Mechanistic-Empirical Pavement Design Guide (MEPDG) Method Implemented to Estimate Damage in Flexible and Rigid Pavements

Thesis

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by

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Abstract

The implementation of the Empirical-Mechanistic Pavement Design Guide (MEPDG) method for flexible and rigid pavements requires numerous input parameters. Most of these parameters can be easily determined while some require best estimates that are usually extracted from available literature. This thesis identifies the most critical input parameters in terms of their effects on the damage of pavements and their influence on the determination of the number of corrective maintenance cycles to be performed during the design life of pavements.

It was found that for flexible pavement, change in the average monthly temperature by as little as 1 °F results in large differences in the number of corrective maintenance cycles. Also, consistently with simple mechanics concepts, pavements on stiffer foundations performed better under the load and hence, required fewer number of the corrective maintenance cycles than those founded on more flexible soils. Also, variations in truck weights affected the outcome in terms of the estimated number of corrective maintenance cycles for flexible pavement. Hence, better estimates of the number of corrective maintenance cycles can be obtained when the analysis was based on larger numbers of truck samples. On the contrary, no significant difference in the final estimation of the number of corrective maintenance cycles was found for rigid pavements even when the average monthly temperatures were increased or decreased by as much as 10 °F. Moreover, no major difference was observed when a larger sample of trucks was used as input for the analysis. Similarly, change in ambient temperature which is directly related to the differential temperature on the top and the bottom of the slab that may lead to the curling of the slab and faulting, was found not to be critical. Similar to the results obtained for flexible pavements, rigid pavement with stiffer foundation properties performed better in terms of the number of corrective maintenance cycles as they required fewer corrective maintenance cycles.
Chapter 1 Introduction

The Mechanistic-Empirical Pavement Design Guide (MEPDG) approach is an improved method to estimate the condition of pavement over time, taking into account vehicular load and environmental conditions. Prior to the MEPDG method, the American Association of State Highway and Transportation Officials (AASHTO) introduced a design guide (Idaho, 2011) that was developed as a result of the American Association of State Highway Officials (AASHO) road test in 1958 (AASHTO, 2008). This former approach was based on the service performance concept. It provided a means for designing pavements based on a specified total traffic volume and a minimum level of serviceability up to the end of the pavement performance period. This approach has been revised in the 1961 AASHTO interim guide, including additional revisions in 1972 and in 1986 by introducing the concept of resilient modulus, as well as the consideration of rehabilitation and reliability. The latest version, the 1993 edition includes the latest improved rehabilitation methods (AASHTO, 2008). However, certain limitations remained, for instance the method assumes only one specific set of pavement materials and only one single environment condition.

In order to provide an improved method and effectively assess pavement damage over time, the Mechanistic-Empirical Pavement Design Guide (MEPDG) method was introduced in 1993 (AASHTO, 2008). The MEPDG method computes pavement responses to vehicular load, traffic and environmental conditions (stresses and strains at different points along the depth of the section) which are used to compute pavement damage (distresses and loss in rideability) over time (R.L. Baus, 2010). Moreover, this method can accommodate better models for pavement materials compared to the previous methods, as well as considers tire pressure instead of just the load under the tire (Tarefder, N/A). The development of the MEPDG method was initiated by a Joint Task Force on Pavements in March 1996. Eventually, the Task force published the National Cooperation Highway Research Program (NCHRP) 1-37A MEPDG (AASHTO, 2008) guide including several interim editions. The MEPDG method proposes an approach that optimizes design and materials by minimizing distresses in the pavement based on the expected truck axle weights including overweight trucks and climate conditions. The MEPDG can accommodate different materials in the pavement such as Hot-Mix Asphalt (HMA), Portland Cement Concrete (PCC) and other aggregate materials. This method is based on both a Mechanistic and an Empirical approach in contrast to traditional pavements design methods which were solely based on an empirical approach.

Since the MEPDG method was introduced, it is being used in the local, state and national levels. For example, the Washington State Department of Transportation (WSDOT) revised its pavement thickness design manual using the MEPDG method concurrently with the 1993 AASHTO Guide (Washington State Department of Transportation, 2011). In 2009, the Illinois Center for Transportation published its “Mechanistic-Empirical Design Concepts for Joined Plain Concrete Pavements in Illinois” (Illinois Center for Transportation, 2009).

In this report, MEPDG method is adopted to estimate the damage of pavement over time and estimate the number of corrective maintenance cycles required to maintain the serviceability of the pavement over its design life. The MEPDG method is used because it is the most comprehensive method in that incorporates the effects of both truck loads and climate as mentioned earlier. For example, by using the Equivalent Single Axle Load (ESAL) method described in the AASHTO Pavement Design Manual (AASHTO, 1993) most state transportation jurisdiction authorities are required to perform corrective
maintenance of flexible pavements every 8 years, and every 13 years for rigid pavements. This approach
of performing corrective maintenance at regular intervals (for both flexible or rigid) are simply general
rules that do not give any consideration to pavement properties (such as layer thickness and stiffness
derived from the material properties), intensity and frequency of truck loads, other environmental and
climate conditions. On one hand, this kind of approach could be very conservative for pavement sections
composed of thick sub layers, good material properties for each sub sections (high elastic modulus or
subgrade modulus), lightly loaded throughout the year and pavement sections located under favorable
environmental conditions. On the other hand, it is questionable that the approach is suitable for pavement
sections designed with minimum thicknesses of each sub layers, very poor material properties, subjected
to heavy truck loading throughout the year and not located in most suitable environmental conditions.
Basically, in the first scenario, corrective maintenance is scheduled to be performed before it is needed
whereas in the second scenario, corrective maintenance of the pavement sections is not performed when
needed. The MEPDG method can be used to optimize pavement corrective maintenance cycles and
economize maintenance costs while maintaining the serviceability of highway pavements.

This thesis builds up on the work performed on a New York State Department of Transportation
(NYSDOT) research study titled “Effects of Overweight Vehicles on NYSDOT Infrastructures”. In that
report Ghosn et al. (2015) studied in depth highway pavement sections of Interstate I-88 that are
considered as representative of the behavior of typical New York State (NYS) pavements. The I-88
Corridor is a 117.7-mile long highway that begins just outside Binghamton in Broome County, New York
and ends just outside Schenectady with a total of 25 exits along the route plus the end points (Ghosn,
2015). The I-88 corridor is composed of both flexible and rigid sections, the latter formed by Jointed
Plain Concrete Pavement (JPCP). In this report, the number of corrective maintenance cycles required to
keep the pavement in good service conditions over a 50-year typical design life is calculated for different
flexible and rigid pavements sections of the I-88 corridor using the MEPDG method. In order to calculate
damage in a pavement segment using the MEPDG method, stresses, strains and displacements due to the
application of truck load or climatic change (especially change in temperature) are required. In this report,
KENPAVE computer program developed by Dr. Yang H. Huang is used to calculate load (vehicular or
thermal) induced stress, strain and displacement. KENPAVE (Huang, 2004) program has two
components; KENLAYER for flexible pavement analysis and KENSLAB for rigid pavements. Damage
calculated is further used to estimate the time when the corrective maintenance cycle would be required
for different pavement segments. Since MEPDG is both a mechanistic as well as empirical method, a
large number of input parameters have to be assembled in order to execute the calculations. These input
parameters pertain to describing the geometry of the pavement segment and its material properties,
subsurface soil property depending upon the geological location of the segment being analyzed,
environmental related parameters such as temperature, precipitation and humidity and truck load
properties including total weight of the truck, axle weights and spacing. Since I-88 corridor was used as
an example in the report by Ghosn et al. (2015), all the input parameters of I-88 are used as base line in
this Thesis, particularly the ones that depend on the location of the pavement segment being analyzed.
Since the MEPDG method is a very comprehensive method that incorporates many input parameters. In
the report by Ghosn et al. (2015), lacking detailed site-specific information, many input parameters used
were default values adopted from different literature. During the work of Ghosn et al. (2015) it was
observed that some input parameters are very critical such that even slight variations in their values could
cause big difference in the final output. It is very important to be able to identify the critical parameters
that have significant impact on the output as well as those that do not have enough influence on the output to help engineers focus on collecting the most critical data during the implementation of the MEPDG method.

The primary objective of this thesis is to identify critical input parameters for the MEPDG analysis of flexible and rigid pavement through a sensitivity analysis performed using the input data of Ghosn et al. (2015) as base line. By categorizing input parameters as “critical” or “not critical”, engineers interested in implementing the MEPDG method to calculate pavement damage and estimating the number of corrective maintenance cycles that are required will know on which input parameters they should focus during the data collection phases.

Chapter 2 of this Thesis, labeled “Methodology”, describes the pertinent sections of the AASHTO (2008) MEPDG approach to estimate the number of corrective maintenance cycles needed to maintain the serviceability of both flexible and rigid pavements. In Chapter 3, a sensitivity analysis is performed to study the effect of various critical input parameters within the MEPDG method on the number of expected corrective maintenance cycles. Chapter 4 gives the conclusions and future work.
Chapter 2 Methodology

The commonly used Present Serviceability Rating (PSR) serves as a criterion to assess the in-situ condition of flexible or rigid pavements and determines the need for corrective maintenance. In practice, PSR is defined as the mean value of the independent ratings obtained by different pavement inspectors who are evaluating the present serviceability condition of a specific roadway section (Huang, 2004). PSR assigns a number between 0 and 5 to a pavement segment with a rating of 0 indicating a pavement segment that is completely damaged and a rating of 5 indicates a perfect condition. Generally, a PSR rating in the neighborhood of 2.5 indicates that the pavement surface is becoming rough. In this report, a PSR of 3.8 (average of PSR = 5 that indicates the perfect condition and PSR = 2.5 that indicates sign of roughness in the pavement) for both flexible as well as rigid pavement is used as indication of need for corrective maintenance (Ghosn, 2015). Although PSR is usually determined through field inspection with the help of various non-destructive instruments, researchers have developed empirical models that correlate field-inferred PSR values to analytical parameters obtained after applying a stress analysis procedure such as the one inherent to the MEPDG method.

An alternative method to assess the condition of pavements is the International Roughness Index (IRI) that provides ride quality statistics which has been used in conjunction with the MEPDG approach (Rajib B. Mallick, 2009). IRI is usually calculated by measuring the longitudinal profile of the pavement in each wheel path, then entering that profile into a computer algorithm to simulate the response of the suspension of a typical sedan car traveling at 50 mph. The resulting accumulated vertical bounce of the vehicle is the IRI, reported in inches per mile. The higher the IRI number, the rougher is the ride quality. An IRI value less than 60 is indicative of a very smooth ride; a value between 61 and 120 indicates a smooth ride. A fair ride is associated with 121<IRI<171. The ride quality is rough when IRI is between 171 and 220 and the ride is very rough when IRI is greater than 220 (NYSDOT, 2010). Researchers have developed models to correlate PSR and IRI values for a given pavement segment. One such model reported by (Huang, 2004) is given as:

\[
PSR = 5e^{-0.0041IRI}
\]  

Pavements should be designed and maintenance actions must be undertaken at regular intervals to provide adequate ride quality throughout the pavement service lives. To help evaluate whether a pavement segment design will meet the required ride quality, the AASHTO manual (2008) provides a set of empirical equations that were calibrated to correlate the pavement condition expressed in terms of IRI and the mechanistic-empirical parameters that describe the effect of loads and environment on the pavement. These empirical equations were developed in order to provide analytical models to estimate the damage to pavements over time for application in pavement design and planning. Two sets of equations are provided: one set is applicable to flexible pavements, and the other is applicable to rigid pavements.
2.0 MEPDG method for Flexible and Rigid Pavements

2.1 Flexible Pavements

A flexible pavement structure is typically composed of several layers of material including Hot Mix Asphalt (HMA) on the top, Asphalt Treated Permeable Base (ATPB), subbase, selected granular subgrade (SGSG) and the subsurface. Figure 1 below shows a typical section for a flexible pavement in Interstate highways in New York State (might be similar for other states).

![Figure 1: Typical Flexible Pavement section as per NYSDOT (Ghosn, 2015)](image)

2.1.1 MEPDG methods for Flexible pavements

According to the MEPDG approach, the International Roughness Index, IRI, of a given flexible pavement section can be estimated by equation (5-15a) in the MEPDG guide (AASHTO, 2008):

\[ IRI = IRI_0 + 0.015(SF) + 0.400(F_{C_{TOTAL}}) + 0.0080(TC) + 40.0(RD) \]  

(2)

where,

- \( IRI_0 \) = Initial IRI of a given section right after construction or rehabilitation (in/mile)
- \( SF \) = Site factor
- \( F_{C_{TOTAL}} \) = Area of fatigue cracking (combined alligator, longitudinal and reflection cracking in the wheel path) as a percent of total lane area. All load related cracks are combined on an area basis; length of cracks is multiplied by 1 foot to convert crack per unit length into crack per unit area.
- \( TC \) = Length of transverse cracking (including the reflection of transverse cracks in exiting Hot Mix Asphalt, HMA, pavements) (feet/mile)
- \( RD \) = Average rut depth (in)

As shown in Eq. (2), IRI of a given flexible pavement section is calculated using five different input parameters: \( IRI_0 \) which represents initial roughness of a new section or a section immediately after maintenance; \( SF \) which is calculated once every year of the design life and the three remaining
parameters \( (F_{TOTAL}, TC \text{ and } RD) \) which reflect the accumulation of damage obtained from the crossing of each truck. Details on the calculation of each parameter are given below.

### 2.1.1.1 Initial and Terminal IRI

For a flexible pavement section, an initial serviceability index of PSR= 4.2 is usually used (Ghosn, 2015). The design of a pavement is usually performed with the goal of reaching a terminal PSR= 2.5. On the other hand, a PSR= 3.8 would correspond to the pavement showing very early signs of reduction in performance. Hence, PSR=3.8 is used as the threshold for corrective maintenance in this report (consistent to the value used in Ghosn et al (Ghosn, 2015)). Executing corrective maintenance at that stage would ensure that the pavement remains smooth and increase its durability to keep it serviceable even beyond its original design life. Using Eq. (1), the initial and corrective maintenance IRI for flexible pavement sections can be calculated as:

\[
IRI_0 = - \frac{1}{0.0041} \ln \left( \frac{4.2}{5.0} \right) = 42.52
\]

\[
IRI_{corrective \ maintenance} = - \frac{1}{0.0041} \ln \left( \frac{3.8}{5.0} \right) = 67
\]

Note: Initial IRI\(_0\) is defined for a new pavement when it is first opened for traffic. In this report, the same initial IRI\(_0\) is used to describe the pavement condition after each corrective maintenance cycle. This is to say that corrective maintenance performed on a pavement section cannot achieve better condition of the pavement than when it was first opened to the traffic. On the other hand, it assures that the corrective maintenance performed on a pavement section is done so well that the original condition of the pavement is restored.

#### 2.2.1.2 Site Factor (SF)

The Site Factor in Eq. (2) is calculated using Eq. (5-15b) of the AASHTO MEPDG guide (AASHTO, 2008):

\[
SF = Age[0.0203(PI + 1) + 0.007947(Precip + 1) + 0.000636(FI + 1)]
\]

where,

- \( Age \) = Pavement age in years
- \( PI \) = Percent Plasticity index of the soil
- \( Precip \) = Average annual precipitation or rainfall in inches
- \( FI \) = Average annual freezing index in °F days

In this report, the design of flexible pavements is based on a soil resilience modulus equal to 28 MPa that corresponds to fine grained soil with plasticity index in the range of 4%-7%, and hence a value of PI=5% is used as an average value.

The freezing index for New York is taken as 753°F days (see Appendix A for calculation of mean freezing index).

Data on annual precipitation is available through the National Oceanic and Atmospheric Administration (NOAA) website at https://www.ncdc.noaa.gov/cdo-web. As an example, for the Interstate I-88 corridor
between Binghamton and Schenectady, the average annual precipitation is equal to 41.22 in. This is based on the average monthly precipitation for Binghamton and Schenectady as presented in Table 1.

<table>
<thead>
<tr>
<th>Month</th>
<th>Binghamton</th>
<th>Schenectady</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>2.45</td>
<td>2.67</td>
</tr>
<tr>
<td>Feb</td>
<td>2.31</td>
<td>2.14</td>
</tr>
<tr>
<td>Mar</td>
<td>2.99</td>
<td>3.06</td>
</tr>
<tr>
<td>Apr</td>
<td>3.43</td>
<td>3.32</td>
</tr>
<tr>
<td>May</td>
<td>3.57</td>
<td>3.61</td>
</tr>
<tr>
<td>Jun</td>
<td>4.31</td>
<td>3.81</td>
</tr>
<tr>
<td>Jul</td>
<td>3.7</td>
<td>3.07</td>
</tr>
<tr>
<td>Aug</td>
<td>3.45</td>
<td>3.36</td>
</tr>
<tr>
<td>Sept</td>
<td>3.63</td>
<td>3.06</td>
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<tr>
<td>Oct</td>
<td>3.33</td>
<td>3.01</td>
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<tr>
<td>Nov</td>
<td>3.3</td>
<td>2.99</td>
</tr>
<tr>
<td>Dec</td>
<td>2.83</td>
<td>2.71</td>
</tr>
<tr>
<td>Sum</td>
<td>42.58</td>
<td>39.88</td>
</tr>
</tbody>
</table>

Average for 2 cities = 41.22

2.2.1.3 Total Area of Fatigue Cracking (FC\textsubscript{Total})
The total area of fatigue cracking FC\textsubscript{Total} as a percent of the total lane area to be entered in Eq. (2) is calculated as the sum of alligator, longitudinal and reflection cracking. However, reflection cracking is calculated only for rigid pavements with Hot Mix Asphalt (HMA) overlays and is not calculated for flexible sections.

The MEPDG approach provides a method to calculate alligator and longitudinal cracking based on the cyclic fatigue Damage Index (DI). The damage caused by a single crossing of an axle group i, \( DI_i \), is defined as the reciprocal of the number of applications of axle group load i that would cause the total damage of the pavement; or as expressed in Eq. (5-5) of the MEPDG guide (AASHTO, 2008):

\[
DI_i = \frac{1}{N_{f-HMA}}
\]

Where \( N_{f-HMA} \) which is the hypothetical number of crossings of axle group i that would cause pavement failure, obtained from Eq. (5-4a) of MEPDG guide (AASHTO, 2008):

\[
N_{f-HMA} = k_{f1}(C)(C_{H})\beta_{f1}(\varepsilon_{t})^{k_{f2}\beta_{f2}(E_{HMA})^{k_{f3}\beta_{f3}}}
\]

where,

\( N_{f-HMA} \) = allowable number of axle-group load applications for the pavement
\( \varepsilon_{t} \) = Tensile strain at critical pavement location (bottom of asphalt layer for bottom up alligator cracking damage, or top of asphalt for top down fatigue damage) due to the crossing of axle group i. The tensile strain is determined using a mechanistic analysis program or a finite element program. In this study, the mechanistic analysis of flexible pavements is executed using the program KENLAYER (Huang, 2004)

\( E_{HMA} \) = Dynamic modulus of the Hot Mix Asphalt (HMA) layer measured in compression, psi
Global field calibration parameters which, based on NCHRP Report 1-40D (2009), are set at 0.007566, -3.91492 and -1.281 respectively. \( \beta_{f1}, \beta_{f2}, \beta_{f3} \): Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0 (AASHTO, 2008).

In Eq. (5), the constant \( C \) is calculated using Eq. (5-4b) of the MEPDG guide (AASHTO, 2008):

\[
C = 10^M
\]  
(6)

And \( M \) is found using Eq. (5-4c) of the MEPDG guide (AASHTO, 2008):

\[
M = 4.84 \left( \frac{V_{be}}{V_a + V_{be}} - 0.69 \right)
\]  
(7)

where,

\( V_{be} \) = Effective asphalt content by volume, taken as 11% (Raul Velasquez, 2009)

\( V_a \) = Percent air voids in the HMA mixture, taken to be 8.5% (Raul Velasquez, 2009)

\( C_H \) = Thickness correction term, depending on type of cracking in question

For alligator cracking, Eq. (5-4d) of the MEPDG guide (AASHTO, 2008) is used to find the thickness correction term:

\[
C_H = \frac{1}{0.00398 + \frac{1}{0.003602 e^{11.02 - 3.49H_{HMA}}}}
\]  
(8)

Similarly, for longitudinal cracking, Eq. (5-4e) of the MEPDG guide (AASHTO, 2008) is used:

\[
C_H = \frac{1}{0.01 + \frac{1}{14 e^{15.676 - 2.8186 H_{HMA}}}}
\]  
(9)

where,

\( H_{HMA} \) = Total HMA thickness in inches

As mentioned in the introduction chapter of this report, I-88 corridor between Binghamton and Schenectady has been chosen as an area of study for the illustration of the MEPDG method and further sensitivity analysis in this report. Thus, pavement section properties are subjected to requirement of NYSDOT as shown on Figure 1 above. As per the requirements of NYSDOT, all flexible pavement section must have a minimum HMA thickness equal to 6.5 in (165 mm) and a minimum selected granular subgrade (SGSG) thickness of 11.81 in (300 mm). Thicknesses of second and third layer remain constant; 3.94 in (100 mm) for the Asphalt Treated Permeable Base (ATPB) and 11.81 in (300 mm) for the sub base. The thickness of 5th layer is taken to be infinite. During the initial design stage (although MEPDG method is not recommended at this stage as it involves iteration process and becomes very time consuming), if the minimum thicknesses of HMA and SGSG are not sufficient to satisfy the design and serviceability requirements, the HMA thickness is increased while keeping the thickness of SGSG constant at its minimum value (11.81 in or 300 mm). Once the HMA thickness has reached the maximum allowable thickness of 9.84 in (250 mm), the subgrade thickness is increased until the design
requirements are satisfied but cannot be increased beyond 35.43 in (900 mm) as per the limitations of NYSDOT.

The dynamic modulus of HMA \( (E_{HMA}) \) in Eq. (5) can only be determined from extensive laboratory testing as it depends on many factors such as loading frequency, temperature and other environmental factors. For the purpose of this study, estimates of the dynamic modulus of asphalt at various temperatures are extracted from data plotted in Figure 2 that were extracted from the report by (Saeed Yousefdoost, N.A). Figure 2 below compares the normalized measured dynamic modulus of typical HMA at different temperatures. The basis for normalization is the modulus measured at a temperature \( T=70^\circ F \).

![Change in Dynamic Modulus](image)

**Figure 2: Change of Dynamic Modulus of Asphalt with Temperature**

Figure 2 plots the change in dynamic modulus against temperature and fits the data into an exponential equation. Accordingly, the dynamic modulus of HMA can be approximated using following equation:

\[
E_{HMA} = 400,000 \times 14.331e^{-0.038T}
\]

where,

\( E_{HMA} \) = Dynamic modulus of HMA in psi

\( T \) = HMA temperature in Fahrenheit

Eq. (10) is used in this report to relate the dynamic modulus to the temperature of the pavement when a truck crosses the pavement section being analyzed.

The damage index \( DI \) for each axle group crossing the pavement as calculated in Eq. (4) is then summed for all axle groups of one truck and for all trucks that cross the pavement section being analyzed during the service period to obtain the cumulative damage.

\[
DI = \sum_{k=1}^{N_{truck}} \sum_{i=1}^{n_{axle\ group}} (DI)_k
\]
Two types of cumulative damages are calculated: Bottom up alligator damage $\text{DI}_{\text{Bottom}}$ which is obtained from the application of Eq. (4) with the tensile strain calculated at the bottom of HMA layer; and top down fatigue damage $\text{DI}_{\text{Top}}$ which is obtained from Eq. (4) with the tensile strain at the top of the HMA pavement.

### 2.2.1.3.1 Area of Alligator Cracking ($\text{FC}_{\text{Bottom}}$)
The area of alligator cracking as a percent of total lane area is calculated using the following equation (AASHTO, 2008):

$$\text{FC}_{\text{Bottom}} = \frac{1}{60} \left( \frac{C_4}{1 + e^{(C_1C_1^* + C_2C_2^* \log(\text{DI}_{\text{Bottom}} \times 100))}} \right)$$ (12)

where,

- $\text{DI}_{\text{Bottom}} = \text{Cumulative damage index at the bottom of the HMA layer (from Eq. 11 and 4)}$
- $C_1, C_2, C_4 = \text{Transfer function regression constants for alligator cracking; given by AASHTO (2008) as 1.00, 1.00 and 6,000 respectively}$
- $C_1^* = -2G_2^*$ (MEPDG equation 5-6b)
- $C_2^* = -2.40874 - 39.748(1 + H_{\text{HMA}})^{-2.856}$ (MEPDG Eq. 5-6c)
- $H_{\text{HMA}} = \text{thickness of HMA layer in inches}$

### 2.1.1.3.2 Length of Longitudinal fatigue cracks ($\text{FC}_{\text{Top}}$)
Using the cumulative damage index calculated in Eq. (11) for top cracking, the length of longitudinal fatigue cracks is calculated using Eq. (5-8) of the MEPDG guide (AASHTO, 2008):

$$\text{FC}_{\text{Top}} = 10.56 \left( \frac{C_4}{1 + e^{(C_1 - C_2 \log(\text{DI}_{\text{Top}}))}} \right)$$ (13)

where,

- $\text{DI}_{\text{Top}} = \text{Cumulative damage index near the top of the HMA surface (from Eq. 11 and 4)}$
- $C_1, C_2, C_4 = \text{Transfer function regression constants for longitudinal cracks; listed in AASHTO (2008) as 7.00, 3.5 and 1,000 respectively}$

From the structural stand point, flexible pavements are continuously supported slab system where the only possible way to obtain tensile strain at the top of the pavement is when it is subjected to temperature gradient, particularly when temperature at the top surface is higher than the bottom surface. The condition is the worst when there are no truck loads applied on the top surface to overcome the tensile strain at the top but the self-weight of the pavement section is always there and can be accounted for. Ultimately, the analysis of the various pavement sections after applying temperature gradient in the presence of no truck load showed that for the range of pavement profiles applicable to this study (described earlier), no tensile strains are obtained on the top of the flexible pavement. Hence, calculation of longitudinal fatigue cracking is ignored in this report.
2.1.1.3.3 Non-Load Related Cracking - Transverse Cracking (TC)
The crack length induced by each thermal cooling cycle is predicted using Paris law of crack propagation, as given in Eq. (5-11a) of the MEPDG guide (AASHTO, 2008):

$$\Delta C = A(\Delta K)^n$$ \hspace{1cm} (14)

Where,

$$\Delta C = \text{Change in crack length due to a cooling cycle}$$
$$\Delta K = \text{Change in stress intensity factor due to a cooling cycle, and}$$
$$A, n = \text{Fracture parameters for HMA mixtures}$$

The parameters A and n can be obtained from the indirect creep-compliance and tensile strength of HMA in accordance with Eq. (5-11b) and (5-11c) of the MEPDG guide (AASHTO, 2008). Thus,

$$A = 10^{k_t \beta_t (4.389 - 2.52 \log (E_{HMA}\sigma_m^n))}$$ \hspace{1cm} (15)

And

$$n = 0.8 \left[ 1 + \frac{1}{m} \right]$$ \hspace{1cm} (16)

where,

$$k_t = \text{Coefficient determined through global calibration}$$
$$E_{HMA} = \text{HMA dynamic modulus (psi) calculated from Eq. (10)}$$
$$\sigma_m = \text{HMA Mixture tensile strength (psi)}$$
$$m = \text{The m-value derived from the indirect tensile creep compliance curve measured in laboratory tests}$$
$$\beta_t = \text{Local or mixture calibration factor}$$

Additionally, the stress intensity factor K, has been incorporated in the MEPDG through the use of a simplified equation developed from theoretical finite element studies as shown in Eq. (5-11d) of the MEPDG guide (AASHTO, 2008):

$$K = \sigma_{tip}[0.45 + 1.99(C_0)^{0.56}]$$ \hspace{1cm} (17)

where,

$$\sigma_{tip} = \text{Far field stress from pavement response model at depth of crack tip (psi) calculated from mechanical analysis of pavement using KENLAYER}$$
$$C_0 = \text{Current crack length (ft)}$$

Ultimately, the degree of cracking is predicted by the MEPDG using an assumed relationship between the probability distribution of the log of the crack depth to HMA-layer thickness ratio and the percent of cracking using the MEPDG guide Eq. (5-11e);
\[ TC = \beta_{11} N \left[ \frac{1}{\sigma_d} \log \left( \frac{C_d}{H_{HMA}} \right) \right] \] 

where,

- \( TC \) = Observed amount of thermal cracking (ft/mi)
- \( \beta_{11} \) = Regression coefficient determined through global calibration (\( =400 \))
- \( N[z] \) = Standard normal probability distribution evaluated at \( z \)
- \( \sigma_d \) = Standard deviation of the log of the depth of cracks in the pavement (\( =0.769 \) in)
- \( C_d \) = Crack depth (in), calculated after each cycle from Eq. (18) \( C_d = C_0 + \Delta C \)
- \( H_{HMA} \) = Thickness of HMA layers (in)

After investigating the effect of TC on IRI, looking at typical values published in the literature and observing that the effect of TC on IRI is less than 10%, a typical value of 1000 feet/mile is used in this study to calculate the IRI in Eq. (2) for each flexible section for each corrective maintenance cycle based on the work presented by (Raul Velasquez, 2009).

2.1.4 Average Rut Depth (RD)

The average rut depth is the sum of the permanent deformations of the HMA layer and all unbound pavement sub layers. Equations to find each deformation are obtained using the MEPDG models provided in the AASHTO (2008).

2.1.4.1 Plastic deformation in HMA layer

The permanent or plastic vertical deformation in the HMA layer is obtained using Eq. (5-1a) of the MEPDG guide (AASHTO, 2008):

\[ \Delta p(HMA) = \beta_{1r} k_2 \varepsilon_{r(HMA)} H_{HMA} 10^{k_{1r}} n^{k_{2r}} \beta_{2r} T^{k_{3r}} \beta_{3r} \] 

where,

- \( \varepsilon_{r(HMA)} \) = Resilient or elastic strain calculated at the mid-depth of each HMA layer
- \( k_{1r}, k_{2r}, k_{3r} \) = Global field calibration parameters which based on NCHRP 1-40D are given as -3.35412, 0.4791 and 1.5605 respectively
- \( \beta_{1r}, \beta_{2r}, \beta_{3r} \): Local or mixture field calibration constants; for the global calibration these constants were all set to 1.0 as per AASHTO MEPDG (2008)
- \( T \) = Mix or pavement temperature, °F
- \( n \) = Number of axle load group repetitions, use \( n=1 \) since one axle group is analyzed at a time
- \( k_z \) = depth confinement factor
- \( H_{HMA} \) = Total HMA thickness in inch

The depth confinement factor \( (k_z) \) is calculated with the MEPDG Eq. (5-1b)

\[ k_z = (C_1 + C_2 D)0.328192^D \] 

The constants \( C_1 \) and \( C_2 \) are found according to AASHTO (2008) the MEPDG Eq. (5-1c) and (5-1d) from:

\[ C_1 = -0.1039(H_{HMA})^2 + 2.4868(H_{HMA}) - 17.342 \]
\[ C_2 = 0.0172(H_{HMA})^2 - 1.7331(H_{HMA}) + 27.428 \]  

where,  

\[ D = \text{Depth to the middle of the thickness} = H_{HMA}/2 \] (Harold L. Von Quintus, 2012)

### 2.1.1.4.2 Plastic deformation in unbound layers

Plastic deformations of the unbound layers are essentially the plastic deformation of the soil supporting the HMA. They are calculated for each layer using Eq. (5-2a) from the MEPDG guide (AASHTO, 2008):

\[
\Delta_{p\text{(soil)}} = \beta_{s1} k_{s1} \varepsilon_r h_{\text{soil}} \left( \frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left( \frac{\varepsilon_r}{n} \right)^p}
\]

where,

\[ \beta_{s1} = \text{Calibration constant for the rutting in the unbound layers; set } = 1.0 \text{ (AASHTO, 2008)} \]

\[ k_{s1} = \text{Global calibration coefficient set at 1.673 for granular base material and 1.350 for fine-grained materials of the subgrade.} \]

\[ h_{\text{soil}} = \text{Thickness of unbound soil layer under the HMA, it varies with pavement section depending on section design of the section. Based on NYSDOT pavement design manual for example, } h_{\text{soil}} \text{ is in the range of 300 mm (11.81 inches) to 900 mm (35.43 inches)} \]

\[ \varepsilon_r = \text{Average vertical resilient or elastic strain calculated at the soil layer from mechanistic analysis of pavement section determined through KENLAYER program in this report} \]

\[ n = \text{number of cycles of the axle group with vertical elastic strain } \varepsilon_r \]

The parameters \( \beta, \rho, \varepsilon_0, \varepsilon_r \) are material properties of the soil as explained below.

From the MEPDG guide Eq. (5-2b):

\[
\log(\beta) = -0.6119 - 0.017638(W_c)
\]

where,

\[ W_c = \text{Water content in soil, chosen to be 5\%. A sensitivity analysis showed that the effect of } W_c \text{ on the results of Eq. (24) is negligible} \]

And, from the MEPDG guide Eq. (5-2c):

\[
\rho = 10^9 \left( \frac{c_0}{1-(10^9)^\beta} \right)^{\frac{1}{\beta}}
\]

where,

\[ c_0 = \text{Ln}(0.0075) \]

\[ \varepsilon_0 = \text{Intercept determined from the laboratory repeated load permanent deformation tests, in/in} \]

\[ \varepsilon_r = \text{Resilient strain imposed in the laboratory test to obtain material properties} \]
For the purposes of this study, typical values for both the intercept and resilient laboratory strain were determined from available sources. Specifically, a value of $\varepsilon_0 = 451.7 \times 10^{-6}$ is extracted from NCHRP REPORT 547. (Witczak, 2005).

The ratio $\frac{\varepsilon_0}{\varepsilon_r}$ can be calculated using the following equation: (US Department of Transportation, Federal Highway Administration, 2011)

$$
Log \left( \frac{\varepsilon_0}{\varepsilon_r} \right) = 0.74168 + 0.08109 W_c - 0.000012157 M_R
$$

where,

$M_R$ = Resilience Modulus, taken to be 39,000 psi

Preliminary investigations of the results of Eq. (23) for the range of pavement layer thicknesses applicable to this study indicated that the permanent deformation of the soil layers is small compared to the HMA deformations. Thus, Eq. (23) was not implemented in the analysis performed in other parts of this report.

2.1.1.5 KENLAYER Program to calculate stresses and strains

As observed above, important components of the MEPDG approach for assessing the damage of flexible pavements is the determination of the strains at different critical locations of a pavement section due to the crossing of the axle groups of heavy vehicles. Specifically, the MEPDG approach for the analysis of the flexible pavements requires the following strains:

- Horizontal tensile strain at the top of HMA layer to obtain fatigue cracking.
- Horizontal tensile strain at bottom of HMA layer to obtain alligator cracking
- Vertical strain at mid-height of each layer to obtain rutting

The evaluation of these strains requires a mechanistic analysis or a finite element program. In this study, the strains induced by each axle load group of each crossing truck are calculated using the program KENPAVE developed by Prof. Huang (2004). The program KENPAVE has two components: KENLAYER for flexible pavements and KENSLAB for rigid pavements.

2.1.1.6 KENLAYER Input Variables

In order to analyze flexible pavements using the MEPDG method, a number of input parameters must be assembled. Of those parameters, some parameters are variables that change based on the location where the analysis takes place, time of the year of the analysis and section being analyzed. The four major variables in MEPDG method for flexible pavements are:

1. **Axle group and Axle weight:** Depending upon the axle-weight and axle-spacing of the tires in the truck, truck loads could be categorized into 4 different load groups based on their axle spacing. It is noted that two different axle groups with the same axle weight may produce two totally different responses. The four load groups considered in as input for KENLAYER analysis are:
   - Single-axle-single-tire (steering axle)
   - Single-axle-dual-tire
   - Tandem axles
   - Tridem axles
2. **Temperature:** Since climate conditions are well incorporated into MEPDG method, pavement responses under the same loading and pavement conditions but different weather conditions (most importantly temperature) are different and hence, temperature is the most important variable that should be considered in the KENLAYER analysis.

3. **Hot Mix Asphalt (HMA) and Subgrade thickness:** The MEPDG method which is partly based on a mechanistic analysis as described before requires the determination of pavement responses (stresses and strains) which depend heavily on the thickness of each layer with the pavement section. Since I-88 corridor is being used for analysis in this report and hence subjected to the requirements of NYSDOT pavement design guides. Thus as per previous discussion, the HMA thickness could range between 6.5 inches (165 mm) to 9.84 inches (250 mm) and subgrade thickness could range between 11.81 inches (300 mm) to 35.43 inches (900 mm). Thicknesses of second and third layer remains constant: 3.94 in (100 mm) for the asphalt treated permeable base (ATPB) and 11.81 in (300 mm) for the sub base and thickness of 5th layer is taken to be infinite.

### 2.1.1.7 KENLAYER Input Constants

In addition to the input variables identified in the preceding section, the analysis of each flexible pavement section requires the input of several material and geometric characteristics. Table 2 lists all the input parameters and the values that have been used for each during the KENLAYER analysis of the flexible pavements performed in this study.

For the purpose of analyzing the flexible pavements using KENLAYER, several other input parameters must be specified. Although these parameters will remain same for the purpose of this report, some of the critical parameters will be varied as indicated and differences in the final outcome (stress, strains and ultimately estimated number of corrective maintenance cycles) will be presented in the next chapter of this report to study the sensitivity of the corrective maintenance cycles to each of the input parameters.

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Remarks/ Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Type (MATL)</td>
<td>3</td>
<td>1 = Linear&lt;br&gt;2 = Nonlinear&lt;br&gt;3 = Viscoelastic&lt;br&gt;4 = Combined&lt;br&gt;MATL=3 because Hot Mix Asphalt is a viscoelastic material</td>
</tr>
<tr>
<td>Number of periods per year (NPY)</td>
<td>1</td>
<td>NPY=1 since each truck is analyzed only once</td>
</tr>
<tr>
<td>Number of Load groups (NLG)</td>
<td>1</td>
<td>Since we are analyzing one load group at a time, NLG=1. Type of load group entered separately see below</td>
</tr>
<tr>
<td>Tolerance for Numerical integration (DEL)</td>
<td>0.001</td>
<td>Lower values would unnecessarily increase program running time</td>
</tr>
<tr>
<td>Number of layers (NL)</td>
<td>5</td>
<td>Total number of layers; will be always 5 but thickness might vary</td>
</tr>
<tr>
<td>Number of Z coordinates for analysis (NZ)</td>
<td>6</td>
<td>&quot;Z&quot; axis is vertical axis. Strain is calculated at top and bottom of HMA layer, mid-height of HMA, base, sub base and subgrade. NOTE: Only vertical strain at the mid-height and radial strain at the bottom of the HMA used in IRI calculation however.</td>
</tr>
<tr>
<td>Maximum cycles of numerical integration (ICL)</td>
<td>80</td>
<td>Higher values would unnecessarily increase program running time</td>
</tr>
</tbody>
</table>
| Type of responses (**NSTD**) | 9 | 1=displacement only  
5=plus stresses  
9=plus strains |
|-----------------------------|---|------------------------------------------------------------------|
| Systems of units (**NUNIT**) | 0 | 0=US units  
1=SI units |
| Thickness for each layer (**TH**) | Varies | Depends on section design. Thicknesses of layers 1 and 4 vary. Thicknesses of layers 2 and 3 remain constant. Thickness of layer 5 (foundation soil) is not needed since default is unbounded thickness. |
| Poisson's ratio for each layer (**PR**) | 0.35  
0.35  
0.35  
0.35  
0.35 | Use same as Asphalt Treated Base, values based on:  
Page 186 (Teruhisa Masada, 2004)  
Page 186 (Teruhisa Masada, 2004)  
Page 186 (Teruhisa Masada, 2004)  
Page 204 (Teruhisa Masada, 2004) |
| All layer interfaces bonded (**NBOND**) | 0 | 1=Yes  
0 = No |
| Elastic Modulus of each layer (**E**) | 0  
Varies  
39050 psi  
39050 psi  
34000 psi | Assign 0 for viscoelastic material  
Equation (12) of this report  
Page 204 (Teruhisa Masada, 2004)  
Page 199 (Teruhisa Masada, 2004)  
Page 211 (Teruhisa Masada, 2004)  
Page 201 (Teruhisa Masada, 2004) |
| Type of Load (**LOAD**) | Varies | 0=Single-axle-single-tire  
1=Single-axle-dual-tire  
2=Tandem-axle  
3=Tridem-axle |
| Contact radius (**CR**) | 6 (inches) | Radius of tire footprint |
| Contact Pressure (**CP**) | Varies | Axle Group Weight/Number of tires in group/area of tire |
| Number of radial coordinates to be analyzed (**NR**) | 1 | Always analyzing point directly underneath the load |
| Radial Distance (**RC**) | 0 | Always analyzing point directly underneath the load |
| Duration of the load (**DUR**) | 0.1 | DUR=0.1 for moving load (Huang, 2004) |
| Number of viscoelastic layer (**NVL**) | 1 | Layer 1 of HMA is the only viscoelastic layer |
| Number of time duration for creep compliance (**NTYME**) | 11 | Recommendation from (Huang, 2004) |
| Times at which creep compliance are specified (**TYME**) | See Table 4 | |
| Creep compliance of viscoelastic material at specified reference temperature (**CREEP**) | See Table 4 | |
| Reference temperature of each viscoelastic layer at which creep compliance have been specified (**TEMPREF**) | 70°F | (Huang, 2004) |
| Temperature shift coefficient (**BETA**) | 0.113 | (Huang, 2004) |
| Pavement temperature of | Varies | Taken as ambient temperature for the month truck crosses |
Pavement temperature throughout its depth is assumed to be same as the ambient temperature for simplicity. Furthermore, the average monthly temperature is used for each month. Table 3 below shows the average monthly temperatures used for the analysis.

<table>
<thead>
<tr>
<th>Month</th>
<th>Average temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>22</td>
</tr>
<tr>
<td>February</td>
<td>25</td>
</tr>
<tr>
<td>March</td>
<td>33</td>
</tr>
<tr>
<td>April</td>
<td>49</td>
</tr>
<tr>
<td>May</td>
<td>56</td>
</tr>
<tr>
<td>June</td>
<td>65</td>
</tr>
<tr>
<td>July</td>
<td>69.5</td>
</tr>
<tr>
<td>August</td>
<td>68</td>
</tr>
<tr>
<td>September</td>
<td>60.5</td>
</tr>
<tr>
<td>October</td>
<td>49</td>
</tr>
<tr>
<td>November</td>
<td>39</td>
</tr>
<tr>
<td>December</td>
<td>26</td>
</tr>
</tbody>
</table>

In order to perform sensitivity of the number of corrective maintenance cycle to the pavement temperature, 2 alternative monthly temperature tables were developed based on the first one; first by increasing the average monthly temperature given in Table 3 by 1 °F and second one by decreasing the
average monthly temperature in Table 3 by 1°F. Table 4 and Table 5 below are alternative monthly average temperatures used for the sensitivity analysis of flexible pavement.

Table 4: Monthly Temperature for I-88 Corridor with an increase of 1°F on average monthly temperature

<table>
<thead>
<tr>
<th>Month</th>
<th>Average temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>23</td>
</tr>
<tr>
<td>February</td>
<td>26</td>
</tr>
<tr>
<td>March</td>
<td>34</td>
</tr>
<tr>
<td>April</td>
<td>50</td>
</tr>
<tr>
<td>May</td>
<td>57</td>
</tr>
<tr>
<td>June</td>
<td>66</td>
</tr>
<tr>
<td>July</td>
<td>70.5</td>
</tr>
<tr>
<td>August</td>
<td>69</td>
</tr>
<tr>
<td>September</td>
<td>61.5</td>
</tr>
<tr>
<td>October</td>
<td>50</td>
</tr>
<tr>
<td>November</td>
<td>40</td>
</tr>
<tr>
<td>December</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 5: Monthly Temperature for I-88 Corridor with a decrease of 1°F on average monthly temperature

<table>
<thead>
<tr>
<th>Month</th>
<th>Average temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>21</td>
</tr>
<tr>
<td>February</td>
<td>24</td>
</tr>
<tr>
<td>March</td>
<td>32</td>
</tr>
<tr>
<td>April</td>
<td>48</td>
</tr>
<tr>
<td>May</td>
<td>55</td>
</tr>
<tr>
<td>June</td>
<td>64</td>
</tr>
<tr>
<td>July</td>
<td>68.5</td>
</tr>
<tr>
<td>August</td>
<td>67</td>
</tr>
<tr>
<td>September</td>
<td>59.5</td>
</tr>
<tr>
<td>October</td>
<td>48</td>
</tr>
<tr>
<td>November</td>
<td>38</td>
</tr>
<tr>
<td>December</td>
<td>25</td>
</tr>
</tbody>
</table>

Note that the analysis is performed for a 50-year period. Although the average monthly temperature for the same month is subject to change in different years (typically increasing due to the effect of global warming), for the sake of simplicity, the change in average monthly temperature over the years was not accounted for in this report. This assumption remains the same for the rigid pavements as well.
Creep compliance constants are used to define the viscoelastic material properties of the HMA. The creep compliance constants and respective times for the reference are presented in Table 6 below based on the data provided by Huang (2004).

Table 6: Creep compliances at given times for the reference temperature (Huang, 2004)

<table>
<thead>
<tr>
<th>Time (Seconds)</th>
<th>Creep Compliance</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>2.50E-06</td>
</tr>
<tr>
<td>0.003</td>
<td>3.51E-06</td>
</tr>
<tr>
<td>0.01</td>
<td>5.81E-06</td>
</tr>
<tr>
<td>0.03</td>
<td>9.80E-06</td>
</tr>
<tr>
<td>0.1</td>
<td>1.69E-05</td>
</tr>
<tr>
<td>0.3</td>
<td>2.70E-05</td>
</tr>
<tr>
<td>1</td>
<td>4.19E-05</td>
</tr>
<tr>
<td>3</td>
<td>5.81E-05</td>
</tr>
<tr>
<td>10</td>
<td>8.11E-05</td>
</tr>
<tr>
<td>30</td>
<td>1.08E-04</td>
</tr>
<tr>
<td>100</td>
<td>1.282E-05</td>
</tr>
</tbody>
</table>

By definition of the creep compliance constants as well as per the Table 6, the first value of the creep compliance corresponds to the time of t = 0.001 seconds. For simplicity, it can be assumed that the first value corresponds to the time of t = 0 seconds and thus, particularly at time t = 0 seconds, creep compliance constant is the reciprocal of the elastic modulus of HMA at t = 0 seconds since it is a time dependent factor. Hence, based on the first value of the creep compliance on Table 6, corresponding initial elastic modulus of asphalt can be back calculated as

\[
E_0 = \frac{1}{2.5 \times 10^{-6}} = 400,000 \text{ psi}.
\]

Thus, it can be concluded that creep compliance constants on Table 6 correspond to HMA with initial elastic modulus of 400,000 psi.

Alternatively, to study the sensitivity of the number of corrective maintenance cycles to the creep compliance, different set of creep compliance constants are also used. To obtain alternative set of creep compliance constant, creep compliance constants in Table 6 is modified. Since the creep compliance constants in Table 6 have been concluded to be corresponding to the initial elastic modulus of 400,000 psi, alternative creep compliance for \(E_0 = 500,000 \text{ psi} \) is obtained. Alternative sets of creep compliance constants can be obtained by multiplying each value in Table 6 by the ratio of two initial elastic moduli, i.e. \( \frac{400,000}{500,000} = 0.8 \). Thus, new sets of creep compliance constants is given in Table 7 below:
Table 7: Modified creep compliance for initial elastic modulus of 500,000 psi

<table>
<thead>
<tr>
<th>Time (Seconds)</th>
<th>Creep Compliance</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>2.00E-06</td>
</tr>
<tr>
<td>0.003</td>
<td>2.81E-06</td>
</tr>
<tr>
<td>0.01</td>
<td>4.65E-06</td>
</tr>
<tr>
<td>0.03</td>
<td>7.84E-06</td>
</tr>
<tr>
<td>0.1</td>
<td>1.35E-05</td>
</tr>
<tr>
<td>0.3</td>
<td>2.16E-05</td>
</tr>
<tr>
<td>1</td>
<td>3.35E-05</td>
</tr>
<tr>
<td>3</td>
<td>4.65E-05</td>
</tr>
<tr>
<td>10</td>
<td>6.49E-05</td>
</tr>
<tr>
<td>30</td>
<td>8.65E-05</td>
</tr>
<tr>
<td>100</td>
<td>1.03E-04</td>
</tr>
</tbody>
</table>

2.2 Rigid Pavements

A rigid pavement structure is composed of a hydraulic cement concrete surface course, underlying base course and often a sub-base course is also used. NYS rigid pavements are typically jointed plain concrete pavement (JPCP). Figure 3 below shows typical rigid pavements in NYS interstate highways.

As was the case with the flexible pavements in the previous section, rigid pavement sections of I-88 corridor between Binghamton and Schenectady will be used in this report for the purpose of illustration. Thus, pavement sections and dimensions used in this study will be limited by NYSDOT pavement design manual. For a typical 2-lane highway where rigid pavement is used, inside lane slab is typically 3.6 m (12-ft) wide and 5.5 m (18-ft) long while outside lanes are normally wider at 4.2 m (14-ft) that includes 0.6m (2-ft) of shoulder. Realistically, rigid pavements are continuous arrangement of each of the slab pieces with the dimension given above and connected at each joint with dowels to achieve continuity between each pieces of slab in each direction. However, for the purpose of studying distress and damages
in the pavement section, it is customary to choose six-slab model (3 in the direction of the traffic and 2 in the transverse direction) as shown in Figure 4 below:

2.2.1 MEPDG Method for Rigid pavements (JPCP)

Following an approach similar to the one described above for the flexible pavements, the MEPDG method estimates the damage to rigid pavements using an empirically derived equation for IRI. Specifically, for Jointed Plain Concrete Pavements (JPCP), IRI can be estimated using Eq. (5-32a) in the MEPDG guide (AASHTO, 2008)

\[
IRI = IRI_0 + C1 \times CRK + C2 \times SPALL + C3 \times TFAULT + C4 \times SF
\]  

Figure 4: Six-Slab model for rigid pavement analysis

where,

- \( IRI \) = Predicted IRI, inch/mile
- \( IRI_0 \) = Initial smoothness measured as IRI, inches/mile
- \( CRK \) = Percent slabs with transverse cracks (all severities)
- \( SPALL \) = Percentage of joints with spalling (medium and high severities)
- \( TFAULT \) = Total joint faulting accumulated per mile, in inches
- \( SF \) = Site factor
- \( C1 = 0.8203 \)
- \( C2 = 0.4417 \)
- \( C3 = 0.4929 \)
- \( C4 = 25.24 \)

The IRI is calculated for JPCP rigid pavement sections using five different input parameters as can be seen in Eq. (27) above. \( IRI_0 \) is the estimate of roughness of the section when the section is newly built or immediately after maintenance is performed, \( SF \) and \( SPALL \) are related to the age of the pavement and
thus, are calculated once a year. The remaining parameters, $CRK$ and $TFAULT$ are the accumulation of two types of damage obtained from the crossing of each truck. Detail explanations for the equations to obtain each parameter are given below.

### 2.2.1.1 Initial and Terminal IRI
For a rigid pavement section, an initial serviceability index $PSR= 4.5$ is usually used (Ghosn, 2015). The design of a pavement is usually performed with the goal of not falling below a terminal $PSR=2.5$. As was the case with flexible pavement, executing corrective maintenance when the PSR reaches 3.8 will ensure a smooth ride and increases the durability of the pavement beyond its originally specified design life. Using Eq. (1), corresponding initial and terminal IRI for rigid pavement sections can be calculated as:

\[
IRI_0 = -\frac{1}{0.0041} \ln \left(\frac{4.5}{5.0}\right) = 25.69
\]

\[
IRI_{Corrective\ Maintenance} = -\frac{1}{0.0041} \ln \left(\frac{3.8}{5.0}\right) = 67
\]

As was the case for flexible pavements, the same initial $IRI_0$ defined for a new pavement when it is first opened for traffic is also used after each corrective maintenance cycle. It is also noted that the IRI criterion for corrective maintenance ($IRI_{Corrective\ Maintenance} = 67$) corresponds to the upper limit for pavement conditions associated with very smooth rides as defined in NYSDOT (2010).

### 2.2.1.2 Site Factor (SF)
The Site Factor in Eq. (27) can be calculated using equation (5-32b) in the AASHTO MEPDG guide (AASHTO, 2008):

\[
SF = AGE(1 + 0.5556 * FI)(1 + P_{200}) * 10^{-6}
\]  

(28)

where,

$AGE$ = Pavement age in years  
$FI$ = Freezing index, °F days and  
$P_{200}$ = Percent subgrade material passing No. 200 sieve

In this report, $P_{200}$ was chosen to be 10% or 0.1 (Raul Velasquez, 2009).

The freezing index for New York State was taken as 753°F days using data for one site in central New York assumed to give an average representation for the entire state (see Appendix A for the calculation of mean freezing index).

### 2.2.1.3 SPALL
The percentage of joints with spalling can be calculated using Eq. (5-33a) in the AASHTO MEPDG guide (AASHTO, 2008):

\[
SPALL = \left(\frac{AGE}{AGE+0.01}\right) \left(\frac{100}{1+1.005^{-12*AGE+SCF}}\right)
\]

(29)

where,
The scaling factor, SCF, in Eq. (29) above is calculated using Eq. (5-33b) in the MEPDG guide (AASHTO, 2008):

\[
SCF = -1400 + 350 * AC_{PCC} * (0.5 + PREFORM) + 3.5 f'c * 0.4
- 0.2(FT_{cycles} * AGE) + 43 H_{PCC} - 536 WC_{PCC}
\]  

(30)

where,
- \(AC_{PCC}\) = Portland Cement Concrete (PCC) air content in percent
- \(AGE\) = time since construction in years
- \(PREFORM\) = 1 if preformed sealant is present; 0 if not
- \(f'c\) = PCC compressive strength in psi
- \(FT_{cycles}\) = average annual number of freeze-thaw cycles
- \(H_{PCC}\) = PCC slab thickness in inches
- \(WC_{PCC}\) = PCC water/cement ratio

The average \(AC_{PCC}\) is 6 percent (AASHTO, 2007). The PCC compressive strength specified in NYSDOT manual is 3,000 psi. The average number of freeze thaw cycles in New York is 39 cycles (WorldNow, 2014). The PCC water to cement ratio is typically 0.45 (AASHTO, 2007). For the parameter \(PREFORM\), a value of 1 is used since sealant is always present between slabs.

### 2.2.1.4 Faulting

Faulting at joints is the difference in elevation across a joint between slabs. It is caused in part by a buildup of loose materials due to pumping or a depression under the slabs. The AASHTO MEPDG guide Eq. (5-20a) measures mean joint faulting at the end of month \(m\), \(Fa\text{ult}_m\) from the equation:

\[
Fa\text{ult}_m = \sum_{i=1}^{m} \Delta Fa\text{ult}_i
\]  

(31)

Where,
- \(Fa\text{ult}_m\) = mean joint faulting at the end of month \(m\), in inches
- \(\Delta Fa\text{ult}_i\) = incremental change (monthly) in mean transverse joint faulting during month \(i\), inches

The Differential Energy (DE) associated with the displacements at the corners of adjacent slabs at the crossing of an axle load is a primary factor for evaluating faulting. DE is calculated using the AASHTO MEPDG guide Eq. (5-23a):

\[
DE = \frac{K}{2} (D_L^2 - D_U^2)
\]  

(32)

Where,
- \(K\) = subgrade reaction
- \(D_L\) = deflection of corner of loaded slab
- \(D_U\) = deflection of corner of unloaded slab

The incremental fault is obtained using Eq. (5-20b) of the AASHTO MEPDG guide:
\[ \Delta \text{Fault}_i = C_{34}(\text{Faultmax}_{i-1} - \text{Fault}_{i-1})^2(DE_i) \]  

(33)

Where,

\[ \Delta \text{Fault}_i \] = incremental change (monthly) in mean transverse joint faulting during month \( i \), in inches

\[ DE_i \] = differential density of energy of subgrade deformation accumulated during month \( i \)

\[ \text{Fault}_{i-1} \] = mean joint faulting at the end of the previous month (= month \( i-1 \)), in inches

\[ \text{Faultmax}_i \] = maximum mean transverse joint faulting for month \( i \), in inches which can be calculated using AASHTO MEPDG equation (5-20c) as given below

\[ \text{Faultmax}_i = \text{Faultmax}_0 + C_7 \left[ \sum_{j=1}^{n} DE_j \log(1 + 5.0^{EROD C_8}) \right] \]  

(34)

Where,

\[ \text{Faultmax}_i \] = maximum mean transverse joint faulting for month \( i \), inches

\[ \text{Faultmax}_0 \] = initial maximum mean transverse joint faulting, inches

\[ DE_j \] = maximum differential density of energy of subgrade deformation during month \( j \)

\[ EROD \] = base/subbase erodibility factor (index of 3)

Initial maximum mean transverse joint faulting in Eq. (34) above is given by the AASHTO MEPDG Eq. (5-20d):

\[ \text{Faultmax}_0 = \delta_{\text{curling}} C_{12} \left[ \log(1 + 5.0^{EROD C_5}) \log \left( \frac{\text{WetDays}(P_{200})}{P_s} \right) \right]^{C_6} \]  

(35)

where,

\[ EROD \] = base/subbase erodibility factor (index of 3)

\[ \delta_{\text{curling}} \] = maximum mean monthly slab corner upward deflection of PCC due to temperature curling and moisture warping

\[ P_s \] = overburden on subgrade, psi where \( P_s = \sum H \times \gamma \) with \( H \) being the thickness of each layer supported by the pavement subgrade and \( \gamma \) is the specific weight

\[ P_{200} \] = percent subgrade material passing No. 200 sieve (10% typical for New York State)

\[ \text{WetDays} \] = average annual number of wet days (greater than 0.1 inch rainfall) (121 days)

The global calibration constants recommended by AASHTO MEPDG guide (AASHTO, 2008) \( C_1, C_2, C_3, C_4, C_5, C_6, C_7, C_{12}, C_{34} \) are listed in Table 8. AASHTO recommends that these constants be calibrated based on local and state-specific calibration process. But, lacking such data, the default values given by AASHTO (2004) are used in this report.
Table 8: Calibration constants for faulting equations

<table>
<thead>
<tr>
<th>Global Calibration Constant</th>
<th>Value provided in AASHTO MEPDG manual</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>1.29</td>
</tr>
<tr>
<td>C2</td>
<td>1.1</td>
</tr>
<tr>
<td>C3</td>
<td>0.001725</td>
</tr>
<tr>
<td>C4</td>
<td>0.0008</td>
</tr>
<tr>
<td>C5</td>
<td>250</td>
</tr>
<tr>
<td>C6</td>
<td>0.4</td>
</tr>
<tr>
<td>C7</td>
<td>1.2</td>
</tr>
<tr>
<td>C12</td>
<td>See equation (39)</td>
</tr>
<tr>
<td>C34</td>
<td>See equation (40)</td>
</tr>
</tbody>
</table>

Two constants, $C_{12}$ and $C_{34}$ in Table 8 are calculated using the AASHTO MEPDG Eqs. (5-20e) and (5-20f) respectively:

$$C_{12} = C_1 + C_2 \cdot FR^{0.25} \quad (36)$$

$$C_{34} = C_3 + C_4 \cdot FR^{0.25} \quad (37)$$

where,

$$FR = \text{Base freezing index defined as percentage of time the top base temperature is below freezing temperature.}$$

The environmental impact on faulting is accounted through delta curling, which is the upward corner deflection due to moisture and temperature. KENSLAB is used to find curling in this report.

2.2.1.5 CRK

The rigid slab may exhibit bottom up and/or top down cracking due to the tensile stresses incurred due to the passing of heavy trucks. The development of such cracks is due to cyclic fatigue from the application of large numbers of loading cycles. The fatigue Damage Index, $Dl_i$, due to a single tensile stress cycle, $i$, is obtained using Eq. (5-17a) of the AASHTO MEPDG guide (AASHTO, 2008):

$$Dl_i = \frac{1}{N_{f \text{-conc}}} \quad (38)$$

where $N_{f \text{-conc}}$ which is the hypothetical number of stress cycles having the same intensity, $i$, that would cause pavement failure in concrete rigid pavements. It is calculated using Eq. (5-17b) of the AASHTO MEPDG guide (AASHTO, 2008):

$$\log(N_{f \text{-conc}}) = C_1 \left(\frac{MR}{\sigma}\right)^{C_2} \quad (39)$$

And,
\( \sigma = \text{applied stress in psi} \)
\( C_1 = 2.0 \)
\( C_2 = 1.22 \)

\[ MR = \text{Modulus of rupture of concrete} = 7.5\sqrt{I_e} = 7.5\sqrt{3000} = 410.79 \text{ psi} \]

The damage index \( D_{II} \) for each stress cycle is then summed for all stress cycles caused by one truck and for all trucks that cross the pavement section being analyzed during the service period to obtain the cumulative damage.

\[
D_I = \sum_{k=1}^{N_{\text{truck}}} \sum_{i=1}^{n_{\text{stress cycles for truck } k}} (D_{II})_k
\]  

(40)

The contribution of \( D_I \) for cracking is found using Eq. (5-16) of the AASHTO MEPDG guide (AASHTO, 2008):

\[
CRK = \frac{1}{1 + 0.6 \frac{D_I}{MR}^{2.06}}
\]

(41)

Two types of cumulative damages are calculated: bottom up and top down cracking. The contributions to cracking from top-down and bottom up cracking can be calculated as a percent using Eq. (5-18) of the AASHTO MEPDG guide (AASHTO, 2008) as shown below:

\[
TCRACK = (CRK_{\text{bottom-up}} + CRK_{\text{top-down}} - CRK_{\text{bottom-up}} \times CRK_{\text{top-down}}) \times 100
\]

(42)

### 2.2.1.6 KENSLAB Program to calculate stresses and deflections

As observed from the above explanations, an important component of the MEPDG approach for assessing the damage of rigid pavements is the determination of the stresses and deflections at different critical locations of a pavement due to the crossing of heavy vehicles and the curling due to temperature changes. Specifically, the MEPDG approach for analysis of rigid pavements requires the following stresses:

- Horizontal tensile strain at the top of rigid slabs to obtain top down fatigue cracking.
- Horizontal tensile strain at bottom of rigid slabs to obtain bottom up cracking
- Vertical deflections at corners of adjacent slabs to obtain differential energy
- Curling of slabs due to temperature differentials

The evaluation of these stresses and deflections requires a mechanistic analysis or a finite element program. As mentioned previously, in this study, the stresses and deflections induced by each axle load group of a crossing truck are calculated using the program KENPAVE developed by Huang (2004). For rigid pavement, KENSLAB component of KENPAVE is used. Because rigid pavements are treated as linear elastic materials, the principle of superposition has been applied in this report. Therefore, the analysis of stresses due to all truck crossings can be performed simply by:

- Analyzing single and dual tire axles having unit load intensities
- Developing the stress (or strain) history at a particular point of interest along the slab due to the crossing of each truck using the principle of superposition after scaling up the appropriate influence lines to account for actual axle weight.

An example set of influence lines used to find the stress at the top of the slab at mid-length of slab 2 on Figure 4 are provided in Figure 5. Figure 6 shows how the influence lines can be superimposed to provide
the time history of the stress at the top of the slab at the mid-length due to the crossing of a semi-trailer truck with split rear tandem. The plot in Figure 6 shows 5 compressive stress cycles at the top of the slab and 4 tensile stress cycles where the crossing of the zero stress line defines a cycle. Thus, top down cracking is evaluated using the four tensile stress cycles because compressive stresses do not cause fatigue fracture. It is noted that due to symmetry in unbonded rigid slab sections, the same stress history shown in Figure 6 is valid with opposite signs at the bottom of the slab. Thus, bottom up cracking will be evaluated using 5 stress cycles.

![Influence lines stress at the mid-slab left wheel for a 1 kip axle load](image)

Figure 5: Example influence line for maximum stress at top of the slab measured at the mid-length

![Stress at the mid-slab under left wheel-path from the truck passing](image)

Figure 6: Example Influence line for maximum stress at top of the slab measured at mid-length due to truck passing

**2.2.1.7 KENSLAB Input Variables**

Because it is impossible to analyze the millions of trucks that are expected to travel over each possible pavement section, this study uses KENSLAB to develop influence lines for each critical point within a typical pavement section induced by different types of axle group loads. Specifically, the stress influence lines are assembled for single-axle-single-tire (steering axle) and single-axle-dual-tire (all other axles) for
different slab thicknesses. Also, deflections at the corners of adjacent slabs were obtained from KENSLAB for unit loads of the different types of axle groups including single-axle-single-tire, single-axle-dual-tire, tandem axles and tridem axles.

2.2.1.8 KENSLAB Input Constants

The calculations of the influence lines as well as the deflections for each typical rigid pavement section are executed using the program KENSLAB which requires the input of several material and geometric characteristics. Table 9 below summarizes all the input parameters and the values that have been used for each KENSLAB analysis.

### Table 9: Summary of KENSLAB Input parameters

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
<th>Remarks/ Reference</th>
</tr>
</thead>
</table>
| Type of Foundation                       | 1     | 0=Liquid or Winkler foundation  
1=Solid foundation  
2=Burmister's foundation |
| Damage Analysis                          | 0     | 0=No damage analysis  
1=Damage analysis based on PCA fatigue cracking criteria  
2=Damage analysis based on user specified fatigue coefficients  
(We are using MEPDG method, we only need stresses at certain critical locations, we perform our own damage analysis) |
<p>| Number of period per year                | 1     | Since each load group is analyzed once, number of period is always 1              |
| Number of Load group                     | 1     | Since we are analyzing one load group at a time, number of load group is 1.      |
| Total number of slab                     | 6     | A six-slab model is chosen for analysis to account for interaction of loaded slab with adjacent slabs (see Figure 5). |
| Total number of Joints                   | 7     | When 6 slabs are arranged in 2 rows and 3 columns, 7 joints are needed to connect slabs to each other, see Figure (5) |
| Number of X-coordinate for each slab     | (table to follow) |
| Number of Y-coordinate for each slab     | (table to follow) |
| Total number of layer                    | 2     | Two layers are analyzed, concrete slab and cement treated subbase                |
| Joint number (JONO1,JONO2,JONO3,JONO4)   | See Figure (5) of this report. |
| Nodal number used to check convergence   | 167   | Any node used for calculating stresses was used as a check node                  |
| Number of nodes NOT in contact           | 0     | NOTCON is always equal to zero when NCYCLE = 1.                                  |
| Number of nodes with initial gaps        | 0     | According to instruction manual of KENSLAB, NGAP must be assigned 0 when NCYCLE = 1 |
| Number of nodes for stress printout      | 1     | We are interested in stresses at a particular point, at midlength of slab 3, right underneath one of the loads |
| Maximum number of cycles for checking    | 1     | USE 1 for full contact                                                          |</p>
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>subgrade contact (NCYCLE)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Input of gaps from previous problem (INPUT)   | 0              | 0=No
1 = Yes                                                            |
| Bond between two slab layers (NBOND)          | 0              | 0=Unbonded
1=Bonded                                                          |
| Temperature Curling (NTEMP)                   | 0 or 1         | 0=No, 1=Yes. This variable decides if or not curling is to be considered. When analyzing the truck, this is set to 0 (No) but when calculating curling of the slab, this is set to 1 (Yes) |
| Weights of the Slab (NWT)                     | 0 or 1         | 0=No – stress cycle analysis does not require slab weight
1=Yes – curling analysis requires slab weight |
| Number of nodes with different thicknesses of slab layer 1 (NAT1) | 0 | All 6 slabs will have same thickness |
| Number of nodes with different thicknesses of slab layer 2 (NAT2) | 0 | All 6 slabs will have same thickness |
| Number of nodes on X axis of symmetry (NSX)   | 0              | We are looking at entire slab, no need to use any symmetry          |
| Number of nodes on Y axis of symmetry (NSY)   | 0              | We are looking at entire slab, no need to use any symmetry          |
| More detail printout (MDPO)                   | 0              | 0=No, 1=Yes
Detail print out is not required as long as we get our output |
| Systems of units (NUNIT)                      | 0              | 0=English
1=SI                                                                |
| Uniform Load (UL)                             | 0              | No uniform load is considered except truck load                      |
| Temperature Gradient (TC)                     | 0              | No temperature gradient was considered                               |
| Concentrated Load (CL)                        | 2              | Indicate, with anything other than 0 that there is a concentrated load |
| Temperature Differential between top and bottom (TEMP) | 0 or Varies | For truck analysis, this is set to zero because effects of curling are not combined. When calculating curling deformation, it is calculated based on ambient temperature. |
| Unit weight of each layer (GAMA)              | 150            | Same unit weight is used for slab and cement treated base.           |
| Modulus of rupture of pavement slab for each layer (PMR) | 0 | Not required if no damage analysis is to be performed (NDAMA=0) |
| Coefficient of Thermal expansion (CT)          | 0.00055        | Default per Huang (2004)                                             |
| Tolerance for iteration (DEL)                  | 0.001          | Default per Huang(2004)                                              |
| Maximum allowable deflection (FMAX)           | 1              | Just to limit the iteration, deflection of 1” due to unit load will not be attended under any circumstances |
| Fatigue properties for each layer (F1,F2)      | 0,0            | Not required if no damage analysis is to be performed (NDAMA=0) |
| X-coordinate of each slab (X)                  |                | Depends on mesh size for each slab that varies based on the purpose of meshing |
| Y-Coordinate of each slab (Y)                  |                |                                                                      |
| Thickness of each slab layer (T)              | Varies 3.94    | Concrete slab thickness vary
base thickness is 3.94 inches (100 mm) |
<p>| Poisson’s ratio of each                        | 0.20           | For concrete and cement treated base                                  |</p>
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer (PR)</td>
<td>0.22</td>
</tr>
<tr>
<td>Young Modulus (YM)</td>
<td>312201</td>
</tr>
<tr>
<td></td>
<td>8 (psi)</td>
</tr>
<tr>
<td></td>
<td>100000</td>
</tr>
<tr>
<td></td>
<td>0 (psi)</td>
</tr>
<tr>
<td></td>
<td>For slab E=57000*$\sqrt{3000}$ = 3122018 for concrete slab</td>
</tr>
<tr>
<td></td>
<td>For cement treated base 1E+6 as per Huang (2004)</td>
</tr>
<tr>
<td>Number of uniformly distributed load for each load group (NUDL)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>No uniform load is considered since UL = 0. Also, only 1 zero value because we are looking at one load group at a time to create out strain database.</td>
</tr>
<tr>
<td>Number of concentrate nodal force (NCNF)</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Single-Axle-Single-tire</td>
</tr>
<tr>
<td></td>
<td>Single-Axle-dual-tire</td>
</tr>
<tr>
<td></td>
<td>(NOTE: This is HOW MANY concentrated force, actual force value is next and we are only creating stress database for single and dual tire, rest all comes from influence line)</td>
</tr>
<tr>
<td>Number of nodal moment in X-direction (NNMX)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>No concentrated moment need to be applied except truck load</td>
</tr>
<tr>
<td>Number of nodal moment in Y-direction (NNMY)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>No concentrated moment need to be applied except truck load</td>
</tr>
<tr>
<td>Nodal number at which load is applied (NN)</td>
<td>Varies</td>
</tr>
<tr>
<td></td>
<td>Different nodes along the truck load path are loaded as the truck moves to create strain database. NOTE: total number of nodes here must be equal to NCNF specified.</td>
</tr>
<tr>
<td>Concentrated force applied at the given node (NCNF)</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>1kip=1000 lbs is used for the influence lines.</td>
</tr>
<tr>
<td>Nodal number at which stresses are computed and printed (NP)</td>
<td>167</td>
</tr>
<tr>
<td></td>
<td>This point is on the load path as well as is the mid-point of the slab</td>
</tr>
<tr>
<td>Foundation Seasonal Adjustment factor (FSAF)</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>No adjustment need to be made</td>
</tr>
<tr>
<td>Young Modulus of subgrade (YMS)</td>
<td>Varies</td>
</tr>
<tr>
<td></td>
<td>Table 4.2.1 (Teruhisa Masada, 2004)</td>
</tr>
<tr>
<td>Poisson’s ratio of subgrade (PRS)</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Pending reference</td>
</tr>
<tr>
<td>Young Modulus of steel bars (YMSB)</td>
<td>290000</td>
</tr>
<tr>
<td></td>
<td>00</td>
</tr>
<tr>
<td></td>
<td>ACI-318</td>
</tr>
<tr>
<td>Poisson’s ratio of steel bars (PRSB)</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>ACI-318</td>
</tr>
<tr>
<td>Spring constant for shear transfer (SPCON1)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>load transfer between slabs is due to dowels at joints</td>
</tr>
<tr>
<td>Spring constant for moment transfer (SPCON2)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>load transfer between slabs is due to dowels at joints</td>
</tr>
<tr>
<td>Modulus of dowel support (SCKV)</td>
<td>150000</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Minimum value suggested by KENSLAB</td>
</tr>
<tr>
<td>Dowel bar diameter (BD)</td>
<td>1.64 in</td>
</tr>
<tr>
<td></td>
<td>Bar No. 14 is assumed</td>
</tr>
<tr>
<td>Dowel bar spacing (BS)</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>Common practice</td>
</tr>
<tr>
<td>Width of joint (WJ)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Not critical</td>
</tr>
<tr>
<td>Gap between concrete and dowel (GDC)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Dowel assumed fully bonded to the slab</td>
</tr>
</tbody>
</table>

Note that values for some of the parameters in Table 9 above are default values. When performing the sensitivity analysis in Chapter 3, numerical values for some of the critical parameters will be varied and
results for each given set of input parameters will be presented. Details on each of the critical parameters will be discussed in chapter 3 as well.

Each slab is meshed in both directions to perform analysis of the slab. This is mainly because of 2 reasons: First, while using KENSLAB to analyze a rigid section, load must be applied at a node and secondly, more accurate results are obtained when finer mesh size are used. On the other hand, KENSLAB has following limitation in terms of mesh size and number of nodes (Huang, 2004):

- Maximum of 6 slabs and 7 joints (another reason for choosing 6-slab model)
- Maximum of 420 nodes
- Maximum of 15 nodes in each of the horizontal and vertical directions in each slab

Thus, nodal coordinates for each of the slabs, 1 through 6 in Figure 4 above are as shown in Table 10 below. Note that each of the coordinates is given in inches and coordinates are given with respect to lower left corner of each individual slab.

<table>
<thead>
<tr>
<th>Slab: 1 and 3</th>
<th>Slab: 4 and 6</th>
<th>Slab: 2</th>
<th>Slab: 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>43.3</td>
<td>24</td>
<td>43.3</td>
<td>24</td>
</tr>
<tr>
<td>86.6</td>
<td>42</td>
<td>86.6</td>
<td>48</td>
</tr>
<tr>
<td>129.9</td>
<td>60</td>
<td>129.9</td>
<td>72</td>
</tr>
<tr>
<td>173.2</td>
<td>78</td>
<td>173.2</td>
<td>96</td>
</tr>
<tr>
<td>216.5</td>
<td>96</td>
<td>216.5</td>
<td>120</td>
</tr>
<tr>
<td>114</td>
<td>144</td>
<td>129.9</td>
<td>144</td>
</tr>
<tr>
<td>132</td>
<td></td>
<td>151.6</td>
<td>132</td>
</tr>
<tr>
<td>150</td>
<td></td>
<td>173.2</td>
<td>150</td>
</tr>
<tr>
<td>168</td>
<td></td>
<td>194.9</td>
<td>168</td>
</tr>
<tr>
<td></td>
<td></td>
<td>216.5</td>
<td></td>
</tr>
</tbody>
</table>

As can be seen from the Table 10 above, more meshes has been provided in x-direction for slab 2. This is because slab 2 is in the middle and as truck travels from the left to right or vise-versa, maximum stresses will be obtained when slab 2 is loaded with truck load, especially when the truck loads is right at the middle of slab 2 as well understood from mechanics. Thus, more mesh is provided where the result is more desired as explained before. Also, under the ideal condition, travel lane is assumed to be along slabs 1 through 3. For single-axle-single-tire, first tire is assumed to at 5 ft from the edge of the slab or 3 ft from the shoulder and second tire in the transverse direction is 6 ft away from the first tire. Thus, there is node in the transverse direction at 60 inches (5 ft) and 132 inches (11 ft).

In order to study sensitivity of the number of corrective maintenance cycle to the pavement temperatures, 3 different sets of average monthly temperature were used. First set of monthly temperature is the average monthly temperature given in Table 3. Based on preliminary study, it was found that changing average monthly temperature by 1 °F as in flexible pavement will not make any difference in the number of
corrective maintenance cycle. Thus, additionally, 2 alternative monthly temperature tables were developed; first by increasing the average monthly temperature given in Table 3 by 10 °F and second one by decreasing the average monthly temperature in Table 3 by 10 °F. Table 11 and Table 12 below are alternative monthly average temperatures for the rigid pavements.

Table 11: Monthly Temperature for I-88 Corridor with an increase of 10 °F on average monthly temperature

<table>
<thead>
<tr>
<th>Month</th>
<th>Average temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>32</td>
</tr>
<tr>
<td>February</td>
<td>35</td>
</tr>
<tr>
<td>March</td>
<td>43</td>
</tr>
<tr>
<td>April</td>
<td>59</td>
</tr>
<tr>
<td>May</td>
<td>66</td>
</tr>
<tr>
<td>June</td>
<td>75</td>
</tr>
<tr>
<td>July</td>
<td>79.5</td>
</tr>
<tr>
<td>August</td>
<td>78</td>
</tr>
<tr>
<td>September</td>
<td>70.5</td>
</tr>
<tr>
<td>October</td>
<td>59</td>
</tr>
<tr>
<td>November</td>
<td>49</td>
</tr>
<tr>
<td>December</td>
<td>37</td>
</tr>
</tbody>
</table>

Table 12: Monthly Temperature for I-88 Corridor with an decrease of 10 °F on average monthly temperature

<table>
<thead>
<tr>
<th>Month</th>
<th>Average temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>12</td>
</tr>
<tr>
<td>February</td>
<td>15</td>
</tr>
<tr>
<td>March</td>
<td>23</td>
</tr>
<tr>
<td>April</td>
<td>39</td>
</tr>
<tr>
<td>May</td>
<td>66</td>
</tr>
<tr>
<td>June</td>
<td>55</td>
</tr>
<tr>
<td>July</td>
<td>59.5</td>
</tr>
<tr>
<td>August</td>
<td>58</td>
</tr>
<tr>
<td>September</td>
<td>50.5</td>
</tr>
<tr>
<td>October</td>
<td>39</td>
</tr>
<tr>
<td>November</td>
<td>29</td>
</tr>
<tr>
<td>December</td>
<td>16</td>
</tr>
</tbody>
</table>
Chapter 3 Implementation of MEPDG Method

In Chapter 3, the MEPDG methods described in Chapter 2 for analyzing flexible and rigid pavements are implemented to study the damage caused by heavy trucks on different segments of the I-88 Corridor to compute the number of corrective maintenance cycles required to maintain the serviceability of each segment throughout its design life (50 years). Note that the pavement segments designs analyzed in this chapter (for both flexible and rigid segments) are not the existing pavement cross sections of the I-88 corridor but idealized cross sections that are designed that exactly satisfies the design requirements of NYSDOT. The MEPDG analysis is based on truck weight data collected on a Weigh-In-Motion station located at one end of the corridor. Also, all climate and environmental related input parameters are those for the I-88 corridor. The need for corrective maintenance is determined based on an analytically estimated PSR and corresponding IRI, computed using Eq. (1). Initial and terminal IRI for flexible and rigid pavements have been defined in section 2.1 and 2.2 respectively. Input parameters required for running KENPAVE programs (KENLAYER for flexible and KENSLAB for rigid pavements) have been discussed in Chapter 2. Because of the uncertainties in estimating the actual values of the MEPDG input parameters and possible changes over time in these parameters, a sensitivity analysis is performed to identify which of the parameters are most critical and which have most influence on the MEPDG output as represented by the number of corrective maintenance cycles in a 50-year pavement design life. The sensitivity analysis is performed in this Chapter for both flexible and rigid pavement segments of the I-88 Corridor.

3.1 Implementation of MEPDG Method for Flexible Pavements

The method described in Chapter 2 of this report for analyzing flexible pavements is implemented in this section to calculate the number of corrective maintenance cycles needed to keep each segment of the I-88 Corridor in good condition. A sensitivity analysis is also performed to study the effect of changes in critical parameters on the final outcome in terms of the number of corrective maintenance cycles. Specifically, the effects of the following input parameters are studied and presented along with explanation for why these inputs parameters were chosen specifically for the sensitivity analysis.

1. Average monthly temperature
   Average monthly temperature is a very critical for estimating the corrective maintenance cycle of the pavement section. Fatigue cracking is a function of dynamic modulus of the HMA according to Eq. (5) where dynamic modulus is an exponential function of HMA temperature as per Eq. (10). On the other hand, unlike other parameters used in the KENLAYER analysis, temperature is one of the parameters which is very difficult to predict. In reality, temperature at a particular location varies from day to day and even throughout the day. It is far beyond the ability of the current program to perform the analysis on the daily basis using average daily temperature of the given location and definitely beyond the ability of the current computer program to analyze for the temperature at the time truck enter arrives at the pavement section being analyzed. Thus, in this report, average monthly temperature was used assuming that temperature throughout the month remains more or less the same. Since, this is simply an approximation to perform the analysis within the reasonable time, it is very important to understand how does change in
temperature effects program outputs in terms of the number of corrective maintenance cycles required for the flexible pavement section. Hence, 3 different sets of average monthly temperatures were used as presented on Table 3, 4 and 5.

2. Creep compliances
   As described in Chapter 2, creep compliance is a function of the initial elastic modulus of the foundation. Since, almost all the time, geotechnical report for the subgrade underneath the pavement are not available, the elastic modulus of the foundation is not a well-known property and thus requires estimation based on the literature and previous studies as was done in this report. Since it is an uncertain property, effects of choosing different numerical value on the final output must be understood. Thus, two different elastic moduli, 400,000 psi and 500,000 psi were used to study the effect of change in the moduli and the corresponding creep compliances in Table 6 and 7 respectively.

3. Thickness of Hot Mix Asphalt (HMA)
   Different pavement sections may have different HMA thicknesses but within the limitation of the jurisdiction of the state authorities. It is well understood from mechanics that thicker sections have higher stiffness and higher section properties (For example: moment of inertia, section modulus etc for beams as per the beam theory). Thus, it is very clear that thicker section performs better and hence, would require lesser number of corrective maintenance cycles compare to thinner section while other parameters are held constant. Thus, different thicknesses of HMA were chosen as one of the parameter in the sensitivity analysis to see how increased HMA thickness affects the number of corrective maintenance cycle.

4. Thickness of Selected Granular Subgrade (SGSG)
   Different thickness of SGSG were chosen in this study for the similar reason why different HMA thicknesses were chosen as explained above.

5. Average Daily Truck Traffic (ADTT)
   Since MEPDG method is both mechanical as well as empirical method, load is not the only component contributing to the damage of the pavement and ultimately, number of corrective maintenance cycle. ADTT is directly proportional to the load on the pavement section. However, since damage of the pavement is contributed by environmental factors as well, increase in load (in another words, increase in the number of truck and daily truck load passing through the section) does not necessarily correspond to the increase in the damage on the pavement. Hence, 3 different ADTT were used in this study: 2000, 4000 and 6000 to see how the change in ADTT affects the final outcomes in terms of the number of corrective maintenance cycles.

6. Number of trucks per class and category used for the analysis
   Truck load and configuration even within the same class and category may vary. Further details on class and category of trucks will follow. If trucks within the same class and the category were to be identical to each other, average damage per truck for the given class and category would have been the same regardless of number of trucks chosen for that class and category. Obviously, to obtain more accurate damage per truck, all the trucks in that class and category must have been
used for the analysis and average damage per truck must have been calculated dividing total damage by the total number of the trucks used. Due to the program limitation, it is impractical to use all the trucks in particular class and category. Thus, either 1 or 3 trucks per class and category were used in this study to see the effect of different sampling approach on the final output in terms of the number of corrective maintenance cycle.

In this study, truck data were extracted directly from Weigh-In-Motion (WIM) Station 9580 located near the North-East end of I-88. This WIM station is located near Exit 23, 20 miles away from the end of the highway near Schenectady as shown in the Figure 7 below (Ghosn, 2015).

![Figure 7: New York State WIM Sites with highlighted station 9580 on I-88](image)

The damage that heavy vehicles cause to pavements depends on the distribution of the weight to the axles and how close the axles are to each other. While most traffic is composed of cars, it is the trucks that cause the most damage due to the significantly heavier weights they carry. According to Federal Highway Administration (FHWA), vehicles travelling along the highway systems can be categorized into 13 different classes based on their vehicle axle configurations. Figure 8 below shows the different configurations of vehicles and their assigned classes. Classes 5 thru 13 are appropriate for trucks where each class has unique axle configuration and gross vehicle weight limitation. Additionally, trucks can be further divided into four different weight categories: 1) legal trucks that adhere to the weight limits imposed by the jurisdiction which in our case is the New York State law as specified on www.dot.ny.gov/nypermits (NYSDOT, 2015); 2) trucks issued permit to carry divisible loads beyond those stipulated by the New York State weight regulations; 3) special hauling vehicles carrying permits to exceed the weight limits on specific routes after evaluation by the bureau of structures because of the
heavier weights or special configurations as compared to those issued divisible load permits and 4) illegally overweight trucks.

![FHWA Vehicle Classifications](image)

In the year 2011, WIM station 9580 recorded data for over 500,000 trucks in classes 5 through class 13 in all 4 weight categories. However, only selected representative samples are used for the analysis of pavement as it is not computationally possible to run all the trucks in the WIM data as this would require an enormous computer time that cannot be accommodated by the available computer facilities. Thus, only 10 trucks from each weight category and each class were chosen.

The pavement analysis used the following parameters for the base case considered as the control case:

- 10 trucks per each trucks class and category
- Average monthly temperature as per Table 3
- Viscoelastic properties as per Table 6
- Thickness of HMA: 9.84 inches

Figure 8: Example of typical vehicles classes per FHWA (photo credit: (Texas DOT, 2015))
Estimated numbers of corrective maintenance cycle for the base case scenario are presented in Table 13 below:

Table 13: Estimated number of corrective maintenance cycles for base case scenario

<table>
<thead>
<tr>
<th>HMA (in)</th>
<th>SGSG (in)</th>
<th>ADTT</th>
<th>Number of Corrective Maintenance Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.84</td>
<td>13.66</td>
<td>2000</td>
<td>2</td>
</tr>
</tbody>
</table>

The input data for the control case are those used by Ghosn et al. (2015) based on information collected for the I-88 corridor as available in the NYSDOT files and related sources. For the purpose of studying the sensitivity of the estimated number of corrective maintenance cycles to critical input parameters (as defined in the beginning of this section), a total of 12 cases of parameter combinations were ran for the flexible pavement. Descriptions of each case are given in Table 14 below.
Table 14: Description of 12 cases of sets of input parameters to perform sensitivity analysis of Flexible Pavement

<table>
<thead>
<tr>
<th>Cases</th>
<th>Parameters Studied</th>
<th>Number of Trucks per Class and Category</th>
<th>Average monthly Temperature Table</th>
<th>Viscoelastic Property Table</th>
<th>HMA (in)</th>
<th>SGSG (in)</th>
<th>ADTT</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Effects of change in average monthly temperature by 1 °F for the range of HMA and SGSG thicknesses and ADTT</td>
<td>1</td>
<td>Table 3</td>
<td>Table 6</td>
<td></td>
<td></td>
<td>2000 4000 6000</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>III</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>Effects of using HMA with ( E_0 = 500,000 ) psi instead of ( E_0 = 400,000 ) psi for the former 3 cases</td>
<td>1</td>
<td>Table 3</td>
<td>Table 7</td>
<td>6.5 in (165 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>V</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.5 in (191 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>8.5 (216 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VII</td>
<td>Effects of using 3 trucks per class and category and finding average damage per truck for that class and category instead of just 1 compared to cases 1 through 3</td>
<td>3</td>
<td>Table 3</td>
<td>Table 6</td>
<td>9.5 (241 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VIII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9.84 in (250 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IX</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X</td>
<td>Effects of using 3 trucks per class and category and finding average damage per truck for that class and category instead of just 1 compared to cases 4 through 6</td>
<td>3</td>
<td>Table 3</td>
<td>Table 7</td>
<td></td>
<td>11.81 (300 mm)</td>
<td></td>
</tr>
<tr>
<td>XI</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19.69 (500 mm)</td>
<td></td>
</tr>
<tr>
<td>XII</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>27.56 (700 mm)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>35.43 (900 mm)</td>
<td></td>
</tr>
</tbody>
</table>
In cases 1 through 6, as discussed previously, only one truck was chosen at random from each vehicle class and category used as representative truck sample of the entire set of trucks. For cases 7 through 12, three randomly selected trucks from each vehicle class and weight category were used. When more than one truck for a given class and category are used, the damages caused by the trucks are accumulated. The average damage for a typical truck (of that class and category) is found by dividing the total damage by the number of trucks used.

Calculating the number of corrective maintenance cycles requires the calculation of IRI as shown in Eq. (2). In order to calculate the overall IRI of the pavement segment at the end of the given month so that the need for corrective maintenance cycles can be determined, all of its contributing factors must be calculated as follows:

- Initial IRI when the pavement is first opened for traffic or after each corrective maintenance cycle has been performed for the flexible pavement as described in section 2.1.1
- Site factor for the pavement segment is calculated using Eq. (3) once every year since it is function of the age of the pavement.
- As discussed in section 2.1.3.3, transverse cracking in Eq. (18) will be entered as 1000 ft/mile, as it is the maximum limit but is computed only once for each corrective maintenance cycle.
- In order to calculate the bottom alligator cracking as given in Eq. (12), firstly, a radial strain database is assembled at the bottom of the HMA for a set of typical trucks representing each truck class (vehicles in classes 5 through 13), overweight category (legal, special hauling permit, divisible load permit and illegally overweight) and average temperature for each month of the year. The radial strains at the bottom of the HMA are obtained using the program KENLAYER. The strain database has a size of 4X9X12 matrix, for total of four categories, nine classes of trucks and twelve months in a year. As discussed before, when more than one truck was used for the analysis (as for the case of Cases 7 through 12), the strain database represents the average strain for each typical vehicle class and class category by finding the sum of all the strains recorded for the particular class, category and month and dividing by the number of trucks used. The strain database is then converted into corresponding database for the damage index calculations using Eq. (4) and thus, the damage index database is assembled into another 4X9X12 matrix, each value corresponding to each vehicle class, weight category, and month.
- In a similar fashion, in order to calculate permanent deformation of the HMA layer, a vertical strain database at the mid-depth of the HMA is assembled for a set of typical trucks representing each truck class, overweight category and average temperature for each month of the year. After assembling the vertical strain database at mid-depth of the HMA, the IRI contribution of each truck for a given class and category due to permanent deformation in the HMA layer is calculated using Eq. (19) for the given month. Hence, 4X9X12 matrix is obtained, with each value as contribution made to the IRI of the pavement segment due to the plastic deformation of the HMA for a typical truck of given class and category and a given month.
- To calculate total IRI contribution made to the IRI due to the bottom alligator cracking or the permanent deformation of the HMA layer, contribution due to each typical class and category of truck for given month is multiplied by the truck distribution table, an another 4X9X12 matrix that represents the percentage of the total truck recorded for given class and category at particular given month. Once a single value if obtained to represent total contribution to the IRI due to
either bottom alligator cracking or the plastic deformation of the HMA, it is multiplied by 365 and the ADTT further to obtain the total contribution made to the IRI for the given month. The final obtained value is then multiplied by 0.5 (because ADTT is the number of trucks recorded in both directions of the corridor), a lane factor of 1 is used to be conservative. This conservatively assumes that all the trucks are traveling in the same lane. In order to account for the potential growth in the average number of trucks and the average weight of trucks over the years, a 1% yearly compound growth factor is included in the computation.

- If the cumulative IRI at the end of a given month is less than the terminal IRI, the calculations continue on a month-by-month basis until the terminal IRI is reached. Once the terminal IRI is reached, the number of corrective maintenance cycles is increased by 1 (starting from zero) and the process is re-started assuming that the maintenance restores the IRI to its initial value. The process is continued for a total pavement service life of 600 months (50 years) and total number of corrective maintenance cycles at the end of the design life is recorded.

The procedure described above is a general procedure for calculating IRI of a given flexible pavement segment at the end of the each month and for determining the need to perform corrective maintenance and for estimating the number of maintenance cycles required within a fifty-year pavement design life. This general procedure is implemented to determine the number of corrective maintenance cycles for different flexible pavement cross sections for different sets of input parameters (described under the 12 cases listed in Table 14 above). Note that except for the changes noted in Table 14, all the input parameters will remain at the values listed in Table 2. The results for the 12 different cases described in Table 13 are summarized in Figures 9 through 20.

Generally speaking, looking at the results for any of the cases with the constant SGSG thickness, the estimated number of corrective maintenance cycles remains unchanged or decreases as the HMA thicknesses increases for the same ADTT. This result is of course expected based on straightforward understanding of the theory of mechanics.

Comparing results for Cases I, II and III in Figures 9, 10 and 11 respectively, the estimated number of corrective maintenance cycles for Case II is higher by one cycle compared to that for Case I with constant HMA. Also, for the constant SGSG thickness cases, the number of corrective maintenance cycles is higher than that of Case II by up to 7 additional corrective maintenance cycles for a constant HMA thickness equal to 6.5 inches. As described in Table 14 above, Case II uses Table 4 for the average monthly temperature where the average monthly temperatures are 1 °F higher than those listed in Table 3. On the other hand, the estimated number of corrective maintenance cycles in CASE III, as shown in Figure 11, remained unchanged in comparison to CASE I for the first 3 thicknesses of SGSG but were reduced by 1 corrective maintenance cycle as the SGSG thickness is increased further. A reduction in the number of corrective maintenance cycles is observed when the SGSG thickness is kept constant as the HMA thickness increased.

Comparing the results for Cases IV, V and VI in Figures 12, 13 and 14 respectively with Figures 9, 10 and 11 for Cases I, II and III respectively, it is observed that the estimated number of corrective maintenance cycles is generally reduced in cases IV, V and VI compared to the latter 3 cases due to the higher HMA initial elastic modulus ($E_0 = 500,000 \, \text{psi}$). This result also is consistent with our understanding of simple mechanics theory.
Looking at the results for the first 6 Cases in Figures 9 through 14, the estimated number of corrective maintenance cycles remained unchanged despite an increase in the ADTT from 2000 to 6000. As noted in Table 14 above, only 1 truck per class and category chosen at random were used for the analysis in these first 6 Cases. This shows that one truck that was chosen in random was not enough to represent typical truck loading as it seems that the effect of truck configurations was not well reflected when only 1 truck was used. When the ADTT is increased from 2000 to 6000, it didn’t make significant difference compared to when a single truck is used from each class. However, looking at the results in Figures 15 through 20, which present the results obtained when using 3 randomly selected trucks for each class and truck category, it is observed that the estimated number of the corrective maintenance cycles increases proportionally with ADTT. This means that by sampling 3 trucks per class and category, we obtained a better representation of the truck load effects. Thus, using a large number of trucks for the analysis is definitely encouraged. Trends for the number of corrective maintenance cycles within Figures 15 through 20 are still consistent with those in Figures 9 through 14.
**Figure 9:** Number of Corrective Maintenance Cycles for CASE I

**Figure 10:** Number of Corrective Maintenance Cycles for CASE II
Figure 11: Number of Corrective Maintenance Cycles for CASE III

Figure 12: Number of Corrective Maintenance Cycles for CASE IV
Figure 13: Number of Corrective Maintenance Cycles for CASE V

Figure 14: Number of Corrective Maintenance Cycles for CASE VI

50
Figure 15: Number of Corrective Maintenance Cycles for CASE VII

Figure 16: Number of Corrective Maintenance Cycles for CASE VIII
Figure 17: Number of Corrective Maintenance Cycles for CASE IX

Figure 18: Number of Corrective Maintenance Cycles for CASE X
Figure 19: Number of Corrective Maintenance Cycles for CASE XI

Figure 20: Number of Corrective Maintenance Cycles for CASE XII
3.2 Implementation of MEPDG method for Rigid Pavements

The method implemented for estimating the damage caused by heavy trucks to rigid pavement is slightly different from the one that was used for the flexible pavement. As described earlier, it was concluded that the principle of superposition holds for rigid pavements because their behavior follows a linear elastic model. Accordingly, a stress influence line for the midpoint of a rigid pavement slab (slab 3 in Figure 4) along the travel lane was obtained for a pair of single tire loads having a total weight equal to 1 kip by moving the load along the length of the slab. Similarly, another stress influence line was obtained by moving 2 pairs of loads (total of 1 kip) for single-axle-dual-tire. Using the two stress influence lines, a new set of influence lines were assembled for four different axle group influence lines: 1) single-axle-single-tire, 2) single-axle-dual-tire, 3) tandem axle and 4) tridem axle for the total load. For the analysis of damage due to curling, the maximum loaded and unloaded slab deflections at the slab corner were obtained for temperatures ranging between 5 °C to 90 °C at an intervals of 5 °C. Linear interpolation is used to obtain stress, deflection or curling for any input parameter that doesn’t exactly correspond to the ones used to create the database.

The rigid pavement influence lines created for each axle group are implemented into the MEPDG method described in Chapter 2 to estimate the number of corrective maintenance cycles required to maintain a pavement section in good condition during its 50-year design life. A sensitivity analysis is also performed to study the sensitivity of the final outcome to each of the input parameters. The sensitivity analysis investigated the effect of the following input parameters on the number of corrective maintenance cycles.

1. **Average monthly temperature**
   One of the major components of initial maximum mean transverse joint faulting in Eq. (35) is the maximum mean monthly slab corner deflection of PCC due to the temperature curling and moisture warping. The maximum monthly slab corner deflection is a function of differential temperature between top and bottom of the slab and the differential temperature is the function of ambient temperature (in other words, temperature at the top of the pavement). It is noted that the temperature at a particular location varies from day to day and even throughout the day. To simplify the analysis process, this report uses the average monthly temperature. Therefore, a sensitivity analysis is performed to understand how does the change in temperature affects the estimated number of corrective maintenance cycles required for rigid pavement sections. Hence, three different sets of average monthly temperatures were used as shown in Tables 3, 11 and 12.

2. **Thickness of PCC**
   Different pavement sections may have different PCC thicknesses. It is well understood from mechanics that thicker sections have higher stiffness and higher section mechanics properties such as stiffness and strength. Thus, it is very clear that thicker section performs better and hence, would require lesser number of corrective maintenance cycles compared to thinner sections when other parameters are held constant. Thus, different thicknesses of PCC are analyzed to study the relation between PCC thickness and the number of corrective maintenance cycles.

3. **Type of foundation and foundation material properties**
   KENSLAB can analyze different types of foundation for rigid pavements. A foundation could be defined as solid, liquid or layered. However, the most suitable and commonly used foundation
under the rigid pavements is solid foundation. Because geotechnical information on subgrade material properties underneath the pavement are not always available two different elastic moduli are compared; with the first is taken as \( E = 30,000 \) psi and the other is set at \( E = 46,500 \) psi. These represent typical maximum and minimum values for the elastic modulus of foundation according to the study by (Raul Velasquez, 2009).

4. **Average Daily Truck Traffic (ADTT)**

Different ADTT values are also chosen for the sensitivity analysis of rigid pavements.

The pavement analysis used the following parameters for the base case considered as the control case:

- 10 trucks per each trucks class and category
- Average monthly temperature as per Table 3
- Thickness of PCC: 8.86 inches
- Elastic modulus: 30,000 psi
- ADTT: 1000

Estimated numbers of corrective maintenance cycle for base case considered are shown in Table 15 below:

<table>
<thead>
<tr>
<th>PCC (in)</th>
<th>ADTT</th>
<th>Number of Corrective Maintenance Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.86</td>
<td>1000</td>
<td>4</td>
</tr>
</tbody>
</table>

The input data for the control case are those used by Ghosn et al. (2015) based on information collected for the I-88 corridor as available in the NYSDOT files and related sources. For the purpose of studying the sensitivity of the estimated number of corrective maintenance cycles to critical input parameters (as defined in the beginning of this section), a total of 4 cases of parameter combinations were ran for the rigid pavement. Descriptions of the four cases are given in Table 16 below:
As done for flexible pavements, checking the sensitivity of the number of corrective maintenance cycles to different trucks within the same class and category, the first two cases in Table 16 were analyzed assuming all the trucks in each class and weight category have the same configuration and weights as one randomly selected truck from that class and weight category. Then for the third and fourth cases, three trucks were chosen at random from each class and category. Again, when more than one trucks for a given class and category are used, any damage caused by the trucks are accumulated. The average damage for a typical truck (of that class and category) is found by dividing the total damage by the number of trucks.

Calculating the number of corrective maintenance cycles for any rigid pavement requires the calculation of IRI as shown in Eq. (27) for the pavement section at the end of each month. When the IRI of a given
pavement reaches the terminal IRI, it is determined that a corrective maintenance is required. The calculations required the following main input:

- Initial IRI when the pavement is first open for the traffic or after each corrective maintenance cycle is calculated for rigid pavement in shown in section 2.2.1.1
- Site factor for the pavement segment is determined using Eq. (28) for each year in the pavement life. Note the years are counted starting from the opening of the pavement to traffic.
- Spall contribution to the IRI is calculated using Eq. (29) where the scaling factor (SCF) is calculated using Eq. (30) for the given age of the pavement. Note again that the age is the age of the pavement since its first opening.
- Faulting is calculated using section 2.2.1.4. To calculate Differential Energy (DE) in Eq. (32), the database for deflection of corner of loaded and the unloaded slab due to unit load for each load group, that was previously assembled due KENSLAB, is used and the total deflection (both of loaded and the unloaded slabs) is found by multiplying the value obtained from the database with the actual weight under the load group. Initial maximum transverse joint faulting ($\text{Faultmax}_0$) in Eq. (35) is calculated using maximum upward deflection of the slab due to maximum mean monthly slab temperature. The database is created based on the detailed description provided in Chapter 2 for the maximum upward deflection of the slab for various slab thicknesses in the range of 8.85 in (225 mm) to 12.79 in (325 mm). The database contains upward slab deflection for a range of ambient temperatures varying between 5 °F to 90 °F because curling is caused by differential temperature between top and bottom of the slab and the differential temperature is a function of the ambient temperature.
- Cracking is calculated using the set of equations listed in section 2.2.1.5. The most important parameter is the applied stress needed to calculate hypothetical number of stress cycles in Eq. (39) and the damage index in Eq. (38). Two databases were created for the stress at the bottom of the pavement due to a unit load; one for single-axle-single-tire and another one for single-axle-dual-tire. Using the superposition theory, stress can be calculated for any of the load groups (4 different types of load groups) and any given total axle load.
- The contributions to the IRI of spall and faulting per average truck of a given class and weight category are calculated for every particular month. The total contribution for each month is obtained by multiplying the contribution of the representative truck by the number of trucks in each class and weight category. The truck distribution table is a 4x9x12 matrix that contains a percentage of each class and category of the truck for each month in the I-88 Corridor. For a given month, the 4x9 values of IRI are multiplied by the 4x9 distribution values then multiplied by 365 and the ADTT to give the total contribution to the IRI for the given month. The obtained value is further multiplied by 0.5 (because ADTT is the number of trucks recorded in both directions of the corridor and damage is calculated for each particular section), the lane factor of 1 is used to be conservative. This conservatively assumes that all the trucks are traveling in the same lane. In order to account for the potential growth in the average number of trucks and average weight of the trucks over the years, a 1% compound growth factor is included in the computation. If the IRI at the end of the given month is less than the terminal IRI, the calculations continue on a month-by-month basis until the terminal IRI is reached. Once the terminal IRI is reached, the number of corrective maintenance cycles is increased by 1 (starting from zero) and
the process is re-started assuming that the maintenance restores the IRI to its initial value. The process is continued for a total pavement service life equal to 600 months (50 years).

The procedure described above is a general procedure for calculating IRI of a given rigid pavement segment at the end of each month to determine the need of corrective maintenance. This general procedure is implemented to determine the number of corrective maintenance cycles for different rigid pavement segments for different sets of input parameters (described under several cases in Table 16 above). Note that except for the input parameters listed in Table 16, all the input parameters remain as shown in Table 9. The results for the four different cases described in Table 16 are presented below:

Comparing the three plots within Figure 21 for the first Case, no difference is observed for the estimated number of corrective maintenance cycles. Note that the first plot corresponds to the analysis using actual average monthly temperatures for the I-88 corridor, the second and third plots correspond to the analysis using temperatures that are 10 °F higher and lower on average for each month. This shows that even temperature changes as significant as 10 °F higher and lower do not affect the number of corrective maintenance cycles.

Results for Case II as shown in Figure 22 show lower number of corrective maintenance cycles compared to those of Case I. As observed from Table 16, the only difference between the second and first case is that the second case uses foundations with a higher elastic modulus (\( E = 46,500 \text{ psi} \)), which is more than 1.5 times the elastic modulus for the first case. As expected, the stiffer elastic modulus reduces the number of the corrective maintenance cycles. Also, it was observed in the second case that the estimated number of corrective maintenance cycles for different ADTT were the same. This could be reasoned in a similar way as was done previously because as the foundation becomes stiffer, the damage induced by the loads is reduced but the environmental related damage is not affected. It can be concluded that the damage in rigid pavements is mostly contributed by the load than the environmental factors because once the stiffer section is used, the damage contribution of the loads is vastly reduced and the damage becomes mainly due to environmental factors.

Comparing Figure 21 with 23 and Figure 22 with 24 where the only difference is that 3 trucks per class and category are used in the latter cases compared to first cases, no difference in the estimated number of corrective maintenance cycles are found. This is different compared to the observation made for flexible pavements. This suggests that different truck sampling approaches are not worthy of consideration especially for the rigid pavements.
Figure 21: Number of Corrective Maintenance Cycles for Case I using 3 different sets of average monthly temperature
Figure 22: Number of Corrective Maintenance Cycles for Case II using 3 different sets of average monthly temperature
Figure 23: Number of Corrective Maintenance Cycles for Case III using 3 different sets of average monthly temperature.
Figure 24: Number of Corrective Maintenance Cycles for Case IV using 3 different sets of average monthly temperature
3.3 Discussion of Results

3.3.1 Flexible Pavement
The number of corrective maintenance cycles for the 50-year design period for flexible pavement sections were calculated for 12 different sets of input parameters in section 3.1 and the results for each set of input parameters were presented in 12 different figures in the same section. The results showed that the number of trucks selected per each class and the category used for the analysis has significant impact on the final output in terms of the number of corrective maintenance cycles. A fewer number of corrective maintenance cycles is required when 3 trucks per class and category were used to obtain the average damage caused by a passing truck as compared to the case when damage is evaluated based on one single truck per each FHWA vehicle class and weight category. This shows that the higher number of trucks selected for the analysis is the more accurate will the results be. Of course, to be more accurate, all the trucks for the given class and the category must have been used to calculate the total damage. However, this becomes impractical due to the large volume of trucks that pass over the pavement. It is also observed that for a given section, the effects of ADTT is minimal when single truck per class and category was used for the analysis in comparison to when average of 3 trucks were used instead.

When the thickness of the top most layer (HMA) is kept to its maximum allowable thickness and the thickness of the 4th layer (SGSG) is increased, the number of maintenance cycles increases. Although the corrective maintenance of SGSG doesn’t directly contribute to the IRI as seen from the equation to calculate IRI for the flexible pavement as seen in section 2, it appears that a thicker SGSG produces higher overall deformations, resulting in higher damage and ultimately larger number of corrective maintenance cycles. On the other hand, increasing the thickness of HMA layer while keeping the thickness of SGSG at the minimum required, will lead to a reduction in the number of corrective maintenance cycles. This confirms the basic theory of mechanics that thicker section results in higher section properties (moment of inertia), inducing lower stresses and strains.

The number of corrective maintenance cycles were also observed to increase with an increase in average monthly temperatures, even by 1 °F. The minimal change in temperature was applied for all 12 months of the year and resulted in increasing the number of corrective maintenance cycles on the average by one. This shows that the MEPDG method for the flexible pavement is very sensitive to the temperature change and fluctuation as climatic conditions are given high importance in MEPDG. Thus, this suggests that one must be very careful about using average monthly temperatures for the analysis.

Lastly, it was observed that the results are very sensitive to the HMA creep compliance coefficients. Generally, the number of rehabilitation cycles using the creep coefficients in Table 6 (corresponding to $E_0 = 400,000 \text{ psi}$) were higher than the ones for Table 10 (corresponding to $E_0 = 500,000 \text{ psi}$). Again, this is consistent with the basic theory of mechanics. Since creep coefficients are analogous to the reciprocal of elastic modulus and lower creep coefficients correspond to higher initial elastic modulus of HMA and thus stiffer section producing lower load related damages and thus requiring fewer number of corrective maintenance cycles.

3.3.2 Rigid Pavement
Unlike flexible pavements, using single truck per class and category compared to 3 trucks didn’t make difference in the final estimation of the number of corrective maintenance cycles of rigid pavements. This
concludes that damage to rigid pavement is mostly related to environmental factors rather than load factors. Moreover, increasing or decreasing average monthly temperatures even by 10 °F for every month didn’t result in any change in number of corrective maintenance cycles. Increasing ADTT by 2000 per day barely caused any change in the final corrective maintenance cycles, again confirming to the first conclusion that the load factors are not crucial for the rigid pavement. Increasing the thickness of PCC didn’t reduce number of corrective maintenance cycles significantly either. Change in elastic modulus of solid foundation however caused significant difference. By increasing elastic modulus from 30,000 psi to 46,500 psi, average number of corrective maintenance cycles per section reduced from 3 down to 1 on average. This concludes that of all the input parameters whose sensitivity for the final number of corrective maintenance cycles were checked, elastic modulus for the foundation seems to have a biggest impact on damage of rigid pavements. Actual elastic modulus of the foundation of the pavement section must be obtained as accurately as possible to obtain good estimates of the number of corrective maintenance cycles.
Chapter 4 Conclusions and Scope of Future Research

4.1 Conclusions

In this Thesis, the MEPDG approach was used to assess damage in flexible and rigid pavements and estimate the average number of corrective maintenance cycles to maintain the serviceability of the pavement during its intended 50-year design life. KENPAVE program developed by Prof. Huang was used to find load induced stresses, strains and deflections which were used to calculate damage in the pavement using the AASHTO MEPDG equations (AASHTO, 2008). KENPAVE consists of the two modules; KENLAYER for flexible and KENLSAB for rigid pavements. In addition, all environmental related damages were calculated using equations available in the same MEPDG manual. The number of corrective maintenance cycles for typical flexible and rigid pavements were estimated for sections with different layer thicknesses and different sets of input parameters to check the sensitivity of different parameters.

The results show that for flexible pavements, obtaining average monthly temperatures is critical because of the large effect temperatures have on the number of corrective maintenance cycles. As expected from the theory of mechanics, the number of corrective maintenance cycles decreases for pavements with higher initial elastic modulus. Unexpectedly, a change in the ADTT did not cause any change in the estimated number of the corrective maintenance cycles.

The results of rigid pavements showed that the average monthly temperature is not a critical parameter. As expected, a higher elastic modulus for the foundation resulted in fewer number of corrective maintenance cycles. Also, as observed with flexible pavements, no difference in the number of corrective maintenance cycles was noted when 1 or different 3 trucks were used to represent the configurations of all the trucks within a FHWA vehicle class and weight category.

4.2 Future Work

The observations made in this report outline the need for additional research to further quantify the effect of changes in the factors that have been identified as being most critical for evaluating the damage to highway pavements. As observed from the results of flexible pavements, identifying the number of representative truck configurations is very crucial for the accurate determination of the number of corrective maintenance cycles. Since it is not practical to use the entire volume of trucks, the selection of the right number of trucks to be used to give satisfactory results must be investigated. Also, developing a procedure to decide on how to establish the average monthly temperatures to be used in the analysis is also important as a slight change in temperature causes significant differences in the number of corrective maintenance cycles especially for flexible pavements. Also, the average monthly temperature changes every year, more than likely increasing slightly every year due to climate change. In order to obtain better estimation of the number of corrective maintenance cycles, further investigation is required to estimate the change in average monthly temperature over the years and incorporate that change in the analysis process. Differences in creep compliance coefficients showed significant change in the number of corrective maintenance cycles for flexible pavements. Further investigation is crucial to better estimate
the creep compliance coefficients of HMA. Moreover, in this report, the maximum allowable limit for the transverse cracking was used even though mathematical equations to calculate it are provided in the AASHTO guide. These calculations could not be executed because the program KENPAVE did not output the necessary parameters. Future research can complement the analysis of KENPAVE to those of other commercial software packages to produce the data needed for calculating transverse cracking. An additional area to look into could be the terminal IRI, which was defined as 67 (corresponding to PSR = 3.8) in this report. The actual terminal IRI might be related to societal and geographical criteria and should be determined by local or state agencies based on their own experiences, public demands and economic constraints. This will cause significant differences in the number of corrective maintenance cycles required for a pavement section.

Whereas for the rigid pavement, further investigation is necessary to better estimate the elastic modulus of the foundation. As discussed earlier, it would be very difficult to get geotechnical report for the foundation where the pavement section is constructed but studies of nearby subgrade conditions, survey or past records would be very helpful to estimate elastic modulus better so that better estimation of number of corrective maintenance cycles can be obtained. As discussed earlier for flexible pavement, further investigation on how to determine the terminal IRI is also important.
Bibliography


Tarefder, R. (N/A). Development of a Flexible Pavement Database for Local Calibration of MEPDG. New Mexico, New Mexico: New Mexico Department of Transportation.


Appendix A: Freezing Index Calculations

The average Freezing Index is dependent on the region in NYS Maps of Freezing Indexes for different regions are provided by Research, Development, and Technology Turner-Fairbank Highway Research Center. For example, Figure D.1 gives the map for New York State. The information in Figure D.1, is applied for finding a weighted average index for the entire state of New York based on the area of each region. The Calculations are shown in the Table D.1.

![Freezing Index Regions of NYS](image)

Figure A.1: Freezing Index Regions of NYS

<table>
<thead>
<tr>
<th>Region</th>
<th>Area [pixel units]</th>
<th>Area Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>52.9372</td>
<td>0.123237</td>
</tr>
<tr>
<td>2</td>
<td>47.9881</td>
<td>0.111715</td>
</tr>
<tr>
<td>3</td>
<td>77.9972</td>
<td>0.181576</td>
</tr>
<tr>
<td>4</td>
<td>137.6994</td>
<td>0.320561</td>
</tr>
<tr>
<td>5</td>
<td>112.9351</td>
<td>0.262911</td>
</tr>
<tr>
<td>Total</td>
<td>429.557</td>
<td>1</td>
</tr>
</tbody>
</table>
Table A.2: Average freezing index in NYS weighted based on the region areas.

<table>
<thead>
<tr>
<th>Freezing index</th>
<th>Contributing region (1:1 ratio of each region)</th>
<th>Weight Contribution Based on Region Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1500</td>
<td>1</td>
<td>92.43</td>
</tr>
<tr>
<td>1250</td>
<td>1 &amp; 2</td>
<td>146.85</td>
</tr>
<tr>
<td>1000</td>
<td>2 &amp; 3</td>
<td>146.65</td>
</tr>
<tr>
<td>750</td>
<td>3 &amp; 4</td>
<td>188.30</td>
</tr>
<tr>
<td>500</td>
<td>4 &amp; 5</td>
<td>145.87</td>
</tr>
<tr>
<td>250</td>
<td>5</td>
<td>32.86</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>752.95</td>
</tr>
</tbody>
</table>

As an example, for a freezing index 1250, the contributing regions are half of region 1 and half of region 2 as shown in Figure A.1. Therefore, the weight contribution of 1250:

\[
\left(\frac{0.123237 + 0.111715}{2}\right) \times 1250 = 146.845
\]

Repeating the process for each Freezing index and region and summing the value, the Average freezing index for the state of New York is Fr=752.95°Fday.