Continuously reinforced concrete pavement research, 1958

Fritz Lab
1958

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May 6, 1958

Dear Sir:

Enclosed for your information is a copy of the final report on the investigation of continuously reinforced concrete pavements conducted for our Committee by Lehigh University.

Very truly yours,

C. A. WILLSON
Research Engineer

CAW:ds
enclosure

(Sent to the Committee on Reinforced Concrete Research and its Technical Subcommittee.)
Mr. C. A. Willson  
American Iron and Steel Institute  
150 East Forty-Second Street  
New York 17, N. Y.

Dear Mr. Willson:

I am sending you the complete report on the work done for your Committee by Lehigh University.

Since you have agreed to reproduce the report copies, I am sending you the original drawings, curves and photographs that will be required for this reproduction.

When you have finished with these originals, please return them for our files.

There will be some requests for report copies and, as we will not reproduce any, we would appreciate it if you would send us several when you make distribution.

Under our contract with the Pennsylvania Department of Highways we will continue to make observations of the pavement at York for another year or more. With our new projects and the considerable amount of research that will be conducted by others during the next few years, it seems likely that an excellent design for continuously reinforced pavement will evolve. Your group deserves a great deal of credit for the interest which has developed in this type of pavement since the start of the York job.

While we have not always agreed upon the information obtained from our research, I have enjoyed my association with you and the men in your group.

If we can be of further assistance to you on the recently completed York project or on any future projects which you may wish to sponsor, please let us know.

Very truly yours,

I. J. TAYLOR

IJT:ml
enclosure
CONTINUOUSLY REINFORCED CONCRETE
PAVEMENT RESEARCH

for

American Iron and Steel Institute

by

Continuous Pavement Research Group
Fritz Engineering Laboratory
LEHIGH UNIVERSITY
Bethlehem, Pa.

1958
This research has been conducted at:

LEHIGH UNIVERSITY
Fritz Engineering Laboratory
Department of Civil Engineering

as part of an investigation sponsored by:

American Iron and Steel Institute
Pennsylvania Department of Highways
U. S. Bureau of Public Roads
Lehigh University

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CONTINUOUSLY REINFORCED CONCRETE PAVEMENT RESEARCH
for: American Iron and Steel Institute

A. SYNOPSIS

In the fall of 1956, the Pennsylvania Department of Highways constructed the first of two experimental continuously reinforced concrete pavements on Route III, near York, Pennsylvania.

Under the sponsorship of the American Iron and Steel Institute, the Fritz Engineering Laboratory at Lehigh University, planned and installed an instrumentation arrangement to measure some of the phenomena which normally occur at cracks in a pavement of this type.

In this project, measurements were made of the strain in the bar-mat reinforcing steel in a uniform 9-inch pavement. In addition, studies were conducted of the crack frequency, the crack width, and the slab temperature.

This report describes a theory governing the behavior, the details of construction and instrumentation, and the results obtained during the first year of the life of the pavement.

B. INTRODUCTION

The trends in pavement design have moved slowly towards the use of longer pavement slabs. Historically, the first concrete pavement in the United States, at Bellefontaine, Ohio, consisted of plain concrete blocks five to six feet square and six inches thick. Under the influences of experience and research in the field, reinforced pavement slabs have increased in size to encompass 12-foot lane widths,
lengths of from 30 to 100 feet, and thicknesses of from 6 to 10 inches or more. The use of steel reinforcement has been utilized as a means of tying cracks in the pavement together, and to give the pavement slabs some additional flexural strength.

The design of conventional reinforced concrete highway slabs, with load-transfer joints every 30 to 100 feet, is based upon the assumption that minor thermal cracking will occur between the joints, and that all deformations will be mobilized at the load-transfer joints. This form of construction, though quite successful in the context of previous experience, is not the optimum condition that can be accomplished with reinforced concrete pavements.

For many years, highway engineers have given thought to the design and construction of reinforced concrete pavements, based on the principle that if cracks which form were held tightly together, there was no reason why the pavement cracking could not be permitted to occur at any natural interval. The mere existence of cracks does no significant damage; the damage occurs when these cracks become of such width as to permit pavement pumping and extreme strains in the steel due to vehicle loads. Thus we have a continuously reinforced concrete pavement where the logical emphasis would be taken away from stress considerations and be placed on the relation between the strain in the steel and the width of crack opening.

Such pavements have been constructed as experimental projects in New Jersey (1), California (2), Illinois (3), Indiana (4), Texas (5), and now in Pennsylvania.
Fundamentally, the behavior of a given continuously reinforced concrete pavement is dependent upon at least three factors: the shrinkage of the concrete; expansion and contraction caused by temperature; and strains induced by imposed external wheel loads.

The initial behavior of concrete is such as to initiate internal capillary forces which attempt to contract the concrete by what is generally referred to as "shrinkage." In reinforced pavements, the restraints offered by the bonded steel and the subgrade will establish net tensile forces in concrete. When this tensile force is of the magnitude of the rupture strength, a shrinkage crack will form.

The temperature mechanism of cracking for continuously reinforced pavements is quite similar, except that the contraction of concrete is not due to shrinkage, but caused by a decrease in temperature.

Once the first crack develops, the steel at the crack will try to maintain the continuity of the pavement. Since practically full restraint exists near each end of the pavement, any tensile strain that develops at 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 certain that the shrinkage and temperature-induced strain are in operation to varying degrees of magnitude at all times. Initially, the shrinkage concept is probably the predominant one, while at a later age the temperature mechanism will be the major one. Initially, all cracks are probably due to shrinkage, and these cracks may well set
the pattern of future behavior.

The effects of wheel loads on a continuously-reinforced pavement are such as to categorize the behavior as no longer rigid, but semiflexible, with almost complete interaction between the bonded cracked slabs, provided the aggregate interlock is not destroyed.

The objectives of the study made at Lehigh University were primarily of a factual nature for a highway under service conditions. To accomplish this, it was necessary to determine the behavior at a crack by measuring the crack width opening and the strains in the steel. In actuality, the strains in the steel are only of importance when they are of such magnitude as to permit a crack to open to objectionable widths. The basic study was made to determine the crack width and steel strain as a function of the thermal oscillations.

C. GENERAL DESCRIPTION

The experimental section of Route III is six miles north of York, and is a part of the main North-South highway in Pennsylvania, linking Harrisburg with Baltimore, Maryland. This highway is a dual-lane road with two 12-foot lanes in each traffic direction.

The total length of the continuously reinforced section is 11,559 feet. The pavement is uniformly 9 inches in thickness, resting on a 6-inch granular base course.

The longitudinal reinforcement for each lane consisted of twenty No. 5 hard-grade, deformed steel bars with a nominal diameter of 5/8-in. These bars, comprising a 0.5% cross-section area of the pavement, had been fabricated into mats 16 feet long, with seven No. 3 deformed bars
to provide transverse reinforcement, as shown in Fig. 1.

The mats were placed at the vertical center of the pavement on a 4-1/2 in. thick spreader run of concrete. Several mats were installed, allowing an end overlap of 1 ft. with adjacent mats to maintain reinforcement continuity. Then the paving equipment was backed up and the strike-off run of concrete was placed.

As shown in Fig. 2, midway between the ends of the pavement, in the outside North-bound 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.

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 at distances 3 ft. and 4 ft. to each side of the crack.

In order 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% reduction of cross-section in each gaged bar, or a 6% 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. Fig. 3.

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-hour period each 30 days as given in "Instrumentation Data" chart.

D. INSTRUMENTATION DETAILS

Six longitudinal reinforcing rods at the selected test panel were gaged to measure the strains occurring at the instigated crack and also at distances of 2 ft. and 4 ft. on each side of the crack. These rods were shipped directly from the manufacturer to Fritz Engineering Laboratory and were prepared for installation several weeks prior to the start of paving operations.

Type AB-3 wire strain gages were attached to the rods as strain transducers, Fig. 4. They were arranged to allow the measurement of direct strain, average strain, and bending strains; depending upon their position in the electrical instrument circuit.
To obtain a smooth surface suitable for attaching the gages, it was necessary to remove the deformations from the deformed bars. Their nominal diameter of 5/8-in. was reduced to 9/16-in. by machining in a lathe. This reduction in diameter, over a 2-in. long section of bar, provided adequate space for attaching and waterproofing the gages. It reduced the cross-sectional area of each rod 20% and, in a 9-in. thick pavement, reduced the total cross-sectional area of the steel reinforcement 6%. Also, it was found that a more ductile material resulted from the removal of the cold-worked outer surface of the deformed bar. Fig. 5.

Cylindrical ovens were designed to produce the correct amount of heat to cure the epoxy resins and synthetic rubber materials used to cement and water-proof the gages on the long rods. Fig. 6. Special jigs and clamps were fabricated and used to apply the required pressure to assure a good bond between the gages and the steel. A procedure was developed that allowed a continuous process with uniform gage installation on each of the reinforcing rods.

After all rods were gaged, completely wired, and checked for electrical stability, they were placed in wooden boxes to await shipment to the test site.

The alloy wire used in SR-4 gages is sensitive to temperature as well as strain. While this temperature sensitivity is very small compared with that of strain, it must be considered in cases where accurate measurements are to be made under varying temperature conditions.

The four-arm bridge circuit used in most electrical strain reading instruments provides a means for the correction of errors due to temperature changes. This involves the use of a temperature compensating gage that must be subjected to all of the conditions of the
active gage except strain. When this gage and the strain measuring gage comprise adjacent arms of a four-arm bridge circuit, only the strains in the active gage influences the output of the bridge.

Two temperature-compensating gages were prepared for use at the test panel, Fig. 7. A strain gage was attached to each side of a mild steel plate 3 inches long by 1 inch wide by 1/4-inch thick. The gages were waterproofed and the plate was inserted into a sponge-rubber envelope. This device, when placed in the concrete pavement, would be subjected to the same temperature conditions as the reinforcing rods, but would not be affected by external strain-producing loads.

As the paving operation approached the site of the test panel, all transducers and gaged rods were put into place. Fig. 8. At the instrumented panel, six of the bars comprising a standard mat section were removed and replaced with gaged bars. Small steel support frames were used to hold the reinforcing rods in the desired position until the concrete was placed.

A conventional Whittemore strain gage, with a 10-in. gage length and a range of 0.10-in. (and least count of 1/10 000-inch) was used to measure the width of the instigated crack at each test panel. A mild steel calibration bar provided a standard gage reference for measurements that would be made at regular intervals for an extended period of time.

Two brass plugs, 1/2-in. in diameter and 3 in. long, provided receptacles for the conic points of the gage. Prior to installation into the pavement, the plugs were placed in a jig and a thin metal strip soldered between them, Fig. 9. This maintained an exact separation of 10 inches between the counter-sunk holes that had been drilled into one end of each plug.
Two sets of plugs were placed on the surface of the pavement at the test panel. The crack was spanned by a set near each edge of the lane where the preformed shrinkage crack would form.

The plugs were inserted into the freshly poured pavement and allowed to remain until the concrete had hardened enough to provide a rigid bond. The metal connecting strip was then removed from the top of the plugs by melting the soft solder joints with the flame from a small gas torch. This left the plugs firmly anchored within the usable 0.10-in. range of the Whittemore gage.

When the metal connecting strips were removed, the "Zero" measurements were made and recorded.

Several types of temperature transducers were considered for use in the measurement of temperature at selected positions within the pavement structure.

While investigating the possibilities of a temperature sensitive resistance-wire measuring device, it became apparent that a properly designed gage of this type could be used with a conventional strain indicating instrument to obtain very accurate measurements. Also, it permitted making all of the electrical measurements with only one instrument, thus reducing the time and equipment required to collect data at the test site.

Small gages to meet this need were not commercially available, so special ones were designed and manufactured at Fritz Laboratory to suit the particular requirements.
The temperature sensitive element consisted of approximately 33 inches of No. Forty, 99.8% nickel wire with Formvar insulation. This wire was cut to a length having a resistance of 17 ohms at a temperature of 70°F. A loop was formed in the wire and the free ends were soldered to standard two-conductor gage leads. Only 1/8 of an inch of insulation was removed from the ends of the wires, and with a small soldering iron it was possible to make a sound solder joint without melting the adjacent insulation.

The attached loop was then wrapped around the two gage leads approximately 1/4 of an inch back from their ends, and the end of the loop was held in place with a drop of plastic cement. The comparative stiffness of the heavier gage lead wires prevented electrical shorting of the soldered ends until the water-proofing and protective coatings were applied to the gage.

The completed gage, Fig. 10, was only about 3/16 of an inch in diameter, and provided a means of obtaining temperature measurements at a precise location within the pavement structure.

In order to use these gages as an active arm of a 120 ohms-per-arm bridge circuit, it was necessary to add a fixed resistance of 103 ohms in series with the 17 ohms provided by the temperature sensitive nickel wire.

The fixed resistors were made with No. 34, cotton insulated, Advance wire, having a resistance of 7.52 ohms per foot. They were wound on fiber-glass cores and given a protective coating of insulating varnish. Several of these resistors were tested and found to have no significant sensitivity to temperature changes between 30°F Fahrenheit and 140°F. Since the nickel wire had a nominal temperature coefficient
of resistance of 0.006, while that of the Advance wire was only ±0.0002, it seemed reasonable to assume that any change in the resistance of the 120-ohm circuit would be due to the temperature sensitivity of the nickel wire in the transducer.

Extreme care in the construction of these gages enabled the use of the same calibration factor with an entire lot of 70 gages and with less than 1% differential error within a temperature range of 0°F and 120°F. When used with standard 120-ohm strain gage instruments, 175 micro in/in. of indicated strain was equal to 1°F change of temperature at the gage.

When an electric current flows in a circuit, heat is generated by the resistance of the conducting materials. With constant input voltage, the temperature of this material will rise until the dissipation of the heat by radiation or thermal conductivity is equal to the heat that is generated. It was found that this phenomenon caused a sudden rise in the indicated temperature from each of the gages as they were switched into the instrument circuit, but that after about 10 seconds, this "self-heating" stabilized and reliable measurements could be obtained.

All gages were calibrated at a known temperature after they were attached to the terminal board at the construction site. Before installation into the pavement, they were submerged in an insulated water bath and checked with a laboratory mercury-column thermometer. The indicated resistance of the gage at this static temperature formed the basis for all subsequent measurement.

Five temperature gages were installed in the pavement and base course. Gages #36 and #40 were placed at each edge of the lane at the center of the pavement depth. Three other gages were placed at the
Thirty days after the pavement was poured, the effects of colder weather started to become noticeable in the pavement, Fig. 11. With decreasing temperature, the instigated crack opened wider and tension strains increased in the reinforcing bars spanning the crack. Strains in the steel 2 feet away from the crack indicated a slight compression.

When strain in the steel bars at the crack had reached 2000 micro-inches per inch, it became apparent that the minor influences of warping, localized temperature distribution, and precise bar alignment has 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 at 52°F., the crack was open to a width of 15,000 micro-inches and the gaged bars across the crack were beginning to yield in tension at 2800 microin/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 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.
longitudinal center line of the pavement with gage #37 one inch into
the base course, gage #38 one inch up from the bottom of the slab and
gage #39 one inch below the pavement surface.

E. PAVEMENT BEHAVIOR

By October of 1957, the pavement at York had been under ob­
servation 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 very 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 very close relationship between strain in the reinforcing
steel, crack width, and air temperature, is shown in Fig. 11, where the
individual strains in the steel bars and pavement crack widths have
been averaged for simplicity of presentation. Individual gage data
have been plotted in Fig. 18 through 41.

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 upon the general strain pattern in the instrumented section. How­
ever, 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.
During January of 1957, when the pavement was 3 months old, measurements were made when the temperature was 22°F. The crack width had increased to 23,000 micro-inches and the gaged bars at the crack were strained to 3600 micro-in/in. in tension, or beyond the yield point. Tension strains in the gaged bars away from the crack had reached only 150 micro-in/in.

This was the lowest temperature at which readings were made on the gages, but temperature records at a nearby airport indicate that a low of 2°F. was recorded 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 4000 micro-in/in. and the crack opened to 27,000 micro-inches.

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 Spring and Summer of 1957, 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, while the crack width remained about 8000 micro-inches, the gaged bars at the crack were yielded in compression until they retained only 100 micro-in/in. of their former tensile strain. These same bars, at a distance of 2 feet away from the crack, were then yielding in compression with a strain of 3300 micro-in/in.
In order 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 hours later when the temperature had dropped to 25°F. During the following 3 days, the temperature fluctuated through a range of 30°F. and the measured crack width responded in proportion to these temperature changes.

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

Although the reinforcing steel was strain-sensitive to the changing temperature, there seemed to be very 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.

Figs. 12 and 13 show the phenomena which occurred during the development of the crack along the induced plane of weakness. Since 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 Al 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 Bl 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 first few days of pavement life prevented the reformation 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. 13.

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 X1 and X2 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 X1 and X2 on the bar indicated only mild tension strain.

Since the coefficient of expansion in 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, and therefore compressive straining occurred at this point before the crack was closed. Since 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 X1 and X2, 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 upon the bar. Compression strains at X1 and X2 were increased to the yield point while the bar at X was yielded in compression and returned very near 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 condition of tension.

**F. CONCLUSION**

The first cracks to appear in a continuously reinforced concrete pavement should be the result of shrinkage. This tends to fix the pattern for future cracking and probably will remain influential throughout the life of the pavement.

Although temperature and pavement base friction are the pre-dominant natural causes of stress in an established pavement, a new pavement is influenced by many other unpredictable variations of nature. Humidity, wind, sunshine and, in fact, anything which effects the dehydration of the fresh concrete and its bonding to the reinforcing steel
may result in a different crack pattern in pavements which are alike in
design but are constructed in different seasons or under different weather
conditions.

The effects of temperature induced contraction in the pavement,
acting against the friction of the base course, became predominate in the
test section at York when the pavement had cured less than twenty-four
hours. With dehydration shrinkage and temperature contraction occurring
simultaneously, the bond of concrete and steel was broken for several
inches on each side of the cracks. This action permitted the existing
cracks to open wider and resulted in longer uncracked sections.

A more ideal crack pattern will result if a pavement is allowed
to cure for approximately twenty-one days before exposure to extreme
changes in temperature. This allows most of the dehydration and shrinkage
to occur and is sufficient time to permit the concrete to establish a satis­
factory bond with the reinforcing steel. Temperature induced contraction
will then form new cracks instead of opening the original shrinkage cracks
to objectionable widths.

Temperature is the most damaging influence to which continuously
reinforced pavements are subjected. Tension strains during cold weather
have an obvious effect upon existing cracks and may cause several new
cracks to form, but the more subtle effects of warm weather compression
are equally important.

Concrete will creep under prolonged straining, and if this creep
is excessive it may not fully recover when the direction of straining is
reversed. Compressive strains measured in the test pavement remained very
high throughout the Summer of 1957. Some of this creep probably influenced
the behavior the following Winter and is responsible for the formation of
new cracks and a slight increase in the width of the older ones.
There is 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.

The first few annual temperature cycles may stabilize the pavement straining within the elastic range of the steel and the recoverable 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.

After one year of pavement life, several of the cracks appear inactive. Slight spalling has occurred at their top edges, but there is no longer any visible separation of the pavement sections. The steel
reinforcing at these cracks probably has never yielded and the bond loss is sufficient to permit the steel to strain within its elastic limit. Infiltration during periods of contraction has filled the cracks and the steel remains in a state of elastic tension.

This crack stabilization has occurred in only a few places. In each case it has occurred in sections where the crack spacing is from two to six feet. The average crack pattern throughout the entire pavement is from seven to ten feet and there are some sections up to twenty feet in length without cracks.

When crack openings exceeded .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.

At extremely low temperatures crack widths up to 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 1/16 in. or more should be looked upon with suspicion. They should not occur in a pavement of proper design.

Strain cycling caused some additional loss of bond along the bar. The approximate amount could be determined when the crack width, steel strain and steel characteristics were known. This bond loss will increase 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 which was used in the pavement at York yielded at a strain of 2,700 micro in/in., yet a strain of 96,000 micro in/in. would have been required to cause rupture.
When observed from the edge of the pavement, many of the cracks which have top surface openings up to .010 inch wide appear to be entirely closed at the bottom surface.

The bottom surface absorbs moisture from the ground and, in cold weather, may obtain some heat. This increases the volume in the lower portion of the pavement and accounts for some of the difference between the top and bottom surface crack width. Also, the slight flexing action of wheel loads causes dislocation of fine particles in the top portion of the pavement. These move downward and, in the moist lower portion, provide an effective sealer. As foreign matter infiltrates the top portion of the crack it is then retained, and if the crack is of moderate width, a process is started which may lead to eventual stabilization.

With 0.5% reinforcing steel the crack pattern is erratic and some of the cracks have opened sufficiently to allow the steel to yield. Although the pavement appears sound and in no immediate distress, it is believed that, in a nine inch thick pavement, 0.5% reinforcement is the minimum amount that should be used.

A more ideal crack pattern will result when the total yield strength of the steel in a pavement is greater than the total tensile strength of the concrete. This would cause new cracks to form before the existing cracks were opened to objectional widths.

It is quite possible that the construction of an eight inch thick pavement, with the same volume and type of reinforcement as that used in the nine inch test pavement, would result in a better crack pattern and hence a more durable highway.
There still remain many questions as to the ideal combination of steel percentage, type of bonding surface, pavement thickness and size of reinforcing bars. The test pavement at York, Pennsylvania will furnish information on one of these combinations and should provide a smooth maintenance-free highway for many years.

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.

G. ACKNOWLEDGMENT

The authors wish to acknowledge the assistance of the American Iron and Steel Institute as sponsors of this project. The suggestions and advice of their Subcommittee on Highway Field Research were invaluable in planning and conducting a test of this type.

We wish to express our appreciation to the personnel of the Pennsylvania Department of Highways and the United States Bureau of Public Roads for their continuing assistance and cooperation.

Much of the work at Lehigh University was performed by technical staff members of the Fritz Engineering Laboratory. Their willing assistance is gratefully acknowledged.
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PROPERTIES OF MATERIALS
A.I.S.I. PROJECT

REPORT ON AIR-ENTRAINED CONCRETE
(Pennsylvania Department of Highways Tests)

1. Proportions Used. 1 - 1.79 - 3.5
2. Cement Factor/Cu. Yd. - 6.25
3. Water per bag - low of 4.88 to high of 5.32
4. Slump - Low of 1-3/8 to high of 2-5/8
5. % Entrained Air - low of 2.0 to high of 3.2
6. Type of Mixer - 34E
7. Air Temperature ranged from 40° to 86°

STRUCTURAL PROPERTIES OF CONCRETE
(Pennsylvania Department of Highways Tests)

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REPORT ON REINFORCING STEEL
(Fritz Laboratory - Lehigh University)
(Average of 10 specimens - variation between specimens less than 1% of maximum)

Size 5/8 inches
Area 0.31 sq. in.
Yield stress 68,000 psi
Ultimate Stress 120,000 psi
Elongation 9%

The steel conformed to ASTM Specifications
## INSTRUMENTATION DATA
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BAR-MAT REINFORCEMENT
(9 INCH PAVEMENT)

FIGURE NO. 1
GAGE POSITIONS IN TEST SECTION - YORK, PA.

1 SR-4 gage
2 SR-4 gages
1' - 6"
1' - 6"
Concrete Strainometer
Temperature Gages
37, 38, & 39
Preformed Crack
 Temperature Compensating Plates
Whittemore Plugs

FIGURE 2
Fig. 3 Gage Terminal-board Installation

Fig. 4 Attaching and Waterproofing SR-4 Gages
LOAD-STRAIN CURVES OF STEEL REINFORCEMENT USED IN YORK AND HAMBURG PROJECTS

A — 5/8" Reinforcing Bar
Ultimate Load: 37,000 lbs.
Ultimate Stress: 120,000 psi
Ultimate Strain: 89,000 micro in/in

B — Reinforcing Bar machined to 9/16"
Ultimate Load: 32,400 lbs.
Ultimate Stress: 130,000 psi
Ultimate Strain: 108,000 micro in/in

FIGURE NO. 5
Fig. 6 Heat Curing Gage Attachment Cement

Fig. 7 SR-4 Gage Temperature Compensation Plates
Fig. 8 Field Installation of Gaged Bars

Fig. 9 Jig for Whittemore Gage Plugs
Fig. 10 Completed Temperature Gages

Fig. 15 Whittemore Plug Installation
FIGURE NO. 12

Area of free movement

FIGURE NO. 13
Crack development near instrumented panel in North-bound traffic lanes after one year of pavement life.

FIGURE NO. 14
Fig. 16 Support Chairs for Bar-Mat

Fig. 17 Instrument Terminal Board in Use
GAGE 6

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

FIGURE 19
Figure 21: Graph showing strain in micro-inches per inch over time elapsed between constant and gage reading, in days.
TIME ELAPSED BETWEEN CONSTRUCTION AND GAGE READING - DAYS

FIGURE 22
FIGURE 23

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

GAGE II

STRAIN - MICRO INCHES PER INCH

0 40 80 120 160 200 240 280 320 360

3000
2000
1000
0 1000 2000 3000
FIGURE 24

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

GAGE 14

STRAIN - MICRO INCHES PER INCH

0 40 80 120 160 200 240 280 320 360

0 1000 2000 3000

-1000 -2000
FIGURE 25

GAGE 15

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

STRAIN - MICRO INCHES PER INCH

0 40 80 120 160 200 240 280 320 360

-1000 1000 2000 3000 4000
FIGURE 27

GAGE 17

STRAIN-MICRO-INCHES PER INCH

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS
FIGURE 30

GAGE 20

TIME ELAPSED BETWEEN CONST. AND GAGE READING — DAYS

STRAIN — MICRO INCHES PER INCH

-3000
-2000
-1000
0
1000
2000
3000

0
40
80
120
160
200
240
280
320
360
FIGURE 31

GAGE 21

STRAIN-MICRO INCHES PER INCH

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

0 40 80 120 160 200 240 280 320 360

0 1000 2000

-1000 -2000 -3000
GAGE 22

TIME ELAPSED BETWEEN CONSTANT GAGE READING - DAYS

FIGURE 32
FIGURE 33

GAGE 23

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

STRAIN - MICRO INCHES PER INCH

0  40  80  120  160  200  240  280  320  360

-4000  -3000  -2000  -1000  0
Figure 35

Time elapsed between const. and gage reading - days

Strain - micro inches / per inch

Gage 25
FIGURE 38

GAGE 28

TIME ELAPSED BETWEEN CONST. AND GAGE READING - DAYS

STRAIN-MICRO INCHES PER INCH

0 40 80 120 160 200 240 280 320 360
0 1000 2000
-1000
-2000
-3000

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FIGURE 39

AIR TEMPERATURE

TIME ELAPSED AFTER CONSTRUCTION - DAYS

0 40 80 120 160 200 240 280 320 360