Calibration and Validation of EverFE2.24: A Finite Element Analysis Program for Jointed Plain Concrete Pavements

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This thesis titled
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ABSTRACT

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Calibration and Validation of EverFE2.24: A Finite Element Analysis Program for Jointed Plain Concrete Pavements (99 pp.)

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This work studies the response of jointed plain concrete pavement to traffic loading as well as temperature variations, using the three-dimensional FE program EverFE2.24. The traffic loading is modeled using ODOT single axle dump truck rolling on top of the pavement, while the environmental loading is modeled using temperature measurements by thermocouples inserted throughout the depth of the slab. Picking a reference point in the temperature data, the change in stresses with respect to time has been computed and compared with the field data. Performing a fatigue analysis, the temperature is found to be considerably more detrimental to the concrete pavement compared to axle loads. Measurement of the surface temperature, especially in the top, is evaluated to be critical in convergence of EverFE2.24 model. The 3D FE program EverFE2.24 results followed the trend of the measured data from the field.

Approved: _____________________________________________________________

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This work is dedicated to my fiancée, Shogofa Hakimi
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CHAPTER 1: INTRODUCTION

1.1 Background

Determination of response of a pavement is a key step in the mechanistic-empirical design of pavements. Based on the mechanistic determination of the pavement response, the empirical design of the pavement can be pursued in terms of strength and serviceability. The more exact the pavement response is determined the more appropriate the pavement can be designed. This response in the pavement is developed due to application of traffic loads and environmental loads. The analysis and prediction of the pavement response to traffic loading has been done before with a good precision (Sargand & Abdalla, 2006). Yet, the analysis and prediction of the rigid pavement response to environmental loads has been difficult and dependent on many variables such as air temperature, humidity, cloud cover, rain, and other environmental factors. In general, the following loads create detrimental response in a concrete pavement: curing of concrete, temperature differential, moisture, and traffic loads. The effect of these loads could be magnified when together with critical slab geometry. For instance, shorter slabs will increase the probability of occurrence of joint loading, and hence, tensile stresses at top.

1.1.1 Curing of Concrete

Generally two types of curing exist: curing at ambient temperatures and curing at elevated temperatures. In either type of curing, the amount of moisture existing in or added to the concrete is important in determining the strength of concrete. Theoretically, a water-cement ratio of at least 0.42 will be enough for a complete hydration of concrete.
On the other hand, the water better be continuously added to the concrete. When the curing process is interrupted, the top surface starts drying while the concrete has not yet developed its strength against tensile stresses. This causes development of premature top-down cracking in the slab due to contraction of the top surface and, hence, progression of tensile stresses.

In addition, the external relative humidity and the curing temperature are also important. In case of low external relative humidity (relatively dry ambient) the concrete consumes its internal water, thus the internal relative humidity declines. This eventually ceases the hydration process, and the concrete fails to develop its full expected strength (Mindess et al., 2003). Hence, external relative humidity is crucial during the concrete early life. A higher temperature during later times of curing causes nonuniform distribution of concrete strength; the weak sections resulted thereby govern the strength of the aged concrete (Mindess et al., 2003). Thus, the curing temperature is, also, important in the development of the concrete strength.

Due to loss of moisture, which is loss of mass and, thus, loss of volume, concrete shrinks as it cures. Usually the top surface of the slab shrinks prior to the bottom since it is exposed to drying factors such as the wind and the sun. Hence, the bottom portion tends to restrain the shrinkage of the top portion. As a result, tensile stresses develop in the top surface of the slab while compressive stresses develop in the bottom of the slab. In addition, casting of concrete in hot temperatures causes a positive temperature gradient (top warmer than bottom) although the slab is flat at this point. Later on, in order for the temperature gradient in the slab to become zero, a negative temperature gradient develops
in the slab to counteract the initial positive temperature gradient (Hansen, Smiley, Peng, & Jensen, 2002). The development of this negative temperature gradient curves the slab upward, causing tension in the top and compression in the bottom.

1.1.2 Temperature Gradient

The so-called temperature gradient describes the direction and rate of the rapidest change of temperature, and is expressed in terms of temperature change per length. In a concrete slab this direction is along the depth since the change of temperature in the two other directions, width and length of the slab, is much slower. This temperature gradient changes with respect to time as a result of changes in the air temperature. The temperature rapidly changes between day and night although it has fluctuations through different seasons of the year. The high frequency of temperature cycling between days and nights can impose a considerable fatigue loading on the pavement.

During the late night and early morning, when the air temperature is relatively low, negative temperature gradients develop in the slab, that is, the top is cooler than the bottom. In this case, the higher temperature in the bottom of the slab forces it to expand more than the top portion of the slab. Due to continuity of concrete, the bottom forces the top to expand as well, and tensile stresses develop at the top portion of the slab. Thus, negative temperature gradient induces tensile stresses at the top and compressive stresses at the bottom of the concrete slab.

During the afternoon, when the air temperature is relatively high, the top surface is warmer than the bottom surface of the slab. The top surface expands and, due to concrete continuity, forces the bottom to expand as well. Because of resistance of the
bottom portion, tensile stresses develop at the bottom, and compressive stresses occur at
top of the concrete slab. Yet, since the overall temperature is relatively high, the entire
concrete slab expands, resulting in shorter joint openings—perhaps, closing the joints. In
this situation, each concrete slab is compressed by its ambient expanding concrete slabs.
An analogy of this situation can be prestressed concrete in which the prestressing steel,
when released, counteracts the dead and live loads. Here the compression applied by the
ambient slabs tends to counteract the effect of positive temperature gradient.

In addition, temperature measurement by thermocouples show that temperature
gradient in concrete pavement is not linear. This nonlinearity of temperature dramatically
affects the thermal stresses in a concrete slab. This point will be shown in the results and
will be discussed later.

1.1.3 Moisture Gradient

Moisture gradient is a function of the ambient moisture existed in the field, free
water in the concrete slab, and water content of the soil beneath the pavement (Huang,
2004). Since the top surface of concrete is exposed to drying factors such as the wind and
the sun, the moisture content increases with the depth of the slab. In this case, the top
portion of the slab experiences more shrinkage compared to the bottom portion. The
continuity of concrete requires the highly-shrinking top portion to shorten compatibly
with the less-shrunken bottom of the slab. Therefore, the top portion is restrained from
shrinking, which undergoes tensile stresses. The bottom portion of the slab undergoes
compression because it is pushed by the top portion to shrink.
1.1.4 Traffic Loading

Traffic loading, when applied at the ends of the slab, induces tensile stresses at the top and compressive stresses at the bottom of the slab. Yet, in general, the truck dynamic loads induce higher tensile stresses at the bottom of the slab (Sargand & Abdalla, 2006). A study by Shoukry, Fahmy, Prucz, and William (2007) shows that the highest stresses can be developed by single axle dump trucks. Furthermore, since both the traffic and the environmental loading are repetitive, the frequency of edge axle load repetition, together with that of negative temperature gradients, has a direct influence on the fatigue life due to tensile stresses in the concrete slab.

1.1.5 Slab Geometry

The geometry of the concrete slab is also a factor that affects the concrete pavement response. The width of the concrete slab is usually picked as 12 ft (3.66 m) in design of rigid pavements while the pavements in SHRP (Strategic Highway Research Program) SPS-2 (Specific Pavement Studies) have a widened width of 14 ft. The slab length also affects the amount of tensile or compressive stresses being developed in the slab. AASHTO (American Association of State Highway and Transportation Officials) recommends a ratio of width to the length of slab no more than 1.25 (as cited in Huang, 2004, p. 176). They also recommend that the length of the slab in feet should not be greater than two times the thickness of the slab in inches (as cited in Huang, 2004, p. 176). Finally, the slab thickness affects the development of tensile stresses in the slab. The thicker the slab is the higher the temperature differential can develop through it. Yet,
the thicker the slab is the less pronounced the effect of traffic loads on the development of stresses will be.

1.2 Research Objectives

Two and three-dimensional finite element programs have been developed to study the rigid pavement behavior to traffic and environmental loads. As comes in chapter two, the general tendency is toward 3D FE programs, which are capable of modeling different aspects of the behavior of rigid pavements. The three-dimensional finite element program EverFE2.24 is to be calibrated and validated in terms of prediction of the concrete pavement response to dynamic and thermal loads. The objectives of this study are here:

- To investigate the effect of temperature changes on the curling stresses of the concrete slab. In other words, the effect of temperature changes between morning and evening, during a day, is to be investigated in this study.
- To investigate the rigid pavement response to nonlinear temperature gradient
- To validate the ability of the 3D Finite Element program EverFE in modeling the rigid pavement under temperature loading.
- To validate the ability of the 3D Finite Element program EverFE in modeling the rigid pavement under traffic loading.
1.3 Outline

Chapter 2 reviews the literature on theoretical, two-dimensional and three-dimensional analysis of rigid pavements.

Chapter 3 describes the geometry of the problem, traffic loading, and temperature data input to the program. A description of the verification of the FE program, EverFE, with analytical solution is also presented in this chapter.

Chapter 4 presents the result of the analysis together with the discussion of the results. Results of the dynamic loading and temperature effects are discussed in terms of the matching between FEM and measured data, and are interpreted. Results of a fatigue analysis of the problem are also discussed in this chapter.

Chapter 5 presents conclusions of the study.
CHAPTER 2: LITERATURE REVIEW

The studies on investigation and modeling of rigid pavements under traffic and environmental loading can be classified into two categories: three and two-dimensional finite element analysis (FEA) and theoretical or closed form solution. Literature reviewed in each category comes next.

2.1 Closed Form Solution

Development of a closed form solution for curling analysis started with the work of Westergaard in early 20th century (Ioannides, Davis, and Weber 1999). As the fast computers emerged, the closed form solution benefited from the numerical solutions, resulting in a combined numerical-analytical solution.

Mohamed and Hansen (1996) divided the analysis of stresses into two stages to derive an equation for curling stresses. First they computed the residual stresses which are caused by internal restraints imposed due to continuity requirements. The authors discussed that residual stresses exist in a completely free slab subjected to nonlinear temperature gradient. At this stage, they derived a relation for equivalent linear temperature gradient (ELTG). In the second stage, they computed the stresses caused by external restraints such as slab self weight (exerted by the earth) and subgrade reaction, using the ELTG found in the first stage. The authors assumed the slab to be elastic, homogeneous, isotropic, and temperature independent. They further assumed that plane sections remain plane, and neglected the stresses and strains in the vertical direction. In comparison between linear and nonlinear temperature gradient, they concluded that linear
analysis calculates tension only at one face while nonlinear analysis predicts tensile stresses at both faces of the concrete slab.

Nishizawa, Fukuda, Matsuno, and Himeno (1996), in an endeavor to derive a curling stress equation for transverse joint edges, considered three load transfer types: shear, bending, and torsion springs. Out of these, shear is the dominant load transfer mode in joints while bending occurs over small cracks, and torsion is negligible. They stated that longitudinal stresses are large in the interior of the slab, and transverse stresses are large in the middle of transverse joints. They, applying the multi-regression analysis to the results of FEA, observed that curling stresses are not proportional to temperature difference, in shorter slabs. This happens because higher temperature difference reduces the area of contact between slab and base, which reduces the friction between them, and hence, reduces the stresses. They observed that the stresses due to combined effect of temperature and traffic loading match with the superposition of the stresses obtained separately due to each.

Ioannides et al. (1999), critiquing the literature in the last 75 years, reviewed the analytical solution, FE analysis, statistical regression analysis, and artificial neural network (ANN) of the rigid pavements. They evaluated the assumptions made by Westergaard in deriving an equation for curling stresses. They reported that the FEM formulation ILSL2 nullifies all the assumptions except the one regarding the absence of a rigid subgrade. The authors evaluated ANN to be more efficient than statistical analysis. They concluded that principle of superposition holds for stresses due to load and those
due to the environment. Also, the Westergaard curl-only stresses should not be compared to the field data.

Some common conclusions in the analytical studies reviewed above would be that the results from traffic loading and temperature variations follow superposition. Also, temperature gradient has a nonlinear trend, and use of computer codes to solve the problem, in other words numerical solutions, is appreciated.

### 2.2 FEM Analysis

Two and three-dimensional FE programs have been proven to be powerful in analyzing the behavior of rigid pavements under traffic and environmental loading. A short review of the related literature comes next. Different features of the two and three-dimensional FE analysis in these studies are summarized in Table 2-1.

**Table 2-1 Modeling Features of Rigid Pavements Using 2D and 3D FE Analyses**

<table>
<thead>
<tr>
<th>Study</th>
<th>Slab</th>
<th>Base/ Subgrade</th>
<th>Dowel and tie</th>
<th>Interface</th>
<th>Program/language</th>
<th>Temperature loading</th>
<th>Verification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bhatti et al. (1998)</td>
<td>Nine-node quadrilateral</td>
<td>Winkler foundation</td>
<td>Beam element with 2 end springs</td>
<td>N/A</td>
<td>2DFE</td>
<td>N/A</td>
<td>Limited data</td>
</tr>
<tr>
<td>Heath et al. (2003)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>ISLAB2000</td>
<td>Nonlinear TG</td>
<td>ISLAB2000</td>
<td>Observed cracking pattern in Palmdale</td>
</tr>
<tr>
<td>Kuo et al. (1995)</td>
<td>27-node 3D element</td>
<td>Liquid foundation with membrane on top</td>
<td>Beam element</td>
<td>INTER9</td>
<td>ABAQUS</td>
<td>Linear TG</td>
<td>AASHO, PCA, and Arlington</td>
</tr>
<tr>
<td>Al-Nasra (1996)</td>
<td>8-node brick</td>
<td>Compressional spring</td>
<td>N/A</td>
<td>N/A</td>
<td>FORTRAN</td>
<td>Equ. linear TG</td>
<td>N/A</td>
</tr>
</tbody>
</table>
2.2.1 Two-dimensional FEM

Bhatti, Molinas-Vega, and Stoner (1998) assumed the concrete in compression as elastoplastic taking the yield and crush into account, and in tension as elastic for the regions below tensile strength of concrete, \( f_t \). The elastic constants vary after the formation of cracks (the modulus of elasticity and Poisson’s ratio, normal to the cracks, are assumed zero). The voids that are caused by pumping decrease the subgrade support;
however, temperature changes or permanent deformation of subgrade cause some voids prior to pumping. The shear deformation of dowel bars, which can be affected by load cycling, creates some additional deformation. They considered the modulus of subgrade reaction as a function of deformation to simulate its resilience. The model in their study was not validated versus sufficient measured data due to unavailability of field data.

Heath, Roesler, and Harvey (2003) utilized ISLAB2000 to model cracks in rigid pavements. The program accounted for quadrilinear temperature and bilinear shrinkage distributions. They treated the shrinkage as linear in top half and zero in the bottom half of the slab. The authors, assuming cracks start where maximum stresses exist, observed the followings. High shrinkage in longer slabs leads to environmental induced cracking. Shrinkage, even with no temperature gradient, leads to maximum tensile stresses at the top. They reported that longitudinal as well as corner and transverse cracks develop at the top. The tensile stresses at both top and bottom faces of the slab can be predicted in case of nonlinear temperature gradient and shrinkage. They concluded that drying shrinkage is one of the most important factors in shaping the response of a concrete slab. Especially when a high shrinkage gradient is existed, only transverse cracking prediction will not be enough for an acceptable study; a thorough analysis is required.

The studies using two-dimensional FEM also state that the temperature gradient is nonlinear. The nonlinear temperature gradient causes tensile stresses at both faces of the slab. As summarized by Kuo et al. (1995), the two-dimensional FEM analysis suffers the limitations of plate theory, which motivates toward three-dimensional FEM.
2.2.2 Three-dimensional FEM

Kuo, Hall, and Darter (1995) listed that the two-dimensional FE modeling has limitations in simulation of features such as loss of contact due to curling, nonlinear and/or unequal temperature and moisture gradient between top and bottom of the slab, unequal dimensions of slab versus base, interface friction and bond, thicker pavement layers, and compressibility. Utilizing a 3D FEM, called 3DPAVE, the authors observed that the modulus of subgrade reaction significantly affects the stresses, and expressed it in terms of speed of the truck. They predicted that, for both thin and thick slabs, cracks—depending on the application of the load at the critical location—start at the bottom of the slab. In the FE analysis, they used the k value found for the subgrade, while in Westergaard, they utilized the k value obtained for the base. For verifying the curling stresses with data from Arlington road test, they assumed linear temperature gradients. The paper concluded that 2D FE resulted in higher stresses while the 3DPAVE gave closer results to the measured ones.

Al-Nasra (1996), considering the problem of slab on grade under combined effect of temperature and moisture gradient, studied the effect of modulus of elasticity of concrete, modulus of subgrade reaction, joint stiffness, and contact surface node coupling on curling of concrete slab. After performing the FE analysis, he concluded the followings. The warping (curling) increases with modulus of elasticity of the concrete. It, also, increases with the decrease of the modulus of subgrade reaction, due to sinking of the slab into the softer subgrade. Stiffer joints restrain the warping and, therefore, increase the slab stresses. Finally, node coupling of the contact surface does not
considerably affect the warping of the slab. Nevertheless, comparison with the field data could further validate the analysis.

Pane, Hansen, and Mohamed (1998) developed a three-dimensional FE program to validate the analytical solution by Mohamed and Hansen (1996), to verify the assumption of plane sections remain plane, and to study the loss of contact between the slab and the base. Comparing the results of linear and nonlinear temperature distribution, they observed that the model with nonlinear temperature distribution is more sensitive to the mesh refinement throughout the slab thickness. For simulating residual stresses (eliminating the external restraints) they assigned stiffness, close to zero, to the springs. These residual stresses from FE matched well with those by Mohamed and Hansen (1996). The authors, studying two severe cases of subgrade support and slab thickness, assigned compression springs (for the case of slab self weight) and compression-tension springs (in the case of loss of support) to the subgrade. They, stating that several factors such as pumping, plastic deformation of subgrade, moisture warping, and temperature gradient lead to loss of support, predicted no LOS by their 3DFE model. The authors, also concluded, that the assumption of plane sections remain plane (PSRP) holds for nearly the entire slab, except the unrestrained edges. The analytical and FE analyses usually match; however, verification of the analyses versus field measured data could further verify the results of their study.

Mahboub, Liu, and Allen (2004), developing a 3D FE program on ANSYS to evaluate temperature loading, used a two-step solution: temperature inputs and then, thermal stresses as input with gravity and static loads. The modulus of subgrade reaction
and other material properties, in different directions, were adjusted in order to match the field data. Only one composite base-subgrade layer was modeled beneath the slab. They assumed that the slab, initially, has some residual stresses, which are created by distributed body forces at the nodes. The magnitude of these distributed body forces was modified to match the field data. They concluded that temperature changes have a higher impact on the trend of slab stresses compared to traffic loading. They recommended that the current fatigue models should be modified to incorporate—and more focus on—temperature effects. The introduction of body forces to create residual stresses and modification of them in order to match the data seem questionable.

Siddique, Hossain, and Meggers, (2006), in a study of curling on new concrete pavements, observed that the range of temperature variations is bigger in the top compared to the bottom. Temperature gradient at the top portion of the slab is more abrupt for positive TG compared to negative TG. They observed that a positive temperature gradient results in larger curling and larger combined stresses compared to a negative temperature gradient with the same magnitude. They assumed that the concrete as well as the base and subgrade are linear elastic. Also, they found that an edge loading with a positive temperature gradient as well as a corner loading with a negative temperature gradient is critical. They reported that the deflection due to linear TG is smaller than that due to nonlinear TG. The difference between FE results and the measured data increase with an increase of temperature gradient. One of the reasons of this difference, they explained, is that deflection measurements are taken with respect to
the first point of measurement. The FE temperature input, on the other hand, is the instantaneous temperature of the pavement.

Shoukry, William, and Riad (2007) set the parameters of the 3DFE model based on the pavement sections on Goshen Road. They attempted to eliminate the effect of the environmental loads other than temperature. To perform this, they considered the temperature variations over a period of six hours, and divided the temperature variations into two categories: temperature differential and uniform temperature change with time. Considering a period of 400 days, they established a relationship between change in the mean slab temperature, and the temperature differential. Using the relationship, they calculated the temperature changes, and inter them into the FE program to compute the stresses in the pavement. They observed that the dowel bending moment follows the slab curling. Bent dowels introduce restraints against contraction of slab due to the uniform temperature drop, and hence increase the slab stresses. In slab length, they observed that with the decrease of slab length the longitudinal stresses decrease while the transverse stresses increase. Hence, they recommended a length of 4.57 m for JPCP. Thickness, the authors concluded, does not affect thermal stresses. In a related manner, although larger dowels reduce curling stresses due to higher stiffness against bending, they increase the dowel-slab contact stresses, and thus were not recommended.

Two points worth discussing here. First, the authors found that FE stresses computed for top matched better than bottom because of the influence of built-in curling on the contact area between slab and base. The built-in curling curves the slab upward, and hence, affects the curling stresses both in top as well as bottom. On the other hand,
since the change of stresses is modeled in their work rather than the absolute stresses (as in the present study), the effect of built-in curling as well as moisture gradient and differential shrinkage are excluded. Hence, the predicted stresses should follow the measured stresses at the top as well as the bottom, similarly. Secondly, they stated that temperature differential is proportional to the slab thickness. Huang (2004, p 152) contradicts this.

Shoukry et al. (2007) stated that the nonlinear temperature-induced stresses do not follow superposition. Equal and opposite nonlinear temperature gradients do not induce equal and opposite stresses. They observed that the combined traffic and thermal maximum principal stresses are greater in the midslab compared to the joint. On the other hand, in applying different types of axle loads, they stated that the stresses decrease with the increase of number of axles. They predicted that larger temperature drops in conjunction with curing effects and negative temperature gradients can cause top-down cracking. The authors concluded that the stresses due to temperature gradient together with traffic loadings do not reach the modulus of rupture of the concrete; yet, the stresses due to contact between dowel and concrete as well as those due to uniform temperature changes can lead the slab to premature failure.

The three-dimensional FE analyses reviewed above studied different aspects of the thermal induced stresses in concrete slabs. The emphasis was done on the longitudinal stresses that cause the prevailing mode of failure, i.e. transverse cracking. The nonlinear temperature gradient, again, was emphasized. The effect of temperature variations was not examined considerably. In some cases of such analyses (Shoukry et al., 2007) the
temperature differential (linear TG) was calculated using a relationship with temperature changes and was input to the 3DFE program. Therefore, need is felt to investigation of the effects of actual nonlinear temperature gradient together with the temperature changes in the course of time, and verification of the results with full scale field data.
CHAPTER 3: METHODOLOGY

3.1 Introduction

Two main stages were executed in this study. First, the model was calibrated using the traffic loading data collected from one section on Delaware 23. At this stage longitudinal strains and longitudinal stresses were compared. Second, using the calibrated model the longitudinal stresses due to temperature effects were predicted and compared with the field data from the section on Delaware 23 and a section on I-86. The pavement sections used in the study are shown next: section 390269 is shown in Figure 3-1 and Figure 3-2.
The abbreviations shown in Figure 3-1 and Figure 3-2 show the strain indicating sensors: K for KMA and KMB strain transducers, and V for Vishay strain gages. For instance, K1/2, in Figure 3-1, represents two sensors K1 and K2 located at the same X and Y coordinates, but different depths. In the same manner, K3/V1, in Figure 3-2, shows the sensors K3 and V1 which are located at the same X and Z coordinates but different points along the width of the slabs. Locations of these sensors are further detailed in Table 3-4.

The rubblized section on I-86, which is modeled for environmental loading, is shown in Figure 3-3.
The strain indicating sensors are positioned in two slabs, each of length 15.58 feet or 4.75 meters. Each of the locations shows two sensors at different depths: 1 inch above the bottom and an inch below the top surface of the slabs. The locations are further detailed in Table 3-5. The sensors designated by f, g, and m are located on wheel paths and the sensors represented by the letters e, n, and o are positioned at the middle of the slabs.

The geometries used in the program for Del-23 and I-86 are shown in Figure 3-4 and Figure 3-5 respectively.
3.2 Brief Description of FE Program EverFE2.24

The three-dimensional finite element program EverFE was developed by University of Maine and University of Washington with the financial support of Departments of Transportation of Washington and California.
This program uses a Finite Element code written in C++ language. For modeling concrete and soil a 20-node brick element has been utilized. Subgrade, as dense liquid foundation (subgrade reaction at each point is proportional to deflection at that point and independent of other points (Huang, 2004)), is modeled using an 8-node planar quadratic element. Aggregate interlock and slab-base interface are modeled using a 16-node quadratic element. In the mean time, 3-node beam elements are used to model ties and dowels (Davids, 2003). The program allows the user to input the dimensions and material properties. The traffic loading is introduced by real axles and wheels attached to the axles. Up to 4 points of temperature changes (trilinear temperature profile) can be introduced to the program. Features such as contact of dowels and ties with concrete, aggregate interlock (by means of joint opening, and joint stiffness), contact and loss of contact between slab and base are accounted for in the program. A list of inputs required by the program is shown in Table 3-1.

Table 3-1 Important Input Features of FE Model Used in the Analysis

<table>
<thead>
<tr>
<th>Concrete Slab</th>
<th>Base and Subbase</th>
<th>Subgrade</th>
<th>Dowel</th>
<th>Tie</th>
<th>Interlock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>Depth</td>
<td>Modulus of subgrade reaction</td>
<td>Dowel length</td>
<td>Tie-slab support modulus</td>
<td>Joint Opening</td>
</tr>
<tr>
<td>Width</td>
<td>Modulus of elasticity</td>
<td>Dowel diameter</td>
<td>Tie-slab restraint modulus</td>
<td>Joint stiffness</td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>Poisson's ratio</td>
<td>Number of dowels</td>
<td>Tie length</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>Density</td>
<td>Dowel-slab support modulus</td>
<td>Spacing of ties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>For Unbonded Base: Initial stiffness, and Slip displacement</td>
<td>Dowel-slab restraint modulus</td>
<td>Tie diameter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient of thermal expansion</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
For the unbonded base, the program uses an elasto-plastic constitutive relationship between shear stresses and relative displacements of slab and base. That requires two new terms: initial stiffness and slip displacement. The term *initial stiffness* defines the stiffness at elastic stage. The term *slip displacement*, on the other hand, defines the relative displacement at the onset of plastic behavior of the interface.

### 3.3 Finite Element Modeling

The pavement was modeled using six slabs, three in the longitudinal direction and two in the transverse direction. The transverse joints between the slabs were doweled and the longitudinal joint was tied. Material properties of concrete were determined by laboratory tests (Pernas, 2009). Properties of base and subgrade were extracted from ORITE database (Database for Ohio SHRP Test Road, Version 2.0). Properties of the lime stabilized subgrade (modulus of elasticity, Poisson’s ratio, and density) were estimated from literature (Mallela et al., 2004), and modulus of subgrade reaction was adjusted in order for the FE results to match the field data. Material properties are shown in Table 3-2.
Table 3-2 Material Properties Used as Input into FE Model

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (in)</th>
<th>Density (kip/in³)</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Poisson's Ratio</th>
<th>Coefficient of thermal expansion (/F°)</th>
<th>Modulus of subgrade reaction (pci)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC 11, (8.86)*</td>
<td>8.14E-05, (8.36E-05)</td>
<td>4220, (3370)</td>
<td>0.25, (0.18)</td>
<td>6.11E-06, (5.78E-6)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ATB 4, (2.95)</td>
<td>8.10E-05</td>
<td>590</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DGAB 4, (9)</td>
<td>7.37E-05</td>
<td>15.8</td>
<td>0.35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lime Stabilized Subgrade 18, (21.6)</td>
<td>6.60E-05</td>
<td>45</td>
<td>0.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Natural Subgrade</td>
<td></td>
<td></td>
<td></td>
<td>350</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dowel and Tie</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Length = 18in, Dia. = 1.5 in, E = 29000 ksi, v = 0.3</td>
<td></td>
</tr>
</tbody>
</table>

Note:
* Values in parentheses show the properties for the pavement in I-86.

Other important program inputs and their values used in this study are shown in Table 3-3. Again the values in parentheses represent the I-86 pavement. For interaction between slab and dowel or tie, two terms were used. Dowel (or tie)-slab support modulus represents the vertical support between dowel (or tie) and the slabs. Dowel (or tie)-slab restraint modulus shows the horizontal support or friction between dowel (or tie) and the slabs (Davids, 2003). Joint opening and joint stiffness introduce the load transfer through aggregate interlock.
Table 3-3 Important Inputs for FE Program

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab length (ft)</td>
<td>13, (15.58)</td>
</tr>
<tr>
<td>Slab width (ft)</td>
<td>14, (12)</td>
</tr>
<tr>
<td>Initial stiffness (ksi/in)</td>
<td>1</td>
</tr>
<tr>
<td>Slip displacement (in)</td>
<td>0.1</td>
</tr>
<tr>
<td>Number of dowels (instrumented slab)</td>
<td>12, (14)</td>
</tr>
<tr>
<td>Dowel-slab support modulus (ksi)</td>
<td>1350</td>
</tr>
<tr>
<td>Dowel-slab restraint modulus (ksi/in)</td>
<td>0</td>
</tr>
<tr>
<td>Tie-slab support modulus (ksi)</td>
<td>150</td>
</tr>
<tr>
<td>Tie-slab restraint modulus (ksi)</td>
<td>1500</td>
</tr>
<tr>
<td>Tie length (in)</td>
<td>40</td>
</tr>
<tr>
<td>Spacing of ties (in)</td>
<td>40</td>
</tr>
<tr>
<td>Tie diameter (in)</td>
<td>0.5</td>
</tr>
<tr>
<td>Joint Opening (in)</td>
<td>0.2</td>
</tr>
<tr>
<td>Joint stiffness (ksi)</td>
<td>0</td>
</tr>
</tbody>
</table>

Sensor locations for Del-23 and I-86 are shown in Table 3-4 and Table 3-5, respectively.

Table 3-4 Locations of Strain Indicating Sensors in Section 390269, Del-23

<table>
<thead>
<tr>
<th>Abbr</th>
<th>Sensor</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Z (in above bottom of slab)</th>
<th>Property measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>KM Strain transducer</td>
<td>36</td>
<td>-54</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K2</td>
<td>KM Strain transducer</td>
<td>36</td>
<td>-54</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K3</td>
<td>KM Strain transducer</td>
<td>78</td>
<td>-54</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K4</td>
<td>KM Strain transducer</td>
<td>78</td>
<td>-54</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K5</td>
<td>KM Strain transducer</td>
<td>120</td>
<td>-54</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K6</td>
<td>KM Strain transducer</td>
<td>120</td>
<td>-54</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K7</td>
<td>KM Strain transducer</td>
<td>258</td>
<td>-54</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K8</td>
<td>KM Strain transducer</td>
<td>258</td>
<td>-54</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K9</td>
<td>KM Strain transducer</td>
<td>36</td>
<td>30</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>K10</td>
<td>KM Strain transducer</td>
<td>36</td>
<td>30</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>V1</td>
<td>Vishay MM strain gage</td>
<td>78</td>
<td>30</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>V2</td>
<td>Vishay MM strain gage</td>
<td>78</td>
<td>30</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>V3</td>
<td>Vishay MM strain gage</td>
<td>234</td>
<td>30</td>
<td>9.5</td>
<td>Longitudinal Strain</td>
</tr>
<tr>
<td>V4</td>
<td>Vishay MM strain gage</td>
<td>234</td>
<td>30</td>
<td>1.5</td>
<td>Longitudinal Strain</td>
</tr>
</tbody>
</table>
The X coordinate of the sensors originates at the edge of the first slab. Therefore, an X coordinate of 234 inches represents a location at the middle of the second slab. The Y coordinate, on the other hand, starts at the center of the first row of slabs (Figure 3-4). The sensors K3, K4, V1, V2, V3 and V4 are located in the middle of the slabs. These locations are important while studying the maximum midslab tensile stresses caused by either dynamic or thermal loading.

Table 3-5 Locations of Strain Indicating Sensors in the I-86 Pavement, Rubblized Section

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Sensor</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Z (in above bottom of slab)</th>
</tr>
</thead>
<tbody>
<tr>
<td>e</td>
<td>VW strain gage</td>
<td>140.15</td>
<td>0</td>
<td>7.86</td>
</tr>
<tr>
<td>e</td>
<td>VW strain gage</td>
<td>140.15</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>f</td>
<td>VW strain gage</td>
<td>94</td>
<td>43</td>
<td>7.86</td>
</tr>
<tr>
<td>f</td>
<td>VW strain gage</td>
<td>94</td>
<td>43</td>
<td>1</td>
</tr>
<tr>
<td>g</td>
<td>VW strain gage</td>
<td>140.15</td>
<td>43</td>
<td>7.86</td>
</tr>
<tr>
<td>g</td>
<td>VW strain gage</td>
<td>140.15</td>
<td>43</td>
<td>1</td>
</tr>
<tr>
<td>m</td>
<td>VW strain gage</td>
<td>280.3</td>
<td>-42</td>
<td>7.86</td>
</tr>
<tr>
<td>m</td>
<td>VW strain gage</td>
<td>280.3</td>
<td>-42</td>
<td>1</td>
</tr>
<tr>
<td>n</td>
<td>VW strain gage</td>
<td>280.3</td>
<td>0</td>
<td>7.86</td>
</tr>
<tr>
<td>n</td>
<td>VW strain gage</td>
<td>280.3</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>o</td>
<td>VW strain gage</td>
<td>327.15</td>
<td>0</td>
<td>7.86</td>
</tr>
<tr>
<td>o</td>
<td>VW strain gage</td>
<td>327.15</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

Locations of these sensors are illustrated in Figure 3-3 and the dimensions with the origin of the coordinates are shown in Figure 3-5. The sensor n is located at the center of the slab, longitudinally and transversely. This location is important for studying midslab stresses due to thermal loading.
3.3.1 Traffic Loading

First the pavement was modeled only for traffic loading, assuming zero temperature gradient throughout the slab thickness. The ODOT dump truck was used for the analysis. Footprint of the single axle truck is shown in Figure 3-6.

![Footprint of Tires for Single Axle Dump Truck](image)

The loads numbers and their magnitudes are shown in Table 3-6.

<table>
<thead>
<tr>
<th>Testing Time</th>
<th>Speed (mph)</th>
<th>Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9:10 AM 26-Sep-07</td>
<td>5</td>
<td>12950</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13950</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6500</td>
</tr>
</tbody>
</table>
The required locations of truck (when sensors are located under either front axle or rear axle) are shown in Table 3-7. These locations are considered with respect to the first joint (left edge of the first slab, longitudinally). A location of 258 inches means that the front axle is located 258 inches from the first joint (the edge) of the three longitudinal slabs considered in the model. This also means that, for single axle truck, the rear axle is located 87 inches from edge of the first slab. The program was run at enough other points to establish a rather smooth curve.

Table 3-7 Required Truck Locations

<table>
<thead>
<tr>
<th>Distance of front axle from first joint</th>
<th>cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>36</td>
<td>91.44</td>
</tr>
<tr>
<td>78</td>
<td>198.12</td>
</tr>
<tr>
<td>120</td>
<td>304.8</td>
</tr>
<tr>
<td>207</td>
<td>525.78</td>
</tr>
<tr>
<td>234</td>
<td>594.36</td>
</tr>
<tr>
<td>249</td>
<td>632.46</td>
</tr>
<tr>
<td>258</td>
<td>655.32</td>
</tr>
<tr>
<td>291</td>
<td>739.14</td>
</tr>
<tr>
<td>405</td>
<td>1028.7</td>
</tr>
<tr>
<td>429</td>
<td>1089.66</td>
</tr>
</tbody>
</table>

For converting the stresses calculated by EverFE to longitudinal strains in concrete the following three-dimensional linear elasticity is utilized (Mase & Mase, 2004).

\[
\varepsilon_{ij} = \frac{1}{E} \left[ (1 + \nu)\sigma_{ij} - \nu \delta_{ij} \sigma_{kk} \right]
\]

Eqn 3-1

Expansion of this equation yields three equations for three directions of strains in terms of stresses and the elastic constants: modulus of elasticity and Poisson’s ratio.
\[ \varepsilon_{xx} = \frac{1}{E} \left[ (1 + \nu)\sigma_{xx} - \nu(\sigma_{xx} + \sigma_{yy} + \sigma_{zz}) \right] \]  
Eqn 3-2

\[ \varepsilon_{yy} = \frac{1}{E} \left[ (1 + \nu)\sigma_{yy} - \nu(\sigma_{xx} + \sigma_{yy} + \sigma_{zz}) \right] \]  
Eqn 3-3

\[ \varepsilon_{zz} = \frac{1}{E} \left[ (1 + \nu)\sigma_{zz} - \nu(\sigma_{xx} + \sigma_{yy} + \sigma_{zz}) \right] \]  
Eqn 3-4

In the equations above, \( E \) represents modulus of elasticity and \( \nu \) shows Poisson’s ratio of concrete.

3.3.2 Environmental Loading

3.3.2.1 Validation of the Model vs. Analytical Solution

To assure the ability of the program in modeling the curling stresses in rigid pavement, the program results were compared to the closed-form solution. In order to do this, the temperature loading for one single slab was solved analytically. The maximum stress in the X-direction as well as in the Y-direction was calculated in the center of the slab at the top. In the following formula \( C_x \) and \( C_y \) are correction factors (Huang 2004).

\[ \sigma_x = \frac{E\alpha \Delta t}{2(1-\nu^2)} \left( C_x + \nu C_y \right) \]  
Eqn 3-5

\[ \sigma_y = \frac{E\alpha \Delta t}{2(1-\nu^2)} \left( C_y + \nu C_x \right) \]  
Eqn 3-6

The results of this analysis are shown in Table 3-8, together with the temperature differential (\( \Delta t \)) and the elapsed time of testing. Since only one slab is considered in closed form solution, the analytical results are compared to those of three consecutive slabs in the FE model. The temperature differential is the difference between change of temperature in the top and change of temperature in the bottom, i.e.

\[ \Delta t = (change \ in \ temperature)_{bottom} - (change \ in \ temperature)_{top} \]  
Eqn 3-7
This temperature differential ($\Delta t$), used for both FE and closed form solution, can be interpreted as follows.

$$\Delta t = (t_2 - t_{2o}) - (t_1 - t_{1o})$$  \hspace{1cm} \text{Eqn 3-8}

Where, $t_2$ is temperature at the bottom of the slab, $t_1$ is temperature at the top of the slab; and $t_{2o}$ and $t_{1o}$ are temperatures at the bottom and top of the slab at zero-strain temperature condition, respectively. The above equation can be rewritten as:

$$\Delta t = (t_2 - t_1) - (t_{2o} - t_{1o})$$  \hspace{1cm} \text{Eqn 3-9}

Yet, usually the zero-strain temperature is chosen to be a point in which the top and bottom of the slab has negligible temperature difference. Hence, the second term of the latter equation drops out, leaving the following equation:

$$\Delta t = (t_2 - t_1)$$  \hspace{1cm} \text{Eqn 3-10}

It is worth mentioning that the temperature distribution through thickness was assumed linear at this point. Table 3-8 presents the results of analytical and FE solutions.

Table 3-8 Verification of FE Model with Analytical Solution for Single Slab

<table>
<thead>
<tr>
<th>Elapsed time (hr)</th>
<th>$\Delta t$ through slab thickness(°F)</th>
<th>Analytical</th>
<th>Finite Element</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\sigma_{xx}$ (psi)</td>
<td>$\sigma_{yy}$ (psi)</td>
</tr>
<tr>
<td>6</td>
<td>-0.36</td>
<td>-4.07</td>
<td>-4.41</td>
</tr>
<tr>
<td>12</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>18</td>
<td>2.5</td>
<td>28.23</td>
<td>30.62</td>
</tr>
<tr>
<td>24</td>
<td>2.8</td>
<td>31.62</td>
<td>34.29</td>
</tr>
<tr>
<td>39</td>
<td>-1.12</td>
<td>-12.65</td>
<td>-13.72</td>
</tr>
<tr>
<td>42</td>
<td>2.25</td>
<td>25.41</td>
<td>27.56</td>
</tr>
<tr>
<td>50</td>
<td>3.7</td>
<td>41.79</td>
<td>45.32</td>
</tr>
</tbody>
</table>
Results are plotted for longitudinal and transverse stresses, in terms of stresses versus elapsed time of testing, in Figure 3-7 and Figure 3-8, respectively.

Figure 3-7 Comparison of Analytical versus FE Solution for Longitudinal Stresses
Comparison of the stresses indicates a very good matching between FE results and the Westergaard analytical solution. In terms of maximum longitudinal and transverse stresses, in average, the difference is 6.1% and 8.6%, respectively. In Westergaard’s solution, a single slab is considered, neglecting the effects of dowel and tie bars as well as the shear friction between slabs and base. Hence, the close matching between FE and Westergaard shows that assumption of a linear temperature gradient, which was done at this stage, does not provide realistic response in the slabs.

3.3.2.2 Nonlinear Temperature Gradient

Since temperature distribution through thickness of the slab is nonlinear, temperature measurements inside the slab is crucial. Temperature readings inside the slab were not available in Del 23 (Pernas 2009). Hence, a section of Interstate 86 (shown in Figure 3-3) was picked for validation of the FE model.
The pavement was modeled for temperature changes during the course of time. The FE modeling results were compared to change of stresses measured in the field during the same period of time. At this stage no traffic loading was included. The program EverFE requires temperature changes as input into the finite element model. The temperature change is relative to a zero-strain temperature, which is calculated by multiplying the coefficient of thermal expansion to the change in temperature at the considered point (Davids, 2003).

The program calculates the curling stresses by subtracting the strains due to change in temperature (pre-strain) from the total strains as follows.

\[
\sigma_{curt} = E [ \varepsilon_{total} - \alpha \Delta t ]
\]

Eqn 3-11

It is capable of modeling a trilinear temperature distribution. Hence, a four point temperature profile was utilized in the model: top, mid top, mid bottom, and bottom. An illustration of trilinear temperature distribution is shown in Figure 3-9.

![Figure 3-9 Illustration of Trilinear Temperature Distribution](image)

The temperature profile of the slab with respect to time is tabulated in Table 3-9, and plotted with slab depth in Figure 3-10.
Table 3-9 Temperature Profile of the Slab on I-86

<table>
<thead>
<tr>
<th>Slab depth (in)</th>
<th>Temperature (°F)</th>
<th>Midnight</th>
<th>6 a.m.</th>
<th>11 a.m.</th>
<th>1 p.m.</th>
<th>4 p.m.</th>
<th>6 p.m.</th>
<th>midnight</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>54.46</td>
<td>50.97</td>
<td>65.28</td>
<td>71.35</td>
<td>63.66</td>
<td>62.04</td>
<td>55.49</td>
</tr>
<tr>
<td>2.95</td>
<td></td>
<td>50.52</td>
<td>47.91</td>
<td>59.78</td>
<td>68.10</td>
<td>68.59</td>
<td>61.66</td>
<td>55.90</td>
</tr>
<tr>
<td>5.91</td>
<td></td>
<td>51.98</td>
<td>49.84</td>
<td>58.42</td>
<td>65.77</td>
<td>66.65</td>
<td>63.39</td>
<td>54.79</td>
</tr>
<tr>
<td>8.86</td>
<td></td>
<td>49.28</td>
<td>47.41</td>
<td>52.90</td>
<td>57.16</td>
<td>59.74</td>
<td>57.47</td>
<td>55.15</td>
</tr>
</tbody>
</table>

Figure 3-10 Temperature Profile of the Slab on I-86

As seen in the figure, the top surface is cold at midnight. With moving towards morning, it further cools down. The top surface starts warming up as the air temperature increases, continuing until afternoon. Late in the afternoon, it again starts cooling down. It is worth mentioning that, for simplicity, the temperature measured by the topmost
thermocouple (an inch below the top) was assumed to be the temperature at the top surface.

Next, to study the effect of reference point on the trend of stresses, the reference temperature point was changed to noon on April 23rd. Temperatures at all other times were considered with respect to the temperature at this point. The temperature profile of the slab with respect to time is tabulated in Table 3-10.

Table 3-10 Temperature Profile of the Slab on I-86, New Reference Point

<table>
<thead>
<tr>
<th>Depth (in)</th>
<th>12:00 PM</th>
<th>3:00 PM</th>
<th>9:00 PM</th>
<th>12:00 AM</th>
<th>7:00 AM</th>
<th>9:00 AM</th>
<th>12:00 PM</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>50.92</td>
<td>60.53</td>
<td>54.82</td>
<td>47.97</td>
<td>43.29</td>
<td>48.13</td>
<td>65.03</td>
</tr>
<tr>
<td>2.95</td>
<td>46.65</td>
<td>60.24</td>
<td>59.13</td>
<td>51.18</td>
<td>42.98</td>
<td>47.57</td>
<td>61.41</td>
</tr>
<tr>
<td>5.91</td>
<td>47.79</td>
<td>56.52</td>
<td>57.34</td>
<td>51.80</td>
<td>44.58</td>
<td>46.62</td>
<td>57.42</td>
</tr>
<tr>
<td>8.86</td>
<td>44.04</td>
<td>50.38</td>
<td>57.27</td>
<td>52.74</td>
<td>45.43</td>
<td>44.13</td>
<td>52.09</td>
</tr>
</tbody>
</table>

The above temperatures are plotted versus depth of the slab in Figure 3-11.
Once the FE model was verified in terms of comparison with the measured data, the model could be used for similar cases. Likely, the Del 23 pavement was, then, modeled for temperature loading using the two measured temperatures and the following estimation procedure:

1. A temperature near to the value measured by the bottom thermocouple was assumed for the bottom surface of the slab (depth = 11 in).
2. According to the air temperature, a value (close to the air temperature, considering the time of the day) was assumed for the top of the slab (depth = 0 in).
3. These four points (two assumed and two thermocouples readings) were fit using a polynomial of order three.

4. Using the equation of the polynomial the temperature at any required point was estimated.

5. Comparison of the FE output with the measured data verified the assumptions.

The measured temperatures (collected in April) as well as the assumed temperatures, at the top and bottom of the slab, are shown in Figure 3-12. Using the third order polynomial, the temperatures at different depths of the slab, mid top and mid bottom, were estimated. These estimated temperatures are shown, together with the assumed top and bottom temperatures, in Table 3-11 and Figure 3-13.

Table 3-11 Estimated Temperature Profile of the Slabs on Del-23, Section 390269

<table>
<thead>
<tr>
<th>depth (in)</th>
<th>12 AM</th>
<th>6 AM</th>
<th>9 AM</th>
<th>12 PM</th>
<th>6 PM</th>
<th>12 AM</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>60</td>
<td>58</td>
<td>70</td>
<td>64</td>
<td>35</td>
<td>28</td>
</tr>
<tr>
<td>3.6667</td>
<td>49.22</td>
<td>50.31</td>
<td>46.90</td>
<td>48.77</td>
<td>54.96</td>
<td>54.86</td>
</tr>
<tr>
<td>7.3333</td>
<td>51.83</td>
<td>51.43</td>
<td>50.48</td>
<td>51.66</td>
<td>49.66</td>
<td>49.77</td>
</tr>
<tr>
<td>11</td>
<td>48</td>
<td>50</td>
<td>50</td>
<td>51</td>
<td>57</td>
<td>50</td>
</tr>
</tbody>
</table>
Figure 3-12 Measured Temperatures and Estimated Temperatures of the Slabs on Del-23

Figure 3-13 Estimated Temperature Profile of the Slabs in Pavement on Del-23
The temperature at the top of the slab varies over a range of around 40°F. This range is much smaller in the bottom of the slab (around 10°F), and in the interior parts of the slab. During the day, the slab surface is exposed to solar radiation, and therefore it absorbs that much thermal energy so that it even becomes warmer than the ambient air. A major portion of this absorbed thermal energy cannot be transferred to the bottom of the slab due to low thermal conductivity of concrete (around 1.5 W/mK) and the convection taking place to the ambient air. Hence the temperature at the bottom surface of the slab does not change much.

In a related manner, during the night the surface of the warm slab emits the radiation back to the night sky (Bentz, 2000). This, again, can be associated with heat convection from the slab to the ambient air. The combination of these heat transfers from top surface of the slab can result finally into making the concrete surface cooler than the ambient air (Bentz, 2000). Again, because the bottom surface of the slab is not imposed to neither convection nor radiation and emission, and because the thermal conductivity of concrete is low, the temperature of the bottom surface does not change much.
CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Verification of Strains Induced due to Traffic Loading

Results for traffic loading are verified in two stages: comparison of strains and comparison of stresses. From the FE program, the stresses are determined in the specific locations of the strain indicators shown in Figure 3-1 and Figure 3-2 as well as Table 3-4. Using these output stresses and utilizing Eqn 3-1 the strains are calculated and are plotted versus distance of front axle from first joint. These strains are compared with measured strains due to single axle dump truck loading, and are presented in Figure 4-1 to Figure 4-14. The required locations of the truck are shown in Table 3-7 and the axle loads are shown in Figure 3-6 and Table 3-6.

In order to compute the dynamic loading strains using FE program, only traffic loads were input. No temperature was involved during this stage of analysis. In the same manner, the measured strains were collected in a short period of time, which includes neither the temperature changes nor any other environmental factors. These strain transducers were initially set to zero to exclude the effects of environmental loadings. Doing so, the strains due to temperature was not considerable in the slab because the temperature does not change considerably in a very short period of time. Hence, the results of FE program was compared with these collected data.
Figure 4-1 Comparison of Longitudinal Strains for Section 390269, Location K1 (Top)

Figure 4-2 Comparison of Longitudinal Strains for Section 390269, Location K2 (Bottom)
Figure 4-3 Comparison of Longitudinal Strains for Section 390269, Location K3 (Top)

Figure 4-4 Comparison of Longitudinal Strains for Section 390269, Location K4 (Bottom)
Figure 4-5 Comparison of Longitudinal Strains for Section 390269, Location K5 (Top)

Figure 4-6 Comparison of Longitudinal Strains for Section 390269, Location K6 (Bottom)
Figure 4-7 Comparison of Longitudinal Strains for Section 390269, Location K7 (Top)

Figure 4-8 Comparison of Longitudinal Strains for Section 390269, Location K8 (Bottom)
Figure 4-9 Comparison of Longitudinal Strains for Section 390269, Location K9 (Top)

Figure 4-10 Comparison of Longitudinal Strains for Section 390269, Location K10 (Bottom)
Figure 4-11 Comparison of Longitudinal Strains for Section 390269, Location V1 (Top)

Figure 4-12 Comparison of Longitudinal Strains for Section 390269, Location V2 (Bottom)
The distance between the two consecutive extreme strains is 171 inches, which is also the distance between the front and rear axles of the single axle dump truck (Figure 3-6). Each sensor reads a peak strain when the front axle places vertically above it. Later
when the rear axle reaches the location of the sensor, it reads higher strains because the rear axle is heavier and also the effects of the front and rear axles accumulate.

The FE results follow the general trend of the measured data. However, the magnitudes are somehow different because the material properties, such as modulus of elasticity, Poisson’s ratio, and density of lime treated subgrade are extracted from the literature. Also, the sensor V1 did not read the proper range of stresses that was measured by all other sensors.

4.2 Results for Stresses

As mentioned in chapter 3, to study the effect of Poisson’s ratio on the rigid pavement under dynamic loading, simplified Hook’s law was utilized. FE results were compared with the stresses computed by multiplying the measured strains by the modulus of elasticity of concrete (Table 3-2). The stresses are compared at the locations of the strain indicating sensors (shown in Figure 3-1 and Figure 3-2), and are plotted versus distance of front axle from first joint (edge of the first slab) in Figure 4-15 to Figure 4-28.
Figure 4-15 Comparison of Longitudinal Stresses for Section 390269, Location K1 (Top)

Figure 4-16 Comparison of Longitudinal Stresses for Section 390269, Location K2 (Bottom)
Figure 4-17 Comparison of Longitudinal Stresses for Section 390269, Location K3 (Top)

Figure 4-18 Comparison of Longitudinal Stresses for Section 390269, Location K4 (Bottom)
Figure 4-19 Comparison of Longitudinal Stresses for Section 390269, Location K5 (Top)

Figure 4-20 Comparison of Longitudinal Stresses for Section 390269, Location K6 (Bottom)
Figure 4-21 Comparison of Longitudinal Stresses for Section 390269, Location K7 (Top)

Figure 4-22 Comparison of Longitudinal Stresses for Section 390269, Location K8 (Bottom)
Figure 4-23 Comparison of Longitudinal Stresses for Section 390269, Location K9 (Top)

Figure 4-24 Comparison of Longitudinal Stresses for Section 390269, Location K10 (Bottom)
Figure 4-25 Comparison of Longitudinal Stresses for Section 390269, Location V1 (Top)

Figure 4-26 Comparison of Longitudinal Stresses for Section 390269, Location V2 (Bottom)
Figure 4-27 Comparison of Longitudinal Stresses for Section 390269, Location V3 (Top)

Figure 4-28 Comparison of Longitudinal Stresses for Section 390269, Location V4 (Bottom)
As seen in Figure 4-1 to Figure 4-28, comparison of the traffic-induced strains gives a better matching (in terms of average of maximum strains, 12.2%) than that of traffic-induced stresses (in terms of average of maximum stresses, 23.3%). This is mainly because in converting the output stresses from the FE program to strains, use is made of the equation of three dimensional elasticity (Eqn 3-1), which includes the effect of Poisson’s ratio. In contrast, the comparison of stresses requires conversion of measured strains to stresses, which, due to unavailability of three-dimensional state of measured stresses, failed to include the effect of Poisson’s ratio (simplified Hook’s law). In other words, determination of longitudinal strains, using a three dimensional state of stress, better simulates the field situation compared to that using a hydrostatic state of stress.

Furthermore, comparison of strains and stresses for the sensors located at right wheel path, K9, K10, and V1 to V4, (Figure 4-23 to Figure 4-28) did not give very good matching. An assumption made in the analysis was that the asphalt shoulder does not provide constraint for the concrete slab. Therefore, the pavement was assumed to consist of two slabs in the transverse direction. The left edge of the slabs was tied to the adjacent slabs, which simulates the field situation very well, while the right edge was assumed free, which does not accurately simulate the in-situ condition.

4.3 Curling Stresses

The environmental stresses can be created in the slab due to a combined effect of different factors such as temperature gradient, permanent loss of support created during curing, and moisture gradient in the slab. Although the concrete slab adjusts to the permanent loss of support created during curing period, and hence, the so-called stress
relaxation occurs, loading a curved slab induces different stresses compared to loading a flat slab.

The permanent loss of support (LOS) created during curing can be assumed constant once concrete hardens. Also, the data collection took place on a dry day (Pernas, 2009). Hence, while subtracting the stresses at any point from those of a reference point, the constant stresses due to LOS (and any stress created during curing) and the identical moisture gradients will drop. Since the sensors were initially set to zero, these measured stresses can be considered only due to thermal effects, and therefore can be used to verify the FE curling stress results.

Likely, in the following figures (Figure 4-29 to Figure 4-60) the stresses relative to a reference stress are compared between FE output and measured data. The temperature differentials between top and bottom are plotted on secondary axis to show their correlation with longitudinal stresses. The temperature profile of the slab with respect to time is tabulated in Table 3-9, Table 3-10, and Table 3-11, and shown in Figure 3-10, Figure 3-11, and Figure 3-13. At locations K2, K3 and K5 only predicted results are reported since no data was recorded.

At this stage only temperature changes were input to EverFE2.24, i.e., no dynamic load is included at this point. The measured strains are plugged into Eqn 3-11 to compute the measured stresses. In Eqn 3-11, the term Δt was considered to be the difference in temperatures of each point through thickness, between any considered time and the reference time (midnight for Del 23, midnight and noon for I-86).
Figure 4-29 Comparison of Curling Stresses for Section 390269, Location K1 (Top)

Figure 4-30 Comparison of Curling Stresses for Section 390269, Location K2 (Bottom)
Figure 4-31 Comparison of Curling Stresses for Section 390269, Location K3 (Top)

Figure 4-32 Comparison of Curling Stresses for Section 390269, Location K4 (Bottom)
Figure 4-33 Comparison of Curling Stresses for Section 390269, Location K5 (Top)

Figure 4-34 Comparison of Curling Stresses for Section 390269, Location K6 (Bottom)
In locations K2, K3, and K5 no temperature was recorded, and hence, no stress data were available there.
Figure 4-37 Comparison of Curling Stresses for I-86, Location e (Top)

Figure 4-38 Comparison of Curling Stresses for I-86, Location e (Bottom)
Figure 4-39 Comparison of Curling Stresses for I-86, Location f (Top)

Figure 4-40 Comparison of Curling Stresses for I-86, Location f (Bottom)
Due to unavailability of temperature, neither the measured stresses are available in the bottom, nor is the temperature differential between top and bottom at location “g”.

Figure 4-41 Comparison of Curling Stresses for I-86, Location g (Top)

Figure 4-42 FE-calculated Curling Stresses for I-86, Location g (Bottom)
Figure 4-43 Comparison of Curling Stresses for I-86, Location m (Top)

Figure 4-44 Comparison of Curling Stresses for I-86, Location m (Bottom)
Figure 4-45 Comparison of Curling Stresses for I-86, Location n (Top)

Figure 4-46 Comparison of Curling Stresses for I-86, Location n (Bottom)
Figure 4-47 Comparison of Curling Stresses for I-86, Location o (Top)

Figure 4-48 Comparison of Curling Stresses for I-86, Location o (Bottom)
The correlation between temperature differential and change in thermal stresses is interesting. The temperature differential is considered as the temperature at the top minus the temperature at the bottom, for stresses at the top; it is considered as the temperature at the bottom minus that at the top, for stresses at the bottom of the slabs. As seen in Figure 4-37 to Figure 4-48, higher tensile stresses develop at the top as the temperature differential decreases and hence, higher negative temperature gradients develop in the slab. Likely, higher compressive stresses develop at the top with higher temperature differential and thus higher positive temperature gradient. This means that the top surface is warmer and, hence, starts expanding and forcing the bottom to comply. The bottom surface is colder and thus cannot expand in the same rate as the top surface. This eventually causes compressive stresses in the top surface of the slab. The same logic holds for the sensors at the bottom of the slab. This point can be seen in Figure 4-29 to Figure 4-60.
Figure 4-49 Comparison of Curling Stresses for I-86, Location e (Top)

Figure 4-50 Comparison of Curling Stresses for I-86, Location e (Bottom)
Figure 4-51 Comparison of Curling Stresses for I-86, Location f (Top)

Figure 4-52 Comparison of Curling Stresses for I-86, Location f (Bottom)
Figure 4-53 Comparison of Curling Stresses for I-86, Location g (Top)

Figure 4-54 FE-calculated Curling Stresses for I-86, Location g (Bottom)
Figure 4-55 Comparison of Curling Stresses for I-86, Location m (Top)

Figure 4-56 Comparison of Curling Stresses for I-86, Location m (Bottom)
Figure 4-57 Comparison of Curling Stresses for I-86, Location n (Top)

Figure 4-58 Comparison of Curling Stresses for I-86, Location n (Bottom)
Figure 4-59 Comparison of Curling Stresses for I-86, Location o (Top)

Figure 4-60 Comparison of Curling Stresses for I-86, Location o (Bottom)
The results presented in Figure 4-37 to Figure 4-60 indicate that the FE output stresses follow the general trend of the measured stresses. However, the difference observed in the magnitudes may stem from the fact that, besides the estimation of material properties for subgrade from the literature, the temperature at the very top as well as very bottom of the slab is not known. It is assumed that the topmost thermocouple (1 inch below top) is located on the top surface of the slab, and the bottommost thermocouple (1 inch above bottom) is located on the bottom surface of the slab. This assumption may not be completely accurate; however, it yields (as shown here) promising results.

Another important point is that the curling stress is not zero whenever the temperature differential is zero. This can be because of three reasons, besides the expansion of the slabs and closure of joints. The first reason is that the thermally induced stresses are created due to nonlinear temperature distribution throughout the thickness of the slab. Hence, even though the temperatures of the outermost fibers become identical, there is still some nonlinear temperature gradient inside the slab that causes some stresses.

Secondly, the thermally-induced stress, calculated herein, is a combination of curling stress due to the temperature gradient with respect to depth of the slab, and the expansion or contraction of a fiber compared to some reference temperature state. Therefore, although the temperature difference between top and bottom becomes zero, yet, the temperature history of the two layers are not identical. Hence, certain amount of
the so-called residual stresses may exist in the slab due to the different temperature
history of the top surface compared to that of the bottom surface.

Finally, the process of heat transfer through a finite temperature difference is not a
reversible process. The heat conduction throughout the slab occurs spontaneously, only,
because of a finite temperature difference. Hence, even though the temperature
differential vanishes, the process of change in the state of the concrete, started from a
zero-strain condition, cannot completely return to the zero-strain condition. From a
mechanics perspective, there exists some hysteresis while loading and unloading the
system since concrete is not ideally elastic.

The FE program EverFE requires the temperature changes through slab thickness
rather than absolute temperatures. This requires assumption of a reference temperature
(zero-strain temperature). As stated earlier, such a situation, in which no stress is induced,
practically does not exist. Even though the top and bottom fibers have the same
temperatures, nonlinear temperature distribution will cause some stresses. To observe the
effect of selection of different reference points on the trend of stresses, the reference
point for temperature gradient was assumed at two different times (once, midnight and
then, noon). The results from these two different reference points show the same range of
stresses.

Furthermore, the stresses calculated based on reference point of noon gives more
tensile stresses at the top of the slab (compare Figure 4-37, Figure 4-39, Figure 4-41,
Figure 4-43, Figure 4-45, and Figure 4-47 with Figure 4-49, Figure 4-51, Figure 4-53,
Figure 4-55, Figure 4-57, and Figure 4-59, respectively). This is because the top surface
of the slab is rather warm at noon; subtraction of other temperatures from this temperature gives negative temperature changes, and hence, tensile stresses at the top of the slab. At midnight, the temperature of the top surface of the slab is rather cold (even colder than ambient air, frequently), and referencing this temperature for consideration of other temperatures gives rather positive temperature changes, and hence, compressive stresses at the top of the slab. Same reasoning can be done about the stresses in the bottom of the slab. However, the bottom surface does not suffer as much stress reversals as the top surface does.

Next, Figure 4-37 to Figure 4-60 show better matching of FE and measured stresses at the sensors located at the bottom compared to those located at the top of the slab. This can be explained as follows. The topmost thermocouple is assumed to be located at the top surface of the slab, and the bottommost thermocouple is assumed to be located at the bottom surface of the slab; however, they are located one inch below and one inch above the concrete surfaces, respectively. In the mean while, the temperature gradient in the upper portion of the slab is much steeper than that at the lower portion of the concrete slab. Hence, the assumption made for the location of the top sensor can cause more error compared to the assumption made for the bottom sensor. Nevertheless, the FE results follow the general trend of the measured stresses.

4.4 Fatigue Analysis

Using the concrete modulus of elasticity considered in the model, fatigue analysis for section 390269, as well as the I-86 pavement, was performed. The number of load
repetitions to failure was calculated using Eqn 4-1 (Huang, 2004) with modulus of rupture from Eqn 4-2.

\[
\log N_f = 17.61 - 17.61 \left( \frac{a}{f_r} \right) \quad \text{Eqn 4-1}
\]

\[
f_r = 7.5 \sqrt{f'_c} \quad \text{Eqn 4-2}
\]

Results are reported in Table 4-1 and Figure 4-61.

<table>
<thead>
<tr>
<th>Applied Stress (psi)</th>
<th>Number of Load Repetitions to Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Del-23</td>
</tr>
<tr>
<td>100</td>
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Comparison of stresses (Figure 4-15 to Figure 4-36) with fatigue results (Table 4-1 and Figure 4-61) yields that the maximum stresses induced due to traffic loads (around 100 psi) can lead the pavement to failure after applying $3 \times 10^{14}$ times. This number of load repetitions is sufficiently enough for the pavement with normal performance. Considering an annual average daily traffic (AADT) of 6000, at most, the pavement will function around 100 million years before failing. On the other hand, the maximum thermal stress (a range of 450 psi) can lead the pavement to failure in 2,180 repetitions. With the assumption that the maximum stresses (and hence, maximum temperature gradients) occur once a day or at least once in two days, the pavement cannot
resist more than 6 to 12 years against thermal fatigue damage. Nevertheless, the effects of traffic loading increase those of thermal loading.

Consideration of range of stresses rather than the instantaneous stress, in thermally-induced stresses, can be justified by the fact that the stresses plotted in Figure 4-29 to Figure 4-60 are change of stresses. Although the range of stresses is immaterial of the selection of reference temperature point (Figure 4-37 to Figure 4-48 can be compared with Figure 4-49 to Figure 4-60), development of either tensile or compressive stresses in the slab depends on the selection of this reference point. Therefore, it is likely that the stresses, for instance at the top, shift toward tensile stresses when the reference point is changed. Hence, consideration of stresses rather than their range will underestimate the pavement response in such severe cases.
CHAPTER 5: CONCLUSIONS

A finite element (FE) computer program and field data were used to investigate the influence of traffic load and thermal load on the response and performance of rigid pavements. Field data collected from Del-23 and I-86 were used. The pavement response investigation focused on stresses and strains, which are very important in failure and cracking of concrete slabs. Based on the analysis and the discussion in this study, the following can be concluded.

The observed fatigue and failure in rigid pavements is due to the combined traffic and thermal loads. The thermal loads provide the major contribution to the total load on the pavement. In case of the pavement on Del-23, the maximum stresses induced by traffic loads, around 100 psi, can lead the pavement to fatigue failure no earlier than 100 million years, greatly exceeding the design period. For instance, the maximum change in thermally-induced stresses in the pavement, around 450 psi on Del-23, can lead the pavement to fatigue cracking in 2,180 repetitions. Assuming this maximum thermal stress occurs once daily, it takes less than 6 years for the pavement to fail. The addition of traffic loading to thermal stress further accelerates the deterioration of the pavement.

The prevalent mode of failure is top-down cracking which accompanies thermally-induced stresses. These stresses are caused by a nonlinear temperature gradient throughout the slab thickness. The changes in temperature are concentrated near the top surface of the concrete slab. A negative gradient induces high tensile stresses in the top surface of the concrete pavement. However, current rigid pavement design procedures consider only bottom-up cracking and neglect thermal stresses (Huang 2004).
Change of stresses with respect to a point is a better tool for studying the effect of thermal loads compared to absolute stresses. Working with change of stresses simplifies the analysis by eliminating or minimizing the effects of permanent loss of support due to curing as well as moisture gradient. Nevertheless, the stresses due to curing decline as concrete relaxes with time. The finite element results, on the other hand, verified the accuracy of the strain gauges.

Since the temperature changes greatly in the top portion of the slab, measurement of the topmost temperature is recommended as a necessary step in studying thermal stresses. Depending on the accuracy required, the temperature measurement at one inch below the top is not sufficient to obtain close matching between FEM and measured data. Also, due to the dramatic effect of nonlinearity of temperature, more temperature measurement points through slab thickness may give a closer prediction of pavement response. However, this analysis suggests four points of temperature are enough.

It is recommended that the finite element program EverFE2.24 should be upgraded to consider different materials for base rather than a monolithic base. Such a feature can be used to analyze the effect of loss of support, by introducing low-strength base materials for the areas where LOS occurs.
REFERENCES


