Design of Continuously Reinforced Concrete Pavement

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Experience gained from existing continuously reinforced concrete pavements is now adequate to suggest certain design principles. Transverse cracks at frequent intervals are caused by drying shrinkage, warping, wheel loads, and falling temperature. It is believed that the stress in the steel would be independent of the temperature drop if the tensile strength of the concrete remained constant. There is evidence that the strength of the concrete increases considerably as the temperature falls and this increased concrete strength causes an increase in steel stress. Neither the amount of increase in concrete strength nor the steel stress over a period of years is definitely known, but experience with existing pavements, including those subjected to low temperatures, has shown that approximately 0.5 percent steel will not break and that the frequent cracks formed in the concrete will be narrow enough in width to result in many years of satisfactory service with very little maintenance.

● THEORETICAL pavement designs should be viewed with skepticism unless proved correct by actual experience. In the case of reinforced concrete pavement design, the best designs often are arbitrarily determined by experienced engineers and perhaps justified later by placing the proper constants in a design formula to make the answer agree with the predetermined design. This paper makes use of the service behavior and research data for continuously reinforced concrete pavement to delineate some design principles which are in agreement with experience.

In a prior paper (1) it was suggested that continuously reinforced pavements were feasible and that this design possessed certain advantages. A formula was presented which indicated that the theoretical amount of longitudinal steel required was 1 percent of the cross-sectional area of concrete. However, limited experience even at that time indicated that, regardless of theory, 0.5 percent steel would not break. Since that time, a number of continuously reinforced pavements have been built. Those containing 0.5 percent or more steel havé excellent service records. On the Illinois continuously reinforced pavement built in 1947, as little as 0.3 percent longitudinal steel has not failed, in spite of reported maximum steel stress measurements of 62,000 psi. This stress was measured in pavement containing 0.7 percent steel and at a pavement temperature of 13 F. How is it possible to explain that 0.3 percent steel did not break, when a short distance down the road more than twice as much steel was stressed to 62,000 psi? One might estimate that the 0.3 percent steel was stressed at 2⅓ times 62,000 psi, which is probably not correct.

The formula originally proposed (1) differs from that proposed by Vetter (2):

$$p = \frac{S_c}{S_s - n\,S_c} \qquad (1)$$

In which:

p = ratio of area of steel to area of concrete;
S_c = tensile strength of concrete;
S_s = yield point of steel; and
n = ratio of moduli of elasticity.

By using S_c = 350 psi, S_s = 50,000 psi, and n = 6, p = 0.007, or 0.7 percent. Eq. 1 assumes that the total tensile strength of the steel must be equal to the total tensile strength of the concrete. To determine the amount of steel required to accomplish this, the unit strength of the concrete is divided by the unit strength of steel. The second term in the denominator represents stress in the steel at points in the concrete away from cracks. This amount of steel stress helps to prevent the concrete from cracking; therefore, the stress in the steel at a crack must exceed the tensile strength of the concrete by the amount of stress in the steel in the uncracked concrete. However, this term affects the result by only about 4 percent, so that the error will be small if Eq. 1 is simplified to

$$p = \frac{S_c}{S_s} \qquad (2)$$

Considering the unknown effect of a number of things to be discussed later, it appears that no formula can be written at this time which correctly estimates the percentage of steel required. Nevertheless, an understanding of Eqs. 1 and 2 is useful in understanding what takes place in continuously reinforced pavement. The author's original formula was also incorrect because of a number of phenomena which were not considered.

During the first 24 hr after a pavement is placed, the temperature first rises, due to heat of hydration, and then starts to drop. Drying shrinkage also occurs during early ages. Both this early drop in temperature and most of the drying shrinkage occur before the concrete gains its full strength. The first cracks are formed while the concrete is relatively weak and the steel stresses relatively low. Warping stresses, due to a difference in temperature between top and bottom of the concrete, also assist in causing the concrete to crack without causing much, if any, steel stress. After the pavement has been opened to traffic, wheel load stresses are added to shrinkage, temperature, and warping stresses, all of which, acting together, form cracks without causing a high stress in the steel.

When cold weather occurs, a number of cracks have already been formed, and additional cracks occur as the temperature falls. During winter months, the concrete is continually in tension, and this tension causes more creep or plastic flow in the concrete than in the steel. Plastic flow in the concrete increases the steel stress between cracks and decreases the steel stress at cracks. Illinois recorded a stress in the steel in uncracked concrete as high as 10,000 psi, whereas by the usual method of calculation this stress should not have exceeded 2,100 psi ($n S_c = 6 \times 350 = 2,100$ psi).

It may be that plastic flow in the concrete subjected to continuous high stress resulted in transferring considerable stress from the concrete to the steel in the uncracked concrete. This would be expected to reduce the steel stress at cracks in the concrete. Under sustained tensile stress it is likely that there is some plastic flow in the steel at cracks, which would be erroneously calculated as stress when reading SR-4 strain gages. The Vetter formula (Eq. 1) provides a means of calculating steel stress due solely to falling temperature, but it ignores all these other factors, which combine to materially reduce the actual steel stress. It must be concluded that the method of calculating stresses in continuously reinforced pavement is not yet known.

The average crack spacing in the central portion of pavements containing 0.5 percent steel has been found to be on the order of 5 to 10 ft after the pavement has been in service several years. This is believed to be the optimum crack spacing. The 0.3 percent steel sections had fewer cracks than did the sections containing heavier steel. These cracks at the age of four years were open an average of 0.031 in. and probably resulted in the steel being overstressed even though it did not break. The steel probably was overstressed and slightly elongated, thus producing at the crack a cold-worked condition, which raised the yield point and helped avoid rupture of the steel. It also resulted in cracks wide enough to permit some water to enter and probably to destroy aggregate interlock. Illinois engineers now suspect that the steel is broken at some of the cracks in the 0.3 percent section. The fact that this steel did not break until the pavement was about nine years old indicates that the breaking force was the shearing developed by heavy wheel loads acting on cracks too wide to develop adequate aggregate interlock.

Steel ratios greater than 0.5 percent caused cracks to form closer together, which appears to cause deflection under wheel loads to occur in a shorter length of pavement. It is believed that the concentrated deflection in an area of very close crack spacing causes the concrete to assume a short radius of curvature, resulting in high shearing and compressive stresses in the concrete. This means that the higher the percentage of steel and the more frequent the cracks, the lower the stress in the steel and the higher the stress in the concrete. Whether or not this is true, cannot be proved at present. From the standpoint of economics alone, it seems safe to conclude that the minimum amount of steel which will not break is the optimum amount to use. Based on the experience in Illinois, 0.5 percent seems safe.

Vetter (2) also pointed out that the maximum steel stress is independent of the amount the temperature falls; there is nothing in Eq. 1 to represent temperature. This is reasonable when one remembers the theory proposed for continuously reinforced concrete: Before the steel across an existing crack becomes overstressed another crack is formed. The steel stress at a crack reaches a maximum just before another crack forms, and this maximum is the same regardless of how low the temperature falls. With the concrete stressed to the breaking point, any force that might be expected to cause an increase in steel stress causes another crack in the concrete instead. Increased contraction due to falling temperature should cause additional cracks in the concrete without any increase in maximum steel stress. If this is correct, the maximum steel stress should be no greater at 0 F than at the time the first temperature crack appears. Although this concept seems reasonable, it does not agree with Lindsay's observations, which showed that the steel stress continued to increase as the temperature dropped. This is believed to be another case where the formula has been considered correct but has given the wrong answer, probably because some of the factors involved are not taken into consideration by the formula.

Many engineers are aware that concrete tested at low temperatures is stronger than the same concrete tested at normal summer temperatures. There are numerous references to this fact in the literature, but complete information to show the exact relation between strength and temperature has not been shown. One such reference (3) notes: "It has been found that the tensile strength of saturated concrete at 40 F is about 75 percent higher than it is at 60 F." At the international Federation of Prestressing, held in London in October 1953, a paper presented by Hill which credits Magnel as observing that at -40 F the modulus of rupture of concrete is from 2.1 to 3.1 times as great as at normal temperature. The direct tensile strength may be assumed to vary in a similar manner. The paper also states that at -40 F the modulus of elasticity is from 1.1 to 1.2 times as great as at normal temperature. On the Illinois test road the maximum steel stress was recorded at a concrete temperature of 13 F. The tensile strength of the concrete at this temperature was certainly considerably higher than at normal summer temperatures existing when the strength was determined. At 13 F the tensile strength may well have been twice as great as at 70 F. Probably the modulus of elasticity was only slightly higher than at normal temperatures.

A drop in the temperature of steel within the limits that occur in concrete pavement has a relatively small effect on its properties. The exact change in tensile strength of steel reinforcing bars between 70 F and 13 F depends on the chemical composition of the steel, but it appears that the strength at 13 F may have been on the order of 5 percent higher than at 70 F. Certainly the gain in strength of concrete due to lowering its temperature is much greater than the gain in strength of reinforcing steel. If this information is substituted in Eq. 1 the result is somewhat surprising. If a concrete strength at low temperatures is assumed twice as great as at normal temperatures, and a steel strength 5 percent greater, the required percentage of steel may be shown to be about 1.4 percent instead of 0.7 percent for normal temperature. It appears that as the temperature declines the stress in the steel increases because the concrete becomes stronger and is harder to break. This line of reasoning satisfied Vetter's statement that steel stress is independent of temperature, provided the strengths of the materials remain the same, as he assumed. It also satisfies observed results in Illinois, which showed that the steel stress increased as the temperature dropped, probably at about the same rate as the strength of the concrete increased.

Even though unproved theory is not reliable for design, theory does serve a useful purpose. For example, if one accepts the theory outlined in this paper, it would appear desirable to take steps to develop a frequent crack interval in the pavement before the weather gets cold. If enough flexibility in pavement length to provide contraction space for the minimum temperature has been developed by a frequent crack interval before cold weather occurs, the steel will not have to break the cold, high-strength concrete. As there is a fairly well-known relation between crack width and steel stress, and, as there is a definite amount of contraction space required to accommodate a definite temperature drop, it seems reasonable to assume that the more cracks there are, the less will be the steel stress.

The available evidence supports this view. The Illinois test section on which the 62,000-psi stress was recorded was laid on October 6, 1947. Construction on the project continued until operations had to be shut down for the winter. The road was not opened to traffic until the fall of the following year. Because of late season construction and the absence of traffic, the spacing of cracks at the time cold weather arrived was not as great as it would have been had the section been placed earlier in the season and promptly opened to traffic. In fact, published data show the average spacing of cracks in the section under observation was about 11 ft some two months after the maximum stress was recorded. Had there been more cracks to absorb the contraction in length, the stress in the steel might be expected to have been lower.

Another similar test section was installed in the spring of 1948, and the pavement was opened to traffic that summer. The crack interval at the time cold weather arrived is not known, but it would be expected to have been somewhat less than was the case on the previous test section. The maximum stress recorded on the second panel was 42,000 psi, or 20,000 psi less than on the first panel. The lower stress may have been partly the result of a smaller crack interval at the time winter arrived. It appears however, that late season construction is not a cause for serious concern because no difference in performance has been reported on any project due to date of construction.

For late season construction some engineers may prefer to take steps to hasten the occurrence of cracking. Two ways of doing this might be to eliminate curing, and to open the road to traffic as soon as possible. Elimination of curing in the fall of the year is not likely to be detrimental to continuously reinforced pavement. There are data to show that enough moisture is available in concrete pavements to cause a steady gain in strength over an extended period of time. It is possible that elimination of curing during late season construction might be beneficial to any concrete pavement constructed in northern states because it would allow a lower moisture content, which woul be helpful in resisting the effects of freezing and thawing during the critical early age of the concrete. Even though it is not possible to open the pavement to the public, crack formation could be promoted by running a few heavy trucks over the pavement at a time when other stresses were near a maximum.

CONCLUSIONS

The following statements appear to be correct:

1. After nine years of experience in Illinois, 0.5 percent steel has been enough to avoid broken steel and there is no indication that more than this amount results in better performan

2. Transverse cracks are formed in continuously reinforced pavement by a combination of drying shrinkage, warping, wheel loads, and falling temperature. The amount of steel required is that amount necessary to cause additional cracks in the concrete as the temperature drops, proper allowance being made for all stresses which help crack the concrete.

3. The maximum stress in the steel, caused by falling temperature, would theoretically be independent of the total temperature drop, if the strength of the constituent materials remained constant. The rather rapid increase in tensile strength of concrete as the temperature drops, probably is the reason steel stresses increase as the concrete gets colder.

REFERENCES

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