MECHANISTIC CRC PAVEMENT DESIGN

Dan Zollinger, Ph.D., PE
Texas A&M University
CE/TTI Building, Room 503e
College Station, TX 77843-3136

ABSTRACT
Traditionally, thickness design of continuously reinforced concrete (CRC) pavements has been more or less based on jointed concrete pavement behavior that has incorporated a procedure to address the design of the steel reinforcement relative to the development of the crack pattern. Unfortunately, this approach has carried little emphasis on how characteristics of the crack pattern influence pavement performance. However, developments over the last 10 years associated with mechanistic design approaches for CRC pavements have led to rational relationships between the design parameters and pavement response to better enable pavement performance predictions. With regard to CRC pavement, performance relations that relate the design parameters of crack spacing distribution, crack width, load transfer efficiency of the transverse cracks, and support conditions to pavement response under traffic and environmental loading provide the means to adequately carry out design analysis that is much more oriented to the behavior of a CRC pavement system. Consequently, an approach can be formulated that results in more realistic designs for CRC pavements.

INTRODUCTION
Continuously reinforced concrete (CRC) pavement is portland cement concrete (PCC) pavement that is constructed with continuous longitudinal steel reinforcement which requires no sawcutting of transverse contraction joints. Shortly after placement, CRC pavement develops a transverse cracking pattern, typically spaced at 0.6 to 1.8 m (2 to 6 ft) that is fully developed within a 2-year period. The purpose of steel reinforcement in CRC pavement is to ensure that the transverse cracks are tightly held together and providing high load transfer over the life of the pavement.

The cracking pattern is affected by many factors, including the environment conditions at the time of construction, the amount and depth of steel reinforcement, cement content and type of coarse aggregate, friction between the slab and the subbase, and concrete strength. Experience indicates the greater the amount of steel reinforcement, the closer and tighter the transverse cracks. The design of continuously reinforced concrete (CRC) pavements has been traditionally mechanistic relative to the design of the steel reinforcement and the development of the crack pattern but with little rationale behind the influence of the slab thickness on pavement performance.

The major structural distress of CRC pavement is punchout, which consists of an area enclosed by two closely spaced transverse cracks, a short longitudinal crack, and the edge of the pavement or a longitudinal joint that is associated with loss of support.
CRCP Design Features

Slab Thickness
This is an important design feature from a performance and slab stiffness standpoint. In general, as the slab thickness of a CRC pavement increases the capacity to resist critical bending stresses increases as does the slabs capability to transfer load across the transverse cracks. Consequently, as slab thickness increases, performance improves since slab stiffness increases concomitant to enhanced load transfer capability afforded by the additional slab thickness. Slab thickness must be selected within the context of other design features including transverse crack width, percent steel reinforcement, PCC mixture properties, and base type and stiffness not to mention weather conditions at the time of construction. In other words, depending upon the construction conditions, coarse aggregate type, one slab thickness may be adequate for a given set of other design features but not for another set of design features. The goal is to select the minimum thickness that provides acceptable levels of aggregate interlock wear-out, punchout development, and smoothness (IRI) over the design period at the desired level of reliability.

Transverse Crack Width
The width of the transverse crack is the primary CRC pavement design feature. It is the feature that is fundamental to many aspects of CRC pavement performance and is used to determine the amount of steel used for design. The smaller the crack width the greater the capacity of the crack to carry shear stress between adjacent slab segments. Crack width is a key focus of the crack pattern since it has a dominant role in controlling the degree of load transfer provided at the transverse cracks. Ultimately, the crack width average and distribution control the life and the quality of CRC pavement performance.

Longitudinal Reinforcement
Longitudinal steel is an important design parameter since it is used to control the opening of the transverse cracks. It is also critical from the standpoint of its effect on crack spacing. Field studies have shown that the longer the crack spacing the greater the potential of widened transverse cracks. The reinforcement in CRC pavement causes a restraining effect to contraction strain that increases as the percentage of steel increases and, relative to the Q value (4p/db), is one of the major steel-related design factors affecting crack development. Decreased crack spacing is associated with increased steel percentages. U.S. experience has indicated that steel percentages of 0.55 to 0.70 have provided suitable cracking patterns and performance in CRC pavement systems. In this regard, it is important to consider the effect of steel content. Steel content is determined within the context of several other design features such as slab thickness, crack width, PCC materials properties, and base type and stiffness. In other words, a specific percentage of steel may be adequate for a given set of other design features, and inadequate for another set of design features particularly with respect to different coarse aggregate types. The goal is to select the minimum steel content that provides an acceptable level of transverse crack widths and resistance to punchout development over the design life at the desired level of reliability. The direct consideration of top down
cracking has made base support, load transfer, and crack width even more critical. Truck axle loading causes a longitudinal bending stress at the top of the slab that increases as the load transverse diminishes over time that can lead to serious top down transverse cracking.

Transverse Crack Load Transfer Efficiency
The load transfer of transverse cracks is a critical factor in controlling development of longitudinal cracking. Field studies have shown that close transverse crack patterns are associated with small crack widths that maintain a high resistance to wear-out of aggregate interlock. Maintaining load transfer of 92% or greater will minimize loss of aggregate interlock over the design life of the pavement and limit the development of punchout distress. Punchout distress is the most critical factor in controlling roughness of CRCP. Joint load transfer can be selected within a variety of combinations of several other design features including slab thickness, percent steel, crack width, crack spacing, PCC materials properties, and base type and stiffness. The goal is to select the crack width and slab thickness corresponding to a load transfer of 92% at a suitable level of steel that provides an acceptable level of punchout development and smoothness over the design life at the desired level of reliability.

CRC Pavement Punchout Mechanism
The causes and factors associated with punchout development in CRC pavement have been the topic of many investigations (1 to 7). One of the first studies, by LaCourserie and Darter (3, 4), describes the mechanism of edge punchout based on the field investigations of punchout distress in CRC pavement in Illinois. This study showed the development of high tensile stress at the top of the slab about 1 m from the longitudinal edge of the slab as a result of poor load transfer at the surrounding transverse cracks. Crack spacing has also been shown to significantly affect the magnitude of the critical tensile lateral stresses on the top of the slab. However, no mechanistic relationship was established between crack spacing and level of load transfer efficiency across the transverse cracks.

Zollinger et al. (7) reported that punchouts in field studies were invariably accompanied by severe subbase erosion and loss of support. As was pointed out by Zollinger and Barenberg (6), poor support conditions can cause rapid deterioration of load transfer capacity due to excessive shear stresses induced by high deflection. Accordingly, it is assumed that environmentally induced upward slab curling and warping in the transverse direction, coupled with loss of load transfer along the transverse cracks, also contribute to high tensile stresses at the top of the slab. Whether the tensile stress is load or environmentally induced, high values will not result unless the load transfer stiffness is significantly diminished.

Deterioration of load transfer effectively isolates the loaded portion of the slab between the deteriorated transverse cracks from the adjacent pavement. As a result, only a narrow strip of concrete bound by two transverse cracks carries the wheel load. This situation leads to development of high top tensile stresses. As repetitive heavy truck loading continues, a short longitudinal fatigue crack forms between the two transverse cracks.
Any further wheel loads cause the portion of the concrete slab bounded by the transverse cracks to develop a short longitudinal crack, and the pavement edge to break off and settle into the eroded area resulting in an edge punchout.

CRACK SPACING CHARACTERISTICS AND PUNCHOUT POTENTIAL

The analysis of the LTPP data for CRCP sections indicates that the punchouts potential is greater in cases where large variability in crack spacing exists resulting in higher probability of short cracking intervals being positioned next to the wide cracking intervals that results in cluster cracking. The LTPP CRCP cracking data obtained from automatic video surveys were used to analyze the relationship between mean crack spacing and standard deviation of crack spacing. The analysis indicates that LTPP CRCP sections with larger crack spacing usually have larger standard deviation of crack spacing, as shown in Figure 1. The method of determining the mean crack spacing and standard deviation is shown in Appendix A.

The size of the CRCP panels that exhibited punchout was analyzed using LTPP data. The results of the analysis, presented in Figure 2, indicate that majority of punchouts develop on the CRCP panels about 0.3 to 0.6 m (1 to 2 feet) wide.

MODELING LOAD TRANSFER

As was indicated by previous investigations, loss of transverse crack load transfer is a precursor of the punchout development. As load transfer wears out, top transverse stresses increase potentially leading
to the formation of a short longitudinal crack between two adjacent transverse cracks as a result of heavy axle load repetitions. Consequently, a relation between the CRC pavement design parameters such as crack spacing, crack width, and load transfer efficiency of the transverse cracks is needed to adequately predict punchouts as a function of traffic.

Crack load transfer efficiency (LTE) due to aggregate interlock can be determined based on:

- **Crack width** – depends on crack spacing, concrete set temperature, steel content, PCC shrinkage and temperature change, and subbase friction
- **Aggregate interlock wear-out** – depends on crack width and governs a cracks’ ability to transfer applied loads from one side of the crack to the other

**Crack width component**
The width of the transverse crack is fundamental to many aspects of CRC pavement performance, since it plays a dominant role in controlling the degree of load transfer provided across a serves as criteria for the required design steel content. Crack width is affected by several time-dependent design parameters, as shown in the following formula for a single layer of steel:

\[
cw_{ki} = L_k \left( \varepsilon_{shr i} + \alpha_{PCC} \Delta T_{\text{zm}} \right) - L_k \frac{c_{2ki}}{E_{PCCI}} \left( \frac{L_k U_m P_b}{c_{1ki} d_b} + C \sigma_0 \left( 1 - \frac{2h_s}{h_{PCC}} \right) + \frac{L_k}{2} f \right)
\]  

Where

- \( cw_{ki} \) = Average crack width at the depth of the steel for each time increment \( i \) and crack spacing \( k \), mm (mils)
- \( L_k \) = \( k \)th crack spacing, mm
- \( \varepsilon_{shr i} \) = Unrestrained concrete drying shrinkage at the depth of the steel for each time increment \( i \) and crack spacing \( k \)
- \( \alpha_{PCC} \) = Concrete CTE, \( ^{\circ} \text{C}^{-1} \left( ^{\circ} \text{F}^{-1} \right) \)
- \( \Delta T_{\text{zm}} \) = Seasonal drop in PCC temperature at the depth of the steel \( ^{\circ} \text{C} \left( ^{\circ} \text{F} \right) \)
- \( c_{1ki} \) = First bond stress coefficient for time increment \( i \) and crack spacing \( k \) – see Appendix A (13)
- \( c_{2ki} \) = Second Bond stress coefficient for each time increment \( i \) and crack spacing \( k \) (22) (typical range = 0.7 to 0.9)

\[ a_i = 0.7606 + 1772.5 \left( \varepsilon_{\text{tot-}\varphi} \right) - 2e06 \left( \varepsilon_{\text{tot-}\varphi} \right)^2 \]
\[ b_i = 9e08 \left( \varepsilon_{\text{tot-}\varphi} \right) + 149486 \]
\[ c_i = 3e09 \left( \varepsilon_{\text{tot-}\varphi} \right)^2 - 5e06 \left( \varepsilon_{\text{tot-}\varphi} \right) + 2020.4 \]
\[ \varepsilon_{\text{tot-}\varphi} \] = Total strain at the depth of the steel for the time increment \( i \) (typical range = 150 to 600 micro-strains)
\( k_{1i} \) = Bond slip coefficient
\( L_k \) = \( k \)th Crack spacing, in
\[ E_{\text{PCC}_i} = \text{Concrete modulus of elasticity for the time increment } i, \text{ kPa (psi)} \] (12)

\[ P_b = \text{Percent steel, fraction} \]

\[ d_b = \text{Reinforcing steel bar diameter, mm (in)} \]

\[ U_m = \text{Peak Bond Stress, kPa (psi)} \] (13)

\[ h_{\text{PCC}} = \text{PCC slab thickness, mm (in)} \]

\[ h_s = \text{Depth to steel, mm (in)} \]

\[ f = \text{Subbase friction coefficient based on subbase type from test data or using AASHTO recommendations.} \]

\[ C = \text{Bradbury’s correction factor for slab size (14)} \]

\[ \sigma_{0,ki} = \text{Westergaard nominal environmental stress factor for slab curling and warping for each time increment } i, \text{ kPa (psi)} \]

\[ = \frac{E_{\text{PCC}}\varepsilon_{\text{tot-}}\Delta m}{2(1 - \mu_{\text{PCC}})} \] (2)

where

\[ \mu_{\text{PCC}} = \text{Poisson’s ratio} \]

\[ \varepsilon_{\text{tot-}}\Delta m = \text{Equivalent total strain difference between the pavement surface and slab bottom (15)} \]

For any given project, crack widths vary widely along the project from crack to crack. One may consider this variability to correlate with the variability in crack spacing. Figure 3 shows differences in crack width predicted for 3 different crack spacing, assuming all other parameters to be constant. Fluctuations in crack width over the design life are affected by changes in thermal and moisture strains for different environmental seasons. However, gradual crack width opening is attributed primarily to drying shrinkage.

**Aggregate interlock wear-out component**

The ability of a crack to carry load is described in terms of shear capacity, which is

![Figure 3. Time history of changes in crack opening over pavement life predicted using data for LTPP GPS-5 Section 175849 in Illinois (Crack spacing are indicated) (24).](image-url)
directly related to aggregate interlock and the thickness of the slab. As a crack opens and closes, its ability to transfer shear load or shear capacity can be described using the following relation (7):

\[ s_{oi} = 0.0312 \cdot (h_{PCC})^{1.4578} \cdot e^{-(0.032)cw_i} \]  

(3)

Where
- \( s_{oi} \) = Dimensionless seasonal shear capacity based on crack width
- \( h_{PCC} \) = Thickness of the slab, mm (in)
- \( cw_i \) = Crack width as a function of time from equation (1), mm (mils)

Based on formula (3), shear capacity varies seasonally with the crack opening and affects crack load transfer efficiency over the life of the pavement. However, as the concrete slab is subjected to axle load applications, vertical crack surfaces are subjected to repetitious shear loading between the two sides of the crack that leads to aggregate wear-out and decreases crack load transfer capacity. Therefore, the crack shear capacity computed in formula (3) is reduced each time a load is applied across a crack. The total shear capacity of the transverse cracks for any given instance in pavement life \( i \) can be characterized using the following formula (7):

\[ s_i = s_{oi} - \sum_{j=1}^{i-1} \sum_{j} \left(0.069 - 2.75 \cdot e^{-cw_j/h_{PCC}} \left(\frac{n_{ji}}{10^6} \right) \left(\frac{\tau_{ij}}{\tau_{ref_i}}\right)\right) \]  

(4)

Where
- \( s_i \) = Total crack shear capacity at time increment \( i \).
- \( s_{oi} \) = Crack shear capacity based on crack width for time increment \( i \).
- \( cw_i \) = Crack width for time increment \( i \).
- \( h_{PCC} \) = Thickness, mm (in)
- \( n_{ji} \) = Number of axle load applications for time increment \( i \), load level \( j \).
- \( \tau_{ij} \) = Shear stress on the transverse crack at the corner due to load level \( j \) applied during time increment \( i \), kPa (psi)
- \( \tau_{ref_i} \) = Reference shear stress derived from the PCA test results for time increment \( i \), kPa (psi)

Coefficients in the equation (3) and (4) were modified from the original values used in the reference (7) based on the analysis of the additional data from CRCP sections from LTPP GPS-5 experiment. Limited verification of the wear-out model is presented in Appendix B.

Load transfer prediction model

To relate crack shear capacity to crack load transfer efficiency, an intermediate parameter that is a function of the aggregate interlock factor (AGG) called a \( J \) factor is employed (16):

\[ \log(J_{ekl}) = ae^{-\frac{J_{ekl}}{\epsilon}} + de^{-\frac{skf}{\tau}} + ge^{-\frac{J_{ekl}}{\epsilon}} \cdot e^{-\frac{skf}{\tau}} \]  

(5)

Where
\[ J_{c_{ki}} = \text{Stiffness of the transverse crack for time increment } i \text{ and crack spacing } k. \]

\[ = \text{AGG/}kl \text{ (dimensionless aggregate interlock factor)} \]

\[ s_{ki} = \text{Dimensionless shear capacity for time increment } i \text{ and crack spacing } k. \]

\[ J_s = \text{Stiffness of the shoulder/slab longitudinal joint with suggested values noted in the table below.} \]

**Table 1. Stiffness of the shoulder/slab longitudinal joint**

<table>
<thead>
<tr>
<th>Shoulder Type</th>
<th>( J_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granular</td>
<td>0.04</td>
</tr>
<tr>
<td>Asphalt</td>
<td>0.04</td>
</tr>
<tr>
<td>Tied PCC</td>
<td>4</td>
</tr>
</tbody>
</table>

Load transfer efficiency on the transverse crack at any instance of time can be found on the basis of the \( J_{c_{ki}} \) parameter using modified formula originally developed by Ioannides (17), as following:

\[
LTE_{TOT_{ki}} = 100 \left(1 - \frac{1}{1 + \log^{-1} \left[ \frac{1}{0.214 - 0.183 \frac{a}{\ell_i} - \log(J_{c_{ki}}) - R} / 1.18 \right]} \right)
\]

(6)

where

\( LTE_{TOT_{ki}} = \text{Total crack LTE due to aggregate interlock, steel reinforcement and base support for time increment } i \text{ and crack spacing } k, \% \)

\( \ell_i = \text{Radius of relative stiffness computed for time increment } i, \text{ mm (in)} \)

\( a = \text{Radius for a loaded area, mm (in)} \)

\( R = \text{Residual dowel-action factor to account for residual load transfer provided by the steel reinforcement.} \)

(#5 bar: 0.5, #6 bar: 1.0, #7 bar: 1.5)

Combining equation (5) with equation (6), the effect of stiffness due to aggregate interlock can be taken into account as illustrated in Figure 4. The effects of aggregate interlock are shown as a function of joint or crack opening. The achievement of a greater load transfer capability can only be accomplished through aggregate interlock and small crack openings. In other words, high load transfer conditions are achieved through aggregate interlock. Perhaps the presence of steel reinforcement makes some contribution to the transfer of load from one segment to another but it is clear that crack width is critical to achieving and maintaining high load transfer conditions.
Performance Criteria for the Design of CRC Pavement Systems

As previously noted, the potential for punchouts may be greater in cases where combinations or short and long crack spacings occur and the loss of shear capacity is excessive. However, CRC pavements with predominantly short crack spacing do not necessarily dictate that poor performance will be the end result, particularly where the support conditions are good and design thickness is adequate. As suggested in equations (1) through (6), combinations of crack spacing and slab thickness can assure proper levels of stiffness at the transverse cracks in a CRC pavement.

It can be stated that in CRC pavement systems, good performance goes hand in hand with adequate stiffness at the transverse cracks. This can not only be assured through adequate crack width/thickness combinations, but also through uniform crack opening and crack space distribution, in addition to the provision of uniform support conditions throughout the life of the pavement. Steel reinforcement also serves to sustain the stiffness of the transverse cracks through crack width as it may be affected by the crack spacing which is influenced by temperature drop and drying shrinkage. The procedure outlined in Appendix A serves to generate steel percentage – crack width relationships for different reinforcement bar sizes and configurations. Figure 5 shows maximum crack width for a given slab thickness to provide sufficient stiffness to prevent aggregate wear-out. This figure demonstrates crack width requirements relative to slab thickness and load transfer requirements. It should be noted that the limits shown in Figure 5 fall between those recommended by Permanent International Association of Roads Congresses (PIARC) (0.5mm [20 mils]) (22) and those recommended by AASHTO (1mm [40 mils]) (23). Figure 5 suggests that the PIARC requirements are too conservative for typical CRC pavement thickness design. From the steel percentage – crack width relationships and a given design slab thickness, a maximum crack width can be associated with a selected design steel percentage. Figure 6 shows how loss of shear capacity

Figure 5. Crack Width – Thickness Combination to Achieve 91% Load Transfer Efficiency (16).

Figure 6. Relationship between LTE and Loss of Shear Capacity.
MECHANISTIC MODELING OF CRITICAL TENSILE STRESSES

To model CRC pavement load response for design purposes, a finite element model, ISLAB2000 was used to develop a load stress algorithm to compute wheel load stress as a function of slab thickness and LTE. Other factors such as seasonal moisture and nighttime temperature gradients through concrete slab thickness can also be considered. Axle loading applied to the central portion of CRC segment in the outside lane yields the critical stress for design purposes. A graphical example of the stress output is shown in Figure 7. The critical top tensile stresses in the longitudinal direction were determined for the various combinations of material characteristic, environmental parameters, and axle loading.

Combination of crack spacing and crack LTE were found to have the most significant affect on CRC pavement load response. Narrow crack spacing coupled with low LTE produce high top tensile stress (as shown in Figures 8 and 9) that could lead to punchout development. This theoretical finding is strongly supported by results from analysis of LTPP data indicating the average width of the PCC segment developing into a punchout is 0.43 m (17 inches). Load stresses on narrow CRC segments are magnified under the influence of slab curling or erosion along the pavement-shoulder edge, as demonstrated in Figures 10 and 11.

Transverse wheel-load stresses should be included in a thickness design process for CRC pavement systems. Using analysis noted above, a database of maximum transverse
wheel-load stresses was generated for a CRC pavement system consisting of a bituminous shoulder and to a lesser degree with other shoulder types. The variation of wheel load stress with load transfer efficiency and thickness is based upon a cracking interval of 0.6 m (2 ft). Transverse wheel-load stresses in a CRC pavement system are therefore, at a minimum, a function of crack spacing and shoulder configuration. (under a free edge condition) for a variety of thicknesses, load transfer efficiencies, and crack spacings. The contribution of bending stresses to fatigue damage is negligible prior to wearout of the aggregate interlock and concomitant loss of load transfer. The level of load transfer may also affect the maximum stress location in a CRC pavement system (independent of environmental transverse stresses) for a CRC pavement with a bituminous shoulder as follows:

$$s = \{a + b \ln \left( \frac{L}{\ell} \right) \}^{-1} \quad (7)$$

where
Figure 10. Effect of loss of support along the edge on critical CRCP stresses for a model with 0.6 m (2 ft) crack spacing, PCC thickness=203 mm (8 in), $E=27.5$ GPa ($4,000,000$ psi), $k=54.2$ MN/m$^3$ ($200$ pci) (24).

Figure 11. Effect of environmental slab curling on critical CRCP stresses for a model with 0.6 m (2 ft) crack spacing, PCC thickness=203 mm (8 in), $E=27.5$ GPa ($4,000,000$ psi), $k=54.2$ MN/m$^3$ ($200$ pci) (24).

\[
a = \exp(-0.930 + 2.84\{1 + \exp[-(L/96.4)/24.6]\})^{-1}
\]
\[
b = (0.427 + 9.73 \times 10^{-7} L^{3.5})
\]
\[
L = \text{mean crack spacing (L)}
\]
\[
\ell = \text{radius of relative stiffness (L)}
\]
\[
\text{LTE} = \text{load transfer efficiency (\%)}
\]
\[
\sigma_{\text{wls}} = \text{wheel load stress (FL}^2)\]
\[
h = \text{pavement thickness (L)}
\]
\[
P = \text{wheel load (F)}
\]
Total stresses will include $\sigma_{\text{wls}}$ along with transverse curl and warping-related stresses.

**FATIGUE DAMAGE MODELING**

Using the critical wheel load stresses, accumulated fatigue damage is estimated and used to predict punchout distress. According to Miner’s hypothesis, each truck axle load passage contributes to overall pavement damage that accumulates over time. For CRC pavements Miner’s hypothesis is implemented in the following form (cracking failure mode) (24):

$$D_{ij} = \sum_i \sum_j n_{ij} / 10^{2.13 \left( \frac{MR_i}{\sigma_{ij}} \right)^{1.2}}$$

(13)

where

- $n_{ij} =$ Number of single or tandem axle loads of the $j$th magnitude applied during time increment $i$
- $MR_i =$ Modulus of rupture during time increment $i$, kPa (psi)
- $\sigma_{ij} =$ Bending stress computed for design conditions of time increment $i$, kPa (psi)

Since the damage prediction is cumulative as a function of tensile stresses at different instances over the life of the pavement, accurate prediction of stress changes over the design life is important. The analysis period is subdivided into time increments based on subgrade support and climatic conditions relative to their effect on crack width and load transfer. The example in Figure 12 demonstrates differences in predicted accumulated damage for 4 different percent of steel reinforcement. Different percent steel resulted in different mean crack spacing and, hence, different LTE of the transverse cracks. All other design inputs and traffic conditions were held constant for all four combinations.

**Figure 12.** Comparison of cumulative damage predicted over time for 4 models with different steel percent (24).

The damage determination is used to predict the probability of punchouts for each crack spacing $k$ as:

$$POU_{yk} = 100e^{-x^{(\text{CESAL}_{yk})}}$$

where
\( POU_k = \) probability of punchouts based on the assumption of uniform \( k^{th} \) crack spacing.

\( D_k = \) maximum accumulated fatigue damage (due to slab bending in the transverse direction) for the \( k^{th} \) crack spacing.

\( b = \) cracking calibration constant (determined from calibration, 1.7534 as shown in Figure 13)

\( m = \) cracking calibration constant (determined from calibration, -0.0141 as shown in Figure 13)

The number of punchouts associated with individual cracking intervals is determined according to:

\[
PO_k = POU_k \times CI_k
\]

where

\( PO_k = \) the average number of potential punchouts in the \( k^{th} \) cracking intervals in 1 mile of pavement

\( POU_k = \) maximum probability of punchouts based on the uniform \( k^{th} \) crack spacing.

\( CI_k = \) the number of possible \( k^{th} \) cracking intervals in 1 mile of pavement

The total number of punchouts over the full range of cracking intervals is accumulated based on the accumulated probabilities of each cracking interval. Given the amount of damage, the number of punchouts could be estimated, on the average. The number of punchouts at a given level of reliability (\( PO_R \)) depends upon the variance associated with the punchout prediction. Given that the variance of punchout can be defined, the punchouts at any level of reliability (assuming a normal distribution):

\[
PO_R = PO_k + Z_R \sigma_{PO} = PO_k + Z_R COV[PO] PO_k = PO_k (1 + Z_R COV[PO])
\]

where

\( COV[PO] = \) Normal Standard Deviate

\( \sigma_{PO} = \) Punchout Standard Deviation

\( VAR[PO] = \left( - \frac{PO_k}{L} \right)^2 VAR[L] + \left( - \frac{PO_k m}{D} e^{b + m L (\bar{L})} \right)^2 VAR[D] \)
The variance of punchouts (VAR[PO]) depends on the variance of the crack spacing (VAR[L]) and the variance of the fatigue damage (VAR[D]). Both of these affect the standard deviation of the punchouts and the level of performance associated with a given design reliability. The variance of the cracking pattern is provided in Appendix A but the variance of damage is not provided due to the complexity associated with its deviation, however if the variability of the cracking pattern can be reduced, the level of performance will increase; as is the case with the fatigue damage variability. There are several material, load, and environmental factors that effect the variance of damage such as load stress, LTE, crack spacing, foundation support, thermal expansion, total shrinkage, etc.

**SUMMARY AND CONCLUSIONS**

A method for predicting CRC pavement performance is based on a process associated with the development of punchout distress. Development of punchout distress is directly related to the formation of a longitudinal crack between two adjacent transverse cracks and the amount of erosion present. This crack initiates at the top of the slab and propagates downward through the CRC slab. The development of the longitudinal crack is, in turn, related to the accumulated fatigue damage caused by a slab bending in the transverse direction.

Based on field and analytical results, previous studies have shown the critical bending stress is located at the slab top located about 1 to 1.5 m (40 to 60 in) from the slab edge. The higher this stress, the greater the fatigue damage and the potential for punchout development. It is critical to control the development of punchouts in CRC pavement to ensure desired performance. Successful CRC pavement design can be assured through adequate crack width/thickness combinations, uniform crack opening and crack spacing distribution, and the provision of uniform support conditions throughout the life of the pavement. To ensure that a CRC pavement design will perform as required over the design period, it is desirable to place limits on crack width/thickness combinations.

In terms of the behavior of CRC pavement, a structural model that can assess the effect of the steel on the crack width relative to the prediction of crack spacing for a given base friction, temperature drop, and applied drying shrinkage is needed to adequately design a CRC pavement system. Crack width varies directly with crack spacing (when all other parameters are constant) and increases with time over the design period.

Crack LTE modeled as a function of crack width, slab thickness, base and shoulder type, and applied traffic loading serve a useful purpose in simulating the punchout process. This process, which includes incremental crack LTE deterioration, is modeled mechanistically through simulation of shear deterioration as a result of the repetitive axle loading. The model was based on performance studies, which indicate very little LTE deterioration occurs over time if the crack width - thickness ratio is above 3.1 (corresponding to 92% LTE or above).

The analytical models for crack width, crack LTE, and subgrade support presented provide the necessary tools to determine incremental damage accumulation over the life of the pavement. The approach presented provides practical means for prediction of
changes in design parameters and associated pavement responses based on sound, mechanistic-based principles coupled with the results of laboratory observations and analysis of long-term pavement performance data.

REFERENCES


