**Development of an Improved Design Procedure for Unbonded Concrete Overlays**

**TPF-5(269)**

**Task 4 Report**

*Develop Mechanistic-Empirical Design Procedure*

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**Table of Contents**

[1.0 Faulting Structural Model and Neural NetWOrk Development 3](#_Toc479768383)

[1.1 Calibration of Totsky Interlayer Parameter 3](#_Toc479768384)

[1.1.1 Reflective Cracking Laboratory Data Analysis 3](#_Toc479768385)

[1.1.2 MnROAD Falling Weight Deflectometer Analysis 8](#_Toc479768386)

[1.2 Modeling Parameters 11](#_Toc479768387)

[1.2.1 Structural Parameters 11](#_Toc479768388)

[1.2.2 Critical Response Parameters 13](#_Toc479768389)

[1.3 Neural Network Development 14](#_Toc479768390)

[2.0 Joint Fauting Model Development for Unbonded Concrete Overlays of Existing Concrete Pavements 19](#_Toc479768391)

[2.1 Previously Developed Faulting Models 19](#_Toc479768392)

[2.2 Faulting Model Framework 23](#_Toc479768393)

[2.2.1 Climatic Considerations 23](#_Toc479768394)

[2.2.2 Model Inputs 25](#_Toc479768395)

[2.3 Calibration Sections 29](#_Toc479768396)

[2.4 Results of Model Calibration 29](#_Toc479768397)

[3.0 SUMMARY AND RECOMMENDATIONS FOR FAULTING MODEL 35](#_Toc479768398)

[4.0 Short Panel Cracking Model Development 36](#_Toc479768399)

[Equivalent Thickness Concept 37](#_Toc479768400)

[Equivalent Temperature Gradient 38](#_Toc479768401)

[Equivalent Structure 39](#_Toc479768402)

[Neural Networks Development 41](#_Toc479768403)

[References 48](#_Toc479768404)

[Appendix A: Calibration Database Information 50](#_Toc479768405)

# Faulting Structural Model and Neural NetWOrk Development

In order to determine joint faulting, the UBOL (Unbonded Concrete Overlays of Existing Concrete Pavements) pavement response was needed from structural modeling. Incremental faulting calculations require many time consuming finite element runs, so the creation of Neural Networks to predict the response greatly decreases run time. Calibration of the Totsky interlayer parameter was needed, which was expanded from the analysis conducted in the Task 3 report. The range of parameters used to generate a factorial of finite element runs and the critical responses to be used in the faulting model were defined. Finally, the development of neural networks to predict the critical responses for the UBOL structure using MATLAB’s Neural Network Toolbox is discussed (MATLAB, 2013).

* 1. **Calibration of Totsky Interlayer Parameter**

In order to accurately model the UBOL structure within ISLAB, the value of the Totsky interlayer k-value must be established for different interlayers. This section details the use of expanded data from the reflective cracking laboratory testing as well as Falling Weight Deflectometer (FWD) data, to establish guidelines for the value of the interlayer Totsky k-value for UBOL design.

* + 1. **Reflective Cracking Laboratory Data Analysis**

The reflective cracking test from Task 2 was modeled in ISLAB and the results from the LVDTs during the test were used to determine the corresponding value of the Totsky interlayer k-value. Figure 1 provides a representation of the model used to determine the Totsky k-value for the different interlayers. Note that the simulated load is applied as a 0.25-in wide line-load along the beam depth of 6 in (indicated in blue in Figure 1a). Thus, the load contact area is 1.5 in2. As the finite element model is static, a single load of 1 kip is applied to determine a response of the beam model to loading.

|  |  |
| --- | --- |
| (a) | |
| (b) | E:\layer.PNG  (c) |

Figure . ISLAB two-dimensional model of Reflective Cracking test, where (a) shows the mesh and load area (plan view), (b) highlights the unsupported area in yellow (plan view), and (c) shows the structure profile view

In ISLAB, the notch at mid-span in the existing concrete is modeled by inserting a joint at mid-span. In the upper layer (the overlay), this joint fully transfers load (the load-transfer efficiency is 100% treated as a rigid joint). However, in the lower layer (the existing concrete), the joint does not transfer the load at all (load transfer efficiency is near-zero). This allows for the test setup to be modeled the same as the laboratory test setup.

With the beam model, a factorial of cases is modeled to observe the response utilizing interlayers of different properties. In each case, only the Totsky interlayer k-value (*ktotsky*) assumed is varied, otherwise the modeled beam has the following properties:

* Layer 1: *hOL* = 6 in, *EOL* = 4,255,000 psi (average of all Reflective Cracking beam overlay elastic moduli), Poisson ratio *ν* = 0.15, unit weight *γ* = 0.087 lb/in3
* Interlayer: *kIL* varied from 100 to 50,000 psi/in
* Layer 2: *hEX* = 6 in, *EEX* = 4,790,000 psi (average of PCC elastic moduli for the “existing” beam of the reflective cracking laboratory specimens), Poisson ratio *ν* = 0.15, unit weight *γ* = 0.087 lb/in3
* Mesh details: Mesh elements are square (0.125 in to a side) for the entire model, as illustrated in Figure 1.1a.
* A static load of 1-kip is applied to determine a linear beam response associated with interlayer properties.

Figure 2 illustrates the final relationship determined for the modeled beam response and the Totsky interlayer stiffness. Also included in the figure is an exponential relationship determined by transforming the variables and finding a linear least-squares fit. As shown in the figure, the R-squared valued for the fitness of the exponential relationship is 0.99, thus the model adequately describes the relationship between model response and the Totsky k-value for this range of values. With the relationship developed in ISLAB, interlayer Totsky k-values can be established for each beam specimen tested and therefore each type of interlayer system included in the laboratory study.

Figure . Relationship between difference in layer deflection (in mils) and Totsky k-value for interlayer from ISLAB

Table 1 presents the reflective cracking beam specimens for each interlayer and the corresponding Totsky k-value. Given the response of the different interlayer beams under a 1-kip load in the lab, the modeled relationship was used to infer an associated Totsky interlayer stiffness. Average and standard deviation of the different interlayers tested in the laboratory are presented in Table 2.

Table . Established Totsky k-values for reflective cracking laboratory testing specimens

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  | **Overlay PCC** | | | **Fabric Type** | **Diff in defl @ 1 kip (mils)** | **Totsky k-value (psi/in)** |
| **Specimen** | **E (psi)** | **f'c (psi)** |
| 15 oz/yd2 geotetextile fabric (Propex Reflectex) | 0429F15OB | 4280000 | 5059 | F15 | 8.27 | 411 |
| 0429F15OC | 4280000 | 5059 | F15 | 10.41 | 325 |
| 0701F15OD | 4430000 | 4632 | F15 | 12.33 | 274 |
|  |  |  |  |  |  |  |
| 10 oz/yd2 geotetextile fabric (Propex) | 0402F10OA | 3880000 | 4512 | F10 | 10.58 | 320 |
| 0501F10OA | 4170000 | 5069 | F10 | 7.76 | 439 |
| 0402F10OA | 3880000 | 4512 | F10 | 9.48 | 358 |
|  |  |  |  | **Asphalt Thickness** |  |  |
| MnDOT Aged, Dense graded Unmilled asphalt | 0417MNDAUA | 3880000 | 4590 | 2.9 | 0.93 | 3824 |
| 0507MNDAUA | 4480000 | 5106 | 2.8 | 2.32 | 1504 |
| 0701MNDAUA | 4430000 | 4632 | 2.8 | 0.76 | 4698 |
|  |  |  |  |  |  |  |
| MnDOT Aged, Dense graded Milled asphalt | 0422MNDAMC | 4300000 | 4696 | 0.9 | 1.37 | 2581 |
| 0507MNDAMB | 4480000 | 5105.75 | 1 | 1.25 | 2828 |
| 0709MNDAMB | 4490000 | 4732 | 0.8 | 0.66 | 5431 |
|  |  |  |  |  |  |  |
| MnDOT New, Open graded Unmilled asphalt | 0507MNONUC | 4480000 | 5106 | 1.7 | 1.52 | 2324 |
| 0522MNONUC | 4650000 | 5131 | 1.7 | 0.93 | 3824 |
| 0701MNONUB | 4430000 | 4632 | 1.8 | 2.3 | 1518 |
|  |  |  |  |  |  |  |
| MDOT Aged, Dense graded Unmilled asphalt | 0424MIDAUC | 4230000 | 5106 | 1.1 | 0.65 | 5521 |
| 0515MIDAUB | 4790000 | 5131 | 1 | 0.99 | 3584 |
| 0701MIDAUC | 4430000 | 4632 | 1.3 | 1.17 | 3033 |
|  |  |  |  |  |  |  |
| MDOT Aged, Open graded Unmilled asphalt | 0513MIOAUC | 4710000 | 5013 | 1.8 | 1.28 | 2760 |
| 0520MIOAUC | 4620000 | 5073 | 1.9 | 0.68 | 5263 |
| 0709MIOAUA | 4490000 | 4632 | 1.8 | 1.32 | 2675 |
|  |  |  |  |  |  |  |
| MDOT New, Dense graded Unmilled asphalt | 0806PADNUC | 4630000 | 4966 | 1.5 | 1.98 | 1766 |
| 0909PADNUA | 4340000 | 4824 | 1.4 | 1.3 | 2717 |
| 0909PADNUC | 4340000 | 4824 | 1.5 | 0.63 | 5690 |

**Table 2.** Average and standard deviation of Totsky k-value for different the different interlayer types

|  |  |  |  |
| --- | --- | --- | --- |
| Interlayer Description | Interlayer Type | Average Totsky k | Standard Deviation |
| 15 oz/yd2 geotetextile fabric (Propex Reflectex) | F15 | 336.7 | 63.4 |
| 10 oz/yd2 geotetextile fabric (Propex) | F10 | 372.2 | 54.9 |
| MnDOT Aged, Dense graded Unmilled asphalt | MNDAU | 3342.3 | 1261.9 |
| MnDOT Aged, Dense graded Milled asphalt | MNDAM | 3613.4 | 1175.1 |
| MnDOT New, Open graded Unmilled asphalt | MNONU | 2555.1 | 900.8 |
| MDOT Aged, Dense graded Unmilled asphalt | MIDAU | 4046.1 | 965.9 |
| MDOT Aged, Open graded Unmilled asphalt | MIOAU | 3566.1 | 1095.2 |
| MDOT New, Dense graded Unmilled asphalt | PADNU | 3390.8 | 1533.4 |

Hypothesis testing is performed to evaluate the effects of the different interlayers and determine if there is any statistical difference between the interlayers. Tukey’s range test is utilized to compare all possible pairs of means (Montgomery, 2012). The null hypothesis is that the means of the two interlayers compared are equal, while the alternative hypothesis is that the mean of one of the two interlayers differs from the other. Table 3 presents all pair-wise comparisons between each interlayer. The difference in means is the result of the subtraction of the averages of the two compared interlayers. The 95 percent confidence intervals on the difference between interlayers are also presented. The two interlayers are statistically different at 95 percent, if the range of the confidence interval does not contain zero. As can be seen from Table 3, the means of the fabric interlayers are statistically different from each of the asphalts with the exception of the open graded asphalt from Minnesota. No statistical difference was detected between any of the asphalt interlayers or between the fabric interlayers.

Note that there does not appear to be a relationship between interlayer asphalt thickness and the inferred Totsky k-value. In addition, no relationship appears to be present between asphalt stiffness and the Totsky k-value. Based on the model and the lab data, other factors, including interlayer bond and perhaps loading/support conditions, must be considered if the inferred Totsky k-value is to be considered beyond an average across all asphalt lab beams.

**Table 3.** Pair-wise Interlayer Comparisons

|  |  |  |
| --- | --- | --- |
| Comparison | Difference of Mean Totsky coeff. Between Interlayers | 95% Confidence Interval of Difference |
| F10 - F15 | 35 | (-2762, 2833) |
| **MNDAU - F15** | **3006** | **(208, 5803)** |
| **MNDAM - F15** | **3277** | **(479, 6074)** |
| MNONU - F15 | 2218 | (-579, 5016) |
| **MIDAU- F15** | **3709** | **(912, 6507)** |
| **MIOAU - F15** | **3229** | **(432, 6027)** |
| **PADNU - F15** | **3054** | **(257, 5852)** |
| **MNDAU - F10** | **2970** | **(173, 5768)** |
| **MNDAM - F10** | **3241** | **(444, 6039)** |
| MNONU - F10 | 2183 | (-615, 4980) |
| **MIDAU- F10** | **3674** | **(876, 6471)** |
| **MIOAU - F10** | **3194** | **(396, 5991)** |
| **PADNU - F10** | **3019** | **(221, 5816)** |
| MNDAM - MNDAU | 271 | (-2526, 3069) |
| MNONU - MNDAU | -787 | (-3585, 2010) |
| MIDAU - MNDAU | 704 | (-2094, 3501) |
| MIOAU - MNDAU | 224 | (-2574, 3021) |
| PADNU - MNDAU | 49 | (-2749, 2846) |
| MNONU - MNDAM | -1058 | (-3856, 1739) |
| MIDAU - MNDAM | 433 | (-2365, 3230) |
| MIOAU - MNDAM | -47 | (-2845, 2750) |
| PADNU - MNDAM | -223 | (-3020, 2575) |
| MIDAU - MNONU | 1491 | (-1306, 4289) |
| MIOAU - MNONU | 1011 | (-1786, 3809) |
| PADNU - MNONU | 836 | (-1962, 3633) |
| MIOAU - MIDAU | -480 | (-3278, 2318) |
| PADNU - MIDAU | -655 | (-3453, 2142) |
| PADNU - MIOAU | -175 | (-2973, 2622) |

**\*Bold font indicates statistically significant comparisons.**

* + 1. **MnROAD Falling Weight Deflectometer Analysis**

To supplement the use of the laboratory beam testing in establishing the Totsky interlayer k-value, an analysis was carried out using FWD data from MnROAD UBOLs to establish the interlayer k-values for comparison and validation of the lab interlayer k relationship. MnROAD Cells 105, 205, 304, 405, 505, and 605 are UBOLs constructed with either an open graded Permeable Asphalt Stabilized Stress Relief Course (PASSRC - denoted MNONU from the laboratory testing) or a non-woven geotextile fabric. The designs of these cells are summarized in Table 4 below. The existing concrete pavement in Cell 5 was constructed in 1993 and consisted of 7.1 in of PCC placed over 3 in of Class 4 aggregate base over 27 in of Class 3 aggregate subbase over a clay subgrade (Watson and Burnham, 2010). Cell 5 had a 20-ft long by 13-ft (passing lane) or 14-ft (driving lane) wide panels and bituminous shoulders. FWD data was available for each cell except 105.

**Table 4.** UBOL MnROAD cells

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Cell | Construction  Date | Slab Size\* (Length x Width)  (ft x ft) | Dowels (in) | Overlay Concrete Thickness (in) | Interlayer Thickness (in) | Interlayer Type | Existing Concrete Thickness (in) |
| 105 | 10/8/08 | 15 x 14 | None | 4 | 1 | Permeable Asphalt (PASSRC) | 7.5 (cracked joints) |
| 205 | 10/8/08 | 15 x 14 | None | 4 | 1 | Permeable Asphalt (PASSRC) | 7.5 |
| 305 | 10/8/08 | 15 x 14 | None | 5 | 1 | Permeable Asphalt (PASSRC) | 7.5 |
| 405 | 10/8/08 | 15 x 14 | None | 5 | 1 | Permeable Asphalt (PASSRC) | 7.5 (cracked joints) |
| 505 | 8/24/11 | 6 x 7 | None | 5 | - | Fabric (15 oz) | 7.5 (cracked joints) |
| 605 | 8/24/11 | 6 x 7 | None | 5 | - | Fabric (15 oz) | 7.5 |

\*NOTE: Sizes shown for driving lane. For sections 15 x 14, passing lane is 15 x 13. For sections 6 x 7, passing lane is 6 x 6.5. This matches the width of Cell 5 driving and passing lanes.

Thermocouple data was available for Cells 205, 305, and 605 and were also used for Cells 105, 405, and 505 respectively since the overlay thickness and design are the same. The temperature profile through the PCC overlay, as well as an approximate temperature of the interlayer at the time of FWD testing, was then established for each cell and testing time. FWD testing performed in the wheelpath and adjacent to the transverse joint was used to establish the LTE to be used in the ISLAB finite element model. The slab stiffness was obtained either directly from an elastic modulus test for the existing PCC or through a correlation with strength for the overlay. The layers beneath the existing PCC are modeled as a Winkler foundation with a k-value of 250 psi/in established from backcalculation from Cell 5 FWD data.

ISLAB’s Totsky formulation was then used to model the structure for FWD testing performed at center slab to establish what interlayer Totsky k-value produces the closest deflection response. Mesh convergence was achieved by examining the deflection and overlay slab stress beneath the center slab load. Three sensors were used to define the deflection, including one directly under the load plate, and the sensors at +/- 12 in from the applied FWD load. Slabs that exhibited cracking and had a corresponding center slab drop after the cracking had initiated were excluded from this analysis in an attempt to isolate the effect of the interlayer on the resulting response. A batch of runs were then generated for Totsky interlayer k-value in increments of 100 psi/in. The FWD deflections were then matched to the Totsky k-value which produced the same deflection using linear interpolation to obtain the interlayer stiffness. The results of the Totsky k-value determination are presented Figure 3. For the cells with the PAASRC interlayer, the range of interlayer k-values is 1180 to 8770 psi/in with an average value of 3900 psi/in. For the nonwoven geotextile fabric interlayer cells, the range of interlayer k-values is 135 to 900 psi/in with an average value of 425 psi/in.

As can be seen in Figure 3, there is no apparent trend between interlayer k-value and asphalt temperature, which is consistent with the laboratory data in that there was no apparent trend between different asphalts with varying stiffness. Statistical testing was carried out to see if a statistical difference could be identified between the k-values obtained from the laboratory specimens and those found from the FWD testing at MnROAD. Student t-tests were carried out using the null hypothesis that the mean laboratory k-values are equal to the mean k-values obtained from the FWD testing. These results are summarized in Table 5 below. Additionally, it can be seen that the FWD results for both asphalt and fabric interlayers are different from one another statistically.

Figure . Interlayer Totsky k-value established from MnROAD FWD

**Table 5.** T-tests comparing FWD Totsky results

|  |  |
| --- | --- |
| Comparison between means of established Totsky values | P-value of t-test for difference in means |
| Fabric LAB vs. MnROAD Fabric FWD | 0.126 |
| MNONU LAB vs. MnROAD Asphalt FWD | 0.137 |
| MnROAD Fabric FWD vs. MnROAD Asphalt FWD | <0.001 |

From the laboratory testing, the only significant comparisons were that all asphalt interlayers, except MNONU, were significantly different from the two fabric interlayers. Additionally, no apparent relationship exists between asphalt stiffness or thickness and Totsky k-values within the different asphalt interlayers tested. The k-values determined using FWD test data are not statistically different from the lab values for the same interlayer type, while the fabric and asphalt k-values established using FWD test data are statistically different from one another.Since there is not an apparent trend between different asphalt types or with temperature, one value is recommended as an average for all asphalt interlayer types and temperatures. Averaging the results from both the laboratory and FWD investigations produces an average Totsky value of approximately 3500 psi/in. This value is recommended for use in the development of a design procedure for UBOL with an asphalt interlayer. No discernable difference was detected between different weight fabrics; however, the fabric stiffness was shown to be statistically different from the asphalt stiffness. Therefore, one value is recommended as an average for all nonwoven geotextile fabrics. The average Totsky value of the laboratory and FWD results is 425 psi/in and this value should be used in the development of a design procedure for UBOL with a nonwoven geotextile fabric interlayer.

* 1. **Modeling Parameters**

In performing the runs necessary to create a database of critical response parameters to train neural networks to predict the critical structural responses, the range of parameters for the UBOL structure had to be established. Additionally, the choice of the critical response parameter to be used as the predictor in the faulting model was made.

* + 1. **Structural Parameters**

ISLAB was chosen as the modeling software for UBOL joint faulting. A convergence analysis was conducted and showed that the element size of 6 inches is sufficient for the analysis. Example output for one of the mesh convergence checks performed is shown in Table 6. This mesh convergence analysis was carried out for a 12-ft joint spacing and a 6-in overlay on a 10-in existing concrete slab with a subgrade Winkler k-value of 150 psi/in. An 18-kip single axle load was applied at the joint. Additionally, validation checks were performed with Falling Weight Deflectometer (FWD) data from UBOLs in Michigan and Minnesota. An example validation with FWD data for two Michigan sections using interlayers tested in the lab study is shown in Table 7.

**Table 6.** Mesh convergence check in ISLAB

|  |  |  |
| --- | --- | --- |
| Mesh size (in) | Corner deflection (mils) | Maximum Interlayer compressive stress (psi) |
| 12 | 70.2 | 25.72 |
| 8 | 80.5 | 26.07 |
| 6 | 80.7 | 26.21 |
| 3 | 80.8 | 26.34 |

**Table 7.** ISLAB validation with FWD data

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
|  | US 131 Kalamazoo (MIOAU) | |  | US 131 Rockford (MIDAU) | |
| FWD Location | FWD (mils) | ISLAB (mils) |  | FWD (mils) | ISLAB (mils) |
| -12 | 4.6 | 5 |  | 3.7 | 3.9 |
| 0 | 5.1 | 5.4 |  | 3.8 | 4.1 |
| 12 | 4.4 | 4.7 |  | 3.5 | 3.7 |

Critical responses from the structural model must be established for every combination of variables considered. The structural model considers a wide range of parameters for the overlay, interlayer, and existing concrete slab. In performing the database of runs to generate critical responses, a baseline case is established and one parameter at a time is allowed to vary. In order to decrease the number of finite element runs required, some parameters within the structure are combined with one another. This can be seen in Figure 4. A list of all variables and range of values considered are included in Table 8. This design matrix results in approximately 100,000 finite element runs to be conducted. The values of the existing thickness, stiffness, and k-value are combined into a radius of relative stiffness. The radius of relative stiffness is adjusted from 20, 50, and 80 inches by leaving the stiffness of the existing concrete as 4,500,000 psi and the k-value as 100 psi/in and only adjusting the thickness. The range of existing thicknesses becomes 3.5 to 22 in. To further decrease the number of finite elements runs that need to be generated, only three different values of flexural stiffness for the PCC overlay are used. The overlay elastic modulus remains 4,000,000 psi and only the thickness of the overlay is increased. The values of flexural stiffness of the overlay are 2\*107, 3\*108, and 9\*108 lb-in. These values of flexural stiffness result in overlay thicknesses between 3.9 and 13.8 in.

**Table 8.** UBOL parameters for structural model

|  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **Parameter** |  | | | | | |  | | | |  | | | |
| **Existing slab and foundation radius of relative stiffness, ℓ (in)** | 20 | | | | | | 50 | | | | 80 | | | |
| **Interlayer Totsky k-value (psi/in)** | 425 | | | 2,000 | | | | 6,000 | | | | 10,000 | | |
| **Overlay Flexural Stiffness, D (lb-in)** | 2.00E+07 | | | | | | 3.00E+08 | | | | 9.00E+08 | | | |
| **Overlay PCC joint spacing x slab width (ft)** | 6 x 6 | | | | | | 12 x 12 | | | | 15 x 12 | | | |
|  |  | | | | | |  | | | |  | | | |
| **Overlay Temp Difference (oF)** | -30 | -20 | | -10 | | 0 | | 10 | 20 | | | 30 | | 40 |
|  |  | | | | | |  | | | |  | | | |
| **PCC Poisson’s ratio** | 0.18 | | | | | |  | | | |  | | | |
|  |  | | | | | |  | | | |  | | | |
| **Longitudinal Lane shoulder LTE (%)** | Tied PCC (90 %) | | | | | | Asphalt (0 %) | | | |  | | | |
| **Transverse Joint AGG Factor (psi)** | 100 | | 1000 | | 10000 | | | 50000 | | 100000 | | | 1000000 | |
|  |  | | | | | |  | | | |  | | | |
| **Wheel wander (in)** | 0 | | | | | | 4 | | | | 16 | | | |
| **Single axle (lb)** | 0 | | | | | | 18 | | | | 30 | | | |
| **Tandem axle (lb)** | 0 | | | | | | 36 | | | | 60 | | | |

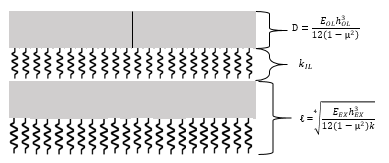


Figure . Consolidation of structural model for UBOL faulting model

The loading configuration of an axle is that shown in Figure 5. When considering tandem axles, the longitudinal spacing between tires is defined as 40 in. For each different structure, 3 slabs are modeled in the driving lane and the passing lane is not modeled. If there is a tied shoulder then there is a shoulder modeled on the edge of the pavement, but in the case of an asphalt shoulder, no shoulder is modeled.

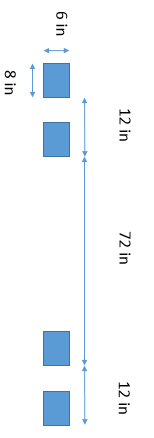


Figure . Axle configuration for structural modeling

* + 1. **Critical Response Parameters**

The critical responses from the model will be used to calculate differential energy of the interlayer, which is shown in Equation 1. Within Pavement ME, the deflections on both the loaded and unloaded sides of the joint are taken to be the deflections at the corners of the approach and leave slabs. In order to more accurately represent the difference in energy density on both sides of the joint, a basin sum deflection is used as the critical response parameter for this structural model and design procedure. All of the vertical nodal displacements within a distance of 2 ft from the joint of interest are summed on both the loaded and unloaded sides of the joint to represent the deflections used to calculate the differential energy. This can be seen in Figure 6 below.

|  |  |
| --- | --- |
|  | Equation |

Where = Differential energy, = Totsky interlayer stiffness, = loaded slab deflection, and = unloaded slab deflection.

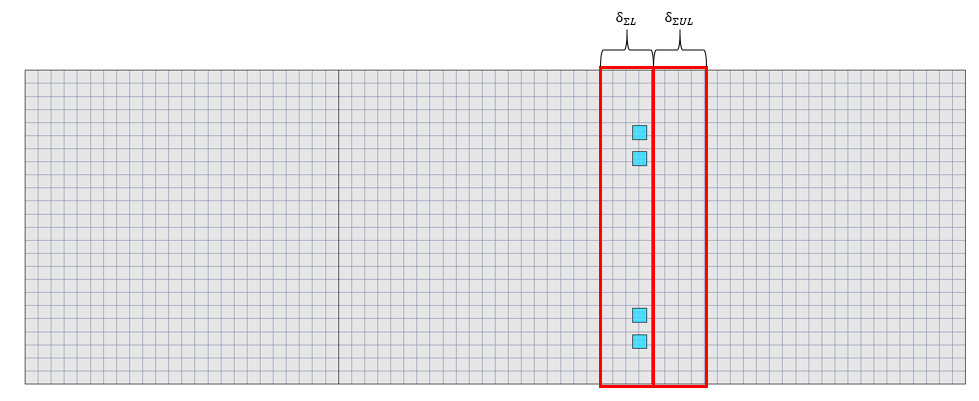


Figure . Representation of basin sum deflection on loaded and unloaded side of joint

For each combination of variables located in Table 8, an ISLAB structure is created and ran taking advantage of ISLABs batching capabilities. Then, all the nodes deflections within 2 ft of the joint on the loaded and unloaded side of the joint are calculated so they can be used to train neural networks for predicting the critical response parameters for any combination of structure, loading condition, joint stiffness, and overlay temperature difference.

* 1. **Neural Network Development**

Neural networks (NNs) are developed to predict the sum of the vertical nodal displacements within a distance of 2 ft from the joint on both the loaded and unloaded sides of the joint for the entire panel. The neural network toolbox in MATLAB is used to train, validate, and test the neural networks (MATLAB, 2013). Separate neural networks are developed for temperature loading only and for a combination of load and temperature. Due to symmetry of the temperature loading condition, only one NN is developed for both the loaded and unloaded sides of the joint. Four NNs are developed for the condition when there is a combination of load and temperature. These consist of the loaded and unloaded side for both single and tandem axles. The predictors for each of these NNs are presented along with pertinent network development information. Finally, the results of the training are presented.

Each of the NNs with each of their predictors are shown in Equation 2 through Equation 4.

|  |  |
| --- | --- |
|  | **Equation 2** |
|  | **Equation 3** |
|  | **Equation 4** |

Where = Neural Network for the sum of the 2-ft deflection basin for the loaded slab for axle type A (= 1 for single and = 2 for double), = Neural Network for the basin sum unloaded deflection for axle type A (= 1 for single and = 2 for double), = Neural Network for the basin sum deflection for the condition when only temperature is present. The predictors for the sum loaded and unloaded deflection are the same, while the NN to predict temperature load excludes the predictors related to axle loading. is the joint spacing in the overlay (ft). is the radius of relative stiffness of the overlay (in), is the radius of relative stiffness of the existing pavement (in). is the lane/shoulder LTE (%). is the nondimensional joint stiffness where = joint load transfer stiffness (psi), = Totsky interlayer k-value (psi/in), and = radius of relative stiffness for the overlay (in). is Korenev’s nondimensional temperature gradient where = coefficient of thermal expansion for the overlay concrete (in/in/oF), = is the unit weight of the overlay concrete (pci), and is the temperature difference in the overlay. is the adjusted load/pavement weight ratio where = axle load (lbs), A = parameter for axle type (= 1 for single and = 2 for tandem axles). = wheel wander (in).

The NN architecture was established through trial and error and the same structure was used for each network. For each of the NNs trained, 2 hidden layers of 20 neurons each is used. The Levenberg-Marquardt backpropagation algorithm is used to train the network, and the default split is used between the training, validation, and the test sets (70%, 15%, and 15% of samples respectively). Each NN was trained 10 times and the results are averaged over the 10 networks. Figure 7 shows the results of the training, validation, and test sets for the single axle loaded and unloaded NNs. Figure 8 shows the results of the training, validation, and test sets for the tandem axle loaded and unloaded NNs. Figure 9 shows the results of the training, validation, and test sets for the temperature loading NN.

|  |  |
| --- | --- |
|  |  |
| (a) | (b) |

Figure . Comparison of (a) and (b) for A = 1 versus ISLAB

|  |  |
| --- | --- |
|  |  |
| (a) | (b) |

Figure . Comparison of (a) and (b) for A = 2 versus ISLAB

Figure . Comparison of versus ISLAB

In addition to the testing of the neural networks using 15% of the runs from the database, an additional test of the networks was performed with independent, randomly generated, parameters which spanned the range of values defined in the matrix provided in Table 8. This was performed to test the robustness of the NNs. Approximately 3000 different points were tested for the combination of load and temperature for the single axle NNs and approximately 3000 different points were tested for the combination of load and temperature for the tandem axle NNs. About 1000 points were tested for the temperature load only NNs. The results of these predictions can be seen in Figure 10 for the single axle NNs, Figure 11 for the tandem axle NNs, and Figure 12 for the temperature load only NNs.

|  |  |
| --- | --- |
|  |  |
| (a) | (b) |

Figure . Predictability of (a) and (b) for A = 1

|  |  |
| --- | --- |
|  |  |
| (a) | (b) |

Figure . Predictability of (a) and (b) for A = 2

Figure . Predictability of (a)

1. **Joint Fauting Model Development for Unbonded Concrete Overlays of Existing Concrete Pavements**

This section details the UBOL faulting model development. First, previously developed faulting models are presented and outlined. Then, the framework that is established for UBOL joint faulting is presented, focusing on the steps which go into the monthly incremental analysis. Information regarding the calibration sections is then shown with detailed section information presented in Appendix A. Results of the initial model calibration are discussed including the calibrated model coefficients as well as a developed standard deviation model for reliability.

* 1. **Previously Developed Faulting Models**

Many of the faulting models developed under previous research were reviewed. Specific attention to the variables chosen for inclusion in the models was made. The details of each of the faulting models reviewed under this study are described separately in the following sections. The faulting models presented are only for Jointed Plain Concrete Pavements (JPCP). Six different models will be presented.

**ACPA JPCP Transverse Joint Faulting Model**

The first model which is presented is a mechanistic-empirical faulting model for doweled and undoweled pavements developed for the American Concrete Paving Association (ACPA) by Wu et al. (1993). These models were expanded from models developed for the Portland Cement Association (PCA) by Packard (1977). The percent erosion damage is established using Miner’s linear cumulative damage concept using Equation 5 (Wu et al, 1993). The allowable number of load applications is computed using Equation 6. The power of each axle pass at the corner of the slab is computed using Equation 7. The faulting for JPCP doweled and undoweled pavements can then be calculated using Equation 8 and Equation 9 respectively.

|  |  |
| --- | --- |
|  | **Equation 5** |

where = Percent erosion damage, = Expected number of axle load repetitions for each axle group I, = Allowable number of axle load repetitions for each axle group I, and = a constant which takes into account the presence of a tied shoulder.

|  |  |
| --- | --- |
|  | **Equation 6** |

where = Allowable number of axle load repetitions to end of design period, = power of each axle pass at the corner of the slab, , = modulus of subgrade reaction (psi/in), and = slab thickness (in).

|  |  |
| --- | --- |
|  | **Equation 7** |

where = power of each axle pass at the corner of the slab, = pressure at slab-foundation interface (psi).

|  |  |
| --- | --- |
|  | **Equation 8** |
|  | **Equation 9** |

where = mean transverse doweled joint faulting (in), = mean transverse undoweled joint faulting (in), = Percent erosion damage (Equation 1.5), = annual precipitation (in), = transverse joint spacing (ft), = 1 (w/ edge drains) = 0 (w/o edge drains).

**SHRP P-020 JPCP Transverse Joint Faulting Model**

Simpson et al. (1994) conducted a Strategic Highway Research Program (SHRP) project looking at early Long Term Pavement Performance (LTPP) General Pavement Study data and developed both doweled and undoweled JPCP faulting models which are presented in Equation 10 and Equation 11 respectively.

|  |  |
| --- | --- |
|  | **Equation 10** |
|  | **Equation 11** |

where = mean transverse doweled joint faulting (in), = mean transverse undoweled joint faulting (in), = cumulative 18 kip ESALs in traffic lane (millions), = transverse joint spacing (ft), = mean backcalculated static k-value (psi/in), = age since construction (yrs), = edge support (1 = tied PCC shoulder, 0 = any other shoulder type), = diameter of dowel in transverse joints (in), = mean annual precipitation (in), = mean freezing index (oF-days), = drainage type (1 = longitudinal subdrainage, 0 = otherwise).

**FHWA RPPR 1997 JPCP Transverse Joint Faulting Model**

Yu et al. (1996) developed both doweled and undoweled faulting models as part of the Federal Highway Administration (FHWA) RPPR project. These models are presented as Equation 12 and Equation 13 below.

|  |  |
| --- | --- |
|  | **Equation 12** |
|  | **Equation 13** |

where = mean transverse doweled joint faulting (in), = mean transverse undoweled joint faulting (in), = cumulative 18 kip ESALs in traffic lane (millions), = modified AASHTO drainage coefficient, = maximum dowel/concrete bearing stress (psi), = transverse joint spacing (ft), = mean freezing index (oF-days), = mean annual precipitation (in), = base type (0 = nonstabilized base, 1 = stabilized base), = widened lane (0 = not widened, 1 = widened), = age since construction (yrs), = drainage type (1 = longitudinal subdrainage, 0 = otherwise), = slab thickness (in), = mean annual number of hot days (days with max temperature greater than 90 oF).

**LTPP Data Analysis Study JPCP Transverse Joint Faulting Model**

Titus-Glover et al. (1999) recalibrated the 1997 Nationwide Pavement Cost Model (NAPCOM) model (Owusu-Antwi et al, 1997) using only LTPP data. Equation 14 is the developed model for both doweled and undoweled pavements.

|  |  |
| --- | --- |
|  | **Equation 14** |

where = mean transverse joint faulting (in), = n/N, n = cumulative 18 kip ESALs applied, N = cumulative 18 kip ESALs allowable, Log(N) = 4.27-1.6\*Log(DE), DE = differential subgrade elastic energy density, = annual average number of wet days, = diameter of dowel in transverse joints (in), = AASHTO drainage coefficient, = base type (0 = erodible base, 1 = nonerodible base).

**NCHRP 1-34 Model**

Yu et al. (1998) developed the model in Equation 15 as part of the National Cooperative Highway Research Program (NCHRP) project 1-34.

|  |  |
| --- | --- |
|  | **Equation 15** |

where = mean transverse joint faulting (in), = n/N, n = cumulative 18 kip ESALs applied, N = cumulative 18 kip ESALs allowable, Log(N) = 0.785983-0.92991\*(1+0.4\*PERM\*(1-DOWEL))\*Log(DE), PERM = base permeability (0 = not permeable, 1 = permeable), DE = differential subgrade elastic energy density, = number of days per year with the maximum temperature greater than 90oF, = annual average number of wet days, = (0 if not stabilized, 1 if stabilized), = presence of dowels (1 = present, 0 = not present), = presence of lean concrete base (1 if present, 0 if not present).

**Pavement ME Model**

The Pavement ME faulting model is a monthly incremental approach developed by ARA (2004). For each month of an analysis a faulting increment is determined which is dependent on the faulting level from the previous month. The faulting is then determined by summing up all of the previous months faulting increments. Equation 16 through Equation 19 detail the faulting models iterative process (ARA, 2004).

|  |  |
| --- | --- |
|  | **Equation 16** |
|  | **Equation 17** |
|  | **Equation 18** |
|  | **Equation 19** |

where Initial maximum mean transverse joint faulting (in), *FR* = Base freezing index defined at the percentage of the time that the top of the base is below freezing, Maximum mean monthly PCC upward slab corner deflection due to temperature curling and moisture warping, *EROD* = Base/subbase erodibility index (Integer between 1 and 5), Percent of the subgrade soil passing No. 200 sieve, *WetDays* = Average number of annual wet days (> 0.1 in of rainfall), overburden on subgrade (lb), Maximum mean transverse joint faulting for month i (in), Maximum mean transverse joint faulting for month i-1 (in) (If i =1, ), Differential energy density of subgrade accumulated during month i, Incremental monthly change in mean transverse joint faulting during month i (in), *FR* = Base freezing index defined at the percentage of the time that the top of the base is below freezing (<32oF), Mean joint faulting at the beginning of month i (in) (0 if i = 1), Calibration coefficients.

The one component of the faulting calculation which changes from month to month is the differential energy. The differential energy is computed using Equation 20. Neural networks are used to calculate the loaded and unloaded slab deflection for each axle and temperature loading condition, and then the differential energy is calculated for each axle crossing the pavement structure for each month of the analysis. This value of differential energy is then used in Equation 16 through Equation 19.

|  |  |
| --- | --- |
|  | **Equation 20** |

where differential energy density of subgrade deformation accumulated for month m, = number of axle load applications for current month and load group i, = modulus of subgrade reaction for month m, corner deflections of the loaded slab caused by axle loading, corner deflections of the unloaded slab caused by axle loading.

Of the procedures which have been presented, important predictive parameters include the following: the differential energy between the loaded and unloaded slabs, an indication of the amount of precipitation, an estimate of the traffic, the presence of dowel bars, and an indication of the erodibility of the base material. The Pavement ME faulting model is the standard mechanistic-empirical framework currently available. Therefore, the framework for the UBOL faulting model will adopt a similar approach to calculate joint faulting.

* 1. **Faulting Model Framework**

The framework to determine faulting will involve using the developed NNs to determine the differential energy. For this model, an iterative monthly incremental analysis is performed. The treatment of climatic considerations as well as calculation of joint stiffnesses is outlined. This is then followed by a discussion on the calculation of differential energy and then the functional form of the current faulting calculation.

* + 1. **Climatic Considerations**

This sections deals primarily with the treatment of temperature gradients in the overlay since it was established that there was no significant relationship between interlayer temperature and the resulting Totsky k-value. Within the current framework, a separate analysis for each structure must be carried out within the Enhanced Integrated Climatic Model (EICM) (Larson and Dempsey 2003). EICM performs an hourly incremental analysis that determines the temperature profile in the pavement structure at specified nodes. This is then used to help establish gradients for use in the design process. Therefore, for each calibration section, an EICM file is created. Within EICM, the structure must be defined including layer thicknesses, the number of nodes for each layer, thermal properties, and permeability, porosity, and water content to model moisture movement in granular layers. Within the overlay, nodes are placed at one inch increments. Additionally, the nearest weather stations to the calibration sites are chosen to give hourly values of air temperature, precipitation, wind speed, and percent sunshine for several years that can be output as a .icm profile. The program is then run to give hourly nodal temperature depths throughout the structure that is output as a .tem file by the software. This information is then used to determine the mean monthly mid-depth overlay temperature, establish hourly equivalent strain gradients, and determine the freezing ratio (FR) which is the percentage of time that the interlayer is less than 32oF. The .icm file for each EICM file is used to establish mean monthly air temperature and the number of wet days in a year

The equivalent strain gradients are calculated using the temperature-moment concept (Janssen and Snyder 2000) that converts the nonlinear temperature profile for a specific hour generated by the EICM into an equivalent linear temperature gradient (ELTG) based on Equation 21 through Equation 23. This conversion was proposed by Janssen and Snyder (2000) to ensure that the resultant strains in the overlay under the ELTG and the nonlinear temperature gradient are the same which results in the same deflections profile of the slab under the two conditions.

|  |  |
| --- | --- |
|  | **Equation 21** |
|  | **Equation 22** |
|  | **Equation 23** |

where ELTG is the equivalent linear temperature gradient (°F/in), is the average temperature (oF), is the temperature moment (°F·in2), is the depth of the *i*th node (in), and is the temperature at depth (°F).

In order to perform a monthly analysis instead of an hourly incremental analysis, it is necessary to create an effective equivalent linear temperature gradient. For each month, the differential energy is summed with the hourly ELTGs for each calibration section. Then, fminsearch in MATLAB is used to find a single temperature gradient which causes the same value of differential energy calculated using the NNs. For this analysis, 1 million ESALs (18 kip single axle loads) are applied over the course of the year, hourly distributed according to the percentages established in Pavement ME based on LTPP traffic data and presented in Table 9 (ARA, 2004). Monthly joint and overlay stiffness are also used in this analysis. The following section describes exactly how the inputs for the NNs are established.

**Table 9.** Hourly truck traffic distributions from Pavement ME (ARA, 2004)

|  |  |  |  |
| --- | --- | --- | --- |
| Time period | Distribution (percent) | Time period | Distribution (percent) |
| 12:00 a.m. - 1:00 a.m. | 2.3 | 12:00 p.m. - 1:00 p.m. | 5.9 |
| 1:00 a.m. - 2:00 a.m. | 2.3 | 1:00 p.m. - 2:00 p.m. | 5.9 |
| 2:00 a.m. - 3:00 a.m. | 2.3 | 2:00 p.m. - 3:00 p.m. | 5.9 |
| 3:00 a.m. - 4:00 a.m. | 2.3 | 3:00 p.m. - 4:00 p.m. | 5.9 |
| 4:00 a.m. - 5:00 a.m. | 2.3 | 4:00 p.m. - 5:00 p.m. | 4.6 |
| 5:00 a.m. - 6:00 a.m. | 2.3 | 5:00 p.m. - 6:00 p.m. | 4.6 |
| 6:00 a.m. - 7:00 a.m. | 5.0 | 6:00 p.m. - 7:00 p.m. | 4.6 |
| 7:00 a.m. - 8:00 a.m. | 5.0 | 7:00 p.m. - 8:00 p.m. | 4.6 |
| 8:00 a.m. - 9:00 a.m. | 5.0 | 8:00 p.m. - 9:00 p.m. | 3.1 |
| 9:00 a.m. - 10:00 a.m. | 5.0 | 9:00 p.m. - 10:00 p.m. | 3.1 |
| 10:00 a.m. – 11:00 a.m. | 5.9 | 10:00 p.m. – 11:00 p.m. | 3.1 |
| 11:00 a.m. – 12:00 p.m. | 5.9 | 11:00 p.m. – 12:00 a.m. | 3.1 |

* + 1. **Model Inputs**

With the equivalent temperature gradients defined for each calibration section, the iterative faulting calculations can then be performed. The primary calculation for each month is to determine the differential energy which can be found using Equation 24 through Equation 26. How each of the inputs to the neural network are defined is outlined next.

|  |  |
| --- | --- |
| ] | **Equation 24** |
|  | **Equation 25** |
|  | **Equation 26** |

where the variables in Equation 24 and Equation 25 are defined previously, = differential energy density deformation accumulated for month m, = number of ESAL applications for current month, = Totsky interlayer coefficient (psi/in), basin sum deflection of the loaded slab for month m (in), basin sum deflection of the unloaded slab for month m (in).

For each calibration section, four files are needed to perform the faulting calculation including input, traffic, .tem, and .icm files. The .tem and .icm EICM files have been previously discussed along with the climatic considerations. An example input and traffic text file are shown in Table 10. Twenty one different inputs are specified for each section as can be seen in Table 10a. The traffic file has four columns which from left to right are the overall month in the analysis, the calendar month of the year, the year, and the ESALs which were observed for that year.

|  |  |
| --- | --- |
|  |  |
| (a) | (b) |

**Table 10.** Examples of (a) an input text file and (b) a traffic text file

Looking at the inputs to the NNs, the joint spacing and the radius of relative stiffness of the overlay and existing pavements can be easily calculated from the input file. Note that a default value of 0.18 is assumed for the Poisson’s ratio of concrete. Additionally, is binary depending on whether there is a tied concrete shoulder (40%) or an asphalt shoulder (0%). The 40% LTE for a tied concrete shoulder matches the long term LTE for a shoulder joint used in Pavement ME (ARA, 2004). The normalized load-pavement weight ratio, . is taken to be 18,000 lbs and is 150 lbs/ft3 for all calibration sections. The wheel wander, s, is a normally distributed in the wheelpath with a standard deviation of 10 in. Korenev’s nondimensional temperature gradient, , according to the equation in the NN Development section. All variables in this equation have been discussed previous with the exception of the temperature difference, . In this procedure, the temperature difference is calculated as the equivalent temperature difference for differential energy plus the default value of the effective built-in temperature difference from Pavement ME of -10 oF (ARA, 2004). The final NN input is . This variable is also referred to as the nondimensional joint stiffness. In order to calculate the nondimensional joint stiffness, the contribution of both aggregate interlock and dowels must be considered.

In order to examine the effects of aggregate interlock on joint stiffness, the joint width must be estimated. The joint width for each month is calculated according to Equation 27. The two variables that still need to be determined to calculate the joint width are the PCC set temperature and the PCC overlay shrinkage strain. The concrete set temperature is estimated using Table 11, which requires the mean monthly temperature for the month of cast as well as the cement content. The concrete overlay shrinkage strain is established from tensile strength (correlated from compressive strength) using the recommendations in AASHTO 93. This recommendation is shown in Table 12. The nondimensional aggregate joint stiffness can then be calculated for each month using Equation 28 and Equation 29 adopted from Zollinger et al. (1998). Note that is equal to zero for the first month of the analysis and the individual monthly increments of loss in shear capacity can be calculated using Equation 30.

|  |  |
| --- | --- |
|  | **Equation 27** |

where = joint width for month m (mils), = friction factor (0.65 for asphalt interlayers, 1.74 for fabric interlayers), = joint spacing in the overlay (ft), = overlay PCC coefficient of thermal expansion (in/in/oF), = concrete set temperature (oF), = mean mid-depth PCC overlay temperature for month m (oF), = PCC overlay shrinkage strain (in/in).

**Table 11.** PCC set temperature for cement content and mean temperature during month of cast (oF)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | Cement Content (lbs) | | | |
| Mean Monthly Air Temp (oF) | 400 | 500 | 600 | 700 |
| 40 | 52 | 56 | 59 | 62 |
| 50 | 66 | 70 | 74 | 78 |
| 60 | 79 | 84 | 88 | 93 |
| 70 | 91 | 97 | 102 | 107 |
| 80 | 103 | 109 | 115 | 121 |
| 90 | 115 | 121 | 127 | 134 |
| 100 | 126 | 132 | 139 | 145 |

**Table 12.** PCC overlay shrinkage strain relationship

|  |  |
| --- | --- |
| Tensile Strength (psi) | Shrinkage Strain (in/in) |
| 400 | 0.0008 |
| 500 | 0.0006 |
| 600 | 0.00045 |
| 700 | 0.0003 |
| 800 | 0.0002 |

|  |  |
| --- | --- |
|  | **Equation 28** |
|  | **Equation 29** |

where = aggregate joint shear capacity, = joint opening (mils), = cumulative loss of shear capacity at the beginning of the current month, = nondimensional aggregate joint stiffness for current monthly increment, = 0.35, = 0.38.

|  |  |
| --- | --- |
|  | **Equation 30** |

where = loss of shear capacity from all ESALs for current month i, = overlay slab thickness (in), = joint opening (mils), = shear stress on the transverse joint surface from the response model, = reference shear stress derived from the PCA test results.

For a doweled pavement, the model adopted for the nondimensional dowel stiffness is that from ARA (2004). The initial nondimensional dowel joint stiffness is calculated using Equation 31 and the critical nondinemsional dowel joint stiffness is calculated with Equation 32. The nondimensional dowel stiffness is then calculated using Equation 33 and the dowel damage parameter is presented Equation 34.

|  |  |
| --- | --- |
|  | **Equation 31** |
|  | **Equation 32** |
| *+(- )* | **Equation 33** |
| *DOWDAM =* | **Equation 34** |

where = area of dowel bar (in2), = overlay PCC thickness (in), = Initial nondimensional dowel stiffness, = critical nondimensional dowel stiffness, = nondimensional dowel stiffness for current month, = cumulative dowel damage for the current month, = dowel bar spacing (in), = dowel bar diameter (in), = PCC compressive stress estimated from the modulus of rupture.

With the differential energy calculated, the faulting can then be predicted using Equation 35 through Equation 38.

|  |  |
| --- | --- |
|  | **Equation 35** |
|  | **Equation 36** |
|  | **Equation 37** |
|  | **Equation 38** |

initial maximum mean transverse joint faulting (in), *FR* = base freezing index defined at the percentage of the time that the top of the base is below freezing (<32oF), maximum mean monthly PCC upward slab corner deflection due to temperature curling and moisture warping, *E =* erosion potential of interlayer: f(% binder content, % air voids, ), Percent of interlayer aggregate passing No. 200 sieve, *WETDAYS* = Average number of annual wet days (> 0.1 in of rainfall), maximum mean transverse joint faulting for month i (in), maximum mean transverse joint faulting for month i-1 (in)(If i =1, ), Differential energy density accumulated during month i, incremental monthly change in mean transverse joint faulting during month i (in), Calibration coefficients, mean joint faulting at the beginning of month i (0 if i = 1), mean joint faulting at the end of month i (in).

* 1. **Calibration Sections**

The calibration database used to calibrate the UBOL faulting model consists of 34 different sections from 9 different states in the United States and 1 province in Canada. The calibration sections are comprised of 14 Long Term Pavement Performance (LTPP) sections, 6 sections from the Minnesota Road Research Facility (MnROAD), and 14 Michigan Department of Transportation (MDOT) pavement sections. Table 13 presents a range for some calibration section parameters. Of the sections, 16 are undoweled while the rest are doweled. The dowel diameter for the doweled sections ranged from 1 - 1.5 in. If the pavement section has a random joint spacing, the mean joint spacing was used in the analysis. Considering the number of time series observations available, a total of 163 data points are available for calibration of the model.

The age of the sections ranged from approximately 2.5 to 33.5 years with an average of 13.5 years of age. In terms of ESALs, the traffic ranged from approximately 0.85 million to 22.4 million with an average value of around 7 million ESALs. Over half of the sections had experienced over 6 million ESALs, while 15% of the sections had experienced over 10 million ESALs. Only one undoweled section was exposed to more than 10 million ESALs. Detailed information for each calibration section can be found in Appendix A.

**Table 13.** Range of parameters for calibration sections

|  |  |  |  |
| --- | --- | --- | --- |
| Parameter | Minimum | Maximum | Average |
| Age, yrs | 2.5 | 33.5 | 13.5 |
| Estmated ESALs | 8.56E+05 | 2.45E+07 | 7.79E+06 |
| Avg Jt Spacing, ft | 6 | 27 | 14.6 |
| Interlayer Thickness, in | 0.1 | 8.6 | 1.5 |
| Overlay thickness, in | 4.5 | 10.3 | 7.1 |
| Overlay EMOD, psi | 3.09E+06 | 4.85E+06 | 3.93E+06 |
| Overlay MOR, psi | 530 | 1022 | 684 |
| Existing thickness, in | 7.1 | 10.2 | 8.5 |
| Existing EMOD, psi | 3.50E+06 | 5.00E+06 | 4.46E+06 |
| Overlay Cement Content, lbs | 354 | 594.5 | 540.7 |

* 1. **Results of Model Calibration**

Calibration of the faulting model requires adjusting the calibration coefficients from Equation 35 through Equation 38 to minimize the error function defined by Equation 39. Additionally, the shape of the erosion function had to be fit based upon the interlayer characteristics chosen to be important to faulting. The fit erosion model can be seen in Equation 40 and Equation 41. A macro driven excel spreadsheet was developed to calibrate the model and the following steps were taken to minimize the error. Several calibration parameters were fixed at a constant value while the remaining coefficients were varied to find the lowest values of the error function. Once the error is minimized for the varied coefficients, these values are kept constant while the coefficients that were previously held constant are allowed to vary until the lowest possible value of the error function is achieved. These two sets of coefficients are varied in this manner until the error can be minimized no further. These steps do not guarantee a global minimum error but should provide a reasonable result. Minimization of the bias in the model with the calibration parameters must also be performed in addition to error minimization when selecting the final set of calibration coefficients. Predicted versus measured transverse joint faulting is presented in Figure 13. Table 14 summarizes all of the calibration coefficients that have been chosen.

|  |  |
| --- | --- |
|  | **Equation 39** |

WhereERROR = error function, calibration coefficients, predicted faulting for ith observation in dataset, measured faulting for ith observation in dataset, N = number of observations in the dataset.

|  |  |
| --- | --- |
|  | **Equation 40** |
|  | **Equation 41** |

where = erodibility index, = calibration coefficients, = binder content of the interlayer (%), = air voids percentage of interlayer, = percent aggregate passing No. 200 sieve in interlayer, = model erosion to be used in predictive equations.

Figure . Measured vs. predicted UBOL transverse joint faulting

**Table 14.** UBOL transverse joint faulting calibration coefficients

|  |  |
| --- | --- |
| Calibration Coefficient |  |
| C1 | 8.3 |
| C2 | 0.9 |
| C3 | 2.3 |
| C4 | 0.001 |
| C5 | 0.17 |
| C6 | 4 |
| C7 | 4.4 |
| C8 | 0.0000036 |

**JPCP Transverse Joint Faulting Model Adequacy Checks**

A series of model adequacy checks were performed to ensure the developed model coefficients provided reasonable values in terms of predictability and reasonableness. The tests outlined by Mallela et al. (2009) have been performed and are summarized below. For the model, an overall SEE of 0.019 in of faulting and a coefficient of determination, R2, of 0.84 was deemed reasonable in comparison to values obtained from Pavement ME JPCP transverse joint faulting model calibration efforts (Sachs et al, 2014). The model bias was checked by using the three hypothesis tests outlined in Table 15. The null and alternative hypothesis outlined in Table 15 were tested and the results summarized in Table 16. A significance level of 0.05 was assumed for hypothesis testing. From Table 16, none of the three null hypotheses are rejected indicating that model bias has been removed through the calibration.

**Table 15.** Null and Alternative hypothesis tested for JPCP faulting

|  |  |
| --- | --- |
| Hypothesis 1 | Null hypothesis Ho: Linear regression model intercept = 0 |
| Alternative hypothesis Ha: Linear regression model intercept ≠ 0 |
| Hypothesis 2 | Null hypothesis Ho: Linear regression model slope = 1.0 |
| Alternative hypothesis Ha: Linear regression model slope ≠ 1.0 |
| Hypothesis 3 | Null hypothesis Ho: Mean ME Design faulting = Mean LTPP measured faulting |
| Alternative hypothesis Ha: Mean ME Design faulting ≠ Mean LTPP measured faulting |

**Table 16.** Results from transverse joint faulting model hypothesis testing

|  |  |  |  |
| --- | --- | --- | --- |
| Hypothesis Testing and t-Test | | | |
| Test Type | Value | 95% CI | P-value |
| Hypothesis 1: Intercept = 0 | 0.00001 | -0.00194 to 0.00196 | 0.968 |
| Hypothesis 2: Slope = 1 | 0.989 | 0.952 to 1.026 | 0.564 |
| Paired t-test | - | - | 0.801 |

**JPCP Transverse Joint Faulting Model Reliability**

The JPCP transverse joint faulting model reliability (standard deviation) was determined in a similar way as was conducted for Pavement ME (ARA, 2004). The resulting standard deviation model developed from UBOL faulting for a design at a specified level of reliability is presented below as Equation 42 and Figure 14 using the data from Table 17, which was determined from the predicted faulting data.

|  |  |
| --- | --- |
|  | **Equation 42** |

Where Stdev(FLT) = transverse joint faulting standard deviation (in), FLT = UBOL model predicted transverse joint faulting (in).

Figure . Predicted faulting versus faulting standard deviation

**Table 17.** Predicted faulting data used to develop faulting standard deviation model

|  |  |
| --- | --- |
| Mean Predicted Joint Faulting, in | Std. Dev. Of Predicted Joint Faulting, in |
| 0.0064 | 0.0063 |
| 0.0189 | 0.0123 |
| 0.0716 | 0.0142 |
| 0.1408 | 0.0247 |

Plots have also been generated showing the faulting versus traffic for three of the calibration sections. Figure 15 shows MnROAD Cells 305 and 405 which consist of the same design. The design is a 5-inch-thick undoweled overlay with an asphalt shoulder, 15 ft joint spacing, and a 1 in PASSRC (MNONU interlayer). Figure 16 shows LTPP section 89\_9018 in Quebec, Canada. The structure is a 6 in undoweled overlay with an asphalt shoulder, 15 ft joint spacing, and a chip seal interlayer Figure 17 is LTPP section 06-9107 in California. The structure is a 9 in undoweled overlay with an asphalt shoulder, 12 ft joint spacing, and a 1 in dense graded interlayer.

Figure . MnROAD Cells 305 and 405 predicted and measured joint faulting

Figure . LTPP section 89\_9018 predicted and measured joint faulting

Figure . LTPP section 6\_9107 predicted and measured joint faulting

1. **Summary And Recommendations For Faulting Model**

This Task report details the development of Neural Networks to predict the critical responses for UBOL joint faulting using MATLAB’s Neural Network Toolbox. Many previous faulting models were then examined looking at key predictive variables and frameworks used to determine faulting for JPCP pavements. The framework for the model to predict faulting for UBOL was then presented. This includes how climatic factors are treated, primarily the temperature gradient for the overlay. Then a discussion of how differential energy is calculated along with all the steps to establish the inputs for the NNs. Finally, the incremental faulting equations are then presented. With the framework presented, a discussion of the data available to calibrate the faulting model is made that includes the location of pavement sections and relevant design features. The initial model calibration was then presented.

A number of improvements need to be made to the model before it can be implemented into a design procedure to UBOL. First, the model needs to be expanded to calibration with shorter panel data (6 by 6 ft slabs). This application is becoming increasingly popular for UBOL. The NNs are able to predict short slab differential energy, however, the only data in the calibration database which has short panels are MnROAD Cells 505 and 605 with nonwoven geotextile fabric. The model must be calibrated with data for short panels with asphalt interlayers to be used for this application. Currently, there is insufficient UBOL data on short slabs to add to the calibration database. Therefore, short slab data for Bonded Concrete Overlays of Asphalt Pavements (BCOA) can be used for calibration if the joints have not propagated through the asphalt.

Next, the model should account for a wide range of axle types and loads instead of only accounting for ESALs. Accounting for different load spectra would provide a more realistic representation of the deflections which are occurring as compared to ESALs. A similar approach to that used in Pavement ME could be used where national average values for load spectra are used for the purposes of calibration. Additionally, the developed Erosion model must also be refined.

Finally, in order to improve the implementation of the design procedure, a framework must be established to determine the effective equivalent temperature gradient without the use of EICM. This would require developing effective equivalent temperature gradients for a wide range of climatic conditions and structures to be able to predict the gradient as a function of structural and climatic features. The design framework must then be transferred from MATLAB into the final design tool.

# Short Panel Cracking Model Development

The critical element in M-E (mechanistic-empirical) design for cracking is stress analysis. The MEPDG considers stresses generated for many cases, including different vehicle loads at different locations on a pavement subjected to different temperature and moisture gradients. The MEPDG accounts for changes in the properties of the pavement structure over time. For example, seasonally adjusted values for subgrade moduli are considered on a monthly basis throughout pavement life. Similarly, load transfer efficiency of joints in jointed plain concrete pavements (JPCP) and the bond condition between the concrete and underlying layer changes as the pavement ages. Accounting for these factors means that hundreds of thousands of individual stress analyses for each design case must be considered in the analysis of a single pavement (NCHRP 2004). The MEPDG is based on stress analyses conducted in the finite element modeling program ISLAB2000 (Khazanovich et al. 2000), which is a widely accepted and used rigid pavement modeling software.

Conducting a suite of finite element analyses for each of these cases for a single pavement design is highly computationally intensive, if not prohibitive due to the large number of runs required for completeness. However, by running a large finite element factorial in advance and using the factorial’s results to train neural networks to predict the pavements stresses, computation of the many stresses for each pavement analyzed in the MEPDG becomes possible (Khazanovich et al. 2001). Using neural networks eliminates the need to embed a time and effort consuming finite element analysis within the pavement design program.

A finite element factorial requires a staggeringly large number of cases to cover all the cases needed to train a neural network for possible design scenarios users will encounter. The MEPDG uses the equivalent structure concept (also known as the similarity concept) to reduce the size of the factorial without introducing any error. Below this concept is reconsidered and adapted for an unbonded overlay system with short panels.

The equivalent structure concept permits the computation of stresses in a multi-layer system (a concrete slab with a base on a subgrade) from those in a similar system. This concept has been used in the MEPDG, (NCHRP 2006), in both the JPCP, (NCHRP 2003) and continuously reinforced concrete pavement (CRCP) (Khazanovich et al. 2001) cracking models. The two systems can be considered as equivalent as long as their deflection basins are scalable, meaning that:

|  |  |
| --- | --- |
|  | **Equation 43** |

where w is deflections, a and b are coordinate scaling factors, x and y define the horizontal coordinate system, λdef is the scaling factor for deflections which is dependent only on properties of the pavement structure, and subscripts 1 and 2 denote pavement systems 1 and 2, respectively.

If system 2 is subjected to axle loading and a linear temperature strain causing temperature distribution throughout the slabs thickness is acting, then if Equation 43 is satisfied, the stresses in system 2 can be found from those in system 1 using the following relationship:

|  |  |
| --- | --- |
|  | **Equation 44** |

where σtotal is the total stress at the surface of the slab at the z distance from the neutral plane, σlinear is the bending stress due to traffic and thermal loading at the surface of the slab independent of coordinates, λstress is the scaling factor for stress which is dependent only on properties of the pavement structures and the distances from the neutral planes, and σnon-linear is the non-linear component of stress due to thermal loading only at the surface of the slab independent of the in-plane coordinates (Ioannides and Khazanovich 1998).

The pavement systems should satisfy several conditions to be similar. Several practically important cases of similar systems are presented below.

**Equivalent Thickness Concept**

Consider a two-layered single slab structure consisting of a concrete slab resting on a base layer on a Winkler foundation, where the Poisson’s ratio of the concrete and the base layer are assumed to be equal. As shown by Ioannides et al (1992), it is possible to find an equivalent single layer system consisting of only a concrete slab on a Winkler foundation. In the absence of a thermal load, the two systems are similar when the following conditions are satisfied:

* Same length and width
* Same Winkler foundation with modulus of subgrade reaction *k*
* Modulus of elasticity of the concrete layers in the two-layered system is equal to the modulus of elasticity of the slab in the single layer system
* Same axle load footprint and magnitude.

Following Ioannides et al (1992), it can be shown that the single layer system is similar to the two-layer system with the unbounded interface between the layers if the following condition is satisfied:

|  |  |
| --- | --- |
|  | **Equation 45** |

where *heff* is the thickness of the single layer system (effective thickness), *hPCC* is the concrete layer thickness, *EPCC*and *EBase* are elastic moduli of the PCC and base layers, respectively, and *hBase* is the base thickness.

When computing deflections and stresses at the top or bottom surfaces of an original system from the stresses at the corresponding locations at the top or bottom surfaces of a similar system using Equations 43 and 44, the coordinate scaling factors should be set to 1, i.e., *a = b = 1*. The deflection and stress scaling factors are given as:

|  |  |
| --- | --- |
|  | **Equation 46** |

**Equivalent Temperature Gradient**

If two systems are subjected to variations in pavement temperature through the pavement thickness and share the criteria listed below, then the two systems will have the same deflection profiles if the temperature distributions satisfy the following conditions (Khazanovich 1994; Ioannides and Khazanovich 1998):

* Same plane view geometry
* Same flexural stiffness
* Same self-weight
* Same boundary conditions
* Same foundations (i.e. same modulus of subgrade reaction)

|  |  |
| --- | --- |
|  | **Equation 47** |

where T is the temperature distribution through the thickness of the slab, and *T0* is the temperature at which the slab is assumed to be flat, *α* is the coefficient of thermal expansion, *z* is the distance from the neutral axis, and *1* and *2* denote the two slab systems.

Let one of the systems be a two-layered system with an unbonded interface subject to a linear temperature distribution through the concrete thickness and constant temperature through the base layer. Consider another single-layered system with layer properties the same as those for the top layer in the two-layered system with the thickness defined as the equivalent thickness in Equation 44. The unit weight of the single-layered system, γ*eff*, is defined in Equation 48 and should ensure the same weight of both systems.

|  |  |
| --- | --- |
|  | **Equation 48** |

where γ*PCC*and γ*base* are effective PCC and base unit weights, respectively.

The linear equivalent temperature differential for the single layer system can be described as (Ioannides and Khazanovich 1998):

|  |  |
| --- | --- |
|  | **Equation 49** |

where ΔTeff is the difference in the top and bottom temperatures for the equivalent single layer slab, *Ttop* and *Tbot* are the temperatures at the top and bottom surfaces of the concrete (top) layer in the two-layered system, respectively.

When computing deflections and stresses in a two-layered system from those in a single-layer system using Equations 43 and 44, a = b = 1, and scaling factors, given as:

|  |  |
| --- | --- |
|  | **Equation 50** |

If the stresses are computed at the top or bottom surfaces of the PCC layer, then the nonlinear stress component, σnon-linear, should be computed as follows:

|  |  |
| --- | --- |
|  | **Equation 51** |

**Equivalent Structure**

While the equivalent thickness concept can be used only for axle loads and the equivalent temperature gradient concept can be used only when thermal loads are applied to the slab, the equivalent slab concept is needed when both traffic and environmental loads must be considered. The equivalent slab concept was originally developed for analysis of circular slabs (Korenev and Chernigovskaya 1962). The equivalent structure concept extended this principle to rectangular, multi-slab systems (Khazanovich et al. 2001). Consider two multi-slab pavement systems subjected to axle and temperature loading. Each system contains the same number of rectangular slabs in the transverse and longitudinal directions. Each slab in the system has the same length, L, and width, *W*. If there is more than one slab in any given direction, then the joint stiffnesses characterized by effective aggregate interlock stiffness, *AGG*, are assumed to be the same for all joints in the system in the same direction. These two systems are similar if the following conditions are satisfied:

|  |  |
| --- | --- |
| ϕ1 = ϕ2  , | **Equation 52** |

where is radius of relative stiffness, ϕ is Korenev’s nondimensional temperature gradient, and *p* is axle pressure distribution (equal to zero when outside of the tire footprint area). For single-layer slabs, the radius of relative stiffness is defined as

|  |  |
| --- | --- |
|  | **Equation 53** |

where *h* is the slab thickness.

For a two-layered slab system with an unbonded interface, the radius of relative stiffness can be computed using Equation 53 by replacing the thickness with the effective slab thickness (defined in Equation 45) and the elastic modulus of the top layer. Korenev’s non-dimensional temperature gradient for a linear temperature distribution in a single layered slab is defined as follows (Korenev and Chernigovskaya 1962):

|  |  |
| --- | --- |
|  | **Equation 54** |

When two slabs have the same length and width, then the first two conditions in Equation 52 can be reduced to ℓ1 = ℓ2 and the axle loading should have the same footprints and locations. Axle weights should be related as follows:

|  |  |
| --- | --- |
|  | **Equation 55** |

where P represents total axle load on the pavement.

If System *1* is a two-layered pavement and System *2* is a similar single-layered pavement, then the stresses and deflections are related using Equations 43 and 44, where the factors *a = b= 1*. The required scaling factors are:

|  |  |
| --- | --- |
|  | **Equation 56** |

where the effective slab thickness and unit weight, and respectively, of the two-layered system is defined by Equations 45 and 48.

To account for the effect of separation of the PCC slab from the base layer, the MEPDG recommends neglecting the self-weight of the base layer. In this case, and Equation 56 can be re-written as follows (Khazanovich et al. 2001):

|  |  |
| --- | --- |
|  | **Equation 57** |

**Neural Networks Development**

As discussed above, to enable the MEPDG-type incremental damage analysis of the short slab unbonded overlay systems, the rapid solutions capable to calculate critical stresses for this type of pavement must be developed. Those solutions should match the ISLAB2000 calculated stresses for the structural model described earlier for the entire range of many parameters, such as concrete slab and base thickness, modulus of elasticity, unit weight and coefficient of thermal expansion, coefficient of subgrade reaction, and a wide combination of axle weights and temperature distributions. That made the development of neural networks a challenging problem. The similarity principles described above were utilized to reduce the dimension of the problem and the number of finite element runs required for the neural networks training. Figure 18 shows the structural model for the short slab unbonded overlay with a void under the overlay slab.

The following ISLAB model parameters will be kept constant for all cases:

* Four slabs in the longitudinal direction
* Three slabs in the transverse direction
* Element size:
  + Transverse direction
    - 6 inches for slab 1 (shoulder) and slab 3
    - 2 inches for slab 2
  + Longitudinal direction
    - 6 inches for slabs 1 and 4
    - 2 inches for slabs 2 and 3
* Single layer system
  + Slab thickness: 5 in
  + Slab Poisson’s ratio: 0.15
  + Slab unit weight: 10 lb/in3
* Longitudinal joint LTE (between unbonded overlay slabs): 50%
* Subgrade support
* Subgrade k-value: 100 psi/in
  + No void or slab-wide
  + 12-in long void with k-value = 0 psi/in (see Figure 1).

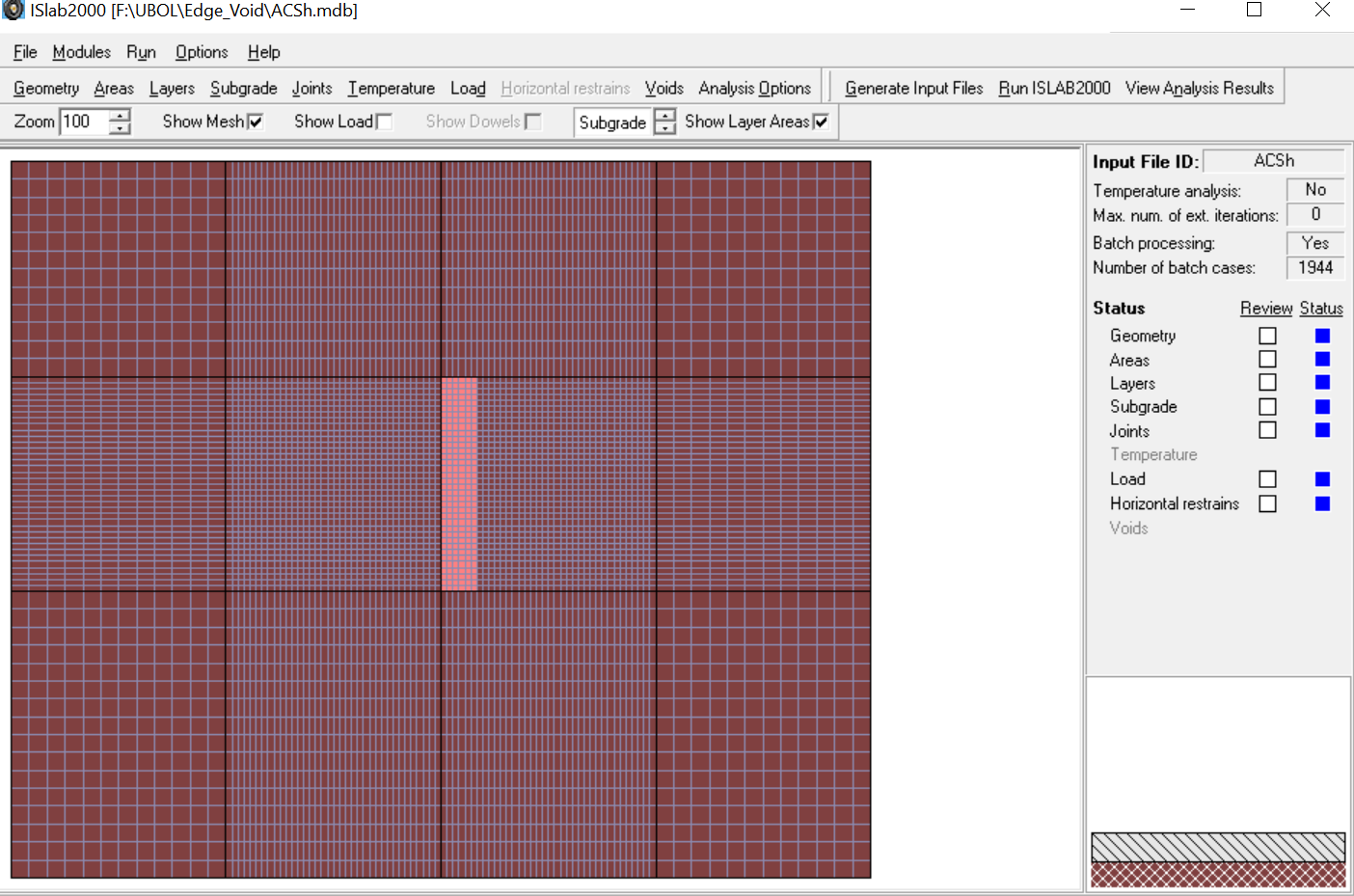


Figure 18. ISLAB structural model for short slab unbonded overlay with a void under the overlay slab

To further reduce the NN factorial, a simplifying assumption of linear superposition of thermal and axial load induced stresses was made. This assumption is justified by the following observations:

* Since the size of the panels is relatively small, curling deformations are not as severe as would be predicted for conventional length overlay slabs
* If an interlayer is not eroded it tends to be in full contact with the overlay
* Even when the linear superposition assumption is violated, it tends to yield to a slightly conservative stress. However, the difference between the “exact” and approximate stresses is not very significant.

Thus, three independent NNs have been designed as follows: one for single axle loading stresses, one for tandem axle loading stresses, and one for curling stresses. An example of a single wheel load on the structural model is shown in Figure 19. The following loading characteristics were assumed in this study which further decreased the dimensionality of the problem:

* Tire width: 8 in
* Tire length: 6 in
* Tire pressure: 100 psi
* Reference point: bottom left
* For tandem loading, the axle spacing was assumed to be equal to 51 in



Figure . Single wheel loading

The remaining parameters are summarized in Table 18 below.

Table . Parameters varied in factorial of ISLAB runs for axle loading

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Case | Slab size, ft | Elastic Modulus, psi | Transverse Joint LTE, % | Lane/Shoulder  LTE, % | Axle Reference Point  Transverse Offset, in\* |
| 1 | 6 x 6 | 200,000 | 35 | 35 | +0 |
| 2 | 7 x 7 | 350,000 | 45 | 50 | +4 |
| 3 | 8 x 8 | 600,000 | 55 |  | +6 |
| 4 | 3 | 1,000,000 | 65 |  | +8 |
| 5 |  | 1,500,000 | 75 |  | +10 |
| 6 |  | 2,200,000 | 85 |  | +12 |
| 7 |  | 3,100,000 | 95 |  | +14 |
| 8 |  | 4,300,000 |  |  | +16 |
| 9 |  | 5,800,000 |  |  | +18 |
| 10 |  | 7,600,000 |  |  | +20 |
| 11 |  | 10,000,000 |  |  | +22 |
| 12 |  | 12,500,000 |  |  | +24 |
| 13 |  | 15,750,000 |  |  | +28 |
| 14 |  | 21,700,000 |  |  | +32 |
| 15 |  | 28,200,000 |  |  | +36 |
| 16 |  | 38,500,000 |  |  | +40 |
| 17 |  | 50,000,000 |  |  | +44 |
| 18 |  | 63,500,000 |  |  | +48 |
| 19 |  | 80,000,000 |  |  | +56 |
| 20 |  | 100,000,000 |  |  | +60 |
| 21 |  | 122,000,000 |  |  | +64 |
| 22 |  | 168,000,000 |  |  |  |
| 23 |  | 225,000,000 |  |  |  |
| 24 |  | 297,000,000 |  |  |  |
| 25 |  | 385,000,000 |  |  |  |

\*measured from the lane/shoulder joint.

Temperature Curling Analysis

The following ISLAB model parameters will be kept the same for all cases:

* Single slab
* Element size: 2 x 2 inches
* Single layer system
  + Slab thickness: 5 in
  + Slab Poisson’s ratio: 0.15
  + Slab unit weight: 10 lb/in3
  + Subgrade k-value: 100 psi/in
* Linear temperature, temperature difference = 10 oF
* The remaining parameters are summarized in Table 19 below.

**Table 19.** Parameters varied in factorial of ISLAB runs for temperature curling

|  |  |  |
| --- | --- | --- |
| Case | Slab size, ft | Elastic Modulus, psi |
|
| 1 | 5 x 5 | 100000 |
| 2 | 6 x 6 | 200000 |
| 3 | 7 x 7 | 350000 |
| 4 | 8 x 8 | 600000 |
| 5 |  | 1000000 |
| 6 |  | 1500000 |
| 7 |  | 2200000 |
| 8 |  | 3100000 |
| 9 |  | 4300000 |
| 10 |  | 5800000 |
| 11 |  | 7600000 |
| 12 |  | 10000000 |
| 13 |  | 12500000 |
| 14 |  | 15750000 |
| 15 |  | 21700000 |
| 16 |  | 28200000 |
| 17 |  | 38500000 |
| 18 |  | 50000000 |
| 19 |  | 63500000 |
| 20 |  | 80000000 |
| 21 |  | 100000000 |
| 22 |  | 122000000 |
| 23 |  | 168000000 |
| 24 |  | 225000000 |
| 25 |  | 297000000 |
| 26 |  | 385000000 |

A critical tensile bending stress occurs at the bottom of the slab under the wheel load which increases when there is a high positive temperature gradient through the slab (the top of the slab is warmer than the bottom of the slab). Repeated loadings of heavy axles under those conditions result in fatigue damage along the bottom transverse joint of the slab. Due to lateral wander of the truck axle load (assumed to be normally distributed), different points along the transverse joint would accumulate fatigue damage at different rates. The software computes multiple points and selects the point of maximum fatigue damage for computation of longitudinal cracking. This point of maximum fatigue damage is where a longitudinal crack will initiate and then propagate to the surface of the slab and then extend along the slab.

The critical tensile bending stress will be at a transverse joint in the wheel path area at the bottom of the slab. Fatigue damage is accumulated at 2 inch points along the transverse joint. The position of the highest damage depends on the truck lateral wheel wander and thermal stresses and may be located between the mean wheelpath point and the midpoint of the slab. The longitudinal cracking program selects the location with maximum damage to predict longitudinal fatigue cracking.

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# Appendix A: Calibration Database Information

For each calibration section, detailed information is presented in the following tables which is required for the faulting model calculation.

Table A.1. Calibration sections project information

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Source | SHRP\_ID or ID | Const. Date | Survey Date | Age, yrs | Est ESALs | Longitude, deg | Latitude, deg |
| LTPP | 6\_9048 | 9-Oct-70 | 24-Mar-04 | 33.48 | 2.41E+07 | 85.55 | 40.59 |
| LTPP | 6\_9049 | 1-Jun-68 | 13-Nov-01 | 33.47 | 6.00E+06 | 95.71 | 39.09 |
| LTPP | 6\_9107 | 1-Oct-88 | 13-Jun-02 | 13.71 | 8.45E+06 | 95.04 | 44.84 |
| LTPP | 8\_9019 | 1-Feb-86 | 14-Aug-98 | 12.54 | 6.15E+06 | 90.7 | 32.36 |
| LTPP | 8\_9020 | 1-Oct-86 | 24-Aug-98 | 11.90 | 7.69E+06 | 78.41 | 41.04 |
| LTPP | 18\_9020 | 1-Jan-87 | 29-Apr-04 | 17.34 | 2.45E+07 | 96.37 | 31.9 |
| LTPP | 20\_9037 | 1-Jan-78 | 12-May-94 | 16.37 | 8.56E+05 | 96.83 | 32.48 |
| LTPP | 27\_9075 | 1-Jan-77 | 1-Jun-95 | 18.42 | 5.89E+05 | 116.7 | 32.84 |
| LTPP | 28\_7012 | 1-Jul-85 | 7-Feb-12 | 26.62 | 1.71E+07 | 121.56 | 38.58 |
| LTPP | 42\_1627 | 1-Sep-88 | 12-Nov-02 | 14.21 | 1.79E+07 | 120.55 | 39.31 |
| LTPP | 48\_9167 | 15-Jun-88 | 29-Oct-12 | 24.39 | 1.50E+07 | 104.98 | 40.22 |
| LTPP | 48\_9355 | 1-Mar-90 | 25-Mar-12 | 22.08 | 2.24E+07 | 104.99 | 40.39 |
| LTPP | 89\_9018 | 1-Aug-87 | 21-Jul-05 | 17.98 | 1.99E+06 | 72.48 | 46.32 |
| MnROAD | Cell105 | 30-Oct-08 | 14-Apr-11 | 2.45 | 2.45E+06 | 93.65 | 45.24 |
| MnROAD | Cell205 | 30-Oct-08 | 14-Apr-11 | 2.45 | 2.45E+06 | 93.65 | 45.24 |
| MnROAD | Cell305 | 30-Oct-08 | 14-Apr-15 | 6.46 | 6.46E+06 | 93.65 | 45.24 |
| MnROAD | Cell405 | 30-Oct-08 | 14-Apr-15 | 6.46 | 6.46E+06 | 93.65 | 45.24 |
| MnROAD | Cell505 | 12-Sep-11 | 16-Apr-15 | 3.59 | 3.59E+06 | 93.65 | 45.24 |
| MnROAD | Cell605 | 12-Sep-11 | 16-Apr-15 | 3.59 | 3.59E+06 | 93.65 | 45.24 |
| MDOT | 03033 | 2009 | 2015 | 6.00 | 3.30E+06 |  |  |
| MDOT | 03111 | 2004 | 2015 | 11.00 | 7.77E+06 |  |  |
| MDOT | 09101 | 1990 | 2013 | 23.00 | 5.81E+06 |  |  |
| MDOT | 16091 | 2008 | 2015 | 7.00 | 1.76E+06 |  |  |
| MDOT | 19022 | 1991 | 2015 | 24.00 | 1.54E+07 |  |  |
| MDOT | 25032 | 2004 | 2015 | 11.00 | 3.59E+06 |  |  |
| MDOT | 39014 | 2004 | 2015 | 11.00 | 7.77E+06 |  |  |
| MDOT | 41026 | 2007 | 2015 | 8.00 | 4.59E+06 |  |  |
| MDOT | 41132 | 2000 | 2014 | 14.00 | 7.69E+06 |  |  |
| MDOT | 47014 | 2001 | 2011 | 10.00 | 8.79E+06 |  |  |
| MDOT | 56044 | 2010 | 2014 | 4.00 | 9.92E+05 |  |  |
| MDOT | 65041 | 2003 | 2015 | 12.00 | 2.62E+06 |  |  |
| MDOT | 70063 | 2004 | 2015 | 11.00 | 6.72E+06 |  |  |
| MDOT | 71111 | 2006 | 2011 | 5.00 | 2.70E+06 |  |  |

Table A.2. Calibration sections design features

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| SHRP\_ID or ID | Lane Width, ft | Lane Width, ft | Tied PCC Shoulder | Avg Jt Spacing, ft | Dowel Diameter, in | Dowel Spacing, in | Drainage Type |
| 6\_9048 | 12 | 12 | No, AC | 15.5 | None | None | None |
| 6\_9049 | 12 | 12 | No, AC | 15.5 | None | None | None |
| 6\_9107 | 12 | 12 | No, AC | 13.5 | None | None | Long Edgedrain |
| 8\_9019 | 12 | 12 | No, PCC | 13 | None | None | None |
| 8\_9020 | 12 | 12 | Yes | 20 | None | None | None |
| 18\_9020 | 12 | 12 | No, AC | 15.5 | None | None | x-drain |
| 20\_9037 | 12 | 12 | No, AC | 15 | 0.5 | 30 | None |
| 27\_9075 | 12 | 12 | No, AC | 15.5 | None | None | None |
| 28\_7012 | 12 | 12 | No, AC | 21 | 1 | 12 | None |
| 42\_1627 | 12 | 12 | Yes | 20.5 | 1.25 | 12 | Long Edgedrain |
| 48\_9167 | 12 | 12 | Yes | 20 | 1.5 | 12 | Long Edgedrain |
| 48\_9355 | 12 | 12 | No | 15 | 1.25 | 12 | None |
| 89\_9018 | 12 | 12 | Yes | 15 | 1.25 | 12 | None |
| Cell105 | 14 | 14 | No, AC | 15 | None | None | Wick Drains |
| Cell205 | 14 | 14 | No, AC | 15 | None | None | Wick Drains |
| Cell305 | 14 | 14 | No, AC | 15 | None | None | Wick Drains |
| Cell405 | 14 | 14 | No, AC | 15 | None | None | Wick Drains |
| Cell505 | 6.5 | 6.5 | No, AC | 6 | None | None | Wick Drains |
| Cell605 | 6.5 | 6.5 | No, AC | 6 | None | None | Wick Drains |
| 03033 | 12 | 12 | Yes | 12 | 1.25 | 12 | yes |
| 03111 | 12 | 12 | Yes | 13 | 1.25 | 12 | none |
| 09101 | 12 | 12 | Yes | 14 | 1.25 | 12 | Varies |
| 16091 | 12 | 12 | Yes | 12 | 1.25 | 12 | yes |
| 19022 | 12 | 12 | Yes | 27 | 1.25 | 12 | PDS at EOP |
| 25032 | 12 | 12 | Yes | 14 | 1.25 | 12 | none |
| 39014 | 12 | 12 | Yes | 12 | 1.25 | 12 | none |
| 41026 | 12 | 12 | Yes | 14 | 1.25 | 12 | 18" PDS at EOP |
| 41132 | 12 | 12 | Yes | 13 | 1.25 | 12 | 18" PDS at EOP |
| 47014 | 12 | 12 | Yes | 13 | 1.25 | 12 | 18" PDS at EOP |
| 56044 | 12 | 12 | Yes | 12 | 1.25 | 12 | 18" PDS at EOP |
| 65041 | 12 | 12 | Yes | 11 | 1.25 | 12 | none |
| 70063 | 12 | 12 | Yes | 14 | 1.25 | 12 | 18" PDS at EOP |
| 71111 | 12 | 12 | Yes | 14 | 1.25 | 12 | 6" open graded underdrain |

Table A.3. Calibration sections structural details

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| SHRP\_ID or ID | Overlay thickness, in | Overlay EMOD, psi | Overlay MOR, psi | Existing thickness, in | Existing EMOD, psi | Overlay CTE, in/in/degF | Overlay Cement Content, lbs |
| 6\_9048 | 6.4 | 3.81E+06 | 808 | 8.1 | 3.90E+06 | 5.50E-06 | 564 |
| 6\_9049 | 7.5 | 3.51E+06 | 829 | 7.7 | 4.80E+06 | 5.50E-06 | 470 |
| 6\_9107 | 8.8 | 3.09E+06 | 530 | 7.6 | 4.75E+06 | 5.50E-06 | 594.5 |
| 8\_9019 | 9 | 3.37E+06 | 572 | 7.9 | 3.50E+06 | 5.50E-06 | 565 |
| 8\_9020 | 8 | 3.44E+06 | 541 | 7.7 | 3.68E+06 | 5.50E-06 | 565 |
| 18\_9020 | 10.2 | 4.05E+06 | 641 | 10.2 | 4.23E+06 | 5.50E-06 | 558 |
| 20\_9037 | 5.8 | 3.31E+06 | 962 | 8.8 | 4.88E+06 | 5.50E-06 | 540 |
| 27\_9075 | 5.9 | 4.25E+06 | 714 | 7.8 | 3.70E+06 | 5.50E-06 | 555 |
| 28\_7012 | 10 | 4.23E+06 | 1022 | 9.4 | 5.00E+06 | 5.50E-06 | 549 |
| 42\_1627 | 10.3 | 3.31E+06 | 696 | 9.7 | 4.25E+06 | 5.50E-06 | 541 |
| 48\_9167 | 10.2 | 4.33E+06 | 858 | 8.4 | 4.85E+06 | 5.50E-06 | 414 |
| 48\_9355 | 10.3 | 4.85E+06 | 877 | 9.9 | 4.98E+06 | 5.50E-06 | 354 |
| 89\_9018 | 6.4 | 4.23E+06 | 810 | 8.9 | 3.80E+06 | 5.50E-06 | 573 |
| Cell105 | 4.5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| Cell205 | 4.5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| Cell305 | 5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| Cell405 | 5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| Cell505 | 5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| Cell605 | 5 | 4.00E+06 | 660 | 7.1 | 4.63E+06 | 5.50E-06 | 550 |
| 03033 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 03111 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 09101 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 16091 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 19022 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 25032 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 39014 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 41026 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 41132 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 47014 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 56044 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 65041 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 70063 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |
| 71111 | 7 | 4.00E+06 | 625 | 9 | 4.50E+06 | 5.50E-06 | 550 |