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Continuously Reinforced Concrete Pavement

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A sampling-type crack survey was made in July 1969; 200 feet in each 1000 feet were sampled. The average interval between cracks was 5.78 feet. Paving was completed in late 1968. The average crack interval in July 1970 was 4.14 feet. The 1970 survey was also a sampling type.

It is apparent that cracking approached maturity or equilibrium during the first year. Apparently, too, most of the cracking occurred during the first few weeks after construction - which suggests curing shrinkage as a contributory cause. A discussion of the significance of time and temperature will follow the presentation of other survey observations.

A frequency-distribution graph of crack intervals was prepared (Figure 1) to illustrate the mode. Although no discrete mode is apparent, it is evident that there was a strong tendency for cracks to occur at intervals ranging between 1 and 6 feet. The dominant interval appears to be 2 feet. Typical cracks are illustrated in the accompanying photographs (Figures 2 through 6).

The initial overall roughness index of this project was 220; considering the outer lanes only, the initial roughness was 215. A rerun of the outer lanes in the summer of 1969 gave 220. A 1970 measurement has not been obtained yet but is scheduled. Project I 71-1(26)28, which was paved by the same contractor, had an initial roughness of 290.

Cracking in the 10-inch, conventional sections at the bridge approaches was quite normal.

A copy of the field strip charts is attached to the original and file copies only.

It is interesting to note that there was no apparent cracking in the first 85 feet from the beginning station (1742+50). Also, there was no obvious cracking in the last 25 feet approaching station 1866. Both are end situations, which are free to expand and contract to some extent. Several other end situations exist within this project, and all conform more or less to this pattern.
The two small sites which required patching were at Stations 1914+40 and 2010+44, in the northbound lanes. Figures 7 and 8 show the situation at Station 1914+40 before repairs; Figures 9 through 12 show stages of repair. Figures 13 through 16 show Station 2010+44 before, during and after repair. Figure 17 shows a spall at Station 1874+44, northbound, which evidently had been repaired previously but now shows additional damage. This site is in a 10-inch, conventional section approaching a bridge.

A deterring consideration associated with concrete patching is the customary necessity of barricading the area during the full curing period. Inasmuch as these were small areas and inasmuch as the patch was well constrained and reinforced, an attempt was made to patch with concrete containing minimum mixing water and thereby enable removal of the barricades before nightfall or within 24 hours. Unfortunately again, the first patch (Station 1914+40) was not placed until about 4 PM; and there was not sufficient time remaining that day for curing; the patch was covered overnight with a steel plate. The steel plate had some ribs on the bottom which grooved and perhaps cracked the surface of the patch. The curing membrane was a chlorinated rubber solution. The second patch was placed earlier in the day, and barricades were removed in the evening.

The close-spaced cracking of continuously reinforced pavements has not yet been explained fully; such a phenomenon presents a challenge. Yoder (1) gives the following equation, from Vetter (2):

\[ l = \frac{A_s(S'_c)^2}{np^2 \Sigma_0 u (E_C t - S'_c)} \]

where
\[ l = \text{interval between cracks}, \]
\[ S'_c = \text{ultimate tensile strength of concrete}, \]
\[ n = \frac{E_S}{E_C}, \]
\[ p = \text{percentage of steel/100}, \]
\[ \Sigma_0 = \text{perimeter of reinforcing bar}, \]
\[ A_s = \text{area of steel bar}, \]
\[ u = \text{bond strength}, \]
\[ E_C = \text{Young's modulus of concrete}, \]
\[ E_S = \text{Young's modulus of steel}, \]
\[ t = \text{temperature (drop)}, \] and
\[ C = \text{coefficient of thermal expansion} \]

Vetter's equation is fraught with difficulties; it does not yield realistic crack spacings. Other analytical theories studied are presented in other references (3, 4, 5). The logic employed in this report follows.

Respecting continuity of strain,
\[ e_s = e_c \quad \text{and} \quad \sigma_0 = \sigma_c. \]

Allowing free expansion and contraction (no external forces),
\[ \frac{\Delta \sigma_s}{\Delta t} = \frac{\Delta \sigma_c}{E_s} + \frac{\Delta \sigma_c}{E_c} \]

where 
\( \Delta \sigma_s / \Delta t = C_s = \) coefficient of thermal expansion of steel \((6.5 \times 10^{-6}/\text{°F})\),
\( \Delta \sigma_s / \Delta t = C_c = \) coefficient of thermal expansion of concrete \((5.5 \times 10^{-6}/\text{°F})\),

\( E_s = \) modulus of elasticity of steel \(= 30 \times 10^6 \) psi,
\( E_c = \) modulus of elasticity of concrete \(= 5 \times 10^6 \) psi,

\( \Delta \sigma_s / E_s \) and \( \Delta \sigma_c / E_c \) = counter strains arising from resisting stresses \( \sigma_s \) and \( \sigma_c \),

\( \Delta \sigma_s = \Delta \sigma_c / A_s \) (for balancing of forces), and

\( A_s = .00677 \) square inch of steel per square inch of concrete.

Substituting and simplifying:
\[ \Delta \sigma_c = \left(C_s \cdot C_c\right) \frac{(t_2 - t_1)}{4.76 \times 10^{-6}} = 1 \times 10^{-6}(t_2 \cdot t_1) / 4.76 \times 10^{-6} \]
\[ \Delta \sigma_c = .195 (t_2 \cdot t_1) \]
\[ \Delta \sigma_s = .195 (t_2 \cdot t_1) / .00677 = 28.8 (t_2 \cdot t_1). \]

\( \Delta \sigma_c \) is the stress rise in the concrete per unit of length. The stress rise in the steel is \( \Delta \sigma_s \). Therefore:

\[ \Delta \sigma_c / \Delta L = d \sigma_c / dL = .195 (t_2 \cdot t_1). \]

Upon integrating,
\[ \sigma_c = .195 (t_2 \cdot t_1) L + C. \]

When \( t_2 \cdot t_1 = 0, C = 0. \) Thus
\[ \sigma_c = .195 (t_2 \cdot t_1) L \] and
\[ \sigma_s = 28.8 (t_2 \cdot t_1) L. \]

When \( \sigma_c(\text{max}) = 600 \text{ psi} \) and \((t_2 \cdot t_1) = 100^\circ \text{F},\)
\[ 600 = .195 \times 100 \times L \]
\[ L = 30.8 \text{ inches}. \]

Also, when \( \sigma_s(\text{max}) = 90,000 \text{ psi} \) (compressive) and \((t_2 \cdot t_1) = 100^\circ \text{F},\)
\[ 90,000 = 28.8 \times 100 \times L' \]
\[ L' = 31.3 \text{ inches}. \]

It seems evident that the distance between cracks may approach 2L. Surely, the interval lies between
L and 2L unless the yield point of the steel is exceeded beforehand.

The foregoing derivation is compatible with other observed phenomena attending continuously reinforced slabs. The crack interval increases in direct proportion to the tensile strength of the concrete and decreases with greater percentages of steel. The versatility of the equations may be extended by superimposing prestresses (due to shrinkage or otherwise) as a bias upon the temperature range. Of course, a prestress, if known, may be subtracted from the limiting critical stresses.

An interesting aspect of this theory is the implication that cracks occur only in a rising-temperature situation.

This theory implies that cracking will occur at a normal distance (between L and 2L) from an end situation. The cracks may not be readily apparent; closure would be almost unrestrained. At farther distances from an end condition, the restraint increases, and closure becomes imperfect.

Apparently, the end movement is in the range between 0.75 and 1.5 inches and varies seasonally. It may be difficult to understand how critical strains or stresses develop. The concept of one or more standing waves or wave fronts along the axis of the pavement may prove satisfying in that respect.

The derivation presented here finds independent verification in the equation for bond strength. Although the bond strength is rational in a constitutive sense, it has some empirical foundation. Simply stated,

$$\sigma_0(\text{max}) = \frac{u \Sigma_o L}{A_s}$$

where

- $u =$ bond strength (pounds per square inch),
- $\Sigma_o =$ perimeter of steel bar (inch),
- $L =$ length of imbedment (inch), and
- $A_s =$ area of steel bar (square inch).

When $A_s = \frac{.31}{\text{in}^2}, \Sigma_o = 1.963 \text{ in},$ and $u = 400 \text{ psi}, \sigma_0(\text{max}) = 60,000 \text{ psi}$ and $L = 60,000 \times \frac{.31}{400} \times 1.936 = 24 \text{ inches}.$

*Note: $\sigma_0/\text{max}/L =$ stress rise per unit of length.*

Further verification of the cracking phenomena may be forthcoming from the several miles of continuously reinforced pavement planned for I 275. Specific attention will be given to the time and rate of crack development.

Corrosion potentials were measured at Station 1915+05, where a patch was made. Exceptionally high potentials were found in the vicinity of the patch but then reduced to a passive level at more remote distances.

Attachments (to original and file copies only)

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LIST OF REFERENCES


2. Vetter, V. P., Stresses in Reinforced Concrete Due to Volume Changes, Transactions, ASCE, Vol. 98, 1933.

3. Teng, T. C. and Coley, J. O., Continuously Reinforced Concrete Pavement Observation Program, Mississippi State Highway Department, Research and Development Division, 1968.


Figure 1 - Frequency Distribution of Crack Intervals

Figure 2 - Typical Cracks
Figure 3 - Typical Cracks

Figure 4 - Typical Cracks
Figure 5 - Typical Cracks

Figure 6 - Typical Cracks
Figure 7 - Sta. 1914 + 40 Prior to Repairs

Figure 8 - Sta. 1914 + 40 Prior to Repairs
Figure 9 - Removal of Pavement at Sta. 1914 + 40

Figure 10 - Sta. 1914 + 40 Prior to Replacement of Steel
Figure 11 - Placement of Concrete at Sta. 1914 + 40

Figure 12 - Screeding Concrete at Sta. 1914 + 40
Figure 13 - Temporary Patch at Sta. 2010 + 44

Figure 14 - Sta. 2010 + 44 Prior to Placement of Steel and Concrete
Figure 15 - Finishing Concrete at Sta. 2010 + 44

Figure 16 - Completed Patch at Sta. 2010 + 44
Figure 17 - Spall at Sta. 1874 + 44