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Continuous Steel Reinforcement for Experimental Concrete Pavements

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Continuous Steel Reinforcement for Experimental Concrete Pavements

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Contents

Analysis of Special Problems in Continuously-Reinforced Concrete Pavements

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In an attempt to fill gaps in the fundamental understanding of the behavior of continuously-reinforced concrete pavements, a number of studies on various topics are presented. Most are presented from a theoretical analytical viewpoint; but some, which are not readily adaptable to analysis, are presented from an experimental viewpoint. Some of the topics studied are pavement thickness, differences in behavior of deformed bars and plain wire mesh, buckling tendencies, horizontal and vertical alignment changes, end anchorage, crack behavior under repeated loading, and reduced slab rigidity due to cracks.

● ALTHOUGH continuously-reinforced highways pavements have been built in many states, the behavior of these pavements is still not fully understood. This is not to say, however, that the general behavior characteristics of why and how such a pavement functions under expansion and contraction are not known, for these facts have already been presented (1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12). It is assumed that the general behavior of such pavements is already known, therefore only additional problems which heretofore have not been considered with any degree of thoroughness are discussed.

Aspects of the continuously-reinforced design discussed in this paper are the following:

1. Pavement thickness.
2. Comparison of behavior between deformed bars and plain wire mesh:
   (a) Percent steel.
   (b) Crack width.
3. Buckling tendency.
4. Movement on horizontal curves.
5. Movement on vertical curves.
6. Terminal anchorage configuration studies:
   (a) Strength.
   (b) Uplift.
7. Cracked slab behavior:
   (a) Reduced bending rigidity.
   (b) Increases in crack width under repeated loading.

Topics 1 through 5 are studied from a theoretical mathematical standpoint, which affords the advantage of generality but is, of course, limited by the validity of the initial assumptions made. General field observations have, however, shown the legitimacy of most of these assumptions.

Topics 6 and 7 are discussed through experimental laboratory data. An attempt is made in the discussion of these experimental tests to indicate the fundamental reasons and trends in such a way that reliable general conclusions may be obtained from them.

PAVEMENT THICKNESS

As shown by Vetter (13), the amount of reinforcing steel necessary in pavements is controlled by such changes in the pavement as shrinkage, moisture-induced swelling, and temperature. In contrast, the thickness of the pavement is controlled by the wheel loads. Westergaard's theories (14) are not entirely applicable in continuously-reinforced pavements, as such pavements have innumerable closely-spaced transverse cracks requiring a new approach to analysis based on cracked-slab behavior, rather than on homogeneous behavior.

Bending

Consider a long slab of thickness t, of width l, with transverse cracks closely spaced at a distance of b, which is assumed small in comparison with l (Fig. 1). Inasmuch as the concrete between cracks is still essentially homogeneous, this portion of slab is extracted as a free body and used as the basis of analysis in accordance with Figure 2, in which

K_1 is the elastic subgrade modulus per width b;
K_2 is the elastic restraint modulus of the adjacent segments transferred by the longitudinal steel; and
K_3 is the aggregate interlock modulus (assumed linear);

The differential equation of behavior of this slab segment is

$$D\frac{d^4 y}{dx^4} + K y = 0 \tag{1}$$

in which

$$D = \frac{E_c bt^3}{12}; \tag{2}$$

$$K = K_1 + K_2 + K_3; \tag{3}$$

E_c = modulus of elasticity of concrete;
and

y = deflection.

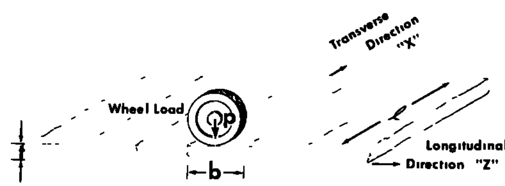

Figure 1. Cracked continuously-reinforced pavement.

The wheel load is positioned at the end to produce the maximum bending moment. The weight of the slab is neglected. The solution to Eq. 1 is

$$y = \frac{2P\beta}{K(\sinh^2 \beta l - \sin^2 \beta l)}$$

$$\Big[\sinh\beta l \cos\beta \;\; \cosh\beta \;\; (l-x)$$

$$\sin\beta \;\; l \cosh\beta \;\; \cos\beta \;\; (l-x)\Big] \tag{4}$$

in which

$$B = \sqrt[4]{\frac{K}{4D}} \tag{5}$$

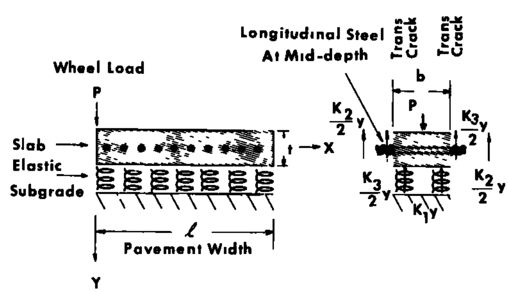

Figure 2. Slab segment.

The maximum bending moment, $D\left[\frac{d^2 y}{dx^2}\right]_{max}$, occurs near the end and is equal to

$$M_{max} = 0.32 \frac{P}{\beta} \tag{6}$$

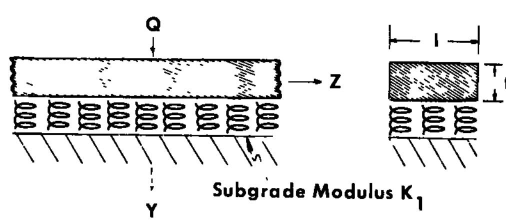

Figure 3. Longitudinal strip.

If the concrete is not to crack in bending, from simple homogeneous beam theory:

$$M_{max} = f_t \left[\frac{b\,t^2}{6}\right] \tag{7}$$

in which f_t is the allowable tensile stress of concrete. Thus, substituting for M_{max} and β, and solving for allowable thickness t based on bending:

$$t = \left[\frac{4.526\,E_c\,P^4}{(f_t)^4\,b^3\,(K_1 + K_2 + K_3)}\right]^{\frac{1}{5}} \tag{8}$$

The subgrade modulus, K_1, may be obtained from tests for small deflections. The aggregate interlock modulus may also be obtained from tests. Information on this value is lacking at present, but it is assumed to be dependent on the crack width. For a conservative analysis K_3 may be neglected. The value of K_2, the elastic restraint modulus, however, may be obtained semi-analytically in the following manner.

Consider a long segmented strip of slab in the longitudinal direction, of thickness t and of unit width. Note that this strip is not homogeneous, as it has transverse cracks. Extract this strip and consider it as a free body, as in Figure 3.

By inspection, the modulus K_2 may be defined as

$$K_2 = \frac{Q}{\delta_q} \tag{9}$$

in which Q is any load and δ_q is the deflection at the load Q. δ_q may be obtained from the differential equation

$$D_r \frac{d^4y}{dz^4} + K_1'y = 0 \tag{10}$$

for y at z = 0. Thus,

$$\delta_q = \frac{Q\ \beta 1}{2\,K_1'} \tag{11}$$

in which

$$\beta_1 = \sqrt[4]{\frac{K_1'}{4\,D_r}} \tag{12}$$

$$K_1' = \frac{K_1}{b} \tag{13}$$

$$D_r = \frac{E_r\,t^3}{12\,(1-\mu^2)} \tag{14}$$

μ = Poisson's ratio for concrete; and

E_r the reduced modulus of elasticity for cracked reinforced concrete (see section on, "Cracked Slab Behavior").

Therefore, upon substitution of these values

$$K_2 = \frac{2K_1'}{\sqrt[4]{\frac{K_1'}{4\,D_r}}} = \frac{2K_1}{b\sqrt[4]{\frac{K_1}{4\,b\,D_r}}} \tag{15}$$

Shear

For design use, a trial value of pavement thickness t may be had from Eq. 8, derived on the basis of bending. The slab should also be checked for shear or diagonal tension. Considering the same slab segment as in Figure 1, the critical position of load P for shear is also at the edge. Figure 4 shows the action of the essential forces, neglecting bending stresses.

From equilibrium of forces in the Y direction,

$$P - b(K_1 + K_2 + K_3)\int_0^t y\,dx - b\,t\,f_t = 0 \tag{16}$$

Thus, the allowable tension stress is controlled by

$$f_t = \frac{P}{bt} - \frac{(K_1 + K_2 + K_3)}{t}\int_0^t y\,dx \tag{17}$$

The value of y may be had from Eq. 4 or from published curves and tables found in such texts as "Strength of Materials II," by Timoshenko, or "Advanced Mechanics of Materials," by Seely and Smith, under the subject of beams on elastic foundations.

Example

As a numerical example of design, consider the following values:

$E_c = 3\text{x}10^6$ psi; $f_t = 90$ psi; b = 6 in. (assumed as tire contact length for limiting case); P = 12,000 lb; $K_1 = 10{,}000$ psi; $K_3 = 1{,}000$ psi; $E_r = 4{,}000$ psi; and $\mu = 0.225$.

Assuming a trial t=6 in., K_2 computed from Eq. 15 is 12,200 psi. t based on bending may then be computed from Eq. 8 as 6.07 in.

The allowable tensile stress f_t is next checked from Eq. 17 to be 121 psi. Because this exceeds the allowable stress of 90 psi, the trial thickness of 6.07 in. is too small. A revision based on Eq. 17 then increases the slab thickness to 6.7 in.

In practice, this thickness would probably be evened off to 7 in. Several continuous pavements of 7-in. thickness have been built in Illinois and have performed well in five years of service (15). This example problem is not intended to be used as a design criterion, but is presented simply to show the use of the basic equations.

This method of analysis for determining the pavement thickness thus provides a rational basis for determining the slab thickness. Certain refinements are still needed in the more complete understanding of the aggregate interlock force and reduced slab rigidity, both of which enter the slab thickness problem.

COMPARISON OF BEHAVIOR BETWEEN DEFORMED BARS AND PLAIN WIRE MESH

To simplify the comparison of deformed bars and plain wire mesh as much as possible, only their behavior in connection with shrinkage of concrete is discussed.

The basic behavior of deformed bars has been studied (13), so that only the behavior of plain welded wire mesh need be studied in this paper. However, to assist in the comparison, a summary of Vetter's results is presented for deformed bars.

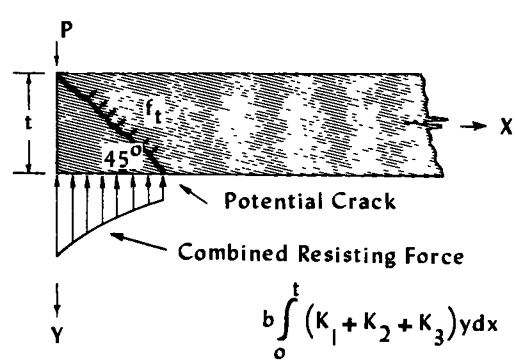

Figure 4. Diagonal tension.

Percent Steel for Deformed Bars

Vetter (13) showed that when reinforced concrete cracks due to shrinkage, the shrinking concrete grips the steel by bond in an extended region near the cracks, causing the concrete to go into tension. The bond force is assumed uniform in the region of grip near the cracks and zero in the central region between cracks. Tests have shown this to be a valid assumption. This action causes the steel near the cracks to go into tension and causes the steel in the central region between cracks to go into compression. For comparison studies it is important to stress the fact that the concrete slips a little in the region of the bond; but since the bars are deformed, bond forces continue to be developed.

Vetter found that for no shrinkage cracks to develop in reinforced concrete, the limiting value of the shrinkage coefficient z must be S'_c/E_c, in which S'_c is the tensile strength of concrete.

The limiting percentage of steel is found as

$$p = \frac{S'_c}{S_c + z\,E_s - n\,S'_c} \tag{18}$$

in which

$$p = \frac{A_s}{A_c} = \frac{\text{area of steel}}{\text{area of concrete}};$$

S_s = Elastic limit of steel;

E_s = Modulus of elasticity of steel; and

$n = E_s / E_c$.

The crack spacing L is found as

$$L = \frac{(S'_c)^2}{n\,p^2\,q\,u(z\,E_c - S'_c)} \tag{19}$$

in which

$q = \Sigma_0 / A_s = \dfrac{\text{perimeter of bar}}{\text{area of steel}}$, and u is unit bond stress.

Percent Steel for Plain Welded Wire Mesh

A complete understanding of the exact bond behavior of wire mesh is not yet known, but based on bond tests by Anderson (16), the following statements appear reasonable. The primary "bond" behavior is really an anchorage behavior, where instead of bond being distributed along the wire (as in the case of deformed bars), it is concentrated at discrete anchorage points where the transverse wires intersect the longitudinal wires. An anchorage strength as strong as the strength of the main steel is achieved for transverse wire sizes not smaller than 4 or 5 wire sizes below that of the main wire size. This means that most of the force exerted on the longitudinal wire is transferred to the concrete through the first transverse wire intersection in the line of action of the force. Furthermore, it is believed that at this ultimate strength any small amount of adhesive bond existing between the concrete and the main wire between the anchorage points is broken by virtue of slip.

Therefore, in the analysis to follow no distributed bond is considered, and all "bond" is assumed concentrated at the first transverse wire intersection nearest a crack in the concrete. Only shrinkage action is assumed to take place. Subgrade frictional forces are neglected, as the real movement of the concrete in contact with the subgrade is quite small. Tests by Friberg (1) have shown that a movement of about 0.1 in. is required before appreciable subgrade friction can develop. The analysis is also limited to the central region of a continuously-reinforced pavement, where no over-all changes in length occur in the steel.

Referring to Figure 5, m is an even integer, A_s is the area of steel, and A_c is the area of concrete. Under the initial assumption that the over-all steel length is unchanged,

$$\frac{T_s\, d}{A_s\, E_s} - \frac{C_s\,(m\, d)}{A_s\, E_s} = 0 \qquad (20)$$

This reduces to

$$f_s = m\, f'_s \qquad (21)$$

in which f_s is the tensile stress of steel and f'_s is the compressive stress of steel. Because the first transverse wire from the crack carries the anchorage force, between these terminal wires the compressive deformation of the steel must equal the net elongation of the concrete.

$$\frac{C_s(m\, d)}{A_s\, E_s} = z(m\, d) - \frac{T_c(m\, d)}{A_c\, E_c} \qquad (22)$$

This reduces to

$$C_s = A_s\, E_s\, z - p\, n\, T_c \qquad (23)$$

Substituting Eq. 23 in Eq. 20 and reducing gives

$$f_s = m(E_s\, z - n\, f'_c) \qquad (24)$$

or

$$m = \frac{f_s}{E_s\, z - n\, f'_c} \qquad (25)$$

where f'_c is the tensile stress in the concrete.

An additional relationship may be obtained from equilibrium of forces at the anchorage:

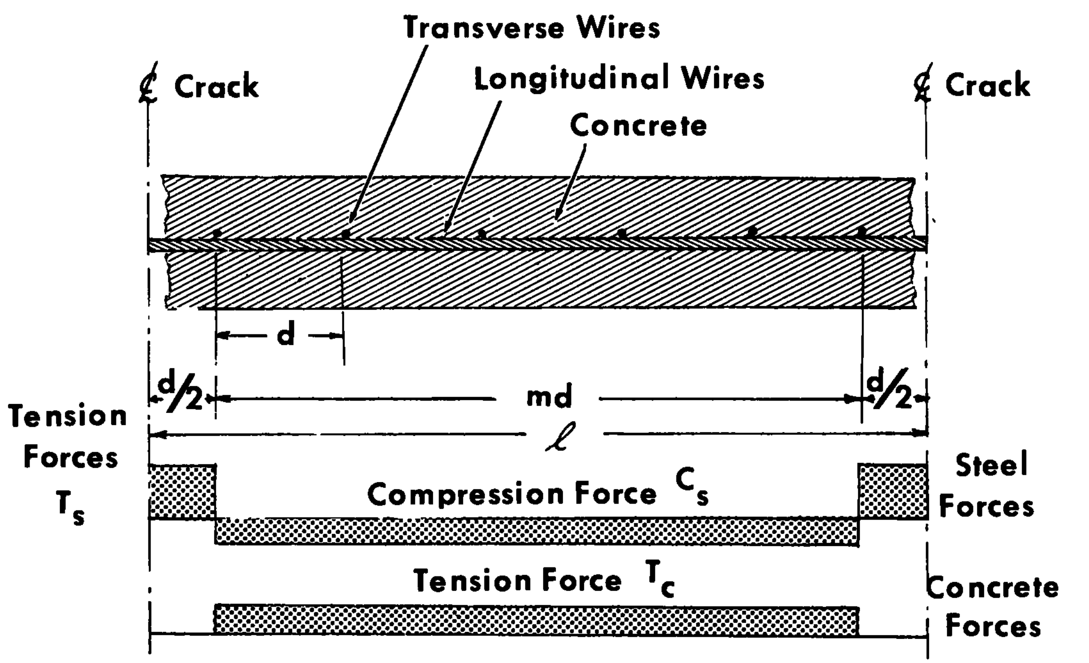

Figure 5. Wire mesh.

$$C_s + T_s = T_c \tag{26}$$

or, in terms of stresses,

$$A_s f'_s + A_s f_s = f'_c A_c \tag{27}$$

Using Eq. 21,

$$f_s = \frac{m f'_c A_c}{A_s + m A_s} \tag{28}$$

Equating Eqs. 24 and 28 and solving for m gives

$$m = \frac{A_s n f'_c + f'_c A_c - A_s E_s z}{A_s E_s z - A_s n f'_c} \tag{29}$$

Thus,

$$L = m d = \frac{A_s n f'_c + f'_c A_c - A_s E_s z}{A_s E_s z - A_s n f'_c} d \tag{30}$$

Note that for no cracks to develop, $L = \infty$, so that the denominator of Eq. 30 equals zero;

$$A_s E_s z - A_s n f'c = 0 \tag{31}$$

or

$$z_{\text{lim for no cracks}} = S'_c / E_c \tag{32}$$

which is the same as Vetter (13) found for slabs reinforced with deformed bars.

To obtain the minimum percentage of steel so that the tensile stress in the concrete will be at its limit of S'_c and the tensile stress in the steel will be at its elastic limit of S_s, set $f'_c = S'_c$ and $f_s = S_s$ in Eqs. 25 and 28 and substitute Eq. 25 in Eq. 28:

$$P_{min} = \frac{S'_c}{S_s + z E_s - n S'_c} \tag{33}$$

The crack spacing for p_{min} may be found from Eq. 25 to be

$$L = d(m + 1) = d \left(\frac{S_s}{E_s z - n S'_c} + 1 \right) \tag{34}$$

It should be noted that the equation for the minimum percentage of steel for slabs reinforced with wire mesh is the same as for slabs reinforced with deformed bars as found by Vetter (13).

It may be of interest to point out in the analysis that the crack spacing L, and consequent steel stress, represent a limiting case on the conservative side. It is entirely possible that through certain regularities or irregularities in the concrete, cracks may form at smaller distances than indicated by Eq. 30. Indeed, cracks could even form near every transverse wire. Assuming no adhesive bond of the plain wire in this case, the concrete will merely crack at an interval d whenever z reaches S'_c / E_c, and the steel stress and concrete stress will both be zero. It is thus seen from this discussion that crack spacing and steel stresses may be predicted within maximum and minimum limits.

Crack Width for Deformed Bars

Under the operating assumptions established, it is seen that there is no distinction in behavior between deformed bars and wire mesh in regard to amount of steel. However, there is a significant difference in crack spacing and crack width. Consider deformed bars first. Vetter (13) showed the stress in the concrete to be as shown in Figure 6. He found the bond length to be

$$x = \frac{A_c f'_c}{u \Sigma_0} \tag{35}$$

The crack width at the steel due to slippage of the concrete is thus

$$W_1 = z\,L - \frac{1}{E_c}\int_0^L f_t\,dL \tag{36}$$

where f_t is the tensile stress of the concrete.

Integrating Eq. 36 and reducing gives

$$W_1 = z\,L - \frac{f'_c}{E_c}\left(L - \frac{A_c\,f'_c}{u\,\Sigma_0}\right) \tag{37}$$

Thus, for p_{min}

$$W_1 = zL - \frac{S'_c}{E_c}\left(L - \frac{A_c\,S'_c}{u\,\Sigma_0}\right) \tag{38}$$

To study the question of the variation of crack width at the steel and at the surface of the concrete, as shown in Figure 7, the following analysis is presented.

If the longitudinal steel is considered to be close together, the concrete behavior may be considered as two-dimensional and the planar methods of the mathematical theory of elasticity may be used. Deduced from Figure 6, the shrinkage forces on the concrete are as shown in Figure 8.

The Airy stress function for this case has been published by Winter (17). After a slight modification in the constant K_n to fit the conditions of the present problem, this stress function is

$$\phi = \sum_{n=1,2,3}^{\infty} (A_n \cosh a_n y + B_n \sinh a_n y + C_n y \cosh a_n y + D_n y \sinh a_n y) \cos a_n x \tag{39}$$

in which

$$a_n = \frac{n\,\pi}{2l} \tag{40}$$

$$A_n = -K_n \frac{\sinh^2 a_n b + (a_n b)^2}{a_n^{\,2}(\sinh 2a_n b + 2a_n b)} \tag{41}$$

$$B_n = \frac{K_n}{2a_n^{\,2}} \tag{42}$$

$$C_n = \frac{K_n}{2a_n} \tag{43}$$

$$D_n = -K_n \frac{\cosh^2 a_n b + 1}{a_n(\sinh 2a_n b + 2a_n b)} \tag{44}$$

$$K_n = \frac{2\,A_c\,S'_c}{n\,L} \sin\frac{n\,\pi}{4} \tag{45}$$

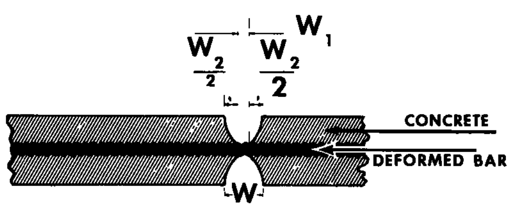

Figure 7. Crack shape.

First, from this stress function (Eq. 39) the exact distribution of stress may be

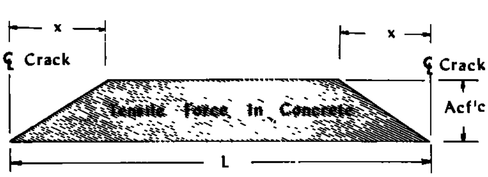

Figure 6. Concrete force.

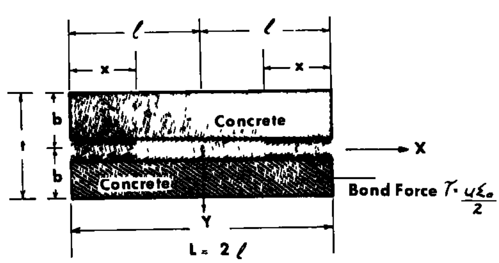

Figure 8. Boundary forces.

found in the concrete from the established principles of elasticity. That is, the longitudinal stress σx at any point in the concrete at coordinates x and y may be found by

$$\sigma x = \frac{\delta^2 \phi}{\delta y^2} \tag{46}$$

Thus,

$$\sigma x = \sum_{n=1,2,3}^{\infty} (A_n a_n^2 \cosh a_n y + B_n a_n^2 \sinh a_n y + C_n a_n^2 y \cosh a_n y + 2C_n a_n \sinh a_n y + D_n a_n^2 y \sinh a_n y + D_n a_n \sinh a_n y + D_n a_n \cosh a_n y) \cos a_n x \tag{47}$$

A study of Eq. 47 reveals that the stress near the surface of the concrete is less than that near the steel and also that the stress near the outside corners is very small.

With the exact stress distribution known, the increase in crack width at the surface over that at the steel may be found as:

$$W_2 = 2 \left[(W')_{y=0} - (W')_{y=b} \right] \tag{48}$$

Neglecting Poisson's ratio,

$$(W')_{y=0} = \frac{1}{E_c} \int_0^1 (\sigma x)_{y=0} \, dx \tag{49}$$

$$(W')_{y=b} = \frac{1}{E_c} \int_0^1 (\sigma x)_{y=b} \, dx \tag{50}$$

After carrying out these operations, W_2 from Eq. 48 becomes

$$W_2 = \frac{2}{E_c} \sum_{n=1,2,3}^{\infty} \left(B_n + \frac{C_n - \psi n}{a_n}\right) \sin a_n 1 \tag{51}$$

in which

$$\psi n = (B_n a_n^2 + 2 C_n a_n + D_n a_n^2 b + D_n a_n)(\sinh a_n b) + (A_n a_n^2 + C_n a_n^2 b + D_n a_n)(\cosh a_n b) \tag{52}$$

Crack Width for Wire Mesh

From Figure 5 it is clear that the crack width at the steel is

$$W_1 = z d + \frac{T_s d}{A_s E_s} = d\left(z + \frac{T_s}{A_s E_s}\right) \tag{53}$$

Thus, for p_{min}

$$W_1 = d\left(z + \frac{S_s}{E_s}\right) \tag{54}$$

W_2 for wire mesh may be computed from the same Eq. 48 as used for deformed bars in view of the fact that this value is generally very small in comparison with W_1.

Example

For a comparative study of crack behavior with deformed bars and wire mesh, consider the following numerical values (reduced physical constants for concrete are used, inasmuch as shrinkage is assumed to take place before the concrete reaches its full strength value): S_s = 50,000 psi (assumed the same for bars and mesh); d = 20 in.; E_s = 30 x 10^6 psi; z = 2 x 10^{-4}; n = 10; E_c = 3 + 10^6 psi; S'_c = 100 psi; t = 8 in.; u = 400 psi; and A = 96 sq in.

From Eq. 33 p_{min} is computed as 0.00182. It should be noted that this is the same for both bars and wires. Selecting No. 4 deformed bars at 13½-in. centers results in Σ_0 = 1.35 sq in. and A_s = 0.175 sq in. For the mesh, 6/0 longitudinal wires and 1/0 transverse wires are selected.

From Vetter's equation (Eq. 19) L for deformed bars is computed as 5.54 ft. From Eq. 38, W_1 for deformed bars is computed as 0.0114 in.

For wire mesh, L is computed from Eq. 34 as 18.4 ft and W_2 from Eq. 54 as 0.0372 in. It may be noted, however, that if the transverse wire spacing is assumed as 6 in. instead of 20 in., the crack width and spacing are about the same as for deformed bars.

A comparison thus reveals that deformed bars tend to cause cracking at closer intervals, with less crack width resulting, and with the crack width being about proportional to the crack spacing. A narrow crack is desirable, as it protects the steel better and provides better aggregate interlock. However, the crack spacing and crack opening may be directly controlled in mesh by adjusting the distance between transverse wires. Closer spacing of transverse wires means closer crack spacing and smaller crack openings.

A numerical check of the value W_2 shows this to be negligible (less than 0.001 in.), and it may thus for practical purposes be ignored. This example problem is not intended to be used as a design criterion, but is presented simply to show the use of the basic equations.

BUCKLING TENDENCY

The phenomenon of blow-ups, or buckling, is well known in standard concrete pavements. It is the purpose of this investigation, therefore, to determine under what conditions buckling may occur in continuously-reinforced pavements. The condition of buckling is shown in Figure 9, in which

s = weight of slab per unit of surface;

F = the incipient buckling force per unit of width; and

M = the bending moment at x = 0 and x = l.

The differential equation of buckling behavior is

$$\frac{d^2y}{dx^2} = -\frac{M_x}{D_r} \tag{55}$$

where D_r is the reduced slab rigidity, defined by Eq. 14. From equilibrium, the bending moment at any distance, x, is

$$M_x = Fy - M + \frac{wx^2}{2} - \frac{wlx}{2} \tag{56}$$

Thus,

$$\frac{d^2y}{dx^2} + \frac{Fy}{D_r} = \frac{M}{D_r} + \frac{wlx}{2D_r} - \frac{wx^2}{2D_r} \tag{57}$$

The general solution to Eq. 57 is

$$y = A\cos kx + B\sin kx + \frac{M}{k^2 D_r} + \frac{w}{k^4 D_r} + \frac{wlx}{2k^2 D_r} - \frac{wx^2}{2k^2 D_r} \tag{58}$$

in which

$$k^2 = \frac{F}{D_r} \qquad (59)$$

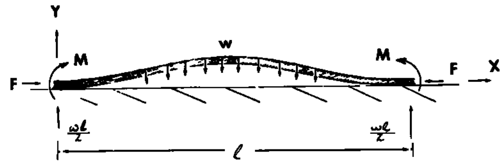

Figure 9. Slab buckling.

To evaluate the unknowns A, B, M, and F, four conditions are needed, as follows:

$$(y)_{x=0} = 0 \qquad (a)$$

$$(y)_{x=1} = 0 \qquad (b)$$

$$\left(\frac{dy}{dx}\right)_{x=0} = 0 \qquad (c)$$

$$\left(\frac{dy}{dx}\right)_{x=1} = 0 \qquad (d)$$

Boundary conditions (a) through (d) are then substituted in Eq. 58 or its first derivative. The following four equations are then obtained:

From (a),

$$A + \frac{M}{k^2 D_r} + \frac{w}{k^4 D_r} = 0 \qquad (60)$$

From (b),

$$A \cos k1 + B \sin k1 + \frac{M}{k^2 D_r} + \frac{w}{k^4 D_r} = 0 \qquad (61)$$

From (c),

$$B k + \frac{w 1}{2 k^2 D_r} = 0 \qquad (62)$$

From (d),

$$B k \cos k1 - A k \sin k1 - \frac{w 1}{2 k^2 D_r} = 0 \qquad (63)$$

To solve these four transcendental equations simultaneously to obtain k as a function of 1, the first several terms of the Maclauren series are substituted for the sine and cosine functions.

$$\sin k1 = k1 - \frac{k^3 1^3}{6} + \ldots.. \qquad (64)$$

$$\cos k1 = 1 - \frac{k^2 1^2}{2} + \frac{k^4 1^4}{24} - \ldots.. \qquad (65)$$

Upon the successive operations of substitution, simultaneous solution of the equations, and reduction, the following relation of k to 1 results:

$$k^2 = \frac{36}{1^2} \qquad (66)$$

But from Eq. 59

$$k^2 = \frac{F}{D_r} = \frac{36}{1^2} \qquad (67)$$

The incipient buckling force is thus

$$F = \frac{36 D_r}{1^2} \qquad (68)$$

It should be noted that the incipient buckling force calculated from Eq. 68 is very nearly equal to Euler's classical critical buckling force for clamped ends and without any lateral force such as the slab weight,

$$F_{cr} = \frac{4\pi^2 D_r}{l^2} \tag{69}$$

If sufficient terms in the Maclauren expansion for the sine and cosine functions were taken, the incipient slab buckling force would be exactly equal to Euler's force. This thus indicates that the weight of the slab has no influence on the buckling force for the slab. Further proof of this is that the unit weight w cancels out in the preceding analysis.

There is, however, a very special and important limitation imposed on continuous pavements, not generally encountered in other buckling problems. This is the fact that the effective ends (where the assumed shapes as shown in Figure 9 meet the ground) are constrained against motion in the x direction by virtue of continuity. It is this fact that prevents actual uplift, despite the preceding analysis which indicates a possibility of incipient buckling.

To visualize the mechanics of actual buckling, consider that there first exists a sufficient axial force on the slab such that the slab is in a state of incipient buckling, given in this case by Eq. 69. If the ends of the slab were free to move under this force, the slab would indeed buckle upward, as in normal buckling action. However, due to the restrained ends, as soon as the slab tends to uplift as shown in Figure 9, the true length of the slab changes from the flat length to the longer length along the curve. Because the force F is imposed internally, as it would be by temperature or volume change in the concrete, this increase in length relieves that force. With F now decreased, the incipient buckling force is no longer materialized and the slab never actually uplifts.

For external forces greater than F, the slab may theoretically uplift slightly; but the uplift occurs gradually and over a very long length, such that the uplift is not visually observable. Furthermore, the uplift occurs gradually, and does not suddenly buckle.

Field observations[1] have verified this conclusion. In all the continuously-reinforced pavements built in the United States, no visible buckling has ever been recorded or observed, even when expansive forces have been large enough to cause compression failure and spalling at the terminal ends of the pavement (18).

MOVEMENT ON HORIZONTAL CURVES

It is conceivable that a roadway may change in horizontal alignment around a curve when a continuous slab of steel and concrete contracts or expands. The tendency to shorten or lengthen may cause the pavement at a curve to move inward or outward in the radial direction.

The simple calculation to follow establishes the conditions under which such movement would take place. It is the subgrade frictional drag which tends to prevent this radial movement. Consider a contraction tendency as shown in Figure 10. (Expansion would lead to the same general conclusion.)

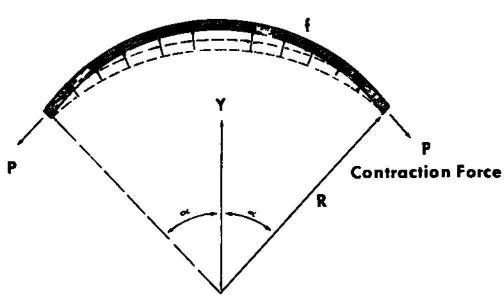

Figure 10. Horizontal curve.

The limiting subgrade drag force f per unit length of slab before appreciable radial movement may take place is μW, where μ is the coefficient of subgrade drag and W is the weight of pavement per unit length of slab.

From equilibrium of forces in the Y direction,

$$2\,\mu\,W R \sin\alpha - 2\,P \sin\alpha = 0 \tag{70}$$

From which it is found that the minimum radius that a roadway should have to prevent horizontal alignment changes is

[1] Confirmed in conversation with H. D. Cashell.

$$R_{limit} = \frac{P}{\mu W} \tag{71}$$

For cracked slabs, the limiting value of P may be taken as the total area of steel A'_s times the elastic limit of steel S_s.
Thus,

$$R_{limit} = \frac{A'_s S_s}{\mu W} = \frac{A_s S_s}{\mu w} \tag{72}$$

in which A_s is the area of steel per unit width and w is the weight of slab per unit of surface.

Considering a numerical example with A_s = 0.5 sq in. per ft, S_s = 50,000 psi, w = 75 psi, and μ = 1.5, R_{limit} is computed from Eq. 72 to be 222 ft. Inasmuch as this radius is well below the normal radius used in highway design, it may be generally concluded that horizontal movements at curves are not a problem.

MOVEMENT ON VERTICAL CURVES

There exists the possibility that a continuous pavement on a vertical curve at the crest of a hill could tend to uplift from its base if sufficient concrete swelling due to moisture penetration and high temperature were to take place. Likewise, at a vertical curve at the bottom of a hill a tendency to uplift would be present if sufficient concrete shrinkage and temperature contraction existed. To investigate these possibilities the following analysis is presented.

Consider the contraction case shown in Figure 11 for a small ratio of h to a. Let ΔT be the total unrestrained contraction due to temperature and ΔS be the total unrestrained contraction due to shrinkage. Under the combined action of ΔT and ΔS the slab will tend to shorten and thus tend to lift off the ground. In doing this, there will be a force F induced throughout the slab caused by the weight of the slab. The exact "shape" of the slab would then be described by a catenary; however, to use a simpler (and almost exact) expression, the parabolic shape is assumed. Thus:

$$F = w\,a\sqrt{1 + \frac{a^2}{4h^2}} \tag{73}$$

This force F then causes the slab to stretch a total length of F l / A E.

When this stretch F l / A E equals $\Delta T + \Delta S$, the slab is recontacted with the base and the induced F thus vanishes. The resulting performance of the slab is as if it were simply straight and horizontal.

The criteria of uplift may then be expressed by

$$\Delta T + \Delta S > \frac{w\,a\,l}{A\,E}\sqrt{1 + \frac{a^2}{4h^2}} \tag{74}$$

For cracked slabs in tension, A may be taken as A_s and E may be taken as E_s.

The behavior of a vertical curve in expansion at the crest of a hill is similar. It is only necessary to modify Eq. 74 by first considering ΔT as the total temperature expansion and ΔS as the total swelling expansion due to moisture and then, because the slab is in compression, all cracks in the concrete close up and

$$A = A_s + \frac{E_c}{E_s} A_c \tag{75}$$

and E should be taken as E_s. (This simply uses the transformed area properties of the concrete.)

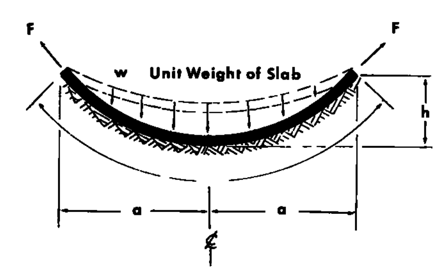

Figure 11. Vertical curve.

Example

Consider the following values for contraction: temperature drop = 80 F, coefficient of expansion $\alpha = 0.0000075$ per degree F, a = 3,000 in., h = 100 in., E_s = 30×10^6 psi, w = 0.5 lb per in. (6-in. slab), A_s = 0.0417 psi, and $\Delta S = 0$. Thus, $\Delta T + \Delta S$ may be computed to be 3.7 in.; and

$$\frac{w\,a\,l}{A_s\,E_s}\sqrt{1+\frac{a^2}{4h^2}}$$

is computed to be 109 in. if the steel is assumed to remain elastic. From Eq. 74 it is thus obvious that there is no danger of uplift.

A comparative check on expansion for a temperature rise of 80 F, $\Delta S = 0$, and with all other values as for contraction, shows $\Delta T + \Delta S = 3.7$ in. and

$$\frac{w\,a}{\left(A_s + \frac{E_c}{E_s} A_c\right) E_s}\sqrt{1+\frac{a^2}{4h^2}} = 7.2 \text{ in.}$$

It should be noted that, due to the added action of the concrete, the deformation caused by the force F in compression is less than for tension. Nevertheless, from Eq. 74 it is clear that no uplift will take place in either case.

TERMINAL ANCHORAGE CONFIGURATION STUDIES

In the construction of existing continuously-reinforced roads, various types of end joints have been tried. Some consist only of standard filler strips as used in normal construction, whereas others are more elaborate, employing such joints as bridge-type expansion joints. Most of these have eventually proven unsatisfactory under service conditions; terminal movements of as much as 4 in. have been observed, resulting in damage to the pavement in the vicinity of the joint. As a possible alternate solution to these troublesome ends, it is suggested that the ends may be anchored instead of allowed to undulate. Several experimental anchors are being planned for a section of continuously-reinforced pavement to be built in southern Virginia. To study this problem a pilot model study was initiated by the Virginia Council of Highway Investigation and Research to determine the best anchorage configuration.

The test bed was dry sand, 2 ft by 14 ft upon which a wooden board (to which anchors were fastened) was placed as shown in Figure 12. The scale factor was 1/24 ft so that

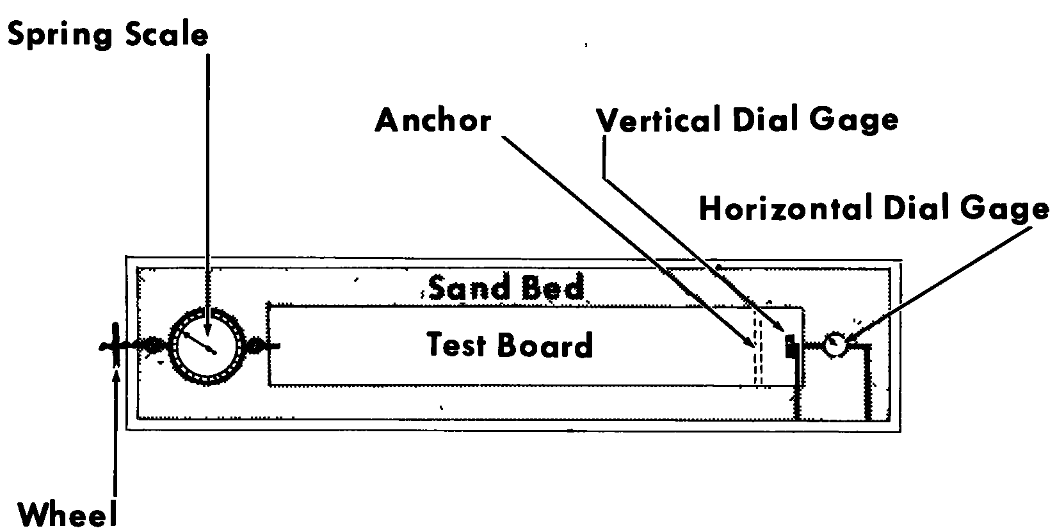

Figure 12. Test bed and apparatus.

the 12-in. by 12-ft board represented the terminal portion of road 24 ft wide and 288 ft long. It was not intended that this pilot study simulate the actual forces and movements at the end of a real slab; it was intended only to offer a qualitative comparison of anchorage resistances. On continuously-reinforced pavements only the last 300 ft or so have been found to move, therefore it is this portion which must be anchored. The anchorage configurations tested are those shown in Figure 13.

These shapes were varied in size, depth, and spacing, to observe relationships. The horizontal strength of this type of anchorage depends on the shape of the failure surface developed. Consider the action of an anchor such as shown in Figure 13a.

It is seen from Figure 14 that, due to the vertical restraining forces exerted by the slab on the soil acting as a surcharge, the failure surface is spread out and thus becomes more effective than if such vertical forces were not present. This vertical restraint can be attributed to a number of factors, including the weight of the slab, the bending resistance of the slab, the weight of the anchor, and the vertical friction of the anchor. Thus, the greater these factors, the greater is the maximum horizontal force P.

It may also be observed from Figure 14 that when the vertical anchor wall moves, it will displace the soil vertically, even allowing for consolidation. Thus, some uplift tendency can be expected. The more the failure surface is spread out due to the surcharge, the less will be the uplift. For a rough approximation in the limiting case of no surcharge, the vertical movement is about the same as the horizontal movement. With surcharge, the vertical movement is much less. The pilot tests have verified this.

Although load-versus-horizontal and vertical movement curves were obtained on all tests, it is felt that because these values were based only on a model study, their quantitative values are not as important as their relative values. Therefore, only a few sample curves are shown in Figure 15.

The important conclusions drawn from these tests may be summarized as follows:

Strength

Consider first a comparison of configurations a, b, and c (Figure 18), which are categorized as single solid anchorages. Taking a as a reference, it is found that due to the added confining action on soil by the side walls of the anchor, extra strength may be attained for the same projected area. Shape b is 14 percent stronger than a and shape c is 55 percent stronger than a. Shape f is also a single anchor, but consists of separated anchors. This shape produces the same strength as shape a for the same depth of embedment, despite the fact that the projected area is only 39 percent that of a. This may be accounted for in view of what happens to the failure surface as described in Figure 14. For a straight solid anchor such as a, the failure surface is essentially two-dimensional; but for an anchor such as in g, the soil flows not only in the direction of pull, but also transversely, creating a much larger three-dimensional failure surface. This thus results in the pile shapes having a much larger resistance than indicated merely by their projected area.

A second comparison may be made on the basis of depth of anchorage. For a given anchorage shape such as a the strength appears to vary linearly with depth, within the range of depths tested.

A third comparison may be made on multiple anchorages as in shapes d, e, and g.

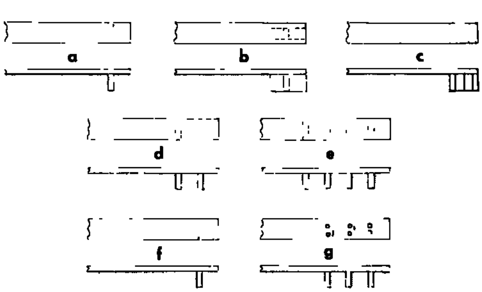

Figure 13. Anchorage configurations.

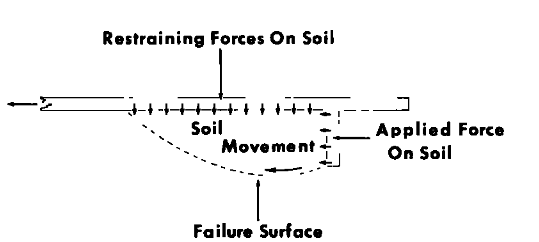

Figure 14. Failure surface.

A series of tests run on shape d with the spacing varying between anchors shows, as might be expected, that if the anchors are too close there is interference of action between the two and the full strength of both of them cannot be realized. It was found that for a scale-model distance of 48 in., or a prototype distance of 96 ft, the interference vanishes, and the full strength of each may be attained. The model depth for this test was 2 in., representing a prototype depth of 4 ft.

The over-all best performance in strength was attained by configuration g, which consists of a series of separated piles. For the same number and size of piles as f, g produced a strength 71 percent greater than f or a.

Uplift

As previously discussed, the problem of uplift is associated with strength. Two important conclusions seem to stand out after consideration of the test results. The first is that a distributed multiple anchorage, such as e or g, produces less uplift than a single large anchorage such as a, b, or c. This is understandable, as the load is more distributed and also the bending resistance of the pavement at interior positions is greater than at the end, allowing less vertical movement.

The second conclusion is that there is less uplift for the deeper anchors. This is also understandable, as a deeper anchor has more weight and more side frictional surface. Incidental to side friction, it was also found that the pile-shaped anchor f produced less uplift than a solid shape like a. This is partly explained by side friction and partly by the fact that the failure surface is three-dimensional, spreading the displaced soil over a larger area.

A comparison of uplift values, again taking shape a as a reference, shows that the uplift in a is 40 percent of the magnitude of the horizontal movement, whereas the uplift

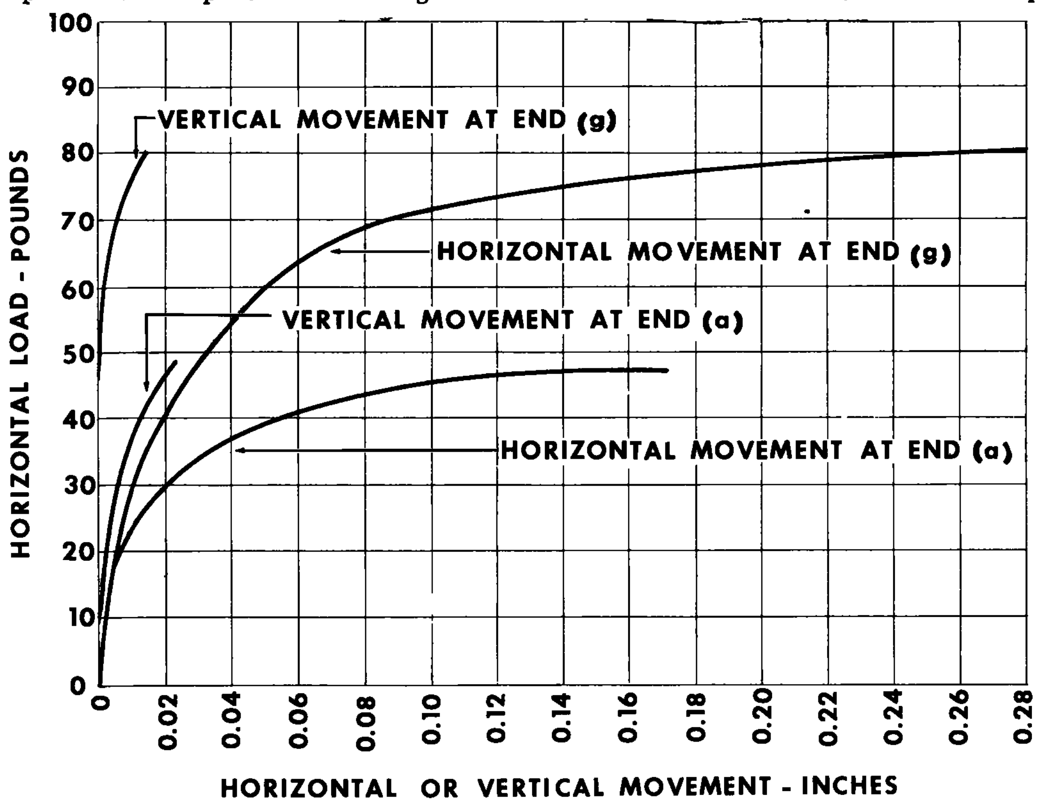

Figure 15. Load-Movement curves for anchorage configuration as in Figure (12 A) and (12 g) (4" Model Depth).

in g is only 11 percent of the horizontal movement just prior to failure (see Fig. 15). In addition, the absolute vertical movement in g for corresponding loads is only about 25 percent of that for a, which shows the over-all superiority of g.

Therefore, as a general conclusion of comparison, it appears that a pile configuration such as Figure 12 g is the best for both maximum strength and minimum uplift.

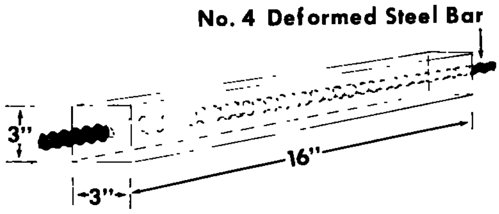

Figure 16.· Test specimen.

Under actual field conditions it is believed that this shape could be quite economical if a truck-mounted earth augur bored holes into which a preassembled reinforcing steel cage could be placed. The holes could be monolithically filled with concrete at the same time the pavement is poured. A preliminary design indicates that about 10 such piles, 18 in. in diameter and 8 ft deep, could resist the terminal forces imposed by a continuously-reinforced pavement in either expansion or contraction.

When the full-scale anchorages are constructed on the proposed highway in southern Virginia, additional field observations will be taken on the horizontal and vertical movements, together with any other unusual cracking or behavior. More conclusive reports may be expected at that time.

CRACKED SLAB BEHAVIOR

As indicated earlier, in connection with the pavement thickness analysis and buckling analysis, there is need for information on the reduced bending rigidity of the slab in the longitudinal direction due to transverse cracking. To this end a pilot test program was conducted to obtain a behavioral trend.

Twenty test specimens (Fig. 16) were cast of concrete. After 28 days of moist curing, a concrete cylinder strength of 2,500 psi was reached. Each specimen was then artificially cracked by a pull on the steel bar sufficient to cause the concrete to crack transversely at various crack widths from 0.010 to 0.193 in. Several specimens were left uncracked to serve as a standard for comparison. The crack widths were measured with a calibrated microscope micrometer.

The specimens were then loaded transversely as beams, such as to cause a bending moment at the cracks of 1,440 in.-lb. This is within the standard allowable limit for normal concrete beams. Load-center deflection readings were then taken for the first, 10th, and 20th loading cycles. The load-deflection readings were not changed from the 10th to the 20th cycle.

Reduced Bending Rigidity

Figure 17 shows several typical load-deflection curves obtained. Note that these curves may be characterized in two stages. The small loads from 0 to about 350 lb are characterized by the fact that in this stage it is the flexing of the steel bar that is primarily controlling the bending of the beam, as the crack is still essentially open (see Fig. 18a). This is naturally a relatively weak phase. The second stage is that beyond 350 lb, where the concrete in compression acts in conjunction with the bending and axial tension in the steel, as shown in Figure 18b. This, or course, accounts for the rapid rise in strength as shown in Figure 17.

Based on the center deflection at the working load, a comparison of flexural rigidity may be made from the known elastic relation of load to deflection for a beam. The flexural rigidity is defined as EI, the modulus of elasticity times the effective moment of inertia. As used in the pavement thickness studies, the reduced rigidity is called D_r.

The plot of percentage of full uncracked rigidity and crack width is shown in Figure 19. Note that the flexural rigidity drops off rapidly with even small crack widths. Slabs with cracks of 0.20 in. have a rigidity of only 1 percent of that of an uncracked slab. The question of reduced rigidity as a function of crack spacing was not studied in this experiment.

Increase in Crack Width Under Repeated Loading

Field observations (12) of pavements in existence for many years have disclosed that lanes with more traffic have wider cracks than lanes with less traffic. This is clearly indicative of a condition resulting from repeated vehicular load, causing the cracks to flex back and forth under the bending moment imposed by the moving loads.

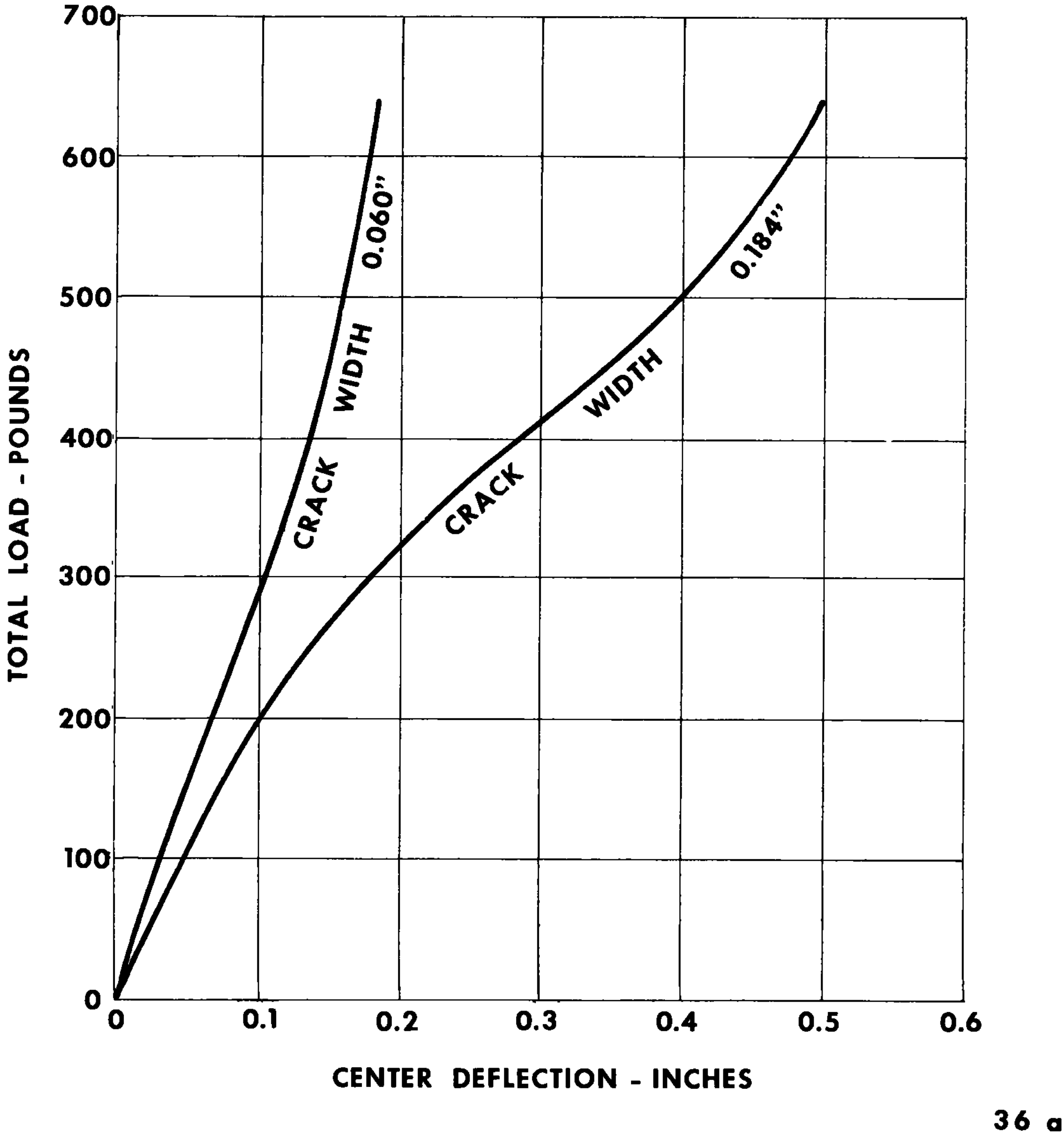

Figure 17. Typical load-deflection curves.

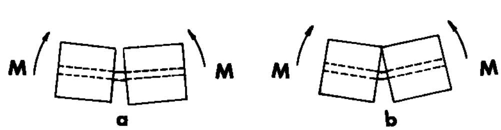

Figure 18. Crack behavior.

The same beam specimens and same apparatus set up to study crack width were used to determine the behavioral trend with repeated loadings. The loads were cycled from 0 to the full working load and back to 0 again. Average crack widths were taken after the 10th and 20th cycles. Only a slight increase was noticed in the 20th cycle over the 10th cycle, so the results reported in Figure 20 are for 10 cycles. Note that although large percentage increases occur for small crack widths, this in reality still represents a very small real width of crack, because the percentage is based on an initially small value.

The increase in crack width under repeated loads is perhaps explained by a certain crushing and agitation of the cracked surfaces in the repeated flexing contact and recontact. For large cracks, this contact is confined to an area very near the surface, as shown in Figure 18b; in narrow cracks, the contact area is much greater. Because in this experiment only average crack widths were measured, the larger cracks did not reflect as much change as the smaller cracks.

Load repetition was not carried out far enough in this pilot study to cause fatigue failure. It is expected that fatigue failure would occur rapidly in the steel in the large crack widths for at least three reasons, as follows:

1. Large bending stresses occur in the steel bar itself, as discussed in connection with Figure 18.

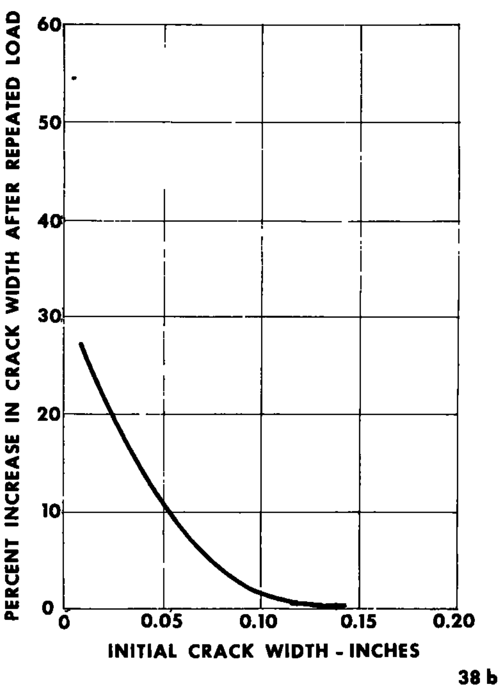

Figure 20. Increase in crack width after repeated loading.

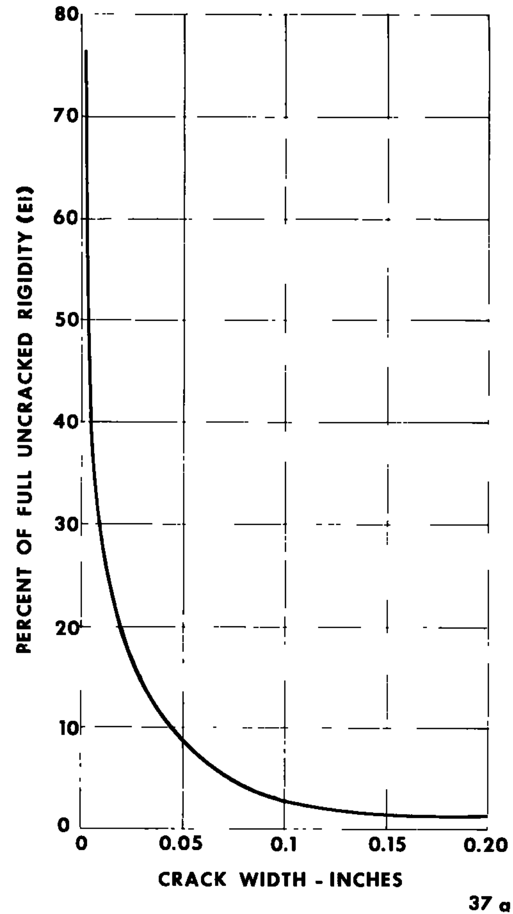

Figure 19. Reduction in rigidity.

2. The large crack results in the loss of aggregate interlock of the concrete, inducing high shear stresses in the steel.
3. Large cracks expose the steel to corrosion deterioration, which accelerates failure by fatigue.

These three reasons combine to cause early failure by fatigue. It is no surprise, therefore, that in all work in these tests and in this whole report, narrow crack widths are indeed the desirable object for the successful performance of a continuously-reinforced concrete pavement.

Because of the preliminary nature of this series of tests, coupled with the importance of information on repeated loads, it is highly desirable that additional test programs be conducted where the specimens are subjected to repeated loadings of many thousands of cycles. From several tests run incidental to the ones described, where the loading was repeated 100 times, it appears that the crack width continues to enlarge. Repeated loading of many thousands of times on various strength concretes and various crack widths would thus reveal a more comprehensive behavior pattern.

ACKNOWLEDGMENTS

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Discussion

VEDAT YERLICI, Asst. Professor of Civil Engineering, Lehigh University, Bethlehem, Pa. — This paper contributes much to further understanding and knowledge of continuously-reinforced concrete highway pavements, and helps to clear up some doubtful points, such as buckling tendency and movement on horizontal and vertical curves. In certain respects it also raises questions.

Under the section on "Pavement Thickness" the slab is analyzed as a transverse beam between cracks. At the cracks the restraint is assumed to consist only of the upward reactions (shears) caused by the "elastic restraint of the adjacent segments transferred by the longitudinal steel" and by the "aggregate interlock". It would appear that the action of the cracked slab as a beam is somewhat questionable. If there is aggregate interlock at the crack there should also be twisting moments, which will reduce the deflections. Also, because there is considerable longitudinal steel, although not at the bottom of the slab, there also will be longitudinal moments. With all of these boundary conditions ignored, it is doubtful if this analysis fits the actual conditions better than the Westergaard theories, which do not assume cracks. Also, in the given differential equation of the beam on elastic foundations the aggregate interlock and the elastic restrain modulus are taken proportional to deflection. It is doubtful if these factors increase linearly with deflection; in the case of aggregate interlock even the reverse might be true.

If there is to be a rigorous mathematical analysis of the pavement, a section between cracks must be assumed partially restrained by shears, by longitudinal and torsional (twisting) moments at the cracked end, and free at the sides, and it should be analyzed as a plate over an elastic foundation. As this solution will be too complicated and time consuming, probably the whole pavement may be analyzed as an infinite or semi-infinite orthotropic plate strip on elastic foundation. In this analysis the effect of transverse cracks may be taken care of by assuming a reduced longitudinal rigidity for the pavement. On the other hand, it must be remembered that the item under consideration is a continuous, cracked, reinforced concrete slab resting on soil and that it is subjected to dynamic concentrated loads. The magnitudes of crack spacing and impact cannot be determined with certainty, and the modulus of elasticity, moment of inertia of reinforced concrete, and foundation modulus of any subbase, are not reliable factors. Therefore, at its best, this sort of analysis, based on such doubtful assumptions, cannot be much more than a help for a qualitative understanding of pavement behavior.

In comparing the behavior between the slabs reinforced with deformed bars and plain wire mesh, to simplify the matter, only the behavior in connection with shrinkage of concrete is discussed. When concrete is at the state of shrinking, its tensile and bond strengths are not fully developed and most probably the steel cannot restrain much of the contraction of concrete because of considerable slippage due to weakness of bond. At this stage the cracks will form mostly due to poor strength of concrete and frictional resistance between the subgrade and the pavement. Later, as the concrete cures, the influence of reinforcement as a crack former increases to be the major effect. Also, mechanical anchorages, such as the transverse bars of the wire mesh, may have an altogether different influence on the uncured concrete pavement than on the cured one. Hence, a study of the reinforcement influence that does not include the temperature effects on the cured pavement may give an incomplete picture and probably should not be generalized.

Anchoring the ends of the pavement, if it could be done cheaply, is a most constructive idea. It should present no problem when the slab is contracting, because the anchorage must only be stronger than the yield strength of steel; but the problem will get more complicated when the pavement expands, because of the high strength of concrete in compression. Thought also must be given to the possible plastic deformation, and creep, of the soil around the anchorages, because it may in time destroy the fixing effect of the anchors altogether.

WILLIAM ZUK, Closure — Most of Mr. Yerlici's comments are well considered and correct from a rigorous point of view, so that there is little to refute from this standpoint. However, he, as well as others, undoubtedly is aware that the simplifications and approach used were instituted not out of ignorance of the factors he mentioned, but in order to achieve a simplified workable design solution to some of the problems encountered in continuously-reinforced pavements. Some rational method of design is needed to replace the "rule of thumb" method now used.

Theoretical, laboratory, and field studies on various controversial phases of this subject are still being continued and it is hoped that eventually answers satisfactory to everyone will be found.

A Ten-Year Report on the Illinois Continuously-Reinforced Pavement

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In the fall of 1947 and the spring of 1948 the Illinois Division of Highways constructed on US 40 an experimental continuously-reinforced pavement consisting of eight test sections ranging in length from 3,500 to 4,230 ft. Four sections were uniform 7-in. pavement and four were uniform 8-in. pavement. The pavement was constructed directly on natural subgrade, 90 percent of which was composed of soils classified as potentially pumping types.

Longitudinal steel amounting to 0.3, 0.5, 0.7, and 1.0 percent of the gross cross-sectional area of the pavement was used with each pavement thickness. The longitudinal reinforcement consists of round deformed rail-steel bars. Number 3 round deformed intermediate grade, billet-steel bars at 12-in. centers in one-half of each section and at 18-in. centers in the other half were used as transverse reinforcement.

The pavement has been carefully observed and detailed surveys to determine its behavior and condition have been made periodically.

This report describes the behavior and performance of the pavement during the 10-year period. The behavior of the pavement, particularly of the more lightly reinforced sections, has been beyond expectations. All of the test sections have given good performance. There have been a few structural failures in the lighter reinforced sections; in those cases local conditions, such as poor subgrade, have been the primary factors. Pumping has been restricted mainly to the 4-in. expansion joints and the construction joints, which are notably points of weakness.

All test sections developed large numbers of transverse cracks, the frequency being proportional to the amount of longitudinal steel. Crack development has been progressive with time, but at a decreasing rate. Crack widths are inversely proportional to the amount of longitudinal steel, but in most cases the cracks have remained narrow, indicating that the steel is effective in holding adjacent slabs together. Only moderate ravelling has occurred at transverse cracks and there has been almost no faulting across cracks. The pavement is noticeably smooth riding, which is further indicated by a recent roughometer test that gave an average reading of less than 72 in. per mile.

There has been a progressive increase in the length of all test sections, resulting in full closure of the 4-in. expansion joints. At several of these joints high localized compressive stresses, probably caused by irregular interfaces, have resulted in rather large surface spalls that have required some maintenance. These repairs, limited amounts of undersealing to reduce pumping at expansion joints and construction joints, and repairs of a few local structural failures, account for the only slab maintenance performed during the 10-year period.

●IN 1947-48 the Illinois Division of Highways, in cooperation with the U. S. Bureau of Public Roads, constructed an experimental pavement to study the effect of continuous reinforcement in long unjointed pavement slabs and to determine, if possible the proper amount of steel necessary to provide a pavement of outstanding performance and service life. Construction began September 25, 1947, and the pavement was completed May 20, 1948.

The design and construction details have been covered in previous reports (1, 2,). It seems desirable, however, to summarize briefly the important facts about its design and construction.

The pavement, which is 22 ft wide and approximately 5½ mi long, is divided into eight test sections, six of which are approximately 3,500 ft and two about 4,230 ft long. The longitudinal reinforcement is continuous from end to end in each test section; consisting of round deformed bars (ASTM 305-47T) meeting the requirements of ASTM Designation A 16 for rail-steel bars. The pavement is reinforced transversely with No. 3 round deformed bars meeting the requirements of ASTM Designation A 15 for intermediate grade billet-steel bars. In one-half of each test section the transverse bars are spaced on 12-in. centers; in the other half, on 18-in. centers. The transverse bars extend the full width of the pavement and the customary center joint was omitted. The physical properties of the reinforcing bars, determined from mill tests, are given in Table 1.

Four of the test sections are uniformly 7 in. thick and four are 8 in. thick. Four percentages of longitudinal steel, based on the gross cross-sectional area of the pavement, were used with each thickness of pavement; namely, 0.3, 0.5, 0.7, and 1.0 percent. These percentages were obtained by using different bar sizes and varying their spacing. The reinforcement bars were assembled as a continuous mat on the subgrade and supported by means of chairs approximately 3 in. below the finished surface of the pavement. Air-entrained concrete was poured in one lift, being deposited through the reinforcement and consolidated and finished by means of a conventional spreader and finishing machine and regular hand operations.

The pavement has been the subject of frequent periodic observations and measurements from the time of its construction in order that a thorough record of its behavior and performance would be obtained. Extensive surveys have been made, usually twice a year (one in extremely warm weather and another in cold). At times, however, cold weather observations had to be omitted because of snow and ice. Observations have also been made immediately following periods of heavy rainfall.

It is the purpose of this report to discuss the data obtained, to describe the performance of the pavement during the 10-year period following its construction, to describe its present condition, and to offer some suggestions as to design of continuously-reinforced pavements.

DEVELOPMENT OF TRANSVERSE CRACKS

One of the distinguishing characteristics of a continuously-reinforced pavement is that transverse cracks develop at very close intervals, the frequency of crack occurrence varying with the amount of longitudinal steel. The cracking pattern near the ends of a long continuously-reinforced slab is similar to that in conventional pavements, but the frequency gradually increases away from the ends and, in long sections, reaches a maximum, which is fairly constant throughout the central portion of the slab (2, 3, 4).

Figure 1 shows the distribution of cracking throughout each of the sections of 7-in. pavement at the age of 3 years and again at 10 years. It will be seen that in 1950 and 1957 the frequency for successive 100-ft increments increased at a reasonably uniform rate from the ends of the slab to a maximum at a distance ranging from 200 to 500 ft from the ends, then decreased uniformly for several hundred feet, and finally became fairly stable over the central 1,600 to 2,000 ft.

It is interesting to note the obvious relationship between crack frequency and the percentage of longitudinal steel. The data for 1950 and 1957 show definitely that the

TABLE 1

RESULTS OF MILL TESTS OF REINFORCING STEEL

Bar No. (in.)	No. of Tests	Yield Point (psi)	Tensile Strength (psi)	Elongation (%)
(a) Longitudinal Reinforcement				
3	6	78,701	119,469	14.3
4	14	66,223	107,355	14.5
5	17	70,778	127,759	12.5
6	21	70,157	124,888	11.7
(b) Transverse Reinforcement				
3	7	43,498	77,356	21.5

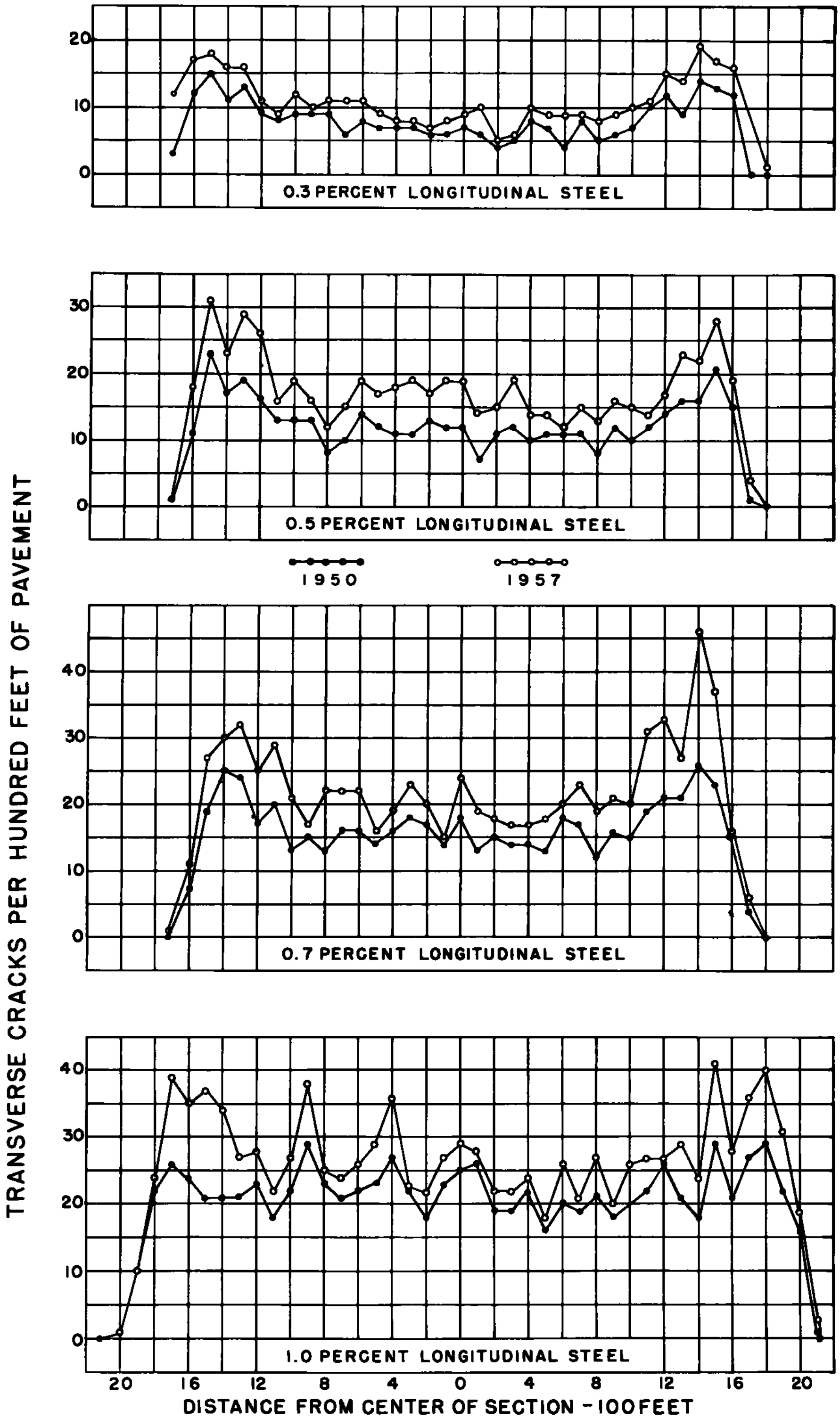

Figure 1. Comparison of frequency distribution of transverse cracks for all sections, 7-in. pavement, 1950 and 1957.

number of cracks increased with the increase in the amount of longitudinal steel. This relationship is illustrated more graphically in Figure 2, which shows the average crack intervals in the four sections of 7-in. pavement at six ages. The first survey was made

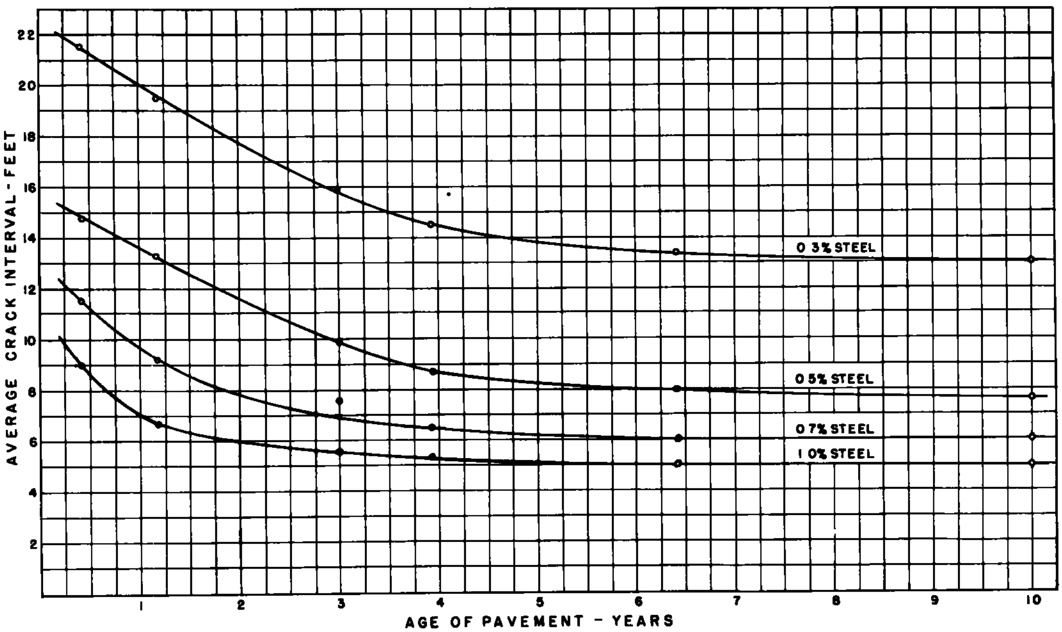

Figure 2. Relationship between average transverse crack interval and age, 7-in. pavement.

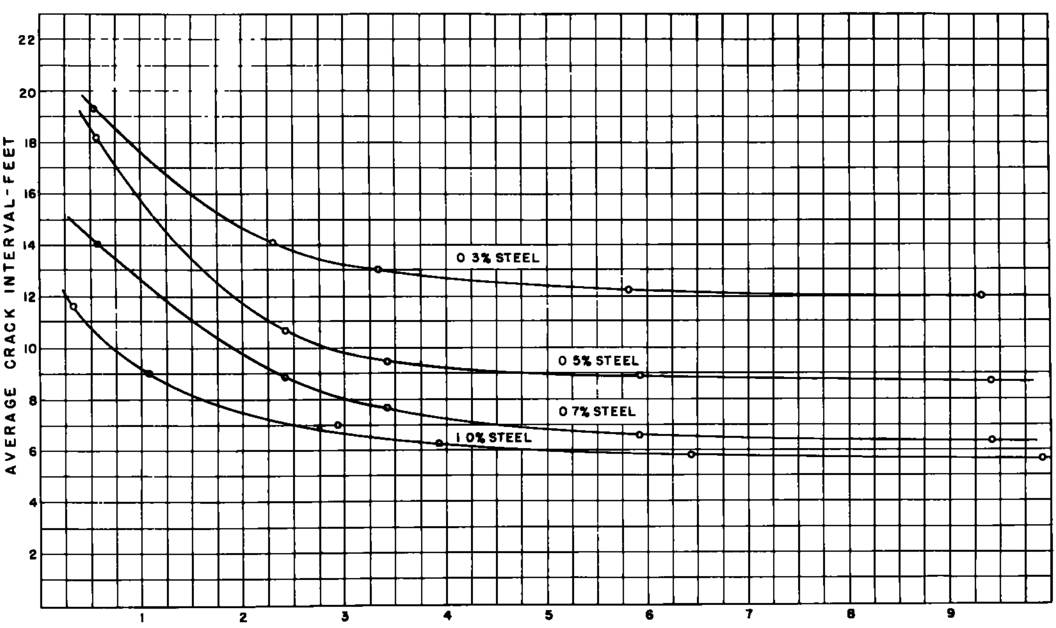

Figure 3. Relationship between average transverse crack interval and age, 8-in. pavement.

in March 1948 after the sections had gone through one winter and were less than six months old. Other surveys were made at approximately 1, 3, 4, 6, and 10 years. This chart shows a definite relationship between frequency of cracks and percentage of steel. Taking the results of the first survey as an example, the average crack intervals were 21.5 ft for the 0.3 percent section, 14.8 ft for the 0.5 percent section, 11.5 ft for the 0.7 percent section, and 9.0 ft for the 1.0 percent section. Expressed as frequency, these values correspond to 4.6, 6.8, 8.7, and 11.1 cracks per 100 ft. Similar relationships are also apparent for the data from the other surveys.

Another significant relationship shown by Figure 2 is that between crack development and age. It is seen that for each section the average crack interval decreased at a rather rapid rate for the first 3 or 4 years and afterward cracks developed at an increasingly slower rate until, at about 6 years the curves became very flat. It can be concluded, therefore, that transverse cracking has practically reached its equilibrium after 10 years and, barring unforeseen structural failures, few new transverse cracks may be expected to occur during the service life of the pavement. It is also apparent from Figure 2 that the development of transverse cracks approached an equilibrium somewhat earlier in the more heavily reinforced sections. The behavior of the 8-in. pavement sections with respect to crack development has been similar to that of the 7-in. sections, as shown by Figure 3.

CRACK WIDTH

Perhaps the requirement most necessary to the success of continuously-reinforced pavement is that the steel reinforcement hold transverse cracks to a narrow width. There are three reasons why narrow transverse cracks are essential. First, they must be narrow to prevent the progressive infiltration of incompressible materials, such as soil, which eventually might cause excessive compressive stress to develop in the pavement and produce blowups. Secondly, the transverse cracks must not admit appreciable amounts of surface water to the subgrade and, by the same token, if the pavement happens to be built directly on soils which are of the potentially pumping types, such as is the case with the Illinois pavement, the cracks must be maintained so tight that pumpable material cannot be ejected through them. Thirdly, the cracks must be held tightly closed so as to maintain effective aggregate interlock between the crack interfaces. The importance of the latter will be shown later in the discussion of failures that occurred at construction joints where no aggregate interlock was present.

The accurate measurement of the width of transverse cracks is difficult. At the time the Illinois pavement was constructed a series of brass reference plugs on 10-in. centers was installed along the edge of the central 30 ft of each test section for the purpose of measuring the widths of cracks which subsequently might form in the areas. Unfortunately, enough cracks did not form between the plugs to give reliable data. Foreseeing that this might occur, a measuring microscope capable of measuring to 0.001 in. was procured and, following the first crack survey in March 1948, 60 transverse cracks were selected from each test section for a program of width measurements, which would be continued as long as the pavement was under active observation.

The cracks were chosen to be representative of those throughout the entire length of the test section by selecting a group of 10 at each end of the section, another 10 at each quarter point, and a group of 20 cracks at the center of each section. Measurements have been made periodically since July 28, 1948, when the first set of readings was taken.

In an attempt to obtain measurements which would be more nearly representative of the true width of the cracks than would be the case if surface widths were measured, the microscope was focused some distance down in the crack and the width was measured at that point. Usually it was possible to register on the matching faces of a broken piece of aggregate. Although this method may not be extremely accurate, it nevertheless is believed that it gave fairly reliable results, although the widths so measured may be somewhat greater than the actual widths.

The data on crack width measurements at the 60 representative cracks are shown in Figure 4 for the 7-in. sections and in Figure 5 for the 8-in. sections. The data for the 8-in. sections are not as complete as those for the 7-in. sections because on several occasions it was not possible to take measurements on all the sections. Nevertheless, they show similar trends.

Except for the July 7, 1949, series of readings, in the case of both 7- and 8-in. pavements and the August 10, 1951, series, in the case of the 8-in. pavement, there is

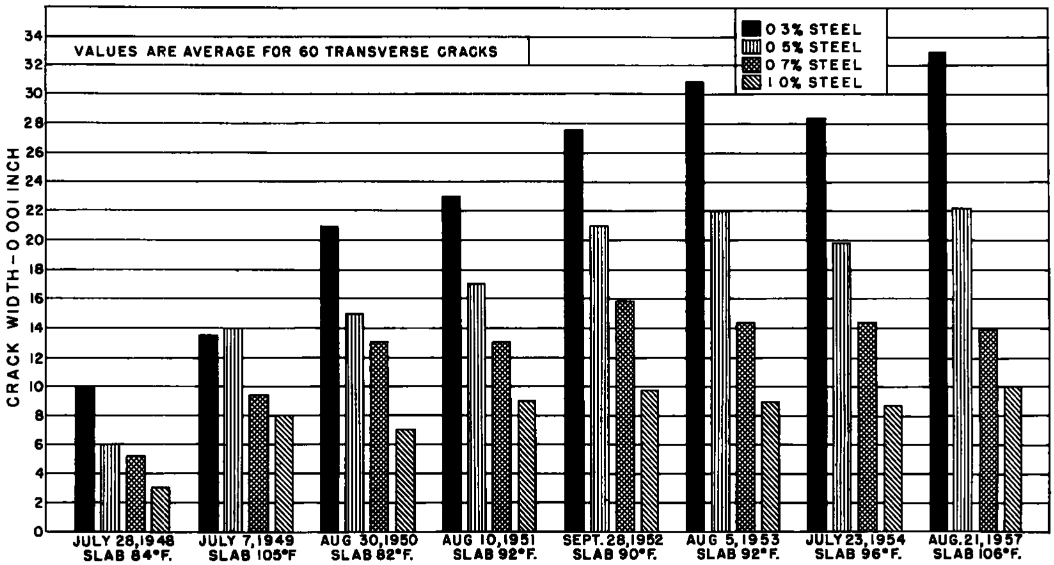

Figure 4. Effect of amount of longitudinal reinforcing steel on width of representative cracks, 7-in. sections.

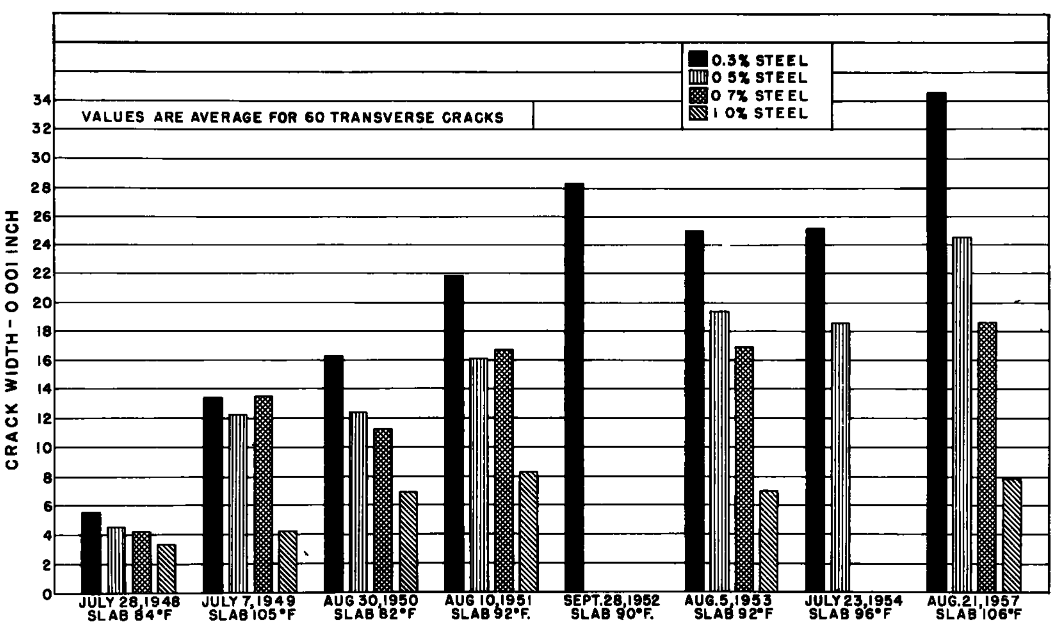

Figure 5. Effect of amount of longitudinal reinforcing steel on width of representative cracks, 8-in. sections.

a definite correlation between crack width and the percentage of longitudinal steel, the crack widths becoming progressively smaller as the percentage of steel increases.

It is readily apparent from these charts that the cracks have become wider with time and also that the influence of time is less in the more heavily reinforced sections. In the 9 years since the first measurements were made on the 7-in. sections, the average crack width in the 0.3 percent section has been increased by 0.023 in.; that in the 0.5 percent section by 0.016 in.; that in the 0.7 percent section by 0.009 in.; and that in the 1.0 percent section by 0.006 in.

The corresponding values for the 8-in. pavement are 0.028 in. for the 0.3 percent steel; 0.020 in. for the 0.5 percent steel; 0.014 in. for the 0.7 percent steel, and 0.0045 in. for the 1.0 percent steel. Except for the section with 1.0 percent steel, the cracks in the 8-in. pavement have shown greater increase in width than those in the 7-in. pavement. The probable reason for this is that 7-in. sections, being built in the fall of 1947, were on the average approximately 6 months older than the 8-in. sections (except Section 7 with 1.0 percent steel, which was also built in 1947) when the initial readings were taken. This difference in age may account for the crack widths at the time of the initial readings being greater in the 7-in. pavement than in the 8-in. pavement.

The appearance of the cracks for the most part is good. Although there has been some raveling along the edges, their appearance is by no means unsightly, even on close inspection. In fact they are not visible from a vehicle travelling 30 mph, except at a few locations where maintenance forces, misunderstanding instructions, poured a number of cracks with asphalt. Figure 6 shows a typical crack in each section of 7-in. pavement at the age of 3 years and the same cracks when the pavement was approximately 10 years old.

What is of real significance is whether the cracks have become so wide that they fail to meet the criteria previously stated; that is, do they permit infiltration of soil, do they allow entrance of appreciable amounts of surface water, and have they lost effective aggregate interlock?

The value of the steel in preserving aggregate interlock appears to be unquestioned. Except at a relatively few cracks in the 0.3 percent sections, no faulting has occurred at transverse cracks. This is in sharp contrast with the serious faulting which has developed at a number of construction joints in the lighter reinforced sections.

Whether the cracks meet the other two requirements is a difficult question to answer. Based on the measurements made, it appears that many of the cracks, particularly those in the 0.3 and 0.5 percent sections, now are so wide that they would allow infiltration of fine incompressible material and entrance and ejection of water. It has been stated previously that because of the method of measurement the measured widths may be somewhat wider than the true widths.

Cashell and Teske (5) has stated: "The real widths of all cracks are many times smaller than their respective surface widths." They reported a surface width of approximately 0.08 in. in pavement with 1.0 percent of steel at the age of 10 years. It is understood that these measurements were made at the surface and thus were influenced to a greater extent by raveling than the measurements made on the Illinois pavement. From limited measurements made midway down the edge of the pavement at the age of 10 years, Indiana found the real widths of cracks in the sections containing 1.82, 0.45, and 0.24 percent steel to be 0.004, 0.011, and 0.013 in., respectively. Based on these data, it would appear that the measured widths obtained in Illinois probably exceed the true widths.

It is understood that cores recently drilled through cracks in the New Jersey continuously-reinforced pavement (6) showed that cracks which appeared very wide on the surface tapered sharply to a fine crack a short distance from the surface. To determine whether such was the case in the Illinois pavement, a core was drilled through at least one of the representative cracks in each test section. Examination of these cores indicated that the cracks were widest at the surface, tapered gradually in width to the approximate depth of the steel, then remained fairly constant in width to the bottom of the pavement. For example, microscope measurements of a core from Section 1 (0.3

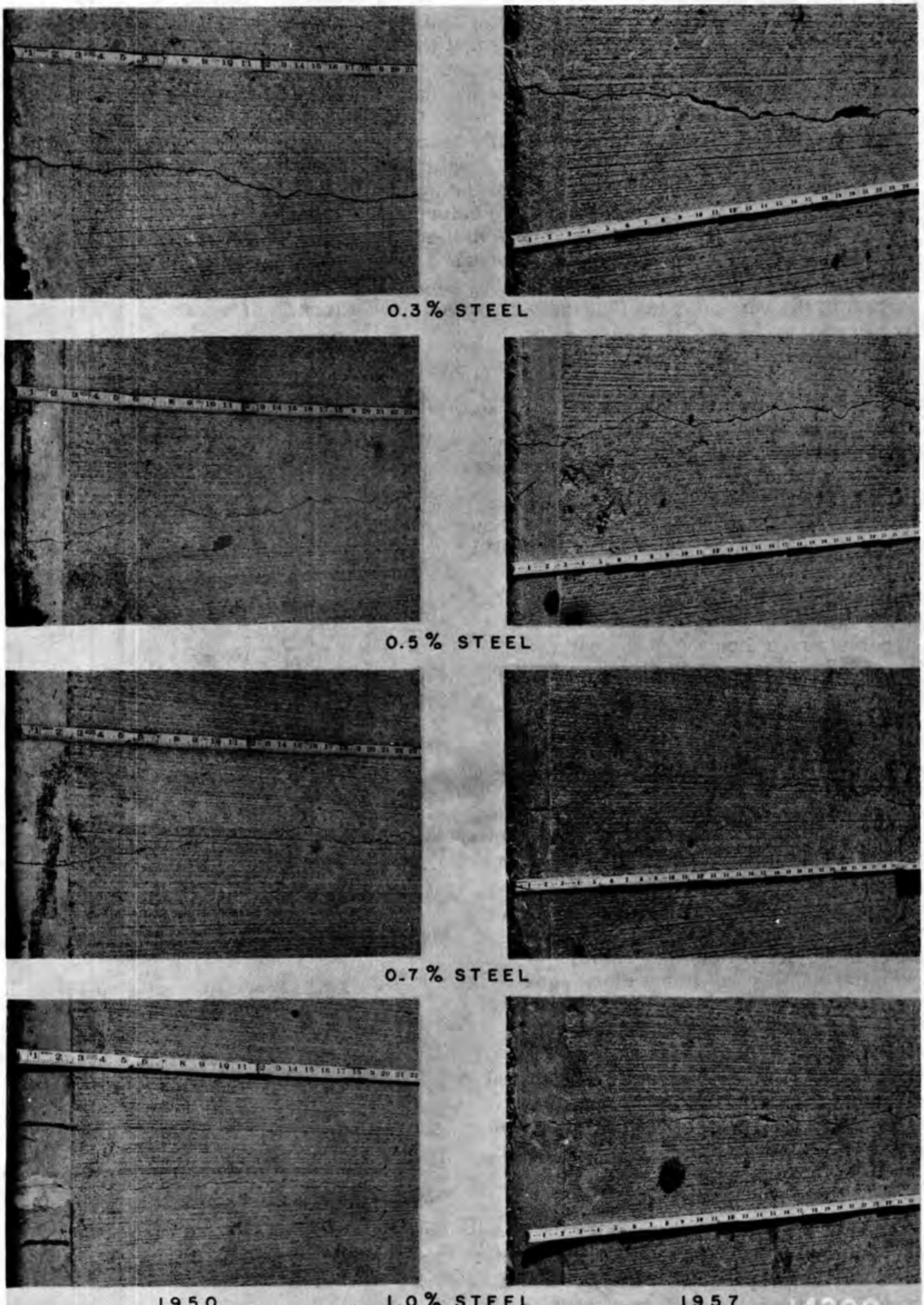

Figure 6. Typical transverse cracks in 7-in. sections, 1950-57.

percent, 7-in.) showed a width of 0.040 in. slightly below the surface, 0.025 in. at 1-in. and 2-in. depths, 0.020 in. at 3 in., and approximately 0.015 in. for the remainder of the depth. It appears, therefore, that the real width of cracks is less than the values given in Figures 4 and 5. On the basis of limited data, perhaps a reasonable approximation of the real width would be one-half the measured surface width.

The cores tend to verify that soil has collected in the cracks. A thin layer of soil was found in the crack in all but one of the cores, and it appears that the permanent opening of the cracks may be due to this condition.

A 4-in. core was taken through one of the cracks in Section 1 which has become quite wide and where faulting has occurred, to determine whether the reinforcing bars have failed, as was suspected. The core was drilled so as to intersect one bar. The bar was found to be necked down and fractured where it intersected the crack. Although there was some evidence of corrosion, the presence of deformations on the "necked down" portion of the bar indicates that the bar had suffered a tensile deformation before it fractured.

Although there are a number of these wide cracks in the 0.3 percent sections, they are by no means representative. To check on the condition of the steel at a more representative crack, a 2-in. core was taken so as to intersect a bar at a crack just 13 ft from the one in which the steel was broken. The crack width at the depth of the steel was approximately 0.015 in. and the bar appeared to be tightly bonded in the concrete, even in the region of the crack, in spite of the presence of a sizeable air void on top of the bar near the crack. This core is shown in Figure 7.

Figure 7. Core through representative transverse crack in 7-in. pavement with 0.3 percent steel.

An interesting sidelight of the coring was that it showed how closely the steel was set to the spacings given in the plans. Eleven 2-in. cores and one 4-in. core were drilled. The locations for ten of the cores were selected so as to fall between two bars and the other two to intersect a reinforcing bar. These locations were determined by measurements based on the planned spacings of the bars. In every case the objective was successfully accomplished. The apparent close agreement between planned and actual spacings thus demonstrated is undoubtedly due to the ingenious design of the chair bars, which supported the longitudinal steel and provided positioning stops at regular intervals for controlling the location of the longitudinal bars.

LONGITUDINAL CRACKS

As has been stated, the pavement was constructed without the conventional center joint for the purpose of finding out how a pavement with continuous transverse reinforcement would act. The results have been quite variable. The five sections constructed in the fall of 1947 all developed a considerable amount of longitudinal cracking the first year. In some locations there were two parallel cracks and in at least two places there were three.

On the other hand, the three sections constructed in the spring of 1948 have developed relatively little transverse cracking. Table 2 gives the projected centerline lengths of longitudinal cracks in each section at six ages. The cracking naturally does not coincide with the centerline of the pavement, but follows an irregular path, meandering as much as 3 ft on either side of the centerline.

In the early years there was a quite definite relationship between the length of longitudinal cracking and the amount of longitudinal steel in the 7-in. sections, and this is still apparent after 10 years in the case of the 0.3, 0.7, and 1.0 percent sections. But for some unknown reason the 0.5 percent section does not follow the same pattern.

The surface width of the longitudinal cracks is probably greater than any of the transverse cracks, although no attempts have been made to measure them because of the extreme hazard involved. However, one core drilled through a representative longitudinal crack in Section 8 (7-in., 0.3 percent) showed that the crack was several times wider than that in the core through a representative transverse crack. The longitudinal cracks are easily visible from a vehicle travelling at normal driving speeds. They probably are more noticeable because there is a continuity of vision and the eye never loses them. Aside from presenting an undesirable appearance, they seem to have had no apparent undesirable effects. Some engineers in the Division have expressed the opinion that they should be sealed, but there seems to be no real need for this. Sealing would be a hazardous task and, unless a material was used whose color blended with that of the concrete, the unsightliness would be increased. Based on this experience, it is believed that a continuously-reinforced pavement should have a joint along the centerline.

PAVEMENT PUMPING

At the time the pavement was planned it was believed, largely on the basis of the behavior of the Indiana pavement and on analytical concepts, that properly designed continuously-reinforced pavement, by reason of the absence of joints, the tightness of cracks, and an inherent flexibility, which would permit it to conform closely to the subgrade, should develop little or no pumping even when constructed over a pumpable subgrade. Also it was realized that if this were true the elimination of a granular subbase would tend to offset the higher cost of the heavy reinforcement. It appeared

TABLE 2

LONGITUDINAL CRACKING IN TEST SECTIONS

Pavement Thickness (in.)	Section No.	Date Constructed (inclusive)	Longitudinal Steel (%)	Longitudinal Cracking (%)					
				March 1948	Dec. 1948	Sept. 1950	Sept. 1951	March 1954	Sept. 1957
7	1	9/25/47 9/30/47	0.3	0	28	34	42	54	64
	2	9/30/47 10/3/47	0.5	0	31	49	72	86	94
	3	10/3/47 10/7/47	0.7	0	42	58	64	66	72
	8	10/14/47 10/17/47	1.0	46	80	82	95	96	97
8	4	4/30/48 5/20/48	0.3	-	0	1	2	3	6
	5	4/26/48 4/30/48	0.5	-	0	<1	3	5	6
	6	11/6/47 12/3/47 4/22/48 4/26/48	0.7	-	0	<1	4	12	13
	7	10/14/47 10/17/47	1.0	52	74	79	83	84	85

logical, therefore, to build the pavement on the natural subgrade in order to confirm these assumptions.

The subgrade is composed of soils of the following groups in the amounts given:

	%
Silty clay and clay of the A-7-4 group	35
Silty clay loam, silty clay, and clay loam of the A-4 group	33
Clay loam of the A-4-2 group	11
Clay of the A-6 group	10
Sandy loam of the A-2 group	11

The A-4, A-6, and A-7 soils are well known for their pumping characteristics when water is present and heavy vehicles in sufficient number travel on a pavement built over them. Hence, 89 percent of the pavement is built over potentially pumping soils.

There is no question about volume and weight of vehicles being sufficient to produce pumping. Table 3 shows the average 24-hr traffic volumes (both directions) in 1950, 1953, 1956, and 1957 through August. It is significant that in that period the number of tractor-semitrailer units has increased from 300 to 1,020 vehicles per day, or 240 percent. Table 4 shows the monthly rainfall from the time the pavement was opened to traffic. It is apparent, therefore, that all the factors necessary to produce pumping have been present many times during the 10-year period.

Serious pumping has occurred at the expansion joints, which separate the sections, and at some of the construction joints, which were built at the end of each day's run. Pumping has been so severe at the expansion joints that the ends of the adjacent slabs at some of them have broken so badly that the concrete had to be removed and replaced with concrete patches. Although pumping at these joints has created a maintenance problem, it should be remembered that they are not a necessary part of a continuously reinforced pavement, having been installed in the experimental pavement only for the purpose of permitting the various test sections to act independently of one another, and the distress at these locations should be disregarded in any evaluation of the performance of this pavement.

Pumping has occurred at one time or another at most of the construction joints in the 0.3 and 0.5 percent sections in both the 7- and 8-in. pavements. At some of these locations major maintenance, such as removal and replacement of small areas, has been required. When the pavement was last surveyed on October 28, 1957, all three of the construction joints in Section 1 (7-in., 0.3 percent steel), two of the three construction joints in Section 2 (7-in., 0.5 percent steel), and all three of the construction joints in Section 4 (8-in., 0.3 percent steel) showed evidence of rather severe pumping.

TABLE 3

TRAFFIC VOLUMES[1] ON EXPERIMENTAL PAVEMENT

Type of Vehicle	1950	1953	1956	1957[2]
Passenger cars	2,060	3,100	2,885	2,850
Single-unit trucks	305	340	335	330
Semitrailers	300	760	1,030	1,020

[1] Average 24-hr count.
[2] January through August.

TABLE 4

MONTHLY RAINFALL IN AREA OF EXPERIMENTAL PAVEMENT[1]

	Rainfall (in.)												
Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual Total
1948	2.26	2.00	7.17	0.87	3.72	6.86	6.07	1.64	2.96	3.95	4.97	2.81	45.28
1949	7.65	3.46	2.97	0.83	4.61	5.01	2.00	1.86	3.97	7.91	-	4.45	
1950	9.06	4.85	2.38	5.00	1.44	4.46	3.25	3.50	1.69	1.38	3.75	0.98	41.49
1951	2.41	4.07	2.73	2.88	1.82	11.54	8.32	1.59	3.03	3.12	3.00	2.67	47.18
1952	0.95	1.56	5.44	3.60	1.90	2.20	4.12	1.35	3.85	0.19	3.00	2.02	30.18
1953	1.03	1.46	4.79	4.74	2.30	3.13	2.33	0.43	0.93	1.84	0.68	1.23	24.89
1954	2.01	0.98	1.21	3.06	2.28	-	-	3.71	1.90	-	0.78	1.62	
1955	2.36	3.02	2.38	4.08	4.17	2.63	6.42	3.30	3.20	4.03	2.92	0.22	38.73
1956	0.97	4.28	1.28	3.20	5.21	3.11	4.00	3.02	1.19	0.94	3.00	4.14	34.42
1957	1.47	2.26	1.73	8.41	9.33	11.58	10.77	3.47	-	-	-	-	

[1] Data from U. S. Weather Bureau station at Vandalia Airport, about 2 mi north of experimental pavement.

All of the joints in Section 1 were open about ¼ in. and it is quite probable that the reinforcing bars are broken. One of the four construction joints in Section 5 (8-in., 0.5 percent steel) was pumping slightly. All construction joints in the sections containing 0.7 and 1.0 percent steel were in excellent condition, were not faulted, and showed no evidence of pumping. This suggests that bars across construction joints should be at least No. 5 at 6-in. centers. Another way of strengthening construction joints would be to use auxiliary plain round dowels. Figure 8 shows pumping along the south edge of the pavement in the vicinity of the construction joint at Sta. 190+40 in Section 1.

In an earlier report (2) it was stated that relatively few cracks in the 8-in. section with 0.3 percent steel, and the 7-in. section with 0.5 percent steel, had shown soil stains, which might be considered as incipient pumping, but that the condition had later improved. It was thought at the time that perhaps the soil collecting in the narrow cracks had acted as a seal. This was confirmed recently when cores drilled over a number of cracks showed a deposit of soil in the bottom of the cracks. Pumping since has been observed at some of the wider cracks in the 0.3 and 0.5 percent sections, but this is by no means a representative condition, as these cracks are ones where broken steel is suspected. In the 0.7 and 1.0 percent sections no pumping has been observed through cracks.

A little edge pumping away from expansion joints and construction joints has been observed in some sections, particularly those with 0.3 and 0.5 percent steel. This pumping is not extensive or severe and does not seem to have affected the performance

Figure 8. Edge pumping near construction joint in 7-in. pavement with 0.3 percent steel.

of the pavement. It seems to occur particularly where the shoulders have become considerably lower than the pavement surface.

In a previous report (2) mention was made of the presence of vertical holes in the earth shoulders where they meet the edge of the pavement. At that time the conclusion was reached that these holes were not caused by pumping, as there was no evidence that subgrade material had been ejected through them. Because at that time they were predominately along the south edge of the pavement, where it was known that the shoulder material had not been as well compacted as along the north edge, it was thought that the holes were the result of water flowing down the edge of the pavement to fill cavities in the soil caused by subsequent settlement of shoulder and subgrade material, and also the space between the subgrade and the slab when the edges of the pavement are curled upward. Later, many of these holes disappeared and it was thought that shoulder maintenance tended to correct the condition.

These holes are still found, particularly along the 0.3 percent sections. They appear to come and go, being more prevalent at one time than another. Careful examinations of these holes have shown no evidence that subgrade material has been ejected. A number have been carefully dug out, and in no case have cavities been found under the edge of the slab. In fact, the slab was observed to be in close contact with the subgrade.

Although pumping, except at expansion joints, which would not be incorporated in a normal continuously-reinforced pavement, and at construction joints, whose weaknesses are now recognized and can be corrected by providing additional load transfer through the use of heavier reinforcing bars or dowel bars, has not been extensive, some pumping has occurred outside these areas.

It is well known that pumping, even though it starts in a small way, is likely to become progressive, and it probably would be a wise practice in anything but an experimental pavement to use a layer of granular material under a continuously-reinforced pavement as a preventive measure where high volumes of heavy vehicles are anticipated. Considering that pumping was not general or of serious consequence away from joints, it is believed that in general a 3-in. layer of granular material would be sufficient.

CHANGE IN LENGTH

The 4-in. expansion joints, which separated the test sections from one another and from the standard pavement adjoining each end of the experimental pavement, were installed so that each section would have considerable freedom to change length independently. Permanent reference monuments were provided at the ends and at selected locations along each section for measurement of longitudinal and vertical movements. Changes in length which have been measured on the sections of 7-in. pavement at various times are shown in Figure 9. Similar data for the 8-in. sections are shown in Figure 10.

It is obvious that the sections change length seasonally, being longer in summer and shorter in winter. Generally speaking, the sections were from 1 to 2 in. shorter in any given winter than they were when the previous summer measurements were taken. One of the concepts of continuous reinforcement is that a large central portion of a long continuously-reinforced pavement remains under restraint and movements occur near the free ends only. When it is considered that the test sections, if entirely free of restraint of any kind, would be expected to undergo seasonal changes in length of 12 to 15 in. due to temperature changes alone, it is apparent that the test sections are under considerable restraint.

Of more significance, however, is the permanent increase that has occurred in the length of every section. Figures 9 and 10 indicate that all the sections are now on the average approximately 4 in. longer than their original lengths. In the summer of 1957 the expansion joints, which were nominally 4 in. wide when constructed, appeared to be tightly closed. In fact, at several of these joints large spalls have occurred due to localized compression where the joint faces were irregular. Some of these areas were so extensive that major repairs were required.

The cause of this growth cannot be stated with any certainty. It is conceivable that the permanent increase in length might be due at least in part to growth of the concrete. Data from the Indiana project indicated that at 10 years the concrete had developed a

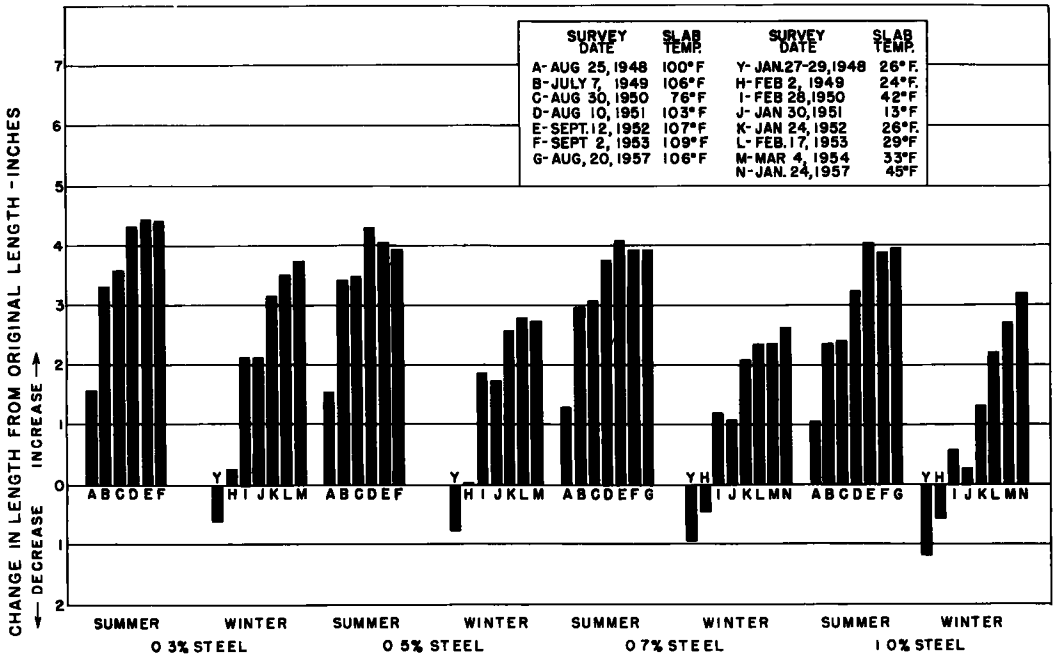

Figure 9. Seasonal changes in length of 7-in. test sections containing various percentages of reinforcing steel.

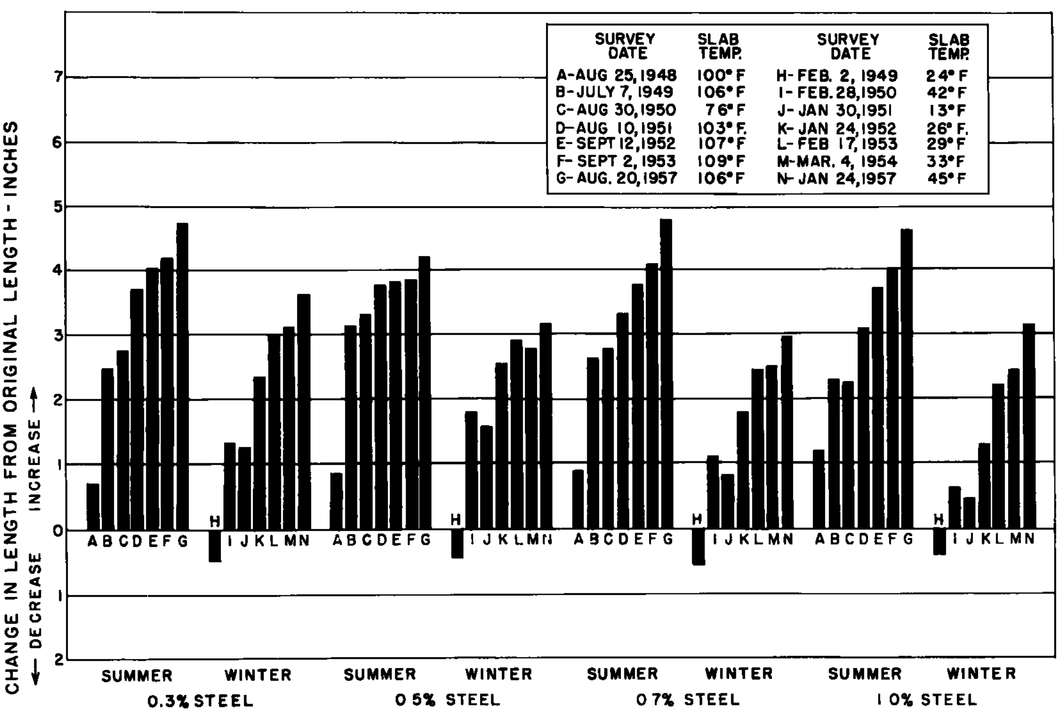

Figure 10. Seasonal changes in length of 8-in. test sections containing various percentages of reinforcing steel.

growth equivalent to $\frac{1}{16}$ in. per 100 ft of pavement (5). A growth of $\frac{1}{4}$ in. per 100 ft was observed at 10 years in the non-reinforced sections in the Bureau of Public Roads investigation at Arlington (5). Thus, the possibility of growth in the concrete cannot be overlooked.

To check this theory a review was made of the Whittemore strain gage measurements which were made periodically between the series of brass plugs set on 10-in. centers along the edge of the pavement in the 30-ft central portion of each test section. Although there are some inconsistencies in the data which need further study, even after eliminating from consideration those readings which were influenced by the presence of transverse cracks, there are strong indications that the concrete has not grown appreciably. The readings taken in August 1957 varied from the initial readings, in most cases, by only 0.001 to 0.002 in., and there was as many minus values as there were plus ones.

It was thought that perhaps the growth may be associated with a permanent opening of the transverse cracks. The cracks, once they form, do not close fully thereafter, even in periods of high temperatures when the pavement is under compression. Whether this permanent opening of cracks is due to infiltration of soil, which may appear logical, is only speculative. Emphasis is given to this speculation by the fact that cores drilled through transverse cracks showed a thin layer of packed soil in the crack. Furthermore, an analysis of crack width data and the permanent increase in length of sections lends some credence to this speculation. When the change in average crack width for a particular section is multiplied by the total number of cracks in the section a value is obtained which, though somewhat greater, is reasonably close to the increase in length of the section. When it is considered that the measured width of cracks, due to the method used, is likely to be somewhat greater than the true width, the agreement is even closer.

Considering the test sections collectively, the increase in length of the 5½ mi of pavement has been between 30 and 40 in. One wonders what would have happened had the entire 5½ mi been built continuous and without any provision for expansion. Would the additional restraint imposed on the pavement have resulted in less over-all growth? Or would excessive compressive stresses have been developed that would have caused blowups? These are questions which cannot be answered on the basis of present knowledge. Neither is it possible to determine how much provision for expansion is required nor the manner in which it should be provided.

RIDING QUALITY OF SURFACE

One of the important claims for continuously-reinforced pavement is that it provides an unusually smooth riding surface. It is believed that there is better opportunity to build the surface to a higher degree of smoothness because it is unnecessary to halt construction and finishing operations for the installation of joints, as is necessary in the case of the conventional pavements in which plate-type joints or groove-type (other than sawed) joints are used. Furthermore, these joints, even when properly installed, may later reduce riding comfort due to curling of slab ends, faulting of slab ends, the joint opening being too wide, and to excess sealing materials which result in a ridge.

The experimental pavement at Vandalia has been recognized for its excellent surface smoothness ever since it was opened to traffic. Some engineers in the Division of Highways have referred to it as the smoothest concrete pavement in the state. Unfortunately, no equipment for measuring surface variations or for determining relative riding quality was available at the time the pavement was opened to traffic; therefore, until recently when the Division of Highways procured a roughometer of the type developed by the U.S. Bureau of Public Roads, any evaluation of riding quality had to be a matter of individual judgment. It was generally agreed that the experimental pavement was exceptionally smooth when built and has retained this characteristic.

Table 5 gives the results of roughometer measurements made August 13, 1957, on the the experimental pavement and also on a 2.32-mi section of standard pavement, which adjoins the experimental pavement of the west and was constructed at the same time and by the contractor who built the experimental pavement. The standard pavement is 10-in. thick, reinforced with 78-lb wire fabric, with plate-type contraction joints at 100-ft intervals.

The readings on the experimental pavement varied from 60 to 90 and averaged 72 in. per mile. Good uniformity existed throughout the length of the experimental pavement, and there were no significant differences between test sections. The average roughness index on the standard pavement was 81 in. per mile. It is doubtful that the difference between the two is of real significance. It is generally considered that an index under 90 in. per mile indicates satisfactory surface smoothness and assures good riding qualities.

TABLE 5

ROUGHNESS INDEXES FOR EXPERIMENTAL PAVEMENT AND ADJOINING STANDARD PAVEMENT
(Readings taken August 13, 1957)

Test Section	Sub-Section	Length (ft)	Roughness Index (in./mi) Westbound Lane OWP	Westbound Lane IWP	Eastbound Lane OWP	Eastbound Lane IWP
(a) Continuously-Reinforced Pavement, Section 0-2						
1	A	1,752	69	72	78	75
	B	1,752	75	66	78	72
2	A	1,752	78	63	72	72
	B	1,752	72	63	69	72
3	A	1,752	66	66	69	75
	B	1,752	63	63	69	72
4	A	1,752	78	66	75	69
	B	1,752	78	69	66	66
5	A	1,752	81	66	72	66
	B	1,752	90	72	69	75
6	A	1,754	87	72	72	69
	B	1,754	84	90	78	81
7	A	2,116	75	75	60	65
	B	2,116	82	77	65	67
8	A	2,116	72	80	62	65
	B	2,116	72	75	65	65
Avg.			77	71	70	70
(b) Standard Reinforced Pavement, Section P-2						
Avg.			80	78	86	80

PAVEMENT SLAB MAINTENANCE

Maintenance of the surface of the experimental pavement has been confined almost exclusively to the repair of distressed areas that developed adjacent to 4-in. expansion joints and at construction joints. These areas of failure are further restricted to the test sections containing 0.3 to 0.5 percent reinforcement. Because, in a regular continuously-reinforced pavement, expansion joints would be installed only infrequently and would be specially designed for the service, no consideration should be given to the maintenance costs at the 4-in. expansion joints. Furthermore, experience has shown that the weakness inherent in the construction joints can be corrected easily, so maintenance at those points would not be a factor in a properly designed continuously-reinforced pavement. Based on the experience in Illinois, therefore, and eliminating the joints from consideration, it can be stated conclusively that the continuously-reinforced pavement has required practically no surface maintenance in the 10-year period since its construction. Surface maintenance costs in the same period for approximately 3.4 mi of adjoining standard pavement west of the experimental pavement amounted to approximately $85 per mile per year. It would appear, therefore, that surface maintenance costs on a properly designed continuously-reinforced pavement could be expected to be low and to compare favorably with those for a pavement of conventional design.

ECONOMIC CONSIDERATIONS

There are three major factors involved in the economic evaluation of continuously-reinforced pavement; namely, first cost, maintenance cost, and service life. Although the experience in Illinois does not as yet give definite data on these factors, it nevertheless furnishes information which permits some general observations.

No accurate information is available to the Division of Highways as to the contractor's true costs for constructing the experimental pavement. The only comparison available is based on his bid prices, and an analysis of these shows that he bid approximately the same unit price for the 7-in. pavement with 0.7 percent steel, the 8-in. pavement with 0.5 percent steel, and the standard 10-in. wire fabric reinforced pavement with 6-in. granular subbase. It is reasonable to assume that the contractor made generous allowances in his unit prices for the continuously-reinforced pavement to cover uncertainties due to the experimental nature of the project. The special requirements for research, the close supervision to be expected from engineers, and the fact that neither his organization nor the Division of Highways had had previous experience with this type of construction. Statements by key personnel of the contractor, which indicated that they were very happy with the manner in which the work proceeded and that their construction costs were well below anticipated costs, tended to confirm this.

It seems reasonable to assume, therefore, that when costs are not influenced by the complications of a research project and when contractors become familiar with the type of construction, the cost of continuously-reinforced pavement of the design recommended hereinafter should compare favorably with the cost of standard 10-in. wire fabric reinforced pavement. It would seem that more conclusive information as to relative costs will be obtained from the projects recently constructed in which a more conventional and perhaps a lower cost method of installing the continuous reinforcement was used.

Practically no surface maintenance has been required on the Illinois pavement outside the areas immediately adjacent to the expansion and construction joints, which are not representative of a regular continuously-reinforced pavement. Furthermore, there is nothing to indicate that any unusual maintenance will be required in the foreseeable future. It appears, therefore, that the cost of maintaining a properly designed continuously-reinforced pavement quite likely will be less than that for a pavement of conventional design.

Both the experimental pavement in Illinois and the adjoining standard pavement are in excellent condition after 10 years of service. There is no evidence now that one will outlast the other. It appears, however, that when a properly designed continuously-reinforced pavement reaches the age when it is no longer adequate as a surface course, it will provide a better base for bituminous resurfacing than a pavement of conventional design.

Summarizing this discussion, it would appear that on the basis of construction cost, maintenance cost, and serviceable life, continuous reinforcement is economically sound.

RECOMMENDED DESIGN FOR FUTURE PAVEMENTS

The data from the several experimental pavements which have been under observation for varying periods of time do not permit formulation of a rational theory of design. The behavior of the various test sections in the Illinois pavement, however, provides an empirical approach to the selection of proper values for certain variables and to the solution of other design features.

Considering first of all pavement thickness, there has been no noticeable difference in performance between the comparable sections of 7- and 8-in. pavements. Therefore, it might be concluded that a 7-in. thickness is adequate. But when one considers the sharp increases which have occurred in recent years in the volume of heavy vehicles on the highways and that this increase is likely to continue, conservatism appears to be the wise course, and 8-in. thickness is recommended.

A more difficult problem is the selection of the minimum percentage of longitudinal steel. The experimental pavement shows definitely that pavement behavior and present condition vary with the amount of longitudinal steel; the 1.0 percent sections being better than the 0.7 percent sections, the 0.7 percent sections being superior to the 0.5 percent sections, and the 0.5 percent sections out-performing the 0.3 percent sections. These four percentages were selected because it was believed they would provide sufficient range in performance that the optimum practical amount could be determined.

It was thought that 0.3 percent would prove wholly inadequate and that those sections would fail extensively and early. It was further believed that 1.0 percent was ultraconservative. In view of the poor expectations for the 0.3 percent sections, their behavior has been amazing. Generally speaking, and with the exception of apparent broken steel and faulting at some wide cracks and failures at expansion joints and construction joints, which, as has been previously stated, should be disregarded in this evaluation, the 0.3 percent sections are in good condition and should continue to give satisfactory performance for a considerable number of years. This is not to say that 0.3 percent is adequate, but it does suggest that the steel requirement may be less than was first supposed.

Stress measurements during the first winter (2) indicated that the steel in the 7-in. section with 0.7 percent steel was at times under a stress of approximately 62,000 psi, which was close to the yield point of the rail-steel bars used. This stress is in excess of minimums specified by ASTM for rail-steel bars, new billet hard grade bars, and wire fabric reinforcement, the logical materials for this purpose. At first glance, this would suggest that perhaps an amount somewhat greater than 0.7 percent, perhaps 0.8 percent, should be recommended.

One cannot disregard the fact that the sections with 0.5 percent have given good performance and in general are in excellent condition. Therefore, 0.5 percent is suggested as an absolute minimum, but 0.6 percent is recommended as a more conservative value. Also, because there is some indication that the steel in pavements built at summer temperatures is subject to higher stresses than when construction temperatures are lower, perhaps 0.7 percent steel should be used in pavements constructed in extremely hot weather.

The unsightly longitudinal cracks which developed in the Illinois pavement leave no doubt about the need for a controlled center joint.

Transverse reinforcement is a matter of what is required to support the longitudinal steel from the subgrade, if the method of installation employed in the Illinois pavement is used. Based on the experience in Illinois, No. 3 bars at 18-in. centers are adequate. If welded fabric or bar mats are used, the size of the transverse members would be dependent on manufacturing considerations.

The requirements for expansion joints are difficult to determine. Certainly some provision for expansion would be required adjacent to structures. But whether expansion joints would be required at regular intervals in long stretches of continuously-reinforced pavement is presently indeterminate. Further research is needed in which the movements in a long (4- or 5-mi) consinuously-reinforced pavement without expansion joints could be studied.

Although pumping has not been a serious problem except at expansion joints and construction joints, some has occurred, and it is recommended that a granular subbase be used. It is believed that a 3-in. layer would be sufficient to control pumping, but greater depths might be required in some cases to counteract the effect of frost action in frost-susceptible soils.

CONCLUSIONS

The information obtained from observations and study of the Illinois experimental pavement support the following conclusions:

1. Large numbers of transverse cracks develop in continuously-reinforced pavements, their frequency being proportional to the amount of longitudinal steel.
2. Cracks are relatively wide-spaced near the ends of the long continuously-reinforced slab; their frequency increases at a fairly uniform rate to a maximum at a distance of 200 to 500 ft from the ends, from whence the frequency remains fairly constant over the long central portion
3. A large number of transverse cracks occurs very early, and thereafter the development of cracks is a function of age, the rate of development decreasing with time. It appears that transverse cracking in the Illinois pavement has reached an equilibrium at the end of 10 years and that cracks can be expected to occur at a very slow rate in the future.
4. After 10 years the average crack intervals for the sections with 0.3, 0.5, 0.7, and 1.0 percent longitudinal steel, respectively, are approximately 12, 8, 6, and 5 ft, considering the 7- and 8-in. pavements collectively. There is no significant difference between the two thicknesses of pavement.
5. The width of cracks is an important factor in the performance of continuously-reinforced pavement. Cracks must be narrow to minimize infiltration of soil and water and to maintain effective aggregate interlock between crack interfaces.
6. Crack width is a function of the amount of longitudinal steel, the greater the percentage of steel the smaller the crack. It also increases with age.
7. At the age of 10 years the crack width measured by focusing a measuring microscope on the point below the pavement surface, so as to reduce the effect of raveling, averages 0.034, 0.023, 0.016, and 0.009 in. for the 0.3, 0.5, 0.7, and 1.0 percent sections, respectively, considering both 7- and 8-in. pavements collectively. There are no significant differences between the two thicknesses.
8. Cores drilled through representative cracks in each section indicate that the real width of the cracks is less than that measured by the microscope. On the basis of limited data, perhaps a reasonable approximation would be that the real width is about one-half of the measured width.

9. Raveling and spalling along the edges of the cracks is of no serious consequence, except at a few cracks in the 0.3 percent sections which are excessively wide and where apparently the steel is broken. The cracks are not visible from a vehicle traveling at 30 mph.

10. Meandering longitudinal cracks will occur in a continuously-reinforced pavement built without a controlled center joint. Although they may be of little significance structurally, they are likely to be considerably wider than the transverse cracks, will ravel more, and become unsightly.

11. Continuous reinforcement will reduce the tendency for pumping, judging from behavior of the Illinois pavement. However, in view of the fact that some pavement pumping occurred, it appears that it would be wise to use a thin granular subbase under this type of pavement as an added safety factor.

12. Construction joints are points of potential weakness and need provision for adequate load transfer. This can be accomplished by using at least No. 5 bars at a maximum of 6-in. centers, or by providing auxiliary dowel bars when a lesser amount of reinforcing steel is used.

13. Sections of continuously-reinforced pavement of the lengths used in this investigation, and separated by wide expansion joints, will show a permanent increase in length with age. The cause of this growth is not readily apparent. Whether a similar increase would occur in sections several times that length is not known.

14. Continuously-reinforced pavement can be built to a high standard of surface smoothness, and can be expected to possess good riding qualities throughout a long life.

15. A properly designed and constructed continuously-reinforced pavement will require little maintenance and will compare favorably in this respect with pavement of conventional design.

16. It appears that on the basis of first cost, maintenance cost, service life, and salvage value, continuously-reinforced pavements are economically sound.

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Ten-Year Report on Experimental Continuously-Reinforced Concrete Pavements in New Jersey

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This paper reports on the behavior and performance of two experimental sections of continuously-reinforced concrete pavement constructed in New Jersey in 1947. These sections, each approximately 1 mi long, have been subjected to relatively heavy truck traffic.

The northerly section is of 8-in. uniform thickness, and contains 0.90 percent of longitudinal reinforcing steel. The southerly section is of 10-in. uniform thickness, and contains 0.72 percent of longitudinal steel. In both sections the steel consists of a double line of welded wire fabric.

The paper includes data relative to the recorded changes in length, crack pattern, crack width, behavior of the terminal joints, and the effect of these sections on the adjacent concrete pavement. It also describes the defects which have developed, and compares the performance and cost of these sections with that of the standard design of reinforced concrete pavement constructed on the same route.

●THE FOLLOWING constitutes a report on two experimental sections of continuously-reinforced concrete pavement constructed in New Jersey during the fall of 1947. Two previous reports have been published (1, 2). Although complete details concerning the design, construction and materials may be found in the first report (1), it appears desirable to include certain of this information in this report.

In particular, both sections are located in the northbound roadway of Route 130, in the vicinity of Hightstown, N. J., and were constructed in connection with a project known as Route 25, Sections 25C and 22D. Traffic is one-directional on these sections; northbound only.

The southerly section, which is separated from the northerly section by a series of slabs and a bridge, is 5,130 ft long, of 10-in. uniform thickness, and contains 0.72 percent of longitudinal steel. The outside (travel) lane was constructed during the period September 8-16. During this period the air temperatures, midnight to midnight, ranged from a minimum of 60 F to a maximum of 89 F. The inside (passing) lane was constructed during the period September 23-30, the air temperatures ranging from 36 F to 70 F.

The northerly section is 5,430 ft long, of 8-in. uniform thickness, and contains 0.90 percent of longitudinal steel. The outside lane was constructed during the period October 3-10, the air temperatures ranging from 43 F to 78 F. The inside lane was constructed during the period October 14-20, the air temperatures ranging from 49 F to 79 F.

Both sections have an over-all width of 24 ft, and consist of two 12-ft lanes, constructed independently. The longitudinal joint between the lanes is of the tongue-and-groove type, without tie bars. Because of this absence of tie bars, each lane has been more or less free to expand and contract independently.

The reinforcing steel in both sections consists of a double line of welded wire fabric, in the form of mats 16 ft 3 in. long, installed by the strike-off method. The plans called for the upper line of steel to be 2 in. below the top of the pavement, and for the lower line to be 3 in. above the bottom, in both sections. A 15-in. lap was employed, except on the resumption of work at the construction joints, where a 4-ft lap was employed. During the first four days work the top mats were installed directly above the bottom mats. In the remainder of the work, however, in order to avoid laps in both mats at the same points, the top mats were installed approximately 4 ft to the rear of the bottom mats.

The longitudinal members consist of cold-drawn wire, 3/8 in. in diameter, spaced 3 in. c. to c. Cross-sectionally, there are 94 of these members per lane. The transverse members consist of No. 5 cold-drawn wire, spaced 12.2 in. c. to c. Tests made on representative samples indicated that the longitudinal members had an average ultimate tensile strength of 84,600 psi.

The mats were manufactured shortly before use, so were practically free from rust at the time of their arrival on the project. Between that time and their installation, however, most of the mats did acquire a film of rust, but of only superficial thickness. Whether or not this practically unrusted condition had any material effect on decreasing the bond is highly problematical; but on the basis of cores recently obtained at a number of the cracks, it appears doubtful.

The pavement immediately adjacent to the ends of both sections consists of 10-in. uniform-thickness reinforced concrete, and the terminal joints at the ends of both sections are ordinary dowelled expansion joints. All of these terminal joints are basically the same, except that 3/4-in. wood filler (cypress) was installed at the ends of the 10-in. section, whereas 1/2-in. cork filler was installed at the ends of the 8-in. section.

Both sections were constructed on a layer of high-quality granular subbase material, the thickness of this layer being 12 in. in the case of the 10-in. section (and also in connection with the conventional concrete pavement incidental to the project) and 14 in. in the case of the 8-in. section. The underlying subgrade soil is of a type that, under the truck traffic at this location, is highly susceptible to pumping.

Figure 1 shows typical cross-sections and incidental construction details.

CONCRETE DATA

Cement: Lehigh, air-entraining.

Fine aggregate: River glacial sand, and quartz sand.

Coarse aggregate: Diabase trap rock, graded 2¼ in. down.

Proportions (by wt): 1:1.7 5:3.50.

Average slump: 4 in. (high slump specified to facilitate embedment of reinforcing steel.)

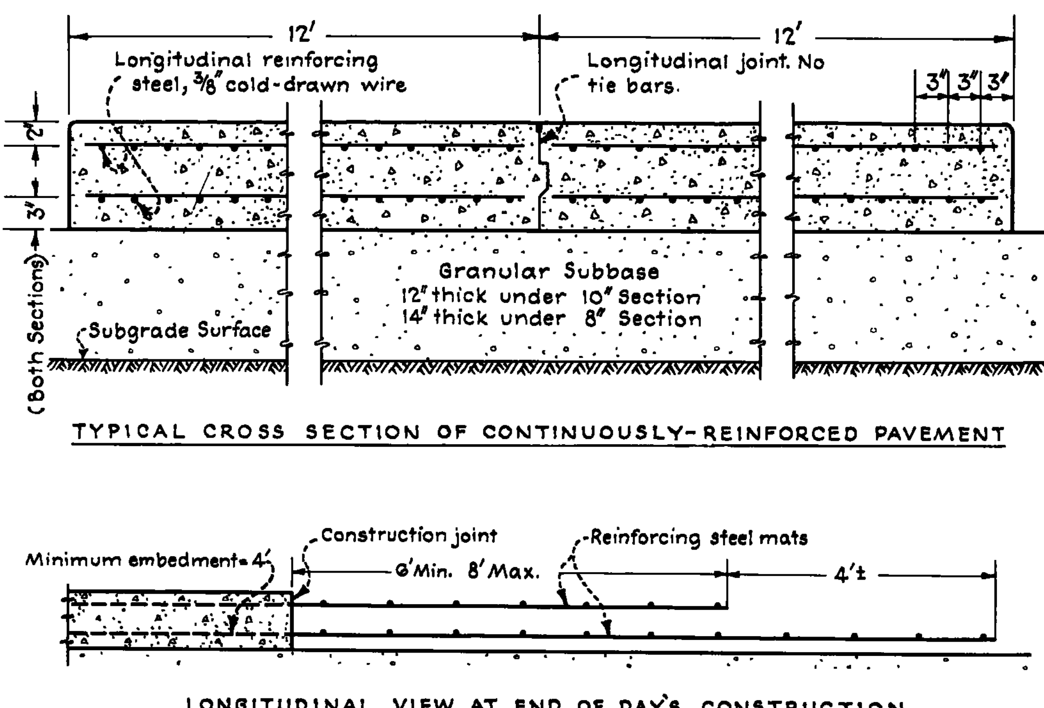


Figure 1.

Curing: "Colorless" membrane spray.

Based on 1957 cores, the concrete has an average compressive strength of about 7,000 psi.

TEMPERATURE CHANGES

Long-term observations indicate that the concrete pavements on this project have undergone an annual change in temperature ranging from a minimum of about 15 F to a maximum of about 110 F, as recorded, by means of temperature wells, midway between the top and bottom.

TRAFFIC

Because of the important influence of traffic on the performance of any pavement, it is necessary to point out that Route 130 is the major north-south trucking route in New Jersey and that, as a result, these sections have carried a considerable amount of heavy truck traffic. The type of heavy-trucking unit which predominates in this area is the tractor-semitrailer combination, which in New Jersey may legally have a single-axle load of 22,400 lb and a tandem-axle load of 32,000 lb, plus a 5 percent tolerance above these limits. From 1947 through 1957 approximately 1,600 vehicles of this type traveled over these sections daily.

Of importance is the fact that a recent visual count showed that about 95 percent of the tractor-semitrailers travel in the outside lanes, which appears to be more or less typical of the divided highways in New Jersey. On this basis, the outside lanes of both sections have thus far carried approximately 5,000,000 vehicles of this type. A breakdown of the average daily axle loads induced by these vehicles is given in Table 1. Figure 2 is a general view of the 8-in. section, the heavier travel in the outside lane being apparent from the greater amount of oil and grease stain on that lane.

BASIC FEATURES

The basic features of a continuously-reinforced concrete pavement are as follows:

1. Installation of a substantially greater amount of longitudinal reinforcing steel than installed in pavements of conventional design.
2. Continuation of the reinforcing steel through the construction joints between the portions of pavement constructed from day to day.
3. Complete omission of transverse joints of any kind, other than the construction joints previously mentioned.

Initially, the function of the reinforcing steel is to induce the occurrence of transverse cracks at relatively close intervals. Subsequently, its function is to prevent these cracks from opening to any detrimental extent, and the success or failure of the pavement depends primarily on the ability of the steel to do so.

TABLE 1

AVERAGE DAILY AXLE LOADS
(Tractor-Semitrailers Only)

Load (lb)	Number of Single Axles 1950	Single Axles 1956	Tandem Axles 1950	Tandem Axles 1956
Total, all weights	4,660	3,717	560	1,053
16,000 and over	1,980	1,270	524	926
20,000 and over	1,020	648	490	871
22,000 and over	515	458	438	791
24,000 and over	276	229	393	749
28,000 and over	50	38	257	551
32,000 and over	8	0	92	320
36,000 and over	0	0	6	65
40,000 and over	0	0	0	8
44,000 and over	0	0	0	0

LONGITUDINAL MOVEMENTS, 10-INCH SECTION

During construction, to determine periodically the magnitude of the longitudinal changes in position at various locations, monumented transverse reference lines were installed at (a) the ends of the sections, (b) points 200, 500, 700, 900, 1,400 and 2,000 ft from the ends, and (c) points midway between the ends. At the time of initial set of the concrete, brass plugs having well-defined center points were installed in the pavement, as precisely as possible on the reference lines.

Subsequently, measurement has involved setting a transit on a given reference line and measuring the offset of the pavement plug, from the reference line, with an engineer's scale. From a purely scientific standpoint this method might not be as precise as desired. But in view of the care exercised in taking these measurements, it is doubted whether the maximum error exceeds 0.04 in. A few of the reference-line monuments have been destroyed by roadside developments, but most of the important monuments are still intact.

The changes in longitudinal position recorded at various points in the 10-in. section are shown in Table 2, which also shows the changes in width recorded at the terminal joints,

In view of the erratic nature of certain of these changes, it is necessary to point out that:

1. Owing to the variations in weather conditions from day to day, certain portions of this section were necessarily constructed at somewhat different temperatures than other portions. When the pavement eventually attained a more or less uniform temperature throughout, this variation in "as-constructed" temperature probably resulted in the development of a differential internal stress condition, which in turn possibly resulted in some longitudinal rearrangement of the various portions of the section. This may account for the small, erratic changes in position recorded at the points within the central region.
2. As will be noted, on January 29, 1948, due to inward movements at the ends

Figure 2. Eight-inch section looking north.

ranging from 0.64 in. to 1.00 in., the terminal joints had undergone an opening ranging from 0.67 in. to 1.18 in. This large amount of opening resulted in complete failure of the joint sealer and, in turn, infiltration of considerable quantities of sand and gravel into the terminal joints. With yearly repetitions of this cycle there has been a further infiltration of solid material. There has also been a progressive loss in the thickness of the joint filler, which now appears to have been compressed to the point of refusal. For these reasons it appears that at present there is far greater restraint to the expansion of this section than existed originally.

3. In May 1949 a localized area in the outside lane, approximately 1,430 ft from the south end, began to undergo failure. In December 1951 the failure had progressed to the point where it became necessary to remove the damaged concrete. This work, which also involved removal of the reinforcing steel, resulted in (a) creation of an open space about 7 ft wide for the full width of the lane, and (b) creation of two free ends. As a temporary measure the space was filled with crushed stone topped with bituminous material. In October 1954 these materials were removed and replaced with concrete, which was securely joined with the existing concrete by means of the reinforcing steel. In conjunction with this work a dowelled expansion joint with 3/4-in. bituminized-fiber filler was installed at the center of the repaired area.

In May 1949, in view of the probability of complete failure at this point, a supplementary reference line was installed 20 ft to the north (shown in Table 2 as 1,450 ft from the south end).

It is apparent that, owing to this occurrence and the work performed the outside lane has been divided into two sections 1,430 and 3,700 ft long. The inside lane, however, is still intact. In any event, this constitutes the reason for the considerable difference in movement recorded since 1949 for the inside and outside lanes in the vicinity of this area.

The maximum inward and outward movements, and the total range of movement, recorded at the ends of this section are as follows:

Lane	Movement of 10-in. Section (in.)					
	South End			North End		
	Max. Inward	Max. Outward	Total Range	Max. Inward	Max. Outward	Total Range
Inside	0.68	0.29	0.97	0.72	0.23	0.95
Outside	0.96	0	0.96	1.00	0.03	1.03

The maximum recorded over-all changes in length of the inside and outside lanes of this section have been, respectively, 1.92 and 1.99 in.

Certain data worthy of note in Table 2 are as follows:

1. All of the terminal joints have undergone an appreciable increase in width, but the amount varies from joint to joint. The greatest increase has occurred at the joint at the north end of the outside lane, which on June 20, 1957, was 1.05 in. wider than at the time of construction, despite a 92 F pavement temperature. There appears to be little doubt that these increases have been due primarily to the infiltration of solid materials into these joints while in an open condition during cold weather.

2. On November 29, 1950, the north end of the outside lane was 0.62 in. south of its as-constructed position. This was also the case on June 20, 1957, despite a 53 F higher pavement temperature. It consequently appears that in this location the outside lane is now being prevented from expanding normally, owing to the impacted condition of the end joint, plus refusal of the adjacent pavement to be displaced.

3. In certain locations the longitudinal movements of the inside lane have differed appreciably from those of the outside lane, especially at (a) the ends of the section and for several hundred feet adjacent thereto, and at (b) the failure in the outside lane and for several hundred feet on each side of this failure. Inasmuch as these differential movements have necessarily involved slippage between the lanes, along the longitudinal joint,

TABL

10" SE

LONGITUDINAL MOVEMENTS AT TRANSVERSE REFERENCE

Date	Slab Temp	Lane	End Joint	← South						Distance
				0	200	500	700	900	1400	1450
As Constructed	Var	Both	0	0	0	0	0	0	0	—
Nov. 13, 1947	46°	Inside	+ .28	.30 →	.08 →	.01 →	—	—	—	—
		Outside	+ .52	.42 →	.10 →	.01 →	—	—	—	—
Jan 29, 1948	35°	Inside	+ .67	.64 →	—	.12 →	—	— .	—	—
		Outside	+ .98	.89 →	—	.10 →	—	—	—	—
July 20, "	91°	Inside	+ .22	← .20	← .05	.07 →	.03 →	← .04	.03 →	—
		Outside	+ .09	.06 →	.10 →	.07 →	—	.03 →	.07 →	—
Dec. 27, "	28°	Inside	+ .70	—	—	—	—	—	—	—
		Outside	+ 1.00	—	—	—	—	—	—	—
June 24, 1949	99°	Inside	+ .01	← .17	—	—	—	—	—	—
		Outside	+ 38	.22 →	—	—	—	—	—	—
Jan. 23, 1950	41°	Inside	+ .69	.41 →	.17 →	.08 →	—	.01 →	0	—
		Outside	+ .96	.70 →	.34 —	.09 →	—	01 →	03 →	—
Oct 6, "	69°	Inside	+ .47	.09 →	0	0	—	← .02	.02 →	—
		Outside	+ .77	43 →	.21 →	.02 →	—	.06 →	06 →	—
Nov. 29, "	39°	Inside	+ .86	43 →	.10 →	—	.06 →	—	—	—
		Outside	+ 1.07	.73 →	.28 →	—	—	—	—	—
July 12, 1951	88°	Inside	+ .26	← .19	← .04	.08 →	.08 →	← .02	.02 →	—
		Outside	+ 71	.32 →	27 →	.12 →	—	.09 →	.18 →	← .1
Feb. 1, 1952	34°	Inside	+ 1.01	.68 →	—	—	—	—	.08 →	—
		Outside	+ 1.22	.96 →	—	—	—	—	← .12	.80 →
June 10, "	90°	Inside	+ .36	← .12	← .04	.08 →	.07 →	0	.04 →	—
		Outside	+ .76	.37 →	.24 →	.12 →	—	.13 →	.63 →	← .8
June 5, 1953	84°	Inside	+ .40	—	—	—	—	—	—	—
		Outside	+ .78	—	—	—	—	—	—	—
Oct. 24, 1955	71°	Inside	+ .70	← .06	—	—	—	—	—	—
		Outside	+ .90	.42 →	—	—	—	—	—	—
Jan. 31, 1956	40°	Inside	+ 1.27	—	—	—	—	—	—	—
		Outside	+ 1.35	—	—	—	—	—	—	—
Oct. 5, "	70°	Inside	+ .77	← .04	← .04	.12 →	.06 →	0	← .02	—
		Outside	+ 1 01	.36 →	.22 →	.16 →	—	36 →	.89 →	← 1.0
Feb 25, 1957	50°	Inside	+ 1.12	.34 →	—	06 →	.02 →	0	← .04	—
		Outside	+ 1.26	.71 →	—	.20 →	—	.29 →	.29 →	← .4
June 20, "	92°	Inside	+ .53	← .29	← .17	0	.04 →	← .02	.04 →	—
		Outside	+ .90	.34 →	.22 →	.18 →	—	35 →	1.10 →	← 1.0

[1] See text.
[2] Actual width of joint space at end joints is obtained by adding 0.50 to changes shown.
[3] Reference line destroyed.

it is speculative whether the installation of tie bars would have proved beneficial, detrimental, or of no apparent consequence.

The movements at the points 1,400 and 1,450 ft from the south end in the outside lane are of particular interest, because these points are immediately south and north, respectively, of the failure which occurred in this lane. As will be noted, on June 20, 1957, these points were 2.19 in. closer together than they were originally. It will also be noted that for a distance of at least 550 ft both north and south of the point of failure the outside lane has undergone an appreciable movement toward the point of failure. Presumably this indicates two things—that this lane has tended to undergo an increase in length, and that there have been times when this lane has been under considerable compression.

LONGITUDINAL MOVEMENTS, 8-INCH SECTION

The changes in longitudinal position recorded at various points in the 8-in. section are shown in Table 3, which also shows the changes in width recorded at the terminal joints.

2

ION

INES, AND CHANGES IN WIDTH OF END JOINTS (in.)

from End (feet)							North →		End
2000	2565	2000	1400	900	700	500	200	0	Joint
0	0	0	0	0	0	0	0	0	0
—	—	—	—	—	.08 →	.05 →	← .10	← .29	+.30
—	—	—	—	—	.06 →	.09 →	← .10	← .43	+.52
—	—	—	—	—	—	—	← .40	← .72	+.88
—	—	—	—	—	—	—	← .47	← 1.00	+1.18
← .01	0	.03 →	0	.04 →	.10 →	*	0	.23 →	- .18
0	.02 →	.03 →	.04 →	.03 →	.09 →	*	← .05	.03 →	+.02
—	—	—	—	—	—	—	—	—	+.83
—	—	—	—	—	—	—	—	—	+1.08
—	—	—	—	—	—	—	—	—	-.02
—	—	—	—	—	—	—	—	—	-.30
0	← .05	*	*	.09 →	.15 →	—	← .12	← .30	+ .65
.03 →	0	*	*	.08 →	.13 →	—	← .22	← .58	+1.01
← .03	← .02	—	—	.10 →	.08 →	—	← .01	.08 →	+ .28
0	0	—	—	.10 →	.05 →	—	← .12	← .21	+ .68
—	—	—	—	—	—	—	← .08	← .33	+ .75
—	—	—	—	—	—	—	← .18	← .62	+1.16
.02 →	0	—	—	.10 →	*	—	.08 →	.20 →	+ .16
.05 →	.04 →	—	—	.10 →	*	—	← .09	← .14	+ .63
—	—	—	—	—	—	—	—	← .54	+ .90
—	—	—	—	—	—	—	—	← .85	+1.34
← .02	← .03	—	—	.05 →	—	—	← .02	10 →	+ .29
0	0	—	—	.06 →	—	—	← .18	← .28	+ .91
—	—	—	—	—	—	—	—	—	+ .37
—	—	—	—	—	—	—	—	—	+ .86
—	—	—	—	—	—	—	—	← .16	+ .45
—	—	—	—	—	—	—	—	← 54	+1.01
—	—	—	—	—	—	—	—	—	+ .07
—	—	—	—	—	—	—	—	—	+1.51
0	← .02	—	—	13 →	—	—	← .12	← .11	+ .37
← .25	← .02	—	—	.14 →	—	—	← .35	← .59	+1.05
—	—	—	—	.10 →	—	—	← .26	← .39	+ .73
—	—	—	—	.12 →	—	—	← .46	← .84	+1.32
0	← .10	—	—	.16 →	—	—	← .05	← .05	+ .30
← .23	← .10	—	—	.18 →	—	—	← .31	← .62	+1.05

As in the case of the 10-in. section, in considering the changes shown in Table 3 it is necessary to bear in mind (a) the variations in as-constructed temperature from day to day, and (b) the infiltration of large amounts of sand and gravel into the terminal joints. In addition, there have been two serious failures in the outside lane. The first of these failures, 569 ft from the north end, became evident in the spring of 1951; the second, 372 ft from the north end, in the spring of 1952.

In October 1954 the damaged concrete in these areas was removed and replaced with new concrete. In conjunction with this work a dowelled expansion joint with 3/4-in. bituminized-fiber filler was installed at the center of each of these areas. Consequently, since October 1954 the outside lane has consisted of three independent sections (from south to north, 4,861, 197, and 372 ft long). The inside lane, however, is still intact.

The maximum inward and outward movements, and the total range of movement, recorded at the ends of this section are as follows:

TABLE 3

8" SECTION

LONGITUDINAL MOVEMENTS AT TRANSVERSE REFERENCE LINES, AND CHANGES IN WIDTH OF END JOINTS (in.)

Date	Slab Temp	Lane	End Joint	← South 0	200	500	700	900	1400	2000	2715	2000	1400	900	700	500	200	North → 0	End Joint
				Distance from End (feet)															
As constructed	Var	Both	0	0	0	0	0	0	0		0	0	0	0	0	0	0	0	0
Nov 13, 1947	46°	Inside	+ 33	22 →	02 →	—	—	—	—	—	—	—	—	—	—	—	← 02	← 24	+ 28
		Outside	+ 30	23 →	0	—	—	—	—	—	—	—	—	—	—	—	0	← 25	+ .22
Feb 1, 1948	–	Inside	+ 97	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ .86
		Outside	+ 94	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ .87
July 22, "	83°	Inside	– 06	← 20	← 09	07 →	03 →	04 →	04 →	.03 →	0	03 →	04 →	.02 →	0	0	11 →	20 →	– 10
		Outside	– 01	← .13	← 06	03 →	.03 →	05 →	0	← 03	← 02	03 →	02 →	← 01	02 →	0	08 →	21 →	– 06
Dec 27, "	28°	Inside	+ .88	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ .80
		Outside	+ 93	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ .89
June 24, 1949	99°	Inside	– 04	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	– 20
		Outside	+ 09	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ 02
Jan 26, 1950	53°	Inside	+ 52	17 →	.06 →	.02 →	← .01	.10 →	01 →	01 →	02 →	.04 →	.04 →	04 →	0	05 →	← 10	← .23	+ .47
		Outside	+ 68	27 →	07 →	04 →	0	.11 →	02 →	0	← .02	04 →	.03 →	.01 →	02 →	07 →	← .05	← .26	+ .67
Oct 10, "	73°	Inside	+ 47	.04 →	02 →	*	0	04 →	07 →	04 →	0	07 →	10 →	.12 →	← .02	.08 →	17 →	.17 →	+ 07
		Outside	+ .62	12 →	0	*	0	04 →	02 →	← 03	0	07 →	10 →	10 →	01 →	.07 →	20 →	15 →	+ 46
Nov 29, "	40°	Inside	+ .88	.32 →	04 →	—	03 →	—	—	—	—	—	—	—	← 04	—	06 →	← 19	+ .51
		Outside	+1.00	37 →	07 →	—	04 →	—	—	—	—	—	—	—	0	—	04 →	← .17	+ .87
July 31, 1951	88°	Inside	+ .47	← .13	← 19	—	.08 →	08 →	07 →	10 →	03 →	03 →	0	02 →	0	10 →	25 →	.35 →	– 15
		Outside	+ .67	← 02	← 12	—	09 →	10 →	02 →	0	01 →	02 →	0	← 01	03 →	.06 →	22 →	32 →	+ 35
Feb. 20, 1952	44°	Inside	+1.12	.45 →	.16 →	—	05 →	05 →	08 →	.10 →	03 →	03 →	.03 →	.03 →	← .05	02 →	← 03	← 28	+ 50
		Outside	+1.24	51 →	.16 →	—	08 →	.06 →	03 →	.02 →	.01 →	03 →	.05 →	.01 →	0	.07 →	← .01	← 22	+ .90
June 3, "	88°	Inside	+ .68	← .13	← 12	—	← 02	.03 →	06 →	04 →	← .02	03 →	0	04 →	← 01	.05 →	23 →	30 →	+ 05
		Outside	+1.01	.04 →	← 02	—	0	05 →	02 →	← 05	← .03	03 →	.02 →	01 →	05 →	10 →	22 →	26 →	+ .55
June 5, 1953	84°	Inside	+ 75	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ .28
		Outside	+1.12	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+ 65
Oct 24, 1955	71°	Inside	+1 16	← .06	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Outside	+1.60	.08 →	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Jan 31, 1956	40°	Inside	+1 60	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+1.22
		Outside	+1 98	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	+1 28
Oct. 10, "	61°	Inside	+1 18	← .06	← 14	—	← .01	.10 →	10 →	05 →	02 →	02 →	02 →	0	← 02	.10 →	17 →	.16 →	+ .97
		Outside	+1 76	.12 →	← 04	—	0	.10 →	.04 →	01 →	← 01	01 →	04 →	06 →	18 →	.10 →	11 →	.13 →	+1.13
Feb 25, 1957	52°	Inside	+1 40	22 →	—	—	02 →	10 →	—	—	—	—	—	10 →	← 04	.02 →	—	← 07	+1 20
		Outside	+1 95	34 →	—	—	04 →	.10 →	—	—	—	—	—	13 →	.06 →	.10 →	—	04 →	+1 24
June 20, "	94°	Inside	+ 68	← 49	← 33	—	← 01	08 →	04 →	.01 →	.10 →	07 →	01 →	07 →	11 →	.12 →	32 →	44 →	+ .67
		Outside	+1 61	← 08	← .16	—	01 →	.10 →	← 02	← 01	05 →	08 →	01 →	18 →	29 →	14 →	.14 →	29 →	+.93

[1] See text.

[2] Actual width of joint space at end joints is obtained by adding 0.50 to changes shown.

[3] Reference line destroyed

Lane	Movement of 8-in. Section (in.)					
	South End			North End		
	Max. Inward	Max. Outward	Total Range	Max. Inward	Max. Outward	Total Range
Inside	0.45	0.49	0.94	0.28	0.44	0.72
Outside	0.51	0.13	0.64	0.26	0.32	0.58

The maximum over-all recorded changes in length of the inside and outside lanes of this section have been, respectively, 1.66 and 1.22 in.

Certain data worthy of note in Table 3 are as follows:

1. All terminal joints have undergone an appreciable increase in width. As in the case of the 10-in. section, there appears to be little doubt that these increases have been due primarily to infiltration of solid materials into these joints while in an open condition during cold weather.
2. The greatest increase in width has occurred at the joint at the south end of the outside lane, which on June 20, 1957, was 1.61 in. wider than at the time of construction, despite a 94F pavement temperature. Conditions at this joint on August 7, 1957, are shown in Figure 3, a portion of the joint space having been cleaned out. As will be noted, the joint space was found to be completely filled with infiltrated sand and gravel, in combination with the cork filler, which had become compressed to the point of refusal. These infiltrated materials were in a highly compacted condition.
3. Corresponding with the behavior of the 10-in. section, the longitudinal movements

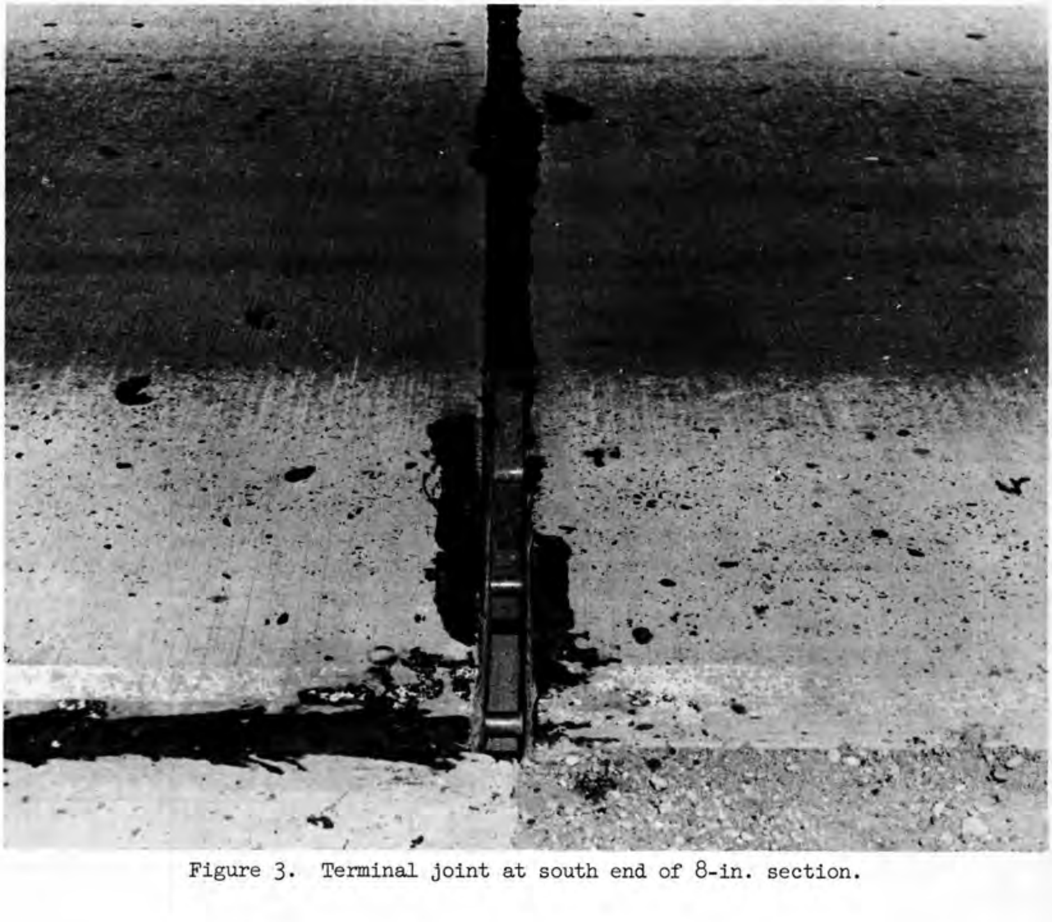
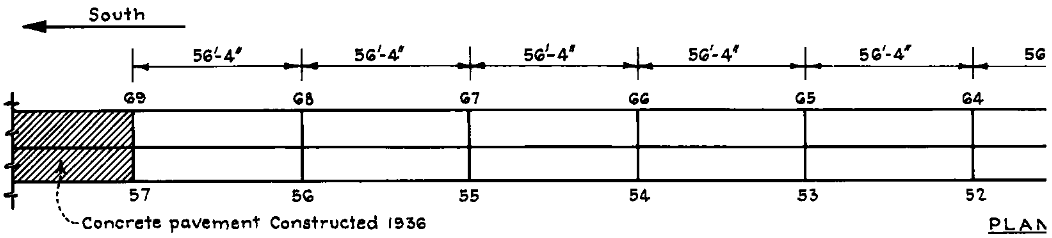


Figure 3. Terminal joint at south end of 8-in. section.



CHANGES IN JOINT

Date	Slab Temp	Inside Lane							
		69	68	67	66	65	64	63	12
		No filler	3/4" Wood	3/4" Wood	3/4" Wood	3/4" Wood	3/4" Wood	3/4" Wood	3/4" Wood
As Constructed	Var	0	0	0	0	0	0	0	0
Jan 24, 1948	35°	+ 11	+ 11	+.12	+.12	+.10	+.10	+.11	+.16
May 10, "	78°	+ 09	-.08	-.02	-.03	- 02	- 02	-.04	+.01
July 15, "	92°	+.04	-.26	-.08	- 08	- 07	-.07	- 09	- 11
Aug 27, "	100°	+.09	-.50	-.10	-.10	-.08	-.09	-.10	-.15
Dec 27, "	28°	-	-	-	-	-	-	-	-
Oct 10, 1950	78°	+.30	- 32	- 04	-.04	-.05	- 05	-.08	- 27
Feb 5, 1951	35°	-	-	-	-	-	+ 06	+.03	-.14
Oct 29, "	65°	+.38	-.31	-.01	-.01	+.01	-.01	-.06	-.24
June 10, 1952	93°	+.38	-.42	-.10	-.10	- 07	-.11	-.16	- 37
June 5, 1953	84°	+.42	-.44	-.11	-.11	-.08	-.14	-.20	- 38
Jan 31, 1956	39°	+.56	-.35	0	+.01	+.04	-.05	-.15	- 25
Dec 28, "	38°	+.60	-.39	-.01	-.02	+.02	- 07	-.17	- 27
June 21, 1957	94°	+ 50	-.51	- 16	-.20	-.14	-.21	-.30	-.42
June 21, 1957 compared with July 15, 1948		+.46	-.25	-.08	-.12	- 07	-.14	-.21	-.31

Figure 4. Pavement adjacent to south end of

of the inside lane in certain locations have differed appreciably from those of the outside lane.

EFFECT ON ADJACENT PAVEMENT

Figures 4 to 7 show the pavement adjacent to the ends of the sections, and the engineer's stations of the ends. They also show the recorded changes in width of (a) the joints in this pavement, and (b) the terminal joints.

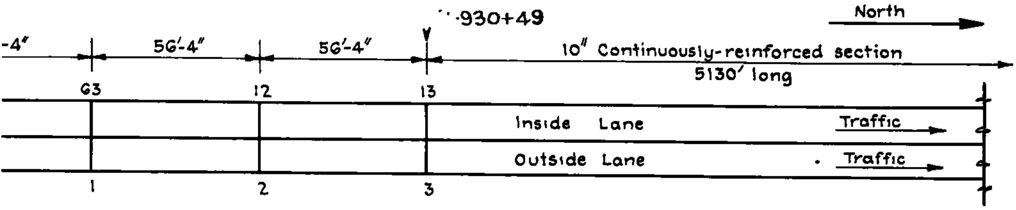
It will be seen that in conjunction with the increase in width of the terminal joints there has been, with certain exceptions, a decrease in the width of the expansion joints in the adjacent pavement. Analysis of the reference-line and joint-width measurements has shown that these sections have exerted pressure on the adjacent pavement, and that this pressure has caused the adjacent pavement to be displaced in various amounts.

During the summer of 1957 the displacements that had taken place in connection with the slabs immediately adjacent to the ends of these sections were approximately as follows:

	Outward Displacement of Adjacent Slabs (in.)			
	10-Inch Section		8-Inch Section	
Lane	South End	North End	South End	North End
Inside	0.82	0.25	1.17	1.11
Outside	0.56	0.43	1.69	1.22

It will be noted that:

1. The displacements have been extremely variable, the range being from 0.25 to 1.69 in.



WIDTH (Inches)

	Outside Lane									
13	57	56	55	54	53	52	1	2	3	← Joint Number
3/4″ Wood	No filler	3/4″ Wood	3/4″ Wood	1″ Wood	3/4″ Wood	3/4″ Wood	3/4″ Wood	3/4″ Wood	3/4″ Wood	← Joint Filler
0	0	0	0	0	0	0	0	0	0	
+.67	+ 17	+.08	+.13	+.11	+ 10	+.11	+.19	+.22	+ 98	
+.03	+.07	-.06	-.02	- 02	-.01	-.03	+.06	+ 07	+.41	
- 22	+ 06	-.22	-.10	-.09	-.06	-.10	+ 01	-.02	+.09	
- 26	+ 06	-.26	-.13	-.11	- 08	-.12	0	-.05	+.06	
+.70	-	-	-	-	-	+.14	+.22	+.23	+1.00	
+.33	+.19	-.22	-.01	-.08	0	-.09	+.04	-.10	+.67	
+ 87	-	-	-	-	-	+.02	+.15	+.01	+1.16	
+ 07	+.18	-.20	-.06	-.05	+.04	-.06	+.07	-.07	+.84	
+.36	+.13	-.30	-.14	-.14	+.04	- 19	-.03	-.19	+.76	
+.40	+.14	-.31	-.14	- 13	-.04	-.21	-.04	-.21	+.78	
+1.27	+.16	-.18	0	-.01	+.08	-.05	+.07	-.13	+1.35	
+1.15	+ 18	-.21	-.02	-.02	+.07	-.07	+.06	-.15	+1.26	
+.52	+.09	- 35	-.16	-.17	-.07	-.25	-.09	-.28	+.89	
+.74	+.03	-.13	-.06	- 08	-.01	-.15	-.10	-.26	+.80	

10-in. continuously-reinforced section.

2. The least displacement has occurred at the north end of the inside lane of the 10-in. section.

3. The displacements in connection with the 8-in. section have been considerably greater than in connection with the 10-in. section.

This erratic behavior has apparently been due to the influence of several factors, as follows:

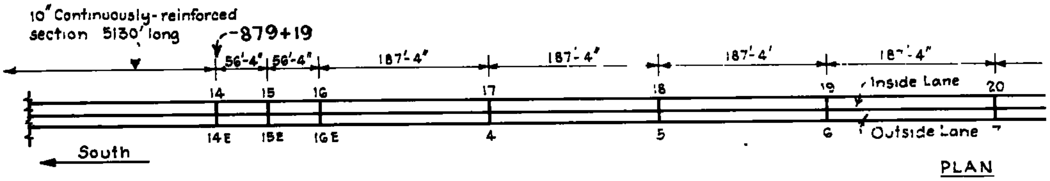
1. Differences in the as-constructed temperatures of the lanes.

2. Differences in the amount of solid material which has accumulated in the terminal joints.

3. Wood joint filler was used in connection with the pavement adjacent to the 10-in. section, whereas cork filler was used in the pavement adjacent to the 8-in. section. Of these two materials, wood offers far more resistance to compression, therefore the pavement adjacent to the 10-in. section has offered considerably more resistance to displacement.

With reference to the relatively small displacement of the slabs immediately adjacent to the north end of the 10-in. section, it is to be noted that the adjacent pavement consists of two 56-ft slabs, which are in turn followed by eight 187-ft slabs, in each lane. And from the tabulation in Figure 5 it will be noted that the joints between the 187-ft slabs have undergone an increase in width and that, conversely, the joints between the 56-ft slabs, both north and south of the 187-ft slabs, have undergone a decrease in width.

From this it is apparent that this series of 187-ft slabs has tended to undergo an increase in length; which, in fact, has actually occurred. At the north, this action has resulted in movement of the 56-ft slabs and the approach slabs, toward the bridge over



CHANGES IN JOINT

Date	Slab Temp	Inside Lane													
		14	15	16	17	18	19	20	21	22	23	24	54W	53W	52W
		3/4"W	3/4 W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	1/2" C	1"C	1"C	1"C
As Constructed	Var	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Jan 26, 1948	24°	+.89	+.15	+.32	+.42	- 52	+.45	+.49	+ 57	+.47	+ 56	+.13	+ 07	+ 10	+ 12
Aug 27, "	100°	- 20	- 20	-.26	- 20	- 16	-.20	- 12	-.12	-.06	-.14	- 32	- 21	- 19	-.17
Dec 27, "	34°	+.79	+.06	+ 26	+ 41	+.53	+ 40	+.50	+.53	+ 54	+ 51	+ 12	-.03	+ 07	+.10
June 24, 1949	99°	-.02	- 35	-.29	-.14	-.11	-.14	-.08	- 01	-.05	- 13	- 32	-.32	-.19	-.17
Jan 20, 1950	38°	+.69	-.18	+ 13	+ 33	+ 39	+.29	+.38	+.42	+.44	+ 37	+ 06	- 10	+.01	+ 06
Oct. 10, "	78°	+ 23	-.35	-.12	-.01	+.03	-.02	+ 05	+ 11	+.10	+.02	-.17	- 23	-.11	- 07
Feb 5, 1951	38°	+.84	- 23	+.14	+.32	+ 40	+.30	+ 39	+ 42	+.44	+ 37	+.05	-.11	0	+.04
Oct 29 "	65°	+ 36	-.32	- 02	+.14	+.15	+.10	+.18	+.25	+.23	+ 16	-.04	-.21	-.07	-.03
July 15, 1952	95°	+ 26	- 42	- 13	- 01	- 07	-.08	+ 01	+.12	+ 05	- 01	- 18	- 47	-.22	-.20
June 6, 1953	84°	+ 37	-.39	- 05	+.05	0	- 02	+.06	+ 17	+ 12	+.09	-.07	-.44	- 21	- 16
Jan 31, 1956	39°	+ 97	- 25	+ 21	+ 38	+.41	+ 31	+ 42	+ 49	+.46	+ 52	+.36	-.31	-.22	-.04
Dec. 28, 1956	38°	+.80	-.27	+ 21	+.38	+.37	+.29	+.38	+ 46	+.47	+.51	+ 38	-.33	-.26	-.09
June 12, 1957	89°	+ 35	- 38	+.03	+ 13	+.06	+.04	+ 10	+ 20	+ 22	+.20	+ 13	-.47	- 40	-.24
June 19, "	104°	+.29	- 40	+ 01	+ 11	+.02	+ 02	+.07	+.17	+.19	+.16	+ 11	- 47	- 43	-.28
June 19, 1957 compared with Aug 27, 1948		+.49	- 20	+ 27	+ 31	+.18	+.22	+.19	+.29	+ 25	+.30	+ 43	- 16	-.24	-.11

Figure 5. Pavement adjacent to north end of

Rocky Brook, and at the south it has resulted in development of pressure against the 10-in. section.

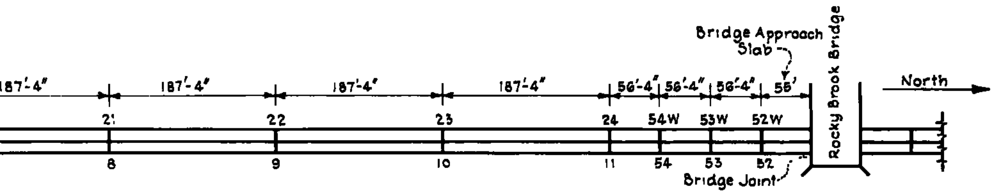
As previously indicated, the largest amount of displacement (1.69 in.) has occurred in connection with the slab adjacent to the south end of the outside lane of the 8-in. section. As shown in Figure 6, this displacement has resulted in considerable closure of Joints 26, 27, and 28, immediately to the south. It has also resulted in about 1 in. of closure of the joint adjacent to the bridge over Rocky Brook. It will also be noted that a similar action has taken place in the inside lane, but to a lesser degree. It therefore is apparent that all of the pavement adjacent to the south end of the 8-in. section has been moved an appreciable amount toward this bridge. And from the data shown in Figure 7 it is apparent that a similar displacement has taken place in connection with the pavement adjacent to the north end.

CONDITION OF PAVEMENT AT ENDS

As measured from the ends, the end portions of the sections are still (1957) uncracked for distances ranging from 64 to 177 ft. The uncracked distances for all lanes are as follows:

	Uncracked Distance (ft)			
	10-Inch Section		8-Inch Section	
Lane	South End	North End	South End	North End
Inside	161	167	152	109
Outside	88[1]	177	64	114

[1] Through core hole.



WIDTH (Inches)

	Outside Lane														
Br Jt.	14E	15E	16E	4	5	6	7	8	9	10	11	54	53	52	Br Jt
2"C	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	3/4"W	1/2"C	1"C	1"C	1"C	2"C
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
+ 09	+1.20	+ 12	+.27	+.61	+ 81	+ 59	+.61	+ 66	+ 62	+.80	+ 25	+.14	+.19	+.20	+.09
-.10	- 01	-.18	- 25	-.13	0	-.09	-.12	-.15	- 13	-.01	-.19	-.25	-.14	- 03	-.10
+ 10	+1 03	+.07	+ 24	+.56	+ 76	+.60	+.55	+.57	+.57	+.73	+.27	+.02	+.14	+.23	+.11
-.10	+ 30	-.39	-.24	-.09	+.06	+.06	-.04	+.01	-.19	-.03	-.04	- 37	-.14	-.03	-.10
+.08	+1 05	- 23	+.14	+.45	+ 59	+ 54	+.44	+.49	+.38	+.56	+.32	-.16	+.07	+ 20	+.08
-.02	+ 64	- 39	-.09	+.12	+.24	+ 29	+ 18	+.21	+ 02	+.18	+.22	-.40	- 08	+.07	- 03
+.09	+1.27	- 28	+.15	+.44	+.61	+.57	+.48	+53	+ 38	+.57	+.42	- 29	+.02	+.17	+.09
+.02	+.80	-.36	+.02	+.24	+ 39	+.39	+.31	+.33	+.18	+.36	+ 41	-.41	-.14	+.06	+.01
-.16	+77	-.44	-.05	+.15	+ 31	+ 32	+ 22	+ 24	+ 07	+.29	+.39	-.53	-.35	-.27	- 22
-.16	+.86	-.42	+ 01	+.22	+ 37	+.37	+.28	+.29	+.12	+.40	+.53	-.48	-.32	-.28	- 26
-.05	+1 51	-.32	+.26	+.53	+ 69	+.65	+.60	+.59	+.51	+.93	+1.08	-.29	-.20	-.22	- 62
-.09	+1 38	-.34	+.26	+.53	+.67	+.65	+.58	+ 57	+.50	+.93	+1.19	-.27	-.20	-.20	- 84
-.30	+1.07	-.44	+.12	+ 31	+ 48	+.41	+ 34	+.34	+.23	+.65	+.98	- 38	-.34	-.34	-113
-.37	+1 04	-.45	+ 10	+.29	+.47	+.39	+.32	+.32	+.21	+.62	+.96	-.40	-.35	- 36	-119
-.27	+1.05	- 27	+.35	+.42	+ 47	+.48	+.44	+.47	+ 34	+.63	+1 15	-.15	-.21	-.33	-1.09

← Joint Number

← Joint Filler

Note

W denotes wood filler

C denotes cork filler

10-in. continuously-reinforced section.

CRACKS

The central regions of both sections now contain a large number of transverse cracks. Many of these cracks occurred immediately after construction—within a matter of days—and practically all of the remainder occurred within the next three years.

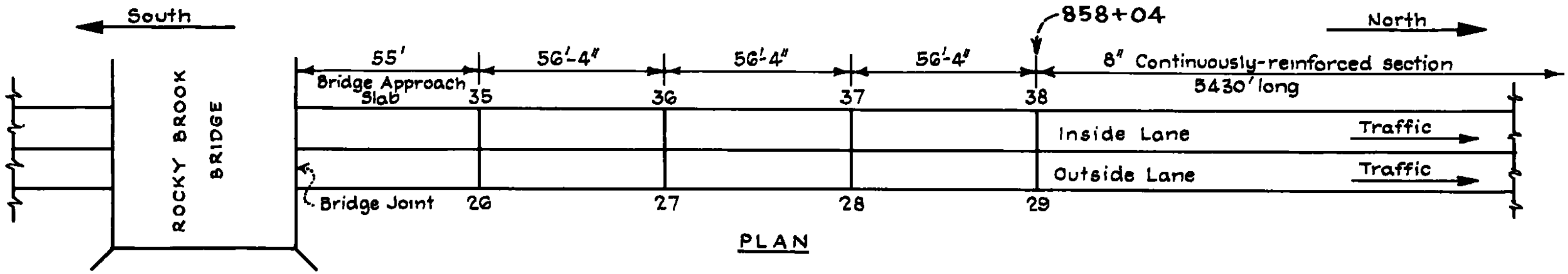
The cracks are of an extremely erratic nature. For example, there are the following types of cracks:

1. Cracks which extend across the full width of the lane.
2. Cracks which originate at an edge, but which become progressively narrower and terminate at some indefinite distance from the opposite edge.
3. Cracks which originate at an edge as a single crack, but which become divided into two cracks, which may or may not extend to the opposite edge.
4. Cracks which originate and terminate within the lane, without extending to either edge.

Although there are other variations in the cracking, these examples will serve to make two things apparent, as follows:

1. It is not possible to establish a true average crack interval, as would be the case if all the cracks were single cracks extending the full width of the lane.
2. It is not possible to establish a true average width of crack, considering, for example, that there are a great many cracks which fall in categories 2 and 3, previously described.

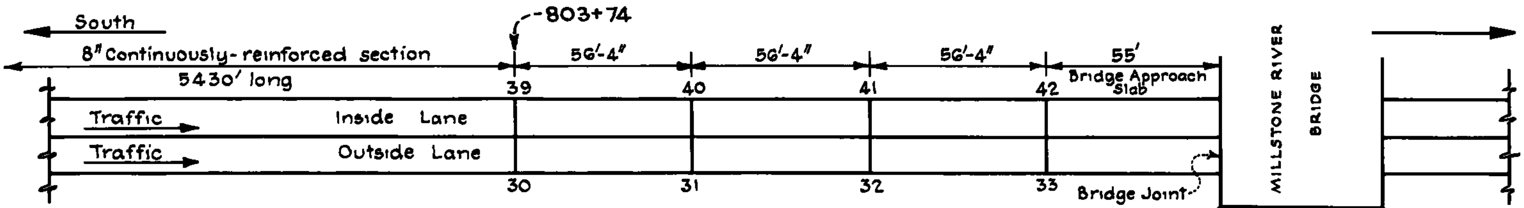
But despite these difficulties, and in order to obtain at least some idea as to the average crack interval, crack surveys were made of a full day's work in each lane of



CHANGES IN JOINT WIDTH (Inches)

Date	Slab Temp.	Inside Lane					Outside Lane				
Joint Number		Br. Jt.	35	36	37	38	Br. Jt.	26	27	28	29
Joint Filler		2" Cork	1" Cork	1" Cork	1" Cork	1/2" Cork	2" Cork	1" Cork	1" Cork	1" Cork	1/2" Cork
As Constructed	Var.	0	0	0	0	0	0	0	0	0	0
Jan 30, 1948	28°	+.09	+.21	+.19	+.16	+.78	+.08	+.22	+.17	+.12	+.89
July 15, "	93°	-.16	-.02	-.04	-.21	-.14	-.18	-.02	-.07	-.32	-.06
June 24, 1949	99°	-.19	+.03	-.07	-.35	-.04	-.22	-.03	-.10	-.52	+.09
Oct. 10, 1950	78°	-.09	+.06	-.03	-.43	+.42	-.14	+.09	-.10	-.58	+.59
Feb. 5, 1951	40°	+.02	+.17	+.08	-.29	+.96	-.02	+.19	0	-.44	+1.07
Oct. 29, "	64°	-.07	+.11	-.05	-.49	+.76	-.11	+.13	-.24	-.51	+.91
July 15, 1952	103°	-.34	-.07	-.32	-.66	+.61	-.38	-.22	-.46	-.59	+.96
June 6, 1953	88°	-.31	-.01	-.30	-.64	+.75	-.35	-.20	- 46	-.55	+1.12
Oct. 24, 1955	69°	-.32	+.10	-.35	-.50	+1.15	-.57	-.20	- 41	-.45	+1.60
Jan 31, 1956	39°	-.23	+.15	-.28	-.52	+1.60	-.47	-.13	-.36	-.37	+1.98
Dec 28, "	38°	-.30	+.16	-.31	-.53	+1.49	-.63	-.14	-.36	-.39	+2.03
June 21, 1957	102°	- 67	0	-.46	-.67	+.67	-1.09	-.30	-.50	-.53	+1.60

Figure 6. Pavement adjacent to south end of 8-in. continuously-reinforced section.



PLAN

CHANGES IN JOINT WIDTH (Inches)

Date	Slab Temp.	Inside Lane					Outside Lane				
		39	40	41	42	Br Jt	30	31	32	33	Br. Jt
		1/2" Cork	1" Cork	1" Cork	1" Cork	2" Cork	1/2" Cork	1" Cork	1" Cork	1" Cork	2" Cork
As constructed	Var.	0	0	0	0	—	0	0	0	0	—
Jan 30, 1948	28°	+.82	+.16	+.22	+.23	—	+.83	+.13	+.21	+.17	—
May 10, "	77°	+ 06	-.01	-.01	+.05	0	+.05	-.04	+.02	-.01	0
Aug. 27, "	100°	-.19	-.25	-.08	+.01	+.05	-.15	-.37	-.11	-.04	+.04
Dec 27, "	28°	+.80	+.06	+.17	+.26	+.15	+.89	-.03	+.18	+.22	+.14
Oct. 10, 1950	78°	+.01	-.18	-.04	+.07	+.02	+.41	-.39	-.25	+.04	-.01
Feb. 5, 1951	40°	+.57	-.06	+ 07	+.19	+.08	+.90	-.28	+ 13	+.16	+.08
Oct. 29, "	64°	+.18	-.14	+.02	+.12	+.02	+.61	- 37	-.26	+.09	+.02
June 12, 1952	95°	+.05	-.39	-.12	-.05	-.15	+.55	-.50	-.44	-.15	-.15
July 15, "	103°	+.02	-.45	-.17	-.09	+.20	+.50	-.51	-.48	-.24	-.21
June 6, 1953	94°	+.28	- 47	-.17	-.07	-.18	+.65	-.44	-.47	-.29	-.19
Nov 23, 1954	47°	+.91	-.42	-.08	+ 08	-.03	+1 20	-.26	-.38	-.23	-.10
Jan 31, 1956	39°	+1.22	-.41	-.15	-.01	-.06	+1.28	-.23	-.38	-.24	-.14
Dec. 28, "	38°	+1.28	-.42	-.18	-.10	-.10	+1.34	-.19	-.37	-.27	-.23
June 21, 1957	102°	+.66	-.57	- 34	-.48	- 45	+.93	- 31	-.52	-.44	-.61
June 21, 1957 compared with Aug 27, 1948		+.85	-.32	-.26	-.49	-.40	+1.08	+.06	-.41	-.40	-.57

← Joint Number
← Joint Filler

Figure 7. Pavement adjacent to north end of 8-in. continuously-reinforced section.

each section, the only cracks recorded being those which appeared to be clearly defined structural cracks adjacent to the longitudinal joint. The results of these surveys are as follows:

AVERAGE CRACK INTERVAL (ft)

Date	8-Inch Section		10-Inch Section	
	Inside Lane	Outside Lane	Inside Lane	Outside Lane
	Sta. 814+62-828+46	Sta. 839+65-849+34	Sta. 915+23-922+43	Sta. 905+18-914+10
Nov. 1947	7.7	6.6	10.7	6.2
Oct. 1950	3.9	3.5	6.2	4.0
Aug. 1957	3.7	3.5	6.2	3.9

It will be noted that:

1. In October 1950, after three years of service, there were nearly twice as many cracks as in November 1947.
2. There has been only a slight increase in the number of cracks during the past seven years.

It also will be apparent from the foregoing tabulation that a substantially greater crack interval was recorded in the inside lane of the 10-in. section than in any of the other lanes. In view of this inconsistency, and doubt as to whether the 6.2-ft recorded interval was representative of the entire lane, a survey was made in September 1957 of three day's work in this lane, among them being the day's work included in the original survey. In this case, however, owing to the hazard involved in taking measurements along the longitudinal joint, the cracks along the westerly edge were recorded. In the day's work originally surveyed, the crack interval was found to be 5.5 feet, and in the two additional day's work the interval was 4.5 and 4.2 ft.

Perhaps about the best that can be said is that in all of the lanes the interval averages from about 3½ to 4½ ft, and ranges from as little as 6 in. to as much as 20 ft.

The present typical crack pattern in the 8-in. section is shown in Figure 8, the cracks in the outside lane having been traced with yellow keel, for photographic reasons. This pattern is also typical of the 10-in. section. It will be noted that there are dark streaks across the lanes, expecially across the inside lane. These streaks, which are about 4 in. wide and dark grey in color, are coincident with the cracks. Apparently they are due to the exudation of a substance derived from the concrete during wet weather, but the nature of this substance is not known.

The variable spacing of the cracks is shown in Figure 9, the lane in the foreground being the outside lane of the 10-in. section approximately 1,800 ft from the north end.

CRACK WIDTHS

During construction, at 12 locations within the central regions brass gage plugs were installed to provide continuing means of determining the amount of crack opening. These plugs, averaging 10 in. apart, were installed while the concrete was still plastic, along lines parallel with the longitudinal axis of the pavement. As many as 60 consecutive plugs were installed in some locations. The distances between these plugs were measured to the nearest 0.001 in. The initial measurements were taken on the day of construction, just as soon as the concrete had definitely hardened, and before any apparent cracking.

Results of the measurements taken from time to time in connection with the gage plugs installed in the 8-in. and 10-in. sections are summarized, respectively, in Tables 4 and 5. With reference to these tables, it is necessary to point out that:

1. The crack widths shown are based on the increase in distance recorded between the plugs. The actual crack width, however, may be slightly greater, as the concrete

Figure 8. Typical crack pattern, 8-in. section.

Figure 9. Variable crack spacing, 10-in. section.

TABLE 4

CRACK WIDTH DATA 8" SECTION

Location and Incidental Data		July 1948	Aug. 1948	Oct. 1950	Dec 1950	July 1951	Nov-Dec 1951	June 1952	Mar. 1953	Sept 1954	Jan 1956	Mar 1957	June 1957
847+50 to 847+95 Inside Lane 49 Spaces @ 10" Const.- Oct. 16, 1947	Slab. Temp	—	—	75°	31°	102°	48°	95°	38°	—	—	43°	101°
	№ Cracks	—	—	7	7	7	7	7	7	—	—	7	7
	Av Spacing	—	—	5.8'	5.8'	5.8'	5.8'	5.8'	5.8'	—	—	5.8'	5.8'
	Av. Width	—	—	.009"	.009"	.010"	.013"	.011"	.011"	—	—	.010"	.010"
	Max Width	—	—	.011"	.011"	.011"	.015"	.013"	.014"	—	—	.012"	.012"
820+52 to 820+84 Outside Lane 38 Spaces @ 10" Const.- Oct. 9, 1947	Slab Temp	90°	—	69°	31°	87°	48°	95°	—	—	—	43°	106°
	№ Cracks	7	—	9	9	9	9	9	—	—	—	9	9
	Av. Spacing	4.5'	—	3.5'	3.5'	3.5'	3.5'	3.5'	—	—	—	3.5'	3.5'
	Av. Width	.002"	—	.007"	.007"	.007"	.008"	.007"	—	—	—	.006"	.006"
	Max. Width	.004"	—	.014"	.012"	.012"	.013"	.012"	—	—	—	.011"	.011"
831+40 to 831+80 Outside Lane 49 Spaces @ 10' Const. - Oct. 8, 1947	Slab Temp.	—	—	69°	31°	86°	48°	94°	38°	—	—	43°	105°
	№ Cracks	—	—	4	4	4	4	4	4	—	—	4	4
	Av Spacing	—	—	10.2'	10.2'	10.2'	10.2'	10.2'	10.2'	—	—	10.2'	10.2'
	Av Width	—	—	.014"	.016"	.015"	.016"	.015"	.015"	—	—	.015"	.015"
	Max Width	—	—	.018"	.020"	.019"	.020"	.019"	.019"	—	—	.020"	.020"
835+00 to 835+20 Outside Lane 23 Spaces @ 10" Const.- Oct. 7, 1949	Slab Temp.	90°	—	75°	31°	84°	54°	94°	—	—	—	42°	105°
	№ Cracks	4	—	6	7	7	7	7	—	—	—	7	7
	Av Spacing	4.8'	—	3.2'	2.7'	2.7'	2.7'	2.7'	—	—	—	2.7'	2.7'
	Av Width	.006"	—	.010"	.012"	.010"	.011"	.010"	—	—	—	.010"	.010"
	Max. Width	.008"	—	.017"	.020"	.019"	.020"	.019"	—	—	—	.019"	.020"
848+00 to 848+50 Outside Lane 60 Spaces @ 10" Const.- Oct. 6, 1947	Slab Temp	—	101°	75	31°	102°	54°	94°	—	82°	42°	43°	100°
	№ Cracks	—	17	21	21	21	21	21	—	21	21	21	21
	Av. Spacing	—	2.9'	2.4'	2.4'	2.4'	2.4'	2.4'	—	2.4'	2.4'	2.4'	2.4'
	Av. Width	—	.001"	.006"	.007"	.006"	.008"	.007"	—	.006	.006"	.006"	.005"
	Max. Width	—	.003"	.011"	.012"	.011"	.012"	.012"	—	.011"	.011"	.011"	.010"

TABLE 5

CRACK WIDTH DATA 10" SECTION

Location and Incidental Data		Nov. 1947	Oct.-Nov. 1950	Dec. 1950	July 1951	Nov.-Dec 1951	March 1952	June 10 1952	June 27 1952	March 1953	Sept 1954	March 1957	June 1957
894+78 to 895+00 Inside Lane 26 Spaces @ 10' Const.- Sept 29, 1947	Slab Temp	—	72°	32°	94°	48°	—	93°	100°	—	—	43°	95°
	Nº Cracks	—	5	5	5	5	—	5	5	—	—	5	5
	Av. Spacing	—	4.3'	4.3'	4.3'	4.3'	—	4.3'	4.3'	—	—	4.3'	4.3'
	Av Width	—	.010"	.013"	.011"	.013"	—	.011"	.010"	—	—	.012"	.012"
	Max Width	—	.018"	.022"	.019"	.022"	—	.020"	.020"	—	—	.020"	.020"
910+00 to 910+50 Inside Lane 60 Spaces @ 10' Const.- Sept. 26, 1947	Slab Temp.	52°	76°	30°	92°	47°	—	93°	—	38°	—	43°	94°
	Nº Cracks	5	8	9	9	9	—	9	—	9	—	9	9
	Av. Spacing	10.0'	6.2'	5.6'	5.6'	5.6'	—	5.6'	—	5.6'	—	5.6'	5.6'
	Av Width	.008"	.010"	.013"	.010"	.012"	—	.010"	—	.011"	—	.011"	.010"
	Max. Width	012"	.017"	.023"	.021"	.022"	—	.019"	—	020"	—	.021"	.021"
921+11 to 921+61 Inside Lane 60 Spaces @ 10' Const.- Sept. 24, 1947	Slab Temp	52°	69°	—	94°	55°	—	88°	100°	—	—	43°	92°
	Nº Cracks	4	10	—	10	10	—	10	10	—	—	10	10
	Av Spacing	12.5'	5.0'	—	5.0'	5.0'	—	5.0'	5.0'	—	—	5.0'	5.0'
	Av. Width	.005"	.007"	—	.008"	.010"	—	.008"	.008"	—	—	.008"	010"
	Max. Width	.008"	.015"	—	.016"	.021"	—	.018"	.017"	—	—	.018"	.020"
886+62 to 886+92 Outside Lane 43 Spaces @ 10' Const.- Sept. 16, 1947	Slab Temp	—	73°	—	94°	34°	—	93°	100°	—	—	43°	96°
	Nº Cracks	—	8	—	8	8	—	8	8	—	—	8	8
	Av Spacing	—	4.5'	—	4.5'	4.5'	—	4.5'	4.5'	—	—	4.5'	4.5'
	Av. Width	—	.013"	—	.014"	.018"	—	.014"	.014"	—	—	.016"	.016"
	Max. Width	—	.019"	—	.019"	.022"	—	.018"	.018"	—	—	.020"	019"
910+25 to 910+50 Outside Lane 30 Spaces @ 10' Const - Sept. 11, 1947	Slab Temp	52°	70°	30°	—	47°	38°	93°	—	—	—	41°	94°
	Nº Cracks	4	6	6	—	6	6	6	—	—	—	6	6
	Av Spacing	6.2'	4.2'	4.2'	—	4.2'	4.2'	4.2'	—	—	—	4.2'	4.2'
	Av Width	.015"	.016"	.020"	—	.020"	.021"	.018"	—	—	—	.022"	.023"
	Max. Width	.020"	.025"	.029"	—	.027"	.028"	.027"	—	—	—	.029"	.030"
921+25 to 921+50 Outside Lane 30 Spaces @ 10' Const - Sept 10, 1947	Slab Temp	52°	77°	32°	—	55°	—	88°	—	—	84°	43°	94°
	Nº Cracks	4	8	8	—	8	—	8	—	—	8	9	9
	Av. Spacing	6.2'	3.1'	3.1'	—	3.1'	—	3.1'	—	—	3.1'	2.8'	2.8'
	Av. Width	.008"	.008"	.012"	—	.010"	—	009"	—	—	011"	.010"	.010"
	Max. Width	.011"	.015"	.018"	—	.018"	—	.016"	—	—	.017"	.017"	.017"
925+34 to 925+50 Outside Lane 19 Spaces @ 10' Const.- Sept. 9, 1947	Slab Temp	70°	77°	30°	95°	40°	—	88°	—	—	82°	43°	94°
	Nº Cracks	2	4	4	4	4	—	4	—	—	4	4	4
	Av Spacing	7.9'	3.9'	3.9'	3.9'	3.9'	—	3.9'	—	—	3.9'	3.9'	3.9'
	Av Width	.008"	.009"	.013"	.010"	.013"	—	.010"	—	—	.013"	.012"	.014"
	Max. Width	.013"	.014"	.017"	.014"	.018"	—	.015"	—	—	.018"	.017"	.017"

between the plugs is susceptible to a slight reduction in length.

2. As will be brought out later, the widths shown are indicative only of the crack widths at the pavement surface, and are by no means necessarily indicative of the widths of these cracks below the surface.

From Tables 4 and 5 it will be apparent that:

1. The greatest increase in crack width occurred during very early life.
2. The increase in crack width during the past seven years has been practically nil.
3. The seasonal variation in crack width has been very small.
4. The cracks have never returned to zero width, nor ever approached doing so, even during periods of high temperature.

Measurements taken at the crack plugs in March 1957 and again in June 1957 showed that the decrease in crack width during this period was practically nil, despite an increase in pavement temperature of more than 50 F. In view of this finding it was decided to install plugs at a number of the wider cracks in both sections, inasmuch as there were many cracks which appeared to be considerably wider than those in connection with the plugs originally installed. Therefore, plugs were installed at about 60 of the wider cracks, and the original measurements were taken in July 1957, the pavement temperature being 102 F at the time. Subsequent measurements taken in December 1957, when the pavement temperature was 60 F lower, showed that:

1. The cracks in the 8-in. section had undergone an average opening of only 0.0010 in., the maximum opening being 0.0023 in.
2. The cracks in the 10-in. section had undergone an average opening of only 0.0015 in., the maximum opening being 0.0029 in.

It was also decided to core-drill the pavement at a few of the wider cracks, to determine whether the failure to undergo closure during hot weather had been due to the accumulation of incompressible material in the cracks. Accordingly, four 6-in. diameter cores were obtained, two from each section. Quite unexpectedly it was found that in all cases the cracks were much narrower in the lower two-thirds of the pavement than at the surface, and of only microscopic width in the lower third. In view of this finding, 12 additional cores were obtained, five from the 10-in. section and seven from the 8-in. section. In nine of these cores the cracks were essentially the same as in the first four cores. In the remaining three cores, however, which were obtained at appreciable localized settlements, a crack of hair-line width extended clear through to the bottom.

Figure 10 shows one of the wider cracks in the outside lane of the 10-in. section, and the exposed shoulder edge of the pavement. It also shows the location of the core obtained at this crack.

In this instance the upper line of steel was found to be 6 in., instead of the specified 2 in., below the surface. This however, is apparently a localized condition, because it was not found to be anywhere near the same degree in connection with the other cores. At any rate, during the drilling operations the core broke in two at this line of steel, and there was a general breaking-up of the concrete for about the next inch. The lower 3 in. of the core remained intact, however, and in this portion the crack was of only microscopic width. From a careful inspection of the entire core it appears that this crack was of appreciable width clear down to the upper line of steel. It therefore seems that installing the steel a considerable distance below the surface might tend not only to promote wide cracks at the surface, but also to promote wide cracks of appreciable depth. Incidentally, measurements show that this crack is not at a hump in the grade line.

Several of these cores were subsequently sawed in two longitudinally to facilitate measuring the widths of the cracks at various levels below the pavement surface. The results of these measurements, which were taken by means of a 20-power scale microscope, are given in Table 6, which also shows the distance below the surface of the upper

Figure 10. Crack in 10-in. section, Sta. 927+81.

and lower lines of reinforcing steel. It must be appreciated, however, that owing to the irregular nature of these cracks, and the breaking-down of their edges, it is extremely difficult, if not impossible, to measure their true widths, and for this reason the figures shown are necessarily only approximate. It is believed, however, that the cracks are at least no wider than indicated.

Figure 11 shows a crack and the exposed edge of the pavement at 894+92 in the outside lane of the 10-in. section, the difference in crack width at the surface and in the lower portion of the pavement being clearly apparent.

The core (C-2) obtained at this crack is shown in Figure 12, the location of the steel

TABLE 6

WIDTH OF CRACKS AND LOCATION OF STEEL IN CORES (in.)

Core No.	Section (in.)	Lane	Eng'rs. Sta.	Distance Below Surface							
				Crack-width Measurement[1]						Steel	
				0 in.	1 in.	3 in.	5 in.	7 in.	9 in.	Upper	Lower
1	10	Outside	927+81	-	-	-	-	.008	.001	6.0	8.5
2	10	Outside	894+92	.060	.020	.016	.008	.004	M	3.0	7.2
3	10	Outside	910+49	.050	.018	.014	.010	.004	M	2.4	7.7
10	10	Outside	886+82	.020	M	M	M	M	M	2.5	7.0
21	10	Outside	905+22	.020	.020	.016	.012	.008	.008	1.7	7.3
9	10	Inside	894+88	.025	.012	M	M	M	M	2.6	7.2
5	8	Outside	835+14	.025	.020	.008	.004	M	-	3.2	5.4
6	8	Outside	835+04	.020	.016	.004	M	M	-	3.2	5.4
7	8	Outside	831+70	.020	.018	.016	.006	M	-	3.0	5.2
19	8	Outside	823+08	.030	.016	.010	.004	M	-	3.2	6.2
22	8	Outside	820+63	.010	.004	.004	.003	M	-	2.2	5.0
23	8	Outside	831+94	.010	.001	M	M	M	-	3.2	5.1

[1] M = crack of microscopic width.

being indicated by dashed lines. In this instance the crack occurred at a lap in the upper line of steel, and at a transverse member in the uppermost line.

Figure 13 shows a core (C-21) obtained at 905+22 in the outside lane of the 10-in. section. It will be noted that in this core the crack, although very narrow, is traceable clear to the bottom. There was a slight sag in the grade line where this core was obtained, and this may account for the more or less uniform crack width below the upper line of steel. As in the case of Core C-2, this crack also occurred at a transverse member in the upper line of steel. Apparently this is typical of a great many of the cracks.

It will also be noted that in this core the upper line of steel is only about $1\frac{1}{2}$ in. below the surface, and that to the right of the vertical crack there is a horizontal crack at the level of the steel. A careful investigation was made to determine whether this crack existed in the pavement or, on the other hand, was induced by the drilling operations. This involved wedging the core apart at the horizontal crack. This resulted in what appeared to be a new fracture, but there were some indications that there might have been a plane of weakness at this point.

The pavement itself was also carefully examined. In addition to a visual inspection, the pavement surface adjacent to the crack was tapped with a hammer. No evidence of a horizontal crack was found. But in view of this occurrence, plus the fact that the reinforcing steel showed some evidence of rusting at the crack, it appears that even though it may be advisable to install the steel above mid-depth, it is also advisable that it not be installed too close to the surface.

Figure 14 shows a core (C-9) obtained at a crack at 894+88 in the inside lane of the 10-in. section. Although this crack was about 0.025 in. wide at the surface, it was of only microscopic width throughout practically the entire depth of the pavement.

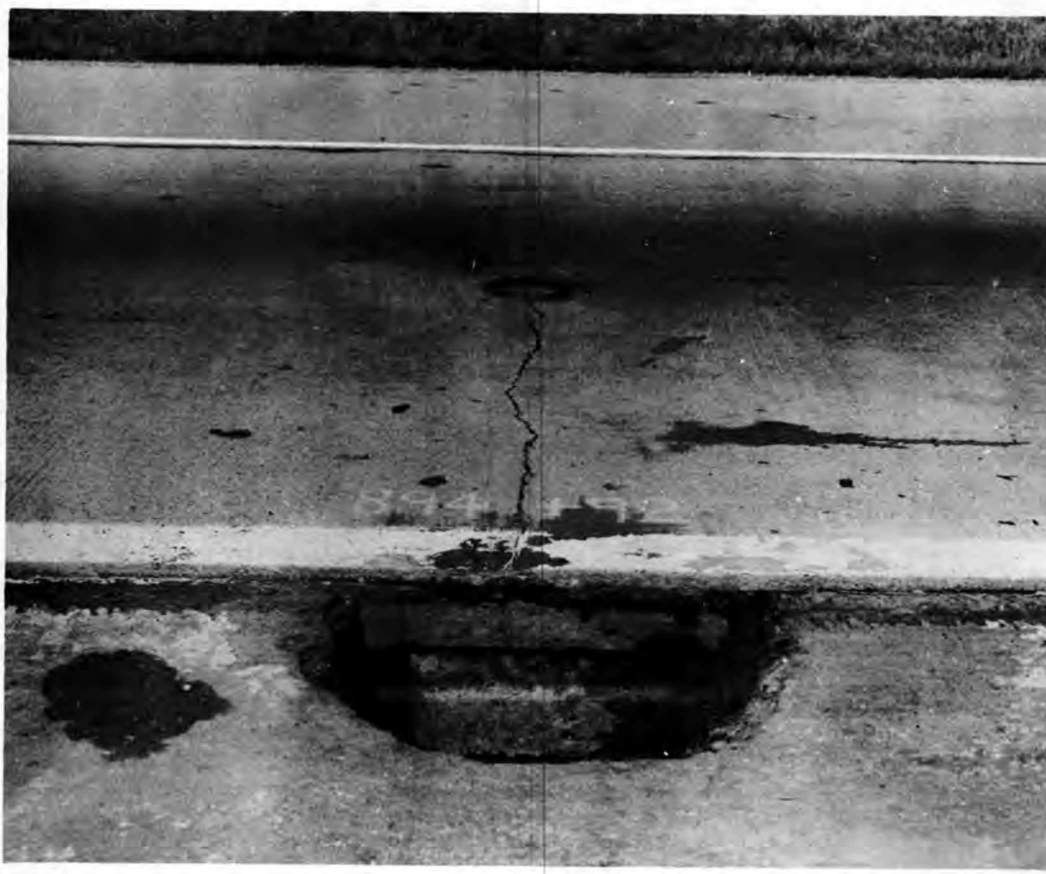


Figure 11. Crack in 10-in. section, Sta. 894+92.

Figure 15 shows a crack and the exposed pavement edge at 835+04.5 in the outside lane of the 8-in. section.

The core (C-6) obtained at this crack is shown in Figure 16, which also shows a thin slice of concrete sawed from the outer surface of the core. Oddly enough, despite the fact that there was no steel holding this slice together, it did not come apart at the crack. It was subsequently determined, however, that the slice was held together by about 2 in. of concrete near the bottom, notwithstanding the presence of a crack of microscopic width through this concrete.

Figure 17 shows a core (C-19) obtained at a ravelled crack at 823+08 in the outside lane of the 8-in. section, and also shows a thin slice of concrete sawed from the side of the core. In this instance the slice did come apart at the crack. Despite the extreme narrowness of this crack in the lower portion of the core, the cracked surfaces were discolored for the full depth of the pavement. Moreover, the crack contained what appeared to be very finely divided silty materials, from brown to almost black in color. Similar materials, incidentally, were found in all of the cracks examined.

Figure 18 shows a core (C-4) obtained at a badly ravelled crack, which had been sealed, about 4 ft north of a construction joint at 849+34 in the outside lane of the 8-in. section. This core broke in two during the drilling operations. As will be noted, the crack is quite wide in the upper third of the pavement (above the upper line of steel). In the lower two-thirds, however, the crack was found to be only about 0.01 in. wide. It is apparent that the conditions at this crack are none too good. In addition to rather serious ravelling at the pavement surface, appreciable rusting was found in connection with the lower line of steel, especially of the transverse member.

Figure 12. Core at crack in 10-in. section, Sta. 894+92.

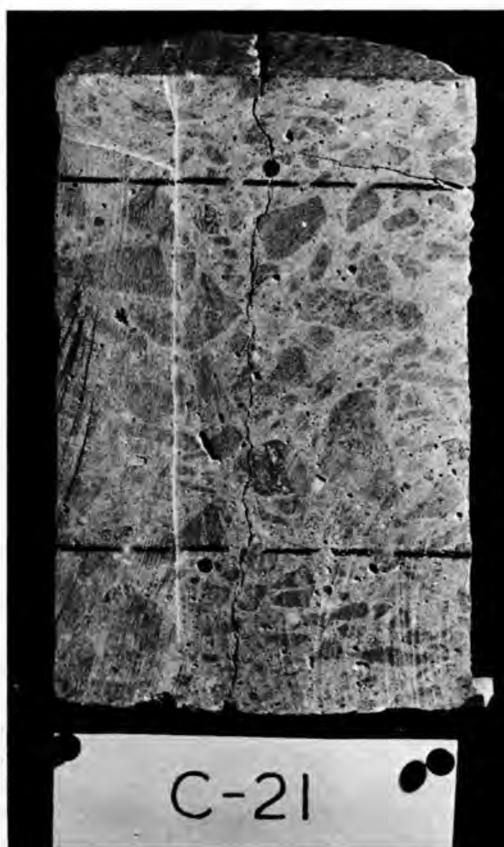


Figure 13. Core at crack in 10-in. section, Sta. 905+22.

The procurement of these cores has resulted in several very important findings, as follows:

1. The measurements at the crack plugs installed during construction are of very limited value, and are indicative only of the surface widths of the cracks.
2. From all indications, the cracks are much narrower in the lower portion of the pavement than they are at the surface. There may, of course, be exceptions. But in a total of 16 cores, no cracks exceeding about 0.015 in. in width were found at the bottom, even at appreciable localized settlements.
3. Owing to the marked difference in crack width from top to bottom, the surface appearance of the pavement, at least in general, is not indicative of its true structural condition.

Figure 14. Core at crack in 10-in. section, Sta. 894+88.

The reason for this difference in crack width is not known. A number of factors may, of course, be involved. However, differential shrinkage of the concrete, which might have been aggravated by the high slump employed, appears to be the most likely predominating cause.

With respect to these findings, it may be stated that because of the serious appearance of the cracks there had been speculation for a number of years as to how the outside lanes of these sections could possibly hold together under the heavy truck traffic they were carrying. Moreover, it was because of their "apparent" poor condition, and the fact that a serious failure had already occurred in the 10-in. section, that the long-term performance of the outside lanes was viewed with skepticism in the 1950 report. Whether or not the two additional failures that occurred a short time later in the 8-in. section justified this point of view is, of course, a matter of opinion. But it has nevertheless become apparent that the life expectancy of the continuously-reinforced pavements cannot be predicted solely on the basis of their surface appearance.

Incidentally, no faulting has been observed at any of the cracks.

CONSTRUCTION JOINTS

In general, the pavement adjacent to the construction joints is still in good condition. Furthermore, there has been no apparent faulting at any of these joints. As of June 25, 1957, the recorded opening of the construction joints in the 10-in. section ranged from 0.004 to 0.045 in., and averages 0.015 in. In the 8-in. section the opening ranged from 0.002 to 0.011 in., and averaged 0.007 in.

Rather severe cracking has occurred, however, in the immediate vicinity of several of these joints, for a distance of 20 ft or more, but only on the northerly side. (Construction was from south to north.) The severity of this cracking is shown in Figure 19, the construction joint being about 3 ft to the left of the core hole. But despite what would appear to be the very poor condition of the pavement, no failures other than excessive ravelling at some of the cracks have occurred in any of these areas. As suggested previously, the reason for the absence of failure appears to be that the cracks are much narrower in the lower portion of the pavement than at the surface.

It is generally believed that the difference in cracking on one side of the joint, as compared with the other, has been due primarily to having attached together two sections of pavement which necessarily had, at the time of attachment, materially different tensile strengths.

CHANGES IN ELEVATION

At certain locations, immediately after construction levels were taken at 5-ft intervals along the edges of the lanes, to determine the amount of subsequent change in elevation. Levels taken one year later indicated that, in general, the changes were limited to less than 1/4 in., but that there were a few localized areas in the outside lanes where, within a distance of about 30 ft, settlements of as much as 5/8 in. had occurred: a slight dip is felt in riding over these areas. A recent re-taking of some of these levels has indicated, however, that little change in elevation has taken place during the past nine years. Probably for this reason, there appears to have been no appreciable change in riding qualities since 1948.

It is to be pointed out, however, that because of the closely spaced cracks these sections appear to lack the longitudinal rigidity and load-distributing capacity of the standard reinforced concrete pavements, and are therefore more susceptible to the development of an undulating profile and localized settlements.

FAILURES

As previously mentioned, in May 1949 a localized area in the outside lane of the 10-in. section approximately 1,430 ft from the south end began to undergo failure,

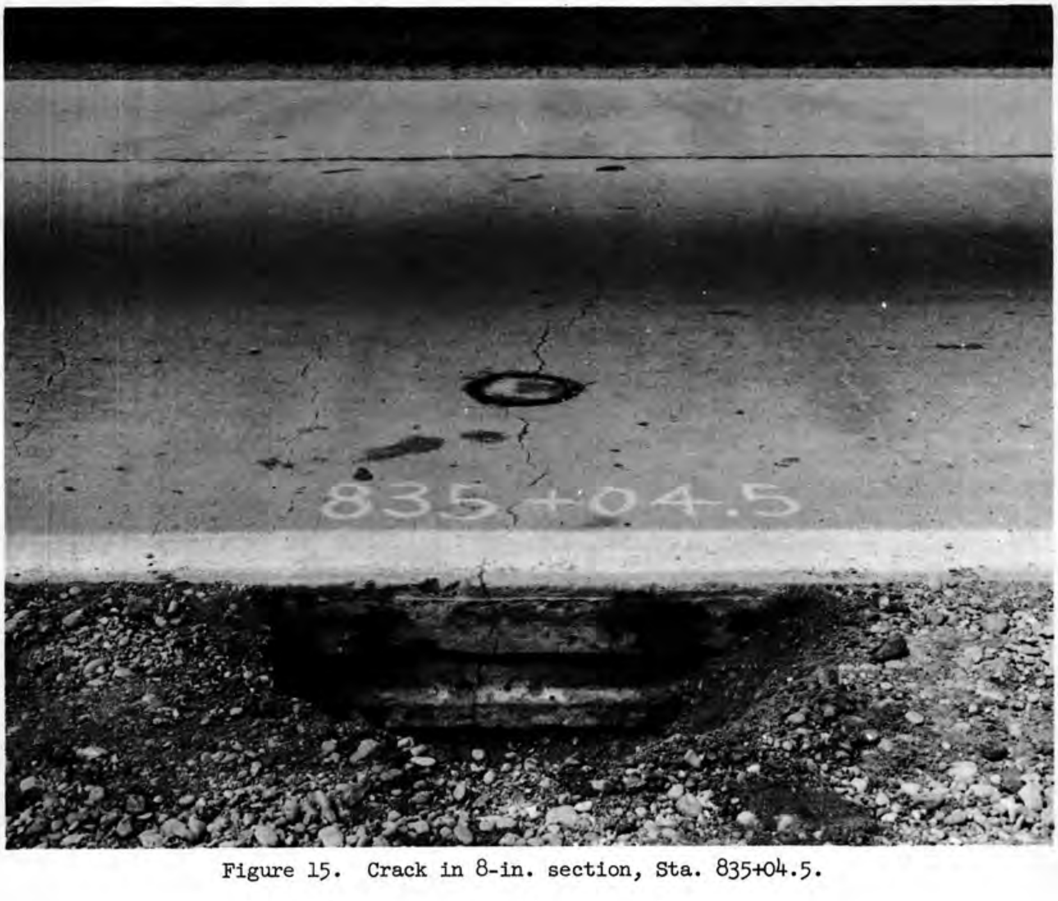


Figure 15. Crack in 8-in. section, Sta. 835+04.5.

this being evident from severe cracking and the exudation of a whitish-colored substance from the cracks during wet weather. Pictures were taken periodically in this location to obtain a photographic record of the failure as it progressed. Figure 20 shows this failure on November 17, 1949.

By the summer of 1951 the failure had advanced to the stage where the entire width of the lane had become involved. The conditions on August 23, 1951, are shown in Figure 21, at which time considerable buckling of both the pavement and the reinforcing steel had occurred.

In December 1951 it became necessary to remove the damaged concrete. This work resulted in creation of an open space about 7 ft wide, for the full width of the lane. As a temporary measure, the space was filled with crushed stone topped with bituminous material.

In October 1954 this area was repaired with concrete, which involved replacement of 25 ft of the lane and installation of a dowelled expansion joint at the center of the repaired area. Figure 22 shows this area in August 1957. The black discoloration on the pavement adjacent to the joint is due to extrusion of a large quantity of joint-sealing compound which had been inadvertently poured into the joint space while in an open condition during cold weather.

Mention was also made previously of two similar failures which occurred in the outside lane of the 8-in. section. The first of these failures occurred at a point 569 ft from the north end, and first became apparent in May 1951. Conditions at this point

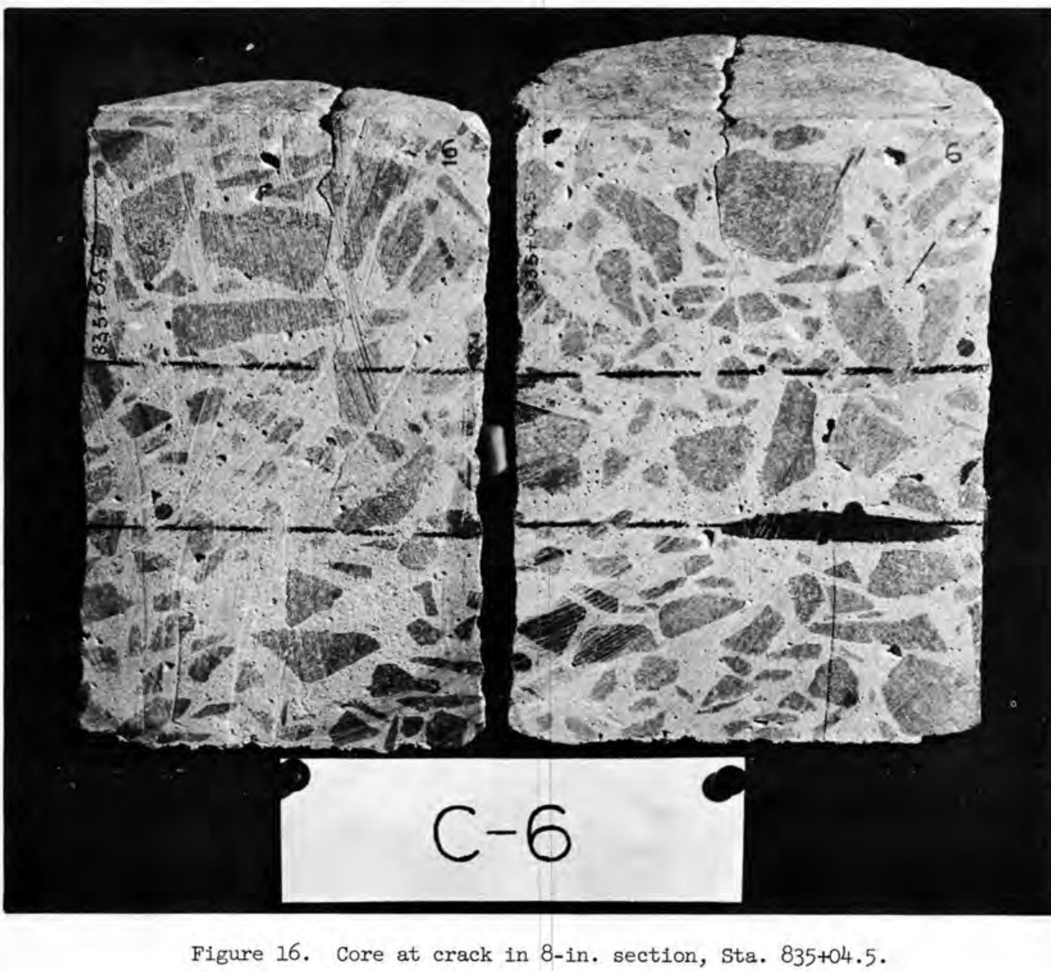


Figure 16. Core at crack in 8-in. section, Sta. 835+04.5.

six months later (November 27, 1951) are shown in Figure 23.

For the next three years the pavement was maintained in passable condition by periodically removing the ruptured concrete and replacing it with bituminous materials. In October 1954 this area was repaired with concrete, at which time a dowelled expansion joint was installed at the center of the repaired area.

The second failure occurred at a point 372 ft from the north end, and first became apparent in 1952. This failure was of the same nature as the one just described, and was also repaired with concrete in October 1954, in the same manner. Each of these repairs involved replacement of about 12 ft of the lane.

The cause of these three failures is unknown. However, there appear to be several possible causes: namely, (a) an excessively wet batch of concrete, (b) a batch of concrete deficient in cement, (c) poor consolidation of the concrete and resulting inadequate bond with the reinforcing steel, (d) a localized weakness in the subgrade, and/or (e) a horizontal plane of cleavage at the upper line of steel. In addition, a pavement of this design is under a high degree of tension in cold weather, at least during early life, and this tension may have been an important contributory cause.

Figure 17. Core at crack in 8-in. section, Sta. 823+08.

Figure 18. Core at crack near construction joint, 8-in. section.

There is also the fact that all of these failures occurred in the outside lanes and that no similar failures have occurred in the inside lanes. Presumably, therefore, the heavy truck traffic in the outside lanes played an important part in their occurrence. On the other hand, the absence of any further failures of this nature since 1952 seems to indicate quite definitely that there was some form of weakness at these failed areas which was not present elsewhere in the outside lanes. Whether or not there are similar weaknesses in the inside lanes, but which have not been brought to light owing to the relative absence of truck traffic in these lanes, is obviously not known.

In view of the indications that these failures may have been caused primarily by construction deficiencies of a type which usually have little or no apparent effect in connection with a conventional pavement, it appears that great care needs to be exercised in the construction of continuously-reinforced pavements.

During the past eight or nine years a progressive deterioration of the concrete has been taking place in a small area in the outside lane of the 8-in. section at a point about 490 ft from the south end. The present condition of this area is shown in Figure 24. Judging from the appearance of the surface, an excessively wet, stoney batch of concrete may have been placed at this point. Whether or not this deterioration will eventually advance to the point of serious structural failure remains to be seen.

In February 1957 a rupturing of the concrete was discovered adjacent to the terminal joint at the north end of the inside lane of the 10-in. section. At that time the rupturing was confined to the end of this section. During the following summer, however, rupturing also occurred in connection with the adjoining pavement. The present conditions are shown in Figure 25, the 10-in. section being to the left of the joint.

Investigations have shown that this rupturing has been due to the combined effects of (a) large seasonal changes in width of this joint and resulting infiltration of large amounts of sandy and gravelly materials, (b) compression of the wood joint filler to the point of refusal, and (c) resistance of the adjoining pavement to displacement. Although there have been no similar failures at any of the other terminal joints, this occurrence serves to illustrate the deficiencies of conventional expansion joints when used at the ends of continuously-reinforced pavements.

RELATIVE CONDITION OF LANES

As previously mentioned, three serious failures requiring replacement of the concrete

Figure 19. Cracking near construction joint, 8-in. section.

have occurred in the outside lanes, and a localized area in the outside lane of the 8-in. section is undergoing slow deterioration. In addition, there has been rather pronounced ravelling at some of the cracks in the outside lanes of both sections. Because of this ravelling, a few of these cracks have been poured with joint-sealing compound. The inside lanes, on the other hand, are still in first-class condition.

From the standpoint of riding qualities, the outside lanes have numerous undulations of noticeable magnitude, whereas the undulations in the inside lanes are much less frequent and much less pronounced.

Needless to say, the relative performance of these lanes serves to emphasize the fact that, as in the case of any pavement, the amount and kind of traffic a continuously-reinforced pavement is to carry must definitely be given serious consideration in the design.

RELATIVE PERFORMANCE OF STANDARD PAVEMENT

In addition to these continuously-reinforced sections, this project included construction of 52 slabs (26 in each lane) containing New Jersey's standard amount of welded wire fabric (612-2/03). These slabs were of 10-in. uniform thickness, 56 ft long, and the joints used in connection therewith were dowelled expansion joints. With the exception of a few experimental joints, the dowels, which were installed 12 in. c. to c., consisted of $1\frac{1}{4}$-in. diameter cold-finished steel bars partly uncased in Monel tubing.

With the exception of a slight sag at one of the joints in the outside lane, and the rupturing previously described at the north end of the 10-in. section, all of these slabs are still in excellent condition and there has been no apparent faulting at any of the joints. Of these 52 slabs, 48 are still uncracked. To date this pavement has required no main-

Figure 20. Failure in 10-in. section, Nov. 17, 1949.

tenance whatever, other than periodic resealing of the joints.

One year after construction of this project a $2\frac{1}{2}$-mi section of concrete pavement was constructed immediately to the north by the same contractor, on the same route, and carrying the same traffic. This pavement was of 10-in. uniform-thickness reinforced concrete, on a 12-in. layer of bank-run gravel subbase. Dowelled expansion joints with 3/4-in. wood filler were installed at 78-ft intervals. The dowelling system was of the type previously described. This, essentially, has been the New Jersey standard design of heavy-duty reinforced concrete pavement since 1948.

All of this pavement is still in excellent condition. There has been no apparent faulting at any of the joints. In a total of 171 slabs in the outside lanes, 164 are still uncracked. Furthermore, there is only one transverse crack, of hair-line width, in each of the remaining seven slabs in this lane. All of the 169 slabs in the inside lane are still uncracked.

Other than periodic resealing of the joints (which would probably not have been necessary had they been sealed originally with the presently used type of rubber-asphalt compound), this pavement has required absolutely no maintenance whatever. Also, there are no indications of incipient failures of any kind, and despite the presence of expansion joints, the riding qualities of this pavement are in no apparent way inferior to the riding qualities of the continuously-reinforced sections probably owing to the care taken in the finishing and edging of the joints, and to adequate dowelling.

RELATIVE COSTS

In the continuously-reinforced sections the cost of the reinforcing steel alone was \$2.78 per square yard of pavement. In consequence, the cost of the 10-in. section was approximately \$1.60 more per square yard than was paid in 1947 for the standard design of 10-in. reinforced pavement. Even the cost of the 8-in. section was approximately \$1.00 more per square yard than the 10-in. standard design. This higher cost would perhaps be of little importance were it reflected in superior performance, but to date there have been no apparent indications that this has been the case.

Figure 21. Failure in 10-in. section, Aug. 23, 1951.

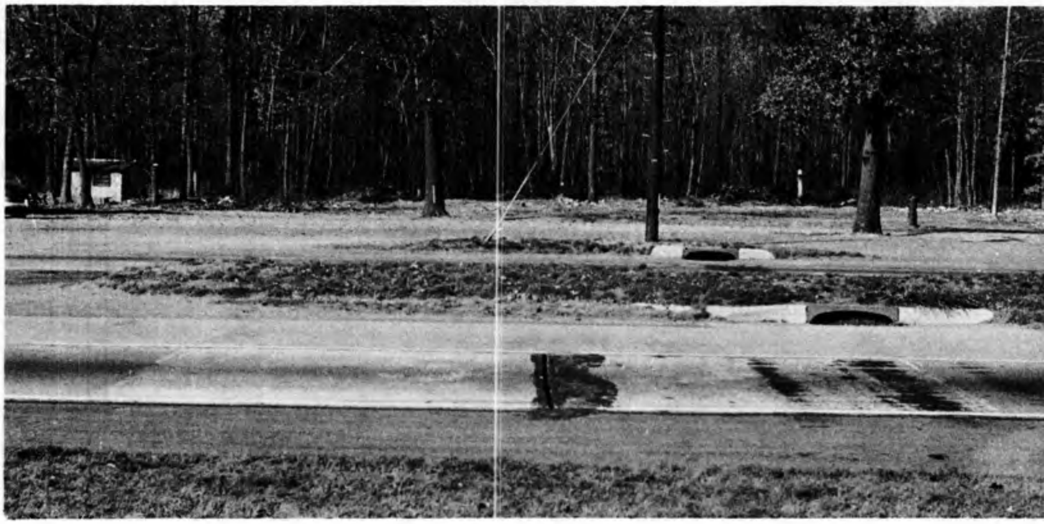


Figure 22. Repaired area, 10-in. section, August 1957.

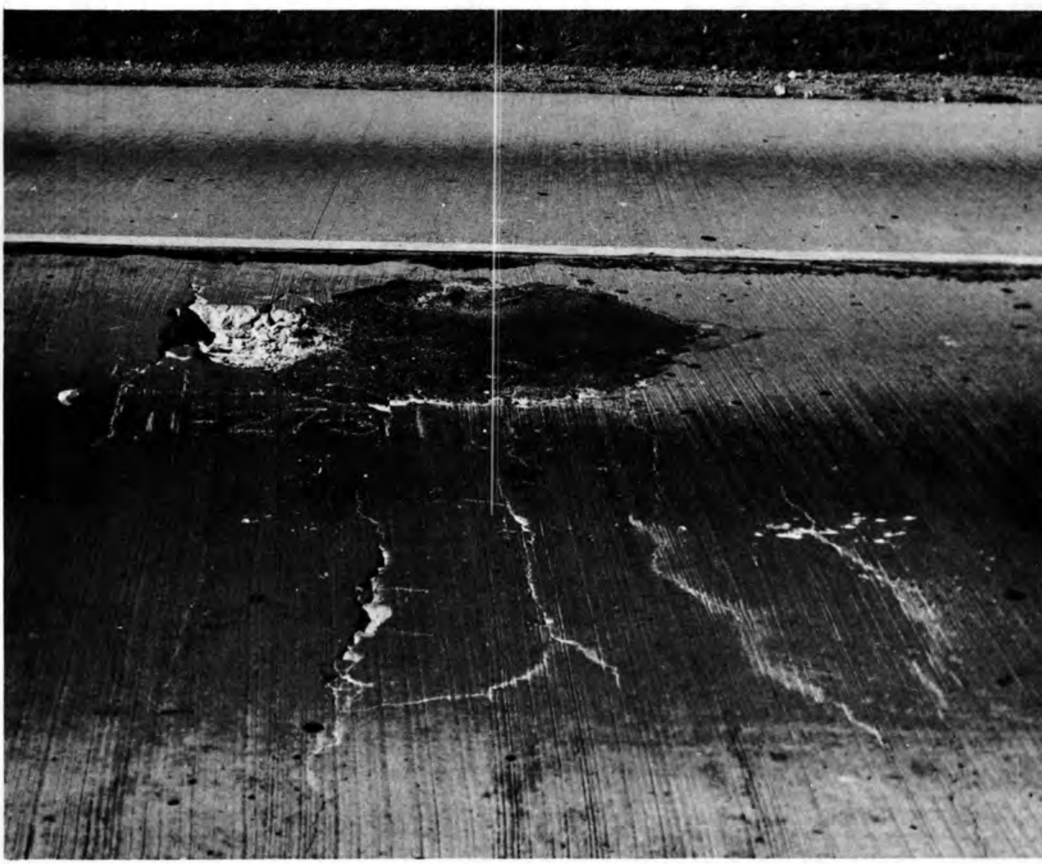


Figure 23. Failed area, 8-in. section, Nov. 27, 1951.

SUPPLEMENTARY EXPERIMENTAL PAVEMENT

As shown in Figure 5, this project included an experimental section of pavement involving slabs 187 ft long. In the end portions of these slabs, for a distance of 34 ft, the reinforcing steel consisted of a single line of the standard type of welded wire fabric. In the remaining 119-ft central portions the steel consisted of a single line of the same kind of wire fabric installed in the continuously-reinforced sections (0.36 percent). The expansion joints between these slabs were of the dowelled type previously described.

With the exception of a corner spall about 6 in. square at the shoulder end of Joint 7, all of these slabs are still in very good condition, and there has been no apparent faulting at any of the joints.

With respect to transverse cracking, the present condition of these slabs is as follows:

Inside lane		Outside lane	
No. of Slabs	Cracks	No. of Slabs	Cracks
6	0	4	0
1	1	3	1
1	2	1	2

The cracks are not concentrated within the central regions. On the contrary, five are within 26 ft of the ends, and only two are within the central regions. All of these

Figure 24. Deteriorating area, 8-in. section.

cracks are very narrow, free from ravelling, and of no apparent structural significance.

Inasmuch as this report lists fewer cracks in these slabs than listed in the 1950 report, it is necessary to state that the 1950 report included a number of cracks which, on the basis of a careful recent inspection, were actually very fine surface checks, whereas this report lists only clearly defined structural cracks.

As previously mentioned, the pavement involving these 187-ft slabs has undergone an over-all increase in length, although the exact amount is not known. But on the basis of reference-line measurements in the vicinity of Joints 10 and 23 at the north end, the inside lane at this point has moved northerly about 5/8 in., and the outside lane has moved northerly about 1 in. It is problematical, however, whether any appreciable outward movement has occurred at the south end, owing to the restraining effect of the 10-in. continuously-reinforced section.

This over-all increase in length is also indicated by the considerable increase in width that occurred at the joints between these slabs during the period August 27, 1948-June 19, 1957, and by the decrease in width of the joints in the adjoining pavement. (See lower line of figures "June 19, 1957, compared with August 27, 1948," in tabulation in Figure 5.)

It will also be noted that the joints between these slabs have undergone large seasonal changes in width. As a result, there has been infiltration of large amounts of solid material. Although this has not caused any apparent damage except the small spall at Joint 7, recent experience in other locations indicates that infiltration can be very harmful in connection with an extensive section of pavement in which the joints are spaced 100 ft or more apart. It is not considered advisable, therefore, to construct slabs of this length.

SUMMARY

The more important results and observations in connection with these continuously-reinforced sections during the past ten years may be summarized as follows:

1. The maximum recorded over-all changes in length of the inside and outside lanes

Figure 25. Ruptured terminal joint, 10-in. section, August 1957.

of the 8-in. section have been, respectively, 1.66 and 1.22 in. For the inside and outside lanes of the 10-in. section these changes have been, respectively, 1.92 and 1.99 in.

2. The 3,500-ft central regions of the inside lanes of both sections have remained essentially at constant length at all times. (This does not apply at present to the outside lanes, owing to certain failures in these lanes.)

3. Neither section has undergone any apparent permanent increase in length. There are indications, however, that there has been a tendency in this direction, which has been counteracted by the restraining effect of the adjacent pavement.

4. The terminal joints at the ends of both sections have undergone an appreciable increase in width, due to the infiltration of solid materials, but despite this increase there has been no apparent faulting at any of these joints.

5. In conjunction with the increase in width of the terminal joints, the slabs adjacent to the ends of both sections have been moved away from these sections, in amounts ranging from approximately 1/4 to 1¾ in.

6. The central regions of both sections have a large number of transverse cracks. These cracks are of an extremely erratic nature. Their spacing averages from about 3½ to 4½ ft, and ranges from as little as 6 in. to as much as 20 ft. Many of these cracks occurred immediately after construction (within a matter of days), and practically all of the remainder occurred within the next three years.

7. In the 12 locations where gage plugs were installed, the maximum recorded crack width has been 0.03 in. There are, however, other locations where the crack width at the surface is as much as 0.06 in.

8. Based on cores recently obtained at 16 cracks in the central regions, the cracks are much narrower in the lower two-thirds of the pavement than at the surface, even at localized settlements. There may, however, be exceptions, but these have not been found.

9. At some of the wider cracks where cores were obtained there was evidence of rusting of the reinforcing steel, but apparently not to a serious degree.

10. Owing to the variation in crack width from top to bottom, the surface appearance of the pavement, at least in the main, is not indicative of its true structural condition.

11. There has been no apparent faulting at any of the cracks.

12. In June 1957 the recorded opening of the construction joints in the 10-in. section ranged from 0.004 to 0.045 in., and averaged 0.015 in. In the 8-in. section the opening ranged from 0.002 to 0.011 in., and averaged 0.007 in.

13. There has been no apparent faulting at any of the construction joints.

14. Many closely-spaced ravelled cracks have occurred immediately north of several of the construction joints in both sections, but no structural failures have occurred in these areas.

15. With the exception of a few localized settlements and undulations in the outside lanes of both sections, the changes in profile which have developed since construction have not resulted in any noticeable impairment in riding qualities.

16. With the exception of serious rupturing at the terminal joint at the north end of the 10-in. section, the inside lanes of both sections are still in very good condition.

17. Three serious localized failures, which required removal and replacement of the damaged concrete, have occurred in the outside lanes; one in the 10-in. section and two in the 8-in. section. The cause of these failures is not known, but the indications are that they were caused primarily by a structural deficiency of some kind in these particular areas. However, the absence of any failures of this nature in the inside lanes would seem to indicate that heavy truck traffic also played an important part in their occurrence.

18. The standard expansion joints have not proved satisfactory as terminal joints.

19. The 16 supplementary 187-ft slabs constructed in conjunction with this project are still in very good condition, and ten of these slabs are still uncracked. There has been no faulting at any of the intermediate joints. However, owing to the large seasonal changes in joint width and resulting infiltration of large amounts of solid material, it does not appear advisable to construct slabs of this length.

CONCLUSIONS

From the foregoing, it is apparent that:

1. Within five years after construction three serious failures occurred in the outside lanes of the continuously-reinforced sections. In addition to constituting a troublesome maintenance problem, these failures eventually required removal and replacement of the damaged concrete.
2. Owing to the close spacing of the transverse cracks, and as indicated by the localized settlements and undulations which have developed in the outside lanes, these sections apparently lack the longitudinal rigidity and load-distributing capacity exhibited by the standard reinforced concrete pavements.
3. The cost of these sections was substantially higher than that of the standard pavements constructed during the same period.
4. The standard pavements on the same route, of essentially the same age, and carrying the same traffic, are still in excellent condition and show no signs of incipient failure. In addition, they have first-class riding qualities.

For these reasons it is felt that, to date, the continuously-reinforced pavements have not proved superior, nor even equivalent, to the standard design of reinforced concrete pavement. Whether or not this will still be the case some years hence remains, of course, to be seen.

It is hoped, however, that this experience will in no way tend to discourage further investigations into the possibilities of this design. Certainly, the fact that a continuously-reinforced pavement constitutes, in effect, a continuous ribbon of concrete without joints of any kind is in itself a strong point in its favor. Furthermore, it is apparent that a pavement of this type is susceptible to a wide range of variation in design, such as in thickness, and in the amount, type, location and tensile strength of the reinforcing steel. Consequently, it is felt that no final conclusions relative thereto should be drawn on the basis of a single experiment, such as this one.

ACKNOWLEDGMENTS

The author wishes to express his sincere thanks to the following individuals and their assistants for their extremely helpful contributions to the preparation of this report: R. H. Stelljes and J. T. Stephan, Bureau of Engineering Research; J. C. Reed, Testing Laboratory; H. A. Hartmann, Bureau of Planning and Economics; J. W. Evans, Maintenance Division; and J. T. Madden, Roads, Location and Design.

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Continuously-Reinforced Concrete Pavement in California After Eight Years Service

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California Division of Highways

An experimental continuously-reinforced concrete pavement one mile in length was constructed in California in 1949. After eight years of heavy traffic, it remains in good condition. Adjacent sections of non-reinforced jointed pavement constructed at the same time are also in relatively good condition, with the exception of a moderate amount of faulting at a few joints. The experimental pavement has not reached sufficient age to warrant comparison with non-reinforced pavement on an economic basis.

IN 1949, the California Division of Highways constructed a test section of continuously-reinforced portland cement concrete pavement one mile in length as part of a project involving non-reinforced pavement with weakened-plane joints at 15-ft intervals. In both the regular and experimental sections the pavement was 24 ft wide and 8 in. thick, constructed lane-at-a-time. The pavement forms one side of a four-lane divided expressway. The experimental section was divided into two equal lengths containing, respectively, 0.50 and 0.62 percent of reinforcement consisting of ½-in. round, deformed billet-steel bars. The reinforcement was continuous through the two portions.

A detailed description of the test project, its instrumentation, and the observed results up to the end of the first year, was reported by Stanton (1). The purpose of the present report is to record the comparative condition of the experimental section and the adjacent non-reinforced sections after eight years of service.

In 1949 the total traffic over the section in one direction was 6,000 vehicles of all types per day. In 1956 the total traffic in one direction was 9,000 vehicles per day, of which 14 percent consisted of heavy trucks and buses. Legal load limits in California are 18,000 lb per single axle and 32,000 lb for tandem axles.

The average uncracked slab length in the outer lane of Section 2 of the reinforced pavement was 4.2 ft in 1950. By 1957 the average length had decreased to 3.2 ft. All of the cracks appear to be tight, with little or no spalling. None shows evidence of pumping. Figure 1 shows one of the most prominent cracks in the reinforced section. Figure 2 shows a typical weakened-plane joint in the section adjacent to the reinforced section. Figure 3 shows a random crack in the non-reinforced pavement.

Movement of the pavement in a longitudinal direction has been measured several times. The results are summarized in Table 1. Longitudinal movement has been confined to a distance of about 400 linear ft at each end.

In June 1957, the reinforced section was 0.98 in. longer than in June 1950. The change took place in about 800 linear ft of pavement. Such an increase could be attributed entirely to thermal expansion only if the pavement temperature in June 1957 was about 18 degrees higher than in June 1950. The actual pavement temperatures are not known. The effects would have been difficult to evaluate, because the entire series of measurements required several days to complete. It must be recognized as a possibility that at least part of the observed increase in length was permanent.

Terminal joints 4 in. wide were constructed of multiple plies of preformed expansion joint filler. The joints have been sealed a number of times and remain in good condition.

Closure of the terminal joints was always greater than the increase in length of the reinforced pavement. This indicates that pressure was being exerted against the jointed pavement at each end. Measurements indicate that the jointed pavement was shortened through a distance of about 300 linear ft from the terminal joint.

Random cracks did not form between installed gage points and no record of their width is available. There were, however, eight pairs of gage points that were installed to span weakened-plane joints and their movement may be considered to be representative of

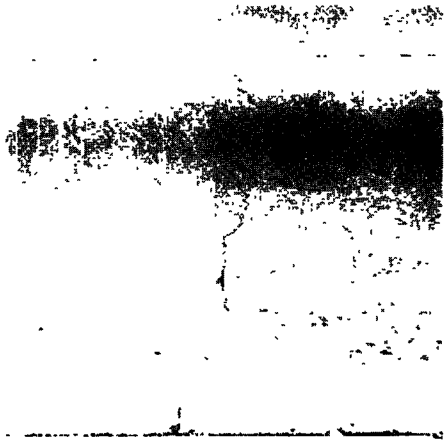

Figure 1. A prominent crack in the continuously-reinforced pavement.

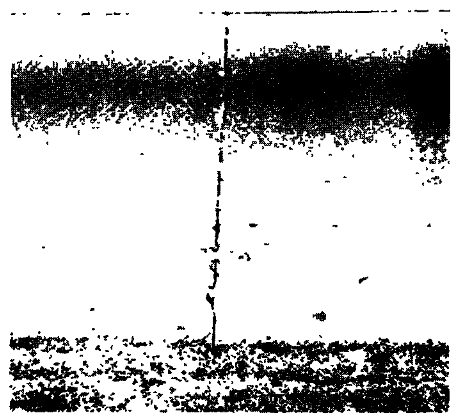

Figure 2. A typical joint in the non-reinforced pavement.

that of the random cracks. Average measured openings are given in Table 2. It will be noted that the joints were not closed as tightly in June 1957 as in June 1950. This finding may afford confirmation of the possibility that the reinforced section has increased permanently in length, as discussed in connection with longitudinal movement.

The condition of the continuously-reinforced pavement after eight years of service appears to be excellent. A comparison of its performance with other pavements of the same age and subjected to the same traffic is afforded by profilograph records of the reinforced section and adjacent portions of the non-reinforced jointed sections placed under the same contract. Throughout the length of the entire pavement there are a number of surface irregularities due to settlement of fills adjacent to culverts. Such portions have been eliminated from consideration.

Figure 3. One of a very few random cracks in the non-reinforced pavement.

Stanton included profilograms (1, Fig. 20) showing short sections of reinforced and non-reinforced pavement. The 10-ft profilograph used at that time (1950) has been superseded by one of 25-ft wheelbase. The shape of the trace produced by the two instruments is somewhat different and simple comparisons are difficult to make.

Short sections of profilograms obtained during the afternoon of June 25, 1957, are shown in Figure 4. These sections include those shown in Stanton's Figure 20. They are fairly representative of the entire reinforced and non-reinforced sections other than those affected by settlement at cross culverts. It will be noted that a moderate amount of faulting is indicated at a few of the joints in the non-reinforced section.

Faulting less than 0.1 in. cannot be

TABLE 1

SUMMARY OF PAVEMENT LONGITUDINAL MOVEMENT MEASUREMENTS

Date	Age (months)	Change in Length of Reinf Section (in.)	Change in Combined Width of Terminal Joints (in.)
May 1949	0	0	0
Dec. 1949	7	-0.50	-0.16
June 1950	13	+0.69	-1.22
Nov. 1950	18	-0.08	-1.72
Jan. 1951	20	-0.41	-0.99
Mar 1952	34	-0.09	-1.35
June 1957	97	+1.67	-3.62

measured accurately from profilograms and only those of greater magnitude are considered in the following discussion. Such faulting has developed only in the non-reinforced section lying east of the reinforced section. In this portion, faulting of 0.10 in. or more has developed in the outer (driving) lane at 17 percent of the joints. The average faulting at these joints is 0.14 in. This faulting and random cracks in 6 percent of the slabs are the only evidence of distress in the non-reinforced pavement.

TABLE 2

AVERAGE MEASURED CRACK OPENINGS

Date	Crack Opening (in.)
Dec. 1949	0.013
June 1950	0.003
Nov. 1950	0.009
Jan. 1951	0.014
Mar. 1952	0.009
June 1957	0.009

Individual opinions as to the relative riding comfort of the two types of pavement are not entirely consistent, probably because of the influence of settlements and because both pavements would be rated as "smooth". An effort has been made to develop a numerical measure of riding comfort from profilograms. Tentatively it has been concluded that noticeable discomfort is felt by automobile passengers only when profilograms show deviations in excess of 0.2 in. from a plane established by the moving wheelbase of the profilograph.

An expression termed the "profile index" is obtained by using a 0.2-in. blanking band on the profilogram, totaling all deviations in excess of this amount, and expressing the result in terms of inches per mile. A composite of personal impressions of many miles of both asphaltic and portland cement pavements indicates that a profile

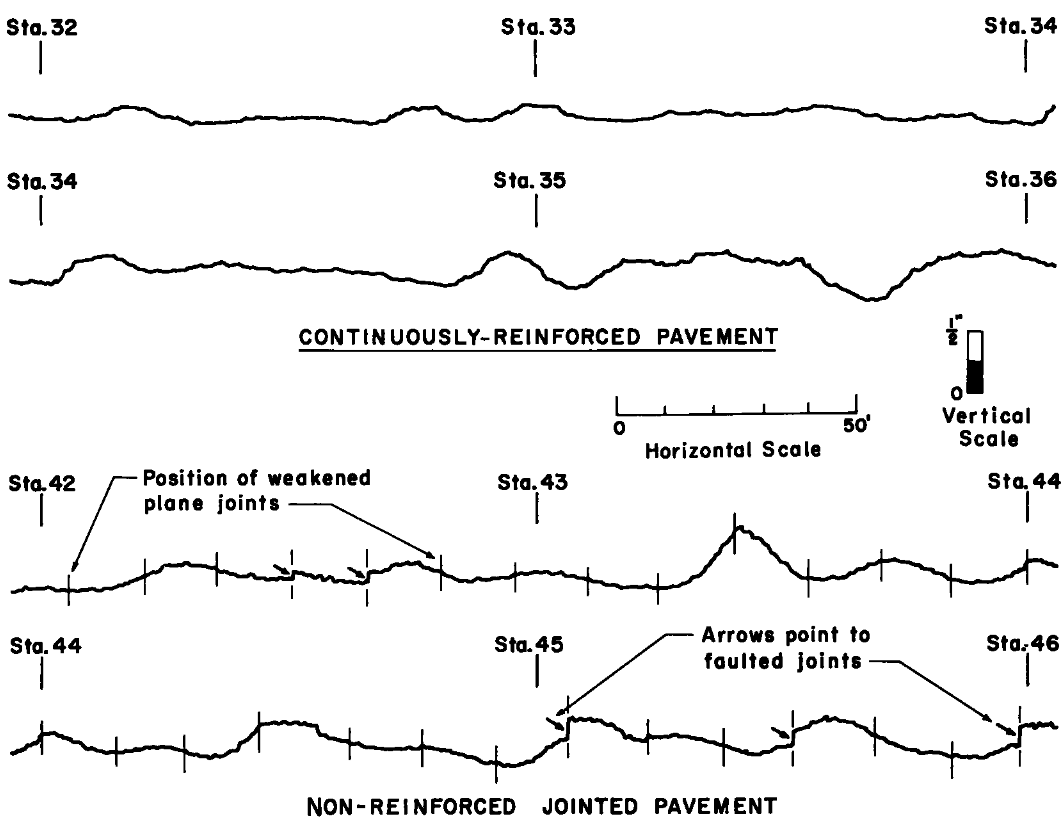

Figure 4. Profilograms of outer (driving) lane taken afternoon of June 25, 1957, temperature 103 F; Fairfield test section.

index of 10 in. or less is typical of well-finished new pavements or of older pavements that have remained exceptionally smooth. If the profile index is greater than 40, the pavement is rated as unquestionably rough and uncomfortable to passengers in automobiles. Present profile indices for the pavements in question, after eliminating irregularities due to local settlements, are 12.6 for both the reinforced and non-reinforced sections. The index does not take account of the moderate faults that have developed in portions of the non-reinforced pavement, some of which may be noticed by more sensitive passengers.

From the standpoint of riding comfort, it cannot be concluded that at present the reinforced section is definitely superior to the non-reinforced pavement. Structurally there is some evidence that the non-reinforced pavement is deteriorating at a slightly greater rate, but at present the difference in performance is not of sufficient magnitude to warrant a conclusion as to the economic value of the continuous reinforcement. It will be recognized, of course, that the comparison is made between pavements of equal thickness supported on presumably equal subgrades, not between pavements of comparable cost.

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Continuously-Reinforced Concrete Pavement in Pennsylvania

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● Pennsylvania's second continuously-reinforced concrete pavement was constructed on a section of Legislative Route 285 Section 6A (Traffic Route US 22), Berks County, during the period from May 8 to July 2, 1957, and from October 16 to 21. The experimental section, running from station 168+99 to station 277+05, is a 10,806-ft length of pavement, which was a portion of a 4.98-mi contract between Hamburg and Lenhartsville. The project was constructed under the supervision of the Pennsylvania Department of Highways in cooperation with the U. S. Bureau of Public Roads. Instrumentation was placed by Lehigh University and will be recorded and reported by that institution.

The objectives of this project can be stated as follows:

1. Study the effect of pavement thickness on service performance.
2. Study the effect of subbase thickness on service performance.
3. Determine the maximum stresses in the reinforcing steel at a crack, considering varying thicknesses of pavement.
4. Determine the effect of traffic upon stresses in the reinforcing steel.
5. Determine the thermal effects on the steel and concrete stresses in the pavements.
6. Determine the warping effects on the pavements.
7. Determine the magnitude of longitudinal movements of the pavements.
8. Determine the ultimate slab length.

SCOPE

The Hamburg project differs in several respects from the York project, which was constructed in 1956. The York pavement was placed in the fall of the year, whereas the Hamburg project was constructed in late spring and early summer. An exception to this is a short length of pavement placed in October.

The York road was designed for a uniform 9-in. pavement with a 6-in. depth of special subgrade material (subbase material) throughout, and 0.48 percent of longitudinal reinforcement steel (bar mats) by area, based on the total cross-sectional area of the pavement (1). At Hamburg, the steel was a uniform 0.50 percent throughout. Pavement thickness was varied to include 7-, 8-, and 9-in. thicknesses and special subgrade was placed in two depths: a minimum of 3 in. and a minimum of 6 in. These depths varied somewhat from the design, but in general, where the special subgrade was designed for 3 in. in the two eastbound lanes under each thickness of pavement, the corresponding thickness in the two westbound lanes was 6 in., and similarly, when the depth of special subgrade was 6 in. in the eastbound lanes, the depth in the westbound was 3 in.

The project is a four-lane, medial-separated roadway, with a maximum grade of 3 percent and two horizontal curves—one a $0^\circ 30'$ curve of 1,100 ft, the other a $1^\circ 00'$ curve of 3,224 ft. The limiting stations of each section are as follows:

Pavement Thickness (in.)	Stations	Length (ft)
7	168+99 to 188+97.5	1,998.5
8	188+97.5 to 232+64	4,366.5
9	232+64 to 277+05	4,441.0

Figure 1 shows the limiting stations of each section and the depth of special subgrade under each pavement thickness, as originally designed.

As mentioned previously, the bulk of the paving was placed in the spring and early summer. In June 1957, however, a serious slide condition developed near station 246, right, necessitating removal of large quantities of cut material (Fig. 2). As a result, a portion of the eastbound lanes from station 237+05 to 255+00 was not paved until October 1957. The pavement was opened to traffic on August 30, 1957, but eastbound traffic was detoured onto one of the westbound lanes, where one-way traffic was maintained from station 233 to 272. The entire roadway was opened to traffic on November 8, 1957.

PROCEDURE

Accurate records were kept throughout the project on every phase of the operation. The thousands of tests conducted make it impractical to list every value obtained, thus the data for the individual tests have been summarized and averaged for ease of analysis.

Table 1 shows the results of the gradings obtained at the batch plant throughout the duration of the project and indicates that all fine and coarse aggregates were ideally graded near the middle of the specification limits.

Tables 2, 3, and 4 list the physical tests conducted on the concrete in the field and other information pertinent to the placing of the concrete. The data have been separated

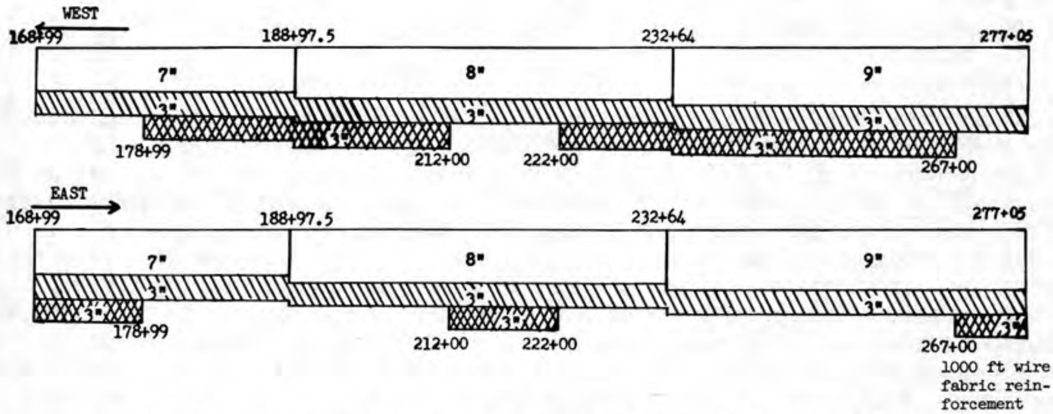

Figure 1. Pavement thickness and special subgrade (subbase) design.

Figure 2. Slide area near Station 246, eastbound lanes.

according to the three thicknesses of concrete and are combined into over all project averages in Table 5.

Concrete for the project was designed for a 2¼-in. slump and 3.5 percent air using 4.99 gal. of water per bag and 6.25 bags of cement per cubic yard. A small portion of high-early strength concrete was used in the right outside lane of the eastbound roadway between stations 237+05 and 255+00, so that the adjacent lane could be placed as soon as possible. In this section, the design called for a 1½-in. slump, 4.0 percent of air, 4.23 gal. of water per bag, and 8.02 bags of cement per cubic yard. Reinforcing steel throughout the project was placed at mid-depth in the pavement thickness. Tables 2, 3, and 4 indicate that the concrete complied closely with the design in every respect.

TABLE 1

AGGREGATE GRADATIONS

(a) Fine Aggregate, 61 Tests

Screen	No. 100	No. 50	No. 30	No. 16	No. 8	No. 4	3/8 in.
High	4.6	20.2	59.6	78.3	87.7	99.0	100
Low	2.0	11.6	45.8	62.1	81.1	96.1	100
Average	3.2	16.8	52.8	71.9	84.5	97.7	100
Spec. Limits	1-8	10-30	30-65	50-80	70-92	90-100	100

(b) 2B Coarse Aggregate, 86 Tests

Screen	No. 8	No. 4	1/2 in.	1 in.	1½ in.
High	3.4	8.3	59.5	99.2	100
Low	0.4	0.8	27.8	93.8	100
Average	1.2	3.5	44.1	95.9	100
Spec. Limits	0-5	0-10	25-60	90-100	100

(c) 3A Coarse Aggregate, 71 Tests

Screen	½ in.	1 in.	1½ in.	2 in.	2½ in.
High	3.4	8.6	66.3	98.2	100
Low	0.5	1.6	39.6	91.9	100
Average	1.5	4.9	49.0	95.7	100
Spec. Limits	0-5	0-15	35-70	90-100	100

CONCRETE PLACING

Concrete was placed in 23 individual runs, in accordance with the specifications of the Pennsylvania Highway Department. Figure 3 shows the location and limiting stations of the individual runs, and the daily air temperatures. The shortest run was 718 ft, the longest 3,953 ft, and the average 1,964 ft per day. (One run of 76 ft to complete a section was not included in the average.) Generally the concrete was consistent in uniformity and was placed on damp subgrade and struck off at half-depth. Vibrators were used along the longitudinal joints; the outside edges were spaded. After placing the steel, the remainder of the concrete was placed and spread to the proper depth. The spreader was followed by the finisher, bull-float, and hand finishers. At this point the pavement was marked off with a chain and station numbers were imprinted in the concrete. When sufficiently set, Kraft-type curing paper was placed on the concrete, which remained covered for a minimum of 72 hr of curing time. No expansion joints were constructed in the continuously-reinforced sections. However, at the end of a day's run, a split wooden bulkhead was used, with the steel protruding a minimum of 5 ft. At the beginning of the next day's run, the bulkhead was removed and concrete was placed immediately against the pavement placed the previous day.

TABLE 2

PHYSICAL TESTS ON CONCRETE, 7-IN. PAVEMENT; STA. 168+99—188+97.5

Lane	Weather	Air Temp. (°F)	Tests	Depth Sp. Subgrade (in.)	Depth Reinf. (in.)	Slump (in.)	Air Entr. (%)	Mod. of Rupture (lb)	Water per Bag (gal)
EB, out	Fair	73	No.	40	9	4	4	1[1]	7
			High	11¾	3⅝	3	4.6	597	5.33
			Low	5	3	1¾	3.7	597	4.65
			Avg.	-	3.3	2¼	4.1	597	4.84
EB, in	Fair	67	No.	39	6	2	2	1[2]	2
			High	3½	3½	2¼	4.2	627	5.04
			Low	7	3¼	2	3.5	627	4.98
			Avg.	-	3.5	2	3.9	627	5.01
WB, out	Partly Cloudy	73	No.	40	6	6	5	2[3]	8
			High	9¼	4	3	5.2	851	5.33
			Low	7	3¼	2	3.7	822	4.74
			Avg.	-	3.5	2½	4.2	837	5.10
WB, in	Partly Cloudy	79	No.	40	7	4	4	0	10
			High	9	3¾	2¾	4.3		5.40
			Low	3	3¼	2¼	2.8		5.07
			Avg.	-	3.4	2½	3.6		5.22

[1] At 9 days.
[2] At 12 days.
[3] At 10 days.

TABLE 3

PHYSICAL TESTS ON CONCRETE, 8-IN. PAVEMENT; STA. 188+97.5—232+64

Lane	Weather	Air Temp. (°F)	Tests	Depth Sp. Subgrade (in.)	Depth Reinf. (in.)	Slump (in.)	Air Entr. (%)	Mod. of Rupture (lb)	Water per Bag (gal)
EB, out	Fair and Windy	85	No.	85	21	7	7	1[1]	13
			High	10½	4	2½	4.5	647	4.89
			Low	4	3¾	1¾	3.2	647	4.65
			Avg.		3.9	2¼	3.9	647	4.81
EB, in	Fair	73	No.	88	18	8	8	1[2]	7
			High	10	4¼	2½	4.5	605	5.16
			Low	3	3½	1⅞	3.1	605	4.98
			Avg.		4.0	2	3.7	605	5.03
WB, out	Fair	74	No.	85	14	7	8	2	14
			High	10½	4¼	2½	4.1	851[2]	5.49
			Low	3½	3½	2	3.0	787[1]	4.82
			Avg.		4.0	2½	3.5	819[2]	5.16
WB, in	Cloudy	83	No.	86	18	8	8	3	16
			High	10	4¼	2½	4.5	692[1]	5.41
			Low	4¼	3¼	2⅛	2.9	532[2]	4.90
			Avg.		4.0	2¼	3.8	604[1]	5.09

[1] At 11 days.
[2] At 10 days.

TABLE 4

PHYSICAL TESTS ON CONCRETE, 9-IN. PAVEMENT; STA. 232+64-277+05

Lane	Weather	Air Temp. (°F)	Tests	Depth Sp. Subgrade (in.)	Depth Reinf. (in.)	Slump (in.)	Air Entr. (%)	Mod. of Rupture[1] (lb)	Water per Bag (gal)
EB, out	Cloudy	77	No.	54	11	6	6	2	5
			High	9½	4½	2¼	4.5	840	4.99
			Low	3½	4	1¾	3.1	647	4.73
			Avg.		4.3	2	3.9	744	4.82
EB, in	Cloudy and Windy	71	No.	54	9	5	5	1	5
			High	12	4½	2⅛	4.5	607	5.17
			Low	3	4¼	1¾	3.6	564	4.91
			Avg.		4.5	2	3.9	586	5.01
WB, out	Fair	84	No.	91	14	10	10	3	16
			High	10¾	4½	2¾	4.2	731	5.33
			Low	3¼	4	2⅛	2.8	598	4.98
			Avg.		4.4	2½	3.4	659	5.12
WB, in	Cloudy and Windy	77	No.	89	17	10	10	1	16
			High	9½	4¾	2½	4.0	492	5.34
			Low	3	3½	1¾	3.0	492	4.74
			Avg.		4.4	2	3.6	492	5.08

[1] At 10 days.

At the beginning and end of the experimental section (at stations 168+99 and 277+05) 24-ft, finger-type bridge expansion joints were installed between the continuously-reinforced pavement and the standard 10-in. pavement. Three inches were allowed for expansion at each of these joints. To remove water and debris which would run down through the joints, concrete boxes, outletting through the shoulder into drainage ditches, were constructed under these joints. The two adjacent lanes were tied together longitudinally by a formed keyway with hook-bolt dowels spaced 5 ft apart.

Additional reinforcement was inserted in the concrete at the end of each day's run to tie into the succeeding run. In the eastbound lanes, and the outside westbound lane, seven ungreased, 1 by 18-in. steel dowels were placed between the horizontal reinforcing bars protruding through the bulkhead. Dowels at station 237+05, eastbound outside, were coated with graphite. At station 178+52, westbound inside, seven graphite-greased dowels were placed. At station 204+50, westbound inside, five graphited dowels were used. As a comparison, no dowels were used in the bulk-

TABLE 5

PHYSICAL TESTS ON CONCRETE; PROJECT AVERAGES

Tests	Depth Sp. Subgrade (in.)	Depth Reinf. (in.)	Slump (in.)	Air Entr. (%)	Mod. of Rupture (lb)	Water Per Bag (gal)
No.	791	150	77	77	18	119
High	12	4¾	3	5.2	851	5.49
Low	3	3	1¾	2.8	492	4.65
Avg.	7	4.0	2¼	3.7	676	5.05

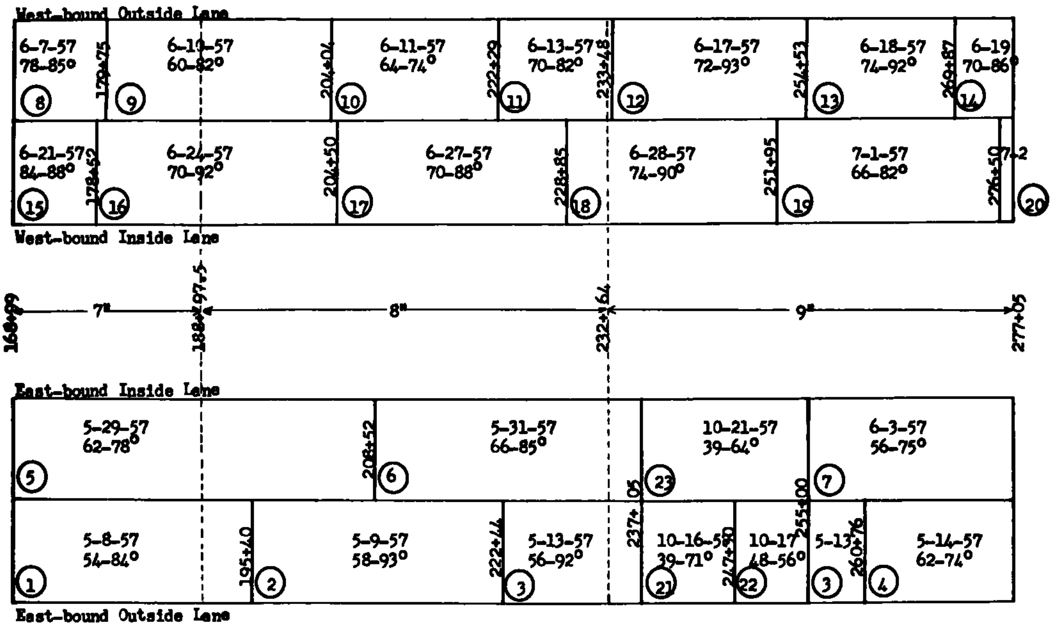

Figure 3. Limiting stations and air temperatures of individual pours.

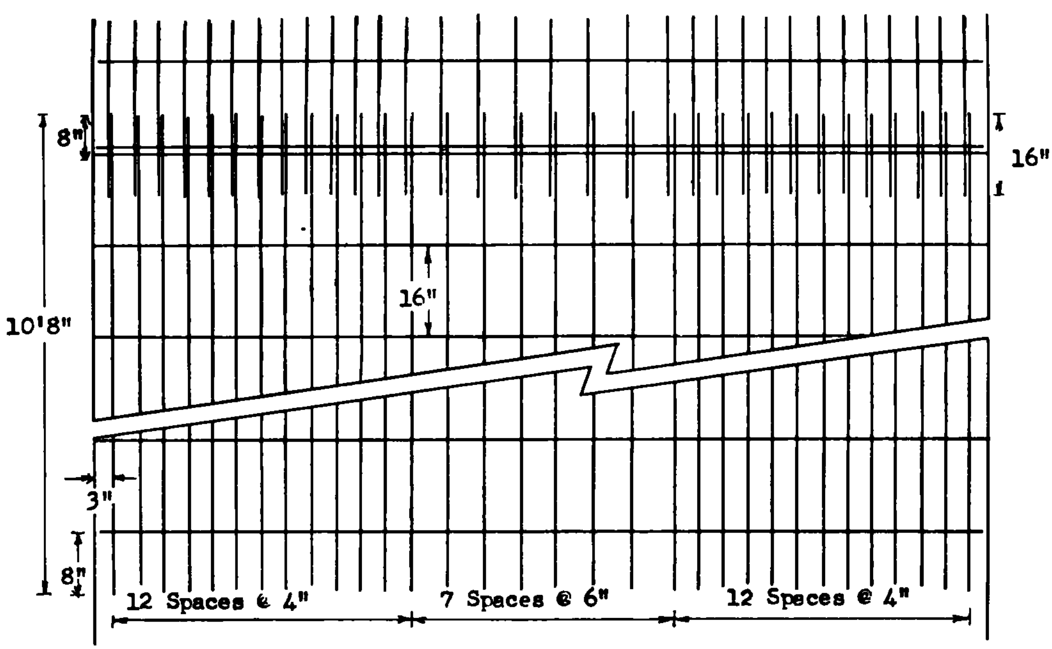

Figure 4. Wire mesh reinforcement, Type K-9.

heads at stations 228+85, 251+92, and 276+50, westbound inside.

Steel reinforcing for this project was Type L-7, L-8, and L-9 bar mats, for the 7-, 8-, and 9-in. pavements, respectively. Two 16-ft by 6-ft 2-in. bar mats were used per 12-ft width pavement lane, overlapped 12 in. longitudinally and 8 in. transversely. The bar mats were fabricated with No. 5 longitudinal bars and No. 3 transverse bars, and conformed to the Department's specifications for hard-grade steel. Reinforcement started and stopped 3 in. from the limits of continuous construction.

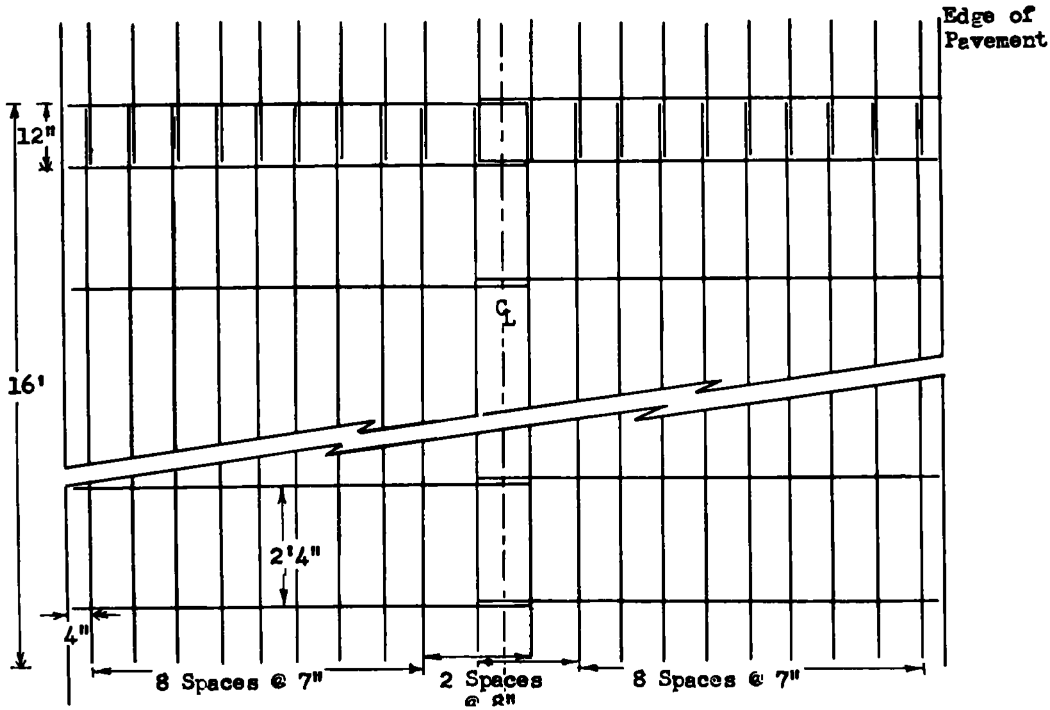

Figure 5. Bar mat reinforcement, Type L-9.

A wire mesh type of reinforcement was used for 1,000 ft in the two eastbound lanes between stations 267+07 and 277+05, right. These were 10 ft 8 in. by 11 ft 8 in. and were overlapped 16 in. longitudinally. They were fabricated transversely with No. 0 wires and longitudinally with ½-in. wires. Steel throughout the project was designed at 0.5 percent of the total cross-sectional area of the pavement. A sketch of the wire mesh reinforcement is shown in Figure 4 and of the bar mats in Figure 5, 6, and 7. Data from physical tests on the steel are given in Table 6.

CRACK FREQUENCY SURVEY

An extensive and comprehensive crack frequency and width survey has been established on this project to determine the pattern of crack development. This report covers the observations of the first three months. It is realized that this represents the early stages of cracking and that the resulting crack pattern in later years may be completely different.

The number of cracks was first counted at seven days. Surveys were made thereafter at 14 and 21 days, and at one, two and three months. It is anticipated that an annual survey will be conducted hereafter, to include the entire length of the experimental concrete slab.

Crack width measurements are not as extensive in scope. In general, three limited areas from each pavement thickness, in each lane, have been selected for measurement. These are located in the first 500 ft of each section, the last 500 ft, and 400 ft near the middle. In selecting these locations, consideration was given to the thickness of special subgrade material where an attempt was made to select contrasting depths.

Crack widths within these 400- and 500-ft sections are being measured by microscope to the nearest 0.002 in. Within these larger sections, selected 100-ft lengths have been chosen and brass plugs have been installed in the pavement at every crack within the 100 ft. The plugs are 10 in. apart and 1 ft from the edge of the pavement. Measurements will be made with an invar-type gage, and a correlation will be made between this measurement and that obtained with a microscope. When the section of pavement was

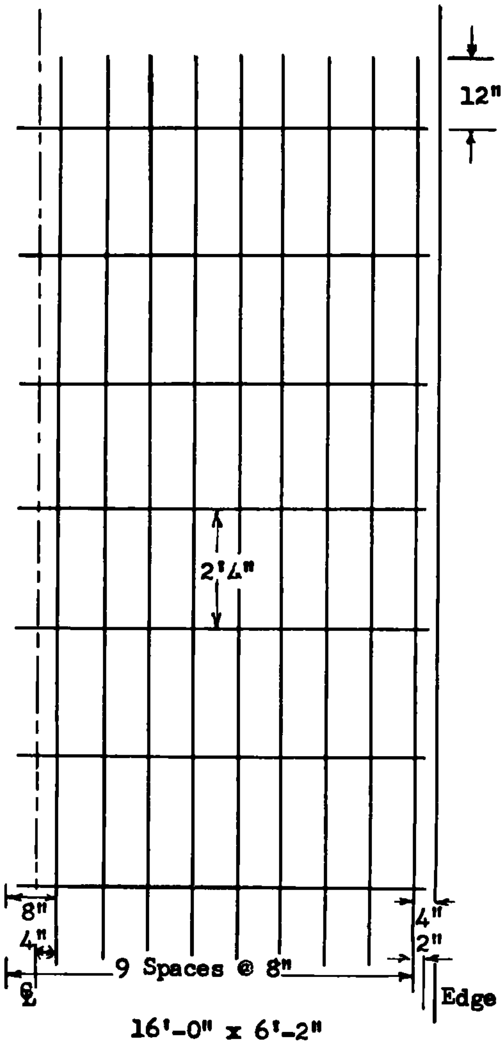

Figure 6. Bar mat reinforcement, Type L-8.

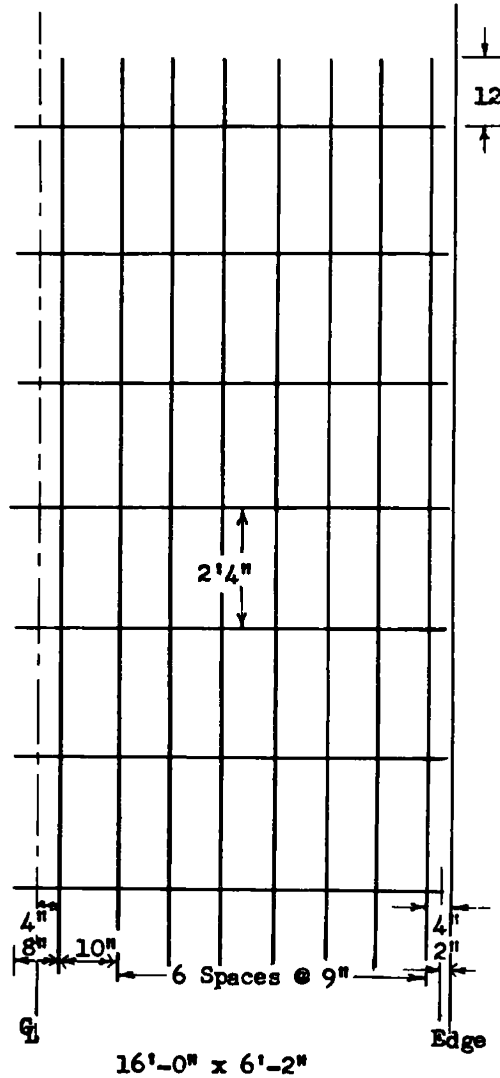

Figure 7. Bar mat reinforcement, Type L-7.

placed in October, a 100-ft section was selected in each of the eastbound lanes and brass plugs were installed in the fresh concrete every 10 in. In this way, measurements were obtained before and after cracking, thus permitting an accurate comparison

TABLE 6

PHYSICAL CHARACTERISTICS OF REINFORCING STEEL

Reinf. Type	Members	Spacing (in.)	Diam. (in.)	Elong. (%)	Area (sq. in.)	Yield Stress (psi)	Tens. Str. (psi)
K-9 Wire mesh	Main	4	½	-	0.20	89,000	95,000
	Sec.	16	5/16	-	0.0767	-	71,707
L-7 Bar mat	Main	9	⅝	-	0.31	67,000	122,581
	Sec.	28	⅜	-	0.11	-	116,364
L-8 Bar mat	Main	8	⅝	10.9	0.31	67,000	110,645
	Sec.	28	⅜	14.1	0.11	-	101,818
L-9 Bar mat	Main	7	⅝	12.5	0.31	67,000	116,452
	Sec.	28	⅜	12.5	0.11	-	129,091

of the two methods of crack width measurement.

Measurements to date have included 2- and 3-month readings with microscope and 3-month readings with gage. Another survey will be made in 6 months and semi-annually thereafter.

The width survey data are presented in several different aspects. Table 7 lists the cracks according to the individual day's runs. Table 8 gives the same information with respect to pavement thickness. Table 9 gives the locations of the selected width measurement sections with the depth of special subgrade under each, and the number and width of cracks in each section.

At present not enough readings have been taken with the gage to establish definite width readings in the selected 100-ft sections. However, the locations of these sections, and other related data, are presented in Table 10.

TABLE 7

CRACK FREQUENCY SURVEY, BY INDIVIDUAL POURS[1]

Section	Length of Pour (ft)	Temp. Range (°F)	Number of Cracks 7 Days	14 Days	21 Days	1 Mo.	2 Mo.	3 Mo.
1	2,641	54-84	127	230	230	236	239	239
2	2,704	58-93	173	238	241	251	255	256
3	2,037	56-92	103	106	107	111	119	119
4	1,629	62-74	44	47	47	58	69	69
5	3,953	62-78	171	246	259	262	265	270
6	2,853	66-85	101	101	186	203	213	218
7	2,205	56-75	76	90	91	91	91	95
8	1,076	78-85	15	46	56	63	70	95
9	2,429	60-82	73	181	189	197	219	219
10	1,825	64-74	112	137	139	147[2]	151[2]	171
11	1,119	70-82	71	79	79[2]	95[2]	96[2]	118
12	2,105	72-93	213	219	230	235	238	248
13	1,534	74-92	151	152	170	179	179	185
14	718	70-86	51	52	63	65	69	71
15	953	84-88	18	44	57	69	72	82
16	2,598	70-92	109	161	183	194	200	219
17	2,435	70-88	178	[2]	[2]	[2]	203	229
18	2,307	74-90	146	206	212	221	239	241
19	2,458	66-82	102	152	152	160	202	234
20	76	-	0	0	0	0	0	0
21	1,025	39-71	32	44	55	57	-	-
22	770	48-56	10	24	31	32	-	-
23	1,795	39-64	74	91	100	102	-	-

[1] See Fig. 2 for stations of each pour.
[2] Partially covered with earth.

A total of 3,569 cracks has been recorded in 43,224 lineal feet of pavement, based on a 3-month survey. Computed on an over-all length this amounts to one crack every 12.1 ft. The true picture is quite different from this, however, as will be discussed later.

Sixty percent of the cracks recorded thus far have occurred within the first seven days; 79 percent occurred within 14 days; 85 percent within 21 days; and 90 percent of the total number of present cracks had occurred at one month.

According to Table 8, there are more cracks per 100 ft in the thinner sections of concrete. It is expected that this pattern will hold true during the life of the pavement. In the 7-in. sections there are 9.0 cracks per 100 ft; in the 8-in., 8.5; and in the 9-in., 8.2 cracks per 100 ft. There is a small increase in the number of cracks in the outside lanes as compared to those in the inside lanes.

It is perhaps too early to try to correlate the number of cracks and crack widths with the corresponding depth of special subgrade as shown in Table 9; but at present, an unusual condition seems to prevail. In most instances, there are fewer cracks and the widths are smaller in the locations where the special subgrade material is thinnest.

The data in Figure 8 show some interesting patterns. This graph indicates the number of cracks per 100 ft, continuously, for all four lanes from the beginning of the experimental section to the end. It is interesting to note the variance in the number of cracks per 100 ft at the beginning and the end of each day's run. This condition is most noticeable for the 7-day count and is a little less obvious at the 1-month count. Considering the 500 ft at each end of all pours, the number of cracks per 100 ft for the 7-day count at the beginning is 6.3; at the end, 3.1.

The same data for the 1-month count reveal that at the beginning of all sections there is an average of 8.8 cracks per 100 ft and at the end the average is 6.2 cracks per 100 ft. Thus there is a growing

TABLE 8

CRACK FREQUENCY SURVEY, BY PAVEMENT THICKNESS

Pavement Depth (in.)	Lane	Length (ft)	Number of Cracks 7 Days	14 Days	21 Days	1 Mo.	2 Mo.	3 Mo.
7	EB, out	1998.5	111	183	183	188	191	191
	EB, in	1998.5	127	150	159	159	161	163
	WB, out	1998.5	56	114	128	140	160	185
	WB, in	1998.5	89	126	143	161	167	183
8	EB, out	4366.5	256	355	359	371	378	379
	EB, in	4366.5	145	197	270	288	296	304
	WB, out	4366.5	213	329	335	359[1]	373[1]	413
	WB, in	4366.5	227	299[1]	319[1]	330[1]	349	390
9	EB, out	4441.0	122	151	169	186	202	202
	EB, in	4441.0	150	181	207	211	214	218
	WB, out	4441.0	418	426	465	484	491	509
	WB, in	4441.0	235	321	327	339	399	432

[1] Partially covered with earth.

tendency for the cracks to level off at a more uniform rate. It might be mentioned that the over-all crack average at one month was 7.2 cracks per 100 ft. It is thought that the crack pattern, as described, can be attributed to the fact that as the previous day's concrete is curing it pulls the freshly placed concrete, resulting in many cracks in the first several hundred feet. As more fresh concrete is placed, the bulk is such that the effect is diminished and fewer cracks appear. This pattern substantiates a similar condition noted by Van Breemen (2) who says, "This method of construction naturally involves the attachment together of sections of pavement having, at the time of attachment, materially different tensile strengths."

One exception to this pattern is a stretch running from station 235 to station 276 in the left outside lane, involving a period of three days when the air temperature ranged from 72F to 93F daily. Here, the crack pattern is quite uniformly high throughout, averaging 9.7 cracks per 100 ft at 7 days and 11.3 cracks per 100 ft at one month. It is believed that the high temperatures were the cause of this higher-than-usual crack pattern.

The average crack width as established by the section under survey is approximately 0.009 in. There are, however, some serious cracks which have developed. These are principally at stations 228+85, westbound inside (Figure 9); 254+46, westbound outside (Figures 10 and 11); 233+51, westbound outside; and 267+96 westbound outside lane in 8-in. and 9-in. concrete with special subgrade of 6- to 9-in. depths. It is perhaps significant that two of these cracks occurred in the same day's run, 3 ft from the beginning and 7 ft from the end. This particular section was placed on one of the hot days (72F - 93F). Another crack also occurred 6 ft from the beginning of a day's run. All of these cracks are severely spalled and in some spots loss of small pieces of concrete has occurred.

TABLE 9

THREE-MONTH MICROSCOPE CRACK WIDTH SURVEY

Station	Lane	Pavement Depth (in.)	Avg. Depth Sp. Subgrade (in.)	No. Cracks	Crack Width (in.) High	Low	Avg.
169+00-174+00	EB, out	7	8½	48	0.016	0.002	0.007
	EB, in		7½	47	0.032	0.004	0.009
	WB, out		7½	42	0.012	0.004	0.007
	WB, in		4½	48	0.012	0.002	0.007
177+00-181+00	EB, out	7	7 -11	41	0.032	0.002	0.010
	EB, in		8	34	0.020	0.002	0.012
	WB, out		8	41	0.016	0.004	0.007
	WB, in		6	30	0.016	0.002	0.008
183+97-188+97	EB, out	7	6½	42	0.020	0.002	0.007
	EB, in		7½	32	0.024	0.004	0.009
	WB, out		8	53	0.012	0.002	0.007
	WB, in		7½	51	0.016	0.002	0.007
188+97-193+97	EB, out	8	6½	36	0.020	0.004	0.009
	EB, in		6½	27	0.020	0.002	0.008
199+50-204+50	WB, out	8	9	46	0.014	0.002	0.008
	WB, in		7½	36	0.012	0.002	0.008
213+00-217+00	EB, out	8	8½	32	0.020	0.004	0.009
	EB, in		8	40	0.028	0.004	0.011
217+00-221+00	WB, out	8	4	36	0.014	0.004	0.008
	WB, in		5	35	0.012	0.004	0.008
227+64-232+64	EB, out	8	4	36	0.012	0.004	0.008
	EB, in		3	36	0.012	0.002	0.008
	WB, out		7½	54	0.016	0.004	0.008
	WB, in		7	47	0.197	0.004	0.013
232+64-237+64	WB, out	9	8	53	0.039	0.006	0.011
	WB, in		7	55	0.020	0.002	0.009
240+00-241+00	EB, out		5½	5			
	EB, in		5½	5			
262+00-266+00	EB, out	9	4½	27	0.012	0.002	0.006
	EB, in		3½	22	0.012	0.004	0.009
	WB, out		8	51	0.014	0.002	0.009
	WB, in		7½	36	0.016	0.004	0.012
272+05-277+05	EB, out	9	8	11	0.016	0.004	0.010
	EB, in		7 -11	16	0.012	0.004	0.009
	WB, out		4	49	0.012	0.002	0.008
	WB, in		4½	44	0.012	0.001	0.008

These cracks appeared approximately two months after completion of the paving. As the cooler weather of late summer and fall approached, the cracks began to widen, and became quite serious. When the last crack width readings were recorded (at the 3-month survey, the weather was still quite warm and the crack widths averaged 0.12 to 0.20 in. With the advent of cold weather, these cracks assumed widths of 0.25 to 0.35 in. When the cracks reached this condition, a decision was made to remove 10-in. cores from the cracked areas to determine the cause of the cracking. These cores revealed that the wide cracks occurred either because the bar mats failed to meet or because of insufficient overlap of the bar mats.

Figure 12 shows a core taken on the crack 5 ft from the beginning of a day's run. The bar protruding from the right half of the core terminates in the core half on the left. The bar mats probably failed to meet by an estimated ½ in. Figure 13 is another view of the same crack.

Figures 14 and 15 show the crack at station 254+46, which was 10 ft from the end of a day's run. No reinforcing steel was found in one-half of this core, indicating that the steel failed to meet by at least 5 in., and perhaps considerably more. The left half of the core shows one bar of steel, the right half a lack of steel; the steel bar is visible in the pavement.

TABLE 10

THREE MONTH CRACK WIDTH MEASUREMENTS
INVAR GAUGE WITH BRASS PLUGS
(100 FOOT SECTIONS)

Stations	Lane	Pavement Depth (in.)	Depth Sp. Subgrade (in.)	No. Cracks
173-174	EB, out	7	9	12
178-179	EB, out	7	10	11
187-188	EB, out	7	7	9
190-191	EB, out	8	6	8
216-217	EB, out	8	8	8
231-232	EB, out	8	4	10
240-241	EB, out	9	5½	5
262-263	EB, out	9	4½	8
274-275	EB, out	9	8½	4
173-174	EB, in	7	8	13
178-179	EB, in	7	8	9
187-188	EB, in	7	7½	6
190-191	EB, in	8	6½	6
216-217	EB, in	8	7½	11
228-229	EB, in	8	3	9
240-241	EB, in	9	5½	5
262-263	EB, in	9	3½	8
275-276	EB, in	9	11	4
172-173	WB, out	7	8	9
180-181	WB, out	7	8	14
185-186	WB, out	7	7	11
201-202	WB, out	8	9	10
217-218	WB, out	8	4	9
229-230	WB, out	8	7½	8
234-235	WB, out	9	7½	12
263-264	WB, out	9	8	12
275-276	WB, out	9	4	9
170-171	WB, in	7	5½	12
180-181	WB, in	7	8	10
185-186	WB, in	7	7½	10
201-202	WB, in	8	7½	7
220-221	WB, in	8	5	10
228-229	WB, in	8	7	3
234-235	WB, in	9	6½	11
263-264	WB, in	9	7½	9
274-275	WB, in	9	4	7

The crack at station 267+96 is shown in Figures 16 and 17. Two bars of reinforcing steel are visible in the core and in the pavement. The crack is 200 ft from the end of a day's run. The core indicated that the crack occurred at an overlap, but it was not possible to determine the length of the overlap, except that one half of the core had one steel bar and the other half had two bars, indicating that the overlap could have ranged from a minimum of 5 in. to the prescribed 12 in.

Another erratic cracking pattern has occurred at stations 223 to 226 in the westbound outside lane and in the outside 5 ft of the lane. The cracks describe an arc pattern by starting and ending at the pavement edge and extending into the lane as much as 5 ft. The average length of the cracks is approximately 12 ft. It is assumed this was caused by loading the pavement with earth and by the use of heavy earth-moving equipment in this area when additional material was removed from a cut after the pavement was placed.

ROUGHNESS SURVEY

The Department's profilometer was used shortly before the road was opened to traffic to determine the roughness or smoothness of the concrete surface. The instrument operates on the mechanical principle of measuring the vertical movements of a wheel suspension with respect to its supported frame. The results of this profilometer survey are as follows:

Date	Stations	Lane	Length (ft)	Meas. Total (in.)	Index (in./mi)
8-27-57	169+50-232+50	EB, out	6,300	30	25.21
8-27-57	169+00-277+00	WB, out	10,800	57	27.80

The profilometer survey will be conducted annually or semi-annually; in addition, a traffic count will be conducted and this information made available in future reports.

COSTS

Pavement unit bids for the various thicknesses of concrete were as follows:

Type of Pavement	Unit Price	Saving (%)
7-in. continuously reinforced	$4.45	+ 15.2
8-in. continuously reinforced	$4.75	+ 9.5
9-in. continuously reinforced	$5.35	- 1.9
10-in. standard reinforced	$5.25	-

INSTRUMENTATION

Four similarly constructed, instrumented test sections were installed in the project. One was located at station 273+00, eastbound outside lane (in the wire mesh section), and three were placed at stations 184+00, 199+50, and 258+00, all in the westbound outside lane. The purpose of these instrumented sections was to induce a crack in the pavement and to have gages properly located so that the strains in the steel and the strains in the pavement itself could be measured. In addition the width of the induced crack could be measured, and temperatures obtained in the concrete and subbase of the test sections. The crack was formed by installing a 4-in. piece of corrugated metal transversely in the center of the instrumented bar mat. The mat itself was placed on chairs so that it was at mid-depth in the concrete pavement. The standard bar mats were carefully placed and overlapped after the first layer of concrete was

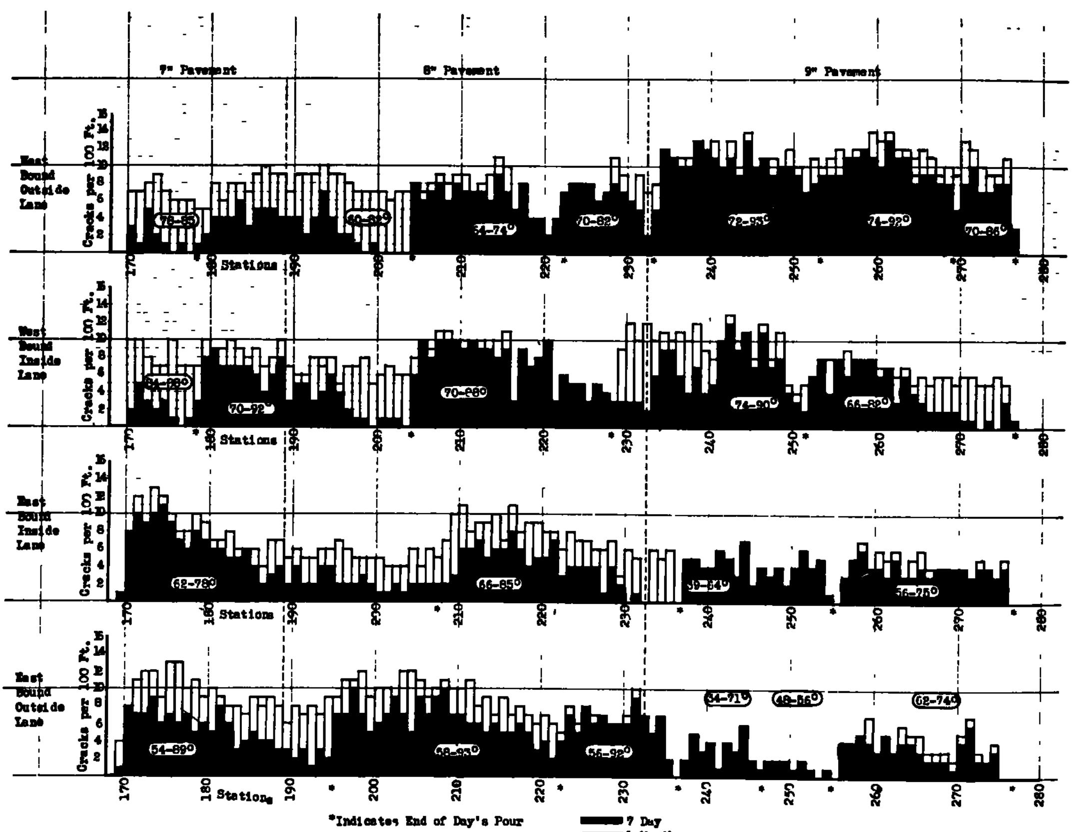

Figure 8. Crack frequency survey.

Figure 9. Crack at station 228+85.

Figure 10. Crack at station 254+46.

Figure 11. Crack at station 254+46.

Figure 12. Separate halves of core at station 228+85.

Figure 13. Location of core removed from pavement at station 228+85.

Figure 14. Core from pavement crack at station 254+46.

Figure 15. Close-up of core shown in Figure 14.

Figure 16. Pavement crack and core at station 267+96.

placed on both sides of the instrumented bar mat. The paver moved back to place the second layer of concrete and then carefully moved through the instrumented section, placing the concrete in small portions over the entire test section to obtain uniformity. The wires connected to the gages were led out through the special subgrade in a conduit terminating in an instrument panel located some distance off the shoulder. It is at this box that all measurements are obtained through instrument readings.

Six bars of the fabricated bar mat were instrumented with Bakelite SR-4 strain gages mounted on the reinforcing steel. These were of the AB-3 type with nominal $^{13}/_{16}$-in. length. Five temperature gages were installed in the pavement and in the special subgrade. These were placed 1 in. into the special subgrade material, 1 in. from the bottom of the slab, and 1 in. from the top of the slab. A typical location of these strain and temperature gages is shown in Figure 18.

Figure 19 shows the location of the Whittemore gage plugs used to measure the crack width. These are located 10 in. apart, spanning the preformed crack, and were set in the fresh concrete, one set near the shoulder and the other set near the longitudinal joint.

Plugs have been installed 100 ft apart on the westbound outside lane to measure the over-all movement of the pavement. Also, monuments have been erected at the quarter points of the test section to measure absolute movement at these points on all four lanes.

Data obtained from the described instrumentation are to be assembled and reported by Lehigh University (see paper by I. J. Taylor and W. J. Eney, this Bulletin).

Figure 17. Close-up of core shown in Figure 16.

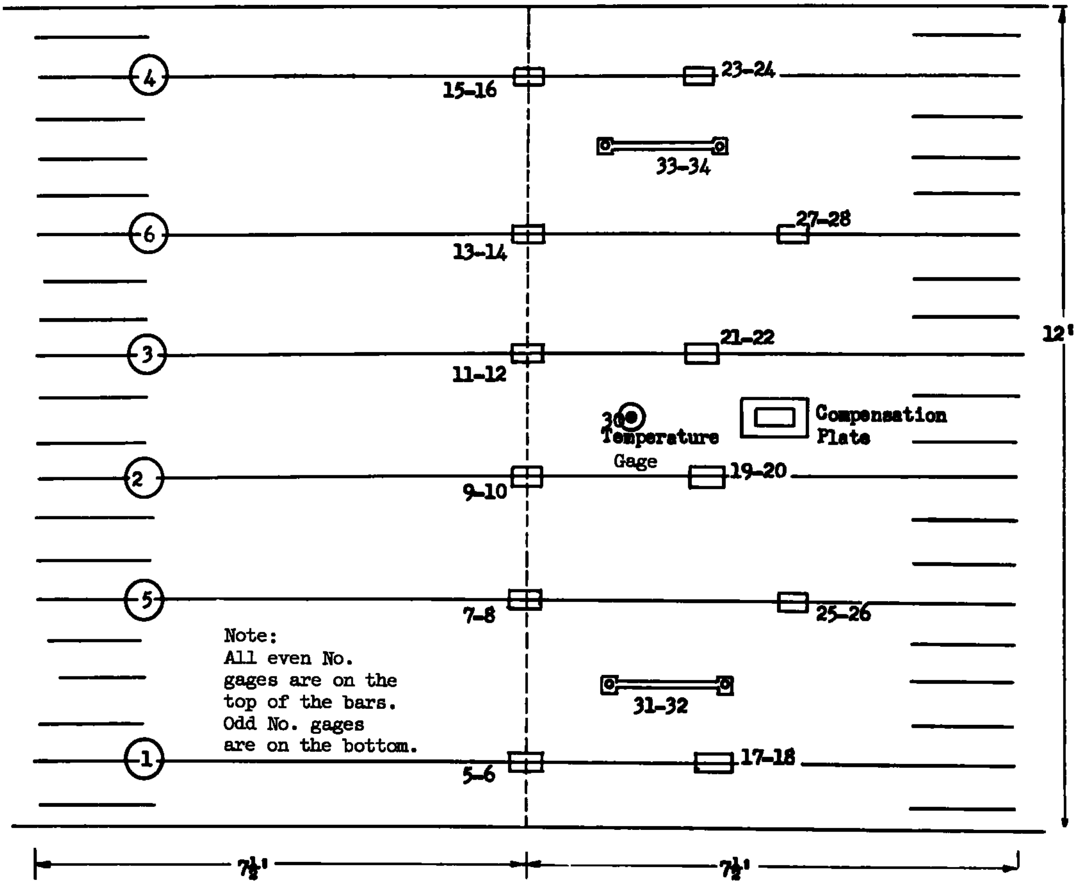

Figure 18. Typical location of strain and temperature gages.

SOIL DATA

At present, supplemental specifications covering embankment, subgrade, and stabilized shoulders are being applied on all major construction. These specifications, among other requirements, provide for moisture control and density measurements in the field.

These supplemental specifications for embankment and subgrade were not incorporated in the proposal for this project; however, the contractor agreed to comply with all of their provisions. The cooperation of the contractor, together with the effort of the Department's soil engineering personnel, made it possible to obtain significant data. The following information is available for completing an evaluation of the experiment:

1. A soil profile showing (a) the type of rock, granular material, or soil as it

TABLE 11

PHYSICAL CHARACTERISTICS[1] OF SOILS

AASHO Group	Liquid Limit	Plasticity Index	Aggregate (%)	Coarse Sand (%)	Fine Sand (%)	Silt (%)	Clay (%)	Optimum Moisture (%)	Maximum Density (pcf)
A-1	28	6	54	17	6	13	10		
A-2	31	9	51	15	10	10	14	15.4	116.0
A-4	29	7	29	16	14	17	24	16.2	112.1
A-6	35	12	13	11	16	30	30	17.0	113.0

[1] Averages.

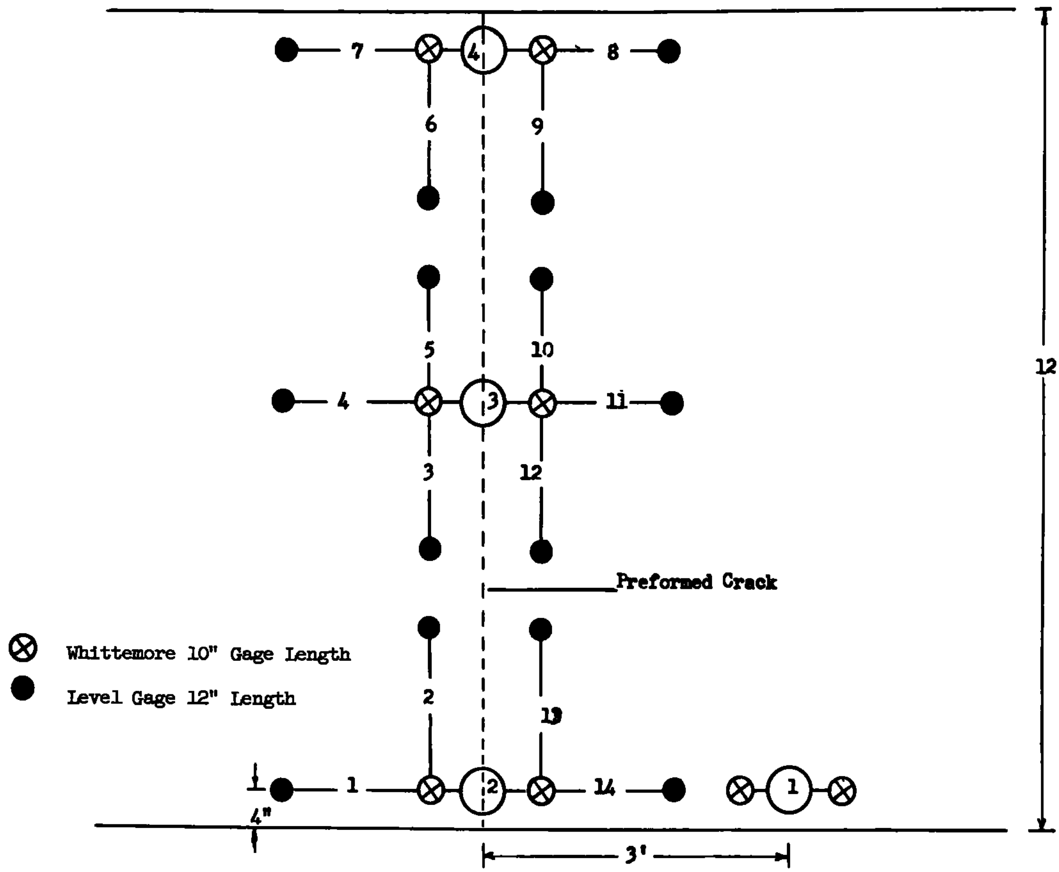

Figure 19. Typical location of Whittemore and warping gage plugs.

originally existed; (b) the general movement of material from excavation to embankment; (c) the location of density determinations by number, station, and elevation.

2. Laboratory reports showing the physical characteristics of special subgrade and stabilized shoulder materials; AASHO classification of soils and granular materials with their moisture-density relationships and California bearing ratio tests of representative samples of these materials.

3. Density determinations made in the field on all embankment, subgrade, special subgrade, and stabilized shoulder materials where they could be performed by the sand cone method.

4. Daily project records showing (a) type of equipment used, (b) weather data, and (c) comments by soils engineer on embankment materials.

5. Ground water fluctuation, measured by means of piezometers.

Embankments and subgrade were composed predominantly of shales and shaley sandy clay loams of the A-2 group (stone or gravel with sand and silt), with a small amount of the A-1 group (stone or gravel and sand with or without fines). Fair quantities of the A-4 group (silt and sand

TABLE 12

PHYSICAL CHARACTERISTICS OF SPECIAL SUBGRADE MATERIAL

Screen	Average Gradation	Spec. Limits	
		Grad. A	Grad. B
3 in.	-	100	-
2½ in.	100	-	100
1½ in.	95	40-90	-
¾ in.	74	-	40-90
No. 4	37	-	-
No. 10	25	15-50	15-70
No. 40	13	-	-
No. 200	9	0-15	0-15
Liquid Limit	22	30 Max.	30 Max.
Plasticity Index	3	6 Max.	6 Max.

with or without coarse fragments) and the A-6 group (clay) were present.

The contractor and Department personnel gained experience with these materials on the project prior to construction of the experimental section. As could be expected with these predominantly granular materials, good compaction and stability were obtained. Table 11 classifies these groups of soils and shows the average values obtained for each group as determined by laboratory tests on samples submitted from the field.

Special subgrade (subbase) material serves as an insulation course between the subgrade and pavement and provides for drainage under the pavement through the shoulder. As previously mentioned, it was placed in 3- and 6-in. depths under the various thicknesses of pavement as prescribed in the design. This material, as supplied from two stone plants, met special subgrade requirements as outlined under gradations A and B in the specifications. The physical characteristics of this material are given in Table 12.

SUMMARY

It should be emphasized that at the time of this report the project described is only six months old and has been open to traffic for a period of three months. It is anticipated that as the pavement ages and more information becomes available, definite conclusions can be drawn concerning its merits.

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First-Year Performance Report on Continuously-Reinforced Concrete Pavements in Pennsylvania

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Extensive mechanical and electrical instrumentation has been utilized to measure and evaluate the performance of two continuously-reinforced concrete highways in eastern Pennsylvania.

The first of these pavements was constructed on US 111 near York in the fall of 1956. A report covering the construction details and early behavior of this pavement was presented at the 1957 Highway Research Board meeting. This highway has remained closed to traffic throughout a complete year of temperature cycling in extreme weather conditions, allowing observation of its performance without the influence of heavy wheel loads.

The second pavement, on US 22 near Hamburg, was constructed in the spring of 1957. The pavement thickness and type of reinforcement were varied, but the cross-sectional area of steel was kept constant. Numerous electrical gages were installed in the pavement at selected locations to permit a careful investigation of the phenomena associated with the formation of transverse cracks. A continuous recording was made of the strains in the steel reinforcing, and also in the concrete, during the formation of one of these cracks.

This report describes the instrumentation used and presents the information obtained during a year of observations. An effort is made to evaluate this information and explain some of the changes that occur during the early life of a continuously-reinforced concrete pavement.

●ALTHOUGH more concrete is used in construction work than all other materials combined, the average engineer probably knows less about it than any of the others. He finds it difficult to understand the complicated chemical reactions that must occur before the separate ingredients become a usable solid. Also, some of the physical conditions which influence its continuing strength and durability are often not fully understood.

He may leave the pursuit of greater knowledge in these matters to the chemists, physicists, and others who can devote their full time to this aspect of the study, but he must never forget some of the fundamental facts about concrete that have been known for several hundred years, as follows:

1. Concrete requires time to gain strength. Fresh concrete is a semi-liquid mixture that must be retained within forms and protected from extreme changes in temperature and strain until it solidifies and develops strength. During this interval all reinforcement within the plastic mass must be rigidly supported to prevent any movement that could interfere with, or destroy, the existing bond between the weak concrete and the steel reinforcement.
2. Concrete shrinks as it hardens. When the water content of fresh concrete is reduced by evaporation, absorption, or other means, the volume of the concrete is also reduced. Friction and other restraints cause stresses throughout the shrinking concrete. If these stresses exceed the tensile strength of the concrete, cracks will form and relieve part of them. Although this cracking is more prevalent during the early curing process, it may occur as long as water is present.
3. Concrete is sensitive to temperature changes. Most materials, including

concrete, decrease or increase in volume in direct relationship with changes in their temperature. An unrestrained and fully-supported concrete beam 10 ft long will increase approximately 0.072 in. in length if its temperature is raised from 0 F to 100 F. However, if the ends of this beam are restrained and it is not allowed to lengthen, compressive forces will develop. Inasmuch as the ultimate compressive strength of concrete is high, no damage is likely to occur. Most concrete beams 10 ft long will withstand a compressive strain of at least 0.20 in. without failing.

A reversal of this temperature change has quite a different effect. If the ends of a 10-ft long fully-supported beam are completely restrained when the temperature is 100 F, failure of the concrete is apt to occur long before a temperature of 0 F is reached. This is due to the relatively low tensile strength of concrete.

These three basic characteristics are common to all portland cement concrete. Addition of reinforcing materials may influence of possibly control their effects, but it must be remembered that these inherent tendencies are always present in concrete structures, regardless of their size, configuration, or reinforcement.

Transverse cracks that form in continuously-reinforced concrete pavements are caused by these same conditions. The longitudinal reinforcement will affect the stress distribution throughout the pavement when bond with the concrete develops, hence will exert a limited influence on the cracking. But the principal forces which cause cracks to form originate in the concrete.

Because practically full restraint exists near each end of the pavement, any tensile strain that develops at these cracks must be carried by the reinforcing steel in the immediate vicinity of the crack. As a result, the reinforcing steel spanning each crack must either contain the strain within its elastic limit by losing bond with the surrounding concrete, or yield in tension.

It is not intended that the rather complex subject of crack development be covered in these few general statements. As this report continues with the presentation of results obtained from the research on two separate continuously-reinforced pavements, it is hoped that the purpose of these introductory remarks will become more obvious.

GENERAL DESCRIPTION

York Pavement

In 1957 a report (1) was presented describing the construction, instrumentation and early behavior of a continuously-reinforced concrete pavement on US 111 near York, Pa.

This pavement was approximately 2 mi long and consisted of four 12-ft lanes with a 20-ft median strip separating the northbound and southbound dual lanes. A concrete thickness of 9 in. on a 6-in. thick granular insulation course was used throughout the project.

The longitudinal reinforcement for each lane consisted of 20 No. 5 hard-grade deformed steel bars with a nominal diameter of 5/8 in. These bars, comprising 0.5 percent of the cross-sectional area of the pavement, had been fabricated into mats 16 ft long with seven No. 3 deformed bars to provide transverse reinforcement. The mats were placed at the vertical center of the pavement on a 4½-in. thick spreader run of concrete. Several mats were installed, allowing an end overlap of 1 ft with adjacent mats to maintain reinforcement continuity. The paving equipment was then backed up and the strike-off run of concrete was placed.

Midway between the ends of the pavement, in the outside northbound traffic lane, special gages were installed to facilitate the measurement of temperature and longitudinal strain in the vicinity of a transverse pavement crack that would develop at an artificially induced plane of weakness (Fig. 1). Brass plugs were installed on each side of the plane of weakness near the edge of the pavement to allow the measurement of crack opening with a 10-in. Whittemore gage. Resistance-wire temperature gages were placed at selected locations in the pavement and in the insulation course to indicate the local temperature and also to permit a study of the effects of vertical temperature gradation.

Bakelite SR-4 strain gages were attached to the surface of six of the longitudinal reinforcing bars comprising a mat. These gages were located at the preformed crack and 2 ft and 4 ft to each side of the crack. To provide a smooth surface for attaching the strain gages, the deformations on each bar were removed for a distance of 2 in. at every gage location. This reduced the nominal diameter of the bar from 5/8 in. to 9/16 in. and resulted in a 20 percent reduction of cross-sectional area in each gaged bar, or a 6 percent reduction in the total steel across the crack. The electrical leads from gages within the pavement were carried underground through a metal conduit to a terminal box at the edge of the highway right-of-way. The pavement was placed at the instrumented panel at 1:10 P.M. on October 10, 1956.

Gage readings taken immediately after the surface finishing operation provided a basis for all subsequent measurements.

During the first 10 days after the pavement was placed, gage readings were taken every few hours throughout the day and night to determine the initial behavior of the panel. For the next 12 months, readings were taken over a 24-hr period each 30 days.

Hamburg Pavement

In May and June 1957 another continuously-reinforced concrete pavement was constructed on US 22 near Hamburg, Pa.

Design and construction of this pavement were essentially the same as for the York pavement, except in vertical dimensions and types of reinforcement. Pavement thicknesses of 7, 8, and 9 in. were poured with insulation courses of both 3 and 6 in. under each different pavement thickness. Also, a 9-in. thick section 1,000 ft long was poured using welded wire mesh as reinforcement instead of the deformed bars common to the major portion of the project. The steel reinforcement throughout the entire project was held constant at 0.5 percent of the cross-sectional area of the pavement.

An instrumented panel similar to the one on the York project was installed in each of the 7-, 8-, and 9-in. pavements containing bar mats, and in the 9-in. pavement

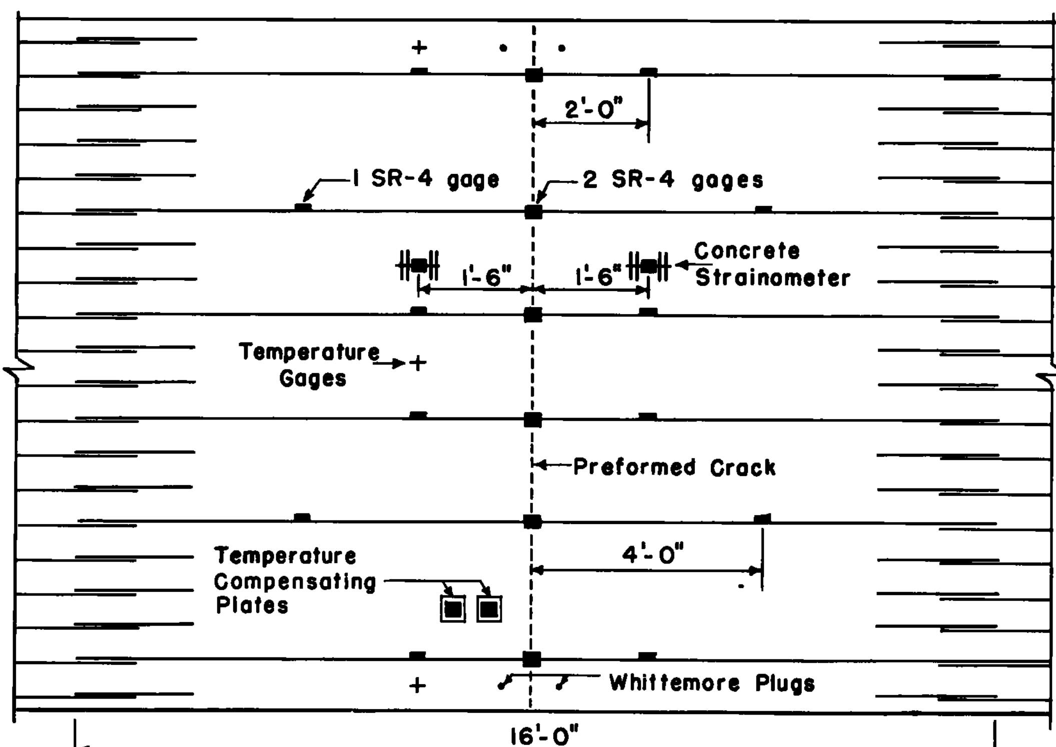

Figure 1.

reinforced with wire mesh. The insulation course at each of these panels was 6 in. thick.

In addition to the four instrumented panels, the Hamburg pavement provided an opportunity to extend the scope of the research and include several other measurements in the observations. An effort was made to provide sufficient instrumentation to allow a complete study of the cracking that occurs in a continuously-reinforced concrete highway. Plugs were installed at 100-ft intervals in the outside westbound lane to permit measurement of relative longitudinal movement of the pavement. Monuments were placed at the ends and quarter points in all four lanes to measure the absolute longitudinal movement, especially that occurring at the bridge-type expansion joints at the extreme ends of the pavement. Devices were installed at selected sections of the pavement to induce cracking, and reinforcement end-laps were marked to determine their influence on the crack pattern of the finished pavement. Special plugs were installed near cracks to measure the longitudinal and transverse warping caused by uneven temperature distributions and surface loading.

Continuous recordings of strains and temperatures were made at the panel in the 8-in. thick pavement, from the time the concrete was poured until a crack developed 36 hr later. Provisions were made to allow continuous recording of the strains imposed by traffic upon the steel and concrete of the completed pavement.

PAVEMENT BEHAVIOR AT YORK

By October 1957, the pavement at York had been under observation for a full year. The highway had remained closed to the public while some of the bridges were being completed, and very little traffic had passed over the instrumented panel.

The behavior of the pavement during the first few days after pouring provided interesting information. Definite trends were evident, and individual gage response fitted well into the expected pattern. However, the most significant feature of the early behavior did not become apparent for several months, and may best be reviewed after a presentation of the strain history throughout the entire first year of the pavement life.

The close relationship between strain in the reinforcing steel, crack width, and air temperature, is shown in Figure 2, where the individual strains in the steel bars and pavement crack widths have been averaged for simplicity of presentation.

During the first few weeks after construction, when the strain in the reinforcing bars remained in the low elastic range, the restraining influence of the tie-bar connected adjacent pavement lane had a measurable effect on the general strain pattern in the instrumented section. However, after a transverse crack had occurred in the adjacent lane within the mutually effective tie-bar area, both lanes tended to move as a unit and the differential movement between the two lanes became much less noticeable.

Thirty days after the pavement was poured, the effects of colder weather started to become noticeable in the pavement. With decreasing temperature, the instigated crack opened wider and tension strains increased in the reinforcing bars spanning the crack. Strains in the steel 2 ft away from the crack indicated a slight compression.

When strain in the steel bars at the crack had reached 2,000 micro-in. per in., it became apparent that the minor influences of warping, localized temperature distribution, and precise bar alignment had become relatively insignificant when compared with the overwhelming effects of the temperature-induced longitudinal straining.

By the end of the second month, when the air temperature was 52 F, the crack was open to a width of 15,000 micro-in. and the gaged bars across the crack were beginning to yield in tension at 2,800 micro-in. per in. strain. The gages on the bars at each side of the crack were indicating a change from compression to tension.

After the steel bars began to yield in tension, determination of the maximum strains within the yield range became more complex. Although yielding was localized within the reduced area of each bar, it was not necessarily confined within the smaller area covered by the resistance-wire gage.

Extrapolation of the temperature-strain and crack width-strain curves, combined with a knowledge of prior strain history and characteristics of the steel bars, provided a reasonably accurate record of strain history within the yield range. Seasonal strain

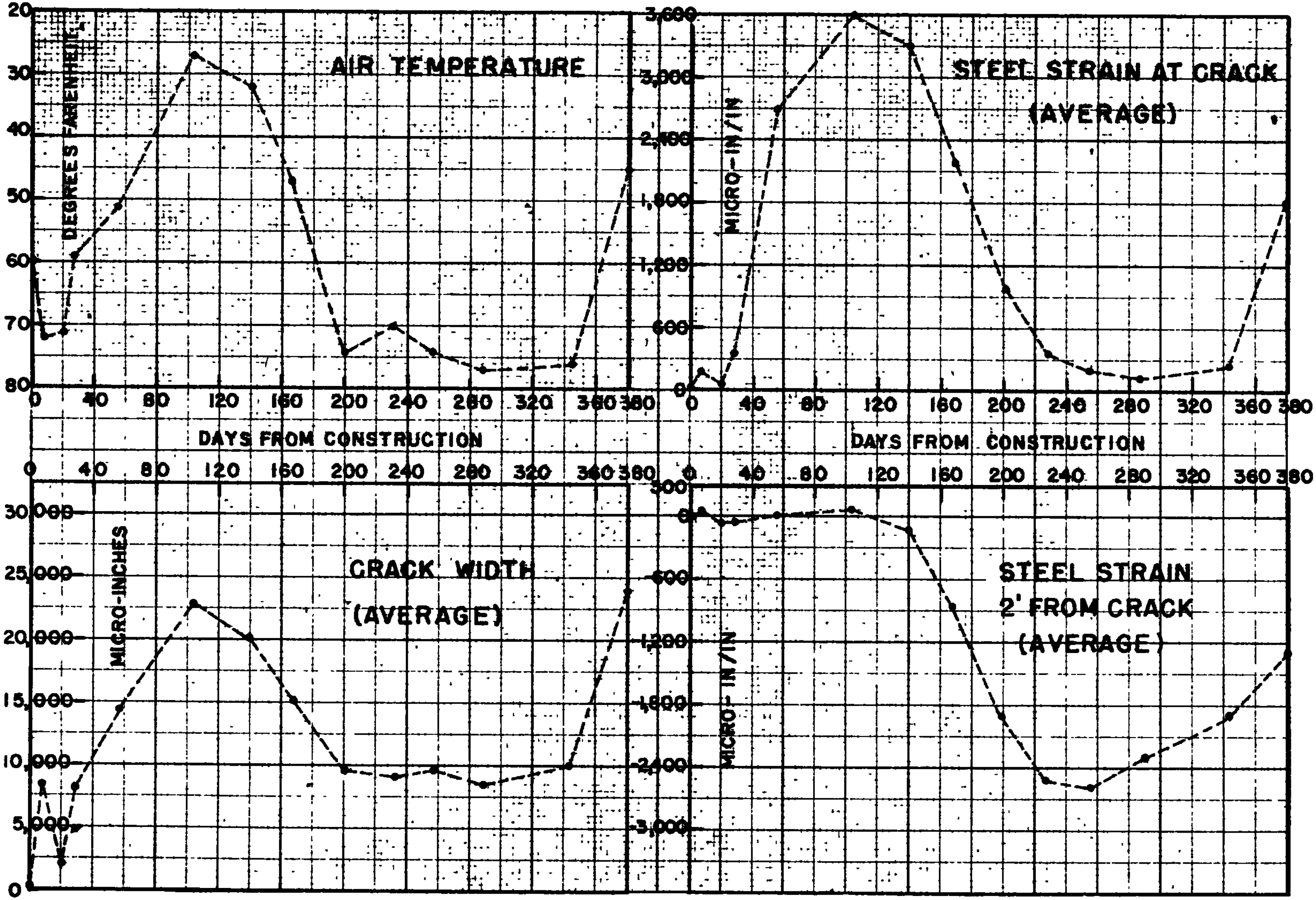

Figure 2. Strain history of York pavement.

reversals, forcing the steel to cycle through the elastic range, would permit frequent opportunities to compare the behavior of the yielded bars with their earlier strain history.

During January 1957, when the pavement was three months old, measurements were made when the temperature was 22 F. The crack width had increased to 23,000 micro-in. and the gaged bars at the crack were strained to 3,600 micro-in. per in. in tension, or beyond the yield point. Tension strains in the gaged bars away from the crack had reached only 150 micro-in. per in.

This was the lowest temperature at which readings were made on the gages, but temperature records at a near-by airport indicate a low of 2 F during this month. Considering the previous ratio of change in strain with change in temperature, it is probable that at this extreme low in temperature, the strains in the bars at the crack reached 4,000 micro-in. per in. and the crack opened to 27,000 micro-in.

For the next three months the temperature was in the warming phase of its yearly cycle. The crack opening and steel strain at the crack reversed their direction and approached the condition that existed shortly after the pavement was poured. Gages located on the bars at each side of the crack, where the strains had previously remained in the low elastic range of the steel, began to indicate significant compression.

Throughout the late spring and summer, measurements were made when the temperature was between 70 F and 80 F, although it was known that in the time interval between these periodic measurements, the temperature fluctuated within a range of 40 F. During this time, although the crack width remained about 8,000 micro-in., the gaged bars at the crack were yielded in compression until they retained only 100 micro-in. per in. of their former tensile strain. These same bars, at a distance of 2 ft from the crack, were then yielding in compression with a strain of 3,300 micro-in. per in.

To explain the development of this unusual strain pattern it was necessary to review and understand the behavior of the pavement when the instigated crack occurred. The pavement was poured at the instrumented panel when the temperature was 60 F and the crack was formed at the induced plane of weakness 40 hr later, when the temperature had dropped to 25 F. During the following three days the temperature fluctuated through a range of 30 F and the measured crack width corresponded in proportion to these temperature changes.

In the test panel, relatively large differential movements occurred between the concrete and steel within 40 hr after the pavement was poured.

Although the reinforcing steel was strain-sensitive to the changing temperature, there seemed to be little relationship between the amplitude of the strain in the bars and the width of the crack opening. This apparent independent action of the steel and the concrete continued for two weeks after the pavement was poured, but when the seasonal cooling cycle started a reasonable ratio could be established between temperature, crack width, and steel strain at the crack.

Figures 3 and 4 show the phenomena that occurred during the development of the crack along the induced plane of weakness. Because the instigated crack was the first crack to form in a center section of the pavement, free end influence did not exist. Therefore, points A and A_1 may be considered as being in areas of complete restraint. Due to shrinkage, and finally to temperature, relatively high tensile forces developed at B and B_1 in the concrete, and a crack formed in the plane of weakness at X. The developing bond along the reinforcing bar C did not have sufficient strength to transfer these forces from the concrete into the bar. As a result, all of the adhesive bond in the vicinity of the crack was destroyed.

Constant temperature cycling during the

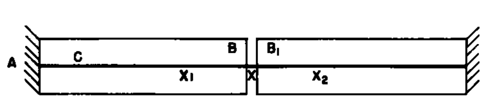

Figure 3.

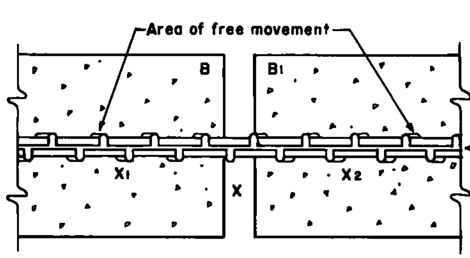

Figure 4.

first few days of pavement life prevented the reoccurrence of adhesive bond, and a purely mechanical bond was developed at the extreme limits of differential movement between the concrete and the deformations on the reinforcing bar (Fig. 4).

As the concrete developed additional strength, the relatively free independent movement between the concrete and the reinforcing bar was confined within the limited range of movement established when the concrete was weak. Strains measured in the bar at X_1 and X_2 indicated that the loss of bond extended to these points, but that the range of free differential movement diminished as the distance from the crack became greater. (This mechanical bond action could be compared with a bolt that fits loosely into a threaded hole; it has adequate strength for transferring forces in either tension or compression, but allows a short range of free movement during a reversal of the loading.)

When lower temperatures and continuing shrinkage opened the crack beyond the range of free movement, the strain was transferred into the reinforcing bar. Crack width and strain in the bar at X increased in proportion with the decrease in temperature throughout the progressive cooling cycle, but gages located at X_1 and X_2 on the bar indicated only mild tension strain. Because the coefficients of expansion of steel and concrete are practically the same, all of the temperature-induced strain in the bar and concrete were mobilized at the crack to produce sufficient tension to cause yielding at X.

With the approach of spring the temperature began to rise. The strain direction reversed and the tension strain at X was gradually relieved.

Earlier yielding had increased the length of the reinforcing bar at X, therefore compressive straining occurred at this point before the crack was closed. Inasmuch as friction was the only restraint to movement within the established free movement area of the mechanical bond, this strain was carried along the bar to some points beyond X_1 and X_2, into the area of true bond.

As the higher temperatures of summer continued to increase the compression stresses in the pavement, creep in the concrete allowed additional strain to be imposed on the bar. Compression strains at X_1 and X_2 were increased to the yield point, whereas the bar at X was yielded in compression and returned almost to its original length.

After one year of pavement life and the beginning of the cooling phase of the temperature cycle, all measurements show a definite reversal in direction of straining and are returning to a condition of tension. It will be interesting to determine the effect of heavy traffic loads during repeated temperature cycling.

PAVEMENT BEHAVIOR AT HAMBURG

The pavement at Hamburg was constructed in the late spring and early summer, at a time when the temperature was in a seasonal warming phase. Cracks at all instrumented panels formed within 40 hr after the concrete was poured.

During the first 10 days of the pavement life, measured strains at the induced cracks were low in amplitude and rather erratic. There was some similarity to the early behavior of the pavement at York, although the temperature at the Hamburg project was more stable and wide crack openings did not occur.

A strip-chart recorder was used at the panel in the 8-in. thick pavement to obtain a continuous amplitude-time history of the formation of a crack. Pavement temperature, strain in the reinforcement at the crack, and longitudinal strain in the concrete adjacent to the crack, were recorded simultaneously over a 12-hr period, beginning 24 hr after the pavement was poured. During this recorded period the crack formed and opened to a width of 5,600 micro-in. The strain in the reinforcing bar increased 196 micro-in. per in., while the temperature in the pavement decreased from 95 F to 88 F. Strain in the concrete adjacent to the crack changed from 50 micro-in. per in. compression to 22 micro-in. per in. tension.

Examination of the chart records revealed that all of the strains in the steel and concrete occurred gradually. There was no sudden increase in strain that would indicate dynamic rupture of the concrete when the pavement cracked. This was considered as further evidence that early cracking is primarily the result of shrinkage in the concrete. The new concrete, being unable to withstand the tension strains developed by shrinkage, permitted a crack to form at the induced plane of weakness. This crack formed with a minimum of tension in the fresh concrete and continued to open slowly without a direct transfer of strain to the reinforcement.

Throughout the summer months all strains in the pavement continued to increase in compression. Crack openings remained small, and in some instances completely closed in a measurable compressed condition. By early October 1957, all of the compression strains had decreased in amplitude and were beginning the seasonal change to a condition of tension.

To determine the possible influence of the end laps of the reinforcing bar-mats on the development of a later crack pattern, a section of the pavement was marked at each lap of the mats. Forty mats with 12-in. overlap were included in this test section. After four months, with a normal crack pattern existing in the test section, there were no cracks found in the immediate area where the steel had overlapped.

Artificial planes of weakness were installed at several points along the pavement. These included insertion of transverse asphalt strips at the center ot the pavement and elimination of effective bonding by covering sections of the reinforcing bars with rubber tape. The asphalt strips reduced the cross-sectional area of the pavement 6 percent and the rubber tape eliminated bond on each longitudinal bar for a distance of 8 in.

These induced planes of weakness were installed at 32 positions in the pavement. A normal crack pattern developed in the area where they existed, but only one crack formed along a plane of induced weakness; this lone crack could be attributed to chance.

Additional information will be available from the several other phases of the testing at Hamburg after a complete cycle of seasonal temperature has had its effect.

CONCLUSIONS

As yet it is impossible to give complete details and explanations of the extensive information obtained during the current investigation of continuously-reinforced pavements in Pennsylvania. Nevertheless, in view of the wide interest and accelerated research involving pavements of this type, it is considered important that these findings be made available to others who may contemplate similar research projects.

With this objective, a brief summary of observations, noticeable trends, and conclusions is presented, as follows:

1. A 12-in. overlap of adjacent reinforcing bar mats was sufficient to maintain continuity of the steel throughout the length of the pavement.
2. Longitudinal continuity of the steel is necessary. Complete local failure of the pavement will occur at places where it is not maintained.
3. The first cracks to appear in the pavement were the result of shrinkage. This tended to set the pattern for future cracking and probably will remain influential throughout the life of the pavement. Shrinkage cracks were formed in the new pavement even after rising temperature had forced the longitudinal reinforcing bars into a state of compression.
4. At extremely low temperatures crack widths up to $\frac{1}{32}$ in. may be expected. Limited infiltration of silt and water does not appear to result in damage, and the cracks close tightly during the warm season. Crack widths of $\frac{1}{16}$ in. or more should be regarded with suspicion; they should not occur in a pavement of proper design.
5. When crack openings exceeded 0.020 in., tension yielding probably had occurred in all of the deformed steel bars spanning the crack. When these cracks were forced to close, the steel yielded in compression. This action caused a loss of bond along the bar. The amount could be determined when the crack width, steel strain, and steel characteristics were known. It is possible that this bond loss increases with the yearly cycle of strain until the steel is capable of responding within its elastic limit.

It should be pointed out that yielding which may occur at normal cracks is well within the working capabilities of the steel reinforcement (Fig. 5). The steel used in the pavement at York yielded at a strain of 2,700 micro-in. per in., yet a strain of 98,000 micro-in. per in. would have been required to cause rupture.

6. The season of the year in which a pavement is constructed has a tremendous influence upon its early behavior. Tension strains during cold weather have an obvious effect on existing cracks and may cause several new cracks to form, but the more subtle effects of warm-weather compression are equally important in a new pavement.

Concrete will creep under prolonged strain, and if this creep is excessive it may not be fully recovered when the direction of the straining is reversed. Compressive strains measured in both pavements described in this report remained very high throughout the summer of 1957. Some of the creep which has occurred will probably influence the cold-weather behavior and result in a slight increase in crack width or the formation of new cracks.

7. The induced cracks at test panels were formed approximately 40 hr after the construction of the pavement, whereas normal cracking in the remainder of the pavement did not start until the sixth or seventh day. A more representative crack at the test panel would have resulted if the metal crack instigator had been omitted and the crack had been induced by transverse sawing after the pavement had developed more strength.

8. Temperature is by far the most damaging influence to which continuously-reinforced pavements are subjected. There is some evidence to indicate that pavement "growth" is not confined to the extreme ends of a continuously-reinforced pavement, but occurs throughout its entire length when hot weather causes the concrete and steel to expand. Only a limited amount of the pavement end is moved by cold-weather contraction. Since this is visibly evident in the crack pattern, it is probably the basis for the belief that all "growth" occurs within this area.

Non-recoverable creep occurs in the concrete when high compressive forces develop during very hot weather. Yearly infiltration of foreign matter and dislocation of sand particles in the cracks may cause additional creep. This will continue until sufficient loss of bond with the reinforcement permits the pavement to expand and contract within the elastic range of the concrete and steel.

The free ends of the pavement will be subjected only to forces equal to the yield strength of the reinforcing steel when the pavement is in a state of tension, but the forces of compression may be of much greater magnitude.

There is some evidence to indicate that the first few annual temperature cycles may stabilize the pavement straining within the elastic range of the steel and the recoverable

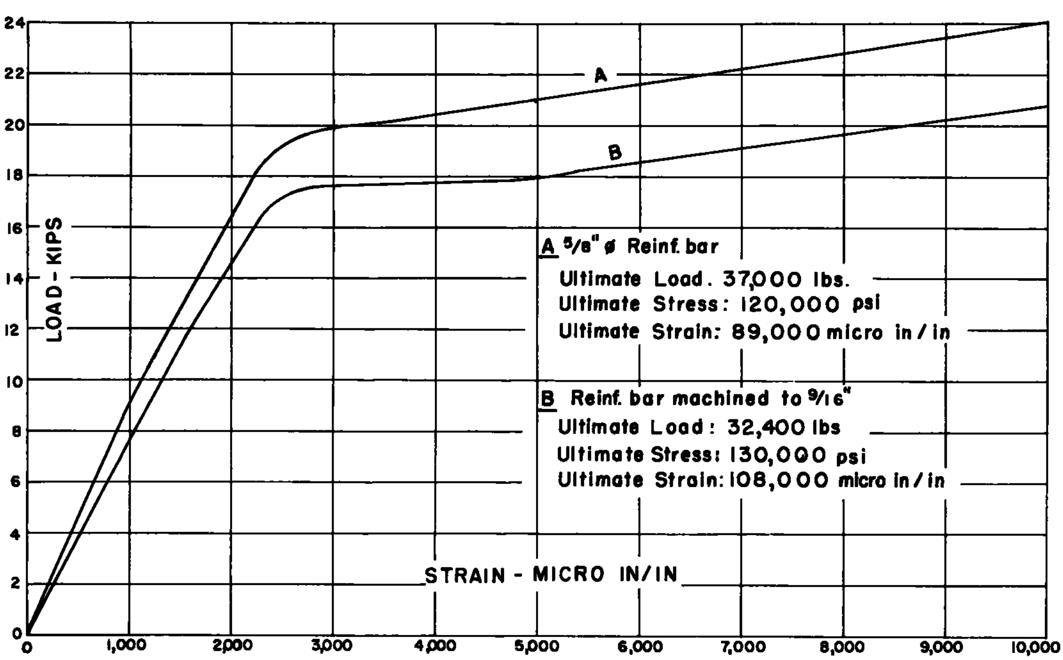

Figure 5. Load-strain curves of steel reinforcement used in York and Hamburg projects.

creep range of the concrete. Subsequent annual cycling may cause additional yielding, but only to the extent that the infiltration of silt into the transverse cracks prevents their complete closure.

The experiences with continuously-reinforced concrete pavements in Pennsylvania have been encouraging. Much remains to be learned before an ultimate design can be specified, but it is believed that design based on currently available knowledge could produce highways of superior riding qualities and greater durability.

ACKNOWLEDGMENTS

This investigation was made possible by the sponsorship and cooperation of the U.S. Bureau of Public Roads and the Pennsylvania Department of Highways. The instrumentation on the York Project was financed by the American Iron and Steel Institute under its Committee on Reinforced Concrete Research.

The authors are indebted to these sponsors, especially F. Witkoski, Director of Research, Pennsylvania Department of Highways. Also to V. Yerlici and W. Shieh of the Fritz Engineering Laboratory, and to R. L. Schiffman, who was associated with the projects until July 1, 1957.

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Discussion

ROBERT L. SCHIFFMAN, Assistant Professor of Soil Mechanics, Rensselaer Polytechnic Institute, Troy N. Y.— This paper is a most interesting contribution to the growing literature on continuously-reinforced concrete pavements and the authors are to be congratulated on a careful and detailed study of the subject. There are, however, several points of interpretation of test data which are subject to question.

The writer questions the use of average values of strain as related to local air temperature, as presented in Figure 2. The authors state that the use of average values was only for ease of presentation, yet they base a quantitative interpretation on numerical values in terms of presented averages. Yet later they treat the strain in each bar, for a given over-all temperature change, as a statistically random event as compared to the other bars. They thus define the statistical population by six replications at the crack, these being the number of gaged bars, and by eight replications for strains 2 ft away from the crack. Under these conditions, even for "simplicity of presentation," the use of mean values may be misleading.

Although the influence of temperature on strain and crack width is probably controlling, it is not believed that all the other effects, dismissed by the authors, are of little or no physical influence. Instead, these effects preclude the use of average values, no matter what the stated purpose. Also, although average values are given, there are no probability statements to indicate the degree of variation from the mean.

In looking at the physical situation, one is first inclined to check the dependence or lack of dependence on strain history by examining the strain gradients at the preformed crack, induced prior to any evidences of yielding of the steel. Such data have been presented elsewhere (1, Fig. 16). The steel strain gradient across the pavement in the period of elastic behavior varied between 250 micro-in. per in. just prior to cracking, and 350 micro-in. per in. at 27.5 days. On the basis that the yield point of the steel is 2,800 micro-in. per in., this gradient is between 9 and 12.5 percent of the yield point strain. In terms of an actual stress gradient it is 7,000 to 10,000 psi, which is not only of significant magnitude, but readings taken through May 28, 1957, indicate that the strain history pattern subsequent to yielding is highly dependent on the pre-yielding pattern.

The physical interactions between bars can be summarized as due to physical restraints of the tie bars, temperature gradients, amount of bond slippage in each bar, individual yield points in each bar, and the position of the elasto-plastic boundaries in the steel with respect to the gage point. These factors, instead of pointing towards randomness, lead to a conclusion that there are physical factors which induce different strain readings in each bar. A statistical hypothesis that the strain readings come from the same population is unjustifiable and implies circumstances of interactions which can be misleading. In fact it can be shown that spread in values is so great that statistical analyses indicate that events are being represented as samples from a single population (single physical occurence) when in fact these events come from many different occurences. This is further proof that the use of averages is not justified. A detailed record of transverse distribution of steel strain at the preformed crack, through May 28, 1957, is presented in Figure 6.

Exactly the same arguments can be applied to averaging the strain of gages placed 2 ft and 4 ft from the crack. In addition, however, this argument is stronger because an additional physical occurence enters the picture. The gages away from the crack were placed on the side of the bar in an effort to eliminate flexural strains. This procedure would eliminate bending strain readings only if such bending occurred about the lateral axis of the bar. Inasmuch as the bars are never truly straight when placed, once the bond is broken (as over the gaged area) these bars may bend about almost any axis. Therefore, there is an additional influence on these gage readings; namely, bending strains.

In summary, the authors, in developing numerical and quantitative interpretations of their data, have employed means of data analysis that are not justifiable. In other words, there is no quantitative validity to the strain instrumentation once the yield point has been exceeded, since the instrumentation itself is influenced by local phenomena. The gage readings beyond the yield point are only representative of what is occuring to a single gage, and cannot be construed as having general applicability.

The writer wishes to emphasize one important aspect of this experiment. The research carried on for the reported projects should put to rest for all time the myth, held by many engineers, that exceeding the yield point is akin to failure. Comparisons between this work and the performance of other, older, continuously-reinforced concrete pavements prove conclusively that the longitudinal steel in all but the heaviest reinforced sections yielded comparatively early in life. In point of fact, this yield took place without any serious effect on over-all performance. It is high time that highway engineers accept the fact that safe economical designs can be accomplished for reinforced pavements by permitting the steel to exceed its yield point.

The authors state categorically that a crack in a continuously-reinforced concrete pavement should not exceed $\frac{1}{16}$ in. in width. It would be most enlightening if they would expand on this statement with respect to the reasoning behind the selection of this particular value.

The authors' fifth conclusion, that the steel at a crack will yield in tension and compression alternately with the seasons, is most interesting, and one which should be developed at greater length. Inasmuch as crack width is the performance criterion, the conclusion of a "Bauschinger effect" (2) should be related to this criterion.

The authors refer to a continuous strain record taken of the steel at the preformed crack, and of longitudinal strain in the concrete adjacent to the crack during the process of cracking. They found "no sudden increase in strain that would indicate dynamic rupture of the concrete when the pavement cracked."

They continue by saying:

"This was considered as further evidence that early cracking is primarily the result of shrinkage in the concrete. The new concrete, being unable to withstand the tension strains developed by shrinkage, permitted a crack to form at the induced plane of weakness. This crack formed with a minimum of tension in the fresh concrete and continued to open slowly without a direct transfer of strain to the reinforcement."

These statements are open to question for several reasons. In the first place, they imply that concrete between 12 and 36 hr old is a perfectly ductile material, which is in opposition to previous measured performance. The statement on lack of dynamic rupture

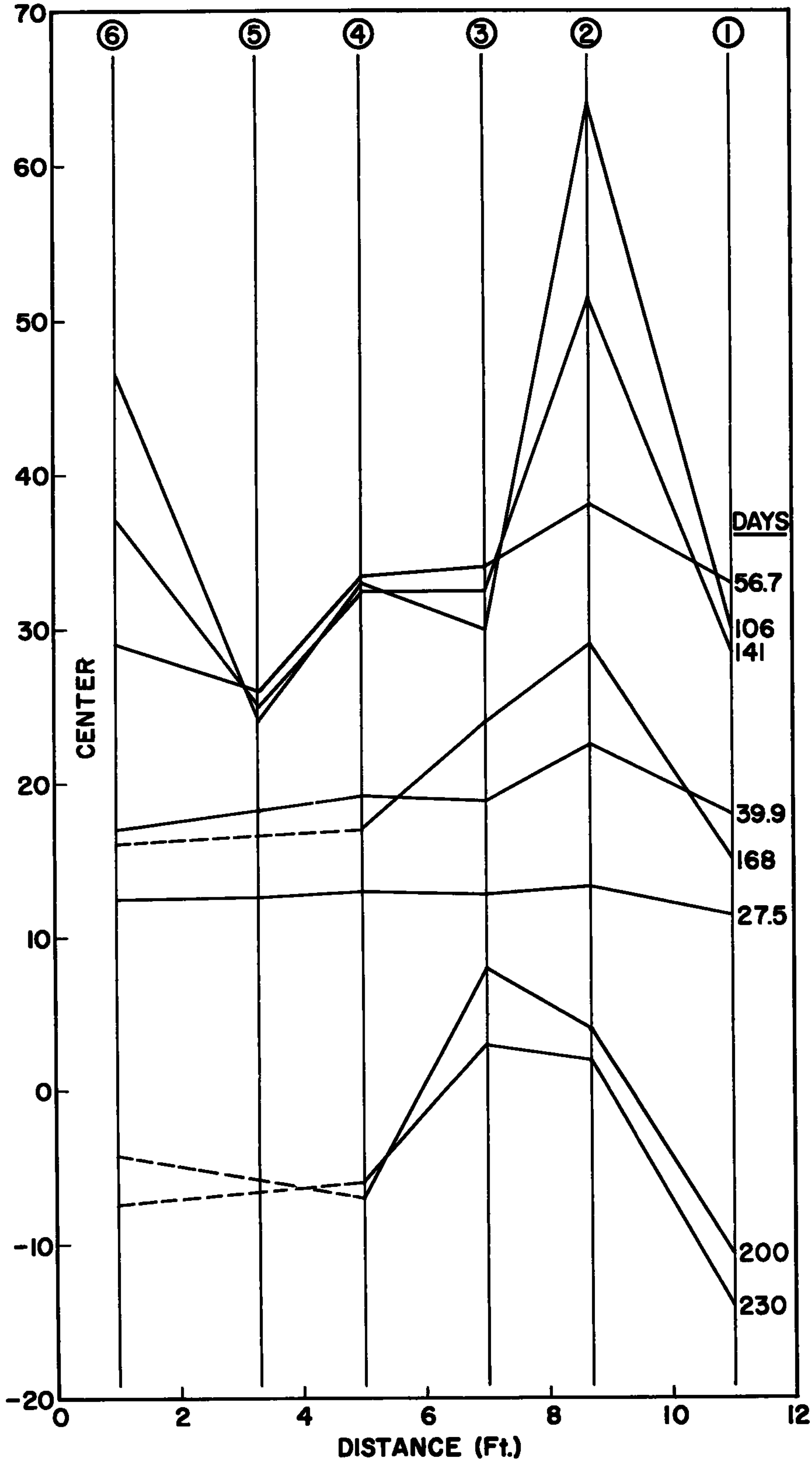

Figure 6. Transverse distribution of steel strain at preformed crack.

does not in fact contradict any previous work, but is simply lack of consideration of the effects of the instrumentation on the pavement, and thus on the data gathered.

Essentially, the strain at the crack was measured by gages on the steel at the crack. All the bars at the crack, including those with instrumentation, were taped so that in this region there was no bond between steel and concrete. Thus, there is a 2-in. length of unbonded steel, and it is in this region that the concrete cracked. Under these conditions, the only way in which these gages at the crack would record rupture strains, is if the energy of rupture was propagated back into the bonded area with undamped motion. The strain in the concrete was measured 18 or more inches away from the crack, which was undoubtedly outside the area of influence of rupture. The authors showed only that there is a St. Venant effect during rupture, and because their instrumentation was effectively outside this range the rupture did not influence the gage readings. The only conclusion that can be drawn from these data is that the tensile strength of the concrete was so low that the region in which rupture influenced the strains was limited in extent.

The paper does not mention the role of the subgrade in the behavior of continuously-reinforced concrete pavements. The strains recorded in this pavement were temperature-compensated. If there were no restraints imposed on this pavement, the recorded strains would have to be zero (assuming equal thermal coefficients for concrete and steel). But the only restraint on the pavement is that due to the shearing stresses mobilized between the pavement and the subgrade (3). As a result, ignoring the effects of the subgrade omits a prime variable.

As of the beginning of 1958, five states have experimental service pavements under continuing investigation. Several others are in the process of installing or planning similar projects. It would seem that in assessing the value of future field service research the law of diminishing returns has set in; although these field experiments have resulted in excellent performance data, not a single experiment of this type has led to a rational design procedure. The reason for this lack of information lies not in the competence of previous investigations, but in the type of project. Any experiment must, to be successful, evaluate the influence and magnitudes of the prime variables.

The following broad program is proposed to develop a design basis for continuously-reinforced concrete pavements.

The first effort should be made in the theoretical area in which a design formula is attempted. The most fruitful effort probably will be a one-dimensional model where tension and compression are handled separately. The tension model would treat the steel as a work-hardening plastic material and the concrete as a brittle elastic material. The compression model would treat the steel as a work-hardening plastic material and the concrete as a relaxing visco-elastic material. This would develop a formulation in which the crack width would be related to the percentage of steel, Young's modulus and the post-yield modulus for the steel, the bulk modulus and viscosity of the concrete, the crack interval, and the bond slippage.

Although a theoretical analysis will ultimately prevail as the lasting method of approach, another, perhaps equally fruitful, avenue of investigation can be opened. This is, essentially, a laboratory study to arrive at optimum conditions likely in field performance. A fundamental plan for this type of investigation is herein proposed.

Consider that the design mix of the concrete is a constant factor, that the steel will be a hard-grade (work-hardening) type, and that the primary motivation will be temperature changes of a cyclic nature. The design variables will thus be:

1. Extreme temperature level.
2. Percentage of steel reinforcement.
3. Size of steel bar.
4. Thickness of slab.

The response variables will be crack interval and crack width, these being interdependent responses. Because a positive but unknown relationship exists between strain and temperature level, the temperature variables can be replaced by a deformation variable. Inasmuch as full restraint is generally the extreme case, the deformation variable is replaceable by a stress or load variable.

The tests performed in the laboratory could be set up with a factorial statistical hypothesis and could be so designed as to test and determine the interactions between the design variables (4).

The test procedure could be as follows: For a given size of slab, size of bar, and percentage of steel, slabs would be constructed in the laboratory and then subjected to a predetermined program of alternating loads. The load programming would include local repetitions to consider the seasonal variations. The effect of temperature variations will be determined by variations in the load level.

Proper time scaling, after the concrete has cured, would enable a test series to be completed in a matter of days, representing five to ten years of pavement life. The only measurements that need be made on these tests are crack spacing and crack width.

With a proper factorial design, the test results can be analyzed statistically, to enable the investigators to ascertain the optimum design conditions, in terms of the preselected variables.

At this point, other variables can be introduced for a more extensive investigation, including the use of wire mesh, and the variations in concrete strength. This can be treated as a sub-factorial design measuring interactions with the basic design.

The program outlined is ambitious in nature, will require a large number of laboratory tests, and may take as much as five years to complete. It should be compared, however, with the various field experiments which have been under investigation for almost 20 years, and the positive end result from each should be compared.

Only when the laboratory study has been completed is a field testing program of real value. In the field program, two or at most three, designs can be used. The optimum design is constructed along with an over-conservative and a safe but under-designed pavement. By evaluating these designs under actual service conditions, a final judgment of design conditions can be established. This field experiment would measure such performance criteria as crack width, crack interval, maintenance, and riding qualities.

Unless future experimental work in this area comes to grips with the actual problems, engineers will know very little more on the subject in the foreseeable future than they know today.

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I. J. TAYLOR and W. J. ENEY, Closure—Lehigh University is one of several participants in a planned program to investigate the possibility of improving the design and construction of continuously-reinforced concrete pavements. Other universities, research foundations, industries, and various State and Federal agencies, are cooperating in this program. It is believed that by this method a thorough investigation will be conducted without excessive duplication of research and field testing.

The work is under the constant surveillance of competent and active committees comprised of outstanding engineers and other recognized authorities in the field of highway design and construction. The knowledge and experience of these man has been combined with the trained personnel and testing facilities of well-equipped engineering laboratories to conduct a specific research program. When the current projects are completed it is not unlikely that all of the problems on continuously-reinforced pavements confronting highway engineers will have been solved. Experience gained in the earlier tests have permitted plans for better practices in subsequent programs. Results on the York and Hamburg projects have not always been as good as desired. However, it is believed that the authors have developed several unique methods of instrumentation and measurement resulting in a better understanding of the origin and magnitude of the forces acting on continuous pavements.

There was no need to first conduct a fundamental laboratory study on continuous pavements and thus delay by two years the opportunity to observe the actual behavior of a pavement which would be satisfactory but not necessarily of optimum design. Controlled laboratory tests at Lehigh upon various pavement specimens under simulated conditions known to influence actual pavements have been considered since the beginning of the York-Hamburg project and an informative series of tests now under way has already provided valuable information.

The complete test data and all related information concerning instrumentation and testing procedures have been furnished to the sponsors. This information is available to those who may wish to seriously review the work or check and compare their theories with test results.

In his discussion Professor Schiffman puts forth many of his own ideas and opinions. It appears that he answers most of his posed questions, at least to his own satisfaction. It is therefore deemed unnecessary to expand or repeat statements taken from the report simply for the sake of argument.

There are only three major items in the report to which he takes particular exception. These are answered as follows:

1. Averaging of gage measurements in order to present a simple comprehensive graph of closely related influences. Twenty-four gages were used to measure the longitudinal strains in the reinforcing bars at the pavement crack. Presentation of separate curves of the strain history at each gage would have added superfluous volume to the report without clearly presenting the very important relationship between the strains in the reinforcement, crack width, and air temperature. Minor local influences are absorbed in the process of averaging the strains, but these are not believed to have an ultimate damaging effect on the pavement so have been considered relatively insignificant when compared with the more important forces which could possibly cause damage. Also, averaging the strains in individual bars at a transverse plane permits a better understanding of the total stress in the longitudinal reinforcement normal to this plane.

In an attempt to explain the fallacies of averaging the strain values, Professor Schiffman has used the maximum strain deviation between individual bars during the early curing period of the pavement. This deviation is approximately 5 percent of the average maximum strain recorded during the first year of pavement life, and is believed to be quite reasonable considering the unstable stress conditions found in new concrete.

2. The influence upon crack behavior of the tie-bar attachment to the adjacent pavement lane. The report stated that this was a minor influence which became progressively less significant with time. A careful inspection of hundreds of other similarly restrained cracks had indicated no distress in the pavement which might be attributed to this influence.

3. The crack formation in new concrete without sudden rupture or dynamic development of tensile strains in the reinforcing bars. To separate effectively the temperature- and shrinkage- induced strains developing in the pavement during the first 36 hr after construction, sensitive strain gages were used on the longitudinal reinforcing bars directly at the crack. The strip-chart equipment used with these gages had a frequency response of 100 cycles per second and any damping of the rupture energy, regardless of the origin, would have been detected.

It is difficult to understand how Professor Schiffman construed the authors' explanation at "proposing that concrete which is between 12 and 36 hr old is perfectly ductile material," or how he derived that, "because their instrumentation was effectively outside this range the rupture did not influence the gage readings."

The authors see no need to elaborate further on their testing methods, or in any way change the analysis or conclusions because of Professor Schiffman's discussion. The results of the continuing studies at Lehigh University and by other research groups are expected to fully support the analysis and conclusions of this progress report.

The National Academy of Sciences—National Research Council is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The Academy itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the Academy and the government, although the Academy is not a governmental agency.

The National Research Council was established by the Academy in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the Academy in service to the nation, to society, and to science at home and abroad. Members of the National Research Council receive their appointments from the president of the Academy. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

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The Highway Research Board was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the National Research Council. The Board is a cooperative organization of the highway technologists of America operating under the auspices of the Academy–Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.