NRC. HIGHWAY RESEARCH BOARD.
11

Bulletin 181

Continuously Reinforced Concrete Pavement

Full-Scale and Model Tests

National Academy of Sciences—
National Research Council
publication 548

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PRESENTED AT THE
Thirty-Sixth Annual Meeting
January 7-11, 1957

1958
Washington, D. C.

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Contents

Design of Continuously Reinforced Concrete Pavement

WAYNE R. WOOLLEY, Highway Research Engineer,
Truscon Steel Division, Republic Steel Corp., Youngstown, Ohio

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

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

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

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

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

In which:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

CONCLUSIONS

The following statements appear to be correct:

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

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

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

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Preliminary Report on Continuously Reinforced Concrete Pavement Research in Pennsylvania

R. L. SCHIFFMAN, Assistant Professor of Civil Engineering, I. J. TAYLOR, Instruments Associate, Fritz Engineering Laboratory, and W. J. ENEY, Professor of Civil Engineering, and Director of Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

In the Fall of 1956, the Pennsylvania Department of Highways constructed the first of two experimental continuously-reinforced concrete pavements. This paper is a preliminary report on the construction procedures and investigatory details of these research pavements.

The first of these projects is on Route 111, near York. In this project, measurements are being made of the strain in the bar-mat reinforcing steel in a uniform 9-in. pavement. Studies are also being conducted on the crack frequency, the crack width, and the slab temperature.

This paper describes a theory governing the behavior, the details of construction and instrumentation, and the results obtained during the first two months of the pavement's life.

The second project will be on US 22, near Hamburg; the pavement thickness will be varied to include sections of 7-, 8-, and 9-in. thickness. The reinforcing will be bar-mat, with the exception of 1,000 ft of welded wire fabric. In addition to strain measurements, studies of temperature distribution, pavement warping, longitudinal movement, crack width, and crack frequency will be made. This paper describes only the plans for this research project.

●THE TRENDS in pavement design have been towards the use of longer pavement slabs. The first concrete pavement in the United States (at Bellefontaine, Ohio) consisted of plain concrete blocks 5 to 6 feet square and 6 in. thick. Under the influences of experience and research in the field, reinforced pavement slabs have increased in size to encompass 12-ft lane widths, 30- to 100- ft lengths and thicknesses of from 6 to 10 in. or more. The use of steel reinforcement has been added as a means of holding pavement cracks together, and it gives the pavement slabs some additional flexural strength.

The design of conventional reinforced highway slabs, with load-transfer joints every 30 to 100 ft, is based on 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, although 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 that form are held tightly together, there is no reason why the pavement cracking cannot be permitted to occur at any natural interval. The mere existence of cracks does no damage; the damage occurs when the cracks become of such width as to permit pavement pumping and extreme strains in the steel due to vehicle loads. Therefore, a continuously reinforced concrete pavement where the reinforcement is of such size as to hold the cracks to non-objectionable width is feasible.

The logical emphasis thus 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, 2), California (3), Illinois 4, 5), Indiana (6, 7, 8, 9, 10), Texas (11) and now in Pennsylvania.

Fundamentally, the behavior of a continuously reinforced concrete pavement is dependent on at least three factors: 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 bonding of the concrete to the steel and the subgrade will establish net tensile forces. When this tensile force is of the magnitude of the rupture strength of the concrete, a shrinkage crack will form.

The temperature mechanism of cracking, for continuously reinforced pavements, is quite similar. For a given temperature drop, there will be less total displacement than for a slab unrestrained by a subgrade. The magnitude of horizontal movement will be dependent on the restraining horizontal shear stresses mobilized between the subgrade and the pavement. The effect within the pavement will be to establish a net tensile force, restraining movement. The magnitude of these forces, at any point, will depend on the temperature drop and the resultant net movement. The distribution of forces will start from zero at the free end, and build up to full restraint at some distance from the free end. At some point relatively close to the free end the tensile strength of the concrete will be reached, and the slab will crack.

Once the first crack develops, the steel will form a bonded dowel at the crack and will maintain the continuity of the pavement.

Under this conception of temperature deformations only, the restraint stresses will build up to a point of complete restraint throughout the central portions of the slab. Subsequently, in theory, the frequency of the initial temperature cracking will increase towards the center section of the pavement. In the subsequent temperature cycles, the inelastic deformation of the subgrade will play a considerable part in the partially restrained portion. At the end of the initial temperature drop the slab, except for the free ends, will be in a state of tension.

When the temperature reverses, there will be a tendency for the cracks to close as the tensile strains are relieved. Because the closing of the cracks depends on the ability of the pavement, in the partially restrained areas, to move over the subgrade, and because the movement is not completely reversible, the pavement will not, for the same temperature change, return to its unrestrained condition. Thus, residual tension strains will be established, and these strains will become cumulative near the ends where relatively large movements can occur, and almost zero in the central portions of the slab where there is little or no movement.

After several cycles of build-up of residual strains, the end portions of the slab will be strained in tension such that they will crack, and thus a high frequency of cracking will occur at some nominal distance from the free end. This process will continue until the subgrade soil is so strain-conditioned as to reach a state of force-deformation reversibility. After a period of several years, excluding load efforts on the pavement, the total range of temperature variations will be achieved, and the crack pattern will stabilize. This type of crack pattern was observed in Illinois (5).

It is certain that the mechanisms of capillary shrinkage and temperature expansion and contraction are in operation to varying degrees of magnitude at all times. Initially, the shrinkage concept is probably the predominant one; at a later age the temperature mechanism will be the major one. Initially all cracks are probably due to shrinkage, and these cracks 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 semi-flexible, with almost complete interaction between the bonded cracked slabs, provided the aggregate interlock is not broken.

In addition to other effects, previously described, there exists a tendency for the pavement to warp under the influence of unequal temperature distributions.

The objectives of the study being made at Lehigh University are primarily of a factual nature for a highway under service conditions. On the hypothesis that the previously mentioned mechanisms are correct approximations of the actual behavior of the pavement, certain measurements are basic to the proof of validity of the behavior mechanism. In the first instance, it is necessary to measure behavior at a crack to determine 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 objectional widths. The basic study being made is to determine the crack width and steel strain as a function of the thermal oscillations.

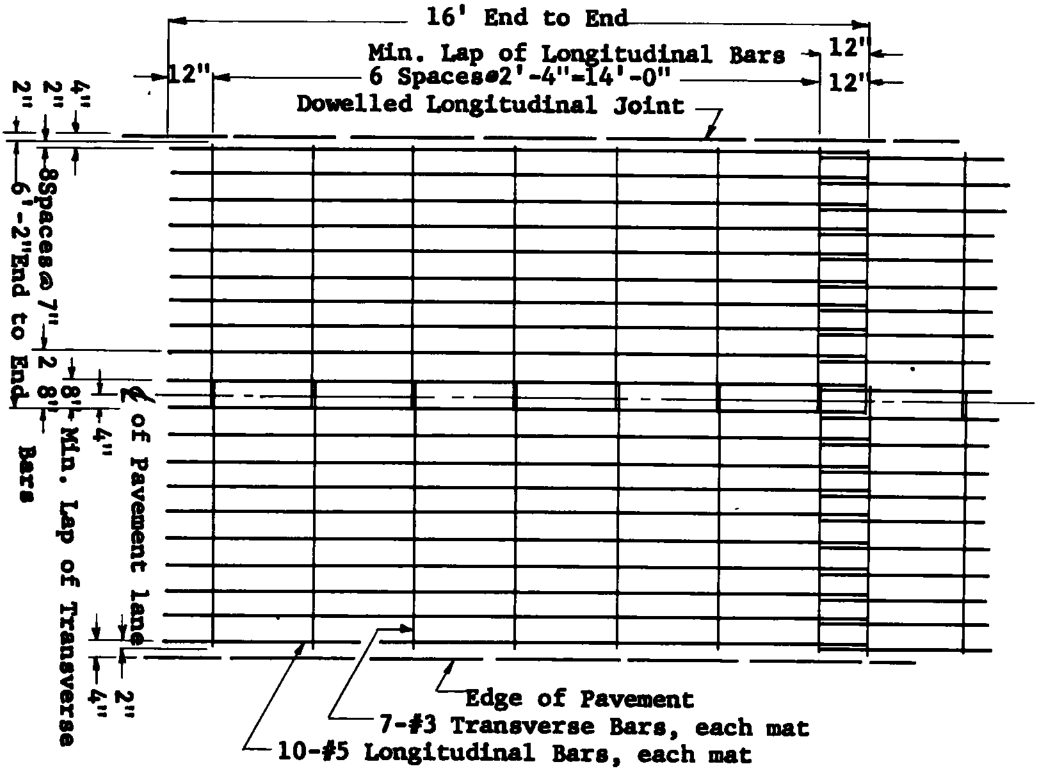

Figure 1. Bar-mat reinforcement (9-in. pavement).

YORK PROJECT

The first study of continuous pavements in Pennsylvania is being made on an experimental section of US 111 six miles north of York. This highway is a dual-lane road with two 12-ft lanes in each traffic direction, and is a part of the main north-south highway linking Harrisburg, Pa., with Baltimore, Md.

The total length of the continuously reinforced section is 11, 559 ft, the pavement being uniformly 9 in. thick and resting on a 6-in. granular base course. The 0.48 percent of longitudinal reinforcement is provided by bar-mats 16 ft long (Fig. 1), the pavement design, set by the Pennsylvania Department of Highways, called for the reinforcement to be placed at the pavement mid-thickness.

The south end of the continuously reinforced concrete pavement joins the standard pavement by means of a finger-type bridge expansion joint at Station 971+70. The north end of the experimental pavement joins the standard pavement by means of four standard expansion joints spaced 61.5 ft apart in the standard pavement, beginning at Station 1087+29.

At the end of a day's pour the construction joint was formed by a bulkhead, with continuous reinforcement running through the bulkhead a minimum distance of 5 ft. Table 1 gives the location of the daily construction joints, the date paved, and the air temperature variations during the day. It should be noted that the south end of the job was bulkheaded at Station 972+99 without placing the finger-type joint, which was to be installed in the Spring of 1957.

The pavement was poured in 12-ft lanes. The juncture between lanes was accomplished by use of a keyed joint and ½-in. round hook bolts, spaced 5 ft on centers.

SUBSURFACE SOIL CONDITIONS

The subgrade construction of this highway was accomplished in accordance with the

TABLE 1
STATIONING OF DAILY CONSTRUCTION JOINTS

Lane	Date Paved	Beginning of Pour	End of Pour	Length, ft	Temperature (° F)
Northbound inside	9-18-56	1087 + 29	1065 + 03	2,226	50 - 71
	9-19-56	1065 + 03	1043 + 91	2,112	42 - 66
	9-20-56	1043 + 91	1024 + 46	1,945	40 - 59
	9-21-56	1024 + 46	998 + 52	2,594	32 - 57
	9-25-56	998 + 52	972 + 99	2,553	44 - 62
Southbound inside	9-26-56	1087 + 29	1064 + 15	2,314	43 - 55
	9-27-56	1064 + 15	1043 + 05	2,110	45 - 53
	10- 1-56	1043 + 05	1018 + 93.5	2,411.5	50 - 68
	10- 2-56	1018 + 93.5	991 + 30	2,763.5	50 - 70
	10- 3-56	991 + 30	972 + 99	1,831	55 - 72
Northbound outside	10- 9-56	1087 + 29	1063 + 22.7	2,406.7	42 - 64
	10-10-56	1063 + 22.7	1035 + 09.2	2,513.2	38 - 56
	10-11-56	1035 + 09.2	1010 + 85	2,724	30 - 59
	10-12-56	1010 + 85	981 + 74	2,911	34 - 61
	10-15-56	981 + 74	972 + 98	876	41 - 83
Southbound outside	10-15-56	972 + 98	984 + 53	1,154	41 - 83
	10-16-56	984 + 53	1013 + 35	2,882.57	43 - 55
	10-17-56	1013 + 35	1043 + 78	3,042.53	54 - 83
	10-18-56	1043 + 78	1071 + 81	2,803	54 - 76
	10-19-56	1071 + 81	1087 + 29	1,547.5	45 - 65

usual procedures of the Pennsylvania Department of Highways. The normal procedure was to place all fills in 8-in. loose lifts, and to compact with a roller until satisfactory compaction, evidenced by non-movement of the soil beneath the roller, was achieved. In general, subgrade compaction was achieved by use of a sheepsfoot roller and/or a flat wheel roller whereas the base course was compacted by a flat wheel roller.

Table 2 gives a record of the subgrade soil conditions, in regard to the soil size and plasticity characteristics, for typical soil groups. These samples were taken at the elevation of the finished grade.

Several in-place density determinations by sand cone method were made in the area of the top lift of the fill sections. The results are presented in Table 3.

The residual soil mantle was derived from the underlying Brunswick red shale. In general terms, this deposit is a triasic deposit with up to 5 ft of soil mantle overlying 10 to 20 ft of weathered rock.

The soil samples were taken at intervals of 200 to 400 ft along the entire length of highway. The maximum depth of sample was 3 ft below subgrade elevation.

In terms of the AASHO soil classification system, the general run of the subgrade soils was in the group classification of A-4, with a maximum group index of 8. There were a few samples of group classification A-2-4, and of A-6. The liquid limit moisture content for most of the subgrade soils was between 23 and 28, with a few between 28 and 35. The plasticity index for the majority of the soils was under 10, with a few exceptions, in which case it was limited to a maximum of 18. The fraction finer than the No. 200 sieve varied from 27 to 84 percent, with the majority of the samples showing about 50 percent passing the No. 200 sieve.

In terms of rating the soils as to potential performance as a highway subgrade (6), the high percentage of silty and clay soils indicates that the potentialities for frost heaving would be moderate to objectionable for ground water conditions within 6 ft of the pavement. The subsurface drainge of these soils would be poor, and they would exhibit a potential capillary rise of from 7 to 20 ft. The over-all expected maximum dry densities would range from 110 to 120 lb per cubic foot. It is to be expected that the sub-

TABLE 2
IDENTIFICATION CHARACTERISTICS OF SUBGRADE AND BASE COURSE SOILS
PERCENT FINER THAN

Size	Sta. 971+70 to 984+50	Sta. 984+50 to 998+50	Sta. 998+50 to 1012+50	Sta. 1012+50 to 1028+50	Sta. 1028+50 to 1042+50	Sta. 1042+50 to 1054+50	Sta. 1054+50 to 1070+50	Sta. 1070+50 to 1087+27	Base Course
1½ in. sieve	100	100	100	100	100	100	100	100	100
¾ in. sieve	97-100	100	98-100	100	100	100	77-100	100	87
No. 4 sieve	91-100	93-100	94-100	93- 99	96-100	98- 99	71- 97	87- 97	53
No. 10 sieve	88- 99	89- 99	90- 97	88- 97	92-100	94- 98	67- 95	80- 92	29
No. 20 sieve	85- 98	83- 98	82- 95	86- 96	86-100	92- 98	65- 93	78- 87	27
No. 40 sieve	64- 95	72- 94	67- 89	79- 92	69- 99	88- 97	59- 92	72- 85	20
No. 60 sieve	46- 92	61- 90	49- 82	66- 81	55- 99	79- 96	50- 90	64- 81	15
No. 100 sieve	37- 89	51- 86	37- 76	66- 73	47- 97	67- 94	40- 87	54- 78	12
No. 200 sieve	29- 84	41- 81	29- 69	42- 69	36- 88	48- 82	31- 80	40- 75	7
0 005mm	23- 49	27- 57	22- 42	26- 40	28- 40	25- 34	21- 29	25- 32	---
Liquid limit	23- 35	24- 38	23- 33	22- 28	24- 31	23- 27	23- 29	24- 29	---
Plasticity index	3- 14	2- 18	2- 13	2- 9	4- 8	2- 5	0- 10	2- 8	---

grade soil would have normal capillary saturation of about 100 percent of optimum moisture, which ranges from 10 to 14 percent by weight. These soils are susceptible to pumping. The foregoing performance ratings indicate that the permanence of the compacted fill, in terms of its relative supporting ability, is fair to poor. These soils are subject to considerable loss of support due to climatic variations.

The measured in-place densities varied from 94 to 105 percent of optimum Proctor density. The densities were taken from an exposed surface that had been subjected to several weeks of heavy traffic and the strengthening effect of capillary evaporation.

Under the local climatic conditions the free-draining 6-in. compacted base course should alleviate most of the potential frost heaving, pumping, and drainage conditions. In addition, the base course will act as a load-distributing mechanism to spread the traffic load over the subgrade, thus tending to alleviate the potential loss of support.

To determine the fluctuations of the ground water level, seven peizometers are being placed in key locations in the experimental section.

PAVEMENT CONSTRUCTION MATERIALS AND PROCEDURES

The 9-in. experimental pavement was constructed as a portion of the regular contract for the entire highway section. The contract specifications and plans call for the Pennsylvania conventional 10-in. pavement to be installed at either end of the experimental pavement. The southern section of the standard pavement was installed in the Fall of 1956. The 10-in. pavement on the northern end is being installed in the Spring of 1957.

Air-entraining cement was used for a design mix of 1: 1.79: 3.5, and 5 gal. of water

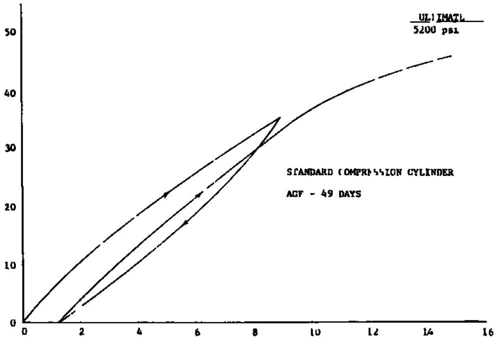

Figure 2. Compression stress-strain curve for concrete cylinders.

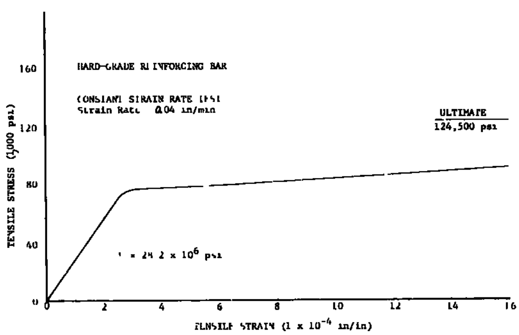

Figure 3. Tension stress-strain curve, reinforcing bars.

per sack of cement. The percentage of entrained air, on a volumetric measure, varied from 2.2 to 3.3. The slump of the concrete varied from 1⅞ to 2⅞ in. The design slump was 1¾ in.

Structural properties of the concrete were determined by the Pennsylvania Department of Highways in the course of its routine highway testing. The tests performed were on plain concrete beams, one being poured on each day of paving, and standard concrete cylinders taken sporadically along the highway. The results of these tests are presented in Table 4.

Six concrete cylinders, taken from the test-panel area, were sent to the Fritz Engineering Laboratory of Lehigh University. Two cylinders were tested on the 49th day after the pour. The average stress-strain results of these tests are shown in Figure 2. The remaining cylinders will be tested at various time intervals in the first year of pavement life.

The reinforcing steel conformed to ASTM Specification A15-54T for hard-grade steel, and A305-54T for deformations. Based on mill reports, the steel yield point had a range of from 60,800 to 90,400 psi, with an average of 77,800 psi. The tensile strength ranged between 92,000 and 134,700 psi, and averaged 118,500 psi. The elongation in an 8-in. gage length varied from 10 to 14 percent, with an average of 11.3 percent.

TABLE 3
SUBGRADE SURFACE COMPACTED DENSITIES

Location	Compaction,[a] %
975+00	99.0-101.7
979+00	95.7-100.7
982+00	96.8- 97.5
987+00	96.3-101.6
990+00	100.3-100.9
993+00	94.1-100.6
996+00	97.9- 99.1
999+00	99.9-102.5
1002+00	98.2- 99.2
1005+00	100.0-102.0
1008+00	102.0-102.4
1011+00	103.0-104.0
1014+00	101.7-102.3
1017+00	101.3
1027+00	100.0
1037+00	100.4
1042+00	95.8- 98.2
1047+00	101.3-105.3
1057+00	102.0
1067+00	95.4-100.3
1077+00	99.8-100.1
1087+00	105.0

[a] Percent of standard Proctor compaction test.

A stress-strain curve (Fig. 3) based on tests at Fritz Laboratory of two randomly-selected bars gaged with a 2-in. extensometer was used as a calibration to convert pavement reinforcing steel strain readings to stresses.

In general, the construction followed the usual procedures used in Pennsylvania; the variation was only slight in the vicinity of the gaged area.

The concrete from the mixer was dropped in front of a screw-type mechanical spreader, struck off at 4½ in., and vibrated directly behind the spreader, along the edges of the lane. The reinforcing steel, fabricated into bar-mats, was placed immediately behind the spreader. Two 6-ft wide mats were then placed and lapped longitudinally a full 12 in. to provide continuity of steel (Fig. 1). After several lengths of reinforcement were placed, the spreader moved back and completed the remaining 4½ in. of pavement, the finishing equipment moving immediately behind the spreader. Curing paper was used on the pavement a minimum of 72 hr after pouring.

At the end of a day's pour, a split-header board bulkhead was constructed,

TABLE 4
STRUCTURAL PROPERTIES OF CONCRETE
(Pennsylvania Department of Highways Test)

Test	Age, days	No. of Samples	Results, psi Min.	Avg.	Max.
Beam	10	7	590	660	710
Beam	11	4	630	660	685
Beam	12	2	610	660	710
Beam	13	1	---	690	---
Cylinder	3	6	1,970	2,210	2,450
Cylinder	7	2	3,110	3,275	3,440
Cylinder	10	8	2,770	3,260	3,720
Cylinder	28	5	3,770	4,200	4,840

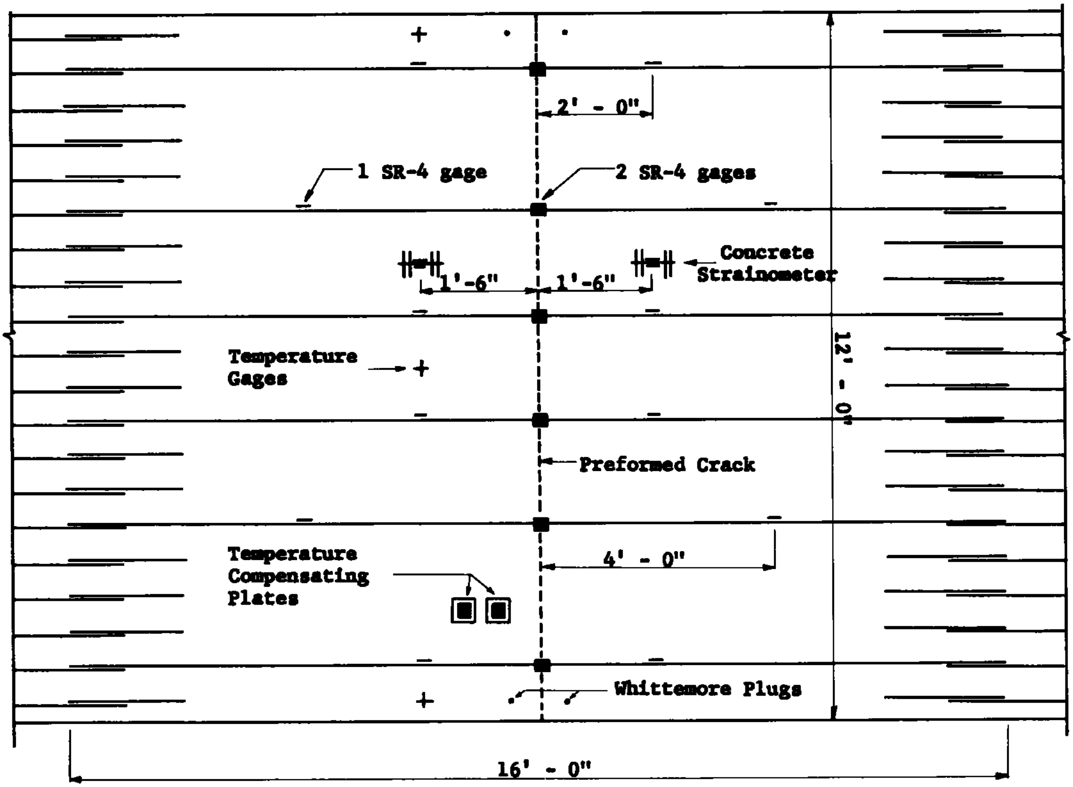

Figure 4. Instrumentation plan, York project.

with reinforcing steel running continuously through the joint.

Test Section Construction

At the test section, where the strain and temperature instrumentation was installed, a slightly modified procedure was followed. In this area, an instrumented bar-mat was installed on chairs well in advance of the pavement and so placed that it would be in the middle of a day's pour. The first lift of concrete was placed up to the instrumented mat. Concrete was omitted over this area, and then placed several mat lengths in advance of the test area. The regular steel was then placed, working back from the test rods a distance of 75 ft.

After the steel was fully placed, both behind and in front of the test area, the spreader was moved back and the second lift was completed. When the spreader reached the test area, a full 9 in. of concrete was poured very carefully to avoid disturbing the instrumentation. No vibration was permitted in this area. Instead, the concrete was carefully rodded to avoid honeycombing. Examination of the pavement edge after stripping the forms indicated that, at least in the examined area, the honeycombing was kept to a minimum. Once the concrete was placed, the finishing procedure followed the normal pattern.

Test Section Instrumentation

A single instrumented test section was installed in the York project. It was at this test panel that the direct measurements of strains in the steel and concrete were made. The test section was located at Station 1051+50, a distance of 3, 569 ft from the north end and 7, 980 ft from the south end of the experimental pavement, in the outside northbound lane.

The purpose of establishing the test panel was to create a crack in as natural a man-

Figure 5. Installed instrumented bar-mat, showing instrumented bars, uninstrumented bars, and crack-former in place.

Figure 6. Bar instrumented 2 ft away from crack, and temperature gage.

ner as possible and to instrument the steel at and in the vicinity of the crack in order to measure the strains at the crack. In addition, it was desired to measure the crack opening, the temperature distribution in the concrete, and the strains in the concrete itself, in the vicinity of the crack. These studies are to be supplemented by an over-all crack-frequency study and a road-roughness study, to be made by others.

A strip of corrugated sheet steel 4 in. high, was installed to cause a controlled crack. Corrugated sheet was used in place of straight metal as a means of simulating, as closely as possible, the effects of a jagged crack, or what is sometimes referred to as aggregate interlock.

The general pattern of the instrumentation is shown in Figure 4. The basic instrumentation consists of bakelite SR-4

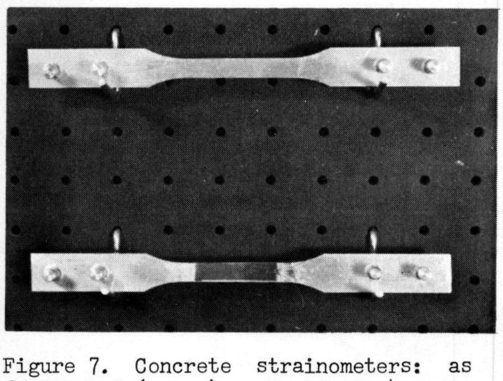

Figure 7. Concrete strainometers: as fabricated (upper), and with SR-4 gages applied (lower).

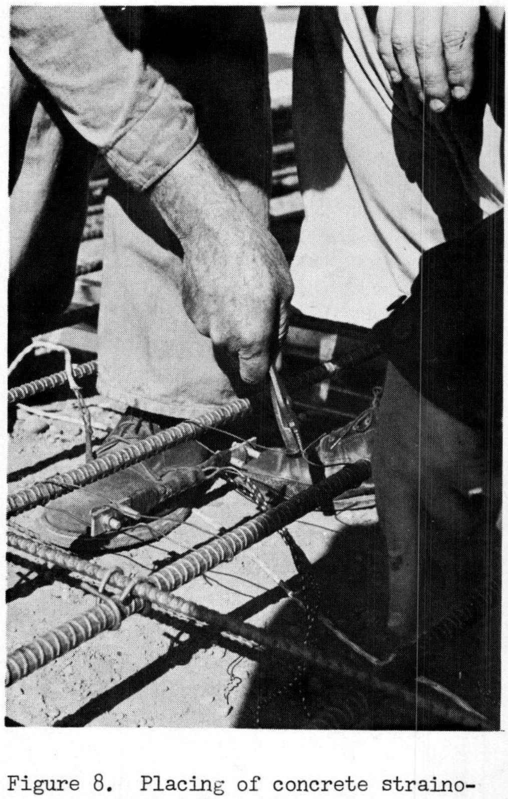

Figure 8. Placing of concrete strainometer in test panel.

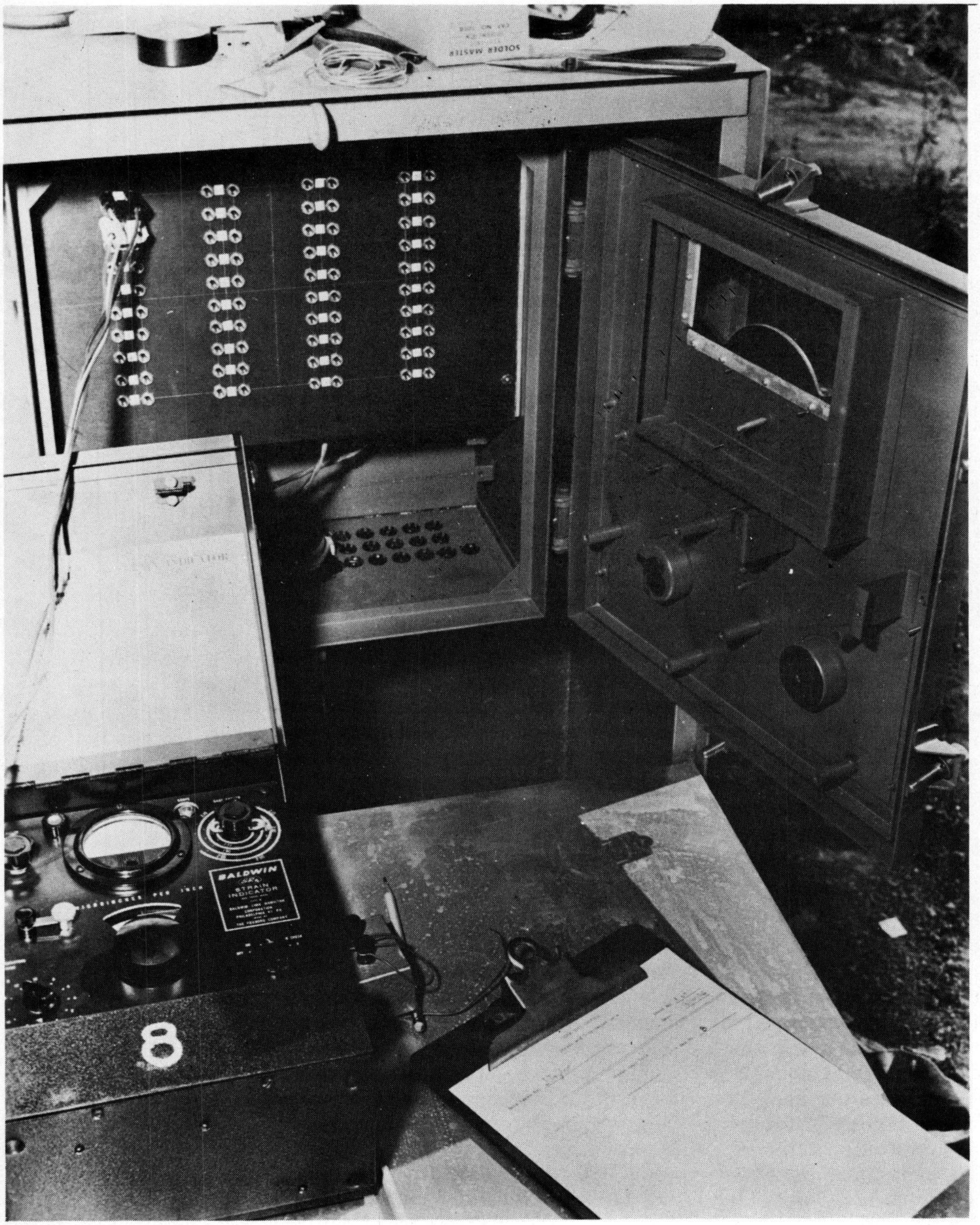

Figure 9. Terminal board ready for gage reading, showing indicator box and jack plugs inserted into board.

strain gages mounted on the reinforcing steel. These gages are of the AB-3 type with a nominal $\frac{13}{16}$-in. length. Of 20 bars, a total of 6 were gaged. The bars at the crack were double-gaged to average out the bending strains; the gages away from the crack were placed on the side of the bar for the same practical purpose. The gages away from the crack were placed at distances of 2 ft and 4 ft from the crack as a means of measuring the bond development. In order to break the bond at the crack, all noninstrumented bars were wrapped with rubber tape at the crack-former. The length of the taped area was 2 inches, the length in which the bond of the instrumented bars was broken.

The installation procedure was to place the instrumented bars on chairs in the test area. The lead wires were pulled through a flexible conduit, which led underground from the edge of the pavement to a control box at the edge of the right-of-way. When the instrumented rods were placed, a bar-mat with spaces for the instrumented bars was dropped over the loose bars, which were then tied to the mat to form a standard bar-mat. The instrumented steel at the crack is shown in Figure 5.

Five temperature gages were installed in the pavement and base course. At the edges of the lane these gages were placed in the center of the pavement depth. At the center of the lane three gages were placed: 1 in. into the base course, 1 in. up from the bottom of the slab, and 1 in. below the top of the pavement.

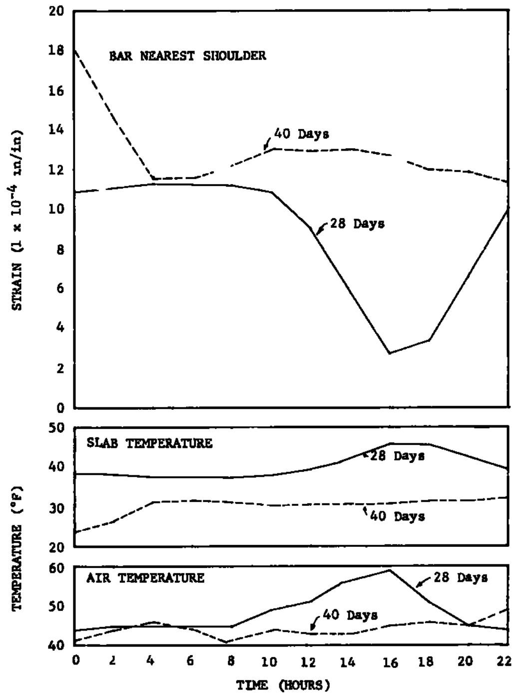

Figure 10. Short-term strain-temperature record for instrumented bar nearest shoulder at preformed crack.

The temperature gages were constructed of a 36-in. length of insulated nickel wire wound around itself to develop 17 ohms of resistance in the coil. The gages were so constructed that by proper calibration temperature recordings were made by placing the gage across the active leg of an SR-4 strain indicator, while using a fixed 120-ohm precision resistance as the compensating leg. In effect, the temperature gages are read in the same manner as the strain gages. Figure 6 shows a temperature gage and one of the SR-4 gages 2 ft away from the crack.

To determine the width of the crack as related to strains in steel at the crack, two sets of Whittemore gage plugs were used. Plugs were placed 10 in. apart, spanning the expected crack. Two sets of plugs were placed; one near the shoulder, the other near the longitudinal joint. The brass plugs to be installed were temporarily bridged by a thin sheet of brass, to which the plugs were lightly soldered. The entire assembly was inserted in the wet concrete. After the initial set, the brass strip was removed and the initial readings taken. At each reading the Whittemore gage was calibrated against a mild steel standard. For extreme computed effects of temperature change and difference in the thermal expansion coefficients of concrete and mild steel, any errors due to calibration would be outside the normal reading accuracy of the instrument. Thus, it was decided that additional precision was unnecessary.

An attempt is being made to determine the strains in the concrete itself, at a known distance away from the crack. To accomplish this, a strainometer was developed. The strainometers described here are only one of two types being developed at Fritz Laboratory, and do not necessarily represent the final thinking of the authors on this subject. As shown in Figure 7, the strainometer itself is machined from aluminum alloy 6063-T5 stock, $\frac{3}{4}$-in. wide and $\frac{1}{8}$-in. thick; the center portion is reduced to $\frac{3}{8}$ in. wide. The instrument has a total length of 8 inches. In order to aid in the bond development, two $\frac{1}{4}$-in. round aluminum pins, 1 in. long, were forced through the stock at each end. The end pin was $\frac{1}{2}$ in. from the end, and the pins were placed 1 in. apart. The total length of the reduced section was $2\frac{1}{2}$ in. On this frame were placed two AB-3 gages, and the entire assembly was waterproofed in the manner usual to this

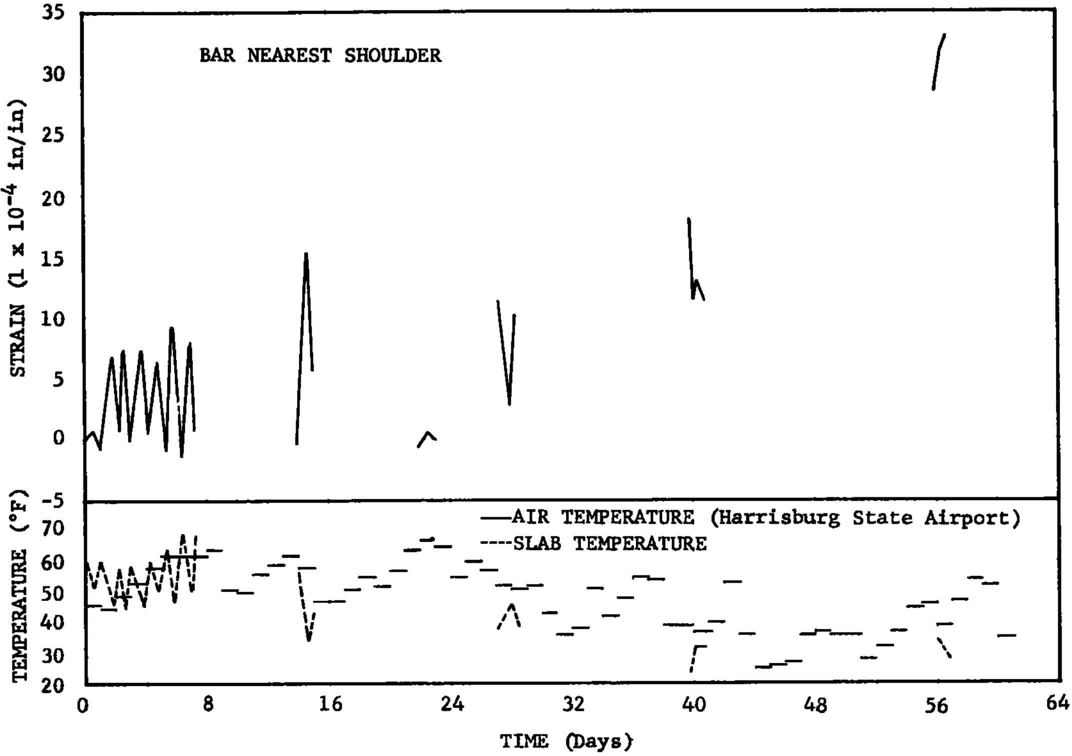

Figure 11. Strain-temperature record for first 56 days of pavement life for instrumented bar nearest shoulder at preformed crack.

project. The strainometers were installed as shown in Figure 8. Due to the developmental nature of these strainometers, they are undergoing an extremely rigorous series of laboratory tests. Although all indications to date are of excellent performance, further details of the strainometer behavior will have to wait until completion of all the tests.

The reinforcing rods were prepared for strain gages by reducing their nominal ⅝-in. diameter to a true diameter of 9/16 in. in a lathe. The length of the reduced section was 2 in., with suitable fillets, to reduce the effects of stress concentrations. The reduction in area resulted in a total reduction in steel at the preformed crack from 0.48 to 0.45 percent. The SR-4 gages were attached to the bar with bakelite cement, and then kept under heat and pressure to ensure a good bond. After the bonding was completed, a thin coating of plastic cement was applied directly over the gage, including ¼ in. of the metal bar. Only the gage leads were left exposed, and these were soldered to a lead cable composed of two No. 18 polythelene insulated shielded copper wires. The wires were looped over the gage, then held in place by glass fiber thread tied around the bar, following which the gage and wires were given another coating of plastic cement for complete waterproofing. Because of the abnormalities to be expected in this use of strain gages, an additional waterproofing was applied by coating the area with a self-vulcanizing neoprene compound. To further protect the gages from the surrounding concrete, abrasion-resistant vinyl tape was wrapped over the entire area.

As a final protection, the gaged area was coated with Glyptal immediately prior to the placing of the concrete. The installation was made on October 10, 1956. The latest set of data reported herein was taken on December 5, 1956, almost two months after the installation. All 24 strain gages were in good operating condition from the start of the test until the end of November 1956. In the December 5 readings, however, one of the gages at the crack indicated an open circuit. The most probable reason for this gage failure was the fatiguing of the small lead wires under the high strains and strain reversals. In no case has there been any indication of gage failure due to poor waterproofing.

As mentioned earlier, the lead wires extended under the base course and shoulder to a waterproof terminal box located at the edge of the right-of-way and mounted in a protected steel housing for security. Each lead wire was connected to female jack connections on the terminal board. Readings were taken by use of an SR-4 indicator that could be arbitrarily plugged into the female connections by means of radio "banana plugs, as shown in Figure 9.

Wherever possible, checks were placed in the system so that, barring gage failures the system would record as true an indication of strain as possible. All SR-4 gages were compensated for temperature by compensating plates cast into the pavement. Although one compensating gage and plate are required for the steel, and one for the aluminum, each compensating plate was double-gaged as a safety measure against failure of one gage.

To determine the drift of the indicator box zero position from one set of observations to the next, two precision resistances were mounted in the terminal box. By completing the bridge circuit with these resistors, and noting the stability of the reading, a correction can be applied for any drift in the instrument. In addition, the compensating gages can be checked for gage drift by using both gages as the active and compensating legs of the bridge. One of the biggest imponderables in the long-time use of SR-4 gages is the change in the relation between indicated resistance of the active gages and the strain. At present there is no known method of determining in an exact or absolute manner the "drift" of an SR-4 gage under strain. However, by taking the elementary precaution of using the most stable type of gages available, it was possible to deduce, from the rate of straining of adjacent rods and the very small probability that all gages were drifting equally, that the total gage drift to date was small. Therefore, the results reported in the next section are believed to be accurate within 1 percent of the indicated strain readings over a short period of time, and not a great deal more over a longer period.

Early Behavior

The test section was poured at 1:10 P.M. on October 10, 1956. The air temperature recorded at a nearby weather station was 50 F; the sun temperature recorded at the site was 60 F. The first two nights subsequent to the pour were unusually cold, with the air temperature down to 25 F. Between 6:00 and 8:00 A.M. on October 12 a crack developed by shrinkage, directly over the corrugated crack-former. Whereas the maximum strain prior to the cracking ranged between a tension of 80 microinches per inch (μ -in./in.) and a compression of 180 μ -in. /in., the first reading after cracking indicated that the strain developed to 675 μ -in. /in. of tension at the shoulder edge. This was an increase in tensile strain in the bar of 541 μ -in. /in. The crack width initially was 0.0053 in. Although the curing paper was on the pavement, the strains fluctuated between zero and tensile strains only slightly higher than the initial recording at the crack. The air temperature during this period was in a warming cycle, with an 8-deg rise in minimum temperatures and a 19-deg rise in maximum tempera-

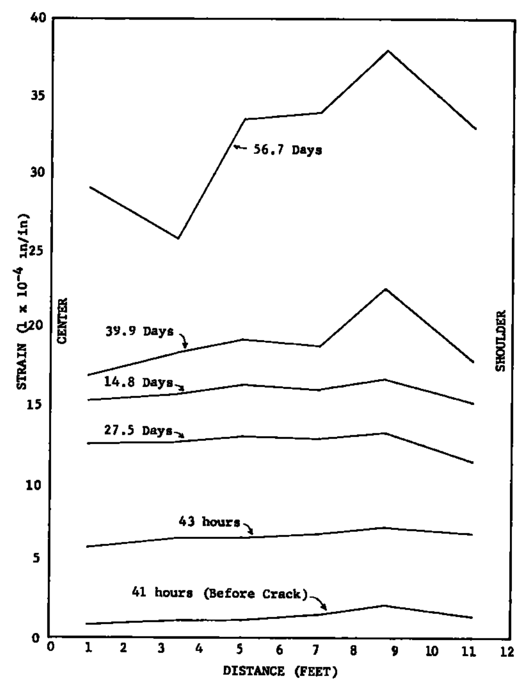

Figure 12. Transverse distribution of local maximum strains across pavement at preformed crack.

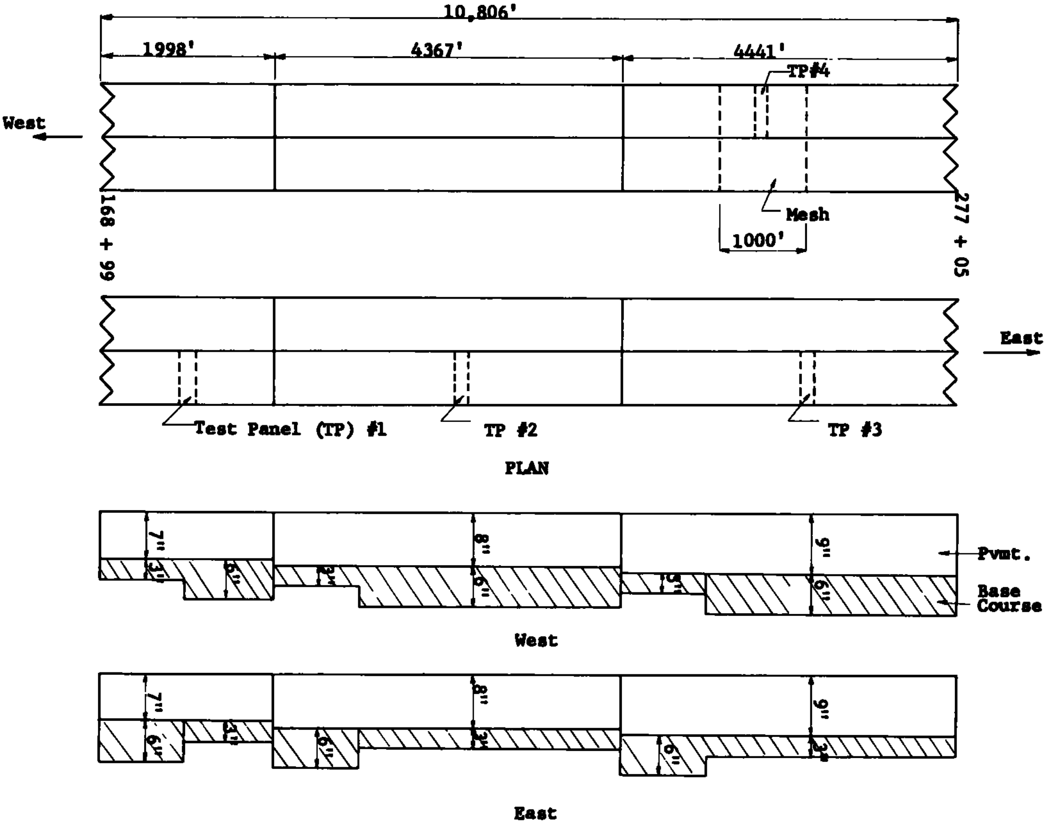

Figure 13. General plan, Hamburg project.

tures. Within 50 μ-in./in., the strains at the pavement edge remained in a constant daily cycle, dropping to zero in the afternoon, and increasing to about 700 μ-in./in. in early morning. The measurements of the crack opening showed similar behavior, with a maximum opening of 0.0087 in. and a minimum opening of zero. The constancy of these fluctuations indicates that the insulating effects of the paper influence the curing, and thus the cooling of the concrete was matched with increase in air temperature.

The curing paper was removed five days after the pour. Simultaneously, the air temperature during that day increased 2 deg and the sun temperature increased about 10 deg above the previous day. As a result of the shrinkage, the range of straining in the steel showed a net increase of 250 μ-in./in. in tension at night and decreased during the day to a total of 150 μ-in./in. in compression.

Considering the mechanism for behavior of continuously reinforced pavements, under the influence of temperature and shrinkage, the observed behavior does not indicate any contradiction. Three related phenomena are occurring simultaneously: (a) The general rise in temperature is greater than before; (b) the curing is increasing the bond properties; and (c) the pavement is tending to "shrink" a greater amount by capillary drying than before. The effect of temperature rise is to impose net compressive strains in the steel, previously tensioned. The increase in bond tends to transfer greater strains to the steel in general, and the tensile shrinkage stresses are also greater than before.

During the hot sunny days just subsequent to removing the paper, the net effect of the heat and shrinkage was a relatively small compression, but larger in magnitude than previously recorded. At night the shrinkage stresses became smaller, the curing rate slowed down, and the temperature decreased. Because the pavement was no longer insulated, the temperature lows, in the warmer period, remained the same as during the colder period when insulated. Thus, the net effects due to the higher shrinkage stresses and bond properties was to increase the net tensile strain in the steel.

A comparison between the slab temperature and the strain in the steel shows that when the temperature fluctuations reversed themselves, and the slab temperature returned to its starting temperature in a given 24 hr period, the strains showed a similar reversal. Because the position of the test section was within the area of complete restraint, this is as it should be. Unfortunately, it is not possible in this experiment to prove the concept of strain residuals quantitatively. When the data on the crack survey and the crack width survey have been analyzed, this tentative hypothesis can be checked.

Figure 10 presents a complete strain-temperature record for the shoulder edge bar on the 28th and 40th days after pour. It should be noted that equal slab temperature gradients result in equal strains over an intermediate period of time. This should not be surprising, except that with complete analysis it should be possible to establish quantitative strain mechanisms based on the temperature gradients. Thus, the design of these pavements can be keyed to the expected temperature fluctuations in a given geographical region.

Figure 11 indicates the strain fluctuations of the shoulder edge bar as compared with the mean air temperatures taken at a nearby weather station, and slab temperatures. Because this air temperature record is the only continuous temperature record available, it is presented as a general indication of over-all behavior. The temperature during the reported period, although averaging normal according to U. S. Weather Bureau records, has shown some of the widest variations in Weather Bureau history. Thus, it can be said that in its early life this pavement underwent a very severe test indeed.

Between 40 and 56 days of age the pavement steel reached such strains as to be in the yield range. Immediately after the 40th-day observation, the temperature dropped suddenly and markedly (Fig. 15). This temperature drop, although abnormal, was not of such magnitude as to be statistically of remote possibility. The increase in the restraint forces was then such as to exceed the yield point of the steel at the crack.

During the 40th day the crack width varied between 0.0157 and 0.0190 in., whereas on the 56th day the crack width fluctuated between 0.0142 and 0.0216-in. On both occasions the nearest crack to the preformed crack was 17 ft to the north and 12 ft to the south. To date, the strain away from the crack has never exceeded 100 μ-in./in.

The transverse distribution of strain is shown in Figure 12. Prior to the formation of the cracks, the strains built up in such a manner that the second instrumented bar from the shoulder developed the highest strains. The strain development was noticeable immediately after the pour, at the first set of readings taken. The strain distribution before and after the crack formation is similar. With aging and temperature effects developing, this relatively highly-strained bar tends to pick up more strain and also relieves the edge bar of strain. By the 40th day the second interior bar had exceeded its yield point and was showing great strain differences. On the basis of a weakest-link theory, this highly-strained bar initiated the process by which all the other bars exceeded the yield point of the steel.

Once yield started, the strain pattern completely readjusted itself within the holding power of the tie-bar and the yield point of the individual bar.

The apparent effect of the tie-bar is to hold the pavement together and thus reduce the strain in the bars at the longitudinal joint. This phenomenon is reflected in the crack width measurements by the fact that the outside crack opening is 0.001 in. wider than the inside crack opening.

The strain indicated, when considered over the entire section, is a direct measure of the force required to maintain complete restraint. To adjust the deformation conditions and to make them compatible with the force conditions, there will be individual bar strain readjustments to conform with the boundary conditions and the material properties of individual bars. Thus, the result is the apparently erratic strain pattern shown in Figure 12. These bars are in the vicinity of the yield point, and will show relatively large strain variations in the readjustments of deformations resulting from the constraints put on the system by the tie-bars, and the metallurgy of individual bars. This is probably a transient phenomenon that will dissipate with additional strai

It is difficult to predict the future, especially since one must postulate the climate; but, extrapolating other experiences to this circumstance, there is every reason to believe that this pavement will perform satisfactorily despite the high strains being measured. Once beyond the yield point, the cyclic nature of the phenomenon is going to introduce, to an unknown degree, the influence of strain hardening of the steel. Any such influence can only affect the pavement in a favorable manner, by increasing the yield point and thus limiting the crack opening.

HAMBURG PROJECT

A second project on continuously reinforced pavements is scheduled for paving in the Spring of 1957, and is located on US 22, near Hamburg.

On this project the pavement thickness is varied to include thicknesses of 7, 8, and 9 in.; the base course will be partly 3 in. and partly 6 in. thick. A schematic layout of the proposed highway is shown in Figure 13.

At about the midpoint stationing of each section of pavement thickness change, and in the section containing wire mesh, an instrumented test panel consisting of a preformed crack will be established. Measurements will be made of the strains in the hard-grade steel at the crack and in the immediate vicinity of the crack. In all cases the longitudinal reinforcing bars will be No. 5, the transverse bars No. 3. As shown, 1,000 ft of pavement will be reinforced with welded wire fabric, using ½-in. longitudinal wires. The steel percentage for the entire job will be constant at approximately 0.5 percent of the pavement cross-sectional area.

To determine the magnitude and distribution of horizontal movement of the Hamburg pavement, surface plugs will be installed in the east-bound lane 100 ft apart and the relative movement of these plugs will be measured. Absolute motion will be ascertained by use of reference monuments off the shoulder.

The study of warping effects of the pavement will consist of three phases. The first will determine the temperature distribution as a function of time and climatic conditions in the pavement, base course, and subgrade. This will be accomplished by the careful placing of resistance-type temperature gages in the pavement-base-subgrade system, at such locations as to develop the vertical and transverse temperature distribution. In the same areas the pavement surface will be spotted with transverse plugs so that the relative profile can be measured. In addition, the transverse No. 3 bars in this area will be instrumented to determine the warping strains in the steel.

In addition to these studies there will be a crack survey, in which the average crack widths at selected stations will be measured, as well as making a total crack count. As a means of quantitatively measuring the riding qualities of the continuously reinforced pavement, it is anticipated that a road-roughness survey will be conducted.

To measure the dynamic effects of traffic on pavement performance, an over-all traffic count will be made. Specific information also will be obtained with regard to the strain behavior of the steel under the influence of moving loads. This will be done by means of strip chart recorders and trucks of known weight.

CONCLUSION

This particular study of continuously reinforced concrete pavements is as yet in its infancy; therefore, no absolute conclusions can be drawn. One fact is apparent; within the region of comparable data, a close parallel exists between the York, Pa., and the Illinois pavements, both being poured in the same period of the year, and both showing high strains at an early age. It will be interesting to see the difference for pavements constructed in warm weather, such as will occur at Hamburg.

The study to date has indicated that the hypothesis of areas of complete restraint is valid. Possibly the crack survey will bear out the remainder of the hypothesis as to residual strains in the end areas. The importance of early behavior being the key to net performance seems to be borne out by the analysis of strain development.

One factor, which was not covered by this report, should not be ignored. This is the part that the bond area of the steel plays in the total picture. The project at Lehigh University is committed to the use of a fixed size and percentage of reinforcing bar, based on standard designs. It is believed that the magnitude of the crack width, crack frequency, and the strains in the steel, are directly relatable to the bond surface available in the steel. It is hoped that other investigations in this field will consider this fact, so that it will be opened to experimental verification.

In addition, it is hoped that other investigations in this area will open the door to studies of the effect of traffic vibrations on pavement behavior and, in particular, to the repetitive action on the shearing strength mobilization between the pavement and the subgrade.

ACKNOWLEDGMENTS

The authors wish to acknowledge the aid of the American Iron and Steel Institute for sponsorship of the instrumentation phases of the York project. Acknowledgment also is made to the support given Lehigh University in its study of continuously reinforced pavements at York and Hamburg by the Pennsylvania Department of Highways and the U. S. Bureau of Public Roads. The work by Lehigh University was carried on by the personnel of the Fritz Engineering Laboratory.

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