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Characterization of Material Properties for Mechanistic-Empirical Pavement Design in Wyoming

By:

Department of Civil & Architectural Engineering
University of Wyoming
1000 E. University Avenue, Dept. 3295
Laramie, Wyoming 82071

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Kam Weng Ng (0000-0001-5099-5454), Harold L. Von Quintus (0000-0003-4030-6162), Khaled Ksaibati (0000-0002-9241-1792), Daniel Hellrung (0000-0003-3400-394X), and Zachary Hutson (0000-0003-3913-9184)</td><td colspan="2">8. Performing Organization Report No.
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The Wyoming Department of Transportation (WYDOT) recently transitioned from the empirical AASHTO Design for Design of Pavement Structures to the Mechanistic Empirical Pavement Design Guide (MEPDG) as their standard pavement design procedure. A comprehensive field and laboratory test program was conducted in Wyoming to characterize the properties of unbound soil materials. The field test program included falling weight deflectometer (FWD), dynamic cone penetration (DCP), standard penetration test (SPT), soil sampling and pavement distress survey. The laboratory test program included standard soil classification tests, R-value test, standard Proctor compaction test, and resilient modulus (M_r) test in accordance with a protocol by modifying the AASHTO T-307 procedure. All test data was stored and managed by an electronic WYOming MEPDG Database (WYOMEP). Using the FWD data, in-place resilient modulus (M_R) of each pavement layer was back-calculated using MODCOMP6 and EVERCALC. For MEPDD Level 2 input, correlation studies were performed to adjust back-calculated modulus to laboratory-derived modulus, calibrate constitutive models, develop relationships between resilient modulus and other soil properties, and develop M_r design tables. Furthermore, tables of unbound soil properties were established for MEPDG Level 3 input. Finally, seven pavement designs were evaluated and compared to achieve the target threshold values and reliability level. The design comparisons and resulting outcomes or predicted distresses for a range of new pavement and rehabilitation designs were presented. The outcomes of these trial examples were used to provide revisions to the 2012 WYDOT MEPDG User Guide.</td></tr>
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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)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		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)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		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

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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CHAPTER 1 - INTRODUCTION

1.1 Introduction

The proper design of pavement structures is a challenging task that requires consideration of a variety of different variables and their interactions that could affect pavement performance. To assist with this task, a research effort initiated by the National Cooperative Highway Research Program (NCHRP) has led to the development of a Mechanistic-Empirical Pavement Design Guide (MEPDG), documented in NCHRP Report 01-37A (2004). This procedure is codified as the American Association of State Highway and Transportation Officials (AASHTO) MEPDG and eliminated limitations of the outdated AASHTO 1993 Guide. The MEPDG procedure requires a large number of input variables for design whereby these variables are broadly classified as traffic, climatic, structural and material inputs.

Currently, Wyoming Department of Transportation (WYDOT) is working towards implementation of the MEPDG. Several research activities have been undertaken to locally calibrate the design guide to reflect design and construction practices in the State of Wyoming. Pavement subgrade materials were characterized and their properties were recommended, electronic data base was developed, and design recommendations were established in this research study by the University of Wyoming (UW) and Applied Research Associates (ARA), Inc. This study facilitates the full implementation of the MEPDG in Wyoming.

1.2 Background

The AASHTO design guide (AASHTO 1993) was developed based on road tests conducted for a limited number of design conditions such as traffic, climate and materials. The limitations encountered with the use of AASHTO design procedure are:

(1) Truck traffic volumes, axle configurations, tire pressures and other traffic conditions used to develop the AASHTO design guide are no longer representative of the current truck traffic on the nation's highways
(2) A more detailed characterization of traffic is necessary to accurately model traffic loading on pavements
(3) Climatic data was not included as a design variable in the AASHTO procedure
(4) Properties of materials used in different pavement layers were not adequately characterized as the AASHTO design guide was developed from a very limited set of base and subgrade materials
(5) The AASHTO method does not directly relate design inputs to different pavement distresses, i.e. pavement performance must be used as the design acceptance criteria
(6) Restrictions on design features such as calibration of design equations for local conditions

The MEPDG was developed under NCHRP Project 1-37A as an enhanced design procedure to overcome the limitations stated above. The MEPDG is a software program that predicts pavement distresses by computing the effects that traffic loads, climate, and pavement structure have on the designed roadway. Mechanistic models in the MEPDG are used to calculate pavement responses such as stresses, strains and deflections by utilizing traffic, climate, and material inputs as explanatory variables. Predicted distresses are computed from the pavement responses using numerical models called transfer functions embedded in the MEPDG over the design life of the pavement.

Local calibration of traffic inputs for Wyoming was performed by ARA, Inc., by analyzing data from several weigh-in-motion (WIM) stations located in Wyoming. The traffic characteristics, including axle load distributions, vehicle class distributions, monthly adjustment factors (MAF), and hourly truck distributions, were determined for various roadway classifications including primary and secondary highways. Wyoming specific climate inputs were established by verifying and cleaning the data obtained from the National Climatic Data Center, along with the inclusion of three additional weather stations (Dzotepe and Ksaibati, 2010).

This research project focused on characterizing the unbound base and subgrade materials of a Plant Mix Pavement (PMP) as it is the primary pavement system in Wyoming. The WYDOT-specific performance indicators for a PMP include International Roughness Index (IRI), thermal cracking, alligator cracking, and rutting in the PMP layer as well as total rutting. The accuracy of the numerical models used to calculate each performance indicator is dependent on the input variables used in the design guide software. In addition to determining appropriate coefficients for the models to reduce standard errors and bias, an accurate characterization of inputs including unbound base and subgrade materials is necessary, which was performed in this study.

Mechanistic analysis of flexible pavements in the MEPDG is performed by using the Jacob Uzan Layered Elastic Analysis (JULEA) model. JULEA is a structural, mechanistic model that incorporates fundamental engineering principles to calculate critical pavement responses that are predicted with the design being analyzed. The typical inputs to characterize the unbound base and subgrade materials for a flexible pavement are resilient modulus (M_r) or Resistance (R) value, plasticity index (PI), and gradation.

Resilient modulus is the primary property input for subgrades in the MEPDG. It is an essential parameter for computing stresses, strains, and deformations induced in the pavement structure by the applied traffic loads. Although a set of default material inputs embedded in the MEPDG software are available for application, they do not reflect local geological and soil conditions.

The material inputs were determined through field and laboratory testing programs and an extensive data analysis program. The in-situ resilient modulus value is typically determined through a back-calculation procedure using measurements obtained from a Falling Weight Deflectometer (FWD). Resilient modulus was determined in the laboratory using standard cyclic triaxial test equipment at multiple stress states and was estimated from the R-value of soils measured with a stabilometer. The general material properties (such as plasticity index and gradation) were determined from standard laboratory tests.

1.3 Research Objectives

The overall objective of this research is to characterize representative, local material properties for unbound subgrade layers to insure a comprehensive MEPDG implementation in the State. Representative material properties will be characterized for various local geological, soil, traffic, climate, and material conditions to facilitate the full MEPDG implementation and to better predict pavement performance in Wyoming. This study will concentrate on determining material characteristics for subgrade layers underneath PMP only since asphalt pavement is the most widely used in the State.

1.4 Research Plan

The research plan was developed based on the aforementioned research objectives. The research objectives were achieved by completing two main phases. Phase I focused on a literature review, a field and laboratory testing program and database development. Phase II focused on data analyses, recommendations, trial designs and examples, design comparison, design guidelines, and implementation. All tasks in both phases are briefly described.

1.4.1 PHASE I−Literature Review, Testing Program and Database Development

Task 1: Literature Review
This task focused on conducting a comprehensive literature review pertinent to the characterization of subgrade material properties for the mechanistic-empirical pavement design procedure.

Task 2: Examine Requirements for MEPDG
This task examined the design requirements relating to material inputs. The global material properties embedded in the MEPDG were examined and served as a reference of comparison with the locally calibrated values determined in Phase II.

Task 3: Identify Test Sections throughout Wyoming
Twelve test locations or 36 test sites throughout the State of Wyoming, including most possible geological, geographic, soil, climate and traffic conditions indigenous to pavement design in Wyoming, were identified for a series of field tests and materials sampling described in Task 4.

Task 4: Field Testing and Samples Collection
A series of field tests and sample collection of unbound subgrades was completed. A falling weight deflectometer (FWD) was deployed to measure the pavement deformation under an impulse load on existing pavements. Undisturbed, disturbed and moisture samples of the unbound subgrade were collected for laboratory tests in Task 5.

Task 5: Laboratory Testing
Unbound subgrade samples collected from the field were used for a series of laboratory tests. The optimum moistures and maximum dry densities of samples as well as in-situ moisture contents were determined. R-values of the unbound subgrade samples at their in-situ moisture and optimum moisture conditions were determined using a stabilometer. Classification follows the AASHTO system. Atterberg limits, gradation, and specific gravity tests were also conducted. A protocol testing procedure for the Resilient Moduli (M_r) was established and used to determine the resilient moduli (M_r) of the unbound subgrade samples at both their in-situ moisture and optimum moisture conditions using the WYDOT cyclic triaxial test equipment.

Task 6: Database Development
The test results were assimilated and compiled in an electronic database known as *WYOMEP* developed using Microsoft Office Access™. The electronic database enables the delivery of an organized storage facility shrouded beneath an appealing user-friendly interface.

1.4.2 PHASE II−Data Analyses, Recommendations and Design Guide Development

Task 1: Data Analyses
Using the project database developed in Phase I, constitutive models were calibrated to estimate resilient modulus of subgrade soils. Back-calculation of in-place M_R values from FWD data was performed, and an adjustment factor (i.e., c-factor) was determined. Recommendations for use of FWD for design were established.

Task 2: Database and Correlation of Unbound Material Properties
A catalog of locally calibrated material inputs that reflect various local geological, soil, traffic, climate, and material conditions was recommended for pavement designs in the State of Wyoming. Relationships between R-value and resilient modulus were established.

Task 3: Trial Designs and Examples
Several trial designs of flexible pavements were completed. The performance of each trial design was evaluated by comparing the predicted reliability with the target reliability criterion at a specified reliability level.

Task 4: Design Comparison
The pavement design outcomes, obtained based on both the AASHTO (1993) Guide and the locally calibrated MEPDG using the DARWin-ME^{TM}, were compared.

Task 5: Develop Design Guidelines
Integrating all the research outcomes and recommendations described, design guidelines were recommended to facilitate the full implementation of MEPDG in the State of Wyoming. Resilient modulus (M_r) values were recommended for each soil type identified in Wyoming.

1.5 Report Outline

The report consists of eight chapters, summarized as follows:

- ***Chapter 1 – Introduction*** introduces the problem and presents research scope, objectives, and research tasks.
- ***Chapter 2 – Literature Review*** reviews and analyzes the literature on pavement design using MEPDG.
- ***Chapter 3 – Field and Laboratory Test Program*** describes field and laboratory tests, resilient modulus test protocol and test results.
- ***Chapter 4 – Electronic Database*** describes the framework and contents of the electronic database (*WYOMEP*).
- ***Chapter 5 – Back-calculation of Pavement Resilient Moduli*** describes the software tools used for back-calculation, back-calculation procedures and a summary of results.
- ***Chapter 6 – Development and Recommendation of Material Properties*** describes the data analyses and correlation studies that led the development of models for subgrade material estimations and recommendations.
- ***Chapter 7 – Trial Pavement Design*** describes the design comparisons and outcomes for a range of new pavement and rehabilitated designs for flexible pavements.
- ***Chapter 8 – Summary, Conclusions and Recommendations*** presents summary and conclusions, suggests changes to current WYDOT pavement design procedures, and makes recommendations for future research.

CHAPTER 2 - LITERATURE REVIEW

2.1 Introduction

This chapter provides a detailed review and background information on MEPDG, laboratory and field test methods for resilient moduli, MEPDG database, and correlation and estimation of resilient moduli.

2.2 Overview of MEPDG

The MEPDG was developed under the NCHRP Project 01-37-A (2004) to replace the outdated AASHTO 1993 Pavement Design Guide. Pavement performance-based designs are accomplished from the MEPDG utilizing mechanistic empirical (M-E) models to predict pavement distresses. In so doing, the MEPDG requires an in-depth analysis of over hundreds to thousands of inputs for both flexible and rigid pavements based on local traffic, climate, and material conditions. However, the M-E models were initially developed based on nationwide long-term pavement performance (LTPP) data that does not necessarily represent a local condition.

The MEPDG has three levels for design inputs. Level 1 is the most accurate and uses site specific data collected at or near the project site. Level 2 requires the designer to input regional data that is representative of the local conditions. Level 3 is the least accurate, utilizing national default values as inputs. The MEPDG provides an output of various performance indicators including rutting in the Hot Mix Asphalt (HMA) and unbound layers, non-load related transverse cracking, load related alligator cracking, load-related longitudinal cracking and smoothness (IRI). The design inputs are adjusted to meet the design criteria.

The resilient modulus is a fundamental material property of the unbound layer. The resilient modulus measures the stiffness of the material and relates it to a measurable variable. Accurate resilient modulus characterization is necessary to model the performance and life span of a given pavement structure (Taylor 2008). The MEPDG allows users to input the resilient modulus one of three ways. Level 1, the most accurate and recommended level, requires determination of the resilient modulus through laboratory or field testing. Level 2 uses empirical relationships to estimate resilient modulus based on common material properties, such as California Bearing Ratio (CBR), resistance value (R- Value) and other common soil properties. Level 3, the least accurate level, uses defaulted resilient modulus values that are given in the MEPDG program.

2.3 Laboratory Test Methods for Resilient Modulus

2.3.1 Long term pavement performance (LTPP)

Long term pavement performance (LTPP) is one of earliest publicized procedures for testing resilient modulus developed by the Federal Highway Administration (FHWA 1996). To produce a dynamic loading sequence simulating the traffic load, LTPP requires a repeated axial cyclic stress of a fixed magnitude with a load cycle duration equal to one second. The LTPP testing protocol is the basis for resilient modulus testing procedures, and other standard test methods have been modified from this procedure. However, it has certain specific test prerequisites, requirements, and definitions (FHWA 1996; Henrichs 2015).

2.3.2 Harmonized resilient modulus test method

In 2004, the NCHRP attempted to harmonize resilient modulus testing under the NCHRP Project 1-28A (Dragos et al. 2004). This procedure considers four material types based upon the material gradation. It uses both 4-inch and 6-inch molds for compaction of disturbed materials while a 2.8-inch mold is applicable to undisturbed materials collected using a standard thin-wall Shelby tube. Material compaction can be achieved by impact, vibratory and kneading methods. Its test procedure requires an increased loading time of 0.2 seconds and a decreased resting time of 0.08 seconds. Also, a lower stress ratio between the confining and axial stresses was recommended.

2.3.3 AASHTO test method

In 2007, AASHTO published a standardized procedure, denoted as AASHTO T 307-99 (2007), for conducting a resilient modulus test. The AASHTO T-307 closely follows the LTPP procedure with a few changes to standardize the testing procedure. This procedure defines materials to be tested as either Material Type 1 or Type 2. Material Type 1 is classified by materials in which less than 70 percent is passing the No. 10 sieve, less than 20 percent passing the No. 200 sieve, and a plastic index of less than or equal to 10. Materials Type 2 is all materials not meeting requirements for Type 1. Disturbed specimens of Material Type 1 shall be 6 inch diameter and undisturbed material specimens of Type 2 shall be 2.8 inch or 3.4 inch diameter. Also, for both Type 1 materials and compacted materials of Type 2, a mold with a minimum diameter equal to five times the maximum particle size shall be used. The length of a specimen shall be at least twice its diameter. The test equipment shall have the capability to apply a repeated axial cyclic stress of fixed magnitude, load duration, and cycle duration as well as a confining pressure. Specimen deformation shall be measured during testing for subsequent determination of resilient modulus. Soil specimens can be prepared at either its in-situ or optimum condition specified by the testing agency. The subgrade soil sample must also be prepared to meet moisture and unit weight tolerances. The moisture content shall not vary more than ±1.0 percent for Type 1 materials and ±0.5 percent for Type 2 materials. The unit weight shall not vary more than ±3 percent of the target wet density. Vibratory compaction method is applicable to both material types while a static loading method is only applicable to Type 2 materials. Table 1 shows the test sequence for subgrade soil.

Table 1 Test sequence for subgrade soil (AASHTO T-307)

Sequence No.	Confining Pressure, S_3		Max. Axial Stress, S_{max}		Cyclic Stress, S_{cyclic}		Constant Stress, $0.1S_{max}$		No. of Load Applications
	kPa	psi	kPa	psi	kPa	psi	kPa	psi	
0	41.4	6	27.6	4	24.8	3.6	2.8	0.4	500-1000
1	41.4	6	13.8	2	12.4	1.8	1.4	0.2	100
2	41.4	6	27.6	4	24.8	3.6	2.8	0.4	100
3	41.4	6	41.4	6	37.3	5.4	4.1	0.6	100
4	41.4	6	55.2	8	49.7	7.2	5.5	0.8	100
5	41.4	6	68.9	10	62.0	9.0	6.9	1.0	100
6	27.6	4	13.8	2	12.4	1.8	1.4	0.2	100
7	27.6	4	27.6	4	24.8	3.6	2.8	0.4	100
8	27.6	4	41.4	6	37.3	5.4	4.1	0.6	100
9	27.6	4	55.2	8	49.7	7.2	5.5	0.8	100
10	27.6	4	68.9	10	62.0	9.0	6.9	1.0	100
11	13.8	2	13.8	2	12.4	1.8	1.4	0.2	100
12	13.8	2	27.6	4	24.8	3.6	2.8	0.4	100
13	13.8	2	41.4	6	37.3	5.4	4.1	0.6	100
14	13.8	2	55.2	8	49.7	7.2	5.5	0.8	100
15	13.8	2	68.9	10	62.0	9.0	6.9	1.0	100

2.4 Field Test Method and Back-Calculation of Resilient Modulus

2.4.1 Falling weight deflectometer (FWD)

FWD measures pavement deflections in response to an applied load. The applied load simulates the traffic loading due to heavy vehicles on a given section of pavement. The field testing with FWD is favored over laboratory testing due to the nondestructive procedure. The FWD can be driven directly to the site to collect the deflection data in a reasonable amount of time. Unlike coring, all the testing equipment needed is on site, and there is no "clean up" after the test. Once the FWD is complete, the equipment is simply driven to the next test section. Additionally, the FWD requires only one operator, and is much faster than coring. The FWD consists of a force generating device, a guide system, a loading plate, multiple deflection sensors, a data processing and storage system, and a load cell. The falling weight is raised to a predetermined height and released to generate an impact with a predetermined magnitude. The magnitude of the impact ranges from 6 kips to 16 kips. The deflections induced by the impact are collected by deflection sensors or geophones. The LTPP Manual by Schmalzer (2006) describes the spacing of the sensors in relation to the center of the loading plate. However, it is important to note that environment and weather conditions influence the data measurements. Because of its highly viscous properties, asphalt becomes brittle leading to smaller deflections in cold temperatures and higher deflections in warm temperatures. Hence, temperature readings should be recorded during FWD testing at various pavement depths.

2.4.2 Back-Calculation of Resilient Modulus

Back-calculation is an iterative process by which pavement layer moduli, or stiffness properties, are estimated from FWD deflection data (Alavi et al. 2008). Several back-calculation methods have been developed for the flexible pavement. Dong et al. (2001) used a 3D finite element method to determine the time-domain back-calculation of pavement properties. Goktepe et al. (2006) considered the static effects of the FWD deflection on back-calculation while Seo et al. (2009) considered its dynamic effects. Gopalakrishnan and Papadopoulos (2011) applied a novel machine learning concept, and Saltan et al. (2011) used a data mining method in the pavement back-calculation. Although the FWD approach is considered a preferred nondestructive testing method that offers many advantages over laboratory M_r testing, especially with its lower cost and higher testing efficiency, its key limitation lies in the back-calculation process. The back-calculation is a user-dependent procedure that requires adequate knowledge of the pavement structure and material properties. It produces a non-unique solution for each test because of the indeterminate nature of the analysis (i.e., number of unknown variables is larger than the number of available solving equations). The resilient modulus (M_R) is back-calculated by matching FWD measured pavement deflections with estimated deflections for each sensor location. During the back-calculation process, the moduli of pavement layers are continuously adjusted until the theoretical or estimated deflection basin matches the measured deflection basin within a given tolerable root mean square error (RMSE) expressed in a percentage. The back-calculated M_R value will be determined during the iteration process based on the best match of deflections with the smallest RMSE. Generally, an acceptable range of the RMSE is between one and two percent (WSDOT 2005). Based on this recommendation, one will strive to achieve the lowest possible RMSE during each back-calculation. However, the back-calculation having the lowest RMSE may not necessarily generate realistic resilient moduli. In fact, Seeds et al. (2000) recommended that any suggested thresholds for RMSE should be used cautiously, and engineering judgment should be used to determine if the back-calculated resilient moduli are reasonable. Mehta and

Roque (2003) also acknowledged that a good fit between the measured and estimated deflections presented in terms of a relatively low RMSE may not necessary yield a reasonable modulus value. There are many back-calculation programs available in the current market, and each has its advantages and disadvantages. The EVERCALC back-calculation program developed by the University of Washington (WSDOT 2005) and MODTAG back-calculation software developed by Cornell University (Irwin 1994) were considered in this study. Their detailed descriptions can be found in the thesis by Hellrung (2015).

A study completed by Dawson et al. (2009) for the Michigan DOT concluded that a relatively good agreement between back-calculated M_R and laboratory-determined M_r values was obtained when the moisture content and boundary condition in terms of confining and axial stresses of the laboratory M_r test resembled the in-situ roadbed condition where the FWD test was performed. Ji et al. (2014) observed a high scatter between their M_R and M_r values due to the variations in moisture content and boundary condition of the FWD and laboratory testing. Dawson et al. (2009) also acknowledged that the shift factor (a ratio of M_R to M_r) depended on the back-calculation software applied in the study but independent of the pavement type. A study by Nazzal and Mohammad (2010) for the Louisiana Department of Transportation and Development (LDOTD) also concluded that the M_R values were significantly affected by the back-calculation method. Mates and Soares (2014) found that the back-calculation approach resulted in higher and more realistic modulus values of granular subgrade soils than that estimated using a calibrated nonlinear constitutive model from a repeated triaxial load testing.

2.5 MEPDG Database

Minimal findings were obtained on DOT's data management for the MEPDG. Although Oklahoma DOT (ODOT) completed thorough tasks for Level 1 MEPDG inputs, little was discussed on the structure of database developed by ODOT. In contrast, New Mexico DOT (NMDOT) thoroughly described the MEPDG database that gathered data from multiple state level databases in order extract required datasets. This setup required the use of an Oracle database, which requires significant experience to setup and maintain. MEPDG databases were developed by Iowa DOT and Tennessee DOT, their structures and uses were briefly described.

2.6 Correlation and Estimation of Resilient Moduli

2.6.1 Correlation between field and laboratory resilient moduli

An adjustment factor (i.e., C-factor) is normally applied to back-calculated M_R value to determine the equivalent laboratory-determined M_r value (i.e., $M_r = CM_R$). Based on the AASHTO Road Test conducted in the late 1950s, AASHTO (1993) suggested using a C-factor of no more than 0.33. Having no local pavement data available during the initial development of the MEPDG, the C-factor of 0.33 was adopted as the default value. Rahim and George (2003) strongly suggested the need to reevaluate this default value. To improve the pavement design efficiency and reflect local practices, several state DOTs initiated independent research to develop their respective locally-calibrated MEPDGs. Outcomes of this effort led to the development of locally-calibrated C-factors summarized in Table 2 (Ng et al. 2016).

2.6.2 Correlation for resilient moduli based on R-value

A subgrade material is typically characterized using the R-value. Hence, R-value is often used in correlation studies to estimate resilient modulus. The correlation equations developed by transportation agencies are summarized in Table 3. Detailed descriptions of the development of these equations can be found in the thesis by Hutson (2016) and the corresponding references given in Table 3.

Table 2 Summary of C-factors

Agency	C-Factor
AASHTO	0.33
Colorado DOT	0.52
Idaho DOT	0.35
Missouri DOT	0.35
Montana DOT	0.50
Utah DOT	0.55 for fine-grained soil
	0.67 for coarse-grained soil

Table 3 Summary of correlation equations for M_r based on R-value

Agency	Equation for M_r (ksi)	Remark
Asphalt Institute (1982)	$M_r = 1.155 + 0.555R$	Overestimate M_r for R>60
Colorado DOT (Yeh and Su 1989)	$M_r = 3.5 + 0.125R$	For all soils
Idaho Transportation Department (Yeh and Su 1989; Buu 1980)	$M_r = 1.455 + 0.057R$ $M_r = 1.6 + 0.038R$ $M_r = 1.004R^{0.6412}$	For fine-grained soils For coarse-grained soils For all subgrade soils
Washington DOT (Mokwa et al. 2009)	$M_r = 0.72(e^{0.0521R} - 1)$	Based on WSDOT R-value test method

2.6.3 Correlation for resilient moduli based on other soil properties

Correlation studies were conducted to estimate M_r values based on some common soil properties. A correlative model was derived by Lofti et al. (1988) in Equation (1) to relate the M_r-value to California bearing ratio (CBR) values and deviator stress (σ_d). The correlation yielded a R^2 value of 0.93 for high plasticity silt.

$$\log(M_r)(\text{ksi}) = 1.0016 + 0.043(\text{CBR}) - 1.9557\left(\frac{\log(\sigma_d)}{\text{CBR}}\right) - 0.1705\log(\sigma_d) \tag{1}$$

Using the M_r data attained from the Highway Research Information Service (HRIS) database, empirical Equations (2) and (3) were developed for coarse and fine-grained soils, respectively (George 2004).

$$\log(M_r)\,(\text{ksi}) = 0.523 - 0.025(\omega) + 0.544(\log\sigma_b) + 0.173(\text{SM}) + 0.197(\text{GR}) \tag{2}$$

$$M_r(\text{ksi}) = 37.431 - 0.4566(\text{PI}) - 0.618(\omega) - 0.1424(P_{200}) + 0.1791(\sigma_3) - 0.3248(\sigma_d) + 36.722(\text{CH}) + 17.097(\text{MH}) \tag{3}$$

where,

ω = moisture content (percent),

σ_b = bulk stress (psi),

SM = 1 for soils classified as silty sand (SM) or 0 otherwise,

GR = 1 for soils classified as silty gravel (GM), well-graded gravel (GW), clayey gravel (GC), or poorly-graded gravel (GP), or 0 otherwise,

PI = plasticity index (percent),

P_{200} = percentage of fines passing the #200-sieve,

σ_3 = confining pressure (psi),

σ_d = deviator stress (psi),

CH = 1 for soils classified as high-plasticity clay (CH) or 0 otherwise, and

MH = 1 for soils classified as high-plasticity silt (MH) or 0 otherwise.

Using 97 undisturbed samples and considering the deviator stress at 6 psi and confining pressure at 2 psi, correlations to estimate M_r values were performed by Jones and Witczak (1977) on California A-7-6 soils that led to Equation (4) in terms of moisture content (w in percent) and degree of saturation (S in percent).

$$\log(\mathrm{M_r})(\mathrm{ksi}) = -0.111\mathrm{w_c} + 0.0217(\mathrm{S}) + 1.179 \tag{4}$$

A correlation for the M_r value was developed by Thompson and LaGrow (1988) for Illinois DOT considering percent clay and its plasticity index (PI) that led to the development of Equation (5).

$$\mathrm{M_r}(\mathrm{ksi}) = 4.46 + 0.098(\text{ percent Clay}) + 0.119(\mathrm{PI}) \tag{5}$$

Several empirical equations had been developed by Farrar et al. (1991) in Wyoming to estimate M_r values for soils. Equation (6) yielded a relatively higher coefficient of determination (R^2) value of 0.663.

$$\mathrm{M_r}(\mathrm{ksi}) = 30{,}280 - 359(S) - 325(\sigma_\mathrm{d}, \mathrm{psi}) + 237(\sigma_3, \mathrm{psi}) + 86(\mathrm{PI}) + 107(\mathrm{P_{200}}) \tag{6}$$

2.6.4 Constitutive models for resilient moduli

A generalized constitutive model of M_r recommended by ARA (2004) in the NCHRP Report 01-37A is given by

$$\mathrm{M_r} = \mathrm{k_1}\sigma_\mathrm{a}\left(\frac{\sigma_\mathrm{b}}{\sigma_\mathrm{a}}\right)^{\mathrm{k_2}}\left(\frac{\tau_{\mathrm{oct}}}{\sigma_\mathrm{a}} + 1\right)^{\mathrm{k_3}} \tag{7}$$

where,

σ_a = atmospheric pressure;

σ_b = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$ (sum of major, intermediate and minor principal stresses);

τ_{oct} = Octahedral shear stress = $\frac{\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}}{3}$; and

k_1, k_2 and k_3 = regression coefficients in terms of significant soil properties.

An alternative generalized constitutive model in terms of confining stress (σ_c), deviator stress (σ_d) and regression coefficients (k_4, k_5 and k_6) developed by Kim and Siddiki (2006) under a Joint Transportation Research Program (JTRP) is given by

$$\mathrm{M_r} = \mathrm{k_4}\sigma_\mathrm{a}\left(\frac{\sigma_\mathrm{c}}{\sigma_\mathrm{a}}\right)^{\mathrm{k_5}}\left(\frac{\sigma_\mathrm{d}}{\sigma_\mathrm{a}}\right)^{\mathrm{k_6}} \tag{8}$$

CHAPTER 3 – FIELD AND LABORATORY TEST PROGRAM

3.1 Introduction

This chapter briefly describes the field and laboratory test program, the experimental results and distress survey. Topics covered in this chapter are selection of test locations and sites, the field test procedure and methods, the labeling and storage system, field results, standard laboratory tests, modified resilient modulus tests, laboratory test results and distress survey. The field and laboratory test program was generally described by Hutson (2016). The detailed description of the pavement distress survey, FWD, and pavement temperature measurements can be referred to the thesis by Hellrung (2016). The laboratory resilient modulus test program was explicitly described by Henrichs (2016).

3.2 Test Location and Site

In order to determine the mechanistic properties of each soil type utilized in Wyoming road construction, soil samples were collected from the road subgrade throughout the entire state. Each test location was chosen based on the soil type reported for each road project, so that the broadest range of AASHTO soil types could be collected for laboratory testing. Figure 1 shows each test location on a map of Wyoming. Field tests were conducted at three distinct test sites denoted A, B, and C at each test location as shown in Table 4. To minimize traveling expenses and to optimize field testing efficiency, weekly field testing was conducted at two to three test locations in proximity to each other.

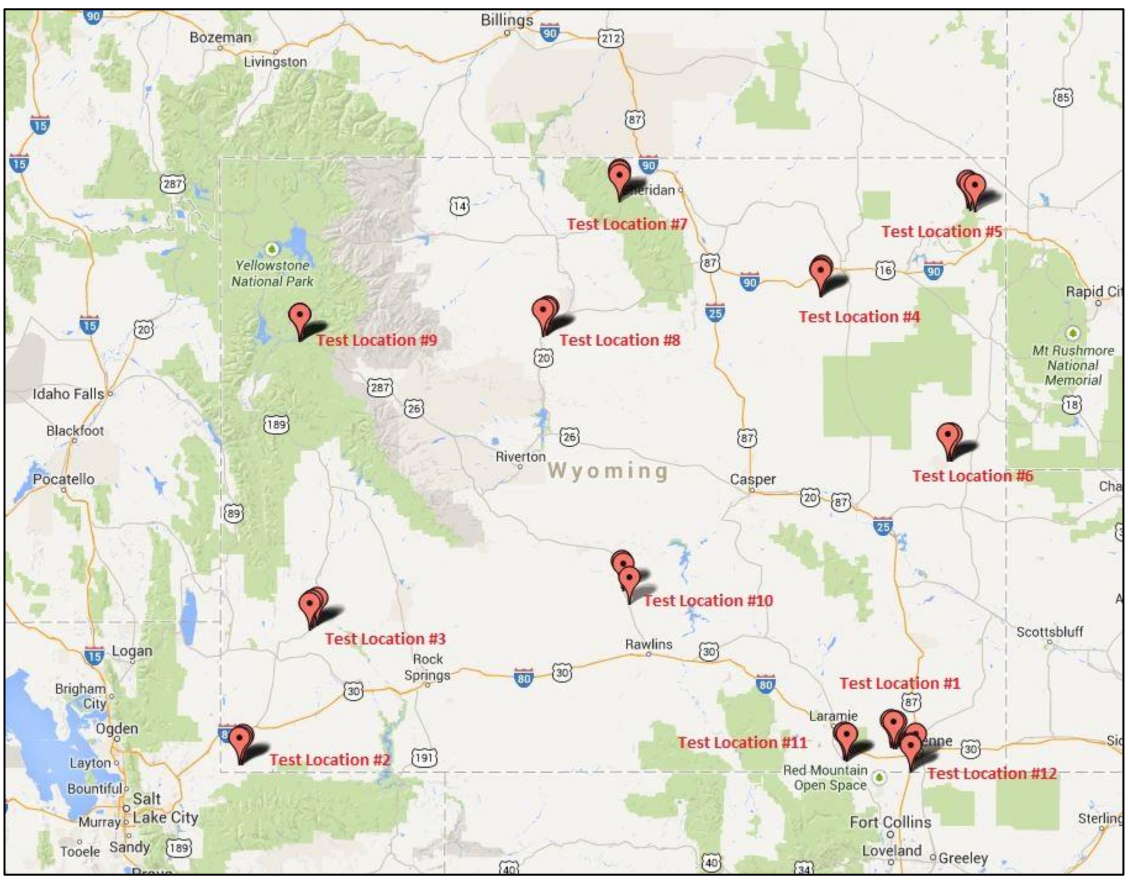

Figure 1 Twelve test locations on the map of Wyoming.

Table 4 Summary of twelve locations and field test results.

Test Loc.	Project Name	Proj. No.	Test Dates	Site	Asphalt/ Base Thk. (in)	Subgrade			
						AASHTO	UCSC	DCP Index (in/blow)	N
1 (W1)	Happy Jack Road (WYO 210)	0107	5/28/13 to 5/30/13	A(4)	12/9.5	A-6(1)	SC	NA	0
				B	12/9.5	A-4(3)	CL	NA	50
				C	12/9.5	A-2-4(0)	SM	NA	10
2 (W2)	Evanston South (WYO 150)	2100	6/4/13	A(3)	N/A	A-1-B(0)	GW-GM	NA	N/A
				B(3)	N/A	N/A	SM	NA	N/A
				C(3)	N/A	A-1-B(0)	SC	NA	N/A
3 (W2)	Kemmerer – La Barge (WYO 189)	0P11	6/5/13	A	13/9.5	A-6(3)	SC	NA	11
				B	6.5/7	A-7-6(6)	CL	NA	20
				C	6/12	A-7-6(9)	CL	NA	6
4 (W3)	Gillette – Pine Tree (WYO 50)	0300	6/11/13	A	4/12	A-6(14)	SC	0.775	16
				B	6.5/12	A-4(1)	CL	0.445	17
				C	5/13	A-6(10)	SM	0.867	14
5 (W3)	Aladdin – Hulett (WYO 24)	0601	6/12/13	A	6/16	A-2-4(0)	SW-SM	0.275	26
				B	6/18	A-2-4(0)	CL	0.184	57
				C	6/12	A-6(13)	CH	1.075	27
6 (W3)	Lance Creek (WYO 270)	1401	6/13/13	A	4/10	A-7-6(29)	CH	0.778	11
				B	5/13	A-7-6(33)	CL	0.401	28
				C	5/11	A-7-6(33)	SM	0.259	50
7 (W4)	Burgess Junction (US 14)	0N37	6/18/13	A	6/12	A-1-B(0)	SM	0.6	10
				B	6/12	A-1-B(0)	SW-SM	0.24	45
				C	5/9	A-1-B(0)	SM	0.264	50
8 (W4)	Thermopolis – Worland (US 20)	0N34	6/19/13	A(5)	11/13	A-2-4(0)	SM	0.64	6
				B(5)	10/12	A-4(0)	SM	0.43	13
				C	9/10	A-4(0)	CL	0.536	10
9 (W5)	Moran Junction (US 26)	0N30	6/25/13	A	4/6	A-6(1)	GW-GM	0.756	11
				B	4/6	A-1-A(0)	SM	0.798	5
				C	4/6	A-4(0)	SW-SM	0.465	18
10 (W5)	Lamont – Muddy Gap (WYO 789)	0N21	6/26/13	A(2)	8/9	A-1-B(0)	CL	0.266	45
				B(2)	8/7	A-6(6)	CL	0.274	11
				C	7/12	A-6(9)	SM	0.867	12
11 (W6)	Laramie – CO. St. Line (US 287)	0N23	7/11/13	A	5/10	A-1-B(0)	SW-SM	0.176	100
				B	5/10	A-1-B(0)	SM	0.36	100
				C	5/10	A-2-4(0)	SM	0.158	33
12 (W6)	Cheyenne – CO. St. Line (I-25)	I025	7/12/13	A(1)	N/A	A-1-B(0)	CL	0.213	N/A
				B(1)	N/A	A-6(4)	SC	1.243	N/A
				C(1)	N/A	N/A	CL	NA	N/A

W−Week; Loc.−Location; Proj.−Project; N/A−Not available; Thk.−Thickness; Temp−Temperature; N−SPT N-value; CO. St.−Colorado State; (1)−Excluded from back-calculation because of rigid pavement; (2)−Excluded from back-calculation because of cement treated base; (3)−Excluded from back-calculation because of unusual stiff granular subgrade encountered during field test; (4)−Excluded from back-calculation because of super-elevation; and (5)−Excluded from back-calculation because of anomalous deflection basins.

3.3 Field Test Program and Results

The first test site – Happy Jack Road – was used to formulate a workable and repeatable field test procedure that the field crews could efficiently execute at the test locations summarized in Table 4. Although the number of soil samples varies among sites, a standardized field test procedure was developed to better manage the testing program. Since this operation was conducted in conjunction with WYDOT, many WYDOT protocols, such as traffic control safety and WYDOT's need for SPT sampling, were implemented as part of the field procedure. Figure 2 is used as a visual representation to demonstrate the layout of each test site at the mile marker.

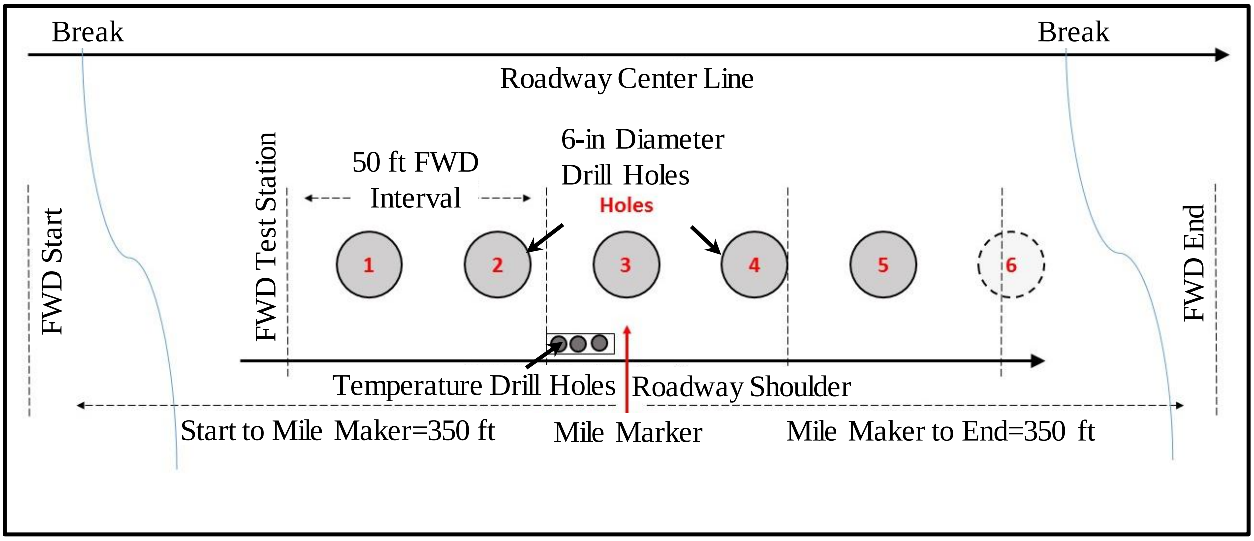

Figure 2 Illustration of a typical test site layout at the mile marker.

The standardized field test procedure for a single test site is briefly described as follows:

1) Falling Weight Deflectometer (FWD) tests were conducted at each test site in accordance with the LTPP procedure (Schmalzer 2006). The FWD test was performed using a KUAB FWD (Figure 3a) with an eight-sensor setup to record deflection measurements for four target loads of 6, 9, 12, and 16 kips. FWD measurements were taken at 50 ft intervals in the right wheel path for a total 15 stations starting 350 ft before the established F.E. mile marker of each test site and finish 350 ft after the F.E. mile marker as illustrated in Figure 2.
2) Temperature readings of the asphalt layer were taken periodically over the course of FWD operation (Figure 3b). Three to four, 1-inch diameter holes was drilled on the shoulder of the pavement at varying depths. The holes were filled with mineral oil and covered with duct tape, and thermometers were inserted into all holes to measure the temperature fluctuation at different depths as illustrated in Figure 3b. The average mid-depth temperature of the asphalt pavement layer is summarized in Table 4.
3) A drill rig and other necessary equipment were setup to begin drilling hole 1, which was approximately 40 ft from the F.E. mile marker (Figure 3c). These holes were drilled in a straight line along the same right wheel path that the FWD readings were taken.
4) The first hole was drilled with a continuous bore auger (Figure 3d) to provide an accurate representation of the road profile. Hole 2 was drilled with a solid bit down to the top of the subgrade soil. Next, a Dynamic Cone Penetrometer (DCP) test (Figure 3f) was conducted first followed by a Standard Penetration Test (SPT) (Figure 3g). Blow

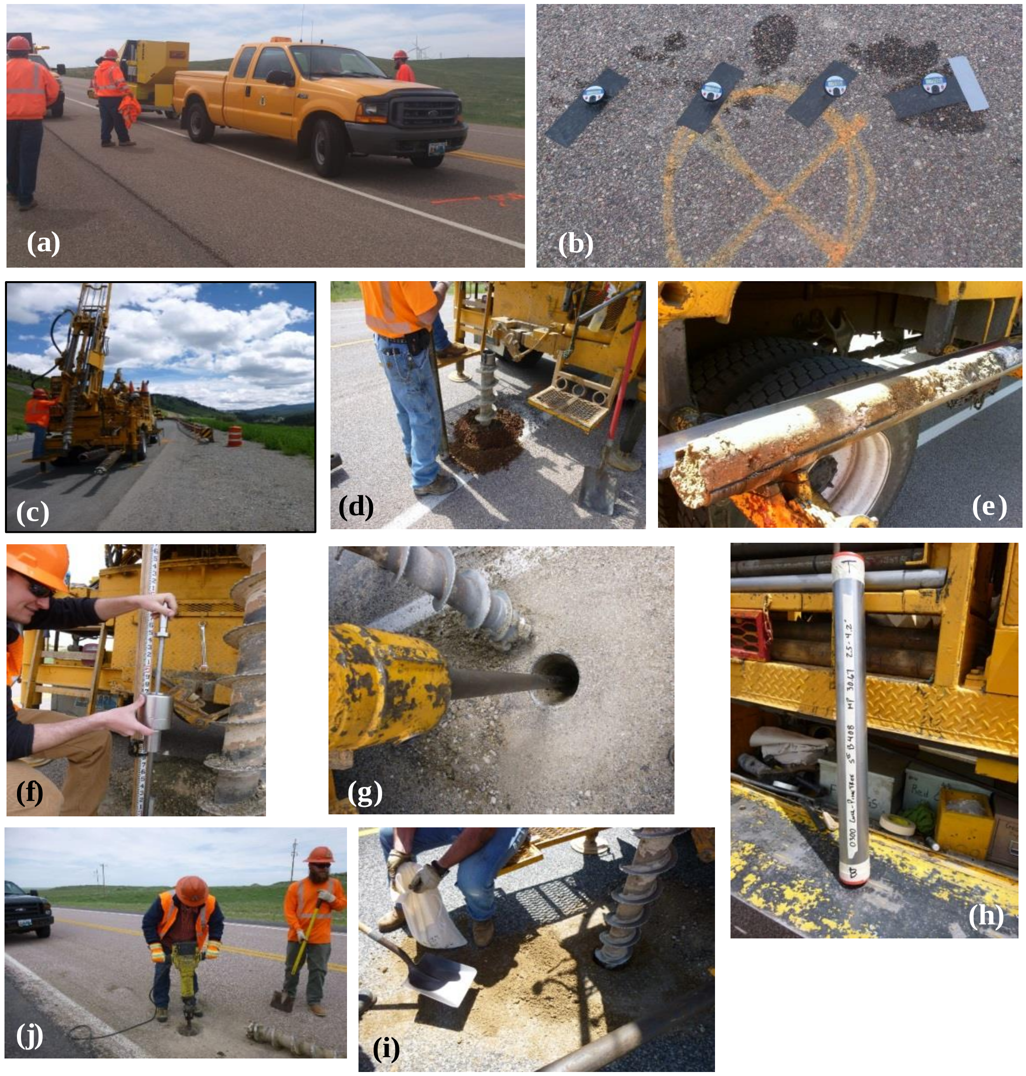

Figure 3 Collection of photos depicting the standardized field testing procedure – (a) FWD, (b) pavement temperature measurements, (c) drill rig setting up on test site, (d) drilling holes to obtain subgrade samples, (e) intact continuous bore auger sample, (f) DCP test, (g) SPT, (h) completed Shelby tube extraction and sealed, (i) collection of disturbed samples, and (j) reclaiming drilled holes at completion of testing for the current site.

counts in terms of SPT N-values were used to determine the likelihood of being able to successfully collect undisturbed subgrade samples using Shelby tubes (Figure 3h). Table 4 summarizes the DCP and SPT test results of subgrade soils. Disturbed samples were collected using the split-spoon sampler as shown in Figure 3e.

5) Assuming that undisturbed subgrade samples through Shelby tubes were possible at the test site, hole nos. 3 to 5 were drilled to collect three Shelby tubes. Concurrently, six canvas bags of disturbed soil samples were collected (Figure 3i) for subsequent laboratory tests.
6) In order to effectively organize the collected materials, a labeling and storage system was implemented. Each of the collected samples had a label identifying the test location (total of 12) and site (total of 3 for each location).

7) Concurrently, a road distress analysis was completed according to Shahin's procedure (2005) to effectively document the physical condition of the roadway. Each test site was divided into fourteen sections with each section 50 ft in length. To ensure consistency, only the lane where the FWD was completed was surveyed for distresses. Photographs were taken at each test section. All distresses were measured and the severity of each distress was recorded and sketched on a distress survey map.
8) Next, WYDOT field crew filled the bore holes with the remaining cuttings and bagged pavement and patched the holes with a bagged asphalt pavement mix (Figure 3j).
9) This testing procedure was repeated at the next test site.

3.4 Laboratory Test Program

3.4.1 Standard laboratory testing

Laboratory testing of subgrade specimens obtained from the field test program was conducted. The testing program for subgrade soils included a comprehensive soil characterization tests and the corresponding WYDOT standard test protocols (WYDOT 2010) to determine the following:

(1) Gradation – WYDOT 814.0 (modified AASHTO T27 and AASHTO T11);
(2) Liquid limit test – AASHTO T89;
(3) Plastic limit and plasticity index test – WYDOT 813.0 (modified AASHTO T90);
(4) Optimum moisture content (AASHTO T180) and maximum dry density (AASHTO T99);
(5) R-value – WYDOT 833.0 (modified AASHTO T190);
(6) Batch in-situ unit weight and optimum moisture content of soil; and
(7) Resilient modulus at in-situ and optimum moisture content – modified AASHTO T307 described in Section 3.4.2.

3.4.2 Modified AASHTO T-307

Resilient modulus test protocol was developed for WYDOT by modifying the AASHTO T-307 procedure after considering research outcomes by other state DOTs and incorporating local test practice and available triaxial test equipment. The modified AASHTO T-307 can be found in the thesis by Henrichs (2015). The specific modifications relating to specimen preparation, compaction, and reporting are briefly described as follows:

(1) *Specimen Preparation*: Subgrade soils should be prepared at target moisture contents within 2 percent below the ω_{opt} to prevent potential deformations greater than 5 percent during M_r testing. Test specimens must be compacted to a minimum dry unit weight greater than 90 percent of its $\gamma_{d\text{-}max}$. The soil specimen was accepted for subsequent compaction only if the measured moisture content varied not more than ±1 percent for Type I materials or ±0.5 percent for Type II materials from the target moisture content.
(2) *Compaction*: A single 4-inch diameter mold was recommended for soils with a maximum particle size of 0.8 inches. This requirement also aligns with current WYDOT practice whereby any soil particles greater than 0.8 inches in size will be removed from the test specimen. To reflect current WYDOT practice, vibratory compaction was used to prepare all soil specimens for M_r testing.
(3) *Reporting*: A single user-friendly report worksheet applicable to all soil specimens was developed to make specimen preparation and result reporting easier.

The proposed resilient modulus test protocol was verified through the Round Robin Test Program reported by Von Quintus et al. (2014). The M_r results determined from this proposed modified AASHTO T-307 agreed well with those produced by other agencies. This comparison confirms the dependability of M_r measurements. The detailed description of this comparison can be found in Henrichs (2015).

3.5 Test Results

The standard properties of subgrade soils are summarized in Table 5. The resilient modulus determined at the optimum moisture and in-situ conditions using the modified AASHTO T-307 at each test sequence are summarized in Table 6 and Table 7, respectively. Results of the pavement distress survey are summarized in Table 8.

Table 5 Summary of laboratory test results.

Project Site	AASHTO Soil Classification	R-Values	Optimum Moisture Content (percent)	In-situ Moisture Content (percent)	Maxium Dry Unit Weight (pcf)	In-situ Unit Weight (pcf)	Liquid Limit (LL)	Plastic Limit (PL)	Plastic Index (PI)
0107-A	A-6(1)	14	11.2	11.8	121.1	133.5	36	21	15
0107-B	A-4(3)	47	23.2	35.2	93.5	102.4	33	18	15
0107-C	A-2-4(0)	19	21.1	27.5	100.3	106.6	35	27	8
2100-A	A-1-B(0)	73	6.1	N/A	132.9	N/A	N/A	N/A	N/A
2100-B	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2100-C	A-1-B(0)	55	7.5	6.7	129.5	N/A	20	N/A	N/A
0P11-A	A-6(3)	10	14.7	17.4	113.3	128.9	36	17	19
0P11-B	A-7-6(6)	12	17	16.7	104.9	127.2	48	20	28
0P11-C	A-7-6(9)	15	17	19.8	105.9	123.9	43	21	22
0300-A	A-6(14)	18	16.4	18.6	109.4	127.1	40	19	21
0300-B	A-4(1)	43	12.8	12.5	114.9	130.4	26	18	8
0300-C	A-6(10)	10	15.3	18.4	112.1	126.9	36	19	17
0601-A	A-2-4(0)	67	8.3	10.8	117.2	139.3	N/A	N/A	N/A
0601-B	A-2-4(0)	61	6.6	1.7	100.7	N/A	N/A	N/A	N/A
0601-C	A-6(13)	18	15.6	11.1	108.7	137.7	34	17	17
1401-A	A-7-6(29)	13	18.5	20.2	99.4	124.5	56	16	40
1401-B	A-7-6(33)	11	23.4	19.3	93.8	107.2	57	18	39
1401-C	A-7-6(33)	13	28.4	25.3	90.4	NA	47	21	26
0N37-A	A-1-B(0)	76	8.2	3.9	126.5	130.5	N/A	N/A	N/A
0N37-B	A-1-B(0)	72	6.1	3.1	127.5	N/A	N/A	N/A	N/A
0N37-C	A-1-B(0)	75	6.3	2.6	129.5	132.6	22	21	1
0N34-A	A-2-4(0)	74	12.2	16.1	116.8	119.6	N/A	N/A	N/A
0N34-B	A-4(0)	47	10.9	9.1	120.1	129.2	19	16	3
0N34-C	A-4(0)	26	11.7	9.8	120	125.0	17	15	2
0N30-A	A-6(1)	14	14.7	13.0	113.8	133.3	27	19	8
0N30-B	A-1-A(0)	65	6.4	N/A	129.1	N/A	21	20	1
0N30-C	A-4(0)	35	11.8	N/A	119.7	N/A	21	19	2
0N21-A	A-1-B(0)	73	7.8	N/A	120.5	N/A	N/A	N/A	N/A
0N21-B	A-6(6)	12	14.9	12.9	111.2	133.0	32	18	14
0N21-C	A-6(9)	12	13.5	N/A	116.8	N/A	34	15	19
0N23-A	A-1-B(0)	79	6.3	N/A	125.6	N/A	N/A	N/A	N/A
0N23-B	A-1-B(0)	75	5.2	N/A	126.6	N/A	N/A	N/A	N/A
0N23-C	A-2-4(0)	59	8.5	14.6	123.1	119.8	20	16	4
I025-A	A-1-B(0)	86	6.6	N/A	129.2	N/A	20	N/A	N/A
I025-B	A-6(4)	22	21.1	N/A	106	N/A	36	18	18
I025-C	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 6 Summary of laboratory resilient modulus values at optimum moisture conditions.

Project/Site	Lab No.	Measured Resilient Modulus for Each Sequence (psi)														
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0N23-B	B1102	18412	19415	20038	20256	21112	13950	14542	15737	16606	17719	10198	11124	12321	13586	14591
2100-A	A202	22346	22884	23488	23671	24124	17317	17368	18095	18945	20295	12026	12161	13234	14401	15646
0N37-B	B706	27633	27573	27101	26883	27080	22673	21794	21958	22417	23154	16727	16431	16938	17689	18816
0N37-C	C704	20917	22155	21827	21771	22305	18796	18430	18212	18270	18945	14517	13944	14186	14604	15407
0N23-A	A1101	19864	22028	22622	23207	23469	14969	16095	17149	18446	19568	10685	11750	12991	14347	15753
0N30-B	B901	28779	27116	26085	25626	25216	24081	22369	21898	21659	21388	17248	16528	16488	16637	16962
I025-A	A1201	21059	25559	26430	26356	26684	17725	19628	20348	21120	22285	13474	14314	15232	16354	17545
2100-C	A203	34063	29795	27682	27366	26782	26239	23118	22396	22284	22450	14959	15360	15683	16225	16966
0N21-A	A1001	15481	18361	19414	20119	20612	12468	13913	14961	16222	17159	9083	10332	11432	12585	13640
0N37-A	A702	18059	20614	21635	21850	22242	15481	16102	16720	17747	18830	11931	12173	12972	14086	15206
0601-B	B503	16916	18163	18897	19270	20210	13824	14161	15010	16200	17393	9527	10186	11459	12876	13873
0601-A	A502	16910	19800	19606	19168	19478	13130	15724	15302	15653	16316	7633	11847	11957	12603	13999
0N23-C	C1103	24635	25516	25543	25802	25893	21188	21370	21524	22119	22222	16355	16436	17039	17662	18406
0N34-A	A802	13964	15065	15596	15964	16322	10857	11350	12055	13036	13884	7938	8380	9299	10469	11408
0107-C	C123	13491	12888	11657	10660	9910	11647	11019	10124	9538	9119	10023	9385	8773	8285	8037
0N34-B	B811	20712	20982	20276	19252	18773	17688	17357	16711	16433	16418	14859	14210	13448	13770	13792
0N34-C	C807	16267	15765	15155	14425	13943	13833	13037	12512	12416	12191	11256	10329	10138	10261	10111
0N30-C	C906	20533	20223	19249	18055	17442	18401	17591	16796	16190	15850	15800	15200	14481	14065	13880
0300-B	B406	14248	13406	12220	11190	10425	13089	11939	11014	10234	9734	11783	10638	9851	9334	8900
0107-B	B119	13960	13203	12161	11138	10582	12571	11091	10046	9551	9425	10424	9188	8543	8333	8341
I025-B	B1202	5537	4786	4060	3628	3611	4436	3451	3180	3255	3358	3536	2808	2697	2836	3003
0300-A	A402	15741	14258	12483	11226	10495	14192	12644	11360	10497	9847	12796	11421	10442	9655	9037
0601-C	C506	18530	17649	16089	14514	13048	17329	16635	15579	14206	12928	14764	14707	13918	12931	11966
0300-C	C411	14248	13406	12220	11190	10425	13089	11939	11014	10234	9734	11783	10638	9851	9334	8900
0N21-B	B1002	16916	16729	15836	14673	13818	15252	14465	13672	13174	12753	12825	12370	11946	11587	11336
0N30-A	A902	15650	14192	12742	11399	10331	14065	12722	11604	10787	9804	11955	11020	10304	9603	8936
0P11-A	A305	20245	19659	17861	16887	16080	17959	17061	16439	15536	15263	15246	14755	14525	13861	13727
0N21-C	C1006	15945	15196	14000	13210	12708	14607	13237	12231	11649	11278	12099	11014	10347	9960	9856
0107-A	A110	30668	30327	29255	28394	28037	26756	25996	25644	24940	25083	20081	20304	20290	20536	20712
0P11-B	B301	15234	14291	13153	12317	11729	13768	12845	12039	11491	11149	12301	11480	10976	10632	10286
0P11-C	C310	15372	15072	13835	12967	12158	13813	13337	12642	12212	11697	12319	11909	11542	11042	10631
1401-A	A602	13382	12528	11610	10746	10213	12543	11649	10847	10171	9704	11395	10528	9900	9433	8968
1401-B	B606	8975	7901	7033	6335	5861	8691	7474	6557	6041	5574	7622	6639	5994	5447	5149
1401-C	C609	8636	7783	6940	6268	5910	7613	6408	5809	5523	5396	6669	5597	5017	4824	4754

Table 7 Summary of laboratory resilient modulus values at in-situ conditions.

Project/Site	Lab No.	Measured Resilient Modulus for Each Sequence (psi)														
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0N37-A	A702	24299	24375	23884	23988	23780	19782	19031	19032	19424	20059	14029	13723	13973	14711	15719
0N37-C	C704	20917	22155	21827	21771	22305	18796	18430	18212	18270	18945	14517	13944	14186	14604	15407
0601-A	A502	6876	7450	7720	8209	8866	4814	5683	6895	7899	8582	4136	4931	6246	7373	8282
0N34-B	B811	20750	21784	20658	19502	18866	18316	17724	17400	17149	17053	15593	15002	14707	14591	14668
0N34-C	C807	16581	17236	17082	16941	16791	14490	13934	13880	14025	14382	11178	11014	10993	11465	11975
0300-C	B406	9953	8821	7148	6006	5381	9185	7677	6431	5660	5124	8312	6902	5843	5158	4690
0107-A	A110	24206	25553	23975	23218	22118	22726	22795	22081	21466	20792	18749	18967	18509	18175	17779
0N30-A	A902	7827	6347	5106	4508	4201	7485	5893	4767	4236	3942	6607	5131	4277	3856	3690
0P11-A	A305	4833	4093	3413	3201	3343	4325	3338	3069	3083	3200	3814	2980	2808	2890	3006
0300-C	C411	9953	8821	7148	6006	5381	9185	7677	6431	5660	5124	8312	6902	5843	5158	4690
0N21-B	B1002	17593	17747	16680	15883	15369	16319	15891	15504	14940	14475	14207	13755	13570	13286	12977
0300-A	A402	18286	16964	14989	13103	12042	16492	14853	12985	11935	11202	14900	13230	11846	10820	10268
0601-C	C506	17446	17400	16802	16402	15794	16917	16304	15678	15102	14585	14727	14061	13602	13104	12817
0P11-B	B301	16760	15126	13920	12747	11851	15356	13949	13078	12223	11587	13170	12524	11903	11305	10785
0P11-C	C310	8437	7011	5777	5041	4747	7513	6069	5195	4673	4414	6361	5101	4336	3960	3784
1401-A	A602	12431	12299	11381	10614	9986	11722	11323	10740	10220	9624	10342	10181	9711	9374	8928
1401-B	B606	9703	8979	8205	7595	7133	9166	8329	7683	7119	6838	7920	7482	7083	6648	6385

Table 8 Summary of pavement distresses.

Project Site	Distress Type	Distress Value	Pavement Condition Index (PCI)	Remark
0107-A	No Crack		100	
0107-B	Transverse Crack	814.6 ft/mile	93.42857	In Some Areas Only
0107-C	Transverse Crack	452.6 ft/mile	95	In Some Areas
2100-A	Distress Survey was not Conducted			
2100-B	Distress Survey was not Conducted			
2100-C	Distress Survey was not Conducted			
0P11-A	No Crack		100	
0P11-B	Longitudinal Crack	445 ft/mile	97.5	In Some Areas
0P11-C	Longitudinal Crack	965.5 ft/mile	93.42857	In Some Areas
0300-A	Patch		94.28571	In Some Areas
0300-B	No Crack		100	
0300-C	No Crack		100	
0601-A	Patch & Rutting	0.5 in.	57.58333	Almost In All Areas
0601-B	No Crack		100	
0601-C	Rutting	0.25 in.	77.92308	Almost In All Areas
1401-A	No Crack		100	
1401-B	No Crack		100	
1401-C	Transverse Crack	452.6 ft/mile	99.75	In One Area
0N37-A	No Crack		100	
0N37-B	No Crack		100	
0N37-C	No Crack		100	
0N34-A	Transverse Crack	618.5 ft/mile	97.67857	In Some Areas Only
0N34-B	Transverse Crack	724.1 ft/mile	97.14286	In Some Areas Only
0N34-C	Transverse Crack	905.1 ft/mile	96.42857	In Some Areas Only
0N30-A	No Crack		100	
0N30-B	No Crack		100	
0N30-C	No Crack		100	
0N21-A	Longitudinal Crack	9307.6 ft/mile	59.14286	In All Areas
	Transverse Crack	11215.9 ft/mile		
0N21-B	Longitudinal Crack	8523.16 ft/mile	62.71429	In All Areas
	Transverse Crack	9164.3 ft/mile		
0N21-C	Longitudinal Crack	11351.64 ft/mile	66.78571	In All Areas
	Transverse Crack	3959.9 ft/mile		
0N23-A	No Crack		100	
0N23-B	Bleeding		97.85714	
0N23-C	No Crack		100	
I025-A	Rigid pavement	Distress Survey was not Conducted		
I025-B	Rigid pavement	Distress Survey was not Conducted		
I025-C	Rigid pavement	Distress Survey was not Conducted		

CHAPTER 4 – ELECTRONIC DATABASE

4.1 Introduction

This chapter briefly described the *WYO*ming *MEP*DG Database (*WYOMEP)* developed for this research project. Microsoft® Access 2013 was used to assemble and organize the test data in an efficient manner for subsequent analyses. The detailed description of WYOMEP can be found in the thesis by Hutson (2015).

4.2 MEPDG Database

WYOMEP was developed to effectively collect, sort, filter, and manage large data sets with the goal of facilitating full MEPDG implementation in Wyoming. The database was created using Microsoft® Access 2013 with an intent to make it easily accessible and useable to WYDOT pavement designers. Also, the *WYOMEP* provides a framework for future augmentation of data. WYOMEP has two main features: 1) the ability to quickly create tables, queries, forms, and macros; and 2) to edit, filter, and organize the data presented. As more pavement data sets are regularly collected in the future by WYDOT, *WYOMEP* can only become more valuable on account of the high quality control and assurance undertaken by the research team, and its ability to continue to improve MEPDG roadway designs in Wyoming.

4.3 Database Framework and Description

WYOMEP was developed with the consideration of providing an easily maneuverable user-interface. Figure 4 illustrates the framework of the *WYOMEP*. The user will first encounter the copyright information followed by the main menu (Figure 5a). Located within the main menu are buttons that allow the user to navigate to the initial project page (Figure 5c), create new data records, access current records, access the data wholesale in the master table (Figure 5b), read the legend entries, open the Wyoming road marker map, and utilize statistical models (see Figure 4). The next layer of the database includes access to the original 36 test sites as well as future data via the WYOMEP form (Figure 5d). The WYOMEP form has data fields that are populated by data collected in the WYOMEP query which binds the WYOMEP table with in-situ properties (Figure 5e), laboratory data, FWD data, and attachments (Figure 5f).

To make forms and queries useful and pull specific information from multiple tables, relationships were established between tables linking specific fields. This link was connected to each primary key field of each table (i.e., the ID field). Figure 6 describes the relationships presented in this database for *WYOMEP* to ensure a proper operation. Figure 6 shows that nine of the fifteen tables were connected via the ID field. The ID field auto generates values starting at 1 and increasing by 1 for every consecutive data input. The remaining six tables, which were not shown in Figure 6 and did not a relationship link, were used for statistical calculations. Currently, *WYOMEP* contained 36 test sites, and their IDs were denoted from 1 to 36. If a new table is needed in *WYOMEP*, the user should determine if the corresponding table needs a link with any of the other tables.

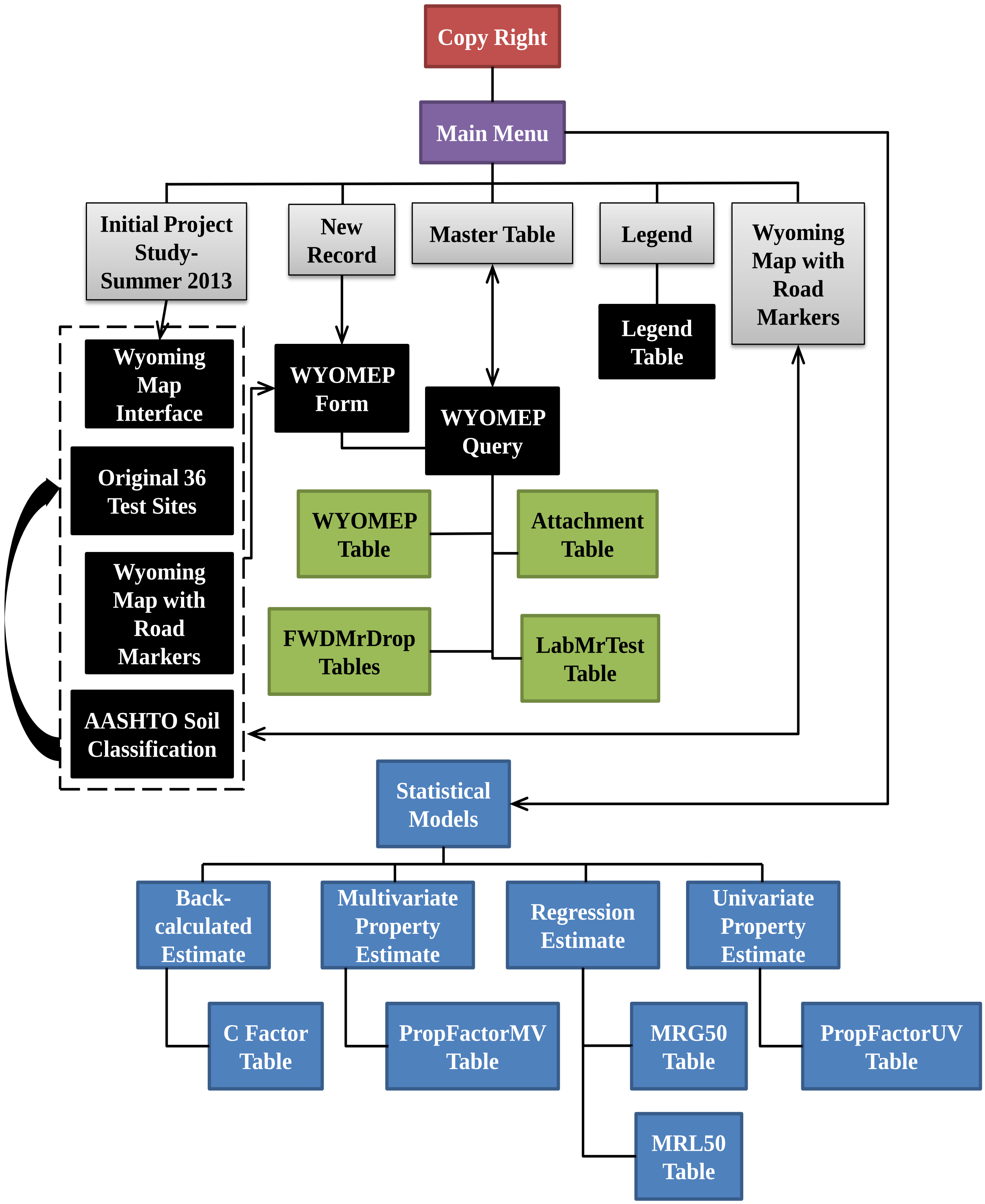

Figure 4 ***WYOMEP*** **framework.**

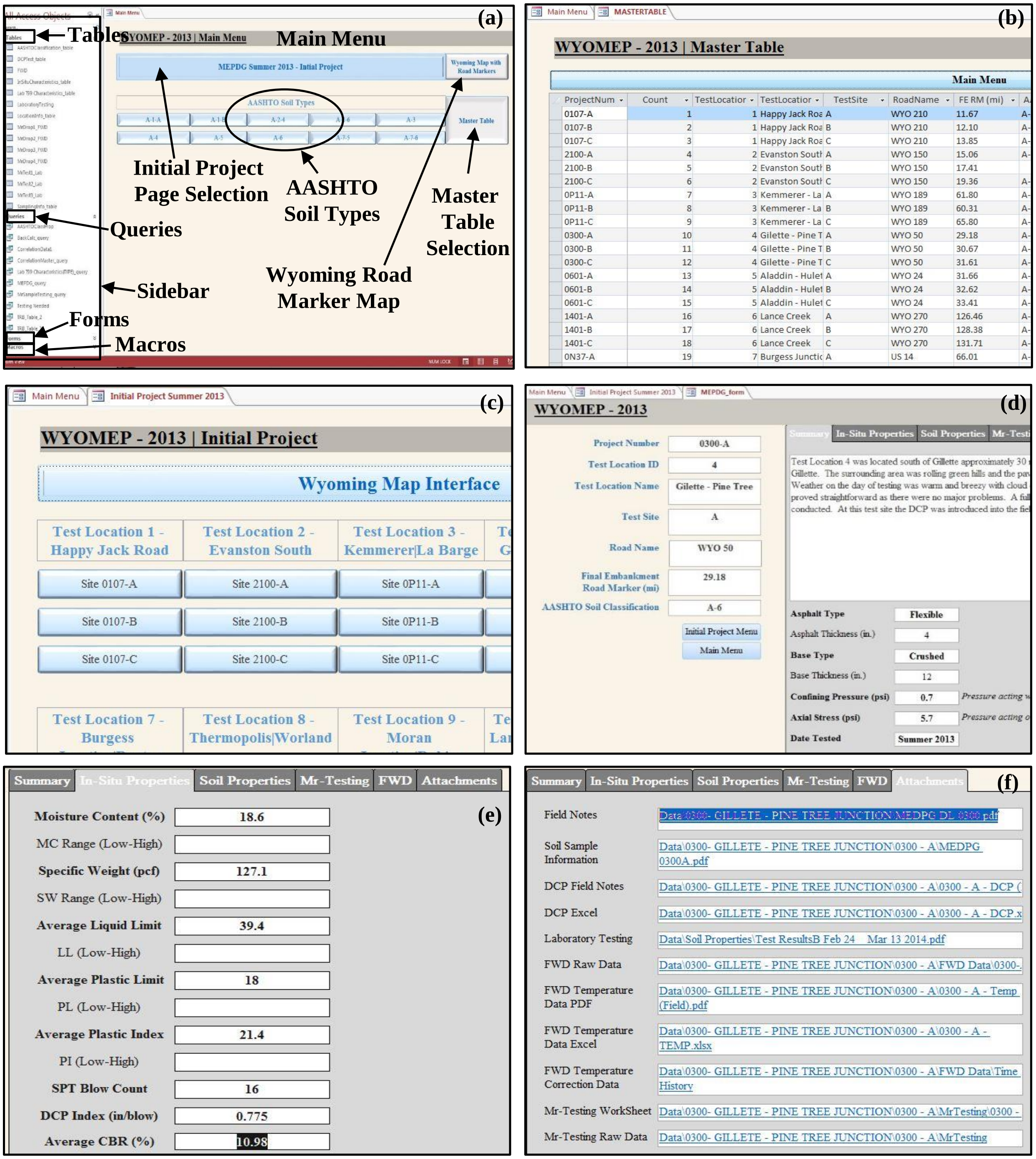

Figure 5 WYOMEP database consisted of: (a) main menu; (b) master table; (c) initial project menu; (d) form for a test site; (e) in-situ properties sub-tab; and (f) attachment sub-tab.

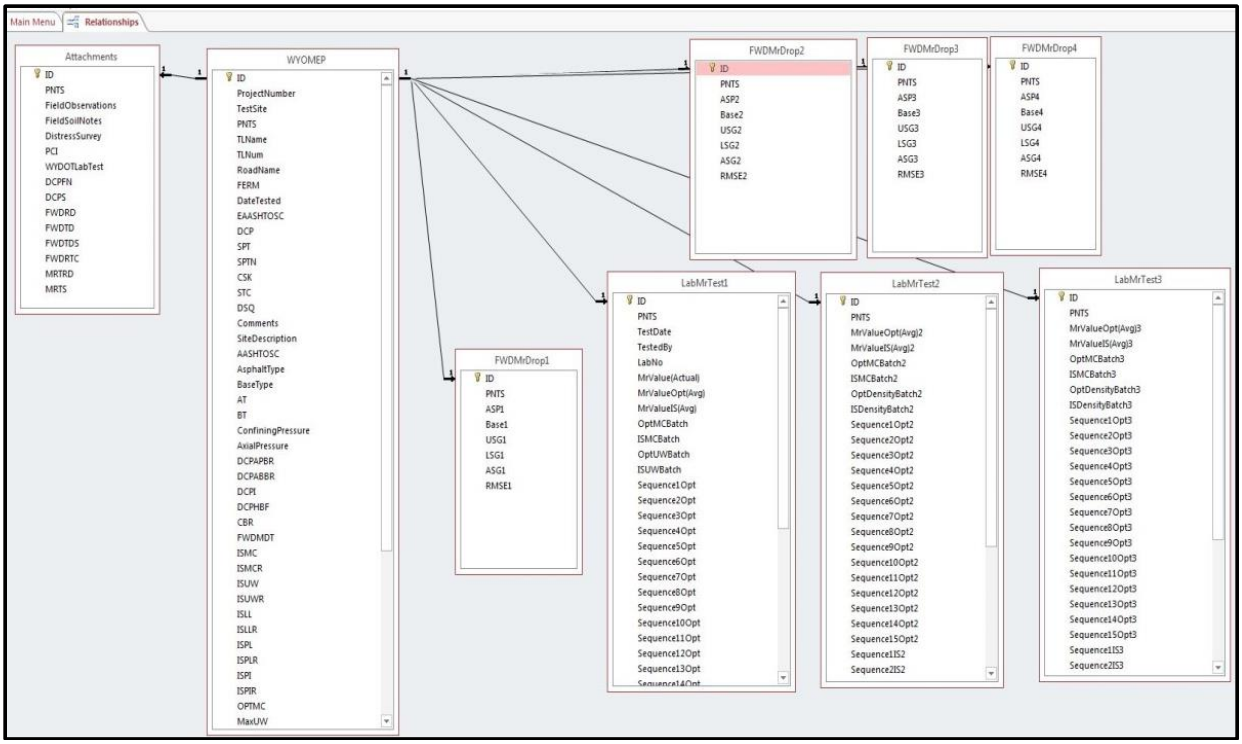

Figure 6 WYOMEP table relationships with primary key links.

The sidebar shown in Figure 5a is the backbone of WYOMEP and is useful for creating, editing, and organizing the data. "Tables" store all data from which *WYOMEP* populates queries and forms. "Tables" allow the user to categorize raw data. "Queries" are customizable tables that can pull data from multiple tables all at once. The master table (Figure 5b) is a query that combines all tables from which *WYOMEP* uses to populate test site forms. "Forms" are used to customize and efficiently display the data from both tables and queries. Lastly, "Macros" are useful for creating a user friendly database through running commands in the background. For instance, when a user wants to access a certain test site from the original field study, he or she can click on the large "MEPDG Summer 2013 – Initial Project" button as shown in Figure 5a, followed by clicking on the desired test site button in Figure 5c. Buttons utilize macros to navigate, and on each form buttons are available for the user to reach a desired form.

The main menu in Figure 5a gives the user the ability to choose between the master table, AASHTO soil types, and the initial MEPDG study from WYDOT. Within the initial study shown in Figure 5a, all the test sites were organized by Test Location number. Each test site button takes the user to the corresponding form similar to that in Figure 5b. This is the main form that displays the data for each site. The AASHTO soil type buttons under initial study direct the user to forms that list each test site containing the soil type. From there, the user is brought to the form for that test site (Figure 5d). The form for each test site has several sub-form tabs which are also shown in Figure 5e and Figure 5f. The sub-form tabbing allows *WYOMEP* to group specific data with the basic information of the test site included outside the sub-form tabs. Each form within *WYOMEP* has built-in with navigation buttons useful for getting the user to the desired form; either the main menu, initial program form, the master table, or a specific test site form while keeping the database cleared of windows.

There are six sub-forms within the main form. Figure 5d shows the “Summary” sub-tab, Figure 5e shows the “In-Situ Properties” sub-tab, and Figure 5f shows the “Attachments” sub-tab. Starting from left to right, the “Summary” is used for general site description, including the road profile (i.e. asphalt thickness), base type (i.e. crushed or cement treated), soil type (i.e. type I or type II in accordance with AASHTO T 307), and the testing date in year and season. The second “In-situ Properties” sub-form lists all the laboratory test data obtained from samples collected using Shelby tubes. The third sub-form, “Soil Properties”, is used for storing all the data pertaining to the disturbed samples collected (i.e. gradation, PI, PL, LL, and mechanical properties). The fourth sub-form, “Mr-Testing”, summarizes the measured M_r values presented in Table 6 and Table 7. The fifth sub-form, “FWD”, summarizes the back-calculated resilient modulus at each pavement layer and its respective root mean square error (RMSE) reported by Hellrung (2015). The six sub-form is designated as “Attachments”, which contains all the linked documents to *WYOMEP*, such as field notes and DCP worksheets.

Since the master table combines data from each table within the *WYOMEP*, the user can easily add new entries without needing to navigate to individual tables. This reduces the chance of input error. However, access to the tables, queries, forms, and macros has not been restricted, and full control is given to the user in case the main menu is closed unintentionally. *WYOMEP* is developed to allow for future editing and for additional data since WYDOT is planning on performing more field and laboratory tests as they continuously improve the implementation of the MEPDG in Wyoming.

CHAPTER 5 – BACK-CALCULATION OF RESILIENT MODULI

This chapter provides a description of two software tools (MODCOMP6 and EVERCALC) used for back-calculation, the back-calculation procedure and a summary of back-calculated resilient moduli of pavement layers. The recommended back-calculation procedure and corresponding results completed by ARA (2015) are primarily described in this chapter. An independent back-calculation study was also completed by Hellrung (2015), and his study outcomes are briefly highlighted in this study.

5.1 Selection of Tools for Back-calculating Pavement Layer Moduli

Back-calculation of in-place layer moduli uses pavement deflection data, typically measured with the FWD. Several back-calculation tools exist that are capable of using deflection data and a set of user inputs to estimate the in-place modulus of pavement layers. These tools can be broadly classified according to the back-calculation procedure into different categories as shown in Table 9 (Von Quintus and Killingsworth 1998, Von Quintus and Rao 2014).

Iterative or gradient-based methods are the most commonly used methods employed in back-calculation programs currently available. Two back-calculation tools – MODCOMP6 and EVERCALC were used in this study. MODCOMP6 is used for pavement layer modulus back-calculation by WYDOT, whereas, EVERCALC was used as the primary back-calculation program in the FHWA study conducted by Von Quintus and Rao (2014). EVERCALC was used to confirm the results obtained from MODCOMP6 computation, and compare the back-calculated layer moduli between WYDOT and LTPP sites. MODCOMP6 was explicitly described in Section 5.2 while EVERCALC was briefly described in Section 5.3. The standardized FWD testing and analysis procedures recommended are described in Appendix A.

Table 9 Back-calculation procedures and list of programs.

Back-calculation Procedure	Programs
Iterative or gradient-search methods	MODCOMP6 (Von Quintus and Simpson, 1998), EVERCALC (WSDOT 2005)
Database search methods	MODULUS (Uzan et al. 1988), COMDEF (Von Quintus et al. 1998), DBCONPAS (Tia et al. 1989), WESDEF
Equivalent thickness methods	ELMOD (Ullitdz, 1978), BOUSDEF (Lytton et al. 1979)
Forward calculation methods or closed-form solutions	AREA (Hoffman et al. 1981), BEST-FIT (Hogg, 1938)
Artificial neural networks	SEARCH, ILLI-SLAB (rigid pavements), ILLI-PAVE (flexible pavements)
Genetic algorithms	MGABPLM (Zhang et al. 2003)
Dynamic analysis methods	DYNABACK-F

5.2 MODCOMP6 Back-calculation Program

MODCOMP6 is a back-calculation software developed at Cornell University and enhanced by Virginia Department of Transportation and Cornell University with the addition of a graphical user interface. MODCOMP6 uses CHEVLAY2, a forward calculation program developed by Chevron to calculate deflections using the layered elastic theory. Von Quintus and Rao (2014) recommended the use of a minimum of four layers (Von Quintus LTPP Report 2014) for back-calculation. Although the program can handle up to fifteen layers, the recommended maximum number of layers including rigid layer is five (Von Quintus and Simpson 2002).

5.2.1 Back-calculation procedure

The MODCOMP program utilizes an iterative-search based algorithm to back-calculate pavement layer moduli. The procedure involves selection of pavement layer thicknesses and Poisson's ratios, and an initial set of modulus values for each layer known as seed moduli. The program also requires FWD load corresponding to each set of deflection readings (or load drop) and sensor spacing as inputs. The forward calculation program calculates the deflections at each sensor location using the user-defined seed moduli (d_{calc}) and the root mean square error (RMSE) between calculated (d_{calc}) and measured deflection data (d_{meas}). RMSE of deflections is calculated using Equation (9) (Von Quintus et al. 1998)

$$\mathrm{RMSE(percent)} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{d_{calc,i} - d_{meas,i}}{d_{meas,i}}\right)^2} \times 100 \text{ percent} \tag{9}$$

where,

$d_{calc,i}$ = deflection calculated by the forward calculation program at the i^{th} sensor,
$d_{meas,i}$ = measured deflection at the i^{th} sensor, and
n= total number of sensors.

The seed moduli are adjusted and the deflections are recalculated using the forward calculation routine iteratively until an RMSE value lower than a specified tolerance is reached. A typical tolerance level of 1 percent is recommended for back-calculation. In addition to the deflection tolerance, modulus rate of convergence can also be specified, which is the percent difference in the modulus of a pavement layer for successive iterations (Irwin 2010). The modulus rate of convergence is calculated using

$$\text{Modulus rate of convergence(percent)} = \frac{E_{k+1,i} - E_{k,i}}{E_{k,i}} \times 100 \text{ percent} \tag{10}$$

where,

$E_{k+1,i}$ = back-calculated modulus of the i^{th} layer at $k+1^{th}$ iteration, and
$E_{k,i}$ = back-calculated modulus of the i^{th} layer at k^{th} iteration.

Figure 7 shows the sequence of steps in the MODCOMP back-calculation procedure. The flowchart is modified from that presented for EVERCALC in the EVERCALC User's Manual (WSDOT 2005), except that different forward calculation routines are used within the two programs.

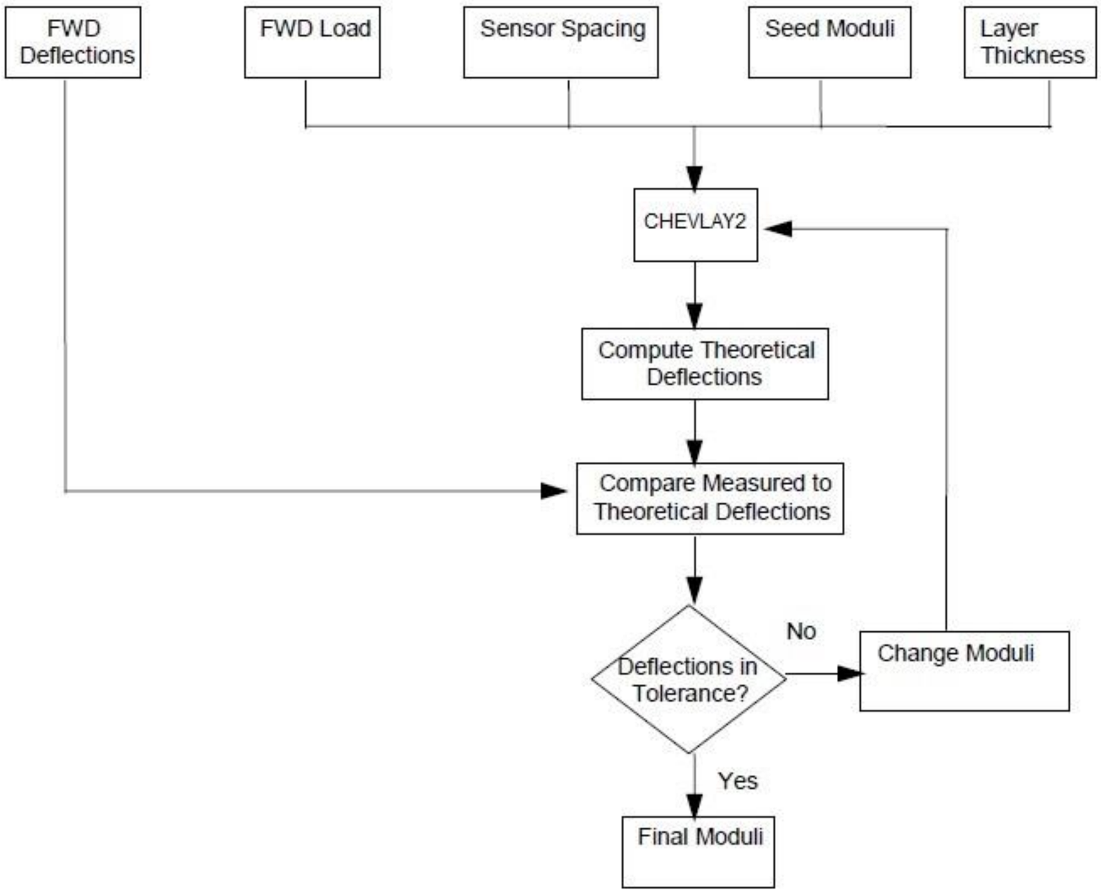

Figure 7 Back-calculation Procedure Flowchart – MODCOMP

5.2.2 Normalization of deflection data

Deflection data measurements are made in the field at different drop heights, and the corresponding load readings for each drop are recorded. The deflection values are normalized to a reference load of 9 kips or a user-defined reference load if the actual load for the drop within 1 kip of apparent target. The normalized deflection is calculated using Equation (11) (Von Quintus et al. 1998)

$$\text{Normalized Deflection} = \text{Actual Deflection} \times \frac{\text{Reference Load}}{\text{Actual Load}} \tag{11}$$

The normalized deflections are used in the calculation of deflection RMSE according to Equation (9).

5.2.3 Error checks

The deflection data used as input for back-calculation was subjected to several error checks. The following checks were performed as part of this exercise:

- Maximum deflection.
- Non-decreasing deflections with increasing distance of sensor from load plate.
- Deflection trend with repeated drops at same load level.

- Deflection trend with increasing load level.
- Linearity of maximum deflection with load level.
- SLIC transformation to check for sensor placement.
- Cumulative differences of deflection.

Basins without maximum deflection and those with non-decreasing deflections were excluded from analysis. Specific drops were selected where distinct trends were observed among repeated drops at the same load level. Similarly, specific load levels were selected where trends were observed among load levels. Basins with sensor placement issues as determined from Stubstad-Lukanen-Irwin-Clevenson (SLIC) analysis were also excluded. The SLIC transformation was used to identify sensor positioning errors. Sensor offset is calculated using Equation (12) (Stubstad et al. 2006)

$$\mathrm{Offset} = \mathrm{a} + \mathrm{b}\ln(|\mathrm{nd}|) + \mathrm{c}\ln(|\mathrm{nd}|)^{\mathrm{e}} + \mathrm{d}(\mathrm{residual}) \tag{12}$$

where,

$|nd|$ = the normalized deflection, and

a, b, c, d and e = regression coefficients.

The calculated offsets are compared to measured offsets by plotting ln(offset) on the X-axis and ln(ln(|nd|)) on the Y-axis for both sets of data. The deflection data was considered free of sensor placement errors if the regression curve matched closely with the actual deflection data curve.

5.2.4 Optimization and convergence criteria

The MODCOMP6 program converges at a solution for the back-calculated layer moduli in either of the following three cases (Deusen 1996):

1) RMSE for deflection readings is lower than the specified tolerance.
2) RMSE of modulus for all layers for a given iteration and the previous iteration is lower than the specified tolerance.
3) The maximum number of iterations is reached.

The RMSE calculated using Equation (9) is used as the tolerance criterion for Case 1, and that calculated using Equation (10) is used for Case 2. When the program reaches the maximum number of iterations as in Case 3, the back-calculation process is terminated irrespective of whether the deflection and modulus error criteria are satisfied or not.

5.2.5 Compilation of deflection data – input file generation

The deflection data and other inputs for back-calculation using MODCOMP6 were compiled in the requisite format for each WYDOT section. MODCOMP6 requires the data to be formatted as a ".DAT" file. The input file contains the following information for each load drop:

- Pavement layer structure, including thickness and Poisson's ratios of each layer.
- Seed moduli.
- Tolerances for deflection error and convergence of modulus.
- Sensor configuration (spacing of sensors from the load plate).
- Allowable range (maximum and minimum) of modulus for each layer.
- Deflection readings.

MODCOMP6 allows layers to have '*fixed'* modulus values, which can be entered by the user. The fixed layer modulus is held constant and not treated as an iteration variable. When stiff layers or bedrock are encountered at a shallow depth beneath the subgrade, they can be treated as fixed layer with a recommended value of 500,000 psi (Irwin 2010) so that the modulus of the actual subgrade can be accurately back-calculated.

5.2.6 Normalization of back-calculated layer moduli

MODCOMP6 does not apply temperature normalization to back-calculated pavement layer moduli. Thus, the in place moduli from the back-calculation process were not adjusted – they represent the actual values at the time of deflection basin testing.

5.2.7 Summarization of Output Data

MODCOMP6 generates two output files for each back-calculation run. The first output file is an ".LST" file, which contains all back-calculated layer moduli, including the deflection basin fit data for each iteration. The second output file, which is a summary of back-calculated modulus values for each iteration, was used to generate station-wise and section-wise summaries for WYDOT sections. Since data from the output ".SUM" files could not be directly converted to spreadsheet format for summarization, a data extraction program was developed in C++ language for extracting the final iteration (converged) back-calculated layer moduli for each load drop in a comma-separated format. Back-calculated data were averaged for different load levels for each station to check for sensitivity of modulus to drop load. The output data was also summarized to obtain the average station-wise moduli as well as the average moduli values for the entire 700-foot section. The summary of MODCOMP6 back-calculation results is presented in Section 5.4 and the detailed results can be found in ARA (2015).

5.3 EVERCALC Back-calculation Program

EVERCALC is an iterative pavement layer modulus back-calculation program developed at the Washington State Department of Transportation (WSDOT) as part of Everseries Pavement Analysis Programs (WSDOT 2005). It is capable of analyzing up to five layers and can handle deflection data from up to ten sensors. EVERCALC uses WESLEA, which is a layered elastic analysis program to perform forward calculation of deflections. Optimization of layer modulus values to minimize error between the user-entered (measured) deflection basin and forward calculated deflection basin is done using an Augmented Gauss-Newton method (Lee et al. 1988). The back-calculation procedure used in EVERCALC is an iterative-search procedure similar to that used in MODCOMP6. The process of normalization of deflection data is similar to that described for MODCOMP6. Error checks described for MODCOMP6 were also applied to EVERCALC. The optimization procedure used in EVERCALC is a modified Augmented Gauss-Newton method (WSDOT 2005), where the modulus for a given iteration is calculated based on the deflection error from the previous iteration. Convergence criteria used to determine the back-calculated layer moduli for each drop are similar to that used for MODCOMP6. EVERCALC requires two different input files for a single analysis. The first file is generated as a ".GEN" file, which contains information about the pavement layer structure, seed moduli, tolerances for error, sensor configuration (spacing of sensors from the load plate) and allowable range (maximum and minimum) of back-calculated modulus values for each layer. The second file is a ".DEF" file, which contains the thicknesses of pavement layers, drop load and measured deflection values. EVERCALC also allows the user to specify a stiff layer with fixed modulus value, which varies between 100,000 psi and 1,000,000 psi (WSDOT 2005). Back-calculated asphalt layer modulus can be adjusted to a reference temperature directly by the software, and the default reference temperature used in EVERCALC is 77^{o}F. The back-calculation results from EVERCALC are saved as a ".SUM" file. The output file contains back-calculated layer moduli for each load drop in a tab-separated data format, which can be easily exported to MS Excel for further calculation. The summary of EVERCALC back-calculation results is presented in Section 5.4 and the detailed results can be found in ARA (2015).

5.4 Summary of Back-calculation Results by ARA (2015)

The back-calculated moduli of pavement layers from WYDOT and LTPP test sites were organized by pavement section and layer type. A summary of complete back-calculation results from each test sites is presented in ARA (2015).

5.4.1 LTPP results

Table 10 shows a summary of back-calculated layer moduli and corresponding layer thicknesses for LTPP test sections in the State of Wyoming. Back-calculated resilient values were extracted for the first available FWD test date. The back-calculation of layer moduli in the LTPP database was performed using EVERCALC software only, hence there is no comparison of the data with back-calculation values from MODCOMP6. The mean asphalt modulus was calculated as 3,307,200 psi, mean treated base course modulus was calculated as 320,000 psi, mean dense graded aggregate base (DBAG) course modulus was 26,100 psi, mean weathered subgrade modulus was 50,8000 psi and the mean natural subgrade was calculated as 33,800 psi. The back-calculated layer moduli had very high standard deviations which were more than 50 percent of the mean values. A high standard deviation of the back-calculated elastic moduli along a roadway is common because of thickness, layer property, and other variations along the roadway.

5.4.2 WYDOT results

Table 11 shows the average back-calculated layer moduli for WYDOT test locations using MODCOMP6 software. The moduli for test locations 0N30-A, B and C could not be back-calculated using MODCOMP6 due to non-convergence of the deflection basin (i.e. the RMSE remained very high after the maximum number of iterations and the program did not converge upon a single modulus value for the layers). Table 12 shows the average back-calculated layer moduli for WYDOT test locations using EVERCALC software.

The mean asphalt concrete layer modulus calculated using MODCOMP6 was 690,000 psi which is higher than 575,000 psi calculated using EVERCALC. Similarly, the average weathered subgrade modulus of 49,200 psi using MODCOMP6 is higher than 20,100 psi. The average asphalt treated base (ATB) course or lower asphalt concrete layer modulus calculated using MODCOMP6 was 199,300 psi, which is lower than that 326,000 psi from EVERCALC. The average back-calculated moduli for DGAB (32,200 psi using MODCOMP6 versus 32,800 psi using EVERCALC) and natural subgrade (28,800 psi using MODCOMP6 versus 27,000 psi using EVERCALC) were very similar.

An excellent agreement was observed between the values back-calculated using MODCOMP6 and EVERCALC for the asphalt concrete layer (HMA), crushed stone (unbound) base course and the natural subgrade layers. Figure 8 shows the plot of MODCOMP6 versus EVERCALC back-calculated values for HMA layers of WYDOT test sites. The back-calculated values from locations 0N34-B and 0N34-C were considered outliers and excluded from this analysis. MODCOMP6 predicted a very high, unrealistic average modulus for these two locations (2,400,000 psi) as compared to EVERCAL (1,600,000 psi), despite both back-calculation runs returning very low RMSE values (less than 1.5 percent). The comparisons for crushed stone (unbound) base course and the natural subgrade layers can be found in ARA (2015).

Table 10 Back-calculation data summary for LTPP test sites in Wyoming.

Section ID	HMA		CTB		DGAB		Weathered Subgrade						Natural Subgrade Modulus (psi)	RMSE (percent)
	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)	AASHTO	R	ω_{opt} (percent)	$\gamma_{d\text{-}max}$ (pcf)		
1007	2.8	413,700		—	6.2	12,000	24	23,700	A-2-4	77	12.5	120	25,300	1.67
2015	3.9	7,259,000	10	201,800		—	13.8	45,100	A-4	74	6	140	54,800	2.20
2017	2.3	4,660,400	11.2	764,600		—	24	17,200	A-6	20	15.7	107	20,800	1.51
2018	5.8	1,075,700	14.4	194,400		—	24	52,200	A-6	6	8.5	112	25,500	1.83
2019	4.2	6,458,200	10.6	279,100	14.4	42,800	24	118,900	A-2-4	62	15.1	109	28,400	1.73
2020	5.0	3,203,900	12.6	354,100		—	24	61,400	A-6	5	14.5	115	29,900	1.42
2037	3.5	5,826,300	16.4	492,200		—	24	37,200	A-4	21	14.8	119	39,500	1.72
6029	3.6	539,700		—	10.9	32,700	24	35,600	A-2-4	73	8.8	134	74,900	2.35
6032	5.2	540,500	9.8	127,200*		—	36	106,300	A-1-a	78	8.9	119	23,600	2.67
7772	2.2	6,845,200	15	110,800		—	24	23,200	A-2-6	50	18.4	107	25,400	2.00
7773	4.6	2,541,700	5.2	163,500		—	15.8	79,000	A-1-b	64	6.1	139	39,000	2.62
7775	4.5	323,000		—	6.8	17,100	15.8	9,900	A-2-4	84	6.2	103	18,700	1.03
Mean		3,307,200		320,000		26,100		50,800					33,800	
SD		2,770,000		216,500		14,100		34,800					16,300	
Min		323,000		110,800		12,000		9,900					18,700	
Max		7,259,000		764,600		42,800		118,900					74,900	

HMA−Hot mix asphalt; CTB−Cement treated base; DGAB−Dense graded aggregate base; and *−Layer is an AC base instead of cement-treated base course.

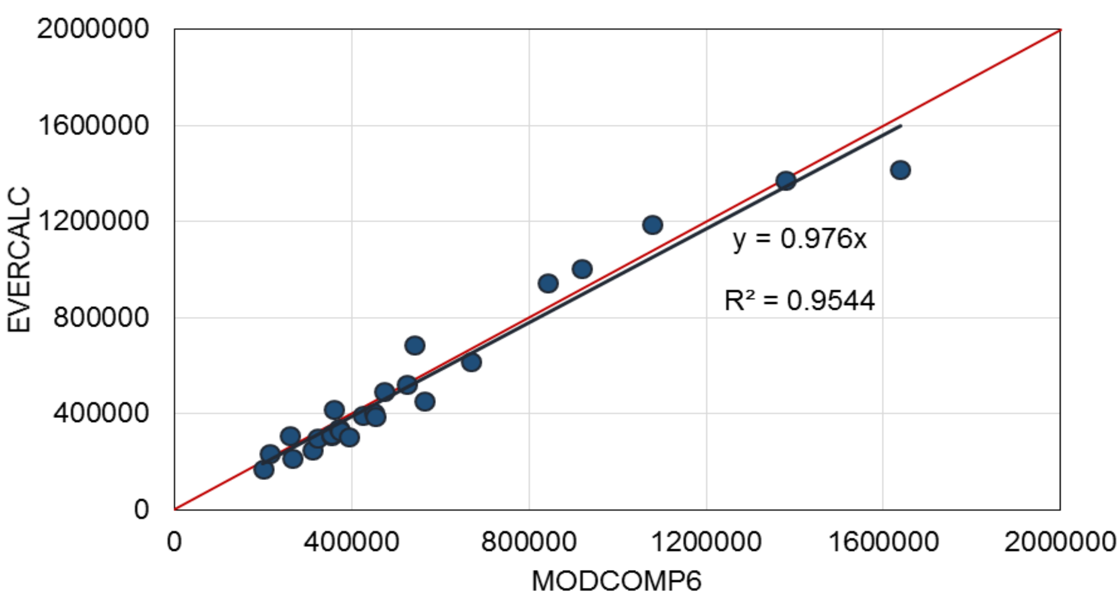

Figure 8 Comparison of MODCOMP6 and EVERCALC back-calculated HMA modulus.

Table 11 MODCOMP6 back-calculation data summary for WYDOT test locations.

Section ID	HMA		ATB/CTB		DGAB		Weathered Subgrade		Natural Subgrade Modulus (psi)	RMSE (percent)
	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)	Thickness (inch)	Modulus (psi)		
0107-A	4	1,380,000	8^{ATB}	450,000	9.5	86,000	24	22,000	37,000	0.79
0107-B	4	920,000	8^{ATB}	460,000	9.5	44,000	24	60,000	31,000	0.63
0107-C	4	1,079,000	8^{ATB}	90,000	9.5	3,000	24	212,000	16,000	0.53
0300-A	4	843,000			12	13,000	24	15,000	19,000	1.88
0300-B	4	425,000			12	5,000	24	71,000	16,000	1.27
0300-C	4	200,000			12	6,000	24	23,000	17,000	3.12
0601-A	6	353,000			16	77,000	24	36,000	41,000	4.13
0601-B	6	451,000			16	98,000	24	32,000	45,000	2.61
0601-C	6	267,000			16	18,000	24	28,000	29,000	3.56
0N21-A	8	259,000	9^{CTB}	82,000			24	37,000	28,000	2.00
0N21-B	8	214,000	9^{CTB}	160,000			24	15,000	21,000	1.56
0N21-C	8	324,000	9^{CTB}	92,000			24	14,000	26,000	1.38
0N23-A	5	523,000			10	69,000	24	24,000	38,000	0.96
0N23-B	5	474,000			10	71,000	24	28,000	34,000	1.16
0N23-C	5	359,000			10	53,000	24	31,000	38,000	1.77
0N30-A	4				6		24		25,000	
0N30-B	4				6		24		35,000	
0N30-C	4				6		24		27,000	
0N34-A	4	1,638,000	7^{ATB}	165,000	13	12,000	24	71,000	16,000	0.83
0N34-B	4	2,391,000	7^{ATB}	141,000	13	36,000	24	53,000	19,000	1.19
0N34-C	4	2,503,000	7^{ATB}	154,000	13	105,000	24	9,000	28,000	0.96
0N37-A	5	453,000			12	13,000	24	41,000	39,000	2.40
0N37-B	5	563,000			12	15,000	24	28,000	33,000	2.10
0N37-C	5	670,000			12	15,000	24	21,000	43,000	2.13
0P11-A	6	371,000			9.5	9,000	24	29,000	16,000	1.59
0P11-B	6	395,000			7	5,000	24	114,000	21,000	4.32
0P11-C	6	310,000			12	3,000	24	251,000	13,000	4.32
1401-A	4	540,000			10	10,000	24	16,000	12,000	1.80
1401-B	4	374,000			10	11,000	24	10,000	19,000	1.93
1401-C	4	355,000			10	11,000	24	38,000	29,000	2.64
Mean		690,100		199,300		32,800		49,200	27,000	
SD		614,300		148,400		33,400		57,500	9,600	
Min.		200,000		82,000		3,000		9,000	12,000	
Max.		2,503,000		460,000		105,000		251,000	45,000	

HMA−Hot mix asphalt; ATB−Asphalt treated base; CTB−Cement treated base; and DGAB−Dense graded aggregate base.

Table 12 EVERCALC back-calculation data summary for WYDOT test locations.

Section ID	HMA Thickness (inch)	HMA Modulus (psi)	ATB/CTB Thickness (inch)	ATB/CTB Modulus (psi)	DGAB Thickness (inch)	DGAB Modulus (psi)	Weathered Subgrade Thickness (inch)	Weathered Subgrade Modulus (psi)	Natural Subgrade Modulus (psi)	RMSE (percent)
0107-A	4	1,373,000	8 ATB	1,147,000	9.5	98,000	24	19,000	48,000	0.93
0107-B	4	1,003,000	8 ATB	569,000	9.5	65,000	24	23,000	35,000	0.67
0107-C	4	1,189,000	8 ATB	83,000	9.5	5,000	24	24,000	19,000	0.62
0300-A	4	945,000			12	7,000	24	11,000	20,000	1.33
0300-B	4	394,000			12	9,000	24	20,000	17,000	1.28
0300-C	4	169,000			12	9,000	24	10,000	19,000	1.41
0601-A	6	309,000			16	80,000	24	41,000	41,000	2.19
0601-B	6	402,000			16	106,000	24	33,000	47,000	1.79
0601-C	6	213,000			16	25,000	24	20,000	30,000	1.73
0N21-A	8	308,000	9 CTB	68,000			24	46,000	29,000	1.42
0N21-B	8	233,000	9 CTB	120,000			24	10,000	22,000	1.23
0N21-C	8	301,000	9 CTB	97,000			24	14,000	29,000	1.02
0N23-A	5	525,000			10	68,000	24	23,000	40,000	0.85
0N23-B	5	495,000			10	70,000	24	26,000	35,000	1.03
0N23-C	5	421,000			10	47,000	24	31,000	41,000	1.33
0N30-A	4	252,000			6	17,000	24	15,000	25,000	1.99
0N30-B	4	392,000			6	36,000	24	23,000	35,000	1.72
0N30-C	4	154,000			6	20,000	24	10,000	27,000	4.16
0N34-A	4	1,420,000	7 ATB	161,000	13	19,000	24	13,000	17,000	0.73
0N34-B	4	1,775,000	7 ATB	245,000	13	13,000	24	31,000	20,000	0.75
0N34-C	4	1,454,000	7 ATB	450,000	13	28,000	24	22,000	24,000	1.46
0N37-A	5	389,000			12	19,000	24	25,000	42,000	1.23
0N37-B	5	456,000			12	26,000	24	18,000	36,000	1.11
0N37-C	5	617,000			12	19,000	24	18,000	46,000	1.01
0P11-A	6	340,000			9.5	14,000	24	18,000	16,000	0.89
0P11-B	6	307,000			7	25,000	24	14,000	22,000	1.03
0P11-C	6	248,000			12	8,000	24	14,000	15,000	1.01
1401-A	4	519,000			10	11,000	24	9,000	13,000	1.37
1401-B	4	330,000			10	14,000	24	9,000	22,000	1.24
1401-C	4	313,000			10	14,000	24	13,000	32,000	1.42
Mean		574,800		326,600		32,200		20,100	28,800	
SD		444,000		354,000		29,000		9,200	10,400	
Min.		154,000		68,000		5,000		9,000	13,000	
Max.		1,775,000		1,147,000		106,000		46,000	48,000	

HMA−Hot mix asphalt; ATB−Asphalt treated base; CTB−Cement treated base; and DGAB−Dense graded aggregate base.

For weathered (compacted) subgrade layers, the modulus values back-calculated using EVERCALC are very low when compared to those back-calculated using MODCOMP6. Figure 9 shows the plot of back-calculated modulus values from both programs, and the regression equation shows that EVERCALC values are about 55 percent of the MODCOMP6 values. The regression was performed after excluding values for sections 0P11-B, 0P11-C, 0N34-C and

0107-C. For MODCOMP6, these sections exhibited compensating layer effects characteristics. The MODCOMP6 modulus values for three of these locations were at least about 10 times higher than the corresponding EVERCALC values and were hence discarded as outliers. The fourth location (0N34C), MODCOMP6 back-calculated modulus was less than half the modulus value back-calculated from EVERCALC. The MODCOMP6 back-calculated modulus for the DGAB at this location was 105,000 psi suggesting compensating layer effects but in the opposite direction from the other three locations. It is unclear or unknown why there was a significant difference in results between EVERCALC and MODCOMP6 for the weathered subgrade layer and not any of the other layers. Based on previous experience from sampling and testing the upper or weathered soil strata, the upper soil is usually wetter or softer.

The back-calculated subgrade moduli from both MODCOMP6 and EVERCALC were also summarized by soil type. Table 13 shows the mean, standard deviation (SD), minimum and maximum subgrade moduli for each AASHTO soil type back-calculated using MODCOMP6 and EVERCALC. The results show that the absolute percentage difference between the mean back-calculated modulus is very small. Comparing the average moduli determined from WYDOT and LTPP data sets, the moduli from LTPP were consistently higher than that from WYDOT. Hence, the LTPP results were excluded in the subsequent analysis described in Chapter 6.

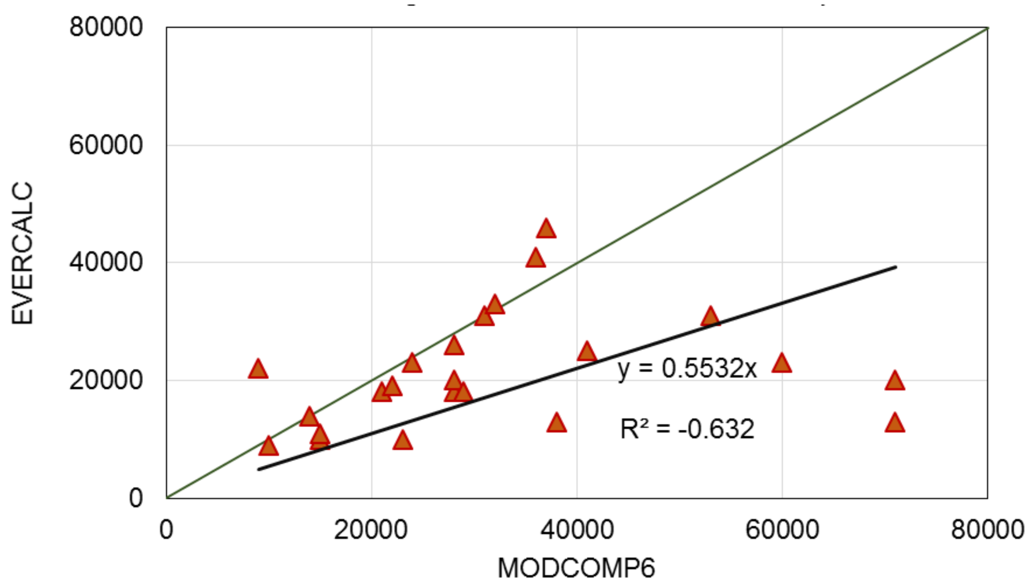

Figure 9 Comparison of MODCOMP6 and EVERCALC back-calculated weathered subgrade modulus.

5.5 Back-calculation by Hellrung (2015)

Recognizing the limitations associated with the back-calculation procedure that produces non-unique resilient moduli, an independent back-calculation study was conducted by Hellrung (2015) using MODCOMP6. Criteria for test site selection, asphalt temperature correction, and material seed modulus selection were established prior to conducting a back-calculation analysis. Seed modulus and pavement structural model adjustments were recommended to determine realistic back-calculated subgrade resilient moduli. To determine when to terminate the back-calculation process, three levels of analysis (A, B, and C) based on specific criteria as shown in

Table B-1 were established based on literature review and typical material moduli ranges (VDOT 2007). The lower and upper bounds of modulus values for subgrade soils were selected to be 4,000 psi based on the VDOT (2007) recommendation and 40,000 psi from the NCHRP (2004), respectively. The proposed back-calculation flow chart is described in Figure B-1 of Appendix B. Adopting the back-calculation protocol, the average back-calculated modulus values for all pavement layers and the twenty-five sites are summarized in Table B-2. The average asphalt modulus of 570,000 psi is well within the typical range. The average base modulus of 25,198 psi is closer to the lower bound of 10,000 psi as the fixed-layer approach was utilized. The realistic difference in modulus values between upper and lower subgrade layers was evident. The average modulus value of 26,090 psi for the upper subgrade is higher than 17,446 psi for the lower subgrade. Detailed description of this alternative back-calculation procedure can be found in the thesis by Hellrung (2015) and a published paper by Ng et al. (2016).

Table 13 Comparison of MODCOMP6 and EVERCALC subgrade moduli.

AASHTO Soil Class	Statistical Parameter	MODCOMP6 M_r (psi)	EVERCALC M_r (psi)	Percent Difference	Average (WYDOT)	Average (LTPP)
A-1-a	Mean		35,000		35,000	106,300
	SD					
	Minimum					
	Maximum					
A-1-b	Mean	35,800	38,000	6.1	36,900	79,000
	SD	5,200	5,900			
	Minimum	28,000	29,000			
	Maximum	43,000	46,000			
A-2-4	Mean	31,200	33,000	5.8	32,100	47,025
	SD	14,000	13,900			
	Minimum	16,000	17,000			
	Maximum	45,000	47,000			
A-4	Mean	23,500	24,600	4.7	24,100	41,150
	SD	7,100	6,900			
	Minimum	16,000	17,000			
	Maximum	31,000	35,000			
A-6	Mean	23,500	26,100	11.1	24,800	43,600
	SD	7,500	10,000			
	Minimum	16,000	16,000			
	Maximum	37,000	48,000			
A-7-6	Mean	18,800	20,000	6.4	19,400	Not Available
	SD	6,800	8,600			
	Minimum	12,000	9,000			
	Maximum	29,000	32,000			

CHAPTER 6 – DEVELOPMENT AND RECOMMENDATION OF MATERIAL PROPERTIES

6.1 Introduction

This chapter describes four different approaches for the determination of resilient modulus of subgrade materials for MEPDG Level 2 inputs. Also, the development of Level 3 material properties database for typical WYDOT unbound base and subgrade materials for M-E design is described. These recommendations were primarily established by ARA (2015). An independent study to determine resilient modulus for subgrade materials for Level 2 inputs was also conducted by the University of Wyoming research team. Their study outcomes are briefly described in this chapter and Appendix C. The detailed descriptions of their studies can be found in Hellrung (2015), Henrichs (2015) and Hutson (2015).

6.2 Determination of Subgrade Resilient Modulus

6.2.1 Development of constitutive models

Regression analysis was conducted on the laboratory resilient modulus test data to determine the constitutive model parameters k_1, k_2 and k_3 as given by Equation (7). R statistical software was used to perform the regression and the data required for analysis was formatted accordingly from the test results. The parameters k_1, k_2 and k_3 were calculated using both R and verified using Microsoft® Excel. Table 14 shows the summary of k-values for each AASHTO soil type from WYDOT and LTPP sections. It is important to note that there is a significant bias between the laboratory-measured resilient modulus stored in the LTPP database and those measured within this study. The reason for this bias is unknown, but restricts combining the values included in the LTPP database and those measured under this study.

$$M_r = k_1 \sigma_a \left(\frac{\sigma_b}{\sigma_a}\right)^{k_2} \left(\frac{\tau_{oct}}{\sigma_a} + 1\right)^{k_3} \tag{7}$$

where,

σ_a = atmospheric pressure;

σ_b = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$ (sum of major, intermediate and minor principal stresses);

τ_{oct} = Octahedral shear stress = $\frac{\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2}}{3}$; and

k_1, k_2 and k_3 = regression coefficients given in Table 14.

Table 14 Summary of resilient modulus constitutive model coefficients.

Soil Class	WYDOT Sections			LTPP Sections		
	Mean k_1	Mean k_2	Mean k_3	Mean k_1	Mean k_2	Mean k_3
Base	—	—	—	665.8	0.481	-0.332
A-1-a	1,544.8	0.626	-0.527	—	—	—
A-1-b	1,505.6	0.619	-1.063	635.3	0.370	-1.205
A-2-4	1,131.2	0.483	-1.056	570.2	0.551	-1.146
A-2-6	—	—	—	843.4	0.1549	-0.6828
A-4	1,003.6	0.52	-0.356	711.7	0.270	-1.284
A-6	801.6	0.294	0.443	712.9	0.243	-1.482
A-7-6	520.4	0.264	0.651	—	—	—

An independent study was conducted by Henrichs (2015) to calibrate the parameters k_4, k_5 and k_6 to estimate resilient modulus using an alternative constitutive model given by Equation (8). These parameters were calibrated using the R statistical software for subgrade soils with R≤50 and R>50. Appendix C.1 describes the statistical analysis, the calibration procedure and the constitutive model.

6.2.2 Determination of back-calculated to laboratory modulus adjustment factor

Subgrade soils are characterized by measuring the resilient modulus using cyclic triaxial load test in the laboratory or by measuring the in-place elastic modulus using FWD testing in the field. However, both tests result in considerably different strength values as the two tests are conducted under different conditions (Kim et al. 2010). In order to determine the subgrade resilient modulus for input in the MEPDG, the sample used for testing must be prepared such that it represents the in-situ condition of the soil as closely as possible. Laboratory testing of soil samples is conducted at different boundary conditions and temperature than those that exist in the field. It is therefore important to determine the typical in-situ pavement stress state in order to convert the field-derived modulus to laboratory-derived resilient modulus.

The in-situ stress state was determined for each WYDOT pavement section as the sum of stresses due to a 9,000 lb load and overburden pressure. Major and minor principal stresses (σ_1, σ_2, and σ_3) were calculated using the EVERSTRESS program, which is a part of the EVERSERIES suite developed by WSDOT (WSDOT 2005). A Poisson's ratio of 0.35 was assumed for all layers and the stresses and strains were computed at a depth of 18 inches below the subgrade surface. This depth was determined by Von Quintus and Killingsworth (1998) to be the effective depth for characterizing the entire subgrade. The vertical and radial stresses computed at this depth using EVERSTRESS program, along with the overburden stress from layers on top of the subgrade, were used to determine the principal stresses. Using these calculated stresses, resilient modulus at the in-situ pavement stress state was calculated using Equation (7) for each WYDOT test location. Table 15 shows the calculated stresses and in-situ resilient modulus for the WYDOT sections.

The field-to-lab adjustment factor or C-factor is defined as the lab-measured resilient modulus (M_r) at the in situ stress condition to the field-measured elastic modulus (M_R) from back-calculation. In other words, the laboratory measured resilient modulus at the in-situ stress condition during FWD deflection basin testing (listed in Table 15) divided by the back-calculated elastic modulus (listed in Table 16). Table 16 summarizes the C-factors for the WYDOT test locations. The LTPP test sites were excluded because of the significant bias between the WYDOT and LTPP laboratory-measured resilient modulus values, as discussed above.

The average C-factor for the WYDOT test sections was calculated as 0.49 with a standard deviation of 0.18. Hence, the recommended equation to correct the field-measured elastic modulus (M_R) from back-calculation to the lab-measured resilient modulus (M_r) is given by

$$\mathrm{M_r} = 0.49\mathrm{M_R} \tag{13}$$

The coefficient of variation for this data set of conventional flexible pavements is about 39 percent. An average C-factor of 0.49 is higher than the value included in the MEPDG Manual of Practice for conventional flexible pavements (0.35), but similar to the value calculated for test location in Montana, an average value of 0.50 (see Table 2). This difference between the global value (0.35) derived for sites across the U.S. is believed to be related to the water content for roadways built in drier or more arid areas. In other words, the lower the in place water content, the higher the C-factor.

Using the back-calculation procedure developed by Hellrung (2015) as described in Appendix B, a similar correlation study was performed to determine the back-calculated to laboratory modulus adjustment factor (C-factor). The study concluded that a C-factor of 0.645 based on the M_r test sequence No.15 and lower subgrade layer yields the best correlation. Detailed description of the study can be found in Ng et al. (2016).

Table 15 In-Situ Laboratory Resilient Modulus and R-Value Data for WYDOT Sections.

WYDOT ID	Soil Class	K1	K2	K3	σ_1 (psi)	$\sigma_2 = \sigma_3$ (psi)	σ_b (psi)	τ_{Oct} (psi)	Calc. M_r (psi)	R-Value
0107-A	A-6	1801	0.489	-0.479	5.628	2.507	10.641	2.548	20931	14
0107-B	A-4	620	0.435	0.249	4.879	2.112	9.102	2.260	7668	47
0107-C	A-2-4	601	0.386	0.302	5.042	2.186	9.414	2.332	7777	19
0300-A	A-6	606	0.262	0.802	5.525	2.173	9.870	2.737	9194	18
0300-B	A-4	1250	0.491	-0.300	5.354	2.217	9.787	2.561	14341	43
0300-C	A-6	625	0.276	0.530	5.503	2.175	9.852	2.718	8993	10
0601-A	A-2-4	1192	0.511	-1.048	5.926	2.556	11.038	2.752	12638	67
0601-B	A-2-4	1413	0.613	-1.499	5.422	2.321	10.063	2.532	12970	61
0601-C	A-6	932	0.122	0.802	6.177	2.545	11.266	2.966	15365	18
0N21-A	A-1-B	1473	0.649	-1.714	5.500	2.333	10.166	2.585	12904	73
0N21-B	A-6	994	0.280	-0.020	5.638	2.349	10.335	2.685	13200	12
0N21-C	A-6	755	0.402	0.171	5.536	2.322	10.179	2.624	9841	12
0N23-A	A-1-B	1637	0.691	-1.571	5.506	2.189	9.883	2.709	14019	79
0N23-B	A-1-B	1457	0.616	-1.433	5.452	2.201	9.854	2.654	13192	75
0N23-C	A-2-4	1713	0.527	-0.892	5.260	2.067	9.394	2.607	17196	59
0N30-A	A-6	621	0.305	0.672	5.620	2.050	9.719	2.915	9083	14
0N30-B	A-1-A	1545	0.626	-0.527	5.500	2.028	9.556	2.835	15799	65
0N30-C	A-4	1091	0.363	-0.026	5.446	1.883	9.212	2.909	13466	35
0N34-A	A-2-4	1143	0.624	-1.485	5.432	2.401	10.233	2.475	10634	74
0N34-B	A-4	1182	0.491	-0.379	5.687	2.526	10.738	2.581	14001	47
0N34-C	A-4	864	0.514	-0.293	5.678	2.491	10.659	2.603	10260	26
0N37-A	A-1-B	1537	0.601	-1.382	5.759	2.268	10.295	2.850	14274	76
0N37-B	A-1-B	1745	0.609	-0.871	5.722	2.274	10.269	2.816	17697	72
0N37-C	A-1-B	1432	0.548	-0.836	5.955	2.269	10.492	3.010	14977	75
0P11-A	A-6	1020	0.313	0.118	5.426	2.198	9.821	2.636	13479	10
0P11-B	A-7-6	728	0.248	0.329	5.251	2.107	9.464	2.567	10112	12
0P11-C	A-7-6	779	0.259	0.217	5.264	2.245	9.753	2.465	10644	15
1401-A	A-7-6	624	0.212	0.485	5.024	2.097	9.218	2.390	8937	13
1401-B	A-7-6	332	0.224	0.991	4.863	1.846	8.554	2.464	5035	11
1401-C	A-7-6	333	0.400	0.574	5.317	2.025	9.367	2.688	4506	13

Table 16 Laboratory-to-Field Resilient Modulus Correction Factor (C-Factors)

WYDOT ID	Soil Class	R-Value	In-situ Lab M_r (psi)	Back-calculated M_r (psi)	C-Factor
0107-A	A-6	14	20931	37000	0.566
0107-B	A-4	47	7668	31000	0.247
0107-C	A-2-4	19	7777	16000	0.486
0300-A	A-6	18	9194	19000	0.484
0300-B	A-4	43	14341	16000	0.896
0300-C	A-6	10	8993	17000	0.529
0601-A	A-2-4	67	12638	41000	0.308
0601-B	A-2-4	61	12970	45000	0.288
0601-C	A-6	18	15365	29000	0.530
0N21-A	A-1-B	73	12904	28000	0.461
0N21-B	A-6	12	13200	21000	0.629
0N21-C	A-6	12	9841	26000	0.379
0N23-A	A-1-B	79	14019	38000	0.369
0N23-B	A-1-B	75	13192	34000	0.388
0N23-C	A-2-4	59	17196	38000	0.453
0N30-A	A-6	14	9083	25000	0.363
0N30-B	A-1-A	65	15799	35000	0.451
0N30-C	A-4	35	13466	27000	0.499
0N34-A	A-2-4	74	10634	16000	0.665
0N34-B	A-4	47	14001	19000	0.737
0N34-C	A-4	26	10260	28000	0.366
0N37-A	A-1-B	76	14274	39000	0.366
0N37-B	A-1-B	72	17697	33000	0.536
0N37-C	A-1-B	75	14977	43000	0.348
0P11-A	A-6	10	13479	16000	0.842
0P11-B	A-7-6	12	10112	21000	0.482
0P11-C	A-7-6	15	10644	13000	0.819
1401-A	A-7-6	13	8937	12000	0.745
1401-B	A-7-6	11	5035	19000	0.265
1401-C	A-7-6	13	4506	29000	0.155

6.2.3 Development of resilient modulus and R-value relationship

A correlation study was performed using R statistical software to develop a relationship between resilient modulus and R-value listed in Table 15. Two models given by Equations (14) and (15) were determined, and their residual standard errors were calculated for comparison. The residual standard errors resulting from both models are about equal.

$$\mathrm{M_r(psi)} = 9713.91 + 61.56\mathrm{R}; \quad \text{Error = 3,347} \tag{14}$$

$$\mathrm{M_r(psi)} = 6644\mathrm{R}^{0.1748}; \quad \text{Error = 3,335} \tag{15}$$

An independent study was conducted by Hutson (2015) to estimate resilient modulus primarily from the R-value. The outcomes of this analysis concluded that M_r value cannot be solely related to R-value. Hence, multi-regression analysis was performed to include other significant soil parameters and stresses in the development of resilient modulus and R-value relationship. Appendix C.2 describes the multi-regression study and the development of two models.

6.2.4 Development of design tables for resilient modulus

To facilitate the implementation of MEPDG Level 2 design through an efficient design process, M_r design tables were developed by Henrichs (2015) to allow for a continuous input of representative M_r values based on a designed pavement structure. Using the M_r data summarized in Table 6 based on σ_c of 2 psi for all sites, two M_r design tables were developed for pavement structures consisting of flexible pavement, crushed base and two subgrade groups in terms of R-values. Two distinct trends of M_r values were observed in Figure 10 as a function of deviator stress for subgrade soils having R-values > 50 and R-values ≤ 50.

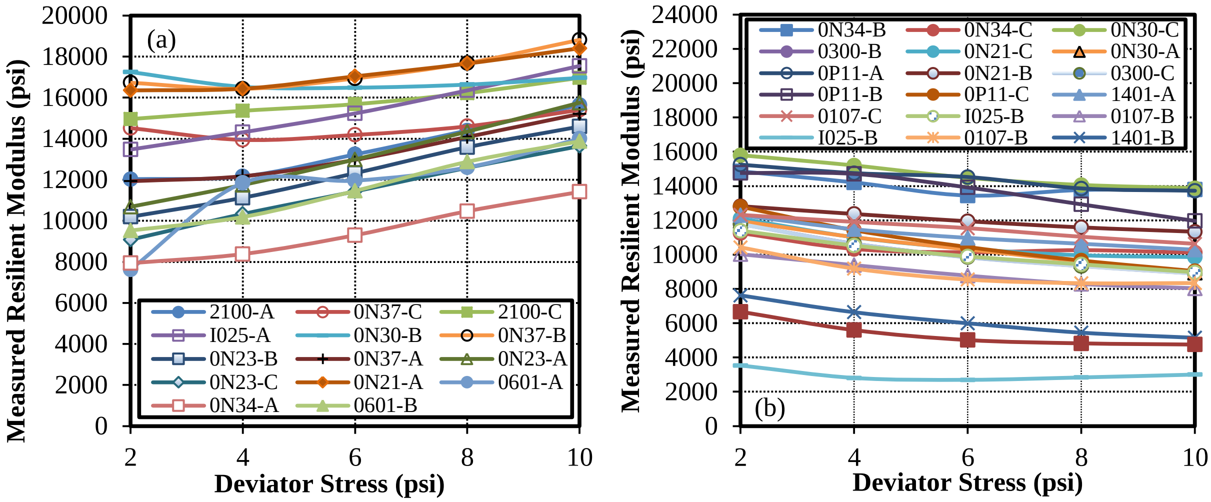

Figure 10 Resilient moduli for subgrade soils having (a) R > 50, and (b) R ≤ 50.

Correlation studies between M_r values and soil properties conducted by Henrichs (2015) discovered that M_r values of subgrade soils having R>50 were best correlated with $\gamma_{d\text{-max}}$ while M_r values of subgrade soils having R≤50 were best correlated with ω_{opt}. To avoid the enormous effort of estimating stresses on top of a subgrade layer, design tables were developed in terms of typical asphalt thicknesses from 4 to 8 inches and base thicknesses from 6 to 18 inches. The M_r design tables for soils with R>50 and R≤50 are presented in Table 17 and Table 18, respectively.

6.3 Summary of Level 3 Unbound Base/Subgrade Material Properties

6.3.1 Overview of M-E design input requirements (Level 3)

According to the M-E design procedure, the pavement layer structure is defined by specifying the type of material used in each layer. The properties of the pavement layer materials are either entered by the user based on available information or assumed as default values available within the MEPDG material database, depending on the level of design. For Level 3 design, the material type must be entered by the user as the primary input. The user is provided with the option to either change the gradation and other engineering properties or accept the MEPDG default values corresponding to the selected material type from database files within the MEPDG.

Table 17 Design table of resilient modulus for subgrade soils (R>50).

Base Thickness (in)	**Estimated Resilient Modulus (psi) for R > 50**														
	Asphalt Thickness (inch) and Corresponding Maximum Dry Unit Weight (pcf)														
	4			5			6			7			8		
	γ=129.6	γ=123.9	γ=119.6	γ=129.6	γ=123.9	γ=119.6	γ=129.6	γ=123.9	γ=119.6	γ=129.6	γ=123.9	γ=119.6	γ=129.6	γ=123.9	γ=119.6
6	16592	15443	14615	15938	14392	13231	15508	13718	12365	-	-	-	-	-	-
8	16095	14641	13556	15621	13894	12589	15299	13397	11958	15051	13018	11483	-	-	-
10	15745	14089	12839	15372	13509	12099	15120	13122	11613	14919	12819	11236	14766	12590	10953
12	15455	13637	12262	15173	13204	11716	14967	12891	11326	14813	12661	11041	14691	12478	10817
14	15222	13279	11809	15013	12960	11411	14841	12702	11092	14721	12523	10871	14617	12368	10682
16	-	-	-	14881	12762	11166	14741	12553	10908	14647	12412	10736	14568	12295	10593
18	-	-	-	-	-	-	14669	12445	10776	14582	12317	10620	14470	12150	10417

*γ−Maximum Dry Unit Weight.

Table 18 Design table of resilient modulus for subgrade soils (R≤50).

Base Thickness (in)	**Estimated Resilient Modulus (psi) for R ≤ 50**														
	Asphalt Thickness (inch) and Corresponding Optimum Moisture Content (percent)														
	4			5			6			7			8		
	ω=11.5	ω=16.9	ω=22.6	ω=11.5	ω=16.9	ω=22.6	ω=11.5	ω=16.9	ω=22.6	ω=11.5	ω=16.9	ω=22.6	ω=11.5	ω=16.9	ω=22.6
6	12471	9332	6218	12832	9904	6710	13084	10313	7067	-	-	-	-	-	-
8	12743	9761	6586	13017	10203	6970	13210	10521	7250	13365	10780	7478	-	-	-
10	12944	10083	6866	13166	10448	7185	13322	10707	7414	13449	10921	7604	13547	11088	7754
12	13116	10365	7112	13288	10651	7364	13418	10869	7558	13517	11036	7707	13585	11172	7829
14	13258	10600	7320	13389	10820	7514	13499	11006	7680	13577	11138	7799	13635	11256	7904
16	-	-	-	13473	10962	7641	13564	11116	7779	13626	11222	7874	13668	11312	7955
18	-	-	-	-	-	-	13611	11197	7851	13668	11295	7940	13734	11426	8057

*ω−Optimum Moisture Content.

6.3.2 Typical WYDOT unbound base and subgrade material properties

Subgrade soils in the State of Wyoming are classified by WYDOT according to the AASHTO soil classification system (AASHTO M 145). No additional criteria are specified by WYDOT for properties of natural subgrade soils. Gradation requirements and aggregate properties for unbound base and subbase materials are provided in WYDOT Standard Specifications for Roads and Structures (WYDOT 2010). Table 19 shows the WYDOT specifications for MEPDG-related aggregate properties of crushed base and subbase materials.

Table 19 WYDOT Specifications for Crushed Base and Subbase Properties.

Properties	Crushed Base	Subbase
Liquid limit, maximum (percent)	30	25
Plasticity index	0 to 3	0 to 6
R-Value, minimum	75	60

6.3.3 Recommended unbound base and subgrade material properties

Material properties of unbound base and subgrade soils were summarized according to material type. A summary of the field and laboratory-derived input level 3 unbound base and subgrade material properties is presented in this section. The testing conducted as part of this study included the following subgrade types for both WYDOT and LTPP test sections: A-1-a, A-1-b, A-2-4, A-2-7, A-4, A-6 and A-7-6. Absolute percentage difference between WYDOT and LTPP section averages is also shown. The column entitled 'Current Level 3' shows the values currently used in the MEPDG as Level 3 defaults.

<u>Optimum Moisture Content</u>

The summary of the subgrade soils optimum moisture content is provided in Table 20. On a project by project basis, there are differences between the WYDOT and LTPP optimum water content for the same type of soil. However, the overall average optimum water content for a soil type measured for the WYDOT test sites is similar to the optimum water contents included in the MEPDG for the same type of soil.

<u>Maximum Dry Unit Weight</u>

The summary of the subgrade soils maximum dry unit weight is provided in Table 21. Maximum dry unit weights for WYDOT and LTPP sections are in reasonable agreement with each other

<u>R-Value</u>

The summary of the subgrade soils R-values is provided in Table 22. Since the absolute percentage difference between R-values of WYDOT test section subgrades and LTPP subgrades is not large, the calculated average R-values for each soil type are reasonable for use in design.

<u>Laboratory Resilient Modulus</u>

The summary of the subgrade soils laboratory-derived resilient modulus is provided in Table 23. Resilient modulus values for soil types encountered in both WYDOT and LTPP test sections are different from each other for A-2-4, A-4 and A-7-6 subgrade soils. The average resilient modulus obtained from this study is lower than those included in the MEPDG.

Table 20 Summary of optimum moisture content (percent).

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average	Current Level 3
A-1-a	Mean	6.4	8.9	39.1	7.7	7.35
	Std. Dev					
	Minimum					
	Maximum					
A-1-b	Mean	6.7		—	6.7	9.1
	Std. Dev	0.9				
	Minimum	5.2				
	Maximum	8.2				
A-2-4	Mean	11.3	12.5	10.6	11.9	8.95
	Std. Dev	5.2				
	Minimum	6.6				
	Maximum	21.1				
A-2-7	Mean		18.4	—	18.4	10.6
	Std. Dev					
	Minimum					
	Maximum					
A-4	Mean	14.1	14.0	0.7	14.1	11.8
	Std. Dev	4.6	1.2			
	Minimum	10.9	13.1			
	Maximum	23.2	14.8			
A-6	Mean	15.3	14.3	6.5	14.8	17.1
	Std. Dev	2.5	1.5			
	Minimum	11.2	12.8			
	Maximum	21.1	15.7			
A-7-6	Mean	20.9		—	20.9	22.2
	Std. Dev	4.4				
	Minimum	17.0				
	Maximum	28.4				

Table 21 Summary of maximum dry unit weight (pcf).

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average
A-1-a	Mean	129.1	119.0	7.8	124.1
	Std. Dev				
	Minimum				
	Maximum				
A-1-b	Mean	127.7		—	127.7
	Std. Dev	3.1			
	Minimum	120.5			
	Maximum	132.9			
A-2-4	Mean	111.6	120.0	7.5	115.8
	Std. Dev	9.4			
	Minimum	100.3			
	Maximum	123.1			
A-2-7	Mean		107.0	—	107.0
	Std. Dev				
	Minimum				
	Maximum				
A-4	Mean	113.6	119.0	4.8	116.3
	Std. Dev	10.3			
	Minimum	93.5			
	Maximum	120.1			
A-6	Mean	112.5	111.3	1.1	111.9
	Std. Dev	4.3	4.0		
	Minimum	106.0	107.0		
	Maximum	121.1	115.0		
A-7-6	Mean	98.9		—	98.9
	Std. Dev	6.1			
	Minimum	90.4			
	Maximum	105.9			

Table 22 Summary of R-values.

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average
A-1-a	Mean	65	78	20	72
	Std. Dev				
	Minimum				
	Maximum				
A-1-b	Mean	73	64	12.3	69
	Std. Dev	8			
	Minimum	55			
	Maximum	86			
A-2-4	Mean	56	69	23.2	63
	Std. Dev	19	7		
	Minimum	19	60		
	Maximum	74	77		
A-2-7	Mean		50	—	50
	Std. Dev				
	Minimum				
	Maximum				
A-4	Mean	40		—	40
	Std. Dev	8			
	Minimum	26			
	Maximum	47			
A-6	Mean	14		—	14
	Std. Dev	4			
	Minimum	10			
	Maximum	22			
A-7-6	Mean	13		—	13
	Std. Dev	1			
	Minimum	11			
	Maximum	15			

Table 23 Summary of laboratory-derived resilient modulus (psi).

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average	Current Level 3
A-1-a	Mean	21800		—	21,800	18,000
	Std. Dev					
	Minimum					
	Maximum					
A-1-b	Mean	19000	9300	51.1	14,100	18,000
	Std. Dev	2500				
	Minimum	15000				
	Maximum	22700				
A-2-4	Mean	13900	7200	48.2	10,500	16,500
	Std. Dev	4100	1500			
	Minimum	10300	5800			
	Maximum	21400	9300			
A-2-7	Mean		11600	—	11,600	16,000
	Std. Dev					
	Minimum					
	Maximum					
A-4	Mean	14400	10600	26.4	12,500	15,000
	Std. Dev	2400	2800			
	Minimum	10500	8300			
	Maximum	17200	13700			
A-6	Mean	12200	10700	12.3	11,400	14,000
	Std. Dev	5300	2400			
	Minimum	3600	7900			
	Maximum	23200	12400			
A-7-6	Mean	9800	14900	52.0	12,300	13,000
	Std. Dev	1800	1800			
	Minimum	7200	13500			
	Maximum	12300	16200			

<u>Liquid Limit</u>
The summary of subgrade soils liquid limit is provided in Table 24. A higher absolute percentage difference was observed between the average liquid limit for WYDOT and LTPP sections for A-1-a and A-2-4 soils. Values calculated for A-1 and A-2 soils were higher than those currently used as default values in the MEPDG.

<u>Plasticity Index</u>
The summary of the subgrade soils plasticity index is provided in Table 25. Plasticity index values determined in this study are similar to the MEPDG Level 3 default for all soils tested except for A-2-7, which is based only on data from one LTPP section. It is therefore reasonable to use the existing default value for this soil.

In-Situ Moisture Content

The summary of the subgrade soils in-situ moisture content is provided in Table 26. In-situ moisture content for WYDOT and LTPP sections are in poor agreement with each other. The average in-situ moisture content for WYDOT sections is generally lower than for the LTPP sites, except for the A-6 soils.

In-Situ Unit Weight

The summary of the subgrade soils in-situ unit weight (pcf) is provided in Table 27. In-situ unit weights for WYDOT and LTPP sections are in excellent agreement with each other except for A-6 soils, where the average in-situ unit weight for WYDOT sections is higher than that for LTPP sections.

Table 24 Summary of liquid limit.

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average	Current Level 3
A-1-a	Mean	21.0	17.0	19.0	19.0	6
	Std. Dev					
	Minimum					
	Maximum					
A-1-b	Mean	20.8		—	20.8	11
	Std. Dev	0.8				
	Minimum	20.0				
	Maximum	22.0				
A-2-4	Mean	27.5	23.5	14.5	25.5	14
	Std. Dev	7.5	3.5			
	Minimum	20.0	20.0			
	Maximum	35.0	27.0			
A-2-7	Mean		45.0	—	45.0	50
	Std. Dev					
	Minimum					
	Maximum					
A-4	Mean	23.2	24.5	5.6	23.9	21
	Std. Dev	5.7	0.7			
	Minimum	17.0	24.0			
	Maximum	33.0	25.0			
A-6	Mean	34.6	31.0	10.4	32.8	33
	Std. Dev	3.4	7.6			
	Minimum	27.0	20.0			
	Maximum	40.0	37.0			
A-7-6	Mean	50.2		—	50.2	51
	Std. Dev	5.4				
	Minimum	43.0				
	Maximum	57.0				

Table 25 Summary of plasticity index.

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average	Current Level 3
A-1-a	Mean	1.0		—	1.0	1
	Std. Dev					
	Minimum					
	Maximum					
A-1-b	Mean	1.0		—	1.0	1
	Std. Dev	0.0				
	Minimum	1.0				
	Maximum	1.0				
A-2-4	Mean	6.0	7.0	16.7	6.5	2
	Std. Dev	2.0	0.0			
	Minimum	4.0	7.0			
	Maximum	8.0	7.0			
A-2-7	Mean		19.0	—	19.0	29
	Std. Dev					
	Minimum					
	Maximum					
A-4	Mean	6.0	7.0	16.7	6.5	5
	Std. Dev	5.0	1.4			
	Minimum	2.0	6.0			
	Maximum	15.0	8.0			
A-6	Mean	16.4	15.8	3.7	16.1	16
	Std. Dev	3.6	3.3			
	Minimum	8.0	12.0			
	Maximum	21.0	20.0			
A-7-6	Mean	31.0		—	31.0	30
	Std. Dev	7.2				
	Minimum	22.0				
	Maximum	40.0				

Table 26 Summary of in-situ moisture contents (percent).

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average
A-1-a	Mean			—	—
	Std. Dev				
	Minimum				
	Maximum				
A-1-b	Mean	13.4		—	13.4
	Std. Dev	0.0			
	Minimum	13.4			
	Maximum	13.4			
A-2-4	Mean	12.1	14.2	17.4	13.2
	Std. Dev	5.9			
	Minimum	6.2	12.9		
	Maximum	21.2	15.5		
A-2-7	Mean		19.5	—	19.5
	Std. Dev				
	Minimum				
	Maximum				
A-4	Mean	9.2	16.0	73.9	12.6
	Std. Dev	0.4	0.0		
	Minimum	8.8	14.8		
	Maximum	9.8	17.2		
A-6	Mean	20.8	16.0	23.1	18.4
	Std. Dev	3.1	0.0		
	Minimum	15.2	13.8		
	Maximum	23.8	17.7		
A-7-6	Mean	27.3		—	27.3
	Std. Dev	5.2			
	Minimum	19.9			
	Maximum	36.1			

Table 27 Summary of in-situ unit weights (pcf).

AASHTO Soil Class		WYDOT Sections	LTPP Sections	Percent Difference	Average	Current Level 3
A-1-a	Mean		115.0	—	115.0	127.2
	Std. Dev					
	Minimum					
	Maximum					
A-1-b	Mean	131.6		—	131.6	123.7
	Std. Dev	1.1				
	Minimum	130.5				
	Maximum	132.6				
A-2-4	Mean	121.3	113.5	6.4	117.4	124.0
	Std. Dev	11.7	10.5			
	Minimum	106.6	103.0			
	Maximum	139.3	124.0			
A-2-7	Mean		106.0	—	106.0	120.8
	Std. Dev					
	Minimum					
	Maximum					
A-4	Mean	121.8	114.5	6.0	118.2	118.4
	Std. Dev	11.4	2.1			
	Minimum	102.4	113.0			
	Maximum	130.4	116.0			
A-6	Mean	131.5	111.8	15.0	121.7	107.8
	Std. Dev	3.7	2.5			
	Minimum	126.9	108.0			
	Maximum	137.7	113.0			
A-7-6	Mean	120.7		—	120.7	97.7
	Std. Dev	7.9				
	Minimum	107.2				
	Maximum	127.2				

CHAPTER 7 – TRIAL PAVEMENT DESIGN

7.1 Introduction

This chapter describes the pavement designs and compares the design outcomes for a range of new and rehabilitated flexible pavement projects. Seven trial designs provided by WYDOT were evaluated using the inputs recommended for use in the WYDOT User Guide (ARA 2012) and the local calibration coefficients that were updated and documented in Byattacharya et al. (2015). The performance of each trial design was evaluated by comparing the predicted reliability with the target reliability criterion. Also, adjusted pavement designs to achieve the target reliability criterion were developed for comparison. These pavement design examples serve to 1) facilitate the full implementation of MEPDG in Wyoming, 2) verify pavement design outcomes obtained from local calibration effort, and 3) provide a basis for revising some of the design guidelines included in the WYDOT 2012 MEPDG User Guide, and in the current WYDOT design and construction manuals (WYDOT 2010). Detailed description of the trial pavement designs can be found in ARA (2016).

7.2 Summary of Design Projects

Seven pavement designs, including new and rehabilitated flexible pavement designs, were evaluated using a target reliability level. The projects or examples are briefly discussed in the following sections:

(1) Section 7.3-Project No. 1: 0803137 ISO RECON – New Flexible Pavement Design;
(2) Section 7.4-Project No. 2: 0803137 Mainline – Rehabilitation of a Semi-Rigid Pavement;
(3) Section 7.5-Project No. 3: N132104 Slide Version – New Flexible Pavement Design;
(4) Section 7.6-Project No. 4: N132104 Rehabilitation – Rehabilitation Design of a Flexible Pavement;
(5) Section 7.7-Project No. 5: P114037 Bridge Ends – New Flexible Pavement Design;
(6) Section 7.8-Project No. 6: Wamsutter Rehabilitation – Rehabilitation Design of a Flexible Pavement; and
(7) Section 7.9-Project No. 7: Wamsutter Reconstruct – New Flexible Pavement Design.

All trial designs (referred as the baseline design) were provided by WYDOT. The baseline designs were evaluated with Pavement ME Design version 2.2.6. The adjusted designs to achieve the target reliability level were compared to that baseline design for each project.

7.3 Project No.1: 0803137 Iso Recon; New Flexible Pavement Design

7.3.1 Baseline design

This project represents a new flexible pavement design on a heavily travelled roadway. The roadway was assumed to be an interstate based on the threshold values used for the baseline design. The general inputs and pavement structure evaluated by WYDOT is included in Figure 11. The 4-inch wearing surface is a polymer modified PG76-28 mixture, while the 6-inch asphalt concrete base layer is a neat PG64-22 asphalt mixture. The new flexible pavement design includes a thick (16 inches) crushed gravel aggregate base layer and 24 inches of a select fill classified as an AASHTO A-1-a soil. The results from the baseline design in terms of predicted distress are provided in Figure 12. The design outcome shows that the baseline design strategy failed with a reliability of 6.9 percent. Transverse cracks, permanent deformation, and bottom-up fatigue (alligator) cracking all exceeded their threshold values.

Design Inputs

Design Life:	20 years	Base construction:	May, 2014	Climate Data Sources	41.806, -107.2
Design Type:	Flexible Pavement	Pavement construction:	June, 2014		
		Traffic opening:	September, 2014		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	4.0
Flexible	Default asphalt concrete	6.0
NonStabilized	Crushed gravel	16.0
Subgrade	A-1-a	24.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	7,545
2024 (10 years)	14,075,300
2034 (20 years)	31,871,000

Figure 11 Baseline design strategy and structure for Project No. 1.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	139.30	95.00	97.95	Pass
Permanent deformation - total pavement (in)	0.50	0.90	95.00	6.93	Fail
AC bottom-up fatigue cracking (% lane area)	10.00	11.99	95.00	46.53	Fail
AC thermal cracking (ft/mile)	1000.00	2547.90	95.00	20.22	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	333.97	95.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.55	95.00	88.88	Fail

Distress Charts

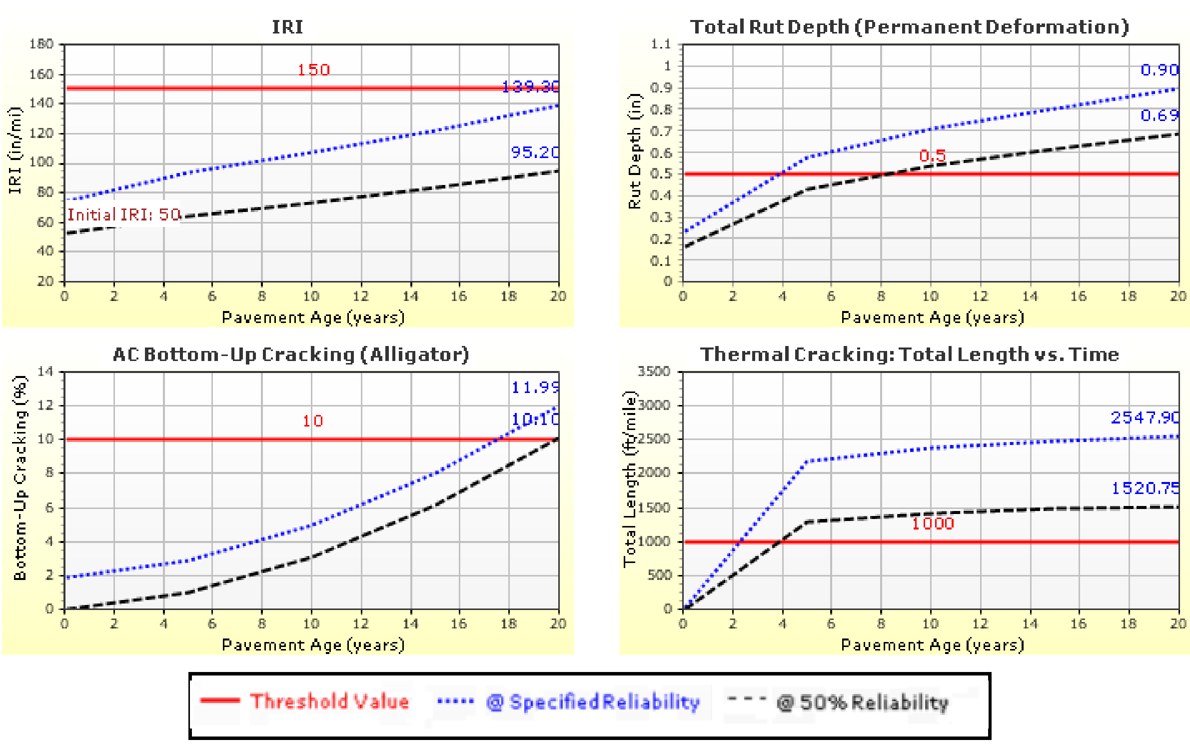

Figure 12 Predicted distress and reliability baseline design for Project No. 1 .

The concerns with this baseline design are briefly highlighted as follows:

(1) A very low resilient modulus of 3,864 psi was used for A-7-6 subgrade soil.
(2) A resilient modulus of 11,592 psi was used for the 24-inch A-1-a subgrade soil. This resilient modulus represents a factor of 3 above the resilient modulus of the A-7-6 subgrade soil.
(3) The local calibration coefficients used in this design were not the values recommended in Byattacharya et al. (2015).
(4) The terminal International Roughness Index (IRI) used in this design was 150 in/mi while a terminal IRI value of 170 in/mi is recommended in the 2012 WYDOT User Manual.

7.3.2 Adjusted design

Figure 13 shows the adjusted design strategy and pavement structure, and Figure 14 shows the corresponding predicted distress and reliability. The following adjustments were made to the baseline design discussed in Section 7.3.1 to achieve the target reliability criterion:

(1) A 9-inch lime stabilized A-7-6 subgrade layer was included and its representative resilient modulus of 11,500 psi was used.
(2) The local calibration coefficients recommended in Byattacharya et al. (2015) were used.
(3) The 16-inch crushed gravel layer was separated into two 8-inch layers. The resilient moduli of the lower and upper layers were 23,000 psi and 37,000 psi, respectively.
(4) A 2-inch stone mastic asphalt (SMA) layer with higher polymer modified asphalt (PG76-34) was used as the wearing surface while maintaining the 4-in asphalt thickness in the baseline design with a 2-inch binder layer using PG76-34.
(5) The thickness of the asphalt concrete base layer was increased to 7.5 inches to reduce the area of bottom-up fatigue cracking below the threshold value.
(6) The target terminal IRI was revised to 170 inches per mile for Interstates and primary arterials with heavy truck traffic.

Figure 14 shows that the reliability of the design strategy exceeds 95 percent or the target reliability level for all distresses except for transverse cracking. Even with the SMA wearing surface with a PG76-34, the predicted length of transverse cracks (1,164 ft/mi) still exceeds the threshold value of 1,000 ft/mi.

Design Inputs

Design Life:	20 years	Base construction:	May, 2014	Climate Data Sources	41.806, -107.2
Design Type:	Flexible Pavement	Pavement construction:	June, 2014		
		Traffic opening:	September, 2014		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	7.5
NonStabilized	Crushed gravel	8.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-1-a	12.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.5
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2014 (initial)	7,545
2024 (10 years)	14,075,300
2034 (20 years)	31,871,000

Figure 13 Adjusted design strategy and structure for Project No. 1.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	170.00	133.66	95.00	99.89	Pass
Permanent deformation - total pavement (in)	0.50	0.47	95.00	97.80	Pass
AC bottom-up fatigue cracking (% lane area)	10.00	9.01	95.00	99.40	Pass
AC thermal cracking (ft/mile)	1000.00	1164.08	95.00	86.06	Fail
AC top-down fatigue cracking (ft/mile)	5000.00	329.52	95.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.24	95.00	100.00	Pass

Distress Charts

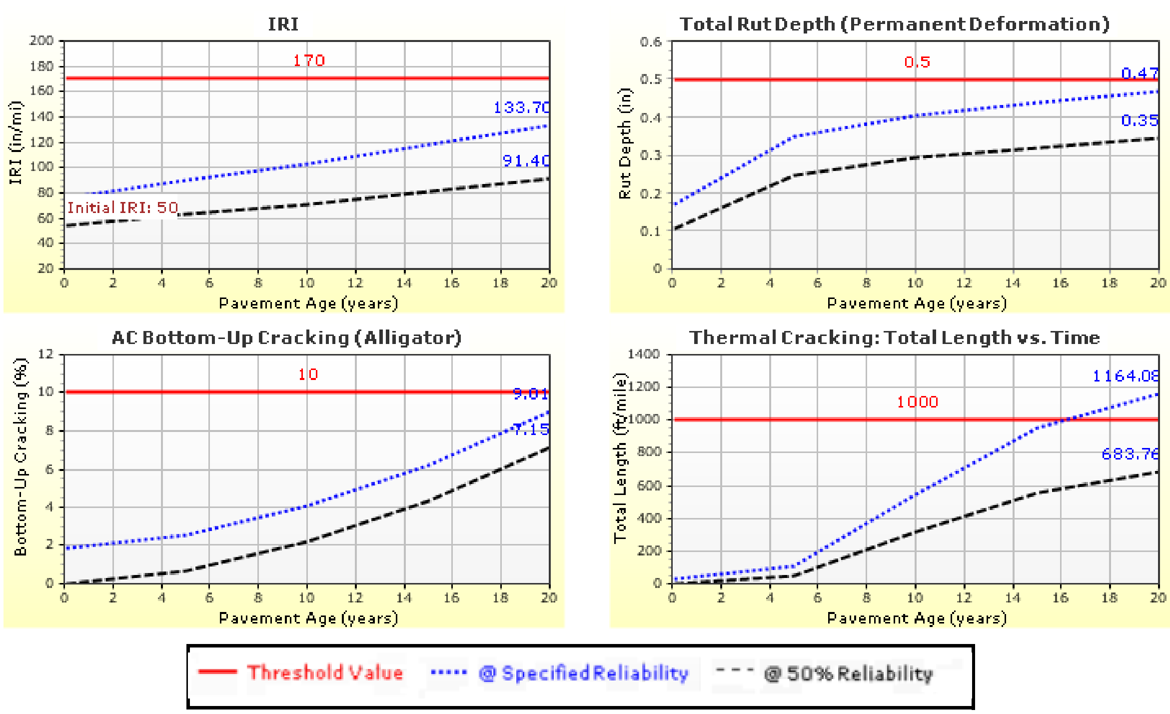

Figure 14 Predicted distress and reliability of adjusted design for Project No. 1.

7.3.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) The default resilient modulus for the different soils should be reviewed and verified or confirmed.

(2) The threshold value for transverse cracks for high traffic roadways (interstates and primary arterials) may need to be increased.

(3) If the higher resilient modulus of the A-7-6 soil (12,000 psi at the optimum water content) and the PG 76-34 asphalt are used, the HMA thickness can be reduced to 10.5 inches and the crushed aggregate base can be reduced to 12 inches (ARA 2016).

7.4 Project No. 2: 0803137; Mainline, Rehabilitation of Semi-Rigid Pavement

7.4.1 Baseline design

Project No. 2 represents a semi-rigid pavement rehabilitation design for a heavily travelled roadway. The truck traffic values used in this project were the same as for Project No. 1. The general inputs and pavement structure evaluated by WYDOT are summarized in Figure 15. The 4-inch asphalt concrete overlay is a polymer modified PG76-28 mixture, while the existing 7.5 inch asphalt concrete layer is a neat PG64-22 asphalt mixture. The existing semi-rigid pavement structure includes a 6-inch cement treated or stabilized aggregate base over a 10-inch crushed gravel aggregate base layer. The results from the baseline rehabilitation design in terms of predicted distress are provided in Figure 16. The baseline design strategy failed because the predicted length of transverse cracks (1,197 ft/mi) exceeds the threshold value of 1,000 ft/mi. The reliability for all other distresses exceeded the target reliability of 85 percent.

The concerns with this baseline design are briefly highlighted as follows:

(1) The target reliability used was 85 percent which was lower than 95 percent recommended in the 2012 WYDOT User Guide.
(2) The threshold total rut depth used was 1.0 in which was higher than the 0.5 inches recommended in in the 2012 WYDOT User Guide.
(3) A total bottom-up fatigue cracking plus reflective fatigue cracks threshold value of 25 percent was used. However, this threshold value should be the same as for the bottom-up fatigue cracking of 10 percent.
(4) A total transverse cracking plus reflective transverse cracks threshold value of 2,500 ft/mi was used. This threshold value should be the same as for thermal cracking of 1000 ft/mi.
(5) A 28-day flexural strength of 650 psi used for the CTB layer was extremely high for an elastic modulus of 3,100,000 psi. A more realistic value would be 450 psi. In addition, a minimum value for the elastic modulus of the CTB layer of 100,000 psi was used while the WYDOT 2012 User Guide suggests a value of 50,000 psi.
(6) The resilient modulus of 5,810 psi for the A-6 soil in accordance with the WYDOT 2012 User Guide was low. More importantly, the default maximum dry density and optimum water content were used.
(7) The local calibration coefficients used in this design were not the values recommended in Byattacharya et al. (2015).
(8) The terminal International Roughness Index (IRI) used in this design was 150 in/mi while a terminal IRI value of 170 in/mi is recommended in the 2012 WYDOT User Manual.

Design Inputs

Design Life:	10 years	Base construction:	May, 2013	Climate Data Sources	41.806, -107.2
Design Type:	AC over Semi-Rigid	Pavement construction:	June, 2013		
		Traffic opening:	September, 2013		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.0
Flexible (existing)	Default asphalt concrete	7.5
Cement_Base	Soil cement	6.0
NonStabilized	River-run gravel	10.0
Subgrade	A-6	24.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2013 (initial)	7,545
2018 (5 years)	6,572,610
2023 (10 years)	14,075,300

Figure 15 Baseline design strategy and structure for Project No. 2.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	92.97	85.00	99.99	Pass
Permanent deformation - total pavement (in)	1.00	0.53	85.00	100.00	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	25.00	1.35	85.00	100.00	Pass
AC total transverse cracking: thermal + reflective (ft/mile)	2500.00	1917.84	85.00	97.57	Pass
AC bottom-up fatigue cracking (% lane area)	10.00	0.01	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	1197.46	50.00	0.00	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	208.03	85.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.25	85.00	100.00	Pass
Chemically stabilized layer - fatigue fracture (% lane area)	25.00	0.50	-	-	-

Distress Charts

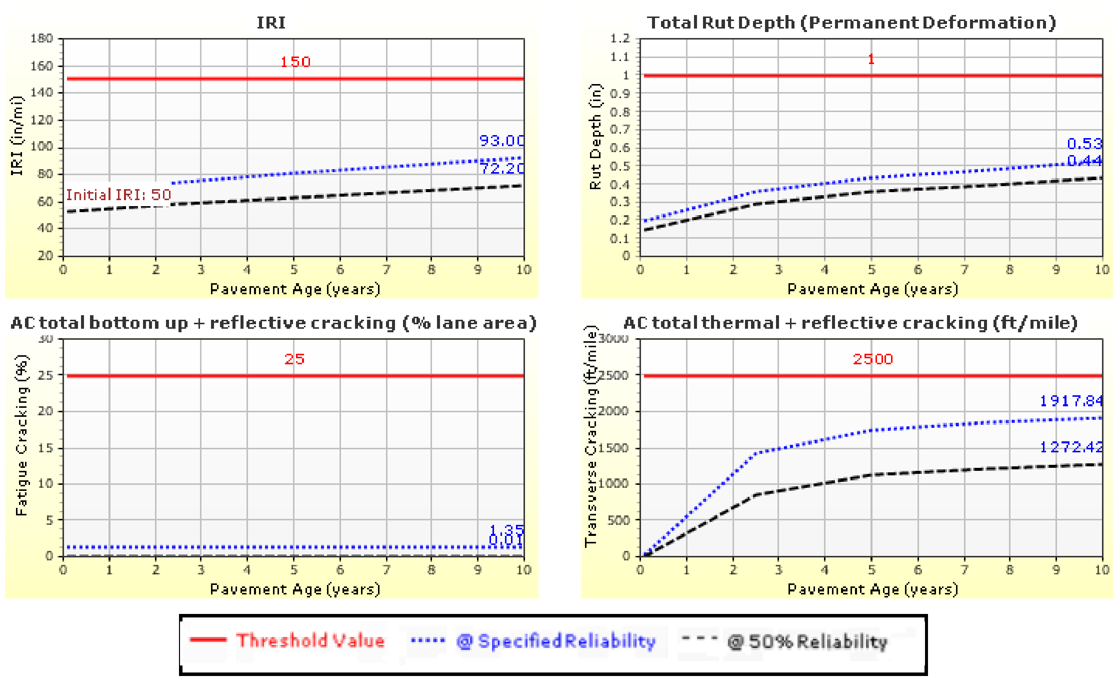

Figure 16 Predicted distress and reliability of baseline design for Project No. 2.

7.4.2 Adjusted design

Figure 17 shows the adjusted design strategy and pavement structure, and Figure 18 shows the corresponding predicted distress and reliability. The following adjustments were made to the baseline design discussed in Section 7.4.1 to achieve the target reliability criterion:

(1) A design resilient modulus of 7,000 psi was used for the A-6 subgrade soil.
(2) The 28-day flexural strength for the CTB layer was reduced to 450 psi, and 50,000 psi was used as the minimum elastic modulus of the CTB layer.
(3) A 2-inch SMA layer with PG76-34 was used as the wearing surface.
(4) The terminal IRI of 170 in/mi was used.

Figure 18 shows that the reliability of the design strategy exceeds 95 percent or the target reliability level for all distresses.

Design Inputs

Design Life:	10 years	Base construction:	May, 2013	Climate Data Sources	41.806, -107.2
Design Type:	AC over Semi-Rigid	Pavement construction:	June, 2013		
		Traffic opening:	September, 2013		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	2.0
Flexible (existing)	Default asphalt concrete	7.5
Cement_Base	Soil cement	6.0
NonStabilized	River-run gravel	10.0
Subgrade	A-6	24.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.5
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2013 (initial)	7,545
2018 (5 years)	6,572,610
2023 (10 years)	14,075,300

Figure 17 Adjusted design strategy and structure for Project No. 2.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	170.00	102.05	95.00	100.00	Pass
Permanent deformation - total pavement (in)	0.50	0.33	95.00	100.00	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	10.00	2.13	95.00	100.00	Pass
AC total transverse cracking: thermal + reflective (ft/mile)	1000.00	963.65	95.00	95.88	Pass
AC bottom-up fatigue cracking (% lane area)	10.00	0.01	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	238.13	50.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	330.01	95.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.17	95.00	100.00	Pass
Chemically stabilized layer - fatigue fracture (% lane area)	25.00	0.49	-	-	-

Distress Charts

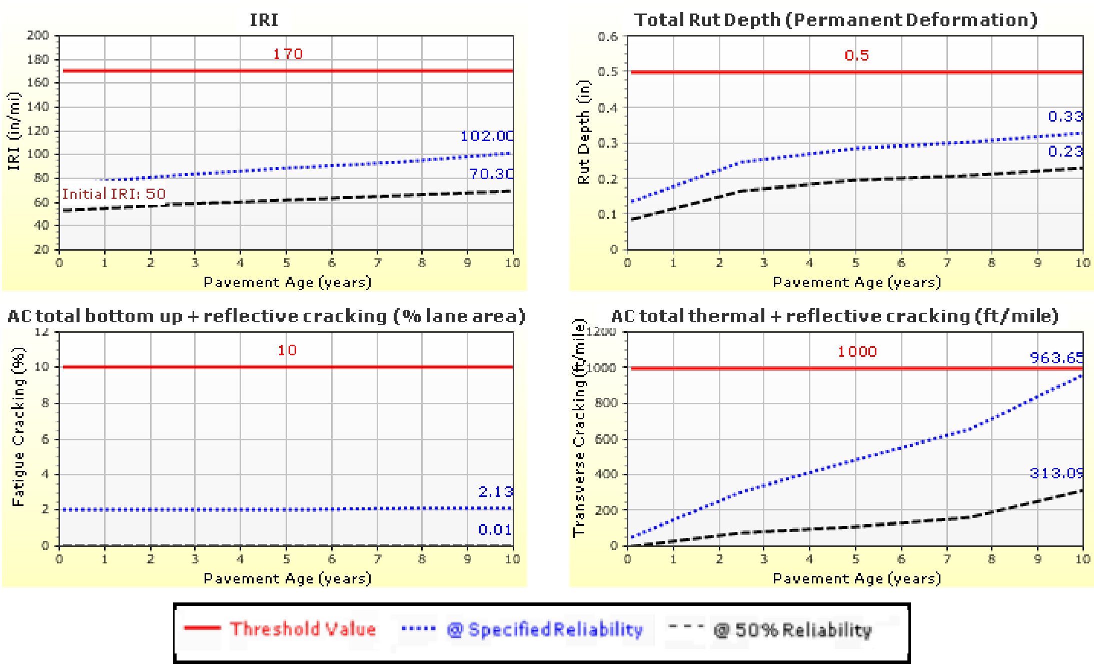

Figure 18 Predicted distress and reliability of adjusted design for Project No. 2.

7.4.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) The default resilient modulus for the different soils should be reviewed and verified or confirmed.

(2) The existing pavement condition is an important input for rehabilitation input level 3 design.

(3) Using the design revisions and adjustments previously discussed, the HMA overlay thickness of 4 inches is still required.

7.5 Project No. 3: N132104, Slide Version, New Flexible Pavement Design

7.5.1 Baseline design

Project No. 3 represents a new flexible pavement design for a low truck volume primary roadway. A primary roadway classification was selected because most of the design criteria used for the baseline design fit this category. However, the truck traffic is low and more representative of a secondary facility (ARA 2012). The general inputs and pavement structure evaluated by WYDOT are included in Figure 19. The 6-inch wearing surface is a polymer modified PG64-28 mixture and the new flexible pavement design includes an 8-inch crushed gravel aggregate base layer. The subgrade soil is an AASHTO A-4 soil classification. The results from the baseline design in terms of predicted distress are provided in Figure 20. The baseline design strategy failed in AC thermal cracking with a reliability of 24 percent. The baseline design failed in both transverse cracks and top-down cracking. The MEPDG 2008 and 2015 Manual of Practice recommend that top-down cracking may not be included in making decisions relative to achieving an acceptable design strategy. Thus, for this example problem and the remainder examples, top-down cracking was not included as a design criterion in comparing and evaluating different design strategies.

The concerns with this baseline design are briefly highlighted as follows:

(1) The target reliability used in the baseline rehabilitation design was 90 percent which was higher than the 80 percent recommended in the 2012 WYDOT User Guide.

(2) The terminal IRI of 200 in/mi was lower than 220 in/mi recommended in the 2012 WYDOT User Guide.

(3) The resilient modulus of 6,085 psi for the 8-inch crushed gravel base layer was low.

Design Inputs

Design Life: 20 years
Design Type: Flexible Pavement
Base construction: May, 2017
Pavement construction: June, 2017
Traffic opening: September, 2017
Climate Data Sources: 42.584, -110.108

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	6.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	152
2027 (10 years)	157,532
2037 (20 years)	356,703

Figure 19 Baseline design strategy and structure for Project No. 3.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	200.00	145.79	90.00	99.90	Pass
Permanent deformation - total pavement (in)	0.75	0.47	90.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	7.61	90.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	3213.25	90.00	23.82	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	2632.07	90.00	82.68	Fail
Permanent deformation - AC only (in)	0.25	0.07	90.00	100.00	Pass

Distress Charts

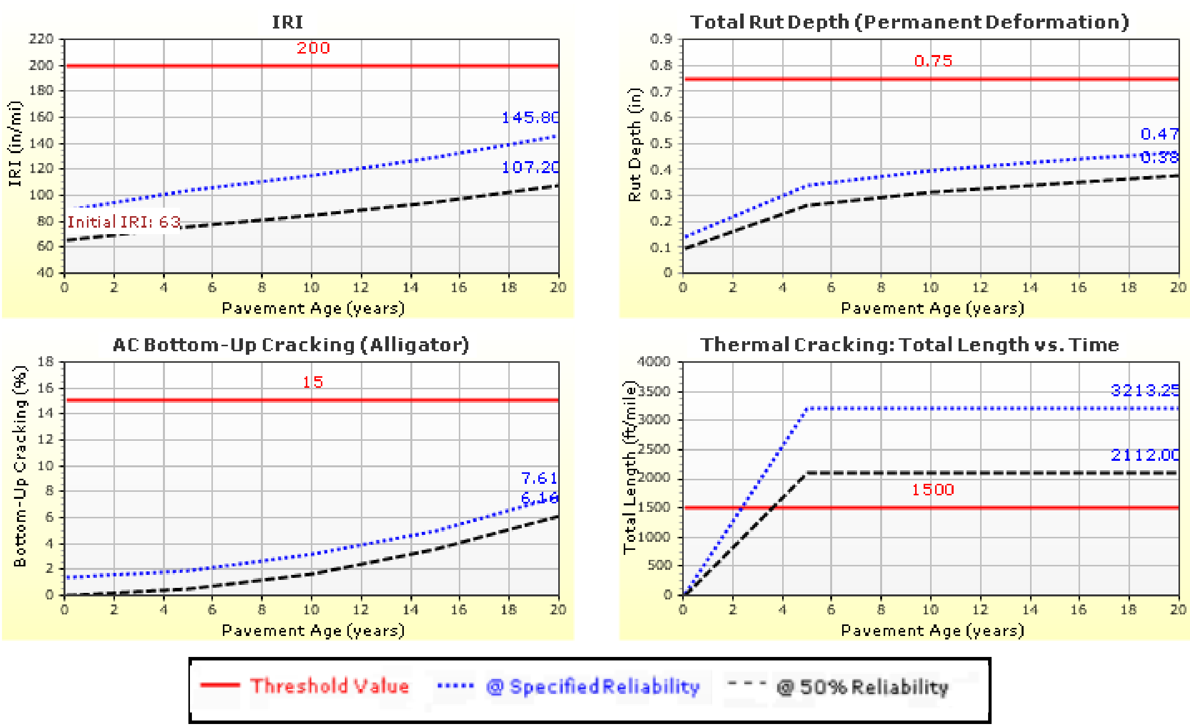

Figure 20 Predicted distress and reliability of baseline design for Project No. 3.

7.5.2 Adjusted design

Figure 21 shows the adjusted design strategy and pavement structure, and Figure 22 shows the corresponding predicted distress and reliability. The following adjustments were made to the baseline design discussed in Section 7.5.1 to achieve the target reliability criterion:

(1) The local calibration coefficients recommended in Byattacharya et al. (2015) were used.
(2) The design resilient modulus for the 8-inch base layer was increased to 15,000 psi.
(3) PG64-34 was used in the wearing surface to reduce the predicted transverse cracking.
(4) The terminal IRI of 220 in/mi was used.

The reliability of the design strategy exceeds the recommended 80 percent or the target reliability level for all distresses except for transverse cracking. Even with a PG68-34 HMA wearing surface, the predicted length of transverse cracks (2,911 ft/mi) still exceeds the threshold value of 1,500 ft/mi resulting in an achieved reliability of 25.6 percent.

7.5.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) The default resilient modulus for the crushed gravel base material should be reviewed.

(2) The threshold value for transverse cracks for low truck volume roadways (primary and secondary facilities) may need to be increased because of the high standard error of the transfer function and large local calibration coefficient of 7.5.

(3) A 4-inch asphalt layer and 8-inch crushed base pavement structure can still achieve the 80 percent reliability level for all distresses providing the threshold value for transverse cracks can be increased. Otherwise, asphalt grade PG64-40 can be considered.

Design Inputs

Design Life:	20 years	Base construction:	May, 2017	Climate Data Sources	42.584, -110.108
Design Type:	Flexible Pavement	Pavement construction:	June, 2017		
		Traffic opening:	September, 2017		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	6.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	152
2027 (10 years)	157,532
2037 (20 years)	356,703

Figure 21 Adjusted design strategy and structure for Project No. 3.

7.6 Project No. 4: N132104, Rehabilitation Design of a Flexible Pavement

7.6.1 Baseline design

Project No. 4 represents a rehabilitation design of the flexible pavement described in Project No. 3. A primary roadway classification was selected because most of the design criteria used for the baseline design fit this category. However, the truck traffic is low and more representative of a secondary facility. The general inputs and pavement structure evaluated by WYDOT are included in Figure 23. The existing 6-inch wearing surface includes AC-20 asphalt. The aggregate base is a 6-inch crushed gravel base layer, and the subgrade soil is an AASHTO A-4 soil. The results from the baseline design in terms of predicted distress are provided in Figure 24. This design failed with a reliability of 0 percent. The design failed in total fatigue cracking (new and reflective fatigue cracks) and total transverse cracks (new and reflective transverse cracks).

The concerns with this baseline design are briefly highlighted as follows:

(1) The design period used was 20 years. However, 10 years were suggested in the 2012 WYDOT Design Guide for all rehabilitation designs.

(2) The target reliability used was 65 percent which was lower than 80 percent recommended in the 2012 WYDOT User Guide.

(3) The terminal IRI of 200 in/mi was lower than 220 in/mi recommended in the 2012 WYDOT User Guide.

(4) The resilient modulus for the 6-inch crushed gravel base layer used was 12,170 psi, while a design resilient modulus of 6,085 psi was selected for the A-4 soil. These values were opposite of those used in the baseline design for Project No. 3. Although the design resilient modulus was selected in accordance with the 2012 WYDOT Design Guide, the modulus of 6,085 psi for the A-4 soil was low.

(5) A structural rating of fair and environmental rating of good was selected to represent the condition of the existing pavement. However, no information was provided regarding the selection of these two categories.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	220.00	117.97	85.00	100.00	Pass
Permanent deformation - total pavement (in)	0.75	0.27	85.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	4.01	85.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	2911.34	85.00	25.57	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	1554.41	85.00	91.11	Pass
Permanent deformation - AC only (in)	0.75	0.08	85.00	100.00	Pass

Distress Charts

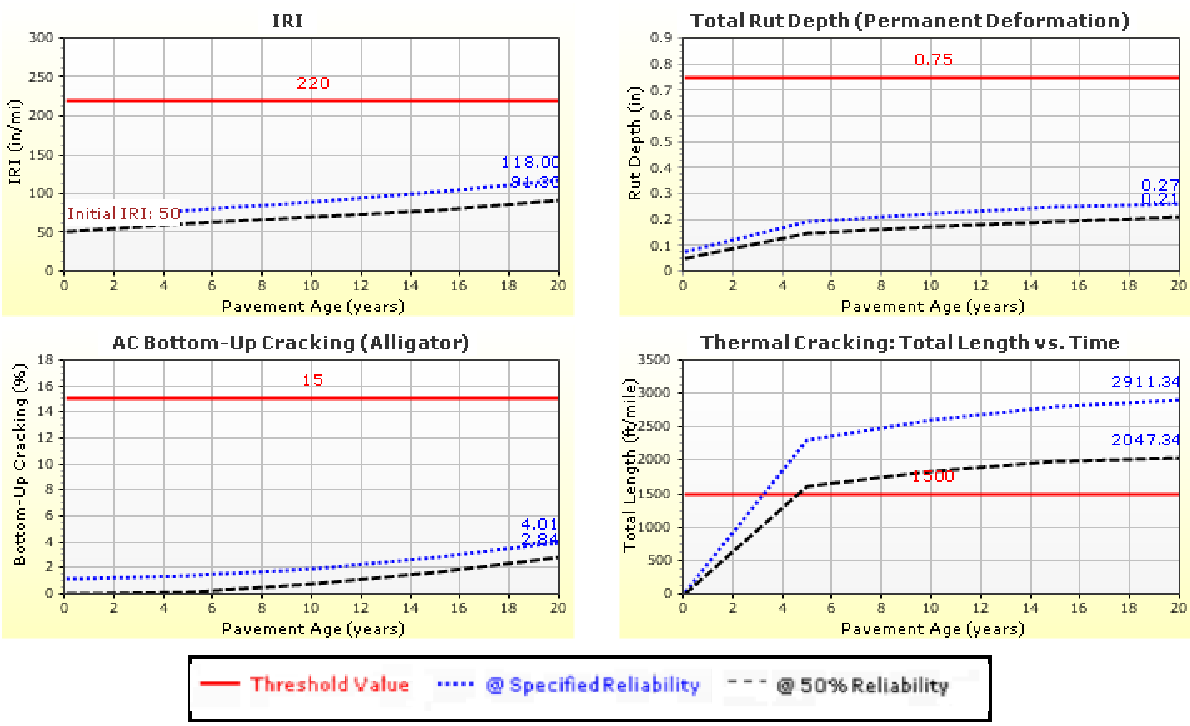

Figure 22 Predicted distress and reliability of adjusted design for Project No. 3.

Design Inputs

Design Life: 20 years
Design Type: AC over AC

Existing construction: August, 1989
Pavement construction: June, 2017
Traffic opening: September, 2017

Climate Data Sources: 42.584, -110.108

Design Structure

Layer type	Material Type	Thickness (in)
Flexible (OL)	Default asphalt concrete	2.0
Flexible (existing)	Default asphalt concrete	3.0
NonStabilized	Crushed gravel	6.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.6
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	152
2027 (10 years)	157,532
2037 (20 years)	356,703

Figure 23 Baseline design strategy and structure for Project No. 4.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	130.13	65.00	83.92	Pass
Permanent deformation - total pavement (in)	0.75	0.66	65.00	90.93	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	15.00	50.53	65.00	0.00	Fail
AC total transverse cracking: thermal + reflective (ft/mile)	1500.00	2469.20	65.00	17.42	Fail
Permanent deformation - AC only (in)	0.25	0.08	65.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	0.00	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	2112.00	50.00	0.00	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	980.38	65.00	82.38	Pass

Distress Charts

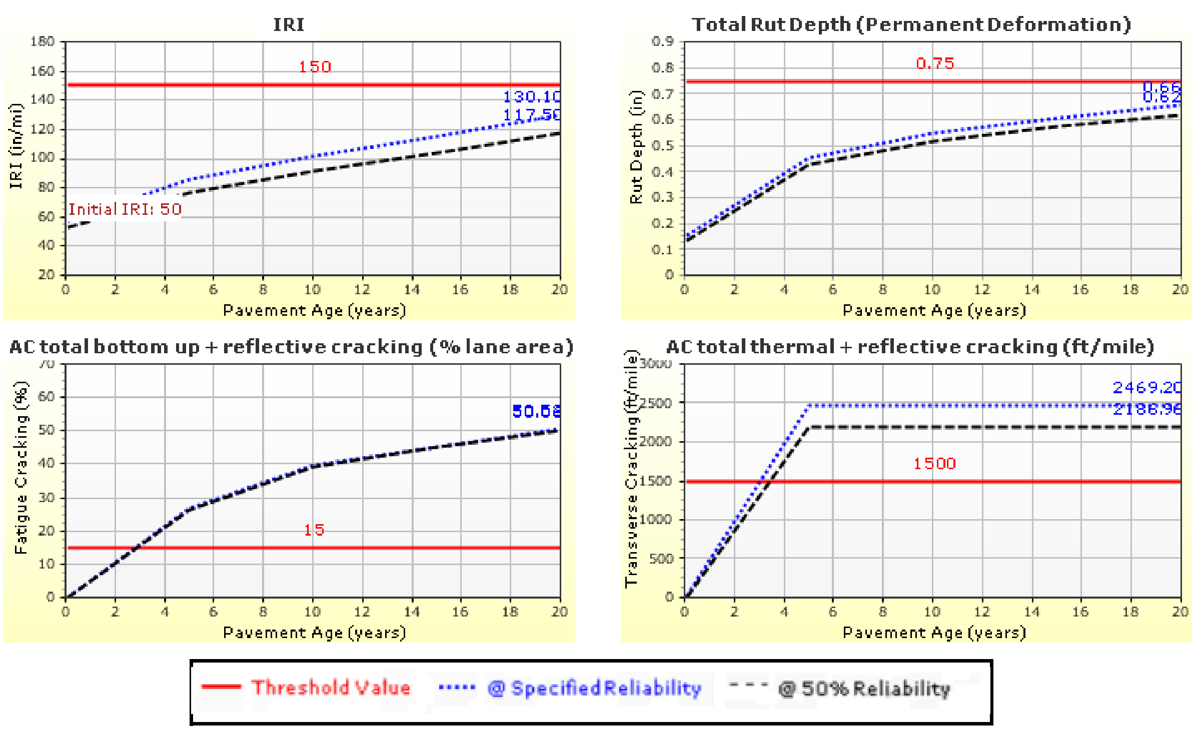

Figure 24 Predicted distress and reliability of baseline design for Project No. 4.

7.6.2 Adjusted design

Figure 25 shows the adjusted design strategy and pavement structure, and Figure 26 shows the corresponding predicted distress and reliability. The following adjustments were made to the baseline design discussed in Section 7.6.1 to achieve the target reliability criterion:

(1) The local calibration coefficients recommended in Byattacharya et al. (2015) were used.
(2) The design life was reduced from 20 years to 10 years.
(3) PG64-34 was used in the wearing surface to reduce transverse cracking.
(4) The terminal IRI of 220 in/mi was used.

Figure 26 shows that the reliability of the design strategy is still 0 percent and controlled by the total area of fatigue cracks. The total length of transverse cracking (2,651 ft/mi) also exceeds the threshold value of 1,500 ft/mi, even if with the PG64-34 asphalt was used as an HMA wearing surface. The reliability of the design based on total transverse cracks is 22 percent.

7.6.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) The condition of the existing pavement is an important input for design. Rehabilitation input level 2 is recommended for all rehabilitation designs.

(2) The default resilient modulus for the crushed gravel base material should be reviewed.

(3) A combination of 3.25-inch asphalt overlay and PG64-40 asphalt will satisfy the reliability level for all distresses.

Design Inputs

Design Life:	10 years	Existing construction:	August, 1989	Climate Data Sources	42.584, -110.108
Design Type:	AC over AC	Pavement construction:	June, 2017		
		Traffic opening:	September, 2017		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible (OL)	Default asphalt concrete	2.0
Flexible (existing)	Default asphalt concrete	3.0
NonStabilized	Crushed gravel	6.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.6
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	152
2022 (5 years)	73,561
2027 (10 years)	157,532

Figure 25 Adjusted design strategy and structure for Project No. 4.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	220.00	115.11	80.00	100.00	Pass
Permanent deformation - total pavement (in)	0.75	0.47	80.00	100.00	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	15.00	36.75	80.00	0.00	Fail
AC total transverse cracking: thermal + reflective (ft/mile)	1500.00	2650.77	80.00	22.34	Fail
Permanent deformation - AC only (in)	0.75	0.18	80.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	0.00	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	1971.42	50.00	0.00	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	1425.19	80.00	88.63	Pass

Distress Charts

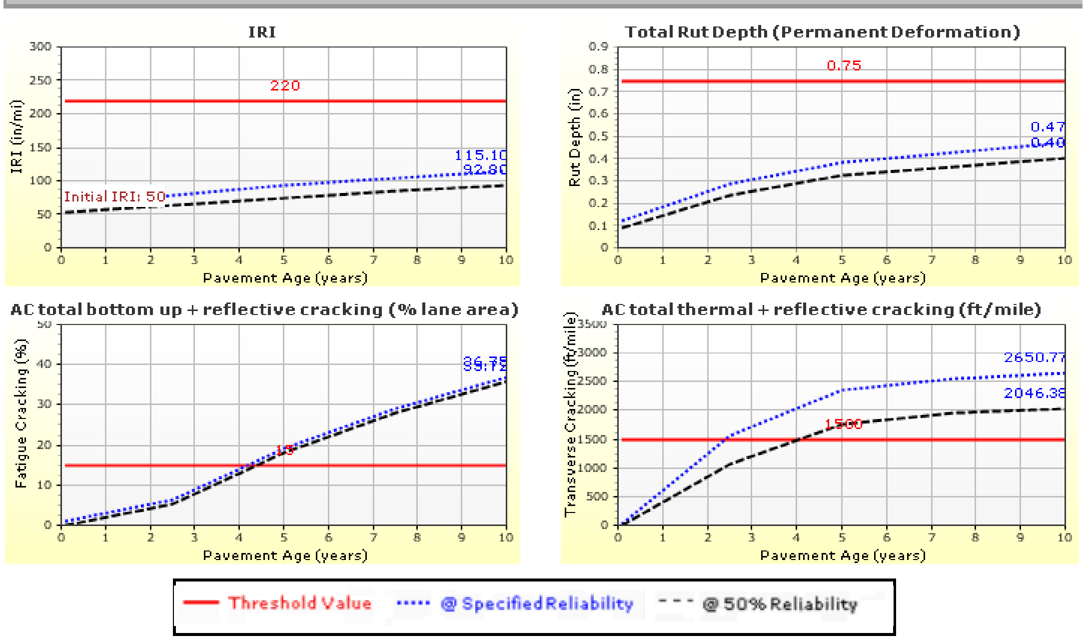

Figure 26 Predicted distress and reliability of adjusted design for Project No. 4.

7.7 Project No. 5: P114037 Bridge Ends, New Flexible Pavement Design

7.7.1 Baseline design

Project No. 5 represents a new flexible pavement design for the ends of a bridge. A primary roadway classification was selected for this roadway because most of the design criteria used for the baseline design fit this category. The general inputs and pavement structure evaluated by WYDOT are shown in Figure 27. The 8-inch asphalt concrete layer is a polymer modified PG64-28 mixture and the new flexible pavement design includes a 12-inch crushed gravel aggregate base layer. The subgrade soil is an AASHTO A-4 soil classification. This pavement structure is considered very thick for the truck traffic. The results from the baseline design are provided in Figure 28. The baseline design strategy passed all design criteria with a reliability of 100 percent.

The concerns with this baseline design are briefly highlighted as follows:

(1) The terminal IRI of 150 in/mi was lower than 220 in/mi recommended in the 2012 WYDOT User Guide.
(2) The threshold value of 3,500 ft/mi for thermal or transverse cracking was much higher than 1,500 in/mi recommended in the 2012 WYDOT Design Guide.
(3) The design resilient modulus of 6,085 psi for the A-4 soil was low.

Design Inputs

Design Life:	20 years	Base construction:	May, 2015	Climate Data Sources	42.584, -110.108
Design Type:	Flexible Pavement	Pavement construction:	June, 2015		
		Traffic opening:	September, 2015		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	8.0
NonStabilized	Crushed gravel	12.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2015 (initial)	190
2025 (10 years)	393,831
2035 (20 years)	891,758

Figure 27 Baseline design strategy and structure for Project No. 5.

7.7.2 Adjusted design

Figure 29 shows the adjusted design strategy and pavement structure, and Figure 30 shows the corresponding predicted distress and reliability. Considering the concerns described in previous Section 7.7.1, the following adjustments are briefly described:

(1) The local calibration coefficients recommended in Byattacharya et al. (2015) were used.
(2) The design resilient modulus for the 12-inch base layer was increased to 15,000 psi.
(3) The terminal IRI of 220 in/mi was used.
(4) The threshold value for thermal cracking was revised to 1,500 ft/mi.

Figure 30 shows that the reliability of the design strategy exceeds 80 percent or the target reliability level for all distresses, except for transverse cracks. The reliability of the design strategy is 24 percent and controlled by the total length of transverse cracking (2,835 ft/mi).

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	116.01	80.00	98.32	Pass
Permanent deformation - total pavement (in)	0.75	0.51	80.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	3.09	80.00	100.00	Pass
AC thermal cracking (ft/mile)	3500.00	2835.21	80.00	94.69	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	202.54	80.00	100.00	Pass
Permanent deformation - AC only (in)	0.75	0.07	80.00	100.00	Pass

Distress Charts

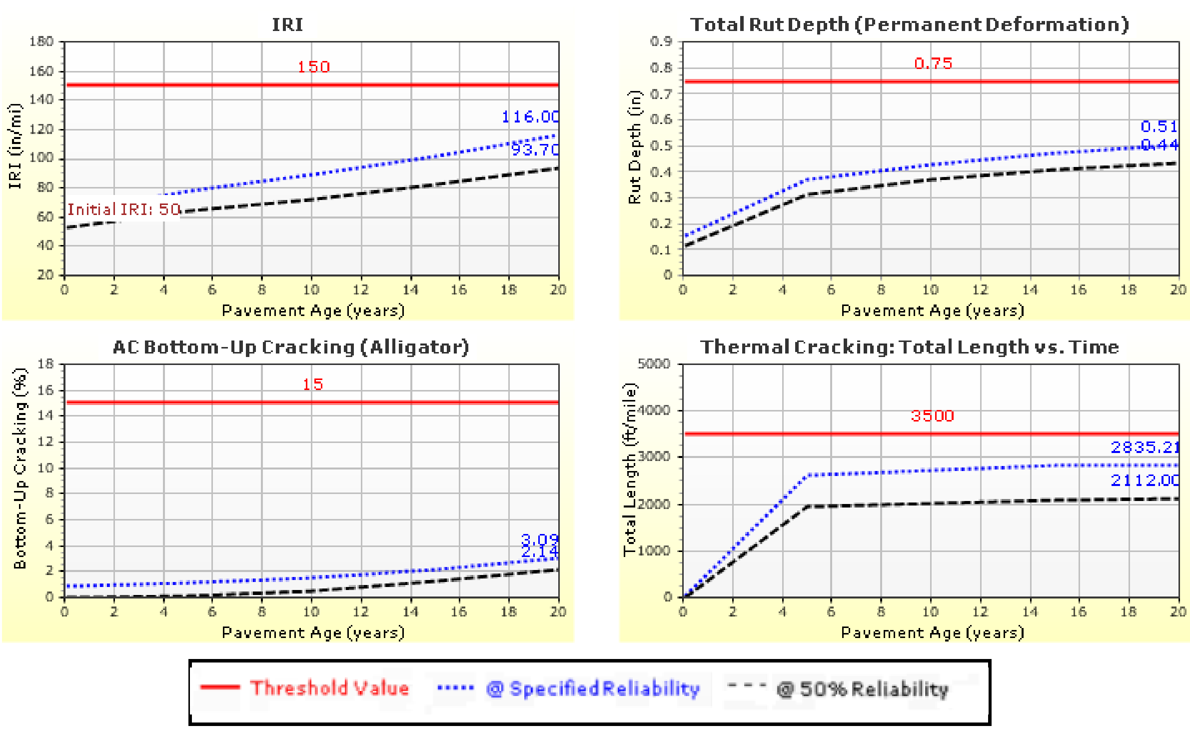

Figure 28 Predicted distress and reliability of baseline design for Project No. 5.

7.7.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) Changing the top 2-inch wearing asphalt surface with PG64-40 achieved the 80 percent reliability.

(2) The pavement design can be optimized with a 2-inch SMA wearing surface and 4-inch asphalt concrete base layer.

(3) Similar recommendations described in aforementioned trial design examples were suggested.

Design Inputs

Design Life:	20 years	Base construction:	May, 2015	Climate Data Sources	42.584, -110.108
Design Type:	Flexible Pavement	Pavement construction:	June, 2015		
		Traffic opening:	September, 2015		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	8.0
NonStabilized	Crushed gravel	12.0
Subgrade	A-4	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2015 (initial)	190
2025 (10 years)	393,831
2035 (20 years)	891,758

Figure 29 Adjusted design strategy and structure for Project No. 5.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	220.00	114.80	80.00	100.00	Pass
Permanent deformation - total pavement (in)	0.75	0.31	80.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	2.61	80.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	2835.21	80.00	23.82	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	189.92	80.00	100.00	Pass
Permanent deformation - AC only (in)	0.75	0.07	80.00	100.00	Pass

Distress Charts

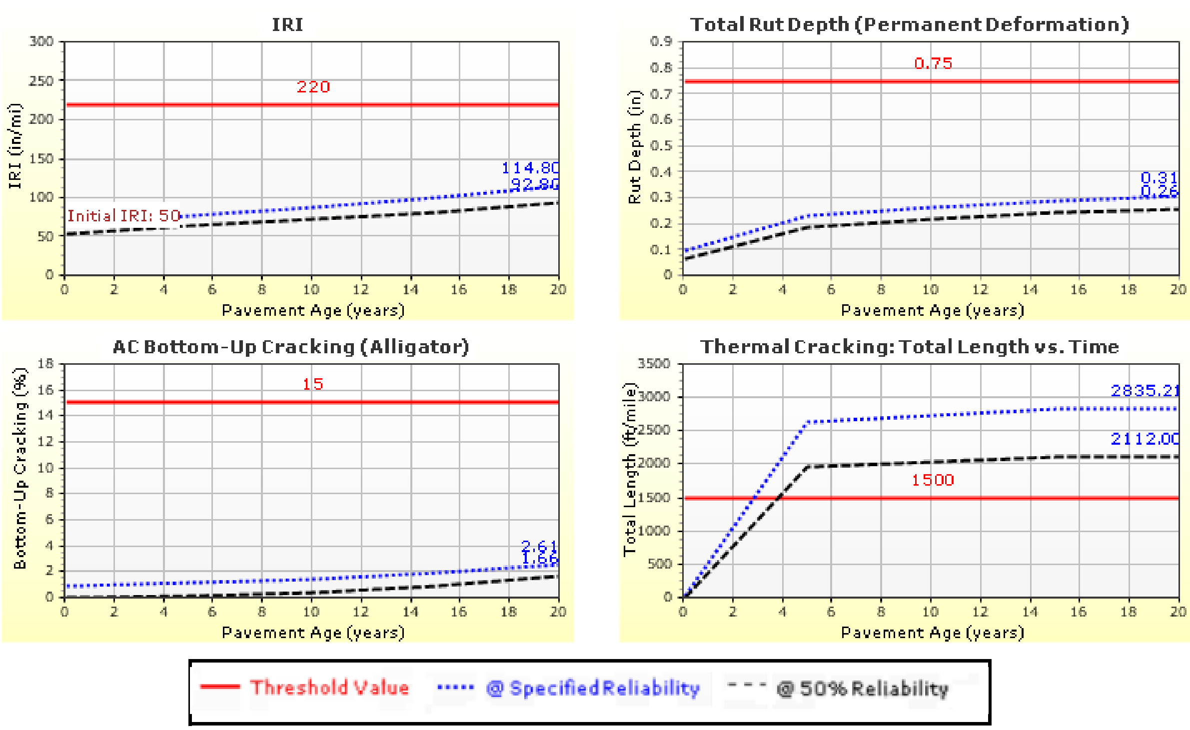

Figure 30 Predicted distress and reliability of adjusted design for Project No. 5.

7.8 Project No. 6: Wamsutter Rehabilitation – Rehabilitation Design of a Flexible Pavement

7.8.1 Baseline design

Project No. 6 represents a rehabilitation design of a flexible pavement for a primary roadway with heavy truck traffic. The general inputs and pavement structure evaluated by WYDOT are included in Figure 31. The existing 6-inch wearing surface includes PG76-28 asphalt. The aggregate base is a 16-inch crushed gravel base layer, and the subgrade soil is an AASHTO A-6 soil classification. The results from the baseline rehabilitation design in terms of predicted distress are provided in Figure 32. The baseline rehabilitation design strategy failed with a reliability of 0 percent. The baseline rehabilitation design failed in total fatigue cracking (new and reflective fatigue cracks) and total transverse cracks (new and reflective transverse cracks).

The concerns with this baseline design are briefly highlighted as follows:

(1) The design period used was 20 years. However, 10 years were suggested in the 2012 WYDOT Design Guide for all rehabilitation designs.
(2) The terminal IRI used in this design was 150 in/mi which was lower than 170 in/mi recommended in the 2012 WYDOT User Manual.
(3) A design resilient modulus of 35,610 psi used for the A-6 subgrade and crushed gravel base layer was extremely high.
(4) A structural rating of fair and environmental rating of good was selected to represent the condition of the existing pavement. However, no information was provided regarding the selection of these two categories.

Design Inputs

Design Life:	20 years	Existing construction:	August, 2008	Climate Data Sources	41.806, -107.2
Design Type:	AC over AC	Pavement construction:	June, 2017		
		Traffic opening:	September, 2017		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible (OL)	Default asphalt concrete	6.0
Flexible (existing)	Default asphalt concrete	8.0
NonStabilized	Crushed gravel	16.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	5,012
2027 (10 years)	9,349,960
2037 (20 years)	21,171,300

Figure 31 Baseline design strategy and structure for Project No. 6

7.8.2 Adjusted design

Figure 33 shows the adjusted design strategy and pavement structure, and Figure 34 shows the corresponding predicted distress and reliability. Considering the concerns described in previous Section 7.8.1, the following adjustments are briefly described:

(1) The local calibration coefficients recommended in Byattacharya et al. (2015) were used.
(2) The design life was reduced from 20 years to 10 years.
(3) A 2-inch SMA wearing surface with a PG76-34 asphalt was used for the overlay.
(4) The crushed gravel base layer was separated into two 8-inch layers. The resilient moduli used for the lower and upper layers were 15,000 psi and 25,000 psi, respectively.
(5) The design resilient modulus for A-6 subgrade soils was reduced to 7,500 psi.
(6) The threshold values were adjusted as shown in Figure 34.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	132.77	85.00	94.80	Pass
Permanent deformation - total pavement (in)	0.50	0.31	85.00	100.00	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	25.00	36.23	85.00	0.00	Fail
AC total transverse cracking: thermal + reflective (ft/mile)	1000.00	2051.99	85.00	27.14	Fail
Permanent deformation - AC only (in)	0.50	0.20	85.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	25.00	0.01	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	1314.26	50.00	0.00	Fail
AC top-down fatigue cracking (ft/mile)	2500.00	263.51	85.00	100.00	Pass

Distress Charts

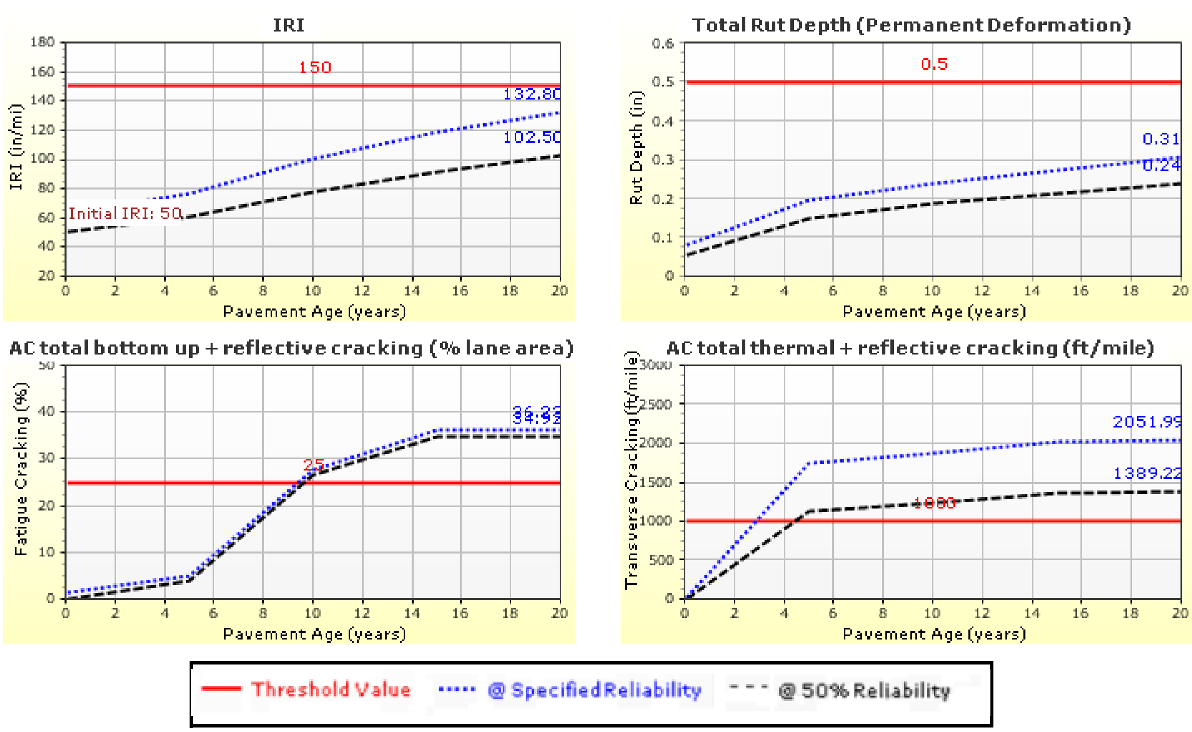

Figure 32 Predicted distress and reliability of baseline design for Project No. 6.

7.8.3 Summary of recommendations

Comparing the outcomes of this trial example, the following recommendations are suggested:

(1) A simple overlay or increasing the overlay from 6 to 8 inches was insufficient for the condition of the existing pavement combined with the higher truck volumes to satisfy the threshold values. Either reconstruction or cold in place recycling are other options that should be considered.

(2) Rehabilitation input level 2 should be used.

(3) Similar recommendations described in aforementioned trial design examples were suggested.

Design Inputs

Design Life:	10 years	Existing construction:	August, 2008	Climate Data Sources	41.806, -107.2
Design Type:	AC over AC	Pavement construction:	June, 2017		
		Traffic opening:	September, 2017		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible (OL)	Default asphalt concrete	2.0
Flexible (OL)	Default asphalt concrete	4.0
Flexible (existing)	Default asphalt concrete	8.0
NonStabilized	Crushed gravel	8.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.7
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	5,012
2022 (5 years)	4,366,060
2027 (10 years)	9,349,960

Figure 33 Adjusted design strategy and structure for Project No. 6.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	170.00	107.04	85.00	99.99	Pass
Permanent deformation - total pavement (in)	0.50	0.23	85.00	100.00	Pass
AC total fatigue cracking: bottom up + reflective (% lane area)	15.00	36.30	85.00	0.00	Fail
AC total transverse cracking: thermal + reflective (ft/mile)	1500.00	805.63	85.00	99.64	Pass
Permanent deformation - AC only (in)	0.50	0.08	85.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	0.00	50.00	100.00	Pass
AC thermal cracking (ft/mile)	1500.00	295.48	50.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	2500.00	207.46	85.00	100.00	Pass

Distress Charts

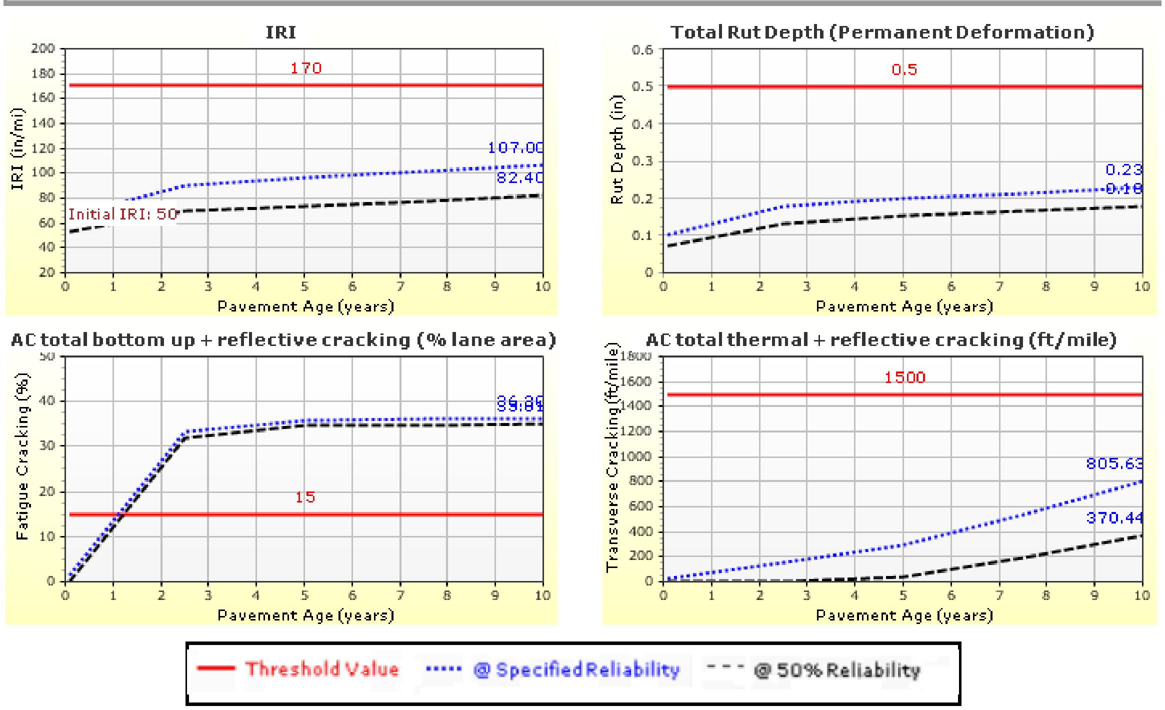

Figure 34 Predicted distress and reliability of adjusted design for Project No. 6.

7.9 Project No. 7: Wamsutter, Reconstruct; New Flexible Pavement Design

7.9.1 Baseline design

Project No. 7 represents a new flexible pavement design or reconstruction of Project No. 6. The general inputs and pavement structure evaluated by WYDOT are included in Figure 35. The 9-inch asphalt concrete layer is a polymer modified PG76-28 mixture and the new flexible pavement design includes a 16-inch crushed gravel aggregate base layer. The subgrade soil is an AASHTO A-6 soil classification. The results from the baseline design in terms of predicted distress are provided in Figure 36. The baseline design strategy failed at 19 percent reliability, and AC transverse cracks and total rut depth exceeded their threshold values. The concerns were similar to those described in aforementioned trial examples.

7.9.2 Adjusted design

The same design revisions or adjustments were made to the baseline design as for some of the other example problems. The design resilient modulus of the A-6 subgrade and crushed gravel base layer used for Project No. 6 were also used for Project No. 7. Figure 37 shows the adjusted pavement structure while Figure 38 shows the predicted distresses for the adjusted design strategy. The adjusted design achieved 85 percent reliability, but with an increase in HMA thickness to 10 inches (2-inch wearing surface and 8-inch asphalt concrete base layer) because of the lower resilient modulus for the crushed gravel and A-6 soil.

7.9.3 Summary of recommendations

Recommendations from this example have already been addressed in the other example problems. A thicker HMA pavement is needed for the heavier truck traffic and softer or weaker soils and crushed gravel base materials. The modulus of the unbound layers should be reviewed more closely and possibly revised in the 2012 WYDOT MEPDG User Guide.

Design Inputs

Design Life:	20 years	Base construction:	May, 2016	Climate Data Sources	41.806, -107.2
Design Type:	Flexible Pavement	Pavement construction:	June, 2017		
		Traffic opening:	September, 2017		

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	9.0
NonStabilized	Crushed gravel	8.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.2
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	5,012
2027 (10 years)	9,349,960
2037 (20 years)	21,171,300

Figure 35 Baseline design strategy and structure for Project No. 7.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	150.00	138.21	95.00	98.16	Pass
Permanent deformation - total pavement (in)	0.50	0.59	95.00	76.72	Fail
AC bottom-up fatigue cracking (% lane area)	10.00	6.97	95.00	100.00	Pass
AC thermal cracking (ft/mile)	1000.00	2629.64	95.00	18.80	Fail
AC top-down fatigue cracking (ft/mile)	2000.00	2507.31	95.00	90.34	Fail
Permanent deformation - AC only (in)	0.50	0.47	95.00	97.48	Pass

Distress Charts

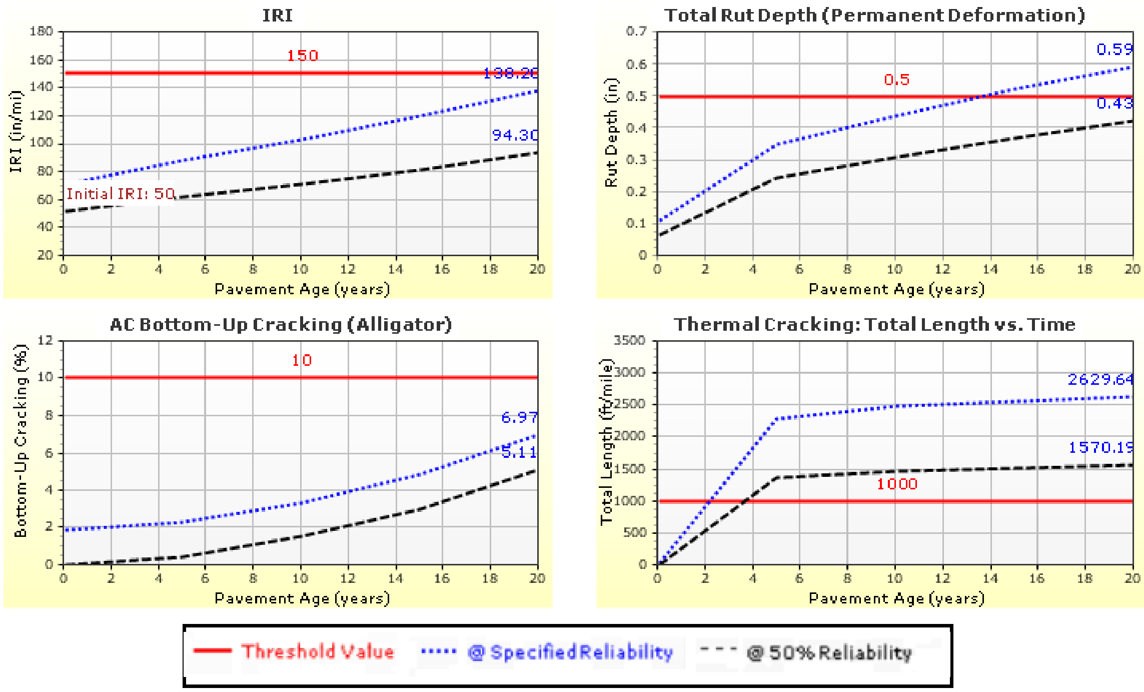

Figure 36 Predicted distress and reliability of baseline design for Project No. 7.

Design Inputs

Design Life: 20 years
Design Type: Flexible Pavement

Base construction: May, 2016
Pavement construction: June, 2017
Traffic opening: September, 2017

Climate Data Sources: 41.806, -107.2

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	2.0
Flexible	Default asphalt concrete	8.0
NonStabilized	Crushed gravel	8.0
NonStabilized	Crushed gravel	8.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.7
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2017 (initial)	5,012
2027 (10 years)	9,349,960
2037 (20 years)	21,171,300

Figure 37 Adjusted design strategy and structure for Project No. 7.

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	170.00	127.62	85.00	99.49	Pass
Permanent deformation - total pavement (in)	0.50	0.37	85.00	99.81	Pass
AC bottom-up fatigue cracking (% lane area)	15.00	14.89	85.00	87.22	Pass
AC thermal cracking (ft/mile)	1500.00	1092.24	85.00	98.94	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	222.04	85.00	100.00	Pass
Permanent deformation - AC only (in)	0.50	0.18	85.00	100.00	Pass

Distress Charts

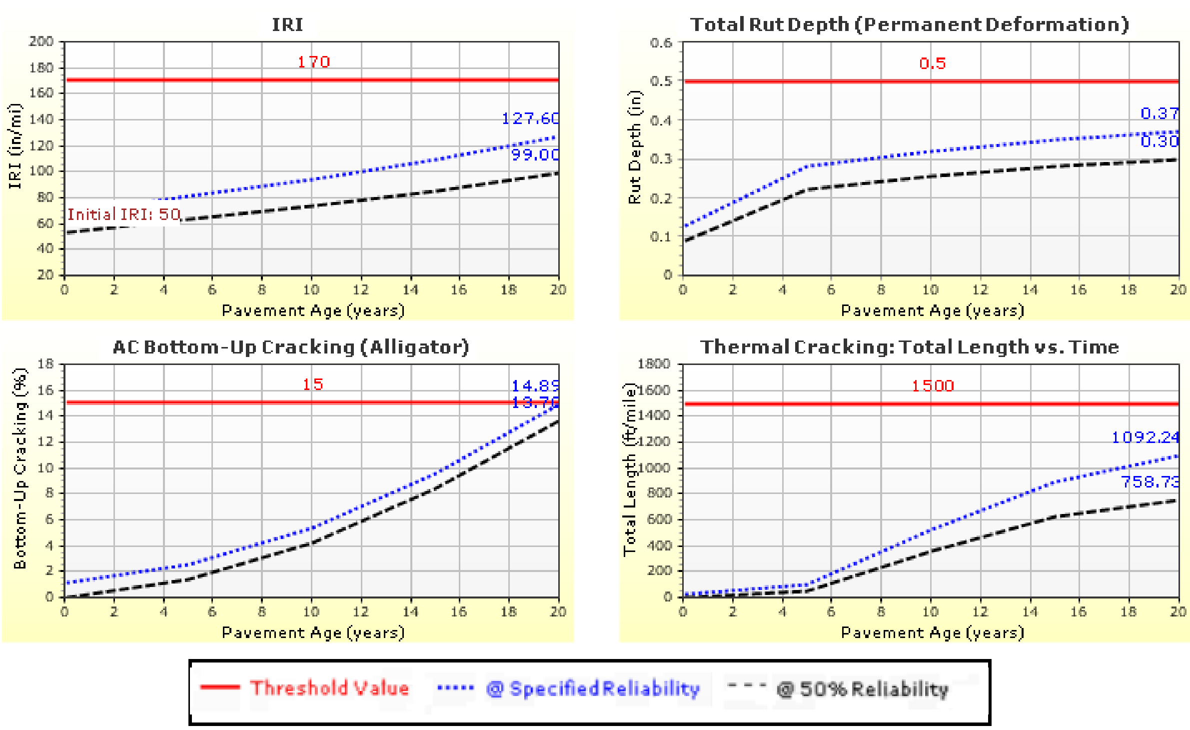

Figure 38 Predicted distress and reliability of adjusted design for Project No. 7.

CHAPTER 8 – SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary and Conclusion

This report focuses on the quantification of local unbound soil material properties used for flexible pavement design in Wyoming. A comprehensive field and laboratory test program was completed. Twelve pavement locations were identified by WYDOT such that these locations consisted of different subgrade AASHTO soil types and pavement structures. Each location was further divided into three sites, resulting in a total of thirty-six sites. An electronic database (*WYOMEP*) was developed to compile, sort, assemble and organize the test data in an efficient manner. In addition to WYDOT sections, test data from twelve LTPP test sections in Wyoming were also evaluated. Using the FWD data, in-place resilient moduli of subgrade soils were back-calculated and corrected to an equivalent laboratory-determined resilient moduli. Constitutive models of resilient modulus were calibrated. Relationships between resilient modulus and other soil properties were established. Soil properties were recommended for Level 3 input. Finally, seven trial design examples on new and rehabilitated flexible pavements were presented to develop recommendations for revisions to the 2012 WYDOT MEPDG User Guide.

8.1.1 Field Test Program

FWD test was performed at each site. The FWD data was used to back-calculate the in-situ pavement layer moduli using two back-calculation programs: MODCOMP6 and EVERCALC. MODCOMP6 is the program used by WYDOT, while EVERCALC was used in the LTPP study. Samples of the unbound layers were recovered for classification, volumetric property determination and laboratory resilient modulus testing. Distress surveys were also performed to identify and measure the distresses observed along each test site, using the procedures established under the LTPP program.

8.1.2 Laboratory Test Program

Laboratory tests on subgrade soil samples collected from WYDOT test sites were conducted at WYDOT. The test results were summarized to obtain average values of the subgrade properties including resilient modulus. Wyoming DOT classifies subgrade soils according to the AASHTO soil classification system, which is consistent with the required input for subgrade layer type in the MEPDG. Subgrade soil properties were summarized based on the AASHTO soil class so that the data could be used to directly update the MEPDG database for subgrade materials. Resilient modulus test was performed in accordance with the modified AASHTO T-307 documented in Henrichs (2015).

8.1.3 LTPP Data

Base and subgrade classification and volumetric properties for the LTPP sections were extracted from the LTPP database. Distress data were also extracted from the LTPP database and summarized for total length of the sections for all survey dates. Pavement deflection basin data and the elastic pavement layer moduli were back-calculated from that deflection data using EVERCALC software from LTPP.

8.1.4 Soil Properties for Level 3 Input

Table 28 summarizes the final data comprising average values of subgrade soil properties obtained from this study for MEPDG Level 3 input. The average gradation values of WYDOT subgrade soils are included in Table 29. The R-value extracted from the LTPP dataset were found to be significantly different in terms of individual values in comparison to the R-values measured within this study for the WYDOT test sections. The average R-values for the same soil classification, however, were found to be similar.

Table 28 Summary of WYDOT subgrade material properties for level 3 input.

Property	AASHTO Soil Type						
	A-1-a	**A-1-b**	**A-2-4**	**A-2-7**	**A-4**	**A-6**	**A-7-6**
ω_{opt} (percent)	7.7	6.7	11.9	18.4	14.1	14.8	20.9
$\gamma_{d\text{-}max}$ (pcf)	124.1	127.7	115.8	107.0	116.3	111.9	98.9
Liquid Limit	19.0	20.8	25.5	45.0	23.9	32.8	50.2
Plasticity Index	1.0	1.0	6.5	19.0	6.5	16.1	31.0
In-situ ω (percent)	0.0	13.4	13.2	19.5	12.6	18.4	27.3
In-situ γ (pcf)	115.0	131.6	117.4	106.0	118.2	121.7	120.7
R-Value	72	69	63	50	40	14	13
Lab M_r (psi)	21,800	14,100	10,500	11,600	12,500	11,400	12,300
Back-calculated M_R (psi)	35,000	36,900	32,100	—	24,100	24,800	19,400

ω_{opt}–Optimum Moisture Content; $\gamma_{d\text{-}max}$–Maximum dry unit weight; ω–Moisture content; γ–Unit weight; M_r–laboratory-determined resilient modulus; and M_R–back-calculated resilient modulus.

Table 29 Summary of gradation data for WYDOT subgrade soils.

SOIL TYPE	Passing Sieve Size (percent)								
	2''	**1.5''**	**1''**	**3/4''**	**3/8''**	**#4**	**#10**	**#40**	**#200**
A-1-A	93	84	66	54	34	28	25	20	9.3
A-1-B	100	100	99.8	99.3	94.3	83.7	72.7	42.8	18.0
A-2-4	100	100	99.8	99.4	97.8	93.6	88.8	74.8	22.3
A-4	100	100	99.8	99.4	97.8	95.6	93.2	85.6	44.08
A-6	100	100	100	99.9	98.5	95.3	91.1	82.1	60.3
A-7-6	100	100	100	99.2	97.8	92.8	87.6	78.8	65.0

8.1.5 Resilient Modulus for Level 2 Input

The LTPP test sections were excluded from the correlation studies to estimate subgrade resilient modulus for MEPDG Level 2 input because of the systematic difference between the WYDOT and LTPP resilient modulus test results.

The C-factor for the subgrade soils was calculated for the WYDOT test sites. If FWD is performed and resilient modulus of subgrade soil is back-calculated, Equation (16) is recommended to correct the back-calculated elastic modulus (M_R) to the lab-measured resilient modulus (M_r).

$$M_r = 0.49 M_R \tag{16}$$

Constitutive model given by Equation (17)(7) was developed to estimate resilient modulus of subgrade soils considering the stress level experienced by the subgrade soil.

Table 30 shows the summary of mean k-values for each AASHTO soil type from WYDOT test sites.

$$M_r = k_1 \sigma_a \left(\frac{\sigma_b}{\sigma_a}\right)^{k_2} \left(\frac{\tau_{oct}}{\sigma_a} + 1\right)^{k_3} \tag{17}$$

Table 30 Summary of resilient modulus constitutive model coefficients.

Soil Class	WYDOT Sections		
	Mean k_1	**Mean k_2**	**Mean k_3**
A-1-a	1,544.8	0.626	-0.527
A-1-b	1,505.6	0.619	-1.063
A-2-4	1,131.2	0.483	-1.056
A-4	1,003.6	0.52	-0.356
A-6	801.6	0.294	0.443
A-7-6	520.4	0.264	0.651

Two relationships were found to provide equal accuracy between the laboratory-derived resilient modulus (M_r) and R-value for subgrade soils (see Section 6.2.3). It is recommended that Equation (18) be used to calculate the laboratory-derived resilient modulus from R-value measurements for Level 2 input. However, it is important to note that the standard error of the estimate using Equation (18) is 3,347 psi.

$$M_r(\text{psi}) = 9713.91 + 61.56R \tag{18}$$

Design tables of resilient modulus for subgrade soils were developed based on thicknesses of asphalt and base layers. For a subgrade soil with R>50, Table 17 can be used to estimate the resilient modulus with respect to its maximum dry unit weight. For a subgrade soil with R≤50, Table 18 can be used to estimate the resilient modulus with respect to its optimum moisture content.

8.1.6 Trial Pavement Design

Seven trial pavement designs provided by WYDOT were evaluated. The outcome or results from evaluating the trial designs to achieve the target reliability value were used to provide revisions to the 2012 WYDOT MEPDG User Guide. The trial pavement designs are summarized in Table 31. Four trial designs were on new flexible pavements and the remaining three on rehabilitated flexible pavements. Both heavy and low traffic conditions as well as interstate and primary roads were considered. The subgrade soils considered in this study were A-4, A-6 and A-7-6. The thickness in inches was included before each pavement layer notation (i.e., O for overlay, F for flexible asphalt, and B for subbase or base). The resilient moduli of the base and subgrade layers used in both baseline and adjusted designs are summarized in Table 32. The threshold values and reliability levels of all distress types used in both baseline and adjusted designs are summarized in Table 33. In all seven trial designs, the predicted length of total transverse cracks exceeded the threshold value using the wearing surface specified by WYDOT. For the higher truck volume roadways, a stone mastic asphalt (SMA) wearing surface was included in the adjusted designs to reduce the predicted length of transverse cracks below the threshold value. For other trial designs, asphalt was designated in the wearing surface that maybe unavailable or uneconomical for use in Wyoming (e.g., PG64-40) in order to achieve the target reliability.

Table 31 Summary of trial pavement designs.

No.	Project Description	Pave-ment	Traffic/ Road	Baseline Design Structure	Baseline Design Per	Adjusted Design Structure	Adjusted Design Per	Design to Pass/ Comment
1	0803137 ISO RECON	New	High/ Interstate	4[a]+6[b]F; 16B; 24A-1-a; A-7-6	Fail[#] [$%*]	2[c]+2[d]+7.5F; 8+8B; 12A-1-a; 9[(1)]A-7-6; A-7-6	Fail[%]	None/Increase AC Transverse Threshold
2	0803137 Mainline	Rehab; Semi-Rigid	High/ Interstate	2[a]+2[a]O; 7.5[b]F; 6C; 10B; 24A-6	Fail [%]	2[c]+2O; 7.5F; 6C; 10B; 24A-6	Pass	−
3	N132104 Slide Version	New	Low/ Primary	6[e]F; 8B; A-4	Fail [%&]	6[f]F; 8B; A-4	Fail[%]	6[g]F; 8B; A-4
4	N132104 Rehabilitation	Rehab	Low/ Primary	2O; 3F; 6B; A-4	Fail [@?%]	2[f]O; 3F; 6B; A-4	Fail[@] [?%]	3.25[f]O; 3F; 6B; A-4
5	P114037 Bridge Ends	New	Low/ Primary	8[e]F; 12B; A-4	Pass [(2)]	8[e]F; 12B; A-4	Fail[%]	2[c]+4[e]F; 12B; A-4
6	Wamsutter Rehabilitation	Rehab	High/ Primary	6[a]O; 8F; 16B, A-6	Fail [@?%]	2[c]+4[a]O; 8F; 8+8B; A-6	Fail[@]	None/Reconstruct Pavement
7	Wamsutter Reconstruct	New	High/ Primary	9[a]F; 8+8B; A-6	Fail[#] [%&]	2[c]+8[a]F; 8+8B; A-6	Pass	−

Per−Performance of the pavement in achieving the target reliability level or threshold values; O−Overlay asphalt layer; F−Existing or new flexible asphalt layer; B−Nonstabilized subbase and base layer; C−Cement-treated base layer; [a]−PG76-28 asphalt; [b]−PG64-22 asphalt; [c]−SMA PG76-34 asphalt; [d]−Binder PG76-34 asphalt; [e]−PG64-28 asphalt;[f]−PG64-34 asphalt; [g]−PG64-40 asphalt; [(1)]−Lime stabilized subgrade layer; [(2)]−Very high threshold value of 3,5000 ft/mi was used for thermal cracking; [@]−Fail in AC total fatigue cracking; [#]−Fail in total pavement permanent deformation; [$]−Fail in AC bottom-up fatigue cracking; [%]−Fail in AC thermal cracking; [&]−Fail in top-down fatigue cracking; [*]−Fail in AC performance deformation; and [?]−Fail in AC total transverse cracking.

Table 32 Summary of resilient moduli used in trial design examples.

No.	M_r (psi) for Baseline: Base	M_r (psi) for Baseline: Subgrade	M_r (psi) for Adjusted: Base	M_r (psi) for Adjusted: Subgrade
1		11,592U; 3,864L	37,000U; 23,000L	11,500L
2	650 for the treated base	5,810	450 for the treated base	7,000
3	6,085	12,170	15,000	
4	12,170	6,085	12,170	6,085
5	12,170	6,085	15,000	
6	35,610	35,610	25,000U; 15,000L	7,500
7	35,610	35,610	25,000U; 15,000L	7,500

U−Upper layer; L−Lower layer

Table 33 Summary of threshold values and reliability levels.

Distress Type	Project	Baseline Design		Adjusted Design	
		Threshold	Reliability (percent)	Threshold	Reliability (percent)
Terminal IRI (in/mile)	1	150	95	170	95
	2	150	85	170	95
	3	200	90	220	85
	4	150	65	220	80
	5	150	80	220	80
	6	150	85	170	85
	7	150	95	170	85
Pavement Deformation-Total Pavement (in)	1	0.5	95	0.5	95
	2	1	85	0.5	95
	3	0.75	90	0.75	85
	4	0.75	65	0.75	80
	5	0.75	80	0.75	80
	6	0.5	85	0.5	85
	7	0.5	95	0.5	85
AC Total Fatigue Cracking (Percent Lane Area)	1	–	–	–	–
	2	25	85	10	95
	3	–	–	–	–
	4	15	65	15	80
	5	–	–	–	–
	6	25	85	15	85
	7	–	–	–	–
AC Total Transverse Cracking (ft/mile)	1	–	–	–	–
	2	2500	85	1000	95
	3	–	–	–	–
	4	1500	65	1500	80
	5	–	–	–	–
	6	1000	85	1500	85
	7	–	–	–	–
Permanent Deformation-AC Only (in)	1	0.5	95	0.5	95
	2	0.5	85	0.5	95
	3	0.25	90	0.75	85
	4	0.25	65	0.75	80
	5	0.75	80	0.75	80
	6	0.5	85	0.5	85
	7	0.5	95	0.5	85
AC Bottom-Up Fatigue Cracking (Percent Lane Area)	1	10	95	10	95
	2	10	50	10	50
	3	15	90	15	85
	4	15	50	15	50
	5	15	80	15	80
	6	25	50	15	50
	7	10	95	15	85

Table 33 Summary of threshold values and reliability levels used (continue).

Distress Type	Project	Baseline Design		Adjusted Design	
		Threshold	Reliability (percent)	Threshold	Reliability (percent)
AC Thermal Cracking (ft/mile)	1	1000	95	1000	95
	2	1000	50	1000	50
	3	1500	90	1500	85
	4	1500	50	1500	50
	5	3500	80	1500	80
	6	1000	50	1500	50
	7	1000	95	1500	85
AC Top-Down Fatigue Cracking (ft/mile)	1	2000	95	5000	95
	2	2000	85	2000	95
	3	2000	90	2000	85
	4	2000	65	2000	80
	5	2000	80	2000	80
	6	2500	85	2500	85
	7	2000	95	2000	85

8.2 Recommendations

Recommendations for characterization of unbound soil properties are described as follows:

(1) A comparison of MODCOMP6 and EVERCALC back-calculation results for WYDOT sites showed that the two programs predicted similar elastic moduli and were in good agreement for HMA, crushed stone base, and the natural subgrade (within 10 percent differences). Significant differences between MODCOMP6 and EVERCALC (about 55 percent difference), however, were found for the weathered subgrade layer. Sites where the ratio of back-calculated moduli between the two programs was greater than 10 were considered outliers and discarded from the comparison. The reason for the difference between the two programs for the weathered soil layer should be further investigated.

(2) There is a consistent relationship between the optimum water content and maximum dry unit weight for the LTPP and WYDOT test sites. The only consistent difference between the two sets of data was confined to the soils and materials with the lower optimum water contents (coarse-grained or higher stiffness soils). It is recommended that the water contents and maximum dry density measured by WYDOT should be used, since these values are based on WYDOT specifications.

(3) There is a significant bias or difference between the resilient modulus extracted from the LTPP database and measured from this study on the subgrade soils for the same soil classification. Caution should be used in combining some of the layer or material properties extracted from the LTPP database because the properties were found to be significantly different than those measured within this study. Future resilient modulus tests will probably be performed by the University of Wyoming or WYDOT, so the resilient modulus measured values reported in this study should be used in the local calibration study.

(4) WYDOT should continue to expand the number of sites to verify and confirm the local calibration coefficients over time.

(5) It is recommended that the average C-factor of 0.49 be used to adjust the field-derived (back-calculated elastic layer moduli) moduli to a laboratory equivalent resilient modulus for the local calibration.
(6) Equation (17) or (18) should be used to estimate the laboratory-derived resilient modulus as an input level 2, when laboratory resilient modulus tests are unavailable. It is recommended, however, that the database for number of soils be expanded to verify or confirm that relationship.

Recommendations for MEPDG pavement designs are described as follows:

(1) The default design resilient modulus values included in the 2012 WYDOT MEPDG User Guide are extremely high for the aggregate base materials typically specified in Wyoming. The repeated load resilient modulus tests from the LTPP database and other sources suggest the default resilient moduli need to be reduced.
(2) The default design resilient moduli for typical soils found in Wyoming are very low in comparison to the repeated load resilient modulus tests and back-calculated elastic layer moduli reported in LTPP and other studies. It is recommended that WYDOT consider revising the default design resilient moduli for the different aggregate base materials and subgrade soils included in the 2012 WYDOT MEPDG User Guide.
(3) Rehabilitation input level 3 was used for all three trial rehabilitation designs of existing flexible pavements. It is recommended that rehabilitation input level 2 be used whenever possible. The performance of the road in terms of distresses and environmental condition rating should be determined.
(4) The 2012 WYDOT MEPDG User Guide includes the threshold values for all distresses which are traffic volume and roadway specific. The thermal cracking threshold value for Interstate and other high truck volume roadways is 1,000 ft./mi. and 2,500 ft./mi. for secondary roadways with low truck volumes.
(5) To satisfy threshold value and achieve the reliability for the thermal cracking, it is recommended 1) to increase the threshold value for transverse cracks to be consistent with standard maintenance operations for all roadway classifications, and 2) identify the asphalt grade to be specified for critical areas in Wyoming and/or specialized mixtures (higher asphalt contents and lower air voids). For pavement areas in a warmer environment, the target reliability for transverse cracks can be reduced to 50 percent because the reliability in the MEPDG is given based on cold temperatures or a single cold temperature event.
(6) WYDOT has built or used semi-rigid pavements in the past, but only one of the trial designs included the rehabilitation of a semi-rigid pavement. Some of the default CTB material properties are believed to be inappropriate and may result in an insufficient pavement structure or overlay thickness. The CTB mixture properties should be verified with construction data.

8.4 Future Research

With respect to the limitations of this research, recommendations for future research are suggested as follows:

(1) Revise the existing MEPDG user manual to facilitate MEPDG pavement design.
(2) Characterize the subbase and base properties for MEPDG input levels 2 and 3.

(3) Investigate the difference between the two back-calculation programs for the weathered soil layer.
(4) Continuously collect laboratory-derived resilient modulus of subgrade soils and populate them to the electronic database (WYOMEP) to improve the local MEPDG calibration.
(5) Expand the number of test sites to verify and confirm the local calibration coefficients over time.
(6) Characterize the properties of CTB materials for MEPDG input levels 2 and 3.
(7) Determine the seasonal effects on material properties using FWD and/or laboratory test methods.
(8) Perform a similar MEPDG study on rigid pavements.
(9) Conduct a pilot study to verify the local calibrated MEPDG recommended in this study.
(10) Perform a pavement performance assessment study by comparing the estimated and measured road performances in terms of distresses over a period of time.

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APPENDIX A – STANDARDIZED FWD TESTING AND ANALYSIS PROCEDURES

The purpose of this appendix is to provide guidance and recommendations to be used for pavement diagnostic and rehabilitation studies. The appendix is grouped into two parts: (A) recommendations for FWD testing and (B) back-calculation or analysis procedures of the deflection basins to determine the in place elastic moduli for individual pavement layers and subgrade.

<u>(A) FWD Testing</u>

It is recommended that the deflection basin data be collected with the FWD in accordance with the LTPP procedure, because that procedure was used to measure deflection basins for both the WYDOT and LTPP sections. The following are exceptions to that procedure that are not needed for rehabilitation design.

(1) *Deflection Basin Testing Segments*: Select representative segments along the project; primarily where the distress survey was conducted or where the distresses change. The length of the segments can vary depending on the site conditions of each project. The MEPDG suggests a windshield distress survey to subdivide any rehabilitation project into segments with similar surface distress. In addition, construction records should be used to determine the locations along the roadway where the pavement structure or subsurface condition changes. Deflection basin tests should be performed within each segment.

(2) *Sensor Location and Spacing*: An adequate number of sensors properly spaced need to be used to measure the actual deflected shape of the pavement from the impact load. The location of the individual sensors is dependent on the pavement structure. Seven to nine sensors are typically used for evaluating flexible pavements. LTPP specifies the spacing of the seven sensors as 0 (under the loading plate), 8, 12, 18, 24, 36, and 60 inches from the loading plate. This number and spacing of sensors have been found to be adequate for most pavement structures. Some agencies (Arizona and Texas DOTs) specify an equal spacing of the sensors; the common sensor spacing is 12 inches. Thinner asphalt concrete surfaces (less than 6 inches in thickness) usually require the variable sensor spacing with some sensors located closer to the loading plate, while an equal 12-inch spacing of the sensors is applicable to the thicker asphalt concrete surfaces (greater than 12 inches in thickness). However, it is highly recommended that the sensor location and spacing be set for all projects to avoid confusion and possible errors in sensor spacing between projects. Based on typical flexible pavements built in Wyoming, the LTPP variable sensor location should be used.

(3) *Deflection Basin Spacing*: Measure deflection basins at a frequency of 100 to 500 feet. If multiple lanes are being tested, the locations for the deflection basin tests should be staggered between the lanes. The spacing of the deflection tests usually depends on how the surface condition varies. Areas with uniform site conditions and surface distresses while a shorter spacing is used when the site conditions and magnitudes of distress vary along the project.

(4) *Load Levels or Drop Heights*: Four FWD drop heights or load levels (about 6, 9, 12, and 15 kips) were used for testing the LTPP and WYDOT test sections. These load levels were used to provide consistency between the LTPP and WYDOT test sections. For typical rehabilitation designs, however, four load levels are not needed because it is difficult to measure the in place stress sensitivity with varying deflection basin load

levels. It is recommended that the seating drops specific in the LTPP test procedure be used as common practice and followed by one drop height – the 9 or 12 kip load level. The load level selected should be representative of the expected heavier wheel load magnitudes for the specific roadway. The use of one drop height allows more deflection basin tests to be completed without increasing the testing time requiring traffic control.

(5) *Path of Deflection Basin Testing*: Deflection basin tests should be located and made along a line in the center of the lane or along the outside wheel path depending on the distresses. Measuring deflection along the inside wheel path is not recommended because of safety regarding oncoming traffic in the opposite lane. The lane of testing should be designated with the safety of field personnel in mind and to not interfere with the traffic in the other lanes.

(6) *Temperatures*: During deflection basin testing, the surface and air temperatures should be recorded. It is also recommended that deflection basins be measured at a couple of locations over different temperature regimes – morning and afternoon testing times, if possible. Measuring the deflection basin at the same point but over different temperatures allows the effect of temperature to be determined. The MEPDG does not require but it is suggested that two test temperatures be used as input level 1 for rehabilitation design.

(B) Deflection Basin Analysis Procedures

This section of Appendix A is grouped into two parts: (1) an analysis of the deflection data termed pre-processing deflection data, and (2) the back-calculation process to determine the field-derived moduli.

Pre-Processing Deflection Data

The following briefly summarizes the steps involved in the pre-processing of the deflection basin data to remove any "problem" deflection basins that cannot be back-calculated using the assumptions of any elastic layered solution.

(1) Review the measured deflection basins to ensure that the deflection basins decrease consistently with the sensors farther from the loading plate. Identify and remove any basin where the deflections do not decrease with sensor location from the loading plate.
(2) Normalize the deflection basin in terms of the deflection measured directly under the loading plate. Identify and segregate the deflection basins with different shapes of the normalized deflection basin. The shape of the deflection basin can designate changes in the pavement structure and underlying subsurface condition not reflected in the construction documents or as-built project plans.
(3) Execute the SLIC program within MODTAG program used by WYDOT to confirm or check any anomaly identified.

Back-calculation of Elastic Layer Moduli from Deflection Basin

The following two statements shall be noted and understood before back-calculating elastic layer moduli from deflection basins:

- There is no unique solution for a specific deflection basin using elastic layered theory. The elastic layer moduli determined for the back-calculation process (defined as field-derived moduli) represent equivalent elastic moduli and should be thoroughly reviewed for reasonableness.
- The general procedure discussed in this section of Appendix A is an iterative process to decrease the error term (difference between the measured and calculated deflection basins) to the lowest value possible. The combination of layers and calculated elastic

moduli resulting in the lowest error should be used for diagnostic studies and rehabilitation design. The magnitude of the error depends on different factors that are listed below:

- Difference between the assumed and actual layer thickness.
- Combining different layers into one structural layer.
- Inaccurate assumption on the existence and depth of an apparent rigid or stiff layer.
- Discontinuities in the pavement, particularly if located between the load and one or more of the sensors.
- Non-linear or anisotropic materials in the pavement structure.
- Inaccuracies contained within the sensor measurement itself.

The following are the steps and criteria recommended for use in back-calculating the elastic layer moduli using MODCOMP6 or EVERCALC.

(1) Pavement simulation rules to determine the structure or layering assumptions to be used. The rules of simulation reported in the MEPDG were used to set up the pavement structure for each LTPP test section. Setting up the initial pavement layering simulation is straightforward, but there are factors that complicate the process. For example, layer thicknesses are not known at every deflection point, and some subsurface layer conditions can be overlooked or not adequately identified throughout the test section. The following lists provide some general rules of simulation for creating the pavement structure used in back-calculating elastic layered modulus for each of the structural layers:
    - Start with the fewest number of layers possible. The number of layers and individual layer thickness, however, are critical parameters for back-calculating the elastic modulus of structural layers—that is, those layers that have a significant impact on the deflection basin with reasonable changes or variation in modulus. Getting a “good” starting pavement simulation for the measured deflection basin is probably the most important activity in the back-calculation process.
    - Identify insensitive layers; assume the modulus of an insensitive layer or combine it with an adjacent layer of “like” material.
    - Combine similar and adjacent layers but separate significantly different and thick soil strata as well as thick aggregate layers.
    - Identify layer anomalies and potential problem layers, such as sandwich sections, stiff soils above weaker or saturated soils, etc.
    - Estimate depth to apparent rigid layer.
    - Estimate depth to the water table.

(2) Thin non-structural layers should be combined with adjacent like layers. Thin is defined as less than 2 inches or less than half the thickness of the layer above the layer in question. As an example, an HMA open-graded friction course should be combined with the supporting HMA layer. In many cases, thin layers are defined as insensitive layers.

(3) Similar or like materials of adjacent layers should be combined into one layer. “Like” is defined as materials or layers with the same AASHTO classification exhibiting similar laboratory-measured modulus values and the same stress-sensitivity and physical properties.

(4) Layer dependent rules for simulating the pavement structure are listed in the following bullets:
  - The lower layers should be combined first, and the upper pavement layers should be treated in more detail, if possible. The discussion and guidance that follows, however, starts with the foundation layers and proceeds up to the surface layers.
  - In most cases, the number of unbound granular base and subbase layers should not exceed two, especially when one of those layers is thick (more than 18 inches in thickness). Sand and other soil-aggregate materials should be simulated separately from crushed stone or crushed aggregate base materials.
  - If thick, unbound aggregate or select materials (i.e., exceeding 18 inches) are used, this layer can be treated as the upper subgrade layer. Thick granular layers are typically used in northern climates as non-frost susceptible materials. When these layers are treated as the upper subgrade, then only one subgrade layer is needed.
  - If a thin aggregate base layer is used between two thick unbound materials, the thin layer should be combined with the weaker or lower layer. For example, a 6-inch sand subbase layer placed between an A-1-b AASHTO classified subgrade soil and crushed stone base can be combined with the upper subgrade layer.
  - Cement-treated and other pozzolanic stabilized materials that are used as a base layer for structural support should be treated as a separate layer. Lime and lime-fly ash stabilized subgrade soils should be treated as a separate layer, if possible. In some cases, a small amount of lime or lime-fly ash is added to soils in the upper subgrade to lower the plasticity index and from a constructability standpoint. For these cases, the lime or lime-fly ash stabilized soil should be combined with the upper subgrade layer.
  - The number of HMA layers should not exceed two, if at all possible. All layers that are dense-graded HMA mixtures should initially be combined. For example, an HMA wearing surface or mix and an HMA binder layer can be combined into one layer without affecting the accuracy of the predictions.

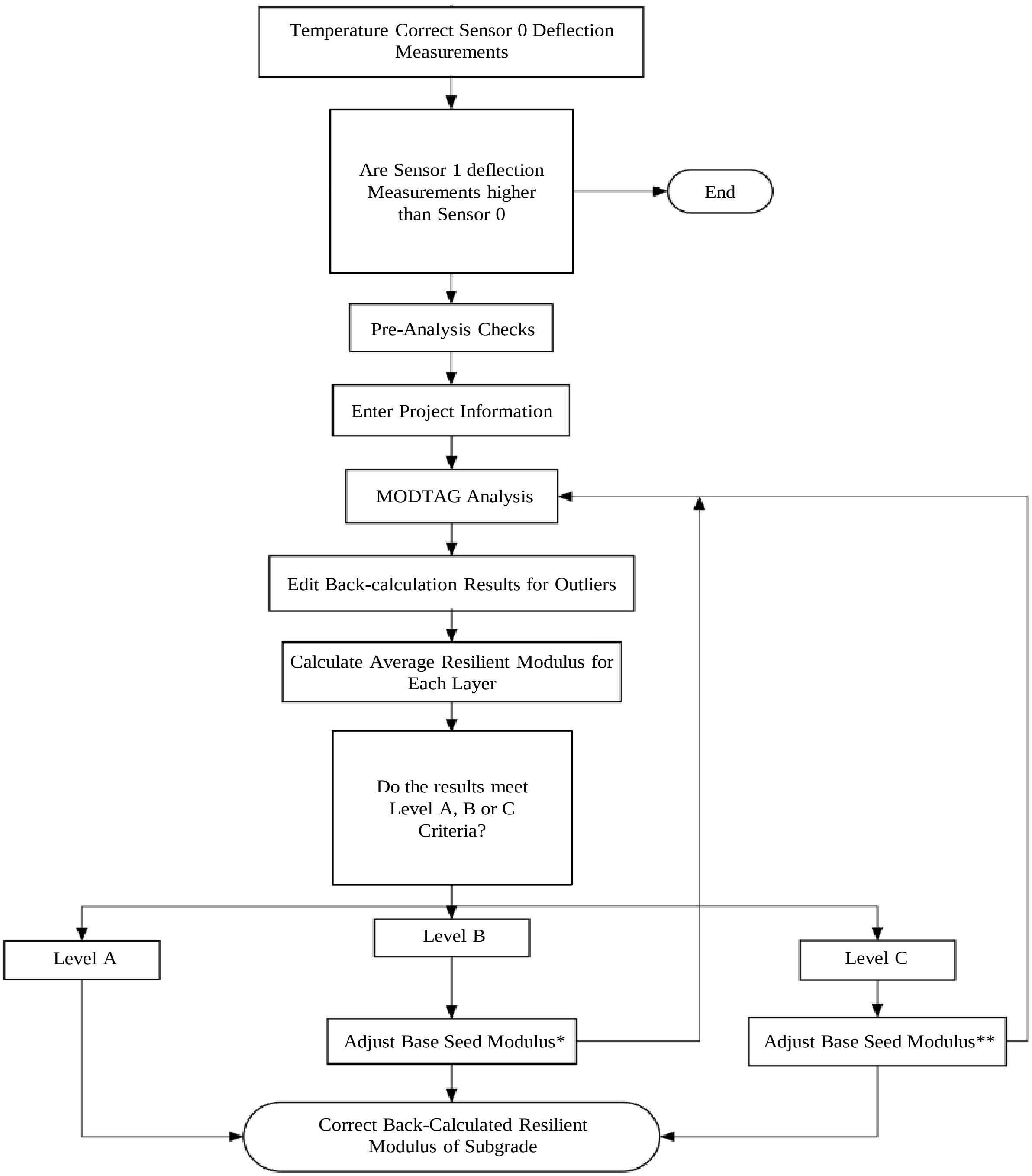
(5) It is suggested and recommended that the deflection not be adjusted to a standard test temperature or load level. After the back-calculation process has been completed, the resulting elastic moduli can be then adjusted to a standard temperature and load level or frequency.

(6) Back-calculate the modulus of each structural layer and calculate the error term for each measured deflection basin. The error should be reviewed and the deflection basins that hit the upper or lower limit set by the user should be investigated as noted below.
  - All of the measurement points that have excessive error terms should not be used in determining the field-derived elastic moduli for the project segment. Excessive is defined as a solution with a root mean squared error (RMSE) greater than 5 percent. Values with a RMSE less than 3 percent are typically considered good solutions when the measured and calculated deflection basins match. Solutions with RMSE values between 3 and 5 percent should be evaluated and reviewed carefully prior to use in rehabilitation design.
  - For those basins that consistently hit the upper limit set for the modulus of a particular layer, the structure should be reviewed in an attempt to reduce the error term while maintaining reasonable modulus values. For basins that consistently

hit the lower limit, the lower limit can be further reduced. Low modulus values may be reasonable because of contamination of underlying materials, the presence of cracks or internal damage (such as stripping), or the weakening of some unbound materials with an increase in moisture or a decrease in density.

The average back-calculated layer moduli from the deflection basins should be used as the design input for rehabilitation design.

APPENDIX B – BACK-CALCULATION PROCEDURE AND RESULTS (HELLRUNG 2015)



* Continue to adjust the base seed modulus in order to achieve Level A criteria. If Level A criteria cannot be met, use best results meeting Level B criteria and then proceed to the next step.
** Continue to adjust the base seed modulus in order to achieve Level A criteria. If Level A criteria cannot be met, adjust the base seed modulus in order to achieve Level B criteria. If Level A or B criteria cannot be met use best results meeting Level C criteria and then proceed to the next step.

Figure B-1 Back-calculation protocol flow chart.

Table B-1 Back-calculation analysis criteria levels

	Level A	Level B	Level C
Back-Calculated Asphalt Modulus (psi)	100,000 - 750,000	100,000 - 750,000	N/A
Back-Calculated Base Modulus (psi)	10,000 - 80,000	10,000 - 80,000	10,000 - 80,000
Back-Calculated Subgrade Modulus (psi)	4,000 - 40,000	4,000 - 40,000	4,000 - 40,000
RMSE	< 7 percent	7 – 11 percent	< 11 percent

Table B-2 Summary of results from back-calculation analysis (Hellrung 2015)

Test Loc.	Proj. No.	Site	Asphalt M_R (psi)	Base M_R (psi)	Subgrade				RMSE (percent)	Level
					Soil Type	Seed M_R^* (psi)	Upper Subgrade M_R (psi)	Lower Subgrade M_R (psi)		
1	0107	B	789,714	45,000	A-4	24,004	34,946	22,038	4.1	C
		C	249,232	15,000	A-2-4	31,995	15,896	11,562	10.4	C
3	0P11	A	404,583	17,000	A-6	16,998	18,588	16,784	8.2	B
		B	410,700	22,000	A-7-6	8,006	20,222	12,844	3.2	A
		C	313,900	12,000	A-7-6	8,006	15,385	8,872	7.7	B
4	0300	A	698,615	12,000	A-6	16,998	11,254	9,609	8.3	B
		B	510,233	17,000	A-4	24,004	16,660	11,831	5.5	A
		C	271,330	12,000	A-6	16,998	12,158	10,181	8.0	B
5	0601	A	775,857	65,000	A-2-4	31,995	54,496	29,359	5.1	C
		B	1,081,679	70,000	A-2-4	31,995	58,845	28,918	4.6	C
		C	308,667	22,000	A-6	16,998	33,355	19,020	3.5	A
6	1401	A	578,161	12,000	A-7-6	8,006	10,397	7,529	6.3	A
		B	421,554	12,000	A-7-6	8,006	14,488	7,740	4.6	A
		C	409,393	12,000	A-7-6	8,006	19,794	13,756	8.0	B
7	0N37	A	361,967	22,000	A-1-B	38,000	31,720	24,838	3.6	A
		B	387,583	30,000	A-1-B	38,000	23,560	19,335	3.0	A
		C	654,783	12,000	A-1-B	38,000	36,583	18,460	5.3	A
8	0N34	C	1,046,537	27,000	A-4	24,004	15,805	16,522	4.3	C
9	0N30	A	404,600	17,000	A-6	16,998	18,589	16,784	8.2	B
		B	763,442	15,000	A-1-A	40,001	37,893	24,767	2.7	C
		C	407,350	12,000	A-4	24,004	13,571	12,436	9.1	B
10	0N21	C	667,571	35,000	A-6	16,998	28,264	14,373	2.5	A
11	0N23	A	793,517	55,000	A-1-B	38,000	32,903	25,492	1.9	C
		B	693,381	22,000	A-2-4	31,995	31,349	24,764	3.2	A
		C	846,933	38,000	A-1-B	38,000	45,525	28,333	3.9	C

Loc.−Location; Proj.−Project; M_R−Back-calculated resilient modulus; RMSE−Root mean square error; and *−Recommended in the NCHRP Report 1-37A (2004).

APPENDIX C – DETERMINATION OF RESILIENT MODULUS FOR SUBGRADE MATERIALS (UNIVERSITY OF WYOMING)

C.1 Development of Constitutive Model

The alternative constitutive model developed by Henrichs (2015) is given by

$$\frac{M_r}{\sigma_a} = k_4 \times \left(\frac{\sigma_c}{\sigma_a}\right)^{k_5} \times \left(\frac{\Delta\sigma}{\sigma_a}\right)^{k_6} \tag{C.1}$$

where,

σ_a = atmospheric pressure;

σ_c = confining pressure or minor principal stress;

$\Delta\sigma$ = deviator stress; and

k_4, k_5 and k_6 = regression coefficients in terms of significant soil properties.

The same statistical software program, the R program, was used to perform this calibration. A series of statistical t-test analyses determined that the maximum dry unit weight ($\gamma_{d\text{-}max}$) and optimum moisture content (ω_{opt}) were significant in defining the k_4 coefficient. The t-test was conducted during nonlinear regression to determine the significant soil index properties. Indicators of significance of a parameter in this test were given by the "t" and "P" values. The t-value measured the differential magnitude relative to the variation in the data. A t-value further away from 0 would represent larger deviation from no effect or no difference. The P-value, Pr (>|t|), is the level of significance that would lead to rejection of the null hypothesis measured. Thus, a smaller P-value would result in more significance of not rejecting that parameter. The standard error (SE) is the squared correlation between the observed values and the predicted values, which was also determined for each regression coefficient. It was determined that the k_4 coefficient depended on both soil types characterized by the R-value described in Section 6.2.4 (i.e., R>50 and R≤50). Thus, it is reasonable to determine the k_4 coefficient for subgrade soils with R>50 using Equation (C.2) in terms of the maximum dry unit weight and R≤50 using Equation (C.3) in terms of optimum moisture content as well as in terms of confining (σ_c) and deviator ($\Delta\sigma$) stresses. On the other hand, the t-test revealed that the confining and deviator stresses have little effect when raised to the other k_5 and k_6 regression coefficient values. This finding implies that the k_5 and k_6 were assumed as constant regression coefficients as summarized in Table C-1.

$$k_4 = 10^{\left[b_0 + b_1\gamma_{d-max} + b_2\left({}^{\sigma_c}/_{\sigma_a}\right) b_3\left({}^{\Delta\sigma}/_{\sigma_a}\right)\right]} \tag{C.2}$$

$$k_4 = 10^{\left[b_0 + b_1\omega_{opt} + b_2\left({}^{\sigma_c}/_{\sigma_a}\right) b_3\left({}^{\Delta\sigma}/_{\sigma_a}\right)\right]} \tag{C.3}$$

where,

b_0, b_1, b_2 and b_3 = regression coefficients given in Table C-1.

Table C-1 Summary of regression coefficients for the M_r constitutive model C.2 or C.3

Subgrade R-value	Statistical Parameters	b_0	b_1	b_2	b_3	k_5	k_6
R > 50	Estimate	4.1633	0.0088	-22.4532	0.9478	1.5585	-0.9574
	Std. Error	0.49982	0.00161	2.71585	0.29956	0.20526	0.29636
	t-value	8.330	5.485	-8.267	3.164	7.593	-3.230
	Pr (>|t|)	0.00114	0.00583	0.00117	0.03406	0.00161	0.03196
R ≤ 50	Estimate	2.8770	-0.0264	4.0018	0.5610	0.0989	-0.3539
	Std. Error	3.083768	0.007289	19.936303	1.038287	2.051692	0.60020
	t-value	0.933	-3.362	0.201	0.540	0.048	-0.590
	Pr (>|t|)	0.36923	0.00352	0.84427	0.59885	0.96235	0.56635

To facilitate the M_r estimation for the MEPDG application, a regression analysis was performed to develop one set of regression coefficients for all subgrade types. The calibration was conducted using the same test data, rather than sorting the data based on R-values. Instead of having two equations for determining the k_4 coefficient, an R category (R_{cat}) was used as a binary indicator of R values. This effort led to the development of a single Equation (C.4) for the k_4 coefficient for all subgrades given by

$$k_4 = 10^{\left[b_0+b_1\gamma_{d-max}+b_2\omega_{opt}+b_3R_{cat}+b_4R_{cat}\gamma+b_5R_{cat}w+b_6\left(\sigma_c/\sigma_a\right)+b_7\left(\Delta\sigma/\sigma_a\right)\right]} \quad \text{(C.4)}$$

where,

σ_c = confining stress;
$\Delta\sigma$ = deviator stress;
σ_a = atmospheric Pressure;
γ_{d-max} = maximum dry unit weight;
ω_{opt} = optimum moisture content;
R_{cat} = 0 for R > 50 or 1 for R ≤ 50; and
b_0, b_1, b_2, b_3, b_4, b_5, b_6 and b_7 = constant coefficients and their statistical parameters are summarized in

Table C-2.

Table C-2 Summary of regression coefficients and statistical parameters for the M_r constitutive Model C.4

Statistical Parameter	b_0	b_1	b_2	b_3	b_4	b_5	b_6	b_7	k_5	k_6
Estimate	2.9573	0.0089	-0.0297	3.4470	-0.0254	-0.0234	-9.5799	0.8153	0.8271	-0.7752
Std. Error	0.9350	0.0033	0.0185	1.6240	0.0113	0.0289	6.4568	0.3333	0.4937	0.2835
t-value	3.163	2.702	-1.605	2.123	-2.38	-0.808	-1.484	2.446	1.675	-2.735
Pr (>|t|)	0.0054	0.0146	0.1258	0.0479	0.0381	0.4297	0.1552	0.0249	0.1112	0.0136

C.2 Development of Resilient Modulus and R-value Relationship

Using the laboratory measured resilient modulus at the optimum condition, resistance-values, and standard Proctor properties of subgrade material, a correlation study was conducted to develop both multivariate and univariate multi-regression models to estimate resilient modulus in terms of the commonly measured R-value for MEPDG Level 2 design inputs. Besides using coefficient of determination (R^2) to assess the predictive ability of the model, a modeling approach was applied to evaluate the significance of subgrade variables through hypothesis tests conducted at the 0.05 significance level.

A multivariate regression model was used to simultaneously model the M_r values in Table 6 for the response values of all 15 test sequences. The resulting multivariate predicted regression model for predicting the $M_{r,i}$ value ($\widehat{M_{r_i}}$) for sequence i=1,2,…,15 is given by

$$\widehat{M_{r_i}}(\text{psi}) = \alpha_i + \beta_i \times R + \mu_i \times \omega_{opt} \tag{C.5}$$

where

α_i, β_i, and μ_i = regression coefficient estimates given in Table C-3 for each test sequence;
R = R-value; and
ω_{opt} = optimum moisture content in percentage.

Table C-3 Summaries for the estimated multivariate multi-regression model

Sequence	σ_c (psi)	$\Delta\sigma$ (psi)	R-value		ω_{opt} (percent)		α_i	R^2
			β_i	p-value	μ_i	p-value		
1	6	2	-75.28	0.0985	-902.34	0.0001	32709	0.4662
2		4	-31.24	0.4114	-897.49	0.0000	30969	0.6228
3		6	-3.60	0.9192	-884.26	0.0000	29119	0.6931
4		8	14.94	0.6643	-883.68	0.0000	27846	0.7350
5		10	31.60	0.3512	-883.24	0.0000	26944	0.7665
6	4	2	-87.28	0.0255	-763.56	0.0001	29007	0.4292
7		4	-62.39	0.0691	-763.13	0.0000	27389	0.5290
8		6	-41.52	0.1955	-776.39	0.0000	26414	0.6085
9		8	-17.20	0.5672	-770.85	0.0000	25320	0.6789
10		10	4.71	0.8741	-775.10	0.0000	24622	0.7264
11	2	2	-90.41	0.0044	-540.66	0.0004	23086	0.3496
12		4	-75.10	0.0076	-577.50	0.0000	22492	0.4476
13		6	-56.37	0.0303	-602.89	0.0000	22041	0.5382
14		8	-34.46	0.1576	-617.20	0.0000	21524	0.6272
15		10	-13.29	0.5814	-625.61	0.0000	21038	0.6858

σ_c–Confining stress; σ_d–Deviator stress; α_i, β_i, and μ_i–Regression coefficient estimates; p-value–p-value from partial MANOVA test; R^2–Coefficient of determination; and ω_{opt}–Optimum moisture content.

Recognizing the challenges with using Equation (C.5) with 15 different sets of coefficients at their respective confining and axial stresses, a single univariate multi-regression model allowing the user to input site specific confining and deviator stresses was developed given by

$$\begin{aligned}\widehat{M_r}(\text{psi}) = 20287.14 - 169.13 \times R - 504.69 \times \omega_{opt} + 5.02 \times R \times \omega_{opt} \\ + 1681.98 \times \sigma_c - 341.77 \times \Delta\sigma - 44.81 \times \sigma_c \times \Delta\sigma + 14.18 \times R \\ \times \sigma_c - 50.65 \times \omega_{opt} \times \sigma_c + 11.10 \times R \times \Delta\sigma\end{aligned} \tag{C.6}$$

The multivariate model provided the best match to measured M_r values while the univariate model provided the least variation in the M_r estimation. On average, both models slightly overestimated the M_r values.

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