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

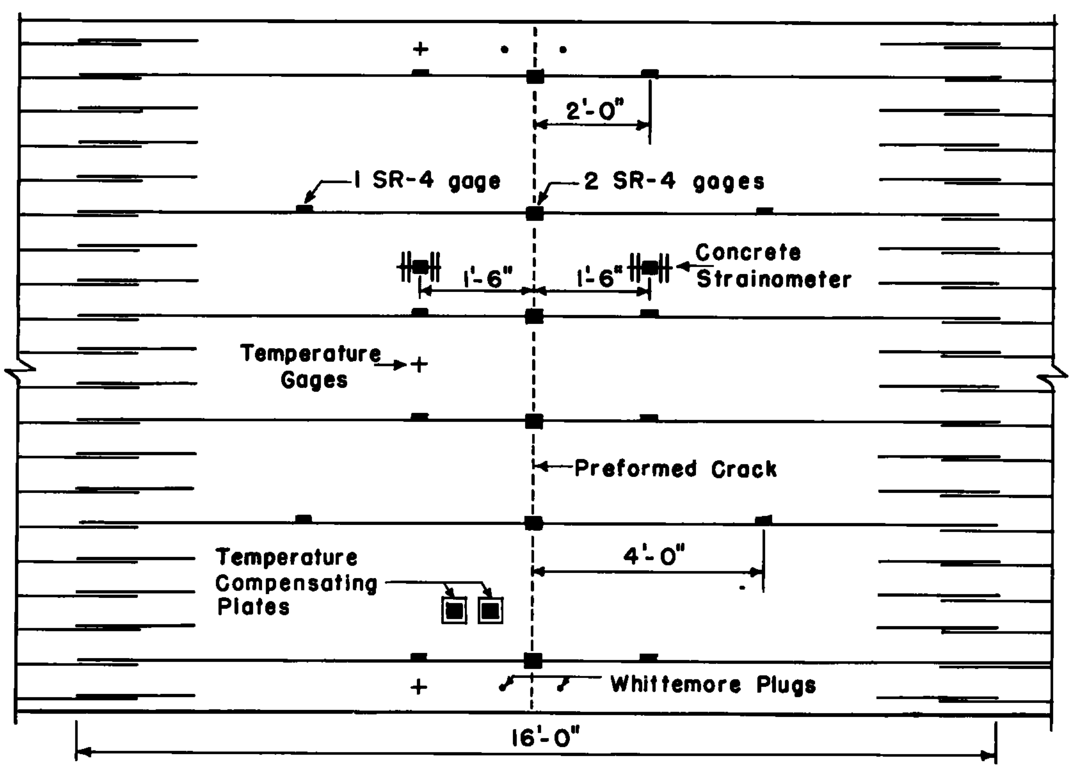

Figure 1.

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

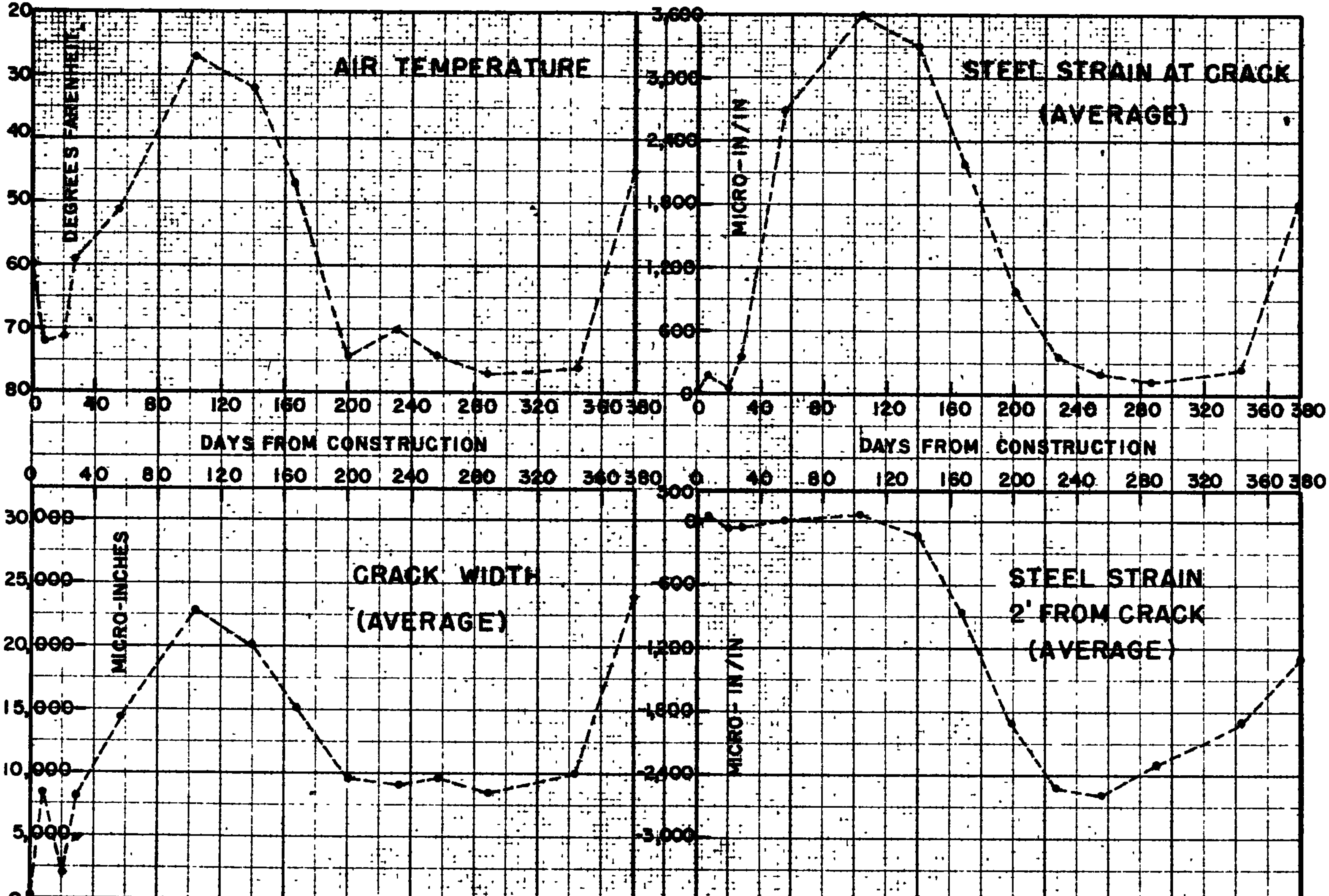

Figure 2. Strain history of York pavement.

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

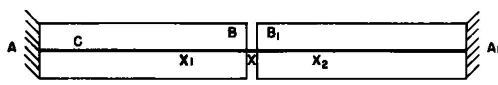

Figure 3.

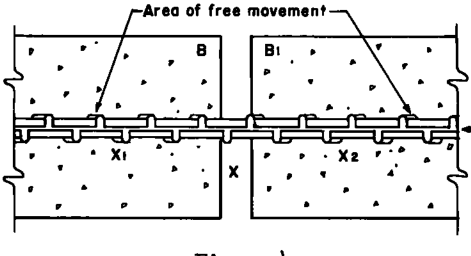

Figure 4.

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

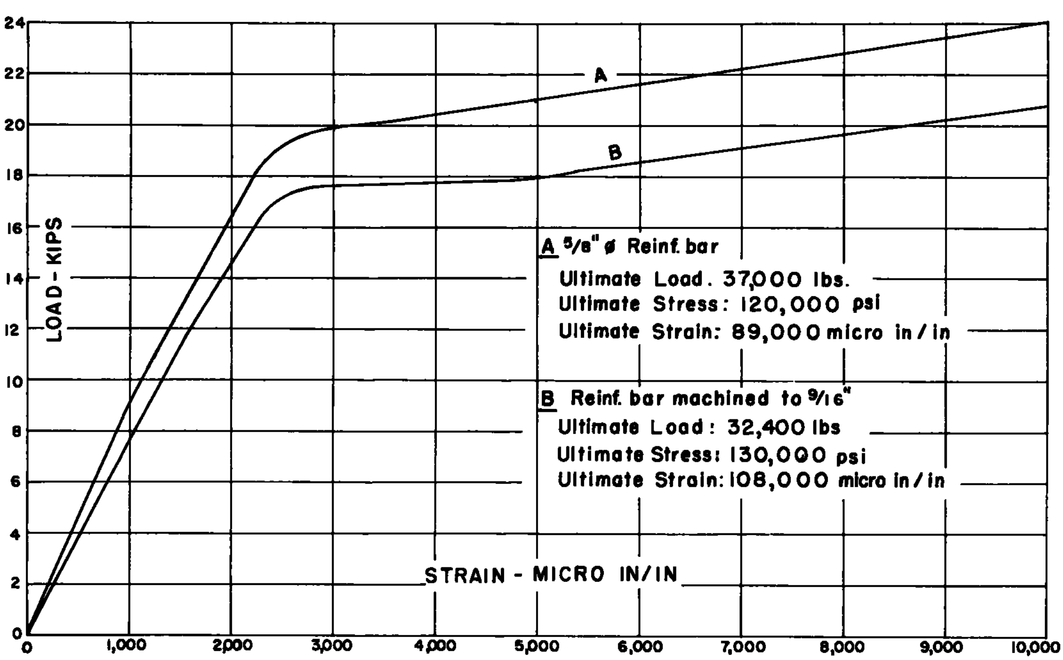

Figure 5. Load-strain curves of steel reinforcement used in York and Hamburg projects.

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

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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

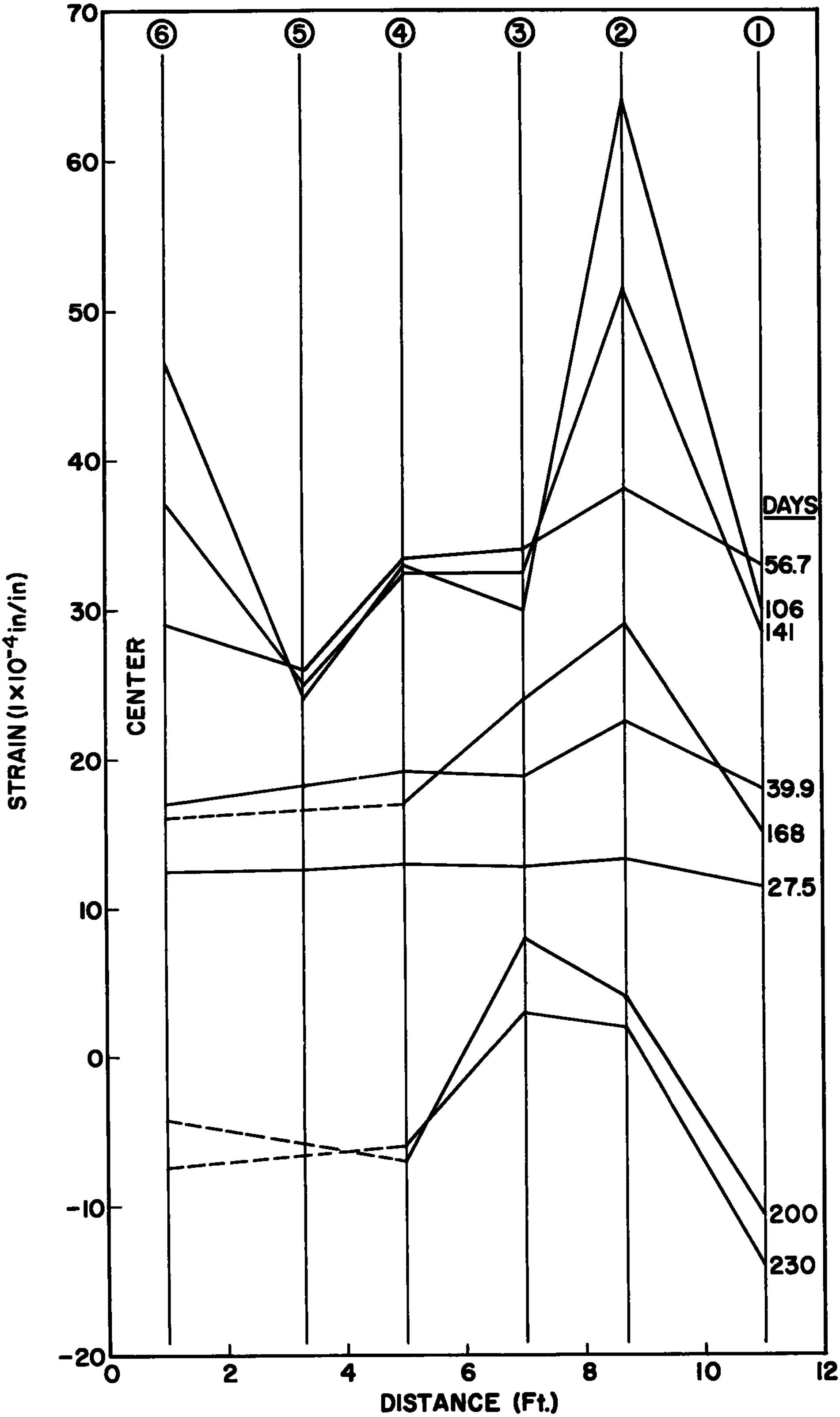

Figure 6. Transverse distribution of steel strain at preformed crack.

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.

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