IDAHO TRANSPORTATION DEPARTMENT
RESEARCH REPORT

RP 211B

Idaho AASHTOWare Pavement ME Design User's Guide, Version 1.1

By
Jagannath Mallela, Leslie Titus-Glover
Biplab Bhattacharya, Michael Darter
and
Harold Von Quintus
Applied Research Associates, Inc.

Prepared for
Idaho Transportation Department
Research Program
Division of Highways, Resource Center
http://itd.idaho.gov/highways/research/

March 2014

<table>
<tr><td>1. Report No.
FHWA-ID/14-211B</td><td>2. Government Accession No.</td><td>3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle

Idaho AASHTOWare Pavement ME Design User's Guide, Version 1.1</td><td>5. Report Date
April 2014</td></tr>
<tr><td>6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Author(s)
Jagannath Mallela, Leslie Titus-Glover, Biplab Bhattacharya, Michael Darter, and Harold Von Quintus</td><td>8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
ARA
100 Trade Centre drive, Suite 200
Champaign, IL 61820</td><td>10. Work Unit No. (TRAIS)</td></tr>
<tr><td>11. Contract or Grant No.
RP211B</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Idaho Transportation Department
Division of Highways, Resource Center, Research Program
PO Box 7129
Boise, ID 83707-7129</td><td>13. Type of Report and Period Covered
Final or Interim Report</td></tr>
<tr><td>14. Sponsoring Agency Code</td></tr>
<tr><td colspan="3">15. Supplementary Notes
e.g. Project performed in cooperation with the Idaho Transportation Department and FHWA.</td></tr>
<tr><td colspan="3">16. Abstract</td></tr>
<tr><td colspan="2">17. Key Words
MEPDG; JPCP; HMA; Idaho; User's Guide</td><td>18. Distribution Statement
Copies available online at
http://itd.idaho.gov/highways/research/</td></tr>
<tr><td>19. Security Classification (of this report)
Unclassified</td><td>20. Security Classification (of this page)
Unclassified</td><td>21. No. of Pages
268

22. Price
None</td></tr>
</table>

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4		mm
ft	feet	0.3048		m
yd	yards	0.914		m
mi	Miles (statute)	1.61		km
		AREA		
in^2	square inches	645.2	millimeters squared	cm^2
ft^2	square feet	0.0929	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
mi^2	square miles	2.59	kilometers squared	km^2
ac	acres	0.4046	hectares	ha
		MASS (weight)		
oz	Ounces (avdp)	28.35	grams	g
lb	Pounds (avdp)	0.454	kilograms	kg
T	Short tons (2000 lb)	0.907	megagrams	mg
		VOLUME		
fl oz	fluid ounces (US)	29.57	milliliters	mL
gal	Gallons (liq)	3.785	liters	liters
ft^3	cubic feet	0.0283	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3

Note: Volumes greater than 1000 L shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
ºF	Fahrenheit temperature	5/9 (ºF-32)	Celsius temperature	ºC
		ILLUMINATION		
fc	Foot-candles	10.76	lux	lx
fl	foot-lamberts	3.426	candela/m^2	cd/cm^2
		FORCE and PRESSURE or STRESS		
lbf	pound-force	4.45	newtons	N
psi	pound-force 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 (statute)	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
km^2	kilometers squared	0.39	square miles	mi^2
ha	hectares (10,000 m^2)	2.471	acres	ac
		MASS (weight)		
g	grams	0.0353	Ounces (avdp)	oz
kg	kilograms	2.205	Pounds (avdp)	lb
mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
mL	milliliters	0.034	fluid ounces (US)	fl oz
liters	liters	0.264	Gallons (liq)	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		TEMPERATURE (exact)		
ºC	Celsius temperature	9/5 ºC+32	Fahrenheit temperature	ºF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/cm^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	pound-force	lbf
kPa	kilopascals	0.145	pound-force per square inch	psi

Acknowledgments

This research was made possible with funding from the Idaho Transportation Department (ITD) under AASHTO Service Unit for ITD. The authors express their sincere gratitude to the ITD personnel who assisted in providing successful implementation of a complex Pavement design and analysis program.

We are particularly grateful to ITD's State Design, Materials and Construction, Engineer Frances Hood, State Materials Engineer Mike Santi and Research Program Manager Ned Parrish for their very helpful and cooperative assistance throughout the implementation. Valuable information was obtained from the report RP193, "Implementation of the MEPDG for Flexible Pavements in Idaho," prepared for ITD by the University of Idaho in 2011.

The Applied Research Associates, Inc. staff that assisted with the *User's Guide* includes: Jagannath Mallela, Leslie Titus-Glover, Biplab Bhattacharya, Michael Darter, and Harold Von Quintus.

Table of Contents

Chapter 1. Overview of the *AASHTOWare Pavement ME Design* Software 1
1.1 Overview 1
1.2 System Requirements 4
1.3 Installing *AASHTOWare Pavement ME Design* Software 4
1.4 Uninstalling *AASHTOWare Pavement ME Design* 6
1.5 Running the *AASHTOWare Pavement ME Design* 6
1.6 *AASHTOWare Pavement ME Design* Database 10
1.7 Hierarchical Approach to Design Inputs 10
1.8 General *AASHTOWare Pavement ME Design* Features and Enhancements 12
Chapter 2. General Information Inputs 13
2.1 Design Life 13
2.2 Construction and Traffic Opening Dates 13
2.3 New and Reconstructed Pavement and Rehabilitated Pavement Types Considered by *AASHTOWare Pavement ME Design* 14
2.4 Site and Project Identification 15
Chapter 3. Performance Criteria Inputs 17
Chapter 4. Design Reliability Input 21
Chapter 5. Traffic Inputs 25
5.1 Introduction 25
5.2 Traffic Volume 26
5.3 Traffic Volume Monthly Adjustment Factors 27
5.4 Vehicle Class Distribution 29
5.5 Hourly Truck Distribution 31
5.6 Truck Traffic Growth Factor 32
5.7 Axle Load Distribution 34
5.8 Number of Axles per Truck Type/Class 54
5.9 General Traffic Inputs 55
Chapter 6. Climate Inputs 57
Chapter 7. Pavement Structure Definition and Materials Characterization 65
7.1 Introduction 65
7.2 "Trial Design" Structure 65
7.3 Step 1: Bedrock Layer Soil Characterization 65
7.4 Step 2: Subgrade and Embankment Soil Characterization 66
7.5 Step 3: Base/Subbase Material Characterization 78
7.6 Step 4: Pavement Surface Materials 85
Chapter 8. JPCP Design Features 103
Chapter 9. Rehabilitation Inputs 105
Chapter 10. Performing New or Reconstructed Pavement and Rehabilitation Designs 109
10.1 Steps Required for New or Reconstructed Pavement Design 109
10.2 Steps Required for Rehabilitation Pavement Design 114
10.3 Local Calibration Factors for Idaho 117
Chapter 11. Input/Output Sensitivity Analysis 121
Chapter 12. AASHTOWare Pavement ME Design Outputs Used for Performance Assessment 133
References 141
Appendix A. Idaho New HMA Pavement Design Example 143

Appendix B. Idaho New JPCP Design Example 171
Appendix C. Idaho HMA Overlay Pavement Design Example 201
Appendix D. Idaho JPCP Restoration Design Example 225

List of Tables

Table 1. Recommended Pavement "Design Life" 13
Table 2. Construction and Traffic Opening Date Description 14
Table 3. Description of New/Reconstructed Pavement Types Considered by *AASHTOWare Pavement ME Design* 14
Table 4. Description of Restored JPCP Considered by *AASHTOWare Pavement ME Design* 14
Table 5. Description of HMA and PCC Overlays Considered by *AASHTOWare Pavement ME Design* 15
Table 6. Initial IRI Values for New and Rehabilitated Pavement Design 18
Table 7. Performance Criteria for Use in New HMA Pavement, HMA Overlays, and Composite (HMA-Overlaid Jointed Plain Concrete) Pavement Design 19
Table 8. Performance Criteria for Use in JPCP New, Concrete Pavement Restoration and JPCP Overlays Pavement Design 19
Table 9. Recommended Level of Design Reliability 22
Table 10. WIM Stations in Idaho 26
Table 11. Current and Future Truck Traffic Volume Estimates for Pavement Design 27
Table 12. Recommended *AASHTOWare Pavement ME Design* MAF Inputs for Design in Idaho 28
Table 13. FHWA Vehicle Class Distribution Inputs for Idaho 30
Table 14. Recommended MEPDG Default Hourly Truck Distribution Inputs for Design in Idaho 32
Table 15. Idaho Statewide Average Single-Axle Distribution 38
Table 16. Idaho Statewide Average Tandem-Axle Distribution 39
Table 17. Idaho Statewide Average Tridem-Axle Distribution 40
Table 18. Idaho Statewide Average Quad-Axle Distribution 41
Table 19. Single-Axle Load Distribution for Primarily Loaded TWRG in Idaho 42
Table 20. Tandem-Axle Load Distribution for Primarily Loaded TWRG in Idaho 43
Table 21. Tridem-Axle Load Distribution for Primarily Loaded TWRG in Idaho 44
Table 22. Quad-Axle Load Distribution for Primarily Loaded TWRG in Idaho 45
Table 23. Single-Axle Load Distribution for Moderately Loaded TWRG in Idaho 46
Table 24. Tandem-Axle Load Distribution for Moderately Loaded TWRG in Idaho 47
Table 25. Tridem-Axle Load Distribution for Moderately Loaded TWRG in Idaho 48
Table 26. Quad-Axle Load Distribution for Moderately Loaded TWRG in Idaho 49
Table 27. Single-Axle Load Distribution for Lightly Loaded TWRG in Idaho 50
Table 28. Tandem-Axle Load Distribution for Lightly Loaded TWRG in Idaho 51
Table 29. Tridem-Axle Load Distribution for Lightly Loaded TWRG in Idaho 52
Table 30. Quad-Axle Load Distribution for Lightly Loaded TWRG in Idaho 53
Table 31. WIM Sites Associated with Idaho TWRG 54
Table 32. Recommended Number of Single-, Tandem-, Tridem-, and Quad-Axles per Truck Class for Idaho 54
Table 33. Axle Configuration for Idaho 55
Table 34. Wheelbase (Based on National Measurements) 55
Table 35. Weather Stations for Idaho and Surrounding States 60
Table 36. Guidance on Bedrock Layer Properties 66
Table 37. Recommended Idaho (Level 3) Lab Resilient Modulus for Embankment/ Subgrade at Optimum Moisture for Flexible and Rigid Pavements 76

Table 38. Recommended Level 3 Inputs for Unbound Soils and Embankment Layers............... 78
Table 39. Recommended (Level 2/3) Lab Resilient Modulus for Unbound Base/ Subbase at Optimum Moisture, and In-Place Recycled Materials for Flexible and Rigid Pavements... 81
Table 40. Recommended Level 3 Inputs for Unbound Aggregate for Base and Subbase Layers.. 85
Table 41. Hot Mix Asphalt Class Requirements 86
Table 42. Typical ITD HMA Mix Locations.. 88
Table 43. Dynamic Modulus Values of Typical ITD HMA Mixtures 89
Table 44. Shear Modulus and Phase Angles of Typical ITD HMA Mixtures............................. 93
Table 45. Level 3 Default HMA Inputs for ITD.. 94
Table 46. Poisson's Ratio Recommended for HMA 97
Table 47. *AASHTOWare Pavement ME Design* National Defaults Based on Concrete Coarse Aggregate Geological Class.. 99
Table 48. PCC Level 3 Material Properties Inputs for *AASHTOWare Pavement ME Design* 99
Table 49. Distress Types and Severity Levels Recommended for Assessing Rigid Pavement Structural Adequacy... 100
Table 50. Existing Intact PCC Typical Modulus Ranges 100
Table 51. Fractured (Rubblized) PCC Resilient Modulus for Design 101
Table 52. PCC Level 3 inputs for Existing Intact and Fractured PCC 102
Table 53. Summary of Design Recommendations for Idaho New/Reconstructed JPCP 103
Table 54. Characterization for HMA Overlay of Existing HMA Pavement 106
Table 55. Characterization for Aggregate Base and Unbound Embankment/Subgrade of Existing HMA Pavement 107
Table 56. Characterization for HMA Overlay of Existing JPCP.. 108
Table 57. Recommendations for Optimizing HMA Pavement Design 113
Table 58. Recommendations for Optimizing JPCP Design... 114
Table 59. Sensitivity Results for New/Reconstructed HMA Pavements................................... 121
Table 60. Sensitivity Results for New/Reconstructed JPCP and Composite Pavements 122
Table 61. Excel Distress Summary Showing IRI, Permanent Deformation, AC Thermal Fracture, Total Cracking (Alligator Reflective and Bottom-Up Alligator) for New/Reconstructed HMA.. 137
Table 62. Excel New/Reconstructed JPCP IRI, Joint Faulting, and Slab Transverse Fatigue Cracking Over Time 140
Table 63. Key Inputs Required for New or Reconstructed HMA Pavement Design 144
Table 64. Traffic Input Data Used for this Design Example 148
Table 65. Required Engineering Properties for the "Trial Design" Subgrade........................... 153
Table 66. Required Engineering Properties for the "Trial Design" HMA Layer 157
Table 67. Key Inputs Required for New or Reconstructed JPCP Design.................................. 173
Table 68. Traffic Input Data Used for This Design Example.. 177
Table 69. Required Engineering Properties for the "Trial Design" Natural Subgrade............... 182
Table 70. Required Engineering Properties for the "Trial Design" PCC Layer 187
Table 71. Summary of New JPCP "Trial Design" Inputs... 189
Table 72. Key Inputs Required for HMA Overlay Pavement Design 202
Table 73. Required Engineering Properties for the Existing Subgrade 207
Table 74. Required Engineering Properties for the Existing HMA Layer 211
Table 75. Characterization of Existing HMA Pavement 213
Table 76. Characterization for Aggregate Base and Unbound Embankment/Subgrade of Existing HMA Pavement.. 214

Table 77. Key Inputs Required for JPCP Restoration or CPR Design 226
Table 78. Required Engineering Properties for the Existing Natural Subgrade 232
Table 79. Required Engineering Properties for the Existing PCC Layer 235
Table 80. Summary of Existing JPCP Design Inputs 237

List of Equations

Equation 1. 68
Equation 2 68
Equation 3 72
Equation 4 72
Equation 5 72
Equation 6 72
Equation 7 82
Equation 8 82
Equation 9 95
Equation 10 96
Equation 11 96
Equation 12 96
Equation 13 97
Equation 14 102

List of Figures

Figure 1. *AASHTOWare Pavement ME Design* Overall Iterative Design Process 3
Figure 2. Location of *AASHTOWare Pavement ME Design* “HELP System” Document 4
Figure 3. *AASHTOWare Pavement ME Design* Default Window 5
Figure 4. *AASHTOWare Pavement ME Design* “HELP System” Document............................. 5
Figure 5. *AASHTOWare Pavement ME Design* Splash Screen 7
Figure 6. Open *AASHTOWare Pavement ME Design* Projects 7
Figure 7. *AASHTOWare Pavement ME Design* Main Window 8
Figure 8. Project Tab.. 8
Figure 9. Color-Coding to Assist User-Input Accuracy 9
Figure 10. Project Identifiers for Site and Project Identification.. 15
Figure 11. Illustration of the Effect of Design Reliability on HMA Thickness for a Project Site Under Heavy Truck Traffic 22
Figure 12. Illustration of the Effect of Design Reliability on Required JPCP Slab Thickness for a Given Project Site Under Heavy Truck Traffic 23
Figure 13. Idaho vs. MEPDG Monthly Adjustment Factors 28
Figure 14. Illustration of FHWA/AASHTO Vehicle Class Type Description 29
Figure 15. Plot Showing Variation of VCD Among WIM Sites in Idaho.................................. 31
Figure 16. Plot Showing MEPDG Default Hourly Truck Distribution 31
Figure 17. *AASHTOWare Pavement ME Design* Screenshot Showing Inputs for Nature and Rate of Traffic Growth Relative to the Base Year 33
Figure 18. Idaho Truck Class 5 Single-Axle Load Distribution.. 35
Figure 19. Idaho Truck Class 9 Single-Axle Load Distribution.. 35
Figure 20. Idaho Truck Class 9 Tandem-Axle Load Distribution 36
Figure 21. Idaho Truck Class 7 Tridem-Axle Load Distribution 36
Figure 22. Idaho Truck Class 10 Tridem-Axle Load Distribution 37
Figure 23. Idaho Truck Class 10 Quad-Axle Load Distribution 37
Figure 24. Schematic Illustration of Mean Wheel Location... 56
Figure 25. Examples of Temperature and Moisture Variations Across the State of Idaho 57
Figure 26. Idaho and Surrounding Weather Stations Available for Pavement Design 62
Figure 27. Well Sites Information on Groundwater in Idaho 63
Figure 28. Typical Correlations Between USCS and AASHTO Soil Classification.................... 77
Figure 29. Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers........... 84
Figure 30. Typical Low Temperature Binder by Geographical Location in Idaho 94
Figure 31. Typical High Temperature Binder by Geographical Location in Idaho...................... 95
Figure 32. Wyoming Calibration Coefficients and Standard Error Prediction Models for New HMA Pavement 118
Figure 33. Wyoming Calibration Coefficients and Standard Error Prediction Models for HMA Over Existing HMA Pavement 119
Figure 34. NCHRP 20-07 Calibration Coefficients and Standard Error Prediction Models for New JPCP.. 120
Figure 35. Effect of HMA Thickness on HMA Bottom-Up Alligator Fatigue Cracking........... 123
Figure 36. Effect of HMA Thickness on Rutting 123
Figure 37. Effect of HMA In Situ Air Void Content on Fatigue (Alligator) Cracking 124
Figure 38. Effect of HMA In Situ Air Void Content on Rutting.. 124
Figure 39. Effect of HMA Volumetric Binder Content on Fatigue (Alligator) Cracking 125
Figure 40. Effect of HMA Volumetric Binder Content on Rutting... 125

Figure 41. Effect of Climate on Fatigue (Alligator) Cracking.. 126
Figure 42. Effect of Climate on Rutting 126
Figure 43. Effect of HMA Binder Grade on Alligator Cracking.. 127
Figure 44. Effect of HMA Binder Grade on Rutting.. 127
Figure 45. Effect of JPCP Transverse Joint Dowel Diameter and PCC Thickness on Joint Faulting 128
Figure 46. Effect of PCC Slab Thickness on Transverse Cracking of JPCP............................. 128
Figure 47. Effect of PCC Coefficient of Thermal Expansion on Transverse Cracking of JPCP.. 129
Figure 48. Effect of Shoulder Support on Transverse Joint Faulting of JPCP 129
Figure 49. Effect of Edge Support on Slab Transverse Cracking for JPCP 130
Figure 50. Effect of Climate on Slab Transverse Joint Faulting for Doweled JPCP................. 130
Figure 51. Effect of Climate on Slab Transverse Cracking for JPCP....................................... 131
Figure 52. Excel Reliability Summary for New/Reconstructed HMA Design........................... 138
Figure 53. Excel Reliability Summary for New/Reconstructed JPCP....................................... 139
Figure 54. New HMA Pavement Design Example Location... 143
Figure 55. New HMA Pavement Design Example Construction Month and Year 145
Figure 56. Performance Criteria and Reliability for New HMA Pavement Design Example 147
Figure 57. General Traffic Inputs for New HMA Pavement Design Example 149
Figure 58. Single-Axle Load Distribution Inputs for New HMA Pavement Design Example .. 149
Figure 59. Selecting Virtual Weather Stations for New HMA Pavement Design Example....... 151
Figure 60. New HMA Pavement Design Structure 152
Figure 61. Subgrade Engineering Properties Input Screen for New HMA Pavement................ 154
Figure 62. Subgrade Level 3 Resilient Modulus Input Screen for New HMA Pavement.......... 154
Figure 63. Selecting Base Modulus for New HMA Pavement Design...................................... 156
Figure 64. Screen for HMA Layer Binder and Mix Inputs for the Example New HMA Pavement Design 157
Figure 65. Screen for HMA Layer Gradation Inputs for the Example New HMA Pavement Design 158
Figure 66. HMA Layer Creep Compliance for the Example New HMA Pavement Design...... 158
Figure 67. *AASHTOWare Pavement ME Design* Input Screen for Additional Inputs\ Required for HMA Surface Layer for the New HMA Pavement Design Example... 159
Figure 68. *AASHTOWare Pavement ME Design* Input Screen for New HMA Pavement Layer Interface Friction 159
Figure 69. New Flexible Pavement Calibration Coefficients 160
Figure 70. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for New HMA Pavement.. 162
Figure 71. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for New HMA Pavement.. 163
Figure 72. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for New HMA Pavement 164
Figure 73. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (Projection of AADTT) for New HMA Pavement...................................... 165
Figure 74. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for New HMA Pavement 166
Figure 75. Optimized New HMA Pavement Design Inputs and Outputs Summary 169
Figure 76. New JPCP Design Example Location 171
Figure 77. New JPCP Design Example Construction Month and Year 174
Figure 78. Performance Criteria and Reliability for New JPCP Design Example 175
Figure 79. General Traffic Inputs for New JPCP Design Example... 178

Figure 80. Single Axle Load Distribution Inputs for New JPCP Design Example 178
Figure 81. Selecting Virtual Weather Stations for New JPCP Design Example 180
Figure 82. New JPCP Design Structure 180
Figure 83. Natural Subgrade Engineering Properties Input Screen for New JPCP 183
Figure 84. Natural Subgrade Level 3 Resilient Modulus Input Screen for New JPCP 184
Figure 85. Compacted Subgrade Engineering Properties for New JPCP 185
Figure 86. Granular Subbase Engineering Properties for New JPCP 186
Figure 87. Screen for PCC Material for the Example New JPCP Design 188
Figure 88. Screen for Cement Type for the Example New JPCP Design 188
Figure 89. Screen for PCC Strength and Modulus for the Example New JPCP Design 189
Figure 90. Screen for New JPCP Design Inputs 190
Figure 91. New JPCP Calibration Coefficients 191
Figure 92. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for New JPCP 193
Figure 93. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for New JPCP 193
Figure 94. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for New JPCP 194
Figure 95. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (Projection of AADTT) for New JPCP 195
Figure 96. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for New JPCP 196
Figure 97. JPCP Optimized Rules 198
Figure 98. Optimized New JPCP Design Inputs and Outputs Summary 199
Figure 99. HMA Overlay Pavement Design Example Location 201
Figure 100. HMA Overlay Pavement Design Example Construction Month and Year 203
Figure 101. Performance Criteria and Reliability for HMA Overlay Pavement Design Example 205
Figure 102. Existing HMA Pavement Structure 206
Figure 103. Existing Subgrade Engineering Properties Input Screen 208
Figure 104. Existing Subgrade Level 3 Resilient Modulus Input Screen 209
Figure 105. Selecting Base Modulus for HMA Overlay Design 210
Figure 106. Screen for Existing HMA Layer Binder and Mix Inputs 211
Figure 107. Screen for Existing HMA Layer Gradation Inputs 212
Figure 108. HMA Overlay Layer Creep Compliance Inputs 212
Figure 109. *AASHTOWare Pavement ME Design* Input Screen for HMA Rehabilitation Inputs 214
Figure 110. *AASHTOWare Pavement ME Design* Input Screen for Additional Inputs Required for HMA Layer 215
Figure 111. *AASHTOWare Pavement ME Design* Input Screen for HMA Pavement Layer Interface Friction 215
Figure 112. Flexible Pavement Rehabilitation Calibration Coefficients 216
Figure 113. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for HMA Overlay Pavement 218
Figure 114. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for HMA Overlay Pavement 219
Figure 115. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for HMA Overlay Pavement 220

Figure 116. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (projection of AADTT) for HMA Overlay Pavement 221
Figure 117. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for HMA Overlay Pavement 222
Figure 118. Optimized Pavement Design Inputs and Outputs Summary 224
Figure 119. JPCP Restoration Design Example Location 225
Figure 120. JPCP Restoration Design Example Construction (month/year) 227
Figure 121. Performance Criteria and Reliability for JPCP Restoration Design Example 228
Figure 122. JPCP Rehabilitation Input Screen 229
Figure 123. Foundation Support Input Screen 230
Figure 124. Existing JPCP Existing Design Structure 230
Figure 125. Natural Subgrade Engineering Properties Input Screen for Existing JPCP 233
Figure 126. Compacted Subgrade Engineering Properties for Existing JPCP 234
Figure 127. Screen for PCC Material for the Example Existing JPCP 236
Figure 128. Screen for Cement Type for the Example Existing JPCP Design 236
Figure 129. Screen for PCC Strength and Modulus for the Example Existing JPCP 237
Figure 130. Screen for Existing JPCP Design Inputs 238
Figure 131. JPCP Restoration Calibration Coefficients 238
Figure 132. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for JPCP Restoration 240
Figure 133. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for JPCP Restoration 241
Figure 134. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for JPCP Restoration 242
Figure 135. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (projection of AADTT) for JPCP Restoration 243
Figure 136. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for JPCP Restoration 244
Figure 137. Optimized JPCP Restoration Design Inputs and Outputs Summary 246

Preface

This design manual is intended for use by Idaho pavement and materials engineers as well as consulting engineers. The design procedure is based on the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed by a team of nationally recognized engineers from Applied Research Associates, Inc. (ARA) and Arizona State University, along with several other expert consultants.

The first phase of research that resulted in the development of MEPDG was completed in 2004 with the release of a research version for peer review and evaluation. After national peer review, the final research version was released in April 2007 for further consideration by the American Association of State Highway and Transportation Officials (AASHTO) as a standard. AASHTO accepted and adopted this version in 2008, and the pavement design procedure was documented as the *AASHTO Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice*. The *AASHTO MEPDG Manual of Practice* provides the best available engineering documentation of new pavement design procedures.

AASHTO further developed *AASHTOWare Pavement ME Design* 1.0™ as the next generation of *AASHTOWare® Pavement ME Design* software, which builds upon the research-grade MEPDG software and is intended to support the *Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice*. *AASHTOWare Pavement ME Design* is a production-ready software tool that supports day-to-day pavement design and analysis. *AASHTOWare Pavement ME Design* represents a major change in the way pavement design is performed, by providing a direct tie between pavement materials, structure, construction practices, climate, traffic, and pavement design features.

This *User's Guide* provides the information necessary for Idaho pavement design engineers and consultants to begin to use the *AASHTOWare Pavement ME Design* software for new and rehabilitated pavement design. This is a stand-alone guide, but it draws from other Idaho pavement materials/construction manuals as needed, as well as research reports from the University of Idaho for traffic, hot mix asphalt (HMA), unbound materials, and various other inputs.

This manual is divided into 12 chapters and 4 appendices of examples, and it covers topics including traffic characterization, materials characterization, flexible pavement design, rigid pavement design, rehabilitation with asphalt concrete or portland cement concrete (PCC), sensitivity of inputs, and design examples.

Special Notice Regarding Calibration of Distress and IRI Models

It is emphasized that only limited local calibrations of the design inputs or distress and International Roughness Index (IRI) prediction models have been accomplished for Idaho. These include traffic inputs and HMA inputs. All other recommended and default inputs and calibration coefficients were based on "global" calibration that utilized several hundred pavements from throughout the United States and southern Canada. A few sites from Idaho were included in the global calibration. In general, the global input defaults and calibration coefficients should work reasonably well in Idaho; however, the following limitations exist:

- **Design Inputs:** It is extremely important to determine proper Idaho input procedures and recommended default values. This has only been accomplished for many of the traffic inputs and HMA inputs to date. Additional local calibration is needed to establish inputs for unbound materials (base, sub-base, embankment, and subgrade), concrete, design factors, climate factors (e.g., water table), and rehabilitation (condition ratings). In addition, selection of suitable Idaho design reliability and performance criteria (e.g., limiting fatigue cracking, rutting, IRI) is absolutely essential to successful implementation.

- **Calibration Coefficients of Distress/IRI Models:** It is extremely important to verify that the global prediction models are unbiased for the design of pavements in Idaho. If one distress or IRI model is consistently over-predicting, for example, the result would be design project problems and, ultimately, designer rejection of the procedure as inadequate.

For these reasons, nearly all Department of Transportations (DOTs) conduct a local calibration to establish local inputs and calibration factors that result in unbiased predictions. This requires Idaho traffic, materials testing, climate, and pavement performance data to establish the accuracy and bias of the distress and IRI prediction models in a State. Thus, it is recommended that Idaho pursue additional local calibration activities, as noted above as soon as possible.

Chapter 1 Overview of the *AASHTOWare Pavement ME Design* Software

1.1 Overview

The *AASHTOWare Pavement ME Design* procedure is based on mechanistic-empirical (M-E) design concepts. This means that the design procedure calculates pavement responses such as stresses, strains, and deflections under axle loads and climatic conditions and then accumulates the damage over the design analysis period. The procedure then empirically relates calculated damage over time to pavement distresses and smoothness based on performance of actual projects throughout the U.S. with a few of these in Idaho and many others in surrounding States. There has been no local calibration to Idaho-specific conditions to date to determine how well the national prediction models and inputs relate to pavement performance in Idaho. (Refer to the special notice in the Preface.)

AASHTOWare Pavement ME Design uses a mix of algorithms and models to

1. Characterize new or existing pavement foundation, structure, layer materials, traffic, and climate.

2. Simulate stress/strains/deflection due to the interactions between applied traffic load and climate.

3. Calculate the resulting damage manifested as distress and smoothness loss over the "Design Life" of a pavement.

AASHTOWare Pavement ME Design performs a wide range of analysis and calculations in a rapid, easy-to-use format. With its many customized features, *AASHTOWare Pavement ME Design* will help simplify the pavement design process and result in improved, cost-effective designs.

The algorithms and models used for pavement design are presented in the 2008 *AASHTOWare Pavement ME Design PDG Manual of Practice* and several National Cooperative Highways Research Program (NCHRP) research reports (see the references listed at the end of this guide). The design models were calibrated and validated using extensive U.S. and southern Canada pavement performance data, with a few sections located in Idaho. As noted in the Special Notice in the Preface, local calibration using Idaho-specific inputs and data has not been accomplished. Therefore, ITD pavement engineers and others initially will provide thickness designs utilizing pavement design methods in Section 500 of the *ITD Materials Manual* and analyze them using the *AASHTOWare Pavement ME Design* with the global calibration factors provided. Caution is advised because some of the models may be biased, meaning they consistently over- or under-predict distress or IRI. Therefore, if the predicted results appear to be unreasonable for a project, this should be reported to the State Materials Engineer for further consideration.

This *ITD User's Guide* presents the following information to assist Idaho pavement design engineers in using the *AASHTOWare Pavement ME Design* software:

- Overview of the *AASHTOWare Pavement ME Design* procedure.
- HELP information on installation of the software.
- Guidelines for obtaining all needed inputs (based on limited testing of Idaho materials, results from other surrounding States, and engineering judgment).
- Guidance to perform pavement design using the software for the following pavement types:
    - New or reconstructed hot mixed asphalt (HMA) pavement.
    - New or reconstructed jointed plain concrete pavement (JPCP).
    - HMA rehabilitation – HMA overlay on existing HMA.
    - HMA rehabilitation – HMA on existing JPCP.
    - JPCP rehabilitation – JPCP overlay on existing JPCP or HMA.
    - JPCP restoration – Surface retexturing and repair of JPCP.
- Examples of pavement design using the *AASHTOWare Pavement ME Design* software.
    - New or reconstructed HMA pavement.
    - New or reconstructed JPCP.
    - HMA overlay on existing HMA.
    - JPCP overlay on existing HMA.
    - Concrete pavement restoration (CPR).

The *AASHTOWare Pavement ME Design* computations are an iterative process**,** as shown in the flowchart in Figure 1. The software provides:

- User interface to enter design variables.
- Computational models for month-by-month analysis and performance prediction.
- Results and outputs from the analyses for decision making.
- Outputs in both PDF and Excel formats suitable for use in design reports.

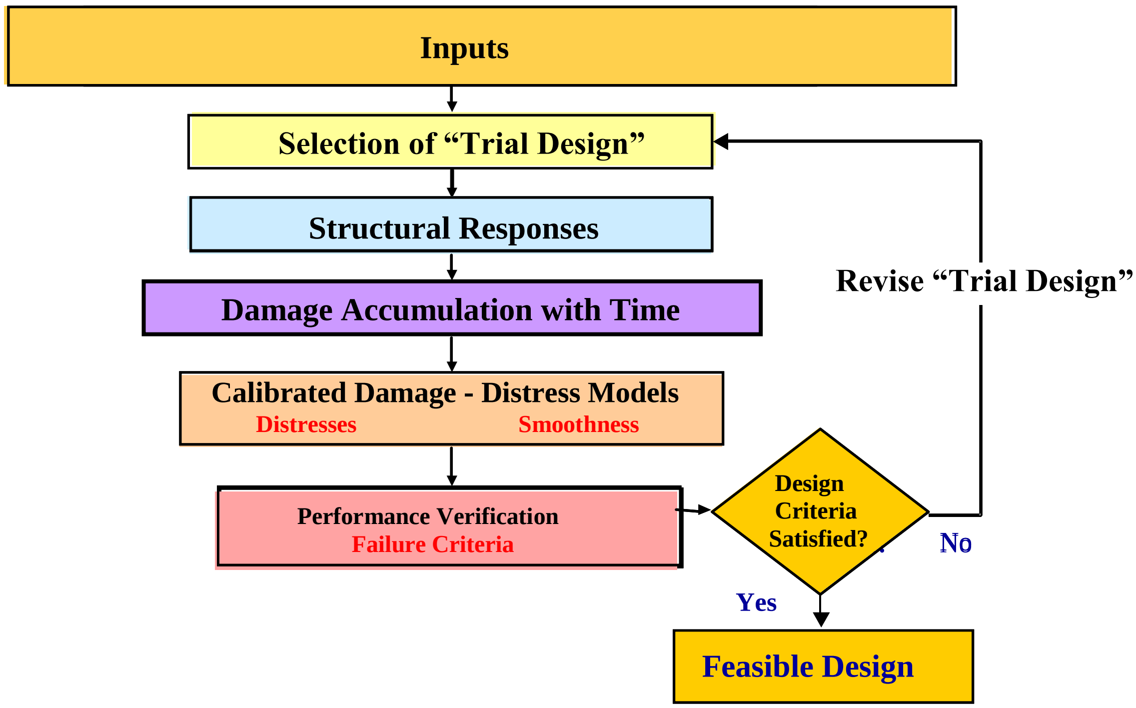


Figure 1. *AASHTOWare Pavement ME Design* Overall Iterative Design Process

***AASHTOWare Pavement ME Design* Iterative Process**

1. The designer develops a "Trial Design" using the existing ITD/AASHTO procedures as a starting point. For widening, use a design similar to the existing pavement and obtain all additional inputs for Pavement ME.

2. The software computes the traffic, climate, damage, key distresses (fatigue cracking, rutting, joint faulting, etc.), and IRI over the "Design Life" on a month-by-month basis (two-week basis for HMA pavement).

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

4. The design may be modified as needed to meet performance and reliability requirements. *AASHTOWare Pavement ME Design* can iterate on design thickness until all of the performance and reliability criteria are met for most projects and most criteria.

1.2 System Requirements

To run *AASHTOWare Pavement ME Design* on your computer, the following minimum hardware requirements must be met.

- Minimum computer hardware requirements are 2GB RAM and 1.9GHz processor clock speed. However, it is strongly recommended to use a computer with minimum 4 GB RAM and 80 GB available hard drive space for multiple project runs such as batch, sensitivity, and thickness optimization. Screen resolution of 1024 × 768 or higher is recommended.

- From a performance standpoint, RAM is very important for computation time. The program is also built to take advantage of multi-core machines to complete analyses faster. A larger hard disk space may be required if program outputs from several projects are to be stored.

1.3 Installing AASHTOWare Pavement ME Design Software

Installation is fully explained in the *AASHTOWare Pavement ME Design* software "HELP System" document. Since the details of this process are likely to change over time, they are not repeated here. The "HELP System" document can be easily obtained in two ways:

- From Program Files under Local Disk (C:) click *AASHTOWare* folder, then select ME Design folder (see Figure 2).

- Press the F1 key after opening the software (see Figures 3 and 4).

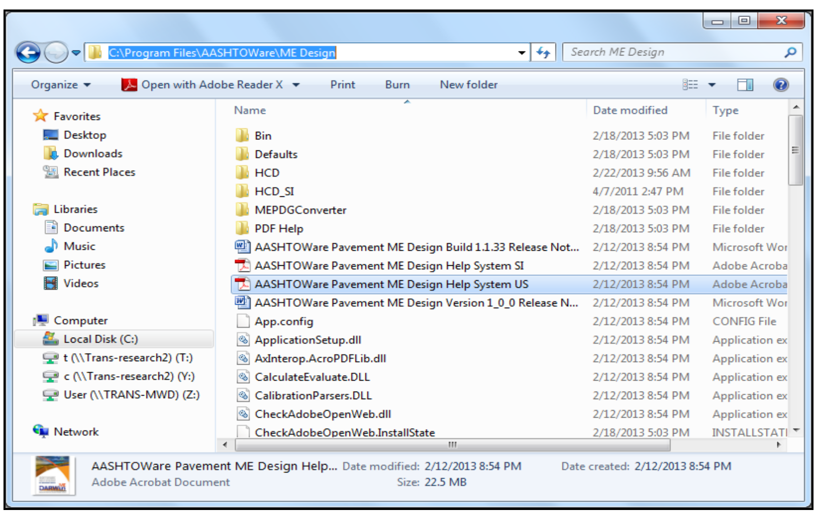


Figure 2. Location of *AASHTOWare Pavement ME Design* "HELP System" Document

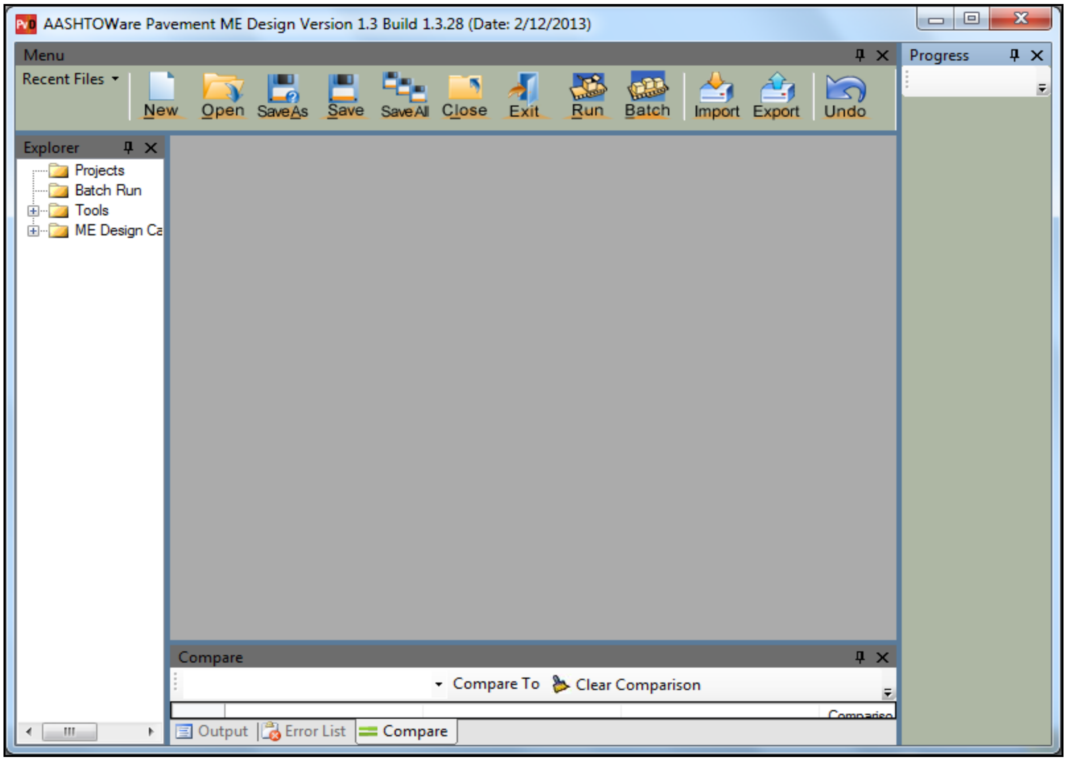


Figure 3. *AASHTOWare Pavement ME Design* Default Window

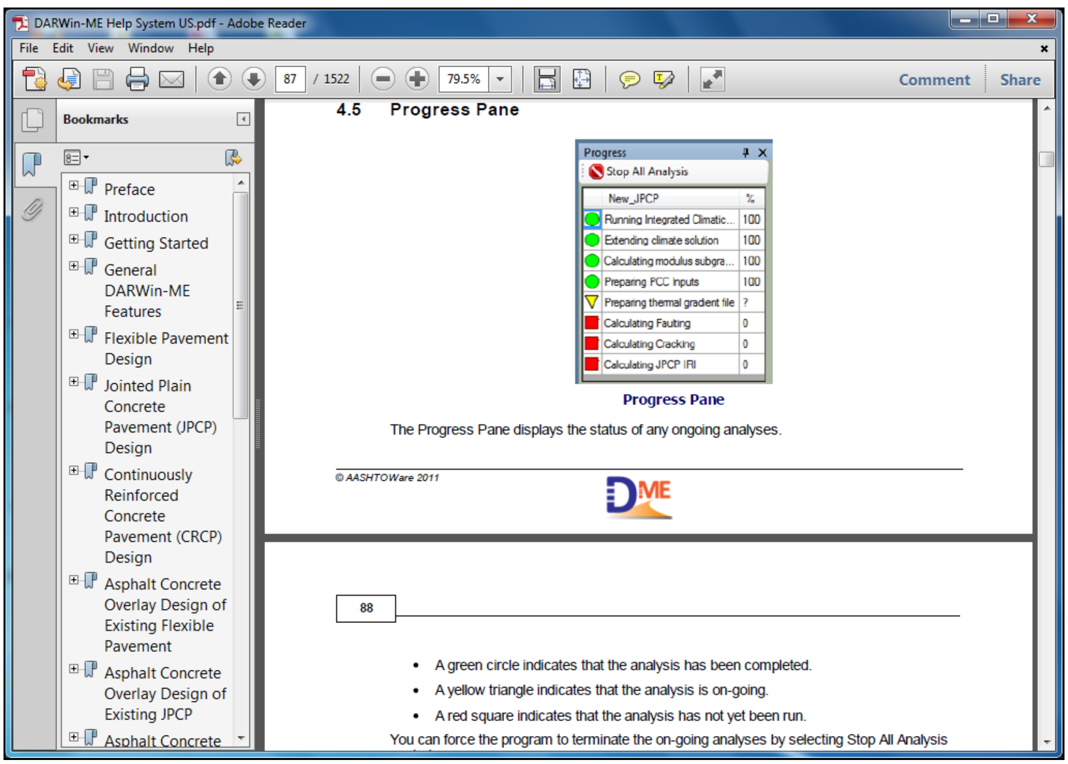


Figure 4. *AASHTOWare Pavement ME Design* "HELP System" Document

1.4 Uninstalling AASHTOWare Pavement ME Design

NEVER just delete the various files of the *AASHTOWare Pavement ME Design* software. If uninstallation of *AASHTOWare Pavement ME Design* is necessary, use the procedure below:

1. Select the Windows Start menu.
2. Select All Programs.
3. Select the *AASHTOWare Pavement ME Design* folder.
4. Select Uninstall *AASHTOWare Pavement ME Design*.
5. Uninstall the software. An updated version of the software can be installed immediately, if desired.

NOTE: This process does not remove the "hcd" (weather station files) under the folder. This folder must be manually deleted, if desired. If old MEPDG weather station files exist, it is recommended to remove all of these and download the new weather stations.

1.5 Running the AASHTOWare Pavement ME Design

An *AASHTOWare Pavement ME Design* program will be added to your Windows Start menu during installation, and an icon will be added to your computer's desktop.

1. Click the Start button in the bottom left corner of your screen to find *AASHTOWare Pavement ME Design*.
2. Go to the Programs option to see a list of folders and programs.
3. Select the *AASHTOWare Pavement ME Design* folder and click on the icon.

The program can also be run by double-clicking the *AASHTOWare Pavement ME Design* icon on the desktop.

The software opens with a splash screen, shown in Figure 5. A new file must be opened for each new project, much like opening a new file for each document in a word processor or other standard Windows applications. However, as many as 10 projects can be opened together by clicking the "Open" menu in *AASHTOWare Pavement ME Design* and selecting 10 projects (see Figure 6). Click on "New" in the tool bar to open a new project. A typical layout of the program is shown in Figures 7 and 8. When more than one project is open, the user should use caution to ensure they are inputting or modifying the specific project of interest. It is easy to modify the wrong project when more than one is open at the same time. Initially, it is best to have only one project open at the same time to avoid this type of error.

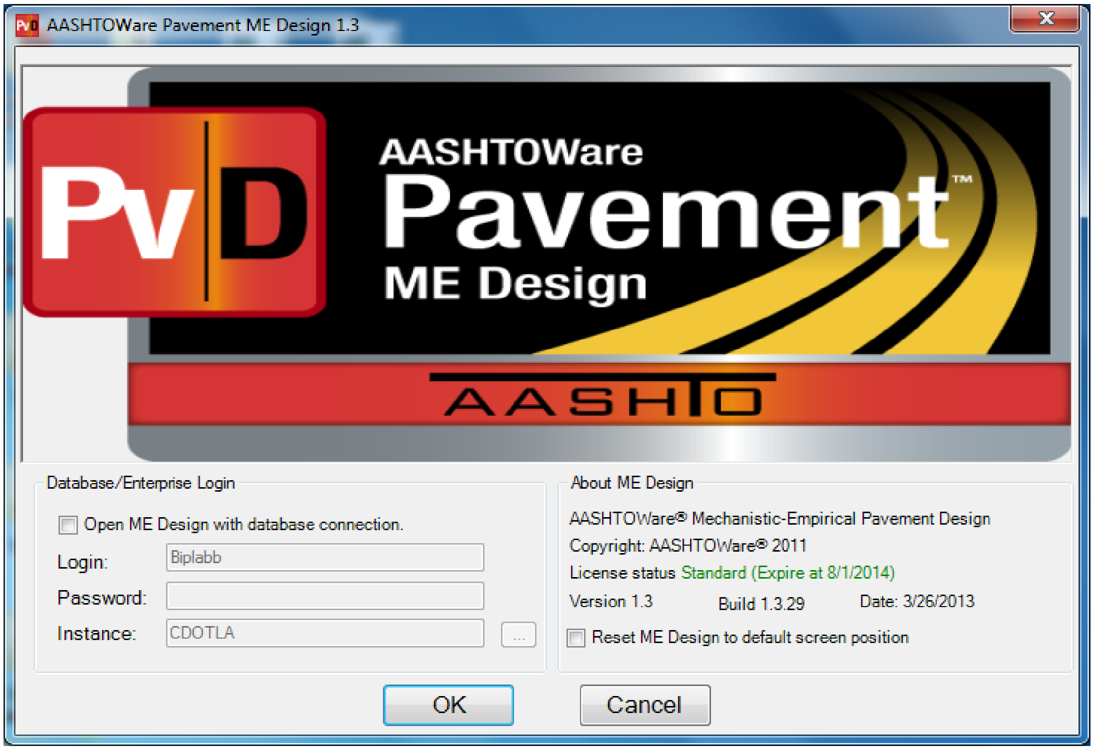


Figure 5. *AASHTOWare Pavement ME Design* Splash Screen

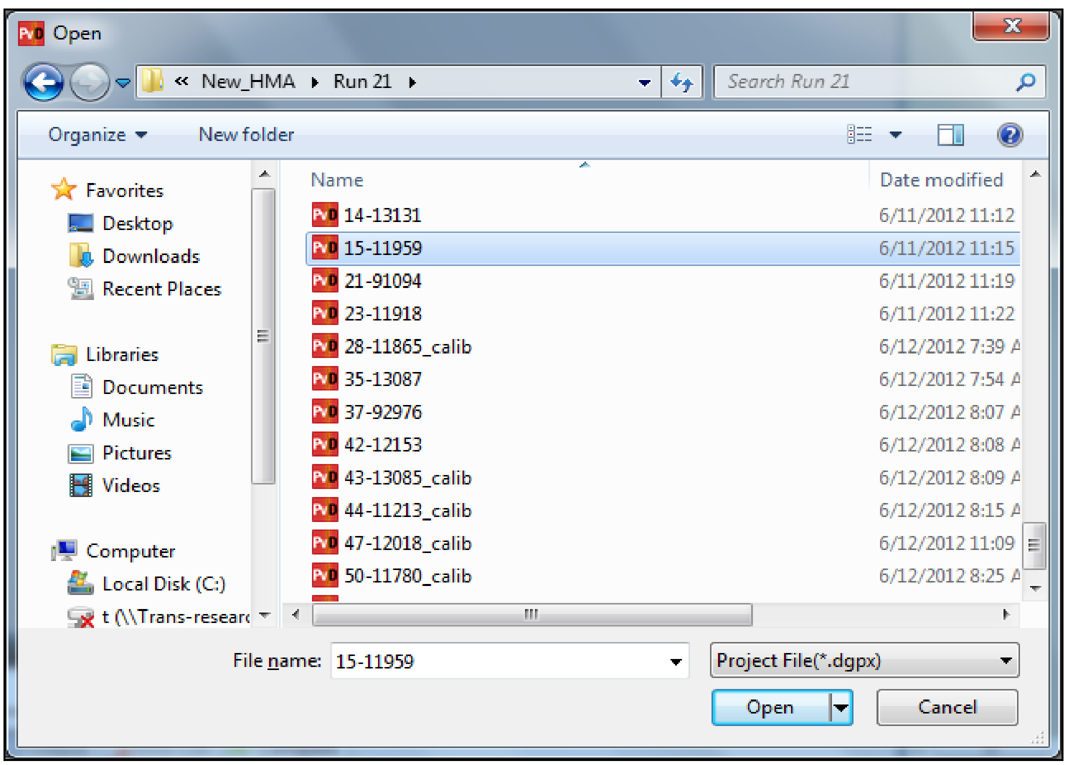


Figure 6. Open *AASHTOWare Pavement ME Design* Projects

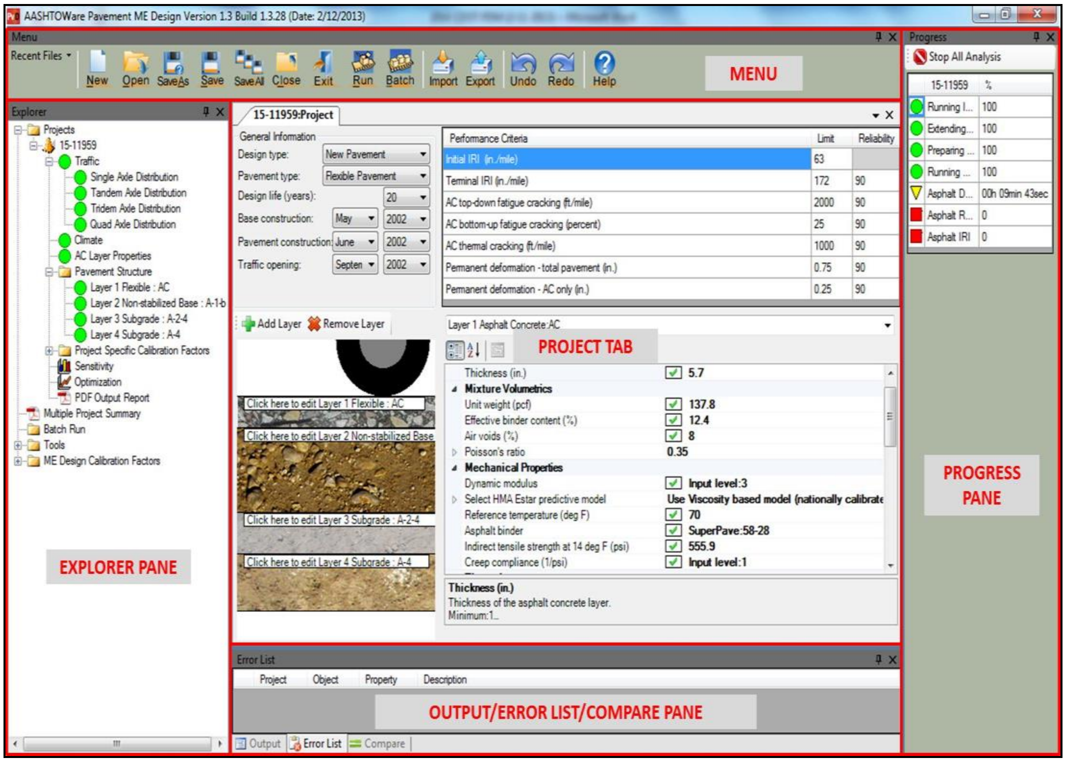


Figure 7. *AASHTOWare Pavement ME Design* Main Window

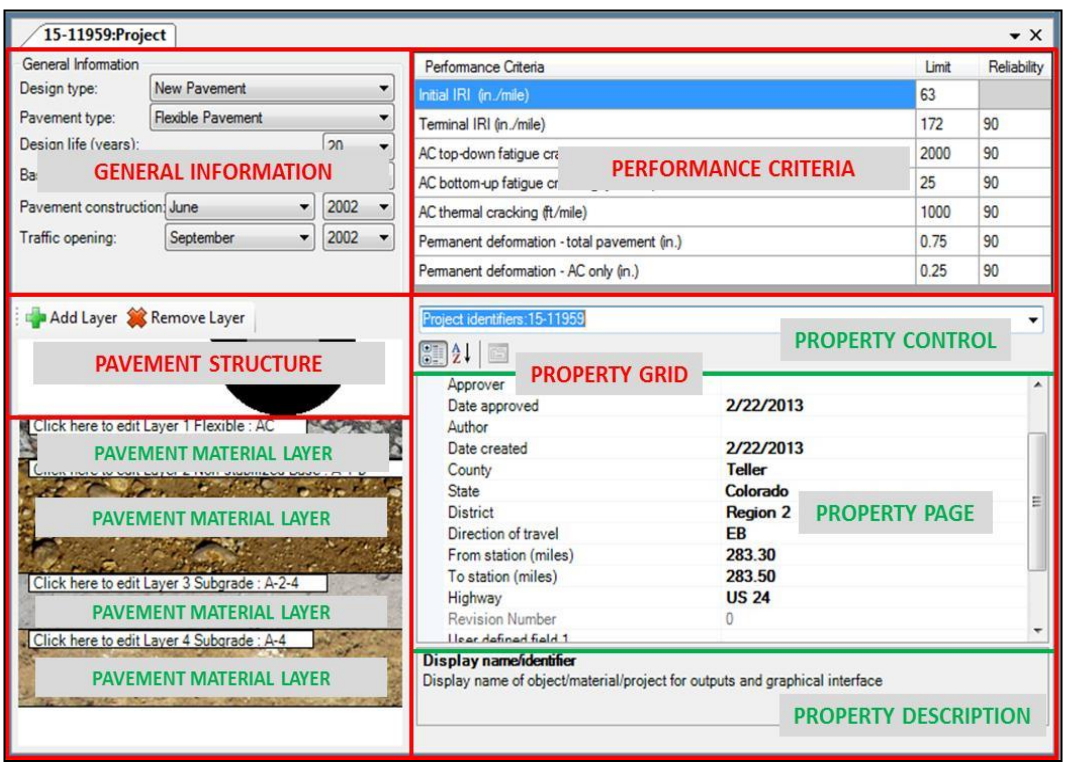


Figure 8. Project Tab

The user first provides the general project information and then enters data in three main categories: Traffic, Climate, and Structure. All inputs for the program are color-coded, as shown in Figure 9. Input screens that require user-entry of data are coded red. Those that have default values but are not yet verified and accepted by the user are coded yellow. Default inputs that have been verified and accepted by the user or any design-specific inputs entered by the user are coded green. The program will not run until all input screens are either yellow or green.

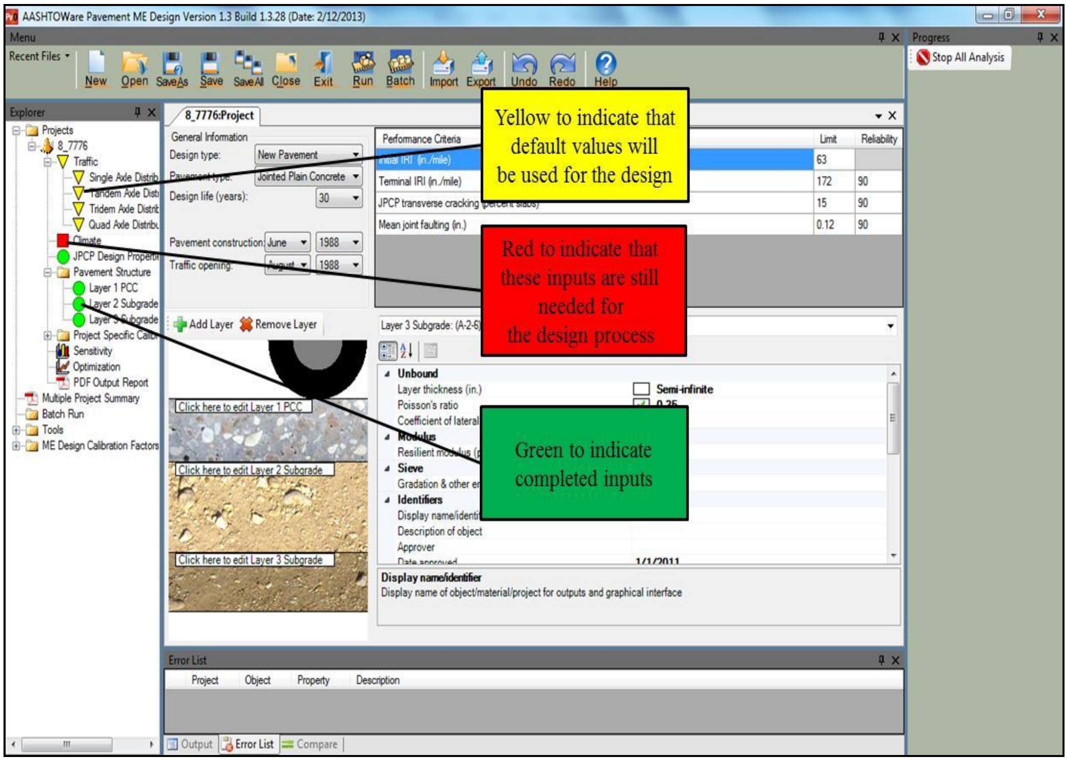


Figure 9. Color-Coding to Assist User-Input Accuracy

The user may choose to run the analysis by clicking on the Run button after all inputs are provided for the "Trial Design." The software executes the damage analysis and the performance prediction engines for the "Trial Design" input when this is done. The user can view input and output summaries created by the program when the execution of the run is complete. The program creates a summary of all inputs of the "Trial Design." It also provides an output summary of the distress and performance prediction in both tabular and graphical formats. All charts are plotted in both PDF and Excel and can be incorporated easily into electronic documents and reports.

1.6 AASHTOWare Pavement ME Design Database

AASHTOWare Pavement ME Design now includes an enterprise option for saving, searching, and loading projects utilizing a relational database. This feature allows users to store and retrieve data at varying degrees of granularity, from entire projects down through data from individual objects such as pavement layers, materials, traffic, climate, back-calculation, etc. MS SQL or Oracle database environments are available.

1.7 Hierarchical Approach to Design Inputs

The hierarchical approach to design inputs is a feature of the *AASHTOWare Pavement ME Design* not found in previous versions of the *AASHTO Design Guide* (i.e., 1993 or earlier). This approach provides the designer with flexibility in obtaining the inputs for a design project based on the criticality of the project and the available resources. The hierarchical approach is employed with regard to traffic, materials, and environmental inputs.

Inputs may be obtained using a mix of levels for a given design project, such as concrete modulus of rupture from Level 1 testing and modulus of elasticity from Level 3 correlation, traffic load spectra from Level 2, and subgrade M_r from Level 3 correlation with soil class. It is important to realize that no matter what input design levels are used, the computational algorithm for damage is exactly the same**.** The same models and procedures are used to predict distress and smoothness no matter what levels are used to obtain the design inputs. There is no such thing as a “Level 1” analysis; rather, a design may be developed using mostly Level 1 inputs. All projects have a wide range of inputs at different levels.

Currently, the *AASHTOWare Pavement ME Design* input level has no effect other than accuracy of the input itself, which of course is important for critical inputs. The only exception to this general rule is the HMA thermal fracture model, which has three different formulations of the design reliability equation corresponding to each of the three input levels. Future versions of *AASHTOWare Pavement ME Design* may link input accuracy level to design reliability for other models.

Of course, the better the design inputs, the better the design. Thus, the more Level 1 and Level 2 design inputs used the better. When local calibration is done in Idaho many default (or typical) design inputs will be established, such as concrete strength, effective air voids of HMA, axle load distributions, initial IRI, and others that can be used to produce more valid and acceptable designs. *ITD's User's Guide* includes many estimated Idaho “Default” design inputs for materials, traffic, climate, and design. Some of the traffic and HMA inputs are based on Idaho data. Others are based on typical values obtained from the global calibration and also from surrounding States.

Eventually, there will be a database of Idaho projects that will have various inputs used in the local calibration. Designers can make use of these inputs for similar projects in their areas of work. This will be another valuable result of local calibration.

AASHTOWare Pavement ME Design **Hierarchical Input Definition**

- **Level 1:** Material input requires laboratory or field testing such as the dynamic modulus (E*) testing of HMA, coefficient of thermal expansion of concrete (CTE), or Falling Weight Deflectometer (FWD) deflection testing. Level 1 inputs for traffic require on-site measurement of axle load distribution, truck lane usage, and truck classification. Obtaining Level 1 inputs requires more resources and time than other levels. Level 1 inputs typically are used for designing heavily trafficked pavements or wherever there are dire safety or economic consequences of early failure. Design-Build (DB) projects often provide the opportunity to test materials and thus use Level 1 inputs.

- **Level 2:** Inputs are estimated through correlations of simpler tests with the more complicated inputs for *AASHTOWare Pavement ME Design*. Examples include estimating HMA E* from binder, aggregate, and mix properties, estimating portland cement concrete (PCC) elastic moduli from compressive strength tests, estimating unbound material M_r from R-value or California Bearing Ratio (CBR) tests, or using traffic classification data based on functional class of highways in the State. This level is used when resources or testing equipment are not available for the tests required for Level 1.

- **Level 3:** Inputs are user-selected values or typical averages for the region. Examples include default unbound materials M_r values from limited research study testing or default PCC CTE for a given coarse aggregate type. This level might be used for designs where there is minimal safety or economic consequences of early failure, such as lower volume roads.

The designer will obtain the inputs that are appropriate and practical for the magnitude of projects under design. Larger, more significant projects require more accurate design inputs.

Examples of new HMA pavement, new JPCP, HMA overlay of existing HMA pavement, and concrete pavement restoration that show the coded *AASHTOWare Pavement ME Design* inputs are presented in Appendices A, B, C, and D.

1.8 General AASHTOWare Pavement ME Design Features and Enhancements

AASHTOWare Pavement ME Design builds upon the research-grade MEPDG software. Key features and enhancements found in *AASHTOWare Pavement ME Design* include:

- Increased computational speed.
- Tool to optimize for thickness design.
- Tool to import back-calculation results for rehabilitation designs.
- Ability to establish agency-specific libraries for materials, traffic, and climate inputs.
- Both U.S. customary units and SI (metric).
- Ability to import third-party traffic data.
- Ability to enforce capacity limits on design traffic volume based on Transportation Research Board (TRB) Highway Capacity Manual (HCM) guidance.
- Ability to perform error checks on input data and report the outcome in a log file.
- Ability to open, view, and edit multiple projects at the same time.
- Ability to save and view structural responses (stress, strain, and deflection).
- Tool to run sensitivity analysis of key inputs of a "Trial Design".
- Ability to run multiple projects in batch mode and generate a multiple project summary.
- Tool to compare and view the differences in inputs between any two "Trial Designs."
- Option for importing and exporting data directly from an agency's enterprise-level relational database in both SQL and Oracle environments.

Chapter 2
General Information Inputs

2.1 "Design Life"

The design life of a new, reconstructed, overlaid, or restored pavement is established by ITD policy. Longer "Design Life" is typically selected for heavier trafficked highways to minimize lane closures for rehabilitation over this time period. The "Design Life" is a critical input since it significantly affects the initial design and thus, construction cost. The *AASHTOWare Pavement ME Design* procedure utilizes "Design Life" as the time from initial construction (or rehabilitation) until significant distress and/or roughness occur that triggers rehabilitation or reconstruction. The "Design Life" is always specified at a selected level of design reliability (e.g., the "Design Life" is 20 years at 95 percent reliability).

AASHTOWare Pavement ME Design can handle design lives from 1 year (e.g., a detour design) to well over 50 years. Recommend design lives are shown in Table 1. Exceptions may be considered for unique situations.

Table 1. Recommended Pavement "Design Life"

Pavement Type	Functional Class (Section 200 of the *ITD Materials Manual*)	"Design Life" New Pavement or Reconstruction (years)
New or Reconstructed HMA	Any Functional Class	20
	Reduced "Design Life" for Special Projects	< 20
New or Reconstructed JPCP	Any Functional Class	40
Flexible Pavement Rehabilitation	Any Functional Class	8 - 20
Rigid Pavement Rehabilitation	Any Functional Class	10 - 36

2.2 Construction and Traffic Opening Dates

Estimated construction and traffic opening dates (month/year; see Table 2) are required inputs for characterizing:

1. Climate properties.
2. Material properties due to climate changes and aging of asphalt and concrete.
3. Future traffic.

The designer must select the most likely month for construction and for opening to traffic. The time reference is keyed to the first day of the month. For example, selecting June means that all timing will begin on June 1. If the project is likely to proceed for several months or years, the month that results in the most distress over time should be selected. Normally this would be the warmest month but it can be determined by running the program over different months.

Table 2. Construction and Traffic Opening Date Description

Construction and Traffic Opening Date Description	
Activity	**Best Estimate**
Base/Subgrade Construction (Flexible Pavement Only)	Month/Year: Program begins with first day of the month to calculate moisture content in unbound layers.
Pavement Construction Month	Month/Year: Program assumes first day of month. Selecting August will result in the August climate being used and the August 1 date for timing of material properties.
Traffic Opening Date	Month/Year: Program begins computing damage on first day of month. Selecting June will start traffic on June 1. June will be the first month listed in the *AASHTOWare Pavement ME Design* output since damage from traffic accumulates from this day forward.

2.3 New and Reconstructed Pavement and Rehabilitated Pavement Types Considered by AASHTOWare Pavement ME Design

New and rehabilitated pavements are described in Tables 3 through 5.

Table 3. Description of New/Reconstructed Pavement Types Considered by *AASHTOWare Pavement ME Design*

Type of Pavement	Description
Flexible Pavement	HMA of all types including conventional thin HMA, deep strength HMA, & full-depth HMA
Semi-Rigid Pavement	HMA over chemically (cement) treated base
Rigid Pavement	JPCP with or without dowels at joints

Table 4. Description of Restored JPCP Considered by *AASHTOWare Pavement ME Design*

Type of Pavement	Description
Existing JPCP	Engineered design that may include cracked slab replacement, joint spall repair, shoulder replacement, dowel bar retrofit, but requires diamond grinding

Table 5. Description of HMA and PCC Overlays Considered by *AASHTOWare Pavement ME Design*

Type of Overlay	Existing Pavement
HMA	Existing Flexible Pavement [AC over AC] Existing Semi-Rigid Pavement (HMA/Cement Treated Base) [AC over AC] Existing Intact JPCP [AC over JPCP] Existing JPCP that has Been Rubblized (into an unbound granular layer) [AC over fractured JPCP]
JPCP	Existing Flexible Pavement [JPCP over AC/white topping] Existing Intact JPCP (minimum 1in. HMA separation layer required) [unbonded JPCP overlay] Existing JPCP that has Been Rubblized/Fractured into an Unbound Granular Layer [unbonded JPCP overlay]

2.4 Site and Project Identification

Enter appropriate information to identify the project for pavement design purposes. Figure 10 shows typical project indentation information used in *AASHTOWare Pavement ME Design*.

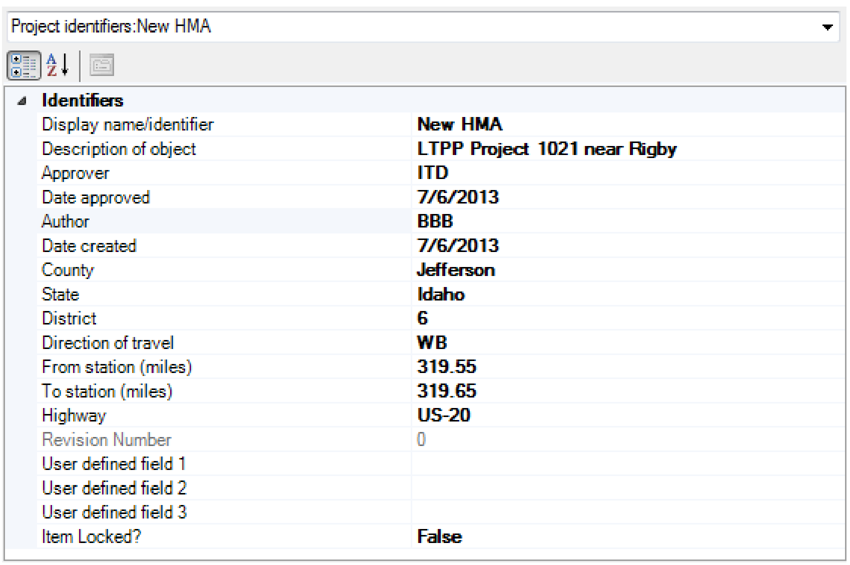


Figure 10. Project Identifiers for Site and Project Identification

Chapter 3
Performance Criteria Inputs

Performance criteria are used to ensure a new pavement design or rehabilitation performs satisfactorily over its “Design Life”. The designer selects performance criteria limits that relate directly to the need for rehabilitation. Performance of a pavement is measured in terms of the key distresses and smoothness as measured by IRI.

IRI (Smoothness). Both an initial IRI and terminal IRI must be selected. Initial pavement performance is characterized using IRI and all other distresses are assumed to be zero just after construction. Initial IRI is influenced mainly by factors associated with pavement construction practices and smoothness specifications. Use of smoothness incentives in recent years has dramatically improved initial smoothness for all types of pavement and rehabilitation. Terminal IRI is described as the lowest acceptable value before resurfacing or reconstruction becomes necessary for that particular class of highway. The distresses used to characterize performance are specific to the particular pavement type (flexible, rigid, composite). It is recommended that the terminal IRI be based on the classification of the highway and traffic level. The terminal IRI typically selected is similar to that used in pavement management to establish when roadways require rehabilitation.


Transverse Fatigue Cracks JPCP


Alligator Fatigue Cracks HMA


Transverse Cracks HMA

Initial IRI is selected at a value being achieved regularly in construction with the ITD smoothness specifications. The initial IRI values for new HMA and new JPCP projects were examined and an average obtained for each value, as presented in Table 6. Unusual conditions for HMA overlays of an existing pavement with heaves or settlements may require a higher value if the effects of the existing settlements or heaves are not removed by the HMA overlay placement.

Distress. Terminal distress is described as the highest acceptable value before resurfacing or reconstruction becomes necessary for that particular class of highway. The distresses used to characterize performance are specific to the particular pavement type (flexible, rigid, composite). It is recommended that the terminal distress be based on the classification of the highway and traffic level. The terminal distress typically selected is similar to that used in pavement management to establish when roadways require rehabilitation. It represents a pavement condition that experienced engineers would generally agree requires rehabilitation or reconstruction. Recommended performance criteria for design are in Tables 6 through 8.

Table 6. Initial IRI Values for New and Rehabilitated Pavement Design

Pavement Type	**Initial IRI** (in./miles)
New/Reconstructed HMA & HMA Overlays	50
New/Reconstructed JPCP, JPCP Overlays, & JPCP Restoration with Diamond Grinding	65

Table 7. Performance Criteria for Use in New HMA Pavement, HMA Overlays, and Composite (HMA-Overlaid Jointed Plain Concrete) Pavement Design

Classification	Performance Indicators (Maximum Value at End of "Design Life" at Design Reliability)*				
	Alligator Cracking (% lane area)**	**Total AC Cracking Through Overlay,** (percent lane area)***	**Total Rutting,** (in.)	**Transverse (Thermal) Cracking** (ft/mile)****	**IRI,** (in./mile)
Interstate/Freeways	10	5	0.40	1,000	160
Primary (Principal Arterials & Minor Arterials)	15	10	0.50	1,500	175
Secondary (Major Collectors)	20	15	0.65	1,500	200

* HMA longitudinal fatigue cracking (top-down) is not considered in HMA pavement design in Idaho.

** HMA alligator cracking: bottom-up alligator (fatigue) cracking in the new HMA layer as well as in the HMA overlay. Alligator fatigue cracking initiates at the bottom of the new HMA layer or new overlay layer in the wheelpaths.

*** Total AC cracking (alligator reflective from existing HMA+ alligator from overlay). Reflective refers to alligator fatigue cracking that initiates in the existing HMA layer and reflects up through the new HMA overlay in the wheelpaths.
NOTE: This value is considered only at 50 percent reliability. The software cannot design for higher levels of reliability at the present time.

**** NOTE: The limits presented do not apply to composite pavements as transverse cracking in composite pavements includes transverse joints and slab cracks reflected through the HMA overlay. A considerably higher limiting transverse cracking value must be assumed for composite Pavements.

Table 8. Performance Criteria for Use in JPCP New, Concrete Pavement Restoration and JPCP Overlays Pavement Design

Classification	Performance Indicators (Maximum Value at End of "Design Life" at Design Reliability)		
	Slabs Cracked (percent)	**Mean Transverse Joint Faulting** (in.[1])	**IRI** (in./mile)
Interstate/Freeways	10	0.12	160
Primary (Principal Arterials & Minor Arterials)	15	0.15	175
Secondary (Major Collectors)	20	0.25	200

[1]A grinding opportunity is allowed approximately 20 - 25 years after initial construction.

The criteria presented in Tables 7 and 8 must also be selected in consideration of the design reliability (discussed in Chapter 4). Selection of too tight of a criterion, such as 0.1 inch rutting at a very high reliability of 97 percent, may make it impossible to obtain an acceptable design, or the design may be excessively costly.

These criteria represent the pavement condition at the end of the design period.

NOTE: Selecting a limiting fatigue slab cracking value of 10 percent slabs at a 90 percent reliability level implies that 9 out of 10 projects will experience fatigue slab cracking levels less than 10 percent over the specified design period.

Use the criteria presented in Tables 7 and 8 to determine whether a pavement design meets minimum performance standards during its "Design Life" for a given level of reliability. These criteria are tentative and need further consideration by the ITD.

Chapter 4
Design Reliability Input

*AASHTOWare Pavement ME De*sign describes design reliability as the probability that the pavement will not exceed specific performance criteria over the selected "Design Life". Reliability is a means to account for random variations in many factors including all design inputs, projections of future climate conditions, future traffic levels, changes in subgrade support along the design route, and residual error in the prediction models.

Design reliability essentially assigns a level of assurance that the pavement section will survive for the "Design Life" under preset terminal distress and IRI levels. Thus, reliability provides a rational "safety factor" that the design will perform at least as well as not exceeding the performance criteria. For example, design reliability of 90 percent for rutting represents the probability (e.g., 9 out of 10 projects) that the mean rutting for the project will not exceed the specified criteria.

A project that exceeds performance criteria usually requires earlier-than-programmed rehabilitation activities that require lane closures. It does not have a dire structural collapse consequence, such as in bridge design. Thus, design reliability in pavement design is lower than in bridge or building design.

Design reliability must be selected for each distress and IRI, and the reliability level can vary between types. It is important to select design reliability that is balanced with the performance criteria. For example, selecting a high design reliability level (such as 99 percent) and a very low performance criterion (such as 3 percent alligator cracking) may make it impossible or very costly to obtain an adequate design. Higher design reliability will require a thicker pavement structure and/or the use of materials with higher durability or structural capability, all of which increase construction costs.

Typically, design reliability is greater for highways with higher traffic volumes, which decreases the probability that the pavement will need earlier-than-programmed lane closures for rehabilitation. In other words, higher reliability is justified for heavily trafficked highways due to the more severe consequences involved (e.g., early closure of traffic lanes for heavy maintenance or rehabilitation).

Recommended reliability values are presented in Table 9 for ITD designs that are compatible with the performance criteria discussed in Chapter 3. Designers must use the same level of reliability for all distress types and IRI. Higher design reliability may be warranted for special designs, such as pavements located in heavily trafficked urban areas. Such designs will require more substantial layer thickness and very specialized materials (e.g., polymer modified asphalt binders) with a corresponding higher first cost but perhaps lower future rehabilitation cost. In addition to the reliability level there is also another input for each distress and IRI and that is the standard error (or deviation). The standard error represents the error associated with each prediction model. These are provided in the calibration default section of the software and should not be changed unless based on results of local calibration.

Table 9. Recommended Level of Design Reliability

Functional Classification	Reliability (percent)	
	Urban	Rural
Interstate/Freeways	95	95
Primary (Principal Arterials and Minor Arterials)	90	85
Secondary (Major Collectors)	80	75

NOTE: These values are tentative only and require ITD evaluation and revision. If, for example, designs are coming out thinner than desired on average, the level of design reliability can be increased

Figures 11 and 12 illustrate the impact of design reliability on HMA and PCC thickness for a given project site under heavy truck traffic. As the design reliability approaches 100 percent, the required increase in thickness is much greater.

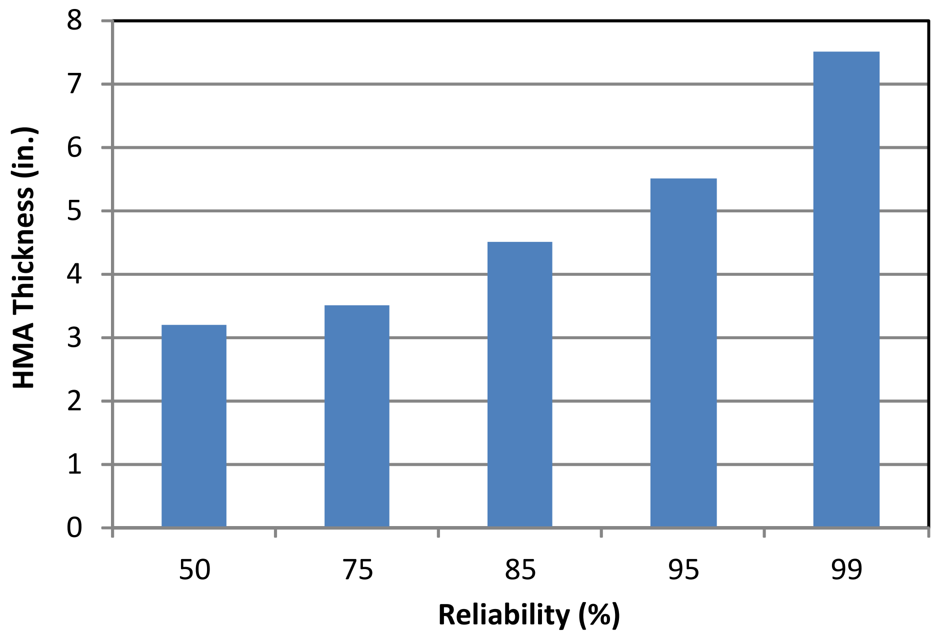


Figure 11. Illustration of the Effect of Design Reliability on HMA Thickness for a Project Site Under Heavy Truck Traffic

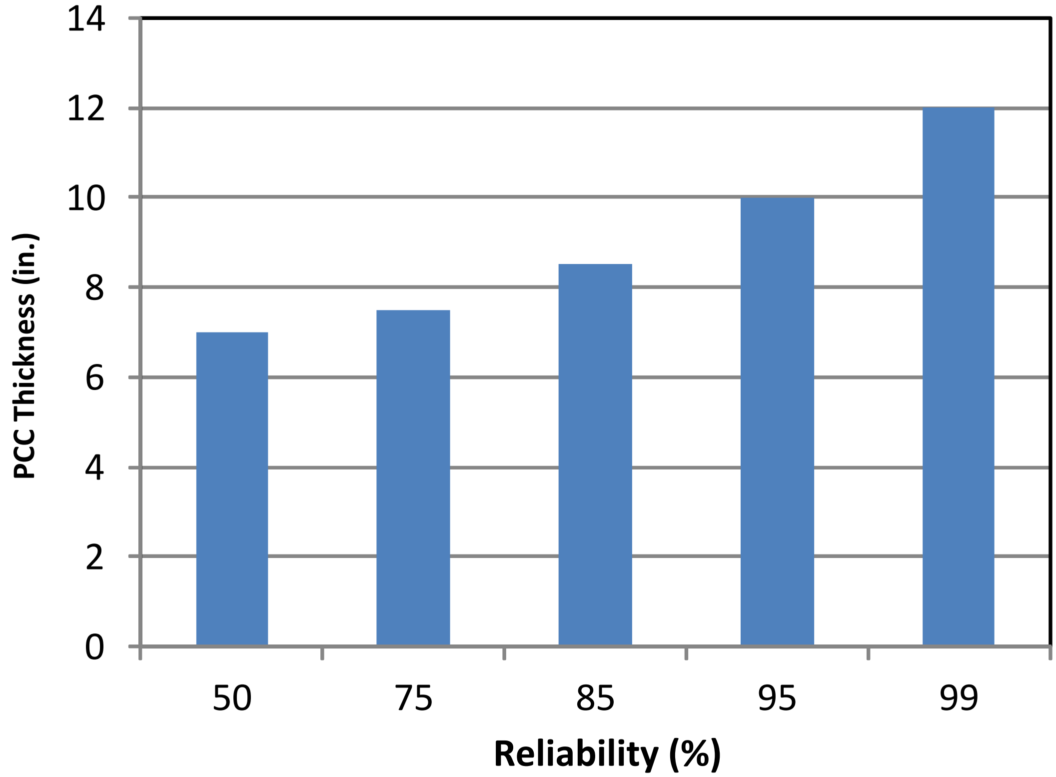


Figure 12. Illustration of the Effect of Design Reliability on Required JPCP Slab Thickness for a Given Project Site Under Heavy Truck Traffic

Chapter 5
Traffic Inputs

5.1 Introduction

Traffic data are typically derived from a variety of sources:

- Automatic Traffic Recorders.
- Portable Traffic Counts.
- Manual Traffic Classification Counts.
- Automatic Vehicle Classification (AVC) Sites.
- Weight-in-Motion (WIM) Sites.

The primary source of traffic inputs in Idaho is WIM sites. Table 10 summaries the location information of 25 ITD WIM sites. The WIM data is divided into two types; vehicle classification data and vehicle weight data. The vehicle classification data contains hourly truck traffic volume by truck class while the weight data contains hourly weights for each truck class and axle type as well as axle spacing. In general, traffic data reported as annual average daily traffic (AADT) and annual average daily truck traffic (AADTT). The AADTs are multiplied by road length to calculate vehicle miles traveled (VMT).

Several inputs are required for characterizing traffic for *AASHTOWare Pavement ME Design*. For most pavement designs, designers will select Level 3 inputs based on similarity of pavement project characteristics such as functional class, location, and so on. Most of the traffic data, from 25 WIM stations, was collected in 2009, with few sites with data for both 2008 and 2009. Information from these WIM sites in Idaho provided traffic data of sufficient detail for developing traffic inputs for the State as part of the University of Idaho Study RP-193 for ITD. Analysis of the data showed that 21 out of the 25 WIM sites contained sufficient classification data for at least 12 consecutive months. The rest of the WIM stations were missing the classification data for some months within the analysis period. Thus, truck volume distribution by class and month of the year were generated using the *TrafLoad* software for the 21 stations with sufficient data. Pavement designers can obtain Level 2 traffic inputs for preliminary designs from this database. Contact the State Materials Engineer for assistance using the ITD Project RP-193 spreadsheet to input traffic data into *AASHTOWare Pavement ME Design*. For Level 1 or Level 2 traffic inputs typically used for the designs of special projects with unique needs, designers must contact the ITD Roadway Data Section and request project specific traffic data. ITD designers will save traffic information to the project design file and to the traffic database.

Table 10. WIM Stations in Idaho

WIM Site ID	Functional Classification	Route	Milepoint	Nearest City
79	Principal Arterial - Interstate (Rural)	I-15	27.700	Downey
93	Principal Arterial - Interstate (Rural)	I-86	25.050	Massacre Rocks
96	Principal Arterial - Other (Rural)	US-20	319.200	Rigby
115	Principal Arterial - Interstate (Rural)	I-90	23.370	Wolf Lodge
117	Principal Arterial - Interstate (Rural)	I-84	231.700	Cottrell
118	Principal Arterial - Other (Rural)	US-95	24.100	Mica
119	Principal Arterial - Other (Rural)	US-95	85.200	Samuels
128	Principal Arterial - Interstate (Rural)	I-84	15.100	Black Canyon
129	Principal Arterial - Other (Rural)	US-93	59.800	Jerome
133	Minor Arterial (Rural)	US-30	205.500	Filer
134	Principal Arterial - Other (Rural)	US-30	425.785	Georgetown
135	Principal Arterial - Other (Rural)	US-95	127.700	Mesa
137	Principal Arterial - Other (Rural)	US-95	37.075	Homedale
138	Principal Arterial - Other (Rural)	US-95	22.720	Marsing
148	Principal Arterial - Other (Rural)	US-95	363.980	Potlatch
155	Minor Arterial (Rural)	US-30	229.620	Hansen
156	Minor Arterial (Rural)	SH-33	21.940	Howe
166	Principal Arterial - Interstate (Rural)	I-84	-	Eden
169	Principal Arterial - Other (Rural)	US-95	56.002	Parma
171	Principal Arterial - Interstate (Rural)	I-84	114.500	Hammett
173	Principal Arterial - Interstate (Rural)	I-15	177.860	Dubois
179	Principal Arterial - Interstate (Rural)	I-86B	101.275	American Falls
185	Principal Arterial - Other (Rural)	US-12	163.010	Powell
192	Principal Arterial - Other (Rural)	US-93	16.724	Rogerson
199	Principal Arterial - Other (Rural)	US-95	441.600	Alpine

This section presents Level 3 traffic inputs for Idaho. These can be improved as more and more traffic data, including truck axle weight and classification data at representative sites throughout the State, are collected and processed.

5.2 Traffic Volume

Designers are required to enter current (design year) truck traffic volume of the given pavement design lane. This is estimated using the parameters presented in Table 11. Additionally, the actual operational speed of the truck traffic is required.

Current truck traffic volume can be estimated by performing traffic counts on-site as needed or through projections of historical traffic volume data, which can be obtained from the ITD Roadway Data Section.

Table 11. Current and Future Truck Traffic Volume Estimates for Pavement Design

Traffic Input	Recommended Value
Initial Two-Way AADTT	Projected for month of opening to traffic from measured historical data at site is desirable. "Trucks" are defined as Federal Highway Administration (FHWA) Classes 4 through 13. The percent trucks to be used to compute AADTT & is available for various previous years in the Transportation Asset Management (TAMS) database. **[Critical]**
Number of Lanes in Design Direction	Actual, from design plans.
Percent of Two-Directional Trucks in Design Direction	50%, unless higher truck volume is measured in design direction NOTE: This is volume, not weight.
Percent of All Trucks in Design Direction in Design Lane (For example, of the trucks in the design direction, 60% may be in the heaviest trafficked design lane & the other 40% in other lanes)	Actual measured truck traffic in the design lane (heaviest truck volume) over 24 hours, or use the following based on Idaho measurements: • 100% for 1 lane in design direction • 90% for 2 lanes in design direction • 80% for 4 lanes in design direction • 60% for 4or more in design direction For unusual truck traffic situations (mountainous terrain or urban usage complexity), conduct onsite truck lane usage counts over a 24-hour period.
Operational Speed (mph)	Posted or design speed

5.3 Traffic Volume Monthly Adjustment Factors

Traffic volume monthly adjustment factors (MAF) are used to adjust truck traffic loadings throughout the year. The MAF is required for each truck class type. A Level 1 MAF is the actual measured site data and must be used for highways with recreational, agricultural, or haulage traffic. Level 2/3 MAFs are defaults estimated using data from WIM sites around the State. The default MAF values were obtained by averaging all available Idaho sites, as shown in Table 12. A comparison of Idaho averages with MEPDG defaults presented in Figure 13 shows reasonably uniform MAFs across months. Again, there may be exceptions for highways used for heavy seasonal recreational, agricultural, or haulage purposes, which is why these require measurement at the site over a 12-month period (Level 1).

NOTE: Summer months (June, July, August, and September) experienced higher levels of truck traffic.

Table 12. Recommended *AASHTOWare Pavement ME Design* MAF Inputs for Design in Idaho

Month	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
January	0.74	0.86	0.91	1.04	0.64	0.98	0.88	0.90	0.93	1.12
February	0.83	0.82	0.87	0.63	0.67	1.00	0.96	0.91	0.67	0.96
March	0.77	0.80	0.83	0.75	0.86	0.95	1.10	0.97	1.48	1.01
April	0.91	0.85	0.86	1.20	1.00	0.95	1.10	0.93	0.79	0.88
May	1.12	0.98	0.90	1.63	1.07	0.95	1.10	1.07	1.20	0.80
June	0.99	1.01	0.84	0.72	1.17	0.94	0.84	1.42	1.69	0.81
July	1.49	1.33	1.30	1.09	1.53	0.97	0.85	1.66	1.08	0.88
August	1.46	1.21	1.45	1.21	1.42	0.98	1.01	0.81	0.96	0.99
September	1.31	1.14	1.29	0.98	1.18	1.06	1.08	0.88	0.71	0.93
October	0.94	1.08	1.26	0.92	1.03	1.16	1.13	0.60	0.76	1.13
November	0.72	0.99	0.75	0.98	0.79	1.07	0.92	0.82	0.67	1.09
December	0.72	0.93	0.74	0.85	0.64	0.99	1.03	1.03	1.06	1.40

NOTE: Each column must add up to 12.

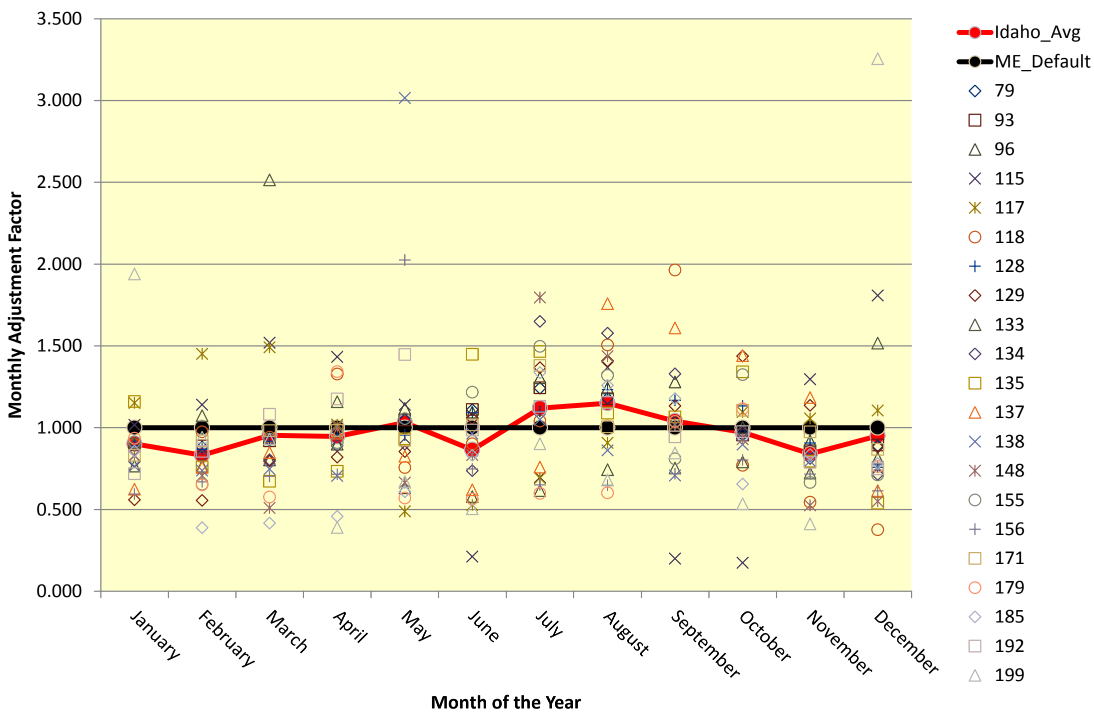


Figure 13. Idaho vs. MEPDG Monthly Adjustment Factors

5.4 Vehicle Class Distribution

Vehicle class distributions (VCD) for *AASHTOWare Pavement ME Design* are basically adjustment factors used to distribute annual truck traffic estimates by vehicle/truck type. Vehicle class types are defined according to FHWA and AASHTO definitions, as shown in Figure 14. Level 1 VCD is the actual measured site data over 24 hours and must be used for highways with heavy seasonal recreational and agricultural traffic (contact the ITD Roadway Data Section). VCD from each of 21 WIM sites are presented in Table 13. Data in this table show that at the majority of the sites, Class 5 and Class 9 are the predominant truck classes.

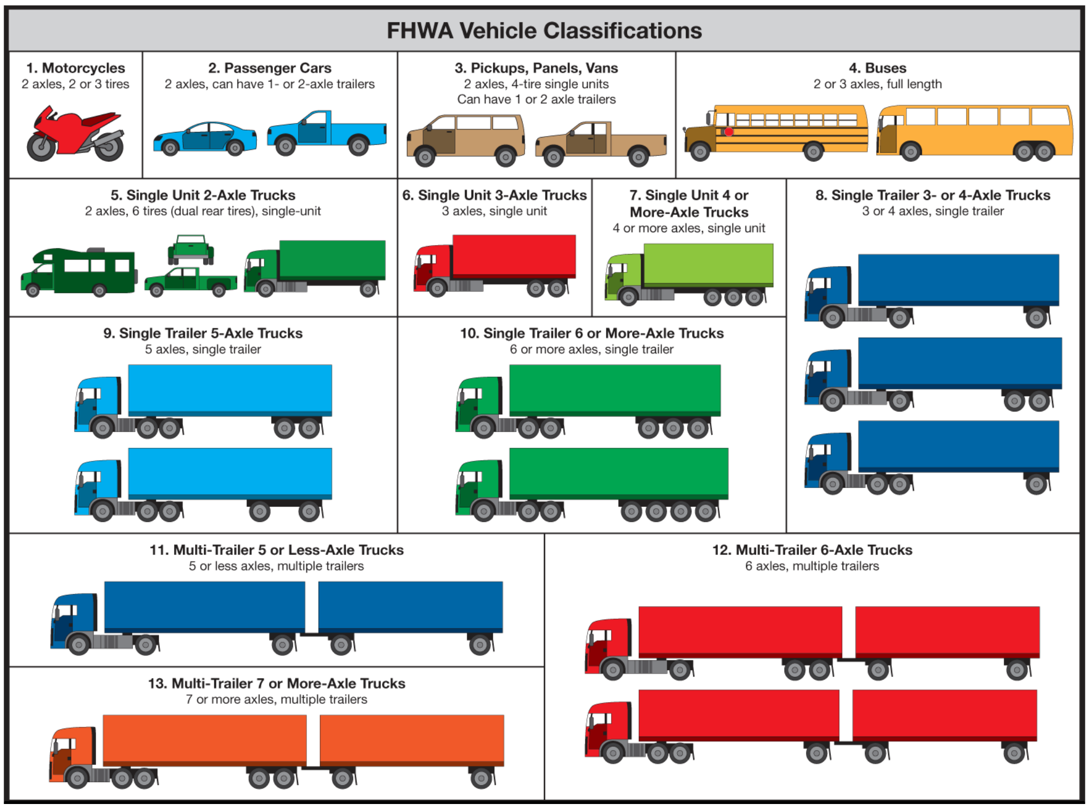


Figure 14. Illustration of FHWA/AASHTO Vehicle Class Type Description

Table 13. FHWA Vehicle Class Distribution Inputs for Idaho

WIM Site ID	FHWA Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
79	1.77	21.20	2.13	0.50	8.35	49.07	5.19	1.11	1.01	9.67
93	0.99	11.21	1.31	0.11	4.09	52.90	12.73	0.76	0.59	15.33
96	1.94	45.59	6.60	0.95	7.64	27.43	6.73	0.18	0.32	2.62
115	2.62	29.15	7.15	10.82	5.31	33.57	7.92	0.26	1.03	2.18
117	1.03	5.96	3.86	7.20	4.56	52.35	15.06	1.45	1.33	7.20
118	2.50	48.01	11.18	14.05	4.19	8.84	10.52	0.02	0.04	0.65
128	1.25	16.44	1.75	0.22	5.49	54.73	9.96	2.28	1.54	6.34
129	5.10	37.84	6.61	0.64	7.29	22.21	11.36	0.45	0.17	8.33
133	1.34	46.53	10.18	7.73	7.54	18.56	5.12	0.08	0.01	2.92
134	2.15	21.28	1.90	0.36	5.51	61.01	3.43	0.19	0.27	3.91
135	1.84	42.40	4.74	0.82	9.71	30.16	7.54	0.53	0.08	2.19
137	5.37	8.56	10.73	0.32	6.94	52.33	8.71	0.61	0.18	6.26
138	1.14	3.82	2.39	0.03	5.18	72.76	6.35	2.23	0.58	5.54
148	2.11	7.69	13.66	1.16	5.02	24.87	41.78	0.00	0.12	3.59
155	17.94	7.73	11.46	3.10	8.46	16.75	15.21	2.07	2.33	14.95
156	1.01	4.00	5.12	0.00	4.96	39.99	12.72	0.00	0.08	32.12
171	1.17	3.37	1.51	0.24	3.46	69.49	9.24	1.64	1.48	8.41
179	0.35	10.37	9.84	0.53	2.64	35.85	13.36	0.00	0.00	27.07
185	0.26	4.77	9.10	0.45	8.05	46.29	21.53	0.00	0.00	9.55
192	3.40	4.90	2.18	0.60	7.24	75.47	3.68	0.50	0.26	1.78
199	2.98	38.76	9.94	12.49	5.12	11.90	11.67	0.68	1.06	5.40

Selection of the appropriate VCD for a given site must thus be based on project location and highway functional class, as a minimum. A preliminary analysis of Idaho traffic data indicates three potential VCD clusters:

- Predominantly Class 5.
- Predominantly Class 9.
- Mixture of Both Class 5 and Class 9.

Variation of VCD among Idaho WIM sites is presented in Figure 15. The VCD clusters represent Idaho average values defined by highway functional class and location can be used as Level 2/3 inputs.

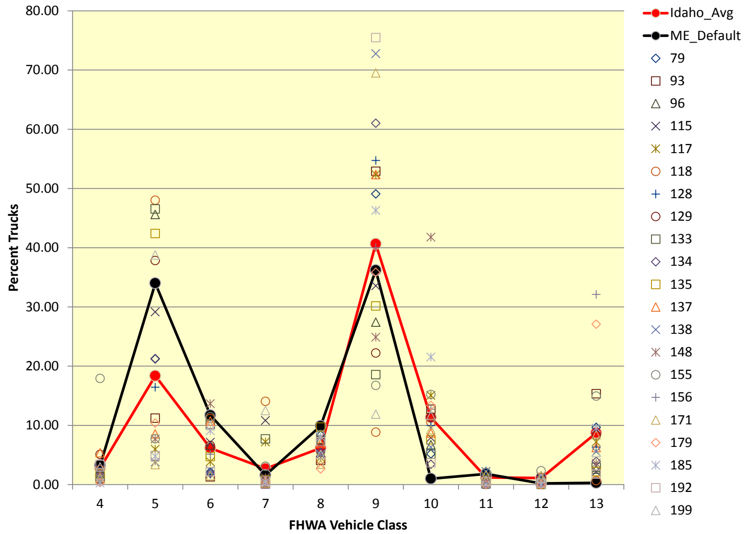


Figure 15. Plot Showing Variation of VCD Among WIM Sites in Idaho

5.5 Hourly Truck Distribution

This input is only needed for rigid pavement design, as it relates number of truck traffic applications to PCC slab curl/warp condition. (Curl/warp is cyclic and repeats itself every 24 hours). Hourly distribution data was not processed or analyzed in the University of Idaho study, as this input is not required for flexible pavement design. MEPDG default hourly distribution can be used temporarily until further data analysis is done. The recommended hourly distribution for design is shown in Figure 16 and Table 14.

NOTE: Highways with heavy seasonal recreational and agricultural traffic may need special 24-hour measurement.

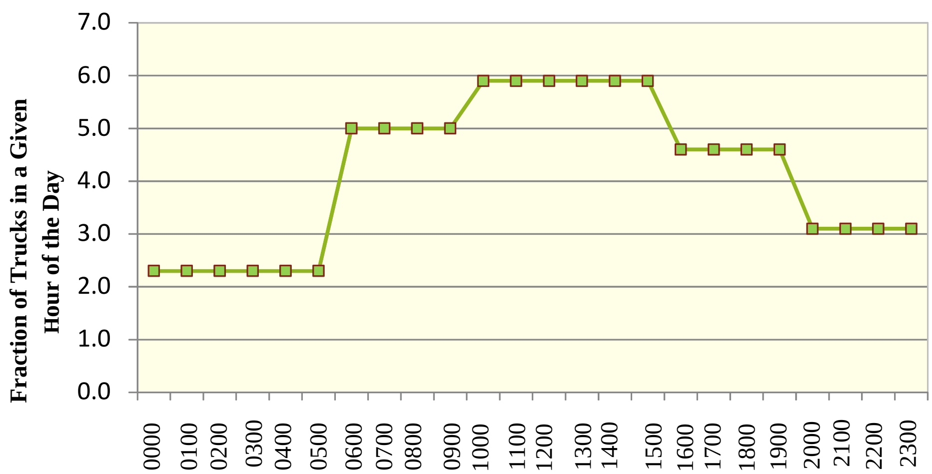


Figure 16. Plot Showing MEPDG Default Hourly Truck Distribution

Table 14. Recommended MEPDG Default Hourly Truck Distribution Inputs for Design in Idaho

Time of Day (Hours)	Rural and Urban Highways
0000	2.3
0100	2.3
0200	2.3
0300	2.3
0400	2.3
0500	2.3
0600	5.0
0700	5.0
0800	5.0
0900	5.0
1000	5.9
1100	5.9
1200	5.9
1300	5.9
1400	5.9
1500	5.9
1600	4.6
1700	4.6
1800	4.6
1900	4.6
2000	3.1
2100	3.1
2200	3.1
2300	3.1
All	100.0

5.6 Truck Traffic Growth Factor

Truck traffic growth factors are used to project total design traffic from the estimated first year traffic to the end of the pavement's "Design Life" by multiplying initial year traffic by an appropriate growth factor.

NOTE: Growth factors can be different for different vehicle classes.

AASHTOWare Pavement ME Design allows the user to specify the nature and rate of traffic growth relative to the base year. Further, the user also has the option of selecting a different growth rate and growth function for each truck class (see Figure 17). This allows the software to consider the growth for each truck class separately.

Vehicle Class Distribution and Growth

Load Default Distribution

Vehicle Class	Distribution (%)	Growth Rate (%)	Growth Function
Class 4	3.3	3	Linear
Class 5	34	3	Linear
Class 6	11.7	3	Linear
Class 7	1.6	3	Linear
Class 8	9.9	3	Linear
Class 9	36.2	3	Linear
Class 10	1	3	Linear
Class 11	1.8	3	Linear
Class 12	0.2	3	Linear
Class 13	0.3	3	Linear
Total	100		

Figure 17. *AASHTOWare Pavement ME Design* Screenshot Showing Inputs for Nature and Rate of Traffic Growth Relative to the Base Year (For All Vehicle Classes)

The designer has the option of choosing one of three traffic growth functions:

- **No Growth:** Truck volume remains the same throughout the "Design Life".
- **Linear Growth:** The truck volume increases by constant percentage of the base year traffic across each truck class.
- **Compound Growth:** The truck volume increases by constant percentage of the preceding year traffic across each truck class.

Traditionally, the ITD has projected future growth of truck traffic using historical AADTT.

NOTE: The proper input for *AASHTOWare Pavement ME Design* is growth in truck/bus traffic (FHWA Vehicle Classes 4 through 13) only.

This implies that future growth will be as significant as past growth, which may or may not be accurate for various highway segments. When assigning truck traffic growth factors, designers must consider future changes in demographics and land use, expected traffic attracted or diverted due to new/improved facility, etc., as these could impact future growth rates and trends. As truck traffic growth is unique to a given pavement project, it is highly recommended that site-specific inputs be used for all interstate and primary route designs. For secondary routes, reasonable estimates can be used if available. Specific recommendations are presented below:

- **Level 1 (Interstate and Primary Routes):** Determine project site historic traffic growth by plotting AADTT over time for as many years as available. Five or more years are desirable to reduce unrealistically high or low values.

NOTE: These data will likely reflect a downturn in AADTT due to the recession from 2007 to 2010.) Then adjust this value up or down based on the relative expected future growth for the project site. The recommended range is from 0 to 10 percent per year. If the value comes out negative, use +3 percent compound growth.

- **Level 2/3 (Secondary Routes):** If no historical data are available for a highway segment, obtain data from another segment as close or as representative as possible to the highway segment under design. If no such data are available, use a value of +3 percent compound growth. Typically, values vary long-term from 0 to 6 percent.

Historical AADTT data may be obtained from the ITD Roadway Data Section.

5.7 Axle Load Distribution

Axle load distributions (ALD) are percentages used to distribute the total number of axles by each axle type (single, tandem, tridem, and quad) and weight, as shown below:

- Single Axles: from 3,000 to 41,000 lb in 1,000 lb increments.
- Tandem Axles: from 6,000 to 82,000 lb in 2,000 lb increments.
- Tridem Axles: from 12,000 to 102,000 lb in 3,000 lb increments.
- Quad Axles: from 12,000 to 102,000 lb in 3,000 lb increments.

For pavement design in Idaho, the following input levels are recommended:

- **Level 1:** The actual measured WIM site data. This should be used for highways with unique traffic characteristics such as mining, recreational, and agricultural routes. Site-specific (Level 1) ALD may be obtained from the ITD Roadway Data Section.

- **Level 2:** Average axle load distributions for three different Truck Weight Road Groups (TWRG) were developed for Idaho using historical WIM data from several sites in the State. The TWRGs representing Idaho traffic loading characteristics are:
    - Primarily Loaded (Heavily Loaded).
    - Moderately Loaded.
    - Lightly Loaded.

A detailed discussion can be found in ITD's Research Report RP193 - *Implementation of the MEPDG for Flexible Pavements in Idaho*.

- **Level 3:** Statewide average axle load distributions were developed for Idaho interstates, primary highways, and secondary routes (rural and urban) using historical WIM data from several sites in Idaho. The historical data represented all the different types of highway functional classes of interest.

Figures 18 through 23 show the Level 2/3 single, tandem, tridem, and quad axle distributions in Idaho for the mix of rural/urban interstate, primary, and secondary highways. ALD must be provided for all four axle types by vehicle class and month of the year. The statewide default (Level 3) axle load distribution factors are presented in Tables 15 through 18. The ALD for the primarily, moderately, and lightly loaded TWRGs are summarized in Tables 19 through Table 30, respectively. WIM sites associated with TWRGs are presented in Table 31.

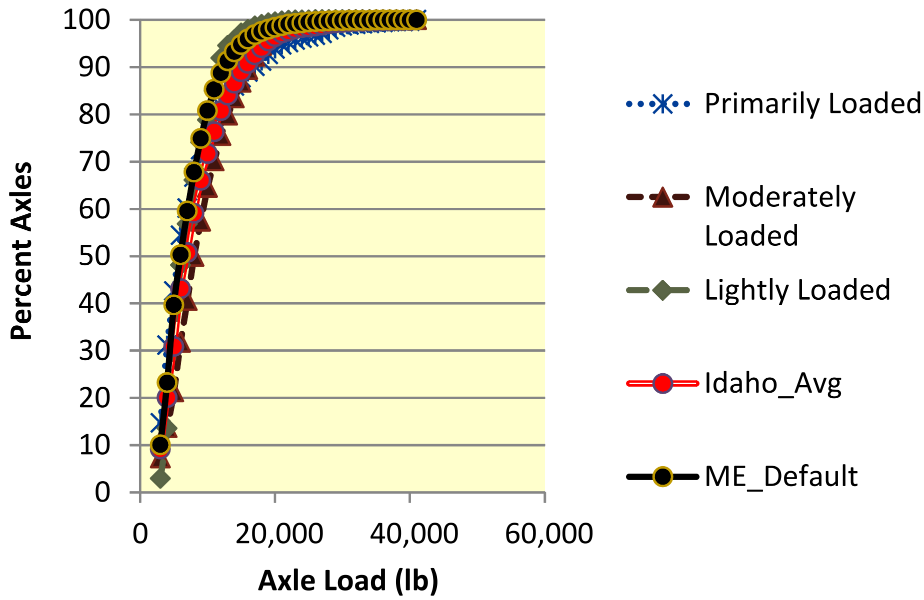


Figure 18. Idaho Truck Class 5 Single-Axle Load Distribution

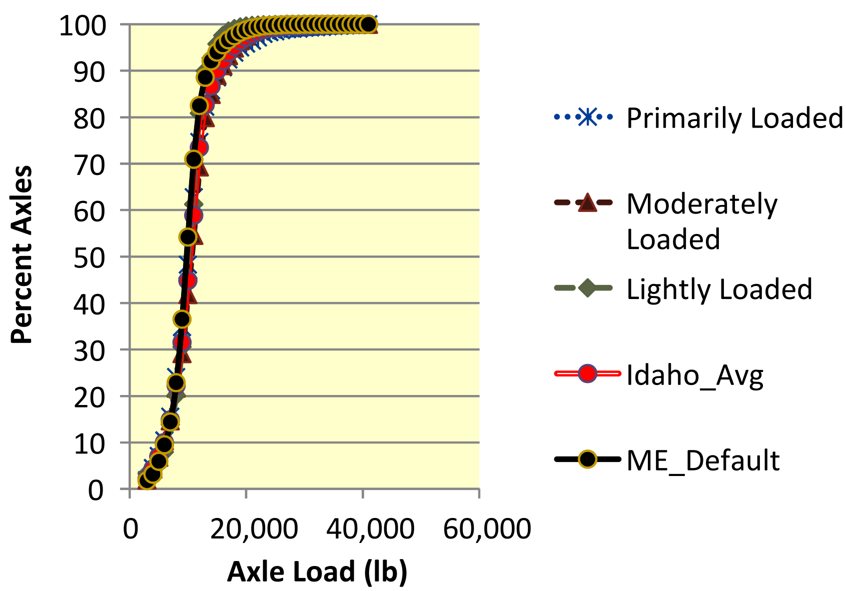


Figure 19. Idaho Truck Class 9 Single-Axle Load Distribution

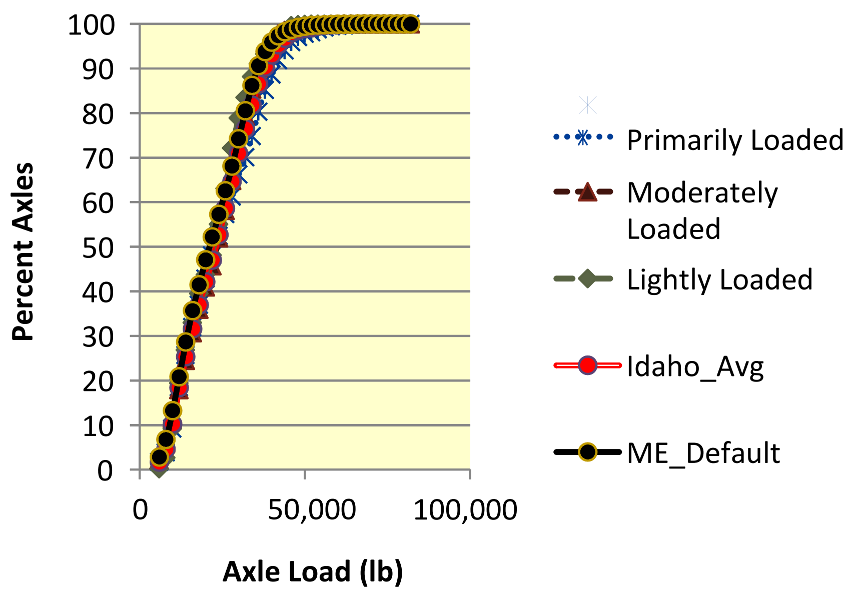


Figure 20. Idaho Truck Class 9 Tandem-Axle Load Distribution

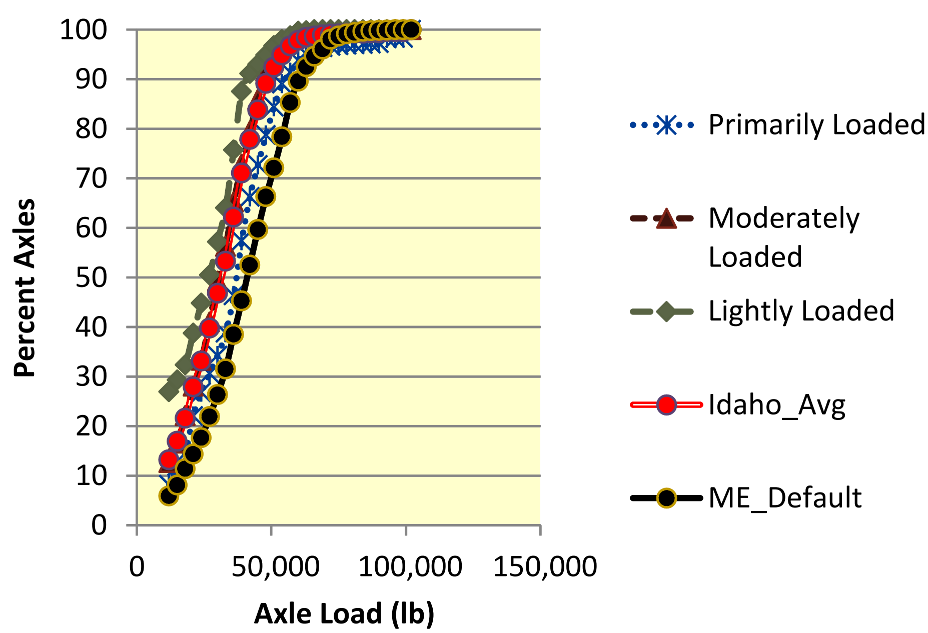


Figure 21. Idaho Truck Class 7 Tridem-Axle Load Distribution

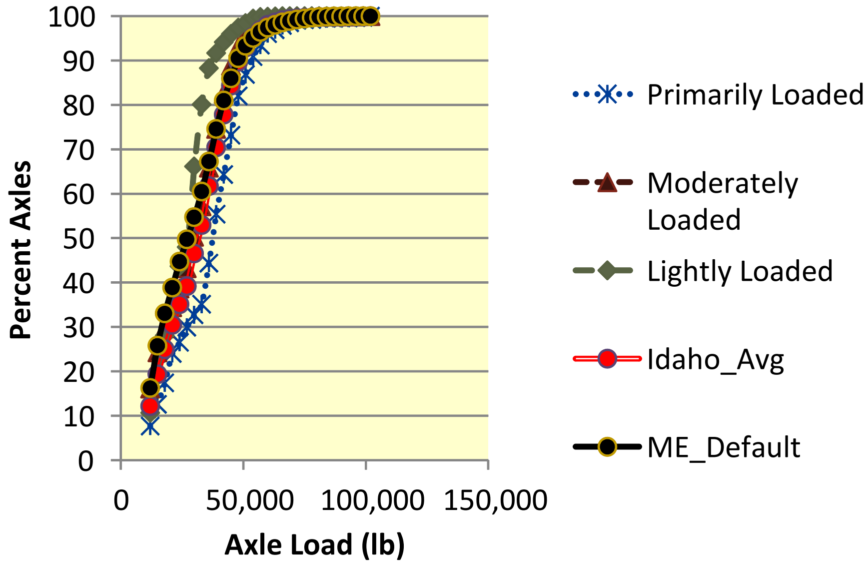


Figure 22. Idaho Truck Class 10 Tridem-Axle Load Distribution

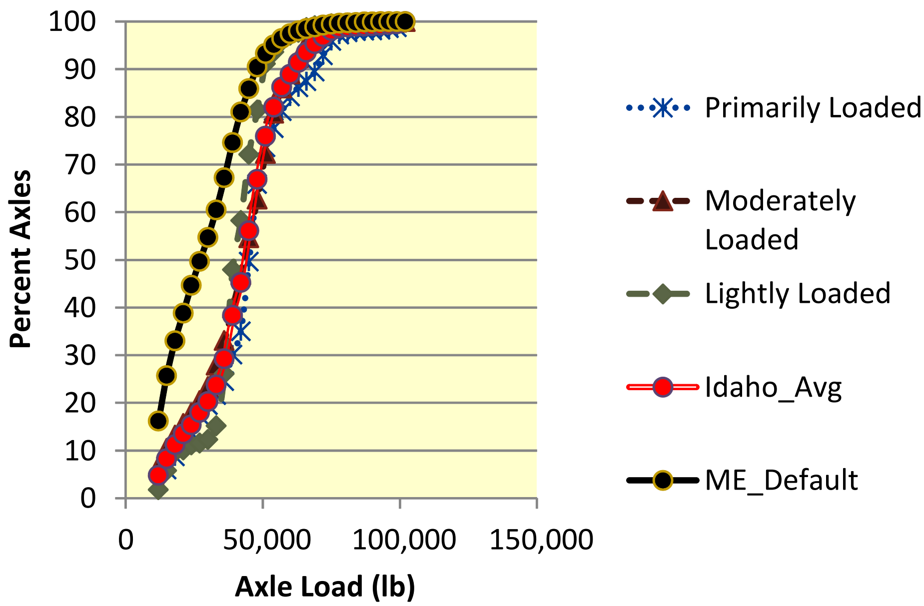


Figure 23. Idaho Truck Class 10 Quad-Axle Load Distribution

Table 15. Idaho Statewide Average Single-Axle Distribution

Axle Load (lb)	Vehicle Class									
	4	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
3,000	4.07	9.14	1.82	5.81	15.18	2.13	1.16	9.74	8.25	5.21
4,000	1.91	10.92	2.83	3.02	10.52	2.15	0.78	6.44	5.84	5.81
5,000	3.18	10.80	3.51	2.44	9.48	2.64	1.72	9.26	4.66	5.87
6,000	6.18	12.22	5.14	5.03	9.05	3.02	2.74	9.79	6.56	6.65
7,000	6.30	7.69	6.82	6.59	7.04	4.89	3.53	7.82	7.12	7.75
8,000	10.77	8.31	9.85	8.93	10.41	7.45	7.30	9.01	10.57	7.20
9,000	8.39	6.94	9.12	9.03	6.37	9.20	10.35	6.72	9.77	8.34
10,000	9.01	5.70	10.59	9.35	7.18	13.36	15.49	7.70	11.94	11.01
11,000	7.49	4.60	9.13	9.15	4.45	14.00	13.92	5.83	9.51	8.15
12,000	7.39	4.47	10.23	9.18	4.00	14.58	15.04	4.73	7.04	8.59
13,000	6.94	3.31	8.47	7.99	3.11	9.22	10.78	3.34	4.67	5.86
14,000	6.22	2.50	5.75	5.07	2.09	4.02	3.94	2.74	2.80	3.48
15,000	6.21	2.40	5.67	3.51	2.15	3.42	3.28	2.82	2.55	3.78
16,000	3.46	1.80	2.97	3.84	1.19	2.05	1.22	2.23	1.78	2.50
17,000	2.68	1.81	2.48	3.13	1.18	1.77	0.96	2.03	1.39	2.63
18,000	1.83	1.48	1.41	2.21	1.01	1.34	0.60	1.72	1.04	1.87
19,000	1.58	1.42	1.18	1.49	1.26	1.18	1.21	1.53	0.71	1.54
20,000	1.02	0.94	0.70	0.87	0.82	0.79	2.29	1.06	0.49	0.96
21,000	0.88	0.74	0.75	0.75	1.01	0.67	1.61	0.83	0.59	0.69
22,000	0.83	0.45	0.80	0.40	0.60	0.52	0.66	0.74	0.31	0.41
23,000	0.74	0.43	0.38	0.66	0.41	0.47	0.24	0.84	0.27	0.27
24,000	0.55	0.29	0.10	0.51	0.23	0.27	0.32	0.56	0.37	0.30
25,000	0.58	0.15	0.12	0.25	0.14	0.14	0.29	0.31	0.31	0.31
26,000	0.43	0.17	0.03	0.13	0.14	0.15	0.11	0.17	0.27	0.12
27,000	0.32	0.19	0.02	0.21	0.11	0.10	0.04	0.22	0.14	0.09
28,000	0.24	0.29	0.02	0.09	0.10	0.06	0.05	0.12	0.11	0.06
29,000	0.15	0.19	0.01	0.16	0.07	0.03	0.02	0.14	0.06	0.06
30,000	0.09	0.11	0.00	0.01	0.07	0.07	0.06	0.25	0.06	0.06
31,000	0.09	0.08	0.00	0.01	0.06	0.04	0.02	0.17	0.06	0.04
32,000	0.11	0.07	0.00	0.04	0.06	0.03	0.01	0.16	0.06	0.04
33,000	0.10	0.04	0.02	0.03	0.06	0.02	0.03	0.15	0.04	0.02
34,000	0.07	0.04	0.00	0.00	0.05	0.05	0.05	0.13	0.05	0.04
35,000	0.04	0.03	0.00	0.00	0.05	0.03	0.02	0.15	0.05	0.03
36,000	0.04	0.04	0.00	0.01	0.04	0.01	0.01	0.09	0.08	0.02
37,000	0.01	0.04	0.01	0.01	0.04	0.03	0.01	0.08	0.09	0.04
38,000	0.01	0.02	0.00	0.01	0.03	0.02	0.01	0.06	0.08	0.03
39,000	0.03	0.01	0.00	0.00	0.04	0.01	0.02	0.06	0.06	0.03
40,000	0.01	0.04	0.02	0.05	0.05	0.01	0.03	0.08	0.09	0.04
41,000	0.05	0.13	0.05	0.03	0.15	0.06	0.08	0.18	0.16	0.10

Table 16. Idaho Statewide Average Tandem-Axle Distribution

Axle Load (lb)	Vehicle Class								
	4	5	6		8	9	10	11	12	13
6,000	4.34	0.00	5.52	11.08	30.69	1.69	3.74	21.91	7.33	6.03
8,000	2.25	0.00	6.01	6.54	11.45	2.92	5.89	9.97	4.42	6.60
10,000	2.60	0.00	6.93	9.47	9.39	5.61	6.01	15.71	8.03	7.20
12,000	3.52	0.00	7.25	9.73	11.11	8.14	7.41	20.39	8.45	9.54
14,000	2.64	0.00	7.09	7.18	7.52	6.94	7.82	13.50	8.20	5.77
16,000	4.20	0.00	6.27	5.76	6.04	6.23	8.24	4.49	10.64	6.20
18,000	4.40	0.00	6.45	5.82	4.66	5.35	5.73	2.91	13.47	6.00
20,000	5.91	0.00	5.45	4.39	3.58	5.22	5.06	1.91	7.83	5.97
22,000	9.56	0.00	5.47	4.15	2.42	4.87	5.70	1.04	8.38	4.79
24,000	10.61	0.00	5.74	4.68	3.64	5.67	6.39	0.57	6.51	5.46
26,000	7.87	0.00	6.18	4.54	3.15	5.93	4.06	0.43	3.84	6.28
28,000	6.64	0.00	5.36	3.97	1.51	6.03	5.21	0.57	3.13	6.13
30,000	6.89	0.00	4.73	3.93	0.90	6.35	5.75	0.86	2.59	5.67
32,000	6.93	0.00	3.75	2.64	0.66	5.48	5.30	0.84	1.88	3.80
34,000	4.51	0.00	3.39	3.24	0.59	5.31	4.04	0.85	1.28	3.37
36,000	3.71	0.00	2.63	3.07	0.55	4.76	2.85	0.89	0.79	2.95
38,000	2.90	0.00	2.43	2.07	0.40	3.81	2.13	0.30	0.68	1.84
40,000	1.72	0.00	1.83	1.68	0.24	2.74	1.83	0.27	0.35	1.79
42,000	1.30	0.00	1.56	1.42	0.18	2.25	1.59	0.20	0.42	1.14
44,000	0.79	0.00	1.88	0.59	0.18	1.47	0.66	0.21	0.36	0.91
46,000	0.76	0.00	1.26	0.45	0.15	1.18	0.54	0.23	0.42	0.53
48,000	0.51	0.00	0.96	0.40	0.12	0.62	0.42	0.17	0.15	0.33
50,000	1.07	0.00	0.46	0.42	0.10	0.38	0.57	0.14	0.10	0.28
52,000	1.41	0.00	0.24	0.35	0.12	0.17	0.24	0.08	0.15	0.44
54,000	0.91	0.00	0.19	0.26	0.08	0.31	0.15	0.10	0.09	0.36
56,000	0.60	0.00	0.55	0.29	0.08	0.19	0.09	0.18	0.04	0.12
58,000	0.16	0.00	0.12	0.15	0.05	0.12	0.08	0.12	0.04	0.06
60,000	0.03	0.00	0.07	0.18	0.04	0.05	0.08	0.13	0.03	0.09
62,000	0.09	0.00	0.07	0.40	0.05	0.05	0.04	0.06	0.06	0.03
64,000	0.22	0.00	0.03	0.32	0.05	0.04	0.02	0.11	0.04	0.06
66,000	0.24	0.00	0.02	0.21	0.08	0.04	0.13	0.12	0.03	0.05
68,000	0.38	0.00	0.01	0.11	0.06	0.03	0.51	0.13	0.05	0.03
70,000	0.16	0.00	0.02	0.08	0.02	0.01	0.71	0.16	0.06	0.01
72,000	0.00	0.00	0.02	0.10	0.01	0.00	0.68	0.09	0.06	0.06
74,000	0.01	0.00	0.03	0.07	0.01	0.00	0.24	0.08	0.04	0.03
76,000	0.01	0.00	0.01	0.05	0.01	0.00	0.02	0.03	0.02	0.01
78,000	0.00	0.00	0.01	0.09	0.02	0.00	0.01	0.02	0.00	0.01
80,000	0.00	0.00	0.00	0.04	0.04	0.03	0.01	0.05	0.01	0.01
82,000	0.15	0.00	0.01	0.08	0.05	0.01	0.05	0.18	0.03	0.05

Table 17. Idaho Statewide Average Tridem-Axle Distribution

Axle Load (lb)	Vehicle Class									
	4	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
12,000	0.00	0.00	42.61	13.22	14.86	40.49	12.16	3.66	30.50	19.41
15,000	0.00	0.00	7.04	3.73	9.56	12.48	7.10	3.84	6.29	7.94
18,000	0.00	0.00	7.37	4.61	25.09	9.37	5.68	16.10	14.17	5.64
21,000	0.00	0.00	9.01	6.32	22.10	7.78	5.51	22.67	3.32	3.85
24,000	0.00	0.00	8.84	5.22	13.32	3.49	4.62	9.36	1.36	3.05
27,000	0.00	0.00	7.59	6.66	2.38	4.49	4.11	8.81	4.76	4.87
30,000	0.00	0.00	7.06	7.04	1.71	6.07	7.31	1.71	8.20	7.18
33,000	0.00	0.00	1.46	6.45	1.08	2.40	6.40	4.17	7.21	10.89
36,000	0.00	0.00	4.40	8.94	0.51	3.14	8.83	2.37	4.84	9.89
39,000	0.00	0.00	1.25	8.90	0.64	1.93	8.71	0.71	3.61	6.94
42,000	0.00	0.00	1.28	6.76	0.68	1.79	7.36	0.68	2.13	5.11
45,000	0.00	0.00	1.20	5.90	0.55	1.63	6.54	1.19	1.91	5.20
48,000	0.00	0.00	0.47	5.37	0.64	1.69	5.39	0.23	1.84	2.64
51,000	0.00	0.00	0.22	3.33	0.28	1.46	3.16	0.74	1.62	1.22
54,000	0.00	0.00	0.18	2.43	0.57	0.29	2.42	5.72	1.76	1.41
57,000	0.00	0.00	0.01	1.82	0.42	0.27	1.48	2.87	1.06	1.22
60,000	0.00	0.00	0.01	1.14	0.46	0.17	1.24	3.80	0.74	0.57
63,000	0.00	0.00	0.00	0.60	0.37	0.09	0.51	4.92	1.03	0.68
66,000	0.00	0.00	0.00	0.27	0.75	0.07	0.48	1.44	0.56	0.51
69,000	0.00	0.00	0.00	0.25	0.71	0.18	0.27	1.95	0.13	0.35
72,000	0.00	0.00	0.00	0.09	0.27	0.09	0.24	1.53	0.33	0.29
75,000	0.00	0.00	0.00	0.09	0.43	0.02	0.08	0.34	0.34	0.10
78,000	0.00	0.00	0.00	0.12	0.67	0.02	0.05	0.00	0.17	0.11
81,000	0.00	0.00	0.00	0.02	0.46	0.05	0.10	0.00	0.59	0.08
84,000	0.00	0.00	0.00	0.02	0.06	0.08	0.01	0.00	0.86	0.13
87,000	0.00	0.00	0.00	0.02	0.11	0.02	0.01	0.40	0.14	0.04
90,000	0.00	0.00	0.00	0.04	0.41	0.05	0.03	0.00	0.16	0.12
93,000	0.00	0.00	0.00	0.21	0.16	0.00	0.02	0.00	0.13	0.11
96,000	0.00	0.00	0.00	0.02	0.14	0.00	0.03	0.08	0.22	0.03
99,000	0.00	0.00	0.00	0.02	0.05	0.08	0.02	0.71	0.02	0.05
102,000	0.00	0.00	0.00	0.39	0.56	0.31	0.13	0.00	0.00	0.37

Table 18. Idaho Statewide Average Quad-Axle Distribution

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	10.85	27.34	18.21	4.77	0.00	14.78	8.29
15,000	0.00	0.00	0.00	3.91	8.72	6.68	3.52	0.00	4.66	2.56
18,000	0.00	0.00	0.00	3.22	6.30	13.83	2.94	2.72	3.31	3.06
21,000	0.00	0.00	0.00	4.57	6.60	10.70	2.27	16.20	5.90	2.04
24,000	0.00	0.00	0.00	6.90	2.62	8.81	1.91	17.69	7.13	1.86
27,000	0.00	0.00	0.00	7.74	5.86	6.19	2.55	10.22	6.20	2.22
30,000	0.00	0.00	0.00	6.54	5.18	3.71	2.34	6.51	7.84	3.20
33,000	0.00	0.00	0.00	4.61	3.54	1.08	3.47	9.77	2.08	6.76
36,000	0.00	0.00	0.00	2.94	1.35	2.05	5.47	13.31	3.97	3.74
39,000	0.00	0.00	0.00	3.88	4.80	4.52	9.09	10.48	9.08	4.61
42,000	0.00	0.00	0.00	3.56	4.73	3.38	6.89	9.99	4.38	4.79
45,000	0.00	0.00	0.00	2.82	5.68	2.40	10.90	2.53	2.93	5.77
48,000	0.00	0.00	0.00	4.11	1.24	2.12	10.80	0.58	1.91	4.29
51,000	0.00	0.00	0.00	4.83	2.22	0.72	9.04	0.00	0.37	5.44
54,000	0.00	0.00	0.00	3.69	2.53	1.13	6.06	0.00	1.22	3.99
57,000	0.00	0.00	0.00	2.84	1.25	2.85	4.23	0.00	0.13	4.85
60,000	0.00	0.00	0.00	1.48	1.64	0.95	2.69	0.00	1.06	4.74
63,000	0.00	0.00	0.00	1.36	2.01	1.80	2.46	0.00	0.13	4.72
66,000	0.00	0.00	0.00	1.27	2.05	1.50	2.16	0.00	0.93	4.02
69,000	0.00	0.00	0.00	1.33	0.51	1.60	1.78	0.00	2.45	4.60
72,000	0.00	0.00	0.00	1.64	0.47	0.74	1.50	0.00	2.40	4.17
75,000	0.00	0.00	0.00	1.28	1.03	0.81	1.23	0.00	3.14	1.83
78,000	0.00	0.00	0.00	1.16	0.00	1.64	0.58	0.00	3.84	1.41
81,000	0.00	0.00	0.00	1.65	0.00	0.70	0.20	0.00	4.12	1.00
84,000	0.00	0.00	0.00	0.75	0.04	1.71	0.11	0.00	1.94	1.13
87,000	0.00	0.00	0.00	1.89	0.21	0.17	0.08	0.00	1.31	1.01
90,000	0.00	0.00	0.00	1.90	0.25	0.00	0.07	0.00	1.00	0.60
93,000	0.00	0.00	0.00	2.42	0.20	0.00	0.14	0.00	0.17	0.58
96,000	0.00	0.00	0.00	1.65	0.20	0.00	0.14	0.00	0.09	0.57
99,000	0.00	0.00	0.00	1.20	0.64	0.00	0.09	0.00	0.26	0.27
102,000	0.00	0.00	0.00	2.01	0.79	0.00	0.52	0.00	1.27	1.88

Table 19. Single-Axle Load Distribution for Primarily Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
3,000	2.74	14.66	2.32	8.76	10.75	1.87	1.08	5.49	10.60	4.28
4,000	1.08	16.44	3.81	3.11	9.48	2.58	1.04	1.88	7.33	2.98
5,000	3.83	11.48	4.11	3.27	13.36	2.60	2.76	7.38	2.73	3.92
6,000	7.04	11.73	4.12	4.90	9.48	3.38	1.72	11.53	6.12	8.17
7,000	7.03	5.78	4.57	2.83	6.56	5.08	2.58	8.37	5.82	9.51
8,000	12.80	6.56	7.40	6.10	10.22	8.57	7.67	8.94	9.81	7.50
9,000	8.33	5.81	5.26	6.34	4.72	10.69	11.02	5.11	7.99	10.03
10,000	7.24	4.48	7.07	7.23	6.82	13.47	16.90	7.09	14.20	11.80
11,000	5.93	2.53	7.55	5.17	3.80	14.52	11.98	5.46	9.95	8.19
12,000	3.64	2.23	12.07	9.87	4.39	11.90	10.07	5.17	6.60	7.58
13,000	5.51	2.43	11.12	11.24	3.00	7.59	7.26	3.89	4.06	4.70
14,000	7.75	1.75	8.17	7.02	2.81	3.49	3.13	4.05	2.09	3.77
15,000	7.13	1.76	8.49	4.38	2.55	2.83	3.50	4.28	2.33	3.36
16,000	3.97	1.26	3.02	2.69	1.17	2.11	0.93	3.56	1.38	2.29
17,000	1.97	1.20	2.79	2.95	0.98	1.72	0.79	2.52	1.25	2.81
18,000	2.13	1.08	1.18	3.09	0.95	1.45	0.63	2.11	0.91	1.93
19,000	1.46	1.40	1.20	1.21	1.73	1.13	2.49	2.36	0.69	1.27
20,000	0.93	1.15	0.84	1.43	0.98	0.85	5.83	1.85	0.44	1.04
21,000	1.09	0.88	1.47	1.20	1.76	0.62	4.12	1.20	0.40	0.84
22,000	1.06	0.52	1.74	0.82	0.98	0.70	1.54	0.82	0.35	0.49
23,000	1.25	0.51	0.86	2.33	0.64	0.73	0.50	1.10	0.42	0.43
24,000	1.04	0.40	0.20	1.64	0.33	0.42	0.70	0.98	0.41	0.60
25,000	1.26	0.26	0.31	0.57	0.23	0.17	0.68	0.57	0.61	0.64
26,000	0.78	0.35	0.04	0.42	0.22	0.25	0.23	0.33	0.59	0.24
27,000	0.71	0.43	0.02	0.78	0.19	0.18	0.08	0.28	0.27	0.17
28,000	0.55	0.76	0.01	0.03	0.21	0.11	0.07	0.25	0.26	0.14
29,000	0.36	0.51	0.00	0.21	0.14	0.05	0.04	0.38	0.15	0.13
30,000	0.13	0.27	0.00	0.03	0.14	0.17	0.09	0.64	0.15	0.14
31,000	0.16	0.20	0.00	0.00	0.11	0.08	0.03	0.32	0.16	0.09
32,000	0.28	0.15	0.00	0.00	0.10	0.06	0.00	0.15	0.14	0.08
33,000	0.25	0.11	0.00	0.00	0.12	0.05	0.04	0.30	0.11	0.05
34,000	0.17	0.11	0.00	0.00	0.12	0.12	0.11	0.29	0.13	0.10
35,000	0.08	0.09	0.00	0.00	0.10	0.07	0.05	0.18	0.13	0.08
36,000	0.07	0.10	0.01	0.00	0.09	0.04	0.01	0.19	0.19	0.06
37,000	0.02	0.10	0.02	0.00	0.09	0.07	0.03	0.13	0.22	0.07
38,000	0.00	0.06	0.00	0.03	0.09	0.06	0.03	0.14	0.21	0.07
39,000	0.06	0.03	0.00	0.01	0.10	0.01	0.04	0.14	0.15	0.09
40,000	0.03	0.10	0.07	0.22	0.12	0.04	0.05	0.19	0.23	0.10
41,000	0.14	0.33	0.16	0.12	0.37	0.17	0.18	0.38	0.42	0.26

Table 20. Tandem-Axle Load Distribution for Primarily Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
6,000	1.87	0.00	6.16	15.63	30.21	1.72	2.70	4.75	8.51	4.58
8,000	1.82	0.00	5.53	7.76	13.38	2.17	9.29	8.44	3.40	5.67
10,000	1.47	0.00	3.73	5.20	8.27	5.28	4.36	8.21	8.06	6.00
12,000	3.92	0.00	4.12	7.78	8.10	9.30	4.09	12.60	4.83	12.96
14,000	2.12	0.00	4.43	6.56	7.21	7.24	5.98	19.12	6.49	5.67
16,000	3.71	0.00	4.89	5.99	6.04	6.25	8.88	10.35	11.40	6.72
18,000	2.84	0.00	5.91	6.33	4.16	4.99	4.73	4.79	19.99	6.17
20,000	4.52	0.00	5.23	4.69	2.61	5.86	4.29	4.32	9.14	6.22
22,000	11.32	0.00	6.59	3.97	2.30	5.24	4.23	1.95	10.45	4.60
24,000	13.62	0.00	7.28	4.73	5.60	4.42	5.17	1.53	5.57	5.25
26,000	8.71	0.00	7.29	4.69	4.93	4.83	2.31	1.04	2.20	5.25
28,000	5.41	0.00	5.88	3.89	1.86	3.99	3.67	1.90	1.26	5.68
30,000	5.17	0.00	4.25	3.85	0.67	4.91	5.29	1.27	0.49	5.03
32,000	5.99	0.00	3.17	2.15	0.48	3.86	7.41	1.51	0.72	2.80
34,000	2.70	0.00	3.52	3.09	0.51	4.75	5.69	1.82	1.50	3.46
36,000	2.37	0.00	2.54	2.73	0.50	5.50	3.42	3.2	0.78	3.71
38,000	1.31	0.00	2.90	3.17	0.38	4.86	2.67	1.03	0.52	2.26
40,000	1.88	0.00	2.46	1.67	0.24	3.45	2.64	1.11	0.40	2.16
42,000	0.9	0.00	2.38	1.43	0.25	3.20	2.54	0.84	0.87	1.30
44,000	1.33	0.00	3.32	0.31	0.27	2.19	0.8	0.88	0.66	1.04
46,000	0.49	0.00	2.40	0.3	0.29	1.87	0.48	0.97	0.74	0.80
48,000	1.03	0.00	2.02	0.55	0.21	1.16	0.7	0.73	0.27	0.60
50,000	2.93	0.00	0.94	0.52	0.16	0.76	1.15	0.60	0.07	0.46
52,000	4.08	0.00	0.40	0.34	0.22	0.29	0.41	0.36	0.14	0.27
54,000	2.69	0.00	0.32	0.06	0.15	0.66	0.30	0.44	0.23	0.13
56,000	1.75	0.00	1.51	0.46	0.14	0.44	0.15	0.77	0.1	0.11
58,000	0.45	0.00	0.29	0.13	0.08	0.26	0.16	0.53	0.11	0.05
60,000	0.07	0.00	0.13	0.03	0.04	0.10	0.21	0.56	0.09	0.20
62,000	0.22	0.00	0.11	0.04	0.12	0.11	0.09	0.25	0.14	0.06
64,000	0.66	0.00	0.05	0.1	0.06	0.09	0.03	0.49	0.10	0.14
66,000	0.71	0.00	0.02	0.48	0.11	0.08	0.31	0.50	0.08	0.14
68,000	1.06	0.00	0.01	0.22	0.09	0.05	1.33	0.55	0.12	0.07
70,000	0.44	0.00	0.01	0.19	0.06	0.01	1.89	0.67	0.15	0.03
72,000	0.00	0.00	0.05	0.10	0.01	0.00	1.81	0.38	0.15	0.14
74,000	0.00	0.00	0.08	0.14	0.01	0.00	0.62	0.33	0.11	0.08
76,000	0.00	0.00	0.02	0.09	0.03	0.01	0.06	0.13	0.06	0.03
78,000	0.00	0.00	0.04	0.18	0.04	0.00	0.02	0.07	0.00	0.02
80,000	0.00	0.00	0.01	0.15	0.1	0.08	0.02	0.23	0.02	0.02
82,000	0.44	0.00	0.01	0.30	0.11	0.02	0.10	0.78	0.08	0.12

Table 21. Tridem-Axle Load Distribution for Primarily Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	22.70	8.17	36.82	28.42	7.66	5.48	37.88	26.67
15,000	0.00	0.00	6.88	3.72	8.75	17.22	4.88	5.76	4.45	6.26
18,000	0.00	0.00	1.63	2.56	5.66	7.44	4.82	6.48	1.53	5.80
21,000	0.00	0.00	6.99	7.57	8.72	4.36	6.61	6.48	2.88	4.23
24,000	0.00	0.00	11.73	4.78	6.34	4.29	2.51	9.51	3.14	2.12
27,000	0.00	0.00	15.01	3.84	2.42	8.30	3.44	12.95	2.93	6.14
30,000	0.00	0.00	14.13	3.59	3.89	9.66	2.79	2.54	3.53	4.26
33,000	0.00	0.00	2.91	4.55	2.70	3.73	2.38	6.26	3.31	8.77
36,000	0.00	0.00	8.80	7.55	1.26	2.10	9.27	3.56	2.89	6.62
39,000	0.00	0.00	2.49	11.05	1.60	0.77	11.03	1.07	3.14	5.87
42,000	0.00	0.00	2.56	8.82	1.69	1.27	8.88	1.02	3.92	4.03
45,000	0.00	0.00	2.40	6.47	1.36	1.29	8.91	1.78	5.33	3.63
48,000	0.00	0.00	0.94	6.06	1.60	3.72	8.81	0.35	5.47	2.10
51,000	0.00	0.00	0.44	5.69	0.69	2.32	4.85	1.12	2.18	1.19
54,000	0.00	0.00	0.36	4.65	1.43	0.67	4.00	8.58	1.39	2.13
57,000	0.00	0.00	0.02	2.30	1.05	0.35	2.63	4.30	1.20	1.98
60,000	0.00	0.00	0.01	2.44	1.14	0.50	2.57	5.70	1.38	1.16
63,000	0.00	0.00	0.00	1.26	0.91	0.29	0.90	7.38	1.95	1.43
66,000	0.00	0.00	0.00	0.53	1.87	0.24	0.91	2.15	0.58	0.78
69,000	0.00	0.00	0.00	0.85	1.78	0.60	0.62	2.93	0.38	0.78
72,000	0.00	0.00	0.00	0.26	0.66	0.30	0.57	2.30	1.07	0.58
75,000	0.00	0.00	0.00	0.11	1.08	0.08	0.15	0.51	1.12	0.05
78,000	0.00	0.00	0.00	0.13	1.68	0.06	0.02	0.00	0.59	0.02
81,000	0.00	0.00	0.00	0.07	1.14	0.18	0.26	0.00	2.16	0.18
84,000	0.00	0.00	0.00	0.07	0.16	0.27	0.01	0.00	3.16	0.38
87,000	0.00	0.00	0.00	0.02	0.28	0.07	0.01	0.60	0.51	0.15
90,000	0.00	0.00	0.00	0.19	1.03	0.15	0.06	0.00	0.59	0.48
93,000	0.00	0.00	0.00	0.91	0.41	0.02	0.05	0.00	0.49	0.43
96,000	0.00	0.00	0.00	0.06	0.35	0.01	0.06	0.13	0.79	0.12
99,000	0.00	0.00	0.00	0.03	0.14	0.27	0.04	1.06	0.06	0.19
102,000	0.00	0.00	0.00	1.70	1.39	1.05	0.30	0.00	0.00	1.47

Table 22. Quad-Axle Load Distribution for Primarily Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	12.99	27.34	18.10	3.43	0.00	14.78	18.51
15,000	0.00	0.00	0.00	5.94	8.71	7.61	2.60	0.00	4.66	5.11
18,000	0.00	0.00	0.00	5.44	6.30	10.46	2.71	2.72	3.31	1.52
21,000	0.00	0.00	0.00	5.62	6.60	8.17	3.00	16.20	5.90	1.86
24,000	0.00	0.00	0.00	5.58	2.62	9.44	1.95	17.69	7.13	0.68
27,000	0.00	0.00	0.00	8.02	5.86	4.65	3.62	10.22	6.20	1.03
30,000	0.00	0.00	0.00	9.60	5.18	3.84	2.23	6.51	7.84	1.41
33,000	0.00	0.00	0.00	6.42	3.54	1.24	2.09	9.77	2.08	7.88
36,000	0.00	0.00	0.00	2.73	1.35	2.95	2.92	13.31	3.97	5.24
39,000	0.00	0.00	0.00	1.74	4.80	6.78	5.58	10.48	9.08	3.57
42,000	0.00	0.00	0.00	2.92	4.73	5.07	4.87	9.99	4.38	2.29
45,000	0.00	0.00	0.00	3.18	5.68	3.60	14.60	2.53	2.93	4.09
48,000	0.00	0.00	0.00	2.00	1.24	3.17	16.27	0.58	1.91	3.42
51,000	0.00	0.00	0.00	2.20	2.22	1.09	7.74	0.00	0.37	3.88
54,000	0.00	0.00	0.00	3.71	2.53	1.69	3.99	0.00	1.22	3.28
57,000	0.00	0.00	0.00	5.68	1.25	4.08	3.55	0.00	0.13	5.65
60,000	0.00	0.00	0.00	3.56	1.65	0.00	3.09	0.00	1.06	3.52
63,000	0.00	0.00	0.00	2.58	2.01	0.00	1.96	0.00	0.13	2.49
66,000	0.00	0.00	0.00	1.25	2.05	0.00	1.25	0.00	0.93	1.78
69,000	0.00	0.00	0.00	0.54	0.51	0.37	2.00	0.00	2.45	1.72
72,000	0.00	0.00	0.00	0.42	0.47	0.97	3.45	0.00	2.40	2.09
75,000	0.00	0.00	0.00	0.49	1.03	1.08	3.00	0.00	3.14	1.22
78,000	0.00	0.00	0.00	0.36	0.00	2.11	1.34	0.00	3.84	2.06
81,000	0.00	0.00	0.00	0.17	0.00	0.71	0.44	0.00	4.12	1.76
84,000	0.00	0.00	0.00	0.19	0.04	2.57	0.26	0.00	1.94	2.52
87,000	0.00	0.00	0.00	0.31	0.21	0.25	0.09	0.00	1.31	2.38
90,000	0.00	0.00	0.00	0.71	0.25	0.00	0.04	0.00	1.00	1.27
93,000	0.00	0.00	0.00	0.49	0.20	0.00	0.16	0.00	0.17	0.51
96,000	0.00	0.00	0.00	0.55	0.20	0.00	0.35	0.00	0.09	1.00
99,000	0.00	0.00	0.00	0.92	0.64	0.00	0.22	0.00	0.26	0.58
102,000	0.00	0.00	0.00	3.69	0.79	0.00	1.24	0.00	1.27	5.68

Table 23. Single-AxleLoad Distribution for Moderately Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
3,000	5.66	7.18	1.80	4.76	18.54	1.88	1.28	12.76	7.42	7.04
4,000	2.79	6.44	2.56	3.35	8.00	2.40	0.71	9.88	5.45	7.25
5,000	2.89	7.57	3.70	2.19	7.93	2.55	1.24	11.33	6.36	6.16
6,000	6.09	10.47	5.84	5.53	8.83	3.52	2.64	9.53	6.35	6.78
7,000	6.24	8.85	6.86	7.87	7.27	4.30	4.28	7.55	8.39	5.51
8,000	9.45	9.39	9.45	9.11	8.61	7.00	8.01	8.20	11.36	6.85
9,000	7.99	7.44	9.56	8.92	8.09	7.45	9.79	6.45	10.93	7.37
10,000	9.50	7.11	12.40	9.28	7.61	12.67	14.26	7.08	10.46	9.93
11,000	7.72	5.66	10.30	9.46	5.38	12.78	15.08	5.41	8.78	7.71
12,000	9.31	5.30	9.70	9.22	4.25	14.74	16.75	3.82	7.14	8.47
13,000	7.60	4.33	7.64	7.89	3.65	10.62	11.46	2.81	4.90	6.76
14,000	5.26	3.65	5.04	4.78	1.84	5.00	5.03	1.90	2.81	3.99
15,000	5.62	3.36	4.76	2.90	2.11	4.07	3.87	2.03	2.53	4.69
16,000	3.23	2.68	2.96	4.66	1.39	2.11	1.59	1.62	2.25	2.62
17,000	3.21	2.76	2.48	3.45	1.41	2.16	1.27	1.83	1.49	2.60
18,000	1.72	2.31	1.66	2.04	1.19	1.62	0.71	1.51	0.96	1.79
19,000	1.77	1.90	1.35	1.73	1.08	1.56	0.56	1.21	0.64	1.48
20,000	1.20	1.07	0.72	0.76	0.79	1.00	0.35	0.73	0.45	0.91
21,000	0.76	0.84	0.44	0.72	0.56	0.92	0.26	0.73	0.35	0.78
22,000	0.72	0.53	0.42	0.26	0.42	0.56	0.20	0.82	0.15	0.47
23,000	0.41	0.52	0.14	0.19	0.31	0.43	0.13	0.81	0.18	0.25
24,000	0.24	0.31	0.06	0.21	0.19	0.23	0.13	0.41	0.38	0.17
25,000	0.15	0.10	0.05	0.17	0.10	0.15	0.06	0.20	0.13	0.16
26,000	0.22	0.07	0.02	0.06	0.10	0.11	0.05	0.10	0.05	0.06
27,000	0.07	0.05	0.01	0.05	0.07	0.07	0.03	0.23	0.07	0.06
28,000	0.03	0.02	0.02	0.12	0.05	0.04	0.05	0.07	0.02	0.02
29,000	0.02	0.01	0.02	0.17	0.04	0.02	0.02	0.03	0.00	0.02
30,000	0.07	0.02	0.01	0.00	0.03	0.01	0.05	0.06	0.00	0.01
31,000	0.04	0.02	0.00	0.02	0.04	0.02	0.03	0.10	0.00	0.01
32,000	0.00	0.03	0.00	0.05	0.03	0.01	0.02	0.20	0.00	0.02
33,000	0.01	0.01	0.03	0.05	0.02	0.00	0.03	0.09	0.00	0.01
34,000	0.00	0.00	0.00	0.01	0.01	0.00	0.02	0.05	0.00	0.00
35,000	0.00	0.00	0.00	0.00	0.03	0.00	0.01	0.17	0.00	0.00
36,000	0.01	0.00	0.00	0.01	0.00	0.00	0.01	0.05	0.00	0.00
37,000	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.07	0.00	0.04
38,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00
39,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00
40,000	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.00	0.00
41,000	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.09	0.00	0.01

Table 24. Tandem-Axle Load Distribution for Moderately Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
6,000	6.47	0.00	5.56	9.97	31.80	2.15	4.94	28.29	6.79	8.27
8,000	2.78	0.00	6.38	6.68	11.06	3.70	4.29	8.59	5.82	7.70
10,000	3.58	0.00	8.27	11.95	9.76	5.59	7.82	18.10	8.25	8.54
12,000	3.70	0.00	8.53	10.93	8.72	6.48	10.74	23.52	10.38	8.86
14,000	2.98	0.00	8.39	7.85	8.22	6.56	10.21	12.62	9.06	7.30
16,000	4.47	0.00	6.87	5.49	6.41	6.22	8.75	2.65	11.47	6.17
18,000	4.77	0.00	6.61	5.76	5.50	5.25	6.35	2.39	9.71	5.33
20,000	5.55	0.00	5.26	4.22	4.65	5.03	5.32	1.09	6.61	4.89
22,000	7.59	0.00	4.89	3.93	2.98	4.72	5.08	0.12	5.63	5.15
24,000	8.22	0.00	4.93	4.44	2.63	6.28	5.71	0.00	5.86	5.87
26,000	7.01	0.00	5.22	4.23	2.21	6.16	4.69	0.00	4.60	5.60
28,000	6.61	0.00	4.85	4.10	1.47	6.72	4.30	0.04	4.48	5.62
30,000	7.73	0.00	4.92	3.42	1.17	7.41	4.43	0.87	4.45	5.47
32,000	7.90	0.00	4.00	2.70	0.88	7.11	4.31	0.74	2.60	4.48
34,000	5.97	0.00	3.54	2.87	0.80	5.99	3.29	0.66	1.20	3.48
36,000	4.76	0.00	2.88	2.80	0.72	4.76	2.75	0.22	0.92	2.40
38,000	4.07	0.00	2.35	1.50	0.47	3.47	2.01	0.09	0.91	1.66
40,000	1.84	0.00	1.75	1.65	0.25	2.53	1.58	0.01	0.36	1.50
42,000	1.74	0.00	1.29	0.97	0.06	1.55	1.23	0.00	0.15	0.64
44,000	0.56	0.00	1.30	0.46	0.10	0.86	0.61	0.00	0.19	0.50
46,000	1.02	0.00	0.79	0.23	0.05	0.59	0.65	0.00	0.24	0.24
48,000	0.29	0.00	0.51	0.32	0.00	0.29	0.30	0.00	0.08	0.16
50,000	0.15	0.00	0.26	0.43	0.01	0.18	0.28	0.00	0.10	0.06
52,000	0.07	0.00	0.18	0.41	0.04	0.12	0.14	0.00	0.14	0.03
54,000	0.01	0.00	0.15	0.36	0.00	0.11	0.06	0.00	0.00	0.03
56,000	0.04	0.00	0.08	0.24	0.00	0.04	0.07	0.00	0.00	0.01
58,000	0.02	0.00	0.03	0.17	0.01	0.04	0.02	0.00	0.00	0.01
60,000	0.01	0.00	0.05	0.28	0.04	0.03	0.01	0.00	0.00	0.01
62,000	0.03	0.00	0.06	0.62	0.00	0.02	0.02	0.00	0.00	0.01
64,000	0.00	0.00	0.02	0.46	0.00	0.01	0.01	0.00	0.00	0.00
66,000	0.00	0.00	0.02	0.14	0.00	0.01	0.01	0.00	0.00	0.00
68,000	0.05	0.00	0.01	0.09	0.00	0.01	0.01	0.00	0.00	0.00
70,000	0.01	0.00	0.03	0.05	0.00	0.01	0.00	0.00	0.00	0.00
72,000	0.00	0.00	0.01	0.12	0.00	0.00	0.00	0.00	0.00	0.00
74,000	0.00	0.00	0.01	0.06	0.00	0.00	0.01	0.00	0.00	0.01
76,000	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.00	0.00	0.00
78,000	0.00	0.00	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.00
80,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
82,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 25. Tridem-Axle Load Distribution for Moderately Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	93.81	12.62	0.00	46.12	16.06	0.00	28.69	17.88
15,000	0.00	0.00	5.96	3.95	7.46	9.66	8.19	0.00	7.90	10.04
18,000	0.00	0.00	0.23	5.55	41.03	8.38	5.36	0.00	21.61	6.93
21,000	0.00	0.00	0.00	5.88	34.63	8.83	4.77	0.00	3.99	4.54
24,000	0.00	0.00	0.00	5.23	14.97	3.35	4.27	0.00	0.78	3.85
27,000	0.00	0.00	0.00	7.76	1.91	2.68	4.55	0.00	6.19	4.91
30,000	0.00	0.00	0.00	8.25	0.00	5.15	7.07	0.00	10.83	7.88
33,000	0.00	0.00	0.00	7.01	0.00	2.12	6.88	0.00	8.62	8.27
36,000	0.00	0.00	0.00	8.95	0.00	4.18	8.72	0.00	4.97	8.36
39,000	0.00	0.00	0.00	7.69	0.00	2.83	8.73	0.00	2.65	7.80
42,000	0.00	0.00	0.00	6.61	0.00	2.26	7.85	0.00	0.18	5.44
45,000	0.00	0.00	0.00	6.39	0.00	1.75	6.32	0.00	0.01	5.25
48,000	0.00	0.00	0.00	5.72	0.00	0.95	4.16	0.00	0.19	3.06
51,000	0.00	0.00	0.00	2.78	0.00	1.28	2.60	0.00	1.22	1.43
54,000	0.00	0.00	0.00	1.88	0.00	0.15	1.68	0.00	1.65	1.42
57,000	0.00	0.00	0.00	1.83	0.00	0.28	1.01	0.00	0.47	1.17
60,000	0.00	0.00	0.00	0.76	0.00	0.03	0.61	0.00	0.05	0.41
63,000	0.00	0.00	0.00	0.47	0.00	0.00	0.38	0.00	0.00	0.30
66,000	0.00	0.00	0.00	0.20	0.00	0.00	0.29	0.00	0.00	0.20
69,000	0.00	0.00	0.00	0.09	0.00	0.00	0.10	0.00	0.00	0.22
72,000	0.00	0.00	0.00	0.05	0.00	0.00	0.08	0.00	0.00	0.23
75,000	0.00	0.00	0.00	0.09	0.00	0.00	0.06	0.00	0.00	0.13
78,000	0.00	0.00	0.00	0.14	0.00	0.00	0.10	0.00	0.00	0.16
81,000	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.05
84,000	0.00	0.00	0.00	0.01	0.00	0.00	0.02	0.00	0.00	0.06
87,000	0.00	0.00	0.00	0.02	0.00	0.00	0.02	0.00	0.00	0.01
90,000	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00
93,000	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.00	0.00	0.00
96,000	0.00	0.00	0.00	0.02	0.00	0.00	0.01	0.00	0.00	0.00
99,000	0.00	0.00	0.00	0.02	0.00	0.00	0.01	0.00	0.00	0.00
102,000	0.00	0.00	0.00	0.02	0.00	0.00	0.05	0.00	0.00	0.00

Table 26. Quad-Axle Load Distribution for Moderately Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	11.08	0.00	0.00	6.55	0.00	0.00	4.84
15,000	0.00	0.00	0.00	3.49	0.00	0.00	3.89	0.00	0.00	1.60
18,000	0.00	0.00	0.00	2.64	0.00	0.00	2.76	0.00	0.00	3.12
21,000	0.00	0.00	0.00	4.07	0.00	0.00	2.45	0.00	0.00	2.42
24,000	0.00	0.00	0.00	6.71	0.00	0.00	2.18	0.00	0.00	2.80
27,000	0.00	0.00	0.00	6.68	0.00	0.00	2.65	0.00	0.00	3.26
30,000	0.00	0.00	0.00	4.09	0.00	0.00	2.94	0.00	0.00	4.73
33,000	0.00	0.00	0.00	3.87	0.00	0.00	4.46	0.00	0.00	7.50
36,000	0.00	0.00	0.00	3.26	0.00	0.00	5.16	0.00	0.00	3.24
39,000	0.00	0.00	0.00	5.12	0.00	0.00	6.88	0.00	0.00	5.04
42,000	0.00	0.00	0.00	4.17	0.00	0.00	6.93	0.00	0.00	5.75
45,000	0.00	0.00	0.00	2.96	0.00	0.00	7.75	0.00	0.00	6.34
48,000	0.00	0.00	0.00	5.37	0.00	0.00	8.11	0.00	0.00	5.13
51,000	0.00	0.00	0.00	6.36	0.00	0.00	9.60	0.00	0.00	7.18
54,000	0.00	0.00	0.00	4.06	0.00	0.00	8.46	0.00	0.00	4.98
57,000	0.00	0.00	0.00	1.99	0.00	0.00	5.27	0.00	0.00	5.40
60,000	0.00	0.00	0.00	0.80	0.00	0.00	2.97	0.00	0.00	6.29
63,000	0.00	0.00	0.00	0.69	0.00	0.00	3.37	0.00	0.00	6.71
66,000	0.00	0.00	0.00	1.02	0.00	0.00	3.07	0.00	0.00	5.09
69,000	0.00	0.00	0.00	1.19	0.00	0.00	2.11	0.00	0.00	2.43
72,000	0.00	0.00	0.00	2.14	0.00	0.00	0.86	0.00	0.00	2.25
75,000	0.00	0.00	0.00	1.51	0.00	0.00	0.48	0.00	0.00	1.00
78,000	0.00	0.00	0.00	1.30	0.00	0.00	0.32	0.00	0.00	0.64
81,000	0.00	0.00	0.00	1.16	0.00	0.00	0.13	0.00	0.00	0.74
84,000	0.00	0.00	0.00	0.99	0.00	0.00	0.06	0.00	0.00	0.57
87,000	0.00	0.00	0.00	2.55	0.00	0.00	0.10	0.00	0.00	0.34
90,000	0.00	0.00	0.00	2.16	0.00	0.00	0.11	0.00	0.00	0.12
93,000	0.00	0.00	0.00	3.35	0.00	0.00	0.09	0.00	0.00	0.08
96,000	0.00	0.00	0.00	2.25	0.00	0.00	0.06	0.00	0.00	0.08
99,000	0.00	0.00	0.00	1.43	0.00	0.00	0.05	0.00	0.00	0.11
102,000	0.00	0.00	0.00	1.54	0.00	0.00	0.18	0.00	0.00	0.22

Table 27. Single-Axle Load Distribution for Lightly Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
3,000	0.57	2.84	0.73	6.20	16.21	3.26	0.96	6.47	2.68	2.66
4,000	0.29	10.63	1.62	0.92	20.68	0.67	0.41	1.71	1.50	8.03
5,000	1.96	17.22	1.32	2.27	4.44	2.92	0.81	3.40	3.14	9.09
6,000	2.90	17.37	4.73	2.21	8.66	1.08	5.32	5.26	9.75	3.29
7,000	3.35	8.73	11.91	6.44	7.56	5.89	3.38	7.56	5.36	9.58
8,000	9.61	9.23	17.16	13.55	16.28	6.30	4.35	14.13	9.26	7.46
9,000	11.06	8.00	16.33	15.09	5.33	10.43	10.48	14.00	10.88	7.30
10,000	14.11	4.71	11.56	13.97	6.79	14.81	16.01	13.58	10.64	12.03
11,000	13.19	6.17	8.10	15.21	3.31	15.91	14.77	9.65	11.86	9.13
12,000	12.84	6.95	8.07	7.58	2.25	19.54	21.10	8.61	8.33	10.91
13,000	9.36	2.66	5.61	2.13	1.74	9.14	16.66	4.62	6.10	5.99
14,000	5.04	1.24	2.94	2.89	1.01	2.75	2.49	3.17	5.95	1.69
15,000	5.57	1.36	2.71	5.46	1.27	3.07	1.01	2.47	3.68	2.48
16,000	2.48	0.78	2.87	1.22	0.68	1.80	0.78	1.24	0.79	2.63
17,000	2.64	0.76	1.79	1.59	1.02	0.94	0.41	1.49	1.39	2.35
18,000	1.19	0.32	0.95	1.47	0.61	0.49	0.22	1.63	2.09	1.93
19,000	1.06	0.33	0.50	0.62	0.61	0.41	0.32	0.52	1.16	2.21
20,000	0.36	0.20	0.32	0.41	0.51	0.18	0.12	0.23	0.96	0.93
21,000	0.68	0.20	0.27	0.05	0.45	0.14	0.03	0.10	2.90	0.17
22,000	0.50	0.13	0.14	0.40	0.21	0.06	0.05	0.00	1.11	0.13
23,000	0.43	0.07	0.20	0.16	0.14	0.05	0.02	0.07	0.17	0.01
24,000	0.16	0.02	0.00	0.07	0.09	0.03	0.03	0.08	0.14	0.00
25,000	0.05	0.02	0.00	0.07	0.04	0.04	0.12	0.01	0.01	0.00
26,000	0.06	0.02	0.08	0.02	0.02	0.03	0.01	0.00	0.15	0.00
27,000	0.01	0.03	0.04	0.00	0.04	0.02	0.03	0.00	0.00	0.00
28,000	0.01	0.00	0.05	0.00	0.01	0.00	0.00	0.00	0.00	0.00
29,000	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30,000	0.07	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
31,000	0.06	0.00	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.00
32,000	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
33,000	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00
34,000	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
35,000	0.06	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
36,000	0.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
37,000	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
38,000	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
39,000	0.12	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00
40,000	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00
41,000	0.00	0.01	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00

Table 28. Tandem-Axle Load Distribution for Lightly Loaded TWRG in Idaho

Axle Load, (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
6,000	0.20	0.00	3.08	6.37	27.90	0.21	2.02	21.16	5.23	2.94
8,000	0.53	0.00	5.41	2.65	6.60	2.48	2.11	20.59	0.58	5.66
10,000	0.67	0.00	10.09	5.21	11.66	6.50	3.73	17.51	6.63	6.20
12,000	1.12	0.00	10.50	7.38	30.71	10.25	4.00	18.74	13.19	3.03
14,000	2.42	0.00	8.57	4.70	5.77	7.36	3.76	7.07	10.68	1.42
16,000	4.37	0.00	7.46	6.69	4.55	6.18	4.32	2.83	2.28	4.99
18,000	7.69	0.00	7.42	4.84	2.96	6.54	6.22	1.98	6.66	7.62
20,000	12.92	0.00	7.30	4.67	2.50	4.18	6.36	1.56	9.34	8.55
22,000	15.18	0.00	5.02	5.89	0.60	4.41	12.63	4.30	15.59	4.18
24,000	14.41	0.00	5.18	5.98	1.16	6.95	12.75	1.82	14.66	4.76
26,000	10.13	0.00	8.16	6.03	0.97	8.00	6.87	1.55	6.68	10.91
28,000	11.15	0.00	6.62	3.37	0.53	9.07	13.50	0.89	3.45	8.82
30,000	7.88	0.00	5.25	7.17	0.57	6.74	12.43	0.00	0.90	7.87
32,000	4.42	0.00	4.18	3.50	0.44	4.60	2.96	0.00	2.81	4.21
34,000	2.14	0.00	2.13	5.84	0.04	4.67	2.06	0.00	0.86	2.83
36,000	2.06	0.00	1.49	5.50	0.00	2.92	1.54	0.00	0.00	2.71
38,000	1.42	0.00	1.26	2.76	0.18	2.20	0.99	0.00	0.00	1.33
40,000	0.38	0.00	0.16	1.89	0.24	1.57	0.38	0.00	0.00	1.72
42,000	0.04	0.00	0.23	4.15	0.42	1.98	0.23	0.00	0.00	2.28
44,000	0.24	0.00	0.33	2.06	0.19	1.52	0.43	0.00	0.00	1.83
46,000	0.11	0.00	0.05	2.20	0.09	1.25	0.27	0.00	0.00	0.75
48,000	0.00	0.00	0.00	0.56	0.30	0.23	0.10	0.00	0.00	0.20
50,000	0.06	0.00	0.03	0.12	0.25	0.05	0.02	0.00	0.21	0.48
52,000	0.12	0.00	0.08	0.03	0.09	0.03	0.14	0.00	0.25	2.08
54,000	0.08	0.00	0.00	0.15	0.11	0.01	0.03	0.00	0.00	1.96
56,000	0.00	0.00	0.00	0.20	0.22	0.02	0.01	0.00	0.00	0.44
58,000	0.00	0.00	0.00	0.09	0.15	0.01	0.03	0.00	0.00	0.23
60,000	0.00	0.00	0.00	0.00	0.05	0.01	0.00	0.00	0.00	0.00
62,000	0.00	0.00	0.00	0.00	0.01	0.02	0.01	0.00	0.00	0.00
64,000	0.00	0.00	0.00	0.00	0.18	0.01	0.00	0.00	0.00	0.00
66,000	0.00	0.00	0.00	0.00	0.32	0.01	0.02	0.00	0.00	0.00
68,000	0.00	0.00	0.00	0.00	0.18	0.01	0.03	0.00	0.00	0.00
70,000	0.09	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
72,000	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00	0.00	0.00
74,000	0.10	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00
76,000	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
78,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
80,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
82,000	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00

Table 29. Tridem-Axle Load Distribution for Lightly Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	26.92	0.70	42.86	10.61	0.00	21.05	13.43
15,000	0.00	0.00	9.70	2.38	15.43	15.16	8.87	0.00	0.58	2.37
18,000	0.00	0.00	38.86	3.03	32.08	21.02	8.54	35.32	0.03	0.25
21,000	0.00	0.00	33.08	6.44	23.79	11.78	5.23	55.06	0.00	0.50
24,000	0.00	0.00	17.85	6.06	24.00	1.95	10.38	9.04	0.00	1.40
27,000	0.00	0.00	0.51	5.63	3.24	3.90	4.30	0.52	0.23	2.61
30,000	0.00	0.00	0.00	6.73	0.76	0.79	18.20	0.06	3.83	9.27
33,000	0.00	0.00	0.00	6.86	0.00	0.08	13.97	0.00	9.03	24.88
36,000	0.00	0.00	0.00	11.66	0.00	0.00	8.14	0.00	9.78	21.47
39,000	0.00	0.00	0.00	11.84	0.00	0.06	3.43	0.00	11.69	5.27
42,000	0.00	0.00	0.00	3.60	0.00	0.48	2.45	0.00	10.47	5.56
45,000	0.00	0.00	0.00	1.89	0.00	1.92	1.87	0.00	4.99	7.57
48,000	0.00	0.00	0.00	1.87	0.00	0.00	1.37	0.00	2.50	1.83
51,000	0.00	0.00	0.00	1.86	0.00	0.00	1.02	0.00	2.65	0.41
54,000	0.00	0.00	0.00	1.24	0.00	0.00	1.08	0.00	3.70	0.18
57,000	0.00	0.00	0.00	0.77	0.00	0.00	0.29	0.00	4.83	0.14
60,000	0.00	0.00	0.00	0.87	0.00	0.00	0.11	0.00	3.69	0.22
63,000	0.00	0.00	0.00	0.13	0.00	0.00	0.06	0.00	5.43	0.92
66,000	0.00	0.00	0.00	0.17	0.00	0.00	0.08	0.00	4.38	1.30
69,000	0.00	0.00	0.00	0.05	0.00	0.00	0.00	0.00	0.28	0.19
72,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.37	0.01
75,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.35	0.04
78,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.14	0.10
81,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08
84,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
87,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
90,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
93,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
96,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
99,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
102,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 30. Quad-Axle Load Distribution for Lightly Loaded TWRG in Idaho

Axle Load (lb)	Vehicle Class									
	4	5	6	7	8	9	10	11	12	13
12,000	0.00	0.00	0.00	0.00	0.00	18.43	1.77	0.00	0.00	0.01
15,000	0.00	0.00	0.00	0.00	0.00	4.84	4.01	0.00	0.00	0.99
18,000	0.00	0.00	0.00	0.16	0.00	20.58	3.88	0.00	0.00	6.60
21,000	0.00	0.00	0.00	5.41	0.00	15.76	0.44	0.00	0.00	0.63
24,000	0.00	0.00	0.00	14.03	0.00	7.56	1.04	0.00	0.00	0.09
27,000	0.00	0.00	0.00	17.16	0.00	9.26	0.42	0.00	0.00	0.00
30,000	0.00	0.00	0.00	18.85	0.00	3.42	0.72	0.00	0.00	0.00
33,000	0.00	0.00	0.00	4.70	0.00	0.73	2.91	0.00	0.00	0.25
36,000	0.00	0.00	0.00	0.51	0.00	0.26	10.88	0.00	0.00	2.49
39,000	0.00	0.00	0.00	0.02	0.00	0.00	21.85	0.00	0.00	5.05
42,000	0.00	0.00	0.00	0.00	0.00	0.00	10.33	0.00	0.00	6.20
45,000	0.00	0.00	0.00	0.00	0.00	0.00	13.87	0.00	0.00	7.12
48,000	0.00	0.00	0.00	0.00	0.00	0.00	9.33	0.00	0.00	2.29
51,000	0.00	0.00	0.00	0.00	0.00	0.00	9.66	0.00	0.00	0.67
54,000	0.00	0.00	0.00	0.00	0.00	0.00	2.49	0.00	0.00	0.79
57,000	0.00	0.00	0.00	0.00	0.00	0.40	2.30	0.00	0.00	0.12
60,000	0.00	0.00	0.00	0.03	0.00	2.86	1.18	0.00	0.00	0.02
63,000	0.00	0.00	0.00	3.15	0.00	5.39	0.60	0.00	0.00	0.37
66,000	0.00	0.00	0.00	3.85	0.00	4.50	1.02	0.00	0.00	4.21
69,000	0.00	0.00	0.00	5.96	0.00	4.05	0.40	0.00	0.00	22.72
72,000	0.00	0.00	0.00	1.47	0.00	0.30	0.00	0.00	0.00	18.94
75,000	0.00	0.00	0.00	2.13	0.00	0.25	0.38	0.00	0.00	7.46
78,000	0.00	0.00	0.00	2.99	0.00	0.71	0.03	0.00	0.00	3.62
81,000	0.00	0.00	0.00	12.40	0.00	0.70	0.00	0.00	0.00	0.37
84,000	0.00	0.00	0.00	0.54	0.00	0.00	0.00	0.00	0.00	0.47
87,000	0.00	0.00	0.00	1.68	0.00	0.00	0.00	0.00	0.00	0.92
90,000	0.00	0.00	0.00	4.08	0.00	0.00	0.00	0.00	0.00	1.31
93,000	0.00	0.00	0.00	0.82	0.00	0.00	0.22	0.00	0.00	3.31
96,000	0.00	0.00	0.00	0.06	0.00	0.00	0.01	0.00	0.00	2.02
99,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.27
102,000	0.00	0.00	0.00	0.00	0.00	0.00	0.26	0.00	0.00	0.69

Table 31. WIM Sites Associated with Idaho TWRG

Idaho Truck Weight Road Groups (TWRG)	WIM Stations
Primarily Loaded	79, 117, 134, 148, 155
Moderately Loaded	93, 137, 138, 156, 169, 185
Lightly Loaded	96, 129, 192

5.8 Number of Axles per Truck Type/Class

The numbers of single, tandem, tridem, and quad axles per truck are basically adjustment factors used to estimate the total number of single, tandem, tridem, and quad axles for a given distribution of truck traffic. Each truck class type has a unique range of axle types. Trucks of specific classes have approximately the same number of axles regardless of which highway they are traveling.

The Level 1 inputs are the actual measured site data and are recommended only for design of atypical highway routes with heavy seasonal mining, recreational, or agricultural traffic. Site-specific axles/truck may be obtained from the ITD Roadway Data Section.

For all other typical routes and designs, it is recommended that designers use Level 2/3 Idaho-specific average values estimated using historical WIM data. The historical data were selected to represent all the different types of highway functional classes of interest and truck classes. Table 32 presents default estimates for the number of single, tandem, tridem, and quad axles per truck.

Table 32. Recommended Number of Single-, Tandem-, Tridem-, and Quad-Axles per Truck Class for Idaho

Vehicle Class	Axle Type			
	Single	**Tandem**	**Tridem**	**Quad**
4	1.59	0.34	0.00	0.00
5	2.00	0.00	0.00	0.00
6	1.00	1.00	0.00	0.00
7	1.00	0.22	0.83	0.10
8	2.52	0.60	0.00	0.00
9	1.25	1.87	0.00	0.00
10	1.03	0.85	0.95	0.26
11	4.21	0.29	0.01	0.00
12	3.24	1.16	0.07	0.01
13	3.32	1.79	0.14	0.02

5.9 General Traffic Inputs

AASHTOWare Pavement ME Design requires designers to characterize typical truck features and interaction with highway pavement for use in pavement loading simulations and response analysis. Information required for this characterization is listed below:

- **Mean Wheel Location** (in.) see Figure 24a.
  - 18 in. from edge of lane stripe to outside of dual tires.
  - Reduce to 12 in. if traffic lane width is less than 12 feet.

- **Traffic Wander Standard Deviation** (in.).
  - 10 in. lateral wander standard deviation.

- **Axle Configuration** (see Figure 24b and Table 33).

- **Truck Wheelbase** (see Figure 24c and Table 34).

- **Design Lane Width** (feet) see Figure 24d.
  - 12 ft (this value is not slab width; it is measured between lane longitudinal paint stripes).

Table 33. Axle Configuration for Idaho (See Figure 24b)

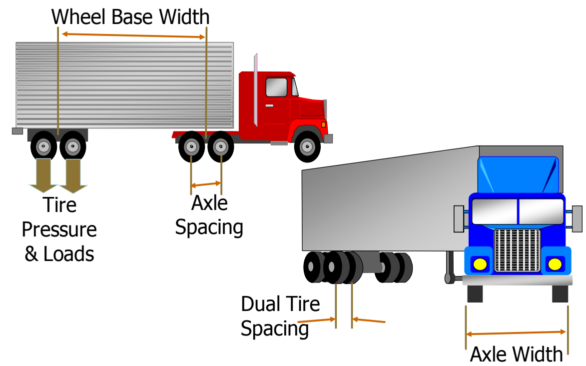
Truck Features	Mean Values
Average Axle Width	8.5 ft (outside to outside of truck tires)
Dual Tire Spacing	12.0 in.
Dual Tire Pressure	120 psi
Tandem Axle Spacing	51.6 in.
Tridem Axle Spacing	49.2 in.
Quad Axle Spacing	49.2 in.

Table 34. Wheelbase (Based on National Measurements). See Figure 24c. [Critical]

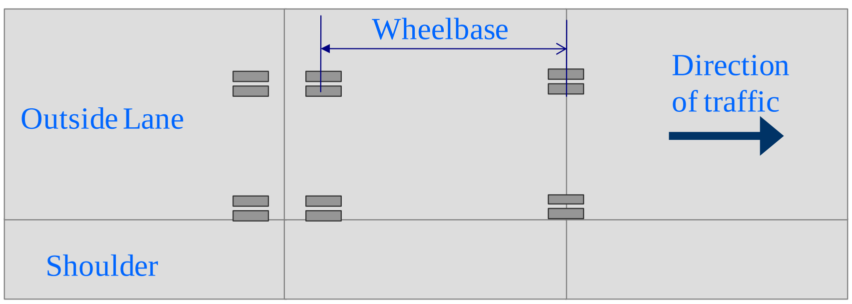
Wheelbase	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Trucks* (percent)	17	22	61

*** Classes 8 to 13.**

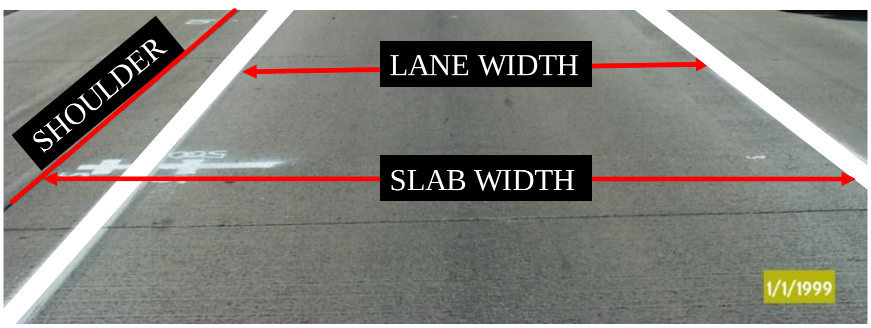
a. **Wheel Location**



b. **Axle Configuration and Wheelbase**



c. **Axle Configuration and Wheelbase**



d. **Lane Width**

Figure 24. Schematic Illustration of Mean Wheel Location

Chapter 6
Climate Inputs

The State of Idaho includes a wide range of climate with annual mean temperatures ranging from 38°F to 54°F and annual precipitation of 7 to 38 inches. Examples of the wide range of temperatures (shown as freezing index, which is closely related to frost depth) and moisture (shown as annual rainfall) throughout the State is shown in Figure 25. Obtaining proper climate data for a given pavement design site is critical to obtaining a reliable design.

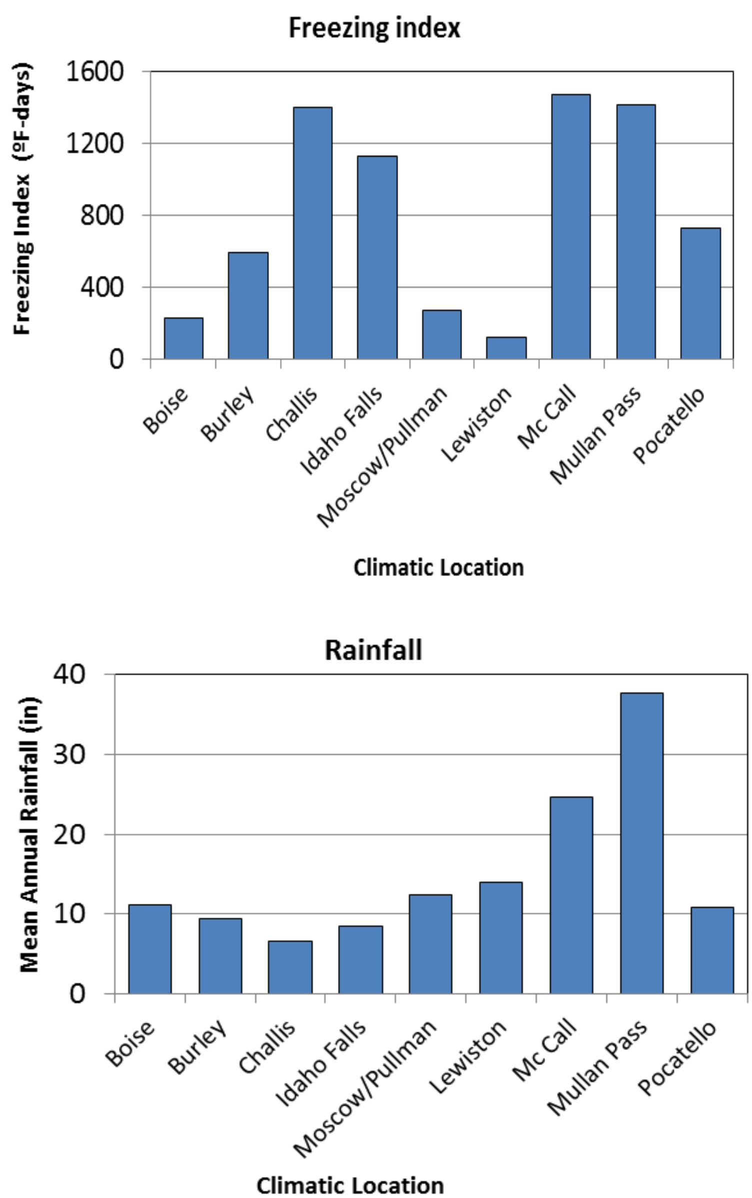


Figure 25. Examples of Temperature and Moisture Variations Across the State of Idaho

AASHTOWare Pavement ME Design requires hourly temperature, precipitation, wind speed, relative humidity, and percent sunshine data. The annual or seasonal depth to groundwater table at the project site is also required. All climate inputs for Idaho can be obtained by following the steps:

1. **Define Project Location**. Site-specific longitude, latitude, and elevation are required. This information can be obtained from various sources (e.g., Google Earth or www.lat-long.com) given the route ID and project milepoint. There are only eight complete weather stations in the software. Four others that were in the MEPDG have some missing data and until corrected, cannot be used.

2. Select from the *AASHTOWare Pavement ME Design* climate database one or more weather stations as close to the project as possible. When the designer enters the project site longitude, latitude, and elevation, *AASHTOWare Pavement ME Design* will identify the closest weather stations. If the closest weather station is 50 miles away or more, the use of more than 1 weather station is recommended, so that a better estimate of the climate at the project site can be obtained. If two or more weather stations are selected, proceed with creating a project-specific "virtual" weather station by weighted interpolation of the data available in the selected weather stations. The software creates the virtual weather station automatically, after the user selects the desired weather station(s).

The Idaho and surrounding weather stations presented in Table 35 contain 6 to 10 years of data, and these data are currently available in *AASHTOWare Pavement ME Design*. Figure 26 shows the location of weather stations in Idaho and surrounding States. The weather stations from surrounding States can be used for projects located near State lines.

Groundwater table is another climate input that must be entered either on a quarterly or annual basis. *AASHTOWare Pavement ME Design* does not contain depth-to-groundwater table data, so the designer must obtain this information from any of the sources listed below:

- **ITD's Research Report RP193** – *Implementation of the MEPDG for Flexible Pavements in Idaho*. Excel spreadsheet: ITD-MEPDG-Final Database

- **The U.S. Geological Survey (USGS) National Water Information System (NWIS)** database: (http://nwis.waterdata.usgs.gov/id/nwis/gwlevels?introduction) Database includes interactive map of well groundwater.

- **Project Geotechnical Reports**.

- **Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database**. http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/survey/geo/?cid=nrcs142p2_053627

- **The Idaho Department of Water Resources (IDWR)**: http://www.idwr.idaho.gov/hydro.online/gwl/default.html

Depth to groundwater table typically ranges widely from 5 to 200 ft or more in Idaho.

Table 35. Weather Stations for Idaho and Surrounding States

City	Location	Latitude (^{0})	Longitude (^{0})	Elevation (ft)	Start Year	End Year	Temperature (°F)	Precipitation	Wet Days	FI °F- Days	Number F-T Cycle
Boise, ID	Boise Air Terminal/Gowen Field Airport	43.565	-116.22	2,861	1996	2006	53.0	10.6	128	603	75
Burley, ID	Burley Municipal Airport	42.5	-113.800	4,137	2000	2006	48.1	9.3	146	1,296	95
Challis, ID	Challis Airport	41.523	-114.218	5,040	1998	2006	43.8	5.2	131	4,631	114
Idaho Falls, ID	Idaho Falls Regional Airport	43.5	-112.100	4,730	1998	2006	44.6	8.6	154	2,304	110
Jerome, ID	Jerome County Airport	42.727	-114.456	4,017	1997	2006	48.8	9.6	128	1,159	96
Lewiston, ID	Lewiston - Nez Perce County Airport	46.375	-117.014	1,425	1996	2006	53.5	12.8	169	302	48
McCall, ID	McCall Municipal Airport	44.889	-116.102	5,008	1997	2006	39.5	18.1	180	3,078	140
Mullan Pass, ID	Mullan Pass	47.457	-115.645	6,014	1996	2006	37.7	39.9	219	2,013	61
Pocatello, ID	Pocatello Regional Airport	42.9	-112.600	4,440	1996	2006	47.4	10.1	160	1,700	107
Rexburg, ID	Rexburg-Madison County Airport	43.834	-111.881	4,859	1998	2006	43.8	8.8	158	2,346	99
Twin Falls, ID	Joslin Field-Magic Valley Regional Airport	42.482	-114.487	4,157	1997	2006	49.7	9.4	135	1,119	108
Big Piney, WY	Big Piney - Marbleton Airport	42.6	-110.100	6,943	1998	2006	37.1	7.7	144	4,617	146
Evanston, WY	Evan - Uinta County/Burns Field Airport	41.3	-111.000	7,140	1999	2006	42.0	9.3	161	2,463	99
Lander, WY	Hunt Field Airport	42.8	-108.700	5,592	1996	2006	45.8	11.6	119	2,291	123
Rock Springs, WY	Rock Springs-Sweetwater County Airport	41.6	-109.100	6,742	2001	2006	43.9	6.6	144	2,396	111
Ogden, UT	Hill Air Force Base	41.1	-112.000	4,447	2006	2011	52.4	11.2	91	1,016	74

Table 35 (cont.). Weather Stations for Idaho and Surrounding States

City	Location	Latitude (0)	Longitude (0)	Elevation (ft)	Start Year	End Year	Temperature (oF)	Precipitation	Wet Days	FI oF- Days	Number F-T Cycle
Ogden, UT	Ogden-Hinckley Airport	41.200	-112.000	4,447	1998	2011	51.6	5.7	48	998	75
Salt Lake City, UT	Salt Lake City International Airport	40.800	-112.000	4,220	1998	2011	53.2	4.5	42	909	75
Vernal, UT	Vernal Airport	40.400	-109.500	5,260	1998	2011	47.1	2.3	40	2,289	113
Logan, UT	Logan-Cache Airport	41.800	-111.900	4,445	1998	2011	46.8	4.3	44	2,092	107
Elko, NV	Elko Regional Airport	40.825	-115.792	5,050	2001	2006	47.5	10.5	143	2,107	136
Bozeman, MT	Gallatin Field Airport	45.800	-111.200	4,427	1996	2006	42.8	12.9	170	2,860	140
Dillon, MT	Dillon Airport	45.300	-112.600	5,200	1997	2006	43.2	9.9	135	2,294	137
Livingston, MT	Mission Field Airport	45.700	-110.400	4,643	2000	2006	45.5	13.9	176	2,019	106
Butte, MT	Bert Mooney Airport	45.953	-112.513	5,506	2000	2006	40.3	11.1	176	3,335	149
Missoula. MT	Missoula International Airport	46.921	-114.093	3,192	1996	2006	45.4	13.8	180	1,528	111
Spokane, WA	Felts Field Airport	47.683	-117.300	1,940	1998	2006	49.3	14.8	160	715	83
Spokane, WA	Spokane International Airport	47.621	-117.500	2,353	1996	2006	48.3	14.9	168	884	86
Baker City, OR	Backer City Municipal Airport	44.838	-117.810	3,361	2001	2006	46.2	10.2	169	1,690	144
Burns, OR	Burns Municipal Airport	43.592	-118.954	4,140	1996	2006	45.5	10.0	151	2,123	156
Meacham, OR	Meacham	45.511	-118.425	3,726	1998	2006	43.0	30.4	178	1,852	157
Ontario, OR	Ontario Municipal Airport	44.021	-117.013	2,184	1997	2006	52.1	8.8	125	868	92

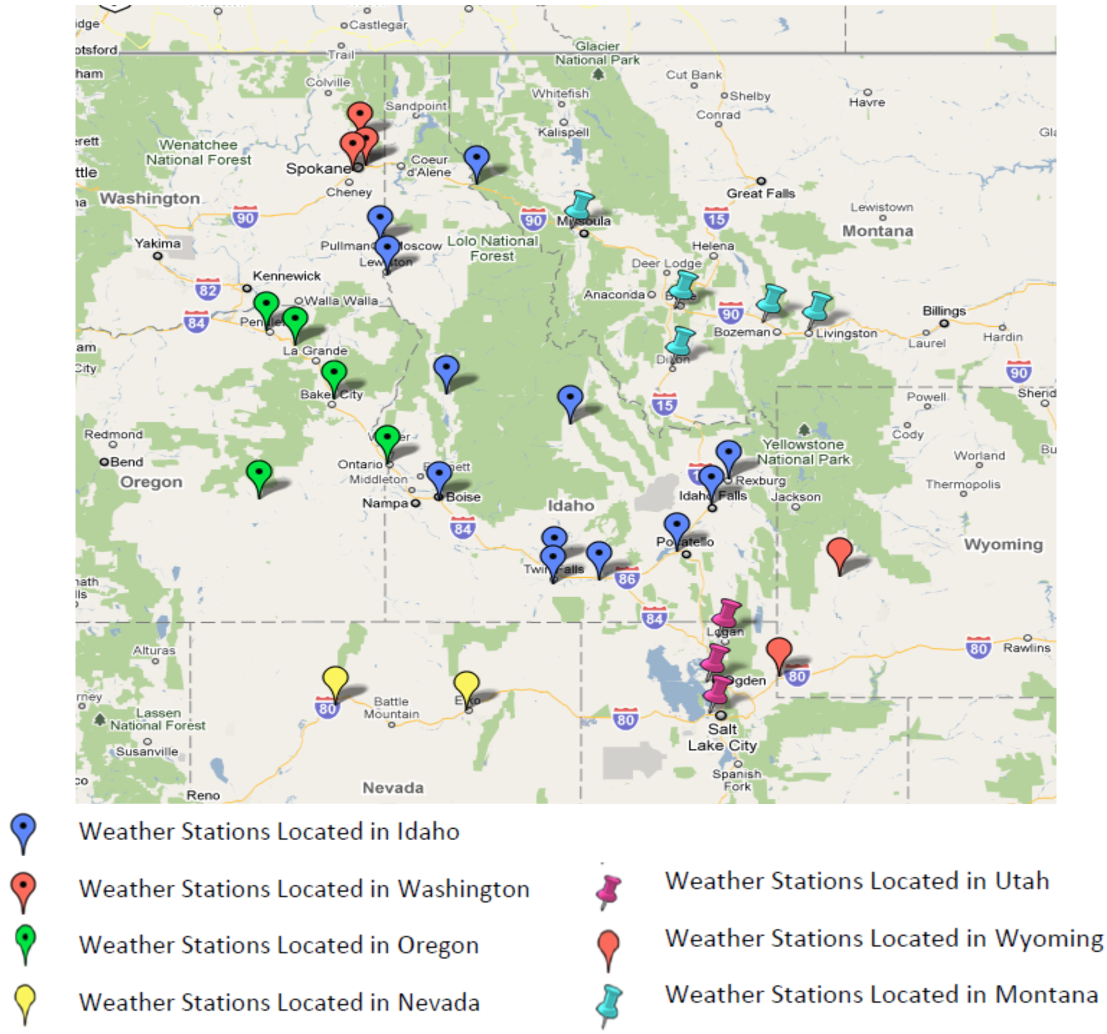


Figure 26. Idaho and Surrounding Weather Stations Available for Pavement Design

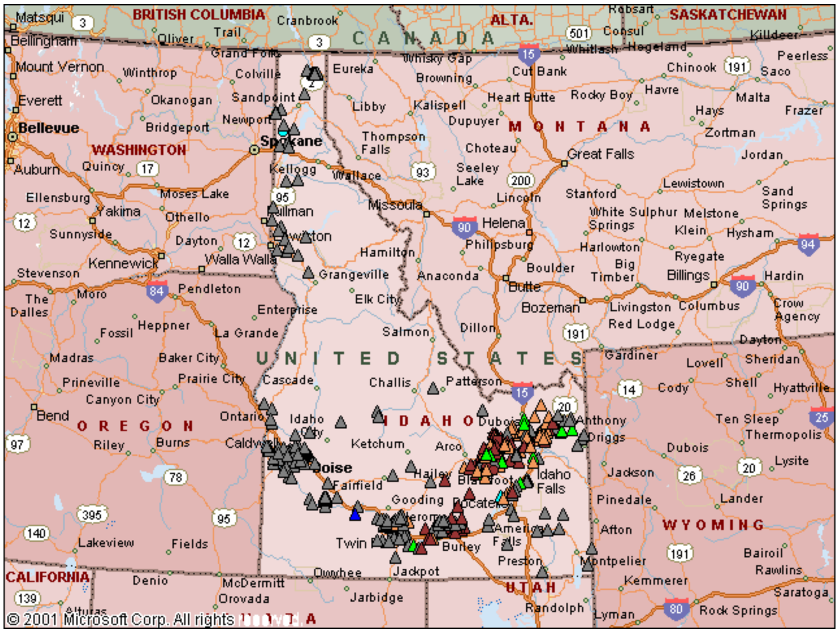


Figure 27. Well Sites Information on Groundwater in Idaho

Chapter 7
Pavement Structure Definition
and Materials Characterization

Pavement design begins with selecting a "Trial Design" that is then evaluated for accuracy using the *AASHTOWare Pavement ME Design* software to simulate effect of the combination of traffic loading, cyclic variation in climate on material properties, materials aging (including HMA, PCC), etc. Adequacy (pass or fail) is determined based on predicted distress/IRI at a preselected reliability level and threshold levels for the distress/IRI of interest.

Through a comprehensive laboratory testing program, ITD developed a database of materials information required by the *AASHTOWare Pavement ME Design* software. The database is presented in a Microsoft Excel spreadsheet entitled *ITD Database for the Mechanistic-Empirical Pavement Design Guide (MEPDG) version 1.100*, developed under ITD Research Project RP193, *Implementation of the MEPDG for Flexible Pavements in Idaho*. Detailed descriptions of the testing and analysis conducted to develop the default material inputs are presented in the report.

7.1 Introduction

This section of the design guide provides guidance to pavement designers for obtaining all materials-related information required by the *AASHTOWare Pavement ME Design* software from the default tables provided by the *ITD Database for the Mechanistic-Empirical Pavement Design Guide*, or directly at the project level from project-level field surveys and testing provided by ITD's Materials Section.

7.2 "Trial Design" Structure

The "Trial Design" structure is determined based on ITD policy regarding pavement design and the engineer's pavement design experience. It is recommended that determination of inputs (materials properties mostly) for the "Trial Design" structure begin from the subgrade/foundation up to the surface layer. Steps for determining the "Trial Design" structure materials properties/inputs are described in the following sections.

7.3 Step 1 - Bedrock Layer Soil Characterization

For some projects, depth to bedrock or very stiff layer may be within 20 feet of the top of the natural subgrade immediately below the proposed grade line. For such projects, *AASHTOWare Pavement ME Design* can include a bedrock layer under the natural subgrade. Table 36 provides guidance on inputs for a bedrock layer when it exists within the project length. Generally, if the depth is less than 20 feet, it can affect deflections at the pavement surface significantly. Otherwise, its effect is minimal and use of bedrock is not warranted.

Table 36. Guidance on Bedrock Layer Properties

Bedrock Parameters	Recommended Input
Depth to Bedrock (ft)	Estimate based on soil borings or topography. Bedrock can have an effect if ≤20 ft deep.
Resilient Modulus (M_r) Highly Fractured & Weathered Bedrock (psi)	500,000 psi
Resilient Modulus (M_r) Massive &Continuous Bedrock (psi)	1,000,000 psi
Thickness (in.)	Actual or semi-infinite if last layer.
Poisson's Ratio	0.30 highly fractured & weathered 0.15 massive & continuous .
Unit Weight (pcf)	140 pcf

7.4 Step 2: Subgrade and Embankment Soil Characterization

The material properties used to classify the subgrade in the *AASHTOWare Pavement ME Design* process are:

- Resilient Modulus (M_r) at Optimum Moisture Content (measured or estimated laboratory value).
- Maximum Dry Density (MDD).
- Specific Gravity.
- Hydraulic Conductivity.
- Optimum Moisture Content.

M_r at optimum moisture content is the most important input required for embankment /subgrade soil materials, as the M_r directly affects the computed deflection, stress, and strain under wheel loads and thus should be estimated properly. Subgrade soil M_r is a key input for all new pavement types and rehabilitation designs with the exception of some existing rigid pavement rehabilitation designs (with HMA or PCC) where the modulus of subgrade reaction (k-value) may also be required.

NOTE: *AASHTOWare Pavement ME Design* takes the input embankment/subgrade M_r and adjusts it internally from optimum moisture to an in situ subgrade moisture content for every month of the analysis period or "Design Life". This often results in a significant increase or reduction of the input M_r to a higher or lower M_r at different in situ moisture contents.

Characterize the subgrade soils and follow the guidance in the *Materials Manual, Section 230.03.01 Soils Profile* and *Section 230.07 Soils Report Summary* to develop a soils profile and soils report summary. The *Materials Manual* is available at: http://itd.idaho.gov/manuals/Manual%20Production/Materials/materials_cover.pdf

Refer to these sections and report the results on ITD-Form 0944, *Phase 2 Soil Report Summary*. http://itdhq1wsp03/Apps/FormFinder2/Home/DownloadFile?storedfilename=%5C%5Citdhq1fsp05%5CForms%5C0501-1000%5C0944.xls&downloadname=0944.xls

Using the information provided in ITD Form 0944, designers must determine the predominant soil type (described using the AASHTO soil classification scheme) along the project length, as well as the soil's mean M_r value).

NOTE: Areas with significantly weak materials may require remedial soil treatment, such as placement of a thick embankment or stabilization of the top 6 to 18 inches of the natural subgrade with lime.

A thick embankment (granular pit run filler) should be considered as a separate compacted subgrade layer overlying the natural subgrade when developing the "Trial Design." The 6 to 18 inches lime-stabilized subgrade must also be treated as a separate compacted subgrade layer. The increased strength of this layer due to the addition of lime can be considered or ignored as a temporary effect for construction purposes.

Specific guidance on Level 1, 2, and 3 inputs for embankment soils, lime-treated subgrade, and natural subgrade is presented in the following sections.

Level 1 Embankment/Subgrade Soil M_r Characterization

This is not available at this time in the software.

Level 2 Embankment/Subgrade Soil M_r Characterization

There are two Level 2 approaches to determine the design M_r for input into the program:

1. Estimate through correlation with R-value and
2. Estimate through FWD testing and back-calculation.

Approach 1: Measure or estimate through correlation the embankment/subgrade R-value and convert to an appropriate design M_r.

This approach should only be used for new alignment HMA and JPCP construction projects or where no FWD testing is available or possible. FWD testing provides much better coverage and estimation of actual subgrade support along a project.

With the R-value approach, estimates of subgrade soil R-value are converted into M_r at the moisture content of the test specimen using the following relationship:

$$M_r = 1004.4\,(R)^{0.6412}$$

where:

M_r = Subgrade Resilient Modulus (psi) (at moisture content of the test specimen).
R = Mean R-value of the subgrade soil using Idaho T-8 procedure.
NOTE: If fine-grained soil, the R-Value obtained from the ITD Test T-8.
Test is multiplied by 1.1. See explanation below.

Equation 1

Subgrade soil R-value is determined using ITD's T-8 procedures which uses a lower exudation pressure (200 vs. 300) and lower compactive effort which results in a higher moisture content. Generally this means that the Idaho R-Value of coarse grained soils is about the same but fine grained soils are 10 percent lower. Thus, it is recommended that the Idaho R-Value test result be multiplied by 1.1 so that it can be used properly in Equation 1.

The R-value from the Idaho T-8 can be obtained directly from lab testing of samples of the embankment/ subgrade soil along the project and the *mean R-value* determined for the project.

NOTE: If there are two or more distinct soil types, the length of project associated with each could be analyzed separately to determine if there is a significant design difference.

If R-value testing cannot be done, the following estimate can be made by measuring the subgrade soil plasticity index and percent passing the No. 200 sieve, and entering the values into the equation below:

$$R = 10^{(1.893 - 0.00159 * P200 - 0.022 * PI)}$$

where:

R = R-value (Idaho T-8 procedure equivalent)
P200 = Percent Passing No. 200 Sieve
PI = Plasticity Index

Equation 2

Finally, the computed subgrade **mean M_r value** is entered into *AASHTOWare Pavement ME Design* directly.

NOTE: The lab-tested for R-value, the moisture content of the R-value test specimen is entered into the **"Optimum Gravimetric Water Content"** input.

Approach 2: Conduct FWD testing and back-calculation to determine an embankment/subgrade design input M_r.

This approach should be used for existing flexible and rigid roadways that can be tested with the FWD. This approach is highly recommended to obtain the design input M_r of existing embankments and subgrades because it reaches deep into the embankment or subgrade layers and, due to the ease in testing, provides the best estimate of existing support along the entire project.

FWD Deflection Testing

The ITD deflection testing procedures outlined in this section are based, in part, on methodologies used by Washington State DOT, Texas DOT, California DOT, and the Strategic Highway Research Program 2 (SHRP2).

FWD testing is performed by ITD's TAMS Section. They will provide deflection testing as described in Section 530.01.01 of *ITD's Materials Manual*.

NOTE: Typically the District Materials Engineer must submit requests for deflection testing to TAMS Section prior to the beginning of the field testing season in April.

The ITD standard deflection testing program is summarized as follows:

- FWD tests will be made in at least 1 direction on two-lane roadways and in the travel lanes (both directions) on four-lane roadways.

- The testing interval is one test every 0.1 mile. Intermediate tests should be made in localized areas of significantly different distress. If the Materials Engineer needs something other than the standard testing these special instructions must be provided.

- Test locations will be in the outer wheel path on flexible pavements unless otherwise directed. On rigid pavement, test locations will be in the center of the slab, except for load transfer across joints.

- Where rutting is too deep to achieve uniform contact with the loading plate, the test point will be relocated so that the plate makes adequate contact.

- Tests will be made with at least 1 force level of 12,000 lb. Once every 10 tests, at least 2 force levels will be used; one at 9,000 lb and the other at 12,000 lb.

- The FWD test report must indicate the degree of distress at the point of test. Degree of distress is determined using the *SHRP Distress Identification Guide*. In addition, the report must indicate whether the test point is in cut or fill.

Back-Calculation of M_r for Existing HMA (Flexible) Pavement

When the existing pavement is HMA, the back-calculation of the subgrade M_r is required and is obtained as follows:

- Conduct FWD testing along the HMA project in the outer wheel path at regular intervals (at least every 0.1 mile apart). The M_r can be calculated for any "heavy" FWD load level, but it is recommended to use the 9,000 lb target to ensure a modulus that matches typical heavy wheel loadings.

- Back-calculate subgrade field elastic modulus ($E_{s)}$, at each FWD deflection point. The E_s is an elastic modulus measured in the field, averaged over the subgrade depth (effects more than 10 ft deep) and radius with in situ moisture and density.

- Plot the E_s along the project and examine the plot for possible division into design segments with average higher or lower series of E_s values. Divide into design segments if desired. ITD uses a cumulative difference approach described in Appendix J of *AASHTO Guide for Design of Pavement Structures, 1993.* An Excel spreadsheet that takes the FWD data and performs the analysis on the deflections is available.

- Then within each segment, check for outliers which are significantly higher or lower than a large majority of sections. Remove these from the data and determine the mean E_s along the project.

NOTE: The computer program MODULUS 6, developed by the Texas DOT and Texas Transportation Institute, is the primary deflection analysis tool used by ITD to back-calculate pavement layer stiffnesses (moduli).

Section 530.08.01 of *ITD's Materials Manual* provides detailed description of analysis using MODULUS. Finally, in back-calculation programs, it is extremely important that layer thicknesses be as accurate as possible. In the calculated moduli, variations of as little as 10 percent in asphalt pavement thickness can make a significant difference.

The mean E_s must be adjusted from a "field" elastic half space to a "lab" test value by multiplying by 0.35 using the field test M_r along the project determine the "mean M_r" for the project.

NOTE: M_r is at field in situ moisture content (not optimum). The in situ moisture content must be determined either through borings into the subgrade or by estimation. This value is commonly 3 to 5 percent above optimum moisture content.

The measured or assumed in situ moisture content is then entered into the program (in the "Optimum Gravimetric Water Content" input location) along with the back-calculated "Field" elastic modulus * 0.35.

For example, FWD testing is performed along a project and the deflection data is used to back-calculate E_s values along the project. After deleting a few obvious outliers, the mean E_s = 20,000 psi. The mean M_r (at in situ moisture) = 0.35*20,000 = 7,000 psi. The measured in situ moisture content of the subgrade from a couple of borings along the project is 19 percent. Therefore, the Pavement ME values for input are as follows:

- **Optimum Gravimetric Water Content = 19.0 percent**
  NOTE: This is NOT actually optimum water content, it is the measured (or estimated) in situ water content but by entering this value into the program, the proper water content is used to calculate the M_r over all months.

- **Mean M_r = 7,000 psi.**

Back-Calculation of M_r and k-Value for Existing JPCP or Composite Pavement (HMA/JPCP)

When the existing pavement is JPCP or composite pavement (HMA/JPCP), the FWD tests the pavement and the deflections are used to back-calculate the dynamic modulus of subgrade reaction (i.e., subgrade dynamic k-value), rather than the E_s modulus.

The effective dynamic k-value can be determined from back-calculation (using MODULUS) or alternately from the area of each deflection basin. The area may be calculated from the deflections at 12, 24, and 36 inches from the center of the FWD loading plate. The AREA method for computing dynamic k-value is described in the *AASHTO Guide for Design of Pavement Structures, 1993* and Section 530 of the *ITD's Materials Manual*.

NOTE: Static k-value is the dynamic k-value/2.

The mean subgrade dynamic k-value along the project is computed (after deleting obvious outliers along the project) and entered into the software along with the month of FWD testing.

The dynamic k-value from FWD back-calculation represents the stiffness of the unbound compressible soils (at least 10 feet or more deep into the subgrade) beneath the JPCP slab. The dynamic k-value typically is twice as high as the conventional static k-value obtained from slow plate loading (which is the traditional input used in the *AASHTO 1993 Pavement Design Guide*). Dynamic k-value is obtained as follows:

- Conduct FWD deflection testing along the project in the center of the slab at regular intervals (0.1 mile apart at the mid-point between nearby transverse joints).

- Back-calculate the subgrade dynamic k-value from FWD deflections at the slab surface using the following equation. The dynamic k-value can be calculated for any sensor location (e.g., 0, 12, 36 inches); however, the sensor at the center of the load plate is recommended. The k-value can be calculated for any "heavy" FWD load level, but it is recommended to use a load at or greater than 9,000 lb to ensure a modulus that matches typical heavy wheel loadings.

$$k = value = \frac{PF}{\ell^2 \Delta_r}$$

where:

P = FWD Load (lb) [example 9,000 lb]
Δ_r = Deflection at r Distance from Center of Load Plate (in.)
[use the deflection at center of the load, i.e., r = 0 inches]

Equation 3

$$F = 0.1245 e^{\left(-0.14707 e^{[-0.07565 \ell]}\right)}$$

ℓ = Radius of Relative Stiffness (inches)

Equation 4

$$\ell = \ln\left(\frac{72 - AREA}{242.385} - 0.442\right)^{2.205}$$

Equation 5

$$AREA = 6 + \left(\frac{12 * \Delta_{12} + 12 * \Delta_{24} + 12 * \Delta_{36} + 12 * \Delta_{48}}{\Delta_0}\right)$$

Equation 6

- Plot the dynamic k-value along the project and examine the plot for possible division into design segments with average higher or lower series of dynamic k-values. Divide into design segments if desired, however, realize that it takes a large difference to make a difference in design thickness. Then within each segment, check for outliers which are significantly higher or lower than a large majority of sections. Remove these from the data and determine the mean dynamic k-value along the project.

The dynamic k-value is a dynamic (impact load) modulus measured in the field, averaged over the subgrade depth and width with in situ moisture and density. Very soft soil has dynamic k-values of 200 psi/in. or less while very stiff soil has k-values of more than double this value. It is used to calculate the stresses and deflections in the slab, which are used to predict fatigue cracking and joint faulting.

Estimating Mean M_r or k-Value for *AASHTOWare Pavement ME Design*

The procedures above describe how to test or back-calculate M_r or dynamic k-value along a given project length for Level 2 inputs. Guidance on determining a representative project input M_r or k-value for use in *AASHTOWare Pavement ME Design* is presented below:

1. **New or Reconstructed PCC Design.** *AASHTOWare Pavement ME Design* requires a subgrade M_r input, and a k-value cannot be entered directly. Obtain the proper subgrade M_r input through an iteration process. Enter a trial M_r (assume the value that corresponds to the mean back-calculated value) and run the program. Examine the output dynamic k-value to see if it is within 10 percent of the field back-calculated value for the month of FWD testing selected. If not, iterate with a new M_r value until agreement is reached.

2. **HMA and PCC Overlay Design Over an Existing PCC Pavement. T**he mean dynamic k-value can be input directly into *AASHTOWare Pavement ME Design* along with the month of FWD testing. This k-value then provides the needed subgrade support modulus for design purposes. It is not required to enter a subgrade M_r.

AASHTOWare Pavement ME Design will internally adjust both M_r and k-value for the effect of moisture and freeze/thaw for each month in the year. Thus, only M_r at optimum, M_r at in situ moisture (with the in situ moisture content also provided), and k-value at in situ moisture (along with the month of testing for moisture and freeze/thaw adjustments) are required.

Level 2 Other Embankment/Subgrade Inputs

Other inputs required for Level 2 embankment/subgrade characterization are listed below:

- **MDD:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Optimum Moisture Content:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Specific Gravity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Saturated Hydraulic Conductivity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Soil Water Characteristic Curve Parameters:** Select based on aggregate/subgrade material AASHTO soil class.

Level 3 Embankment/Subgrade Soil Characterization

The recommended input M_r depends on the soil class, as provided in Table 37 from the Unified Soil Classification System (USCS). The same recommendations are provided for flexible and rigid/composite pavements for embankment/subgrade according to current ITD practice. There is a wide variation in recommended M_r from project to project within a given soil class (the coefficient of variation between projects with the same AASHTO soil class is typically 50 percent), so this Level 3 approach can result in a very poor estimate of the M_r for a specific project design and should only be used if FWD testing data for the project cannot be obtained.

NOTE: *AASHTOWare Pavement ME Design* software only allows AASHTO soil classification as input. Designers must convert USCS soil class to equivalent AASHTO soil class in order to use M_r from Table 37.

NOTE: Both Unified and AASHTO classifications are now provided in the soils report for ease and accuracy of use. However, if this information is not available, an approximate AASHTO soil class can be determined from the USCS soil class as presented in Figure 28.

Level 3 Other Embankment/Subgrade Inputs

The following additional inputs are required for Level 3 embankment/subgrade soils characterization for *AASHTOWare Pavement ME Design*:

- **MDD:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Optimum Moisture Content:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Specific Gravity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

- **Saturated Hydraulic Conductivity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.

Soil Water Characteristic Curve Parameters: Select based on aggregate/subgrade material AASHTO soil class.

Summary of Level 3 Embankment/Subgrade Inputs for AASHTOWare Pavement ME Design

Table 38 presents a summary of inputs for embankment/subgrade soils in *AASHTOWare Pavement ME Design* for Level 3 design input.

Table 37. Recommended Idaho (Level 3) Lab Resilient Modulus for Embankment /Subgrade at Optimum Moisture for Flexible and Rigid Pavements

Soil Type (Unified Soil Classification System)	ITD Recommended R-Value	Estimated M_r (psi)**	ITD Recommended M_r Range (psi)	
			Lower Bound M_r	Upper Bound M_r
OH	32	9,268	5,702	12,180
OL	44	11,368	8,893	13,571
CH	15	5,702	2,032	8,113
MH	28	8,508	4,942	11,533
CL	27	8,312	4,942	10,865
CL - ML	45	11,533	9,082	13,869
ML	60	13,869	11,859	15,728
SC	35	9,817	6,178	12,963
GC	38	10,348	6,857	13,269
SC - SM	53	12,809	9,817	15,310
GC - GM	60	13,869	11,697	15,728
SM	66	14,743	12,653	16,679
GM	72	15,589	13,721	17,209
SP - SC*	15	5,702	1,004	9,268
SW - SC	71	15,450	14,164	16,679
SP - SM	74	15,866	14,455	17,209
SW - SM	77	16,275	15,589	16,945
GP - GC	65	14,600	12,180	16,945
GW - GC	68	15,028	12,809	17,077
GP - GM	78	16,410	15,170	17,471
GW - GM	79	16,545	15,728	17,340
SP	74	15,866	15,450	16,410
SW	75	16,003	15,170	16,679
GP	77	16,275	15,310	17,209
GW	79	16,545	15,589	17,601

* NOTE: These values are based only on limited number of data points.
** M_r obtained from correlation with R-value (M_r = 1004.4(R-Value)$^{0.6412}$.
The R-value is obtained from lab testing with Idaho T-8 procedure.

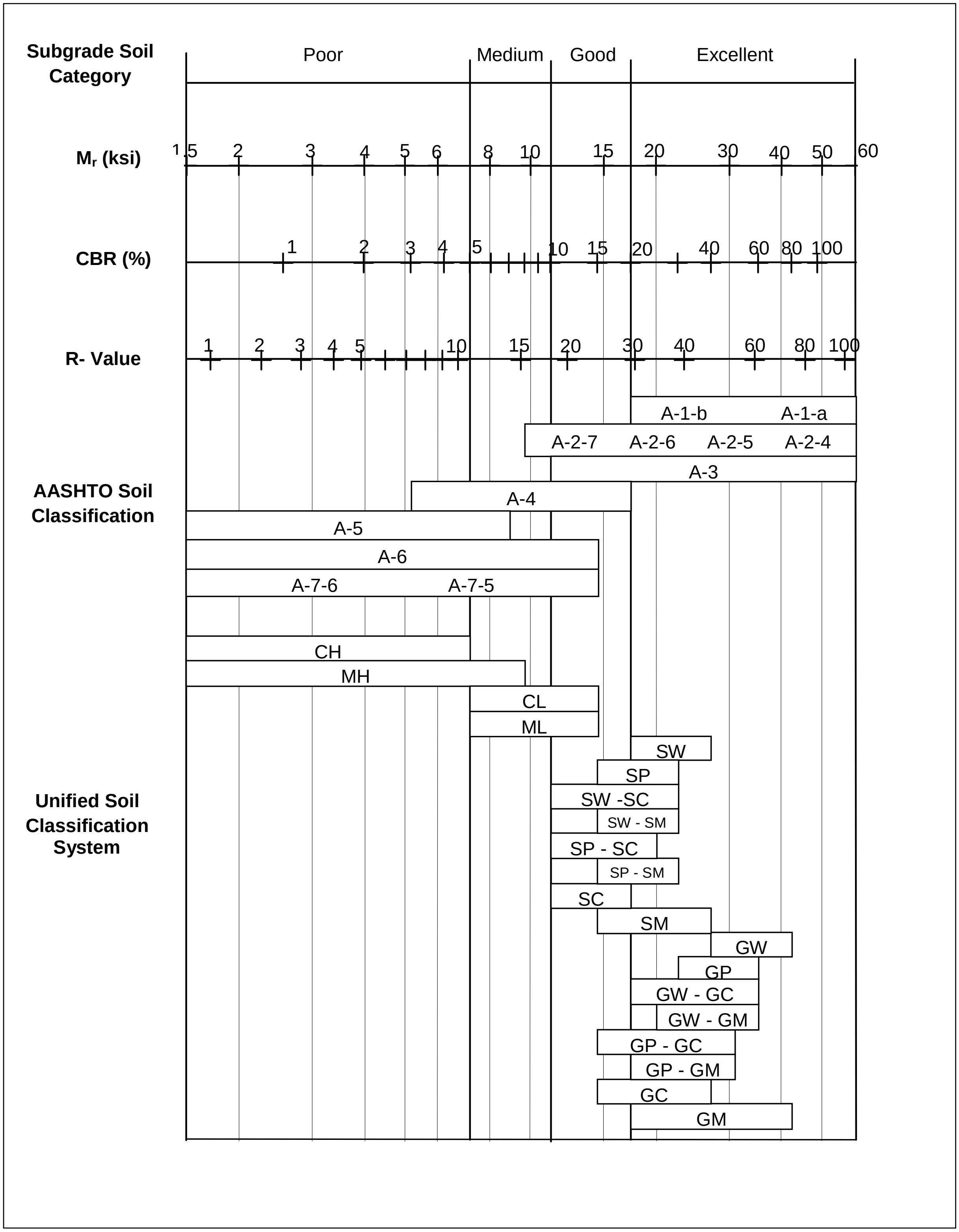


Figure 28. Typical Correlations Between USCS and AASHTO Soil Classification

Table 38. Recommended Level 3 Inputs for Unbound Soils and Embankment Layers

Embankment/Subgrade Material	Inputs
Thickness (in.)	Embankment: Actual Compacted Lime Stabilized Subgrade: Actual Natural Subgrade: Semi-infinite if last layer
Strength Properties Input Level	Level 2 or 3 (see descriptions in previous sections)
Poisson's Ratio	0.4
Coefficient of Lateral Pressure	0.5
Compacted Unbound Material or Uncompacted Natural Unbound Material	Click on "Uncompacted" option for natural subgrade only. All others embankment or stabilized must be compacted.
M_r at Optimum Moisture Content & Density (psi)	Level 2 or 3 (see descriptions in previous sections)
Plasticity Index, PI	Actual or use default for soil classification. NOTE: Use PI = 1 for drainage reasons if non-plastic
Liquid Limit, LL	Actual or use defaults for soil classification.
Gradation	Actual or use defaults for soil classification.
User Override Index Properties: • Unit Maximum Dry Unit Weight • Specific Gravity • Saturated Hydraulic Conductivity • Optimum Gravimetric Water Content • Degree of Saturation at Optimum	Actual or use defaults for soil classification. Enter specific values for these parameters if available. Measured values will be more accurate than these estimated values.

7.5 Step 3: Base/Subbase Material Characterization

The common base and subbase types applied by ITD for new flexible and rigid pavement design are:

- Asphalt (Emulsion) Treated Base (ATB, *ITD Standard Specification for Highway Construction*, Item 302).
- Asphalt Treated Permeable Base (*ITD Standard Specification for Highway Construction*, (Supplemental) ATPB, SSP 413).
- Untreated Aggregate Base (*ITD Standard Specification for Highway Construction*, Items 303 and 307; includes open graded shot rock base material).
- Granular Subbase (*ITD Standard Specification for Highway Construction*, Item 301; mostly granular borrow material designated as improved subgrade and should have an

R-value greater than the natural subgrade to be improved. Granular borrow may include cinder aggregate and selected granular excavation if quality is satisfactory).

From the time of initial construction of the State system until the early 1990s, the predominant method of pavement rehabilitation was to place a plant mix overlay on the existing pavement. A relatively small number of Cold-in-Place Recycle (CIR) and Hot-in-Place Recycle (HIR) projects have been constructed since around the mid-1980s. Since the early 1990s, the predominant method of pavement rehabilitation has been Cement Recycled Asphalt Base Stabilization (CRABS) followed by a plant mix overlay. While use of plant mix overlays without treating the existing pavement has remained significant, CRABS is the most widely used method for major rehabilitation of existing pavement in Idaho.

For rehabilitated pavement design or reconstruction of existing pavements, ITD typically includes the following material types as bases:

- **Full-Depth Reclamation (FDR/CRABS):** The CRABS process is a FDR process consisting of pulverizing the existing asphalt materials and a portion of the base. The materials are then mixed with cement, usually at the rate of 2 percent, and then recompacted.

- **CIR:** This procedure consists of cold-milling up to 4 inches depth, mixing emulsified asphalt and additives, relaying the material, and recompacting. Additives are included for timely stabilization of the material.

- **HIR:** This process recycles the top 1 to 1½ inches of the asphalt pavement layer. This is accomplished by heating the existing pavement, hot-milling the material, and relaying the material as new hot mixed pavement. Current practice is to use emulsified asphalt as a rejuvenating agent. HIR is predominantly a pavement preservation application. However, if HIR is used for pavement rehabilitation, it should include a structural overlay that meets pavement design requirements.

Reclaimed/Recycled Asphalt Materials

Use of the *AASHTOWare Pavement ME Design* for FDR including CRABS, CIR, and HIR recycled HMA currently requires (without local Idaho calibration) that these materials to be treated as “non-stabilized granular bases” with an appropriate constant modulus (e.g., the modulus of the material does not vary from month to month). This is accomplished by choosing the following selections:

- Add a layer below the HMA layer.

- Select “Non-Stabilized Base” and A-1-a type.

- Select "Resilient Modulus" and Input Level 3.
- Under "Analysis Types" select "Annual Representative Value."
- Enter the annual representative value modulus appropriate to the material being used for that layer (FDR, CRABS, CIR, HIR, etc.)

Guidance on selecting the "Annual Representative Value" is provided in Table 39. These moduli values (and those labeled "To Be Determined") are important in performance prediction and will require further evaluation and revision as well as providing those missing during the calibration effort.

NOTE: This layer is assumed (in the software) to not exhibit fatigue damage even though it has tensile strength and a higher modulus.

Future local calibration may provide the capability to consider these materials more comprehensively such as including their fatigue capabilities.

Table 39. Recommended (Level 2/3) Lab Resilient Modulus for Unbound Base/Subbase at Optimum Moisture, and In-Place Recycled Materials for Flexible and Rigid Pavements

ITD "Crush Base" Classification	AASHTO Classification	Level 3 Estimated Lab M_r at Optimum Moisture (psi)	Level 3 Annual Representative Modulus Value (psi)*
Item 301 - Granular Subbase	A-2-4, A-2-6, A-3	26,000 - 32,000	NA
Item 303 - Aggregate Base	A-1-a, A-1-b	38,000 - 40,000	NA
Item 307 - Open Graded Base			
Class I - Rock Cap	NA	25,000 - 60,000	40,000
Class II	NA	To Be Determined	
Class III	NA	To Be Determined	
Full-Depth Reclaimed			
With Cement (CRABS)	NA	NA	80,000*****
With Asphalt	Consider as HMA Equivalent**	NA	NA
With Lime Only	A-1-a, A-1-b	30,000 - 100,000	60,000***
Cold-in-Place Recycled with Emulsified Asphalt Added	Consider as HMA Equivalent in ME Design****	NA	NA
Hot-in-Place Recycled	Consider as HMA Equivalent in ME Design****	NA	NA

* The values included in this column need to be verified or confirmed from deflection basins over time and adjusted to laboratory equivalent values, or represent laboratory measured values at the equilibrium moisture content.

** The air voids and percent binder should be the in place material considering the asphalt in the existing layer and the amount of aggregate base included in the recycled layer.

*** The elastic modulus values should be selected by the percentage of HMA versus aggregate base included in the recycled layer.

**** The air voids and percent binder should represent the in place material considering the amount of asphalt binder added to the recycled HMA. The amount of asphalt binder should have been determined using laboratory mixture design procedures.

***** This material is considered an unbound layer with only a small percentage of Portland cement added to the recycled material. The amount of Portland cement added results in an increase in stiffness, but not a layer that is subject to fatigue cracking. The increase in stiffness should be determined based on deflection basins measured over time.

NOTE: Several material moduli need to be determined during calibration.

Untreated Aggregate Base and Granular Subbase (*ITD Standard Specification for Highway Construction*, Items 301, 303, and 307)

Input for unbound aggregate bases is very similar to that for untreated subgrade soils and embankments.

Level 1 Untreated Aggregate Base and Granular Subbase Resilient Modulus (M_r) and Other Material Properties

Not applicable.

Level 2/3 Untreated Aggregate Base and Granular Subbase Resilient Modulus (M_r) and Other Material Properties

For untreated aggregate base and granular subbase materials with thickness greater than 8 inches, divide the base layer into sublayers with thickness between 4 to 6 inches on average. Determine initial Level 2 or 3 M_r as follows:

- For Level 2 M_r, perform lab testing and obtain base material R-value using Idaho T-8. Convert the estimated base/subbase material R-value into M_r at optimum moisture using the following relationship:

$$M_r = 1155 + 555(R)$$

where

M_r = Base/Subbase Resilient Modulus (psi)
R = R-Value of the Base/Subbase Material
NOTE: Since the Materials for a base course are always granular, there is no adjustment to convert the Idaho T-8 procedures to the AASHTO T-190 procedures.

Equation 7

Although obtaining R-value directly from lab testing is recommended, base/subbase R-value estimates can also be obtained by measuring aggregate material plasticity index and percent passing the No. 200 sieve and entering the values into the equation below:

$$R = 10^{(1.893 - 0.00159 * P200 - 0.022 * PI)}$$

where

R = R-Value
P200 = Percent Passing No. 200 Sieve
PI = Plasticity Index

Equation 8

Computed untreated aggregate base/subbase mean M_r must be input directly into *AASHTOWare Pavement ME Design*.

- For Level 3 inputs, M_r recommendations for untreated aggregate base and granular subbase materials are provided in Table 39.

Ratio of Unbound Material Layer Moduli

The M_r of aggregate or granular base/subbase, and full-depth recycled layers is dependent on the M_r of the supporting layers; thus, as a rule of thumb, the Level 2 or 3 M_r entered into *AASHTOWare Pavement ME Design* for a granular base layer must not exceed 3 times the M_r of the supporting layer to avoid decompaction of that layer.

Figure 29 may be used to adjust the M_r of the unbound aggregate base layer to ensure that it is in agreement with the above rule of thumb. As shown in Figure 29, M_r adjustment depends on layer thickness and the M_r of the supporting layers (see Appendix A). Thus, for pavement design using *AASHTOWare Pavement ME Design*, the M_r bottom-most base sublayer must first be adjusted using the sublayer thickness and subgrade M_r. This is followed by each overlying layer until M_r values for each base sublayer are determined.

NOTE: As the base comprises a single layer, only a single adjustment based on base layer thickness and subgrade M_r is required.

The maximum base layer/sublayer M_r must not be greater than the values presented in Table 39.

Other Inputs Required for Level 2/3 Unbound Aggregate Base Characterization are Listed Below:

- **MDD:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.
- **Optimum Moisture Content:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.
- **Specific Gravity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.
- **Saturated Hydraulic Conductivity:** Compute using *AASHTOWare Pavement ME Design* predictive equations based on the following inputs: Gradation, Plasticity Index, and Liquid Limit.
- **Soil Water Characteristic Curve Parameters:** Select based on aggregate/subgrade material AASHTO soil class.

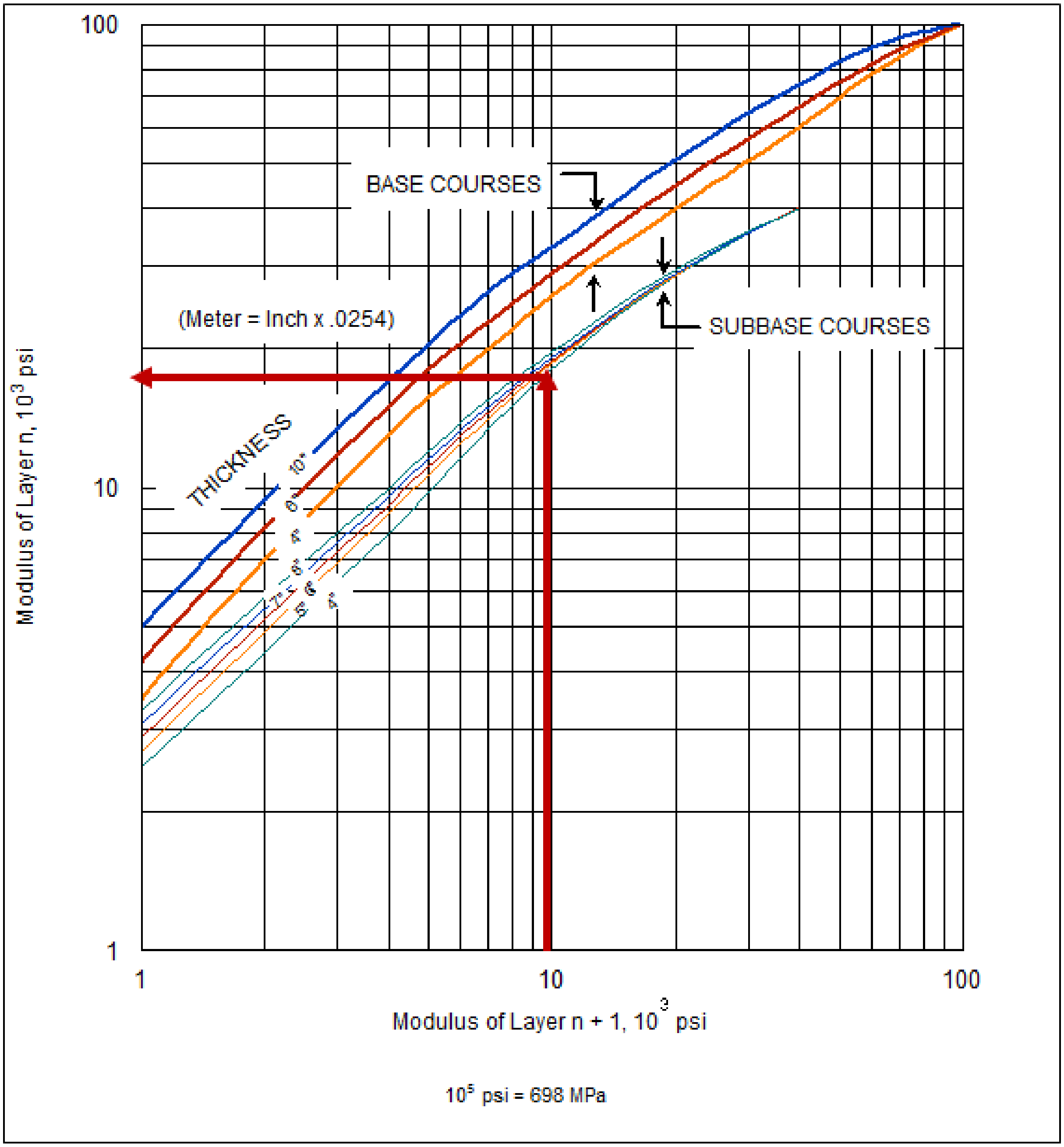


Figure 29. Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers (Barker & Brabston, 1975)

Summary of Level 3 Unbound Aggregate for Base and Subbase Inputs for AASHTOWare Pavement ME Design

Table 40 provides guidance on various inputs for unbound base and subbase layer properties for *AASHTOWare Pavement ME Design*.

Table 40. Recommended Level 3 Inputs for Unbound Aggregate for Base and Subbase Layers

Unbound Material	Crushed Stone, Gravel, or AASHTO Class A-1-a through A-1-b
Thickness (in.)	Actual if less than 8 inches. Otherwise, divide the base layer into sublayers with thickness between 4 and 6 inches on average.
Strength Properties Input Level	Level 3
Poisson's Ratio	0.35
Coefficient of Lateral Pressure	0.5
Compacted Unbound Material or Uncompacted Natural Unbound Material	Click on "Compacted" option for all base/subbase layers.
M_r at Optimum Moisture Content (psi)	Use recommended M_r values from Table 39. NOTE: That base/subbase single layer or sublayers to underlying layer (other base sublayer or subgrade) M_r ratio should be less than 3 to prevent decompaction of the base/subbase. Thus, the maximum base M_r should be the subgrade/embankment M_r*3. NOTE: The maximum base layer/sublayer M_r must not be greater than the values presented in Table 39.
Plasticity Index	Actual or default, always use 1 minimum, even if non-plastic for drainage reasons.
Liquid Limit	Actual or default
Gradation	Use actual or defaults for AASHTO soil class in *AASHTOWare Pavement ME Design*.
User Override Index Properties • **Unit Maximum Dry Unit Weight** • **Specific Gravity** • **Saturated Hydraulic Conductivity** • **Optimum Gravimetric Water Content** • **Degree of Saturation at Optimum**	User may enter specific values for these parameters if available. Measured values will be more accurate than these estimated values.

7.6 Step 4: Pavement Surface Materials

The two types of pavement surface materials used by the ITD are HMA and PCC. Descriptions of these materials and how they are characterized for *AASHTOWare Pavement ME Design* are presented in the following sections.

HMA

Designers must select an appropriate mix and binder type. HMA mix types selected based on anticipated cumulative truck traffic over pavement "Design Life". The binder type is selected based on geographic area, pavement temperature, and air temperature.

ITD HMA mixes are classified based on cumulative truck traffic (FHWA Truck Classes 4 through 13) applications within a 20-year design period (see Table 41). Regardless of mix type, inputs required by *AASHTOWare Pavement ME Design* at Levels 1, 2, and 3 remain the same as described in the following sections.

Table41. Hot Mix Asphalt Class Requirements

Class of HMA	SP2	SP3	SP5	SP6
Design ESALs* (millions)	< 1	1 to <10	10 to <30	≥ 30
Design Trucks** (millions)	<0.2 to <0.6	0.6 to 1.8	1.8 to 18	≥ 18

*Regardless of the actual "Design Life" of the roadway, determine the design ESALs or trucks for 20 years.
** The anticipated project traffic level expected on the design lane over a 20-year period.

NOTE: The ESALs for a Pavement ME project is provided in an intermediate file named FlexibleESAL.txt.

The type of PG binder selected is based on geographic area, pavement temperature, and air temperature using the Superpave software, LTPPBIND (select 98 percent reliability for both the low and high temperature value) must be used to determine the appropriate binder type for a given project location.

NOTE: LTPPBIND considers the depth of the HMA layer within the pavement structure and cumulative traffic over the design period (ESALs) in determining appropriate binder type.

Typical binder grades used in Idaho are PG 58-28, PG 58-34, PG 64-28, PG 64-34, PG 70-28, and PG 76-28.

NOTE: These are without upward adjustments for traffic or downward adjustments for use of RAP.

For designs where low temperature transverse cracking is critical (extreme cold and high elevations), low temperature would mostly be -34°C or -40°C. Typical low temperature is -28°C.

Level 1

For atypical or critical designs, designers are encouraged to adopt Level 1 inputs. For all other designs, designers must use Level 1 inputs if available. Required Level 1 HMA inputs are as follows:

- Dynamic Modulus (E*) (30 Individual Values Representing 5 Different Temperatures and 6 Different Frequencies).
- Shear Modulus (G*) and Phase Angle (δ) at 4 Different Temperatures.
- Volumetric Binder Content (as constructed).
- Percentage of Air Voids in HMA Mix (as constructed).
- HMA Unit Weight.

As part of the process of implementing *AASHTOWare Pavement ME Design* in Idaho, ITD performed comprehensive characterization of commonly used HMA mixes. The laboratory testing program comprised of identifying typical projects across the State, field sampling, and laboratory testing. Table 42 presents descriptions of project location. The outcome of the laboratory testing program was the development of the default Level 1 HMA inputs. Tables 43 through 44 present default Level 1 inputs developed. Designers must select the properties of mix type of interest. It is recommended that designers select the properties from projects closest to their project location. A detailed description of Level 1 testing for HMA materials characterization is presented in the *AASHTO MEPDG Manual of Practice*. Recommendations for default volumetric binder content, percentage of air voids, and HMA unit weight are presented in Table 45.

Table 42. Typical ITD HMA Mix Locations

Mix ID	Highway	Project ID	Project Number	Key Number	ITD Class
SP1-1*	-	STC-3840, Ola Highway, Kirkpatrick Road North	A 011(945)	11945	SP1
SP2-1	US-20	Cat Creek Summit to MP 129 to Camas County	A 009(867)	09864 09867	SP2
SP2-2	SH-6	Washington State Line to US 95/SH6	S07209A	08883	SP2
SP3-1	I-15	Sage Junction to Debois, SBL	A 010(010)	10010	SP3
SP3-2	US-20	Junction US-26 to Bonneville County Line	STP 6420(106)	09239	SP3
SP3-3	SH-75	Bellevue to Hailey	A 009(865)	09865	SP3
SP3-4	US-20	Rigby North & South	NH 6470(134)	09005	SP3
SP3-5	SH-62 SH-162	Oak Street, Nez Perce	ST 4749(612)	09338	SP3
SP3-6	US-30	Topaz to Lava Hot Springs	NH A010(455)	10455	SP3
SP3-7	US-95	Lapwai to Spalding	NH 4110(144)	08353	SP3
SP3-8	US-20	MP 112.90 to MP 124.63	NH 3340(109)	09106	SP3
SP3-9	WA-270	Pullman to Idaho State Line, (1/2 inch Mix)	01A-G71985(270)	07120	SP3
SP3-10	WA-270	Pullman to Idaho State Line, (1 inch Mix)	01B-G71974(270)	07120	SP3
SP4-1*	-	Broadway Ave. Rossi St. to Ridenbaugh Canal Bridge	A 009(812)	09812	SP4
SP4-2*	I-84	Cleft to Sebree	A 010(533)	10533	SP4
SP4-3*	US-30	Alton Road to MP 454 / Dingle	NH 1480(127)	09543	SP4
SP4-4*	I-84	Jerome Interchange	IM 84-3(074)165	08896	SP4
SP5-1	I-84	Ten Mile Rd to Meridian Interchange, Reconstruction	A 0011(003)	11003	SP5
SP5-2	I-15	Deep Creek to Devil Creek Interchange	A 011(094)	11094	SP5
SP5-3	SH-55	EP Ramps to Fairview Avenue	A 010(527)	10527	SP5
SP5-4	US-95	Moscow Mountain Passing Lane	A 011(031)	11031	SP5
SP6-1	I-84	Burley to Declo & Heyburn Interchange O'Pass	IM 84-3(071)211	09219	SP6
SP6-2	-	Garrity Bridge Interchange & 11th Avenue to Garrity	A 010(915) & A 011(974)	10915 & 11974	SP6

*SP-1 and SP-4 mixes are no longer used.

Table 43. Dynamic Modulus (E*) Values of Typical ITD HMA Mixtures

Mix ID	Binder Grade	Temperature (°F)	Testing Frequency (psi)					
			0.1	0.5	1	5	10	25
SP1-1	PG 58-28	14	1,697,895	1,953,622	2,055,862	2,272,131	2,355,781	2,457,557
		40	773,198	1,023,243	1,140,289	1,420,212	1,541,246	1,705,792
		70	181,370	295,878	363,538	552,885	648,320	791,835
		100	35,926	67,566	86,066	156,859	198,267	263,316
		130	9,282	17,143	22,720	45,665	62,098	88,596
SP2-1	PG 58-28	14	1,440,000	1,720,000	1,810,000	2,010,000	2,070,000	2,150,000
		40	604,000	884,000	1,020,000	1,350,000	1,490,000	1,680,000
		70	82,400	174,000	230,000	412,000	509,000	658,000
		100	11,500	26,100	39,200	95,600	136,000	206,000
		130	3,740	5,950	7,540	16,600	25,500	44,600
SP2-2	PG 58-34	14	1,350,000	1,510,000	1,580,000	1,690,000	1,730,000	1,780,000
		40	586,000	815,000	921,000	1,180,000	1,290,000	1,430,000
		70	104,000	197,000	249,000	418,000	504,000	633,000
		100	12,700	28,500	40,100	88,700	120,000	177,000
		130	3,540	5,790	8,110	19,400	28,100	46,400
SP3-1	PG 64-28	14	2,350,000	2,710,000	2,850,000	3,150,000	3,270,000	3,410,000
		40	1,070,000	1,430,000	1,600,000	2,010,000	2,190,000	2,430,000
		70	234,000	399,000	485,000	754,000	887,000	1,090,000
		100	37,000	75,600	100,000	194,000	250,000	340,000
		130	7,800	14,900	20,400	44,400	62,200	93,100
SP3-2	PG 64-28	14	2,220,000	2,530,000	2,640,000	2,880,000	2,960,000	3,070,000
		40	899,000	1,240,000	1,410,000	1,810,000	1,990,000	2,290,000
		70	179,000	319,000	397,000	637,000	762,000	954,000
		100	28,800	58,100	77,800	155,000	203,000	280,000
		130	9,510	16,700	22,300	47,200	65,000	96,500
SP3-3	PG 58-28	14	2,090,000	2,230,000	2,270,000	2,350,000	2,370,000	2,400,000
		40	881,000	1,260,000	1,430,000	1,850,000	2,020,000	2,250,000
		70	137,000	279,000	363,000	633,000	770,000	976,000
		100	14,200	32,600	48,700	118,000	168,000	253,000
		130	7,310	11,300	15,000	34,400	51,600	88,300
SP3-4	PG 70-28	14	2,040,000	2,400,000	2,530,000	2,800,000	2,900,000	3,020,000
		40	903,000	1,270,000	1,440,000	1,860,000	2,040,000	2,290,000
		70	161,000	308,000	389,000	648,000	781,000	979,000
		100	21,700	50,400	71,000	158,000	212,000	311,000
		130	5,060	9,130	12,500	29,600	44,000	74,500

Table 43.(Cont.) Dynamic Modulus (E*) Values of Typical ITD HMA Mixtures

Mix ID	Binder Grade	Temperature (°F)	Testing Frequency (psi)					
			0.1	0.5	1	5	10	25
SP3-5-1	PG 58-28	14	1,600,000	1,870,000	1,980,000	2,220,000	2,310,000	2.420,000
		40	746,000	1,010,000	1,120,000	1,410,000	1,530,000	1.690,000
		70	154,000	269,000	330,000	527,000	625,000	769,000
		100	27,100	52,400	69,600	135,000	176,000	241,000
		130	5,240	9,240	12,500	27,700	39,700	61,800
SP3-5-2	PG 58-28	14	1,660,000	1,890,000	1,970,000	2,150,000	2,220,000	2.290,000
		40	740,000	996,000	1,110,000	1,400,000	1,520,000	1.690,000
		70	153,000	265,000	326,000	514,000	610,000	751,000
		100	21,700	45,500	61,600	124,000	162,000	224,000
		130	4,740	9,380	12,900	29,000	41,400	64,100
SP3-5-3	PG 58-28	14	1,860,000	2,070,000	2,150,000	2,300,000	2,360,000	2.430,000
		40	783,000	1,060,000	1,190,000	1,500,000	1,640,000	1.830,000
		70	159,000	278,000	343,000	546,000	650,000	803,000
		100	22,400	47,400	64,200	129,000	169,000	235,000
		130	6,240	12,600	17,800	40,000	56,500	83,500
SP3-5-4	PG 58-28	14	1,970,000	2,230,000	2,330,000	2,540,000	2,630,000	2.720,000
		40	908,000	1,190,000	1,330,000	1,640,000	1,790,000	1.980,000
		70	214,000	354,000	425,000	646,000	758,000	925,000
		100	36,200	72,200	94,500	178,000	228,000	305,000
		130	8,620	17,000	23,500	49,900	69,900	103,000
SP3-5-5	PG 58-28	14	1,540,000	1,710,000	1,780,000	1,910,000	1,960,000	2.010,000
		40	771,000	1,000,000	1,100,000	1,340,000	1,440,000	1.540,000
		70	179,000	300,000	361,000	550,000	641,000	770,000
		100	28,300	57,800	76,600	147,000	188,000	253,000
		130	6,470	12,000	16,400	35,400	49,600	74,600
SP3-6	PG 64-34	14	1,190,000	1,450,000	1,550,000	1,740,000	1,800,000	1,880,000
		40	361,000	585,000	696,000	991,000	1,130,000	1,320,000
		70	48,700	100,000	133,000	257,000	328,000	441,000
		100	9,150	16,800	22,100	47,200	64,800	101,000
		130	4,180	5,950	7,020	13,100	17,900	30,200
SP3-7	PG 70-28	14	1,610,000	1,820,000	1,900,000	2,050,000	2,110,000	2,170,000
		40	712,000	972,000	1,090,000	1,380,000	1,500,000	1,660,000
		70	148,000	264,000	325,000	522,000	622,000	771,000
		100	24,300	50,100	66,600	134,000	176,000	246,000
		130	7,500	13,800	18,300	38,400	52,600	81,500

Table 43.(Cont.) Dynamic Modulus (E*) Values of Typical ITD HMA Mixtures

Mix ID	Binder Grade	Temperature (°F)	Testing Frequency (psi)					
			0.1	0.5	1	5	10	25
SP3-8	PG 70-28	14	1,670,000	1,930,000	2,030,000	2,230,000	2,310,000	2,400,000
		40	805,000	1,070,000	1,190,000	1,480,000	1,600,000	1,760,000
		70	191,000	329,000	399,000	616,000	720,000	876,000
		100	39,000	72,600	94,200	179,000	229,000	314,000
		130	9,550	17,700	23,600	48,800	65,600	98,400
SP3-9	PG 70-28	14	1,680,000	1,920,000	2,020,000	2,210,000	2,280,000	2,360,000
		40	838,000	1,100,000	1,220,000	1,510,000	1,630,000	1,790,000
		70	202,000	338,000	407,000	621,000	726,000	877,000
		100	38,100	74,400	97,300	184,000	235,000	317.,000
		130	9,840	17,200	22,500	46,200	62,900	96,600
SP3-10	PG 70-28	14	1,160,000	1,370,000	1,460,000	1,640,000	1,710,000	1,790,000
		40	509,000	715,000	811,000	1,050,000	1,150,000	1,280,000
		70	97,700	182,000	228,000	377,000	453,000	569,000
		100	15,000	31,000	42,100	88,500	118,000	169,000
		130	2,920	5,550	7,220	16,200	23,200	40,100
SP4-1	PG 70-28	14	1,630,000	1,800,000	1,860,000	1,970,000	2,010,000	2,050,000
		40	704,000	971,000	1,090,000	1,390,000	1,500,000	1,670,000
		70	140,000	255,000	317,000	521,000	624,000	770,000
		100	22,000	44,200	58,700	121,000	160,000	232,000
		130	8,250	14,800	19,200	38,700	53,100	78,700
SP4-2	PG 76-28	14	1,830,000	2,110,000	2,230,000	2,460,000	2,550,000	2,660,000
		40	958,000	1,240,000	1,370,000	1,670,000	1,800,000	1,930,000
		70	248,000	405,000	486,000	723,000	839,000	1,000,000
		100	59,500	102,000	128,000	226,000	284,000	377,000
		130	14,500	24,500	30,700	58,700	77,200	117,000
SP4-3	PG 64-34	14	1,270,000	1,600,000	1,730,000	1,990,000	2,090,000	2,210,000
		40	389,000	626,000	743,000	1,070,000	1,220,000	1,430,000
		70	55,600	112,000	146,000	279,000	355,000	474,000
		100	10,900	20,300	26,500	55,500	75,500	110,000
		130	4,470	6,800	8,040	14,600	20,100	30,300
SP4-4	PG 70-28	14	2,610,000	2,900,000	3,000,000	3,210,000	3,280,000	3,360,000
		40	1,280,000	1,690,000	1,880,000	2,320,000	2,510,000	2,750,000
		70	296,000	521,000	631,000	973,000	1,130,000	1,370,000
		100	46,400	96,800	130,000	264,000	341,000	467,000
		130	13,000	23,200	31,200	67,700	93,600	145,000

Table 43. (Cont.) Dynamic Modulus (E*) Values of Typical ITD HMA Mixtures

Mix ID	Binder Grade	Temperature (°F)	Testing Frequency (psi)					
			0.1	0.5	1	5	10	25
SP5-1	PG 70-28	14	1,650,000	1,700,000	1,720,000	1,740,000	1,750,000	1,760,000
		40	744,000	1,020,000	1,140,000	1,430,000	1,550,000	1,710,000
		70	143,000	266,000	335,000	549,000	653,000	803,000
		100	21,000	43,900	59,900	126,000	167,000	236,000
		130	16,600	31,600	42,500	83,900	110,000	157,000
SP5-2	PG 64-34	14	1,310,000	1,580,000	1,680,000	1,880,000	1,950,000	2,040,000
		40	443,000	680,000	799,000	1,120,000	1,280,000	1,480,000
		70	72,200	140,000	180,000	327,000	407,000	528,000
		100	13,900	25,800	33,600	69,700	93,900	154,000
		130	6,190	8,840	10,500	19,800	27,200	52,100
SP5-3	PG 70-28	14	2,130,000	2,400,000	2,510,000	2,720,000	2,790,000	2,880,000
		40	1,040,000	1,360,000	1,510,000	1,850,000	2,000,000	2,190,000
		70	246,000	412,000	497,000	756,000	883,000	1,070,000
		100	44,700	86,800	112,000	210,000	268,000	362,000
		130	12,100	21,300	27,700	56,200	75,700	108,000
SP5-4	PG 70-28	14	1,710,000	1,950,000	2,040,000	2,220,000	2,280,000	2,350,000
		40	740,000	1,040,000	1,180,000	1,500,000	1,650,000	1,830,000
		70	139,000	259,000	324,000	539,000	647,000	810,000
		100	23,000	46,300	62,100	128,000	171,000	242,000
		130	8,080	12,900	16,200	31,900	44,500	66,400
SP6-1	PG 76-28	14	1,540,000	1,740,000	1,820,000	1,950,000	2,000,000	2,060,000
		40	786,000	1,050,000	1,170,000	1,470,000	1,600,000	1,760,000
		70	244,000	411,000	494,000	760,000	892,000	1,090,000
		100	40,500	81,100	106,000	203,000	260,000	350,000
		130	10,400	18,000	23,500	48,200	66,600	96,600
SP6-2	PG 76-28	14	2,090,000	2,310,000	2,400,000	2,560,000	2,620,000	2,690,000
		40	1,040,000	1,350,000	1,490,000	1,810,000	1,950,000	2,120,000
		70	270,000	434,000	518,000	769,000	891,000	1,080,000
		100	47,600	91,200	117,000	217,000	276,000	367,000
		130	13,900	24,900	31,900	63,300	84,200	118,000

Table 44. Shear Modulus (G*) and Phase Angles (δ) of Typical ITD HMA Mixtures

Binder Grade	Temperature (°F)	G* (Pa)	δ (deg)
PG 58-28	40	2,4571,802	57.96
	70	1396,791	60.92
	100	68,395	73.70
	130	5,776	82.02
PG 58-34	40	4,490,000	56.13
	70	228,000	63.32
	100	25,100	68.09
	130	3,490	70.34
PG 64-28	40	5,893,366	58.87
	70	1,616,897	60.97
	100	103,989	66.79
	130	10,735	73.77
PG 64-34	40	8,420,687	46.93
	70	504,367	60.75
	100	39,119	66.87
	130	5,945	61.47
PG 70-28	40	9,963,942	58.22
	70	1,886,139	59.61
	100	111,078	61.85
	130	13,355	67.88
PG 76-28	40	21,980,433	42.28
	70	2,190,720	59.11
	100	133,602	58.16
	130	18,570	63.63

Level 2

Level 2 inputs are not recommended.

Level 3

Level 3 inputs are recommended for non-critical designs such as low-volume highways, urban low truck or bus roadways, and local roads. For Level 3, *AASHTOWare Pavement ME Design* will internally estimate E* using the E* predictive equation that is incorporated in the software, along with the inputs listed below:

- **Gradation.**
- **Binder Grade** (used to obtain typical Ai-VTS values and viscosity based on asphalt binder grade (PG, or viscosity, or penetration grades) as included in the software).

- **Volumetric Binder Content** (as constructed, not lab).
- **Percentage of Air Voids in HMA Mix** (as constructed, not lab).
- **HMA Unit Weight** (as constructed, not lab).

Table 45 presents ITD defaults for Level 3 HMA materials inputs. It is recommended that the PG binder designation be used.

Figures 30 and 31 show typical ITD binders by geographic location.

Table 45. Level 3 Default HMA Inputs for ITD

ITD Mix ID	HMA Aggregate Gradation				Effective Binder Content (percent)	Air Voids (percent)	Unit Weight (pcf)
	Passing ¾ inch	Passing ⅜ inch	Passing No. 4	Passing No. 200			
SP1	100.0	86.0	54.0	5.20	11.5	7.7	137.9
SP2	100.0	82.0	55.0	5.95	12.1	7.6	141.9
SP3	99.0	76.7	50.1	6.20	11.3	7.5	145.1
SP4	96.5	72.5	46.8	4.70	10.1	7.2	141.5
SP5	99.3	69.8	46.3	4.20	10.1	7.6	141.8
SP6	98.5	73.5	51.5	4.15	9.5	6.5	142.2
ATB							
ATPB							

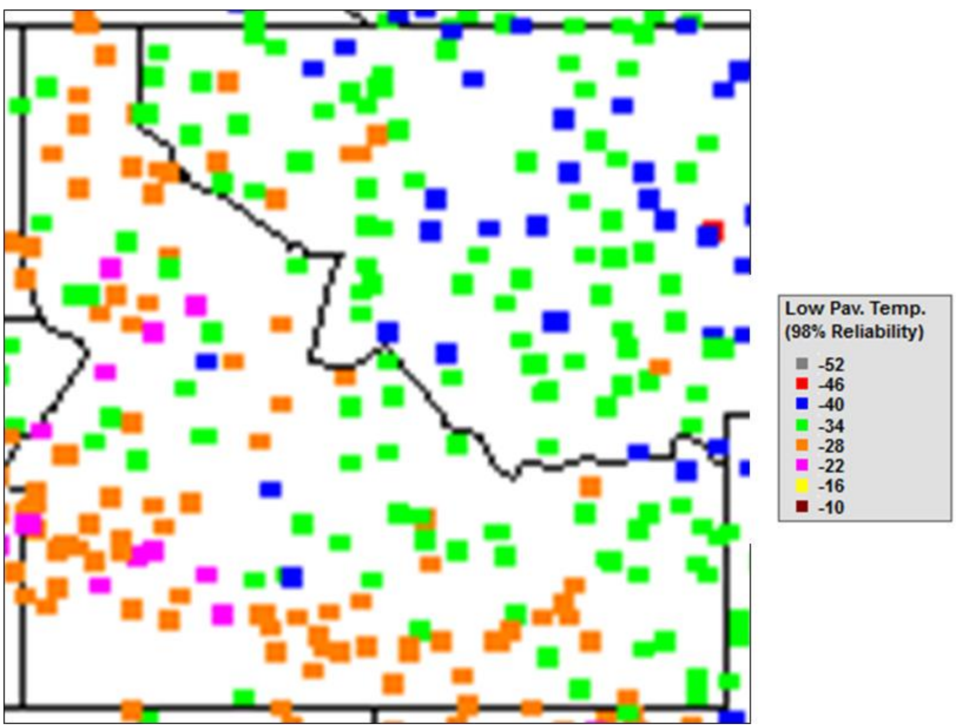


Figure 30. Typical Low Temperature Binder by Geographical Location in Idaho

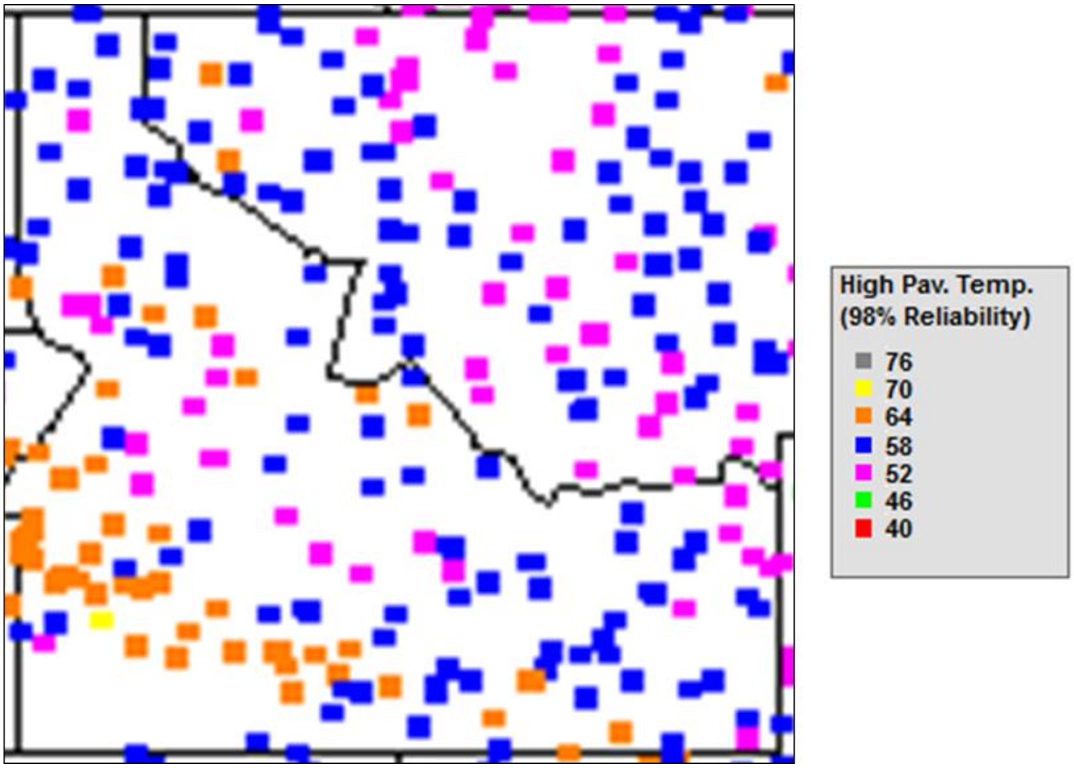


Figure 31. Typical High Temperature Binder by Geographical Location in Idaho

NOTE: that these are without traffic bump.

Other new and existing HMA properties required for HMA characterization for *AASHTOWare Pavement ME Design* are presented in the following sections.

Tensile Strength

Select Level 3 and the software will internally compute tensile strength using the inputs previously provided, using the relationship below:

$$TS(psi) = 7416.712 - 114.016 * Va - 0.304 * Va^2 - 122.592 * VFA + 0.704 * VFA^2$$
$$+ 405.71 * Log10(Pen77) - 2039.296 * log10(A)$$

where:

TS = Indirect Tensile Strength at 14^oF
Va = As-Constructed HMA Air Voids (percent)
VFA = As-Constructed Voids Filled With Asphalt (percent)
Pen77 = Binder Penetration at 77^oF (mm/10)
A = Viscosity-Temperature Susceptibility Yntercept

Equation 9

Input variables can be obtained through testing of lab-prepared mix samples, extracted cores (for existing pavements), or from agency historical records.

Creep Compliance D(t)

Select Level 3 and the software will internally compute creep compliance using the inputs previously provided, using the relationship below:

$$D(t) = D_1 * t^m$$

Equation 10

$$\log(D_1) = -8.524 + 0.01306 * Temp + 0.7957 * \log10(Va) + 2.0103 * \log10(VFA) \\ -1.923 * \log10(A)$$

Equation 11

$$m = 1.1628 - 0.00185 * Temp - 0.04596 * Va - 0.01126 * VFA + 0.00247 * Pen77 \\ + 0.001683 * Temp * Pen77^{0.4605}$$

where:

t = Time
Temp = Temperature at Which Creep Compliance is Measured (^{o}F)
Va = As-Constructed Air Voids (percent)
VFA = As-Constructed Voids Filled with Asphalt (percent)
Pen77 = Binder Penetration at 77^{o}F (mm/10)

Equation 12

Input variables can be obtained through testing of lab-prepared mix samples, extracted cores (for existing pavements), or from agency historical records.

Poisson's Ratio

Recommended values for Poisson's ratio are provided in Table 46. Select "True" on the box that asks, "Is Poisson's ratio calculated?" This automatically provides Poisson's Ratio as a function of temperature.

Table 46. Poisson's Ratio Recommended for HMA

HMA Temperature (°F)	Dense-Graded HMA* ($\mu_{typical}$)	Open-Graded HMA* ($\mu_{typical}$)
< 0 °F	0.15	0.35
0 – 40 °F	0.20	0.35
40 – 70 °F	0.25	0.40
70 – 100 °F	0.35	0.40
100 – 130 °F	0.45	0.45
> 130 °F	0.48	0.45

*Level 3

Surface Shortwave Absorptivity

Use default of 0.85.

Thermal Conductivity

Typical values for asphalt concrete range from 0.44 to 0.81 BTU(ft)(hr)(°F). Use default value set in program = 0.67 BTU(ft)(hr)(°F).

Heat Capacity

Typical values for asphalt concrete range from 0.22 to 0.40 BTU(lb)(°F). Use default value set in program = 0.23 BTU/lb °F.

Coefficient of HMA Thermal Contraction

Use the relationship below:

$$L_{MIX} = \frac{VMA * B_{ac} + V_{AGG} * B_{AGG}}{3 * V_{TOTAL}}$$

where:

LMIX = Linear Coefficient of Thermal Contraction of the Asphalt Concrete Mixture (1/°F)
Bac = Volumetric Coefficient of Thermal Contraction of the Asphalt Cement in the Solid State (1/°F)
BAGG = Volumetric Coefficient of Thermal Contraction of the Aggregate (1/°F)
VMA = Percent Volume of voids in the mineral aggregate (= % Volume Air Voids + 5 Volume of Asphalt Cement - % Volume of Absorbed Asphalt Cement)
VAGG = Percent Volume of Aggregate in the Mixture
VTOTAL = 100 Percent

Equation 13

Typical values for linear coefficient of thermal contraction, volumetric coefficient of thermal contraction of the asphalt cement in the solid state, and volumetric coefficient of thermal contraction of aggregates measured in various research studies are as follows:

Where:
LMIX = 2.2 to 3.4*10-5 /°C (linear)
Bac = 3.5 to 4.3*10-4 /°C (cubic)
BAGG = 21 to 37*10-6 /°C (cubic)

Portland Cement Concrete (PCC)

Level 1

Level 1 PCC material characterization (flexural strength) is not recommended at this time. A detailed description of Level 1 PCC testing for PCC materials characterization is presented in the Mechanistic-Empirical Pavement Design Guide, A Manual of Practice.

Level 2

Level 2 PCC material characterization requires the following:

- **PCC Compressive Strength at 7-, 14-, 28-, and 90-day**. Long-term to 28-day PCC compressive strength of 1.44 is recommended. The ITD materials lab is setup to perform PCC compressive strength. Designers must obtain mix-specific compressive strength values from the ITD Materials Lab.

- **PCC Coefficient of Thermal Expansion (CTE).** Default Level 2 CTE values are determined based on PCC coarse aggregate geological class. Designers must determine the source of PCC coarse aggregate and thus, the predominant geological class. With this information, select the most appropriate CTE value from the recommendations presented in Table 47.

- **For All Other Inputs, Assume Level 3 Values.**

A detailed description of Level 2 PCC testing for PCC materials characterization is presented in the Mechanistic-Empirical Pavement Design Guide, A Manual of Practice.

Level 3

New PCC

Level 3 PCC material characterization requires the following

NOTE: Idaho testing data are needed to establish these values more accurately.

- **28-day PCC Mean Flexural Strength:** 700 psi.
- **28-day PCC Mean Elastic Modulus:** 4,200,000 psi.
- **PCC Mean Coefficient of Thermal Expansion (CTE):** 4.7 x 10^{-6} in./in./°F (or if geological source of coarse aggregate is known, use Table 47.
- **For All Other Inputs,** see Table 48.

Table 47. *AASHTOWare Pavement ME Design* National Defaults Based on Concrete Coarse Aggregate Geological Class

Coarse Aggregate Type	CTE (10^{-6}/°F)
Basalt	4.4
Diabase	5.2
Granite	4.8
Schist	4.4
Chert	6.1
Dolomite	5.0
Limestone	4.4
Quartzite	5.2
Sandstone	5.8

NOTE: CTE is very critical input and tests on Idaho materials are needed to establish these values more accurately.

Table 48. PCC Level 3 Material Properties Inputs for *AASHTOWare Pavement ME Design*

Level 3 Input Category Data Items	Input Values
Unit Weight (pcf)	145
Poisson’s Ratio	0.2
Thermal Conductivity (BTU/hr-ft-°F)	1.25
Heat Capacity (BTU/lb-°F)	0.28
Cement Type	Type I
Cementitious Material (PCC + Pozzolans) (lb/yd^{3})	Basic Mix: 660 Cement + Fly Ash: 688
Water to Cement Ratio (w/c)	Basic Mix: 0.44 Cement + Fly Ash: 0.42
Coarse Aggregate Type	See list above
PCC Zero Stress Temperature (°F)	Computed internally by the software. (see Equation 14)
Ultimate Shrinkage, microstrain	Computed internally by the software.
Reversible Shrinkage	Use default of 50 percent.
Time to Develop 50 Percent of Ultimate Shrinkage	Use default of 35 days.
Curing Method	Curing compound.

Existing Intact PCC

Existing intact PCC properties are required only for HMA overlay, unbonded PCC overlay and for concrete pavement restoration. The designer must assess the overall condition of the existing pavement PCC using the guidelines presented in Table 49. Select typical modulus of elasticity values from the range of values given in Table 50 based on the pavement condition.

Table 49. Distress Types and Severity Levels Recommended for Assessing Rigid Pavement Structural Adequacy (at the time of evaluation)

Load-Related Distress	Highway Classification	Current Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
JPC Deteriorated Cracked Slabs (medium- and high-severity transverse and longitudinal cracks and corner breaks) (% slabs)	Interstate/ Freeway	>10	5 to 10	<5.000
	Primary	>15	8 to 15	<8.000
	Secondary	>20	10 to 20	<10.000
JPC Mean Transverse Joint/Crack Faulting (in.)	Interstate/ Freeway	>0.15	0.1 to 0.15	<0.100
	Primary	>0.20	0.125 to 0.20	<0.125
	Secondary	>0.30	0.15 to 0.3	<0.150
CRC Punchouts (medium- & high-severity) (#/lane-mile)	Interstate/ Freeway	>10	5 to 10	<5.000
	Primary	>15	8 to 15	<8.000
	Secondary	>20	10 to 20	<10.000

Table 50. Existing Intact PCC Typical Modulus Ranges

Qualitative Description of Pavement Condition	Typical Modulus Ranges (psi)	Mean Modulus (psi)
Good/Adequate	3 to 4 x 10^6	3.5 x 10^6
Marginal	1 to 3 x 10^6	2.0 x 10^6
Poor/Inadequate	0.3 to 1 x 10^6	0.65 x 10^6

Existing Fractured PCC

Existing fractured PCC properties are required for HMA or PCC overlays over fractured PCC pavements. The two common methods of fracturing JPCP slabs include:

1. Crack and Seat.
2. Rubblization.

Of the two methods, the most effective design to minimize reflection cracking is rubblized concrete material where it is broken into smaller aggregate-sized pieces that behave similar to a high-quality crushed aggregate layer. The Pavement ME can be used directly to design an HMA overlay of rubblized concrete similar to a flexible pavement design.

Crack and seat involves cracking the slab into larger pieces (e.g., 3 to 6 ft pieces) where the key design approach is to provide adequate HMA thickness to reduce deflections in the cracked JPCP to prevent the pieces from becoming loose and rocking which leads to reflection cracking. The Pavement ME cannot be used to directly design a crack and seat project because HMA over a cracked and seated slab behaves totally different than a flexible pavement. Only with the selection of a very conservative modulus of the cracked slab can a reasonable design be obtained using *AASHTOWare Pavement ME Desig*n (the program does not model reflection cracking originating from crack and seated PCC pieces. Thus, it is recommended to assume conservative reflection cracking values to predicted transverse cracking values.)

Select the M_r value in Table 51.

NOTE: Selection of too high of a modulus for rubblized material will prevent obtaining an adequate fatigue-based design for the HMA or PCC surface.

Table 51. Fractured (Rubblized) PCC Resilient Modulus for Design

Fractured PCC Type	**Resilient Modulus (psi)**
Rubblized (Into Crushed Granular Like Material)	50,000
Crack & Seat	70,000*

* The actual modulus may be much higher, however, this will result in far too thin HMA overlay to prevent rocking of the cracked pieces. Either the *AASHTOWare Pavement ME* should not be used to design HMA overlay of crack and seated pavement or additional research is needed to establish an appropriate input value.

Other PCC Inputs (existing intact and fractured PCC) are presented in Table 52.

Table 52. PCC Level 3 inputs for Existing Intact and Fractured PCC

Level 3 Input Category Data Items	Existing PCC Inputs	
	Intact	Fractured
28-Day Flexural Strength (psi)*	700	N/A
28-Day Elastic Modulus (psi)*	4,200,000	N/A
CTE (in./in./°F)*	4.7 x 10^{-6} (see Table 47)	N/A
Unit Weight (pcf)	145	145.00
Poisson's Ratio	0.2	0.2.
Thermal Conductivity (BTU/hr-ft-°F)	1.25	1.25
Heat Capacity (BTU/lb-°F)	0.28	0.28
Cement Type	Type I	N/A
Cementitious Material (PCC + Pozzolans) (lb/yd³)	Basic Mix: 660 Cement + Fly Ash: 688	N/A
Water-to-Cement Ratio (w/c)	Basic mix: 0.44 Cement + Fly Ash: 0.42	N/A
Coarse Aggregate Type	Limestone	N/A
PCC Zero Stress Temperature (°F)	Computed internally by the software. (see Equation 14)	N/A
Ultimate Shrinkage (microstrain)	Computed internally by the software.	N/A
Reversible Shrinkage	Use default of 50%	N/A
Time to Develop 50 percent of Ultimate Shrinkage	Use default of 35 days.	N/A
Curing Method	Curing compound	N/A

*Required for HMA over PCC and bonded PCC over PCC pavements.

Zero-Stress Temperature (New and Existing Intact PCC)

Zero stress temperature (Tz) occurs after placed concrete has cured and hardened sufficiently that the temperature begins to drop, resulting in tensile stress. It can be input directly or estimated from monthly ambient temperature and cement content using the equation shown below:

$$Tz = (CC*0.59328*H*0.5*1000*1.8/(1.1*2400) + MMT)$$

where:

Tz = Zero Stress Temperature (allowable range: 60°F to 120°F).
CC = Cementitious Content (lb/yd³).
H = $-0.0787+0.007*MMT-0.00003*MMT^2$.
MMT = Mean Monthly Temperature for Month of Construction (°F).

Equation 14

Chapter 8
JPCP Design Features

JPCP design features have a significant impact on predicted performance. Designers can optimize JPCP design to produce the most cost-effective pavement solution by selecting these inputs carefully. General guidance on selection of JPCP design inputs is provided in Table 53.

Table 53. Summary of Design Recommendations for Idaho New/ Reconstructed JPCP (Bare or as Composite Pavements)

JPCP Design Parameter	Recommended Inputs for JPCP Optimization
Slab Thickness (in.)	A minimum thickness for new concrete pavement is 9 in. & the design thickness should be rounded to the nearest inch. *AASHTOWare Pavement ME Design* allows PCC thickness to range from 6 - 16 in.
Permanent Curl/Warp Effective Temperature Difference (ºF)	-10 **DO NOT CHANGE THIS INPUT.**
Joint Spacing (ft)	≤ 10 in. concrete pavement thickness: 12 ft design joint spacing. > 10 in. concrete pavement thickness: 15 ft design joint spacing. (All joints should be perpendicular & of uniform spacing)
Sealant Type	Single sawcut with hot applied sealant, or as specified in plans.
Load Transfer Mechanism for Transverse Joints (round dowel bars)	*AASHTOWare Pavement ME Design* software analyzes the adequacy of the load transfer for transverse joints. Designers should follow these recommendations to determine if dowels are required to control joint faulting to achieve the design reliability level. Dowels are typically required when there are more than 250 trucks per day in the design lane. Dowels can be used in any thickness of slab ≥7 in. ITD policy is to use dowels for all new JPCP because faulting must be controlled to provide smoothness.
Dowel Diameter for Transverse Joints (in.)	Required dowel diameter typically increases with slab thickness. Dowels are typically available commercially with diameters of 1.00, 1.25, and 1.50 inches. Others are available but at a much higher cost. *AASHTOWare Pavement ME Design* software will indicate joint faulting as “Failed” if the chosen bar is too small. The dowel diameter should be increased until faulting has “Passed” the criteria. Minimum dowel bar diameters is keyed to slab thickness from *ITD’s Standard Drawing* C-1-B: 1.25 in. for less than ≤11 in. thickness. 1.50 in. for 11to 13 in. thickness. 1.75 in. for ≥13 in. thickness. Transverse joint load transfer efficiency (LTE) is shown graphically over time in the output. This should be above 90% over the analysis period.
Dowel Bar Spacing (in.)	12 in. (Use 12 in. even for designs with 5 dowels per wheelpath). Transverse joints can be designed with 5 dowels per wheel path spaced at 12 in.
Edge Support from 3 Alternatives: Asphalt or Turf Shoulder, Tied PCC Shoulder and Widened Slab	Conventional 12-ft traffic with HMA shoulder: None Conventional 12-ft traffic lane plus tied PCC: use tied PCC shoulder option with long term load transfer: 40%. Widened 13-ft traffic lane (Maximum) plus tied PCC: use tied PCC shoulder option & input: 13-ft slab width. ITD uses tied PCC shoulder or widened lane for all new JPCP.
Base Type	Actual specified
PCC-Base Interface Friction	The following lengths of time for full contact friction between the PCC slab & base course are recommended (means & range obtained from national calibration): Asphalt (permeable or dense graded) base: Use full design analysis period. Cement stabilized: ITD does not use this type of base. Unbound material base: Use full design analysis period.
Erodibility Index of Base	Recommendations: Asphalt (permeable or dense graded) base: Select 1 / 2, very erosion resistant Granular unbound aggregate base: Select 4, fairly erodible

Chapter 9
Rehabilitation Inputs

Rehabilitation design is very similar to new pavement design. However, rehabilitation design requires several new inputs and some modifications of other inputs that are related to the existing pavement. In rehabilitation design, the existing pavement typically has deteriorated from its original condition through all types of fracture, distortion, or material disintegration. Some of the material properties may also have changed over time, such as the oxidation of asphalt and the strengthening of concrete. *AASHTOWare Pavement ME Design* can account for these changes through modifying various design inputs and through a few new inputs related to the condition of the existing pavement. These modifications are basically used to adjust the various moduli of the existing pavement.

Recommendations for inputs similar to new pavement design are not repeated in this chapter; this chapter covers the modifications required of previously described inputs and the new inputs required for rehabilitation design. These inputs vary depending on the existing pavement and on the type of rehabilitation. Input recommendations are given for the following combinations of existing pavement and rehabilitation type:

- HMA Overlay of Existing HMA Pavement (see Tables 54 and 55).
- HMA Overlay of Existing JPCP (see Table 56).

Table 54. Characterization for HMA Overlay of Existing HMA Pavement

<table>
<tr><th>Rehabilitation Inputs Level</th><th>Rehabilitation Design Inputs
Existing HMA Pavement</th></tr>
<tr><td>1</td><td>Not used.</td></tr>
<tr><td>2</td><td>Requires measurement of wheelpath fatigue alligator cracking & total mean wheelpath rutting. For mill & fill HMA overlay, planned milling thickness is also required.

Alligator Cracking:
Identify representative length of heaviest trafficked lane along project (that has typical alligator cracking, if any).
Measure the alligator cracking in each wheel path & compute percent of total lane area alligator cracking. Include all severities of cracking including longitudinal wheelpath cracking.
Enter percent lane area alligator cracking into software.
Measure Mean Wheelpath (both wheelpaths) Rutting along the Project. Estimate total rutting in each layer & compute individual HMA, base, & subgrade rutting using the following typical values (enter values into software):
HMA Layer: 70%.
Base Layer: 5% (unbound aggregate), otherwise 0%.
Subgrade Layer: 25%.
Depth of Milling of Existing HMA.</td></tr>
<tr><td>3</td><td>Requires estimate of condition rating (based on alligator cracking) & total surface mean rutting.
Condition Rating: Based solely on alligator cracking in wheelpaths. Enter condition rating into software.
Estimate Percent Lane Area of Alligator Cracking In Wheelpaths & Determine Rating Below:
Excellent: < 3 percent area
Good: 4 - 5 percent area
Fair: 6 - 10 percent area
Poor: 11 - 20 percent area
Very Poor: > 20 percent area
Measure Rutting Along Each Wheelpath Throughout the Project & Average (examine & eliminate outliers). Enter mean value into software.
Depth of Milling.</td></tr>
</table>

Table 55. Characterization for Aggregate Base and Unbound Embankment/Subgrade of Existing HMA Pavement

Rehabilitation Inputs Level	Rehabilitation Design Inputs Existing HMA Pavement Base & Embankment/Subgrade
1	**Not Recommended.**
2/3	**Unbound Aggregate Base Course M_r:** Level 2: Rehabilitation – Back-calculate elastic modulus from FWD testing of existing pavement, determine mean, adjust to lab values at in situ moisture content by multiplying by 0.62. Level 2: Rehabilitation - Alternatively, estimate from R-value tests. NOTE: The in situ base material moisture content of the R-Value test specimen must also be input into the program in the location titled "Optimum Gravimetric Water Content" (percent). Level 3: Use default values from Table 39. For all designs, limit input M_r of unbound base to 3 times that of the subgrade as described in Section 7.5.
2/3	**Subgrade M_r:** Level 2: Rehabilitation - Back-calculate from FWD testing of existing pavement, determine mean, & adjust to lab values at in situ moisture by multiplying by 0.35. Measure in situ moisture content of the subgrade and enter this into "Optimum Gravimetric Water Content" (percent). Level 2: Rehabilitation - Alternatively, if subgrade moisture content cannot be measured, multiply the mean back-calculated modulus by the following & assume the moisture content is optimum (calculated by the program): Fine Grained Soil M_r = 0.55 * Back-calculated elastic modulus Coarse Grained Soil M_r = 0.67 * Back-calculated elastic modulus Level 2: Rehabilitation – Alternatively, estimate subgrade M_r from R-value tests as above for the base course. Determine the moisture content of the R-Value test specimen & enter that into the "Optimum Gravimetric Water Content" (percent) Level 3: Use global default M_r values at optimum moisture content from Section 7.4, Table 37.

Table 56. Characterization for HMA Overlay of Existing JPCP

Rehabilitation Input Type	Rehabilitation Inputs for Existing Pavement
Existing Fatigue Damage of Existing Concrete Slab	Determine slabs distressed (transverse cracked)/replaced (any replaced slabs) before restoration (or overlay) as percent of all slabs in design traffic lane section. For example, over a segment of the project, 10% with transverse cracks + 5% replaced totals 15% slabs entered into program. The 15% cracked/repaired slabs figure is used to determine past fatigue damage. Determine total slabs repaired/replaced that exist after restoration (or overlay) as percent of all slabs in design traffic lane section. For example, over the same segment of the project, all cracked slabs were replaced, and thus a total of 15% is entered into the program. This would leave 0% cracked slabs that were not replaced.
Elastic Modulus of Existing Intact Concrete Slab	Estimate elastic modulus of existing slab by testing of cores using ASTM C469, or estimate using 28-day modulus and multiplying by 1.2 for approximate long-term modulus.
Modulus of Fractured JPCP (for HMA Overlay)	Rubblized JPCP: 0,000 psi Cracked & Seated JPCP: 70,000 psi* (needs further validation) Unbound Base Course Modulus: Use default values from Section 7.5. (Limit M_r of unbound base to 3 times that of subgrade).
Stabilized Base Course Modulus	Estimate asphalt stabilized dynamic modulus through volumetric & gradation inputs (Level 3) from Section 7.5.
Subgrade/Unbound M_r	Level 1: Rehabilitation - Back-calculate dynamic k-value from FWD testing of existing pavement. Enter mean project k-value& testing month into software. See Section 7.4. Level 2: New construction - Estimate M_r from R-value tests. See Section 7.4. Level 3: Use default M_r values for soil class at Optimum Moisture Content from Section 7.4.

Chapter 10
Performing New or Reconstructed Pavement and Rehabilitation Designs

This section details the basic steps required to perform a pavement design using the *AASHTOWare Pavement ME Design* software:

1. Select a "Trial Design" alternative in *AASHTOWare Pavement ME Design*.
2. Select performance criteria and design reliability.
3. Obtain and enter required inputs for a "Trial Design".
4. Run the software, check all inputs, and review outputs for reasonableness.
5. Determine if the design meets the reliability criteria.
6. If the design meets the reliability criteria, review design to see if overdesigned; otherwise, accept as "passing."
7. If the design does not meet the reliability criteria, determine what design features require revision to improve reliability.
8. Revise "Trial Design" and repeat process until design meets criteria.

The following sections provide detailed descriptions of these steps for new/reconstruction designs and for rehabilitation designs.

NOTE: *AASHTOWare Pavement ME Design* includes a thickness optimization routine. This routine can be misleading and should only be used after experience has been gained. There are several other design features and materials properties that may provide for better optimization.

10.1 Steps Required for New or Reconstructed Pavement Design

The following major steps should be followed when designing a pavement structure for new alignment, reconstruction, or widening an existing pavement.

1. Select a "Trial Design." The ITD *AASHTOWare Pavement ME Design* procedure provides for the following new, reconstructed, or widening pavement designs:

    a. HMA Pavements of All Types (conventional, deep strength, full-depth).
    b. JPCP (with and without dowels).
    c. HMA Overlay Over Existing JPCP (composite pavement).
    d. JPCP Overlay Over Existing JPCP and Existing HMA Pavement.

    For both new designs and overlays, "Trial Design" must begin with the characterization of the subgrade M_r. Once the subgrade M_r is known, the designer must decide (for new pavements) what types of layers must be placed over the subgrade to obtain a feasible "Trial Design". Determining the feasible "Trial Design" requires significant engineering experience. The following is provided as guidance.

Idaho recommends special consideration for soils with low R-value (< 5). The design recommendations must address the isolated areas by either replacing the top 2 ft of subgrade with better material or providing an increased surfacing section for the areas having low M_r soils. In all cases, the appropriate recommendations for designs over such subgrade materials must be incorporated into the "Trial Design" structure and layer thicknesses (lime stabilization, placement of embankment, etc.). Although it is preferable to replace these soils, this is not always a viable option. In cases where the subgrade is identified as unable to support construction equipment, the material will either be replaced with better material or be treated in-place with hydrated lime, cement, or another material.

Use the existing ITD/AASHTO procedure or the experience of the designer as a starting point. For pavement widening or lane addition, using a "Trial Design" similar to the existing pavement section is a good starting point.

NOTE: The following minimum surface layer and base thicknesses in developing a "Trial Design".

Functional Class	Minimum Thickness (in.)				
	HMA	HMA Overlay over HMA	PCC	Base (ATB, ATPB, UTB)	Subbase (if used)
Interstates	6	3	9	4	4
All Other Routes	3	2	7	4	4

ATB Asphalt Treated Base ATPB Asphalt Treated Permeable Base UTB Untreated Base
NOTE: A surface treatment placed as part of design is not considered a structural layer.

2. **Select the appropriate performance criteria and design reliability level for the project.** See Chapters 3 and 4 of this *User's Guide* for guidance on these inputs.

3. **Obtain all inputs for the pavement design under consideration.** These inputs can be obtained using any of the three levels of effort depending on resources available for the project as defined in previous chapters.

4. **Run the *AASHTOWare Pavement ME Design* software and assess inputs and outputs**. It is recommended to make an initial run to check all inputs and outputs. After all inputs are correct, focus on optimizing the thickness and other design features using the optimization option in the software. For JPCP, both thickness and dowel diameter must be optimized together (e.g., thicker PCC slabs require larger diameter dowels).

See Appendix B for JPCP optimization rules. For flexible pavement, it is recommended to optimize only HMA thickness after selecting the base and subbase thicknesses.

a. **Examine carefully the Excel input data summary.** Ensure that the inputs are correct and are what the designer intended.

b. **Review the climatic outputs from PDF or Excel.** Many graphics are provided for quick review. Check the error list for reasonableness and for the five key hourly climate inputs (temperature, precipitation, percent sunshine, humidity, and wind speed).

c. **Review all of the traffic outputs.** Check the reasonableness of the number of trucks in the design lane, as tabulated in the distress output column, for the first month and the total trucks over design period in the design lane.

d. **Review all layer material moduli and other outputs.** Do this month-by-month over time to determine their reasonableness.

5. **Assess the** "Trial Design"**.** The ITD uses the following performance criteria to assess design reliability for new or reconstructed pavements. Other performance criteria are not to be used at this time.

   a. HMA: IRI, total rutting, alligator fatigue (bottom-up) cracking.
   b. JPCP: IRI, joint faulting, slab fatigue transverse cracking.
   c. HMA/JPCP: IRI, total rutting, slab fatigue transverse cracking.

   Has the "Trial Design" met each of the performance criteria at the design reliability level? If YES, then the design is nominally acceptable. If NO, the design is not acceptable and must be revised.

6. **Performance and reliability criteria met.** If the "Trial Design" meets the criteria, check to see if the reliability level is far above that required. If so, the "Trial Design" may be over-designed and could be reduced to a more optimum design.

7. **Performance and reliability criteria not met.** Determine how this design deficiency can be remedied by altering the materials used, layer thicknesses, and other design details. This requires knowledge of how various inputs affect performance outputs. Recommendations for optimizing "Trial Designs" are presented in Tables 57 and 58 for HMA pavements and JPCP, respectively. These recommendations also apply to HMA overlays over existing HMA pavements and JPCP. Chapter 11 of this *User's Guide* provides some information on how inputs affect performance.

8. **Revise "Trial Design" as needed.** Revise the inputs/"Trial Design" and rerun the program. Repeat until the reliability and performance criteria have been met. This design is then a feasible design for further consideration in the pavement selection process. However, since the *AASHTO Pavement ME Design* has not been specifically calibrated for Idaho conditions, designers should use engineering judgment when assessing the reasonableness of the design.

Table 57. Recommendations for Optimizing HMA Pavement Design

Issue	Recommendation/Description
Excessive HMA Rutting	• Increase the quality of the HMA layer. Use stiffer binder grade, reduce binder content, & reduce as placed (field compacted) air voids. • Enhance HMA mix stability (use crushed particles, increase nominal maximum aggregate size, etc.). • Majority of HMA rutting occur within the top 3 to 5 inches. Use of better quality HMA for the top 5 in. should improve rut resistance • Locally calibrate rutting model to Idaho NOTE: Most other State calibrations have shown that rutting is significantly over predicting.
Excessive Unbound Base & Subgrade Rutting	• Improve base material quality (M_r or R-value). • Place a thick (12 to 24 in.) embankment of superior material over the subgrade. • Increasing thickness of poor base/subgrade material will only tend to increase rutting, not decrease it. • Presence of excess moisture in any base or subgrade (particularly materials with high amount of fines) decreases M_r & increase rutting. This situation is improved by providing positive drainage.
Excessive Alligator Cracking	• Increase HMA thickness. Thicker HMA layers combined with high HMA stiffness (high E*) decreases critical tensile strains at the bottom of the HMA layer and increases resistance to alligator cracking. • If thin HMA layers are used, it is highly desirable to have a low stiffness (low E*). Thin, very stiff HMA layers have a high susceptible to alligator cracking • For HMA, increasing effective volume of bitumen & decreasing air voids results in significant increase in HMA fatigue life. • Ratio of the stiffness of the HMA (E*) & underlying unbound aggregate material modulus does influence alligator cracking. Thus any Pavement structural changes that reduce this ratio will significantly decrease the likelihood of fatigue damage. As a result, any changes that increase the base stiffness (i.e., by chemical stabilization; use of higher quality /stiffer layers; or increasing the thickness of high quality unbound base/subbase layers) will improve the alligator cracking Resistance.
Excessive Transverse “Thermal” cracking	• Thermal cracking is controlled by the HMA stiffness, tensile strength, & creep compliance all of which are highly influenced by HMA mix properties & binder grade. • Use of a less stiff binder grade is 1 way of decreasing transverse cracking. Other options include thicker HMA, decreased air voids, & increasing binder content
Construct the Pavement Very Smooth	• Smoothness specifications that offer significant incentives to build a smooth pavement are standard in many states

Table 58. Recommendations for Optimizing JPCP Design

Recommendation	Description
Include Dowels or Increase Dowel Diameter	The use of properly sized dowels is the most reliable & cost-effective way to control joint faulting.
Use a Treated Base (if non-stabilized dense graded aggregate was specified)	The treating of non-stabilized aggregate base with asphalt or cement will reduce the erosion potential of the base.
Widen the Conventional Traffic Lane Slab by 1 ft	Widening the slab effectively moves the wheel load away from the longitudinal free edge of the slab, thus, greatly reducing the critical bending stress and the potential for transverse cracking.
Decrease Joint Spacing	Reducing joint spacing is an effective means of reducing cracking & faulting, which directly affect pavement smoothness.
Increase Slab Thickness	Slab thickness affects slab cracking very significantly & faulting to a lesser extent. At some thickness, however, a point of diminishing returns is reached & fatigue cracking can no longer occur. Do not increase slab thickness to control faulting, specify dowels or larger diameter dowels.
Provide a PCC Shoulder (if AC shoulder was specified)	A tied PCC shoulder (especially those constructed monolithically with the mainline) provides better edge & corner support than an AC shoulder. Tied PCC shoulders reduce the deflection of the slab and the potential for erosion and pumping, especially for non-doweled pavements
Decrease Slab Permanent Curl/Warp	Permanent curl/warp increases voids under PCC slab corners & increases corner deflections. Depending on curing conditions, permanent curl/warp may either increase or decrease from mean conditions corresponding to the equivalent temperature gradient -10°F.
Decrease PCC Zero-Stress Temperature	Paving in hot weather may result in a high PCC zero-stress temperature. That may lead to high joint opening, accelerated loss of aggregate shear capacity, & low load transfer efficiency.
Construct the Pavement Very Smooth	Smoothness specifications that offer significant incentives to build a smooth pavement are standard in many states

10.2 Steps Required for Rehabilitation Pavement Design

The following steps should be followed in designing rehabilitation for an existing Pavement:

1. **Select a "Trial Design".** The *AASHTOWare Pavement ME Design* procedure provides for the following rehabilitated designs:

    a. HMA Rehabilitation - HMA overlay on existing HMA.
    b. HMA Rehabilitation - HMA on existing JPCP.

2. **Select the Appropriate Performance Criteria and Design Reliability Level for the Project.** See Chapters 3 and 4 for guidance on these inputs.

3. **Obtain All Inputs for the Existing Pavement and Rehabilitation Design Under Consideration.** These inputs can be obtained using any of the 3 levels of effort depending on resources available for the project as defined in Chapters 5 through 7 of this *User's Guide*. Chapter 9 of this *User's Guide* explains the various input levels available for rehabilitation.

   a. **Collect As-Built Design and Materials Data.** Layer thicknesses and materials information are critical inputs that must be obtained either from historical records or from boring and coring the pavement.

   b. **Conduct a Condition Survey.** The following are minimum distress inputs required for each type of existing pavement. Detailed recommendations are provided in Chapter 9 for each rehabilitation level.

      i. **Existing HMA**: percent area alligator (fatigue) cracking in the wheelpaths, and mean rutting in the wheelpaths.

      ii. **Existing JPCP**: percent slabs transverse (fatigue) cracking.

   Pavement condition information can be obtained from the ITD TAMS database. Condition information provided includes IRI, transverse profile, and photos of the pavement surface, etc.

   c. **Conduct Other Testing.** Additional testing that may be needed includes the following:

      i. **FWD** testing along the project can provide the best estimate of the subgrade support for rehabilitation design for both HMA and PCC pavements. Back-calculated values for other layers, including existing HMA, aggregate base, and PCC, can also be obtained through back-calculation.

      ii. **Coring and Boring** at selected locations along the project can provide some important details, including layer thicknesses (critical for back-calculation) and material properties. In addition, cores can reveal material durability problems (e.g., stripping of asphalt) for some materials.

      iii. **Profile Measurements** to identify any significant heaves or settlements that may need pre-rehabilitation treatment.

Run the *AASHTOWare Pavement ME Design* Software and Assess Inputs and Outputs. It is recommended to make an initial run to check all inputs and outputs. After all inputs are correct, focus on optimizing the thickness and other design features using the optimization option in the software.

a. **Examine Carefully the Excel Input Data Summary.** Ensure that the inputs are correct and are what the designer intended.

b. **Review the Climatic Outputs from PDF or Excel.** Many graphics are provided for quick review. After the program runs, check the error list for reasonableness and for the five key hourly climate inputs (temperature, precipitation, percent sunshine, humidity, and wind speed).

c. **Review All of the Traffic Outputs.** Check the reasonableness of the number of trucks in the design lane, as tabulated in the distress output column, for the first month and the total trucks over the design period in the design lane.

d. **Review All Layer Material Moduli and Other Outputs.** Do this month-by-month over time to determine their reasonableness.

5. **Assess the Trial Rehabilitation Design.** ITD uses the following performance criteria to assess design reliability for rehabilitated pavements. Other performance criteria are not to be used at this time.

   a. **HMA Overlay of Existing HMA**: IRI, total rutting, total cracking (reflective alligator cracking from existing pavement plus alligator fatigue (bottom-up) cracking from overlay).

   NOTE: The total cracking can only be assessed at 50 percent reliability at the current time.

   b. **HMA Overlay of Existing JPCP**: IRI, total rutting, slab fatigue transverse cracking.

   Has the "Trial Design" met each of the performance criteria at the design reliability level? If YES, the design is nominally acceptable. If NO, the design is not acceptable and must be revised.

6. **Performance and Reliability Criteria Met.** If the "Trial Design" meets the performance and reliability criteria, check to see if the reliability level is far above that required. If so, the "Trial Design" may be over-designed and could be reduced to a more optimum design.

7. **Performance and Reliability Criteria Not Met.** Determine how this design deficiency can be remedied by altering the materials used, layer thicknesses, and other design details. This requires knowledge of how various inputs affect performance outputs. (see Tables 57 and 58).

8. **Revise "Trial Design" As Needed.** Revise the inputs/trial design and rerun the program. Repeat until the reliability and performance criteria have been met. This design is then a feasible rehabilitation design for further consideration in the pavement selection process.

10.3 Local Calibration Factors for Idaho

Local calibration and validation has not been conducted to date for Idaho HMA and JPCP or for the overlays of these pavements. To ensure that the design inputs are reasonable for Idaho conditions and that the distress and IRI models were unbiased (on average did not over- or under-predict rutting, fatigue cracking, or IRI), it is highly recommended to conduct a local calibration.

When completed, the Idaho-specific local calibration coefficients will be entered into the most current version of *AASHTOWare Pavement ME Design*. Idaho designers should always check to make sure that they are using the Idaho local coefficients in their designs. The coefficients used are always output with every run of the software and located under the Calibration tab in the Excel output and at the end of the PDF output file.

Temporarily, the following local calibration coefficients from Wyoming are provided in Figures 32 through 33 for new HMA and HMA/HMA overlays. Calibration coefficients from NCHRP 20-07/Task 288 *National Recalibration of MEPDG Concrete Models Based on CTE Values* are provided in Figure 34 for JPCP. Coefficients may be changed in the future as the DOT conducts additional local calibration efforts over time.

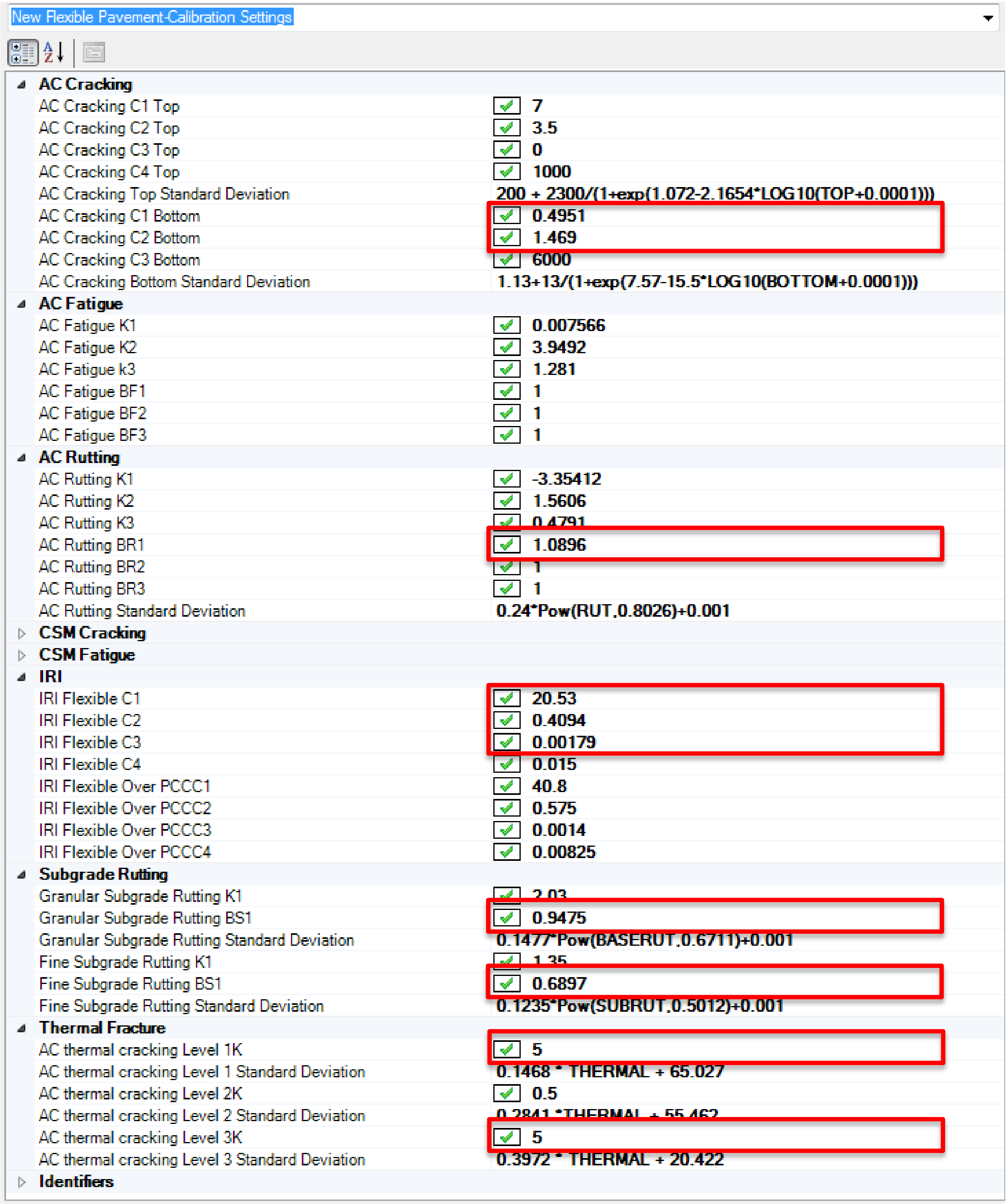


Figure 32. Wyoming Calibration Coefficients and Standard Error Prediction Models for New HMA Pavement

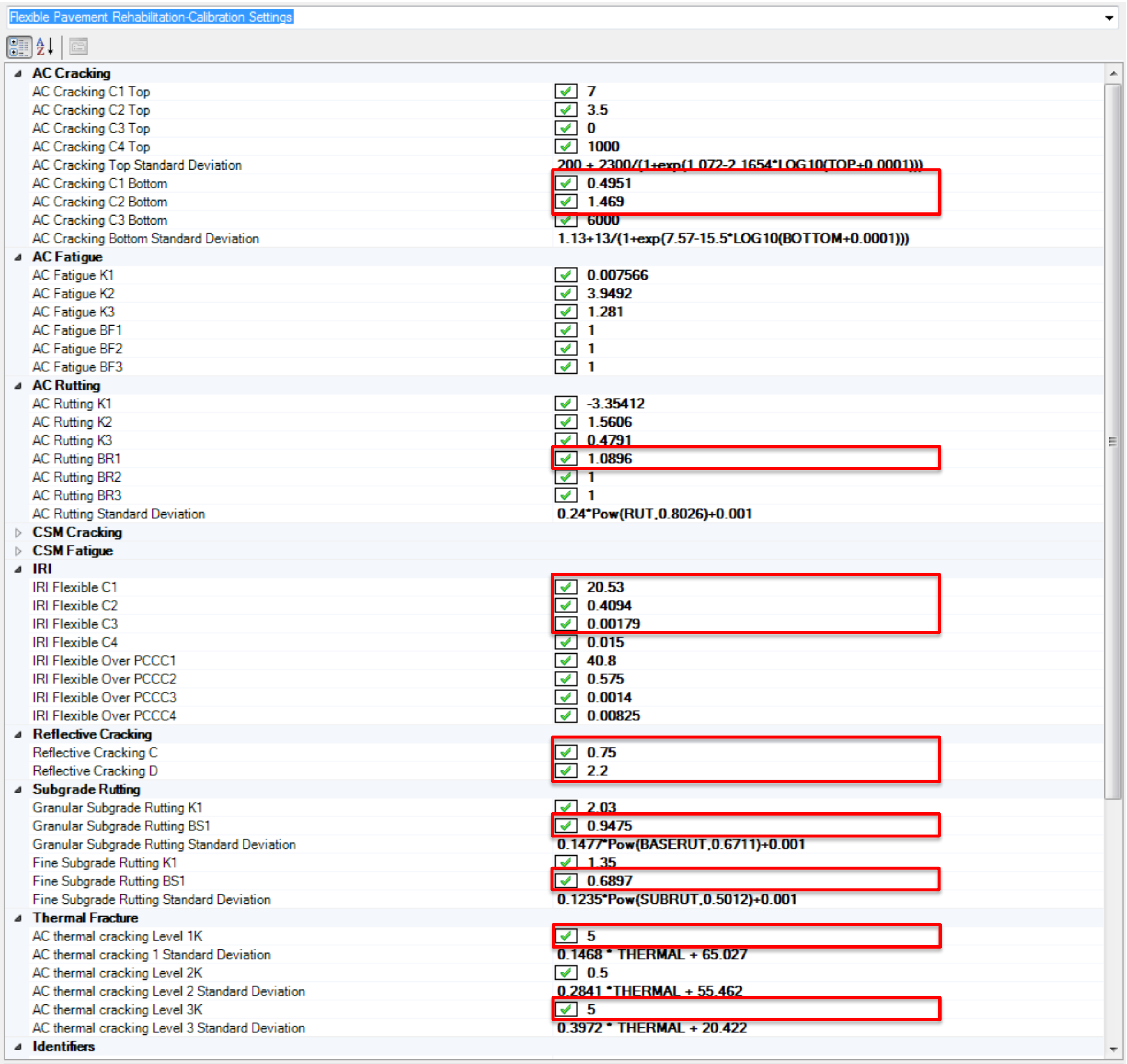


Figure 33. Wyoming Calibration Coefficients and Standard Error Prediction Models for HMA Over Existing HMA Pavement

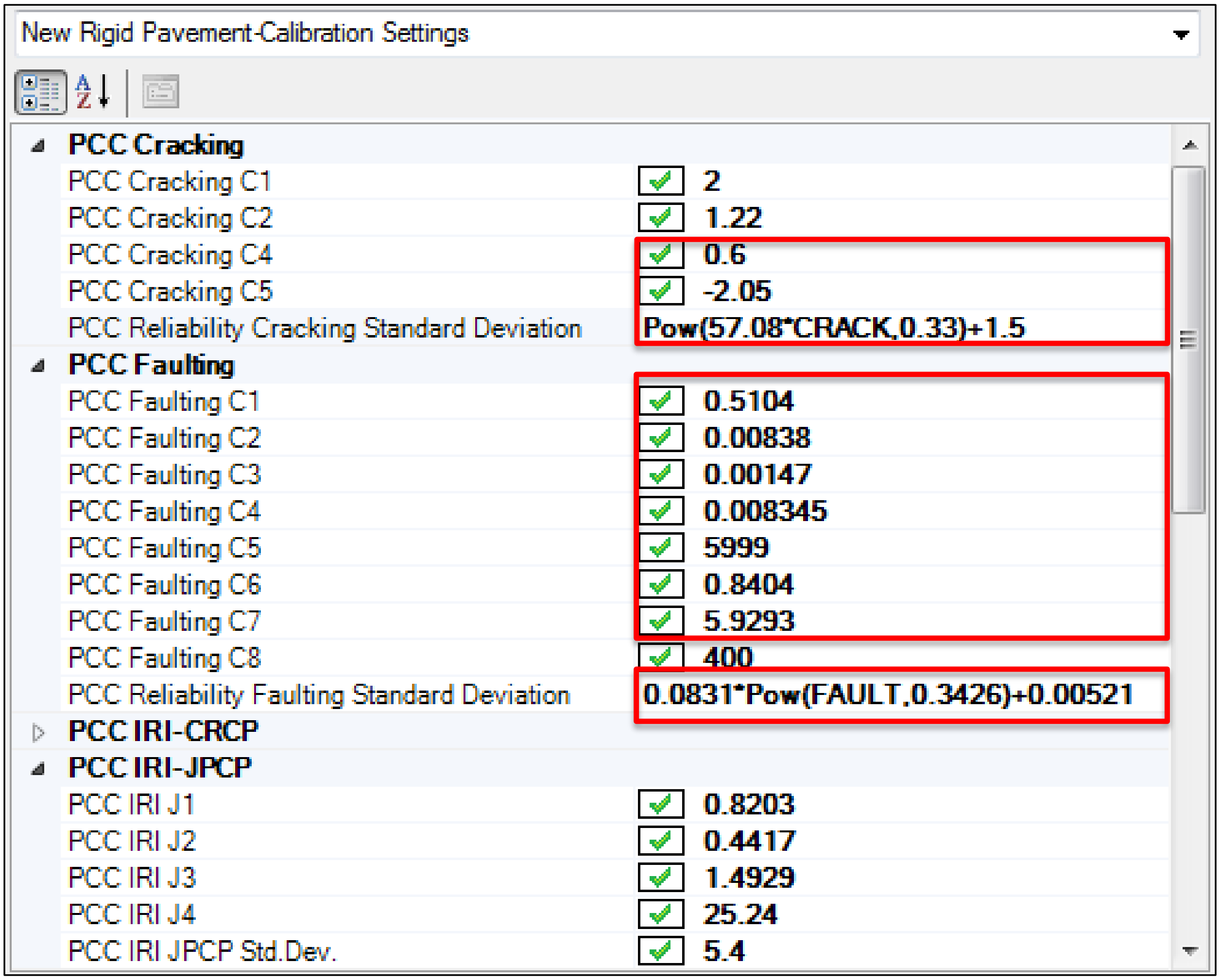


Figure 34. NCHRP 20-07 Calibration Coefficients and Standard Error Prediction Models for New JPCP

Chapter 11
Input/Output Sensitivity Analysis

Sensitivity analysis was conducted to determine the impact of inputs on predicted pavement performance. Knowledge of these effects will help designers to improve their trial designs to meet the performance criteria.

Table 59 shows overall results for HMA pavements and Table 60 shows overall results for JPCP. Figures 35 through 44 show the effects of various factors on predicted HMA pavement performance. Figures 45 through 51 show the effect of various factors on predicted JPCP performance.

Table 59. Sensitivity Results for New/Reconstructed HMA Pavements

Design/Material Variable	Distress/Smoothness			
	Alligator Fatigue Cracking	Rutting	Transverse Cracking	IRI
HMA Thickness	XXX	XX	X	XX
Tire Load, Contact Area, & Pressure	XX	XXX		XX
HMA Tensile Strength			XXX	
HMA Coefficient of Thermal Contraction			XX	
Mixture Gradation	XX	XXX		
HMA Air Voids In Situ	XXX	XX	XX	XX
Effective HMA Finder Content	XXX	XX	XX	X
HMA Binder Grade	XX	XX	XXX	XXX
Existing HMA Condition	XXX			X
Bonding with Base	XXX	X		
Base Modulus	XXX	XX		
Base Thickness	X			
Subgrade Modulus	XX	XX		
Groundwater Table	X	X		
Climate	XX	XX	XXX	X
Truck Volume	XXX	XXX		
Truck Axle Load Distribution	XX	XX		
Truck Speed	XX	XXX		
Truck Wander	XX	XX		
Initial IRI				XXX

X Factor has small effect on distress/IRI
XX Factor has moderate effect on distress/IRI
XXX Factor has large effect on distress/IRI

Table 60. Sensitivity Results for New/Reconstructed JPCP and Composite Pavements

Design/Material Variable	Distress/Smoothness		
	Transverse Joint Faulting	**Transverse Cracking**	**IRI**
PCC Thickness	XX	XXX	XXX
PCC Modulus of Rupture & Elasticity		XXX	XX
PCC CTE	XXX	XXX	XXX
Existing PCC Condition		XXX	X
PCC Unit Weight	X	XX	X
Joint Spacing	XX	XXX	XX
Joint LTE	XXX		XXX
Edge Support*	XXX	XXX	XX
Permanent Curl/Warp	XXX	XXX	XXX
Zero-Stress Temperature	XX		X
Friction Between Slab & Base		XXX	XX
Base Type	XXX	XX	X
Climate	XX	XX	XX
Subgrade Type/Modulus	X	XX	X
Groundwater Table	X	X	X
Truck Speed		X (with HMA base only)	
Truck Axle Load Distribution	X	XXX	X
Truck Volume	XXX	XXX	XXX
Tire Pressure		X	
Truck Lateral Offset	XX	XXX	XX
Truck Wander		XX	X
Initial IRI			XXX

X Factor has small effect on distress/IRI.
XX Factor has moderate effect on distress/IRI.
XXX Factor has large effect on distress/IRI.
* Free edge vs. tied shoulder vs. widened slab.

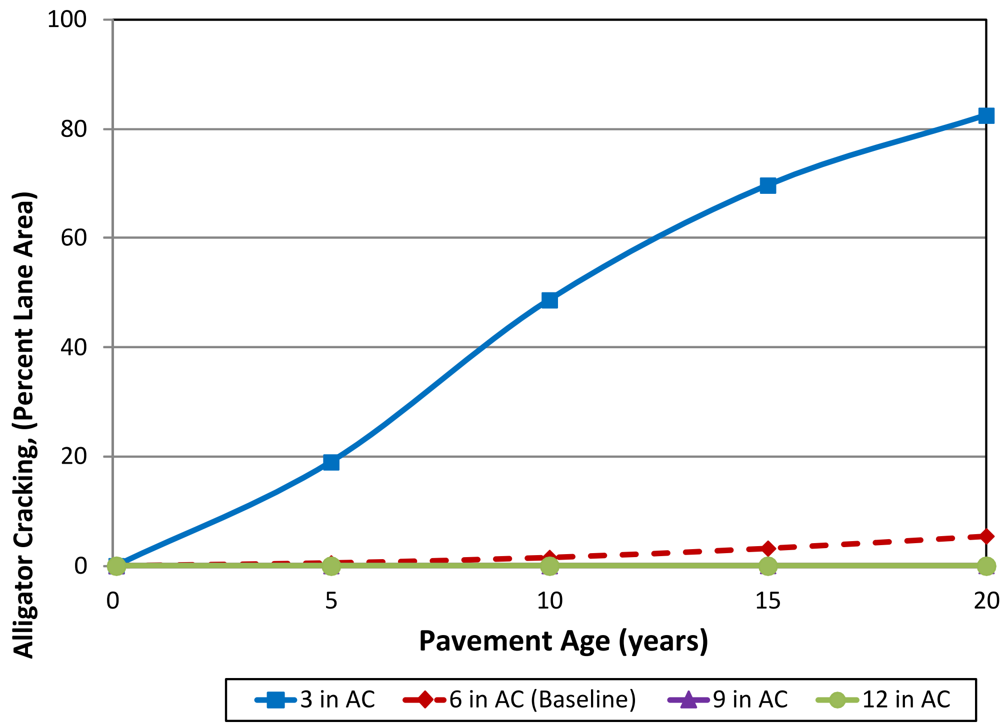


Figure 35. Effect of HMA Thickness on HMA Bottom-Up Alligator Fatigue Cracking

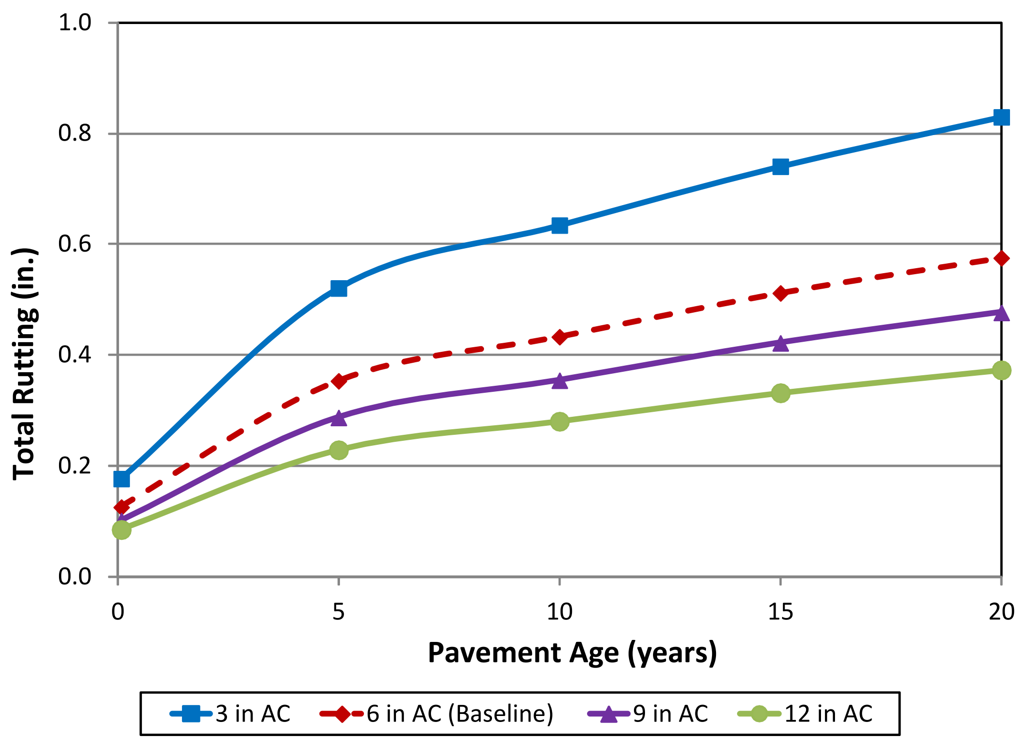


Figure 36. Effect of HMA Thickness on Rutting

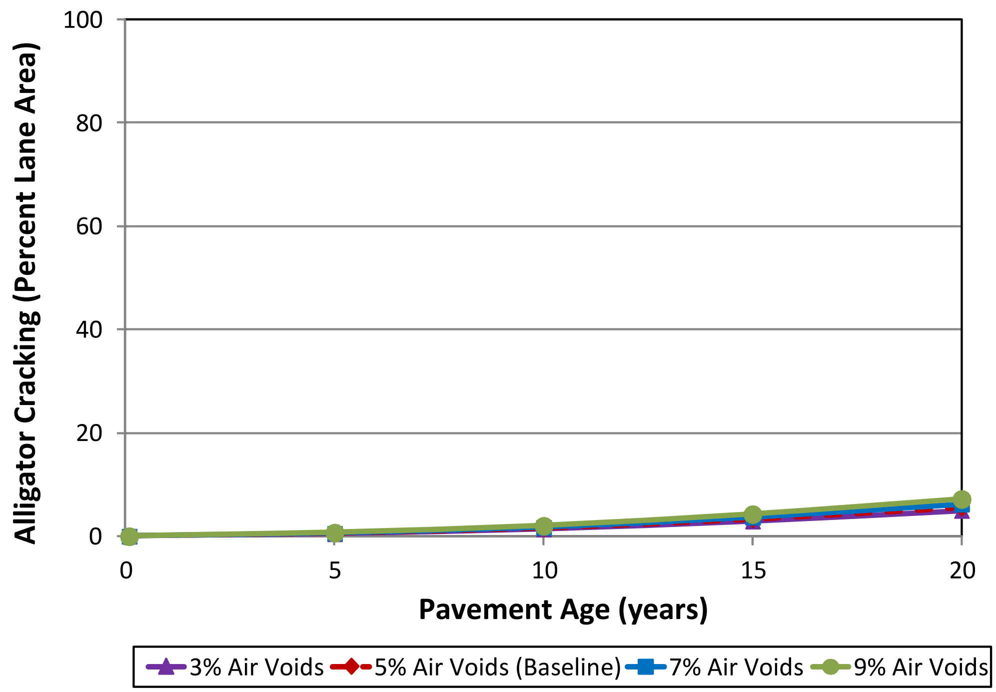


Figure 37. Effect of HMA In Situ Air Void Content on Fatigue (Alligator) Cracking

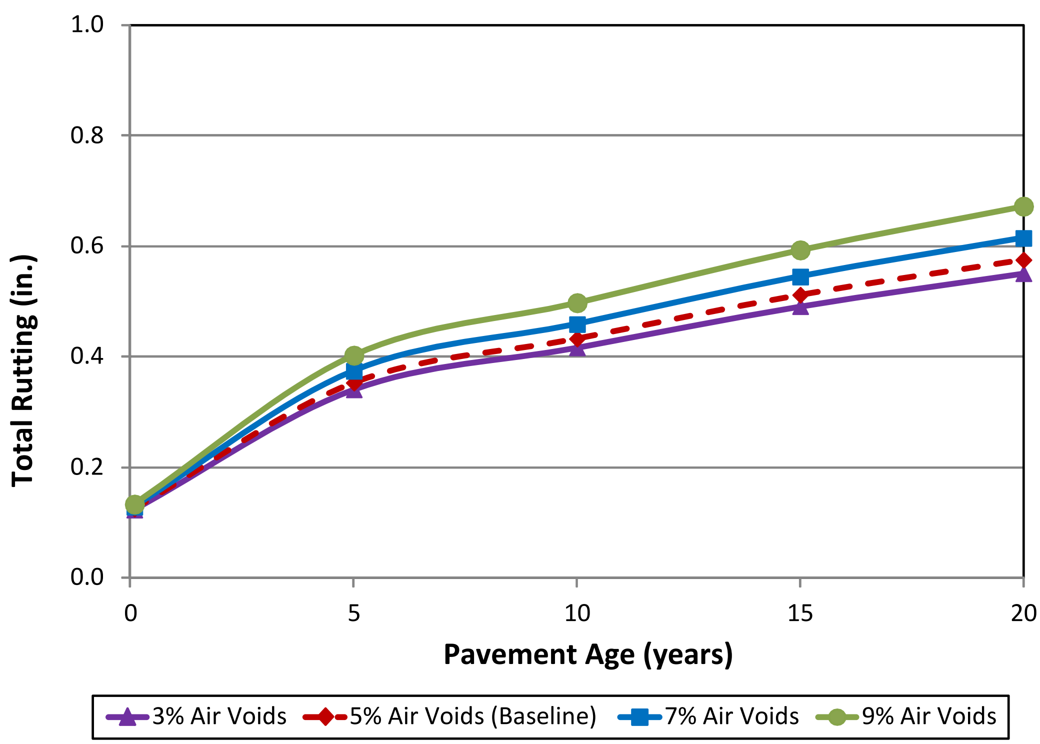


Figure 38. Effect of HMA In Situ Air Void Content on Rutting

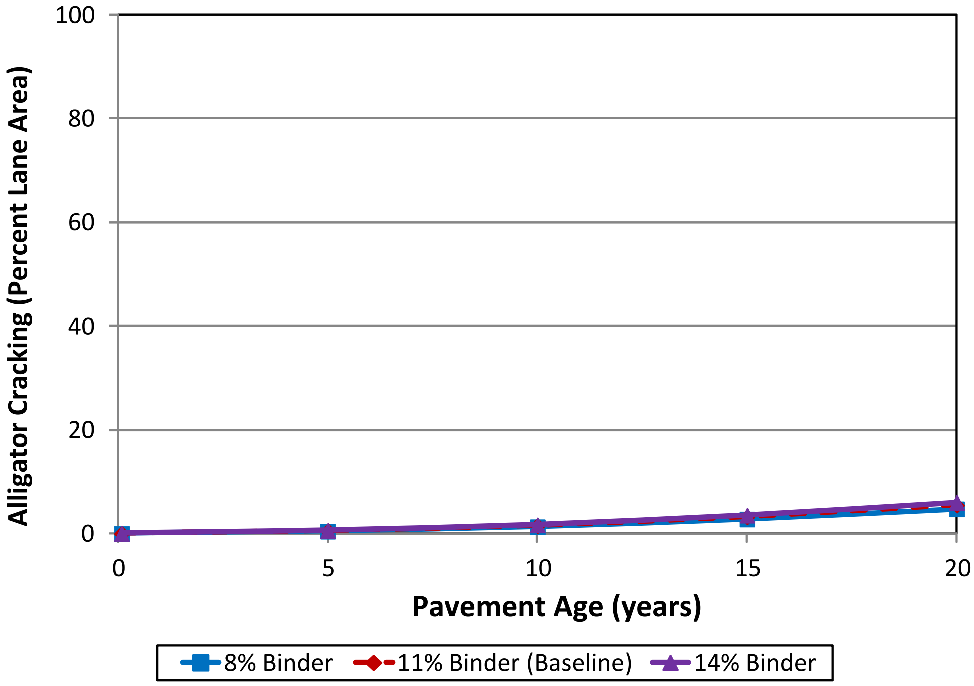


Figure 39. Effect of HMA Volumetric Binder Content on Fatigue (Alligator) Cracking

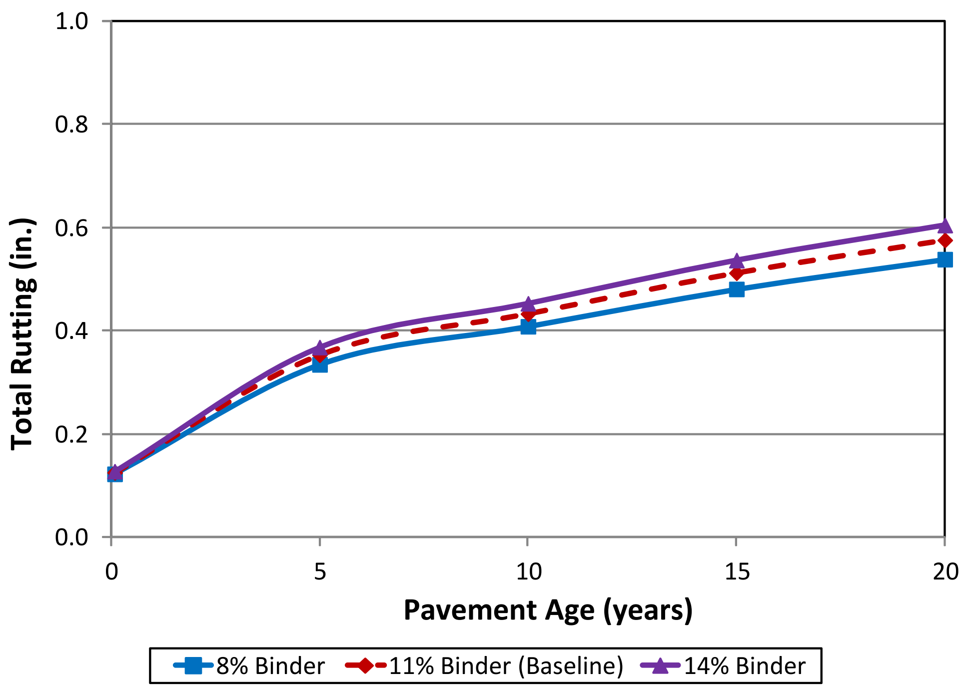


Figure 40. Effect of HMA Volumetric Binder Content on Rutting

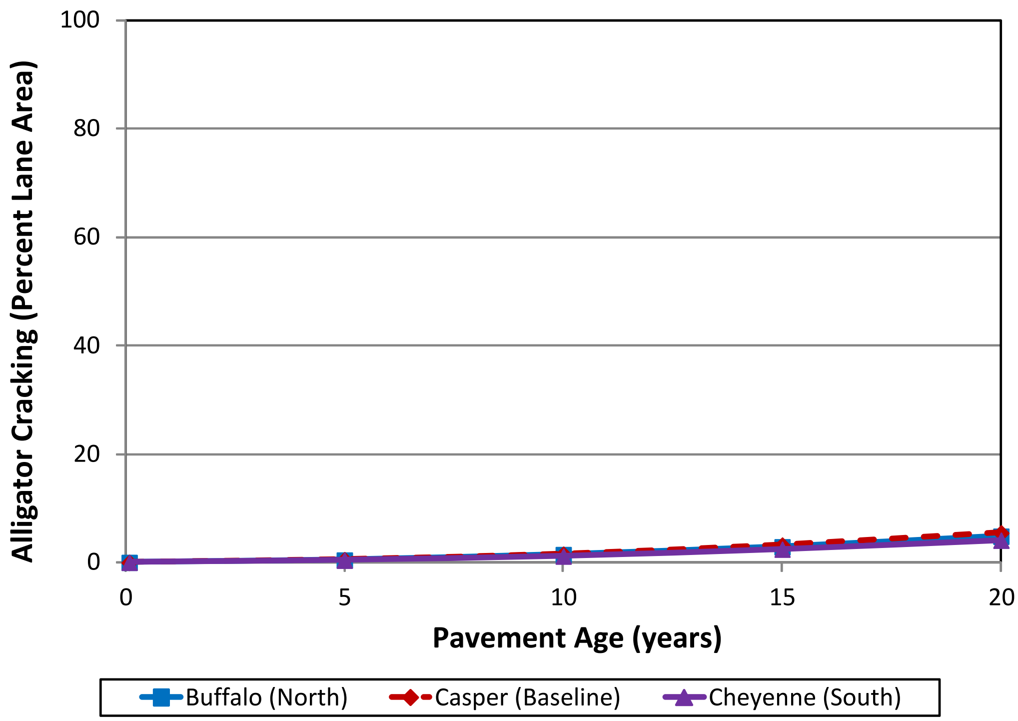


Figure 41. Effect of Climate on Fatigue (Alligator) Cracking

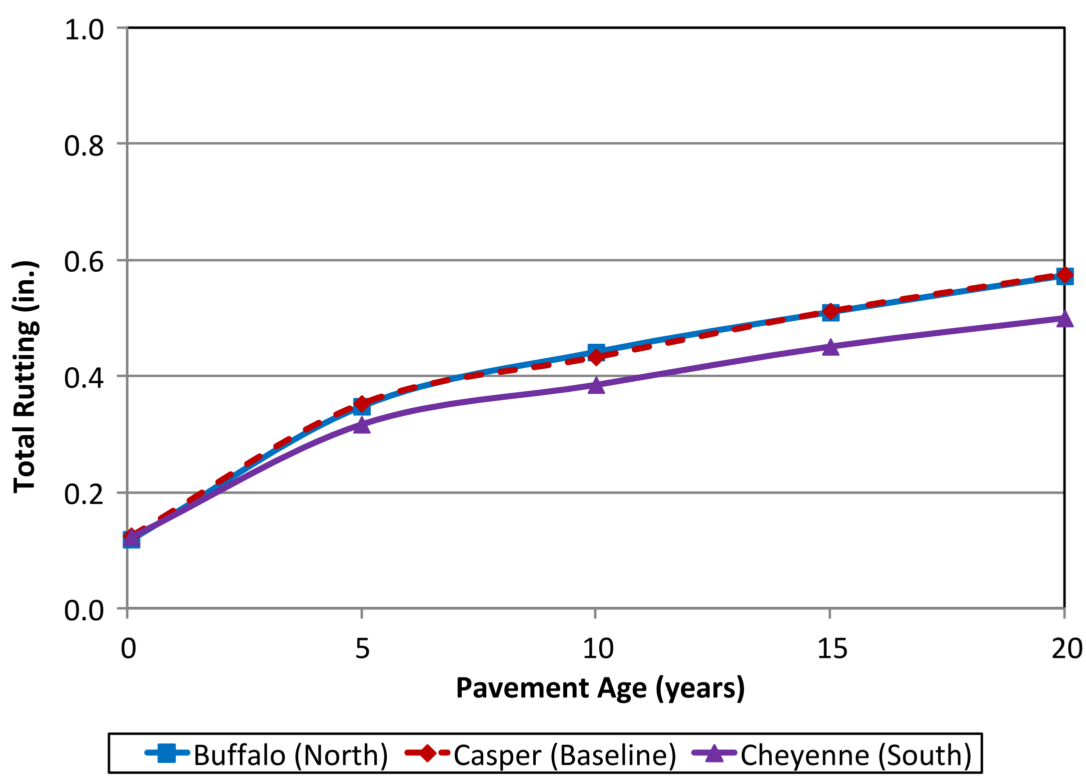


Figure 42. Effect of Climate on Rutting

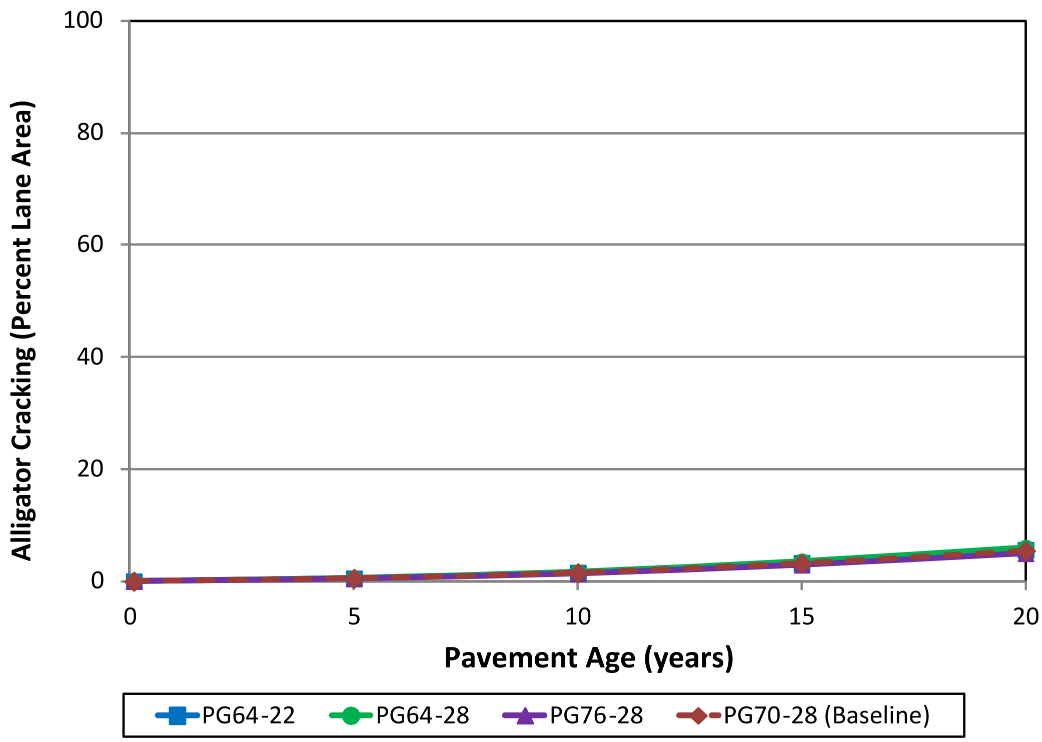


Figure 43. Effect of HMA Binder Grade on Alligator Cracking

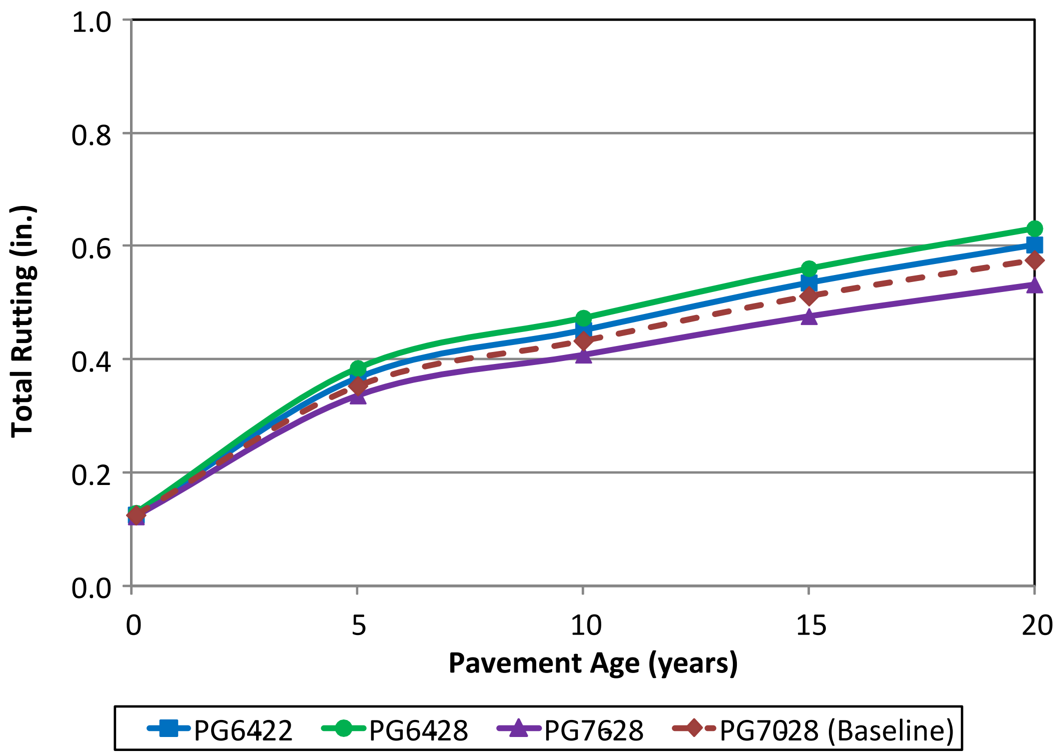


Figure 44. Effect of HMA Binder Grade on Rutting

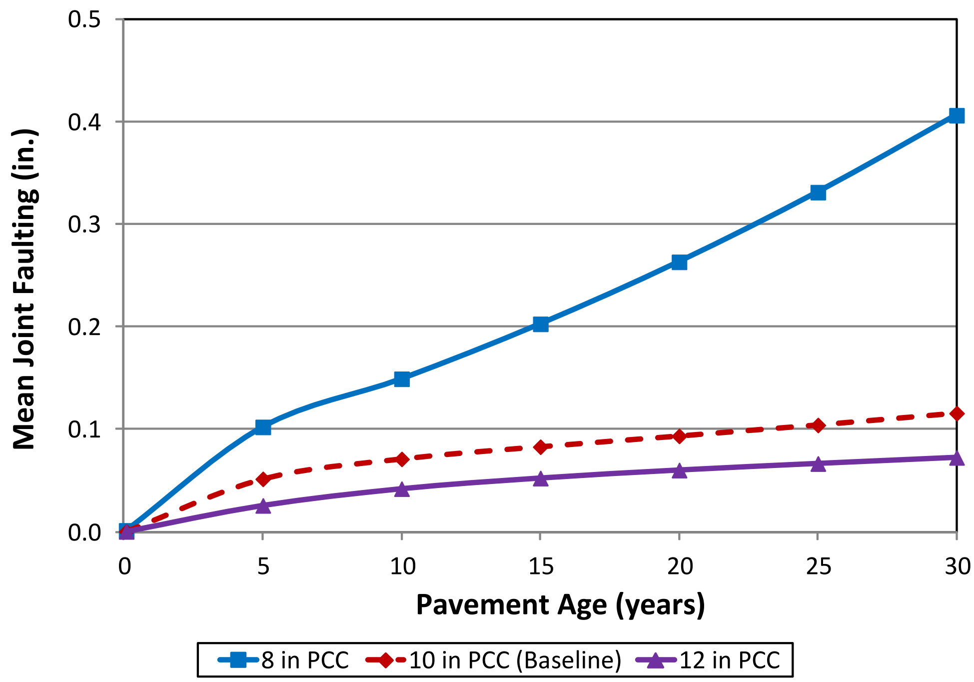


Figure 45. Effect of JPCP Transverse Joint Dowel Diameter and PCC Thickness on Joint Faulting

NOTE: That dowel diameter = PCC thickness, (in inches), divided by 8.

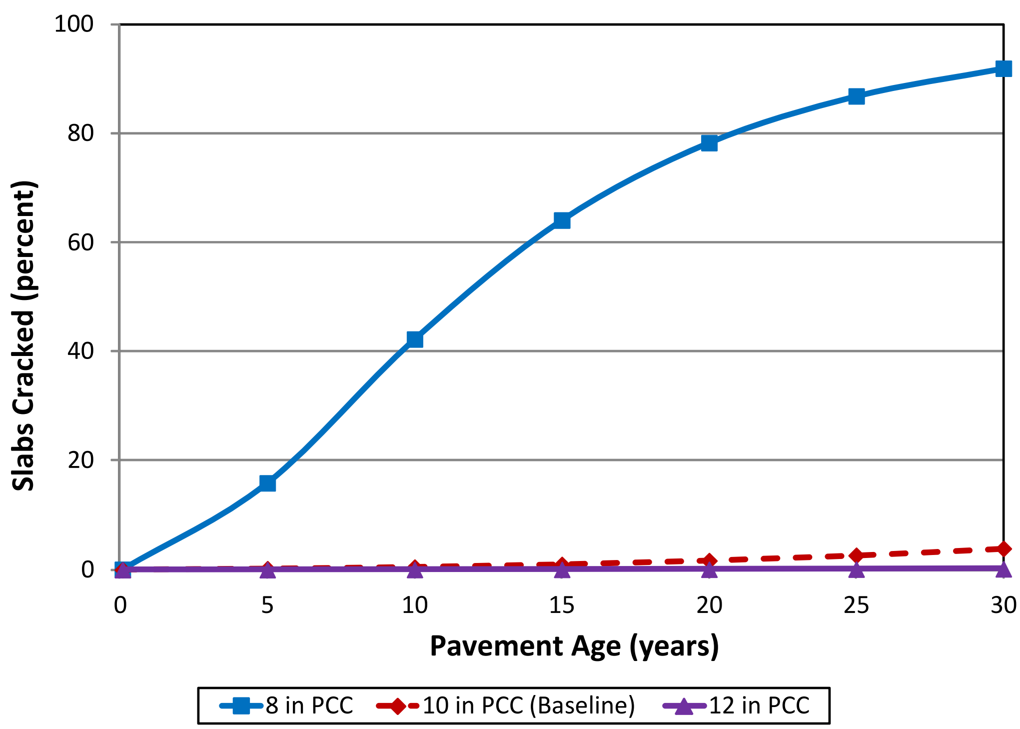


Figure 46. Effect of PCC Slab Thickness on Transverse Cracking of JPCP

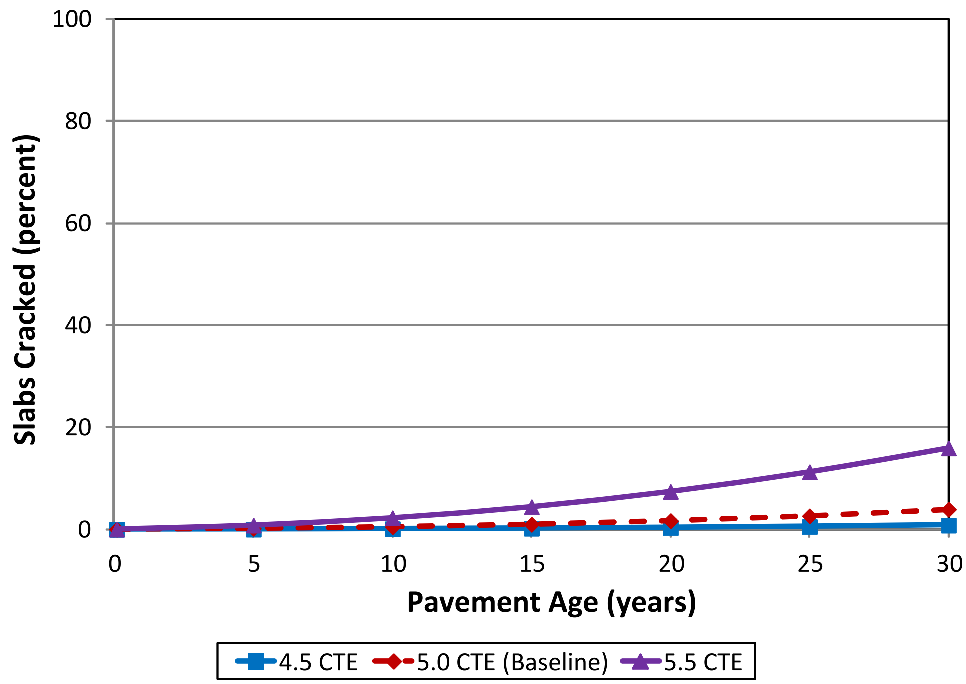


Figure 47. Effect of PCC Coefficient of Thermal Expansion on Transverse Cracking of JPCP

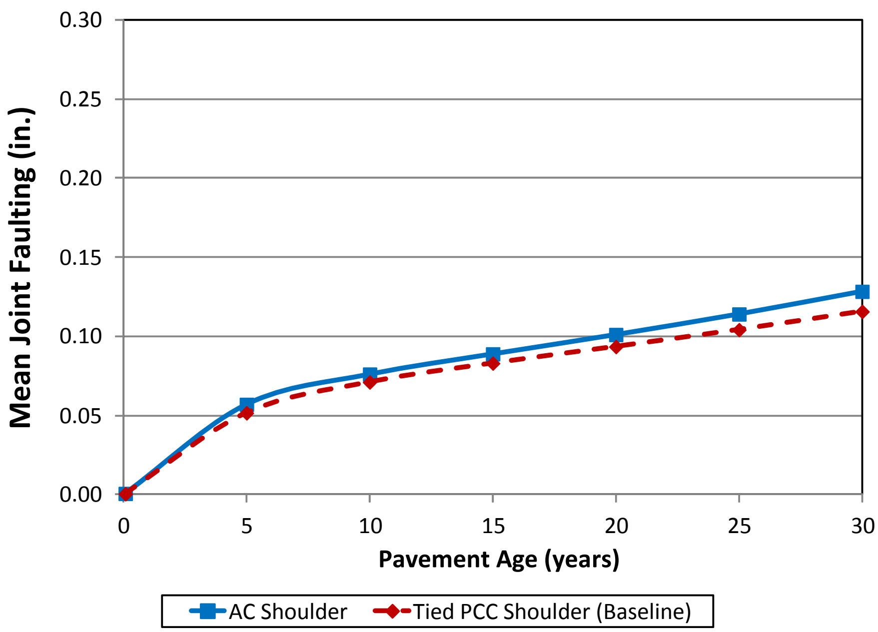


Figure 48. Effect of Shoulder Support on Transverse Joint Faulting of JPCP

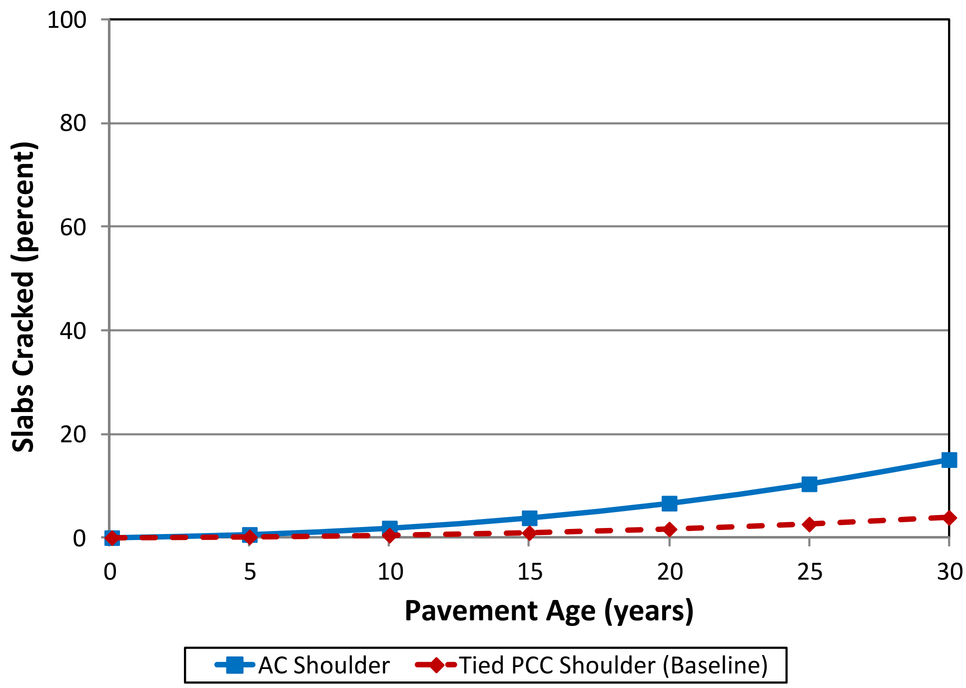


Figure 49. Effect of Edge Support on Slab Transverse Cracking for JPCP

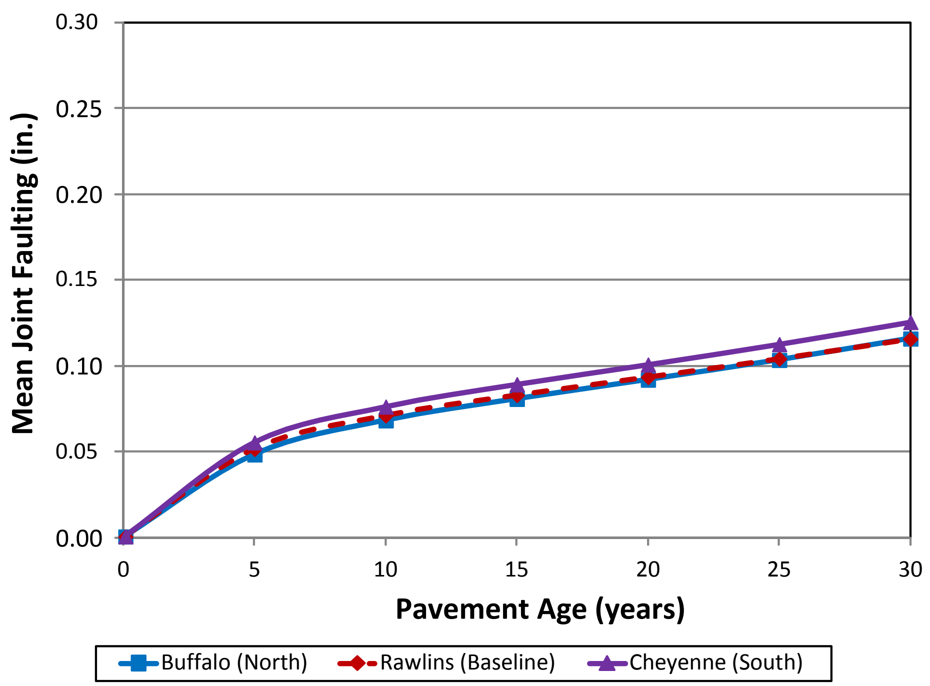


Figure 50. Effect of Climate on Slab Transverse Joint Faulting for Doweled JPCP

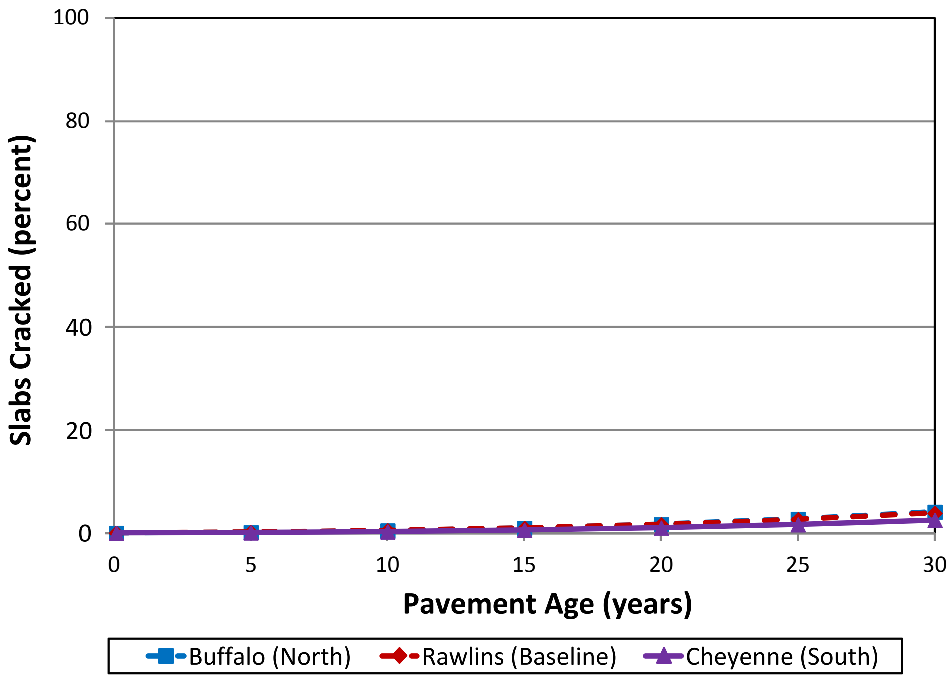


Figure 51. Effect of Climate on Slab Transverse Cracking for JPCP

CHAPTER 12
AASHTOWare Pavement ME Design Outputs Used for Performance Assessment

The *AASHTOWare Pavement ME Design* software analyzes a given trial design and predicts its performance in terms of key distress types and smoothness. Materials properties and other factors are output on a month-by-month basis over the design period. Each pavement type and rehabilitation type has its own specific output tables and charts. The designer examines the output materials properties and other factors to see if reasonable results are being obtained. *AASHTOWare Pavement ME Des*ign provides two different output documents:

- PDF Summary: Provides a summary of key design outputs.
- Excel Summary: Provides detailed tables and graphics of design inputs and outputs (when more detail is needed to check or analyze a design).

For asphalt pavements, the output provides the HMA E* and M_r for unbound layers for each month over the design period. Vehicle speed and temperature affect the HMA material E* greatly. Moisture content and frost conditions affect the unbound materials and soils M_r greatly. The designer can observe these and assess their reasonableness.

For concrete pavements, the output provides the PCC modulus of rupture and modulus of elasticity for each month over the design period. The base course modulus is also output monthly over the design period. The back-calculated subgrade dynamic k-value is also output monthly. Load Transfer Efficiency (LTE) at joints is also output. Use a larger dowel if the LTE drops below 70 percent. Moisture content and frost conditions affect the unbound materials M_r and k-value greatly. The designer can observe these and assess their reasonableness.

The designer must examine the key distress type outputs and smoothness to see if they are meeting the performance criteria. The distress and IRI are output at the end of each month over the design period. The number of cumulative heavy trucks (Class 4 and above) is also shown for the design traffic lane.

Examples of *AASHTOWare Pavement ME Design* outputs for new HMA pavement and new JPCP analysis are presented in Tables 61 through 64 at the end of this chapter. The red horizontal line for all distress/IRI plots represents the limiting performance criteria at a given level of reliability. The design is acceptable if distress/IRI at the specified reliability is lower than the red line over the entire design period.

Another method for assessing design adequacy is to review the reliability output. The distress target and its corresponding reliability target are the first right-hand columns, followed by the distress predicted and the reliability predicted. The pavement passes if the reliability predicted is greater than the reliability target, and the pavement fails if the reverse is true. The designer must alter the "Trial Design" to correct the problem if any key distress fails. This "trial and error" process allows the pavement designer to "build" the pavement in the software prior to building it in the field to determine if it will perform.

The *AASHTOWare Pavement ME Design* software also has limited optimization capabilities that iterate on HMA or PCC thickness until an acceptable design is achieved. Problems with the design and materials for the given subgrade, climate, and traffic can be corrected and an early failure avoided. This is the power of the *AASHTOWare Pavement ME Design* methodology.

New HMA and Rehabilitation with HMA

AASHTOWare Pavement ME Design creates an Excel output file with the following worksheets (key HMA output file tabs):

- **Grand Summary:** A few key design, climate, and traffic inputs and distress outputs showing reliability and if performance criteria were satisfied.
- **Traffic Input Charts:** Graphical representation of key traffic inputs, including vehicle class distribution, traffic growth, axles/truck, and monthly distribution factor.
- **Traffic Distributions:** Tabular representation of monthly adjustment factors, vehicle class distribution, and hourly truck distribution.
- **Axle Configurations:** Tabular traffic wandering, average axle spacing, axle configuration, truck wheelbase, and axles per truck.
- **AADTT Growth:** Graphical representation of the growth of trucks over the design analysis period.
- **AADTT Growth by Class:** Tabular truck growth by class over the design analysis period.
- **Design Properties:** Tabular HMA design properties.
- **Climatic Inputs:** Graphical displays of climate stations used, annual statistics, precipitation, temperature, percent sunshine, relative humidity, wind speed, frost penetration, and number of wet days.
- **HMA Layer 1 Master Curve Inputs:** Parameters for master curve plot and statistics of top HMA layer.
- **HMA Layer 1 Shift Curve Inputs:** Inputs for shift factor curve for top HMA layer.
- **HMA Layer 1 Viscosity Curve Inputs:** Parameters of plot for top HMA layer.
- **HMA Layer 2, 3, etc.**: Similar inputs/outputs for each HMA layer.

- **Distress Charts:** Graphical displays for IRI, rutting, thermal crack total length versus time, total (fatigue) cracking (reflective alligator plus alligator fatigue cracking), thermal crack spacing, and thermal crack depth.
- **Fatigue Charts:** Graphical displays for top-down and bottom-up fatigue damage and cracking.
- **Rutting Charts:** Graphical display of rutting of all layers at 50 percent reliability level.
- **Sublayer Modulus Charts:** Graphical display of HMA, base, and subgrade moduli.
- **AC Thermal Cracking:** Tables and graphics of data and plots related to low-temperature thermal cracking.
- **Calibration Coefficients:** Coefficients used in the current run of the software for all distresses and IRI.
- **Layer 1, 2, etc.:** Inputs associated with each layer beginning from the top of the pavement structure to the subgrade or bedrock.
- **Distress Data:** Month-by-month summary of key performance outputs, including time since opening to traffic, cumulative number of heavy trucks in the design lane, transverse low-temperature crack depth and spacing, IRI, permanent total deformation (mean rutting both wheel paths), AC thermal fracture (low-temperature transverse cracking), total cracking (reflective cracking from existing HMA plus overlay bottom-up alligator cracking**—**this is all fatigue bottom-up cracking; for new design there is no reflective cracking), and the same distresses at the selected level of design reliability.
- **Sublayer Modulus Data:** Table of modulus data for each sublayer over each month of the entire design analysis period.
- **Fatigue Data:** Table of predicted AC fatigue damage and cracking, both top-down and bottom-up.
- **Rutting Data:** Table of predicted mean permanent deformation in AC layer, base layer, subgrade, and total (at surface).

New JPCP and Rehabilitation with JPCP (including CPR)

AASHTOWare Pavement ME Design creates an Excel output file with the following worksheets (key JPCP output file tabs):

- **Grand Summary:** A few key design, climate, and traffic inputs and distress outputs showing reliability and if performance criteria were satisfied.
- **Traffic Input Charts:** Graphical representation of key traffic inputs, including vehicle class distribution, traffic growth, axles/truck, and monthly distribution factor.

- **Traffic Distributions:** Tabular representation of monthly adjustment factors, vehicle class distribution, and hourly truck distribution.
- **Axle Configurations:** Tabular traffic wandering, average axle spacing, axle configuration, truck wheelbase, and axles per truck.
- **AADT Truck Growth:** Graphical representation of the growth of trucks over the design analysis period.
- **AADT Truck Growth by Class:** Tabular truck growth by class over the design analysis period.
- **Design Properties:** Tabular JPCP design features, including joint spacing, dowel design, shoulder type, PCC slab/base friction permanent curl/warp effective temperature difference, and erodibility index of base.
- **Climatic Inputs:** Graphical displays of climate stations used, annual statistics, precipitation, temperature, percent sunshine, relative humidity, wind speed, frost penetration, and number of wet days.
- **Distress Charts:** Graphical displays for IRI, transverse joint faulting, and transverse fatigue cracking.
- **PCC Strength, Modulus Charts:** Graphical displays over time of PCC modulus of elasticity, flexural strength, base modulus, and subgrade dynamic k-value.
- **PCC Damage, LTE Charts:** Graphical displays of cumulative fatigue damage at top and bottom of PCC slab and transverse joint LTE.
- **Calibration Coefficients:** Coefficients used in the current run of the software for all distresses and IRI.
- **Layer 1, 2, etc.:** Inputs associated with each layer beginning from the top of the pavement structure down to the subgrade or bedrock.
- **Distress Data:** Month-by-month summary of key performance outputs, including time since opening to traffic, cumulative number of heavy trucks in design lane, IRI, transverse joint faulting, transverse fatigue slab cracking; and the same IRI and distresses at the selected level of design reliability.
- **PCC Strength and Modulus Data:** Table of month-by-month cumulative heavy trucks in design lane, PCC modulus of elasticity, PCC flexural strength, base modulus, and dynamic subgrade k-value over each month of the entire design analysis period.
- **Faulting Data:** Table of various details of predicted transverse joint faulting-related data and joint opening and LTE over each month of the entire design analysis period.
- **Cracking Data:** Table of predicted fatigue damage data, both top-down and bottom-up over each month of the entire design analysis period.

Table 61. Excel Distress Summary Showing IRI, Permanent Deformation, AC Thermal Fracture, Total Cracking (Alligator Reflective and Bottom-Up Alligator) for New/Reconstructed HMA

Predicted Distress

			Mean Predicted Distress					Predicted Distress @ Reliability		
Month	Pavement Age (years)	Heavy Trucks (cum.)	IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal fracture (ft/mile)	Total Cracking (Reflective + Alligator)	Reflective Cracking (%)	IRI (in/mi)	Permanent deformation - total pavement (in.)	AC thermal fracture (ft/mile)
10/2013	0.08	88,515	45.1	0.039	0.00	0.000	0.000	61.8	0.163	39.41
11/2013	0.17	198,688	45.1	0.043	0.00	0.000	0.000	61.8	0.169	39.41
12/2013	0.25	308,301	45.1	0.044	0.00	0.000	0.000	61.9	0.170	39.41
1/2014	0.33	383,613	45.2	0.045	0.00	0.000	0.000	62.0	0.172	39.41
2/2014	0.42	464,556	45.2	0.046	0.00	0.000	0.000	62.1	0.173	39.41
3/2014	0.50	547,815	45.3	0.048	0.00	0.000	0.000	62.2	0.176	39.41
4/2014	0.58	628,742	45.3	0.055	0.00	0.000	0.000	62.3	0.186	39.41
5/2014	0.67	710,437	45.4	0.068	0.00	0.000	0.000	62.4	0.205	39.41
6/2014	0.75	796,180	45.5	0.082	0.00	0.000	0.000	62.6	0.224	39.41
7/2014	0.83	878,276	45.6	0.096	0.00	0.000	0.000	62.7	0.242	39.41
8/2014	0.92	971,006	45.7	0.105	0.00	0.000	0.000	62.9	0.254	39.41
9/2014	1.00	1,050,840	45.7	0.109	0.00	0.000	0.000	63.0	0.258	39.41
10/2014	1.08	1,142,010	45.8	0.110	0.00	0.000	0.000	63.2	0.259	39.41
11/2014	1.17	1,255,490	45.9	0.110	0.00	0.000	0.000	63.3	0.259	39.41
12/2014	1.25	1,368,390	46.0	0.110	0.00	0.000	0.000	63.5	0.259	39.41
1/2015	1.33	1,445,960	46.1	0.110	0.00	0.000	0.000	63.7	0.260	39.41
2/2015	1.42	1,529,330	46.2	0.110	0.00	0.000	0.000	63.8	0.260	39.41
3/2015	1.50	1,615,090	46.2	0.111	0.00	0.000	0.000	64.0	0.260	39.41
4/2015	1.58	1,698,440	46.3	0.112	0.00	0.000	0.000	64.2	0.261	39.41
5/2015	1.67	1,782,590	46.4	0.115	0.00	0.000	0.000	64.4	0.265	39.41
6/2015	1.75	1,870,900	46.5	0.122	0.00	0.000	0.000	64.6	0.274	39.41
7/2015	1.83	1,955,460	46.7	0.128	0.00	0.000	0.000	64.8	0.281	39.41
8/2015	1.92	2,050,970	46.8	0.133	0.00	0.000	0.000	65.0	0.287	39.41
9/2015	2.00	2,133,200	46.9	0.135	0.00	0.000	0.000	65.2	0.290	39.41

Design Inputs

Design Life:	20 years	Base construction:	September, 2013	Climate Data Sources	34.518, -109.379
Design Type:	Flexible Pavement	Pavement construction:	September, 2013		
		Traffic opening:	October, 2013		

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	Default asphalt concrete	11.0
NonStabilized	A-1-a	10.0
Subgrade	A-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.3
Air voids (%)	6.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2013 (initial)	7,200
2023 (10 years)	12,046,700
2033 (20 years)	28,236,400

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	150.00	151.55	97.00	96.60	Fail
Permanent deformation - total pavement (in.)	0.50	0.56	97.00	89.84	Fail
AC bottom-up fatigue cracking (percent)	10.00	6.90	97.00	99.68	Pass
AC thermal fracture (ft/mile)	20000.00	39.41	97.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	20000.00	455.11	97.00	100.00	Pass
Permanent deformation - AC only (in.)	0.50	0.47	97.00	98.84	Pass

Distress Charts

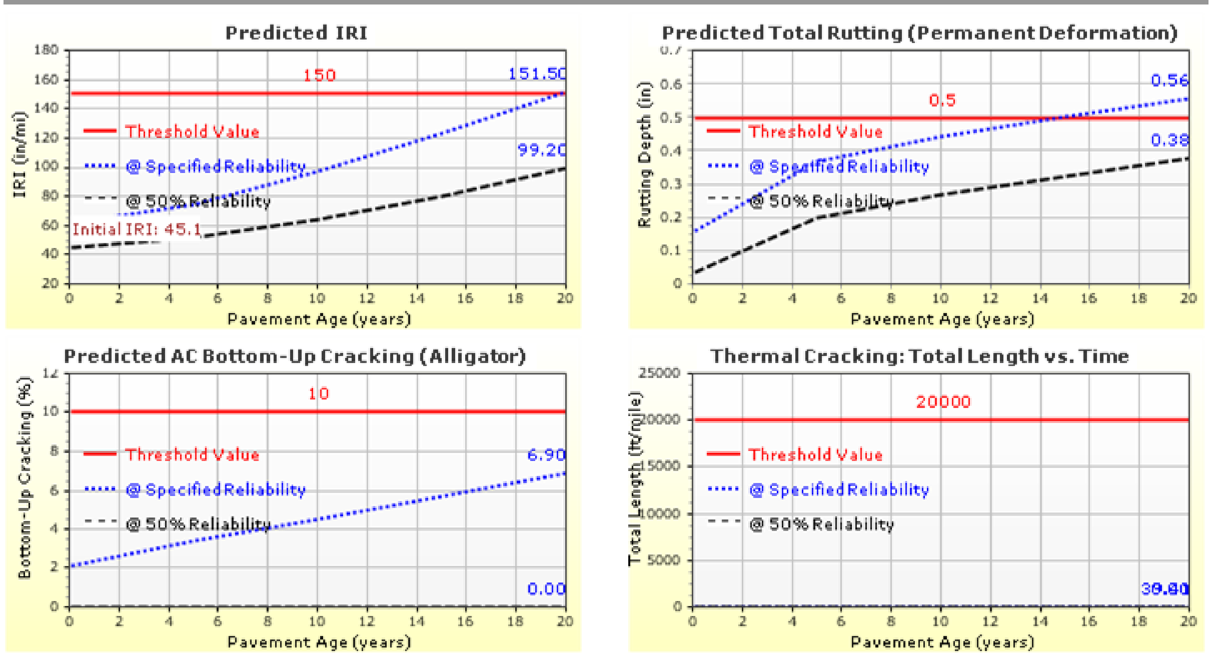


Figure 52. Excel Reliability Summary for New/Reconstructed HMA Design

Design Inputs

Design Life:	30 years	Existing construction:	-	Climate Data Sources	33.688, -112.082
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	September, 2013		33.443, -111.99
		Traffic opening:	October, 2013		33.623, -111.911

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP	10.0
NonStabilized	A-1-a	6.0
Subgrade	A-2-4	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2013 (initial)	10,000
2028 (15 years)	27,145,100
2043 (30 years)	69,436,200

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	150.00	218.30	97.00	58.14	Fail
Mean joint faulting (in.)	0.12	0.19	97.00	0.00	Fail
JPCP transverse cracking (percent slabs)	10.00	10.23	97.00	96.70	Fail

Distress Charts

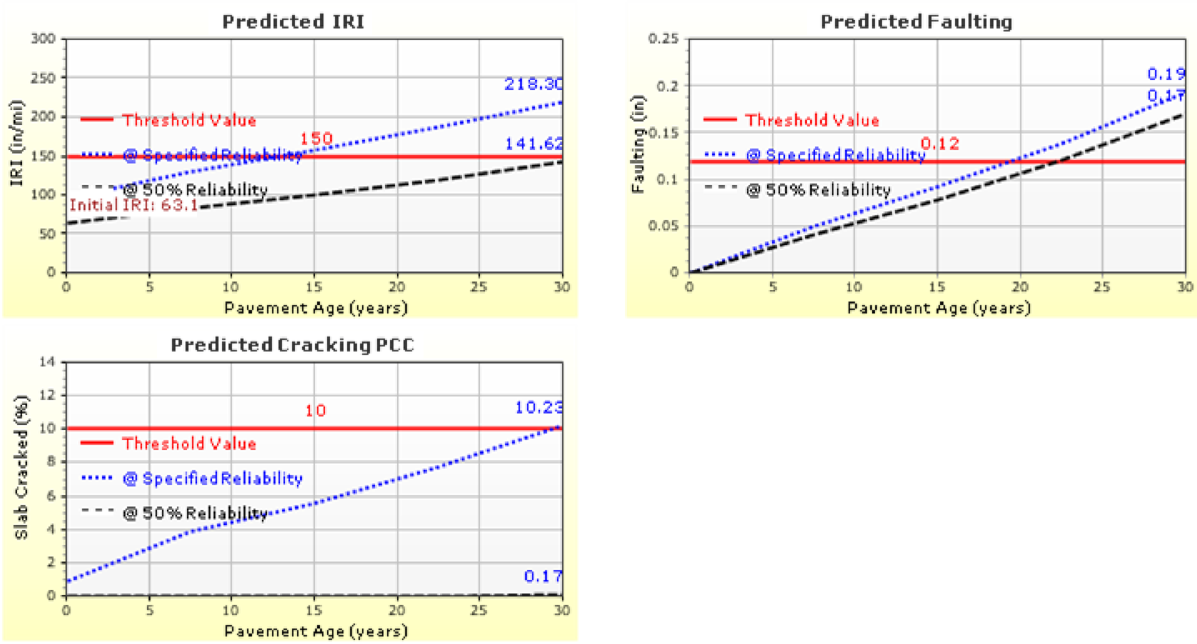


Figure 53. Excel Reliability Summary for New/Reconstructed JPCP

Table 62. Excel New/Reconstructed JPCP IRI, Joint Faulting, and Slab Transverse Fatigue Cracking Over Time

Predicted Distress

Month	Pavement Age (years)	Heavy Trucks (cum.)	Mean Predicted Distress			Predicted Distress @ Reliability		
			IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent	IRI (in/mi)	Mean joint faulting (in.)	JPCP transverse cracking (percent
10/2013	0.08	122,937	63.1	0.000	0.00	95.7	0.000	0.94
11/2013	0.17	275,956	64.2	0.001	0.00	97.7	0.001	0.94
12/2013	0.25	428,196	64.4	0.001	0.00	98.1	0.002	0.94
1/2014	0.33	532,796	64.6	0.001	0.00	98.5	0.002	0.94
2/2014	0.42	645,217	64.8	0.002	0.00	98.8	0.003	0.94
3/2014	0.50	760,855	65.0	0.002	0.00	99.2	0.003	0.94
4/2014	0.58	873,253	65.2	0.003	0.00	99.6	0.004	0.94
5/2014	0.67	986,718	65.5	0.003	0.00	100.0	0.005	0.94
6/2014	0.75	1,105,810	65.7	0.004	0.00	100.4	0.005	0.94
7/2014	0.83	1,219,830	65.9	0.004	0.00	100.7	0.006	0.94
8/2014	0.92	1,348,620	66.1	0.005	0.00	101.1	0.007	0.94
9/2014	1.00	1,459,500	66.3	0.005	0.00	101.5	0.007	0.94
10/2014	1.08	1,586,120	66.6	0.006	0.00	102.0	0.008	0.94
11/2014	1.17	1,743,730	66.9	0.006	0.00	102.5	0.009	0.94
12/2014	1.25	1,900,540	67.2	0.007	0.00	103.1	0.010	0.94
1/2015	1.33	2,008,280	67.4	0.008	0.00	103.5	0.010	0.94
2/2015	1.42	2,124,070	67.6	0.008	0.00	103.9	0.011	0.94
3/2015	1.50	2,243,180	67.8	0.009	0.00	104.3	0.012	0.94
4/2015	1.58	2,358,950	68.1	0.009	0.00	104.7	0.012	0.94
5/2015	1.67	2,475,820	68.3	0.010	0.00	105.0	0.013	0.94
6/2015	1.75	2,598,480	68.5	0.010	0.00	105.4	0.013	0.94
7/2015	1.83	2,715,920	68.7	0.010	0.00	105.8	0.014	0.94
8/2015	1.92	2,848,580	68.9	0.011	0.00	106.1	0.015	0.94
9/2015	2.00	2,962,780	69.1	0.011	0.00	106.5	0.015	0.94

References

American Association of State Highway and Transportation Officials. *Guide for Design of Pavement Structures, 4th* Edition. Washington, DC: American Association of State Highway and Transportation Officials, 1993.

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

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

ASTM International. ASTM C469 - Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression. West Conshohocken, PA: ASTM International.

Barker, W. R., and W. N. Brabston. *Development of a Structural Design Procedure for Flexible Airport Pavements*. Washington, DC: Federal Aviation Administration, FAA Report Number FAA-RD-74-199, 1975.

Bayomy, F., S. El-Badawy, and A. Awed. *Implementation of the MEPDG for Flexible Pavements in Idaho*. Boise, ID: Idaho Transportation Department, Research Report RP 193, 2011.

Idaho Transporation Department. Idaho Standard Method of Test for Resisstance R-Value and Expansion Pressure of Compacted Soils and Aggregates. Idaho IT-8-11. Boise, ID: Idaho Transportation Department, 2011. http://itd.idaho.gov/manuals/Manual%20Production/QA/files/2013Jan/500/550/IT-8.pdf

Transportation Research Board. *2002 Guide for Design of New and Rehabilitated Pavement Structures*. Washington, DC: Transportation Research Board, NCHRP Project 1-37A., 2004.

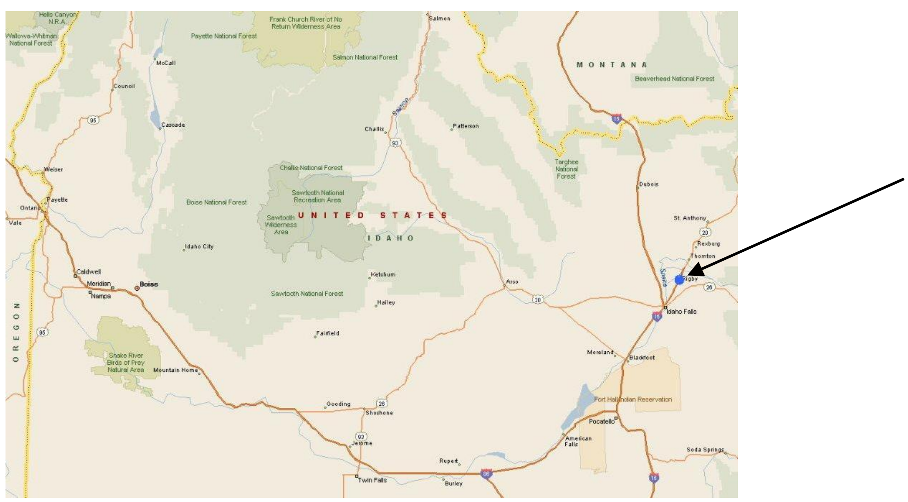
Transportation Research Board. *Version 1.0 – Mechanistic-Empirical Pavement Design Guide Software*. Washington, DC: Transportation Research Board, NCHRP, 2007.

Appendix A
Idaho New HMA Pavement Design Example

Project Description

This design example is for the new construction of a four-lane divided flexible pavement (HMA over granular base) located on US-20, milepoint 319.55 to 319.65. The project location is close to Rigby, between Idaho Falls and Rexburg, in Jefferson County, as shown in Figure 54.

a. Project Locations in Idaho



b. Close-Up View of Project Surroundings

Figure 54. New HMA Pavement Design Example Location

The roadway was originally constructed in August 1985 and later adopted in the Long Term Pavement Performance (LTPP) program with WIM Site ID 1021. The AADTT for 1985 was

873 trucks with 5.68 percent compound growth. This example uses August 1985 as the construction month and October 1985 as the traffic opening month.

Pre-Design Issues

Prior to the start of design and analysis, the pavement designer must assemble the key inputs required for this pavement type and decide on the hierarchical level of inputs for each key input category (traffic, climate, materials, etc.). Key inputs required for new or reconstructed HMA pavement design are presented in Table 63. Based on the functional class (U.S. highway) and location (rural) of the roadway under design, Level 2/3 inputs were generally assumed to be adequate.

NOTE: Inputs such as initial truck traffic volume (AADTT) and projected future growth rate must always be estimated at Level 1.

Table 63. Key Inputs Required for New or Reconstructed HMA Pavement Design

Input Category		Input Variables
General Information		Design Type & Pavement Type
		Base Construction Date (month/year)
		Pavement Construction Date (month/year)
		Traffic Opening Date (month/year)
"Design Life" & Reliability		"Design Life" (years, ####)
		Design Reliability, (years)
Performance Criteria		Initial IRI (in./mile)
		Terminal IRI (in./mile)
		Alligator Cracking (%)
		Thermal Cracking (ft/mile)
		Total Rutting (in.)
Traffic		Initial Two-Way AADTT
		Number of Lanes
		Directional Distribution
		Lane Distribution
		Truck Growth
		Vehicle Class Distribution
		Monthly Adjustments
		Number of Axles per Truck
		Axle Load Distribution
Structure & Materials Properties	**HMA Course**	Binder Grade, Binder Content, In-Place Air Voids, Aggregate Gradation, & Thickness. HMA course includes multiple lifts with same layer properties.
	Crushed Base	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, & Thickness
	Subgrade	Engineering Properties & Atterberg limits, M_r at Optimum Moisture Content, & Thickness
	Bedrock	Elastic/M_r, Unit Weight, & Poisson's Ratio
Project-Specific Calibration Factors		ITD Local Calibration Coefficients for New HMA Pavement (adopted from Wyoming DOT local calibration coefficients)

Develop a "Trial Design"

"Trial designs" begins with opening *AASHTOWare Pavement ME Design* and selecting the appropriate design type and pavement type, which for this design example are "New Pavement" and "Flexible Pavement." Additional information is presented in the *Pavement ME Design* "HELP System." Next, to create the "Trial Design" by populating several screens of the user interface. The "Trial Design" file, once completed, must be saved and reviewed for accuracy and erroneous entries. Files should be named using standard ITD conventions. For this example, the filename "New HMA.dgpx" is assumed.

NOTE: The output summary file names will be based on the input file name.

The following sections provide details regarding how the *AASHTOWare Pavement ME Design* project is created and populated with "Trial Design" inputs.

"Design Life"

For new or reconstructed HMA pavements, the recommended "Design Life" is 20 years. Thus, a 20-year "Design Life" was selected (see Figure 55).

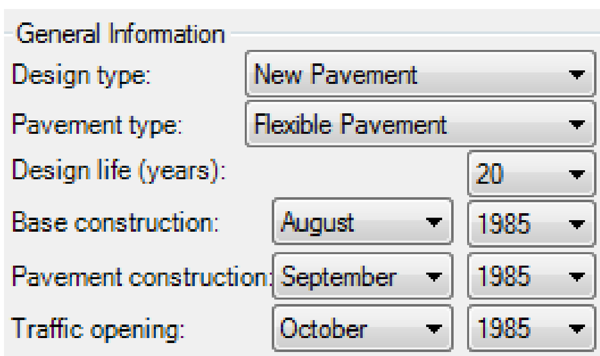


Figure 55. New HMA Pavement Design Example Construction Month and Year

Construction and Opening Dates

AASHTOWare Pavement ME Design requires information on anticipated construction or placement date (month and year) for both the base layer and HMA layer. This information is used for setting the baseline climate and traffic at construction. Anticipated month and year of base and HMA layer placements must be estimated based on typical ITD practices (i.e., the seasons in which pavements are normally constructed). Also required are the anticipated month and year for which the complete pavement will be opened to traffic. Again, this input must be selected based on typical ITD construction practices. As shown in Figure 55, for this example the following were selected:

- Base Layer Placement (month/year): August 1985.
- HMA Layer Placement (month/year): September 1985.
- Traffic Opening (month/year): October 1985.

Performance Criteria & Design Reliability

Designers must select pavement performance criteria from which the "Trial Design" is accepted or rejected. Performance criteria are basically critical distress and smoothness levels that ITD allows for a given pavement type and functional class. *AASHTOWare Pavement ME Design* predicts distress and smoothness over a specified analysis period "Design Life", and these predictions at the end of the "Design Life" are compared to the selected threshold values. If the predicted distress and smoothness are greater than the threshold values, the "Trial Design" is rejected. AASHTOWare Pavement ME Design allows designers to predict distress and smoothness at various levels of reliability. See Chapters 3 and 4 for guidance on selecting performance criteria and reliability levels.

For this HMA design example, the performance criteria recommended for a primary highway (Principal Arterial) were selected (see Table 7). A reliability level of 85 percent was selected based on the functional class (see Table 9).

NOTE: ITD does not include longitudinal (top-down fatigue) cracking in the mix of distress types used in assessing HMA pavement performance.

Therefore, even though *AASHTOWare Pavement ME Design* produces predictions for this distress type, the predictions are ignored. One way of doing this is to set very high threshold values for this distress type. In addition, ITD does not include the asphalt layer permanent deformation in assessing HMA pavement performance. This distress type prediction is ignored. One way of doing this is to set the same threshold value as for total permanent deformation.

AASHTOWare Pavement ME Design requires an estimate of initial pavement smoothness (i.e., IRI right after surface HMA layer placement). This is an important input, as the time from initial construction to attaining the threshold IRI value depends greatly on the initial IRI obtained at the time of construction. The initial IRI value provided in the design must be attained in the field and thus must reflect mean ITD results. As shown in Figure 56, an initial IRI of 50 in./mile is used in this example. Designers can vary this input if there is reason to believe a different value would better reflect initial smoothness for a given project.

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	50	
Terminal IRI (in./mile)	175	85
AC top-down fatigue cracking (ft/mile)	5000	85
AC bottom-up fatigue cracking (percent)	15	85
AC thermal cracking (ft/mile)	1500	85
Permanent deformation - total pavement (in.)	0.5	85
Permanent deformation - AC only (in.)	0.5	85

Figure 56. Performance Criteria and Reliability for New HMA Pavement Design Example

Traffic

AASHTOWare Pavement ME Design hierarchical Levels 1 through 3 define how representative traffic inputs are for a particular site. Level 1 data are considered the best representation of past and future traffic characteristics, as traffic inputs are obtained from measurements and counts of actual axle weights and truck traffic volume (from WIM sites). Statewide averages of traffic inputs computed from historical traffic data from ITD’s Roadway Data Section (analyzed by the University of Idaho in RP193 *Implementation of the MEPDG for Flexible Pavements in Idaho*) are considered Level 2 data, while the Level 3 traffic inputs are national averages.

With the exception of traffic volume data, Level 2/3 traffic inputs are used in this design example. This project is not considered critical, based on its functional class, traffic volumes, and location. Pavement for a more critical roadway (e.g., eight-lane urban freeway with heavy truck traffic) would require mostly Level 1 traffic inputs. Regardless of how critical a design is, initial AADTT and growth rate must always be site-specific (Level 1).

The traffic input data used for this design example are a described in Table 64. Figures 57 and 58 present the *AASHTOWare Pavement ME Design* traffic module with key inputs populated for this example design.

Table 64. Traffic Input Data Used for This Design Example

Traffic Input Variable	Level of Input	Source and Value of Input
Initial Two-Way AADTT	1	The designer must always use Level 1 initial two-way AADTT data. Initial two-way AADTT data must be obtained from the ITD Roadway Data Section. For this example, initial two-way AADTT of 873 was obtained from measured data.
Number of Lanes (in each direction)	1	Two lanes in the design direction.
Percent Trucks in Design Direction	2	ITD recommendations are provided in Table 11. The assumed statewide default is 50 percent. Designers can vary this input if there is reason to believe it would not reflect actual conditions.
Percent Trucks in Design Lane	2	ITD recommendations are provided in Table 11. For this example, 90 percent trucks in the design lane was selected.
Operational Speed	2	Posted speed limit for this highway and location was 65 mph.
Axle Configuration & Lane Wander	3	Level 3 defaults were assumed, as there are no ITD-specific recommendations.
Wheelbase	3	National averages of 17, 22, and 61 percent were assumed for the percentages of trucks with short-, medium-, & long-axles. Recommendations are provided in Table 34.
Vehicle Class Distribution	1	Project-specific vehicle class distribution was obtained from Table 13. The project location is very close to WIM Site No. 96.
Truck Traffic Growth	1	The designer must always use Level 1 inputs obtained from the ITD Transportation Systems section. For this example, a growth rate of 5.68 percent compounded over a 20-year "Design Life" was obtained based on historical data.
Monthly Adjustment	2	Idaho statewide average vehicle class distribution was obtained from Table 12.
Axles per Truck	2	Idaho statewide average vehicle class distribution was obtained from Table 12.
Hourly Distribution	2	The hourly truck distribution is not required for: 1. new or reconstructed HMA pavements. 2. HMA-overlaid existing HMA pavements. 3. HMA-overlaid existing fractured JPCP or CRCP. For all other pavement types, estimates of hourly truck distribution are needed.
Axle Load Distribution	2	Idaho statewide average axle load distribution for lightly loaded traffic was obtained from Tables 15 through 18.

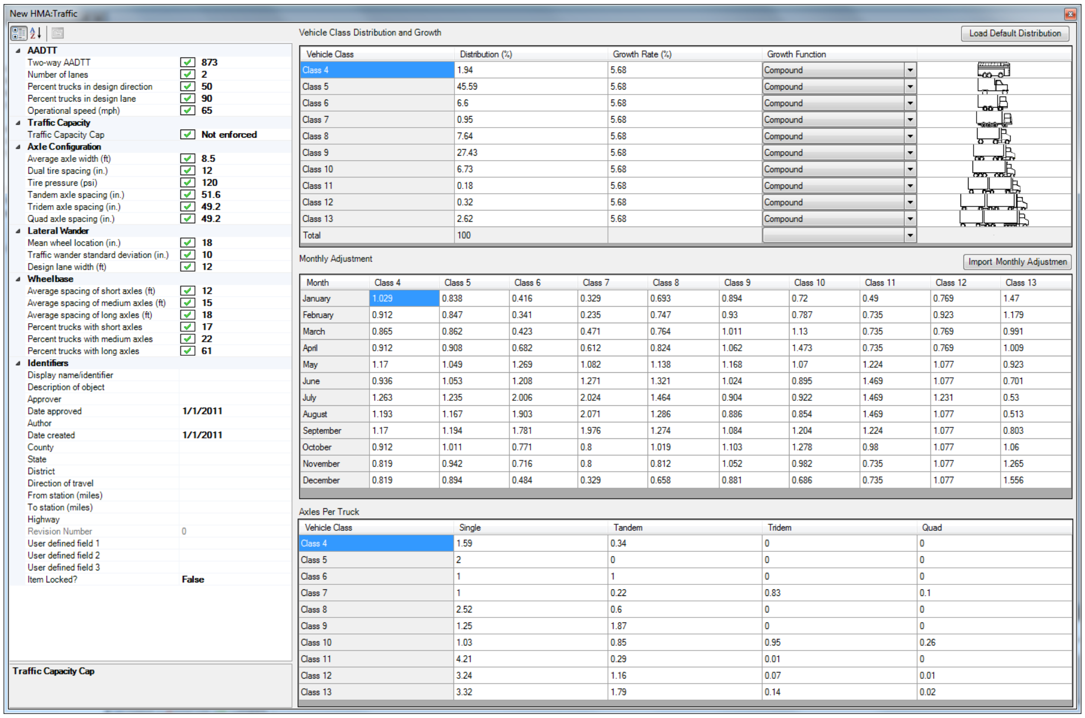


Figure 57. General Traffic Inputs for New HMA Pavement Design Example

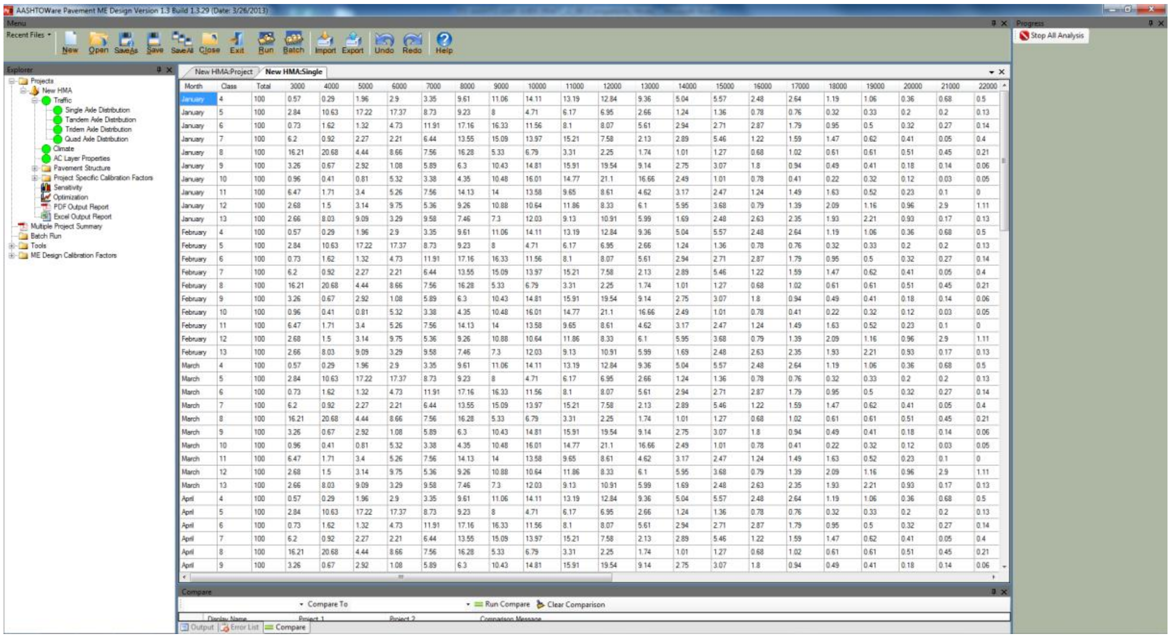


Figure 58. Single-Axle Load Distribution Inputs for New HMA Pavement Design Example

Climate

AASHTOWare Pavement ME Design requires historical climate data to simulate temperature and moisture conditions within the “Trial Design” structure. For most design situations, climate data available at Level 2 and embedded in the software are adequate. *AASHTOWare Pavement ME Design* allows users to create their own weather stations from which project-specific climate data can be obtained. The methodology for creating project-specific weather stations is presented in the *Pavement ME Design* "HELP System."

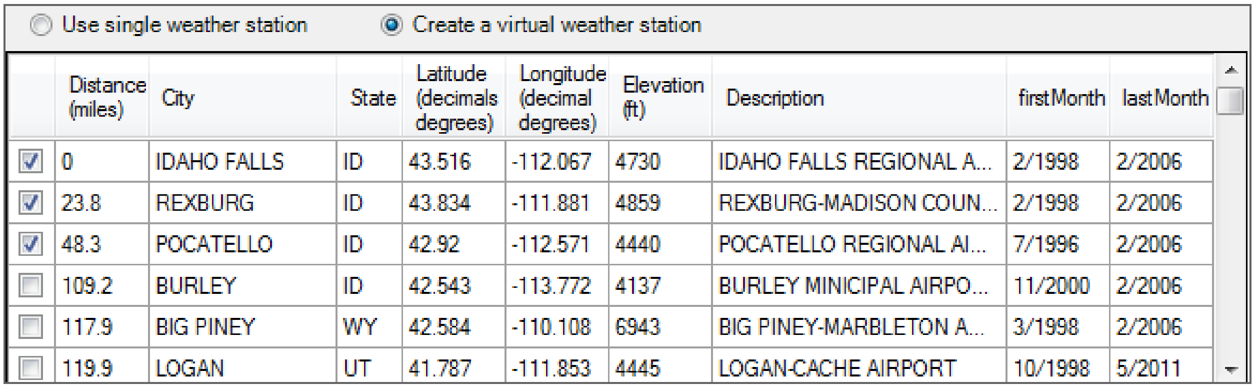
For this design example, climate data were obtained from weather stations in Idaho or nearby States. As noted, historical climate data from these weather stations have been included in the software. Designers can query the climate data to obtain information on the appropriate weather station or stations to use for design analysis. This requires the designer to produce project location coordinates (i.e., latitude and longitude) and elevation. For existing alignments, project-specific location coordinates and elevation typically are available from project design documents, plan sheets, or the ITD TAMS database. Other online tools also could provide the necessary information, such as Google Earth, Map Point, and Google Map.

AASHTOWare Pavement ME Design uses project location coordinates to identify nearby weather stations for use in creating a Level 2 virtual weather station. Once the nearby weather stations are identified, the designer can selected as many weather stations as they want to create the virtual weather station. Care must be taken in making this selection, to ensure that the selected weather stations are representative of conditions at the project location. In addition to project coordinates and elevation, the designer must provide an estimate of the location's depth to water table. This is mostly determined based on the designer's local experience or historical data available from nearby wells.

For this example the project coordinates and elevation are as follows:

- Latitude: 43.516 decimal degrees.
- Longitude: -112.067 decimal degrees.
- Elevation: 4,730 feet.

Depth to water table of 10 feet was assumed. Based on the location of the project, it was determined that the closest weather station was approximately 23 miles away, and 3 weather stations were within a 50-mile radius. Thus, these three weather stations were selected for use in creating the project virtual weather station. Figure 59 presents the *AASHTOWare Pavement ME Design* inputs for climate for this design example.



Use single weather station　　Create a virtual weather station

	Distance (miles)	City	State	Latitude (decimals degrees)	Longitude (decimal degrees)	Elevation (ft)	Description	firstMonth	lastMonth
☑	0	IDAHO FALLS	ID	43.516	-112.067	4730	IDAHO FALLS REGIONAL A...	2/1998	2/2006
☑	23.8	REXBURG	ID	43.834	-111.881	4859	REXBURG-MADISON COUN...	2/1998	2/2006
☑	48.3	POCATELLO	ID	42.92	-112.571	4440	POCATELLO REGIONAL AI...	7/1996	2/2006
☐	109.2	BURLEY	ID	42.543	-113.772	4137	BURLEY MINICIPAL AIRPO...	11/2000	2/2006
☐	117.9	BIG PINEY	WY	42.584	-110.108	6943	BIG PINEY-MARBLETON A...	3/1998	2/2006
☐	119.9	LOGAN	UT	41.787	-111.853	4445	LOGAN-CACHE AIRPORT	10/1998	5/2011

Figure 59. Selecting Virtual Weather Stations for New HMA Pavement Design Example

Pavement Structure

AASHTOWare Pavement ME Design allows for the design of three types of new or reconstructed HMA pavement design: Conventional (HMA over granular base), Full-Depth (HMA over asphalt treated base), and Semi-Rigid (HMA over chemically treated base). Deciding which HMA pavement type to select is based on ITD practices and policy. For this design example, a conventional HMA pavement structure was selected.

Based on ITD's HMA pavement design philosophy, conventional HMA pavements will typically consist of 4 layers, as shown in Figure 60. The general description of 4 layers, starting from the bottom foundation support, are as follows:

- **Bedrock:** Highly fractured and weathered or massive continuous (intact) rock within 10 to 20 ft of the pavement foundation, if present.

- **Subgrade**: The nature of the subgrade foundation (including depth to bedrock and groundwater table) is mostly determined directly from subsurface exploration and testing activities. Key for pavement design is to determine the natural/compacted subgrade properties and depth, as well as depth to bedrock. Natural and compacted subgrade soil properties are obtained from tests on the natural foundation soil in place and in its compacted state as the upper layers (12 to 24 in.) are rolled and compacted or removed and replaced during construction.

- **Crushed Gravel Base**: ITD specifies a range of aggregate/granular materials for use as base materials. The materials are mostly classified as AASHTO A-1-a and A-1-b soils.

- **HMA Layers:** ITD specifies a minimum two courses for the HMA (wearing course and intermediate course). In this example, the two HMA layers were combined, as their material properties are the same.

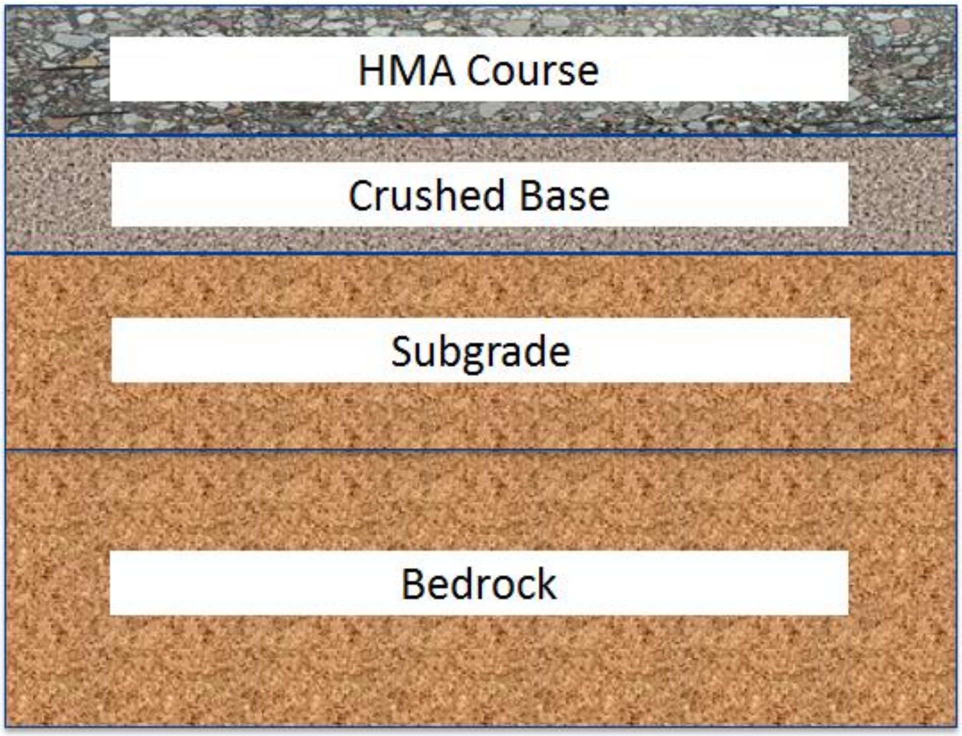


Figure 60. New HMA Pavement Design Structure

Guidance for obtaining pavement layer properties and thicknesses to define the trial HMA pavement structure has been presented in Chapter 10, Section 10.1 of this *User's Guide*. For this design, a Level 2/3 input was adopted. *AASHTOWare Pavement ME Design* recommends that once the "Trial Design" is defined, material properties must be populated starting from the lowest layer bedrock or natural subgrade to the surface layer.

The *AASHTOWare Pavement ME Design* "HELP System" provides detailed guidance on how to enter pavement structure and layer materials input data.

Bedrock

Review of historical subsurface exploration and testing reports for this location showed there was highly fractured bedrock under the natural subgrade. Thus, a bedrock layer was included. A highly fractured and weathered bedrock layer was selected with elastic modulus of 500,000 psi, which is MEPDG default.

Subgrade Layer

Subsurface exploration and testing reports from the LTPP database indicate the subgrade for this location is AASHTO A-1-a soil. Engineering properties required at Level 2/3 for the natural subgrade are presented in Table 65.

Table 65. Required Engineering Properties for the "Trial Design" Subgrade

Engineering Properties	Level of Input	Source of Data
Gradation	Level 2	Obtained through subsurface exploration and testing.
Atterberg Limits (liquid limit & plasticity index)	Level 2	Obtained through subsurface exploration and testing.
Maximum Dry Unit Weight	Level 3	Computed internally by the *AASHTOWare Pavement ME Design.*
Saturated Hydraulic Conductivity	Level 3	Computed internally by the *AASHTOWare Pavement ME Design.*
Specific Gravity Of Solids	Level 3	Computed internally by the *AASHTOWare Pavement ME Design.*
Optimum Gravimetric Water Content	Level 3	Computed internally by the *AASHTOWare Pavement ME Design.*
Soil Water Characteristic Curve	Level 3	Computed internally by the *AASHTOWare Pavement ME Design.*
Resilient Modulus (M_r)	Level 2	Obtained elastic modulus back-calculated from FWD deflection testing data, then converted field modulus to laboratory condition. (see Table 37 for guidance)

Figure 61 shows the subgrade engineering properties (gradation and Atterberg limits) obtained from laboratory testing coded into *AASHTOWare Pavement ME Design*. Based on these two properties, the software internally estimates maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum gravimetric water content, and the soil water characteristic curve. The designer must check the estimated soil engineering properties for accuracy and reasonableness. If the estimates are deemed unreasonable, the designer can override the internally estimated values. Guidance for overriding the engineering properties is provided in the *AASHTOWare Pavement ME Design "*HELP System."

For this example, the FWD deflection back-calculated project mean elastic modulus was 23,807 psi. The corrected M_r of 23,807*0.67 = 15,951 psi at optimum moisture content (calculated by the program as 9.9 percent) was entered into the software, as shown in Figure 62. Again, the designer can override this value if warranted.

NOTE: The "layer compacted" box on the input screen was uncheck to reflect field conditions (as the subgrade layer is not compacted).

The thickness of the subgrade layer was 30 inches, as there was an immediate underlying layer or bedrock.

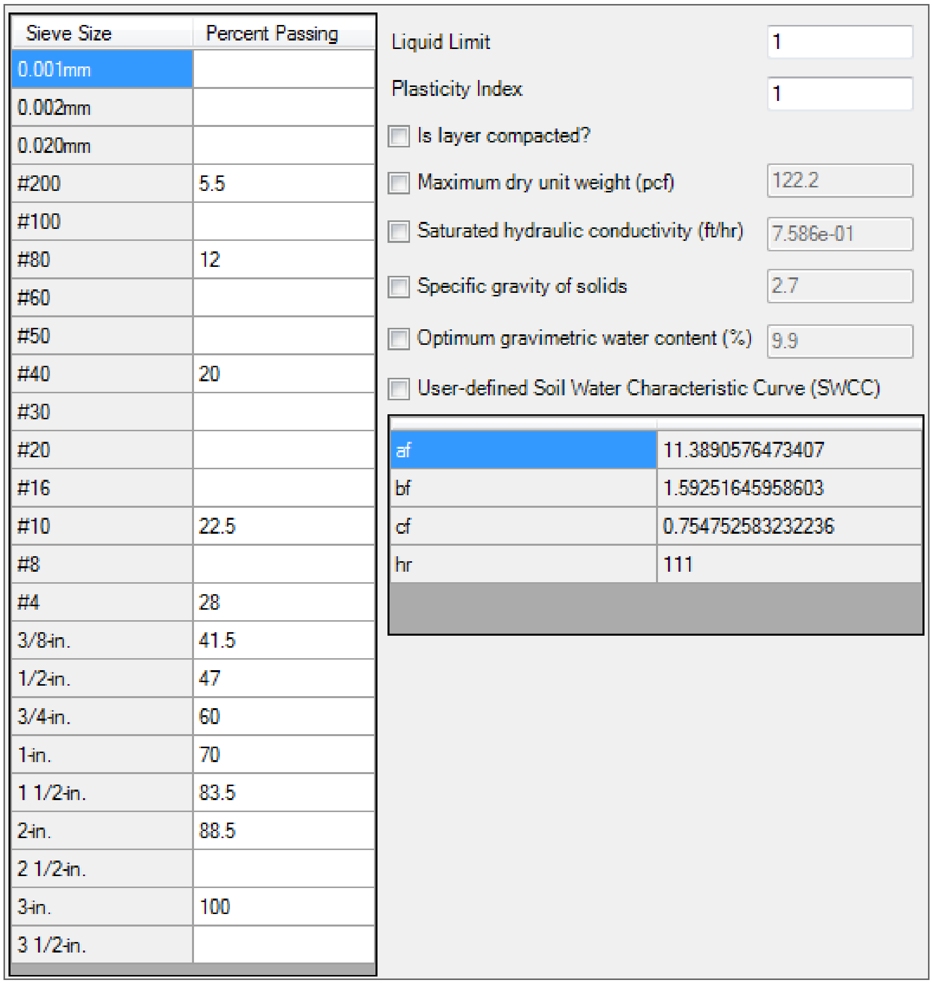


Figure 61. Subgrade Engineering Properties Input Screen for New HMA Pavement

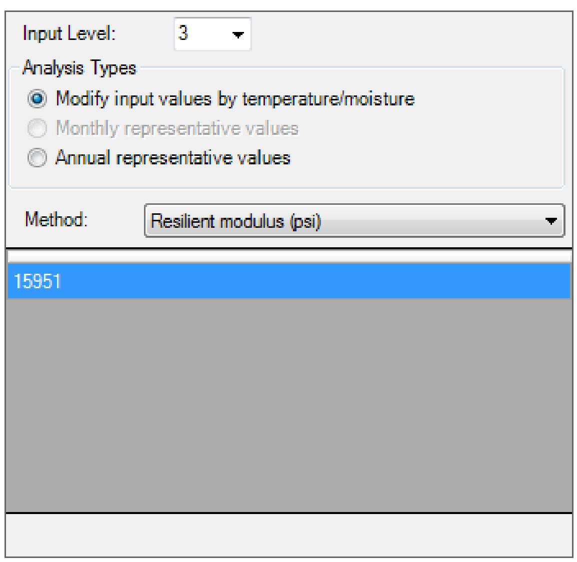


Figure 62. Subgrade Level 3 Resilient Modulus Input Screen for New HMA Pavement

Crushed Gravel Base Layer

The aggregate base material type for use in pavement construction is selected based on the project location (mostly, the nearest source of high-quality aggregate material is selected). The ITD "crushed gravel" material classification and properties are determined based on the source of the material.

For this design example, material classification was selected as A-1-a. As shown in Table 39, crushed gravel has properties comparable to AASHTO soil class A-1-a, and M_r at optimum moisture for this material is assumed to be 35,524 psi. This value came from Figure 63 by entering the modulus of lower layer axis at 15,951 psi, turning on the 6 inch base line and intersecting the modulus of upper layer axis. Because the M_r of aggregate or granular base/subbase layers depends on the M_r of the supporting layers, as a rule of thumb, the aggregate base M_r entered into the software for a granular base layer must not exceed three times the M_r of the supporting subgrade or subbase layer to avoid decompaction of that layer (see Figure 63).

Therefore, it is critical for designers to check whether $M_{rbase}/M_{rsubgrade}$ is more than or less than 3. For this example, $M_{rbase}/M_{rsubgrade} = 2.23$.

Once the base modulus is selected, the designer can enter the aggregate base engineering properties and M_r into *AASHTOWare Pavement ME Design*. For this example, AASHTO A-1-a soil gradation and Atterberg limits were obtained from measurement. A layer thickness of 5.3 inches was assumed for the "Trial Design" and entered into the software. See Table 40.

HMA Layer

The required inputs and values entered for this design example are presented in Table 66.

NOTE: The input requirements were for Level 2/3 for all inputs.

Figures 64 through 66 present the *AASHTOWare Pavement ME Design* screens used to enter HMA material properties for the HMA layer.

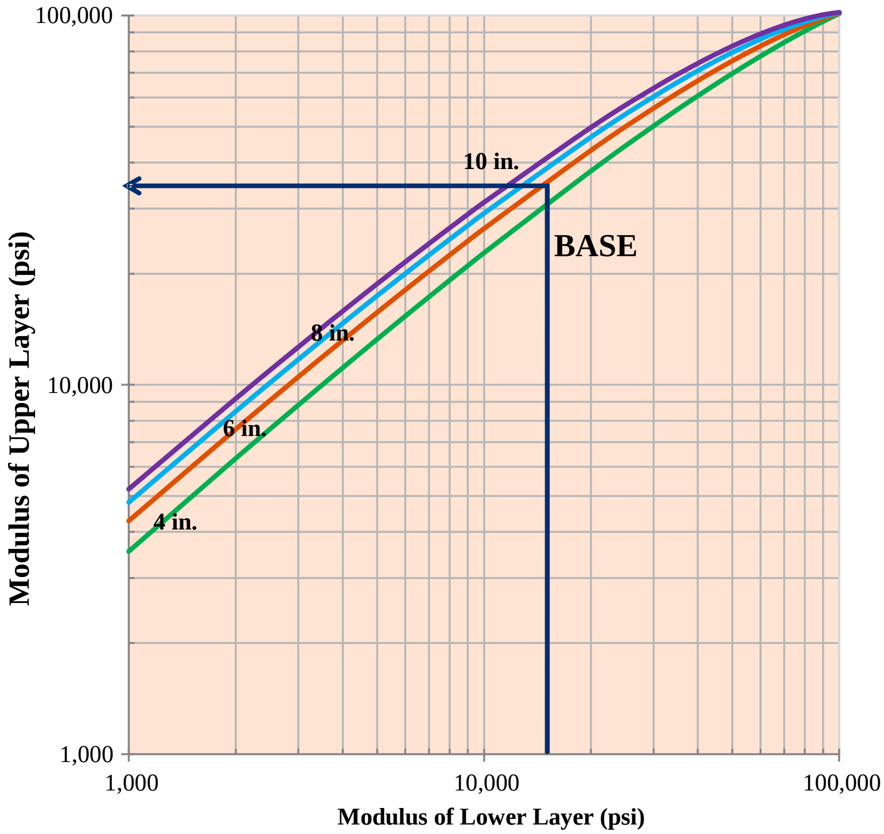


Figure 63. Selecting Base Modulus for New HMA Pavement Design

Table 66. Required Engineering Properties for the "Trial Design" HMA Layer

Engineering Properties	Level of Input	Source of Data
Layer Thickness	1	ITD design policy recommends a minimum total HMA thickness of 2 in. *AASHTOWare Pavement ME Design* does not allow for HMA layer thicknesses of less than 1in. An HMA layer thickness of 4.5 in. was assumed for "Trial Design".
Gradation (found under the Dynamic Modulus input screen)	3	Gradation for this HMA mix type was obtained from mean gradation test results. Percent passing the ¾ in., ⅜ in., No. 4 & No. 200 sieves were 100%, 92%, 72%, and 6.8%, respectively.
Asphalt Binder Type	3	Asphalt binder type PG 58-28 (Superpave) was selected. See Section 7.6.
Asphalt Binder Content (volumetric, as placed)	3	A value 12.23% was selected. See Section 7.6.
HMA Mix Air Voids Content (as placed)	3	A value 5.5% was selected. See Section 7.6.
HMA Unit Weight	3	A value 139 pcf was selected. See Section 7.6.
Test Reference Temperature	3	Assume *AASHTOWare Pavement ME Design* defaults.
HMA Creep Compliance*	3	Assume *AASHTOWare Pavement ME Design* defaults.
HMA Indirect Tensile Strength*	3	Assume *AASHTOWare Pavement ME Design* defaults.
Coefficient of Thermal Contraction*	3	Assume *AASHTOWare Pavement ME Design* defaults.
Thermal Conductivity	3	Assume *AASHTOWare Pavement ME Design* defaults.
Heat Capacity	3	Assume *AASHTOWare Pavement ME Design* defaults.

*HMA creep compliance, indirect tensile strength, and coefficient of thermal contraction. for the HMA layer are used in computing HMA thermal cracking distress.

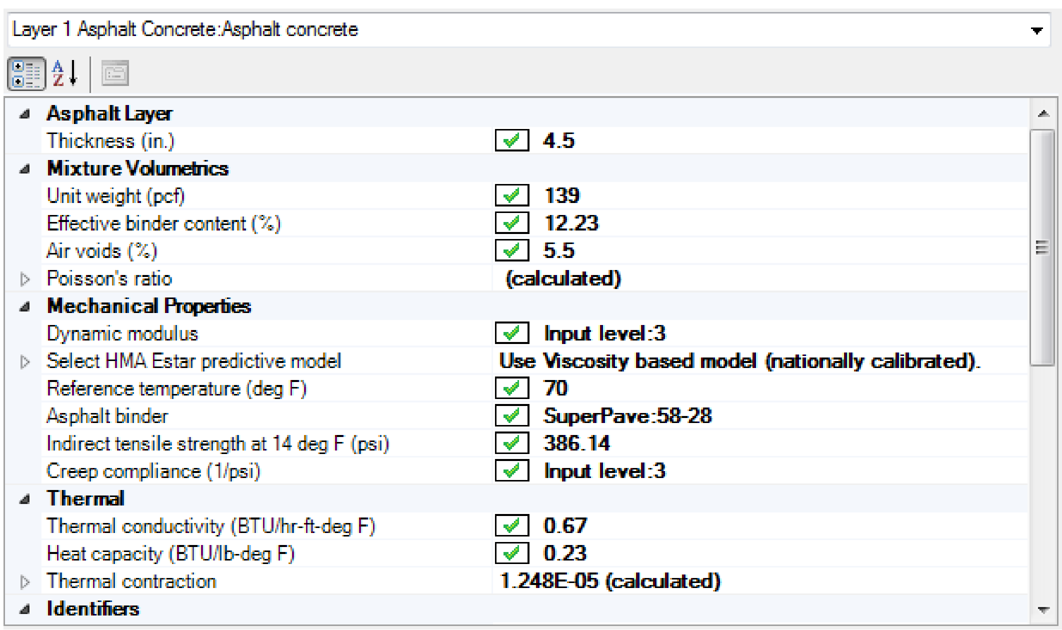


Figure 64. Screen for HMA Layer Binder and Mix Inputs for the Example New HMA Pavement Design

Dynamic modulus input level: 3

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	92
No.4 sieve	72
No.200 sieve	6.6

Figure 65. Screen for HMA Layer Gradation Inputs for the Example New HMA Pavement Design

Creep compliance level: 3

Loading Time(sec)	Low Temp (-4 deg F)	Mid Temp (14 deg F)	High Temp (32 deg F)
1	3.168712E-07	4.684324E-07	6.102071E-07
2	3.530102E-07	5.55981E-07	8.115491E-07
5	4.071849E-07	6.973144E-07	1.183094E-06
10	4.536241E-07	8.276403E-07	1.573463E-06
20	5.053597E-07	9.823238E-07	2.092638E-06
50	5.829147E-07	1.232036E-06	3.050693E-06
100	6.493959E-07	1.462299E-06	4.05729E-06

Figure 66. HMA Layer Creep Compliance for the Example New HMA Pavement Design

Additional HMA Layer Properties

The following additional HMA layer properties are required inputs for *AASHTOWare Pavement ME Design*:

- **Surface HMA Layer Surface Shortwave Absorptivity**: This input is used to estimate heat flow within the HMA layers. The *AASHTOWare Pavement ME Design* default value of 0.85 was assumed.

- **Endurance Limit**: HMA endurance limit is required only for the design of perpetual HMA pavements. As this design procedure is not calibrated and not recommended by ITD, designers must set the endurance limit to "False" in *AASHTOWare Pavement ME Design*, as shown in Figure 67.

- **Layer Interface:** This defines the friction levels between the HMA, base, and subgrade layers. As ITD recommends full bonding between all layers for HMA pavements, a default value of 1 (implying full friction between the layers) is recommended, as shown in Figure 68.

Designers can override any of these additional HMA layer inputs if warranted.

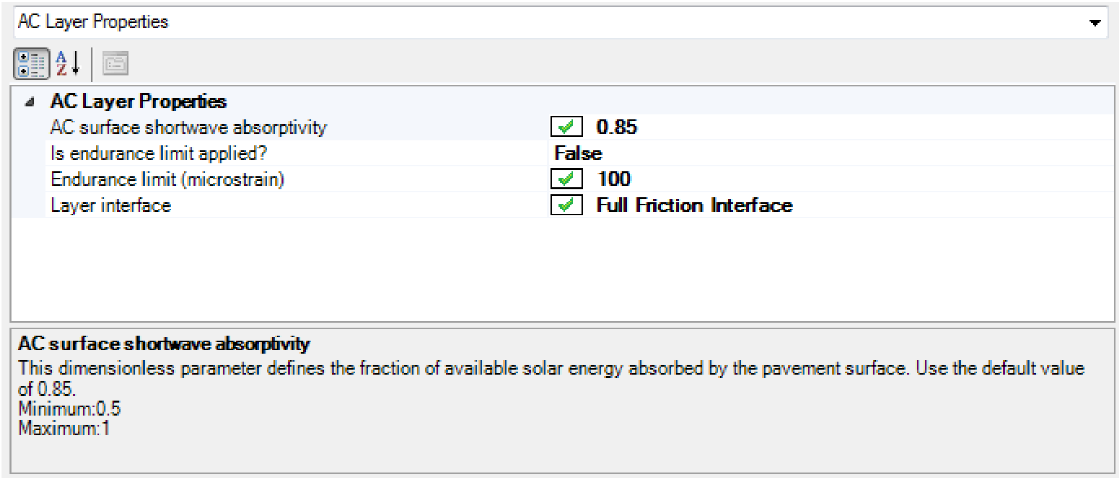
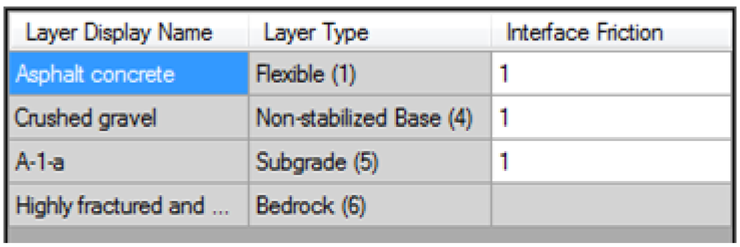


Figure 67. *AASHTOWare Pavement ME Design* Input Screen for Additional Inputs Required for HMA Surface Layer for the New HMA Pavement Design Example

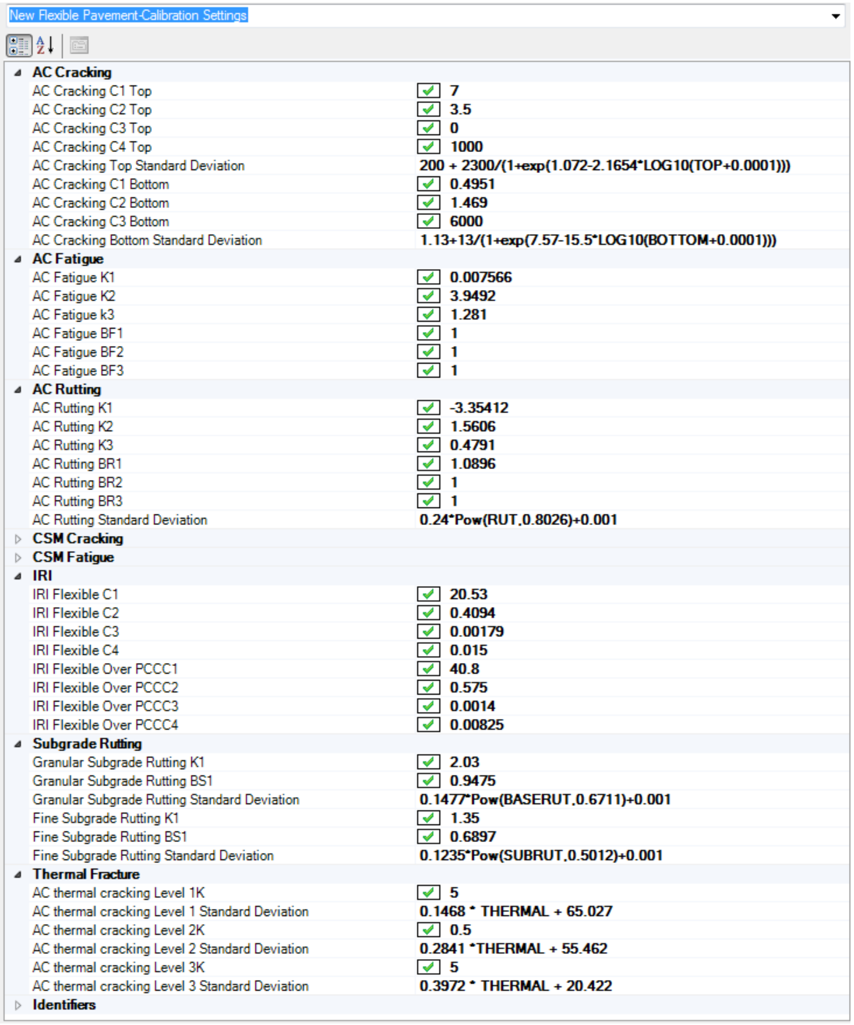


Layer Display Name	Layer Type	Interface Friction
Asphalt concrete	Flexible (1)	1
Crushed gravel	Non-stabilized Base (4)	1
A-1-a	Subgrade (5)	1
Highly fractured and ...	Bedrock (6)	

Figure 68. *AASHTOWare Pavement ME Design* Input Screen for New HMA Pavement Layer Interface Friction

ITD HMA Pavement Project Specific Calibration Coefficients

When *AASHTOWare Pavement ME Design* is used for Idaho conditions, ITD recommends the calibration coefficients presented in Figure 69 for distress and IRI models. These coefficients were adopted from Wyoming DOT and should be used until ITD establishes its own local calibration coefficients. Designers must enter these values and then check if the new HMA pavement project under design is outputting the calibration coefficients presented. If not, guidance is provided in the *AASHTOWare Pavement ME Design* "HELP System" on how to replace the global calibration coefficients with ITD-recommended values.



New Flexible Pavement Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.4951
AC Cracking C2 Bottom	1.469
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1.13+13/(1+exp(7.57-15.5*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	1
AC Fatigue BF2	1
AC Fatigue BF3	1
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.4791
AC Rutting BR1	1.0896
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.24*Pow(RUT,0.8026)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	20.53
IRI Flexible C2	0.4094
IRI Flexible C3	0.00179
IRI Flexible C4	0.015
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.9475
Granular Subgrade Rutting Standard Deviation	0.1477*Pow(BASERUT,0.6711)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.6897
Fine Subgrade Rutting Standard Deviation	0.1235*Pow(SUBRUT,0.5012)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	5
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	5
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 69. New Flexible Pavement Calibration Coefficients

Run "Trial Design" and Analyze Results

Pavement design using *AASHTOWare Pavement ME Design* is iterative. This means that the designer must

1. Run the software.
2. Check key outputs for reasonableness.
3. Check the "Trial Design" for adequacy (i.e., it should be able to carry anticipated traffic over its "Design Life" at the preset performance criteria recommended by ITD).

The check for adequacy must be done at the ITD-recommended reliability level. If the "Trial Design" is deemed inadequate, appropriate modifications must be made, such as increased thickness, or modification of binder type and HMA mix properties, to obtain a feasible final design.

Check of Key *AASHTOWare Pavement ME Design* Outputs for Reasonableness

It is very important for designers to review key inputs and outputs for each run to ascertain whether inputs were entered correctly in the software and the software processed input data correctly and produced the expected results (e.g., whether the climate statistics produced by the software correspond to what is expected of the given location). *AASHTOWare Pavement ME Design* produces two output files with a summary of key inputs and design outcomes (FILENAME.PDF and FILENAME.XLS) that can be used for this review. The XLS file contains significantly more detailed information for this review. Less information is contained in the PDF output file, which may be adequate and is presented under the following general headings:

- **Design Inputs**: Contains information about key designs inputs such as pavement structure definition, layer thicknesses, and traffic projections (see Figure 70).

- **Design Outputs:** Distress prediction summary in tabular and graphical forms (See Figure 71).

- **Traffic Input and Output Summary:** Graphical and tabular representation of key traffic inputs and projected growth and seasonal adjustments (see Figures 72 and 73):
    - **Traffic Distributions**: Tabular representation of traffic inputs.
    - **Axle Configuration**: Axle configuration summary.
    - **AADT Truck Growth:** Plots showing trends in AADTT growth.
    - **AADT Truck Growth by Class**: Tabular representation of growth in AADTT.

- **Climate Inputs and Output Summary**: Graphical and tabular representation of key climate inputs and climate variable statistics (see Figure 74):
    - **Climate data sources** (weather stations).
    - **Annual statistics of key variables:** temperature, precipitation, freezing index, etc.
    - **Monthly statistics of key variables**: temperature, precipitation, freezing index, etc. in a graphical format.

- **Design Properties:** Key pavement design input summary information.

- **Key HMA Material Inputs and Computed Parameters**:
    - Thermal (transverse) cracking inputs such as creep compliance, coefficient of thermal contraction, and so on.
    - HMA master curve and shift factors in graphical format.

- **Analysis Output Charts:**
  - Plots of predicted IRI, rutting, alligator cracking, and thermal cracking versus age in graphical format.
  - Detailed breakdown of predicted distress and IRI:
    - Plots of predicted bottom-up and top-down damage versus age in graphical format.
    - Components of total rutting.
    - Thermal cracking spacing and depth.

- **Layer Information:** Detailed summary of data for all layers within the pavement structure.

- **Calibration Coefficients:** Detailed summary of project-specific distress/IRI models calibration coefficients.

Designers are encouraged to thoroughly examine the information presented under these headings. Possible discrepancies between input data summaries and what was entered into *AASHTOWare Pavement ME Design* must be resolved.

Design Inputs

Design Life:	20 years	Base construction:	August, 1985	Climate Data Sources	43.516, -112.067
Design Type:	Flexible Pavement	Pavement construction:	September, 1985		43.834, -111.881
		Traffic opening:	October, 1985		42.92, -112.571

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	Asphalt concrete	4.5
NonStabilized	Crushed gravel	5.3
Subgrade	A-1-a	30.0
Bedrock	Highly fractured and weathered	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	12.2
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
1985 (initial)	873
1995 (10 years)	1,863,080
2005 (20 years)	5,100,200

Figure 70. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for New HMA Pavement

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	175.00	119.28	85.00	99.92	Pass
Permanent deformation - total pavement (in.)	0.50	0.45	85.00	95.03	Pass
AC bottom-up fatigue cracking (percent)	15.00	19.81	85.00	4.26	Fail
AC thermal cracking (ft/mile)	1500.00	2340.01	85.00	41.61	Fail
AC top-down fatigue cracking (ft/mile)	5000.00	3742.03	85.00	94.47	Pass
Permanent deformation - AC only (in.)	0.50	0.25	85.00	100.00	Pass

Distress Charts

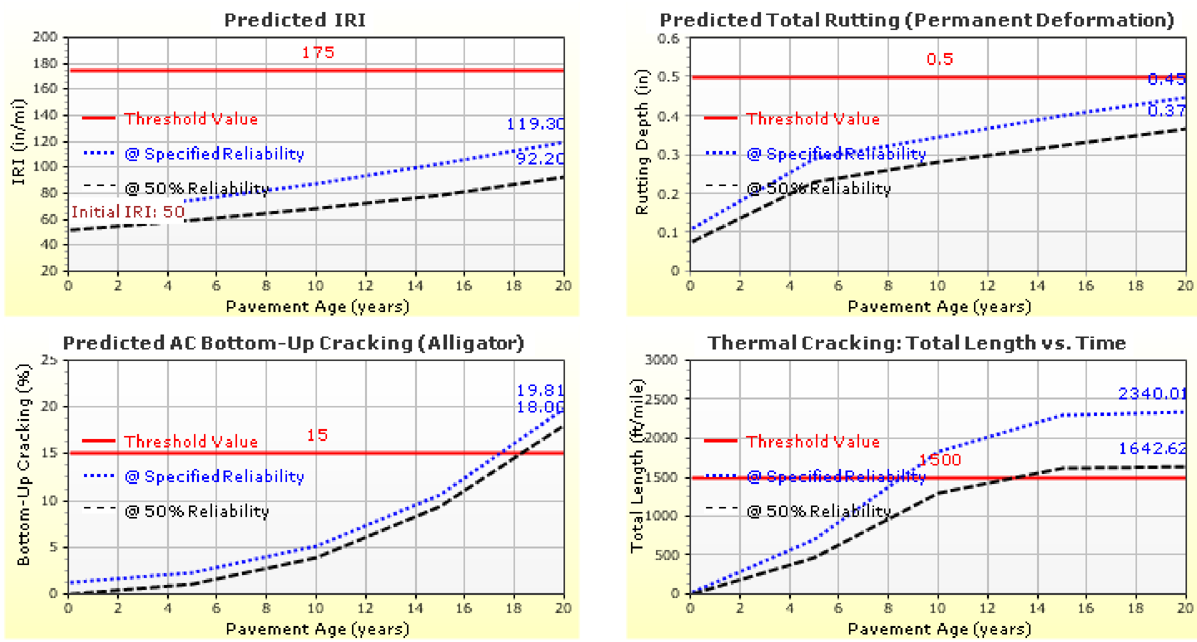


Figure 71. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for New HMA Pavement

Traffic Inputs

Graphical Representation of Traffic Inputs

Initial two-way AADTT:	873	Percent of trucks in design direction (%):	50.0
Number of lanes in design direction:	2	Percent of trucks in design lane (%):	90.0
		Operational speed (mph)	65.0

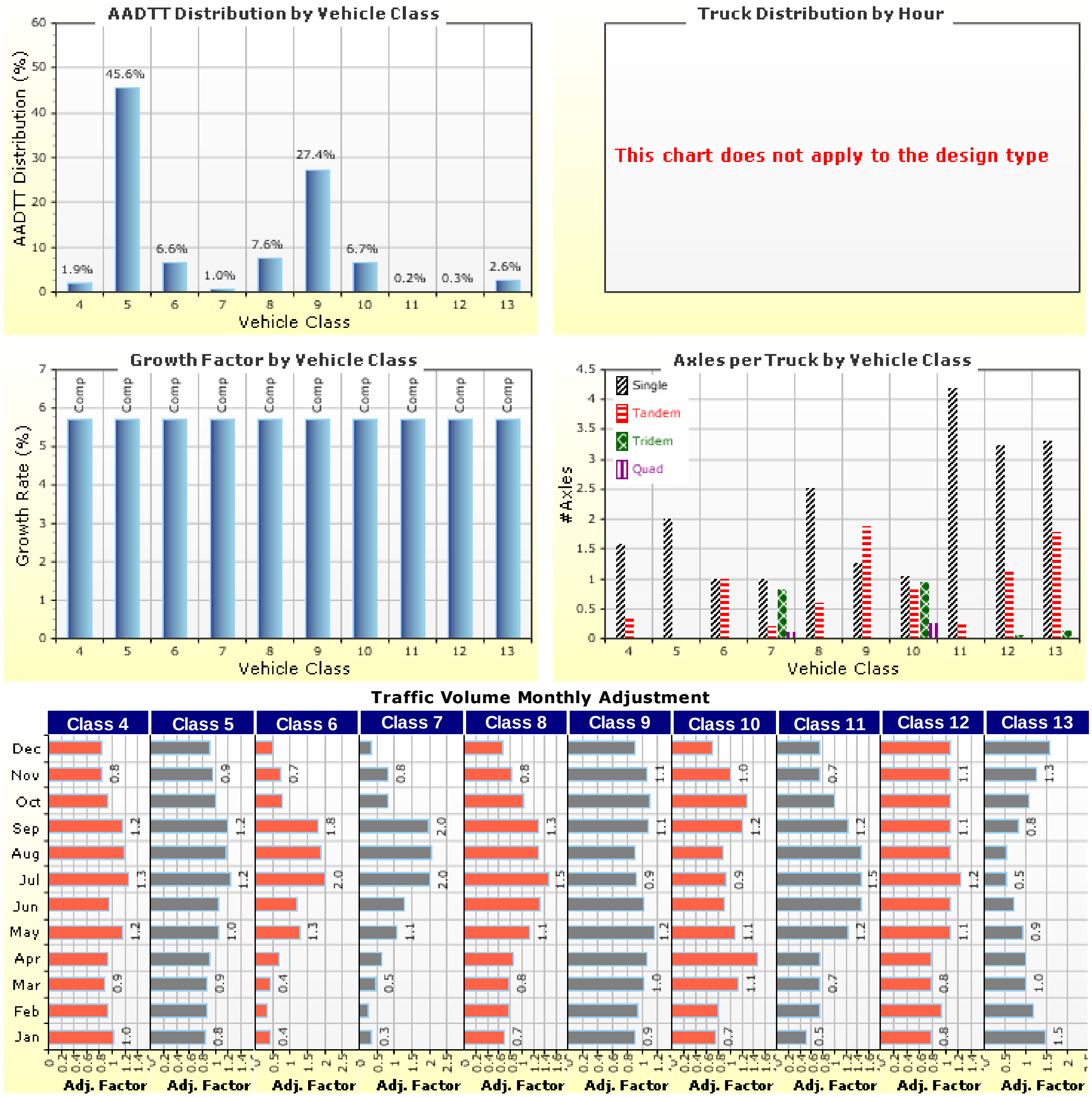


Figure 72. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Traffic Inputs for New HMA Pavement**

AADTT (Average Annual Daily Truck Traffic) Growth

*** Traffic cap is not enforced**

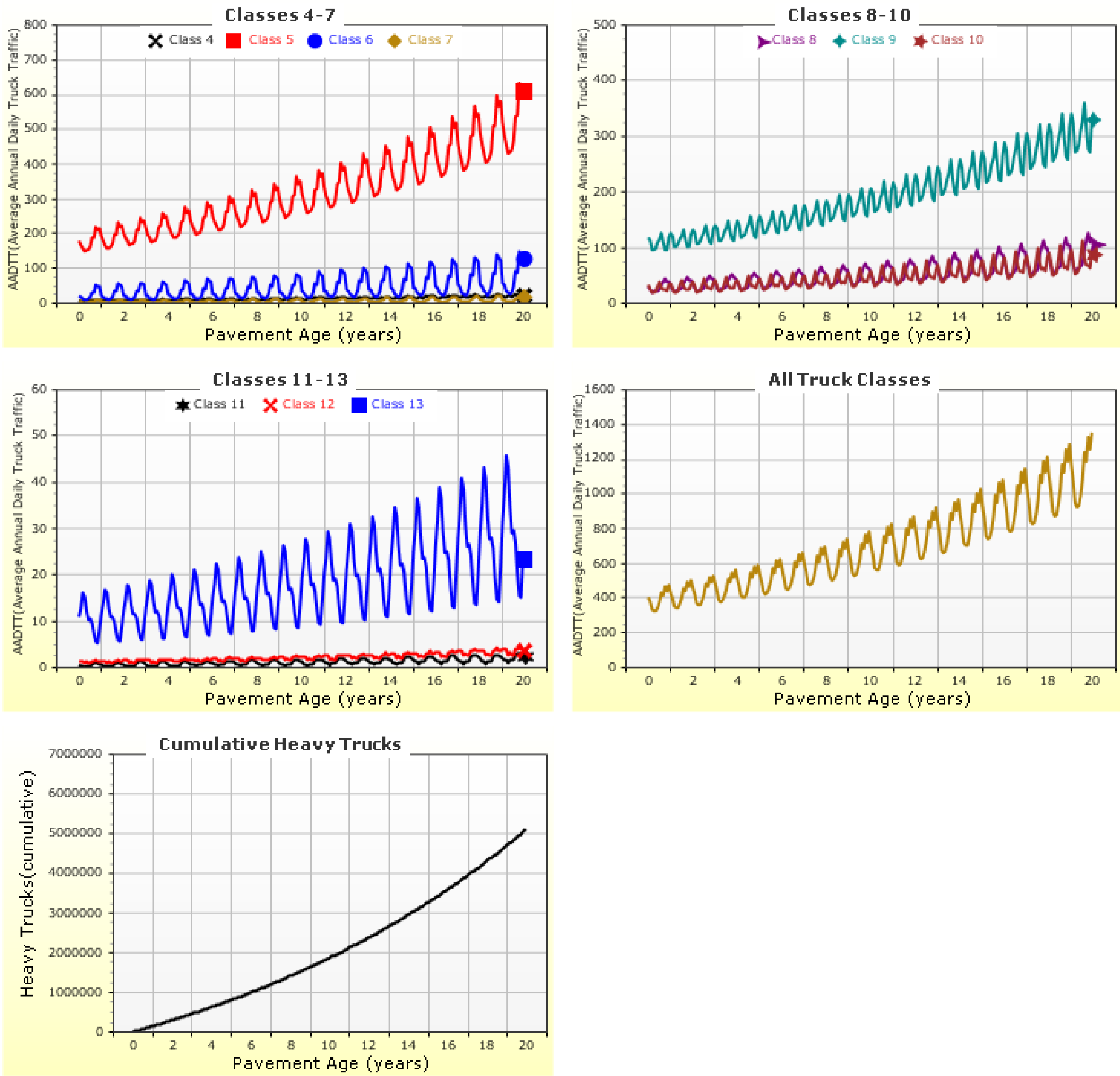


Figure 73. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (Projection of AADTT) for New HMA Pavement

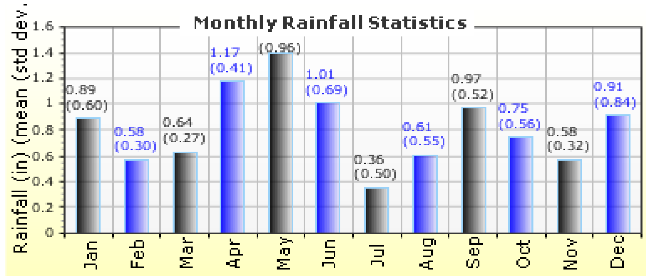
Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
IDAHO FALLS, ID	43.51600 -112.06700 4730
REXBURG, ID	43.83400 -111.88100 4859
POCATELLO, ID	42.92000 -112.57100 4440

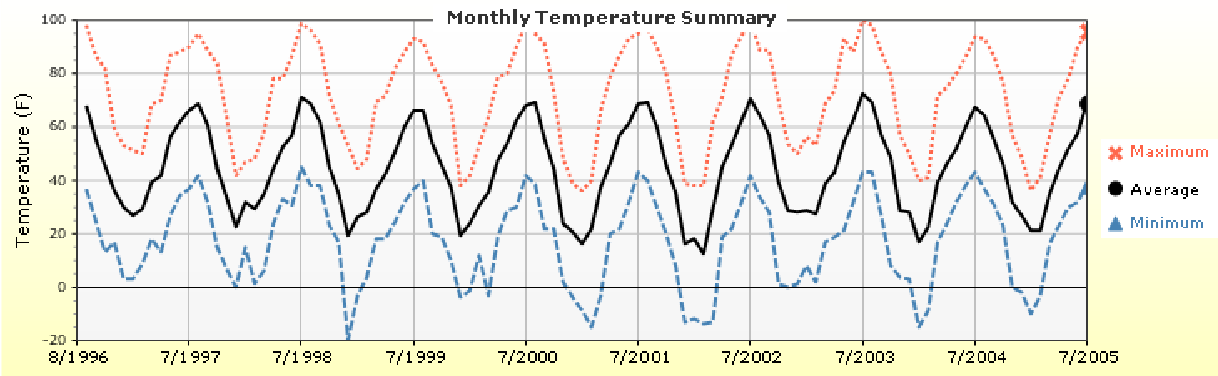
Annual Statistics:

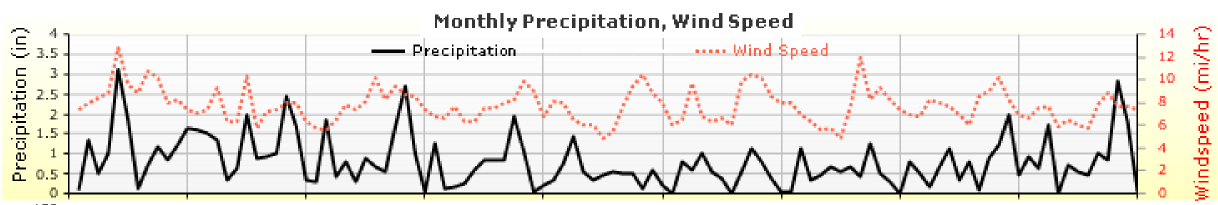
Mean annual air temperature (ºF)	44.92
Mean annual precipitation (in.)	9.87
Freezing index (ºF - days)	1063.24
Average annual number of freeze/thaw cycles:	116.86

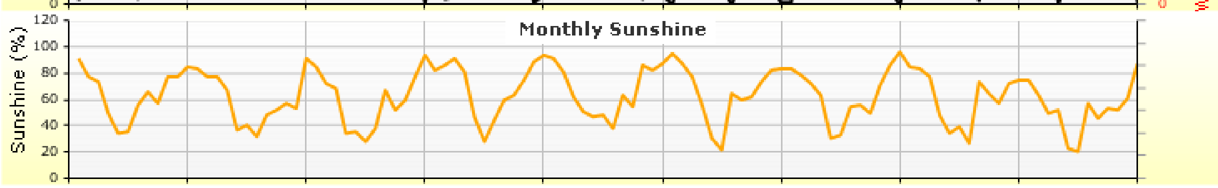


Water table depth(ft)	10.00

Monthly Climate Summary:







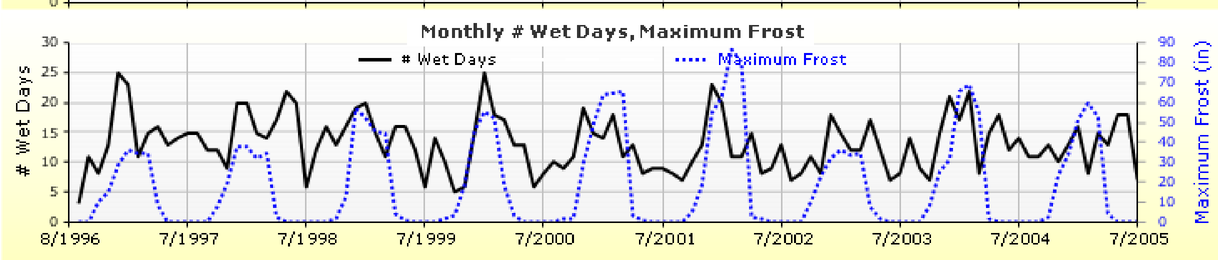


Figure 74. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for New HMA Pavement

Check "Trial Design" for Adequacy

The final step is to check the "Trial Design" for adequacy. It should be able to carry anticipated traffic over its "Design Life" at the preset performance criteria recommended by ITD. The outcome of this example "Trial Design" is presented in Figure 71. Designers must check this output summary to determine whether the design performance criteria are satisfied. This is done as follows:

1. Review the column called Criterion Satisfied? In the tabular output and determined for each distress type when the trial design "passed" or "failed."

2. If the "Trial Design" "passes" the criteria set for all distress types, then the design is deemed adequate and acceptable.

3. If one or more of the criteria "fail," then the design is deemed inadequate and the "Trial Design" must be revised as needed and checked again.

For this example, the trial design did not meet the performance criteria for alligator cracking and thermal cracking. This "Trial Design" needs to be revised.

Revise "Trial Design" and Rerun *AASHTOWare Pavement ME Design* as Needed

With the causes of "Trial Design" pavement inadequacy determined (failed to meet alligator cracking and transverse cracking performance criteria), the designer must determine reasons for failure to meet the performance criteria and adopt feasible solutions to improve the "Trial Design". For this design example, common reasons for not meeting alligator cracking and transverse cracking performance criteria are:

- Alligator cracking is caused by horizontal strain at the bottom of the HMA layer and is highly influenced by the HMA thickness and dynamic modulus (i.e., includes all the variables that significantly influence HMA dynamic modulus).

- Transverse cracking is caused by horizontal stress in the HMA layer and is highly influenced by the HMA creep compliance and indirect tensile strength (thus, HMA properties such as binder type, aggregate type, air voids, and binder content influence these factors).

As needed, the designer must adjust the "Trial Design" properties to improve performance.

A careful review of the design inputs for this example indicated a need to modify the binder type and thickness to improve performance. This was done using the *AASHTOWare Pavement ME Design* thickness optimization tool. A detailed description of this tool is provided in the *AASHTOWare Pavement ME Design* "HELP System."

Acceptance of Finalized Design

Figure 75 shows the final new HMA pavement design structure. *AASHTOWare Pavement ME Design* analysis shows that this design structure, along with the layer material types and properties under the prevailing site conditions in Idaho, would be able to carry approximately 5.1 million trucks over a 20-year "Design Life". The design outputs also show clearly a more than 85 percent chance that the distress and IRI over the 20-year "Design Life" will be less than the thresholds recommended by ITD. This design is thus deemed adequate. However, it must be noted that adequate designs are not achieved only by increasing HMA thickness or changing binder type. All of the options available through modifying materials properties and so on must be examined to produce a cost-effective design.

Design Inputs

Design Life:	20 years	Base construction:	August, 1985	Climate Data Sources	43.516, -112.067
Design Type:	Flexible Pavement	Pavement construction:	September, 1985		43.834, -111.881
		Traffic opening:	October, 1985		42.92, -112.571

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	Asphalt concrete	5.0
NonStabilized	Crushed gravel	5.3
Subgrade	A-1-a	30.0
Bedrock	Highly fractured and weathered	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	12.2
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
1985 (initial)	873
1995 (10 years)	1,863,080
2005 (20 years)	5,100,200

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	175.00	111.55	85.00	99.99	Pass
Permanent deformation - total pavement (in.)	0.50	0.45	85.00	94.58	Pass
AC bottom-up fatigue cracking (percent)	15.00	12.02	85.00	99.98	Pass
AC thermal cracking (ft/mile)	1500.00	49.57	85.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5000.00	3493.08	85.00	95.68	Pass
Permanent deformation - AC only (in.)	0.50	0.26	85.00	100.00	Pass

Distress Charts

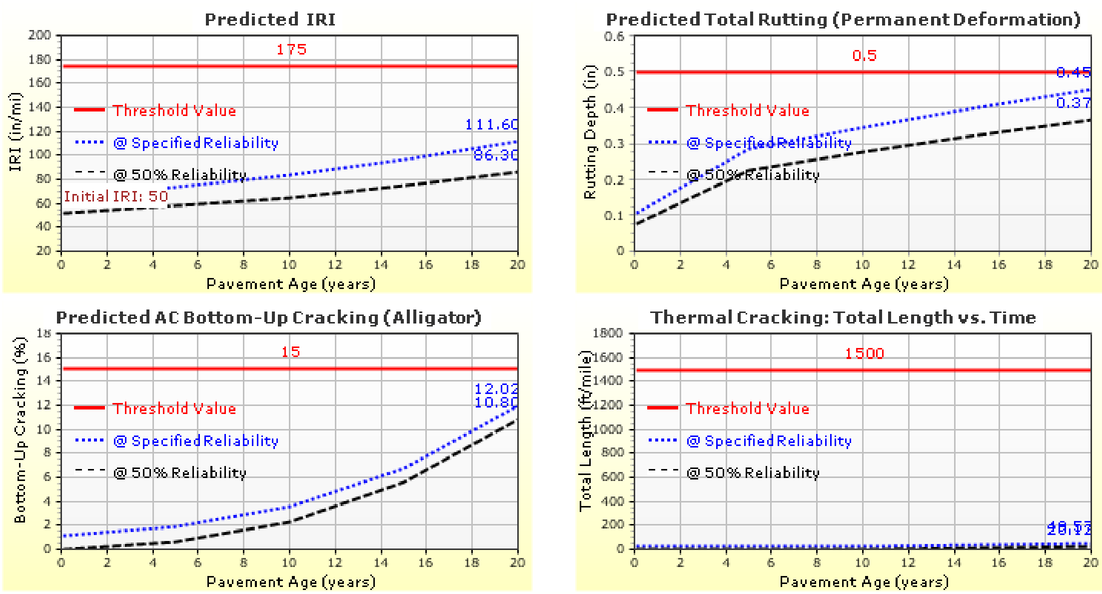


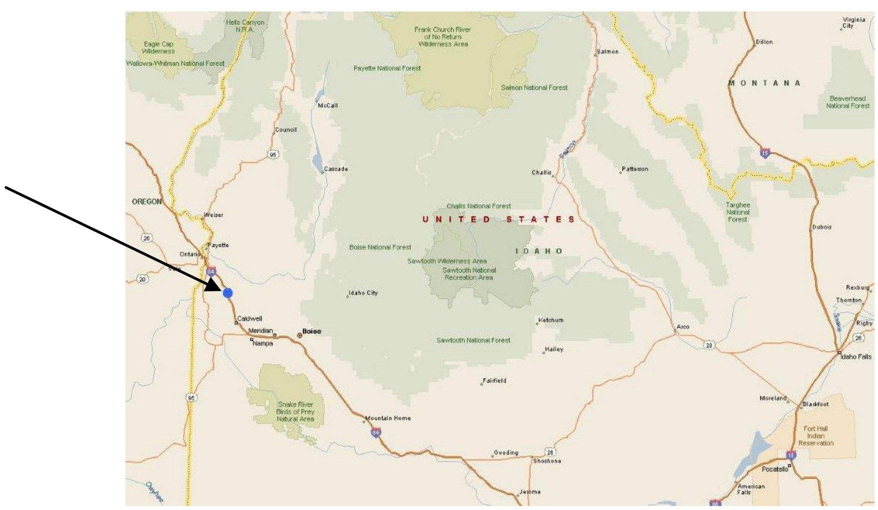
Figure 75. Optimized New HMA Pavement Design Inputs and Outputs Summary

Appendix B
Idaho New JPCP Design Example

Project Description

This design example is for the new construction of a four-lane divided JPCP located on I-84 eastbound, milepoint 15.08 to 15.18. The project location is in Payette County, Idaho, close to Ontario, Oregon, as shown in Figure 76

a. Project Location in Idaho



b. Close-Up View of Project Surroundings

Figure 76. New JPCP Design Example Location

The roadway was originally constructed in October 1983 and later adopted in the LTPP program with Site ID 3023. Average traffic (AADTT) for 1983 was 1,900 trucks with 6.4 percent linear growth. This example uses October 1983 as the construction month and December 1983 as the traffic opening month.

Pre-Design Issues

Prior to the start of design and analysis, the pavement designer must assemble all key inputs required for this pavement type and decide on the hierarchical level of inputs for each key input category. Key inputs required for new or reconstructed JPCP design are presented in Table 67. Based on the functional class (interstate) and location (rural) of the roadway under design, Level 2/3 inputs were generally assumed to be adequate.

NOTE: Inputs such as initial truck traffic volume (AADTT) and projected future growth rate must always be estimated at Level 1.

Table 67. Key Inputs Required for New or Reconstructed JPCP Design

<table>
<tr><th colspan="2">Input Category</th><th>Input Variables</th></tr>
<tr><td colspan="2" rowspan="3">General Information</td><td>Design Type & Pavement Type</td></tr>
<tr><td>Pavement Construction Date (month/year)</td></tr>
<tr><td>Traffic Opening Date (month/year)</td></tr>
<tr><td colspan="2" rowspan="2">“Design Life” & Reliability</td><td>“Design Life” (years)</td></tr>
<tr><td>Design Reliability (%)</td></tr>
<tr><td colspan="2" rowspan="4">Performance Criteria</td><td>Initial IRI (in./mile)</td></tr>
<tr><td>Terminal IRI (in./mile)</td></tr>
<tr><td>Transverse Cracking (% slabs cracked)</td></tr>
<tr><td>Transverse Joint Faulting (in.)</td></tr>
<tr><td colspan="2" rowspan="10">Traffic</td><td>Initial Two-Way AADTT</td></tr>
<tr><td>Number of Lanes</td></tr>
<tr><td>Directional Distribution</td></tr>
<tr><td>Lane Distribution</td></tr>
<tr><td>Truck Growth</td></tr>
<tr><td>Vehicle Class Distribution</td></tr>
<tr><td>Hourly Adjustments</td></tr>
<tr><td>Monthly Adjustments</td></tr>
<tr><td>Number of Axles per Truck</td></tr>
<tr><td>Axle Load Distribution</td></tr>
<tr><td rowspan="5">Structure & Materials Properties</td><td>PCC
(surface layer)</td><td>28-Day Flexural Strength, 28-Day Elastic Modulus, Coefficient Of Thermal Expansion, Cement Type, Cement Content, Water-To-Cement Ratio</td></tr>
<tr><td>Crushed Base</td><td>Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness</td></tr>
<tr><td>Granular Subbase</td><td>Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness</td></tr>
<tr><td>Compacted Subgrade</td><td>Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness</td></tr>
<tr><td>Natural Subgrade</td><td>Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness</td></tr>
<tr><td colspan="2">Project-Specific Calibration Factors</td><td>ITD Local Calibration Coefficients for New JPCP
(adopted from NCHRP 20-07/Task 288 study)</td></tr>
</table>

Develop a “Trial Design”

“Trial Design” begins with opening the software and selecting the appropriate design type and pavement type, which for this design example are “New Pavement” and “Jointed Plain Concrete Pavement (JPCP).” Additional information is presented in the *AASHTOWare Pavement ME Design* “HELP System.” Next is to create the “Trial Design” by populating several screens of the software user interface. The “Trial Design” file, once completed, must be saved and reviewed for accuracy and wrong entries. Files should be named using standard ITD conventions. For this example, the filename “New JPCP.dgpx” is used.

NOTE: The names of the output summary files will be based on the name of the input file.

Details of how the project is created and populated with "Trial Design" inputs are presented in the following sections.

"Design Life"

Table 1 of this *User's Guide* provides information on pavement "Design Life". For new or reconstructed JPCP, the recommended "Design Life" is 40 years. Thus, a 40-year "Design Life" was selected (see Figure 77).

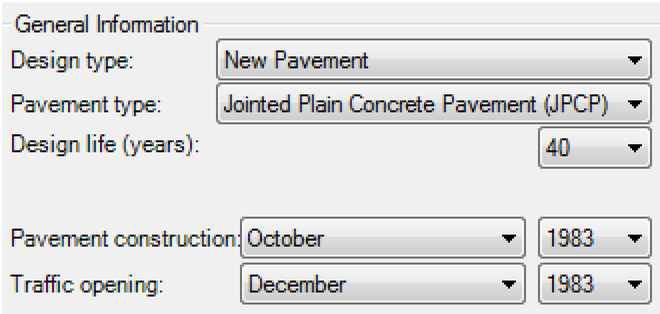


Figure 77. New JPCP Design Example Construction Month and Year

Construction and Opening Dates

AASHTOWare Pavement ME Design requires information on anticipated construction or placement date (month/year) of the PCC layer. This information is used for setting the baseline climate and traffic at construction. Anticipated month and year of PCC layer placements must be determine based on typical ITD practices (i.e., the seasons in which pavements are normally constructed). Also required are the anticipated month and year in which the completed pavement will be opened to traffic. Again, this input must be selected based on typical ITD construction practices. As shown in Figure 77, the following were selected for this example:

- PCC Layer Placement Month/Year: October 1983.
- Traffic Opening Month/Year: December 1983.

Performance Criteria & Design Reliability

Designers must set pavement performance criteria on which a "Trial Design" is accepted or rejected. Performance criteria are basically critical distress and smoothness levels that ITD allows for a given pavement type and functional class. As part of its evaluation of a "Trial Design," *AASHTOWare Pavement ME Design* predicts distress and smoothness over a specified analysis period "Design Life", and these predicted values at the end of the "Design Life" are compared to the preset threshold values. If the predicted distress and smoothness are greater than the preset threshold values, the "Trial Design" is rejected. *AASHTOWare Pavement ME Design* allows designers to predicted distress and smoothness at various levels of reliability. Chapters 3 and 4 of this *User's Guide* present guidance for selecting performance criteria and reliability levels.

For this JPCP design example, the relevant performance criteria are those recommended for an interstate highway (see Table 7). A reliability level of 95 percent was selected based on the pavement's functional class (see Table 9).

AASHTOWare Pavement ME Design requires an estimate of initial pavement smoothness (IRI right after the PCC layer placement). This is an important input, as the time from initial construction to attaining the threshold IRI value depends greatly on the initial IRI obtained at the time of construction. The initial IRI value provided in the design must be attained in the field and, thus, must reflect ITD practices. An initial IRI of 65 in./mile was selected. Designers can vary this input if there is reason to believe a different value would better reflect initial smoothness values for a given project.

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	65	
Terminal IRI (in./mile)	160	95
JPCP transverse cracking (percent slabs)	10	95
Mean joint faulting (in.)	0.12	95

Figure 78. Performance Criteria and Reliability for New JPCP Design Example

Traffic

AASHTOWare Pavement ME Design hierarchical Levels 1 through 3 define how representative traffic inputs are for a particular site. Level 1 data are considered the most representative of past and future traffic characteristics, as traffic inputs are obtained from measurements and counts of actual axle weights and truck traffic volume (from WIM sites). Statewide averages of traffic inputs computed from historical traffic data obtained from the ITD Roadway Data Section are considered Level 2 data, while Level 3 traffic inputs are national averages.

With the exception of traffic volume data, Level 2/3 traffic inputs were used in this design example. This is because the project is not considered critical based on its functional class, traffic volumes, and location. Pavement for a more critical roadway would require mostly Level 1 inputs. Regardless of how critical a design is, initial AADTT and growth rate must always be Level 1.

Traffic input data used for this design example are described in Table 68. Figures 79 and 80 show the *AASHTOWare Pavement ME Design* traffic module with key inputs populated for this example design.

Table 68. Traffic Input Data Used for This Design Example

Traffic Input Variable	Level of Input	Source & Value of Input
Initial Two-Way AADTT	1	The designer must always use Level 1 initial two-way AADTT data. Initial two-way AADTT data must be obtained from the ITD Roadway Data Section. For this example, initial two-way AADTT of 1,900 was obtained from measurement data.
Number of Lanes (in each direction)	1	Two lanes in the design direction.
Percent Trucks in Design Direction	2	ITD recommendations are provided in Table 11. The assumed statewide default is 50 percent. Designers can vary this input if there is reason to believe it would not reflect actual conditions.
Percent Trucks in Design Lane	2	ITD recommendations are provided in Table 11. For this example (four-lane divided roadway), 90 percent trucks in the design lane was selected.
Operational Speed	2	Posted speed limit for this highway & location was 65 mph.
Axle Configuration & Lane Wander	3	Level 3 defaults (national averages) were assumed, as there are no ITD-specific recommendations.
Wheelbase	3	National averages of 17, 22, & 61 percent, respectively, were assumed for the percentage of trucks with short-, medium-, & long-axles. Recommendations are provided in Table 34.
Vehicle Class Distribution	1	Project-specific vehicle class distribution was obtained from Table 13. The project location is very close to WIM Site No. 128.
Truck Traffic Growth	1	The designer must always use Level 1 inputs obtained from the ITD Roadway Data Section. For this example, a growth rate of 6.4 percent linear over 40-year "Design life" was obtained based on historical data.
Monthly Adjustment	2	Idaho statewide average vehicle class distribution was obtained from Table 12.
Axles per Truck	2	Idaho statewide average vehicle class distribution was obtained from Table 12.
Hourly Distribution	2	Idaho statewide average hourly distribution was obtained from Table 14.
Axles Load Distribution	2	Idaho statewide average axle load distribution for primarily (heavily) loaded traffic was obtained from Tables 19 through 22.

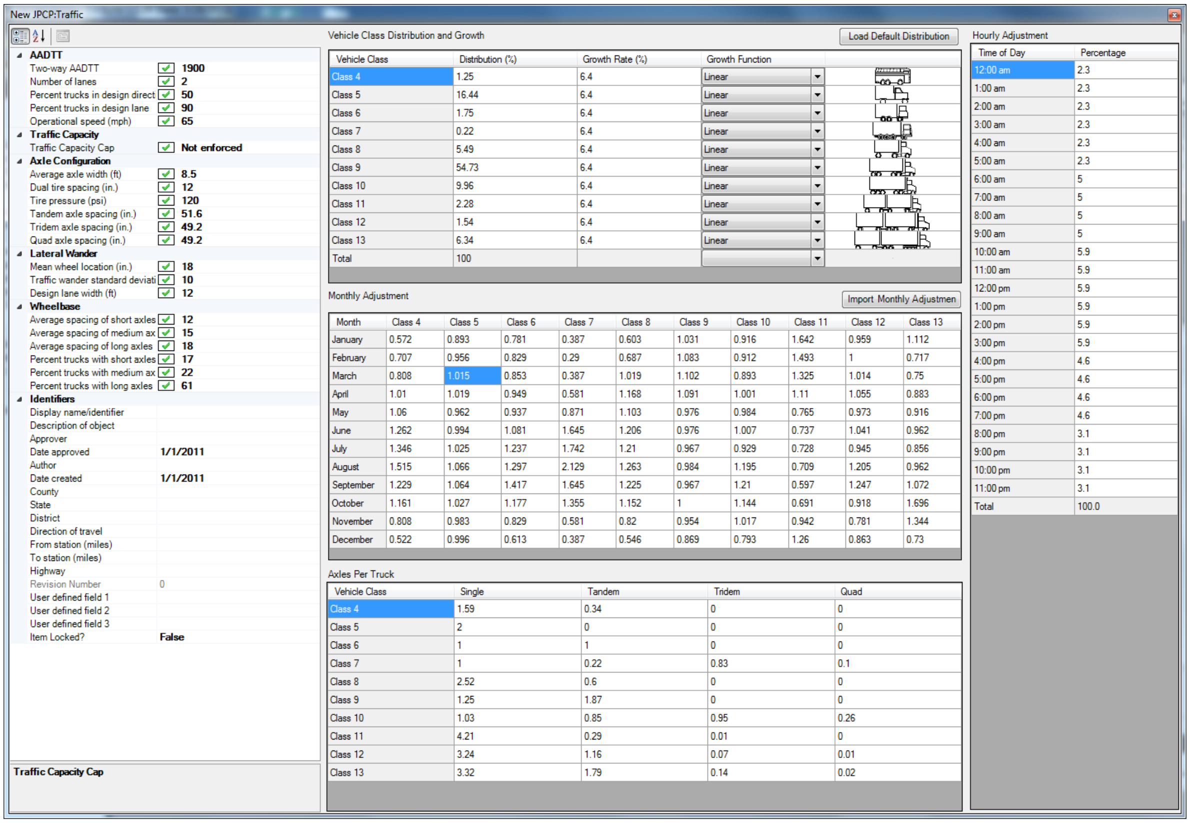


Figure 79. General Traffic Inputs for New JPCP Design Example

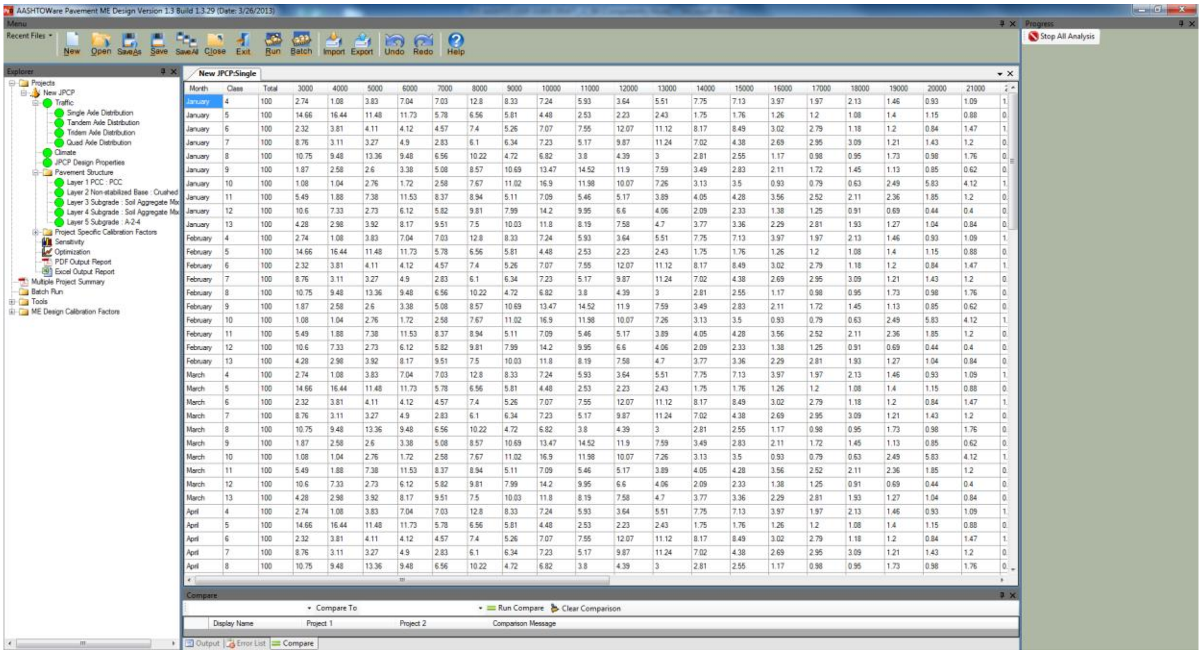


Figure 80. Single Axle Load Distribution Inputs for New JPCP Design Example

Climate

AASHTOWare Pavement ME Design requires historical climate data to simulate temperature and moisture conditions within the "Trial Design" structure. For most design situations, climate data available at Level 2 and embedded in the software are adequate. The software allows users to create their own weather stations from which project-specific climate data can be obtained. The methodology for creating project-specific weather stations is presented in the *AASHTOWare Pavement ME Design* "HELP System."

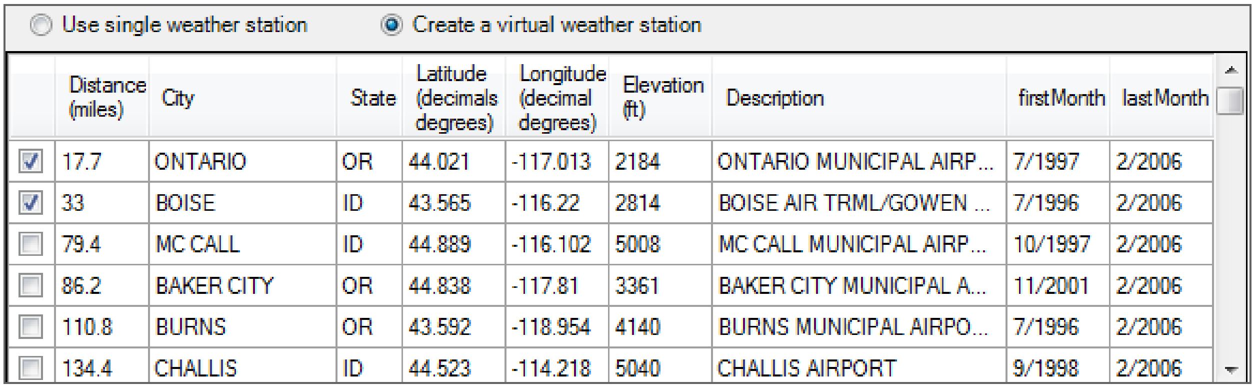
For this design example, climate data were obtained from weather stations located in Idaho or nearby States. As noted, historical climate data from these weather stations are included in the software. Designers can query the *AASHTOWare Pavement ME Design* climate data to obtain information on the appropriate weather station or stations to use for design analysis. This requires the designer to produce project location coordinates (i.e., latitude and longitude) and elevation. For existing alignments, project-specific location coordinates and elevation typically are available from project design documents, plan sheets, or the ITD TAMS database. Online tools such as Google Earth, Map Point, and Google Map also could provide the information required.

AASHTOWare Pavement ME Design uses project location coordinates to identify nearby weather stations for use in creating a Level 2 virtual weather station. Once the nearby weather stations are identified, the designer can selected as many weather stations as desired to create the virtual weather station. Care must be taken to ensure that the selected weather stations are representative of conditions at the project location. In addition to project coordinates and elevation, the designer must also provide an estimate of the locations depth to water table. This is mostly determined based on the designer's local experience or historical data available from nearby wells.

For this example project, the project coordinates and elevation are as follows:

- Latitude: +43.84 decimal degrees.
- Longitude: -116.76 decimal degrees.
- Elevation: 2,503 feet.

A depth to water table of 10 feet was assumed. Based on the location of the project, it was determined that the closest weather station was approximately 17 miles away, another was within a 33 mile radius. Thus, these two weather stations were selected for use in creating the project virtual weather station. Figure 81 presents the *AASHTOWare Pavement ME Design* inputs for climate for this design example.


○ Use single weather station ◉ Create a virtual weather station

	Distance (miles)	City	State	Latitude (decimals degrees)	Longitude (decimal degrees)	Elevation (ft)	Description	firstMonth	lastMonth
☑	17.7	ONTARIO	OR	44.021	-117.013	2184	ONTARIO MUNICIPAL AIRP...	7/1997	2/2006
☑	33	BOISE	ID	43.565	-116.22	2814	BOISE AIR TRML/GOWEN ...	7/1996	2/2006
☐	79.4	MC CALL	ID	44.889	-116.102	5008	MC CALL MUNICIPAL AIRP...	10/1997	2/2006
☐	86.2	BAKER CITY	OR	44.838	-117.81	3361	BAKER CITY MUNICIPAL A...	11/2001	2/2006
☐	110.8	BURNS	OR	43.592	-118.954	4140	BURNS MUNICIPAL AIRPO...	7/1996	2/2006
☐	134.4	CHALLIS	ID	44.523	-114.218	5040	CHALLIS AIRPORT	9/1998	2/2006

Figure 81. Selecting Virtual Weather Stations for New JPCP Design Example

Pavement Structure

AASHTOWare Pavement ME Design allows for the design of new or reconstructed JPCP with three base types: dense graded aggregate, cement stabilized or lean concrete, and asphalt treated materials. Selection of the base type must be based on ITD practices and policy. For this design example, JPCP constructed over an aggregate base course placed over the granular subbase and natural subgrade was selected.

Based on ITD's JPCP design philosophy, JPCP over an aggregate base typically will consist of 5 to 6 layers, as shown in Figure 82.

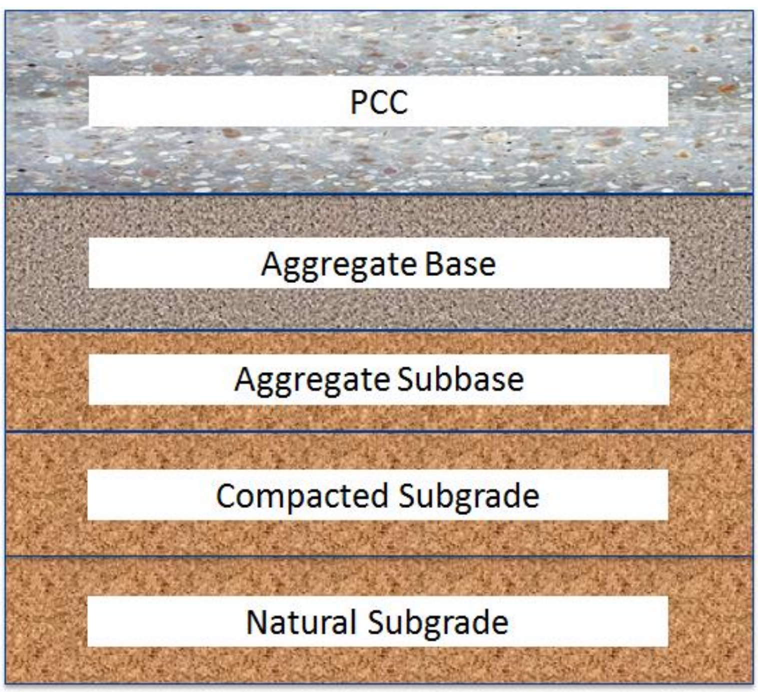


Figure 82. New JPCP Design Structure

The general description of pavement layer structure, starting from the bottom foundation support, is described in detail as follows:

- **Bedrock:** Highly fractured and weathered or massive continuous (intact) rock within 10 to 20 feet of the pavement foundation, if present.

- **Natural Subgrade:** The nature of the subgrade foundation (including depth to bedrock and groundwater table) is mostly determined directly from subsurface exploration and testing activities. Key for pavement design is to determine the natural/compacted subgrade properties and depth, as well as the depth to bedrock. Natural and compacted subgrade soil properties are obtained from tests on the natural foundation soil in place and in its compacted state as the upper layers (12 to 24 in.) are rolled and compacted or removed and replaced during construction.

- **Compacted or Prepared Subgrade:** This is typically wetting, rolling, and compacting the top 12 to 24 in. surface of the natural subgrade to produce a firm, compact surface with sufficient strength to support construction equipment and other activities. Subgrade preparation may also include stabilization with lime or other chemicals to reduce plasticity, improve workability, minimize shrinkage/swell, increase compressive strength CBR and M_r, and provide long-term durability in very adverse conditions.

- **Granular Subbase:** ITD specifies a range of aggregate/granular materials for use as subbase materials. The materials are mostly classified as AASHTO A-1-a and A-1-b soils.

- **Crushed Base**: ITD specifies a range of aggregate/granular materials for use as base materials. The materials are mostly classified as AASHTO A-1-a and A-1-b soils.

- **PCC Layer:** ITD specifies a single PCC layer.

Guidance for obtaining pavement layer properties and thicknesses to define the trial JPCP structure is presented in Section 10.1 of this *User's Guide*. For this design example, a Level 2/3 hierarchical input was adopted. *AASHTOWare Pavement ME Design* recommends that, once the "Trial Design" (layer types and initial thicknesses) is defined, material properties must be populated starting from the lowest layer bedrock or natural subgrade to the surface layer.

The *AASHTOWare Pavement ME Design* "HELP System" provides detailed guidance on how to enter pavement structure and layer materials data.

Bedrock

Review of historical subsurface exploration and testing reports for this location showed there was no bedrock within a 50-ft depth. Thus, a bedrock layer was not needed.

Natural Subgrade

Subsurface exploration and testing reports indicate the natural subgrade for this location is AASHTO A-2-4 soil. Engineering properties required at Level 2/3 for the natural subgrade are as presented in Table 69.

Table 69. Required Engineering Properties for the "Trial Design" Natural Subgrade

Engineering Properties	Level of Input	Source of Data
Gradation	2	Obtained through subsurface exploration & testing
Atterberg Limits (Liquid Limit & Plasticity Index)	2	Obtained through subsurface exploration & testing
Maximum Dry Unit Weight	3	Computed internally by *AASHTOWare Pavement ME Design*
Saturated Hydraulic Conductivity	3	Computed internally by *AASHTOWare Pavement ME Design*
Specific Gravity of Solids	3	Computed internally by *AASHTOWare Pavement ME Design*
Optimum Gravimetric Water Content	3	Computed internally by *AASHTOWare Pavement ME Design*
Soil Water Characteristic Curve	3	Computed internally by *AASHTOWare Pavement ME Design*
Resilient Modulus (M_r)	2	Back-calculated elastic modulus from FWD deflection testing data then converted field modulus to laboratory condition. (see Table 37)

Figure 83 shows the subgrade engineering properties (gradation and Atterberg limits) obtained from laboratory testing, which are coded into the software. Based on these two properties, the software internally estimates maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum gravimetric water content, and the soil water characteristic curve. The designer must check the estimated soil engineering properties for accuracy and reasonableness. If the *AASHTOWare Pavement ME Design* estimates are deemed not reasonable, the designer can override the internally estimated values. Guidance for overriding the engineering properties is provided in the *AASHTOWare Pavement ME Design* "HELP System."

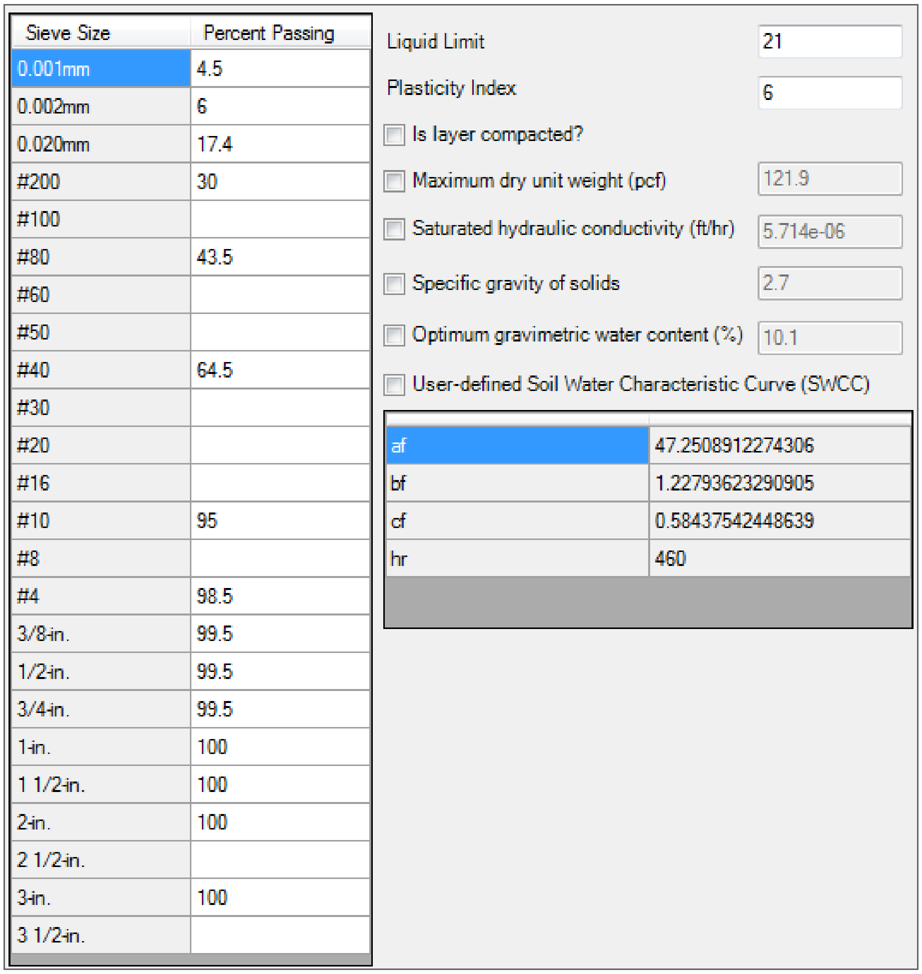


Figure 83. Natural Subgrade Engineering Properties Input Screen for New JPCP

For this example, the back-calculated elastic modulus was 23,880 psi. The corrected M_r of 23,880*0.67 = 16,000 psi at optimum moisture content was entered into the program, as shown in Figure 84. Again, the designer can override this value if warranted.

NOTE: The “layer compacted” box on the input screen was uncheck to reflect field conditions (as the natural subgrade layer is not compacted). The thickness of the natural subgrade layer is semi-infinite, as there was no underlying layer or bedrock.

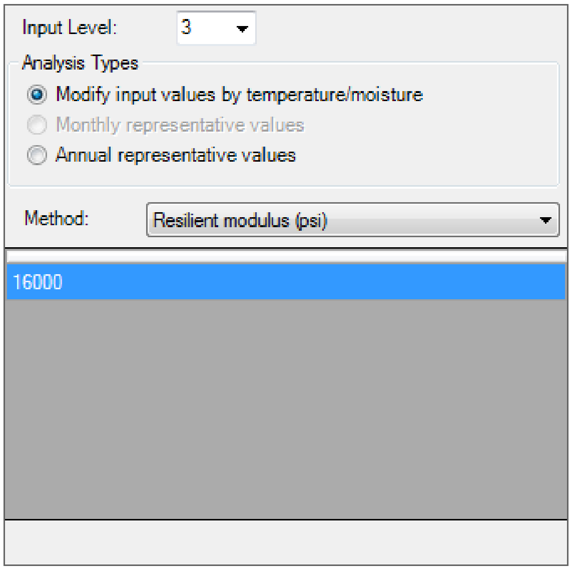


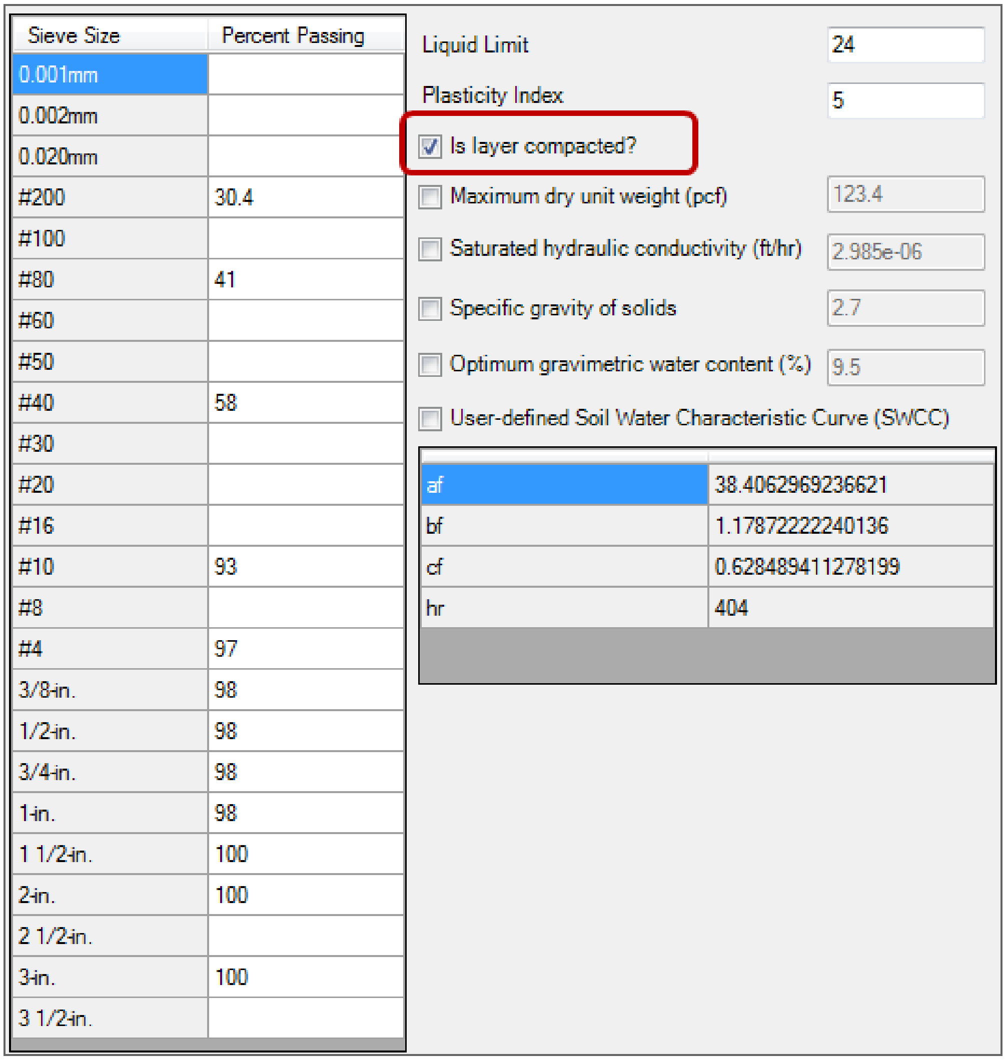
Figure 84. Natural Subgrade Level 3 Resilient Modulus Input Screen for New JPCP

Compacted Subgrade Layer

Records indicate 9 inches of the natural subgrade was rolled and compacted. This layer was not chemically treated. The engineering properties and M_r for this layer were similar to the natural subgrade. The M_r was assumed as 20,000 psi. The main distinction between these layers is that the "Layer Compacted" box on the input screen was checked to reflect field conditions (rolled and compacted subgrade layer). See Figure 85. In addition, a layer thickness of 9 inches was entered into *AASHTOWare Pavement ME Design*.

Granular Subbase Layer

For this example, 5.3 inches of granular subbase was determined from borings. This layer was not chemically treated. The engineering properties and M_r for this layer were similar to the natural subgrade. The M_r was assumed as 25,000 psi. The "Layer Compacted" box on the input screen was check to reflect field conditions (rolled and compacted layer). See Figure 86. Also, a layer thickness of 5.3 inches was entered into *AASHTOWare Pavement ME Design*.



Sieve Size	Percent Passing
0.001mm	
0.002mm	
0.020mm	
#200	30.4
#100	
#80	41
#60	
#50	
#40	58
#30	
#20	
#16	
#10	93
#8	
#4	97
3/8-in.	98
1/2-in.	98
3/4-in.	98
1-in.	98
1 1/2-in.	100
2-in.	100
2 1/2-in.	
3-in.	100
3 1/2-in.	

Liquid Limit	24
Plasticity Index	5
Is layer compacted?	☑
Maximum dry unit weight (pcf)	123.4
Saturated hydraulic conductivity (ft/hr)	2.985e-06
Specific gravity of solids	2.7
Optimum gravimetric water content (%)	9.5
User-defined Soil Water Characteristic Curve (SWCC)	☐

af	38.4062969236621
bf	1.17872222240136
cf	0.628489411278199
hr	404

Figure 85. Compacted Subgrade Engineering Properties for New JPCP

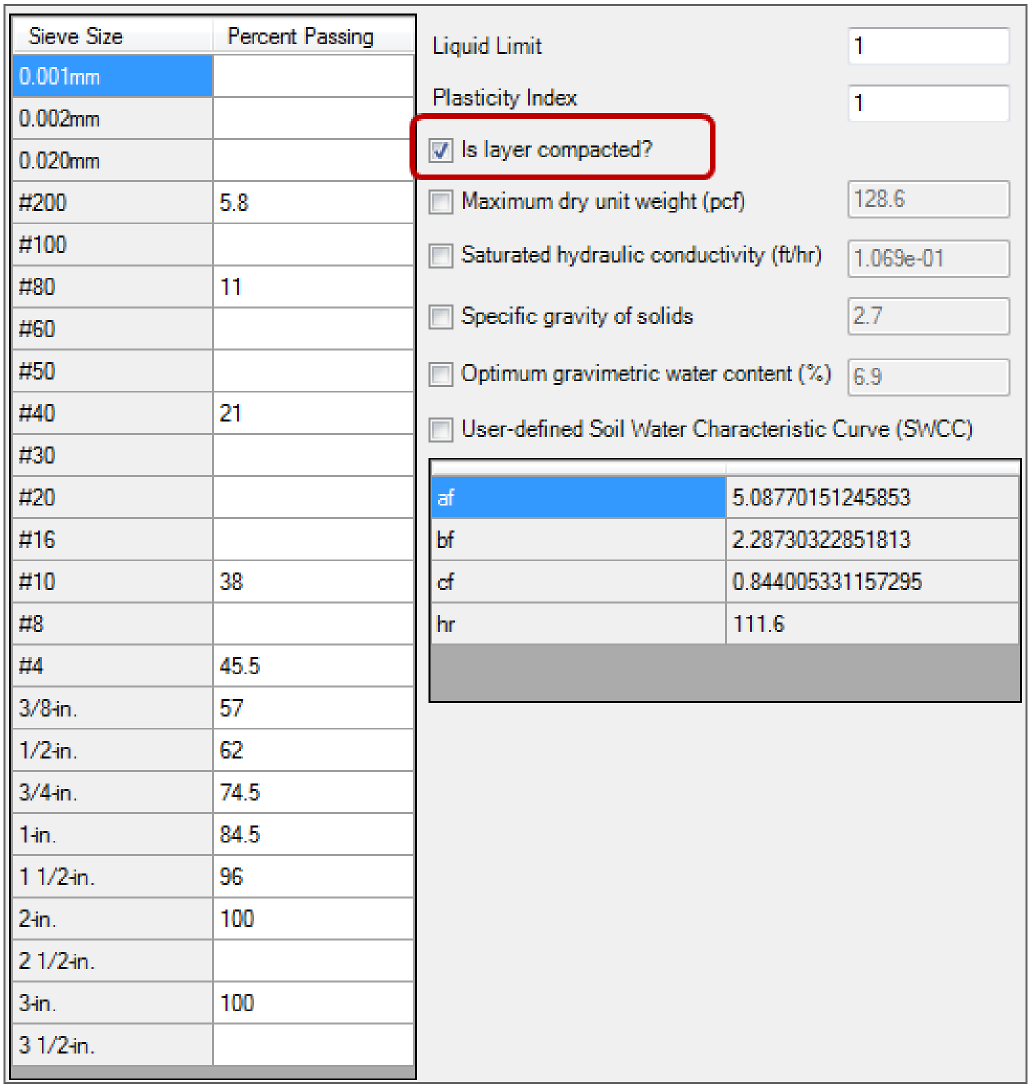


Figure 86. Granular Subbase Engineering Properties for New JPCP

Crushed Gravel Base Layer

The aggregate base material type for use in pavement construction is selected based on the project location (mostly, the nearest source of high-quality aggregate material is selected). The ITD "Crushed Gravel" material classification and properties are determined based on the source of the material.

For this design example, the material classification was obtained from borings. As shown in Table 39 of this *User's Guide*, crushed gravel has properties comparable to AASHTO soil class A-1-a, and M_r at optimum moisture for this material is assumed to be 40,000 psi.

Once the base modulus is selected, the designer can enter the aggregate base engineering properties and M_r into *AASHTOWare Pavement ME Design*. For this example, AASHTO A-1-a soil gradation and Atterberg limits were determined from previous borings. A layer thickness of 4.4 inches was the thickness entered into the software.

PCC Layer

The required inputs and values entered for this design example are presented in Table 70.

NOTE: The input requirements were for hierarchal Level 2/3 (default inputs based on ITD practices and policy).

Designers are encouraged to use the best estimates of inputs available. Thus, any or all of these inputs can be replaced if Level 1 data are available. Figures 87 through 89 presents the *AASHTOWare Pavement ME Design* screens used to enter PCC material properties for the PCC layer.

Table 70. Required Engineering Properties for the "Trial Design" PCC Layer

Engineering Properties	Level of Input	Source of Data
Layer Thickness	1	ITD design policy recommends a minimum total PCC thickness of 9 inches. Thus, 9 inches was assumed for "Trial Design."
Flexural Strength (M_r)	1	Flexural strength value from material testing.
Elastic Modulus	1	Elastic modulus value from material testing.
Unit Weight (pcf)	1	Unit weight of 140.5 from material testing.
Poisson's Ratio	1	Poisson's ratio of 0.16 from material testing.
CTE (per °F)	1	CTE of 4.31 from material testing.
Cement Type	1	Type II cement type was specified.
Cementitious Material (PCC + pozzolans) (lb/yd^3)	1	564 was specified.
Water-to-Cement Ratio (w/c)	1	0.4 was typical.
Coarse Aggregate Type	3	Not required.
PCC Zero Stress Temperature, (°F)	3	Computed internally in the software.
Ultimate Shrinkage, Microstrain	3	Computed internally in the software.
Reversible Shrinkage	3	Use *AASHTOWare Pavement ME Design* default of 50% unless more accurate information is available.
Time to Develop 50% of Ultimate Shrinkage	3	Use *AASHTOWare Pavement ME Design* default of 35 days unless more accurate information is available.
Curing Method	3	Curing compound.
Thermal Conductivity	3	Assume *AASHTOWare Pavement ME Design* defaults.
Heat Capacity	3	Assume *AASHTOWare Pavement ME Design* defaults.

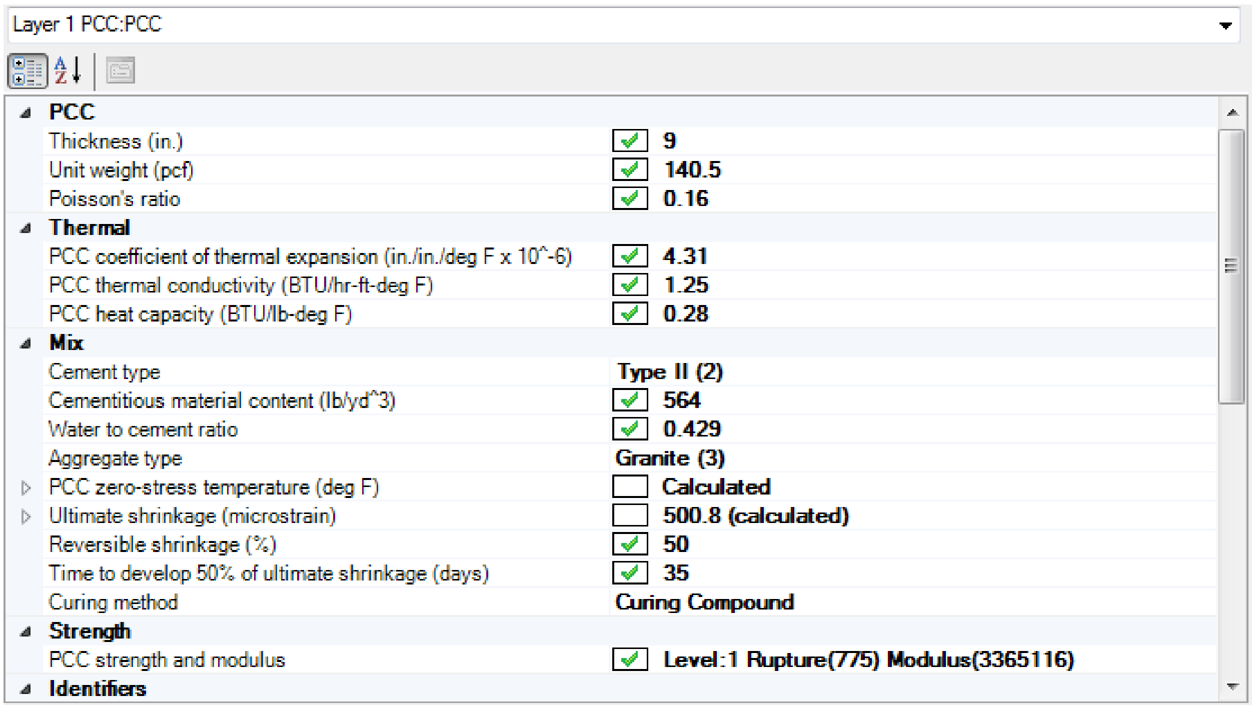


Figure 87. Screen for PCC Material for the Example New JPCP Design

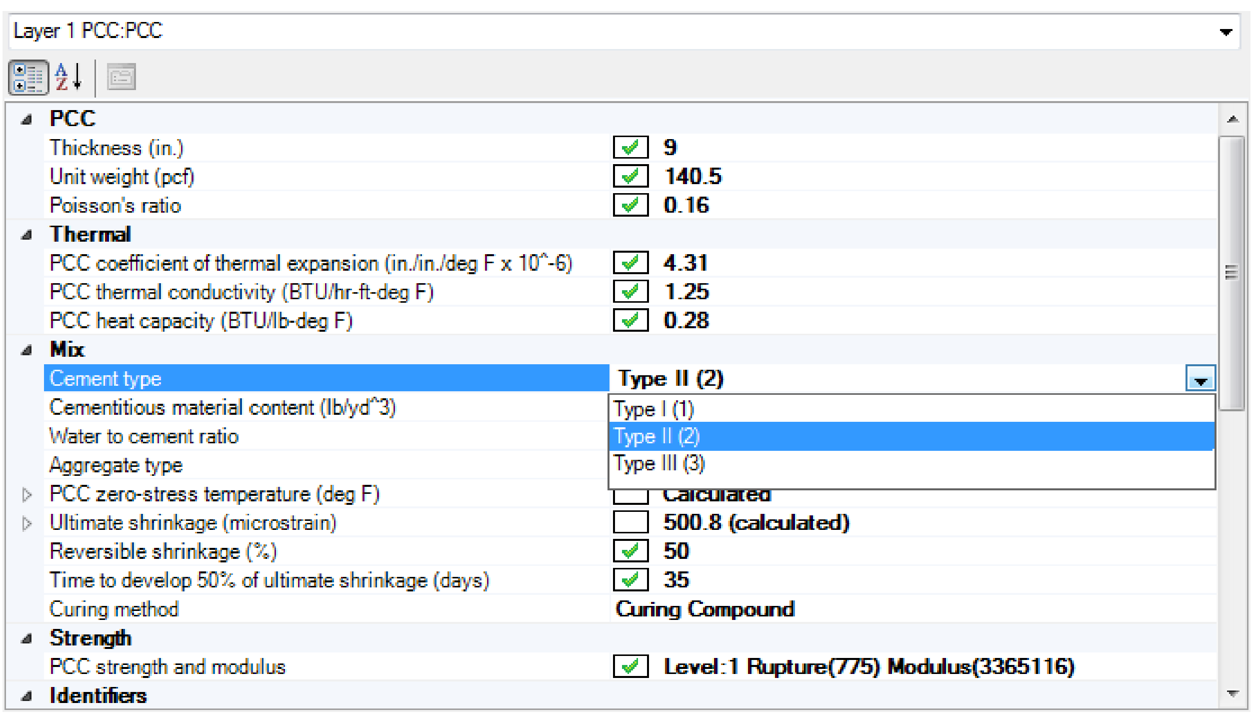


Figure 88. Screen for Cement Type for the Example New JPCP Design

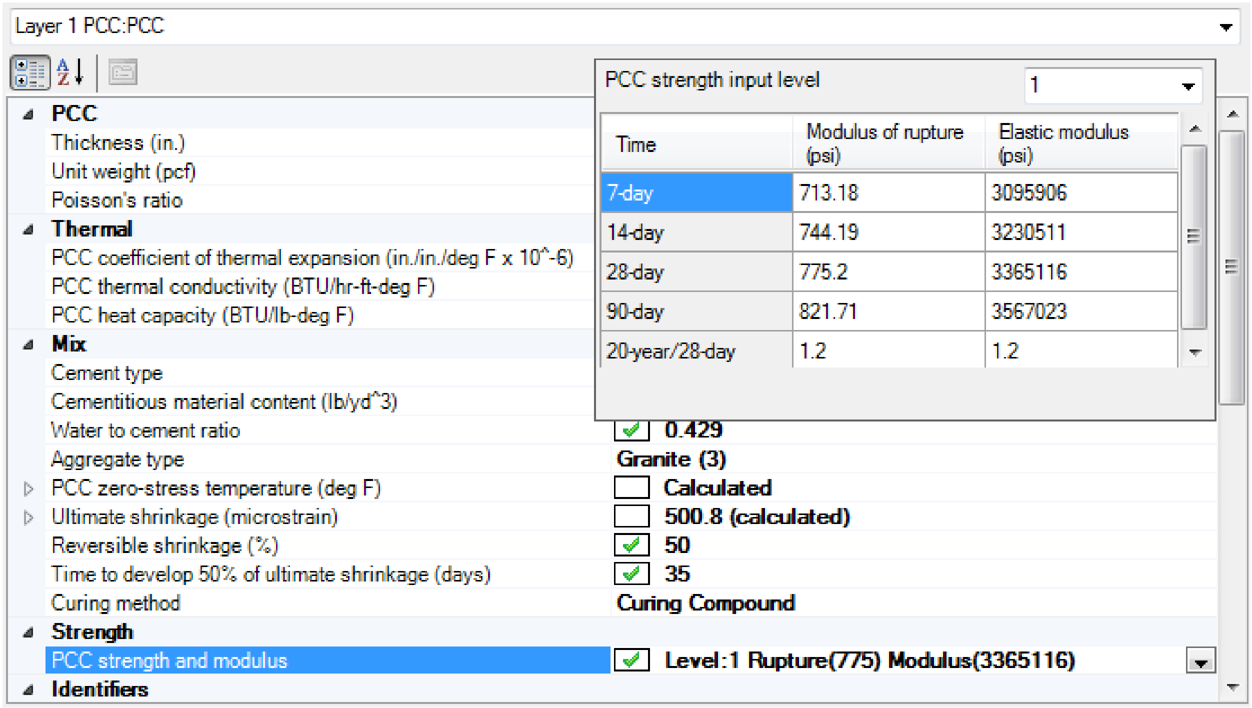


Figure 89. Screen for PCC Strength and Modulus for the Example New JPCP Design

JPCP Design Inputs

Selecting representative design features for the "Trial Design" is very important. Table 71 presents a summary of the design inputs selected for this project based on recommendations presented in the *User's Guide* and ITD policy and practices.

Table 71. Summary of New JPCP "Trial Design" Inputs

Trial JPCP Design Inputs	Recommended Values
Permanent Curl/Warp Effective Temperature Difference (°F)	-10
Joint Spacing (ft)	Project-specific joint spacing of 15 feet was selected.
Sealant Type	Silicone.
Load Transfer Mechanism (round dowel bars)	ITD recommends the use of dowels for all designs. For this example, no dowels were assumed as an initial trial to see what would be predicted. NOTE: Based on joint faulting predictions for this "Trial Design;" the designer may have to reconsider this option.
Dowel Diameter (in.)	Minimum dowel diameter must be used in order to pass the faulting performance criteria at the design reliability level, (if needed).
Dowel Bar Spacing (in.)	12 in. (if needed).
Edge Support	Conventional 12-ft slab with tied PCC shoulders.
Base Type	Crushed gravel base material.
PCC-Base Interface Friction	The following lengths of time for full contact friction between the PCC slab & base course are recommended by ITD & thus adopted for this "Trial Design": For unbound crushed gravel base, enter full friction over the design analysis period (480 months).
Erodibility Index of Base	Granular aggregate base provides a value of 4 (i.e., fairly erodible material).

Designers can override any of these additional inputs if warranted. JPCP design features used in this example are shown in Figure 90.

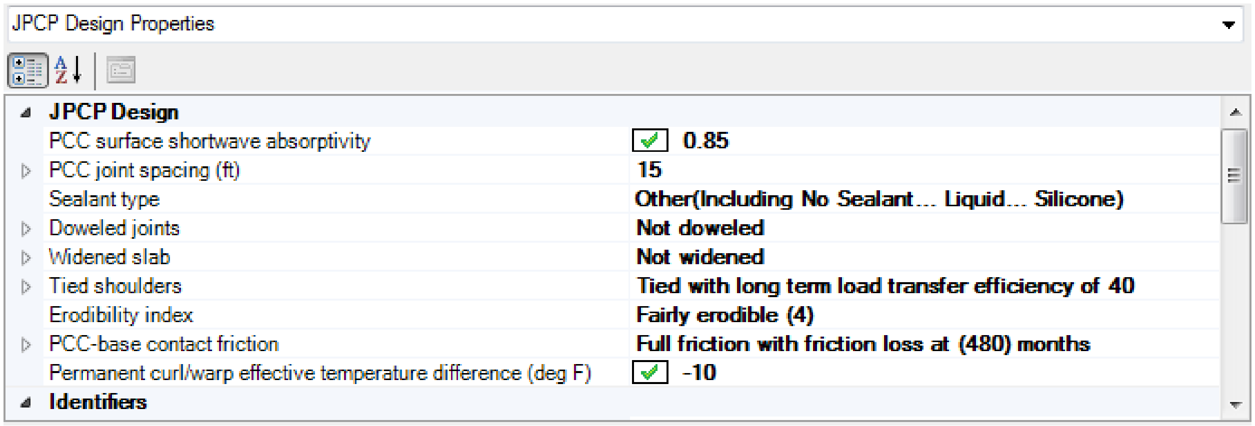


Figure 90. Screen for New JPCP Design Inputs

ITD New JPCP Project-Specific Calibration Coefficients

When *AASHTOWare Pavement ME Design* is used for Idaho conditions, ITD recommends the calibration coefficients presented in Figure 91 for distress and IRI models. These coefficients were adopted from the NCHRP 20-07/Task 288 study and should be used until ITD establishes its own local calibration coefficients. Designers must check if the new JPCP project being designed is applying the calibration coefficients presented. If not, guidance is provided in the *AASHTOWare Pavement ME Design* "HELP System" on how to replace the global calibration coefficients with ITD recommended values.

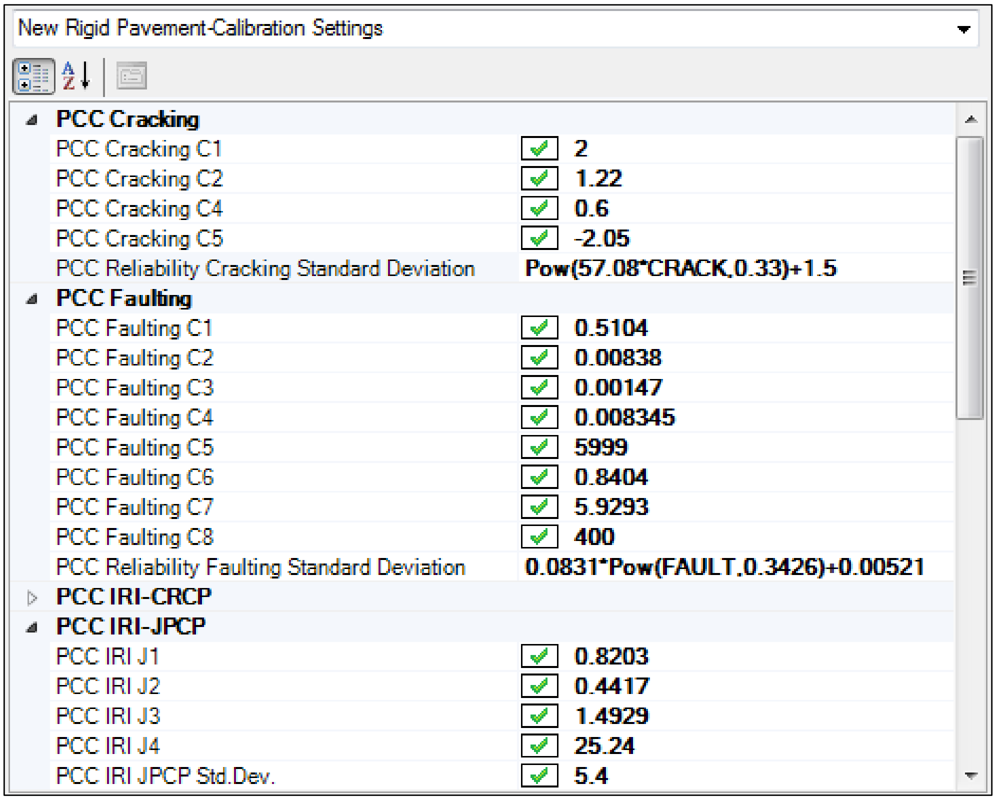


Figure 91. New JPCP Calibration Coefficients

Run *AASHTOWare Pavement ME Design* and Review/Analyze Outputs

Pavement design using *AASHTOWare Pavement ME Design* is iterative. The designer must run the software, check key outputs for reasonableness, and check the "Trial Design" for adequacy - it should be able to carry anticipated traffic over its "Design Life" at the preset performance criteria recommended by ITD. Check for adequacy must be done at the ITD recommended reliability level. If the "Trial Design" is deemed inadequate, appropriate modifications must be made to obtain a feasible "Final Design."

Check of Key *AASHTOWare Pavement ME Design* Outputs for Reasonableness

It is important for designers to review key inputs and outputs for each *AASHTOWare Pavement ME Design* run to ascertain whether inputs were entered correctly and the software processed input data correctly and produced expected results (e.g., the climate statistics produced by the software correspond to what is expected of the given location). *AASHTOWare Pavement ME Design* produces two output files with a summary of key inputs and design outcomes The .xls file contains significantly more detailed information which may not be needed for this review. The information contained in the PDF output file is deemed adequate and is presented under the following general headings:

- **Design Inputs:** Contains information about key designs inputs such as pavement structure definition, layer thicknesses, and traffic projections (see Figure 92).

- **Design Outputs:** Distress prediction summary in tabular and graphical forms (See Figure 93).

- **Traffic Input and Output Summary:** Graphical and tabular representation of key traffic inputs and projected growth and seasonal adjustments (see Figures 94 and 95):
    - Traffic Distributions: Tabular representation of traffic inputs.
    - Axle Configuration: Axle configuration summary.
    - AADTT Growth: Plots showing trends in truck growth.
    - AADTT Growth by Class: Tabular representation of truck growth.

- **Climate Inputs and Output Summary**: Graphical and tabular representation of key climate inputs and climate variable statistics (see Figure 96):
    - Climate data sources (weather stations).
    - Annual statistics of key variables such as temperature, precipitation, freezing index, etc.
    - Monthly statistics of key variables such as temperature, precipitation, freezing index, etc. in a graphical format.

- **JPCP Design Features:** Key pavement design input summary information.

- **Key PCC, Base, Subbase, and Subgrade Material Inputs and Computed Parameters**: including plots of seasonal effects on base and subgrade.

- **Analysis Output Charts:**
    - Plots of predicted IRI, transverse cracking, and faulting versus age in graphical format.
    - Detailed breakdown of predicted distress and IRI:
        - Plots of predicted bottom-up and top-down damage versus age in graphical format.
        - LTE versus age.

- **Layer Information:** Detailed summary of data for all layers within the pavement structure.

- **Calibration Coefficients:** Detailed summary of project-specific distress/IRI models calibration coefficients.

Designers are encouraged to examine the information presented under these headings. Possible discrepancies between input data summaries and what was entered into *AASHTOWare Pavement ME Design* must be resolved.

Design Inputs

Design Life:	40 years	Existing construction:	-	Climate Data Sources	44.021, -117.013
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	October, 1983		43.565, -116.22
		Traffic opening:	December, 1983		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	PCC	9.0
NonStabilized	Crushed Gravel	4.4
Subgrade	Soil Aggregate Mixture	5.3
Subgrade	Soil Aggregate Mixture	9.0
Subgrade	A-2-4	Semi-infinite

Joint Design:

Joint spacing (ft)	15.0
Dowel diameter (in.)	-
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
1983 (initial)	1,900
2003 (20 years)	10,043,400
2023 (40 years)	28,081,500

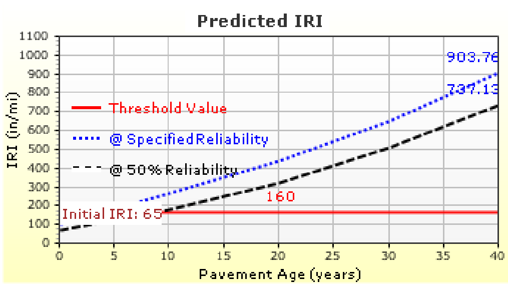
Figure 92. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for New JPCP

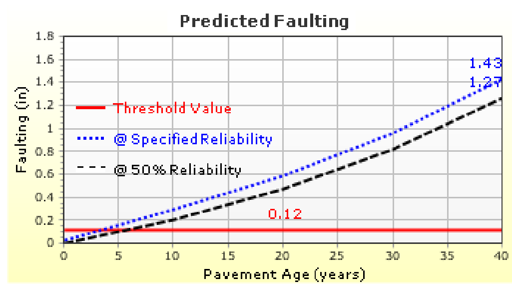
Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	903.76	95.00	0.00	Fail
Mean joint faulting (in.)	0.12	1.43	95.00	0.00	Fail
JPCP transverse cracking (percent slabs)	10.00	7.76	95.00	98.42	Pass

Distress Charts





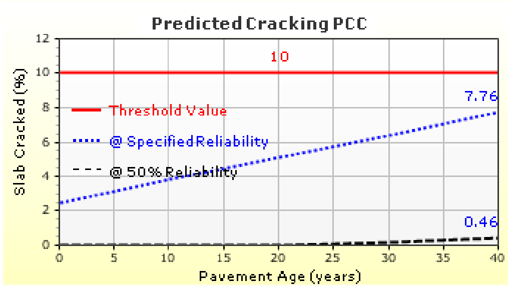


Figure 93. *AASHTOWare Pavement ME Design* PDF Output File Summary of Design Outputs for New JPCP

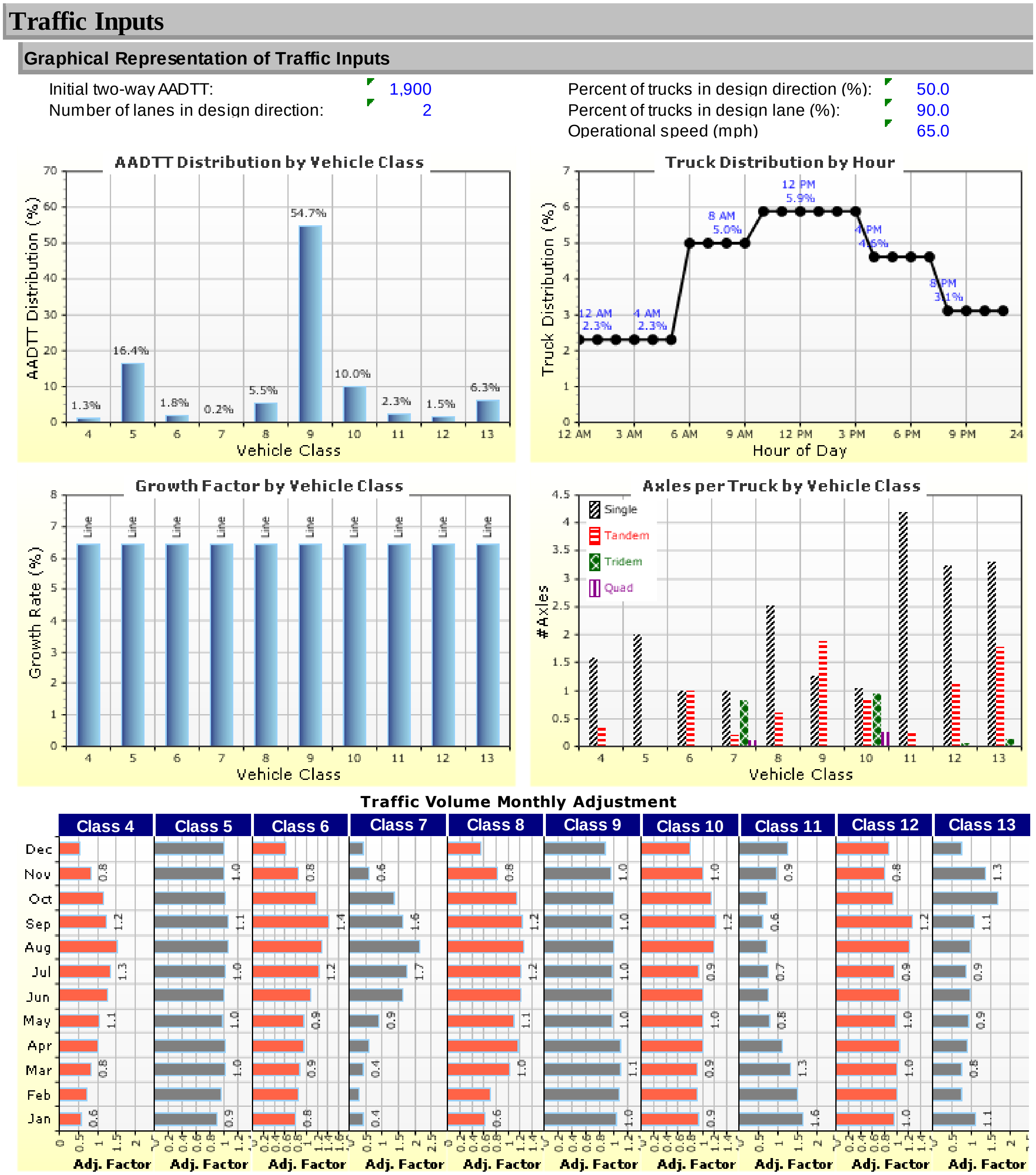


Figure 94. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for New JPCP

AADTT (Average Annual Daily Truck Traffic) Growth

*** Traffic cap is not enforced**

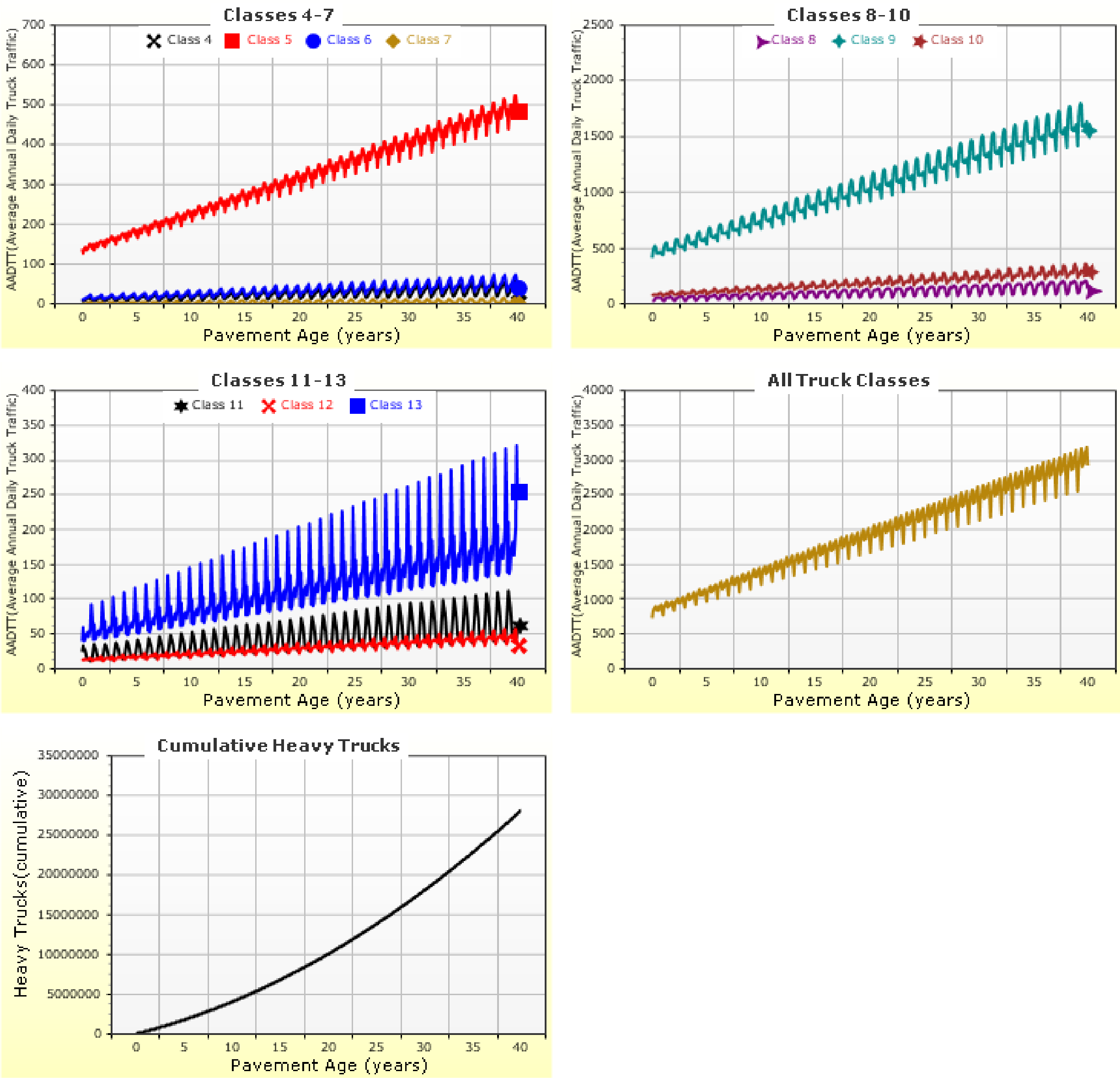


Figure 95. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Outputs (Projection of AADTT) for New JPCP

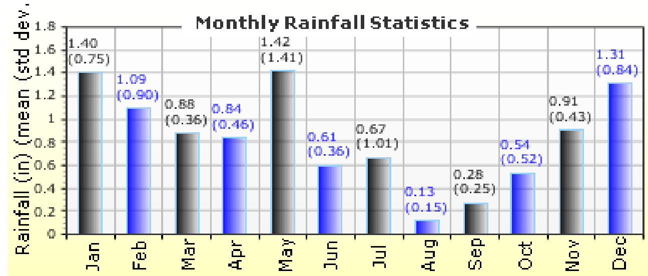
Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
ONTARIO, OR	44.02100 -117.01300 2184
BOISE, ID	43.56500 -116.22000 2814

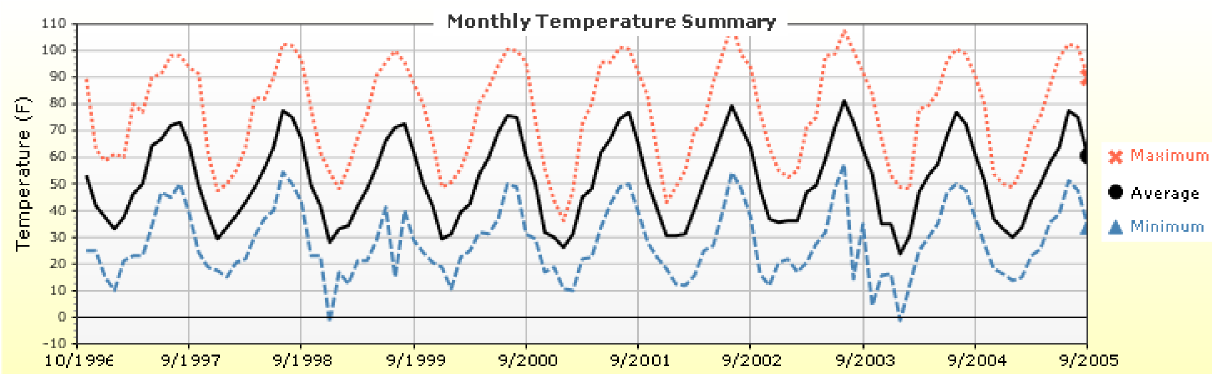
Annual Statistics:

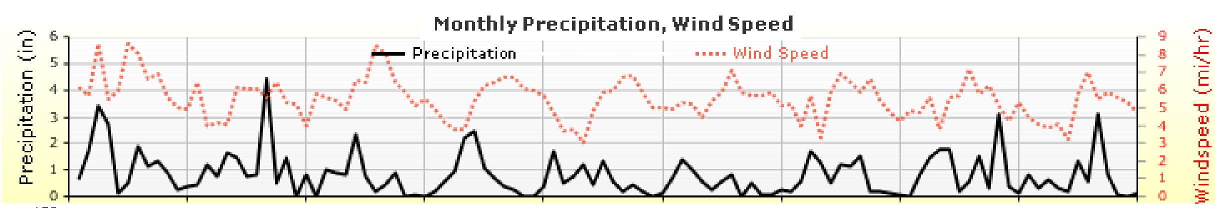
Mean annual air temperature (ºF)	51.83
Mean annual precipitation (in.)	10.12
Freezing index (ºF - days)	314.44
Average annual number of freeze/thaw cycles:	102.20

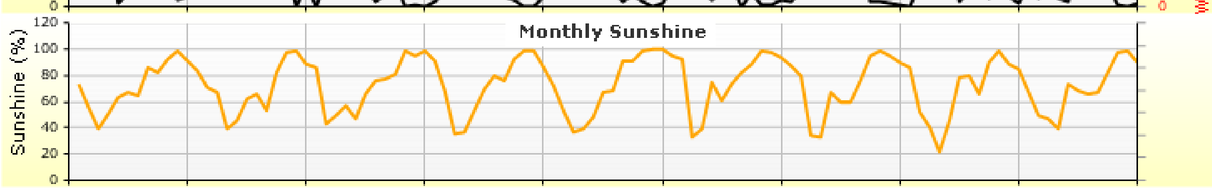


Water table depth(ft)	10.00

Monthly Climate Summary:







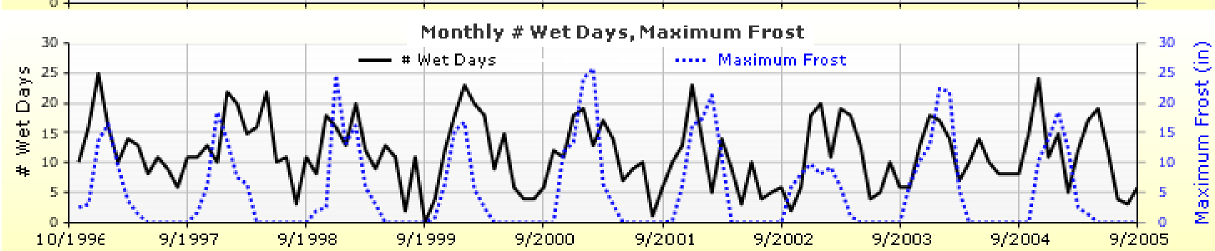


Figure 96 ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Climate Inputs and Outputs for New JPCP**

Check "Trial Design" for Adequacy

The final step is to check the "Trial Design" for adequacy. It should be able to carry anticipated traffic over its "Design Life" at the preset performance criteria recommended by ITD. The outcome of this example "Trial Design" is presented in Figure 93.

Designers must check this output summary to determine whether the design performance criteria are satisfied. This is done as follows:

1. Review the column called Criterion Satisfied? In the tabular output and determine for each distress type when the "Trial Design" "passed" or "failed."

2. If the "Trial Design" passes the criteria set for all distress types, then the design is deemed adequate and acceptable.

3. If one or more of the criteria "fail," then the design is inadequate and the "Trial Design" must be revised as needed and checked again.

For this example, the "Trial Design" did not meet the performance criteria for faulting and IRI. Revision is required.

Revise "Trial Design" and Rerun *AASHTOWare Pavement ME Design* as Needed

With the causes of "Trial Design" pavement inadequacy determined (failed to meet faulting and IRI performance criteria), the designer must determine reasons for failure to meet the performance criteria and adopt feasible solutions to improve the "Trial Design". For this design example, common reasons for not meeting faulting and IRI performance criteria were as follows:

- **Excessive Faulting:** Faulting has a significant impact on IRI. Various studies have reported that once faulting exceeds 0.15 inches, pavement smoothness will be adversely affected. Excessive faulting can be controlled by using appropriately sized dowels and by providing adequate levels of edge support. For this "Trial Design", faulting was considered excessive (predicted faulting at 95 percent reliability after 40 years was 1.43 inches).

- **Transverse Cracking:** A significant amount of high-severity transverse cracking has a significant impact on IRI, as such cracks tend to be highly faulted. Reducing the occurrence of such cracks through the use of thicker PCC slabs, higher flexural strength, and low modulus and CTE could help in reducing IRI. For this "Trial Design", transverse cracking was not considered excessive (predicted transverse cracking at 95 percent reliability after 40 years was 7.76 percent).

- **IRI:** Roughness can be minimized by constructing smoother pavements or minimizing the development and deterioration of the distress.

As needed, the designer can adjust the "Trial Design" PCC thickness or design properties listed above to meet performance criteria.

A careful review of this new JPCP "Trial Design" indicated that excessive IRI may be due mostly to excessive faulting. Applying stricter quality controls and incentive-based contracting methods could impact initial IRI, making initial IRI values of 50 in./mile achievable. The use of such methods is worth investigating. For this "Trial Design", however, a more appropriate solution is to reduce faulting by applying 1.25 in.-diameter dowels. This was done using the software thickness optimization tool (see Figure 97). A detailed description of this tool is provided in the *AASHTOWare Pavement ME Design* "HELP System."

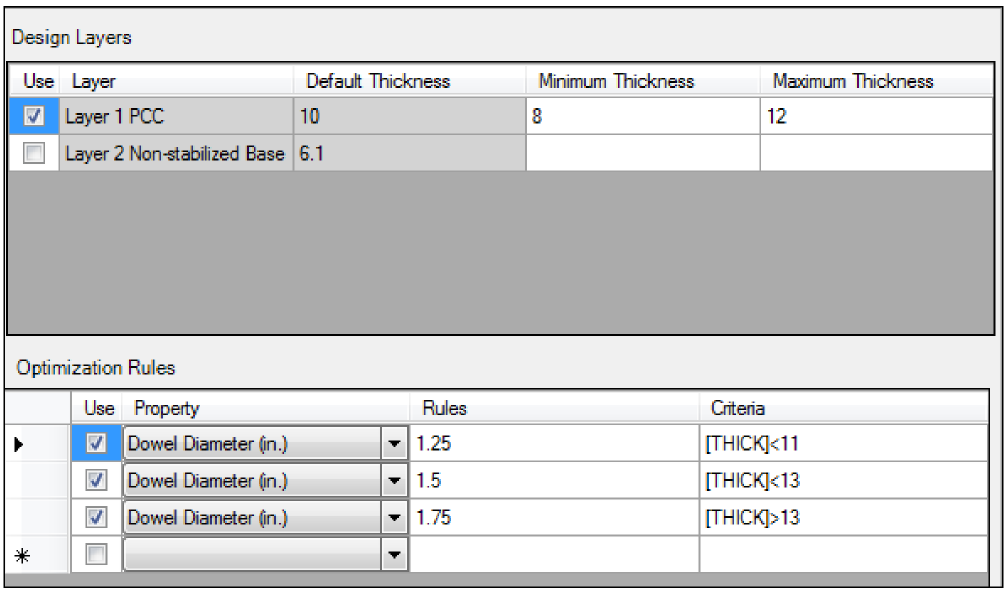


Figure 97. JPCP Optimized Rules

The modified design was rerun, and the outputs from the revised "Trial Design" are presented in Figure 98. The results show that the revised design satisfied performance criteria thresholds beyond 25 years. ITD typically performs diamond grinding after 25 years.

Acceptance of Finalized Design

The optimized thickness required was 9 inches. *AASHTOWare Pavement ME Design* analysis shows that this design structure, along with the layer material types and design properties under the prevailing site conditions in Idaho, would be able to carry approximately 28 million trucks over a 40-year "Design Life." The design outputs also show a more than 95 percent chance that the distress and IRI over the 25-year period will be less than thresholds recommended by ITD. This design is thus deemed adequate.

NOTE: Adequate designs can be achieved through the application of the many options available, such as modifying materials properties and design features.

Design Inputs

Design Life:	40 years	Existing construction:	-	Climate Data Sources	44.021, -117.013
Design Type:	Jointed Plain Concrete Pavement (JPCP)	Pavement construction:	October, 1983		43.565, -116.22
		Traffic opening:	December, 1983		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	PCC	9.0
NonStabilized	Crushed Gravel	4.4
Subgrade	Soil Aggregate Mixture	5.3
Subgrade	Soil Aggregate Mixture	9.0
Subgrade	A-2-4	Semi-infinite

Joint Design:

Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
1983 (initial)	1,900
2003 (20 years)	10,043,400
2023 (40 years)	28,081,500

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability Target	Distress @ Specified Reliability Predicted	Reliability (%) Target	Reliability (%) Achieved	Criterion Satisfied?
Terminal IRI (in./mile)	160.00	185.71	95.00	84.68	Fail
Mean joint faulting (in.)	0.12	0.16	95.00	75.46	Fail
JPCP transverse cracking (percent slabs)	10.00	7.76	95.00	98.42	Pass

Distress Charts

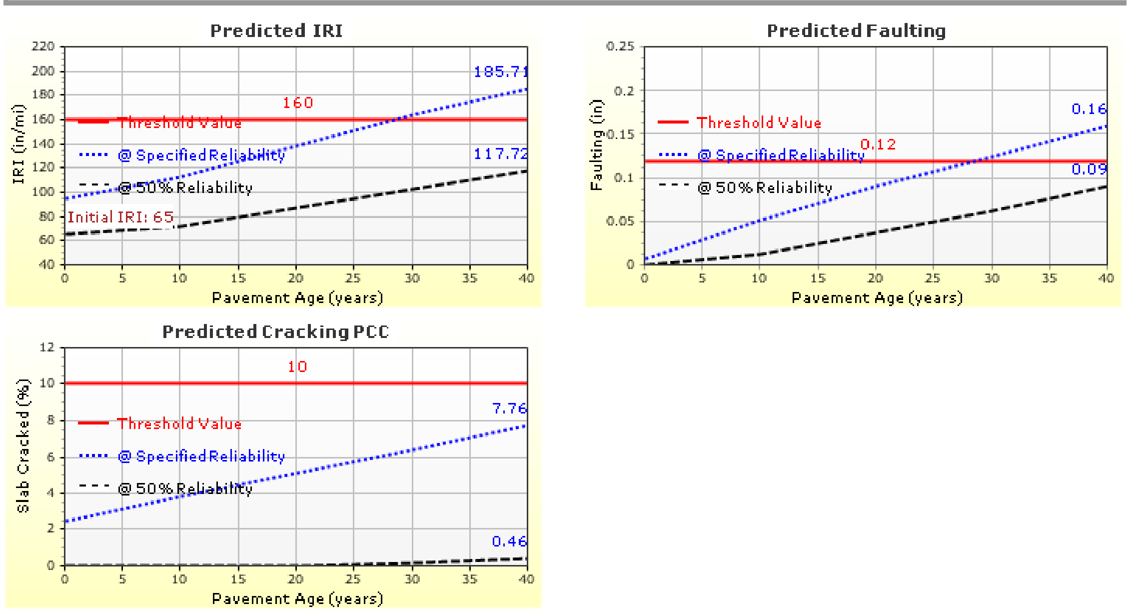


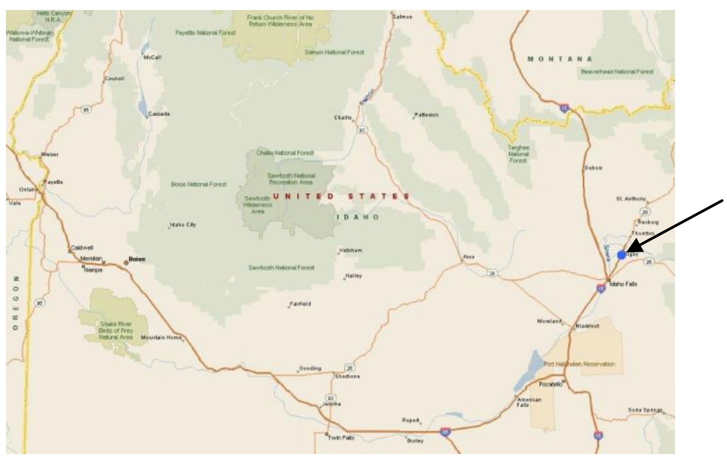
Figure 98. Optimized New JPCP Design Inputs and Outputs Summary

Appendix C
Idaho HMA Overlay Pavement Design Example

Project Description

For this overlay design example, the pavement section from new HMA design example (Appendix A) was used as the existing pavement. As of 2005, moderate amounts of cracking and rutting were predicted on this pavement, precipitating the need for mill and fill overlay. The AADTT in 2005 was 1,353 trucks with 5.68 percent compound growth.

As noted in Appendix A, this section is a four-lane divided flexible pavement (HMA over granular base) located on US-20, milepoint 319.55 to 319.65. The project is close to Rigby, between Idaho Falls and Rexburg, in Jefferson County, as shown in Figure 99. The roadway was originally constructed in August 1985 and later adopted in the LTPP program with WIM Site No. 1021.



Close-Up View of Project Surroundings

Figure 99. HMA Overlay Pavement Design Example Location

Pre-Design Issues

Prior to the start of design and analysis, the pavement designer must assemble all key inputs required for this pavement type and decide on the hierarchical level of inputs for each key input category. Key inputs required for HMA overlay pavement design are presented in Table 72. Based on the functional class (U.S. highway) and location (rural) of the roadway being designed, Level 2/3 inputs were generally assumed to be adequate. Inputs such as initial AADTT and projected future growth rate must always be estimated at Level 1.

Table 72. Key Inputs Required for HMA Overlay Pavement Design

Input Category		Input Variables
General Information		Design Type & Pavement Type
		Existing Construction Date (month/year)
		Pavement Construction Date (month/year)
		Traffic Opening Date (month/year)
"Design Life" & Reliability		"Design Life" (years) Design Reliability (%)
Existing Pavement Condition		Mill Thickness (in.)
		Existing Fatigue Cracking (%)
		Existing Rutting of Each Existing Pavement Layer (in.)
Performance Criteria		Initial IRI (in./mile)
		Terminal IRI (in./mile)
		Alligator Cracking (%)
		Thermal Cracking (ft/mile)
		Total Rutting (in.)
		Total Cracking – Alligator + Reflective (%)
Traffic		Initial Two-Way AADTT & Truck Growth
		Number of Lanes, Directional & Lane Distribution
		Vehicle Class Distribution
		Monthly Adjustments
		Number of Axles Per Truck
		Axle Load Distribution
Structure & materials properties	HMA Overlay	Binder Grade, Volumetric Binder Content, In-Place Air Voids, Aggregate Gradation, & Thickness
	Existing HMA	Binder Grade, Volumetric Binder Content, As-Constructed Air Voids, Aggregate Gradation, & Thickness
	Existing Crushed Base	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness
	Existing Subgrade	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness
	Existing Bedrock	Elastic/M_r, Unit Weight, & Poisson's Ratio
Project-Specific Calibration Factors		ITD local calibration coefficients for HMA overlay pavement. (adopted from Wyoming DOT local calibration coefficients)

Develop a "Trial Design"

"Trial Design" begins with opening the *AASHTOWare Pavement ME Design* software and selecting the appropriate design type and pavement type, which for this design example are "Overlay" and "AC over AC." Additional information is presented in the *AASHTOWare Pavement ME Design* "HELP System." Next - create the "Trial Design" by populating several screens of the user interfaces. The "Trial Design" file, once completed, must be saved and reviewed for accuracy and wrong entries. Files should be name using standard ITD conventions. For this example, the filename "AC Overlay.dgpx" is used.

NOTE: The output summary file names will be based on the input file name. Details of how the project is created and populated with "Trial Design" inputs are presented in the following sections.

"Design Life"

Table 1 of this *User's Guide* provides information on pavement "Design Life". For rehabilitated HMA pavements, the recommended "Design Life" is 20 years. Thus, a 20-year "Design Life" was selected (see Figure 100).

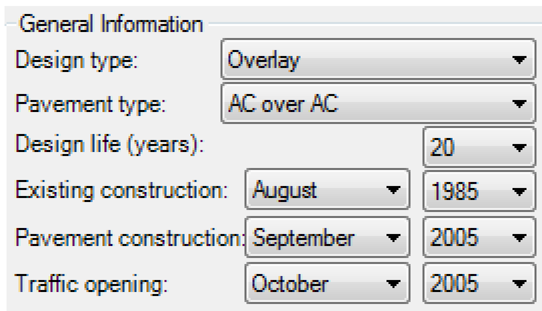


Figure 100. HMA Overlay Pavement Design Example Construction Month and Year

Construction and Opening Dates

AASHTOWare Pavement ME Design requires information on the anticipated construction or placement date (month/year) for both the existing pavement and HMA overlay layer. This information is used for setting the baseline climate and traffic at construction. The month and year of placing the existing pavement and HMA overlay layers must be determined based on typical ITD practices (i.e., the seasons in which pavements are normally constructed). Also required is the anticipated month and year for which the complete pavement will be opened to traffic. Again, this input must be selected based on typical ITD construction practices. For this example, the following were selected (see Figure 100):

- Existing Pavement Construction (month/year): August 1985.
- HMA Overlay Layer Placement (month/year): September 2005.
- Traffic Opening (month/year): October 2005.

Performance Criteria & Design Reliability

Designers must select pavement performance criteria from which the "Trial Design" is accepted or rejected. Performance criteria are basically critical distress and smoothness levels that ITD allows for a given pavement type and functional class. *AASHTOWare Pavement ME Design* predicts distress and smoothness over a specified analysis period "Design Life", and these predictions at the end of the "Design Life" are compared to the selected threshold values. If the predicted distress and smoothness are greater than the threshold values, the "Trial Design" is rejected. *AASHTOWare Pavement ME Design* allows designers to predict distress and smoothness at various levels of reliability. See Chapters 3 and 4 for guidance on selecting performance criteria and reliability levels.

For this HMA overlay design example, the performance criteria recommended for a primary highway (principal arterial) were selected (see Table 7). A reliability level of 85 percent was selected based on the functional class (see Table 9). For total cracking, a design reliability of 50 percent must be selected because the software cannot select other values. Thus, the level of the performance criteria selected must consider this value (e.g., total cracking should be lower than that selected at a higher level of reliability).

NOTE: ITD does not include longitudinal (top-down fatigue) cracking in the mix of distress types used in assessing HMA pavement performance.

Therefore, even though *AASHTOWare Pavement ME Design* produces predictions for this distress type, the predictions are ignored. One way of doing this is to set very high threshold values for this distress type. In addition, ITD does not include the asphalt layer permanent deformation in assessing HMA pavement performance. This distress type prediction is ignored. One way of doing this is to set the threshold value the same as the total permanent deformation.

AASHTOWare Pavement ME Design requires estimates of initial pavement smoothness, IRI (i.e., right after HMA overlay layer placement). This is an important input as the time from initial construction to attaining threshold IRI value is very much dependent on the initial IRI obtained at the time of construction. The initial IRI value provided in the design must be attained in the field and thus must reflect ITD practices. As shown in Figure 101, an initial IRI of 50 in./mile is used in this example. Designers can vary this input if there is reason to believe it would not reflect initial smoothness values for a given project.

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	50	
Terminal IRI (in./mile)	175	85
AC top-down fatigue cracking (ft/mile)	*5000*	85
AC bottom-up fatigue cracking (percent)	15	85
AC thermal cracking (ft/mile)	1500	85
Permanent deformation - total pavement (in.)	0.5	85
Permanent deformation - AC only (in.)	0.5	85
AC total cracking - bottom up + reflective (percent)	10	50

Figure 101. Performance Criteria and Reliability for HMA Overlay Pavement Design Example

Traffic

Traffic inputs are the same as for the new HMA design example (Appendix A), except the initial two-way AADTT is 1,353 trucks with 5.68 percent compound growth.

Climate

Climate inputs are the same as for new HMA design example (Appendix A).

Existing Pavement Structure

AASHTOWare Pavement ME Design allows for HMA overlay design on three types of existing HMA pavement design: conventional (HMA over granular base), full-depth (HMA over asphalt treated base), and semi-rigid (HMA over chemically treated base). For this design example, a conventional HMA existing pavement structure was assumed. The existing conventional HMA pavement is shown in Figure 102.

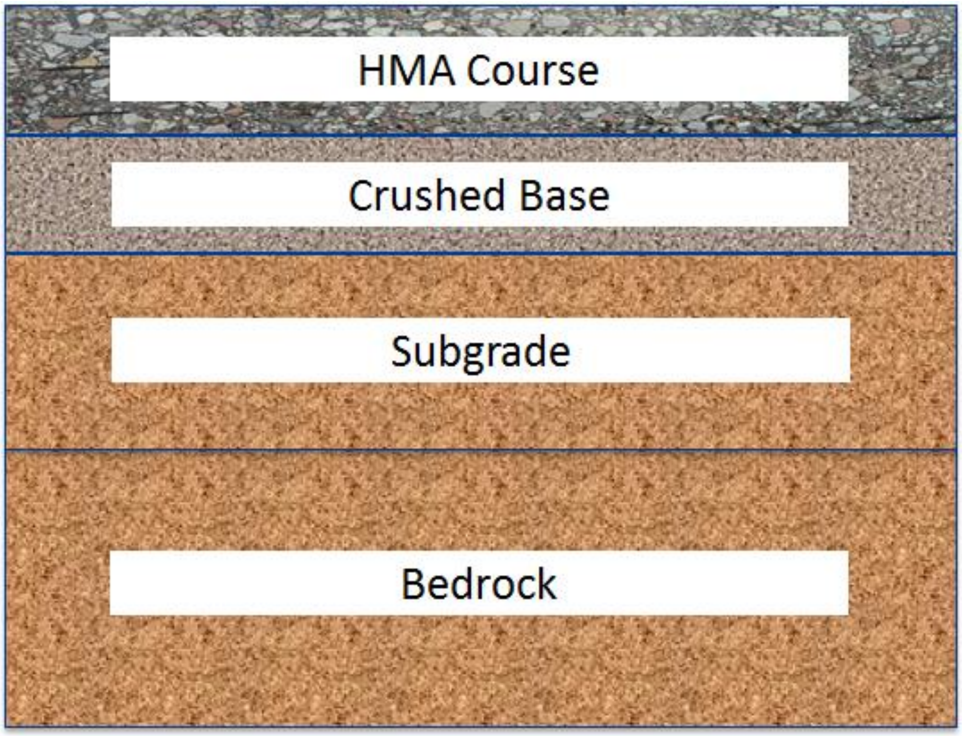


Figure 102. Existing HMA Pavement Structure

The general description of existing pavement structure, starting from the bottom foundation support, is as follows:

- **Bedrock:** Highly fractured and weathered or massive continuous (intact) rock within 10 to 20 ft of the pavement foundation, if present.
- **Subgrade:** The nature of the subgrade foundation (including depth to bedrock and groundwater table) is mostly determined directly from subsurface exploration and testing activities. Key for pavement design is to determine the natural/compacted subgrade properties and depth, as well as depth to bedrock. Natural and compacted subgrade soil properties are obtained from tests on the natural foundation soil in place and in its compacted state as the upper layers (12 to 24 in.) are rolled and compacted or removed and replaced during construction.
- **Crushed Gravel Base:** ITD specifies a range of aggregate/granular materials for use as base materials. The materials are mostly classified as AASHTO A-1-a and A-1-b soils.
- **HMA Layers:** ITD specifies a minimum two courses for the HMA (wearing course and intermediate course). In this example, the two HMA layers were combined, as their material properties are the same.

Guidance for obtaining pavement layer properties and thicknesses to define the trial HMA pavement structure has been presented in Section 10.2 of this *User's Guide*. For this design, a Level 2/3 input was adopted. *AASHTOWare Pavement ME Design* recommends that once the "Trial Design" is defined, material properties must be populated starting from the lowest layer bedrock or natural subgrade to the surface layer.

The *AASHTOWare Pavement ME Design* "HELP System" provides detailed guidance on how to enter pavement structure and layer materials input data.

Existing Bedrock

Review of historical subsurface exploration and testing reports for this location showed there was bedrock under the natural subgrade. Thus, a bedrock layer was included. A highly fractured and weathered bedrock layer was selected with elastic modulus of 500,000 psi, which is MEPDG default.

Existing Subgrade Layer

Subsurface exploration and testing reports from the LTPP database indicate the subgrade for this location is AASHTO A-1-a soil. Engineering properties required at Level 2/3 for the natural subgrade are presented in Table 73.

Table 73. Required Engineering Properties for the Existing Subgrade

Engineering Properties	Level of Input	Source of Data
Gradation	2	Obtained through subsurface exploration & testing
Atterberg Limits (Liquid Limit & Plasticity Index)	2	Obtained through subsurface exploration & testing
Maximum Dry Unit Weight	3	Computed internally by the software
Saturated Hydraulic Conductivity	3	Computed internally by the software
Specific Gravity of Solids	3	Computed internally by the software
Optimum Gravimetric Water Content	3	Computed internally by the software
Soil Water Characteristic Curve	3	Computed internally by the software
Resilient Modulus (M_r)	2	Obtained elastic modulus back-calculated from FWD deflection testing data, then converted field modulus to laboratory condition (see Table 37 for guidance)

Figure 103 shows the subgrade engineering properties (gradation and Atterberg limits) obtained from laboratory testing, coded into the *AASHTOWare Pavement ME Design* software. Based on these two properties, the software internally estimates maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum gravimetric water content, and soil water characteristic curve (see Figure 103). The designer must check the estimated soil engineering properties for accuracy and reasonableness. If the *AASHTOWare Pavement ME*

Design estimates are deemed unreasonable, the designer can override the internally estimated values. Guidance for overriding the engineering properties is provided in the *AASHTOWare Pavement ME Design* "HELP System."

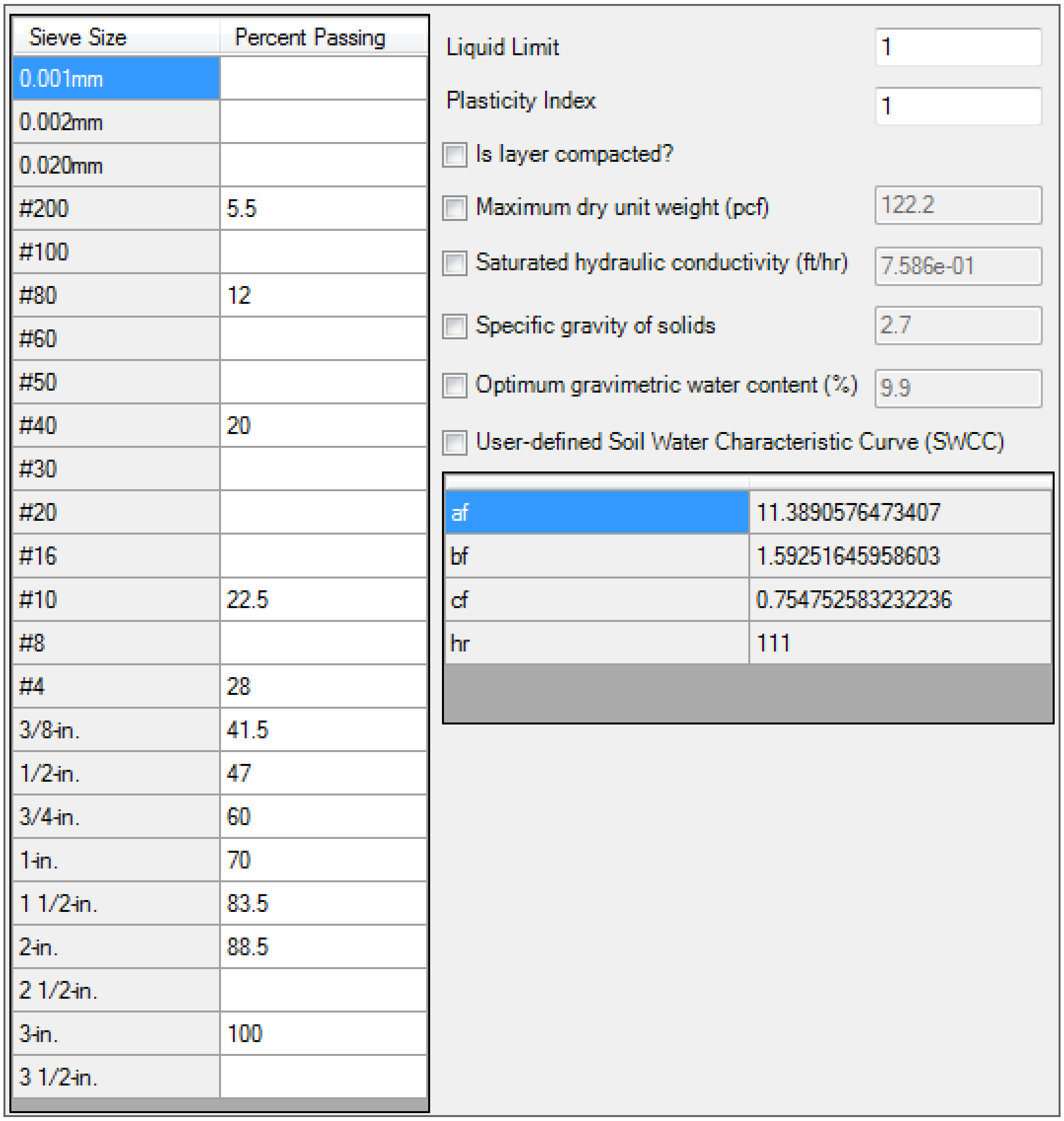


Figure 103. Existing Subgrade Engineering Properties Input Screen

For this example, the back-calculated elastic modulus was 23,807 psi. The corrected M_r of 15,951 psi at optimum moisture content was entered into *AASHTOWare Pavement ME Design*, as shown in Figure 104. The designer can override this value if warranted.

NOTE: The "Layer Compacted" box on the input screen was unchecked to reflect field conditions (as the subgrade layer is not compacted). The thickness of the subgrade layer was 30 inches, as there was an underlying layer or bedrock.

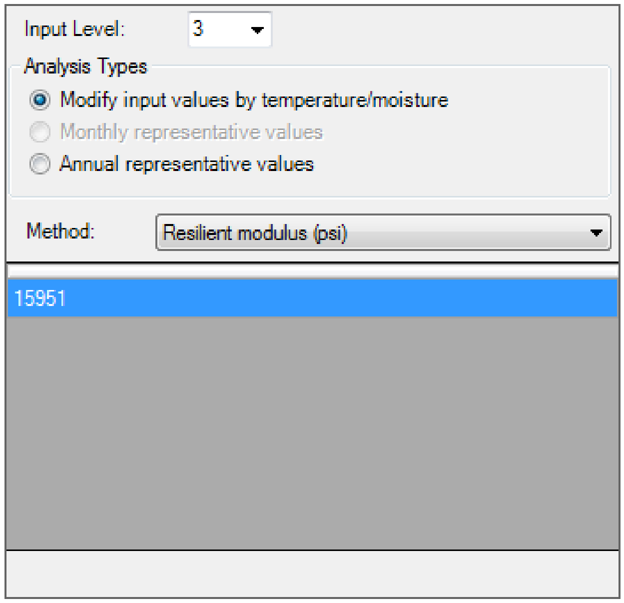


Figure 104. Existing Subgrade Level 3 Resilient Modulus Input Screen

Existing Crushed Gravel Base Layer

The aggregate base material type for use in pavement construction is selected based on project location (mostly, the nearest source of high-quality aggregate material is selected). ITD's "crushed gravel" material classification and properties are determined based on the source of the material.

For this design example, a material classification of A-1-a was selected. As shown in Table 39, crushed gravel has properties comparable to AASHTO soil class A-1-a, and M_r at optimum moisture for this material is assumed to be 35,524 psi. This value came from Figure 105 by entering the modulus of lower layer axis at 15,951 psi, turning on the 6 in. baseline and intersecting the modulus of upper layer axis. M_r of aggregate or granular base/subbase layers depends on the M_r of the supporting layers. Therefore, as a rule of thumb, the aggregate base M_r entered into *AASHTOWare Pavement ME Design* for a granular base layer must not exceed three times the M_r of the supporting subgrade or subbase layer to avoid decompaction of that layer (see Figure 105).

Therefore, designers must check whether $M_{rBase}/M_{rSubgrade}$ is more than or less than 3. For this example, $M_{rBase}/M_{rSubgrade}$ = 2.23, and thus the base M_r was found to be adequate.

Once the base modulus is selected, the designer can enter the aggregate base engineering properties and M_r into *AASHTOWare Pavement ME Design*. For this example, AASHTO A-1-a soil gradation and Atterberg limits were assumed from the LTPP database. A layer thickness of

5.3 inches was assumed for the "Trial Design" and entered into *AASHTOWare Pavement ME Design*.

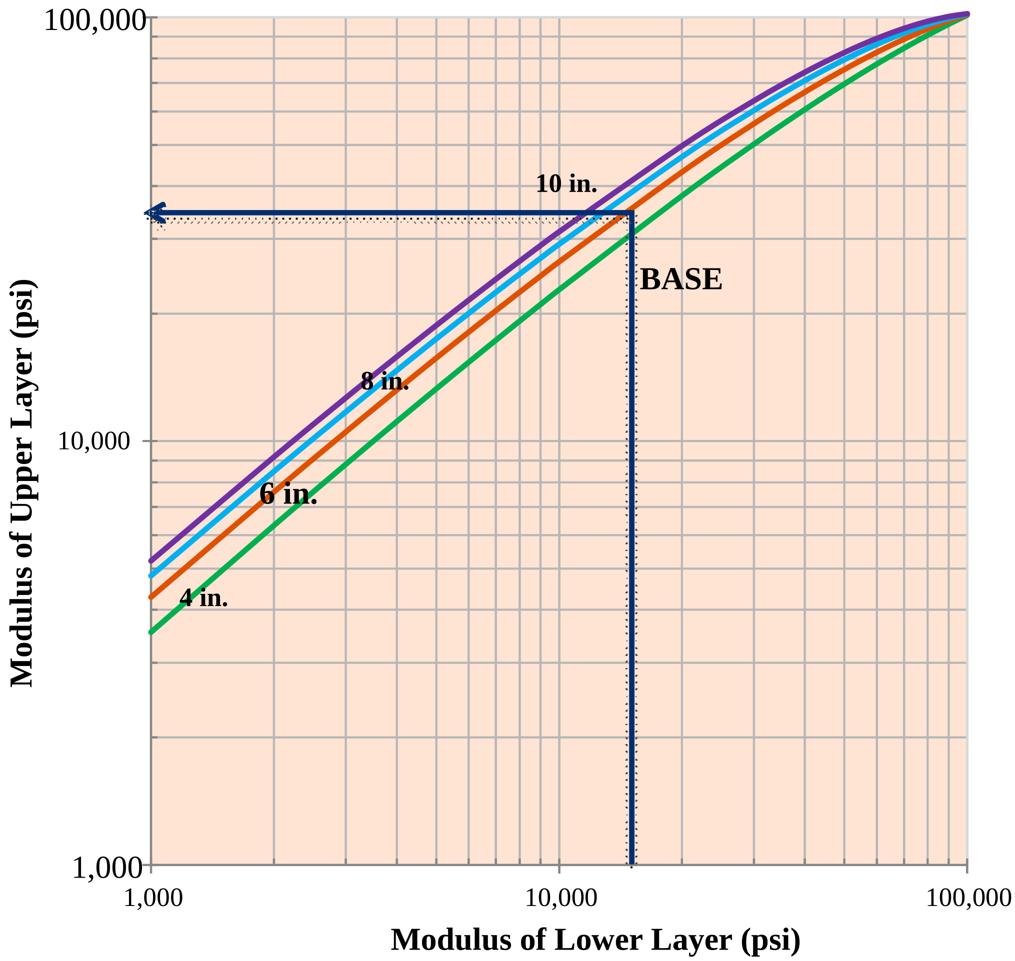


Figure 105. Selecting Base Modulus for HMA Overlay Design

Existing HMA Layer

Required inputs and values entered for this design example are presented in Table 74.

NOTE: The input requirements were for hierarchal Level 2/3 for all inputs. Figures 106 and 107 present the *AASHTOWare Pavement ME Design* screens used to enter HMA material properties for the existing HMA layer.

Table 74. Required Engineering Properties for the Existing HMA Layer

Engineering Properties	Level of Input	Source of Data
Layer Thickness	1	*AASHTOWare Pavement ME Design* does not allow for HMA layer thicknesses of less than 1 inch. An HMA layer thickness of 3 inches was assumed for "Trial Design" (assumed 2-inch milling).
Gradation (found under the Dynamic Modulus input screen)	3	Gradation for this HMA mix type was obtained from mean gradation test results. Percent passing the ¾ in., ⅜ in., No. 4, & No. 200 sieves were 100, 92, 72, & 6.8 percent, respectively.
Asphalt Binder Type	3	Asphalt binder type PG 58-34 (Superpave) was selected.
Asphalt Binder Content (volumetric, as placed)	3	A value of 12.23 percent was selected.
HMA Mix Air Voids Content (as placed)	3	A value of 5.5 percent was selected.
HMA Unit Weight	3	A value of 139 pcf was selected.
Test Reference Temperature	3	Assume *AASHTOWare Pavement ME Design* defaults.
HMA Creep Compliance*	3	Assume *AASHTOWare Pavement ME Design* defaults.
HMA Indirect Tensile Strength*	3	Assume *AASHTOWare Pavement ME Design* defaults.
Coefficient of Thermal Contraction*	3	Assume *AASHTOWare Pavement ME Design* defaults.
Thermal Conductivity	3	Assume *AASHTOWare Pavement ME Design* defaults.
Heat Capacity	3	Assume *AASHTOWare Pavement ME Design* defaults.

* HMA creep compliance, indirect tensile strength, & coefficient of thermal contraction for HMA layer is used in computing HMA thermal cracking distress.

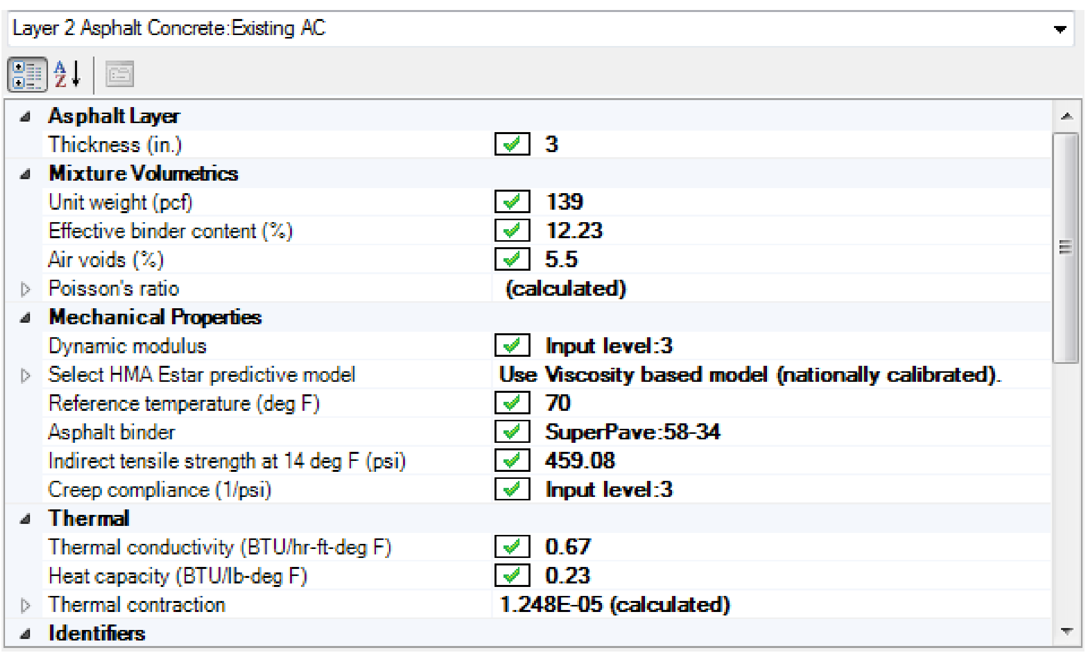


Figure 106. Screen for Existing HMA Layer Binder and Mix Inputs

Dynamic modulus input level: 3

Gradation	Percent Passing
3/4-inch sieve	100
3/8-inch sieve	92
No.4 sieve	72
No.200 sieve	6.6

Figure 107. Screen for Existing HMA Layer Gradation Inputs

HMA Overlay Layer

AASHTOWare Pavement ME Design allows for the design of up to three HMA overlay layers. For this design example, a single HMA overlay layer was assumed. The material properties for the HMA overlay layer were assumed to be the same as the existing HMA layer. A layer thickness of 2 inches was assumed for the "Trial Design". The creep compliance inputs for the surface layer (overlay) is shown in Figure 108.

Creep compliance level: 3

Loading Time(sec)	Low Temp (-4 deg F)	Mid Temp (14 deg F)	High Temp (32 deg F)
1	4.512738E-07	6.185922E-07	7.98589E-07
2	5.116777E-07	7.485776E-07	1.105714E-06
5	6.041101E-07	9.632418E-07	1.700033E-06
10	6.849715E-07	1.165649E-06	2.35384E-06
20	7.766563E-07	1.410588E-06	3.259091E-06
50	9.169559E-07	1.815092E-06	5.010844E-06
100	1.039692E-06	2.196499E-06	6.937939E-06

Figure 108. HMA Overlay Layer Creep Compliance Inputs

Rehabilitation Inputs

Rehabilitation design requires a few additional inputs and some modifications of other inputs that are related to the existing pavement. In rehabilitation design, the existing pavement typically has deteriorated from its original condition through all types of fracture, distortion, or material disintegration. Some of the material properties may also have changed over time, such as the oxidation of asphalt. *AASHTOWare Pavement ME Design* can account for these effects through modifying various design inputs and through a few new inputs related to the condition of the existing pavement. These modifications are used to adjust the various moduli of the existing pavement.

Rehabilitation inputs vary depending on the existing pavement and on the type of rehabilitation (i.e., Level 1, 2, or 3). The inputs for Level 3 rehabilitation require estimation of pavement condition rating (based on alligator cracking) and total rutting. Level 2 rehabilitation requires measurement of wheelpath alligator cracking and rutting for each existing pavement layer. Level 1 rehabilitation requires rutting inputs for each existing pavement layer plus additional inputs such as nondestructive testing back-calculated modulus for each existing pavement layer.

Level 1 rehabilitation is not recommended at this time. Thus, Level 2 rehabilitation was used. For this design example, pavement condition information was obtained from predictions from the new HMA design example (Appendix A). For mill and fill HMA overlay, planned milling thickness is also required for all rehabilitation levels. The Level 2 rehabilitation inputs used in this example are presented in Tables 75 and 76 and Figure 109.

Table 75. Characterization of Existing HMA Pavement

<table>
<tr><th>Rehabilitation Input Level</th><th>Rehabilitation Input Type</th><th>Rehabilitation Design Inputs
Existing HMA Pavement</th></tr>
<tr><td rowspan="3">2</td><td>Alligator Cracking</td><td>Identify the representative length of heaviest trafficked lane along project (that has typical alligator cracking, if any). Then measure the alligator cracking in each wheelpath & compute the percentage of lane area of alligator cracking. Include all severities of cracking, including longitudinal wheelpath cracking.

Alligator cracking of 5.25 percent lane area was entered into the software.</td></tr>
<tr><td>Wheelpath Rutting</td><td>Measure mean wheelpath rutting along project. Estimate total rutting in each layer using default percentages & compute individual HMA, base, & subgrade rutting. The following were entered into the software:
• HMA Layer: 0.1962 in.
• Base Layer: 0.0453 in.
• Subgrade Layer: 0.1266 in.</td></tr>
<tr><td>Mill Thickness</td><td>Assume HMA mill thickness of 2 in. for this example.</td></tr>
</table>

Table 76. Characterization for Aggregate Base and Unbound Embankment/Subgrade of Existing HMA Pavement

Rehabilitation Input Level	Existing Layer Type	Rehabilitation Design Inputs Existing HMA Pavement Base & Embankment/Subgrade
2/3	Unbound Aggregate Base Course M_r	• Level 2: Back-calculate from FWD testing of existing pavement & adjust to laboratory values (multiply by 0.35) at in situ moisture content OR estimate from R-value tests. NOTE: The in situ soil moisture content must also be entered into *AASHTOWare Pavement ME Design.* • Level 3: Use default values from Table 39. **Limit Input M_r of Unbound Base to 3 Times that of Subgrade.**
2/3	Subgrade M_r	• Level 2: Back-calculate from FWD testing of existing pavement and adjust to laboratory values at optimum moisture OR estimate M_r from R-value tests. • Level 3: Use default M_r values at optimum moisture content; see Table 37.

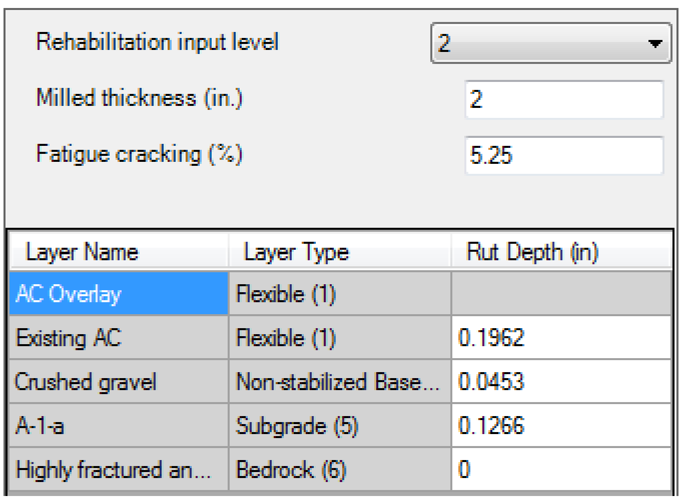


Figure 109. *AASHTOWare Pavement ME Design* Input Screen for HMA Rehabilitation Inputs

Additional HMA Layer Properties

The following additional HMA layer properties are required by *AASHTOWare Pavement ME Design:*

- **Surface HMA Layer Surface Shortwave Absorptivity:** This input is used to estimate heat flow within the HMA layers. The *AASHTOWare Pavement ME Design* default value 0.85 was assumed.

- **Endurance Limit:** HMA endurance limit is required only for the design of perpetual HMA pavements. As this design procedure is not calibrated and not recommended by ITD, designers must set the endurance limit in *AASHTOWare Pavement ME Design* to "False," as shown in Figure 110.

- **Layer Interface:** This defines the friction levels between pavement HMA overlay, existing HMA, base, and subgrade layers. As ITD recommends full bonding between all layers for HMA pavements, a default value of 1 (implying full friction between the layers) is recommended, as shown in Figure 111.

Designers can override all of these additional HMA layer inputs if warranted.

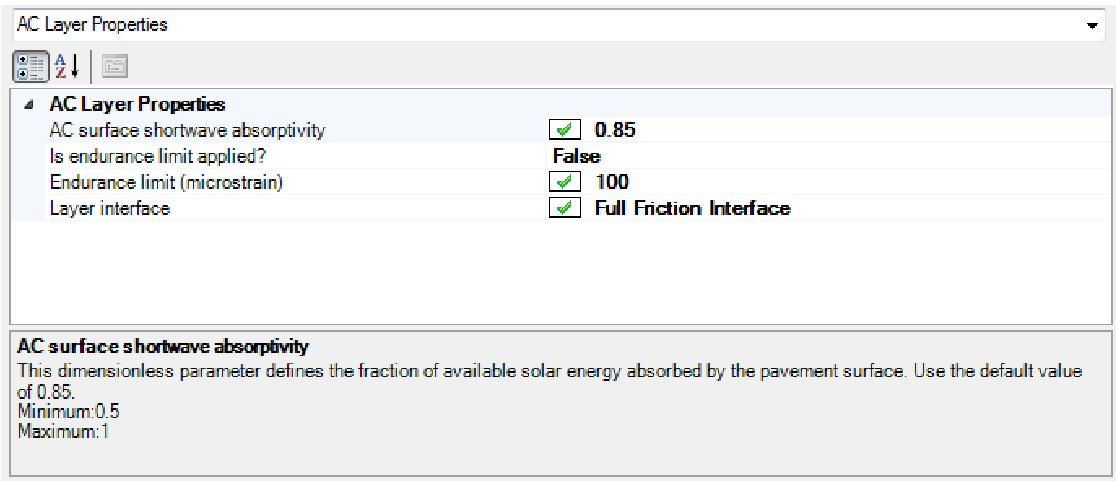
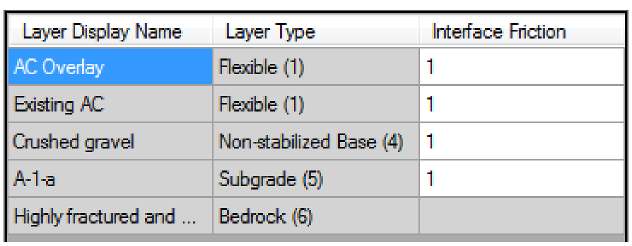


Figure 110. *AASHTOWare Pavement ME Design* Input Screen for Additional Inputs Required for HMA Layer



Layer Display Name	Layer Type	Interface Friction
AC Overlay	Flexible (1)	1
Existing AC	Flexible (1)	1
Crushed gravel	Non-stabilized Base (4)	1
A-1-a	Subgrade (5)	1
Highly fractured and ...	Bedrock (6)	

Figure 111. *AASHTOWare Pavement ME Design* Input Screen for HMA Pavement Layer Interface Friction

ITD HMA Rehabilitation Pavement Project-Specific Calibration Coefficients

When *AASHTOWare Pavement ME Design* is used for Idaho conditions, ITD recommends the calibration coefficients presented in Figure 112 for distress and IRI models. These coefficients were adopted from Wyoming DOT (which are based on a combined set of LTPP flexible pavements from Idaho, Wyoming, Colorado, and South Dakota) and should be used until ITD establishes its own local calibration coefficients. Designers must check if the HMA rehabilitation pavement project is applying the calibration coefficients presented. If not, guidance is provided in the *AASHTOWare Pavement ME Design* "HELP System" on how to replace the global calibration coefficients with ITD-recommended values.

Run "Trial Design" and Analyze Results

Pavement design using *AASHTOWare Pavement ME Design* is iterative. The designer must run the software, check key outputs for reasonableness, and check the "Trial Design" for adequacy. The check for adequacy must be done at the ITD-recommended reliability level. If the "Trial Design" is deemed inadequate, appropriate modifications must be made to obtain a feasible design.

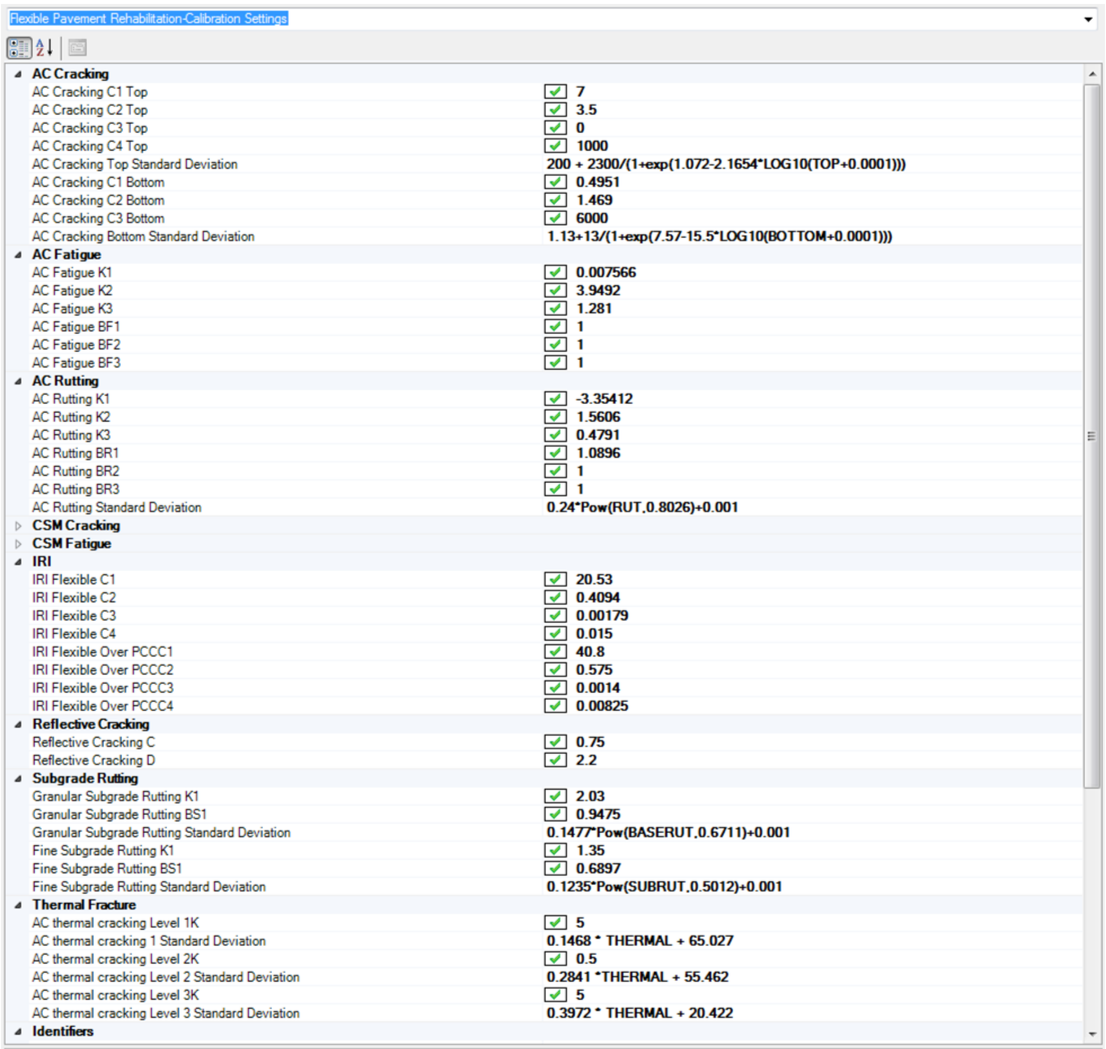


Figure 112. Flexible Pavement Rehabilitation Calibration Coefficients

Check of Key *AASHTOWare Pavement ME Design* Outputs for Reasonableness

It is important for designers to review key inputs and outputs for each *AASHTOWare Pavement ME Design* run to ascertain whether inputs were entered correctly and the software processed input data correctly and produced expected results. *AASHTOWare Pavement ME Design* produces two output files with a summary of key inputs and design outcomes, a .pdf file and an .xls file. The information contained in the .pdf output file is deemed adequate for this review and is presented under the following general headings:

- **Design Inputs:** Contains information about key designs inputs such as pavement structure definition, layer thicknesses, and traffic projections (see Figure 113).

- **Design Outputs:** Distress prediction summary in tabular and graphical forms (See Figure 114).

- **Traffic Input and Output Summary:** Graphical and tabular representation of key traffic inputs and projected growth and seasonal adjustments (see Figures 115 and 116):
  - Traffic Distributions: Tabular representation of traffic inputs.
  - Axle Configuration: Axle configuration summary of truck growth.
  - AADTT Growth by Class: Tabular representation of growth in trucks.

- **Climate Inputs and Output Summary:** Graphical and tabular representation of key climate inputs and climate variable statistics (see Figure 117):
  - Climate data sources (weather stations).
  - Annual statistics of key variables: temperature, precipitation, freezing index, etc.
  - Monthly statistics of key variables: temperature, precipitation, freezing index, etc. in a graphical format.

- **Design Properties**: Key pavement design input summary information.

- **Key HMA material Inputs and Computed Parameters:**
  - Thermal (transverse) cracking inputs such as creep compliance, coefficient of thermal contraction, and so on.
  - HMA master curve and shift factors in graphical format.

- **Analysis Output Charts:**
  - Plots of predicted IRI, rutting, alligator cracking, and thermal cracking versus age in graphical format.
  - Detailed breakdown of predicted distress and IRI:
    - Plots of predicted bottom-up and top-down damage versus age in graphical format.
    - Components of total rutting.
    - Thermal cracking spacing and depth.

- **Layer Information:** Detailed summary of data for all layers within the pavement structure.

- **Calibration Coefficients:** Detailed summary of project-specific distress/IRI models calibration coefficients.

Designers are encouraged to examine this information. Possible discrepancies between input data summaries and what was entered into *AASHTOWare Pavement ME Design* must be resolved.

Design Inputs

Design Life:	20 years	Existing construction:	August, 1985	Climate Data Sources	43.516, -112.067
Design Type:	AC over AC	Pavement construction:	September, 2005		43.834, -111.881
		Traffic opening:	October, 2005		42.92, -112.571

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	AC Overlay	2.0
Flexible	Existing AC	3.0
NonStabilized	Crushed gravel	5.3
Subgrade	A-1-a	30.0
Bedrock	Highly fractured and weathered	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	12.2
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2005 (initial)	1,353
2015 (10 years)	2,887,450
2025 (20 years)	7,904,430

Figure 113. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for HMA Overlay Pavement

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	175.00	100.65	85.00	100.00	Pass
Permanent deformation - total pavement (in.)	0.50	0.20	85.00	100.00	Pass
Total Cracking (Reflective + Alligator) (percent)	10	31.02	-	-	Fail
AC thermal cracking (ft/mile)	1500.00	49.81	85.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	15.00	1.17	85.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5000.00	4449.24	85.00	89.89	Pass
Permanent deformation - AC only (in.)	0.50	0.20	85.00	100.00	Pass

Distress Charts

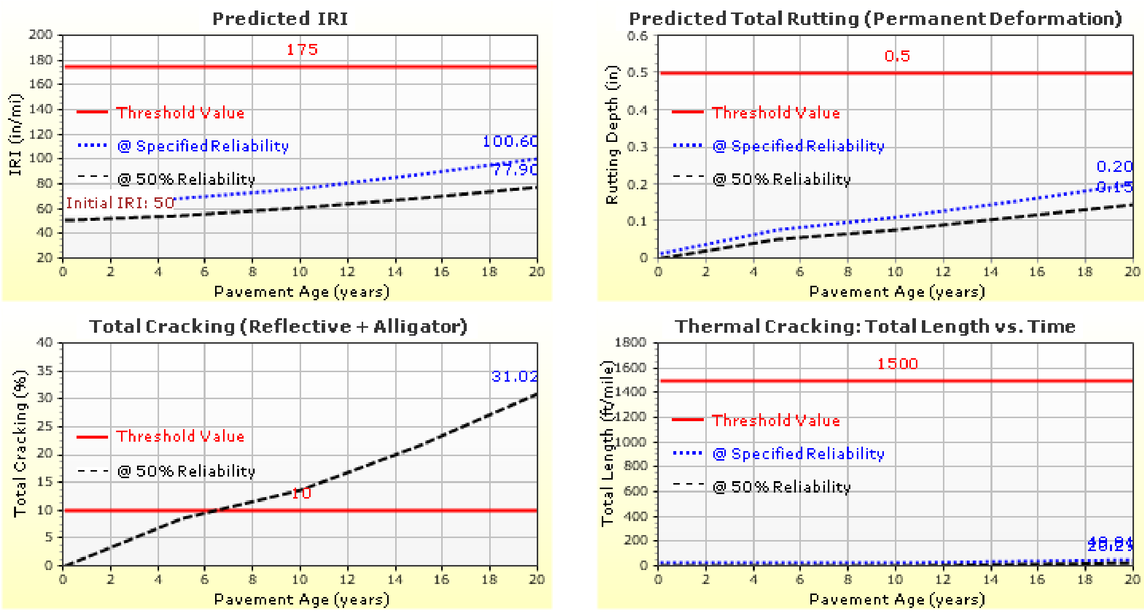


Figure 114. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Design Outputs for HMA Overlay Pavement**

Traffic Inputs

Graphical Representation of Traffic Inputs

Initial two-way AADTT:	1,353	Percent of trucks in design direction (%):	50.0
Number of lanes in design direction:	2	Percent of trucks in design lane (%):	90.0
		Operational speed (mph)	65.0

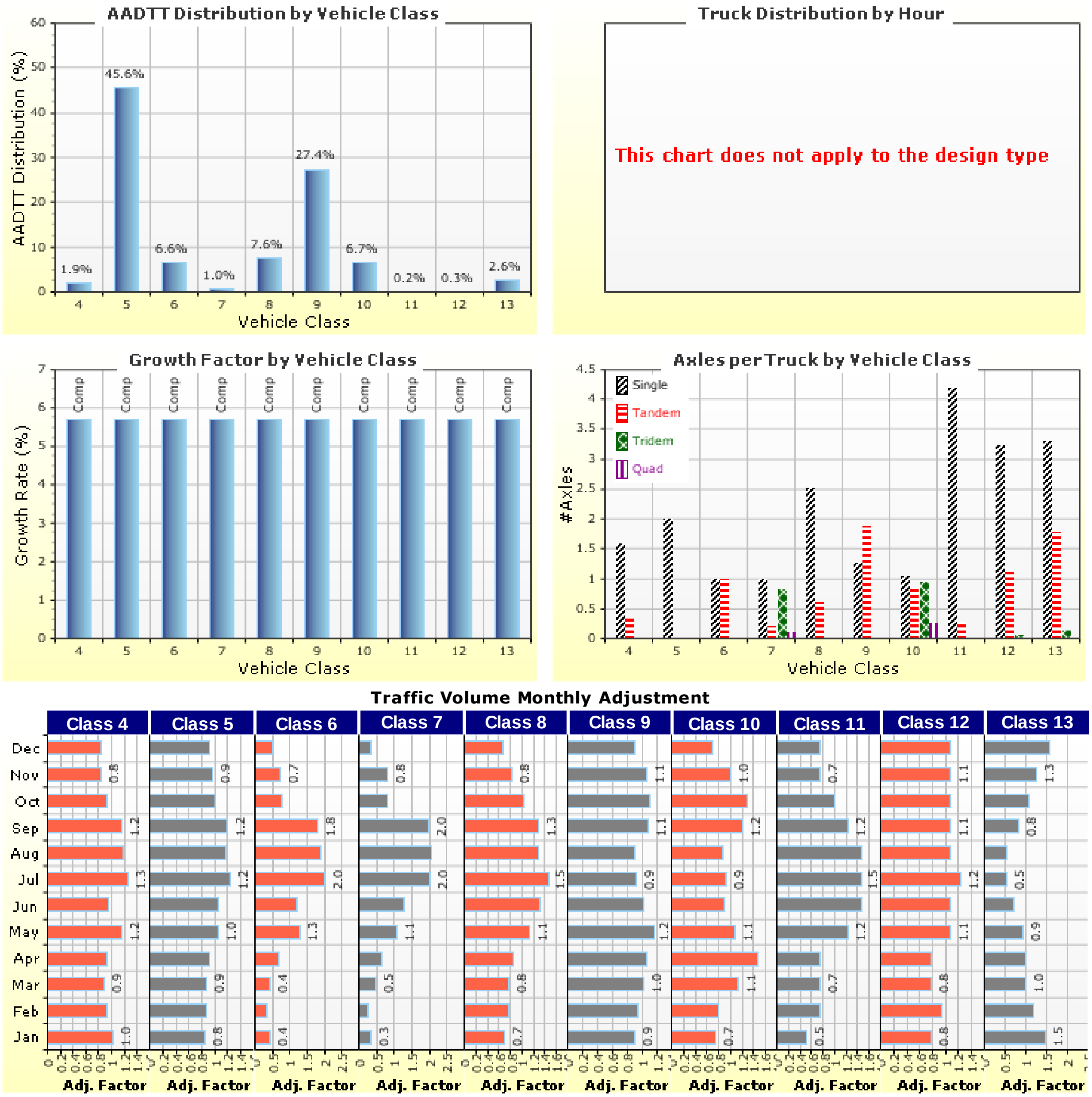


Figure 115. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Traffic Inputs for HMA Overlay Pavement**

AADTT (Average Annual Daily Truck Traffic) Growth

*** Traffic cap is not enforced**

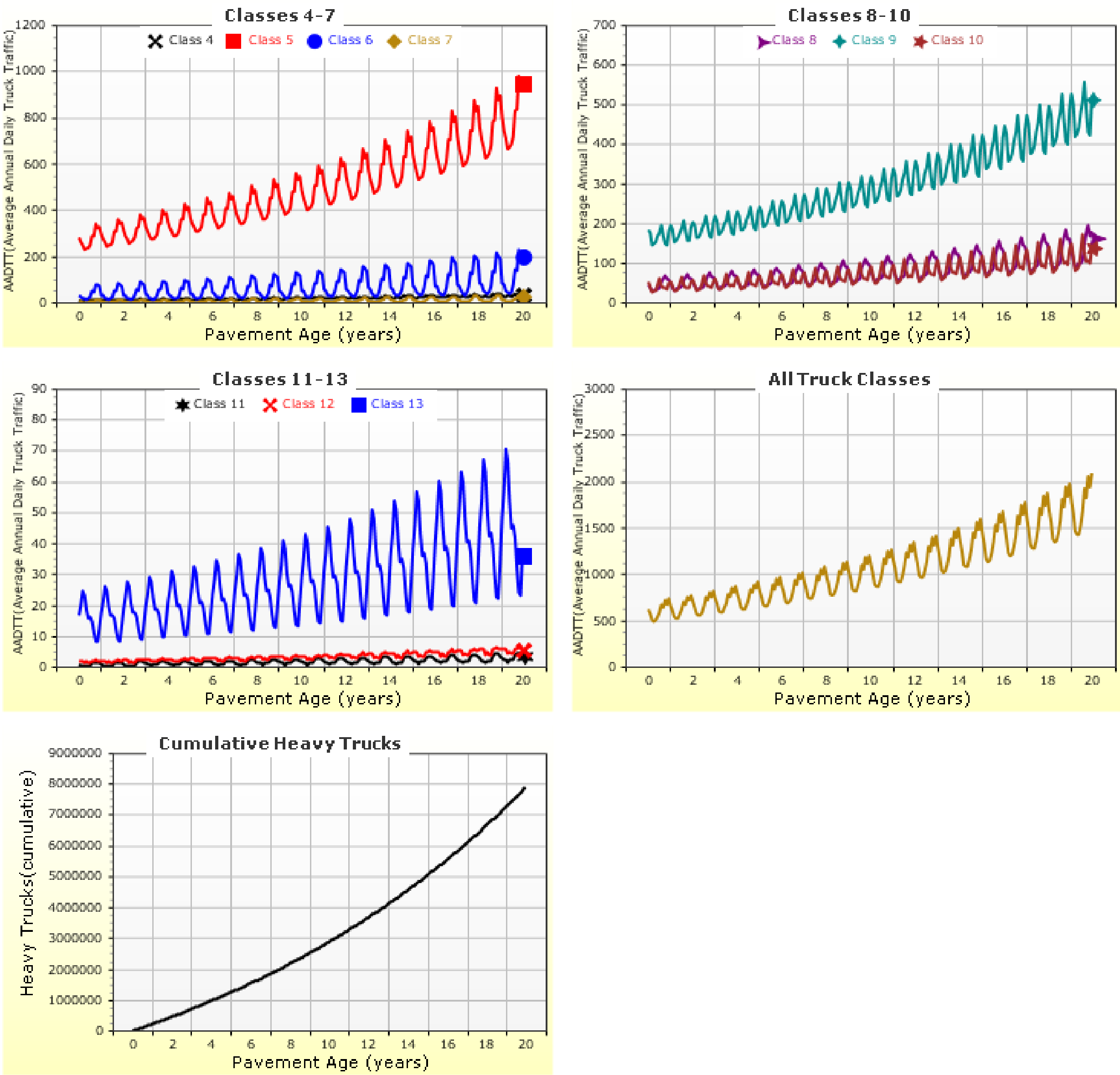


Figure 116. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Traffic Outputs (projection of AADTT) for HMA Overlay Pavement**

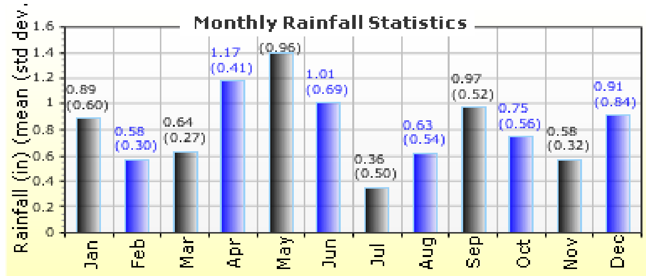
Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
IDAHO FALLS, ID	43.51600 -112.06700 4730
REXBURG, ID	43.83400 -111.88100 4859
POCATELLO, ID	42.92000 -112.57100 4440

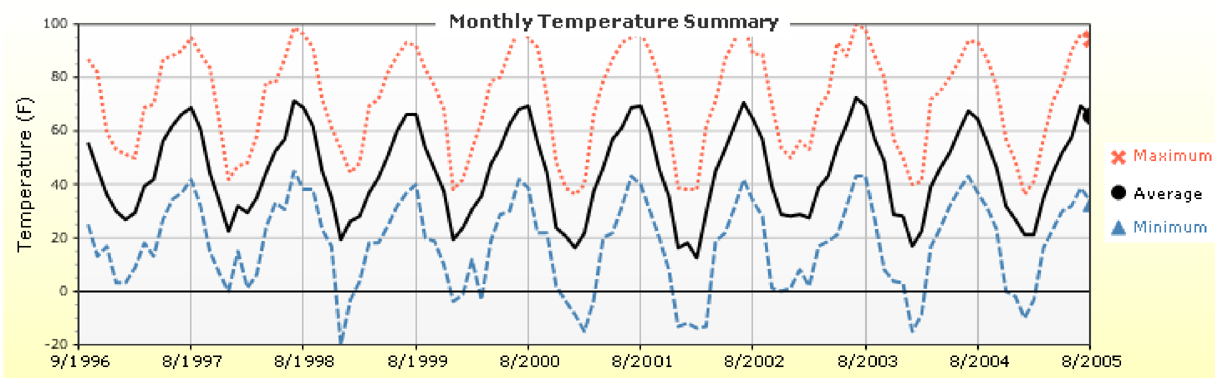
Annual Statistics:

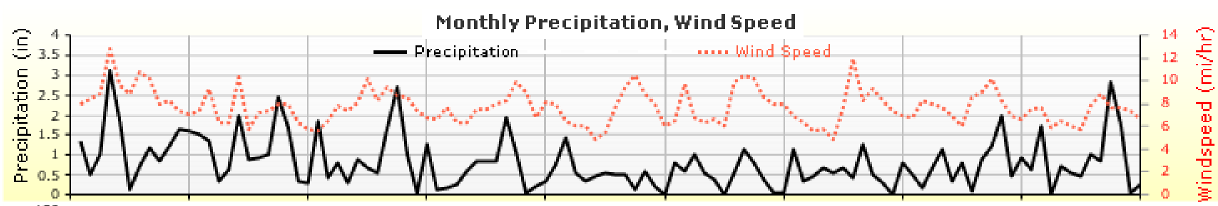
Mean annual air temperature (ºF)	44.90
Mean annual precipitation (in.)	9.89
Freezing index (ºF - days)	1063.24
Average annual number of freeze/thaw cycles:	116.86

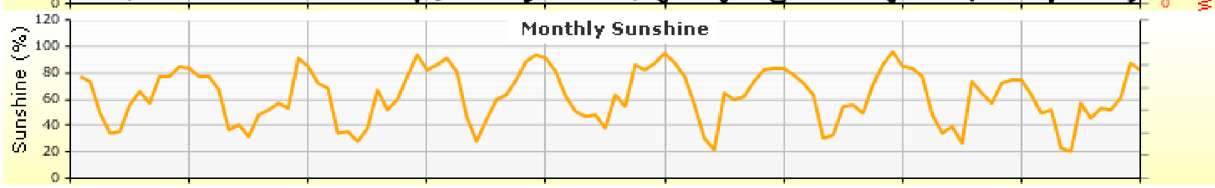


Water table depth(ft) 10.00

Monthly Climate Summary:







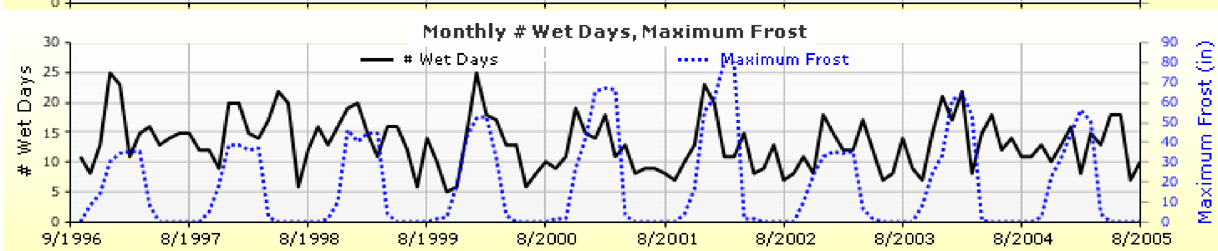


Figure 117. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for HMA Overlay Pavement

Check “Trial Design” for Adequacy

The final step is to check the “Trial Design” for adequacy. It should be able to carry anticipated traffic over its “Design Life” at the preset performance criteria recommended by ITD. The outcome of this example “Trial Design” is presented in Figure 114.

Designers must check this output summary to determine whether the design performance criteria are satisfied. This is done as follows:

1. Review the column called Criterion Satisfied? In the tabular output and determine for each distress type when the “Trial Design” “Passed” or “Failed.”

2. If the “Trial Design” passes the criteria set for all distress types, then the design is deemed adequate and acceptable.

3. If one or more of the criteria fail, then the design is deemed inadequate and “Trial Design” must be revised as needed and checked again.

For this example, the “Trial Design” did not meet the performance criteria for total alligator (fatigue) cracking (alligator + reflective). Thus, revision of this “Trial Design” was warranted.

Revise “Trial Design” and Rerun *AASHTOWare Pavement ME Design* as Needed

The designer must determine reasons for failure to meet the performance criteria and adopt feasible solutions to improve the “Trial Design”. Relevant to this design example, reflective cracking refers to alligator fatigue cracking that initiates in the existing HMA layer and reflects up through the new HMA overlay in the wheelpath. Alligator fatigue cracking initiates at the bottom of the new HMA overlay layer in the wheelpath.

A careful review of this HMA overlay example design inputs indicated a need to increase HMA overlay thickness to improve performance. This was done using the *AASHTOWare Pavement ME Design* thickness optimization tool. A detailed description of this tool is provided in the *AASHTOWare Pavement ME Design* “HELP System.” The outputs from the HMA layer thickness optimizations process are presented in Figure 118. The results show that, for an HMA overlay thickness of 4 inches, all performance criteria are satisfied.

Acceptance of Finalized Design

Figure 118 shows the final HMA overlay pavement design structure. *AASHTOWare Pavement ME Design* analysis shows that this design structure, along with the layer material types and properties under the prevailing site conditions in Idaho, would be able to carry approximately

7.9 million trucks over a 20-year "Design Life." The design outputs also show clearly a more than 85 percent chance that the distress and IRI over the 20-year "Design Life" will be less than the thresholds recommended by ITD. This design is thus deemed adequate. Designers must note, however, that adequate designs are not achieved only by increasing HMA thickness. All of the options available through modifying materials properties and so on must be considered to produce a cost-effective design.

Design Inputs

Design Life:	20 years	Existing construction:	August, 1985	Climate Data Sources	43.516, -112.067
Design Type:	AC over AC	Pavement construction:	September, 2005		43.834, -111.881
		Traffic opening:	October, 2005		42.92, -112.571

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	AC Overlay	4.0
Flexible	Existing AC	3.0
NonStabilized	Crushed gravel	5.3
Subgrade	A-1-a	30.0
Bedrock	Highly fractured and weathered	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	12.2
Air voids (%)	5.5

Traffic

Age (year)	Heavy Trucks (cumulative)
2005 (initial)	1,353
2015 (10 years)	2,887,450
2025 (20 years)	7,904,430

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	175.00	100.70	85.00	100.00	Pass
Permanent deformation - total pavement (in.)	0.50	0.27	85.00	100.00	Pass
Total Cracking (Reflective + Alligator) (percent)	10	8.95	-	-	Pass
AC thermal cracking (ft/mile)	1500.00	37.88	85.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	15.00	1.17	85.00	100.00	Pass
AC top-down fatigue cracking (ft/mile)	5000.00	2500.83	85.00	98.88	Pass
Permanent deformation - AC only (in.)	0.50	0.27	85.00	100.00	Pass

Distress Charts

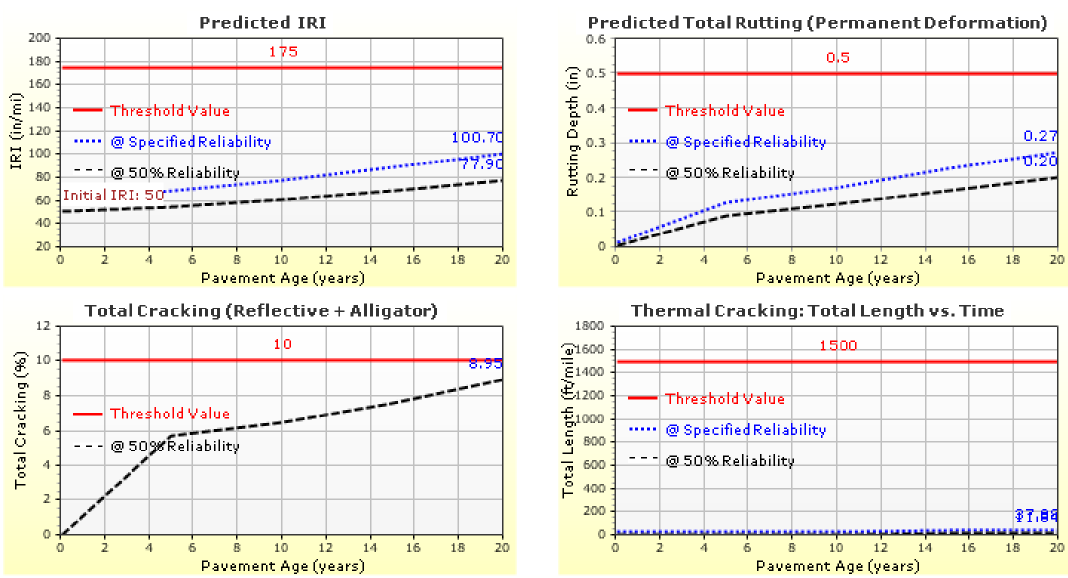


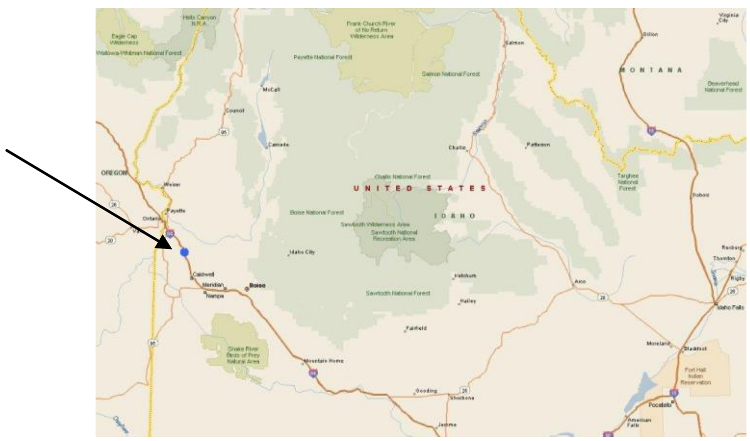
Figure 118. Optimized Pavement Design Inputs and Outputs Summary

Appendix D.
Idaho JPCP Restoration Design Example

Project Description

For this concrete pavement rehabilitation (CPR) design example, the pavement section from the new JPCP design example (Appendix B) was used as the existing pavement. The project location on I-84 near Payette, ID is shown in Figure 119. The roadway was originally constructed in October 1983 and later adopted in the LTPP program with WIM Site No. 3023.

As of 2008, the existing pavement exhibited moderate amounts of cracking and severe amounts of faulting and IRI, thus precipitating the need for diamond grinding. The AADTT in 2008 was 2,113 trucks with 6.4 percent linear growth.



Close-Up View of Project Surroundings

Figure 119. JPCP Restoration Design Example Location

Pre-Design Issues

Prior to the start of design and analysis, the pavement designer must assemble all key inputs required for this pavement type and decide on the hierarchical level of inputs for each key input category. Key inputs required for JPCP restoration or CPR design is presented in Table 77. Based on the functional class (Interstate) and location (rural) of the roadway, Level 2/3 inputs were generally assumed to be adequate. Inputs such as initial truck traffic volume (AADTT) and projected future growth rate must always be estimated at Level 1.

Table 77. Key Inputs Required for JPCP Restoration or CPR Design

Input Category		Input Variables
General Information		Design Type & Pavement Type
		Existing Construction Date (month/year)
		Pavement Construction Date (month/year)
		Traffic Opening Date (month/year)
"Design Life" & Reliability		"Design Life" (years)
		Design Reliability (%)
Existing Pavement Condition		Slabs Distressed Before Restoration (% slab)
		Slabs Repaired After Restoration (% slab)
Performance Criteria		Initial IRI (in./mile)
		Terminal IRI (in./mile)
		Transverse Cracking (% slabs cracked)
		Transverse Joint Faulting (in.)
Traffic		Initial Two-Way AADTT & Truck Growth
		Number of Lanes
		Directional Distribution & Lane Distribution
		Vehicle Class Distribution
		Hourly Adjustments
		Monthly Adjustments
		Number of Axles per Truck
		Axle Load Distribution
Structure & Materials Properties	PCC (surface layer)	28-Day Flexural Strength, 28-Day Elastic Modulus, Coefficient of Thermal Expansion, Cement Type, Cement Content, Water-to-Cement Ratio
	Crushed Base	Engineering Properties &Atterberg Limits, M_r at Optimum Moisture Content, Thickness
	Granular Subbase	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness
	Compacted Subgrade	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness
	Natural Subgrade	Engineering Properties & Atterberg Limits, M_r at Optimum Moisture Content, Thickness
Project-Specific Calibration Factors		ITD Local Calibration Coefficients for New JPCP (Adopted from the NCHRP 20-07/Task 288 study)

Develop a "Trial Design"

"Trial Design" begins with opening the *AASHTOWare Pavement ME Design* software and selecting the appropriate design type and pavement type, which for this design example are "Restoration" and "JPCP Restoration." Design for a restoration project does not involved thickness but determines

1. If the existing slab has sufficient structural capacity to handle the future traffic loadings without excessive fatigue cracking.
2. If retrofit dowels are required to control faulting.
3. If retrofit PCC tied shoulders are required to control faulting and fatigue cracking.

Additional information is presented in the *AASHTOWare Pavement ME Design* “HELP System.”

Next is to create the “Trial Design” by populating several screens of the *AASHTOWare Pavement ME Design* user interface. The “Trial Design” file, once completed, must be saved and reviewed for accuracy and wrong entries. Files should be name using standard ITD conventions. For this example, the filename “CPR.dgpx” is used.

NOTE: The output summary file names will be based on the input file name.

Details of how the *AASHTOWare Pavement ME Design* project is created and populated with “Trial Design” inputs are presented in the following sections.

“Design Life”

Table 1 of this *User’s Guide* provides information on pavement design life. For JPCP restoration or CPR, the recommended “Design Life” is 20 years. Thus, a 20-year “Design Life” was selected (see Figure 120).

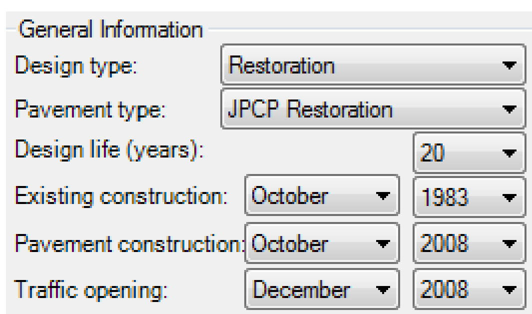


Figure 120. JPCP Restoration Design Example Construction (month/year)

Construction and Opening Dates

AASHTOWare Pavement ME Design requires information on anticipated construction or placement date (month/year) for both the existing pavement and restoration. This information is used for setting the baseline climate and traffic at construction. Anticipated month and year of existing PCC layer placements must be determine based on typical ITD practices (i.e., the seasons in which pavements are normally constructed). Also required is the anticipated month and year for which the completed pavement will be opened to traffic. Again, this input must be selected based on typical ITD construction practices. For this example, the following were selected:

- Existing Pavement Construction (month/year): October 1983.
- PCC Layer Placement (month/year): October 2008.
- Traffic Opening (month/year): December 2008.

Performance Criteria & Design Reliability

Designers must set pavement performance criteria from which a "Trial Design" is accepted or rejected. Performance criteria are basically critical distress and smoothness levels that ITD allows for a given pavement type and functional class. *AASHTOWare Pavement ME Design*, as part of its evaluation of a "Trial Design", predicts distress and smoothness over a specified analysis period "Design Life". Predicted distress and smoothness at the end of the "Design Life" are compared to the preset threshold values. If predicted distress and smoothness are greater than the preset threshold values, the "Trial Design" is rejected. *AASHTOWare Pavement ME Design* allows designers to predict distress and smoothness at various levels of reliability. Chapters 3 and 4 present guidance for selecting performance criteria and reliability levels.

For this JPCP restoration design example, the performance criteria recommended for an interstate highway were used (see Table 7). A reliability level of 95 percent was selected based on the pavement's functional class (see Table 9).

AASHTOWare Pavement ME Design requires an estimate of initial pavement smoothness (IRI right after restoration or diamond grinding). This is an important input, as the time from initial construction to attaining the threshold IRI value depends greatly on the initial IRI obtained at the time of construction. The initial IRI value provided in the design must be attained in the field and thus must reflect ITD practices. For new and restored JPCP, an initial IRI of 65 in./mile is used in this example. Designers can vary this input if there is reason to believe a different value would better reflect initial smoothness values for a given project. The 10 percent cracking would be new fatigue cracking occurring over the design period of JPCP restoration.

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	65	
Terminal IRI (in./mile)	160	95
JPCP transverse cracking (percent slabs)	10	95
Mean joint faulting (in.)	0.12	95

Figure 121. Performance Criteria and Reliability for JPCP Restoration Design Example

Traffic

Traffic inputs are the same as for the new JPCP design example (Appendix B), except the initial two-way AADTT is 2,113 trucks with 6.4 percent linear growth.

Climate

The climate inputs are the same as for the new JPCP design example (Appendix B).

Rehabilitation Inputs

Rehabilitation design requires a few additional inputs and some modifications of other inputs that are related to the existing pavement. In rehabilitation design, the existing pavement typically has deteriorated from its original condition through all types of fracture, distortion, or material disintegration. Some of the material properties may also have changed over time. *AASHTOWare Pavement ME Design* can account for these effects through modifying various design inputs and through a few new inputs related to the condition of the existing pavement. These modifications are used to adjust the various moduli of the existing pavement.

For JPCP restoration or CPR, two rehabilitation inputs are required: slabs distressed before restoration (percent) and slabs repaired after restoration (percent). For this design example, pavement condition information was obtained from field measurements. After 25 years in service, the existing JPCP reached the threshold for faulting and IRI. Thus, ¼ inch diamond grinding was considered. In addition, 0.5 percent slabs were also repaired (roughly 2 slabs per mile). Thus, the percentage of slabs distressed before restoration was 0.5 percent, and the percentage of slabs replaced after restoration was 0.5 percent, leaving 0 percent slabs cracked at the beginning of the "Design Restoration Period" (see Figure 122).

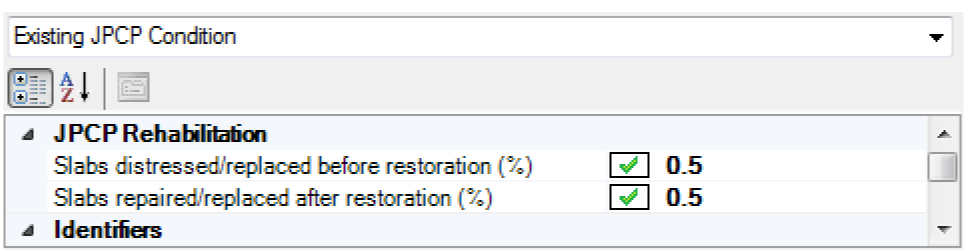


Figure 122. JPCP Rehabilitation Input Screen

Foundation Support

Modulus of subgrade reaction (subgrade dynamic k-value) is used for characterizing foundation support for existing JPCP, rather than the E_s modulus. The effective dynamic k-value can be determined from back-calculation (MODULUS 6) or alternately from the AREA of each deflection basin. The mean subgrade dynamic k-value along the project is then entered into the software along with the month of FWD testing. The k-value from FWD back-calculation represents the stiffness of the unbound compressible soils (at least 10 or more feet deep into the subgrade) beneath the JPCP slab. This k-value provides the needed subgrade support modulus for design purposes. *AASHTOWare Pavement ME Design* does not use the subgrade M_r value, although it is entered. A dynamic k-value of 240 psi/in. and month of testing as June were entered into the software.

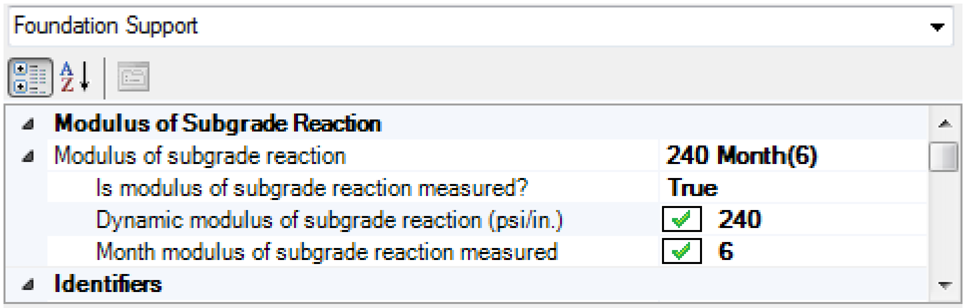


Figure 123. Foundation Support Input Screen

Existing Pavement Structure

This design example includes an existing JPCP constructed over an aggregate base course placed over the granular subbase and natural subgrade. The JPCP over an aggregate base layer structure is shown in Figure 124.

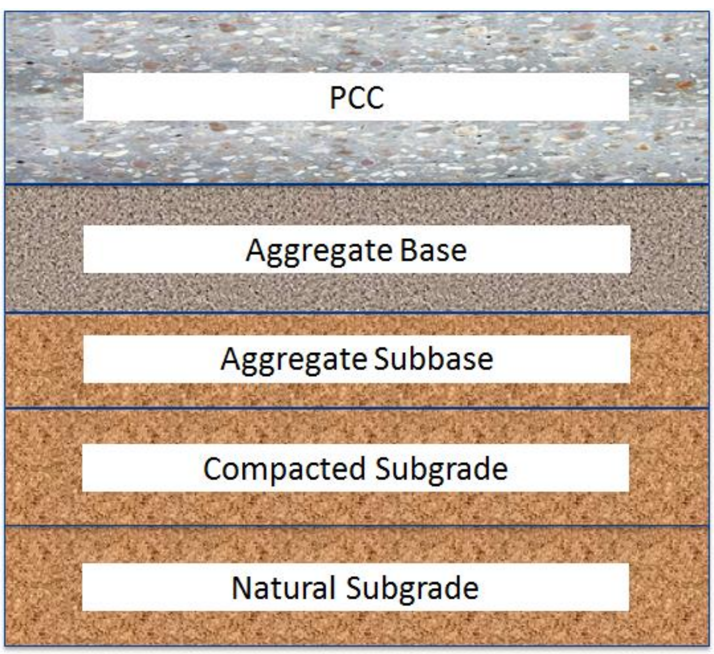


Figure 124. Existing JPCP Existing Design Structure

The general description of pavement layer structure, starting from the bottom foundation support, is described in detail as follows:

- **Bedrock:** Highly fractured and weathered or massive continuous (intact) rock with 10 to 20 ft of the pavement foundation, if present.

- **Natural Subgrade:** The nature of the subgrade foundation (including depth to bedrock and groundwater table) is mostly determined directly from subsurface exploration and testing activities. Key for pavement design is to determine:

    1. Natural/compacted subgrade properties and depth.
    2. Depth to bedrock.

    Natural and compacted subgrade soil properties is obtained from tests on the natural foundation soil in its in-situ condition and in its compacted state as the upper layers (12 to 24 in.) is rolled and compacted or removed and replaced during construction.

- **Compacted or Prepared Subgrade:** This is typically wetting, rolling, and compacting the top 12 to 24 in. surface of the natural subgrade to produce a firm compact surface with sufficient strength to support construction equipment and other activities. Subgrade preparation may also include stabilization with lime or other chemicals to reduce plasticity, improves workability, minimize shrinkage/swell, increases compressive strength CBR and M_r, and provide long term durability in very adverse conditions.

- **Granular Subbase:** The existing subbase material is classified as AASHTO A-1-a and A-1-b soils.

- **Crushed Base**: The existing material is classified as AASHTO A-1-a and A-1-b soils.

- **PCC Layer:** The existing PCC layer was 9 inches thick.

Guidance for obtaining pavement layer properties and thicknesses to define the trial JPCP structure is presented in Chapter 10 of this *User's Guide.* For this design example, a Level 2/3 hierarchical input was adopted. *AASHTOWare Pavement ME Design* recommends that, once the "Trial Design" is defined, material properties must be populated starting from the lowest layer bedrock or natural subgrade to the surface layer.

The *AASHTOWare Pavement ME Design* "HELP System" provides detailed guidance on how to input pavement structure and layer materials input data.

Bedrock

Review of historical subsurface exploration and testing reports for this location showed there was no bedrock within a 50-ft depth. Thus, a bedrock layer was not needed.

Natural Subgrade

Subsurface exploration and testing reports indicate the natural subgrade for this location is AASHTO A-2-4 soil. The engineering properties required by *AASHTOWare Pavement ME Design* at Level 2/3 for the natural subgrade are presented in Table 78.

Table 78. Required Engineering Properties for the Existing Natural Subgrade

Engineering Properties	Level of Input	Source of Data
Gradation	2	Obtained through subsurface exploration & testing.
Atterberg Limits (Liquid Limit And Plasticity Index)	2	Obtained through subsurface exploration & testing.
Maximum Dry Unit Weight	3	Computed internally by the software.
Saturated Hydraulic Conductivity	3	Computed internally by the software.
Specific Gravity of Solids	3	Computed internally by the software.
Optimum Gravimetric Water Content	3	Computed internally by the software.
Soil Water Characteristic Curve	3	Computed internally by the software.
Resilient Modulus (M_r)	2	Dynamic k-value or elastic modulus is back-calculated from FWD deflection testing data, then the field modulus is converted to laboratory conditions (see Table 37).

Figure 125 shows the subgrade engineering properties (gradation and Atterberg limits) obtained from laboratory testing, coded into *AASHTOWare Pavement ME Design*. Based on these two properties, *AASHTOWare Pavement ME Design* internally estimates maximum dry unit weight, saturated hydraulic conductivity, specific gravity of solids, optimum gravimetric water content, and soil water characteristic curve. The designer must check the estimated soil engineering properties for accuracy and reasonableness. If the *AASHTOWare Pavement ME Design* estimates are deemed unreasonable, the designer can override the internally estimated values. Guidance for overriding the engineering properties is provided in the *AASHTOWare Pavement ME Design* "HELP System."

FWD testing was performed on the existing JPCP, and the back-calculated k-value was entered directly into the software. Therefore, a dummy M_r value of 16,000 psi was entered into the software since the software does not use subgrade M_r value.

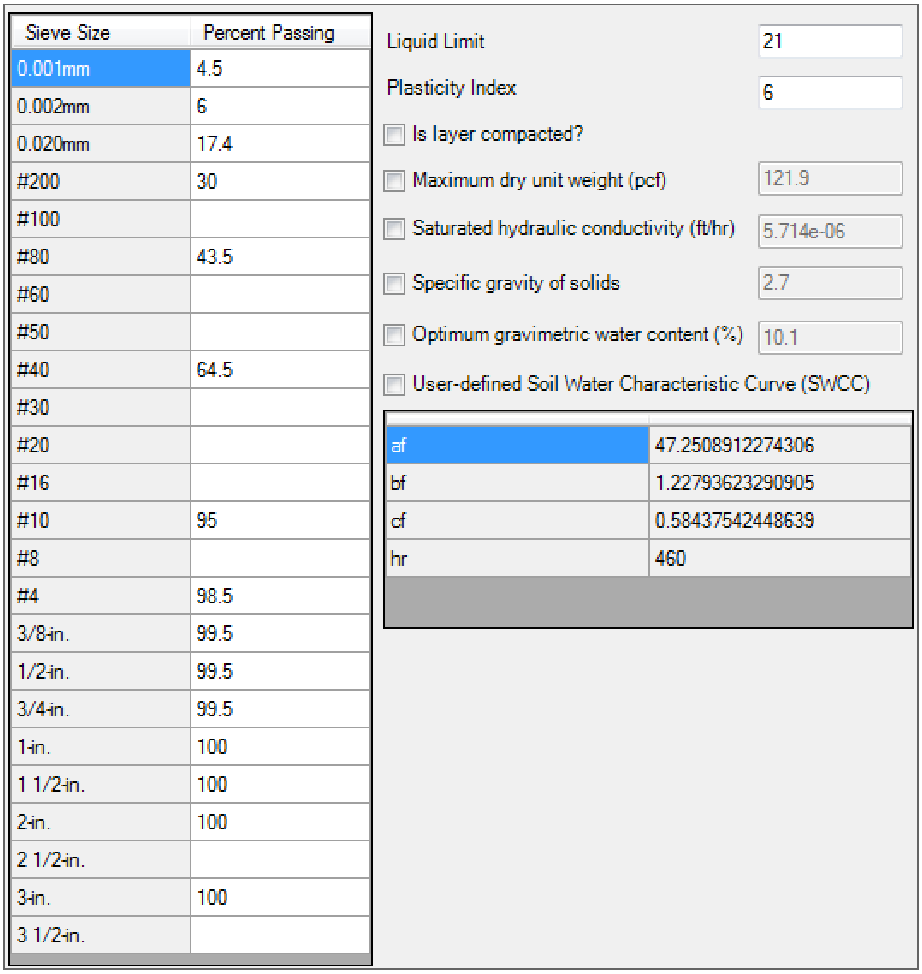


Figure 125. Natural Subgrade Engineering Properties Input Screen for Existing JPCP

Compacted Subgrade Layer

Records from the LTPP database indicate that 9 in. of the natural subgrade was rolled and compacted. This layer was not chemically treated. The engineering properties and M_r for this layer were similar to those of the natural subgrade. The M_r was assumed as 20,000 psi. The main distinction between these layers is that the "Layer Compacted" box on the input screen was check to reflect field conditions (rolled and compacted subgrade layer). See Figure 126. Also, a layer thickness of 9 in. was entered into *AASHTOWare Pavement ME Design.*

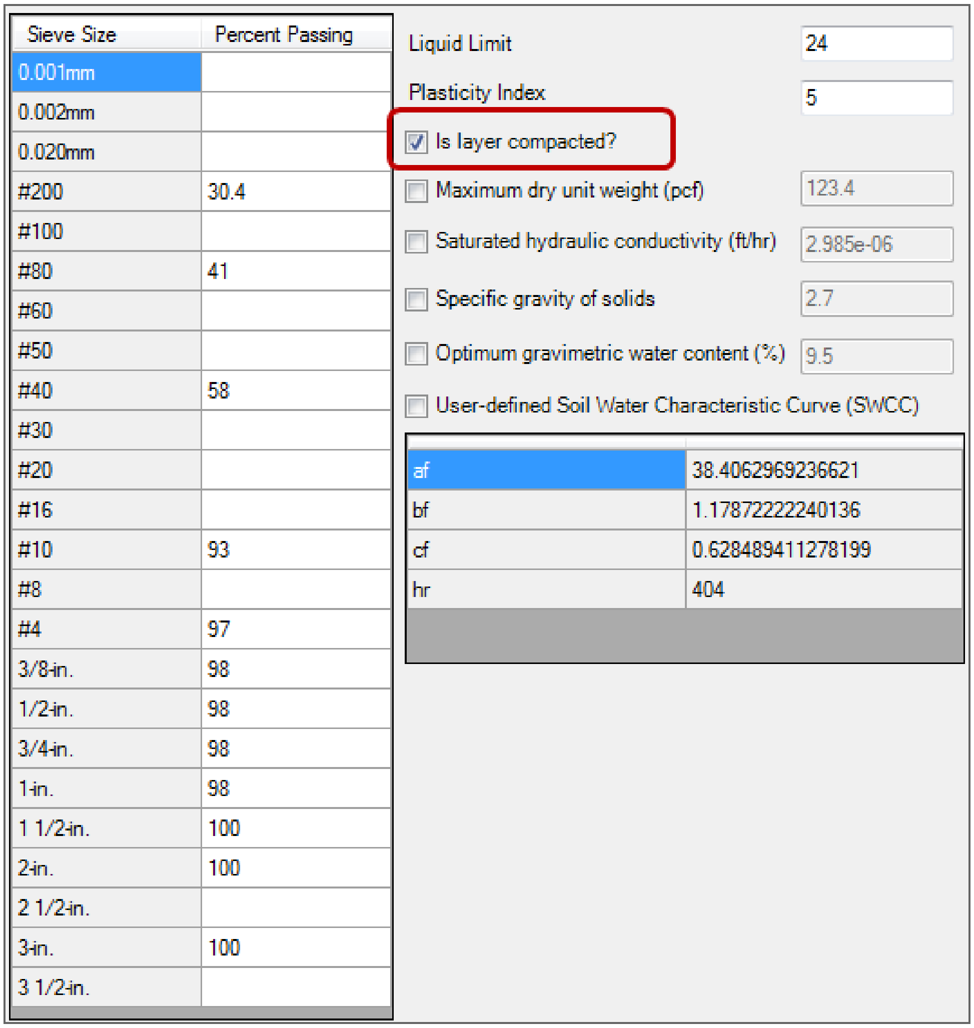


Figure 126. Compacted Subgrade Engineering Properties for Existing JPCP

Granular Subbase Layer

For this example, 5.3 in. of granular subbase was assumed from the LTPP database. This layer was not chemically treated. The engineering properties and M_r for this layer were similar to those of the natural subgrade. The M_r was assumed as 25,000 psi. The "Layer Compacted" box on the input screen was check to reflect field conditions (rolled and compacted layer). A layer thickness of 5.3 in. was entered into *AASHTOWare Pavement ME Design*.

Crushed Gravel Base Layer

The aggregate base material type for use in pavement construction is selected based on the project location (mostly, the nearest source of high-quality aggregate material is selected). The ITD "crushed gravel" material classification and properties are determined based on the source of the material.

For this design example, the material classification was selected as A-1-a. As shown in Table 39, crushed gravel has properties comparable to AASHTO soil class A-1-a, and M_r at optimum moisture for this material is assumed to be 40,000 psi.

Once the base modulus is selected, the designer can enter the aggregate base engineering properties and M_r into *AASHTOWare Pavement ME Design*. For this example, AASHTO A-1-a soil gradation and Atterberg limits were assumed from LTPP database. A layer thickness of 4.4 inches was assumed and entered into *AASHTOWare Pavement ME Design*.

PCC Layer

Required inputs and values entered for this design example are presented in Table 79.

NOTE: The input requirements were for Level 2/3. Designers are encouraged to use the best estimates of inputs available.

Thus, any or all of these inputs can be overridden if more accurate Level 1 data are available. Figures 127 through 129 presents the *AASHTOWare Pavement ME Design* screens used to enter PCC material properties for the PCC layer.

Table 79. Required Engineering Properties for the Existing PCC Layer

Engineering Properties	Level of Input	Source of Data
Layer Thickness	1	The existing PCC thickness of 9 in. obtained from new JPCP design example. ¼ in. diamond grinding was assumed. Therefore, PCC thickness of 8.75 in. was entered into the software.
Flexural Strength (M_r)	3	28-day flexural strength of 775 psi from original project files.
Elastic Modulus	3	28-day elastic modulus of 3,365,116 psi estimated from the flexural strength.
Unit Weight (pcf)	1	Unit weight of 140.5 from the files.
Poisson's Ratio	1	Poisson's ratio of 0.16 was assumed.
CTE (^{o}F)	1	CTE of 4.31 was measured from cores in the lab.
Cement Type	1	Type II cement type.
Cementitious Material (PCC + Pozzolans) (lb/yd^3)	1	564 from construction database.
Water-to-Cement Ratio (w/c)	1	0.4 was assumed.
Coarse Aggregate Type	3	Not required.
PCC Zero Stress Temperature (^{o}F)	3	*AASHTOWare Pavement ME Design* computes internally.
Ultimate Shrinkage (microstrain)	3	Computed internally by the software.
Reversible Shrinkage	3	Use *AASHTOWare Pavement ME Design* default of 50% unless more accurate information is available
Time to Develop 50% of Ultimate Shrinkage	3	Use *AASHTOWare Pavement ME Design* default of 35 days unless more accurate information is available.
Curing Method	3	Curing compound.
Thermal Conductivity	3	Assume *AASHTOWare Pavement ME Design* defaults.
Heat capacity	3	Assume *AASHTOWare Pavement ME Design* defaults.

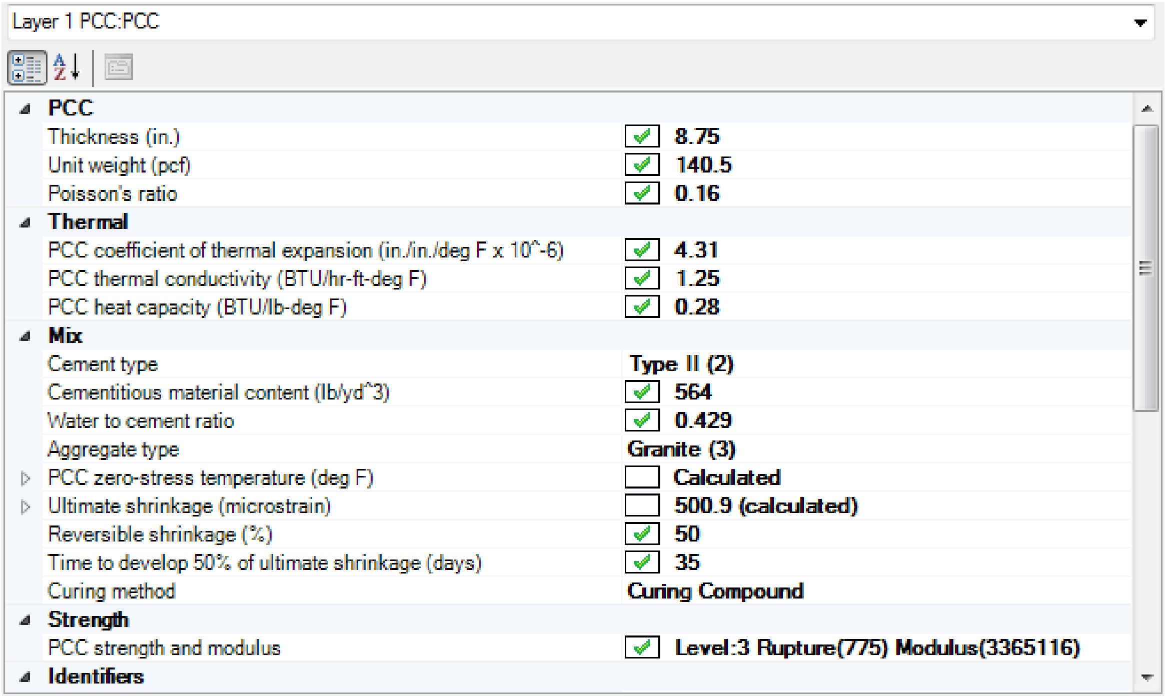


Figure 127. Screen for PCC Material for the Example Existing JPCP

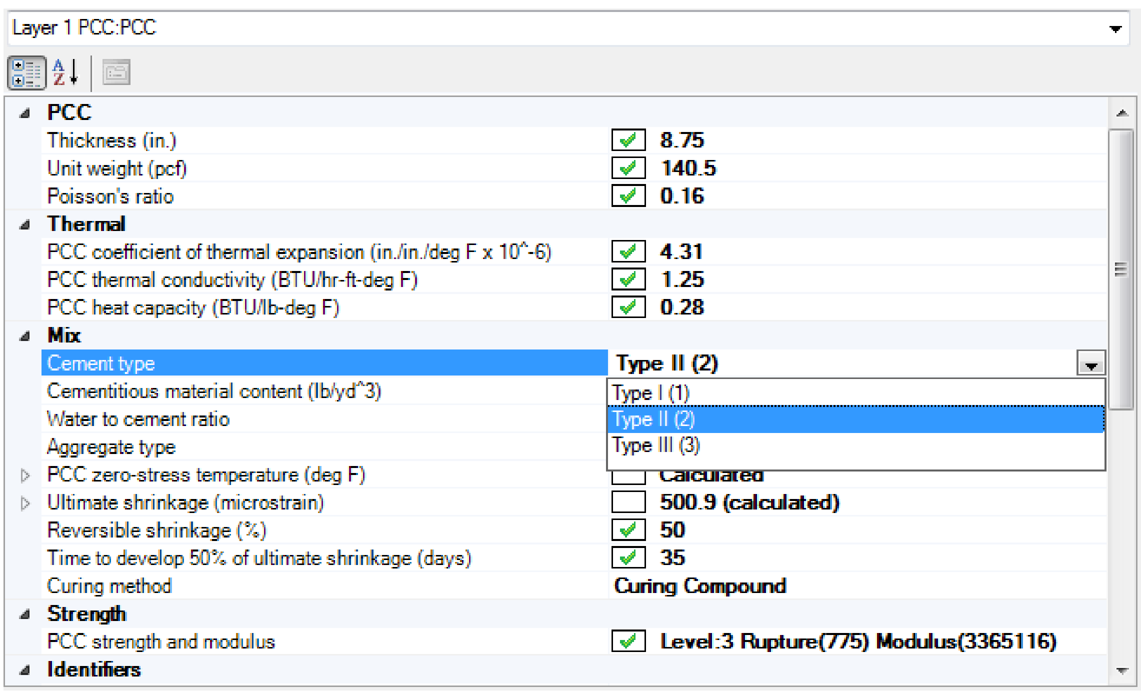


Figure 128. Screen for Cement Type for the Example Existing JPCP Design

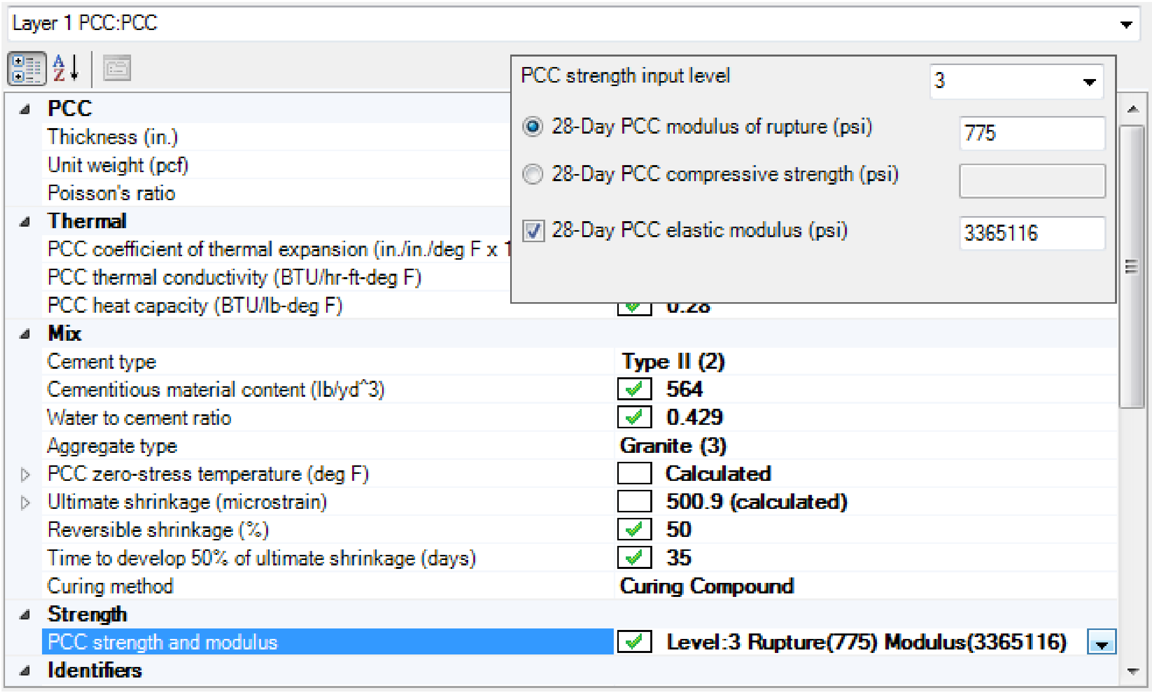


Figure 129. Screen for PCC Strength and Modulus for the Example Existing JPCP

JPCP Design Inputs

Selecting representative design features for the "Trial Design" is very important. Table 80 presents a summary of the design inputs selected for this project based on recommendations presented in the *User's Guide* and ITD policy and practices.

Designers can override all of these additional inputs if warranted. JPCP design features used in this example shown in Figure 130.

Table 80. Summary of Existing JPCP Design Inputs

Trial JPCP Design Inputs	Recommended Values
Permanent Curl/Warp Effective Temperature Difference (ºF)	-10
Joint Spacing (ft)	Project specific joint spacing of 15 ft was selected.
Sealant Type	Silicone.
Load Transfer Mechanism (round dowel bars)	No dowels were used in the existing project. NOTE: Based on joint faulting predictions for this "Trial Design," retrofit dowels may be required.
Dowel Diameter (in.)	If needed, 4 dowels per wheelpath will be placed with a diameter of 1.25 or 1.5 in.
Dowel Bar Spacing (in.)	12 inches if needed.
Edge Support	Conventional 12 ft slab with tied PCC shoulders.
Existing Base Type	Crushed gravel base material.
PCC-Base Interface Friction	The following lengths of time for full contact friction between the PCC slab & base course are recommended by ITD & thus adopted for this "Trial Design." For unbound crushed gravel base, use full design analysis period (240 months).
Erodibility Index Of Base	ITD recommends for granular aggregate base a value of 4 (i.e., fairly erodible material).

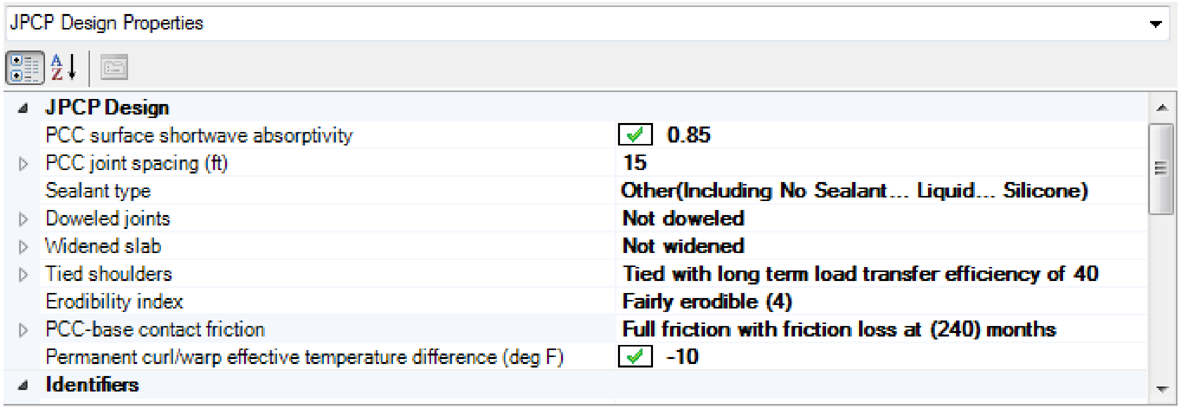


Figure 130. Screen for Existing JPCP Design Inputs

ITD JPCP Restoration Project-Specific Calibration Coefficients

When *AASHTOWare Pavement ME Design* is used for Idaho conditions, ITD recommends the calibration coefficients presented in Figure 131 for distress and IRI models. These coefficients were adopted from the NCHRP 20-07/Task 288 study and should be used until ITD establishes its own local calibration coefficients. Designers must check if the new JPCP project being designed is applying the calibration coefficients presented. If not, guidance is provided in the *AASHTOWare Pavement ME Design* "HELP System" on how to replace the global calibration coefficients with ITD-recommended values.

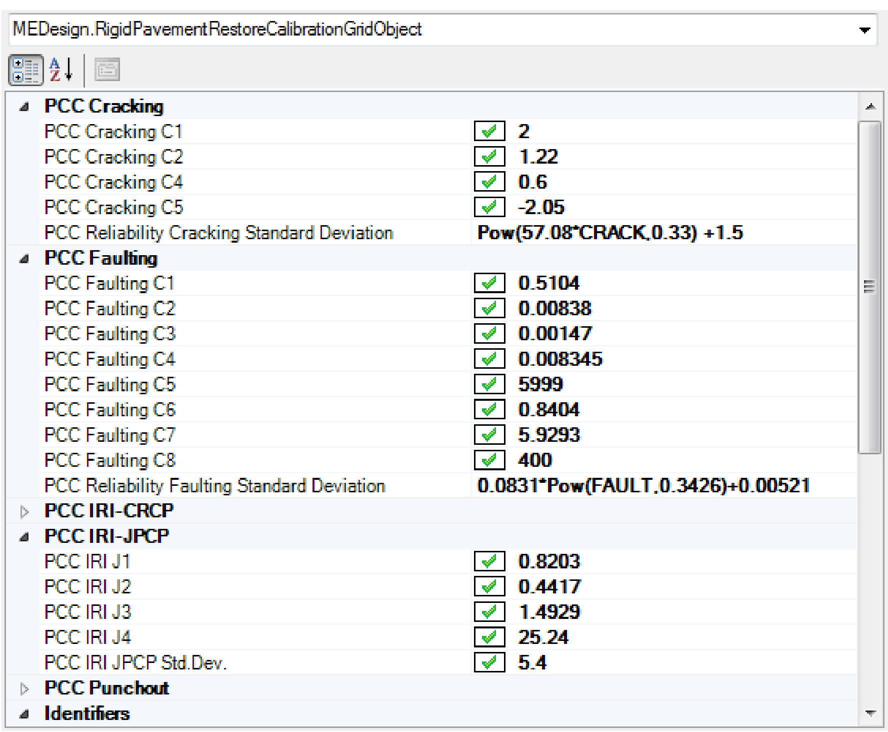


Figure 131. JPCP Restoration Calibration Coefficients

Run *AASHTOWare Pavement ME Design* and Review/Analyze Outputs

Pavement design using *AASHTOWare Pavement ME Design* is iterative. The designer must run the software, check key outputs for reasonableness, and check the "Trial Design" for adequacy. The check for adequacy must be done at the ITD-recommended reliability level. If the "Trial Design" is deemed inadequate, appropriate modifications must be made to obtain a feasible "Final Design."

Check of Key *AASHTOWare Pavement ME Design* Outputs for Reasonableness

It is important for designers to review key inputs and outputs for each *AASHTOWare Pavement ME Design* run to ascertain whether inputs were entered correctly and the software processed input data correctly and produced expected results. *AASHTOWare Pavement ME Design* produces two output files with summary of key inputs and design outcomes, a PDF file and an XLS file. The information contained in the PDF output file is adequate for this review and is presented under the following general headings:

- **Design Inputs:** Contains information about key designs inputs such as pavement structure definition, layer thicknesses, and traffic projections (see Figure 132).

- **Design Outputs:** Distress prediction summary in tabular and graphical forms (See Figure 133).

- **Traffic Input and Output Summary:** Graphical and tabular representation of key traffic inputs and projected growth and seasonal adjustments (see Figures 134 and 135):
  - Traffic Distributions: Tabular representation of traffic inputs.
  - Axle Configuration: Axle configuration summary.
  - AADTT Growth: Plots showing trends in truck growth.
  - AADTT Growth by Class: Tabular representation of growth in trucks.

- **Climate Inputs and Output Summary:** Graphical and tabular representation of key climate inputs and climate variable statistics (see Figure 136):
  - Climate data sources (weather stations).
  - Annual statistics of key variables such as temperature, precipitation, freezing index, etc.
  - Monthly statistics of key variables: temperature, precipitation, freezing index, etc. in a graphical format.

- **JPCP Design Features:** Key pavement design input summary information.

- **Key PCC, Base, Subbase, and Subgrade Material Inputs and Computed Parameters:** including plots of seasonal effects on base and subgrade.

- **Analysis Output Charts:**
    - Plots of predicted IRI, transverse cracking, and faulting versus age in graphical format.
    - Detailed breakdown of predicted distress and IRI:
        - Plots of predicted bottom-up and top-down damage versus age in graphical format.
        - LTE versus age.

- **Layer Information:** Detailed summary of data for all layers within the pavement structure.

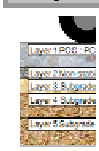
- **Calibration Coefficients:** Detailed summary of project-specific distress/IRI models calibration coefficients.

Designers are encouraged to examine the information presented under these headings. Possible discrepancies between input data summaries and what was entered into *AASHTOWare Pavement ME Design* must be resolved.

Design Inputs

Design Life:	20 years	Existing construction:	October, 1983	Climate Data Sources	44.021, -117.013
Design Type:	JPCP Restoration	Pavement construction:	October, 2008		43.565, -116.22
		Traffic opening:	December, 2008		

Design Structure



Layer type	Material Type	Thickness (in.):
PCC	PCC	8.8
NonStabilized	Crushed Gravel	4.4
Subgrade	Soil Aggregate Mixture	5.3
Subgrade	Soil Aggregate Mixture	9.0
Subgrade	A-2-4	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	-
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2008 (initial)	2,113
2018 (10 years)	4,473,270
2028 (20 years)	11,169,300

Figure 132. *AASHTOWare Pavement ME Design* PDF Output File Summary of Structural Design Inputs for JPCP Restoration

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	497.90	95.00	0.38	Fail
Mean joint faulting (in.)	0.12	0.70	95.00	0.00	Fail
JPCP transverse cracking (percent slabs)	10.00	8.35	95.00	97.70	Pass

Distress Charts

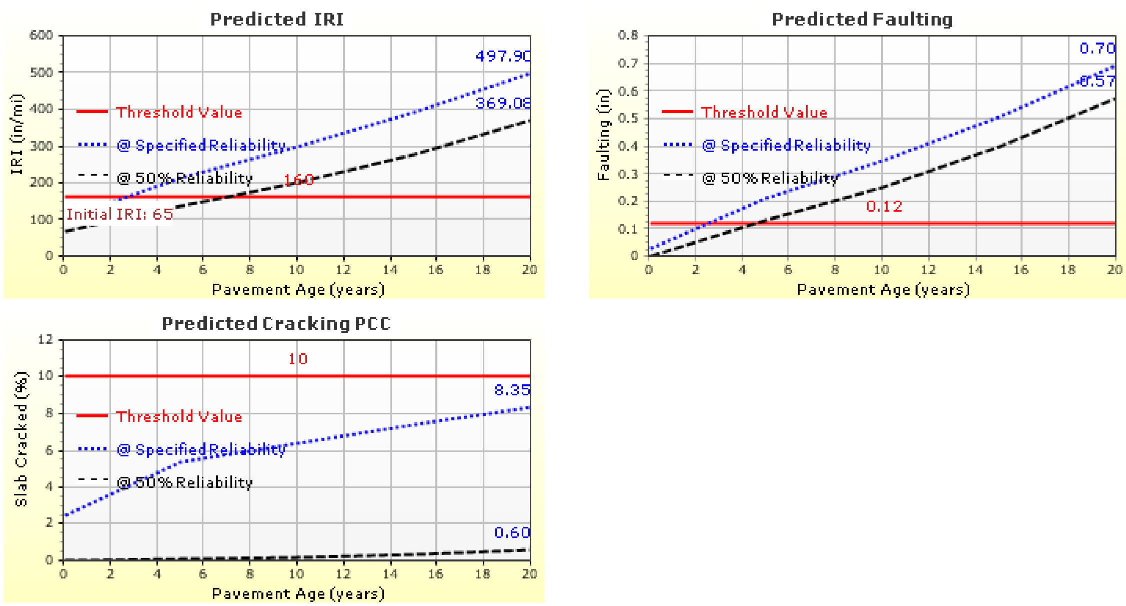


Figure 133. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Design Outputs for JPCP Restoration**

Traffic Inputs

Graphical Representation of Traffic Inputs

Initial two-way AADTT:	2,113	Percent of trucks in design direction (%):	50.0
Number of lanes in design direction:	2	Percent of trucks in design lane (%):	90.0
		Operational speed (mph)	65.0

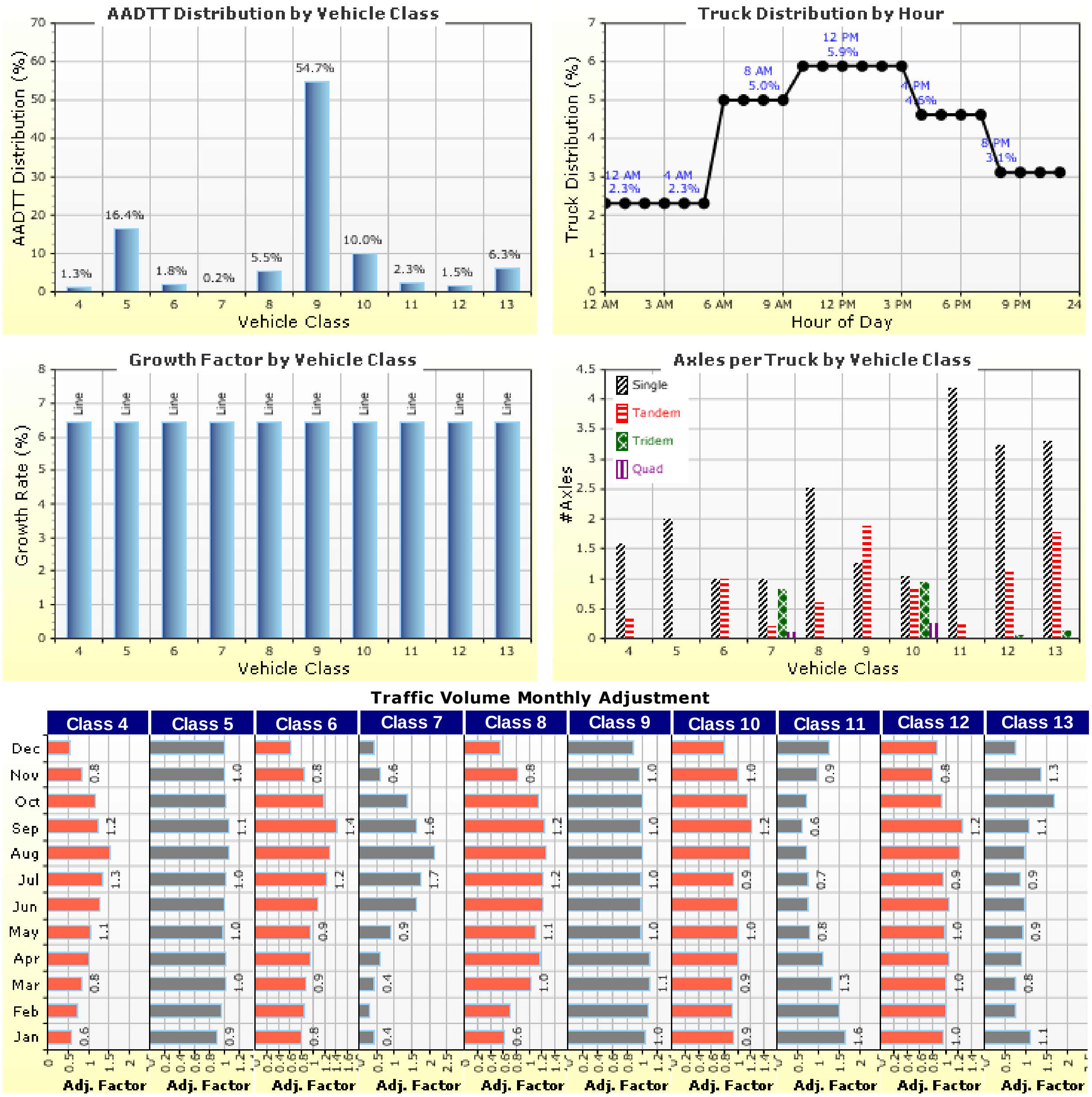


Figure 134. *AASHTOWare Pavement ME Design* PDF Output File Summary of Traffic Inputs for JPCP Restoration

AADTT (Average Annual Daily Truck Traffic) Growth

*** Traffic cap is not enforced**

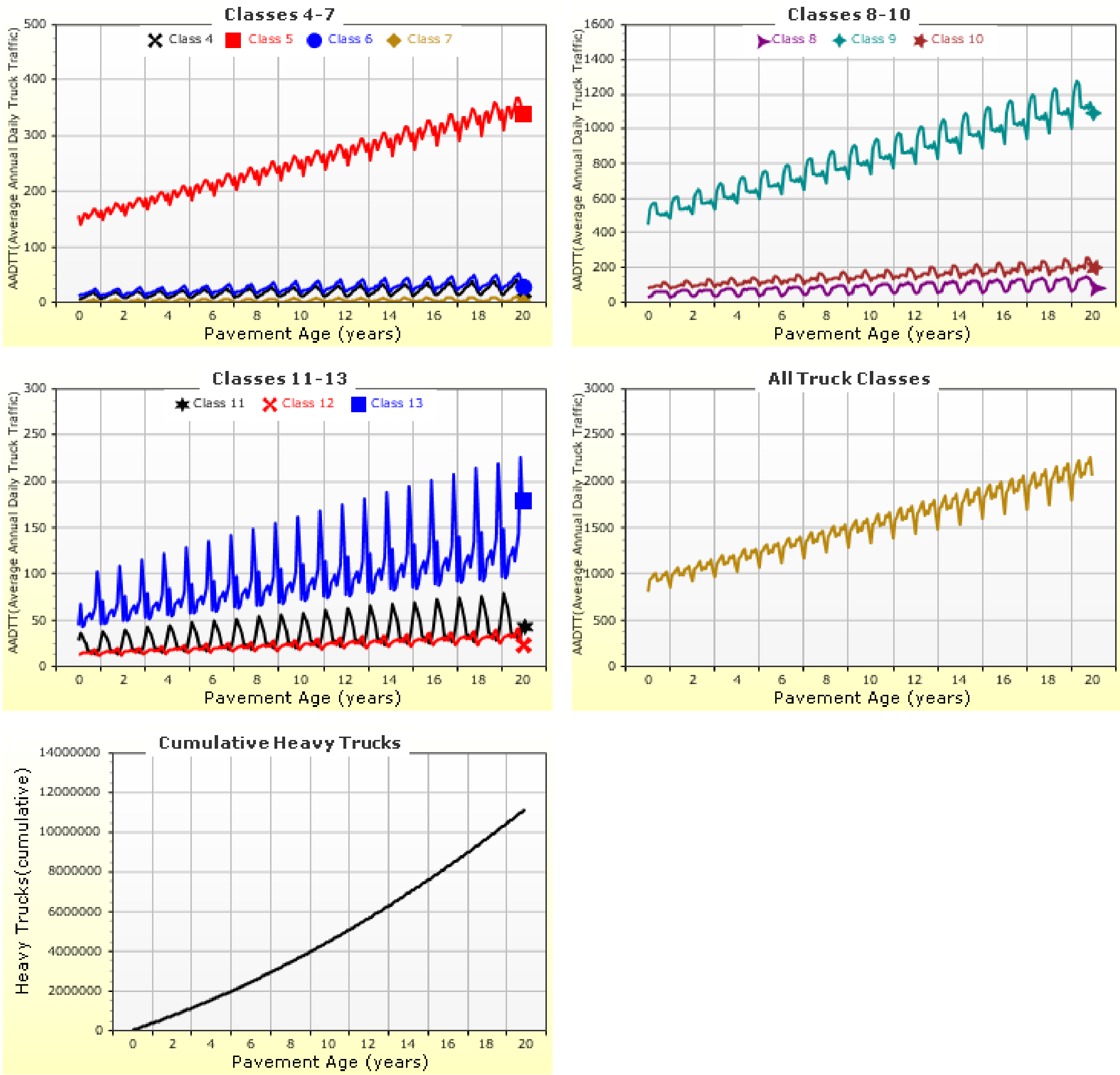
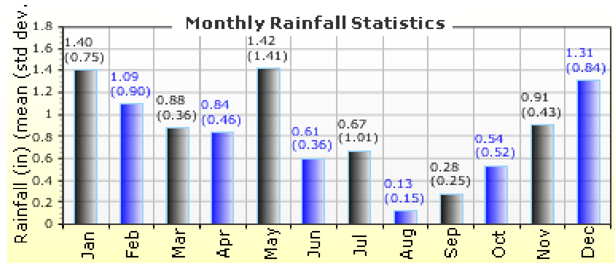


Figure 135. ***AASHTOWare Pavement ME Design*** **PDF Output File Summary of Traffic Outputs (projection of AADTT) for JPCP Restoration**

Climate Inputs

Climate Data Sources:

Climate Station Cities:	Location (lat lon elevation(ft))
ONTARIO, OR	44.02100 -117.01300 2184
BOISE, ID	43.56500 -116.22000 2814

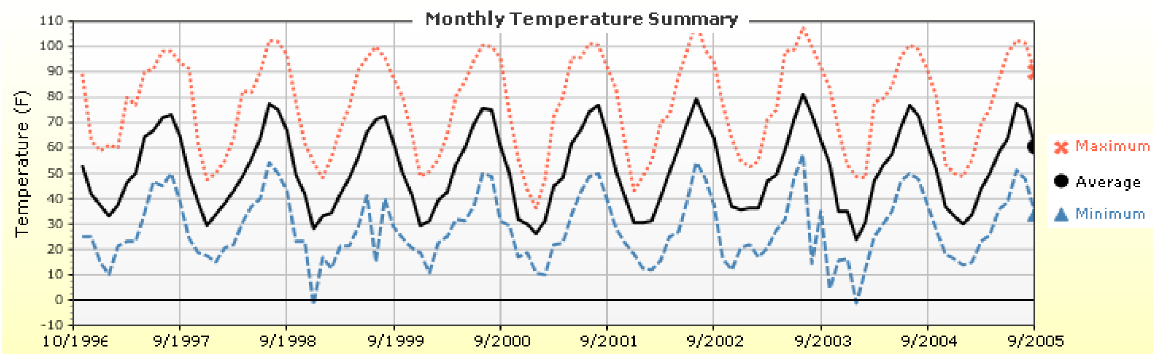


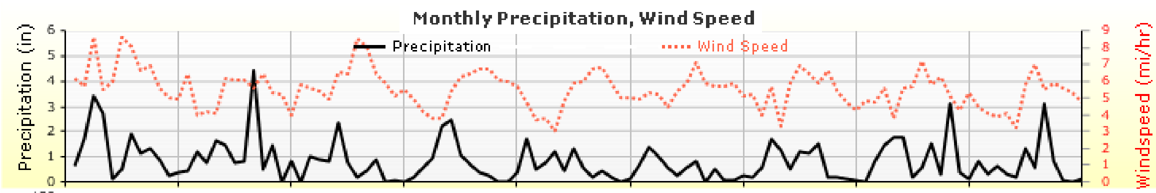
Annual Statistics:

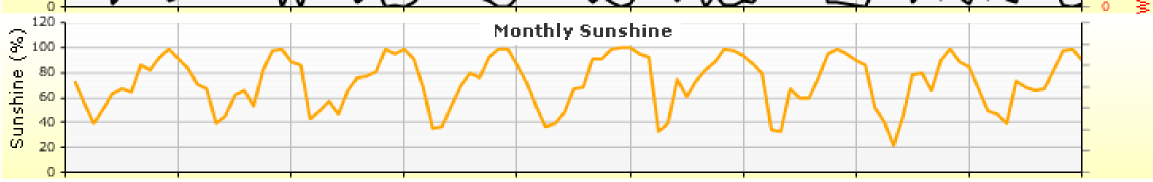
Mean annual air temperature (ºF)	51.83
Mean annual precipitation (in.)	10.12
Freezing index (ºF - days)	314.44
Average annual number of freeze/thaw cycles:	102.20

Water table depth(ft) 10.00

Monthly Climate Summary:







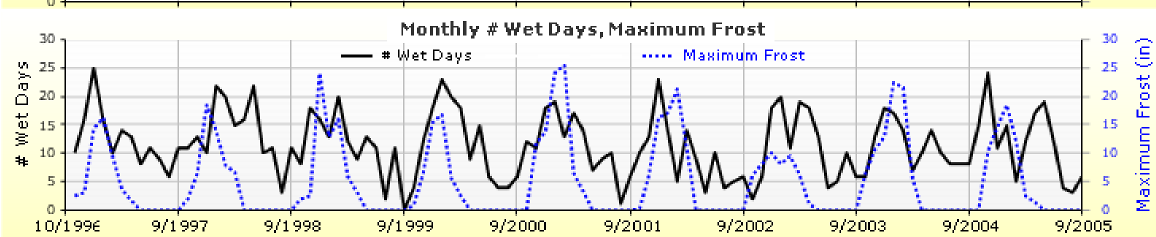


Figure 136. *AASHTOWare Pavement ME Design* PDF Output File Summary of Climate Inputs and Outputs for JPCP Restoration

Check "Trial Design" for Adequacy

The final step is to check the "Trial Design" for adequacy. It should be able to carry anticipated traffic over its "Design Life" at the preset performance criteria recommended by ITD. The outcome of this example "Trial Design" was presented in Figure 133.

Designers must check this output summary to determine whether the design performance criteria are satisfied. This is done as follows:

1. Review the column called "Criterion Satisfied?" in the tabular output and determine for each distress type when the "Trial Design" "passed" or "failed."

2. If the "Trial Design" passes the criteria set for all distress types, then the design is deemed adequate and acceptable.

3. If one or more of the criteria fail, then the design is deemed inadequate and "Trial Design" must be revised as needed and checked again.

For this example, the initial trial run showed excessive faulting and IRI. Thus, this "Trial Design" must be revised.

Revise "Trial Design" and Rerun *AASHTOWare Pavement ME Design* as Needed

A careful review of this JPCP restoration "Trial Design" indicated that excessive IRI is due to excessive faulting. An appropriate solution was to reduce faulting by dowel retrofit. This was done with four 1.25-inch-diameter dowels per wheelpath. The "Modified Design" was rerun, and the outputs from the "Revised "Trial Design" are presented in Figure 137. The results show that, with the revised design, performance criteria such as faulting and IRI satisfied the threshold criteria.

Acceptance of Finalized Design

The final diamond ground thickness was 8.75 inches. Figure 137 shows the final JPCP restoration design structure. *AASHTOWare Pavement ME Design* analysis shows that this design structure, along with the specified layer material types and design properties, would be able to carry approximately 11 million trucks over a 20-year "Design Life." The design outputs also show clearly a more than 95 percent chance that the distress and IRI over the 20 years will be less than thresholds recommended by ITD. This restoration design is thus deemed adequate.

Design Inputs

Design Life:	20 years	Existing construction:	October, 1983	Climate Data Sources	44.021, -117.013
Design Type:	JPCP Restoration	Pavement construction:	October, 2008		43.565, -116.22
		Traffic opening:	December, 2008		

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	PCC	8.8
NonStabilized	Crushed Gravel	4.4
Subgrade	Soil Aggregate Mixture	5.3
Subgrade	Soil Aggregate Mixture	9.0
Subgrade	A-2-4	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

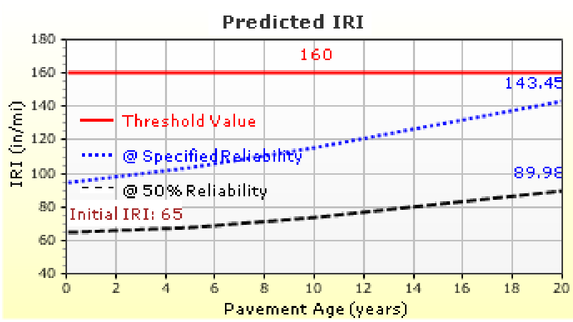
Age (year)	Heavy Trucks (cumulative)
2008 (initial)	2,113
2018 (10 years)	4,473,270
2028 (20 years)	11,169,300

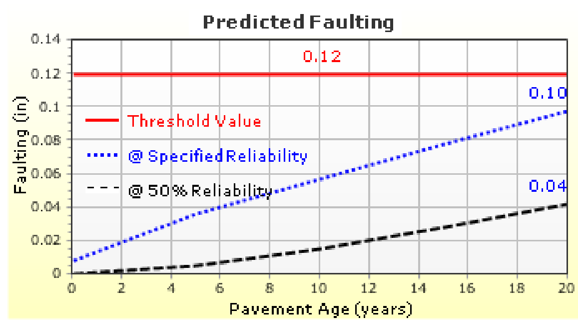
Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	143.45	95.00	98.44	Pass
Mean joint faulting (in.)	0.12	0.10	95.00	98.99	Pass
JPCP transverse cracking (percent slabs)	10.00	8.35	95.00	97.70	Pass

Distress Charts





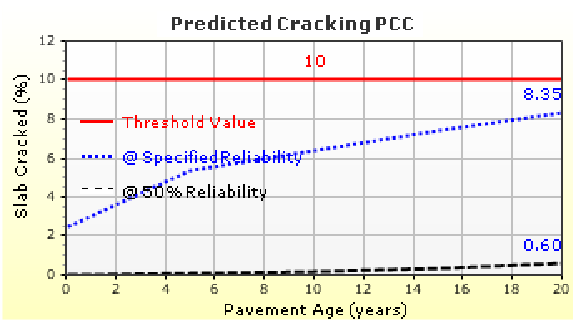


Figure 137. Optimized JPCP Restoration Design Inputs and Outputs Summary