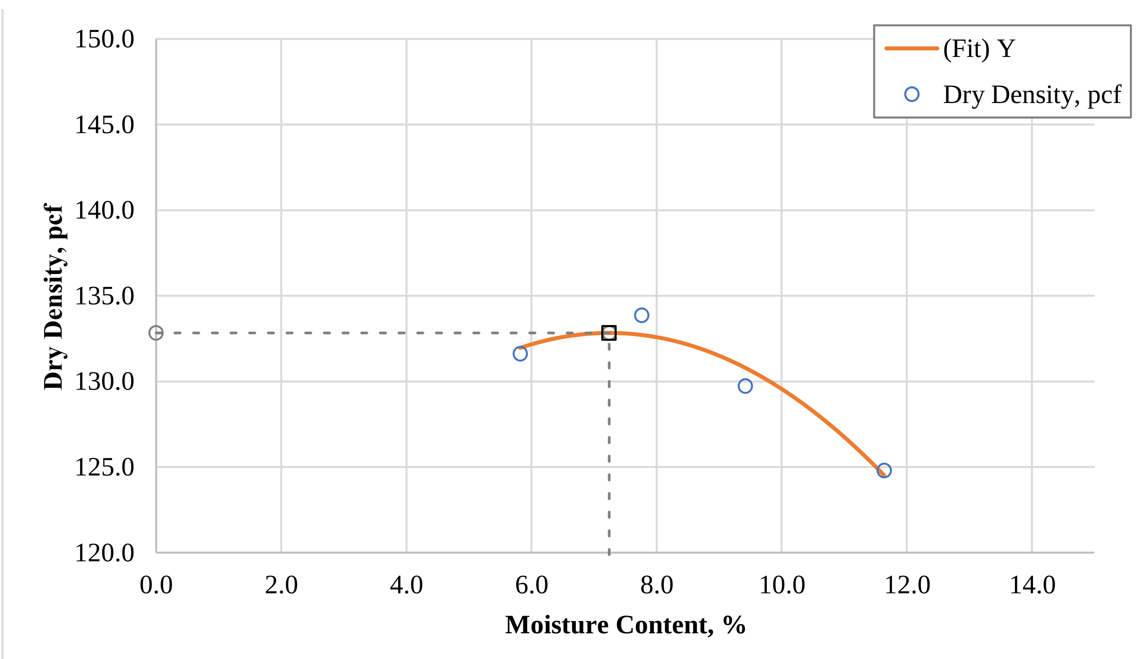

Figure 103. Moisture-density curve for Hunnewill base.

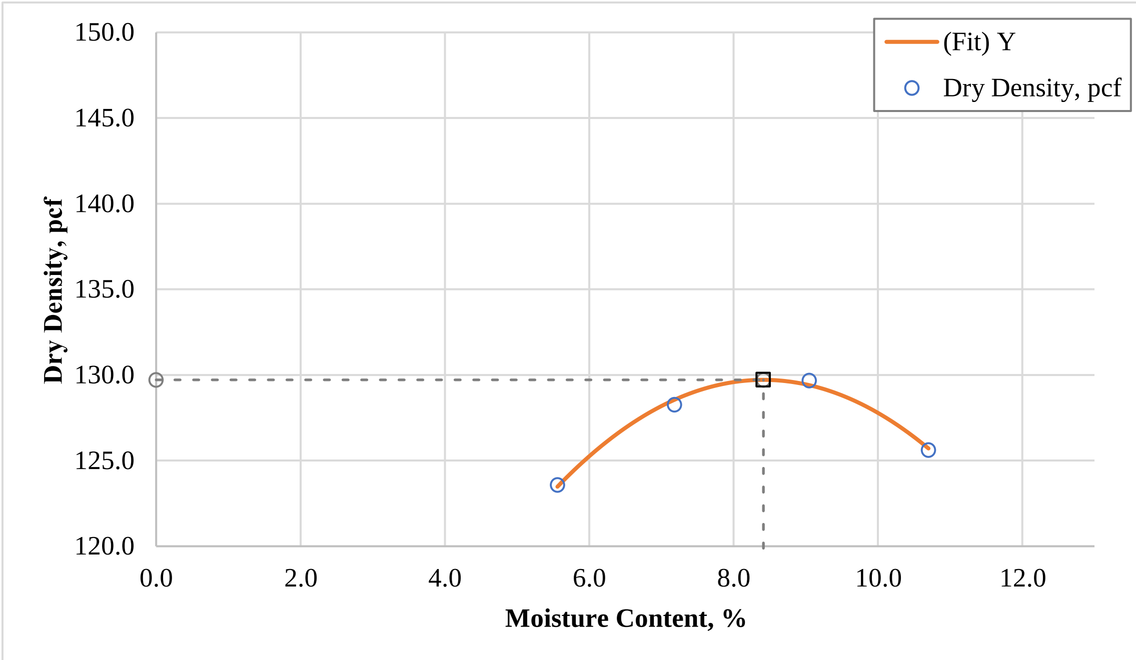

Figure 104. Moisture-density curve for Elko base.

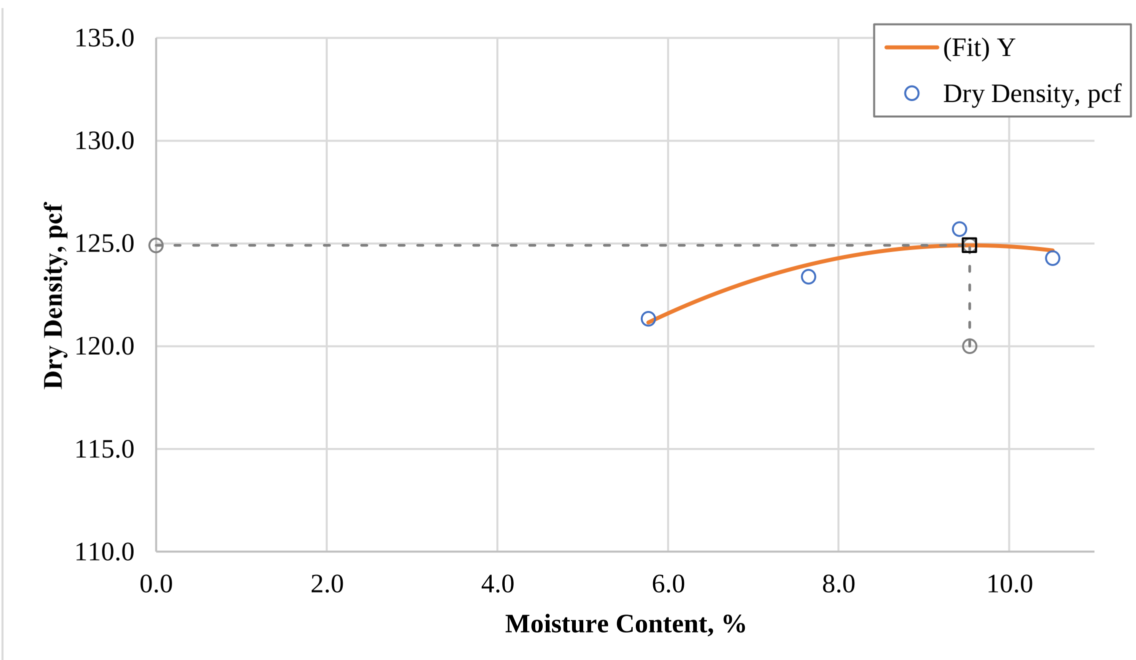

Figure 105. Moisture-density curve for Elko borrow.

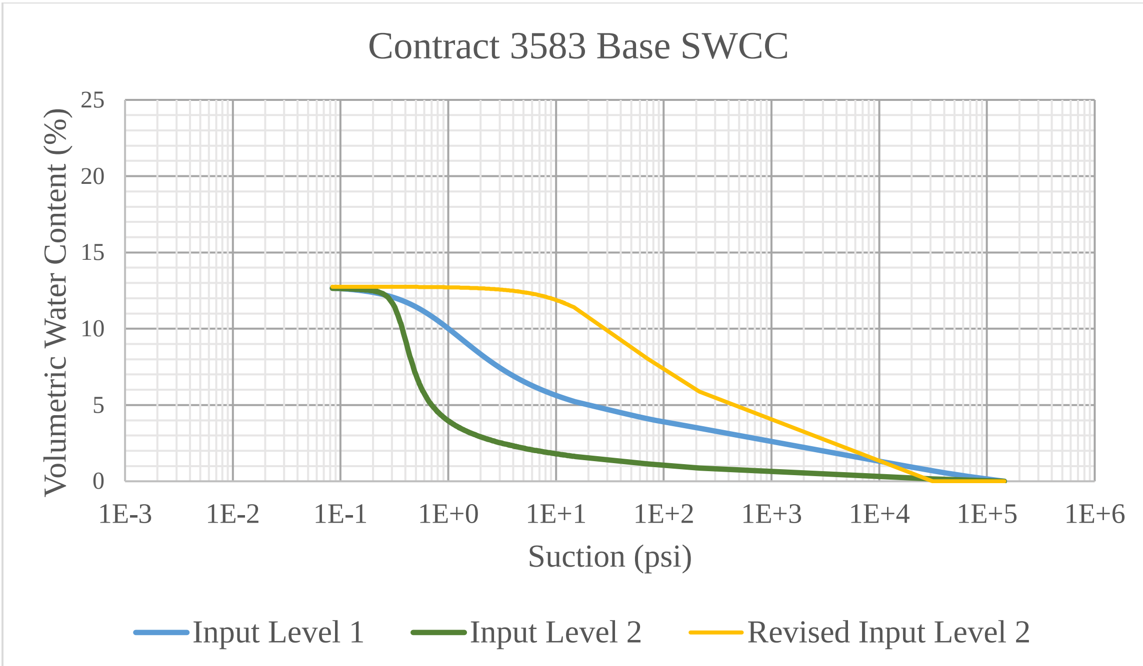

Figure 106. Contract 3583 base SWCC.

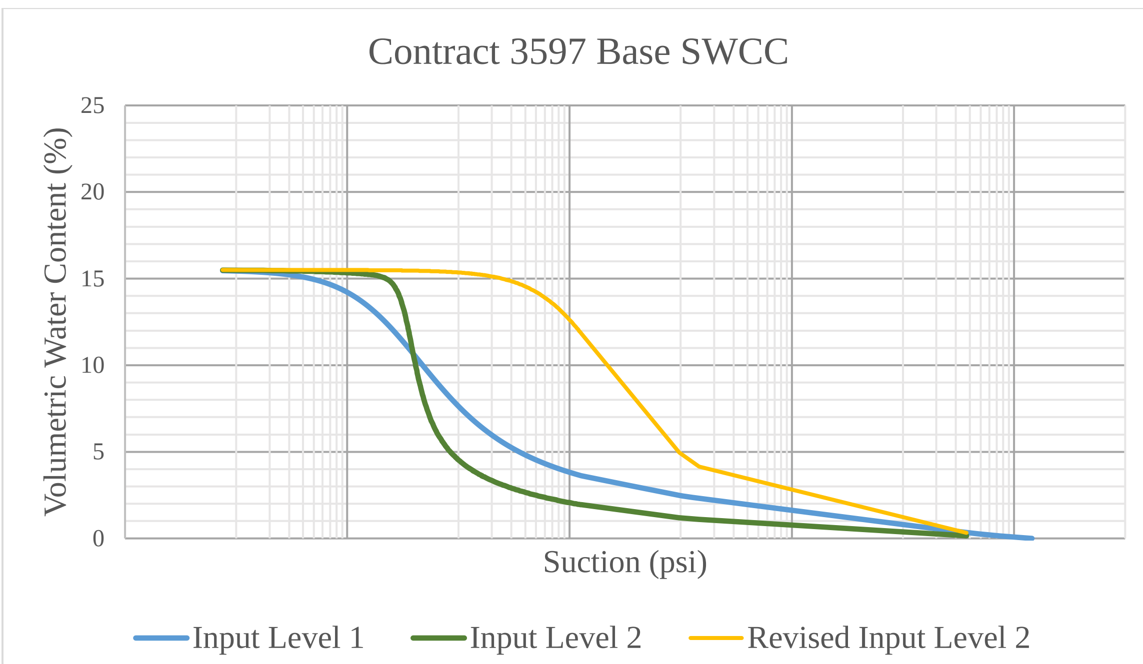

Figure 107. Contract 3597 base SWCC.

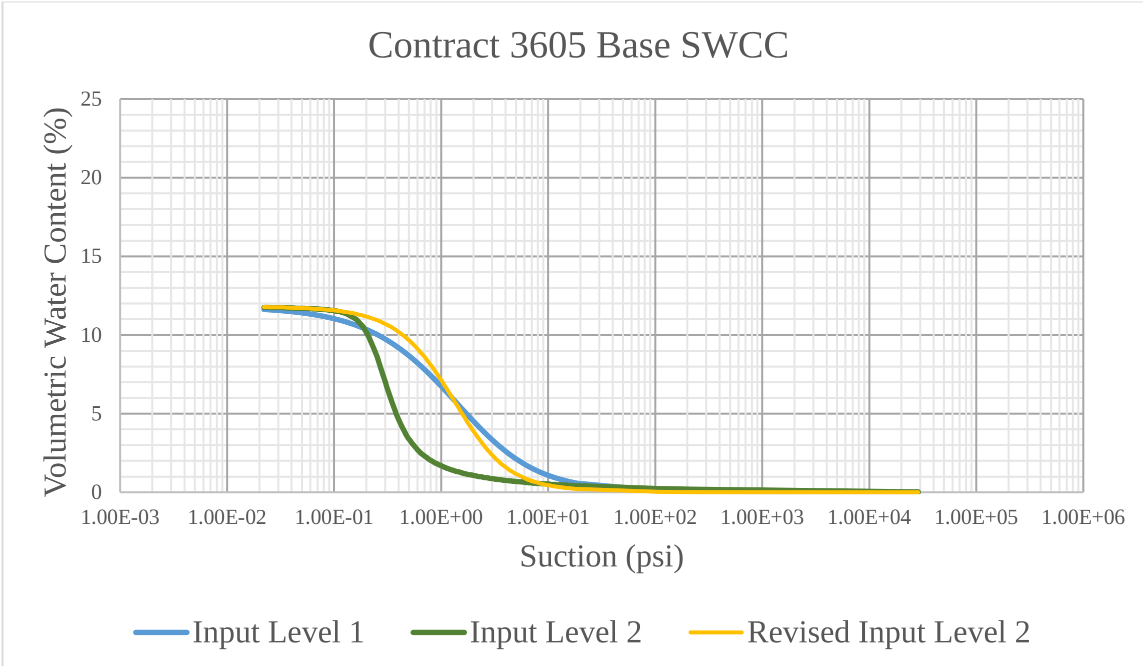

Figure 108. Contract 3605 base SWCC.

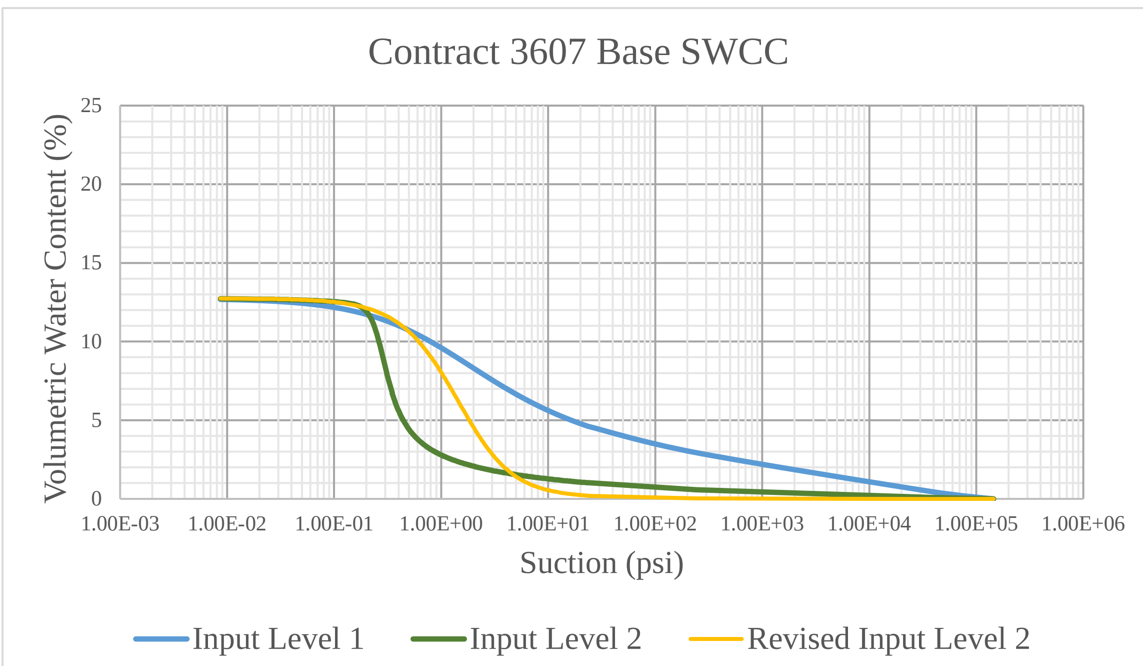

Figure 109. Contract 3607 base SWCC.

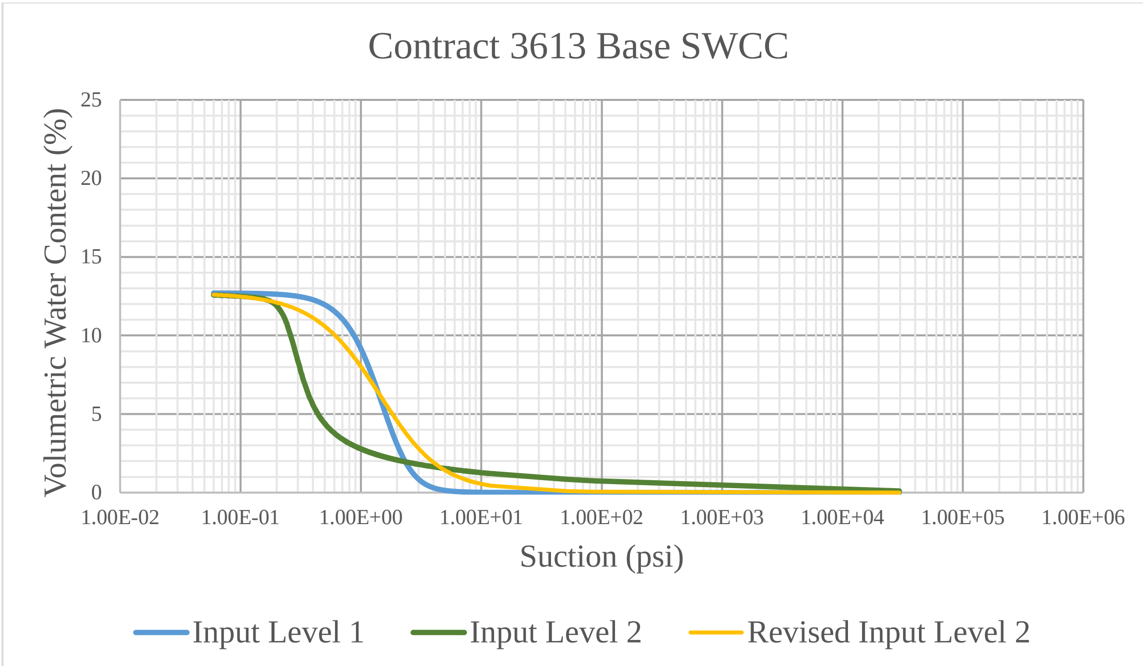

Figure 110. Contract 3613 base SWCC.

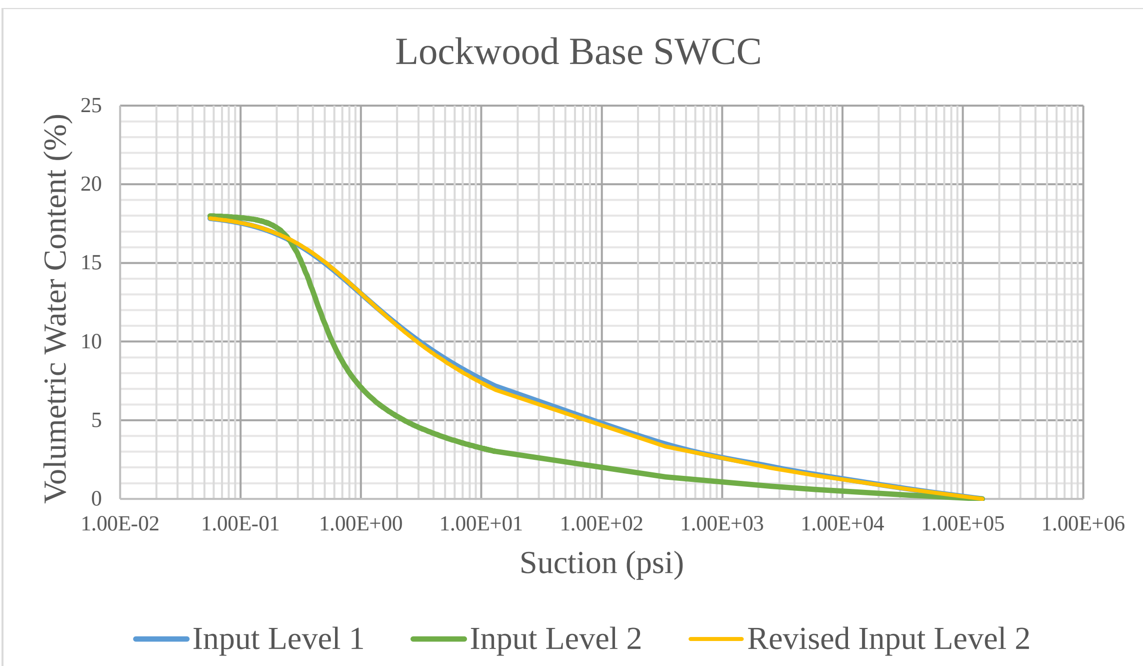

Figure 111. Lockwood base SWCC.

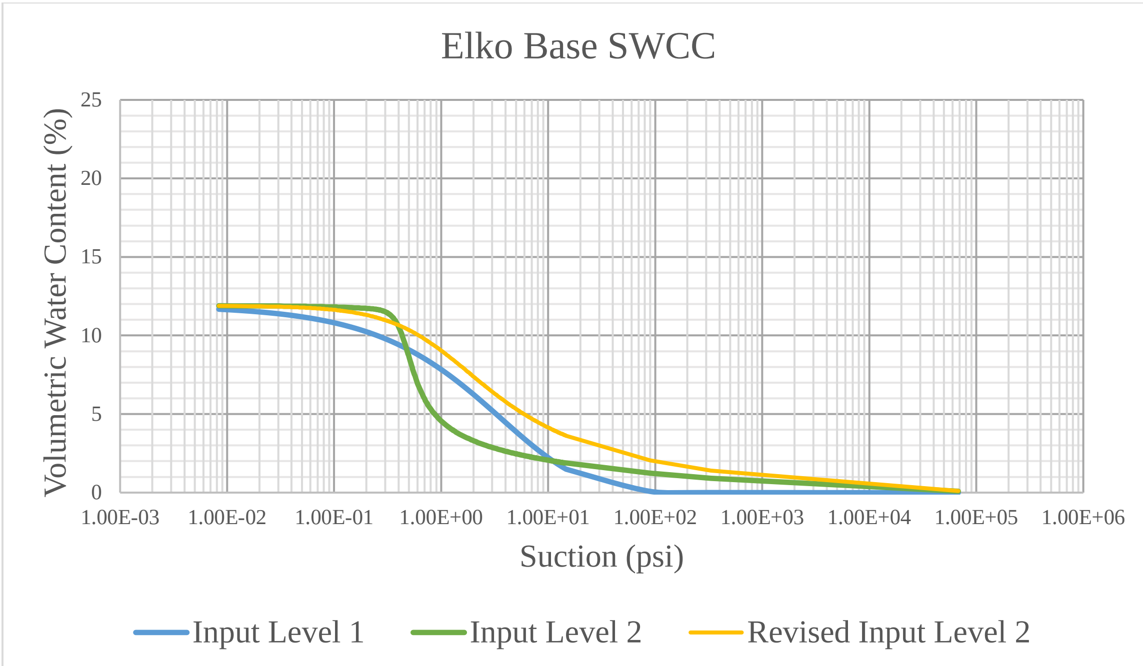

Figure 112. Elko base SWCC.

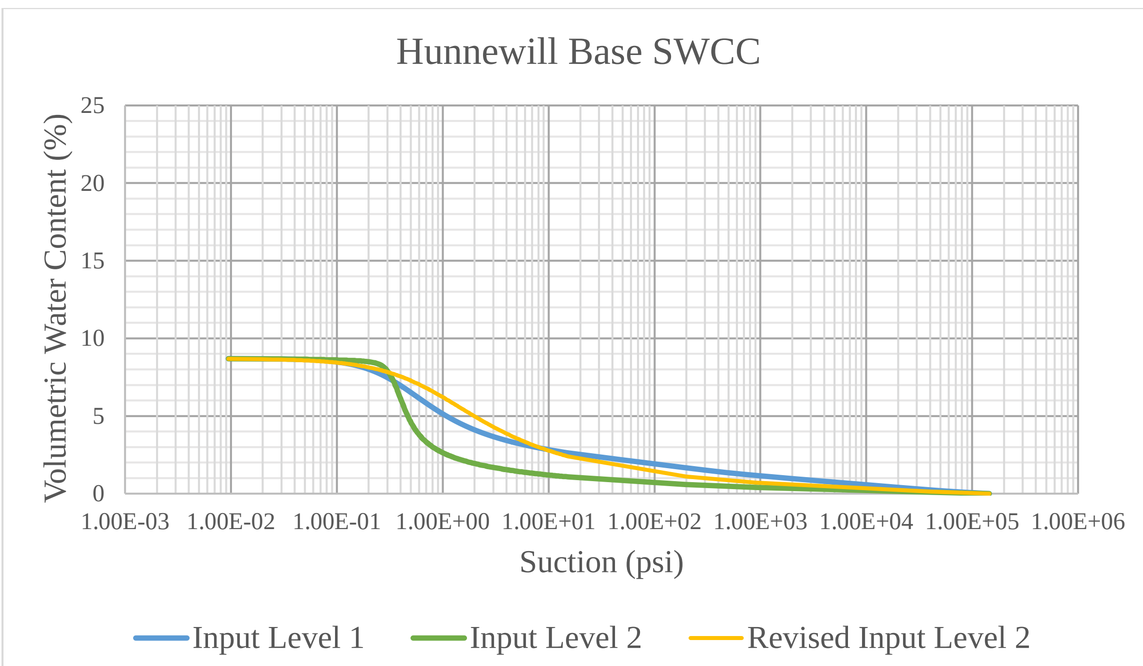

Figure 113. Hunnewill base SWCC.

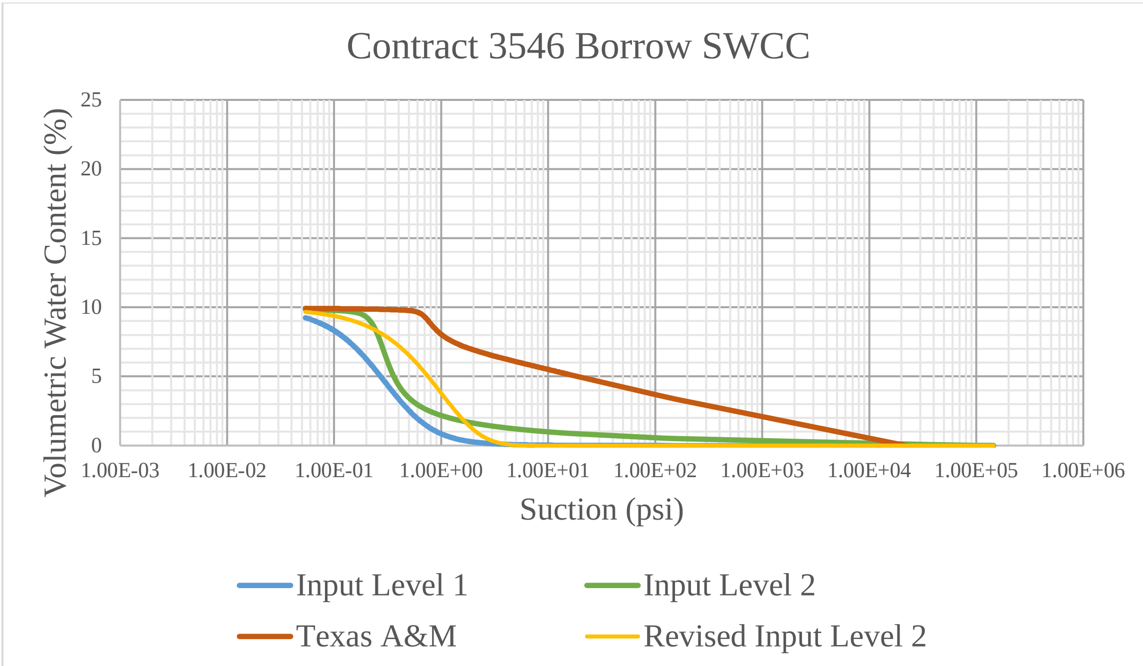

Figure 114. Contract 3546 borrow SWCC.

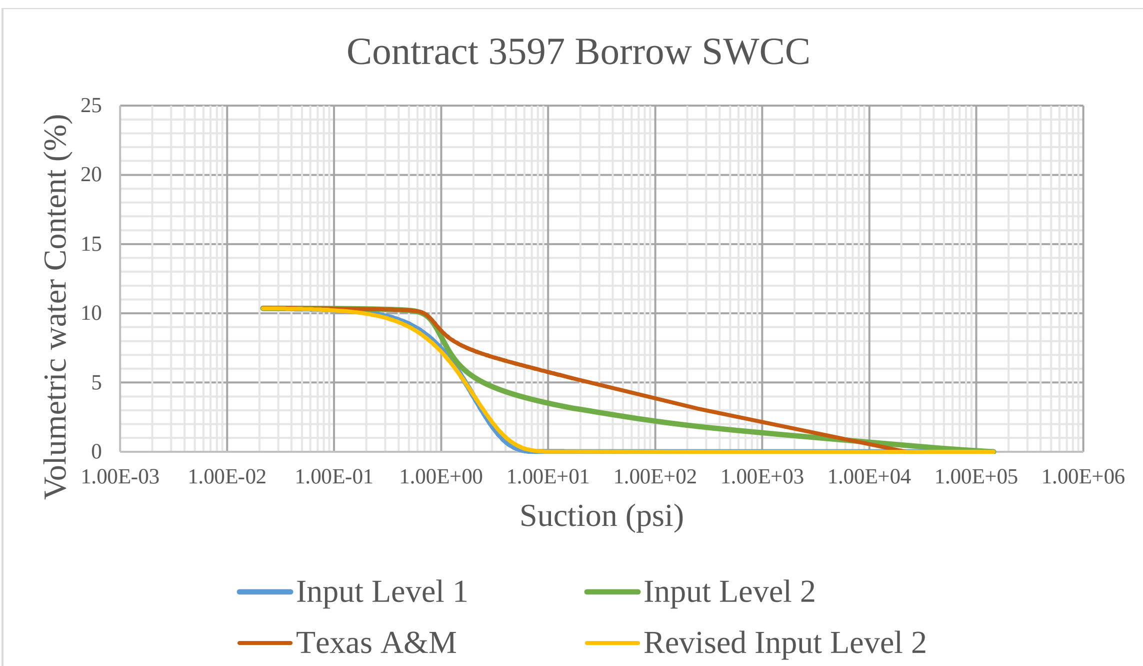

Figure 115. Contract 3597 borrow SWCC.

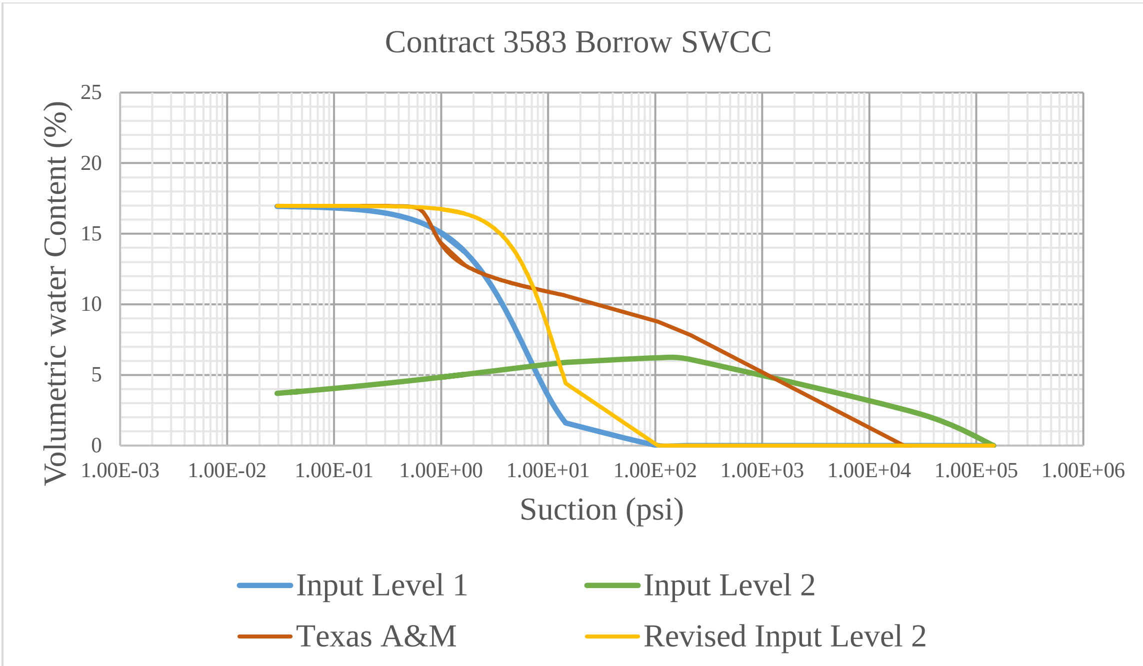

Figure 116. Contract 3583 borrow SWCC.

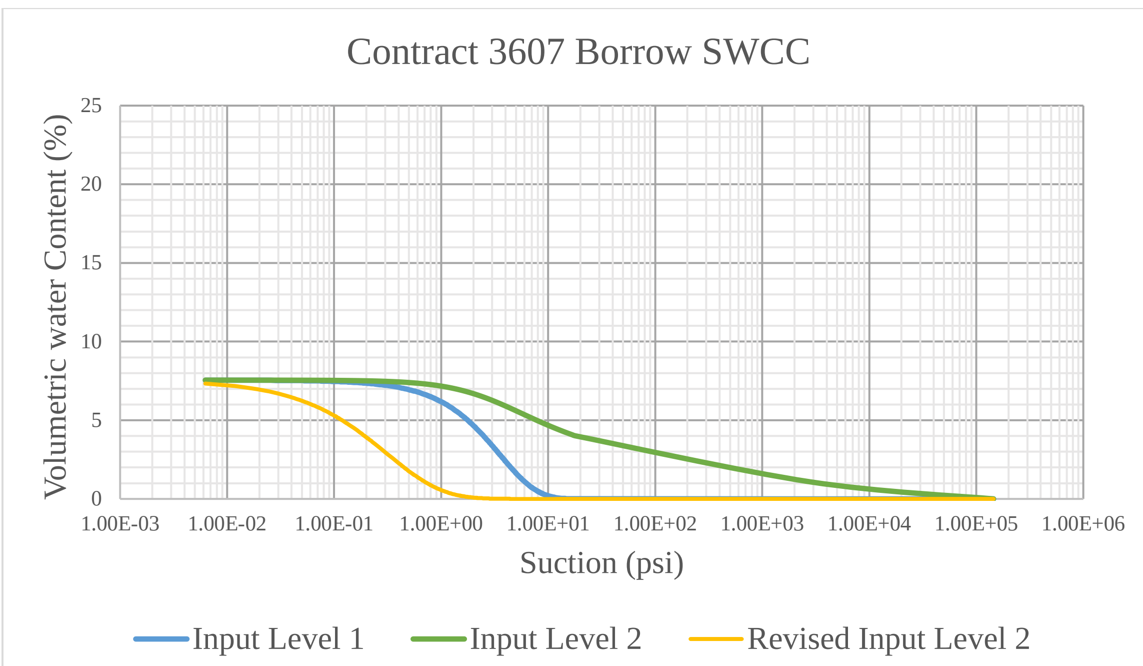

Figure 117. Contract 3607 borrow SWCC.

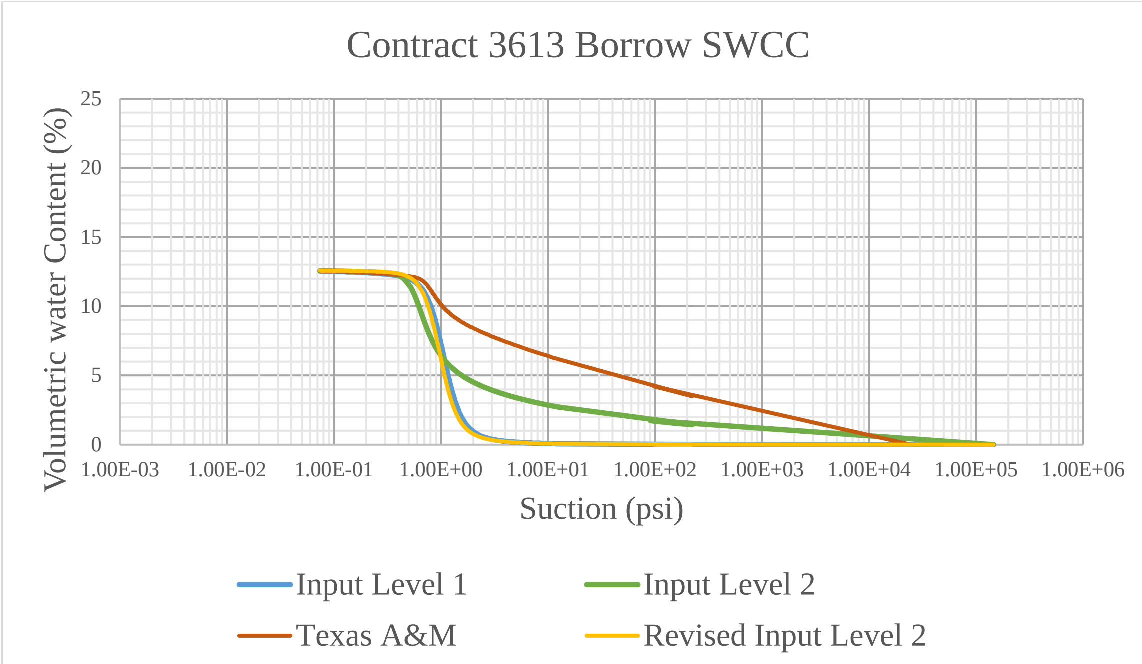

Figure 118. Contract 3613 borrow SWCC.

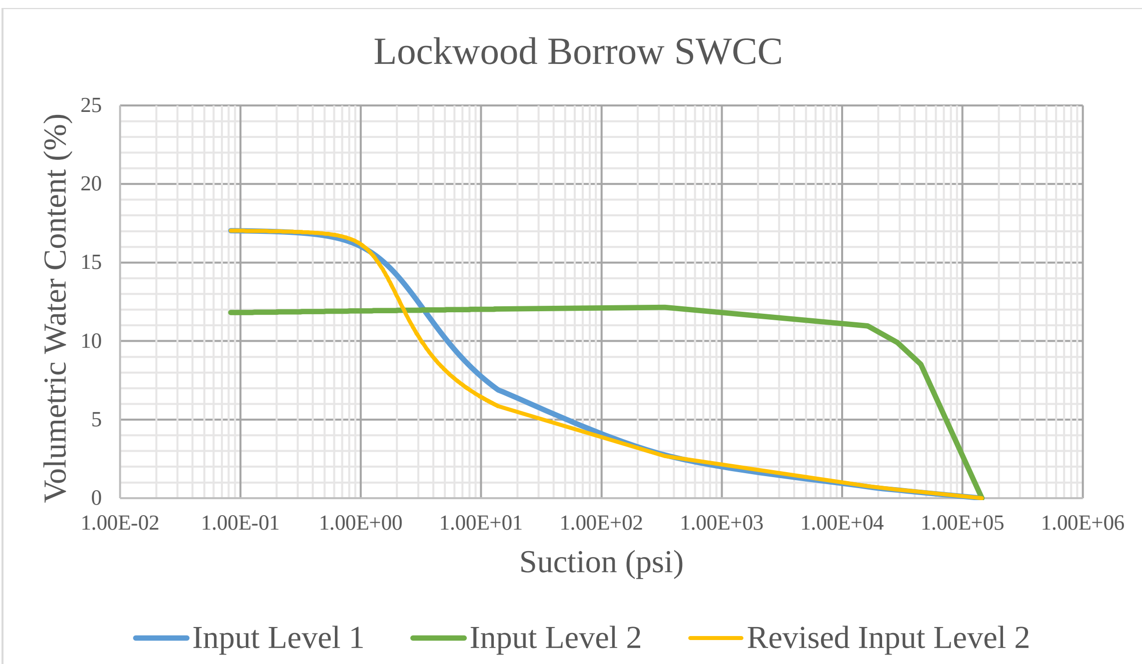

Figure 119. Lockwood borrow SWCC.

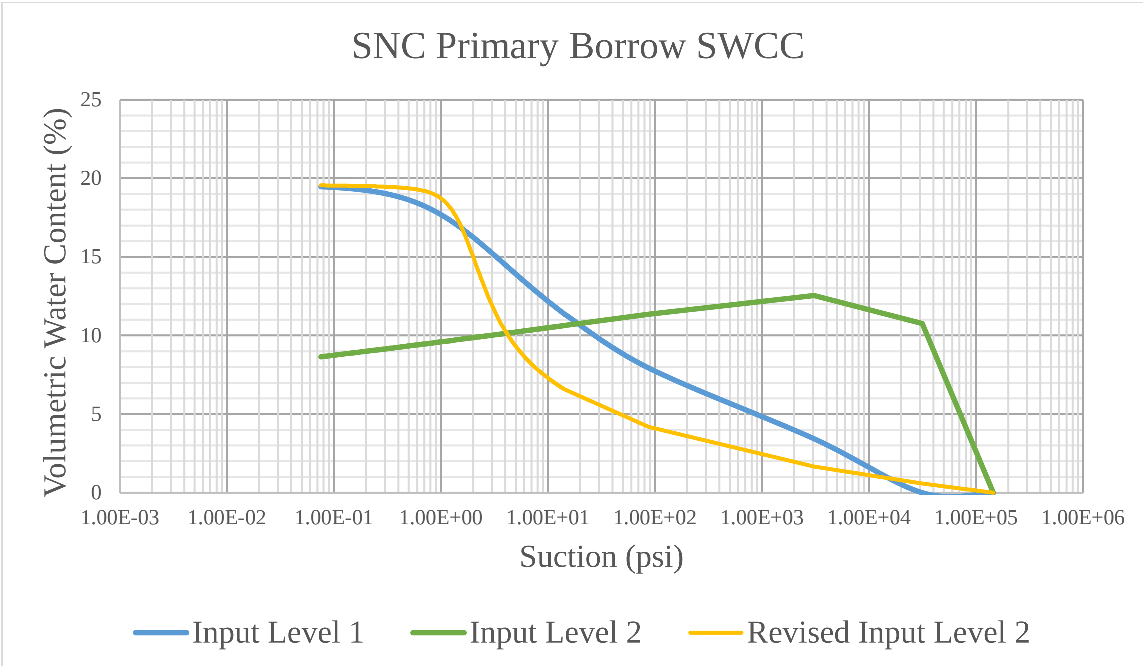

Figure 120. SNC Primary borrow SWCC.

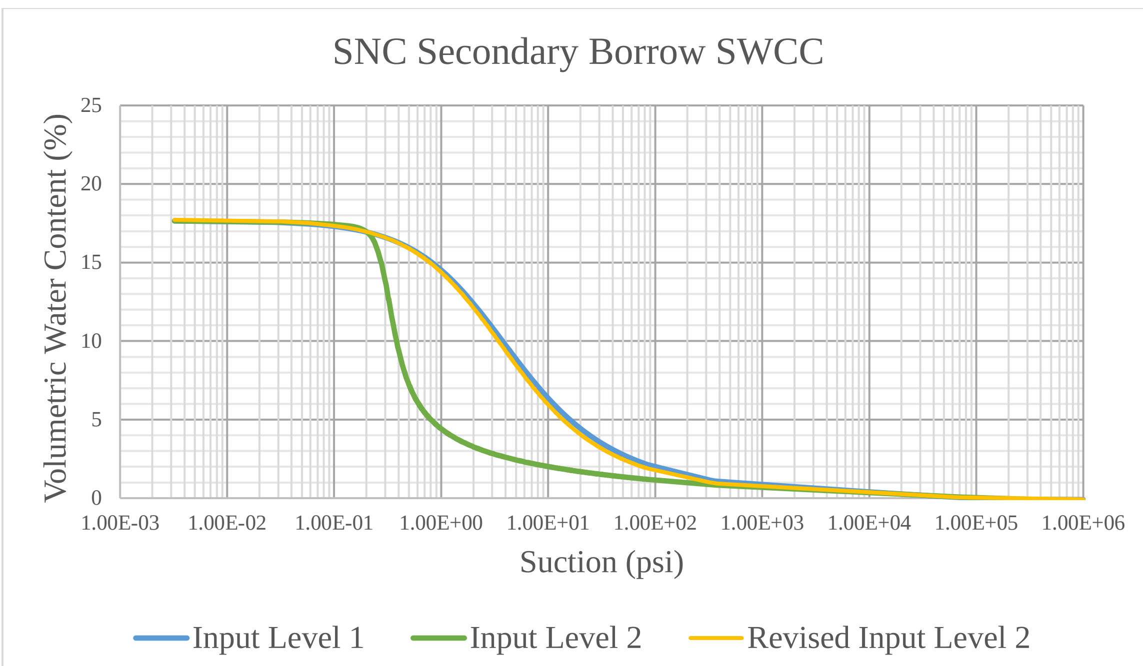

Figure 121. SNC Secondary borrow SWCC.

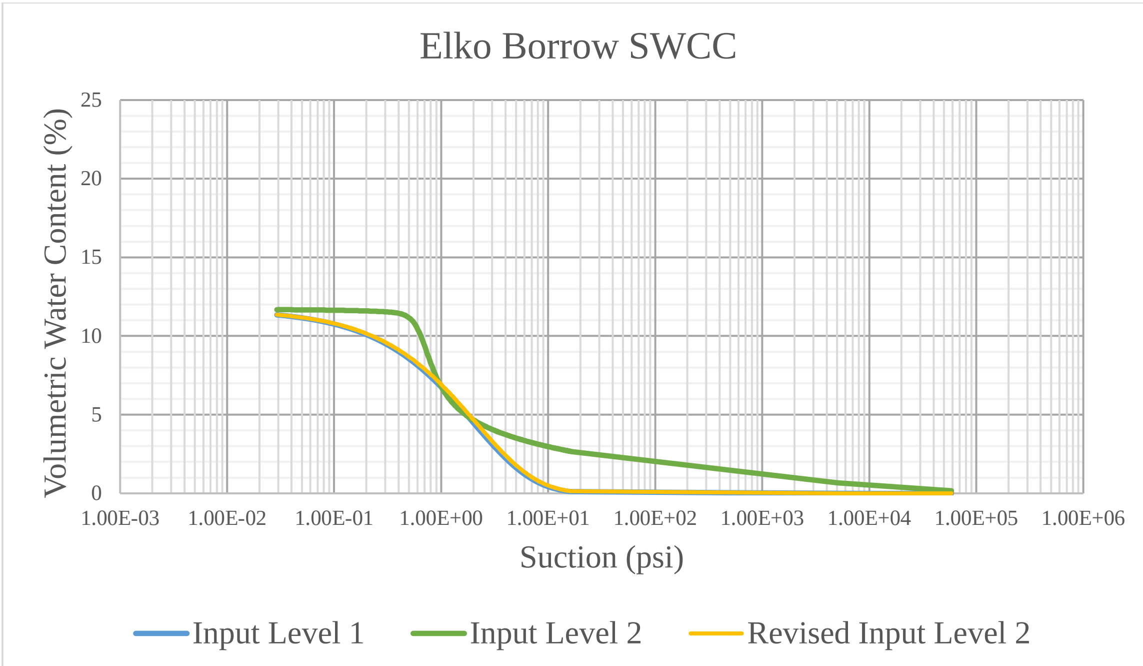

Figure 122. Elko borrow SWCC.

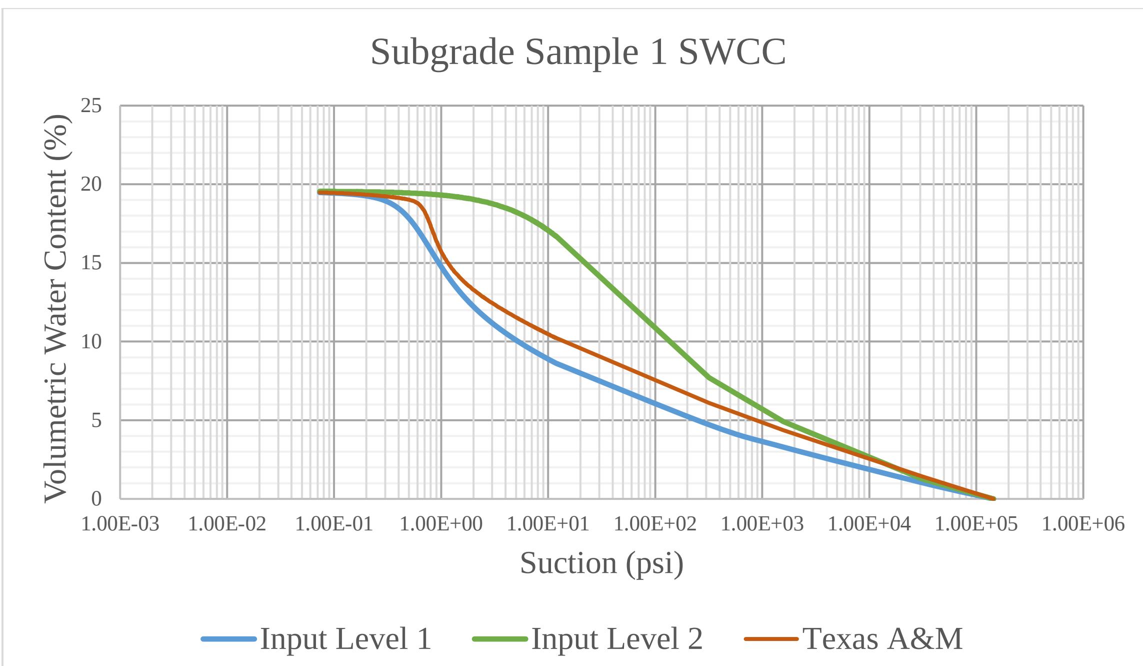

Figure 123. Sample 1 subgrade SWCC.

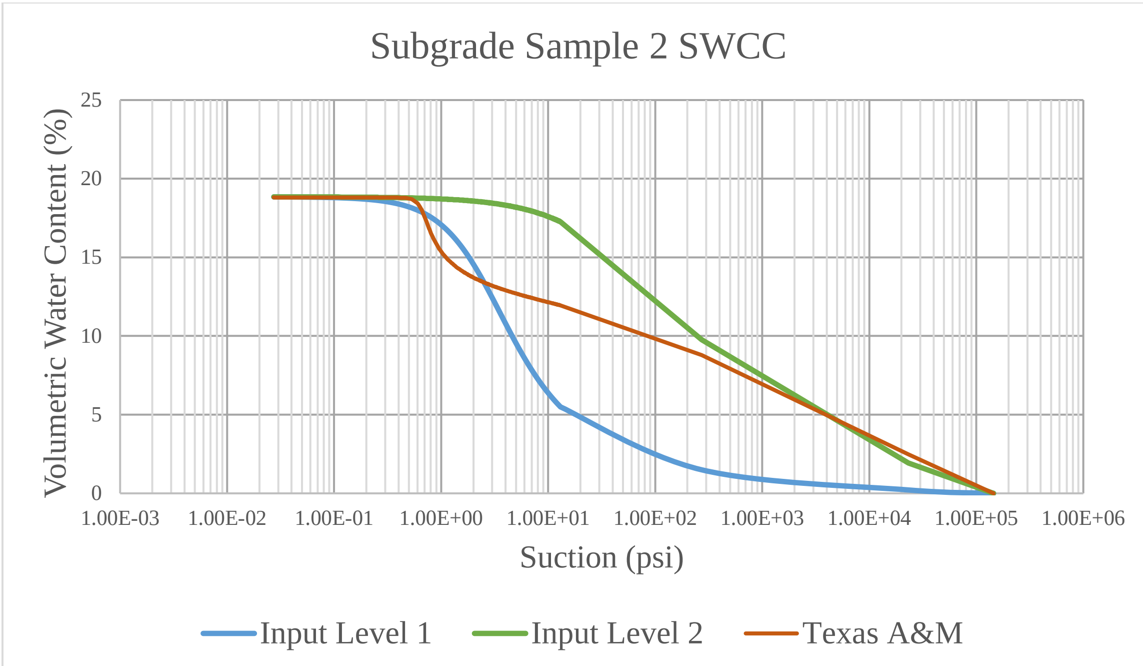

Figure 124. Sample 2 subgrade SWCC.

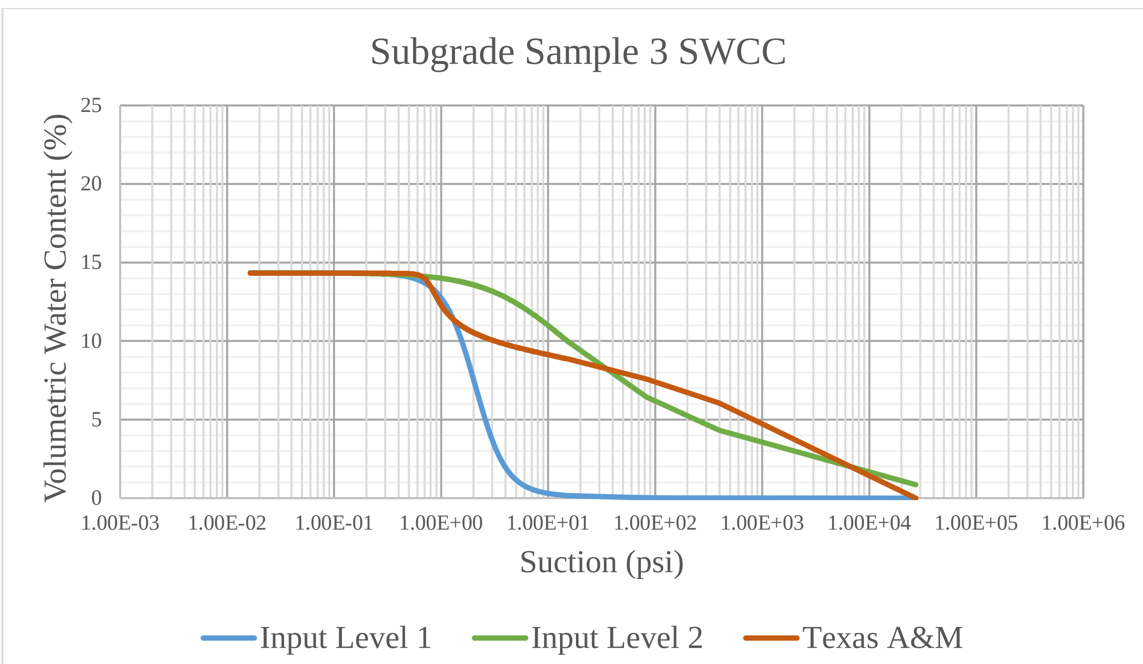

Figure 125. Sample 3 subgrade SWCC.

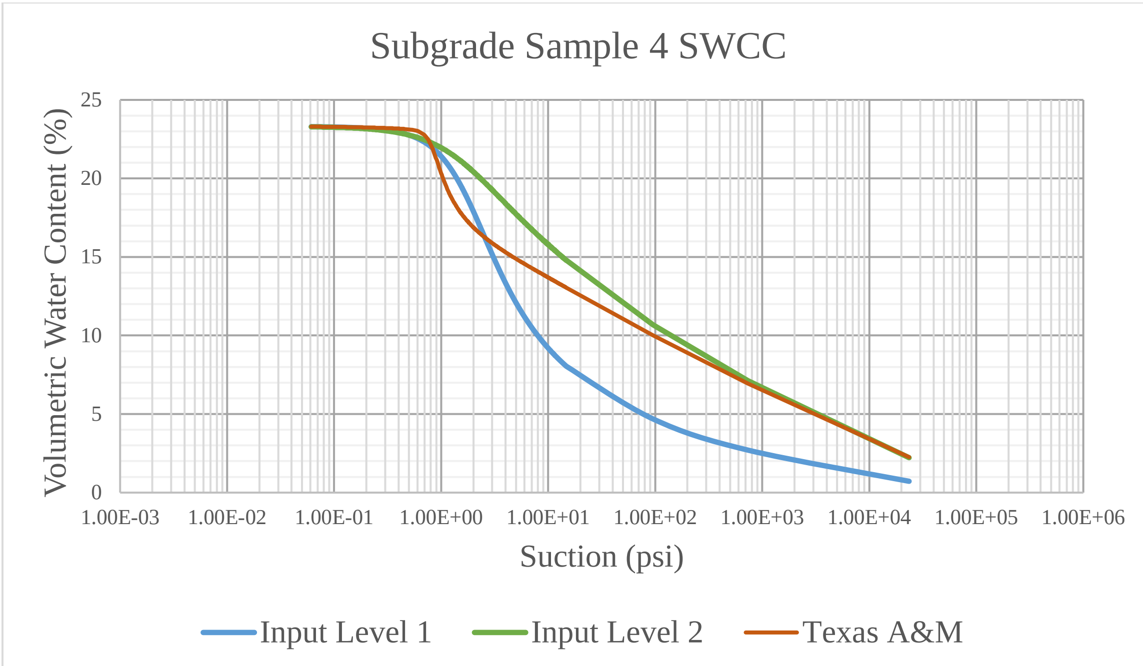

Figure 126. Sample 4 subgrade SWCC.

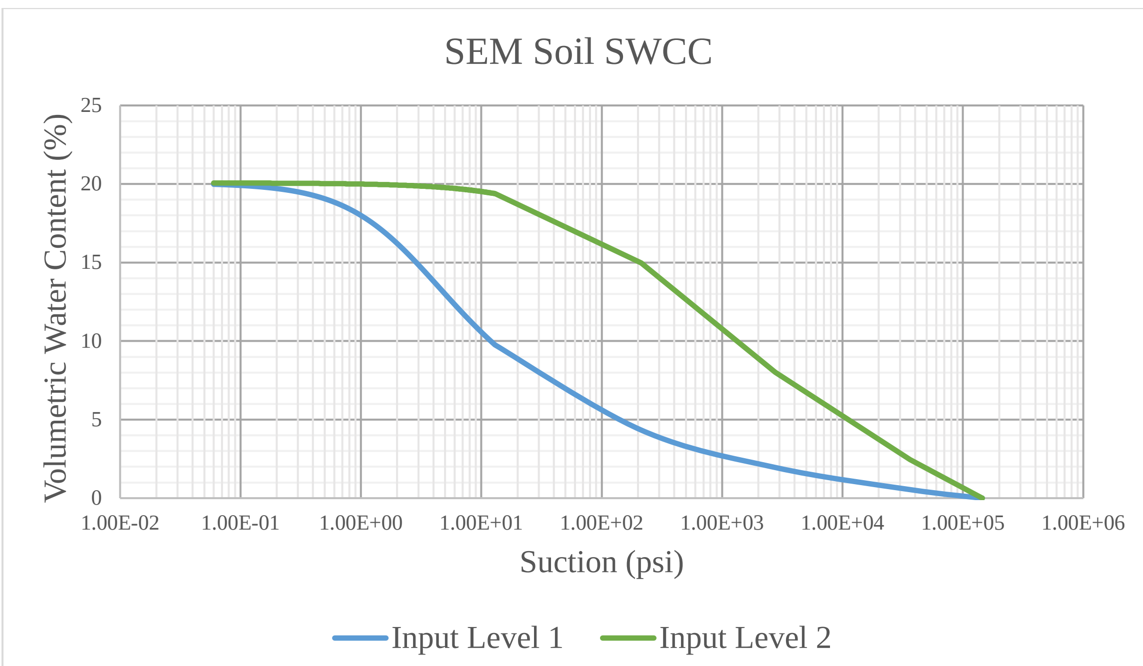

Figure 127. SEM Soil subgrade SWCC.

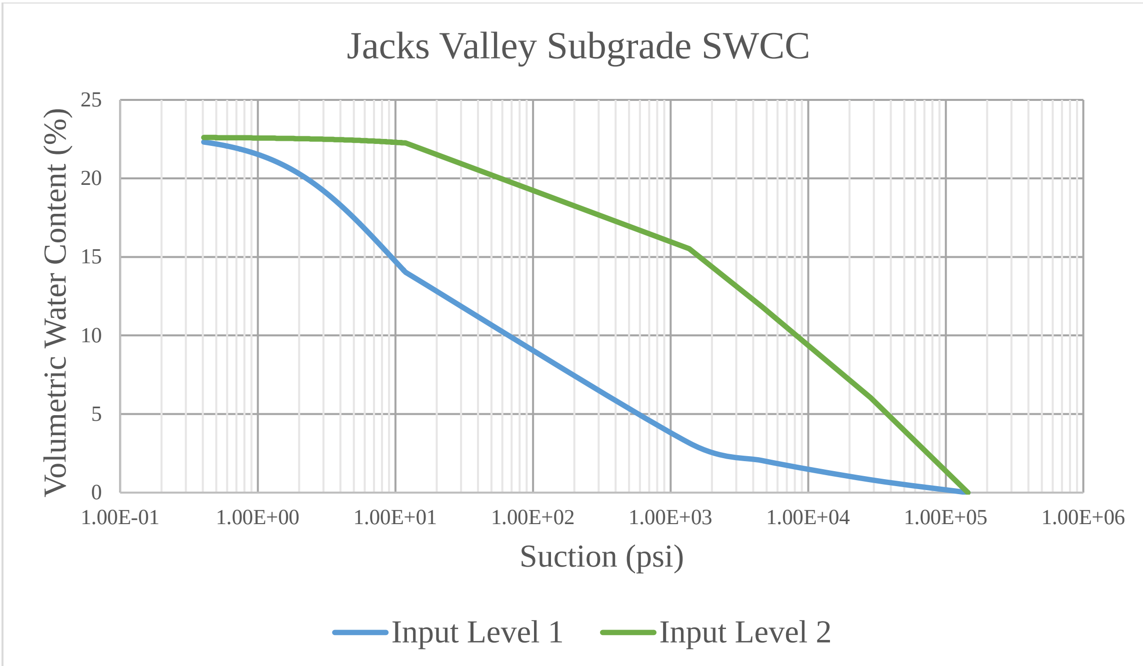

Figure 128. Jacks Valley subgrade SWCC.