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CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

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CONTINUOUSLY REINFORCED CONCRETE PAVEMENT RESEARCH IN PENNSYLVANIA

PRELIMINARY REPORT

by

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A. SYNOPSIS

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-inch pavement. In addition, studies are being conducted of 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 life of the pavement.

The second project will be on Route 22, near Hamburg, in which the pavement thickness will be varied to include sections of 7,8, and 9-inch thickness. The reinforcing will be bar-mat, with the exception of 1000 feet 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 impending research project.

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 added as a means of tieing cracks in the pavement together, and to give the pavement slabs some additional flexural strength.

The design of conventional reinforced highway slabs, with load-transfer joints every 30 to 100 feet, 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, 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 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. Thus we have a continuously reinforced concrete pavement where the reinforcement is of such size as to hold the cracks to non-objectionable width.

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), California (2), Illinois (3), Indiana (4), Texas (5), and now in Pennsylvania.

Fundamentally, the behavior of a given continuously reinforced concrete pavement is dependent on 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 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 upon the restraining horizontal shear stresses mobilized between the subgrade and 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 very 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. Since the closing of the cracks depends on the ability of the pavement, in the partially restrained areas, to move over the subgrade, and since 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 (3b).

It is certain that the mechanisms of capillary shrinkage and temperature expansion and contraction are in operation to varying degrees of magnitude at all time. 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 set the pattern of future behavior.

The effects of wheel loads on a continuously reinforced pavement are such as to catagorize 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 will exist 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.

C. YORK PROJECT

1. GENERAL DESCRIPTION

The first study to be made of continuous pavements in Pennsylvania is being made near York on Route 111.

The experimental section of Route 111 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 0.48 percentage of longitudinal reinforcement is provided by bar-mats 16 feet long, as shown in Figure 1.

The pavement design was set by the Pennsylvania Department of Highways and 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. The location of this joint is at station 971+70. The north end of the experimental pavement joins the standard pavement by means of four standard expansion joints, the first one being at station 1087+29. The expansion joints were spaced 61.5 feet apart in the standard pavement.

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 feet. Table 1 shows a tabulation of 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 the placing of the finger-type joint. This joint will be installed in the Spring of 1957.

The pavement was poured in 12-foot lanes. The juncture between lanes was accomplished by use of a keyed joint and 1/2-inch round hook bolts, spaced 5 feet on centers.

2. SUBSURFACE SOIL CONDITIONS

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

A record of the subgrade soil conditions, in regard to the soil size and plasticity characteristics, are presented in Table 2 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 of these determinations, as expressed in terms of the per cent of a standard Proctor compaction test, 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 feet of soil mantle overlying 10 to 20 feet of weathered rock.

The soil samples were taken at intervals of 200 to 400 feet along the entire length of highway. The maximum depth of sample was 3 feet 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 the PI was limited to a maximum of 18. The per cent finer than the No. 200 sieve varied from 27 to 84 per cent, with the majority of the samples in the vicinity of 50 per cent 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 claysoils would indicate that the potentialities for frost-heaving would be moderate to objectionable for ground water conditions within 6 feet of the pavement. The subsurface drainage of these soils would be poor, and they would exhibit a potential capillary rise of from 7 to 20 feet. The overall expected maximum dry densities would range from 110 to 120 pounds per cubic foot. It is to be expected that the subgrade soil would have normal capillary saturation of about 100 per cent of optimum moisture, which ranges from 10 to 14 per cent by weight. These soils are susceptible to pumping. The above 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 per cent 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-inch 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, and thus tend to alleviate the potential loss of support.

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

3. PAVEMENT CONSTRUCTION MATERIALS AND PROCEDURES

The 9-inch 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-inch 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-inch pavement on the northern end will be installed in the Spring of 1957, in anticipation of opening the highway to traffic in July 1957.

Air-entraining cement was used for a design mix of 1 : 1.79 : 3.5, and 5 gallons of water 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 1-3/8 inches to 2-5/8 inches. The design slump was 1-3/4 inches.

Structural properties of the concrete were determined by the Pennsylvania Department of Highways in the course of their 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 life of the pavement.

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-inch gage length varied from 10 to 14 per cent, with an average of 11.3 per cent.

A stress-strain curve, based on tests at Fritz Laboratory, of two randomly-selected bars gaged with a 2-inch extensometer, is shown in Figure 3. This curve was used as a calibration to convert strain readings in the pavement reinforcing steel to stresses in the steel.

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

The concrete from the mixer was dropped in front of a screw-type mechanical spreader. The concrete was struck off at 4-1/2 inches and was 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-foot wide mats were then placed and lapped longitudinally a full 12 inches to provide continuity of steel, as shown in Figure 4. After several lengths of reinforcement were placed, the spreader would move back and complete the remaining 4-1/2 inches of pavement.

At this time the finishing equipment would move immediately behind the spreader. The pavement was cured by using curing paper which remained on the pavement a minimum of 72 hours after pouring.

At the end of a day's pour, a split-header board bulkhead was constructed, with reinforcing steel running continuously through the joint.

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 was so placed that it would be in the middle of a day's pour, as shown in Figure 5. 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 feet.

After the steel was fully placed, both behind and in front of the test area, the spreader was moved back and proceeded to complete the second lift. When the spreader reached the test area, a full 9 inches of concrete was poured very carefully to avoid disturbing the instrumentation. No vibration was permitted in this area. In it's stead, the concrete was carefully rodded to avoid honeycombing. This procedure is shown in Figure 6. 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.

4. TEST SECTION

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 3569 feet from the north end, and 7980 feet 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 manner 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 overall crack-frequency study and a road-roughness study, to be made by others.

A strip of corrugated sheet steel, 4 inches 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 7. The basic instrumentation consists of bakelite SR-4 strain gages mounted on the reinforcing steel. These gages are of the AB-3 type with a nominal 13/16-inch length. Out of 20 bars, a total of 6 were gaged. The bars at the crack were double-gaged to average out the bending strains, while 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 feet and 4 feet from the crack, as a means of measuring the bond development. In order to break the bond at the crack, all non-instrumented bars were wrapped with rubber tape at the crack-former. The length of the taped area was 2 inches, this being the length in which the bond of the instrumented bars was broken.

The installation procedure was to place on chairs, the instrumented bars 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; these bars were then tied to the mat to form a standard bar-mat. A photograph of a section of the instrumented steel at the crack is shown in Figure 8.

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; the bottom one, one inch into the base course, the second, one inch up from the bottom of the slab, and the third, one inch below the top of the pavement.

The temperature gages were constructed of a 36-inch length of insulated nickel wire wound around itself to develop

17 ohms of resistance in the coil. The gages were constructed in such a manner that by proper calibration, temperature recordings were made by placing the temperature 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. A photograph of the temperature gage and one of the SR-4 gages two feet away from the crack, is shown in Figure 9.

In order 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 inches 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 end, 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 10, the strainometer itself is machined from Reynolds aluminum alloy 6063-T5 stock, 3/4-inch wide and 1/8-inch thick. As shown, the center portion is reduced in width to 3/8-inch. The instrument itself has a total length of 8 inches. In order to aid in the bond development, two 1/4-inch round aluminum pins, 1 inch long, were placed by being forced through the stock at each end. The location of the end pin was 1/2-inch from the end, and the pins were placed 1 inch apart. The total length of the reduced section was 2-1/2 inches. On this frame were placed two AB-3 gages, and the entire assembly waterproofed in the manner usual to this project. The strainometers, as installed, are shown in Figure 11. Due to the developmental nature of these strainometers, they are at the present time undergoing an extremely rigorous series of laboratory tests. Although all indications to date are of excellent performance, further details of the behavior of the strainometer will have to wait until all the tests have been exhausted.

The reinforcing rods were prepared for strain gages by reducing their nominal 5/8-inch diameter to a true diameter of 9/16-inch in a lathe. The length of the reduced section was 2 inches, with suitable fillets, to reduce the effects of stress concentrations. The reduction in area, effected a total reduction in steel percentage at the preformed crack from 0.48 to 0.45 per cent. 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 Armstrong plastic cement was applied directly over the gage, including 1/4-inch of the metal bar. Only the gage leads were left exposed, and these were soldered to two conductor-shielded cables which formed the lead wire. The lead wires were No. 18 polythelene insulated shielded copper wire cable. At this stage, the lead wires were soldered to the gage and the wires looped over the gage. The wires were then held in place by glass fiber thread tied around the bar. After tying down the wires, the gage and wires were given another coating of plastic cement. At the end of this operation, the gage under normal conditions, would be completely water-proofed. Because of the abnormalities to be expected in this usage of strain gages, an additional waterproofing was applied by coating the area with a self-vulcanizing neoprene compound. In order to protect the gages from the surrounding concrete, abraison-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 being reported here 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. The December 5, 1956 readings showed that one of the gages at the crack indicated an open circuit. It is the authors' opinion that the most probable reason for this gage failure was the fatiguing of the small lead wires under the high strains, and strain reversals, leading to a rupture of the wire. In no case has there been any indication, to date, of gage failures due to poor waterproofing.

As was mentioned earlier in this paper, the lead wires extended under the base course and shoulder to a water-proof terminal box located at the edge of the right-of-way. The terminal box was mounted in a protected steel housing for security. Each lead wire was connected to female jack connections on a terminal board, as shown in Figure 12. Readings were taken by use of an SR-4 indicator that could be arbitrarily plugged into the female connections by means of radio "bananna" plugs, as shown in Figure 13.

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 a failure of one.

In order 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 the present time there is no method known of determining the "drift" of an SR-4 gage under strain, in an exact or absolute manner. However, by taking the elementary precautions of using the most stable type of gages available, the authors were able 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. Based on the above reasoning, the authors believe that the results reported in the next section of this report are accurate within one per cent of the indicated strain readings over a short period of time, and not a great deal more over a longer period.

5. 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. Wheras the maximum strain prior to the cracking ranged between a tension of 80 micro-inches per inch and a compression of 180 micro-inches per inch, the first reading after cracking indicated that the strain developed to 675 micro-inches per inch of tension at the shoulder edge. This was an increase in tensile strain in the bar of 541 micro-inches per inch. The crack width initially was 0.0053-inch. While 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-degree rise in minimum temperatures and a 19-degree rise in maximum temperature. Within 50 micro-inches per inch, the strains at the pavement edge remained in a constant daily cycle, dropping to zero in the afternoon, and increasing to about 700 micro-inches per inch in early morning. The measurements of the crack opening showed similar behavior, with a maximum opening of 0.0087-inch and a minimum opening of zero. The constancy of these fluctuations indicate 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 and coincidentally, the air temperature during that day increased 2 degrees, and the sun temperature increased about 10 degrees above the previous day. As a result of the shrinkage, the range of straining in the steel showed a net increase of 250 micros in tension at night and decreased during the day to a total of 150 micros in compression.

Considering the mechanism for behavior of continuously reinforced pavements, under the influence of temperature and shrinkage, the above observed behavior does not indicate any contradiction. Three related phenomena are occurring at the same time. The general rise in temperature is greater than before, the curing is increasing the bond properties, and 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 that occurred, just subsequent to removing the paper, the net effect of the heat and shrinkage was a relatively small compression which was larger in magnitude than previously recorded. At night the shrinkage stresses became smaller, the curing rate slowed down, and the temperature decreased. Since 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 temperatures 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-hour period, the strains showed a similar reversal. Since 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 14 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 15 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. Since this air temperature record is the only continuous temperature record available, it is presented as a general indication of overall behavior. The temperature over 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 this pavement underwent, in its early life, 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, as shown in Figure 15. This temperature drop, although abnormal, was not of such magnitude as to be statistically of remote possibility. The increase in the restraint forces were then such as to exceed the yield point of the steel at the crack.

During the observations taken on the 40th day, the crack width varied between 0.0157-inch and 0.0190-inch, while on the 56th day, the crack width variation fluctuated between 0.0142-inch and 0.0216-inch. The nearest crack to the preformed crack on both occasions, was 17 feet to the north and 12 feet to the south. The strain away from the crack has never exceeded 100 micros, to date.

The transverse distribution of strain is shown in Figure 16. 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. This 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 on the pavements 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 now 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 re-adjusted 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-

inch 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. In order 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 16. 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 which will dissipate with additional straining.

It is difficult to predict the future, especially since one must postulate the climare, 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.

D. HAMBURG PROJECT

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

This project will be one in which the pavement thickness is varied to include thicknesses of 7, 8, and 9 inches; the base course will be varied to include a 3-inch thick base course and a 6-inch base. A schematic layout of the proposed highway is shown in Figure 17. 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. It is at these preformed cracks that measurements will be made of the strains in the steel at the crack, and in the immediate vicinity of the crack. The reinforcing bars in all cases, will be No. 5 longitudinal bars and No. 3 transverse bars. The steel will be hard-grade steel. One thousand feet of pavement, as shown, will be reinforced with welded wire fabric, using 1/2-inch longitudinal wires. The steel percentage for the entire job will be constant at approximately 0.5 per cent of the pavement cross-sectional area.

In order to determine the magnitude and distribution of horizontal movement of the Hamburg pavement, surface plugs 100 feet apart, will be installed in the east-bound lane, 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 area that the temperature gages are placed, 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 the above-mentioned studies, there will be a crack survey in which the average crack widths, at selected stations, will be measured in addition to 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.

In order to measure the dynamic effects of traffic on pavement performance, an overall traffic count will be made. In addition to this general information, specific information will be obtained with regard to the strain behavior of the steel under the influence of moving loads, by means of strip chart recorders, and trucks of known weight.

E. CONCLUSION

This particular study of continuously reinforced concrete pavements is as yet in its infancy, and as a result, no absolute conclusions can be drawn. One fact is apparent; within the region of comparable data, a close parallel exists between the York, Pennsylvania, 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 the opinion of the authors 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, the authors hope that other future investigations in this area 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 subgrade.

F. ACKNOWLEDGEMENTS

The authors would like to acknowledge the aid of the American Iron and Steel Institute for their sponsorship of the instrumentation phases of the York project. Acknowledgement 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 - Professor William J. Eney, Director.

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H. TABLES AND FIGURES

TABLE 1

STATIONING OF DAILY CONSTRUCTION JOINTS

Date Paved	Beginning of Pour	End of Pour	Feet	Temperature (degrees F)
		Northbound Inside Lane		
9-18-56	1087 + 29	1065 + 03	2226	50 - 71
9-19-56	1065 + 03	1043 + 91	2112	42 - 66
9-20-56	1043 + 91	1024 + 46	1945	40 - 59
9-21-56	1024 + 46	998 + 52	2594	32 - 57
9-25-56	998 + 52	972 + 99	2553	44 - 62
		Southbound Inside Lane		
9-26-56	1087 + 29	1064 + 15	2314	43 - 55
9-27-56	1064 + 15	1043 + 05	2110	45 - 53
10- 1-56	1043 + 05	1018 + 93.5	2411.5	50 - 68
10- 2-56	1018 + 93.5	991 + 30	2763.5	50 - 70
10- 3-56	991 + 30	972 + 99	1831	55 - 72
		Northbound Outside Lane		
10- 9-56	1087 + 29	1063 + 22.7	2406.7	42 - 64
10-10-56	1063 + 22.7	1035 + 09.2	2513.2	38 - 56
10-11-56	1035 + 09.2	1010 + 85	2724	30 - 59
10-12-56	1010 + 85	981 + 74	2911	34 - 61
10-15-56	981 + 74	972 + 98	876	41 - 83
		Southbound Outside Lane		
10-15-56	972 + 98	984 + 53	1154	41 - 83
10-16-56	984 + 53	1013 + 35	2882.57	43 - 55
10-17-56	1013 + 35	1043 + 78	3042.53	54 - 83
10-18-56	1043 + 78	1071 + 81	2803	54 - 76
10-19-56	1071 + 81	1087 + 29	1547.5	45 - 65

TABLE 2 - A

IDENTIFICATION CHARACTERISTICS OF SUBGRADE AND BASE COURSE SOILS

Location	971+70 to 984+50	984+50 to 998+50	998+50 to 1012+50	1012+50 to 1028+50
% Finer than:				
1-1/2" Sieve	100	100	100	100
3/4" Sieve	97-100	100	98-100	100
No. 4 Sieve	91-100	93-100	94-100	93-99
No. 10 Sieve	88- 99	89- 99	90- 97	88-97
No. 20 Sieve	85- 98	83- 98	82- 95	86-96
No. 40 Sieve	64- 95	72- 94	67- 89	79-92
No. 60 Sieve	46- 92	61- 90	49- 82	66-81
No.100 Sieve	37- 89	51- 86	37- 76	66-73
No.200 Sieve	29- 84	41- 81	29- 69	42-69
0.005mm.	23- 49	27- 57	22- 42	26-40
Liquid Limit	23- 35	24- 38	23- 33	22-28
Plasticity Index	3- 14	2- 18	2- 13	2- 9

TABLE 2 - B

IDENTIFICATION CHARACTERISTICS OF SUBGRADE AND BASE COURSE SOILS

Location	1028+50 to 1042+50	1042+50 to 1054+50	1054+50 to 1070+50	1070+50 to 1087+27	Base Course
% Finer than:					
1-1/2" Sieve	100	100	100	100	100
3/4" Sieve	100	100	77-100	100	87
No. 4 Sieve	96-100	98-99	71-97	87-97	53
No. 10 Sieve	92-100	94-98	67-95	80-92	29
No. 20 Sieve	86-100	92-98	65-93	78-87	27
No. 40 Sieve	69- 99	88-97	59-92	72-85	20
No. 60 Sieve	55- 99	79-96	50-90	64-81	15
No.100 Sieve	47- 97	67-94	40-87	54-78	12
No.200 Sieve	36- 88	48-82	31-80	40-75	7
0.005 mm.	28- 40	25-34	21-29	25-32	--
Liquid Limit	24- 31	23-27	23-29	24-29	--
Plasticity Index	4- 8	2- 5	0-10	2- 8	--

TABLE 3

SUBGRADE SURFACE COMPACTED DENSITIES

Location	Per Cent Compaction
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

TABLE 4

STRUCTURAL PROPERTIES OF CONCRETE

(Pennsylvania Department of Highways Tests)

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	1970	2210	2450
Cylinder	7	2	3110	3275	3440
Cylinder	10	8	2770	3260	3720
Cylinder	28	5	3770	4200	4840

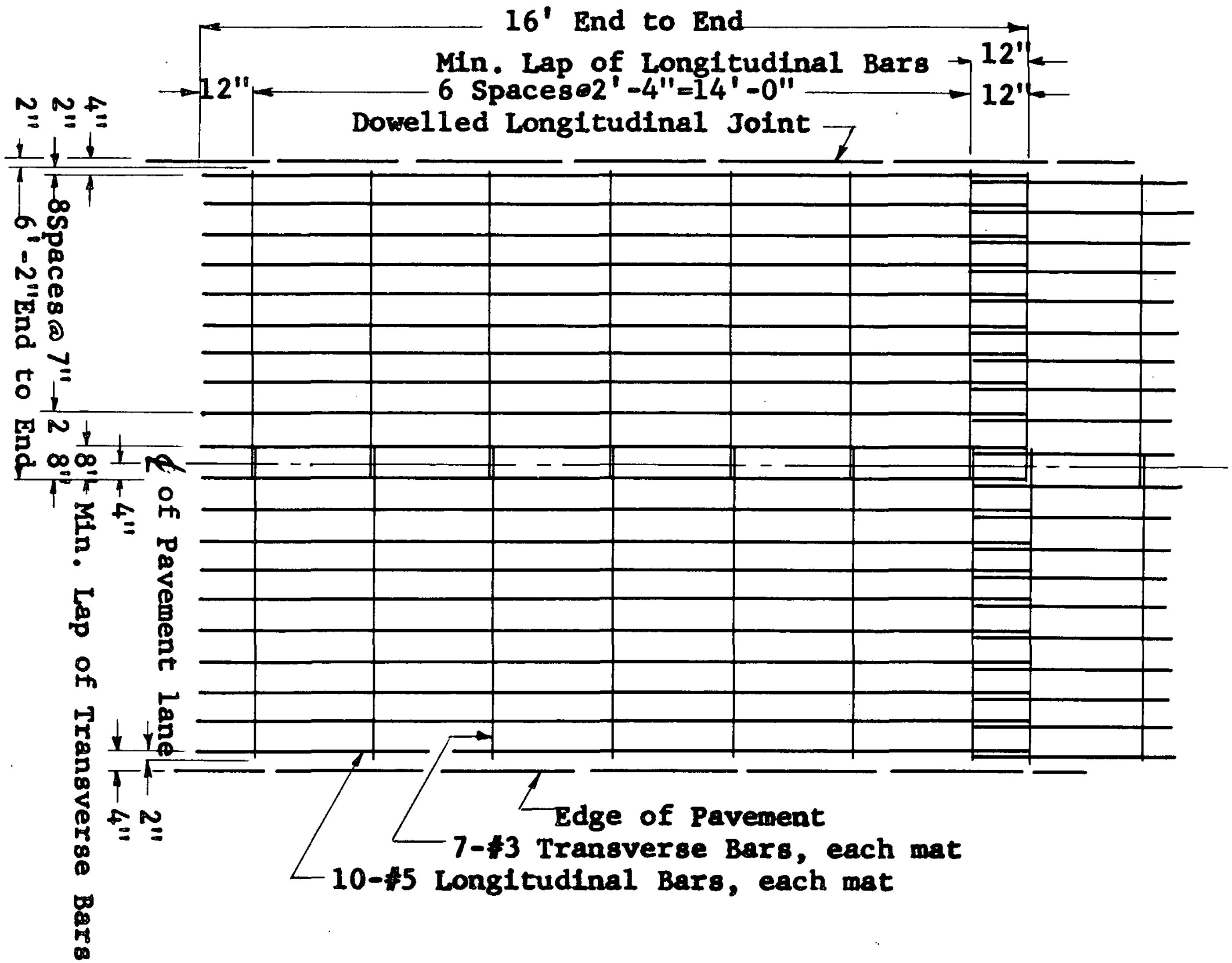

Figure 1 Bar-Mat Reinforcement (9-inch Pavement)

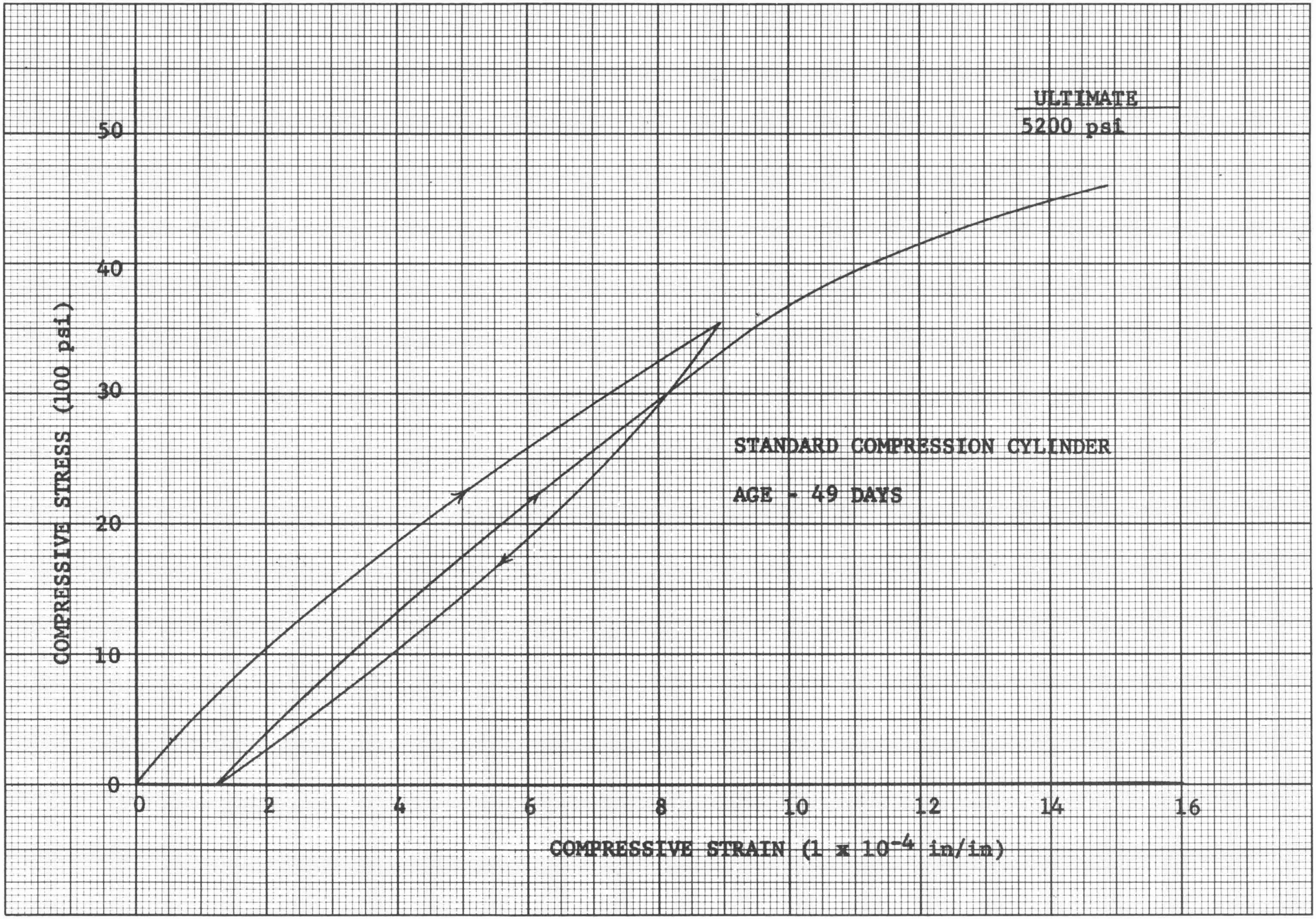

Figure 2 Compression Stress-Strain Curve For Concrete Cylinders

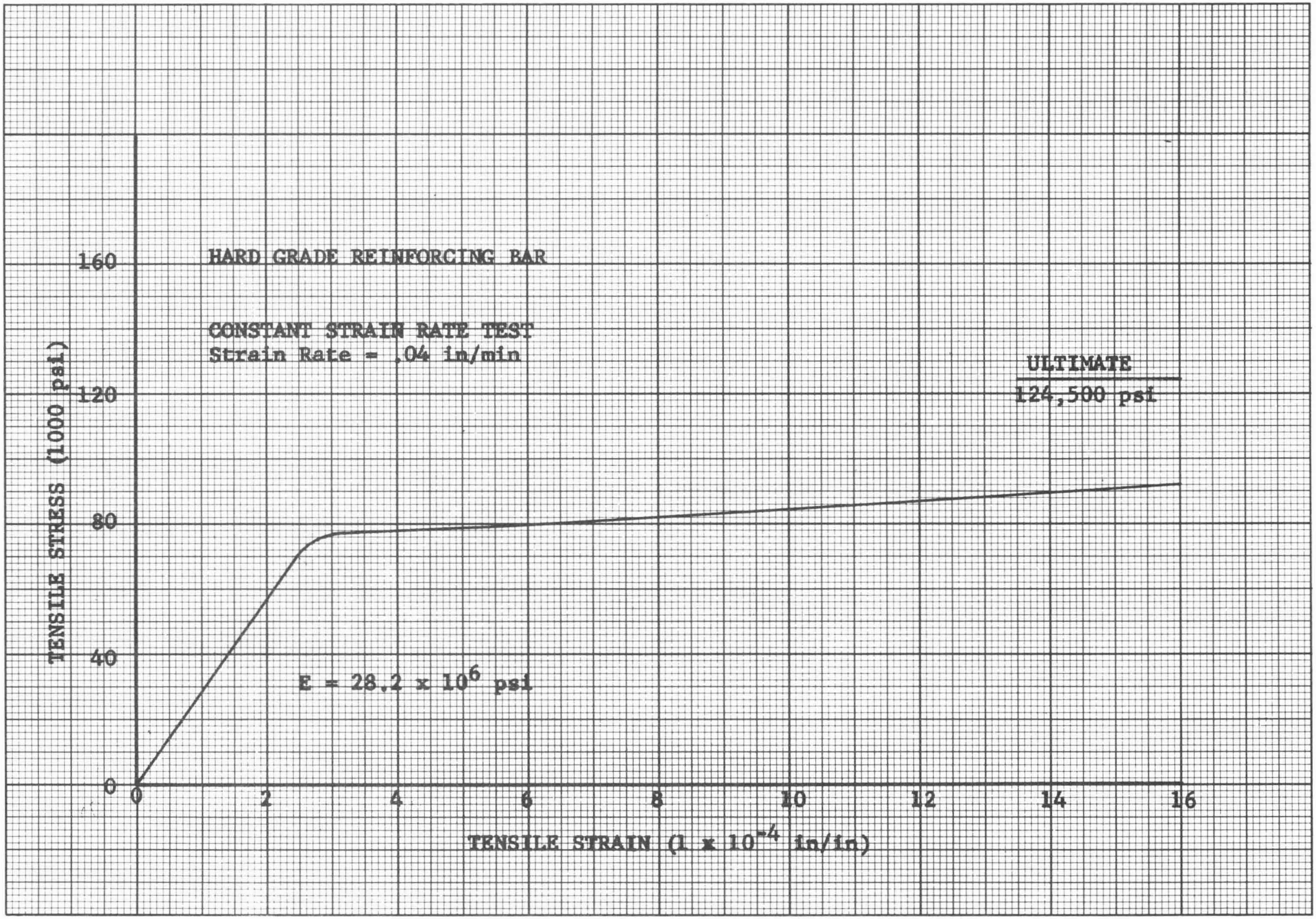

Figure 3 Tension Stress-Strain Curve — Reinforcing Bars

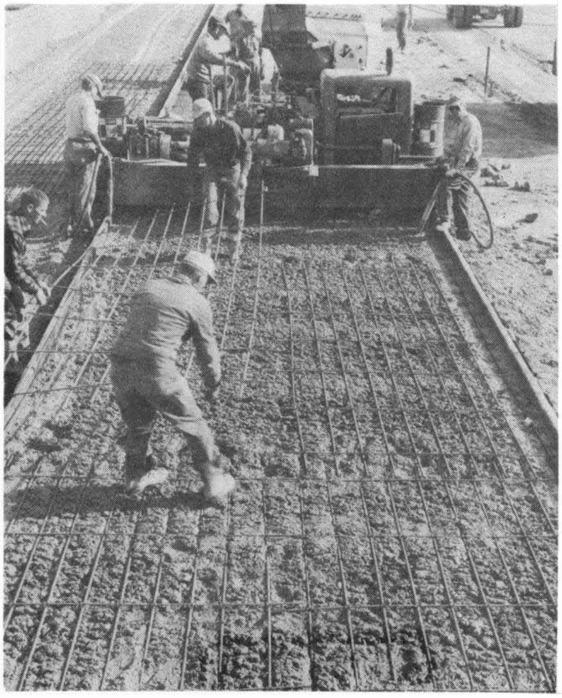

Figure 4 Placement of Bar-Mat Reinforcement in Continuously Reinforced Pavement

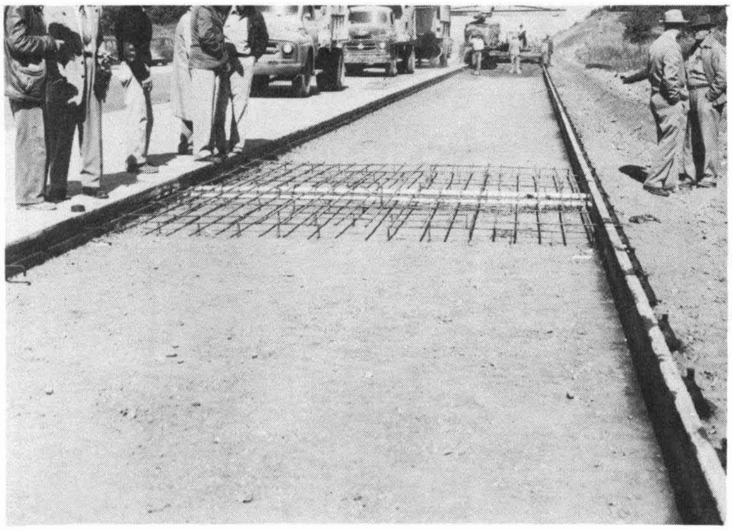

Figure 5 Instrumented Bar-Mat Prior to Pouring of Concrete

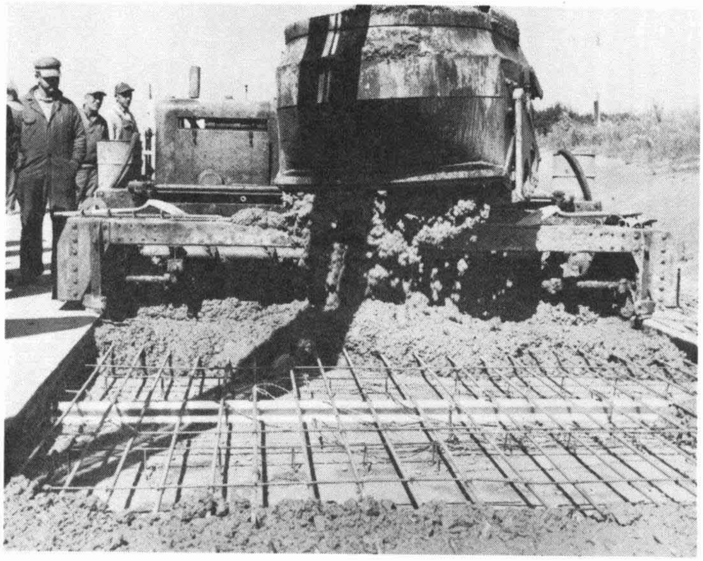

Figure 6 Pouring Concrete in Instrumented Test Area

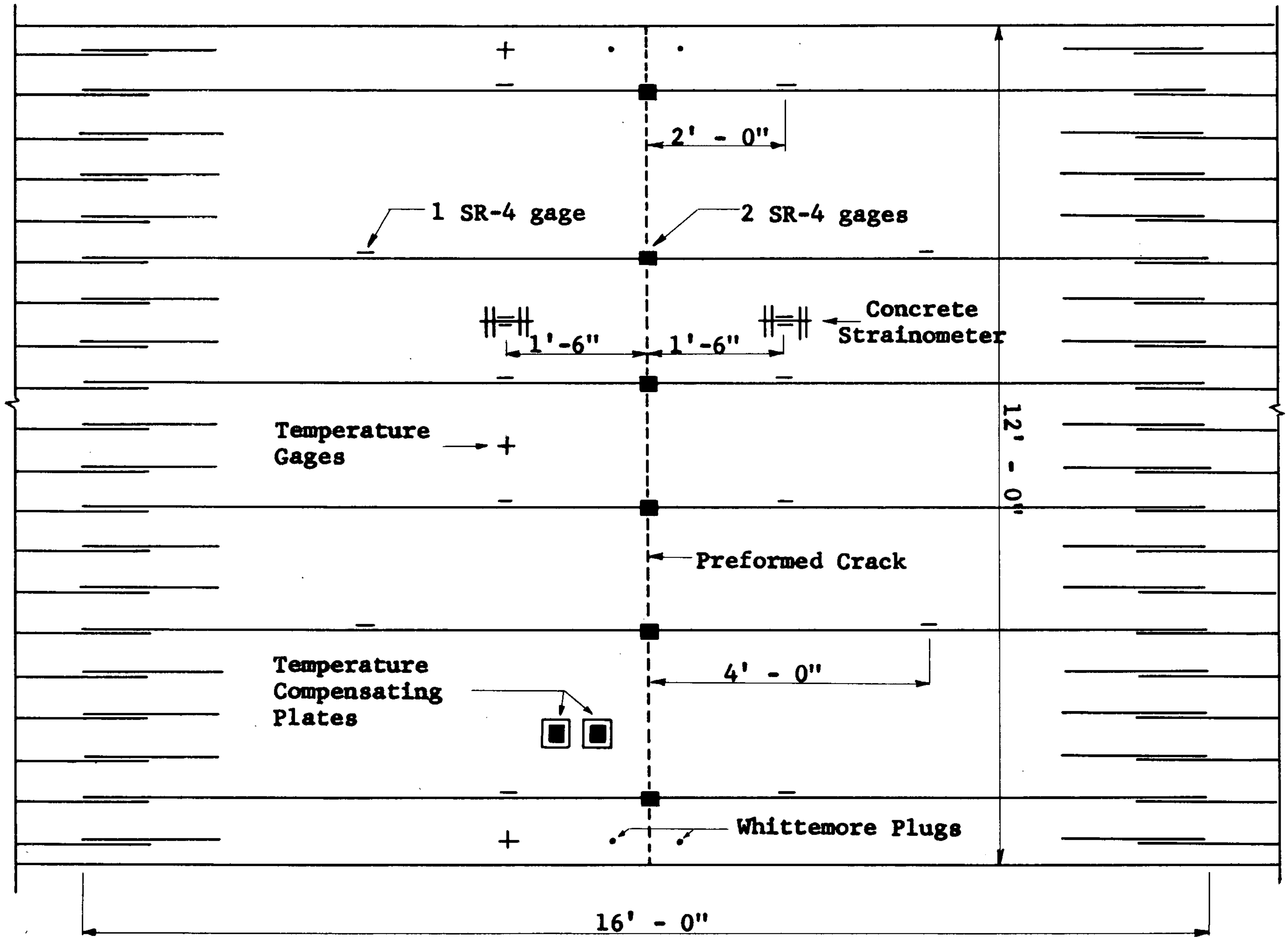

Figure 7 Instrumentation Plan—York Project

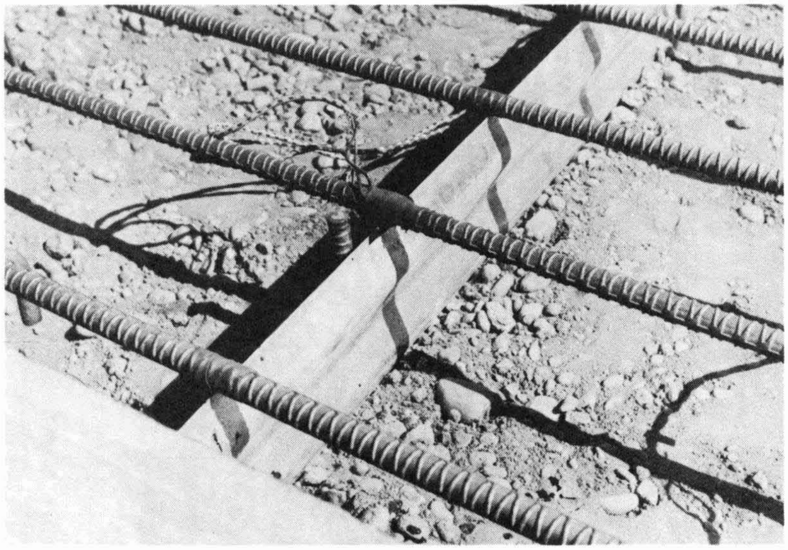

Figure 8 Installed Instrumented Bar-Mat Showing Instrumented Bars, Uninstrumented Bars, and Crack-Former in Place

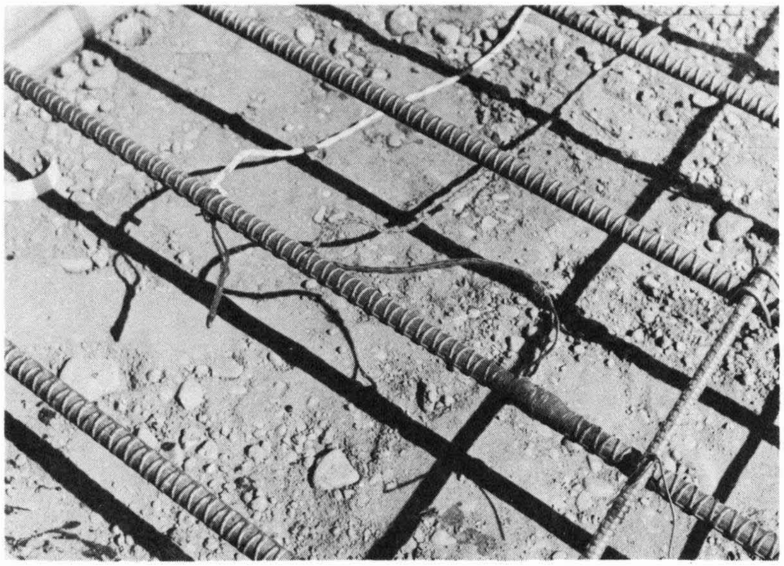

Figure 9 Bar Instrumented Two Feet Away from Crack, and Temperature Gage

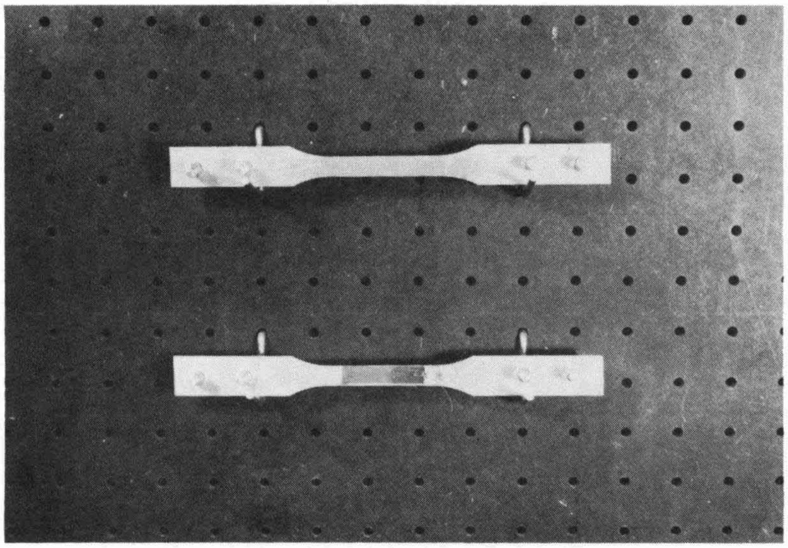

Figure 10 Concrete Strainometers, as Fabricated, and with SR-4 Gages Applied

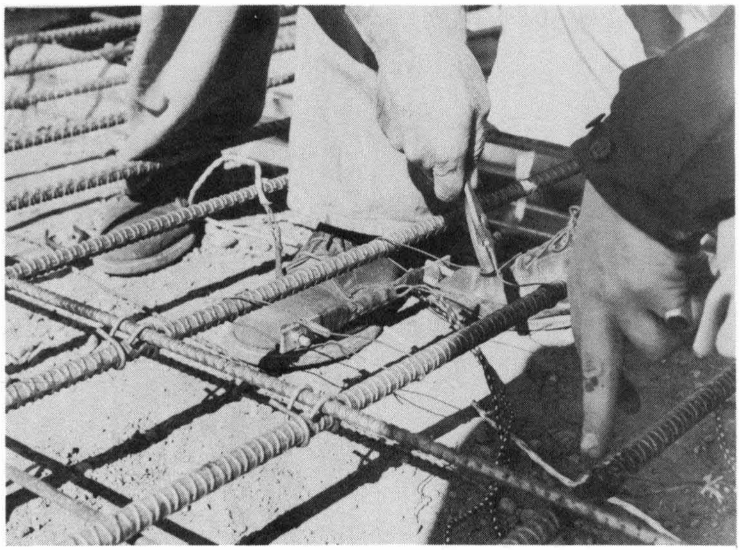

Figure 11 Placing of Concrete Strainometer in Test Panel

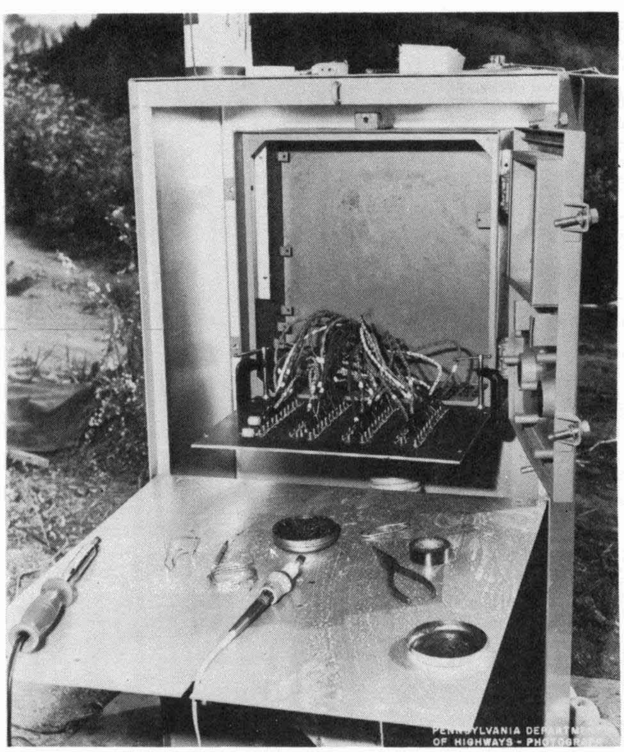

Figure 12 Terminal Board Showing Lead Wire Connections

Figure 13 Terminal Board Ready for Gage Reading, Showing Indicator Box and Jack Plugs Inserted into Board

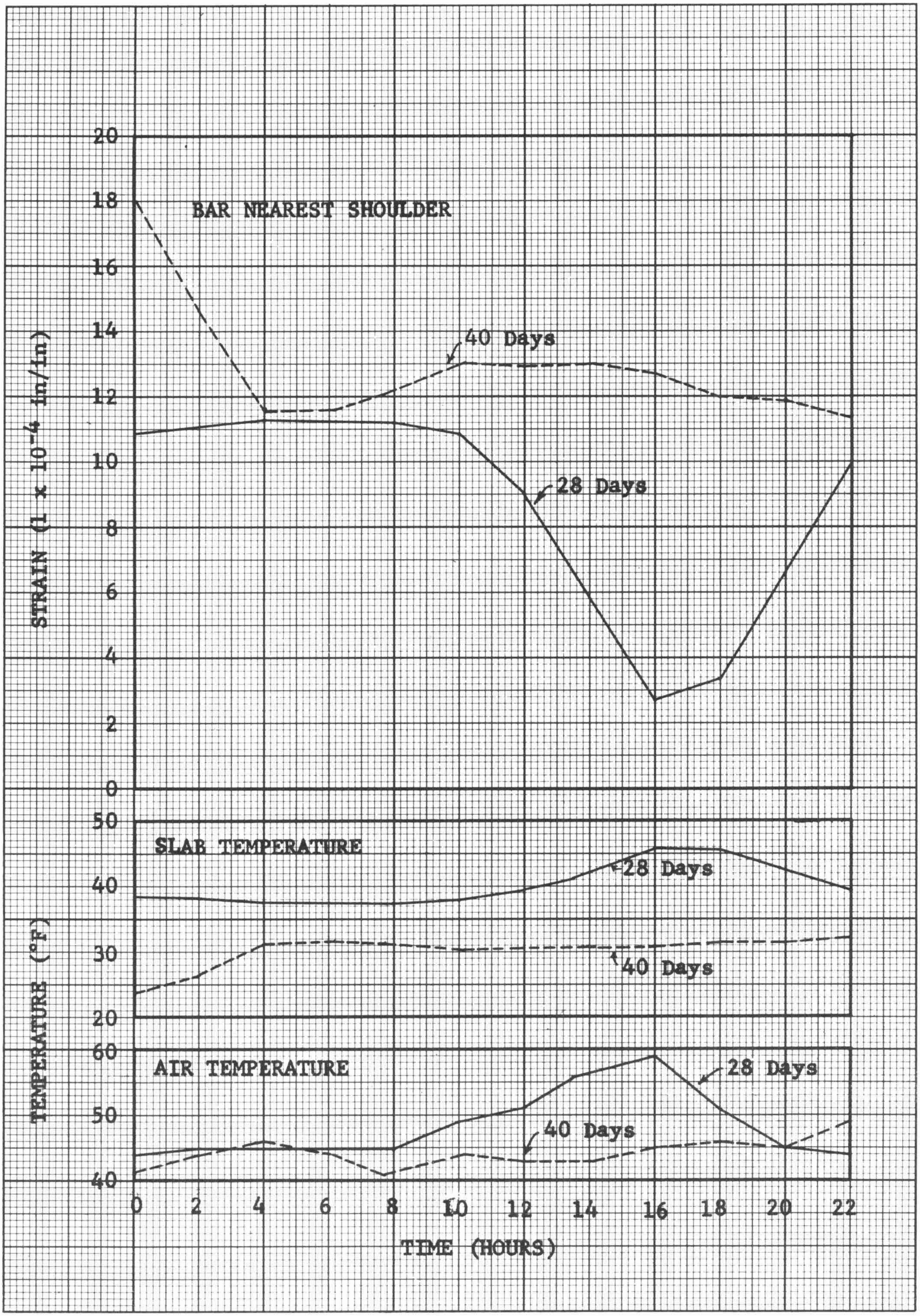

Figure 14 Short-Term Strain-Temperature Record for Instrumented Bar Nearest Shoulder at Preformed Crack

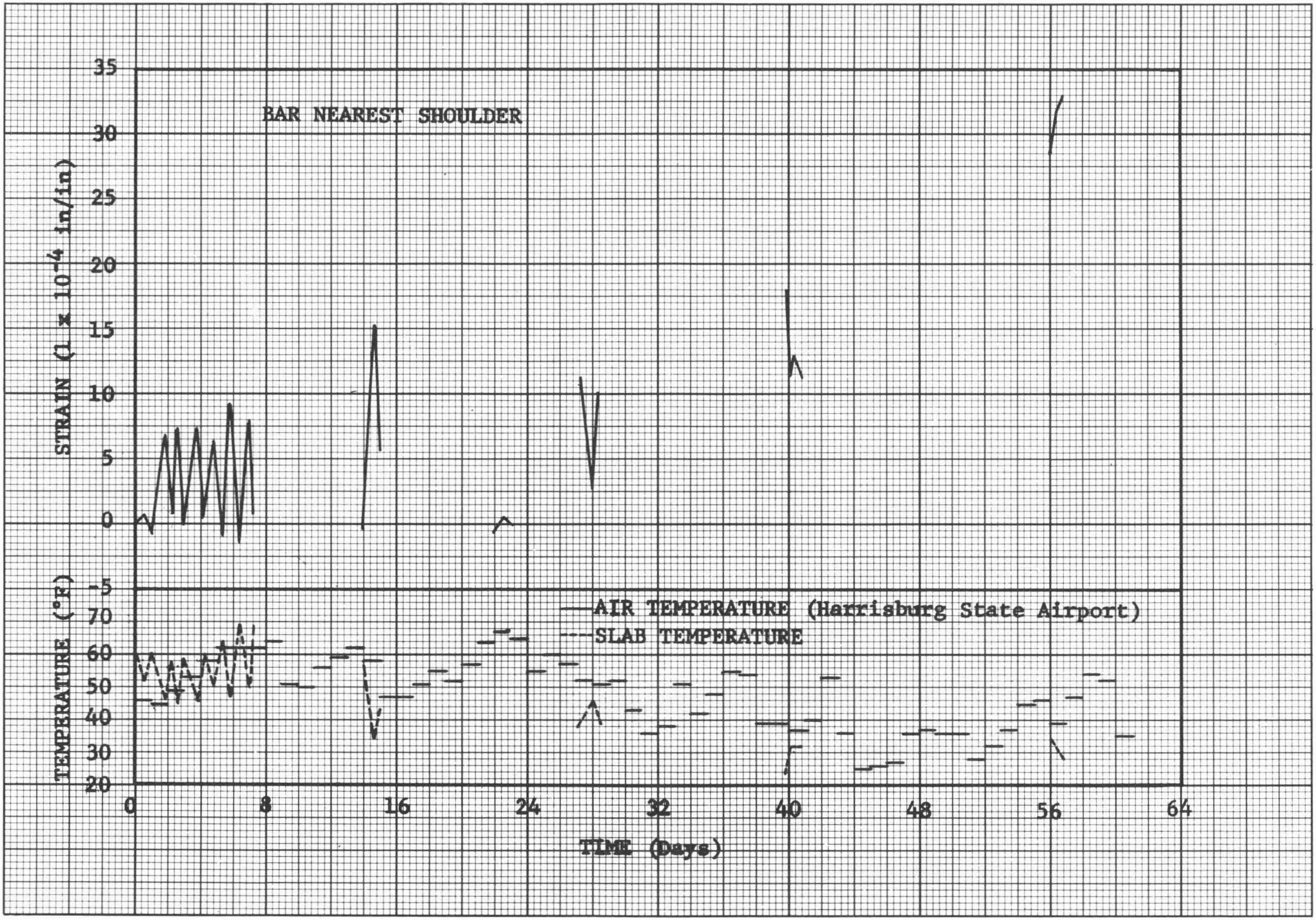

Figure 15 Strain-Temperature Record for First Fifty-Six Days of Pavement Life
for Instrumented Bar Nearest Shoulder, at Preformed Crack

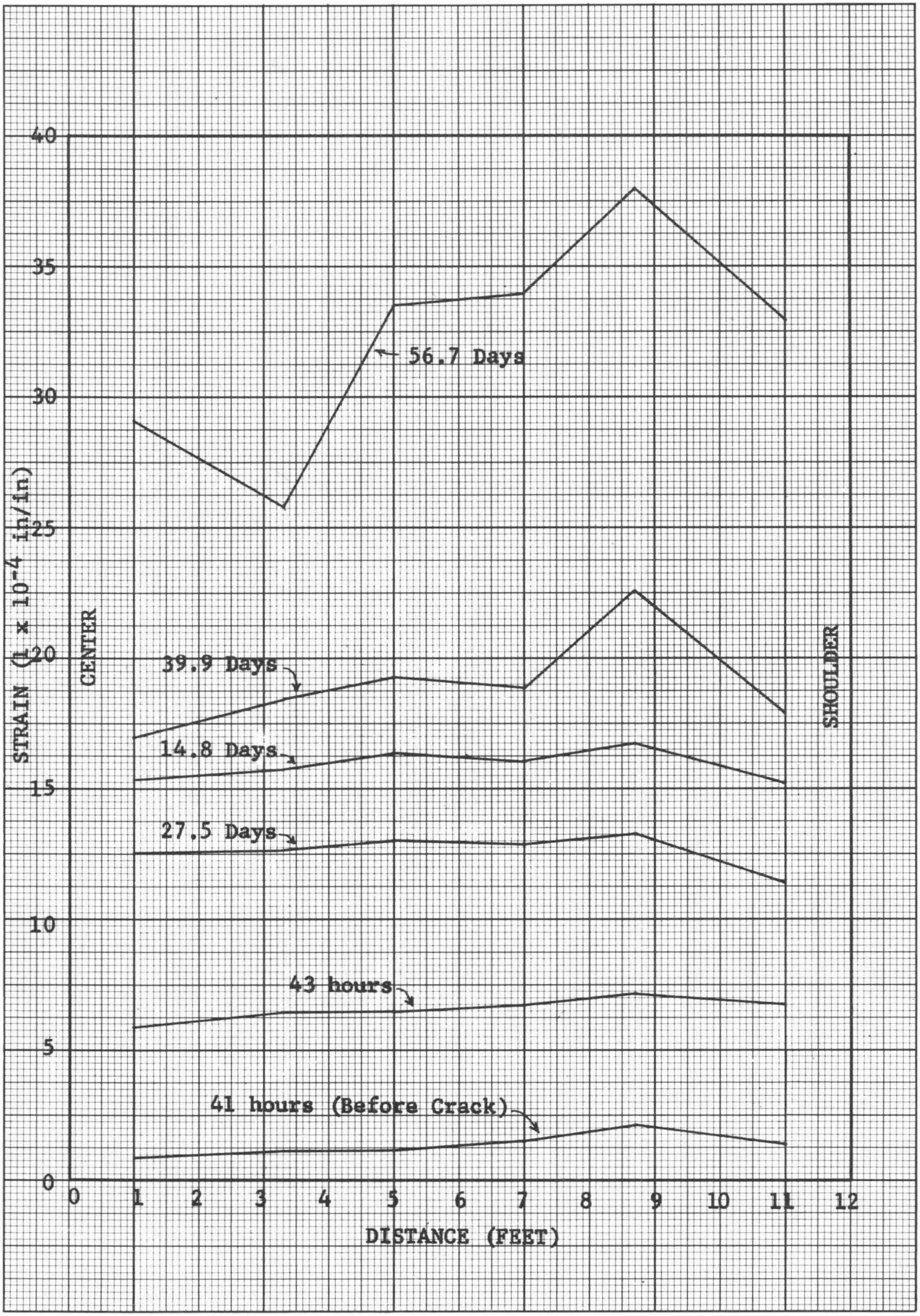

Figure 16 Transverse Distribution of Local Maximum Strains Across Pavement at Preformed Crack

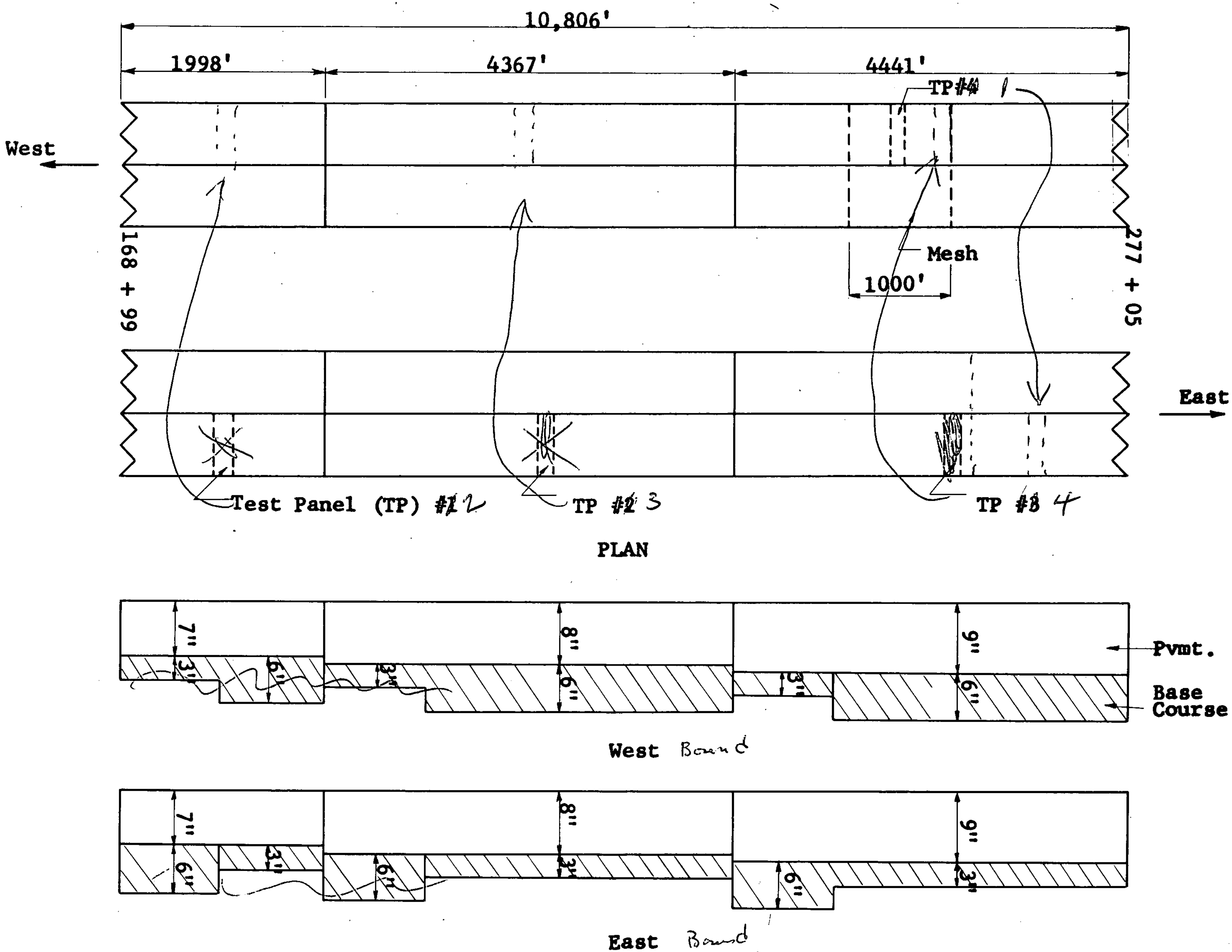

Figure 17 General Plan, Hamburg Project