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Seasonal Thermal Expansion and Contraction Characteristics of a Precast Prestressed Concrete Composite Pavement on U.S. Highway 14 - Bypass

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SEASONAL THERMAL EXPANSION AND CONTRACTION CHARACTERISTICS

OF A PRECAST PRESTRESSED CONCRETE COMPOSITE PAVEMENT

ON U.S. HIGHWAY 14 - BYPASS

BY

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This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Advisor / Date

Head, Civil Engineering Department / Date

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INTRODUCTION

Justification

Existing highway pavements are being taxed to their economic limit by present traffic volumes and wheel loads. The trucking industry contributes extensively to these conditions because a single truck wheel load is equivalent to a number of automobile wheel loads. The number of equivalent wheel loads increases significantly for each load increment with the increase being much larger for heavier loads than for lighter loads.(1) Development of long distance freight trucks will further increase highway traffic volumes and wheel loadings in the foreseeable future.

With present design standards, highway pavements must be much thicker to withstand increased traffic volumes and wheel loads. This increased thickness has exhausted aggregate supplies in some geographic areas. The scarce aggregate supplies must be transported over increased distances at increased costs to construct necessary pavements. Thus, present highway pavements are becoming economically unfeasible and a look at other types of pavements is warranted.

Extended use of prestressed concrete pavements may be the answer. Concrete has a high compressive strength, and steel has a high tensile strength. Prestressed concrete is a workable combination of the two which produces a higher strength concrete. Usually, highly tensioned steel strands are placed in the concrete to obtain the prestress. The tension is obtained by either the pre-tensioning or the post-tensioning method.

Pre-tensioning is accomplished by stressing steel strands against exterior abutments prior to pouring the concrete around the strands. After the concrete has reached sufficient strength, the strands are released from the abutments. This transfers the tensile stress of the strands into compressive stress in the concrete. The result is prestressed concrete.

Post-tensioning is accomplished by a different procedure. Conduits are placed within the form and the concrete is poured around them. After the concrete has reached sufficient strength, steel strands are threaded through the conduits and tensioned by jacking against the concrete. The strands are anchored and the jacks are removed. This also results in prestressed concrete. Another post-tensioning method is accomplished by compressing the member between two abutments by jacking. This method is used for prestressing concrete without the use of steel strands.

Prestressed concrete is stronger and more flexible than regular concrete when used as a pavement. Economy can be realized from prestressed concrete pavements because the increased flexibility makes more effective use of the subgrade. Shrinkage cracks are eliminated after the prestress is initiated. The prestress controls pavement cracking because it keeps cracks closed. This prevents the entrance of moisture and other foreign material into the subbase. These desirable qualities of prestressed concrete make possible a pavement of reduced thickness.

Reduced pavement thickness and short term traffic interruption are measures of economy in the construction of a highway or airport pavement. Areas of extreme traffic volumes are seeking a pavement which can be placed in a minimum amount of time. Periods of low traffic volume can be used for the placement of the pavement. This may be realized with the use of precast prestressed concrete pavement panels.[1]

Previous Prestressed Concrete Pavements

Prestressed concrete pavements have advanced extensively since the first prestressed concrete test slab was constructed at the Orly International Airport near Paris, France, in 1946. This pavement proved to be stronger than regular concrete pavements. Based upon the favorable results of the test pavement, a section of a main runway was constructed of prestressed concrete. Thirty-eight inch square, 6 1/4-inch thick precast concrete slabs were placed and post-tensioned

[1]Letter from Mr. Bernard J. Geisher, Public Works Department, Milwaukee, Wisconsin, August 13, 1969.

to form the pavement.(2) The first significant application of prestressed concrete used as a pavement occurred in 1954 at the Maison Blanche Airport at Algiers. The 7.1-inch thick runway and taxiway were prestressed by jacking between abutments with hydraulic flatjacks. Based upon construction costs, the prestressed pavement was selected in preference to a 14.5-inch regular concrete pavement.(3)

The European trend to prestressed concrete pavements moved forward in 1959. Melsbroek Airport at Brussels, Belgium, used prestressed concrete for a runway. The 7.1-inch thick pavement was prestressed by jacking between transverse ribs with flatjacks. The airport also constructed an experimental taxiway. Four-inch thick precast prestressed concrete slabs measuring 39 feet long by four feet, one inch wide were used.(4) The Vienna, Austria, airport used six-inch thick prestressed concrete pavement for both the runway and taxiway. This post-tensioned pavement included a horizontal curve located in the taxiway.(3) A Swiss road used reinforced concrete wedges at given intervals for the prestressing force. Other European roads used transverse ribs or rows of short batter piles as abutments for prestressing the pavement.(4)

The United States' interest in prestressed concrete pavements began in the middle nineteen-fifty's. A study at the University of Missouri in 1958 studied creep and warping in prestressed concrete.(5) In 1957, the Jones and Laughlin Steel Company of Pittsburgh, Pennsylvania, constructed a 530 foot long, five-inch thick experimental prestressed concrete highway pavement. Load tests showed the pavement to be satisfactory in load carrying capabilities. A rubber type expanding material was used at the pavement's joints. It was found that a problem of providing adequate shear transfer at the joints may exist, so a concrete sleeper slab was placed beneath the pavement joints to provide load transfer. An indication of the pavement's flexibility was exhibited when a 16,000 pound wheel load caused a 0.048-inch deflection under the load, and three to four feet away the pavement was unstressed.(6) Then, in 1958, a four-inch thick prestressed concrete pavement overlay was placed over an existing pavement for load testing by the United States Corps of Engineers at their Sharonville, Ohio, facility.(7)

Application of prestressed concrete pavements in the United States began with a four-inch thick post-tensioned overlay pavement at the airport near San Antonio, Texas. This pavement was laid over an existing pavement which had been damaged by large aircraft.(8) In 1959, a prestressed concrete pavement was constructed at Biggs Air Force Base near El Paso, Texas. The taxiway consisted of different pavement sections. It included 24 inches of regular concrete, 19 inches of reinforced concrete, and nine inches of prestressed concrete, respectively. The pavement was constructed with 500-foot joint spacings. The joint opening variations reached 0.5-inch daily and

1.5-inch seasonal readings.(9) The military has also used prestressed concrete for other airport pavements.(3)

In 1961, Emil Hargett of South Dakota State University envisioned the use of precast prestressed concrete pavement panels in a composite highway pavement. Gorsuch constructed half-scale panels in 1962 for laboratory testing to determine the structural behavior of the prestressed panels. The six panels were post-tensioned to form one panel. The resulting panel acted as a single slab structurally and held promise for future pavements.(10) In 1966, Kruse further studied the load carrying capabilities of prestressed panels. He used repetitive loads on panels which were tested with and without an asphaltic concrete covering. The 4 1/2-inch thick panels proved to be of sufficient thickness for load carrying capability. However, it was indicated that subbase pumping may occur.(11)

Jacoby conducted a two phase research study on panel connectors in 1967. One phase was a laboratory study of tongue and fork panel connectors on one-half scale width panels. The second phase was a study of temperature expansion and contraction on a 96 foot long by 24 foot wide tongue and fork connected test section. This section was constructed near the South Dakota Department of Highways building east of Brookings, South Dakota.(12) The favorable results of this research led to the present five-year study of a prestressed concrete composite pavement to be tested under regular highway traffic conditions.

Haug, in 1969, prepared a cost analysis of this pavement.(13) Hargett also suggested the use of these panels as an overlay in the strengthening of existing airport pavements.(14)

Scope of Study

An important structural consideration of rigid pavements is providing for expansion and contraction in the pavement. The purpose of this thesis is to determine the thermal expansion and contraction characteristics of a precast prestressed concrete composite pavement on U. S. Highway 14-Bypass near Brookings, South Dakota. Expansion and contraction measurements, obtained once every three months, and the corresponding temperatures were used to reach the objective. Precast prestressed concrete panels arranged transversely and longitudinally to the roadway will be considered.

Present research on the U. S. Highway 14-Bypass composite pavement is part of a five-year cost and performance study. The research project is a cooperative research study sponsored by the South Dakota Department of Highways and the U. S. Bureau of Public Roads.

PREPARATION OF TEST SECTION

Construction of the Roadbed

The 900-foot long precast prestressed concrete highway pavement test section is located midway between U. S. Highway 77 and Interstate 29 on the U. S. Highway 14-Bypass one-half mile north of Brookings, South Dakota. Grading of the naturally silty subgrade was accomplished during the 1967 construction season with the west portion of the subgrade being a fill section and the east portion a cut section. A three foot undercut was specified for the subgrade. The subgrade was constructed and compacted in eight to ten inch lifts until the designated grade elevation was reached. At this time, a three inch layer of subbase gravel was placed. See Figure 1. This concluded the construction of the subgrade and the 1967 construction season.

Description of the Pavement Panels

The precast prestressed concrete pavement panels were 24 feet long, six feet wide, and 4 1/2 inches thick. Four half-length panels, which were required at the ends of the longitudinal panel section, were exceptions to this basic panel size. See Figure 2. A keyway was provided on all adjoining panel edges. The keyway was filled with concrete grout to provide shear transfer from one panel to the next.

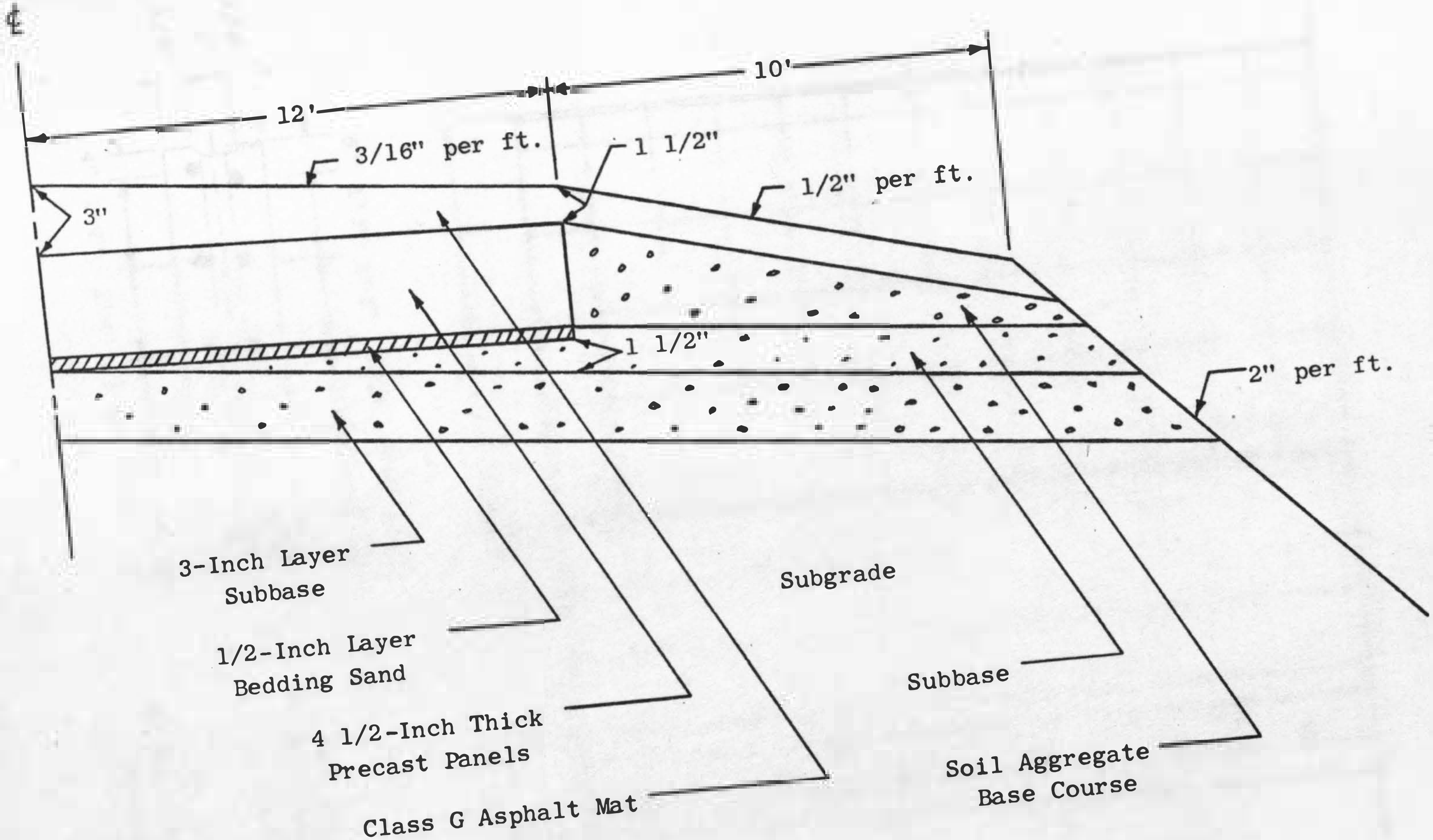


Figure 1. Typical Cross Section

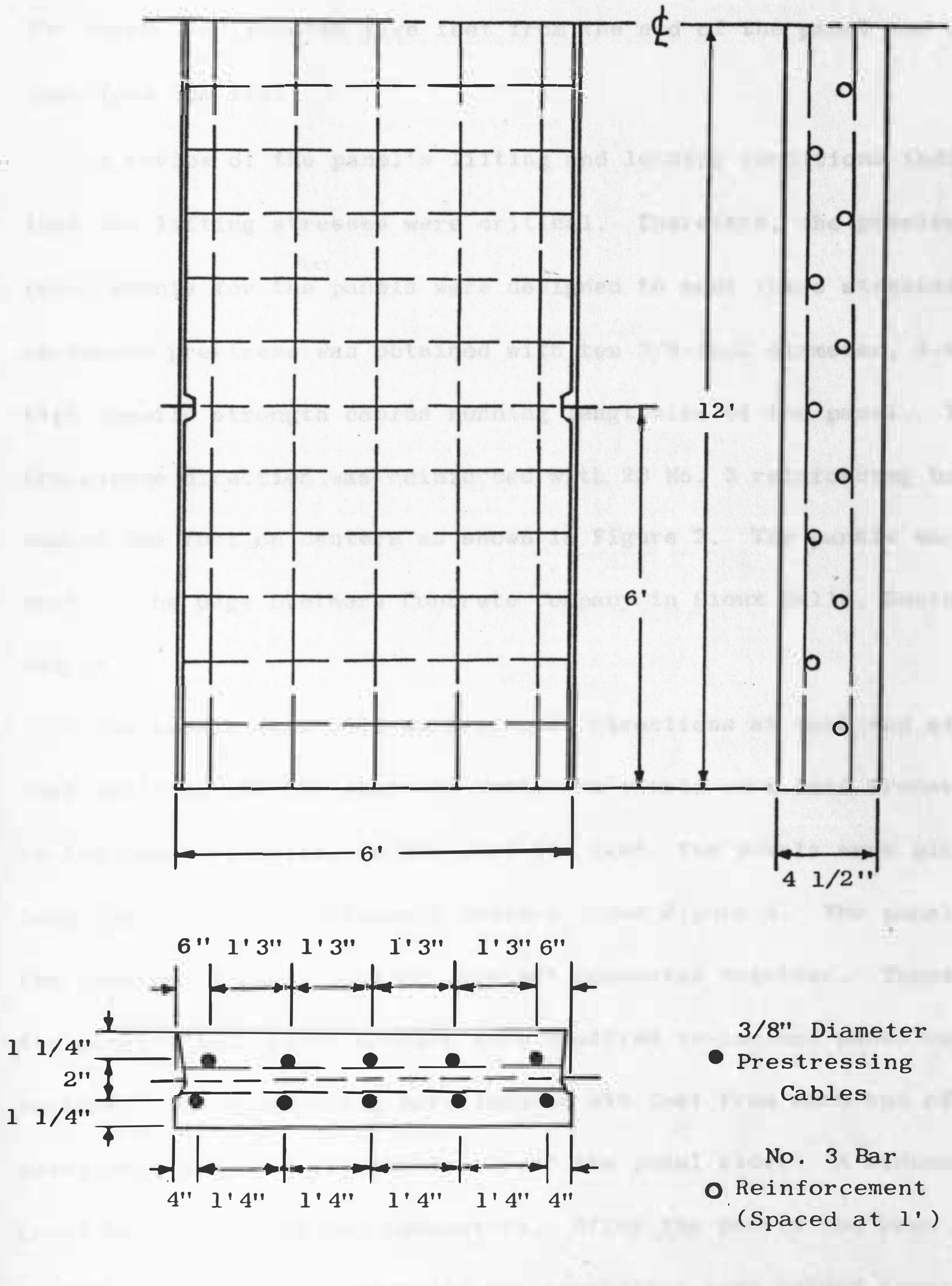
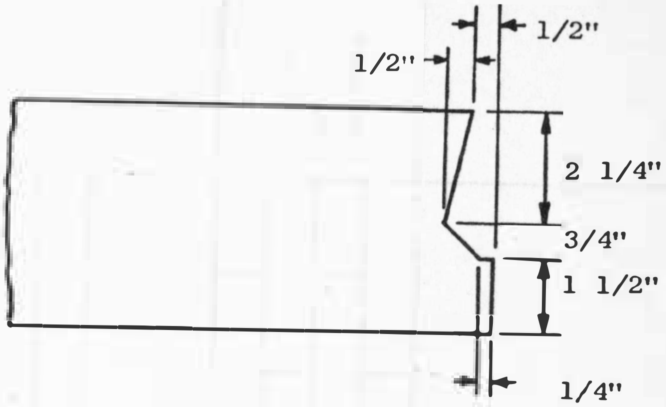


Figure 2. Typical Panel Section

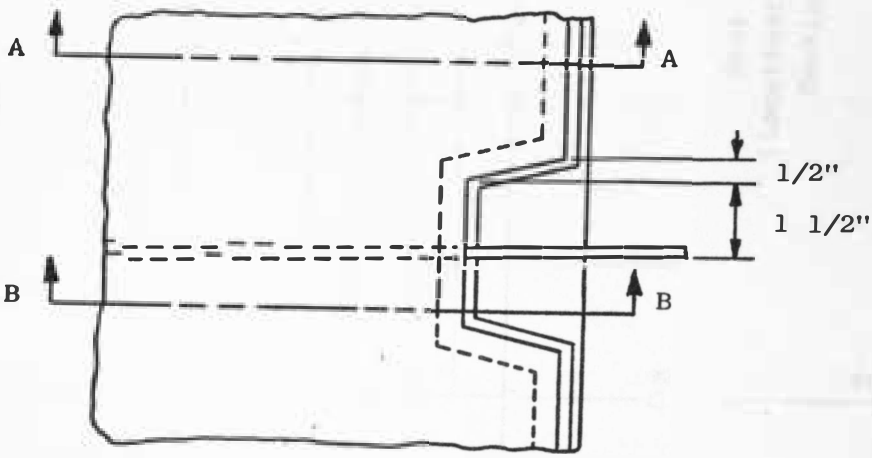
See Figure 3. Four steel lifting loops were provided on each panel. The loops were located five feet from the end of the panel and one foot from the side.

A review of the panel's lifting and loading conditions indicated that the lifting stresses were critical. Therefore, the prestressing requirements for the panels were designed to meet these stresses. The necessary prestress was obtained with ten 3/8-inch diameter, 7-strand, high tensile strength cables running lengthwise of the panel. The transverse direction was reinforced with 23 No. 3 reinforcing bars spaced one foot on centers as shown in Figure 2. The panels were pre-cast by the Gage Brothers Concrete Company in Sioux Falls, South Dakota.

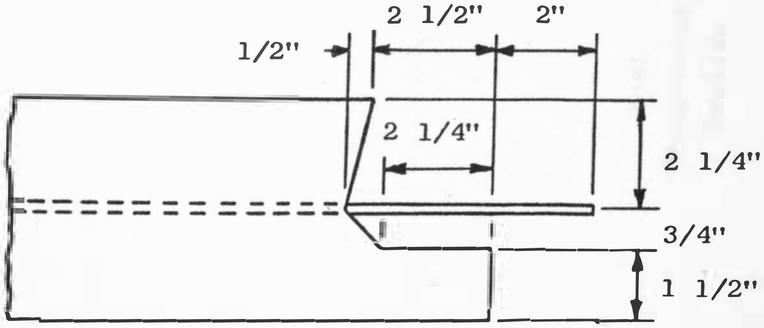
The panels were laid in different directions at each end of the test section. In the west 396 feet, the panels were laid transverse to the roadway; while, in the east 504 feet, the panels were placed longitudinally in a brickwork pattern. See Figure 4. The panels in the transverse panel section were not connected together. Those in the longitudinal panel section were modified to include panel connectors. The reinforcing bars located six feet from each end of the panel were extended two inches beyond the panel sides. A widened grout key existed at the connectors. After the panels had been placed on the subbase, the reinforcing bar connectors were welded together.



Grout Key Section
(Section A-A)



Connection Joint Detail



Connection Joint Section
(Section B-B)

Figure 3. Grout Key and Connection Joint Details

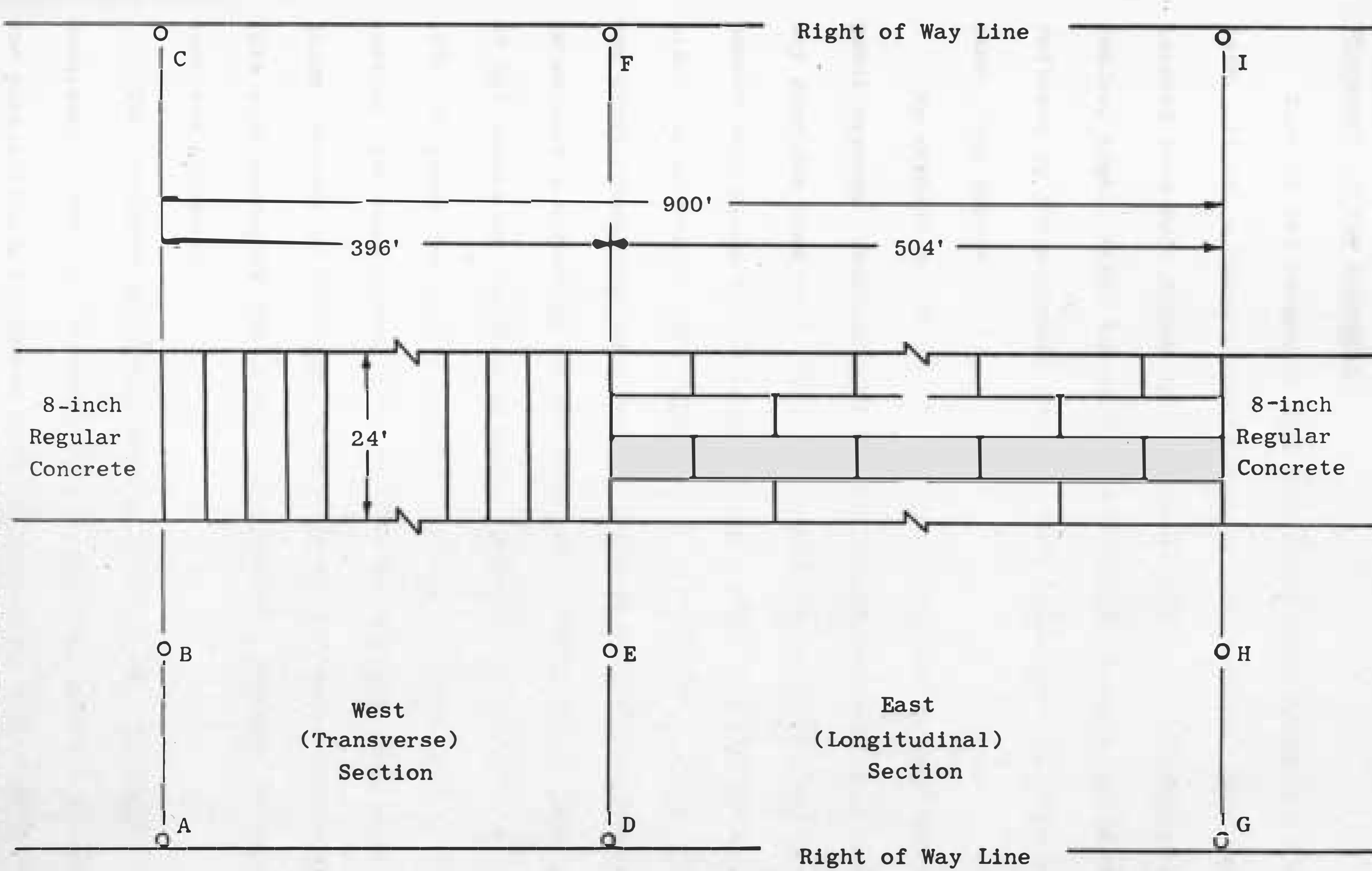


Figure 4. Layout of Test Section

Placement of the Pavement

Most of the composite pavement was placed during the summer of 1968. First, a layer of subbase gravel was placed. This layer tapered from zero inches at the center-line to 1 1/2 inches at the roadway edge. Next, the surface was leveled with an autograder followed by the placement of a one-half inch layer of fine bedding sand. See Figure 1.

An asphalt RC 250 bond breaker was applied to one edge of the panel keyways. This allowed expansion and contraction at the grout key from one side only. A truck mounted crane lifted and lowered the panels into place on the bedding sand. After the panels were in place, an acetylene torch was used to cut off the lifting loops, and the panel connectors were welded. Timbers were placed on the panels to support a vibrating roller which was used to assure firm placement of the panels on the bedding sand. Concrete grout was then pumped into the grout keyway and allowed to cure. At each end of the test section, an underpinning sill was poured monolithically with the adjacent section of the eight inch concrete pavement. See Figure 5. This sill connected the prestressed concrete pavement to the regular concrete pavement.

The remainder of the subbase gravel was then placed on the shoulders. Next, an asphalt concrete wearing surface was placed over the panels with a thickness varying from three inches at the roadway

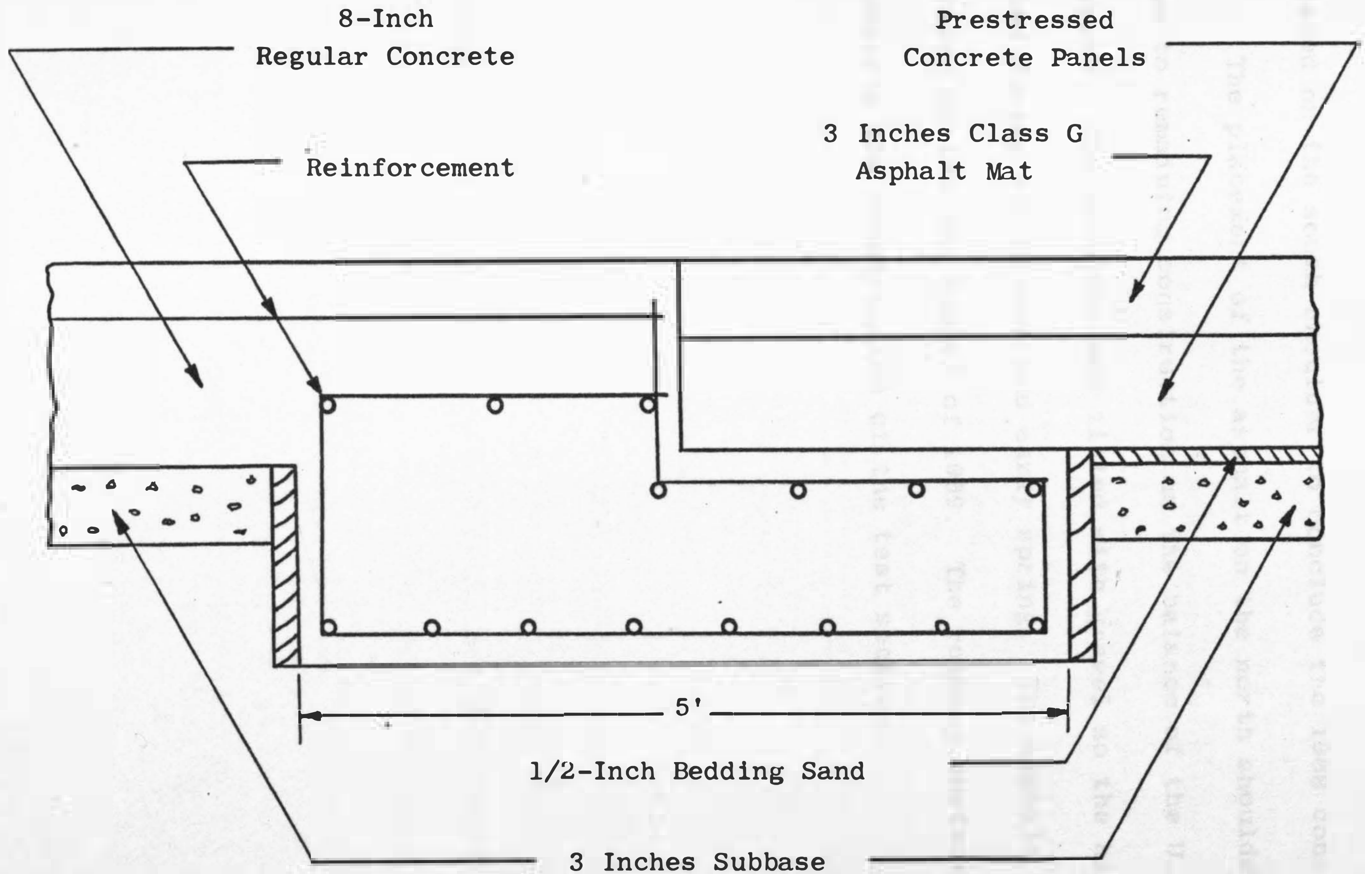


Figure 5. Detail of Underpinning Sill

center-line to 1 1/2 inches at the roadway edge. See Figure 1. This provided the crown for the pavement surface. Finally, asphalt was placed on the south shoulder to conclude the 1968 construction season.

The placement of the asphalt on the north shoulder was delayed due to remaining construction on the balance of the U. S. Highway 14-Bypass. The shoulder was filled with gravel so the highway could be used during the winter and early spring. The asphalt shoulder was placed during the summer of 1969. The roadway surface was primed to complete the construction of the test section.

TESTING

Instrumentation

Shortly after the construction of the prestressed concrete composite pavement, test points for obtaining research data were installed. Concrete control monuments were placed at three stations along the pavement, namely at each end of the test pavement and at the station where the panel alignment changed from transverse to longitudinal. Three monuments were installed along a transverse line through each of the above indicated stations. A monument was located near the right of way line on each side of the roadway; the third was located in the ditch at the base of the south shoulder. These are indicated as A, B, C, D, E, F, G, H, and I in Figure 4. For each monument, five foot deep holes were dug and sheet metal forms were used to contain the concrete. A brass cap was set in the concrete surface and a punch mark was added to locate transverse alignment. The line of sight was obtained by means of a theodolite.

The elevation of each monument was obtained by using precise differential leveling. Monument "A" was assigned an elevation of 100.000 feet. The distance between monuments "B" and "E" was taped as a base line for a triangulation network. The angles between each two adjacent monuments were obtained with a theodolite. This network may be retraced in case of suspected movement in order to relocate any monument which may have been disturbed.

In order to measure expansion and contraction of the composite pavement, 40 test points were installed as shown in Figure 6. The points were grouped in sets of two or four at panel joints. Two sets consisting of two points were located near the shoulder of the longitudinally placed panels while nine sets consisting of four points in the form of a square 12 inches on a side were located within the test section as shown. These were located in the eastbound lane between the south shoulder and the center-line throughout the entire length of the pavement. Marking gages were built by the Engineering Shops at South Dakota State University to accurately locate the inspection points. The gage for the sides of the square was 12.000 inches long while the one for the diagonal was 16.970 inches long. These are shown in the foreground of Figure 7.

Two-inch diameter cores of the asphalt wearing surface, extending down to the concrete panels, were removed as shown in Figure 8. At the bottom of the core holes, one-fourth inch diameter holes were drilled into the concrete panels to allow the placement of zinc plated steel studs. The studs were fixed in place with a mortar grout. The marking gages were used to locate the inspection points upon the surface of the studs. The inspection points were center punched to allow the use of a Berry strain gage for making measurements. Two-inch

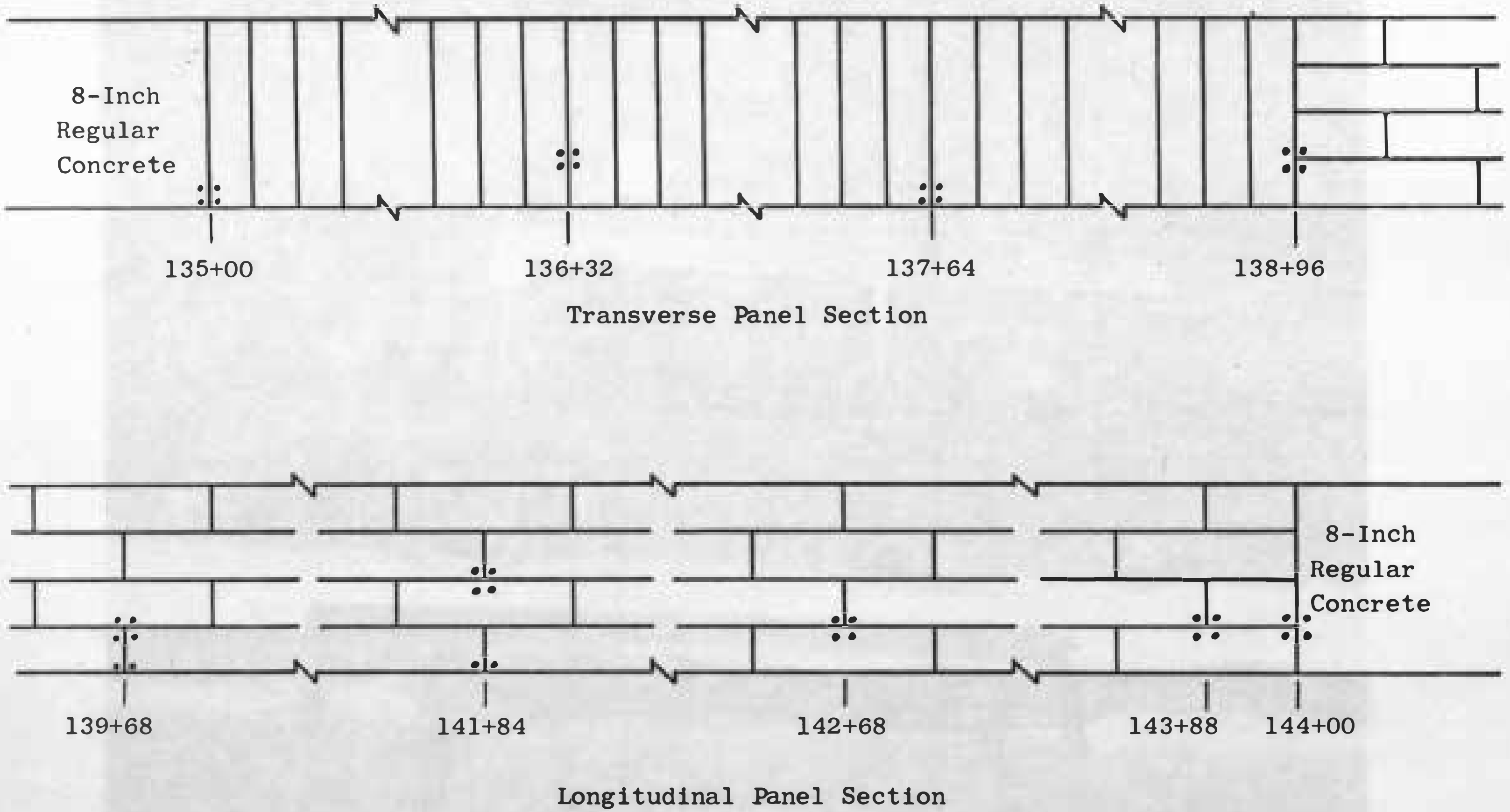


Figure 6. Location of Inspection Points

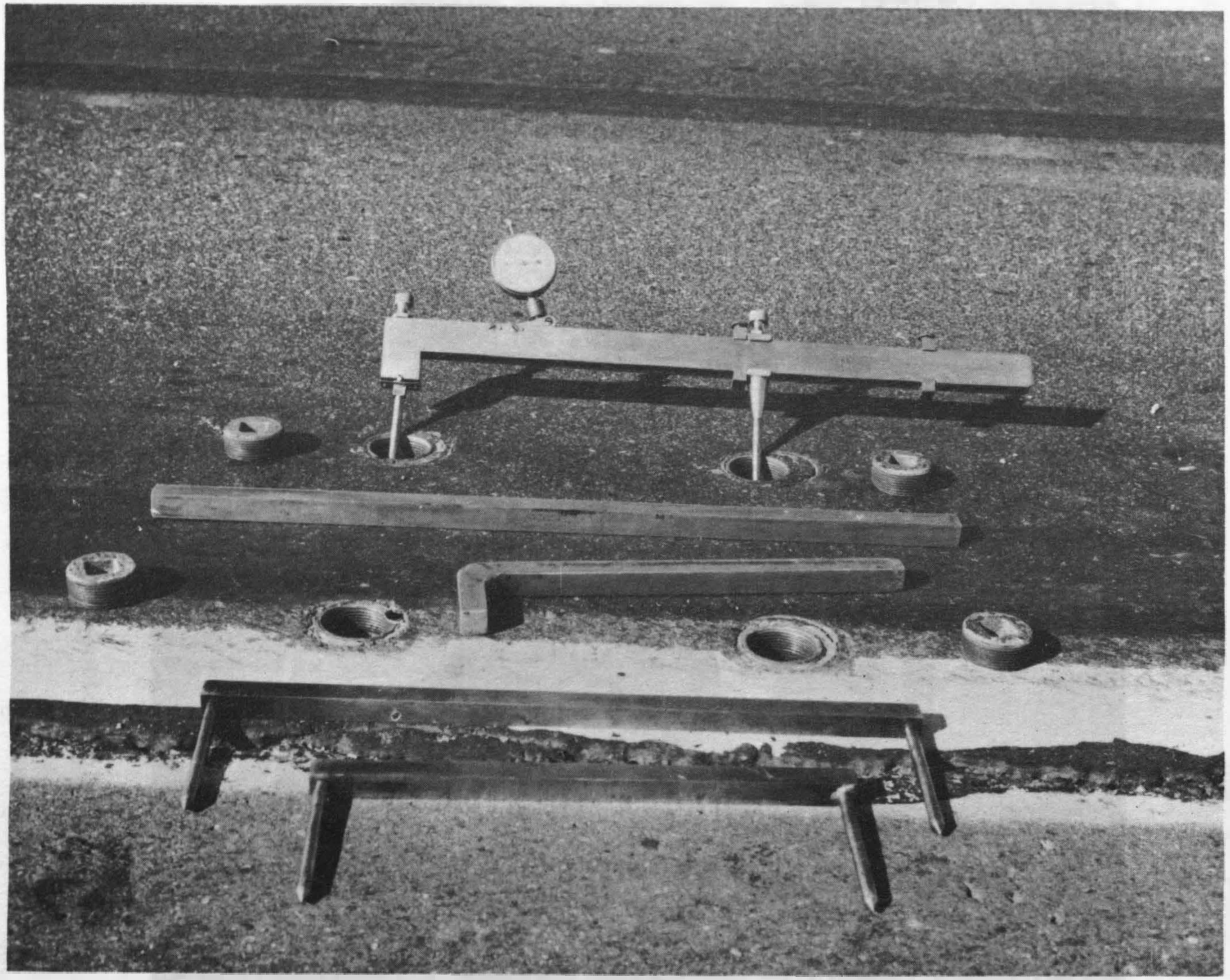


Figure 7. Equipment for Expansion and Contraction Measurements

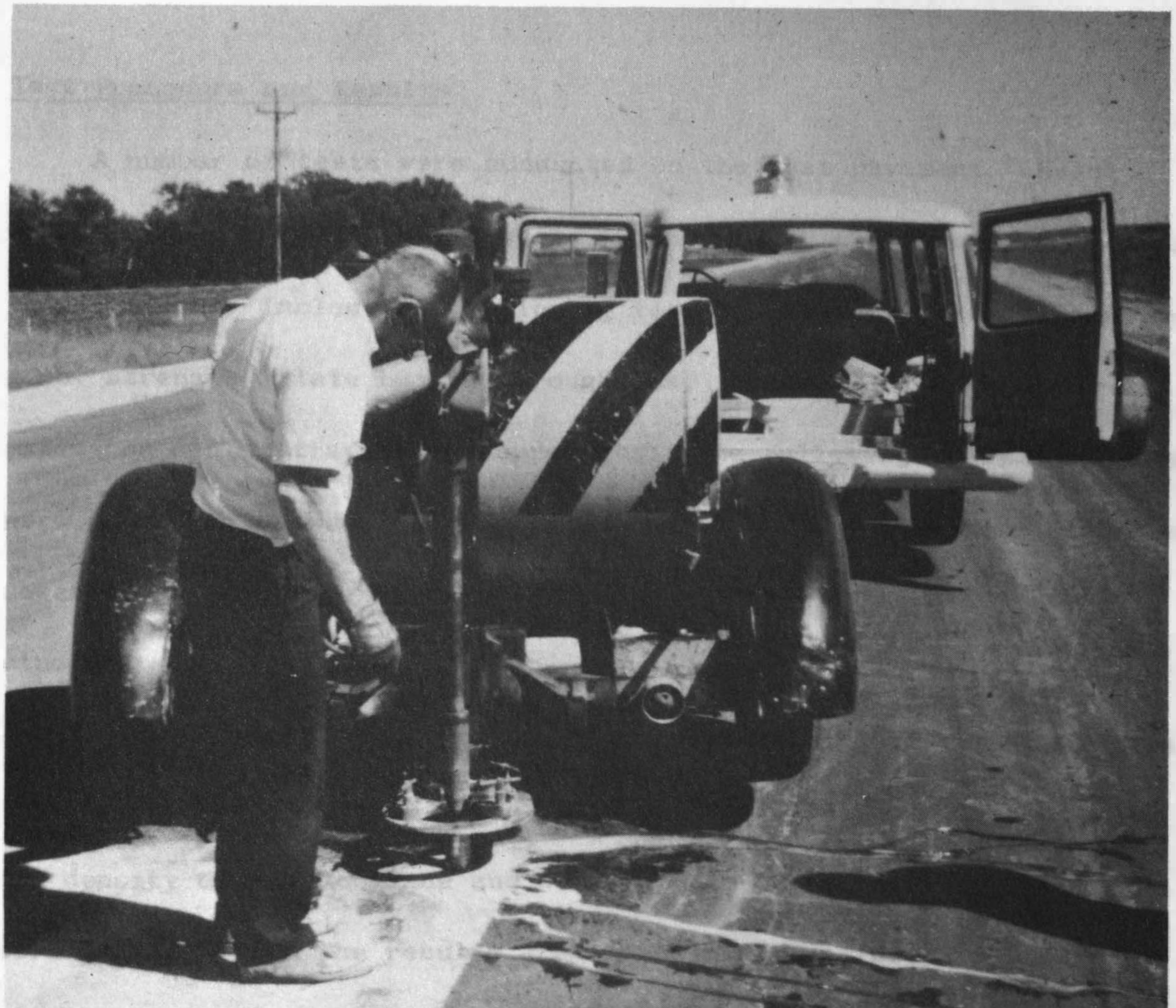


Figure 8. Core Drilling for the Inspection Holes

diameter pipe couplings were placed in the inspection holes and set with epoxy. Metal plugs were screwed into the couplings to protect the inspection holes from highway traffic and the elements. See Figure 9.

Test Procedure and Results

A number of tests were conducted on the test pavement. Results will be given only for those tests which support the objective of this thesis. These include the following tests: moisture content, density, grout strength, plate bearing, roughness, transverse alignment, and expansion and contraction measurements. The following additional tests were conducted to meet the overall objective of the five-year sponsored research project: Benkleman beam, vertical movements, and traffic studies.

A South Dakota Department of Highways construction crew conducted the first series of tests on the test section. The moisture content and density of the subgrade and subbase were obtained during the construction period. The results are shown in Figure 10. The moisture content is given in terms of a percentage while the density is given in terms of pounds of soil per cubic foot. Test cylinders of the concrete grout were also prepared and tested. After 28 days of curing, the grout had obtained strengths ranging from 6640 to 6995 pounds per square inch.

Figure 9. Placing Couplings in the Inspection Holes

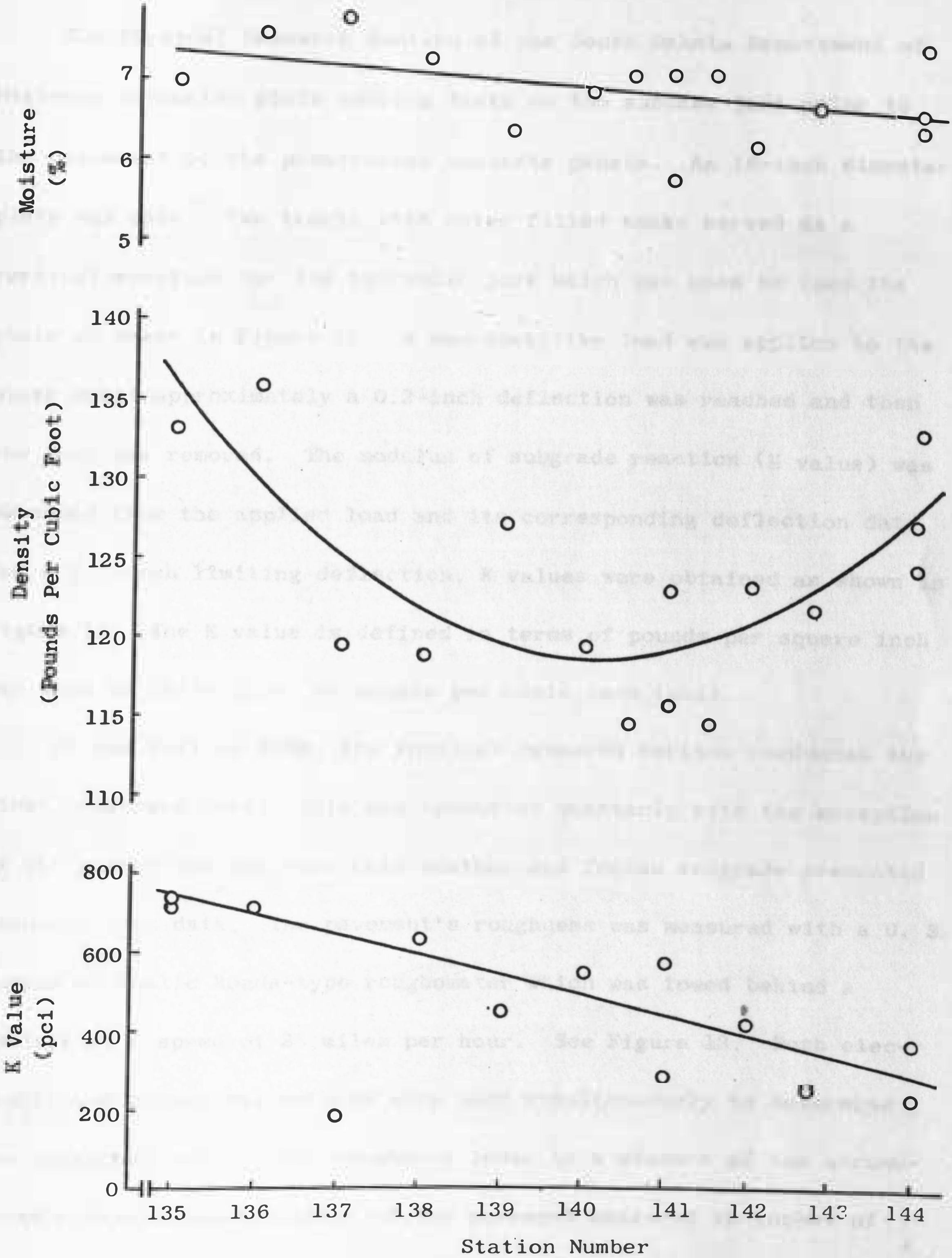


Figure 10. Moisture - Density - K Value Relationships

The Physical Research Section of the South Dakota Department of Highways conducted plate bearing tests on the subbase just prior to the placement of the prestressed concrete panels. An 18-inch diameter plate was used. Two trucks with water filled tanks served as a vertical reaction for the hydraulic jack which was used to load the plate as shown in Figure 11. A nonrepetitive load was applied to the plate until approximately a 0.2-inch deflection was reached and then the load was removed. The modulus of subgrade reaction (K value) was obtained from the applied load and its corresponding deflection data. For a 0.1-inch limiting deflection, K values were obtained as shown in Figure 10. The K value is defined in terms of pounds per square inch per inch of deflection, or pounds per cubic inch (pci).

In the fall of 1968, the Physical Research Section conducted the first roughness test. This was conducted quarterly with the exception of the winter quarter when cold weather and frozen subgrade prevented accurate test data. The pavement's roughness was measured with a U. S. Bureau of Public Roads-type roughometer which was towed behind a vehicle at a speed of 20 miles per hour. See Figure 12. Both electronic and mechanical methods were used simultaneously to determine the roughness index. The roughness index is a measure of the accumulated riding characteristics of the pavement measured in inches of

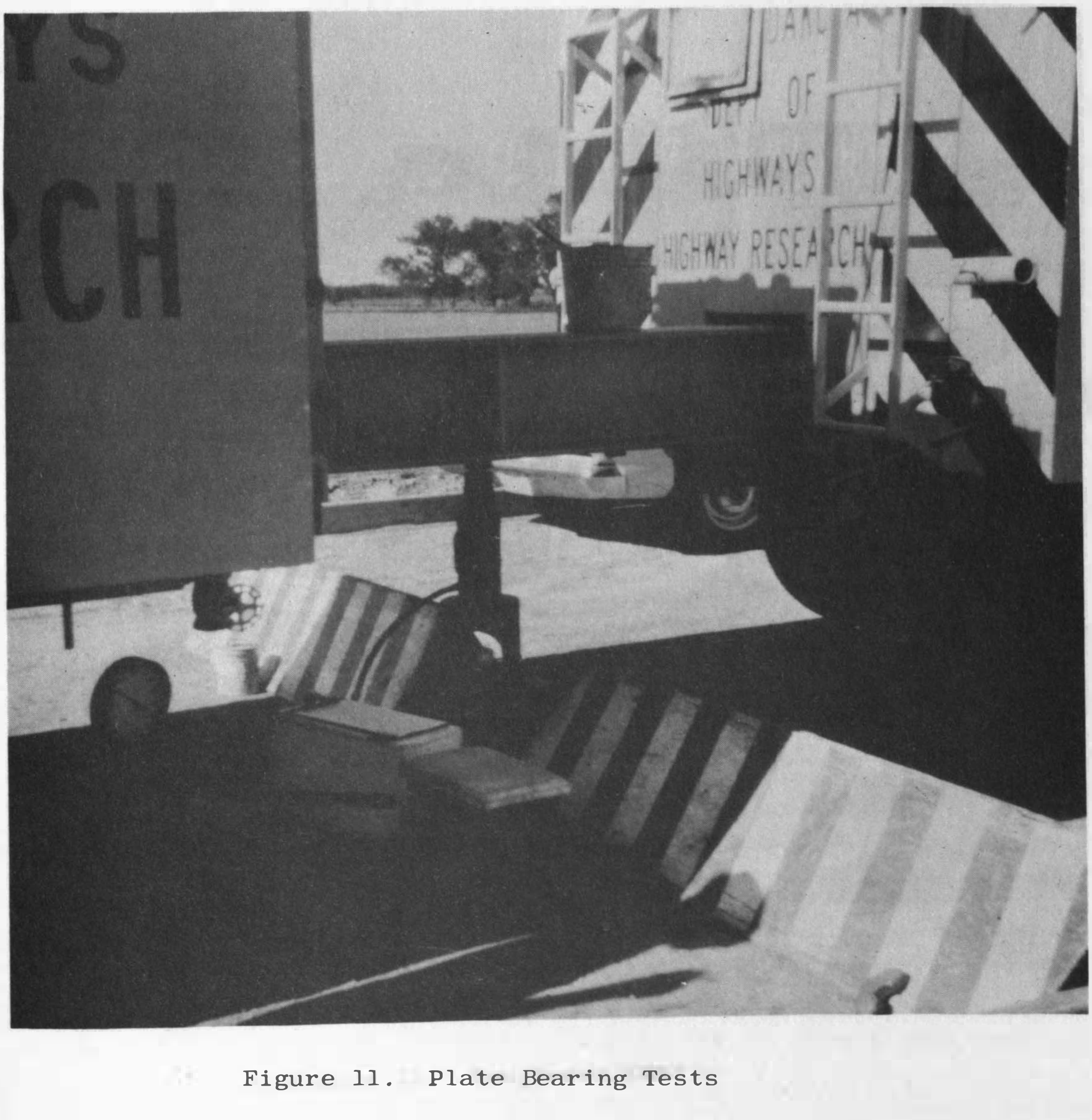


Figure 11. Plate Bearing Tests

Figure 12. Roughness Test

roughness per mile. To date, the roughness index of the test section has ranged from 111 to 146 inches per mile as shown in Table 1, Appendix A.

The study of expansion and contraction at the pavement's panel joints was initiated in the fall of 1968 by the Civil Engineering Department at South Dakota State University. One phase of this included checking the transverse alignment of the concrete monuments. After the line of sight had been established, the distance from the line to each of the four nearby inspection points was measured to the nearest 0.01-inch. The original measurement was subtracted from each new measurement to give the net movement of the points with respect to the line of sight. See Table 2, Appendix A.

The expansion and contraction of the panel joints were studied by measuring between inspection points with a Berry type strain gage. See Figure 13. Due to the depth of the inspection holes, three inch extensions were added to the gage points by the Engineering Shops. These extensions made it necessary to recalibrate the gage. The two foot long, three-fourths inch square steel calibration bar shown in the center of Figure 7 had drill marks indicating 12.000-inch and 16.970-inch gage lengths. These were modified to give a plus and minus 0.1-inch change for each gage length. A 0.125-inch dial reading was obtained for each 0.100-inch change in gage length. Therefore, the dial readings were multiplied by an 0.8 calibration factor to

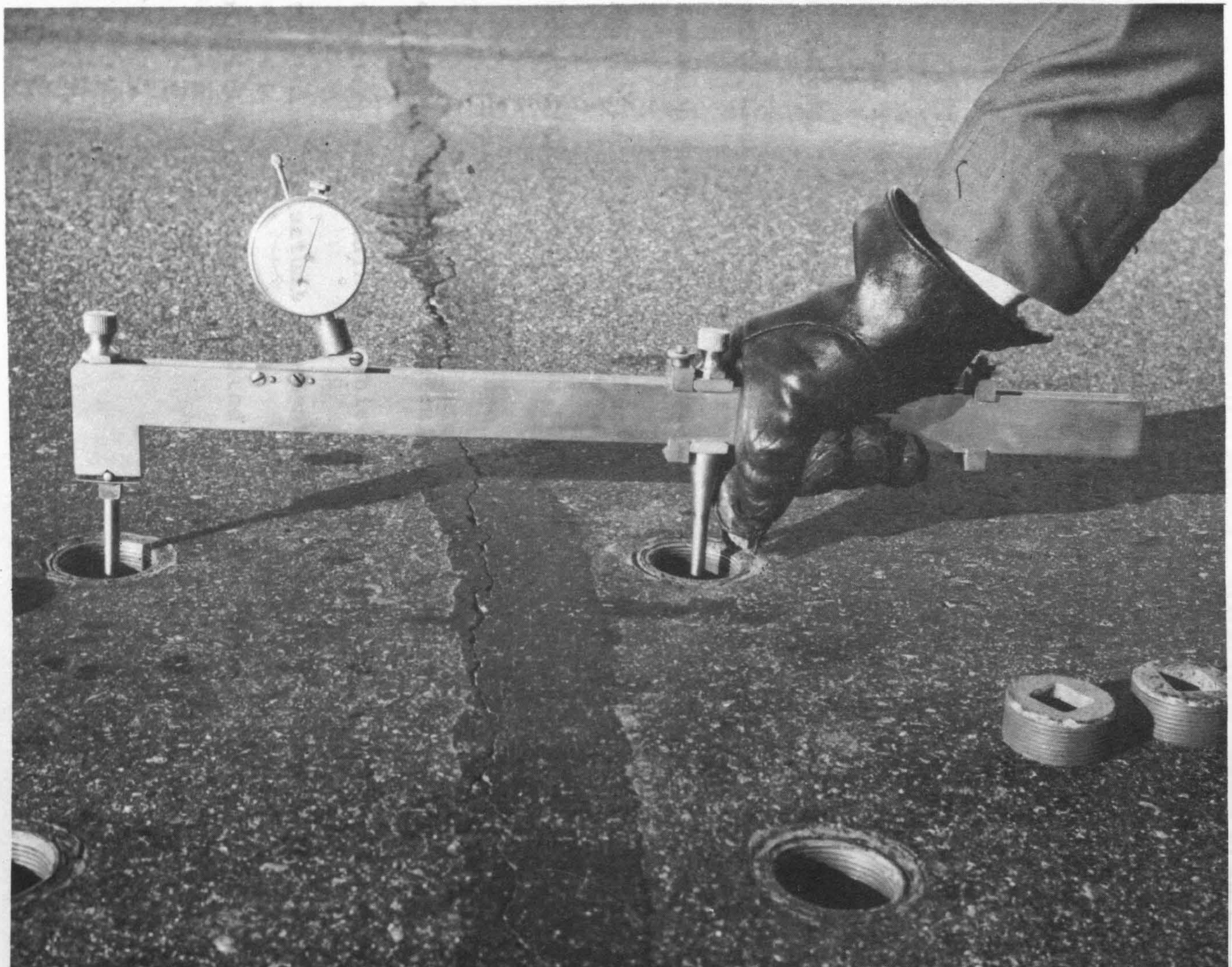


Figure 13. Test Measurement With A Berry Type Strain Gage

obtain the expansion or contraction between two inspection points measured in inches. The gage length was set for 12.000 inches or 16.970 inches by placing the gage points into the drill marks of the calibration bar. Accordingly, the same initial dial reading was obtained each time. This eliminated reducing the dial readings to their initial value.

After the gage length had been preset, the gage points were placed into the punch marked inspection points and a dial reading was obtained. This value was subtracted from the initial reading and multiplied by 0.8 to obtain the desired measurement. The temperature for each test period was obtained from the Weather Division of the Agricultural Engineering Department at South Dakota State University located three-fourths mile southwest.

Knowing the temperature change since the first measurement, the change of length in the strain gage due to thermal expansion or contraction was obtained. This length was added to the net measurement to obtain the actual movement between the two inspection points. The resulting data are tabulated in Table 3, Appendix A.

ANALYSIS OF TEST RESULTS

Discussion

A temperature change in highway pavements creates expansion or contraction within the pavement. Subgrade resistance causes stresses to develop within the pavement when the expansion or contraction occurs. When the tensile stresses exceed the tensile strength of the concrete or asphalt in the pavement, cracks appear. The cracks reduce the load carrying capacity of the pavement because the crack will not transfer a load. Moisture and other foreign materials often enter the cracks to further reduce the effectiveness of the pavement.

Throughout this study, thermal expansion and contraction of a prestressed concrete composite pavement on the U. S. Highway 14-Bypass have been observed. Visual observations were conducted in addition to the regular tests. No sign of pavement distress was observed. However, pavement reflection cracking was noted in the asphaltic concrete wearing surface above a number of panel joints. Some cracks were large enough to allow infiltration of foreign materials such as sand which was used for ice conditions during the winter. This was most noticeable at the terminal joints at each end of the transverse and longitudinal panel sections. Major reflection cracks of this nature should be filled with an asphaltic material to prevent further entrance of foreign materials which could eventually reduce the load carrying capability of the pavement. A crack was considered a major reflection crack if it was one-eighth inch wide and extended the full length of the panel joint. Minor reflection cracks were narrower and did not extend the full length of the panel joint.

In the transverse panel test section, the randomly spaced cracks were transverse to the road center-line. In the longitudinal panel test section, transverse cracks appeared at the ends of each panel. Therefore, as one looked across the pavement, two six-foot-long cracks separated by a six-foot-long gap were observed. No cracking was observed for the longitudinal panel joints in the longitudinal panel test section.

Major reflection crack spacings in the transverse panel test section are listed in Table 4, Appendix A. The mean value for these spacings is 39.6 feet as compared with the median and mode values of 48 feet. Several minor cracks appeared between the major ones. The panels in the transverse panel test section were not connected together as in the longitudinal panel test section. However, it is felt that some connection existed through the bonding of the grouted keyways and asphaltic concrete wearing surface. Considering the absence of panel connectors, the major crack spacings were significantly large. This may be due to a relatively low coefficient of subgrade resistance which would be obtained with the smooth precast panel undersurfaces. A larger mean crack spacing would have been obtained if the six-foot crack spacing next to the regular eight-inch pavement had not appeared. It is felt that the underpinning sill below this panel caused the crack. Bonding between the panel and the sill or a non-uniform subgrade condition may have existed.

At the time of the November 1969 measurements, corroded inspection points were encountered. Erroneous data were obtained from some of these points. Two measurements were not obtained at that time because of the corroded points. All inspection points were repunched at the time of the February 1970 measurements. Stainless steel studs should be used in place of the zinc plated studs to eliminate this problem in the future.

Adequate subgrade and subbase were provided for the pavement according to Figure 10. Some variation in density occurred with a lower value existing at the center of the test pavement than at the ends. Proceeding from the west end to the east end, both the K value and the moisture content decreased. Thus, a slightly reduced subgrade reaction was indicated for the east end.

The grout strength, being over 6000 pound per square inch, was acceptable according to the specifications for the project. Accordingly, it is not probable that the expansion and contraction would cause failure in the grout keyways.

Movements of nearby inspection points about the line of sight through the control monuments indicated an overall shortening of the test pavement. A study of the movements at station 138+96 indicated an expansion of the longitudinal panel arrangement during the summer and fall followed by contraction during the winter. The results for May, 1969, showed nearly equal expansion in each panel arrangement. This indicates that a large section of the longitudinal panel arrangement acted as an independent unit more so than the transverse panel arrangement. Individual panels were probably restricted from individual expansion and contraction by the panel connectors and by bonding and frictional resistance of the grout key. A portion of the expansion and contraction in a panel was probably transferred to the adjacent panels. It is felt that longitudinal panel test pavements of greater length may exhibit major transverse cracking.

Thermal expansion and contraction and the resulting reflection cracking apparently have not affected the roughness of the pavement. During the 1969 season, the roughness index decreased with each consecutive measurement. See Table 1, Appendix A. A portion of the reduction in the last measurement of 1969 may be attributed to the asphalt prime coat applied to the surface of the pavement at the end of construction just after the July, 1969, measurement.(15)

The expansion and contraction measurements were divided into a number of categories depending upon the location and direction of the measurements. The two major categories were longitudinal and transverse measurements. Each of these was subdivided into across-panel joint or in-panel measurements. The longitudinal across-panel joint measurements were further subdivided into the following types of measurements: panel section terminal joints, transverse panel section joints, longitudinal panel section joints, and longitudinal panel section shoulder joints. The transverse in-panel measurements were within either the prestressed concrete panels or the eight-inch regular concrete pavement located at each end of the test pavement. An average measurement was computed when two parallel measurements were obtained for the same category at a given station. Notations used to describe each category are given in Appendix B.

The strain gage measurements and the corresponding temperatures were evaluated to obtain an understanding of expansion and contraction in a precast prestressed concrete composite pavement. The accumulated data were entered into the I. B. M. cataloged Polynomial Regression, scientific subroutine, computer program to obtain curves to represent the data. Four measurements were considered erroneous due to corroded inspection points and therefore excluded from the program. The least squares method for obtaining the best fitting curve was used. Curves ranging up to fourth degree could be obtained without modifying the program. Although second and third degree curves were obtained, first degree curves (straight lines) were considered most representative for the limited amount of data available at this time. The curves indicated trends in expansion or contraction as shown in

Figures 14 through 17. Three of the measurements used in the computer analysis were excluded from Figure 17 since they were beyond the scale of the graph.

The expansion characteristics within the prestressed panels and the regular concrete pavement are shown in Figure 14. The least expansion was obtained for the longitudinal in-panel measurements (curve X2) which indicated the smallest slope. This shows that some expansion occurred even though the panels were prestressed.

Without the end restraints acting upon each end of the test pavement by the regular concrete pavement, a larger expansion value may have been obtained. This is shown by the transverse in-panel expansion value (curve Y2) which was the largest in-pavement expansion value. Less restraints in the transverse direction may have accounted for this larger expansion. A portion of this value may also have been due to the erroneous data from the corroded inspection points. Except for the lower temperature values, the expansion value for the regular eight-inch concrete pavement (curve Y3) lies between those given for the prestressed panels.

The transverse across-joint expansion in the longitudinal panel section (curve Y1) shown in Figure 15 is almost identical to the longitudinal in-panel value (curve X2) of Figure 14. The absence of a cracked longitudinal joint may explain this low value. The uncracked

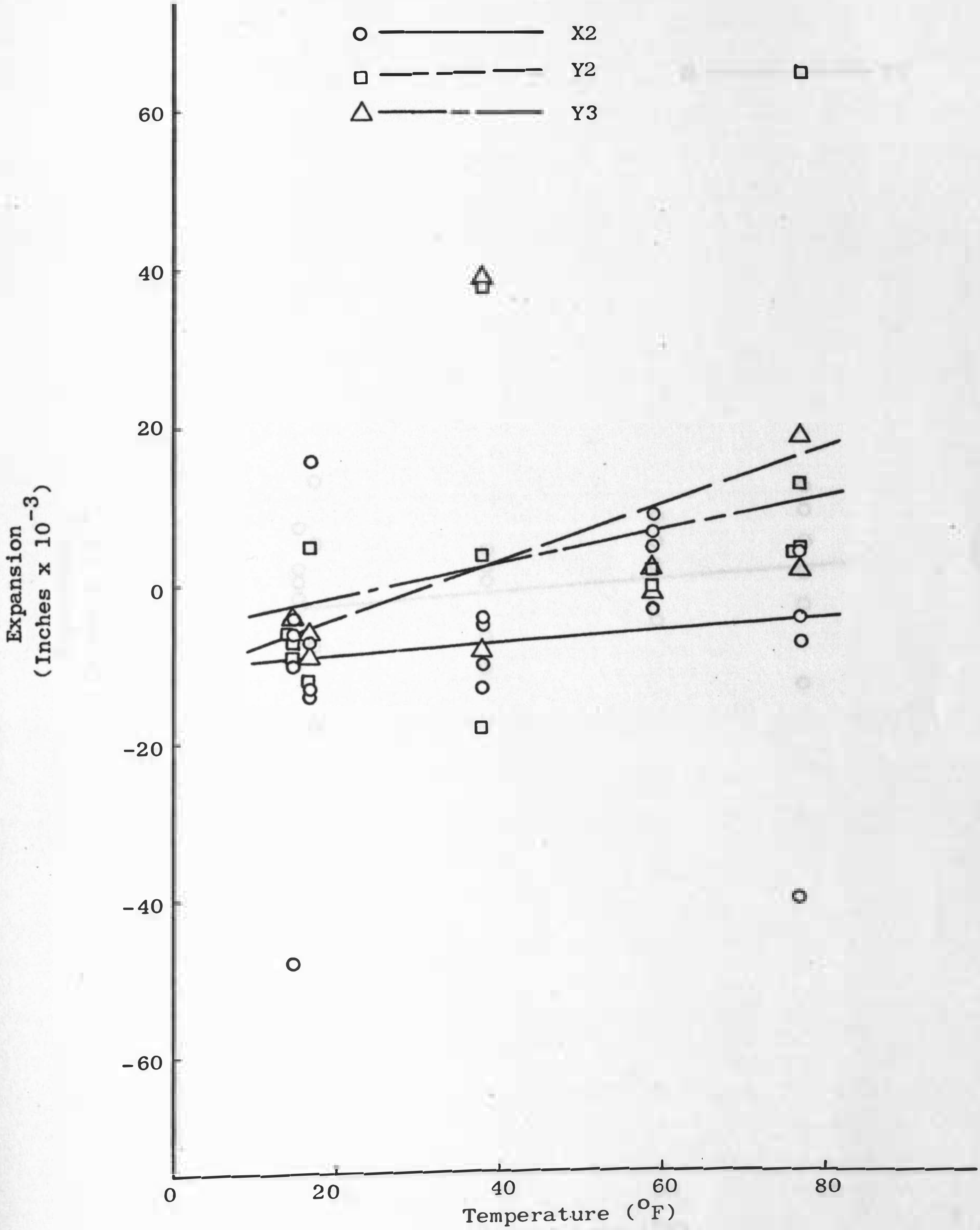


Figure 14. Expansion within the Pavement Panels and the Eight-Inch Regular Concrete Pavement

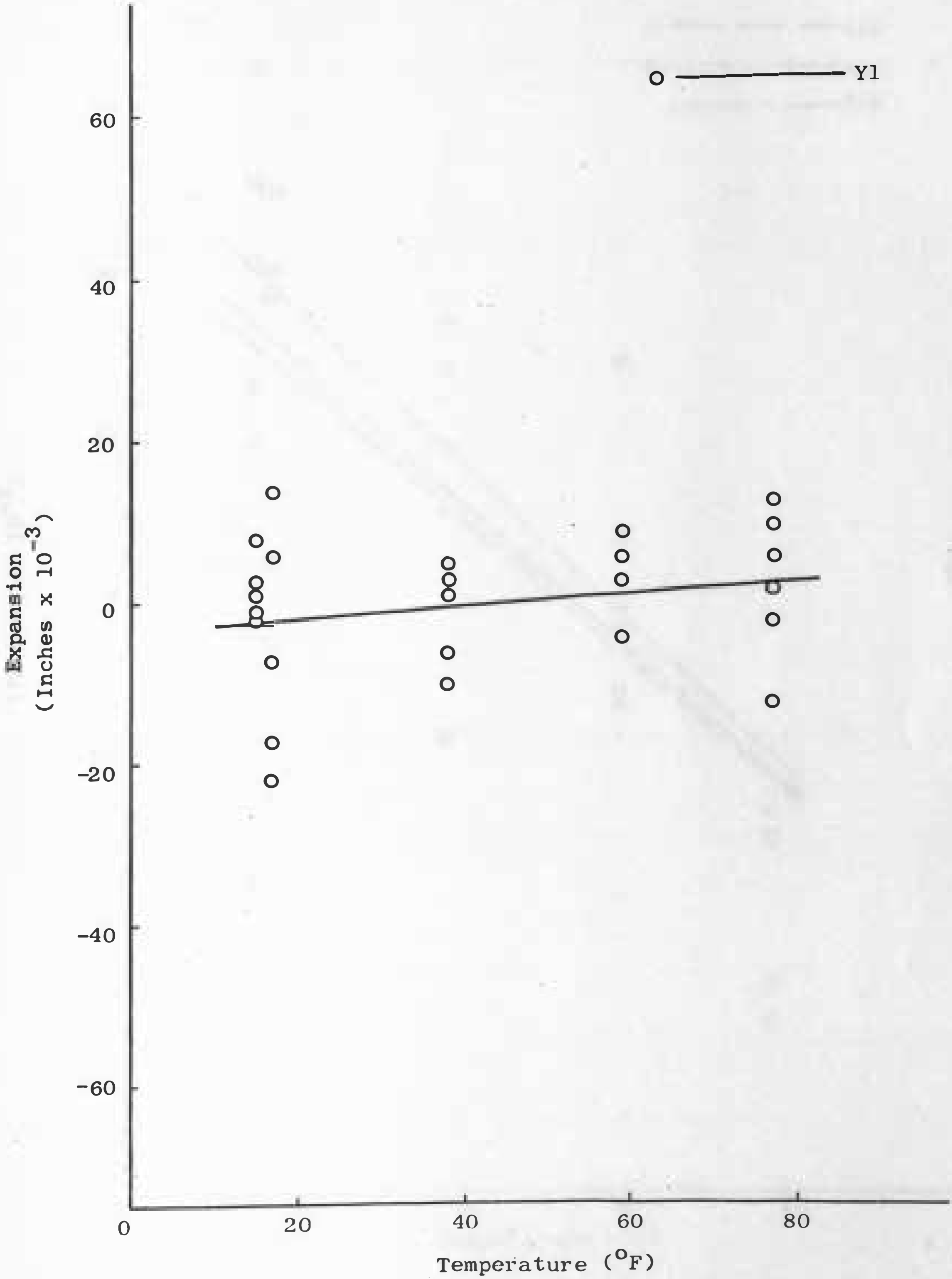


Figure 15. Transverse Expansion in the Longitudinal Panel Section

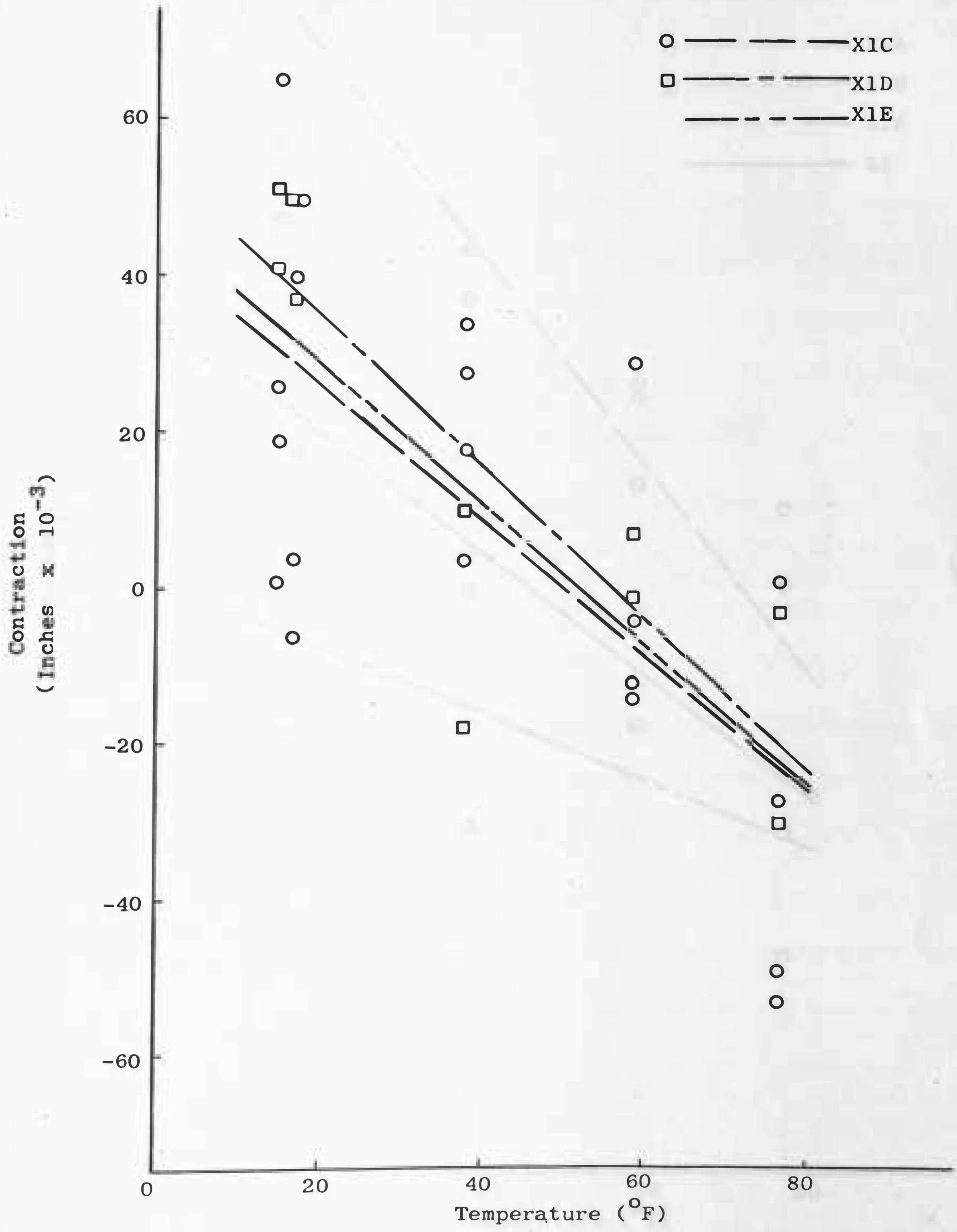


Figure 16. Longitudinal Contraction across Transverse Joints in the Longitudinal Panel Section

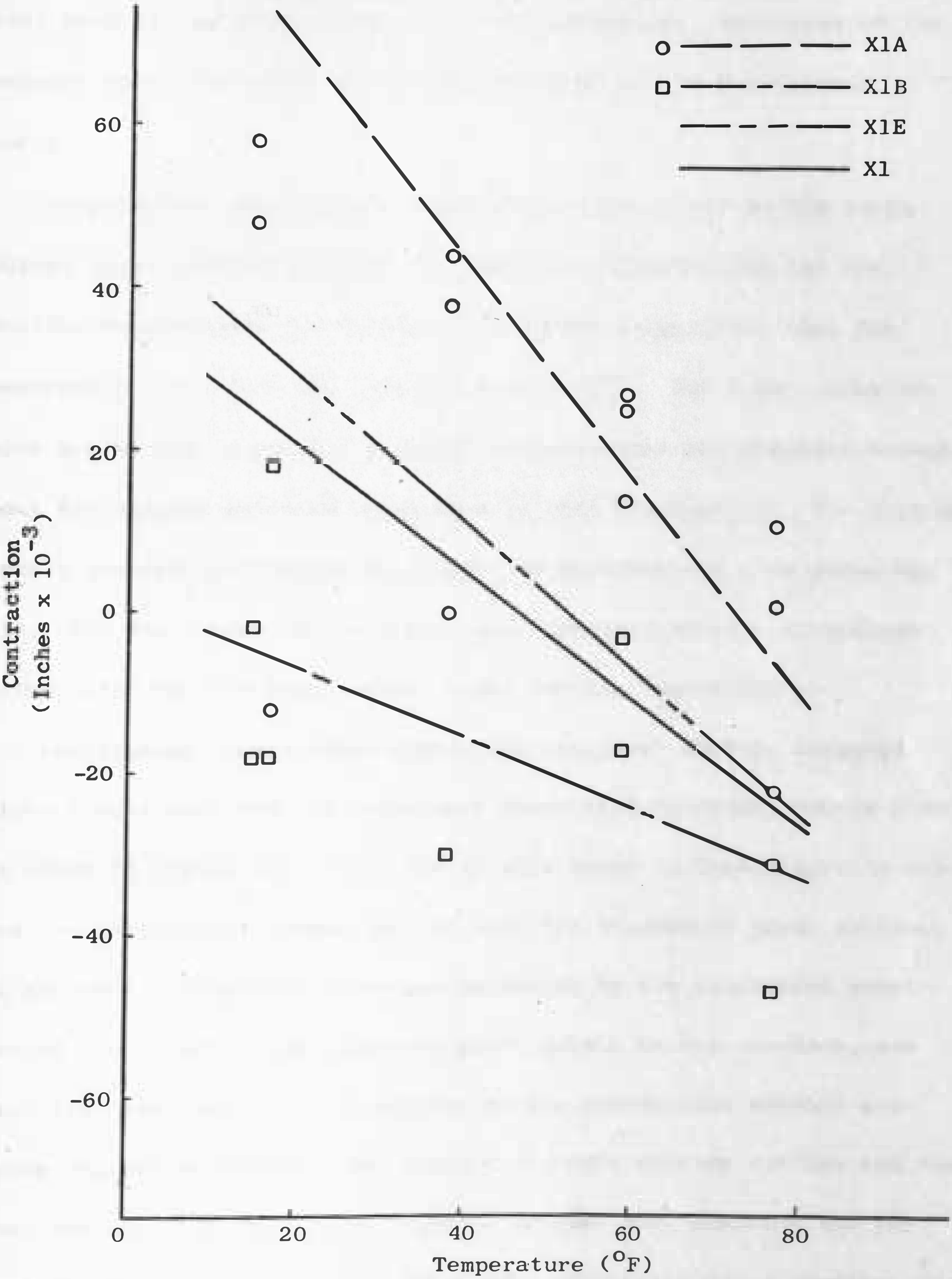


Figure 17. Longitudinal Contraction across Transverse Joints in the Test Pavement

joint possibly absorbed a portion of the expansion. This area of the pavement acted similarly to interior portions of the prestressed panels.

Longitudinal contraction across transverse joints in the longitudinal panel section is shown in Figure 16. Contraction for the shoulder measurements (curve X1D) is slightly larger than that for measurements away from the shoulder (curve X1C). The higher shoulder value may be due to greater pavement distress near the shoulder brought about by changing moisture conditions at that location.(1) The thinner asphalt concrete wearing surface near the shoulder may also encourage this. The two curves (X1C and X1D) were combined to give a representative value for the longitudinal panel section (curve X1E).

Longitudinal contraction curves for the panel section terminal joints (curve X1A) and the transverse panel section joints (curve X1B) are shown in Figure 17. Curve X1E is also shown in this figure to compare the longitudinal panel section with the transverse panel section. The smallest contraction value was displayed by the transverse panel section (curve X1B). The numerous panel joints in this section, one every six feet, may absorb a portion of the contraction without producing reflection cracks. The asphalt concrete wearing surface and the grout key may bond the panels together so that they overcome the resistance of the subgrade since the panels apparently acted together as panel groups. The crack spacings may have been affected by varying subgrade resistances resulting from the various moisture conditions in the subbase.

The panel section terminal joints (curve X1A) exhibited the largest contraction of the entire pavement. Changing from one type of panel arrangement to another may encourage this condition. The transverse panel arrangement and the longitudinal panel arrangement appeared to act as independent units.

Curve X1 represents longitudinal contraction across transverse joints throughout the test section being a combination of the transverse panel section (curve X1B) and the longitudinal panel section (curves X1C and X1D). Curve X1A was excluded from this combination because it was felt that the panel section terminal joints were not typical joints for the prestressed pavement.

As a means of studying the opening and closing of transverse joints in the longitudinal panel section, the Williot diagram for determining displacements in structures was used. Only inspection points in the interior portion of the longitudinal panel section, stations 139+68 through 143+88 were analyzed by this method. The longitudinal measurements which did not cross a joint were used as the base lines of the respective Williot diagrams. The two diagonal measurements were used while the measurements parallel to the base line were excluded. See Appendix C. The relative longitudinal movements of the two adjacent panel corners were obtained from the Williot Diagrams and are listed in Table 5, Appendix A. Positive values indicate relative movement away from the joint while negative values indicate relative movement toward the joint. In some Williot diagrams, both panel corners moved in the same direction requiring the determination of the difference for the relative movement.

It is noted that in some cases the joints opened during the fall and winter seasons. The most recent winter results indicated a closing of the joint at station 143+88. A portion of this may be due to erroneous data from the corroded inspection points. Also foreign material may have infiltrated into the joints to restrict free expansion and contraction.

The July, 1969, and February, 1970, results for station 143+88 did not follow the trend of the other results. For example, the joint opened in July while the others closed. The reverse occurred in February. This may be due to the station being near the end of the pavement test section where less stress is transferred from one panel to the adjacent panels.

Conclusions

The following conclusions have been reached from this investigation:

1. Major pavement distress was not observed.

2. Major reflection cracks should be filled with an asphaltic material to prevent the entrance of foreign materials.
3. Smooth undersurfaces and bonding by the concrete grout key and the asphalt concrete wearing surface apparently caused the transverse panels to act together in panel groups during cycles of expansion and contraction.
4. Stainless steel studs should be used for the inspection points to prevent corrosion problems.
5. Large sections of the longitudinal panel arrangement apparently acted as independent units during expansion and contraction. The panel connectors and the bonding and frictional resistance of the grout key may restrict individual panel expansion or contraction. It is possible that transverse cracking in longer test sections may appear in the longitudinal panel arrangement.
6. The riding quality of the pavement improved throughout 1969.
7. Measurements across longitudinal joints showed that this region acted similarly to interior regions of the prestressed panels in terms of expansion and contraction.
8. The transverse panel arrangement showed a lower contraction value than the longitudinal panel arrangement.

9. Terminal joints at the end of the transverse and the longitudinal panel arrangements are not representative of the interior pavement joints.

10. Expansion and contraction cause the transverse joints in the longitudinal panel section to open and close without showing signs of pavement distress.

The following areas of future study are recommended:

1. Another study of expansion and contraction should be conducted at the end of the five-year pavement study. The added time would give more data for the regression analysis.

2. A study of deflections should be undertaken to evaluate the load carrying capacity of the pavement under traffic and weathering conditions. This could be utilized in evaluating the efficiency of grouted keyways for shear transfer.

3. A study of subgrade pumping should be conducted.

4. The roughness index of the transverse panel arrangement should be determined separately from that of the longitudinal panel arrangement. The high speed roughometer should be used for this.

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APPENDIX A

TABLE 1

PAVEMENT SURFACE ROUGHNESS

(ROUGHNESS - INCHES PER MILE)

Date	Lane	Method of Measurement Electronic	Mechanical
Sept. 23, 1968	Eastbound	126	132
	Westbound	127	135
April 29, 1969	Eastbound	135	146
	Westbound	127	129
July 22, 1969	Eastbound	129	134
	Westbound	122	127
Oct. 21, 1969	Eastbound	117.3	117.3
	Westbound	111.4	114.4

TABLE 2

PAVEMENT MOVEMENT AT MONUMENT LINE

(MOVEMENT* - INCHES x 10^{-2})

Station Number	Time of Measurement May, 1969	July, 1969	Nov., 1969	Feb., 1970
134+99.5	- 4	-27	-12	-15
135+00.5	0	11	2	6
138+95.5	-10	10	17	-14
138+96.5	-10	-26	-10	16
143+99.5	2	17	17	32
144+00.5	13	-14	-14	-20

*Positive values indicate longitudinal movement away from the line of sight while negative values indicate movement toward the line of sight.

TABLE 3

EXPANSION AND CONTRACTION MEASUREMENTS

(MEASUREMENTS* - INCHES x 10^{-3})

Station Number	Measurement**	Feb. 1969	May 1969	July 1969	Nov. 1969	Feb. 1970
Temperature (°F)		15	59	77	38	17
135+00	X1A	58	14	1	0	-12
	Y2	-9	0	65	38	-81
	Y3	-4	2	19	39	-6
136+32	X1B	-18	-17	-31	-30	-18
	Y2	-6	0	4	-18	-12
137+64	X1B	-2	-3	-47	145	18
	Y2	-6	2	5	4	5
138+96	X1A	108	25	-22	38	173
	Y1	-1	-4	-2	5	-22
	Y2	-7	0	13	-117	-89
139+68	X1C	26	-12	-53	34	40
	X1D	41	-1	-30	10	50
	X2	-6	7	-7	-10	16
	Y1	-2	6	13	3	-7
141+84	X1C	19	-4	-27	28	-6
	X1D	51	7	-3	-18	37
	X2	-10	-3	-4	-13	-13
	Y1	3	9	6	1	14
142+68	X1C	65	-14	-49	18	50
	X2	-4	5	4	-4	-7
	Y1	3	-4	-12	-10	-17

Table 3. Expansion and Contraction Measurements (Continued)

Station Number	Measurement**	Feb. 1969	May 1969	July 1969	Nov. 1969	Feb. 1970
143+88	X1C	1	29	1	4	4
	X2	-48	9	-40	-5	-14
	Y1	8	9	2	-6	6
144+00	X1A	48	27	11	44	104
	Y1	1	3	10	-6	-7
	Y3	-4	-1	2	-8	-9

*Positive values indicate expansion between the inspection points while negative values indicate contraction between the inspection points.

**Refer to Appendix B for notation.

TABLE 4

LOCATION AND SPACING OF MAJOR TRANSVERSE CRACKS

IN TRANSVERSE PANEL SECTION

(SPACING - FEET)

Station	Crack Spacing
135+00	
	6
135+06	
	48
135+54	
	48
136+02	
	30
136+32	
	48
136+80	
	48
137+28	
	48
137+76	
	60
138+36	
	36
138+72	
	24
138+96	

TABLE 5

RESULTS OF WILLIOT DIAGRAMS

FOR THE ANALYSIS OF JOINT MOVEMENTS

(MOVEMENT - INCHES x 10^{-3})

Station Number	Moving Point*	Feb. 1969	May 1969	July 1969	Nov. 1969	Feb. 1970
139+69	b	-7	-19	-4	-4	-8
	c	45	-15	8	59	51
141+84	a	23	-13	-23	15	17
	d	13	6	-1	19	11
142+84	a	42	-11	-3	26	-36
	d	44	5	-3	60	41
143+88	a	41	-10	84	13	-21
	d	119	-10	31	23	-10

*Refer to Appendix C for notation

APPENDIX B

NOTATION FOR EXPANSION AND CONTRACTION MEASUREMENTS

Symbol	Definition
X1	A measurement made parallel to the road's center-line across a transverse joint.
X1A	An X1 measurement at stations 135+00, 138+96, and 144+00 (panel section terminal joints).
X1B	An X1 measurement at stations 136+32 and 137+64 of the transverse panel section.
X1C	An X1 measurement at stations 139+68, 141+84, 142+68, and 143+88 of the longitudinal panel section.
X1D	An X1 measurement at the shoulder of stations 139+68 and 141+84 in the longitudinal panel section.
X1E	A combined curve of curves X1C and X1D.
X2	A measurement made parallel to the road's center-line within a prestressed concrete panel.
Y1	A measurement made transverse to the road's center-line across a longitudinal joint.
Y2	A measurement made transverse to the road's center-line within a prestressed concrete panel.
Y3	A measurement made transverse to the road's center-line within the regular eight-inch concrete pavement.

APPENDIX C

USE OF WILLIOT DIAGRAM FOR DETERMINING JOINT MOVEMENTS

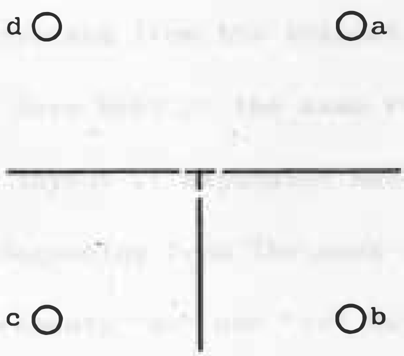
Background

The Williot diagram, which was developed for analyzing structural displacements, was applied to a precast prestressed concrete pavement to study the movements of panel joints in the longitudinal panel arrangement. In place of the strains in a trussed structure, the expansion or contraction movements between inspection points of the prestressed pavement were used.(16) Due to the graphical nature of these diagrams, the procedure is described in an example which follows.

Example Williot Diagram

The July 1969 expansion and contraction movements at station 139+68 were used for the example Williot diagram as shown in Figure 18. The figure also shows the layout of the inspection points relative to the panel joints. The expansion or contraction movement is also given for each strain gage measurement.

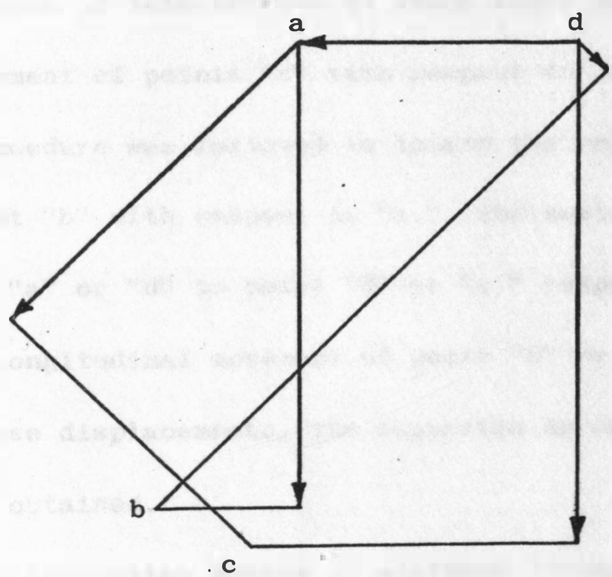
Because measurement "ad" did not cross a panel joint, it was selected as the base line of the Williot diagram. This lines gives the orientation of the diagram. Beginning at point "d," a line was drawn toward the left to represent the movement of point "a" with respect to point "d." The length of the line represents the amount of movement between points "a" and "d." Since there was a contraction



Inspection Point Layout

Inspection Point Movements

Measure- ment	Movement (Inches x 10^{-3})
ab	12
cd	13
da	-7
ac	10
bd	1



Williot Diagram

Figure 18. Use of Williot Diagrams for Determining Joint Movements

in this measurement, the relative location of points "a" and "d" are reversed from the inspection point layout. Points "a" and "d" would have been in the same relative locations as in the inspection point layout if expansion had occurred.

Beginning from the ends of the base line, the movements of measurements "ac" and "dc" were drawn in the same manner as line "ad." These lines were drawn downward because there was expansion away from the base line "ad." Next, lines were drawn at right angles to the free ends of the lines representing measurement changes in "ac" and "dc." The point of intersection of these lines indicated the relative displacement of points "c" with respect to "d."

The same procedure was followed to locate the relative displacement of point "b" with respect to "a." The horizontal distance from point "a" or "d" to point "b" or "c," respectively, represented the longitudinal movement of point "b" or "c," respectively. From these displacements, the expansion or contraction of panel joints was obtained.

Movements of inspection points at stations 141+84, 142+68, and 143+88 were obtained in the same manner except that side "bc" was used as the base line. Movements of points "a" and "d" were desired in these cases.

Interpretation of Table 5

The following descriptions refer to Table 5:

Moving Point a -- indicates movement of point "a" with respect to point "b."

Moving Point b -- indicates movement of point "b" with respect to point "a."

Moving Point c -- indicates movement of point "c" with respect to point "d."

Moving Point d -- indicates movement of point "d" with respect to point "c."

Positive values -- indicates opening of the joint.

Negative values -- indicates closing of the joint.