THE EARLY SERVICE LIFE CHARACTERISTICS OF A CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN COMAL COUNTY, TEXAS

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SYNOPSIS

The performance record of continuously reinforced concrete pavement has been the subject of discussion for numerous research papers during the last few years. This paper constitutes a report on the early service life characteristics of a continuously reinforced concrete pavement in Comal County, Texas.

Gage plugs were placed longitudinally along the pavement surface at spacings of 20 inches in two short test areas covering distances of 28 feet and 38 feet, respectively. The movement between adjacent gage plugs was measured by the use of a 20-inch Berry Strain Gage at various periods during the early life of the pavement. In conjunction with these studies of movement, measurements of air temperature, transverse crack width, and concrete properties were performed. A test section approximately 1500 feet long was observed for the development of the crack pattern for a period of one year.

The experimental data were used to analyze the interaction among air temperature, pavement age, and concrete properties, as these variables influence the development of the crack pattern in a continuously reinforced concrete pavement. In addition, an expression for predicting the width of a transverse crack in terms of pavement age and air temperature was developed.

DEFINITIONS

1. Slab - The portion of a continuously reinforced concrete pavement placed during one consecutive period of paving operations, such as a day. The length of the slab would be measured between consecutive transverse construction joints.
2. Slab Segment - The portion of a slab between any two consecutive transverse cracks in the pavement.
3. Test Section - A slab or part of a slab used to determine crack pattern development.
4. Test Area - The area within a slab where strain and/or movement measurements are conducted.
5. Average Crack Spacing - The average length of the slab segments within a designated test section.
6. Cycle - Any 24 hour period of time in the life of a pavement during which measurements of movement, strain, etc, are performed. The cycles are numbered consecutively with the first cycle starting with the initial placement of concrete.
7. Plugs - Gage plugs installed in concrete to provide reference points from which to measure minute movements between any two adjacent plugs.
   a. Opening - distance between any two adjacent plugs increasing.
   b. Closing - distance between any two adjacent plugs decreasing.

I. Introduction

Explanation of CRCP

Continuously reinforced concrete pavement (CRCP) is an unusual type of pavement structure. In this type of pavement the transverse contraction joints, long considered to be essential in the construction of concrete pavements to prevent the volume changes in the concrete from causing excessive damage to the pavement, have been eliminated. In convential jointed concrete pavements, the contraction joint spacings range from 15 feet to 125 feet, depending upon the amount of longitudinal reinforcement used. In continuously reinforced concrete pavement, a series of seemingly uncontrolled, apparently randomized cracks which generally run transversely across the pavement develop. These cracks are hairline in width and are generally spaced at closer intervals than common contraction joint spacings. Since the contraction joints, which are an inherent source of weakness in convential jointed concrete pavements, are deleted in continuous pavement, the problems normally associated with pavement performance at joints is alleviated. (9/1)*. The only joints used in continuous pavement are construction joints which are installed at the end of a day's placement and expansion joints at structures.

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* Numbers in parentheses refer to items in the Bibliography. The number before the slash(/) refers to the numerical order in the Bibliography; whereas, the number after the slash refers to the page number in the enumerated reference.

A designer for this type of pavement must take into account the stresses induced into the concrete and the steel by temperature changes, shrinkage, and externally applied loads (wheel loads). Due to the complexity of the problem in evaluating the effects of these combined factors, several simplified design procedures have been proposed. (11/6) (4/3). These design methods propose that the stresses induced by wheel loads be accounted for by slab thickness, and that the stresses induced by the volumetric changes in the concrete be taken care of by the longitudinal steel. In essence, the steel serves the purpose of holding the transverse cracks in the pavement tightly closed so that slab continuity is maintained. If the crack openings are held to a minimum magnitude, then full load transfer is achieved across the cracks.

This brief description indicates that crack width and crack spacing are of primary importance in the performance of a continuously reinforced concrete pavement. Numerous publications by various investigators have reported on factors which influence the crack pattern (20/61) (10/22) (19/76) (16/1), but only limited amounts of data are available concerning the actual development of the crack pattern. The goals of this study are to examine and to explain the phenomenon of the crack pattern development. A full understanding of this phenomenon, along with the other factors which affect the pavement performance, can serve as a basis for a rational pavement design approach.

History of CRCP

The use of continuously reinforced concrete pavement started in 1938, when Indiana constructed a continuous pavement with a longitudinal steel percentage* of 1.82. (2/49) In various experimental pavements constructed since this time, the longitudinal steel percentage has ranged from a maximum of 1.82 per cent to a minimum of 0.3 per cent (10/22). At the present time, most states use a longitudinal steel percentage of 0.5 to 0.6. Generally, the lower the steel percentage, the cheaper the unit cost of the pavement. Therefore, the engineer is faced with the problem of selecting a minimum steel percentage in order to insure that continuous pavement is a competitive alternate design. Although very little is known about the design of continuous pavement, this type of pavement has commenced to experience widespread usage throughout the country. Recent reports (October 1961) by the Concrete Reinforcing Steel Institutes Committee on Continuously Reinforced Concrete Pavement show that 489 miles of equivalent two lane continuously reinforced concrete pavement have been constructed or contracted for construction in the United States. (18/1)

This relatively widespread adoption can be attributed largely to good performance records. Practically every

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* Per cent steel refers to the ratio of the cross-sectional area of the steel to the cross-sectional area of the concrete times 100.

continuous pavement constructed thus far has an excellent performance record in comparison with conventional pavement. This, of course, makes it desirable for use on high-volume highways, since obstruction of traffic for joint repairs is costly as well as dangerous to the workmen. A study of annual costs of a mile of 24 foot pavement made by the Concrete Reinforcing Steel Institutes Committee on Continuous Pavement and based on a 35 year service life shows that continuously reinforced concrete pavements cost $1,445 per mile per year (first cost and maintenance cost) and conventional jointed concrete pavements cost $1,730 per mile per year. (1/31). This particular economic comparison implies that continuously reinforced concrete pavement can be competitive, as well as being equal to or superior to convential jointed pavement in performance. A better understanding of the basic design factors influencing the performance of continuous pavement could possibly reduce the first cost and the maintenance cost, and could result in a superior product at a cheaper cost.

II. General Information Pertaining to The Project Under Investigation

Background

The first projects in Texas using continuously reinforced concrete pavement were located in Fort Worth, Texas, and these projects have a longitudinal steel percentage of 0.7 per cent. The excellent performance received from the Fort Worth pavements over an eight year period, along with design investigations, indicated the possibility of reducing the longitudinal steel percentage to 0.5 per cent. In the early part of 1959, a continuous pavement with 0.5 per cent steel was constructed approximately 15 miles south of Waco, Texas. The pavement used for the investigation in this report is located in Comal County, Texas, and is the second continuously reinforced concrete pavement in Texas to be constructed with 0.5 per cent longitudinal steel. The Comal County project consists of 2.4 miles of freeway-type construction running north from a point about 0.2 of a mile south of the New Braunfels city limits to the south bank of the Guadalupe River. Figure 1 shows the location and layout of the project, while Figure 2 portrays the typical cross-section of the pavement structure on this project. The Comal County project was actually "let" for construction under two separate contracts, but the same design was used in both cases, and one contractor performed all paving operations. Therefore, for the purpose of this report, the two contracts will be considered as one

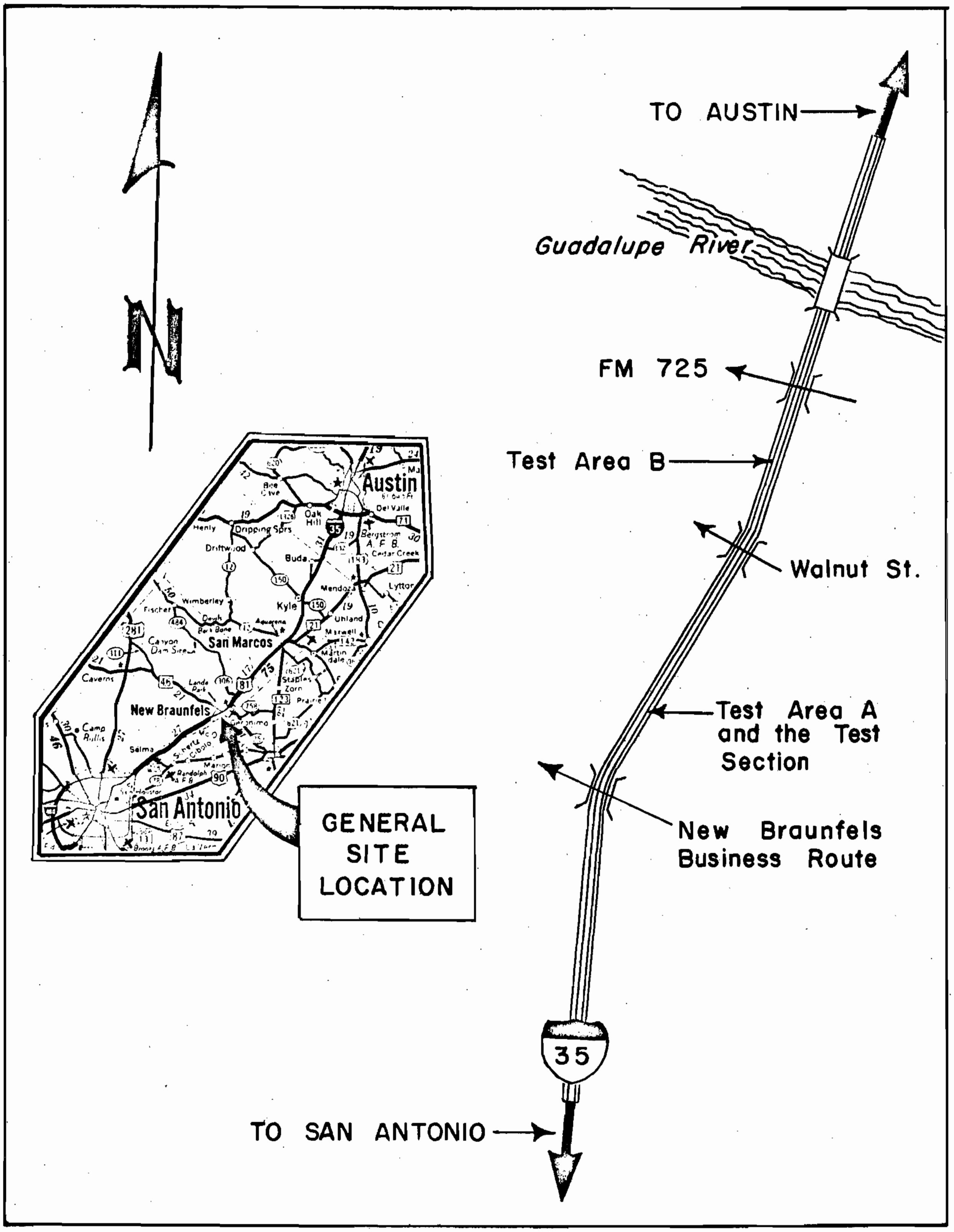


FIG. 1 — LOCATION AND LAYOUT OF COMAL COUNTY PROJECT

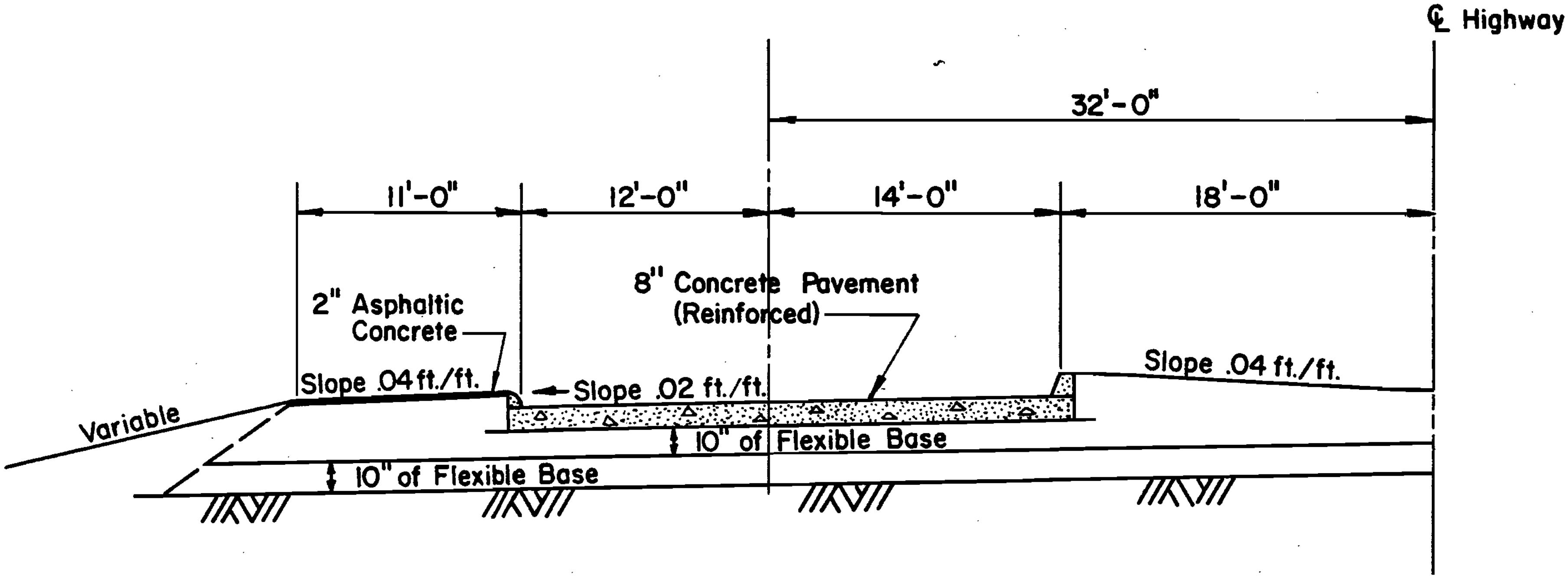


FIGURE 2

TYPICAL CROSS SECTION

project. Mr. R.F. Stotzer, Jr., Resident Engineer for the Texas Highway Department, was in charge of all construction operations.

Continuously reinforced concrete pavement was used on all ramps, speed-change lanes, and the through lanes; and a flexible pavement was used on the frontage roads. Expansion joints were only used at the structure ends, and no provisions were made for terminal anchorage of the free ends of the pavement on this project. Free ends refer to the end area of a slab or a series of slabs that experience contractive and expansive movement. The expansion joints were of the "finger" type used on many bridges. (14/1).

Design and Construction Procedures

As mentioned previously, the bar size and spacing for the longitudinal steel was based on a design requirement of 0.5 per cent of the cross-sectional area of the concrete, and the transverse steel size and spacing was based on a standard requirement of 0.1 per cent. The plans called for the longitudinal steel bars to be 3/4 inch in diameter spaced at 11 inches center to center, and the transverse bars to be 1/2 inch in diameter spaced at 24 inches center to center. The longitudinal steel was required to be at mid-depth, with the transverse bars being located immediately beneath. Deformed bars of a hard grade steel or an equivalent thereof were specified for all steel reinforcement. (6/1). All laps of the longitudinal steel were staggered in such a manner as to provide a minimum number per running foot of pavement (see Figure 3). This method of lap staggering was not a specification requirement, but was done as verbal agreement between the contractor and the resident engineer. The use of the staggered lap reduces the possibility of bond slippage along a transverse plane in the pavement. (7/1).

The concrete pavement was placed monolithically with the steel bars being tied and placed on the subbase immediately in front of the concrete placement operations. The steel mats were kept at the desired elevation by the use of metal chairs. Because of the possible effects on the results obtained in this investigation, it is important to note here that the type of steel placement used on this project differs considerably from the procedures used in many other states. (20/63) (3/36). Many states require the concrete pavement to be placed in two layers, with preassembled mats of reinforcing steel installed on top of the first layers of concrete. Figure 2 shows that a uniform slab eight inches thick over 20 inches of subbase was used throughout the project. The concrete was placed in 27 foot widths, with the curb being placed monolithically with the pavement. All concrete was placed in accordance with Item 320 of the Standard Specifications of Texas. (17/232). For a resumé of the mix proportions, refer to Table 1 in Appendix A.

FIGURE 3

METHOD OF LAP STAGGERING USED ON THE COMAL COUNTY PROJECT

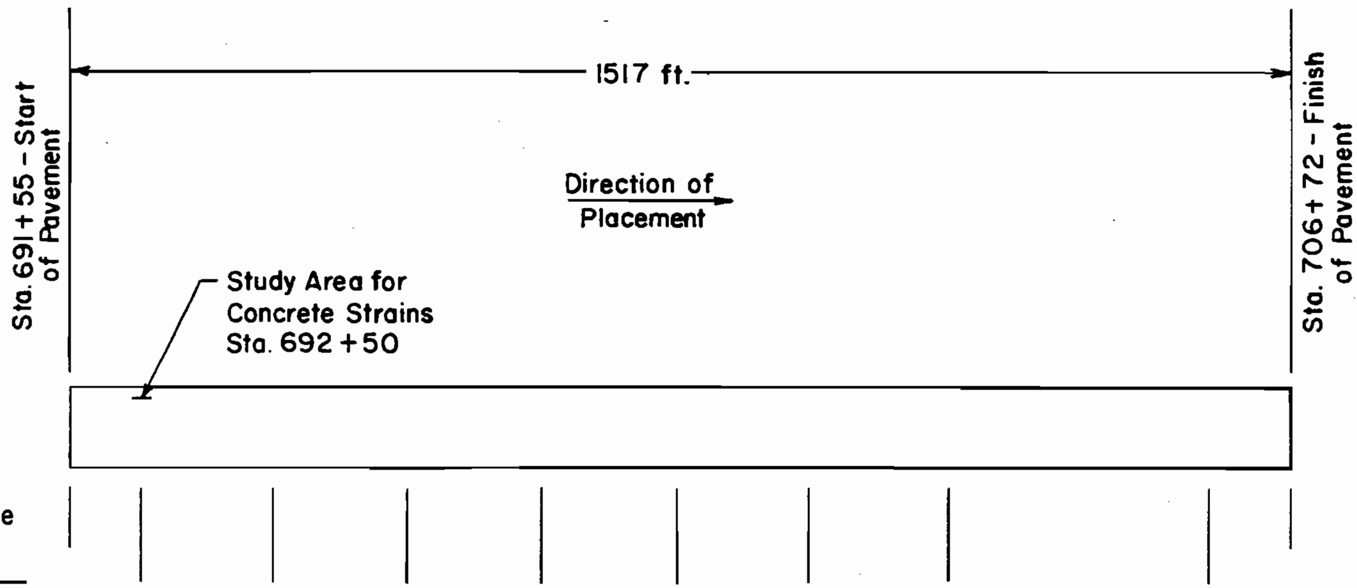
III. Scope of Investigation

The investigation described in this thesis was undertaken to study the configurations of crack patterns which developed in a continuously reinforced concrete pavement under field conditions; and to relate the development of cracks to the properties of the concrete and to existing environmental conditions. Provisions were also made to study the terminal or free end movements experienced at all expansion joints and at a construction joint.

Crack Development

The crack development phase of the investigation consisted of determining when and where the concrete cracked during the life of the project. The data for this study were obtained by taking crack surveys of a test section at periodic intervals after the initial placement. These surveys consisted of measuring the actual distance between cracks throughout the length of the test section. A section representing one day's placement (approximately 1500 feet) was used for observing the crack development (see Figures 1 and 4).

In order to determine the strain at which the concrete cracked and the distribution of movement within a slab segment, gage plugs were installed in the pavement. The gage plugs were 5/8 inch by 2 inch reinforcing bars, with a reference hole in the center, spaced at 20 inches center to center. A detailed description of the procedure used in installing the gage plugs is contained in Appendix A. The plugs



Physical Tests on Concrete at age of seven days								
1. Flexural Strength, psi.	673	741	651	699	694	762	822	715
2. Tensile Strength, psi.	—	—	100	106	100	75	88	94
3. Bond Strength, psi.	—	1172	753	901	1098	880	954	960

Note:

1. The slab was placed on 6 May, 1960 from 6:00 A.M. - 6:00 P.M.
2. The air temperature ranged from 70°F to 85°F.
3. The test section is located in the northbound roadway.

FIG. 4 — CONCRETE PROPERTIES AND OTHER PERTINENT DATA FOR THE TEST SECTION AND TEST AREA - A

were placed with the long axis of the plugs perpendicular to the surface, and with the end containing the reference hole flush with the surface. The strains were measured with a 20-inch Berry Strain Gage, which is capable of detecting movements of 0.0002 of an inch (see Figure 5). The surface widths of the crack were measured by using an optical comparator with 0.001 of an inch graduations.

Two separate locations on the project were chosen for installing gage plugs. Test Area A was at Station 694+20 in the northbound lanes, and Test Area B was at Station 769+00 in the southbound lanes. Test Area A had 18 plugs at spacings of 20 inches center to center (28.3 feet overall between the first and last plug), while Test Area B had 24 plugs at spacings of 20 inches center to center (38.3 feet overall between the first and last plug). In both cases, the plugs were installed approximately 30 inches from the inside edge (median edge) of the pavement. A narrow line was painted along the plugs with a thin mixture of lime and water in Test Area B. The line proved to be of considerable aid in locating cracks at the time of occurrence. The data obtained from measurements in the test areas are presented in Tables 1, 2, and 3 in Appendix B. The procedure used in analyzing these data is found in Appendix C.

Concrete Properties

Physical tests were performed on concrete specimens taken in the field to determine the bond strength, the modulus of

FIGURE 5

BERRY STRAIN GAGE USED FOR MEASURING
MOVEMENT BETWEEN THE GAGE PLUGS

elasticity, the tensile strength, and the modulus of rupture of the in-place concrete. The latter test was performed by the construction forces; whereas, the other tests were performed by personnel from the Materials and Tests Division of the Texas Highway Department. Determinations of the flexural strength, tensile strength, and bond strength of the concrete at seven days were made at approximately 175 foot intervals on the test section containing Test Area A. The location and magnitude of the concrete properties in relation to the total slab under consideration are shown on Figure 4. At Test Area B concrete specimens were taken to determine the tensile strength at different ages and the seven day modulus of elasticity. These data are shown in Table 4 of Appendix B. A word of caution is inserted here in relation to the tensile strength values shown. It is felt that due to the testing procedure used, eccentric loads were introduced in the specimen, hence the values may not be representative of the actual strengths obtained in the slab.

Terminal Movements

Field tests were set up to determine the magnitude of movement occurring at construction joints and expansion joints in the pavement. Gage plugs identical to those previously described were installed on each side of a construction joint. The Berry Strain Gage was used to measure the movements experienced.

Punch marks were made at a spacing of 10 inches center to center on each side of the "finger" expansion joints for the purpose of measuring terminal movements. The distance between punch marks was measured periodically with a scale having 0.02 of an inch graduations. Table 1 shows the distance between expansion joints ranges from about 400 feet to over 5,000 feet.

TABLE I

DISTANCE BETWEEN EXPANSION JOINTS

Description	Lane	Feet
1. South Terminal to Business Route Interchange	NBL	3,448.8
2. South Terminal to Business Route Interchange	SBL	3,501.2
3. Bus. Route Interchange to Walnut St. O.P.	NBL	5,232.4
4. Bus. Route Interchange to Walnut St. O.P.	SBL	5,180.0
5. Walnut St. Overpass to FM 725 Overpass	NBL	5,368.9
6. Walnut St. Overpass to FM 725 Overpass	SBL	5,368.9
7. FM 725 Overpass to Guadalupe River Bridge	NBL	411.5
8. FM 725 Overpass to Guadalupe River Bridge	SBL	411.5

NBL - Northbound Lanes
SBL - Southbound Lanes

IV. Results

In the first year after construction of the pavement, numerous observations were made in accordance with the previously defined scope of investigation. First, the data for crack development in the test section will be presented. Next, the data for the measured movement in the test areas will be proffered; after which, the data on terminal movement will be presented.

Crack Development

As pointed out previously, a test section approximately 1,500 feet long, representing one day's placement, was used to observe the development of the crack pattern. (see Figure 4). The studies made in relation to crack development consist of determining the age-crack spacing relation, and determining the area of occurrence of a new crack within a slab segment.

Age-Crack Spacing Relation - Crack surveys have been made periodically on the test section since its initial placement. The surveys were made at frequent intervals during the early life, and less frequently as the age increased. Figure 6 shows the age-crack spacing relation for this project during the first year. Each point on the plot represents the average crack spacing of the test section for the pavement's age at the time of the survey. The data fall into four distinct stages, each stage represented by a straight line when plotted on log-log paper. The derived equations for each of these lines are shown on the graph. The equations show that new

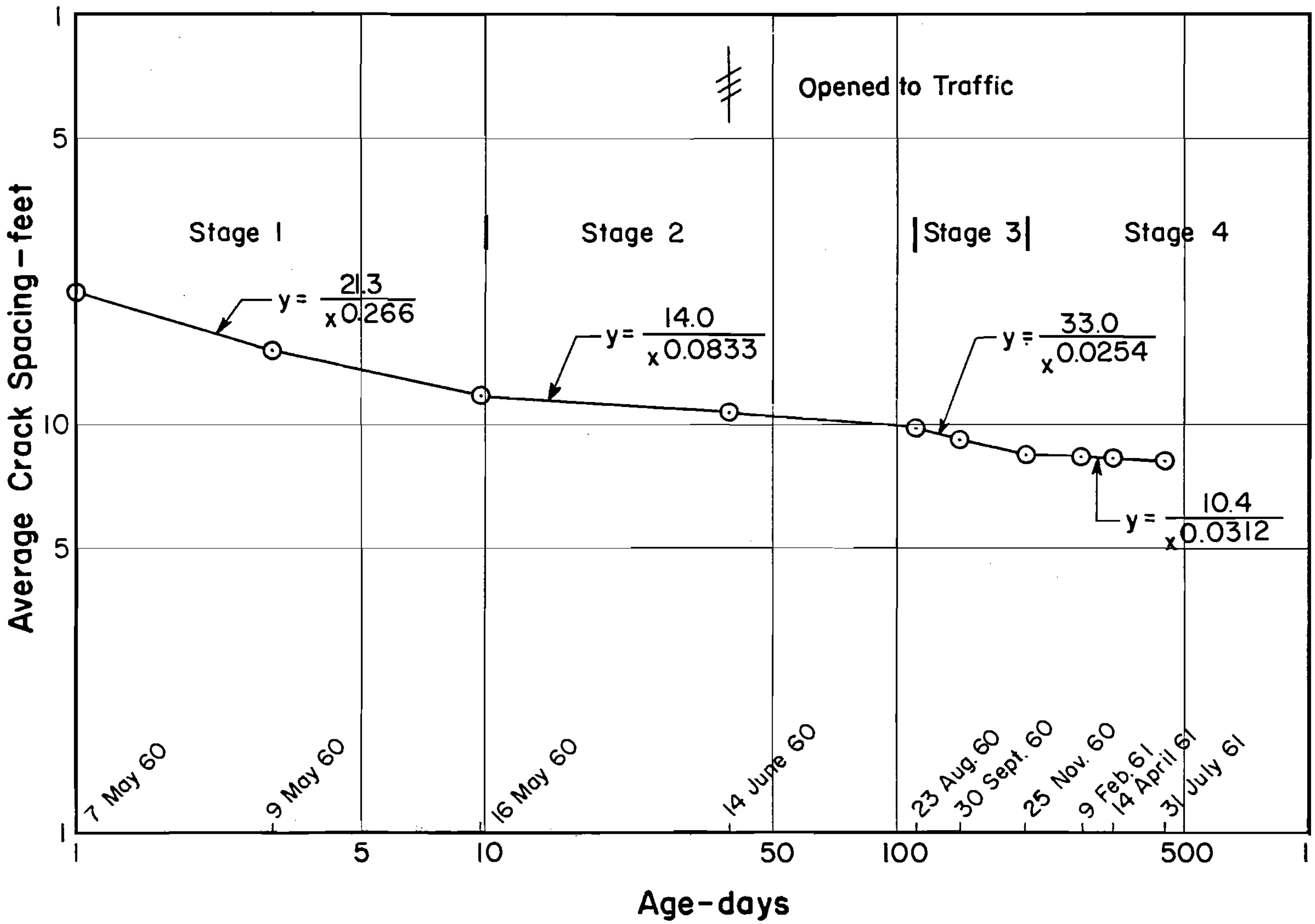


FIG. 6–EFFECT OF AGE ON THE AVERAGE CRACK SPACING IN TEST SECTION

cracks appear in the pavement at a specific rate. The rate of crack development is influenced by various factors such as curing and weather. Note that in Stage 1 - the first 10 days - cracks appear in the pavement more rapidly than during any other period. This ten-day period represents the initial curing period when the concrete properties, such as shrinkage, thermal coefficient, and strength, are experiencing the greatest change. (13/661) (15/417). During this period the basic crack pattern for the section is formed; hence, the first ten days are probably the most important time from the standpoint of crack development. The change in magnitude of the average crack spacing experienced after the age of ten days is small compared to that experienced during the first ten days.

Stage 2 basically represents the relation for the summer months. Very little change in the average crack spacing is experienced during the summer months, even though the pavement is in an early period of life. It might be pointed out that the test section was opened to traffic during this period, with no resulting alteration in the rate of crack development. It is evident here that summer traffic does not alter the average crack spacing.

During Stage 3, the pavement experiences another period of rapid change. This stage occurs in the fall period when the climatic conditions are such that large temperature differentials are experienced during short cycles, as a result of hot days and cool nights or the arrival of a "norther".

Note that this stage of rapid change terminates around the first of December; hence, it does not extend through the severe winter months when the lowest temperatures are experienced. Evidently, a balanced condition between concrete stresses and crack spacing is again obtained, and the rate of change again decreases. Stage 4 is again a period experiencing a low rate of change. Further observation will indicate whether the pavement will again experience a rapid change in crack development, or whether a static condition has been obtained.

An overall examination of the age-crack spacing relation shows that a change is experienced during two periods. One is during the early period (1-10 days) when shrinkage stresses are high, and the other is during the period when temperature changes of large magnitude within a cycle are experienced. Another interesting observation is that traffic (wheel loads) has very little effect on the crack pattern.

Several other pertinent observations in relation to crack spacing and age were made on this project. One is that only the first 500 feet of the test section experienced cracking during the first night. The rest of the slab cracked during the second night. This could probably be attributed to the fact that either the concrete in the last 1000 feet of the slab had not acquired its initial "set", or that the right combination of thermal conditions and concrete properties (which change with time) required for cracking had not occurred during the first night. This area not experiencing cracking during the first night tends to point out the delicate relation between age, concrete properties, and thermal conditions which affect crack development. It is pointed out that this interval of no cracks is not a function of slab length, but rather the time of day the placement was made. Another observation is that the average crack spacing of the first one-third of the slab has always been smaller than the rest of the slab.

Position of Cracks - Figure 7 gives a graphical presentation of the location where new cracks occur in the existing slab segments. The graph is a plot of the cumulative total number of new cracks versus the distance from an existing crack that a new crack occurs in a slab segment. The data for the graph were obtained from the crack surveys described previously. Each new crack found in a slab segment during a survey was designated by the distance from an existing crack. The distances for all the surveys were then arranged in a numerical sequence, with the smallest distance designated as number one, and the next smallest distance designated as number two, etc. The distances were then plotted in terms of the numerical sequence (or cumulative total number of new cracks). Each new crack is located between two existing cracks, so the distance used was the least of the two possible choices. An examination of the data shows that the cumulative total should terminate at a distance of approximately eight feet. Since

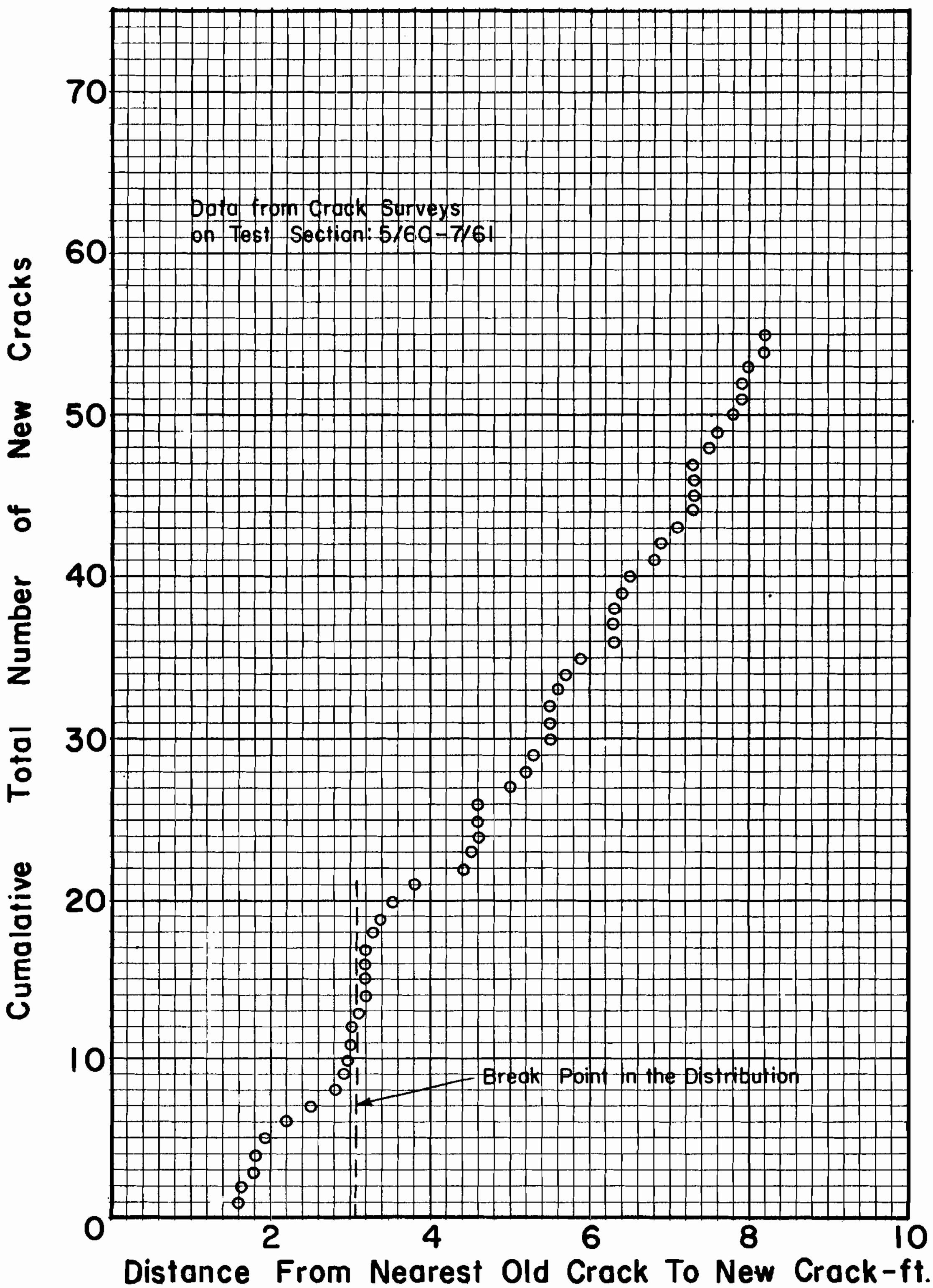


FIG. 7 - AREA OF SLAB SEGMENT
WHERE NEW CRACKS OCCUR

the average crack spacing was 21.3 feet at an age of one day, the chance of a distance in excess of eight feet was relatively small. The maximum distance from the standpoint of the average crack spacing would be 10.2 feet. Therefore, the distribution was terminated at the distance of eight feet.

The data on the graph brings out two pertinent points. One point is that no cracks occurred at a distance less than 1.5 feet from an existing crack. Second, there is a fairly uniform distribution over most of the curve with the exception of the offset at about three feet. A rapid increase in the number of cracks occurring is experienced at this distance. The data tend to indicate that the majority of the new cracks occur at a distance of three feet or more from an existing crack. Therefore, it may be tentatively concluded that distance from an existing crack is a factor influencing the occurrence of a new crack.

Localized Movement

The term localized movement refers to the movement occurring between any two adjacent gage plugs. These movements are cyclic in nature; therefore, the distance between the plugs is constantly changing through the day as the temperature varies. The greatest magnitude of movement between any two adjacent plugs is experienced at transverse cracks and at transverse construction joints. Cyclic movements are experienced in other areas of the slab segments, but the relative magnitude of the movements at the interior positions generally decreases with an increase in distance from the crack. The procedure used for calculating the movement experienced between any two gage plugs is presented in Appendix C.

Movement at Transverse Crack - Figure 8 portrays the relationship between crack width and air temperature. Crack width, as used here, refers to the movement recorded between the two gages across a crack in the pavement. The data show these two factors to be inversely proportional. Consequently, as the air temperature increases, the crack width decreases. The lines shown on the figure represent the relation between these two factors for three specific 24-hour cycles. Within the scope of the data, the slope of the lines for each specific day after cracking is approximately equal, which means that the crack width varies about the same for a given temperature change for any cycle. Note that the air temperature-crack width relations assume a higher position on the plot as the age of the pavement increases. This indicates that the crack is developing a permanent set or width at a given temperature as the age increases. The magnitude of the set is shown on the figure, and can be further demonstrated by extrapolating the data to where the crack is completely closed. A temperature of 95°F would be required for complete closure on the second day, 107.5°F on the fourth day, and 117°F on the seventeenth day. Attention is directed to the fact that the increase in

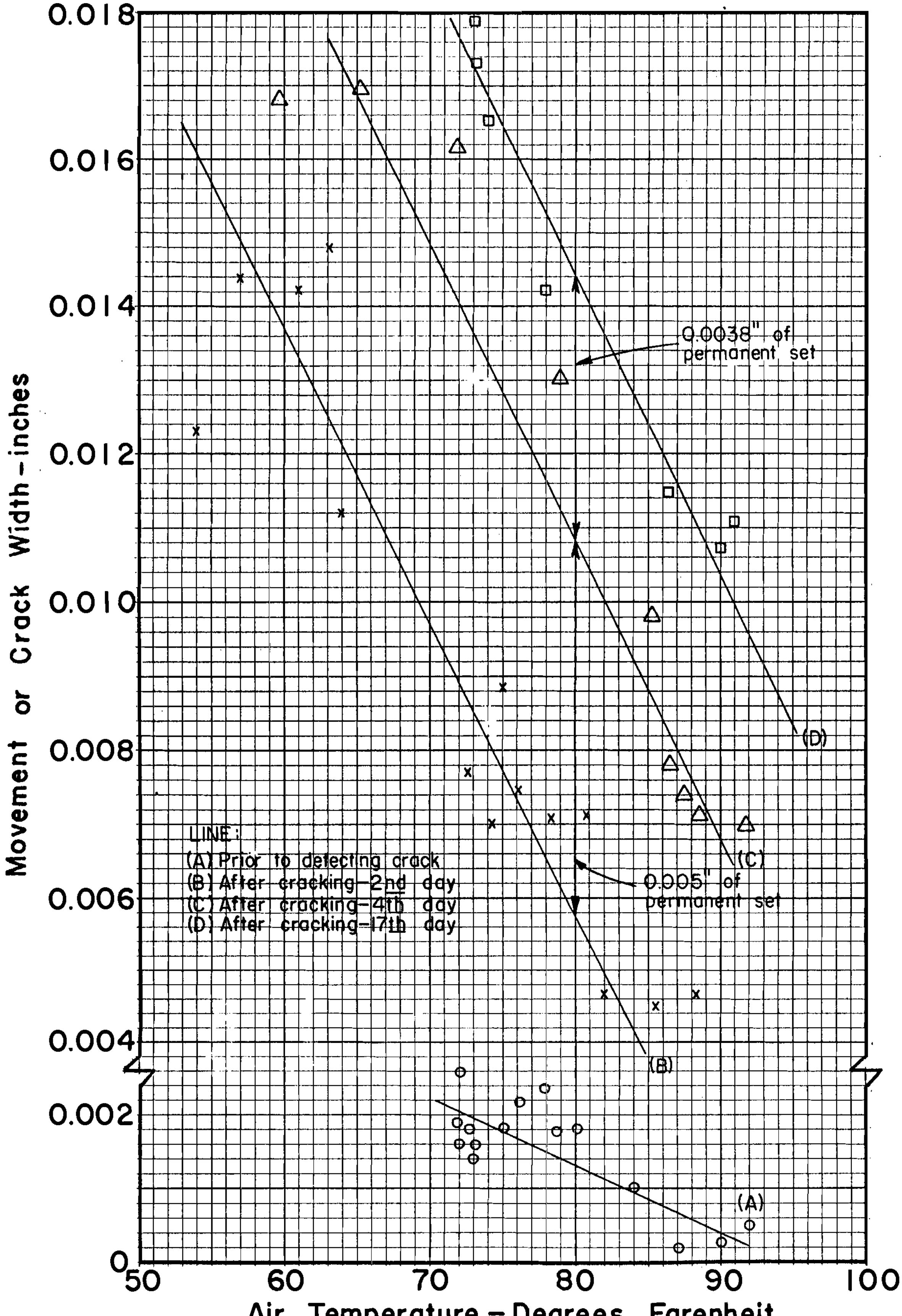


FIG. 8 - CRACK WIDTH vs. AIR TEMPERATURE FOR TEST AREAS A and B

magnitude of the permanent width apparently becomes smaller with each succeeding cycle. In other words, these data seem to manifest that the crack width will reach some upper limit. This fact indicates that the observed phenomenon is dependent on some property of the concrete which is changing with time.

It might be pointed out that the lower line (Line A) shows the relationship between air temperature and movement between two adjacent gage plugs before the concrete cracks. Note the slope of this line is much less than the lines for the temperature-crack width correlations after the concrete cracks.

The data shown on Figure 8 are a combination of the data obtained from the initial cracks in both Test Area A and Test Area B. It is interesting to note that the initial cracks in both of the test areas exhibited identical performance characteristics. The data are portrayed separately for each crack in Figures 1 and 2 in Appendix C. A much better correlation would probably be obtained if slab temperature had been available for use in lieu of air temperature.

<u>Movement at a Transverse Construction Joint</u> - Figure 9 gives the relation between construction joint width and the air temperature. As was the case for the transverse crack, these two factors are inversely proportional. Attention is drawn to the fact that the width of a transverse crack and of the construction joint for a specific temperature cannot be compared, since the data for the construction joint do not

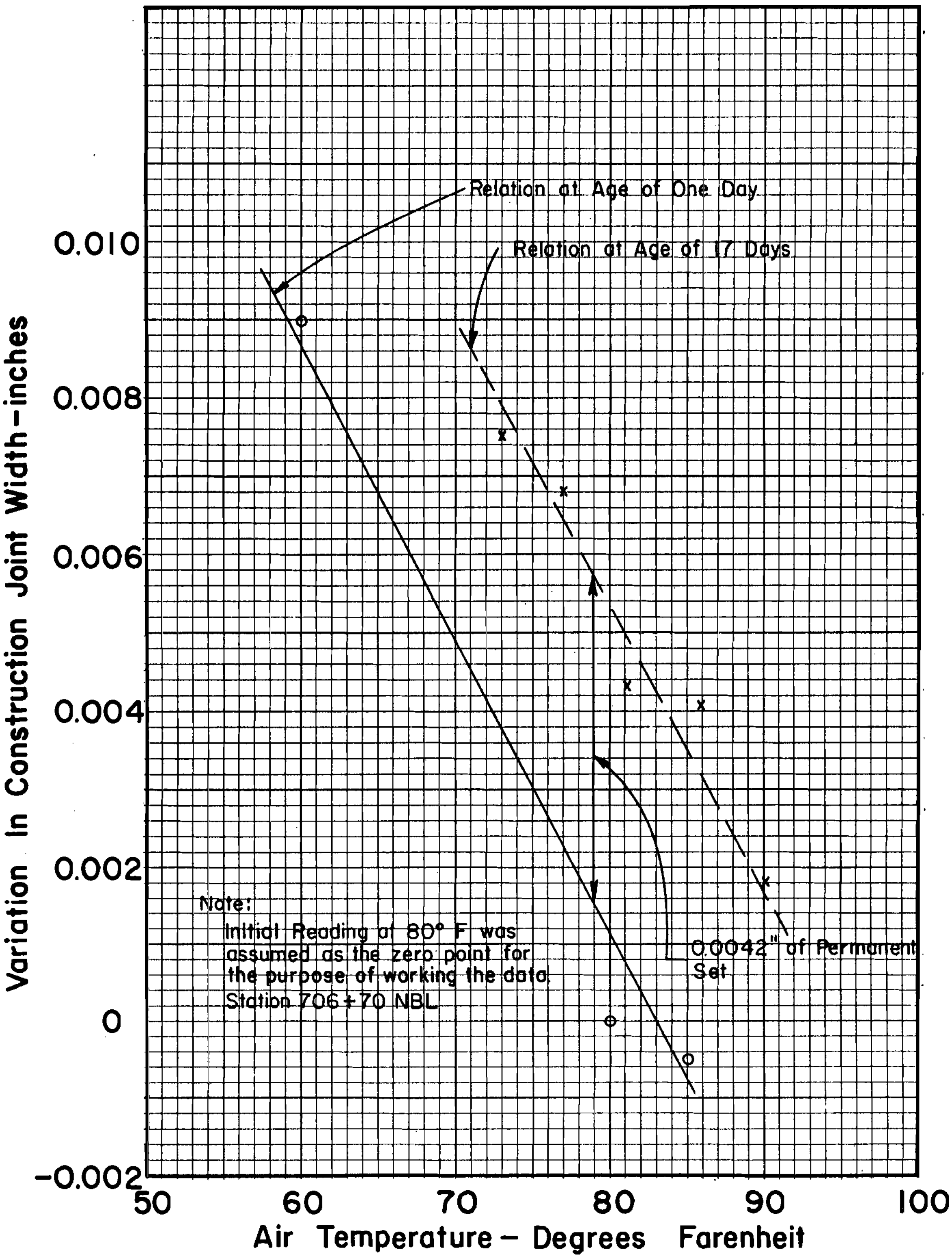


FIG. 9-VARIATION IN CONSTRUCTION JOINT WIDTH FOR CHANGES IN AIR TEMPERATURE

represent total movement. This is due to the fact that the concrete on one side of the construction joint was placed approximately twelve hours later than that on the other side; hence, the movement which occurred during this period was not accounted for by the measuring techniques employed in this investigation.

In general, the characteristics of crack movement observed in connection with the transverse cracks are applicable to the movements at the construction joint. The movement is of a cyclic nature, with a certain amount of permanent width or set developing at the joint. In addition, the slopes of the lines in Figure 9 are almost identical with the slopes of the relations for the transverse crack as shown in Figure 8. This observation indicates, for all practical purposes, that transverse construction joints are performing in a manner similar to the transverse cracks on this project. These observations contrast rather sharply with the excessive movements observed at several transverse construction joints on the first continuous pavement in Texas with 0.5 per cent longitudinal steel. (3/2) (7/1). Several of the construction joints on the first project with 0.5 per cent longitudinal steel experienced openings of approximately 1/4 of an inch, and these excessive openings eventually resulted in a pavement failure at these locations. It might be pointed out that the staggered lap, along with an ample protrusion of steel through the construction joint, was used on the Comal

County project in contrast to the absence of lap staggering on the first project. This difference in construction procedure is the factor resulting in the performance difference.

Distribution of Movement

To obtain a better understanding of what the movements between gage plugs mean, these movements should be studied as a unit. In other words, a study of the longitudinal distribution of movements along the pavement should be made. This can be accomplished by plotting the movement between each set of gage plugs versus the distance from gage plug number one to the mid-point between the set of gage plugs for a specified age of the pavement. In this way the distribution pattern of the movements can be investigated in relation to age, temperature, and slab cracking. The effect of these previously mentioned factors can be determined by studying movement distribution at the minimum temperature, the maximum temperature, and then comparing these movements for different daily cycles.

Distribution of Movement at Minimum Temperature - In a cycle the cracks in the pavement reach the maximum width when the temperature is at a minimum. This relationship may be observed in Figures 8 and 9. Figure 10 portrays the longitudinal distribution of movement between adjacent gage plugs in Test Area A for cycles 1 and 3 at 6:00 A.M. During the season of the year when this study was made, the minimum temperature usually occurred at or near 6:00 A.M. on a

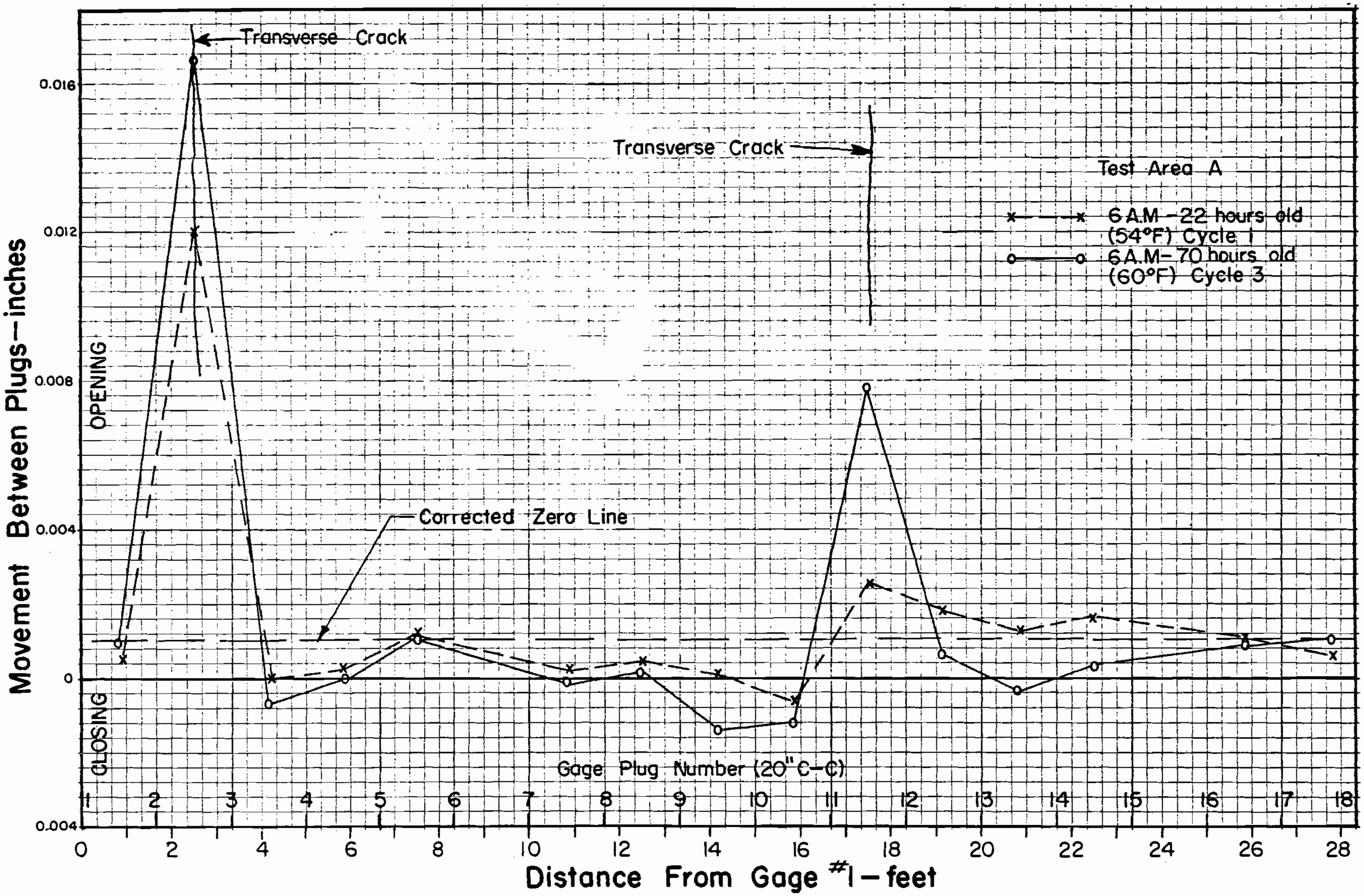


FIG 10- COMPARING MOVEMENT DISTRIBUTION AT MINIMUM TEMPERATURE CYCLES 1 AND 3 typical day.

A correction to compensate for changes in the length of the standard bar used with the Berry Strain Gage was made after studying and re-evaluating the data. The corrected zero line on the graph is felt to be much more representative of the data. For an explanation of the corrective procedure used, see Appendix C.

A number of pertinent observations concerning the distribution of movement at minimum temperature (see Figure 10) are listed below:

(1) Two cracks had occurred in Test Area A between Gages 2 and 3 and Gages 11 and 12 before the completion of the first cycle.

(2) The greatest magnitude of movement occurred at the crack, with progressively smaller movements occurring toward the middle of the slab segment. An examination of the graph seems to indicate that the slab is moving for a distance of approximately 60 to 70 inches away from the crack. The movement of a portion of the slab segment means the concrete within this distance is relieving itself of tensile stresses developed by temperature drop, since absolute fixity would be required for full development of stress. The middle of the slab segment is not moving, hence, concrete stress is greatest in the area of least movement.

(3) The crack between Gages 11 and 12 was not visible on the surface at the time of the first cycle readings, but

the magnitude of movement indicates cracking had occurred. This crack was not visible on the surface until about two hours later.

(4) The openings at the cracks are greater during the third cycle than the first, even though the temperature drop is less. This increase in crack opening is related to the permanent set of the crack, which phenomenon has been discussed previously. Similarly, the closing movements between gage plugs in the interior of the slab are greater for the third cycle than for the first cycle. This balance of movement indicates that the algebraic summation of movement along a slab segment must be equal to zero.

(5) The crack between Gages 11 and 12 is not opening as wide as the one between Gages 2 and 3. This observation is logical, since the crack between Gages 2 and 3 was the first to develop.

Figure 11 shows the longitudinal distribution of movement between adjacent plugs at Test Section A for the third and for the eighteenth cycle at 6:00 A.M. (time of minimum temperature for the cycles). In addition to the previous comments, the following observations apply to the data presented on Figure 11:

(1) A third crack had developed in the test section between Gages 17 and 18 by the eighteenth cycle. Note that this crack occurred where the pavement had experienced very little movement in the previous cycles.

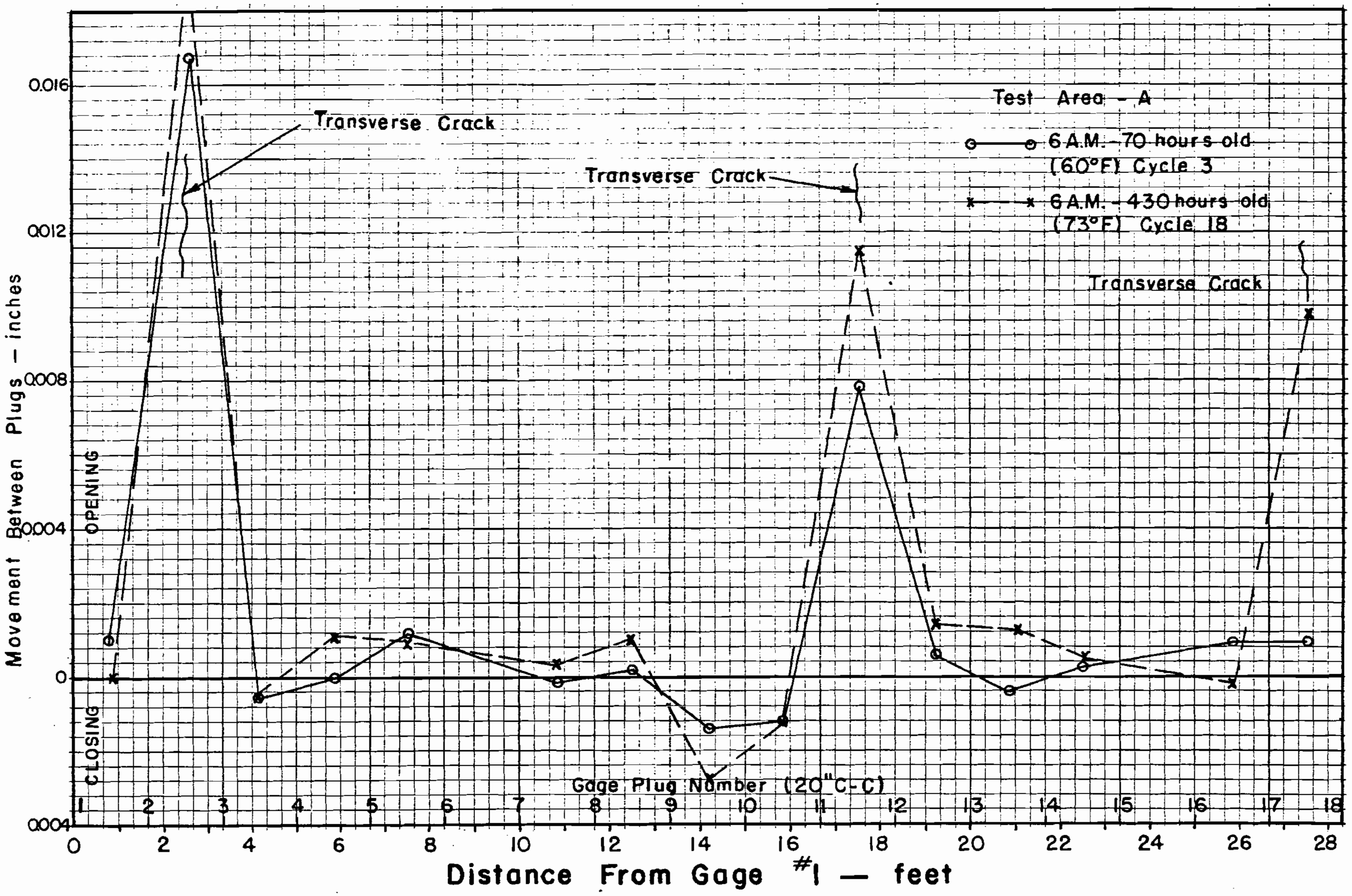


FIG. 11 - COMPARING MOVEMENT DISTRIBUTION AT MINIMUM TEMPERATURE CYCLES 3 & 18

(2) The older the crack, the greater the magnitude of movement. The oldest crack in the test area is between Gages 2 and 3, while the youngest crack is between Gages 17 and 18.

Distribution of Movement at Maximum Temperature - The data discussed previously in the section on localized movement show the cracks to be at minimum width at the maximum temperature. On the Comal County project during the season of year during which the pavement was placed, the maximum temperature generally occurred around 2:00 P.M. Figure 12 shows the distribution of movements between adjacent gage plugs for the same test area and for the same cycles presented in Figure 10, but during the period of maximum temperature.

A number of pertinent observations concerning Figure 12 are listed below:

(1) The width of crack openings has reduced considerably; whereas, the interior portion between cracks has expanded in comparison to the distribution of movement at the minimum temperatures.

(2) The opening at the crack is greater during the first cycle than during the third. This relationship is the reverse of the previous observations at minimum temperatures.

(3) The crack between Gages 11 and 12 had closed completely for all practical purposes at the time of maximum temperature during both cycles.

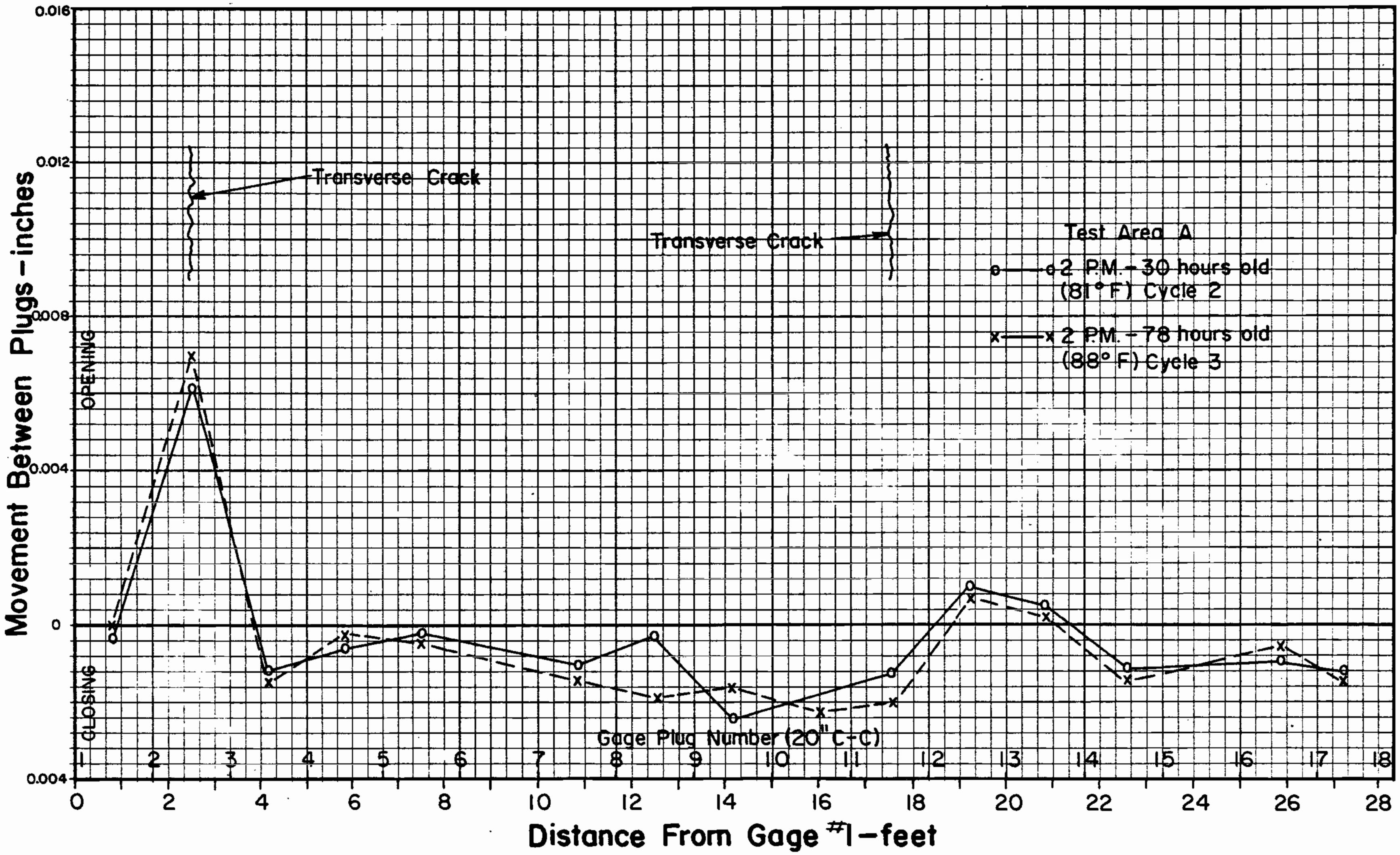


FIG. 12 - COMPARING MOVEMENT DISTRIBUTION OF MAXIMUM TEMPERATURE CYCLES 1 AND 3

Figure 13 is inserted to compare the distribution of movements between adjacent gage plugs which occur at the maximum and at the minimum temperature within a cycle. From the data shown in this figure, an apparent observation is that the opening at the crack and the closure in the interior is greatest at the minimum temperature. This balance of opening and closing movements points out the concept that the summation of movements within any slab segment must be equal to zero.

Terminal Movements

The measured movement at the expansion joints can be used to determine the coefficient of friction between the subbase and the pavement. The first step in such a determination is to use the movement measured at the joint to calculate the length of the slab which moves over the subbase. Once the length of the slab which moves over the subbase is defined, the coefficient of friction can be determined by using the "subgrade drag" formula and the computation techniques shown in detail in Appendix D. (5/2).

The calculations show that the average length of slab moving over the subbase is 107 feet, with an average deviation of ± 17 feet. The average coefficient of subbase friction is 2.3, with a range of 1.9 to 2.7. These values represent the maximum range of values that could be expected on this particular subbase, since a yield point stress was used in the calculations. The actual values are probably a

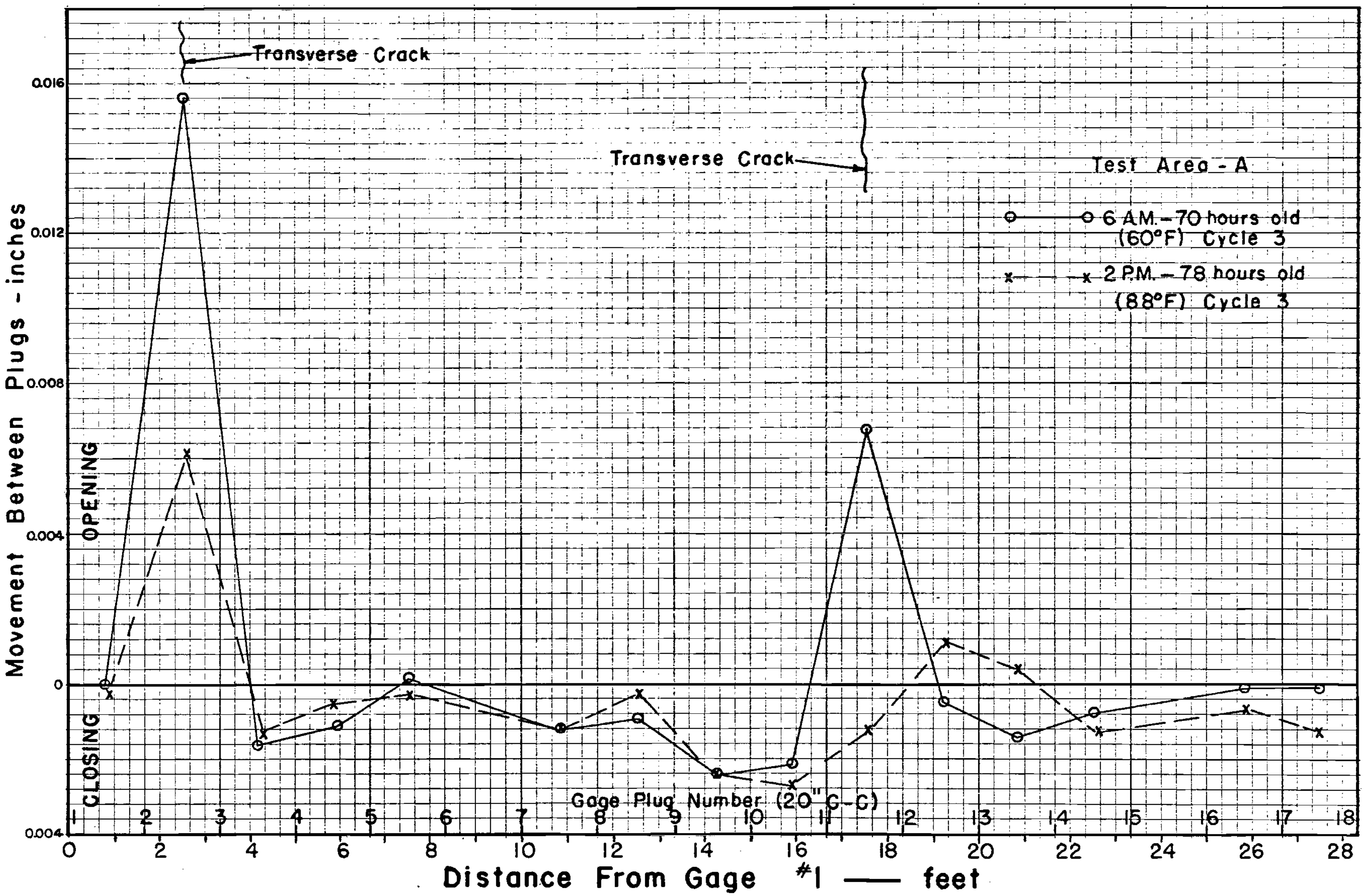


FIG. 13 COMPARING MOVEMENT DISTRIBUTION AT MIN. & MAX. TEMPERATURE
CYCLE 3

little smaller than those presented, since the working stress in the steel is less than the yield stress.

Miscellaneous Observations

In the process of accumulating the data to fulfill the scope of this investigation, several ancillary observations were made. In the test section, the transverse bars were referenced accurately on the forms during construction operations for a distance of approximately 300 feet. For the first ten days, the location of all transverse cracks were studied in relation to the transverse bars, and there was no apparent relation between the location of the transverse cracks and the transverse bars.

Numerous cracks in the test section were examined quite closely along the edge of pavement. In all cases the slab was cracked from top to bottom and on both edges of the pavement. This observation is fairly indicative that the slab is completely cracked throughout its transverse width.

V. Discussion of Results

Formation of Cracks

The relationship between the pavement age and the average crack spacing for the test section is presented in Figure 6. In Figures 10 through 13 the longitudinal distribution of movement between adjacent gage plugs along the pavement is portrayed for various ages. Analysis of the data presented in all of these figures indicates that temperature is a prime factor influencing both the movement in a slab segment and the average crack spacing during any period. The purpose of the following discussion is to analyze the interaction among the movement in a slab segment, the change in the average crack spacing, and the change in air temperature. In explaining this interaction, an approximation of the magnitudes of the longitudinal stress distribution will be presented along with the effect of concrete properties on this stress distribution.

Stress Distribution - Stress distribution in a concrete slab is related to movement. A slab segment completely restrained and subjected to a decrease in temperature would develop a uniform tensile stress in the slab. Any contractive movement occurring in the slab segment results in stress relief, and the magnitude of the stress relief is directly proportional to the movement. The partial restraint of movement that occurs at a transverse crack and distributes along the slab segment is due to the resistance of the steel and of the subbase; hence, a variation in the magnitude of movement is experienced in the area adjacent to the transverse crack. The magnitude of movement is inversely proportional to the distance from the transverse crack; therefore, the magnitude of the stress increases as the distance from the crack increases.

A relatively long slab would have (1) a central portion that is completely restrained, consequently, an area of constant stress, and (2) an area at each end of the slab segment that experiences a stress build-up as the distance from the crack increases. As long as the concrete is of relatively uniform strength, the central portion of a slab segment experiences a greater degree of cracking than the areas of the slab segment adjacent to the crack. The occurrence of an area of maximum stress results in the explanation of an earlier observation which showed that the majority of new cracks occur at a distance of three feet or more from an existing crack. (See Figure 7). This magnitude of distance must be covered before sufficient stresses are experienced in the pavement to rupture the concrete.

As the relative length of the slab segments decreases as a result of the new cracks appearing in the slab, the slab segments will no longer have an area of constant stress. If the temperature differentials remain approximately equal during the daily cycles, the pavement will attain a stable crack pattern, since the summation of the flexural and tensile stresses in the pavement will not rupture the concrete. This postulate explains why the traffic in the summer season did not influence the crack pattern. Before cracking will resume, a greater relative temperature differential will have to be experienced.

Figures 14 and 15 portray the longitudinal distribution of stresses in the concrete at the age of 22 hours (first cycle) and 70 hours (third cycle). The figures confirm the general discussion of stress distribution presented above. These stresses occurred during the periods of minimum temperature for each cycle, hence, the periods of maximum tensile stress. The magnitudes of the stress are rough approximations arrived at by the use of elastic theory, movement data obtained in this investigation, and concrete properties obtained from testing laboratory mixes similar to those used on the project. (Refer to Appendix E for the method of calculation.)

From Figures 14 and 15, it may be noted that stresses in the concrete are small near a crack, and that the stresses become progressively greater as the distance from a crack increases. The area of stress build-up or stress increase occurs in the section of a slab segment which experiences movement.

The movement and the stress build-up in the slab segments bring in the concept of subbase friction. A subbase with a high friction factor will result in a shorter stress transition area and a greater area of no movement within a slab segment. The increased subbase friction offers a greater

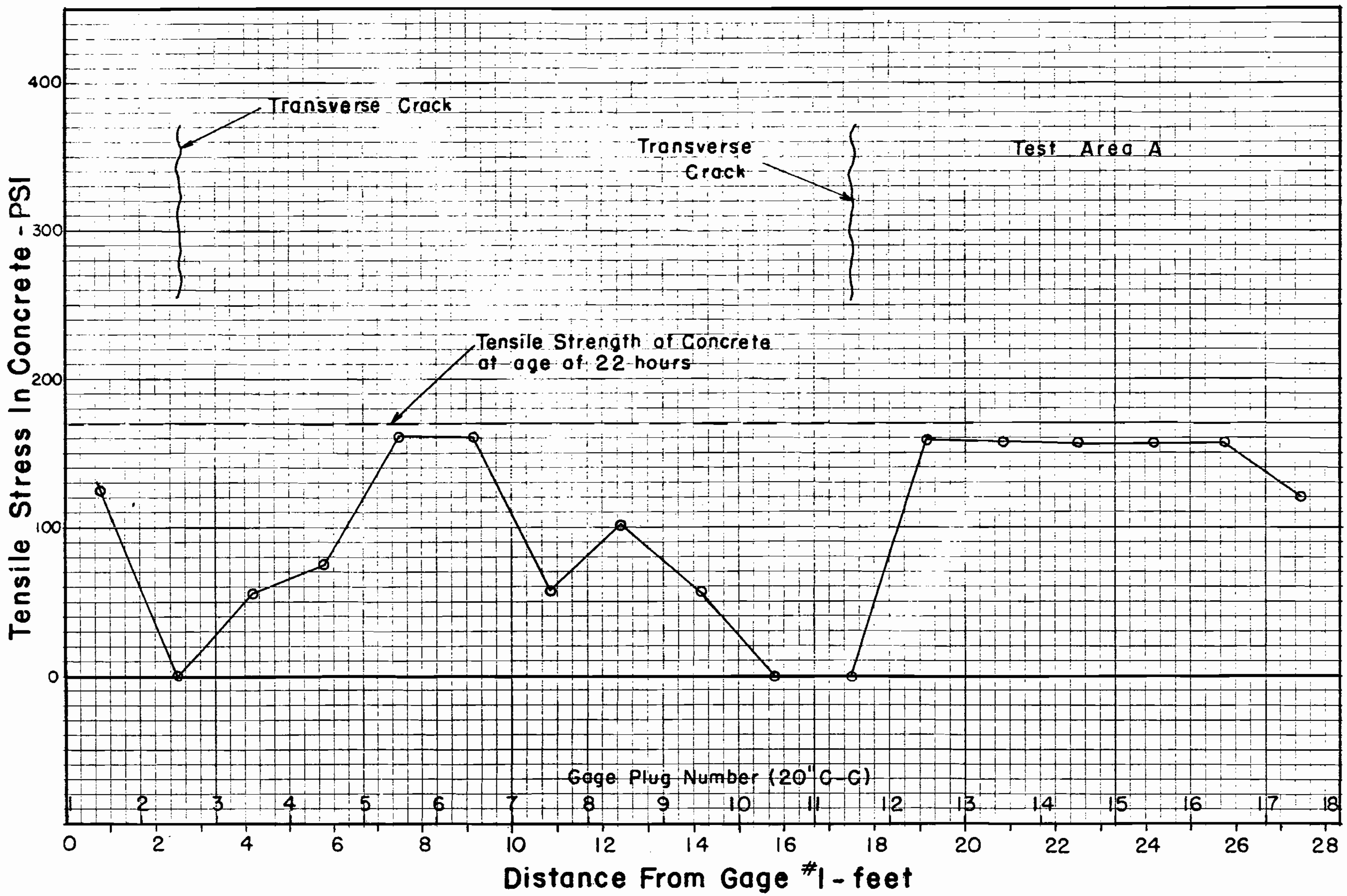


FIG. 14 - STRESS DISTRIBUTION AND TENSILE STRENGTH
CYCLE I

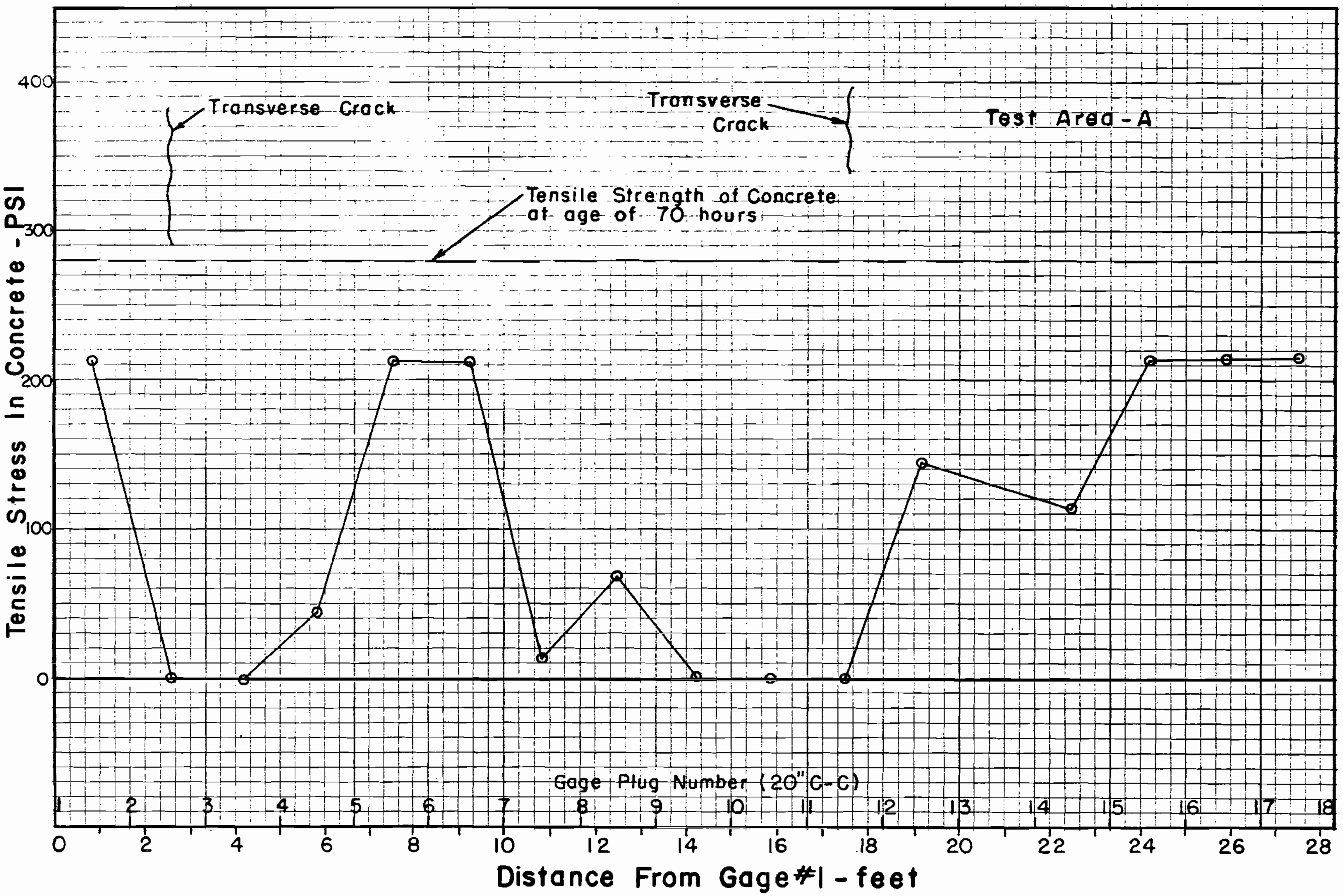


FIG. 15 - STRESS DISTRIBUTION AND TENSILE STRENGTH
CYCLE 3

resistance to movement; hence, there will be less stress relief due to movement. The greater the area of no movement, the greater the chance for tensile stresses near the tensile strength to be developed in the concrete and the resulting additional cracking. Therefore, high friction subbases will result in a smaller average crack spacing. With the benefits derived from close crack spacings, reported elsewhere, an appropriate statement would be that high friction subbases are a desirable design feature for continuously reinforced concrete pavement. (9/13).

Effect of Concrete Properties - In previous discussions, the fact was pointed out that the pavement showed its most rapid change in crack spacing during the initial ten days of life. This initial ten days corresponds to the curing period for the pavement, at which time the concrete properties are undergoing their greatest degree of change. The strength, the modulus of elasticity, and the thermal coefficient are increasing at various rates as the concrete ages. In general, the thermal coefficient and the modulus of elasticity are increasing at a greater rate than the tensile strength of the concrete, especially during the first few days after placement of the concrete. The rate of change for the thermal coefficient and for the modulus of elasticity tend to stabilize within 20 to 50 hours (2 days), while the rate of change for tensile strength does not level out for 200 to 240 hours (10 days). (8/51-67).

The stress in the concrete is a direct function of the first two properties mentioned above (see Equation (3) in Appendix E). The direct dependance of stress on the thermal coefficient and on the modulus of elasticity, in essence, means a greater stress will result from an equal temperature drop as the pavement age increases. This concept is of major importance during the first two days after placement of the concrete, since the stresses due to thermal changes are building up much more rapidly than the tensile strength of the concrete. Hence, the pavement experiences a high rate of cracking during this initial period. As the tensile strength of the concrete continues to increase, and the thermal coefficient and the modulus of elasticity of the concrete become more stable, the rate of cracking reduces markedly from its previous rate, if no sudden change in the weather conditions is experienced.

This reduced rate of cracking marks the end of Stage 1 and the beginning of Stage 2, discussed previously in connection with Figure 6. During Stage 2, the thermal changes experienced during the summer months are not of sufficient magnitude to develop failure stresses in the concrete. Therefore, Stage 2 shows only a small change in the average crack spacing, since the concrete properties are changing at a very slow rate. Once the fall weather arrives with its cool nights and warm days, there is a tremendous increase in the daily temperature differentials. By the time fall weather occurs, the pavement age is such that the tensile strength of concrete has tended to reach a leveling out point; therefore, the thermal changes, along with the wheel loads, again develop stresses of sufficient magnitude to cause cracking. This condition marks the end of Stage 2 and the beginning of Stage 3. Once again the rate of change in the average crack spacing assumes a high rate as in Stage 1. When the average crack spacing diminishes to a point where the slab segment movement, discussed previously, will not allow a large build up in stresses, the rate of change again decreases to a condition similar to that experienced in Stage 2. This change in the rate of crack development marks the end of Stage 3 and the beginning of Stage 4.

The effect of the different rates of increase for the thermal coefficient and the strength of the concrete on stresses is demonstrated by comparing Figures 14 and 15. Note that the stresses in Figure 14 are very near the tensile strength of the concrete in the area of fixity. The data are for the age of 22 hours, when the thermal coefficient and modulus of elasticity are increasing much more rapidly than the tensile strength. Figure 15 shows a greater difference between maximum stress in the concrete and tensile strength of the concrete for approximately the same temperature drop. The age of the pavement for the data is 70 hours, a period when the tensile strength is increasing faster than the other properties. In connection with this discussion of crack pattern formation, it is pointed out that variations in atmospheric conditions from those experienced would alter the curing and the concrete properties, and, consequently, the crack pattern development.

Predicting Crack Width

Figure 8 presented the general relation between air temperature and crack width for several cycles on this project. An examination of the data shows that it would fit the following mathematical expression:

$$W = K_1 t^n - K_2 T \quad . \; . \; . \; . \; . \; (1)$$

Where:

W = Crack width in inches

t = Age of pavement in days

T = Air temperature in ^oF

K_1, K_2, n = Constants

Using this general expression, and the data used in Figure 8, the constants were evaluated, and the following equation was derived:

$$W = 0.037\ t^{0.095} - 4.03 \times 10^{-4}\ T \quad . \; . \; . \; . \; (2)$$

Limits: $1 < t < 20$ days

$T > 60^oF$

Accuracy = $\pm$ 0.001 inch

This expression can be used for calculating the crack width at any specified air temperature and age within the specified boundary conditions. In general, the first term expresses the degree of permanent set occurring in crack width with age, whereas, the latter terms express the effect of temperature. In the discussion of movement at a transverse crack, it was pointed out that the permanent set of the crack width was probably a function of some concrete property.

Shrinkage, in all probability, is the concrete property contributing to the permanent set developed. One investigator, working with reinforced concrete beams, has developed a hypothesis that predicts the width of cracks in relation to concrete shrinkage. (12/105). This apparent importance of shrinkage indicates that any further research in crack widths of continuously reinforced concrete pavement should definitely consider shrinkage of the concrete. It is pointed out that equation (2) could be safely extrapolated to 30 days. The permanent crack width or set at this age would probably represent the maximum value, since most of the concrete shrinkage has developed by this age. (8/47).

VI. Conclusions

On the basis of results obtained from this investigation, the following conclusions are made:

1. Each slab segment within a slab experiences a fluctuating movement during each daily cycle. The magnitude of movement is a direct function of temperature and of the properties of the concrete. These factors have a definite influence on the magnitude of stress developed in the steel, and the degree of load transfer across a crack.
2. During the first ten days, the most significant changes in the crack pattern occur. The distribution of movement in a slab segment, the properties of concrete, and the location where a new crack occurs, are all interrelated phenomena. Therefore, the proportions of a concrete mix are of prime importance in determining a crack pattern, since the properties of the concrete are definitely related to the material proportions. This importance of mix design points out that further research should be directed toward the effect of the material proportions of the concrete on the crack pattern.
3. The width of a transverse crack is a direct function of shrinkage, and an indirect function of temperature. This study has evaluated the effect of temperature; therefore, further studies should consider the effect of shrinkage.
4. The lap staggering procedure used on this project has

evidently resulted in a much tighter construction joint than those obtained on previous projects. The results show that the transverse cracks and the transverse construction joints have approximately equal reaction to a change in temperature.

5. The distribution of movement in a slab segment on this project indicates subbase friction is definitely a factor to be considered in design. The results indicate why a higher subbase friction will result in a closer average crack spacing.

6. The location of transverse cracks are not influenced by the location of the transverse reinforcing bars.

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A P P E N D I X   A

General Information

TABLE I

MIX PROPORTIONS AND GENERAL INFORMATION PERTAINING TO CONCRETE PAVEMENT

1. Cement Factor = 5 sacks/cubic yard
2. Water-Cement Ratio = 5.8 gallons/sack
3. Coarse Aggregate Factor = 0.80
4. Type I cement was purchased from the Longhorn Cement Company
5. The coarse aggregate and the fine aggregate was purchased from the New Braunfels Sand and Gravel Company
6. The mixing water was obtained from a water pond adjacent to the project's right of way

PROCEDURE FOR INSTALLING GAGE PLUGS

1. Begin installation operation after the straight edge operation has passed and before belting of the surface.

2. Lay gage plug templates on the pavement and obtain proper alignment of the plugs.

3. Punch holes in the concrete by use of small bar or screwdriver.

4. Place plugs in the template holes and adjust to proper spacing by using the standard bar of the Berry Strain Gage.

5. Tap plugs through the template into the pavement.

6. Pick up the template and check the spacing of the holes.

7. After belting operations, clean out holes in the plugs and recheck spacing. Recheck the spacing after the curing agent is applied.

8. Observe plugs until all construction activity has passed the area: cures, curbing, etc.

9. Start reading the plugs after an age of five hours.

10. White wash the surface of the pavement at an age of eight to nine hours.

A P P E N D I X B

Test Values

Gage Plugs		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Date		5/6	5/6	5/6	5/6	5/6	5/6	5/7	5/7	5/7	5/7	5/7	5/7	5/9	5/9	5/9
Time		12:30P	2:10P	3:40P	4:45P	5:45P	6:45P	6:15A	7:15A	8:15A	9:15A	10:10A	1:45P	6:20A	7:25A	8:20A
Temp.		82°	80°	80°	80°	78°	72°	54°	57°	61°	63°	64°	81°	60°	65°	70°
1-2	B	300	294	295	298	302	305	316	315	312	308	303	296	312	312	312
	P	169	176	177	176	175	176	179	179	177	178	178	178	177	180	179
2-3	B	299	294	295	298	302	305	316	315	312	308	303	295	312	312	312
	P	137	143	141	138	137	136	88	78	78	83	86	109	65	67	68
3-4	B	297	294	295	298	302	305	316	315	311	307	302	295	312	312	311
	P	366	365	371	368	368	368	371	372	372	371	371	374	374	373	370
4-5	B	297	295	295	298	302	305	315	315	310	307	302	295	312	312	311
	P	301	302	302	302	301	302	305	304	304	304	303	304	307	307	307
5-6	B	297	295	295	298	303	305	315	314	310	306	302	294	312	312	311
	P	175	178	175	173	172	174	176	178	178	178	179	178	179	179	181
6-7	B	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	P	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
7-8	B	297	296	296	298	303	305	315	314	310	306	302	293	312	313	311
	P	86	89	90	82	90	89	92	91	92	93	92	93	94	95	95
8-9	B	295	296	296	298	303	305	315	314	310	306	302	293	312	313	310
	P	565	563	566	564	564	568	564	564	563	565	563	569	566	564	564
9-10	B	295	296	295	297	303	305	315	313	309	306	302	293	312	313	310
	P	302	314	315	314	314	316	317	319	318	319	318	320	325	324	324
10-11	B	295	297	295	297	303	305	315	313	308	305	302	293	312	313	310
	P	544	550	551	547	554	543	558	556	558	558	554	559	559	560	561
11-12	B	295	297	295	297	303	305	315	313	308	305	302	293	312	313	309
	P	518	518	529	525	526	523	507	504	506	502	508	525	482	482	482
12-13	B	293	297	295	297	303	306	315	313	308	305	301	293	312	313	309
	P	502	503	504	502	505	508	496	498	497	498	493	495	503	504	502
13-14	B	293	298	297	297	303	306	315	313	308	305	301	293	312	313	309
	P	—	207	202	201	199	199	202	202	200	201	206	200	211	204	211
14-15	B	293	298	297	297	303	307	315	312	308	305	301	293	313	313	309
	P	—	562	557	562	564	565	555	562	564	566	564	564	563	563	564
15-16	B	293	298	297	297	303	307	315	312	308	305	301	293	314	313	309
	P	—	434	444	437	440	442	447	447	446	445	448	446	453	452	450
16-17	B	292	295	297	297	303	307	316	312	308	305	301	292	314	313	309
	P	68	82	83	82	82	81	82	81	82	83	82	82	82	83	84
17-18	B	292	295	298	297	302	308	316	312	307	305	301	292	315	313	309
	P	526	525	518	525	525	526	527	520	517	525	529	530	528	529	529

Gage Plugs		16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
Date		5/9	5/9	5/9	5/9	5/9	5/9	5/9	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23
Time		9:20A	10:20A	11:20A	12:20P	1:20P	2:20P	3:20P	1:50P	3:25P	4:30P	7:30P	9:30P	11:30P	1:30A	4:25A	6:25A
Temp.		72°	79°	85°	86°	87°	88°	91°	91°	90°	86°	78°	74°	73°	73°	73°	73°
1-2	B	308	301	293	291	289	290	290	297	294	295	301	305	304	325	305	308
	P	180	179	178	176	178	178	177	179	182	183	184	185	186	185	187	186
2-3	B	308	301	293	291	289	290	290	296	294	295	301	305	304	305	305	308
	P	71	84	96	105	111	113	111	96	95	90	78	68	62	59	55	54
3-4	B	308	301	293	291	289	290	289	295	294	295	301	304	305	305	305	307
	P	374	373	365	373	371	372	370	359	362	363	373	363	362	376	375	376
4-5	B	308	300	293	291	289	290	288	294	294	294	301	304	305	305	305	307
	P	306	306	306	306	306	304	304	306	306	307	306	307	307	304	306	304
5-6	B	308	300	293	290	289	290	288	294	294	294	301	304	305	305	305	307
	P	182	181	179	179	180	178	178	181	182	179	179	180	180	180	180	181
6-7	B	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	P	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
7-8	B	308	300	293	290	289	290	287	295	294	295	302	305	305	305	305	307
	P	95	94	95	95	94	94	93	96	97	95	95	96	95	96	97	94
8-9	B	307	300	293	290	289	290	287	294	294	295	302	305	305	305	305	307
	P	564	563	563	563	563	563	563	563	564	563	563	563	563	563	565	564
9-10	B	307	300	293	289	289	290	287	293	294	295	302	305	305	305	305	307
	P	324	324	322	323	324	325	320	328	330	326	332	330	330	336	331	335
10-11	B	307	300	292	290	289	290	287	293	293	295	302	306	306	306	305	307
	P	561	550	556	560	560	560	557	555	560	555	562	553	553	560	560	562
11-12	B	307	300	291	290	289	290	287	293	293	295	302	306	306	306	305	307
	P	481	492	494	510	516	509	517	499	495	490	478	460	460	458	467	466
12-13	B	308	299	291	290	288	289	287	293	294	295	302	306	306	306	306	307
	P	501	491	488	492	492	494	501	499	498	502	501	516	516	513	509	502
13-14	B	308	299	291	290	288	288	287	293	294	295	303	306	306	306	306	307
	P	210	201	199	205	208	199	202	210	203	213	213	207	207	218	219	205
14-15	B	307	299	290	290	288	288	286	293	294	295	303	306	306	306	306	307
	P	563	564	563	562	569	563	563	562	564	564	563	567	567	562	564	564
15-16	B	307	299	290	290	288	287	286	293	294	295	303	306	306	306	306	307
	P	447	446	435	452	448	449	443	460	456	446	446	460	460	457	460	459
16-17	B	307	299	291	290	288	286	286	293	294	295	303	306	306	306	305	308
	P	84	83	84	83	85	81	83	87	87	87	87	88	88	88	89	90
17-18	B	306	299	291	289	287	285	285	293	294	296	303	306	306	306	305	308
	P	524	520	529	530	529	526	527	507	497	492	496	488	488	481	485	484

B- Readings on the Standard Bar   P-Readings on the Gage Plugs

1. Temperature shown is the Air Temperature taken in the shade.
2. Reading #2 selected as the zero base.
3. Multiply all reading by 10^{-3} inches.

TABLE 1 - DATA FOR TEST AREA -A, COMAL COUNTY, STA. 694+20, NBL

Gage Plugs		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Date		5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/23	5/25	5/25	5/25	5/25
Time		1:10P	2:10P	3:00P	4:05P	5:10P	6:05P	7:05P	8:10P	9:10P	10:10P	11:10P	12:10A	1:10A	2:10A	4:10A	5:10A	6:10A	7:10A	8:10A	9:10A	5:40A	6:10A	7:05A	8:10A
Temp		89	92	90	89	87	84	79	76	75	73	73	72	73	72	73	72	73	74	78	82	72°	72°	75°	76°
1-2	B	294	294	295	294	295	297	299	305	302	302	303	306	304	304	303	304	304	316	315	309	293	296	294	294
	P	208	207	208	208	209	209	208	209	209	210	209	208	209	209	209	209	210	214	221	220	201	200	200	200
2-3	B	294	294	295	294	294	297	299	305	302	302	303	306	304	304	303	304	304	315	315	309	294	295	294	294
	P	526	530	534	533	544	530	530	530	531	530	531	532	529	527	529	529	531	543	536	539	519	524	530	530
3-4	B	294	294	295	294	294	297	299	305	301	301	303	306	304	304	303	304	305	315	315	310	294	295	294	293
	P	252	264	254	264	263	263	264	262	262	263	263	264	264	264	263	263	263	264	268	263	243	238	253	253
4-5	B	294	293	294	294	294	297	299	305	301	301	303	306	304	304	303	304	305	314	315	310	295	294	294	293
	P	372	382	382	380	382	382	379	379	378	379	377	365	378	377	378	378	376	377	372	382	352	364	367	357
5-6	B	294	293	294	294	294	297	299	304	302	302	303	306	304	304	303	304	305	314	315	310	295	294	294	293
	P	199	201	199	202	200	201	200	203	201	200	202	200	200	200	200	200	201	213	211	201	192	192	192	192
6-7	B	294	293	294	293	294	298	299	303	302	302	302	306	304	304	304	305	305	315	315	309	295	294	294	293
	P	505	501	500	502	506	509	500	508	507	504	507	508	508	508	507	510	508	521	512	522	498	500	490	491
7-8	B	294	293	294	293	294	298	299	303	302	302	302	306	304	304	304	305	305	315	315	309	294	295	294	293
	P	375	365	368	367	375	373	374	374	375	373	375	373	378	375	377	378	378	379	378	375	357	368	368	361
8-9	B	294	292	294	292	293	297	299	303	301	302	303	305	304	304	304	305	305	316	315	309	294	295	294	293
	P	554	562	560	563	563	564	564	564	564	563	562	561	563	562	562	565	560	563	563	565	552	554	553	554
9-10	B	294	291	293	292	292	296	299	303	301	302	303	305	304	304	304	305	305	316	314	309	294	295	294	293
	P	349	349	352	364	362	362	363	363	363	351	360	358	358	365	363	361	369	375	376	375	223	334	342	334
10-11	B	294	291	293	292	292	295	299	303	301	302	303	305	304	304	304	305	305	316	314	309	294	295	294	293
	P	228	225	230	223	230	229	228	227	227	225	225	227	228	226	225	226	229	240	240	238	210	218	210	205
11-12	B	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	P	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
12-13	B	294	291	292	291	292	295	299	303	302	302	303	305	304	304	304	305	305	316	314	309	295	294	294	293
	P	391	387	388	394	395	395	393	393	393	393	393	393	392	393	393	394	378	376	374	383	343	347	338	345
13-14	B	294	291	292	291	292	296	299	303	302	302	303	305	304	304	304	305	305	316	313	309	295	294	294	293
	P	563	565	563	564	564	564	564	565	564	564	564	567	564	563	566	565	567	564	564	564	563	564	563	565
14-15	B	294	291	292	291	292	296	299	303	302	302	303	305	304	304	304	305	305	316	313	309	295	294	294	293
	P	337	336	339	335	347	345	348	347	349	346	347	345	346	348	348	347	351	350	362	354	340	341	340	330
15-16	B	294	291	292	291	293	296	299	303	302	303	303	305	304	304	304	305	305	316	313	309	296	294	295	293
	P	491	483	480	485	492	492	492	489	491	490	490	483	490	490	490	491	491	502	492	502	470	468	475	471
16-17	B	294	291	292	291	294	296	300	303	302	303	304	304	304	304	304	305	304	316	313	309	296	294	295	293
	P	75	75	75	77	76	76	75	74	75	74	75	75	75	75	75	76	76	88	87	88	86	68	68	68
17-18	B	294	291	292	291	294	296	300	303	302	303	304	304	304	304	304	306	304	316	313	309	296	295	295	293
	P	108	108	108	108	109	107	107	107	106	106	107	107	107	107	106	107	107	119	119	119	100	99	95	100
18-19	B	294	291	291	291	293	296	300	303	302	303	304	305	304	304	304	306	304	316	313	286	296	295	295	293
	P	111	110	111	112	111	111	111	110	110	110	110	110	110	110	110	110	111	123	121	101	102	102	100	102
19-20	B	294	291	291	291	293	295	300	302	302	303	304	305	304	304	304	305	304	316	313	286	296	295	295	293
	P	35	32	35	36	31	36	34	31	33	35	34	33	34	35	34	34	37	48	42	25	28	29	28	29
20-21	B	293	291	291	291	293	295	300	302	302	303	304	305	304	304	304	305	304	316	313	286	296	295	295	293
	P	175	174	170	175	175	174	174	174	173	175	174	173	173	173	170	173	173	185	185	162	163	164	165	162
21-22	B	293	291	291	291	293	296	300	303	302	303	304	305	304	304	304	305	304	316	313	286	296	295	295	293
	P	437	441	432	434	432	430	429	441	440	442	441	441	437	437	439	440	443	440	438	415	429	430	428	425
22-23	B	294	291	291	291	294	296	300	303	303	302	305	305	304	304	304	304	304	315	313	286	297	295	295	293
	P	38	38	41	39	39	39	39	38	38	38	35	37	38	38	38	37	40	49	47	20	28	28	29	30
24-25	B	294	291	291	291	294	296	300	303	303	302	305	305	304	304	304	304	304	315	313	286	297	295	295	293
	P	242	242	243	240	243	242	242	241	241	240	240	241	240	239	240	240	240	253	251	228	229	221	229	230

B – Readings on the Standard Bar P – Readings on Gage Plugs

1. Temperature shown is the Air Temperature taken in the shade.
2. Reading #1 selected as the zero base.
3. Multiply all readings by 10^{-3} inches.

TABLE 2 – DATA FOR TEST AREA - B, COMAL COUNTY, STA. 769+00, SBL.

TABLE 3

READINGS ON GAGE PLUGS ACROSS CONSTRUCTION JOINT[2]

Reading No.	Standard Bar Reading[1]	Plug Reading[1]	Air Temp.	Date	Time
1	298	535	80	5/7/60	12:00 AM
2	314	494	60	5/8/60	6:45 AM
3	306	500	75	5/8/60	9:45 AM
4	289	532	86	5/8/60	1:00 PM
5	295	529	90	5/23/60	1:30 PM
6	303	504	77	5/23/60	7:30 PM
7	307	502	73	5/24/60	6:30 AM
8	290	510	86	5/24/60	10:40 AM
9	492	708	81	5/25/60	7:30 AM

[1]Multiply readings by 10^{-3}

[2]Station 706+70 NBL - Comal County

TABLE 4

LABORATORY TEST RESULTS FROM FIELD SPECIMENS

TAKEN AT TEST AREA B

Tension Determinations

Age in Days	Ultimate Stress in psi.
2	11
4	57
7	63

Compressive Strength of Concrete at 7 days = 4610 psi

Modulus of Elasticity at 7 days = 5.55×10^6 psi

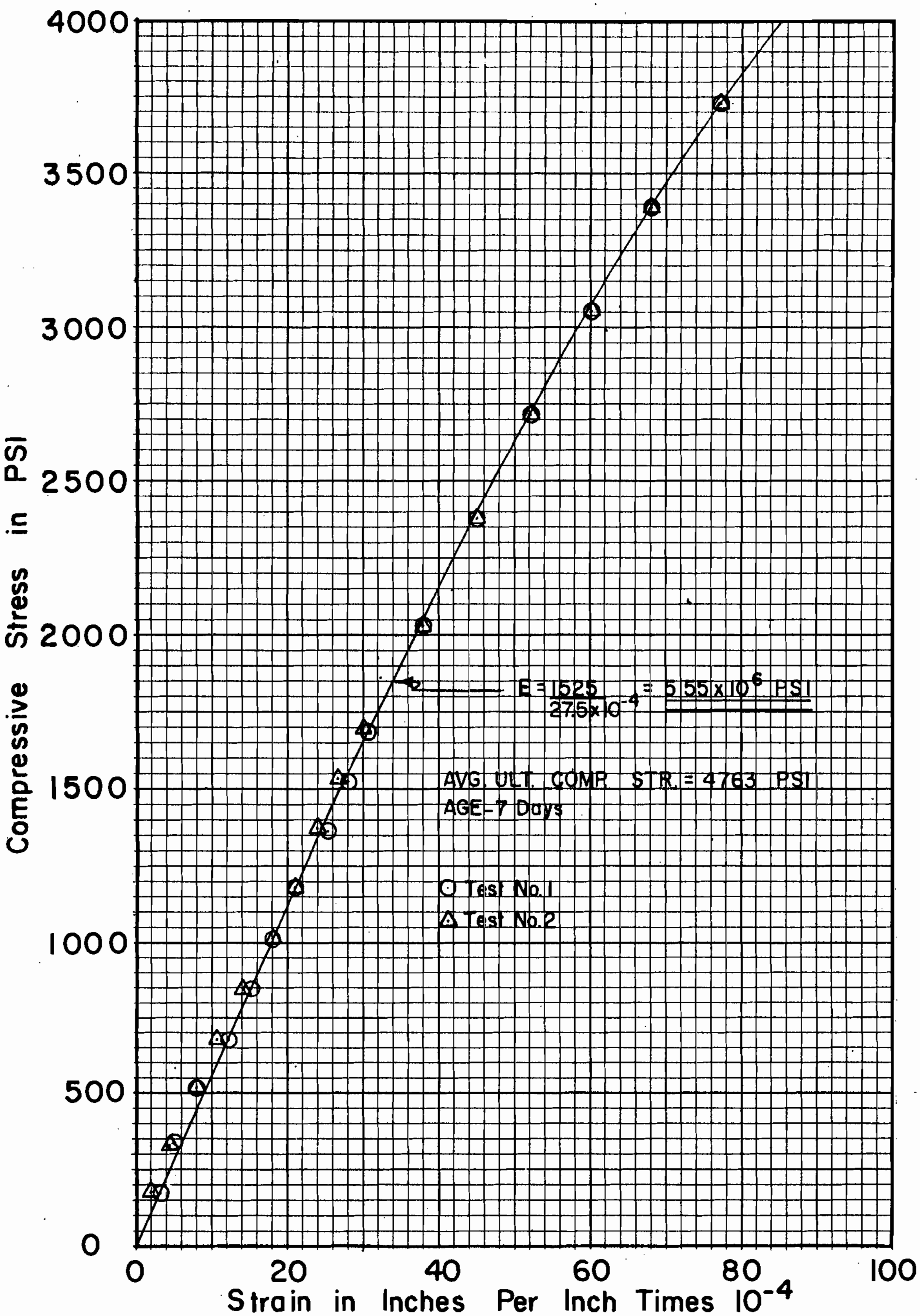


FIG. 1 – COMPRESSIVE STRESS-STRAIN CURVE FOR THE CONCRETE AT THE AGE OF SEVEN DAYS

A P P E N D I X C

Calculation of Movement

PROCEDURE FOR CONCRETE MOVEMENT CALCULATIONS

A 20-inch Berry Gage was used to measure the movement in the concrete between adjacent gage plugs. The Berry Gage has 0.001 of an inch graduations with a 5 to 1 multiplication factor, hence movements to the closest 0.0002 of an inch were detected. The recorded data are translated to relative movement as follows:

$$C = A - B \quad \ldots\ldots\ldots\ldots (1)$$

A = Reading with the Berry Gage on the standard bar.

B = Reading with the Berry Gage on adjacent plugs in the pavement.

(Note: Keep track of the signs)

The values from equation (1) are then substituted into the following equation:

$$\Delta X_G = \frac{Cn - Co}{M} \quad \ldots\ldots\ldots\ldots (2)$$

Where:

ΔX_G = Movement experienced between adjacent gage plugs after the reference readings, inches.

Co = Initial or base difference, inches. (Zero reading).

Cn = Difference at any interval after the base difference, inches.

M = Multiplication ratio of the gage (5:1).

Equation (2) is in the general form used to make strain calculations from data taken in laboratory experiments other than being divided by the gage length. Movement is used in lieu of strain in this investigation, since any measured movement is not due to strain of the concrete, but to relief from restraint stresses developed due to suppressed volume changes of the concrete. In other words, if no movement was recorded after a temperature drop, then restraint stresses are developed in the slab. Any movement of the slab segment is relief from the restraint stresses.

Another factor that must be considered with the form of application used in this investigation is the changes in length experienced by the standard bar. In normal laboratory uses, this change in length is not a point of consideration, since very little temperature differential is experienced; but with long periods of outdoor use, large magnitudes of temperature differentials are experienced during daily cycles. The standard bar used with the Berry Gage is made from cold-rolled keyway steel (thermal coefficient = $6 \times 10^{-6}/^{\circ}F$). The correction in length change of the standard bar would be of the following form:

$$\Delta X_T = \gamma X \ (Tn - To) \ . \ . \ . \ . \ . \ (3)$$

Where:

ΔX_T = Correction of standard bar length due to changes in temperature from that recorded with the initial reading, inches.

X = Gage length of standard bar, inches (20 inches).

γ = Thermal coefficient of the standard bar steel, $/^{o}F$.

To = Temperature at time of the base reading, ^{o}F.

Tn = Temperature at any interval after the base reading, ^{o}F.

Substituting the known values into equation (3):

$$\Delta X_T = 120 \times 10^{-6}(Tn - To) \quad . \; . \; . \; . \; . \; . \; (4)$$

Equations (2) and (4) can be combined for a net expression of movement from a reference measurement.

$$\Delta X_N = \Delta X_G + \Delta X_T \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (5)$$

Where:

ΔX_N = Net movement between adjacent gage plugs after correcting for changes in length of the standard bar, inches.

By substituting and combining terms, the following expression is derived:

$$\Delta X_N = \frac{Cn - Co + 6 \times 10^{-4} \; (Tn - To)}{5} \quad . \; . \; . \; (6)$$

If the algebraic sign is:

(+) positive - the distance between adjacent gage plugs is increasing.

(-) minus - the distance between adjacent gage plugs is decreasing.

For the purpose of selecting the base reading, all the values of relative movement between adjacent gage plugs were plotted versus the time after placement of the concrete for each gage interval. Since the set of readings made when the concrete had aged 7 1/2 hours (second set of readings) gave readings which were consistent with readings taken at a later time, the distance between gage plugs recorded at 7 1/2 hours were taken as the base measurements. The difference, Co, in equation (6) is the second set of readings.

EVALUATING THE UNKNOWN VARIATION IN LENGTH OF THE STANDARD BAR

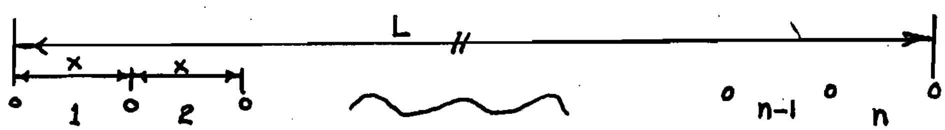
The air temperature was used to correct the standard bar for changes in length due to the changes in temperature. These corrections were approximations, since the temperature of the standard bar was not measured directly. The bar was used in a shady area on top of the pavement surface. The variation between the maximum and the minimum surface temperature of the pavement during a cycle is greater than that experienced by the air temperature. Therefore, the bar actually went through a greater temperature change than the use of the air temperature would indicate.

Two different methods were used to evaluate the effects of this unknown variation in bar length on the movement between adjacent gage plugs.

Method I - Summation of Movements

The following derivation is based on the assumption that the summation of movement for all of the gage plug intervals is equal to zero.

Before movement between gage plugs occurs:



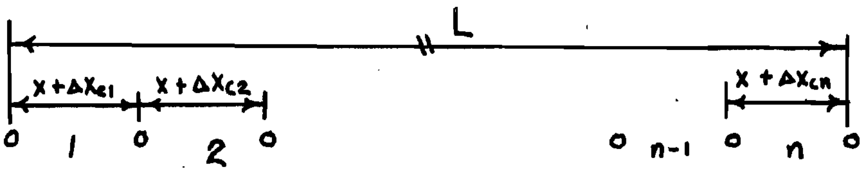
L = Distance between 1st and last gage plug, inches.

X = Uniform distance between gage plugs, inches.

n = Total number of spaces between gage plugs.

$$L = n\,X \qquad (1)$$

If movement occurs:



Assuming L remains constant, the following expression can be developed:

$$L = (X + \Delta X_{c1}) + (X + \Delta X_{c2}) \ . \ . \ . + (X + \Delta X_{cn}) \ . \ . (2)$$

Where:

ΔX_{ci} = Corrected movement between adjacent gage plugs encompassing all the variables - known and unknown, inches.

Using the basic expression for movement derived in the procedure for concrete movement calculations and revising for the needed correction, the following equation is obtained:

$$\Delta X_c = \Delta X_N - \Delta X_k \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ (3)$$

Where:

ΔX_N = Net movement between adjacent gage plugs from readings and temperature corrections, inches.

ΔX_k = Unknown movement of the standard bar applicable to each set of gage plugs that cannot be evaluated with present data, inches.

Combining equations (2) and (3).

$$L = (X + \Delta X_{N1} - \Delta X_k) \ . \ . \ . + (X + \Delta X_{Nn} - \Delta X_k)$$

$$L = n X + \left[(\Delta X_{N1} - \Delta X_k) \ . \ . \ . + (\Delta X_{Nn} - \Delta X_k)\right]$$

Substituting equation (1) in the above expression and combining terms:

$$L = L + \left[(\Delta X_{N1} - \Delta X_k) \; . \; . \; . + (\Delta X_{Nn} - \Delta X_k) \right]$$

$$(\Delta X_{N1} \; . \; . \; . + \Delta X_{Nn}) - n \Delta X_k = 0$$

$$X_k = \frac{\sum \Delta X_N}{n} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (4)$$

Using the readings taken during the periods of minimum temperature, the correction movement can be calculated.

Setup A

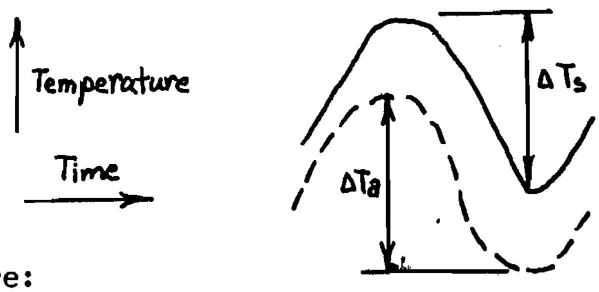
Reading	n	$\sum \Delta X_N$	ΔX_k
7	16	0.02108	0.00132
13	16	0.02320	0.00145
31	9*	0.00756	0.00084

Average $\Delta X_k = + 0.00120$ inches at 6:00 AM.

* Only the readings between transverse cracks used.

Method II. - Approximating Temperature Change

This derivation is based on the assumption that the surface temperature of the pavement would give a better correction than the air temperature. The corrections thus far were made in terms of air temperature.



Where:

—— Ts = Temperature of the surface.

--- Ta = Air temperature.

Subscripts o and n refer to zero reading and readings at any hour, respectively.

The differences from 0-hour to nth hour:

in terms of air - $\Delta Ta = Tao - Tan$ . . . (1)

in terms of surface - $\Delta Ts = Tso - Tsn$ . . . (2)

A correction in the movement between gage plugs for changes in the standard bar length due to variations in the air temperature was made previously. Therefore, for this method, the difference between the variation in the air temperature and the variation in the surface temperature must be accounted for.

$$\Delta T_k = \Delta Ts - \Delta Ta \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

In a recent field investigation by the Highway Design Division of the Texas Highway Department, a relationship between slab temperature and air temperature was obtained. The following table is a presentation of a portion of this data for a weather season similar to the Comal County investigation.

Tsn[1] ($^{\circ}F$)	Tso[2] ($^{\circ}F$)	ΔTs ($^{\circ}F$)	Tan[1] ($^{\circ}F$)	Tao[2] ($^{\circ}F$)	ΔTa ($^{\circ}F$)	ΔTk ($^{\circ}F$)
72.5	94	21.5	67	84	17	4.5
67	98	31	64	85	21	10
65.5	100	34.5	60.5	88.5	28	6.5
69.5	100	30.5	66	87.5	21.5	9
69	101	32	65.5	91.5	26	6
72	100	28	69	93	24	4

$$\frac{40.0}{6} = 6.7$$

1 - Temperature at 6:00 AM

2 - Temperature at 2:00 PM

By applying the above value of ΔTk with the gage length and thermal coefficient of the standard bar, a correction movement is obtained.

$$\Delta X_k = (6 \times 10^{-6} in/in/^{\circ}F)\ (6.7^{\circ}F)\ (20\ in.)$$

$$\Delta X_k = 0.0008 \text{ inches at 6:00 AM}$$

Averaging the values obtained by the two methods.

$$\Delta X_k = + 0.0010 \text{ inches.}$$

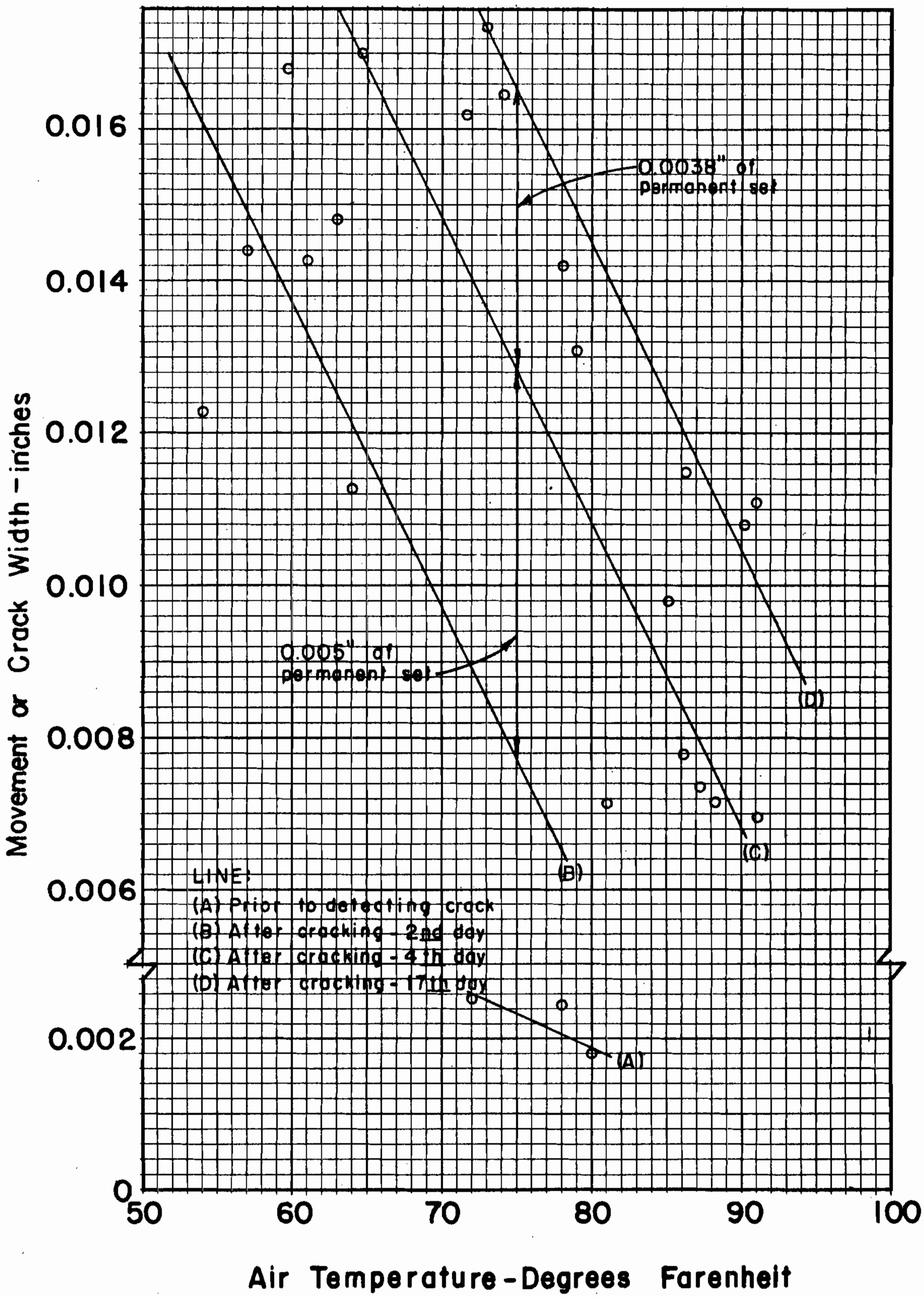


FIG. I CRACK WIDTH vs. AIR TEMPERATURE FOR TEST AREA A

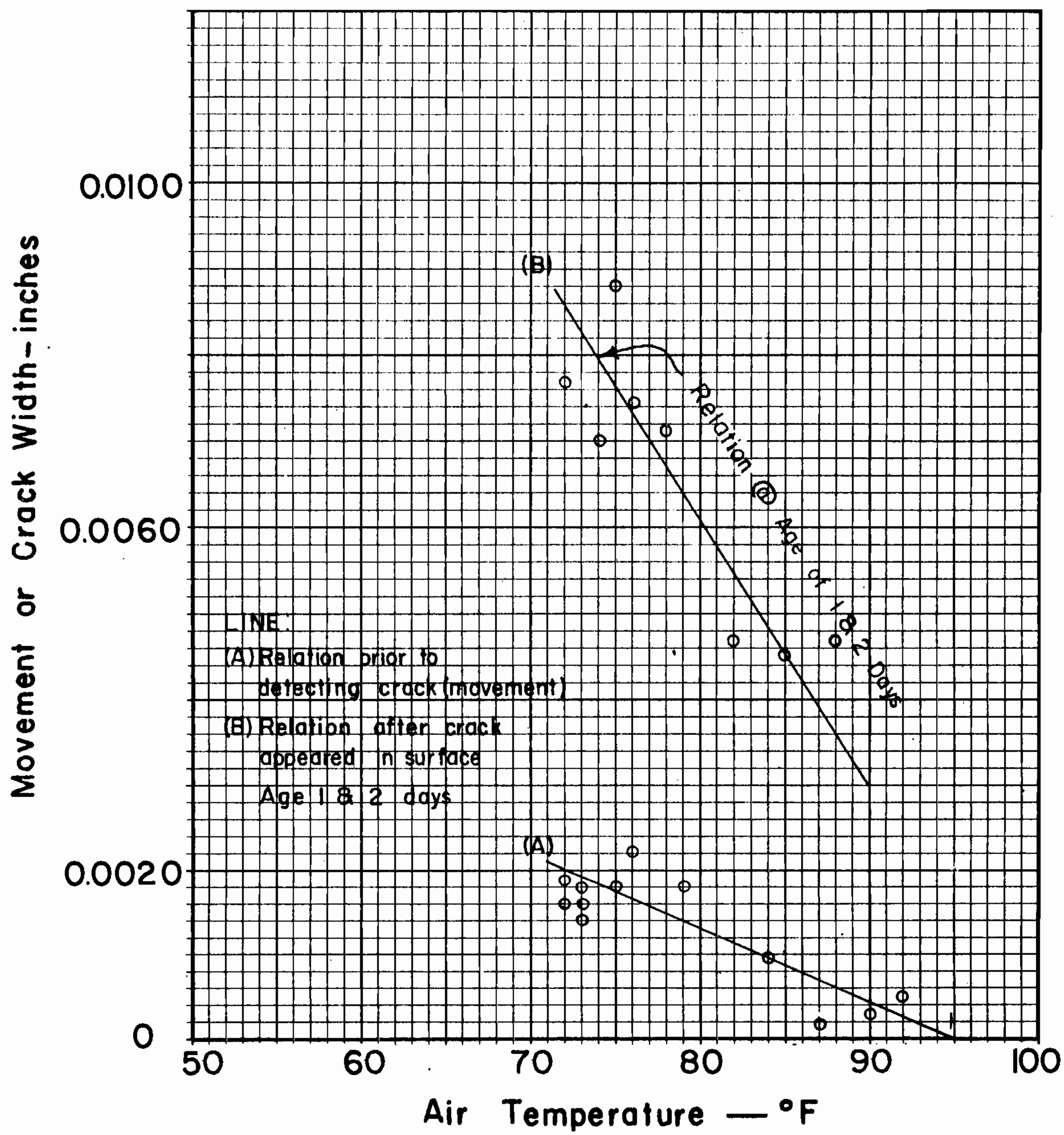


FIG. 2 — CRACK WIDTH VS AIR TEMPERATURE FOR TEST AREA B

A P P E N D I X   D

Calculation of Subbase Friction

PROCEDURE FOR SUBBASE FRICTION CALCULATIONS

The object of this set of calculations is to obtain a value of subbase friction by using the data obtained from the terminal movement measurements. The first step was to plot the readings obtained at each of the terminals against the air temperature at the time of the measurement. Figure 1 in this Appendix portrays the results obtained at the south end of the Walnut Street Overpass. The slope of the relation on the graph was calculated as follows:

$$M = \frac{\Delta L}{\Delta T} \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Where:

ΔL = The change in width of the expansion joint - inches.

ΔT = The change in temperature for corresponding change in width of the expansion joint, ^{o}F.

The results of these calculations are tabulated in Table 1 of this Appendix. By using the linear thermal change formula, the length of slab moving over the subbase could then be calculated.

$$L_c = \frac{\Delta L}{12 \alpha \Delta T} \quad \ldots\ldots\ldots\ldots (2)$$

Where:

L_c = The length of slab moving over the subbase, feet.

α = The thermal coefficient of the concrete, $in/in/^{o}F$.

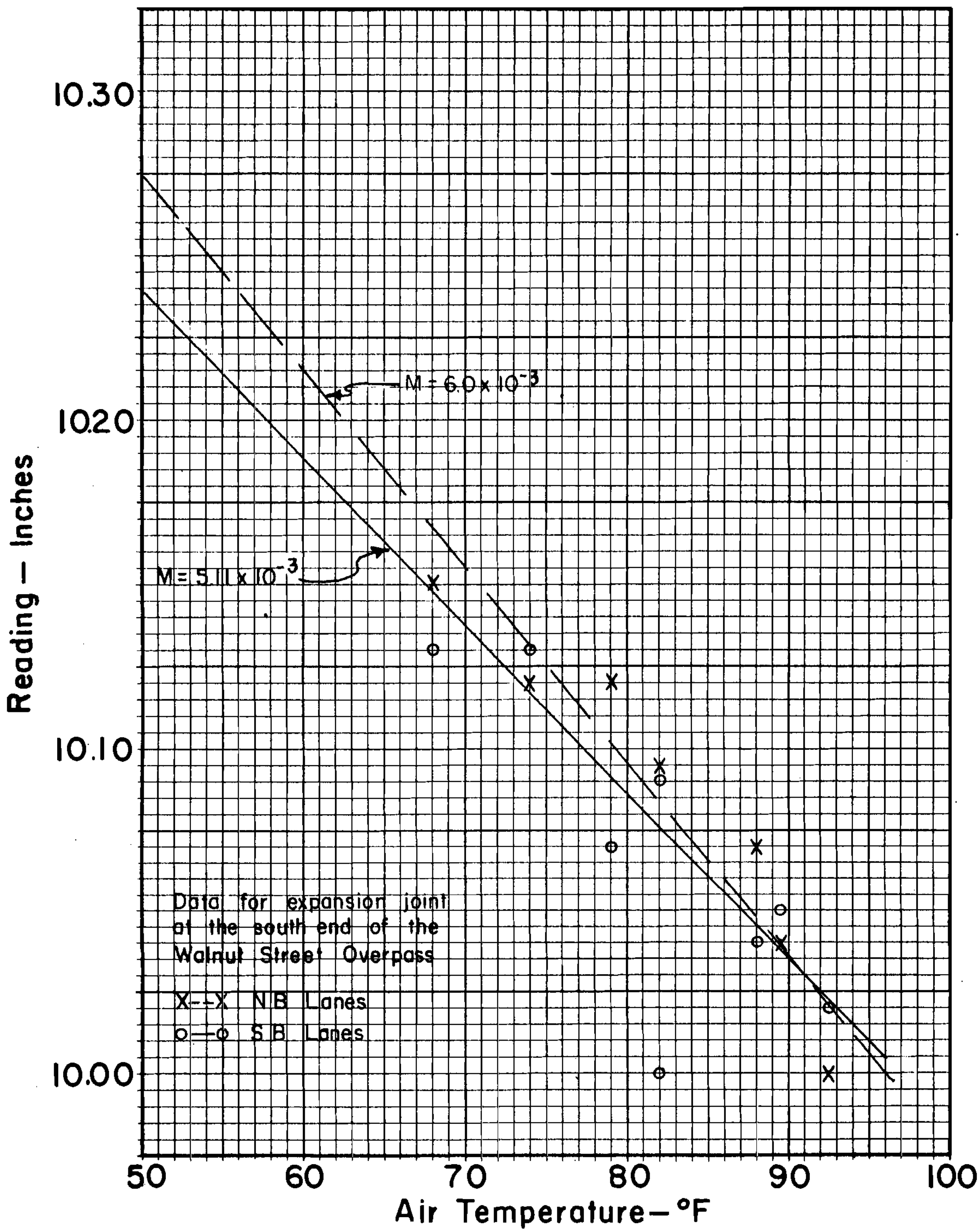


FIG. 1— READINGS AT AN EXPANSION JOINT VS AIR TEMPERATURE

TABLE 1

TABULATION OF CALCULATIONS FOR LENGTH OF SLAB MOVING OVER THE SUBBASE

Location of Terminal	M in/°F 10^{-3}	Lc (ft.)	Lc - $\bar{L}$
South Business Route O/P			
North end - NBL	4.0	98	5
North end - SBL	4.95	121	14
Walnut Street O/P			
South end - NBL	6.0	147	40
South end - SBL	5.11	125	18
State Highway 46 O/P			
North end - NBL	2.94	72	35
North end - SBL	3.47	85	22
South end - NBL	4.22	103	4
South end - SBL	4.30	105	2
		Σ856	140
		$\bar{L}$ = 107 ft	A.D. = $\pm$ 17 ft.

Note:
1. Expressway type facility with twin bridges.
2. A.D. = Average deviation
3. O/P = Overpass
4. Lc = calculated length of slab end moving over the subbase.

In connection with work being performed by the Highway Design Division of the Texas Highway Department, numerous tests of concrete properties have been performed by the Texas Transportation Institute. These data are contained in their final report on Research Project-19 (see reference 8 in the Bibliography). In order to obtain the concrete properties needed in certain calculations in this study, batch designation AU-1 was selected as the reference batch from the Research Project-19 study. This batch had similar aggregates and mix proportions as the concrete used in the Comal County paving project. Figures 1 and 2 in Appendix E show the age relation of the various concrete properties, i.e. strength, thermal coefficient, modulus of elasticity. Using these data, a thermal coefficient of 3.4×10^{-6} in/in/°F was selected for the subbase friction calculations. Substituting this value of the thermal coefficient and equation (1) into equation (2), the following expression for length of slab moving over the subbase is obtained.

$$L_c = \frac{M}{4.08 \times 10^{-5}} \quad \dots\dots\dots\dots (3)$$

The values of length calculated with equation (3) are also tabulated in Table 1, along with the average value of the calculated lengths and the average deviation. Using these average values, then the subbase friction can be calculated with the "subgrade drag" formula.

$$f = \frac{2\ Ss\ p}{10^4\ (2L)} \quad \dots\dots\dots\dots (4)$$

Where:

f = The coefficient of subbase friction.

p = The percentage of longitudinal steel, percent. (0.5%)

Ss = The yield point stress of the steel, psi, (50,000 psi)

L as used in the standard formula is the length between expansion joints; therefore, the end of slab moving must be doubled (2L) before using it in this formula.

The following results were obtained after substituting the proper values into equation (4).

Average value of $f = 2.3$

Range with average deviation of length: $1.9 < f < 2.7$.

A P P E N D I X   E

Calculation of Stress Distribution

CALCULATING STRESS DISTRUBUTION

By combining concrete properties and measured movements, a rough calculation of the stresses in the concrete can be made. Elastic theory (which is not completely applicable to concrete) must be applied to determine the stresses.

Theoretical Stress Derivation

If the concrete was completely fixed, the stress would be as follows:

$$Sc = \epsilon E \quad \ldots\ldots\ldots\ldots \quad (1)$$

Where: S = The stress in the concrete, psi.

E = The modulus of elasticity of the concrete, psi.

ϵ = Concrete strain due to a change in temperature, in/in.

The theoretical strain is equal to the following:

$$\epsilon = \alpha \Delta T \quad \ldots\ldots\ldots\ldots \quad (2)$$

Where: α = The thermal coefficient of concrete, in/in/^{o}F.

ΔT = The change in temperature, ^{o}F.

Substitute equation (2) into equation (1)

$$Sc = \alpha \Delta T E \quad \ldots\ldots\ldots\ldots \quad (3)$$

To make these calculations, the concrete properties will be required.

Concrete Properties

Since the test results of the specimens taken on the project were incomplete and erratic, supplementary tests must be used. Research Project-19, conducted at the Texas Transportation Institute, offers basic data for the concrete properties. Using this source, Figures 1 and 2 were compiled to show influence of age on the properties of a batch similar to concrete used on the project.

From graphs:

Age (Hours)	E*(psi)	α (in/in/°F)	Tensile strength (psi)
22	2.3×10^6	2.9×10^{-6}	165
70	3.35×10^6	3.18×10^{-6}	280

* Tensile E used (ultimate)

Calculating Theoretical Stress

The air temperature at 22 hours and 70 hours is 54° F and 60° F, respectively. These values result in temperature changes of - 26° F and - 20° F. By using the temperature change along with concrete properties in equation (3) the theoretical stresses may be calculated.

$$Sc = (2.9 \times 10^{-6})\ (24)\ (2.3 \times 10^{6}) = 160 \text{ psi @ 22 hours}$$

$$Sc = (3.18 \times 10^{-6})\ (20)\ (3.35 \times 10^{6}) = 213 \text{ psi @ 70 hours}$$

Calculating Stress Distribution

If no movement occurred within the slab segment, the theoretical stresses would apply. Therefore, any movement would be stress relief, so it must be combined with the theoretical stress as follows:

$$Sa = Sc \neq \frac{\Delta Xc}{X} E \quad \ldots\ldots\ldots\ldots (4)$$

Where:

Sa = Actual stress in area between adjacent gage plugs, psi.

Δ Xc = Corrected movement experienced between adjacent gage plugs, inches. (See procedure for calculating correction movements in Appendix C).

X = Gage length, inches. (20 inches)

Gage Interval	Age of 22 hours		Age of 70 hours	
	$\epsilon_c \times 10^{-6}$ **	Sa	$\epsilon_c \times 10^{-6}$ **	Sa
1-2	-16	123	0	213
2-3	Crack	0	Crack	0
3-4	-46	54	-80	0
4-5	-36	77	-50	45
5-6	+14	*160	-10	180
6-7	-	*160	-	-
7-8	-46	54	-60	12
8-9	-26	100	-40	79
9-10	-46	54	-120	0
10-11	-106	0	-110	0
11-12	Crack	0	Crack	0
12-13	+44	*160	-20	146
13-14	+14	*160	-70	0
14-15	+34	*160	-30	112
15-16	-	*160	-	*213
16-17	+4	*160	+20	*213
17-18	-16	123	0	213

* Maximum stress assumed as the theoretical stress.

** Data from Table 1 in Appendix B with the appropriate correction applied. (Note $\epsilon_c = \Delta Xc/X$)

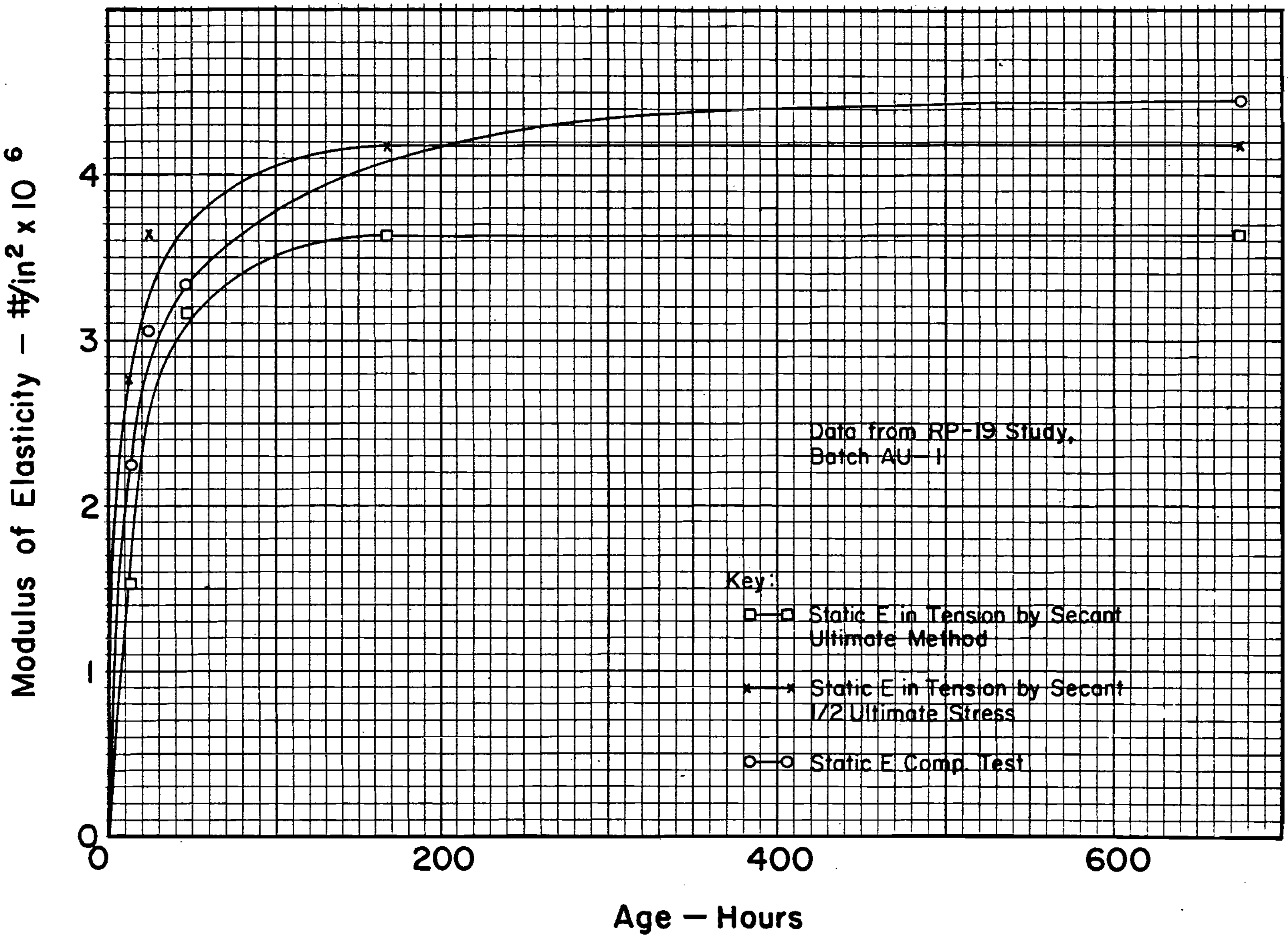


FIG. 1 - EFFECT OF AGE ON MODULUS OF ELASTICITY
OF THE CONCRETE

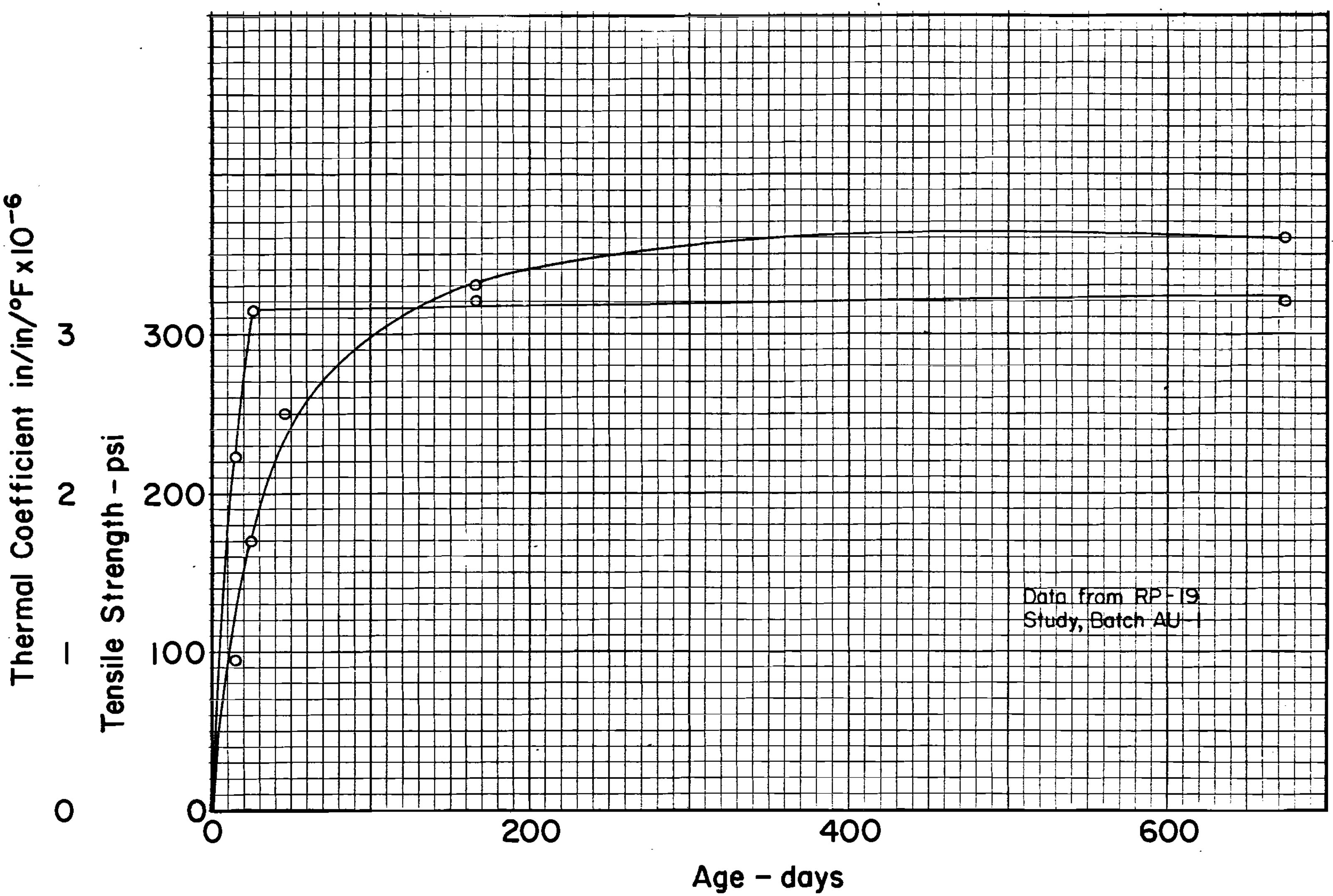


FIG. 2 - EFFECT OF AGE ON THE THERMAL COEFFICIENT AND TENSILE STRENGTH OF THE CONCRETE