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DESIGN OF THE TEXAS PRESTRESSED CONCRETE PAVEMENT OVERLAYS IN COOKE AND McLENNAN COUNTIES AND CONSTRUCTION OF THE McLENNAN COUNTY PROJECT

Alberto Mendoza Diaz, B. Frank McCullough,
and Ned H. Burns

RESEARCH REPORT 555/556-1

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16. Abstract

This report describes the development of the design of prestressed overlays to be installed in Cooke and McLennan Counties, Texas, and includes a detailed description of the construction and placement methods used. The design was developed from state-of-the-art design procedures and experience gained on past prestressed concrete pavement projects in the United States. Construction problems encountered in the McLennan County project are reported. Eighteen 240-foot and fourteen 440-foot slabs were cast in McLennan County in two lanes with a concrete paver. The slabs were post-tensioned by applying the prestress force in pockets located near mid-slab. This central stressing technique was introduced in prestressed pavements in the Texas projects and eliminated the need for gaps between adjacent prestressed slabs for the post-tensioning operations.

Another innovation introduced with the Texas projects was the use of prestress in both directions. The need for transverse prestress had been recognized because in-service prestressed projects had often developed longitudinal cracking. Transverse prestress permits better control of the load stresses in the pavement and kepps the longitudinal construction joint between the lanes tightly closed. The slabs were keyed together at the transverse joints with load transfer dowel bars clad with stainless steel. The longitudinal joints were designed and constructed to allow slab length changes. During the construction of the project, the pavement was instrumented and monitored at specific sections. It is being closely observed and a long-term plan for analyzing performance is being developed.

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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report describes work done on Research Projects 555-556, "Demonstration of Prestressed Concrete Pavement in McLennan and Cooke Counties." The study was conducted at the Center for Transportation Research (CTR), The University of Texas at Austin, as part of a cooperative research program sponsord by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration.

Many people contributed their help toward the completion of this report. Thanks are extended to all the CTR personnel, especially Lyn Gabbert for typing this report. Invaluable comments were provided by James Brown from the Texas State Department of Highways and Public Transportation.

Alberto Mendoza Diaz
B. Frank McCullough
Ned H. Burns

LIST OF REPORTS

Report No. 555/556-1, "Design of the Texas Prestressed Concrete Pavement Overlays in Cooke and McLennan Counties of the McLennan County Project," by Alberto Mendoza Diaz, B.Frank McCullough, and Ned H. Burns, presents the design procedure for the prestressed overlays in Cooke and McLennan counties and the construction report for the McLennan County Section. February 1986.

ABSTRACT

This report describes the development of the design for the prestressed overlays that were to be installed on southbound IH-35 in Cooke and McLennan Counties, Texas. The report also includes a detailed description of the construction and placement methods used in the McLennan County project. There was no construction in Cook County.

The design presented in this report was developed from state-of-the-art design procedures backed up with the experience gained on past pretressed concrete pavement projects in the United States. Construction problems encountered in the construction of the McLennan County project are reported to provide valuable experience as to what can be expected on future prestressed concrete pavement projects.

Between 17 September and 20 November 1985, eighteen 240-foot and fourteen 440-foot slabs were cast in McLennan County, 15 miles north of Waco, Texas. The concrete was placed in two lanes with a concrete paver. The slabs were post-tensioned by applying the prestress force in pockets located near midslab. This central stressing technique was introduced prestressed pavements in the Texas projects. The central stressing technique eliminated the need for gaps between adjacent prestressed slabs for the post-tensioning operations.

Another innovation in prestressed pavements for highway applications introduced with the Texas projects was the use of prestress in both directions. The need for transverse prestress had been recognized in the past since in-service prestressed projects had often developed longitudinal cracking. Transverse prestress permits better control of the load stresses in the pavement and keeps the longitudinal construction joint between the lanes tightly closed. The slabs were keyed together at the transverse joints with load transfer dowel bars clad with stainless steel. The longitudinal joints were designed and constructed to allow slab length changes.

The paving work in Waco was completed in late November, 1985, and was opened to traffic in December, 1985. During the construction of the project, the pavement was instrumented and monitored at specific sections. The pavement is being closely observed and a long-term plan for analyzing performance is being developed. The results from this plan will be documented in future reports.

SUMMARY

This report presents the development of the design for two prestressed overlays, in Cooke and McLennan Counties, Texas. The planned projects consisted of two one-mile experimental prestressed pavements, one in each county, that were to be placed on 24-foot-wide reinforced jointed pavements with shoulders added on each side for a total width of 38 feet. This report also includes a detailed description of the construction and placement of the McLennan County overlay, the only one actually constructed. The material in this report may be used later in the development of a design manual for prestressed pavement and overlays.

The design covered in this report includes the selection of the location for the experimental section in Cooke County from a set of possible candidate sections proposed by the SDHPT. The procedure for determining the thickness and prestress level at midslab from the structural conditions of the original pavements in both locations is presented. The design of the transverse joint detail based on a review of experience with different types of details used in past projects is also shown. Recommendations for initial joint openings are also presented. The design included the selection of the strand spacing to obtain the required prestress level at midslab. With respect to the prestress force, the development of a strategy for applying the post-tensioning is also shown. This strategy was defined basically from consideration of the stresses produced by the frictional restraint when the slabs tend to experience length changes due to temperature variations and early shrinkage. The rationale for the selection of the tendon layout and stressing method is also included. The stressing method was a post-tensioning technique utilizing internal pockets near midslab. A detailed description of the method for constructing the slabs and applying the post-tensioning is presented. A plan for short-term and long-term instrumentation is also included in this report.

The construction part of this report is written to document the problems encountered during the construction phase of the McLennan overlay for future prestressed concrete pavement projects. It may provide the basis for future improvments in construction procedures.

Finally, conclusions are drawn concerning design and construction factors, and recommendations on several aspects of prestressed concrete pavement which require further research are made.

IMPLEMENTATION STATEMENT

Based on theoretical analysis and the experience gained from past prestressed concrete pavement projects, two prestressed overlays to be placed in Cooke and McLennan Counties were designed. The designs and the details of the construction of the McLennan County overlay are reported here. There was no construction in Cooke County.

It is recommended that the experience documented in this report be used later in the development of a manual to be implemented for the design of prestressed concrete pavements. Pavements of this type should be considered for future construction in Texas and other parts of the United States.

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CHAPTER 1. INTRODUCTION

Prestressed concrete pavement is a type of pavement in which internal compressive stresses have been introduced in the concrete to counteract the stresses generated by wheel loads and environmental effects to a certain degree. The resulting pavement requires thinner sections to withstand the traffic loads than other more conventional types of concrete pavements. Also, through early application of the precompression stresses, shrinkage cracks can be minimized, and those that occur are kept tightly closed so that the pavement can be built in longer slabs. This reduces the required number of transverse joints, and a better riding quality can be expected.

Important advantages of prestressed pavement over other types of concrete pavements include the following:

(1) Important savings in materials are obtained because prestressed pavement requires less concrete and reinforcement steel.

(2) Since the number of joints is reduced and the amount of cracking in this type of pavements is generally lower, prestressed pavement requires less routine maintenance.

(3) Improved performance is derived from the smaller number of transverse joints and the lower pavement deterioration with high traffic volume projected for the 20-year design life.

HISTORICAL BACKGROUND

The concept of prestressed pavement originated in Europe over 40 years ago where it found applications in both airfield and highway pavements. Prestressed concrete pavements were investigated first in England in 1943. The first prestressed pavement was constructed at the Paris International Airport at Orly in 1946. Prestressed concrete highway applications began with two short sections built in France in 1946 and 1949. This work was followed by British projects totaling 6000 feet that were constructed between 1950 and 1952.

In the United States, the first applications of prestressed pavements were made during the 1950's in the construction of several experimental airfield pavements. Prestress applications on highways were preceded by the construction and testing of an experimental section built in 1956 at Pittsburgh, Pennsylvania. The first known prestressed concrete highway section in the U.S. was a short pavement slab built in Delaware in 1971. The need for more information on prestressed highway pavements resulted in several test programs which were carried out in various parts of the United States during the 1970's. Tables 1.1 and 1.2 summarize the features of the most recent demonstration programs. Today, a great number of prestressed pavements have been built in various parts of the world in the different types: pretensioned, post-tensioned and post-stressed pavements. However, in the United States, only post-tensioned concrete pavements have been used in highway projects.

OBJECTIVE

In spite of the advantages cited above and the fact that the experimental installations in the U.S. have shown that prestressed pavements generally perform well, this concept has not received widespread acceptance as a viable solution for either new pavements or overlays of existing pavements. This is primarily due to two important problems:

(1) Past technology, materials, and equipment made the construction of this pavement system, with an associated high initial cost, impractical.

(2) There is no acceptable design procedure.

In order to research these problems, the Texas State Department of Highways and Public Transportation (TSDHPT) and the Federal Highway Administration (FHWA) decided to sponsor the construction of two demonstration projects, of approximately one mile each, in the southbound lanes of Interstate 35. One mile was to be placed south of Gainesville, Texas (Cooke County), and the other north of Waco, Texas (McLennan County). The work plan for both demonstration projects consisted of four distinct phases: design, construction, monitoring, and reporting. The design steps, developed by the Center for Transportation Research (CTR) under Projects 555 and 556, encompassed the latest procedures in pavement design and prestressing technology. The development of a rational design manual was

TABLE 1.1. MOST RECENT PRESTRESSED PAVEMENT PROJECTS IN U.S.A.

Date	Project	Length	Thickness
1972	Virginia (Dulles Int'l Airport)	3360 ft total (400-ft to 760-ft sections)	6 inches
1973	Pennsylvania (Harrisburg)	13,333 ft total (600-ft sections)	6 inches
1976	Mississippi (Brookhaven)	25,100 ft total (450- ft sections)	6 inches
1977	Arizona (Tempe)	12,000 ft total (400-ft sections)	6 inches

TABLE 1.2. PAST PROJECT DATA

Variables	Virginia	Pennsylvania	Mississippi	Arizona
Length-Miles	0.6	1.5	2.5	1.2
Year Built	1971	1973	1976	1977
Lanes	2 (One Direction)	4 (Divided)	4 (Divided)	4 (Divided)
Slab Thickness-Inches	6	6	6	6
Subbase	6-inch cement-treated aggregate	6-inch aggregate bituminous base course	4-inch hot mix bituminous concrete	4-inch lean concrete
Subgrade - CBR	3	4	20 (a)	3
Soil Classification	Clay	Silty clay	Sandy clay	Silty clay
Climate	Wet, freeze-thaw	Wet, freeze-thaw	Wet, no freeze-thaw	Dry, no freeze-thaw

(a) Improved subgrade layer on A-2 to A-7 subgrade.

also required of the CTR, under Project 401. The objective of this report is to present the steps of the procedure followed for defining the most relevant design aspects of the projects in Cooke and McLennan Counties.

SCOPE

This report, although descriptive in nature, is not intended to be a design manual. It discusses some of the principles of the design and basically presents the results of the analyses.

On the Cooke County Project, the location of the experimental section was chosen from a set of possible candidate sections. Chapter 2 presents the criteria followed to select the location of the prestressed pavement and two additional miles to be overlaid with 8 and 10-inch–thick continuously reinforced concrete pavement (CRCP), respectively. The CRCP sections were to be used as control sections for direct comparison in performance with the prestressed pavement.

Chapter 3 presents the procedure followed to determine the thickness and prestress level required at the center of the slab from fatigue requirements of the section. These two design variables were chosen from the structural condition of the existing pavement and the traffic expected on the section for a 20-year design period. The particular design for each project is described in this chapter.

Chapter 4 presents a discussion on the slab lengths selected in both projects. This variable was defined from the experience of past experimental projects and a series of considerations of movements produced by temperature variations, concrete shrinkage effects, and creep.

Chapter 5 shows the design of the transverse joint detail. The final choice was made based on the observed performance of joint details of past projects with similar joint spacings. Measured and computed movements from a theoretical analysis were considered in this part of the study. The selection of the longitudinal joint detail is also presented.

Chapter 6 presents the recommendations developed with respect to the post-tensioning forces. First, the fatigue design for thickness and prestress level from Chapter 3 was checked using an elastic approach. Then, the strand spacing needed to provide the necessary prestress level between fatigue and elastic analyses at the slab mid-length was determined. This required the estimation of prestress losses. Finally, since it was decided to apply the total post-tensioning force in two stages, a series of provisions was developed for the initial stage of prestress application, based on the calculations and the analysis of a series of anchorage tests at very early ages.

Chapter 7 presents the basis for selection of the tendon layout. The final tendon layout selected is practical to implement from a construction standpoint and adequate for providing the required prestress level at the center of the slab. The advantages of the system selected are compared to the alternatives for providing the post-tensioning force.

In Chapter 8 a preliminary plan for instrumentation is presented. Two types of measurements are discussed in this chapter: short-term and long-term. Short-term measurements are required for verifying the values of slab movement and concrete stress assumed in the design and to calibrate the design procedure. Long-term measurements are required for monitoring the performance of the pavement during its life. The plan for instrumentation shown in this chapter covers both types of measurements.

Chapter 9 reports important aspects of the construction of the Waco project (McLennan County).

In Chapter 10 a set of conclusions and recommendations derived from the construction of the Waco section is presented.

The developments in the chapters described above are supported with the information presented in three appendices.

Appendix A presents the plan views of the candidate sections studied for locating the experimental and control sections. A series of deflection plots used in the selection of the experimental and control sections is shown in this appendix.

Appendix B contains the supporting information for evaluating the structural capacity of the original pavement at the time of overlay placement. Dynaflect deflections and spectral analysis of surface waves (SASW) were used for evaluation. A mechanistic analysis was based on this information performed in order to define thickness and prestress level.

Appendix C contains information used in Chapter 5 for the design of the joint detail. A series of charts of joint width versus temperature, developed from past prestressed experimental projects, is presented. Computations of expected joint movements for different slab lengths are also shown in this appendix.

CHAPTER 2. SELECTION OF THE LOCATION OF THE PRESTRESSED PAVEMENT AND CONTROL SECTIONS FOR THE COOKE COUNTY PROJECT

This chapter presents the criteria used to select a one-mile section of IH-35 in Cooke County for the experimental prestressed concrete pavement and two additional miles for overlaying with 8 and 10-inch-thick CRCP. The CRCP sections, one mile each, were to be used as control sections for a direct comparison of factors in the experiment that affect the performance of both types of pavements, prestressed and CRCP. There was no construction in Cooke County.

The candidate sections were originally proposed by District 3 of the SDHPT. The locations of the sections are indicated in Table 2.1. The existing pavement structure in most of the sections to be overlaid was ±4 inches of ACP on a 10-inch jointed concrete pavement with a 38-foot width and a 20-foot joint spacing.

This chapter does not describe the selection of the location of the McLennan section because it was defined by the SDHPT.

METHODOLOGY

The factors considered for comparing the candidate sections were horizontal alignment, vertical alignment, and structural condition of the existing pavement. The adequacy of the horizontal alignment was evaluated by counting the number of potential interferences along the section. The vertical alignment was evaluated based on the accumulated change of the vertical profile along the section. The comparison of the structural condition of the pavement sections was based on the analysis of a set of Dynaflect deflections taken in 1980. The deflection readings were taken at the pavement joints and at the mid-spans of the slabs. The magnitudes of the mean deflections along the sections and their standard deviation were used for the comparisons.

The procedure for evaluating and comparing the proposed sections using the factors mentioned above was as follows:

(1) A detailed rating was made of each section's best mile with regard to horizontal alignment. The range of conditions for horizontal alignment was rated as shown below.

TABLE 2.1. LOCATION OF CANDIDATE SECTIONS IN COOKE COUNTY

Candidate Section	Stationing
1	372 - 437
2	469 - 540
3	615 - 680
4	728 - 890
5	895 - 980

TABLE 2.2. SUMMARY TABLE OF RATINGS (RT) FOR BEST MILE OF CANDIDATE SECTIONS

		Ratings							
Section	Best Mile Stationing	RH	RV	RW1	RW5	RSW1	RSW5	RT	Ranking
1	384 - 437	1	1	3	3	1	1	10	5
2	469 - 522	3	5	4	3	2	2	19	3
3	628 - 681	1	4	3	3	5	3	19	2
4	729 - 782	4	4	3	3	4	5	23	1
5	927 - 980	3	3	3	3	2	3	17	4

Straight section with < 2 interfaces	5
Straight section with 3 or more interfaces	4
Curved section with < 2 interfaces	3
Curved section with 3 or more interfaces	2
Any other more critical condition	1

(2) The vertical alignment was rated based on the weighted average of the profile's slopes along the one–mile section. In order to compute the weighted mean each section was divided into a total of n elements. The expression used for determining the weighted mean slope from the slope and the length of each element was

$$\overline{M} = \frac{\sum_{i=1}^{n} (\text{SLOPE X LENGTH})\ i}{5280}$$

The rating for the range of conditions of M was

$\overline{M}$ (Percent)	Rating
0.00 - 0.30	5
0.30 - 0.60	4
0.60 - 0.90	3
0.90 - 1.20	2
> 1.20	1

(3) The supporting characteristics of the sections were evaluated from the mean and standard deviations of the the deflections W1 and W5 recorded by sensors 1 and 5 of the Dynaflect. The ratings developed for mean and standard deviations of the deflections, respectively, are presented below.

Mean Sensor 1 Readings (W1)	Rating
0.00 - 0.20	5
0.20 - 0.40	4
0.40 - 0.60	3
0.60 - 0.80	2
> 0.80	1

Mean Sensor 5 Readings (W5)	Rating
0.00 - 0.05	5
0.05 - 0.10	4
0.10 - 0.15	3
0.15 - 0.20	2
> 0.20	1

Standard Deviation of Sensor 1 Readings (SW1)	Rating
< 0.090	5
0.090 - 0.100	4
0.100 - 0.110	3
0.110 - 0.120	2
> 0.080	1

Standard Deviation of Sensor 5 Readings (SW5)	Rating
< 0.050	5
0.050 - 0.060	4
0.060 - 0.070	3
0.070 - 0.080	2
> 0.080	1

(4) The section that had the highest accumulated rating and showed additional advantages in terms of distress condition, time of construction, geographical location, etc. was selected as the optimal for placement of the prestressed overlay.

(5) The next two lower rated sections were selected as control sections for placement of the 8 and 10-inch CRCP overlays. A special consideration was made to select those sections closer to the prestressed section.

RESULTS

According to the methodology presented earlier, the following results were obtained:

(1) The best mile for each candidate section was located between the following stations:

Section	Stationing of Best Mile
1	384 - 437
2	469 - 522
3	628 - 681
4	729 - 782
5	927 - 980

(2) The ratings (RH) developed for these one-mile sections with respect to horizontal alignment were

Section	Rating (RH)
1	1
2	3
3	1
4	4
5	3

(3) The following ratings (RV) were developed for vertical alignment:

Section	$\overline{M}$ (percent)	Rating (RV)
1	1.23	1
2	0.28	5
3	0.58	4
4	0.43	4
5	0.72	3

(4) The ratings for the mean and standard deviations of the Dynaflect deflections were

	Mean Deflections		Ratings	
Section	W1	W5	Rw1	Rw5
1	0.58	0.22	3	3
2	0.40	0.27	4	3
3	0.43	0.24	3	3
4	0.50	0.29	3	3
5	0.46	0.24	3	3

	Standard Deviations		Ratings	
Section	SW1	SW5	Rw1	Rw5
1	0.1860	0.0812	1	1
2	0.1200	0.0742	2	2
3	0.0860	0.0700	5	3
4	0.0990	0.0460	4	5
5	0.1112	0.0630	2	3

(5) The summation of ratings, RT, and the final ranking of the sections are presented in Table 2.2.

Section 4 was selected for placement of the prestressed concrete overlay for the following reasons:

(a) It obtained the highest RT.

(b) It also showed a small amount of distress during past condition surveys.

(c) The distress was distributed uniformly along the length of the section.

(d) The direction of overlay construction, from south to north, made Section 4 one of the first to be overlaid.

(6) Sections 3 and 5 were selected as control sections for placing the CRCP for the following reasons:

(a) They showed very high ratings (RT).

(b) The existing pavements presented adequate structural conditions.

(c) Sections 3 and 5 were inmediately north and south, respectively, of Section 4.

(7) Since Section 3 presents smaller values of W1 and W5 (0.429 and 0.239, respectively) than Section 5 (0.46 and 0.241), it was recommended that Section 3 be overlaid with the 8-inch-thick CRCP and Section 5 with the 10-inch-thick CRCP.

Appendix A contains the plan sheets showing the horizontal and vertical alignments for the one-mile elements in each candidate section and a series of plots of the Dynaflect readings W1, W5, and W1-W5.

CHAPTER 3. FATIGUE DESIGN OF THICKNESS AND PRESTRESS LEVEL

This chapter presents the procedures followed to determine the thickness and prestress level necessary to meet the fatigue requirements of the prestressed sections in Cooke and McLennan Counties. The problem was to select a feasible design which would provide good service over the design period. The system consisted of analyzing different alternative solutions.

DESIGN METHODOLOGY

The design approach followed was based on the methodology proposed by Seeds and McCullough to determine the life of a specific overlay design strategy (Ref 1). It was based on the application of multi-layered elastic theory to three different structural stages presented in the pavement during its entire service life. These stages are illustrated graphically in Fig 3.1. The deterioration of the pavement with time is shown in terms of the remaining life of the fundamental layers: the original JCP and the prestressed concrete overlay. Figure 3.2 presents a general block diagram of the design system showing the steps that correspond to each structural stage. A detailed explanation of each structural stage is presented later in this report.

The design procedure consisted of the following steps.

Base Information

Before generating alternative design solutions, some initial base information was needed. The following items were defined for this purpose.

Layer Properties. Different methods for non-destructive evaluation of the layer properties can be used. Most of them are based on measurements of the response of a pavement structure to an external force or energy input and the results of these measurements are then related in some way to the structural properties of the pavement section. These measurements are referred to as "non-destructive" because the structure of the pavement is not altered by the measurements and such measurements can be repeated at the same location as often as

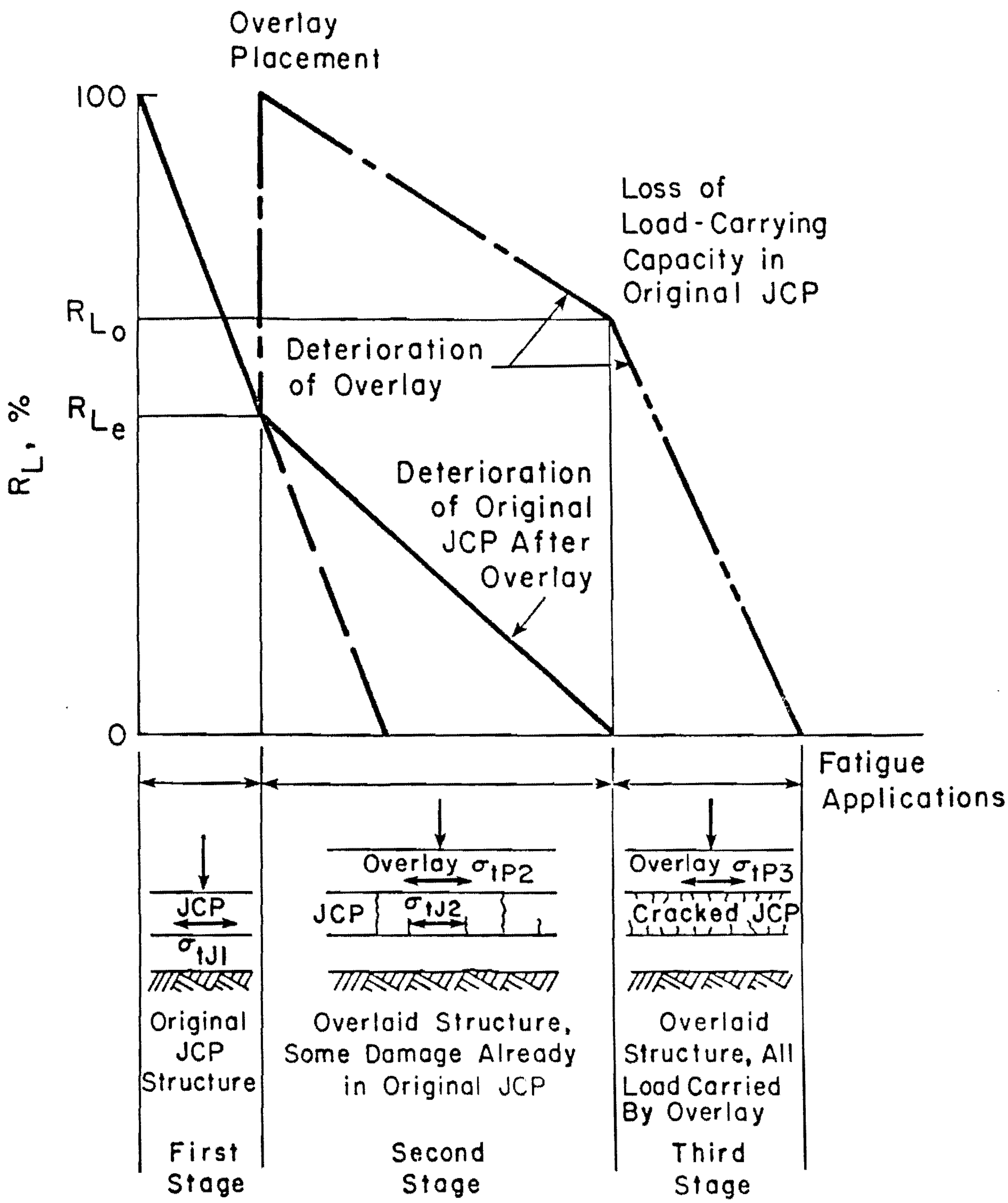


Fig 3.1. Structural stages of the pavement during its entire service life, after Seeds and McCullough (Ref 1).

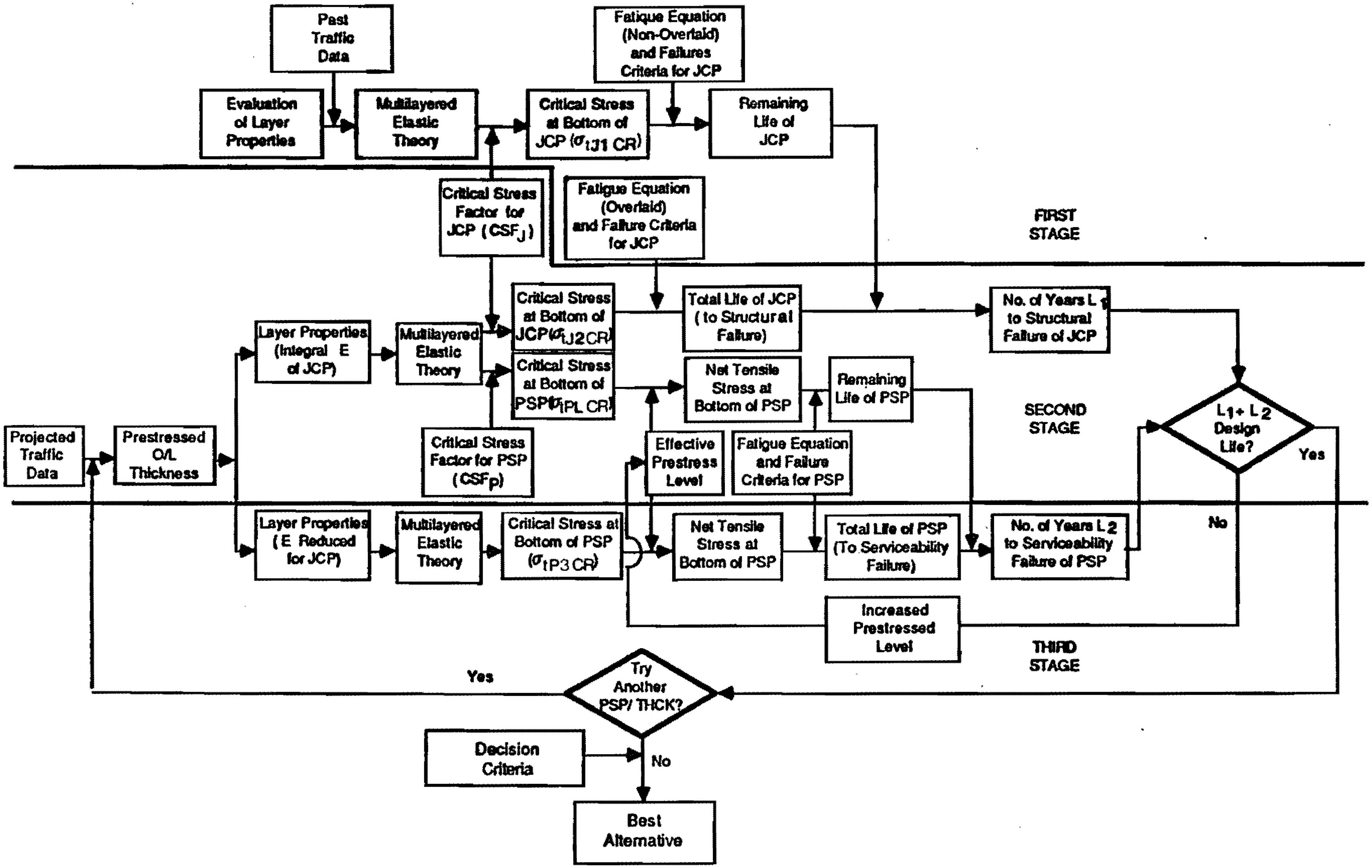


Fig 3.2. Block diagram for determining thickness and prestress level.

necessary. Highly recommended are the non-destructive testing methods that use responses to a repeated or dynamic load.

Critical Stress Factors. The stresses computed by layered theory are assumed to be interior stresses. However, the design of JCP and prestressed pavements (PSCP) is normally based on corner stresses, which are more critical than interior loading stress conditions. Stress adjustment factors to convert interior stresses into edge or corner stresses were developed by Treybig, McCullough, et al (Ref 2) using discrete element theory as well as Westergaard and Picket theory. Two different stress factors were defined for use along with this design procedure. If the critical stress at the bottom of the JCP before overlay placement was desired, the condition of the structure will be the one presented in Fig 3.3(a). The critical stress factor should be defined based on interior as well as corner deflections taken on the existing pavement. Figure 3.4 should be used for this purpose. This critical stress factor is noted throughout this report as CSF_J.

Once the overlay has been placed, since the locations of the JCP joints generally do not coincide with the location of the PSCP joints, the stress factor has to be changed. If the critical stress at the bottom of the PSCP at the PSCP joints is desired, the condition of the structure will be the one presented in Fig 3.3(b). The critical stress factor recommended for this condition was 1.3. This critical stress factor is referred to throughout this report as CSF_P.

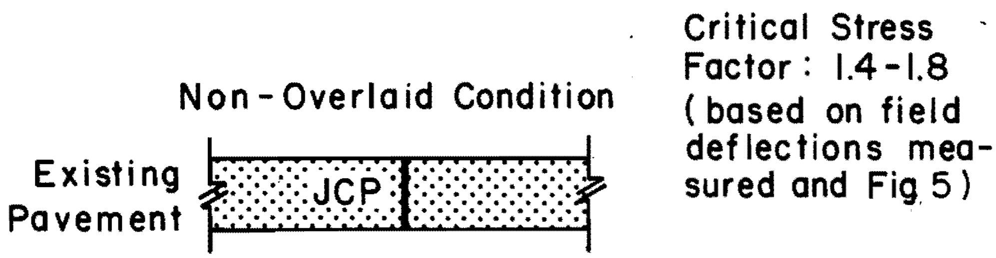
Design Period. The design period is the service life of the prestressed section from the time of overlay placement to the time of overlay failure.

Traffic Data. Traffic data on the section should be acquired for two different periods: (a) past traffic from year zero to the time of overlay placement and (b) projected traffic during the design period.

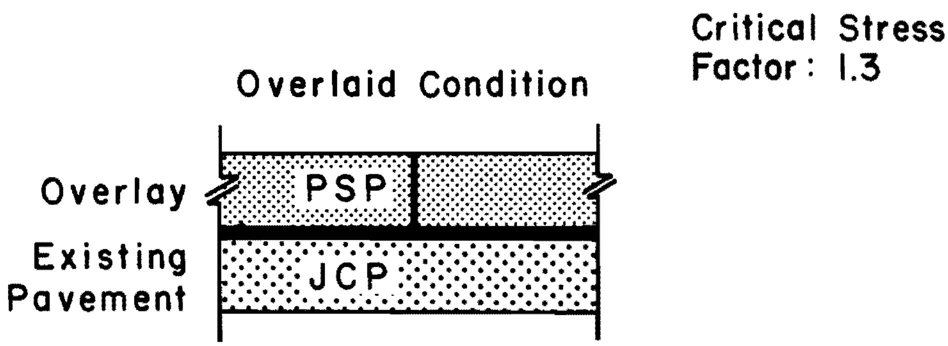
Analysis of Stages

The next step in the design procedure was the analysis of structural stages. From the first stage, the remaining life of the original JCP was determined. Alternative design solutions were generated from the two other stages.

Remaining Life of Original JCP.- First Stage. The layer properties of the original structure and the past traffic data were used to determine the structural remaining life of the JCP at the time of overlay placement. Elastic layered theory was used to determine the tensile



(a) Critical stress factor CSF_J for stress at the bottom of JCP, at the JCP joints.



(b) Critical stress factor CSF_P for stress at the bottom of PSP, at the PSP joints.

Fig 3.3. Stress adjustment factors selected to convert interior stresses to stresses for use in design.

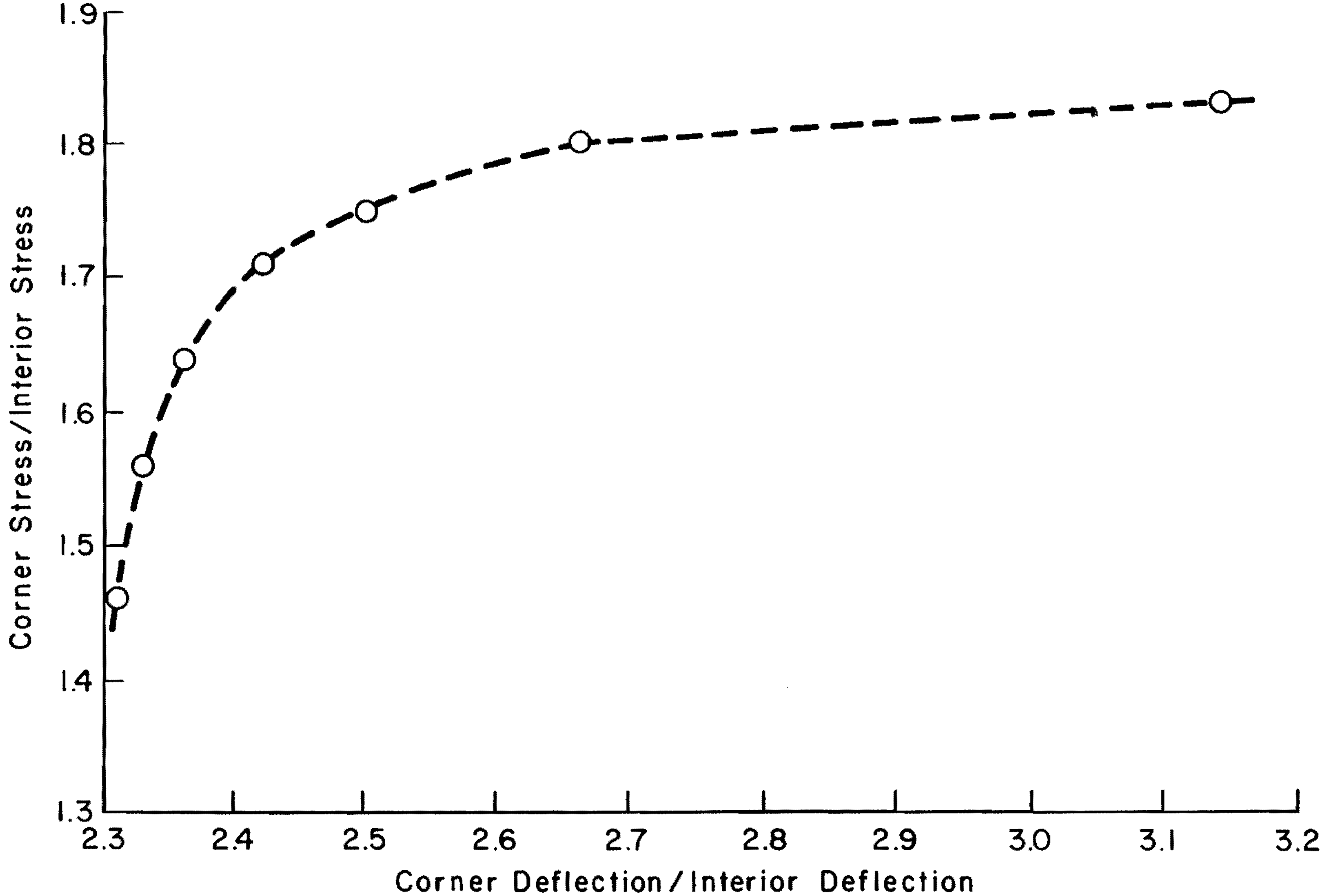


Fig 3.4. Stress ratio curve for relating interior to corner stresses for a given deflection ratio.

stress generated at the bottom of the JCP under a standard 18-kip ESAL. Then, depending on the ratio of corner to interior deflections of the existing JCP, the critical stress factor CSF_J was selected from Fig 3.4. This factor was used to approximate the tensile stress on the JCP for the critical loading condition near the pavement corners. Once the critical stress on the JCP was obtained, a fatigue equation for non-overlaid PCC pavements was used to determine the number of 18-kip axle repetitions the pavement could sustain before reaching a specific failure criterion. Failure was viewed in this stage as a structural failure and not as a serviceability failure. The original JCP was considered to have structurally failed when it lost its load carrying capacity by excessive cracking.

Generation of Design Alternatives

Second Stage. The second stage of the pavement design started when the prestressed overlay was placed and traffic was allowed on the section. A lower stress level is generated on the JCP by the application of wheel loads. Thus, the rate of deterioration of the structural capacity of the original JCP decreases. Also, the prestressed overlay starts deteriorating during this stage due to the development of tensile stresses at the bottom. This is where the generation of alternatives starts. An initial overlay thickness must be input in the procedure and the new stress level on the JCP must be determined by multi-layered elastic theory, assuming that this layer keeps its modulus of elasticity as evaluated earlier. The tensile stress at the bottom of the prestressed pavement must also be obtained.

The stresses in the JCP should be converted into critical stresses by applying the stress factor CSF_J. This critical stress should be used with a fatigue equation for overlaid PCC pavements, the structural failure criterion, and the remaining life of the JCP, as obtained from the first stage, to determine the number of years L_1 to structural failure of the JCP.

A different critical stress factor should be defined for the PSCP, since the location of the JCP joints generally will not coincide with the location of the joints of the PSCP. The stress on the PSCP should also be converted into critical stress by applying the stress factor CSF_P. Then, an iterative procedure starts by assuming an initial effective prestress level. This prestress level should be subtracted from the critical stress on the PSCP to obtain a net tensile stress on the PSCP. The net stress is then used in combination with a fatigue equation of the PSCP and the conventional serviceability failure concept to determine the remaining life of the PSCP at the time of structural failure of the original JCP.

Third Stage. During this stage, the pavement shows its last structural condition. The JCP has already failed and its original modulus of elasticity needs to be reduced to represent its cracked condition. Once the modulus of the JCP has been reduced, the overlaid pavement is analyzed by multi-layered elastic theory to determine the tensile stress at the bottom of the prestressed overlay. This stress is then converted into critical stress through the stress factor CSF_p and reduced to net stress by subtracting the effective prestressing. The net stress is then input in the fatigue equation of the PSCP. Then, by applying the serviceability failure criterion and the remaining life of the PSCP, the number of years L_2 to serviceability failure of the PSCP is obtained. If the sum of L_2 and L_1 from stage 2 does not equal the design life required for the overlay, then the prestress level must be modified. Changes in prestress level directly affect L_2, whereas changes in thickness affect both L_1 and L_2.

Once a thickness is selected, L_1 is automatically fixed and the required L_2 to match the design life can be obtained by making adjustments s on the prestress level.

If a different alternative is desired, another thickness has to be input and all computations starting from stage 2 have to be repeated in order to obtain the prestress level required to match the overlay design life.

Selection of a Suitable Design Alternative

After a series of alternatives were developed, the last step in the design procedure consisted of using a set of decision criteria to select the best alternative.

COOKE COUNTY

The experimental section in Cooke County was located on southbound IH-35 between stations 728 + 10 and 781 + 50. The existing pavement structure consisted of ±4 inches of ACP on a 10-inch jointed concrete pavement 38 feet wide with a 20-foot joint spacing. The proposed project included removing the existing ACP, patching the jointed pavement where necessary, sealing, and overlaying with approximately 2 inches of new ACP prior to placing the prestressed concrete pavement. Figure 3.5 presents a sectional view of the existing pavement structure.

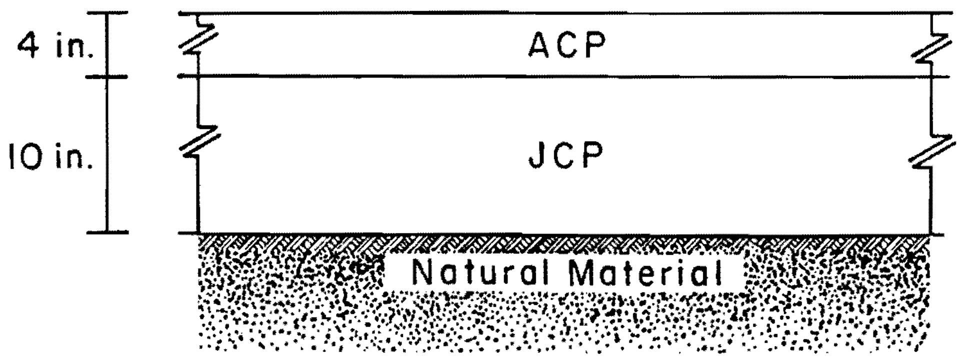


Fig 3.5. Typical section in Cooke county.

The JCP section in Cooke County was opened to traffic for the first time in 1949. Eventually, some parts of the section had to be overlaid with asphaltic concrete for general improvement of ride quality, pavement performance, and skid resistance. The number of 18-kip ESAL accumulated since the time of construction and the projected repetitions for the next 20 years are presented in Table 3.1.

Base Information

According to the design procedure outlined earlier, the following initial base information was obtained:

Layer Properties. Layer properties were estimated for the Cooke County section through two different evaluation methods: (a) evaluation of the elastic moduli of the layers from spectral analysis of surface waves (SASW) and (b) back calculation of the elastic properties of the layers from the deflections measured with the Dynaflect. The theoretical bases of and evaluation procedures for both methods are presented in Refs 3 and 4, respectively. Here, only a brief introduction and a comparison of the results obtained with both methods are presented.

Spectral Analysis of Surface Waves Method (SASW). In this method, a transient vertical impulse is applied to the surface of the pavement and a group of surface waves with different frequencies is generated in the medium. These waves propagate along the surface with velocities which vary with frequency and the soil stiffness profile. Propagation of the waves is monitored with two vertical geophones a known distance apart on the pavement surface. Results from analysis of the phase information from the cross power spectrum are used with the spacing between the geophones to determine the moduli of each layer, as well as layer thicknesses.

The typical structure in Cooke County is shown in Fig 3.5. The SASW test was performed in three different locations along the one-mile section: station 742 + 23, station 760 + 02, and station 776 + 00. The plots of Young's modulus varying with depth are presented in Figs B.1, B.2, and B.3 (in Appendix B) for the three locations analyzed. The values of Young's modulus for the two layers as well as the average for the three stations are reported in Table 3.2.

Back Calculation of Elastic Properties from Dynaflect Deflections. The process to back calculate the elastic properties of the layers is trial-and-error in nature. Initially, a set of

TABLE 3.1. TRAFFIC DATA FOR THE EXPERIMENTAL SECTION IN COOKE COUNTY

Period	Total Number of 18-kip Equivalent Single Axle Load Applications (one direction)
1949-1984	17,236,000
1984-2004	45,047,000

TABLE 3.2. ELASTIC MODULI DETERMINED BY SASW

Station	E_1 (psi)	E_2 (psi)
742 + 23	3,000,000	25,000
760 + 02	3,200,000	37,000
776 + 00	3,000,000	29,000
Average	3,000,000	27,000

Dynaflect deflections, W_1 through W_5, are obtained for the site where the properties are required . Then, reasonable values of Young's modulus and Poisson's ratio were assigned to each layer and are used along with the corresponding thicknesses into a multi-layered elastic theory program. The load applied to the structure must be the standard Dynaflect peak dynamic force. The desired output is an array of computed surface deflections for the relative positions of the five Dynaflect sensors. If the computed deflections did not match the measured values, a new set of moduli was assigned and the deflections recomputed. The iterative procedure was continued until a "best" fit to the measured deflection basin was achieved.

For the one-mile experimental section in Cooke County, Dynaflect deflections were provided by the SDHPT. A plot of the deflection profile is included in Appendix B as Fig B.4. Back calculation was performed on every point reported along the section and the cumulative frequency distributions of the elastic moduli of the two layers were determined and plotted. The plots are presented in Appendix B, in Figs B.5 and B.6. The values of Young's modulus for the two layers corresponding to 50 percent in the cumulative distribution are presented in Table 3.3. Table 3.4 shows the moduli obtained by the SDHPT by back calculation from the deflection basins of the stations closest to where the SASW tests were performed.

The results reported in Tables 3.2, 3.3, and 3.4 from both methods showed fairly similar values. The moduli from the back calculation method were lower than the ones obtained from the SASW method. The reason for this difference is that, although the SASW method is a very precise evaluation of the elastic properties of the pavement, it is performed under non-loading conditions. On the other hand, for the back calculation method, some fundamental conditions, such as cracking, that affect the behavioral response of the pavement under wheel loads directly affect the deflection readings under dynamic load sensed by the Dynaflect geophones. Therefore, the moduli obtained from back calculation resulted in lower values.

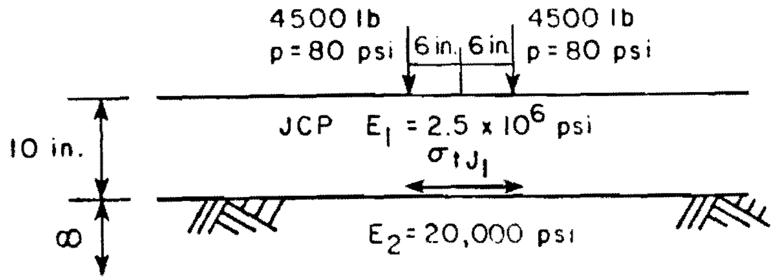
For design purposes, the elastic moduli of the layers were selected from the cumulative frequency distributions in Figs B.5 and B.6 derived from back calculation, since they result in more conservative values and take into account the variability of the material properties along the section. The design values were selected based on confidence levels. To design on the conservative side, the 10 and 90 percent values were chosen, as convenient, for each one of the layers. The values assigned to the moduli depended on the location where the tensile stresses were desired and the structural stage of the pavement. Figure 3.6 shows the four

TABLE 3.3. FIFTY PERCENT VALUES ON CUMULATIVE FREQUENCY DISTRIBUTIONS FOR E_1 AND E_2 FROM BACK CALCULATIONS

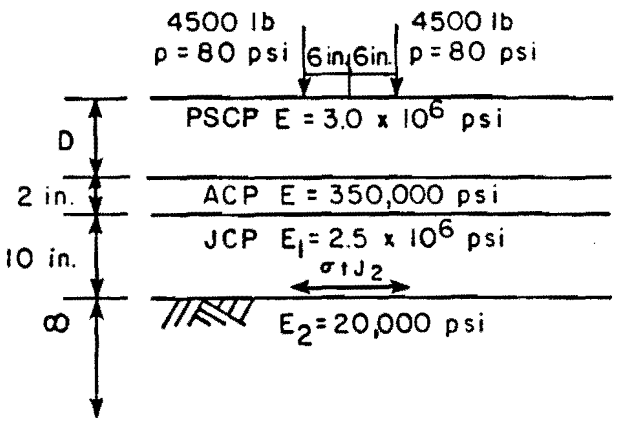
Percent	E_1 (psi)	E_2 (psi)
50	1,800,000	25,000

TABLE 3.4. ELASTIC MODULI DETERMINED BY BACK CALCULATION

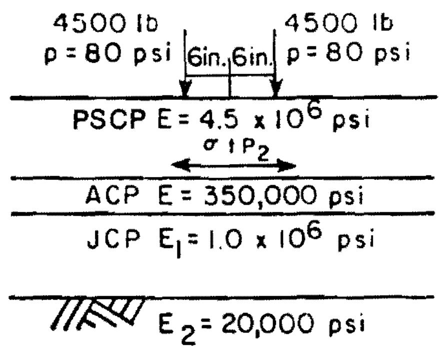
Station	E_1 (psi)	E_2 (psi)
742 + 65	3,000,000	23,000
760 + 00	1,500,000	21,000
776 + 80	1,500,000	23,000
Average	2,000,000	22,000



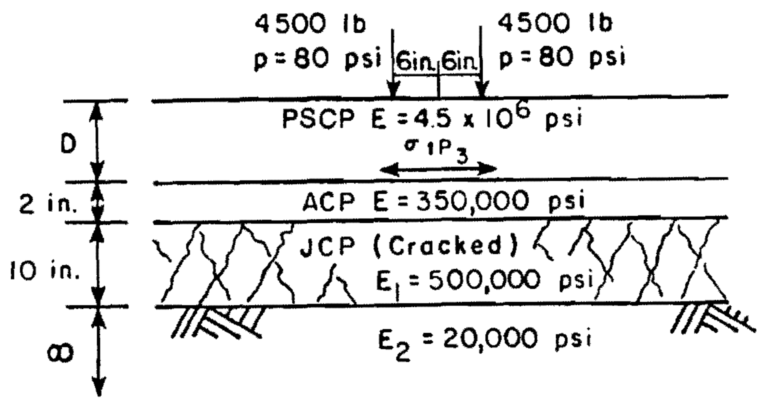
(a) Stress on JCP first stage.



(b) Sress on JCP second stage.



(c) Stress on PSCP second stage.



(d) Stress on PSCP third stage.

Fig 3.6. Models to be analyzed by multi-layered elastic theory to determine thickness and effective prestress level.

models required to be analyzed by multi-layered elastic theory for the design. The moduli selected and the locations where the stresses were desired for each structural stage are shown.

Critical Stress Factors. The critical stress factors selected for the design of the Cooke section were

$$CSF_J = 1.4$$

and

$$CSF_P = 1.3 \tag{3.1}$$

Design Period. Twenty years was selected as the design period for the prestressed section.

Traffic Data. The traffic data for the section as reported by the SDHPT were presented in Table 3.1. Since most of the heavy traffic is carried by the inside lane, it is generally the "design" lane. The lane distribution factor that defines the percentage of the 18-kip ESAL traffic carried by the design lane was assumed to be 0.9 for this two-lane facility. The final traffic data used in the design are presented in Table 3.5.

Analysis of Stages

First Stage - Remaining Life of Original JCP. The tensile stress σ_{tJ1} at the bottom of the JCP was computed by running the elastic layer program ELSYM5 (Ref 5) on the model presented in Fig 3.6(a):

$$\sigma_{tJ1} = 85 \text{ psi} \tag{3.2}$$

Then, the critical stress at the bottom of the JCP was determined by applying the stress factor CSF_J:

$$\sigma_{tJ1\,CR} = 119 \text{ psi} \tag{3.3}$$

The fatigue equation used was developed by Taute (Ref 6) for non-overlaid PCC pavements from AASHO Road Test data and data from statewide condition surveys in Texas:

$$N_{18} = 46{,}000 \left(f/\sigma_{t\,CR} \right)^{3.00} \tag{3.4}$$

where

N_{18} = number of 18-kip ESAL to serviceability failure or approximately 5 percent of the total area of the JCP cracked,

f = concrete flexural strength, assumed to be 700 psi at 28 days, according to the minimum value required by the SDHPT construction standard specifications (Ref 7), and

$\sigma_{t\,CR}$ = critical tensile stress on the PCC pavement.

Therefore,

$$N_{5\%} = N_{18} = 9.36 \times 10^{6} \text{ 18-k ESAL} \tag{3.5}$$

In order to extrapolate $N_{5\%}$ to any other percent of the total area of the original JCP cracked, the probabilistic concepts developed by Darter and Hudson for the design of flexible pavement systems (Ref 8) were applied. The distribution of fatigue cracking was assumed to be log-normally distributed with standard deviation equal to 1.698.

Therefore,

$$N_{\alpha\%} = 10^{0.23} \left(Z_{\alpha}\% = Z_{5\%} \right) N_{5\%} \tag{3.6}$$

where

$$0.23 = \log 1.698$$

TABLE 3.5. TRAFFIC ON DESIGN LANE

Period	Total Number of 18-k Equivalent Single Axle Load Applications in Design Lane
1949-1984	15,500,000
1984-2004	40,500,000

$Z_{\alpha\%}$ and $Z_{5\%}$ = standard normal variate for α and 5 percent, respectively.

Figure 3.7 was developed from the equation above to check the accuracy of the approach followed here. The theoretical curve percent area cracked versus N_{18} was plotted, as was the point representing the condition of the JCP in 1984.

A cracking index equal to 18.2 feet per 1,000 square feet was obtained from a condition survey conducted on the section after 15.5 x 10^6 18-kip ESAL repetitions. Since the 1984 point lies below the theoretical curve, this approach looks rather conservative but is sufficiently accurate.

Structural failure of the JCP was considered to be reached when 90 percent of the total area of the pavement cracked. Then, $N_{90\%}$ was computed:

$$N_{90\%} = 44.06 \times 10^6 \text{ 18-K ESAL} \qquad (3.7)$$

Finally, the remaining life of the JCP, R_L, was obtained:

$$R_L = \left(1 - \frac{N_{1984}}{N_{90\%}}\right) \times 100$$

$$R_L = \left(1 - \frac{15.50 \times 10^6}{14.40 \times 10^6}\right) \times 100$$

$$R_L = 65\% \qquad (3.8)$$

Generation of Design Alternatives. The design alternatives were generated in this section following the principles presented in the block diagram of Fig 3.8 but using a graphical procedure.

Second Stage. The tensile stress σ_{tJ2} at the bottom of the JCP once the overlay had been placed was determined by running the elastic layer program BISAR (Ref 9) on the model presented in Fig 3.6(b). BISAR was used in the analysis, because, unlike ELSYM5, it allows for varying the amount of slip at the interface between the PSP and the ACP. ELSYM5 restrains the analysis by considering full friction at all interfaces. The tensile stress σ_{tJ2}

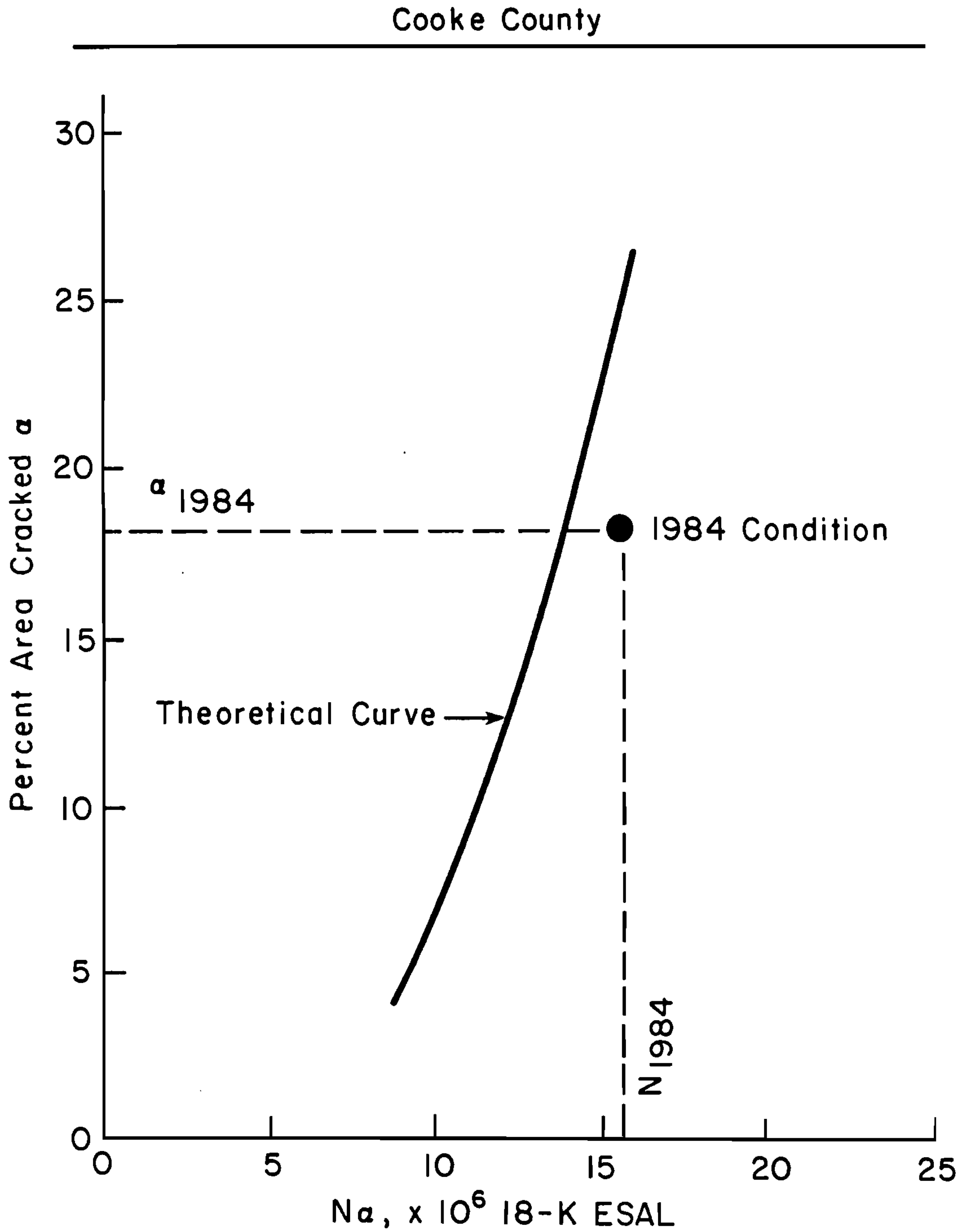


Fig 3.7. Comparison of theoretical approach and actual condition of the JCP section in 1984.

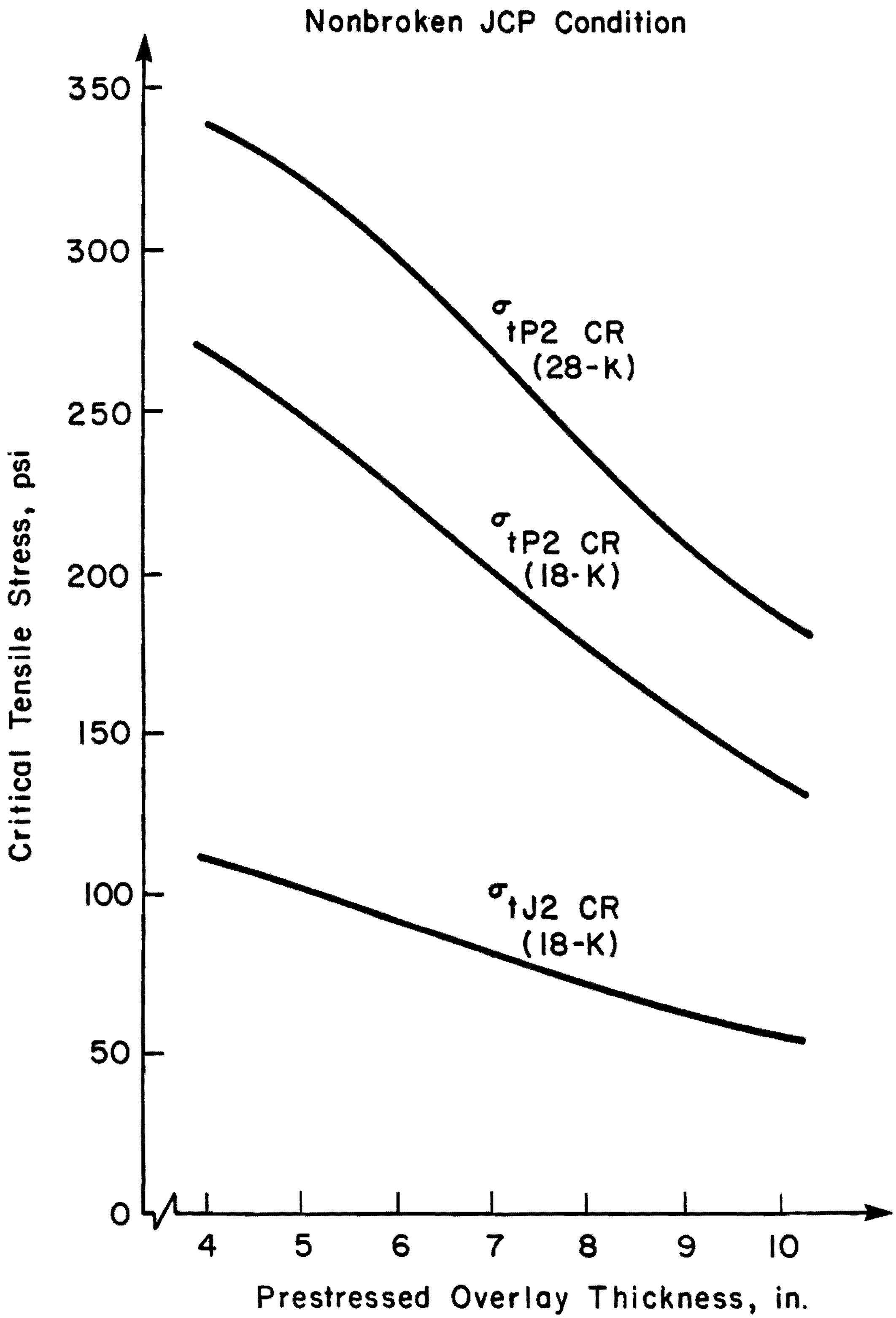


Fig 3.8. Critical stresses versus prestressed overlay thickness for the nonbroken JCP condition.

was computed for a range of overlay thicknesses varying from 4 to 10 inches. Then, the stresses computed were converted into critical stresses by applying the stress factor CSF_J = 1.4.

The tensile stress σ_{tP2} at the bottom of the overlay was determined by running BISAR on the model presented in Fig 3.6(c). σ_{tP2} was computed for the same range of overlay thicknesses and converted into the critical stress, $\sigma_{tP2\ CR}$. The stress factor CSF_p = 1.3 was used for this purpose.

Table B.1 in Appendix B shows the critical stress $\sigma_{tJ2\ CR}$ computed for varying overlay thicknesses and Fig 3.8 is the corresponding plot.

The fatigue equation used in this stage was also developed by Taute (Ref 6) for overlaid PCC pavements:

$$N_{18} = 43{,}000\left(f/\sigma_{t\,CR}\right)^{3.2} \tag{3.9}$$

From this equation, the numbers of repetitions, N_{L1}, to structural failure of the JCP were determined for overlay thicknesses ranging from 4 to 10 inches. Table B.2 shows the intermediate steps followed in the computation of N_{L1}. Figure 3.9 is a plot of overlay thickness, D, versus number of repetitions, N_{L1}. From Fig 3.9 it can be observed that, for the range of thicknesses analyzed, the projected traffic for the design period will never be high enough to cause structural failure of the JCP. Therefore, the design procedure should be reduced to the analysis of stages one and two, since the cracked condition will never appear on the JCP. In other words, the design alternatives can be generated directly from stage 2 by working with the critical stresses $\sigma_{tP2\ CR}$.

Different prestress levels were imposed on the critical stresses $\sigma_{tP2\ CR}$ presented in Table B.1. Then, by applying the fatigue equation for non-overlaid PCC pavements, the plot shown in Fig 3.10 was developed. By superimposing the projected design repetitions, the alternatives presented in Table 3.6 were obtained.

Selection of a Suitable Design Alternative

Alternative 1 was selected as the most suitable alternative since 100 psi effective prestress is an acceptable prestress level in an 8-inch PCC slab.

TABLE 3.6. SET OF GENERATED ALTERNATIVES

Alternative	Overlay Thickness (inch)	Prestress Level (psi)
1	8	100
2	9	75
3	10	55

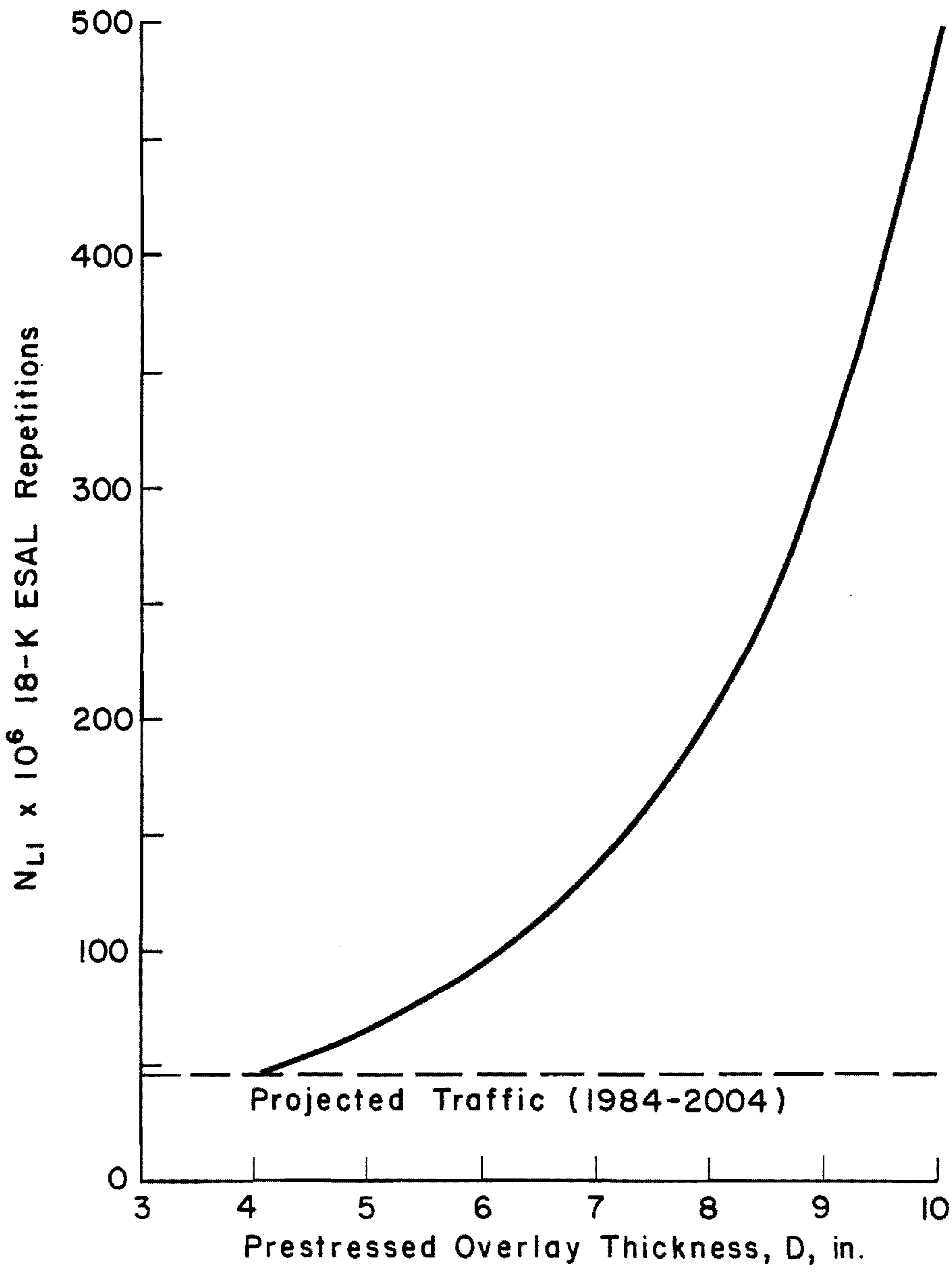


Fig 3.9. Prestressed overlay thickness D versus number of repetitions N_{L1} to structural failure of JCP.

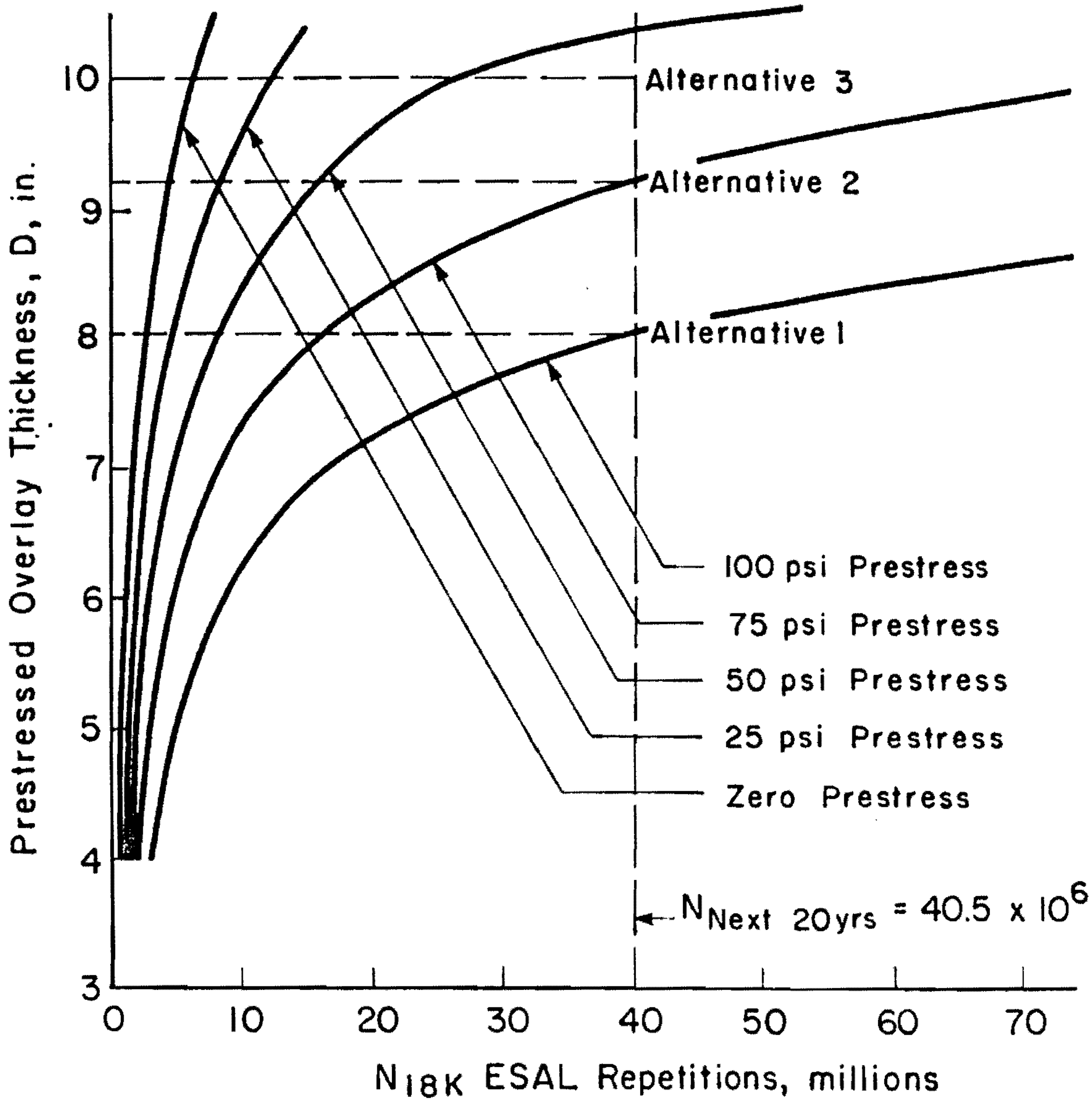


Fig 3.10. Number of 18-k ESAL repetitions N_{18} versus prestressed overlay thickness.

Cross Section

Figures 3.11(a) and 3.11(b) show two cross sections corresponding to the existing JCP pavement, one for the non-overlaid sections and the other for the sections that eventually required an ACP overlay.

Figure 3.11(c) shows the cross section of the proposed prestressed overlay as defined from the analysis presented earlier. The old ACP should be removed and 2 inches of new ACP placed on top of the existing JCP all the way along the entire experimental section. A friction reducing membrane should placed between the new ACP and the PSCP overlay. The longitudinal joints of the PSCP and the existing JCP should be displaced with respect to each other to avoid a plane of weakness on the longitudinal direction.

McLENNAN COUNTY

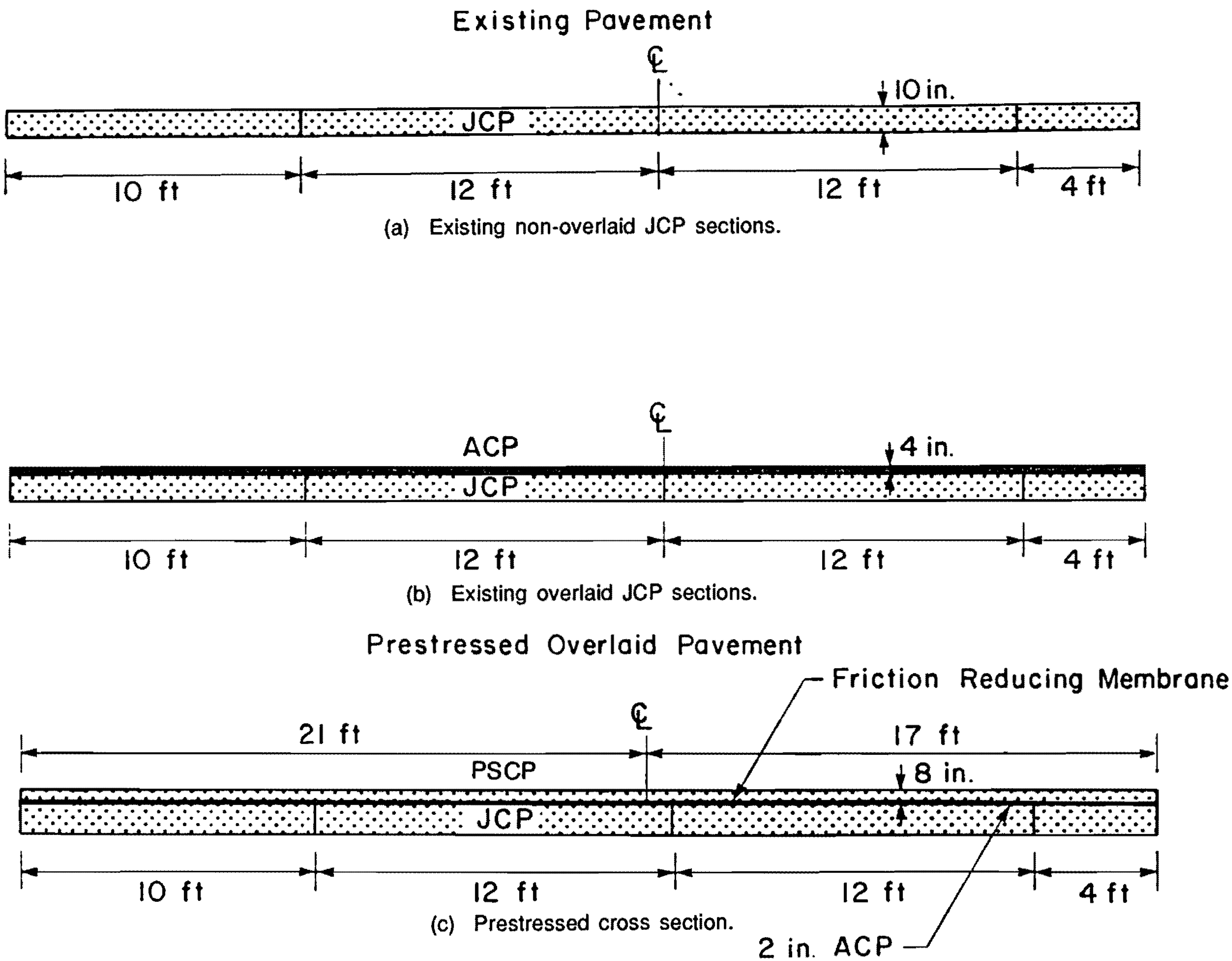
The section in McLennan County, like the section in Cooke County, was located on southbound IH-35, between stations 696 + 00 and 749 + 00. The present pavement structure was 4 inches of existing ACP on a 12-inch jointed concrete pavement 24 feet wide, with shoulders on each side, as shown in Fig 3.16. The project consisted of removing the ACP, patching the jointed pavement where necessary, sealing and overlaying with approximately 2 inches of new ACP, and placing the prestressed concrete pavement on top. Figure 3.12 shows a sectional view of the existing pavement structure.

The JCP section in McLennan County was opened to traffic for the first time in 1952. Like the section in Cooke County, this section had to be overlaid with asphaltic concrete for general improvement of ride quality, pavement performance, and skid resistance. Table 3.7 presents the performance and skid resistance. Table 3.7 presents the number of 18-kip ESAL accumulated from the time of construction until 1984 and the projected repetitions for the next 20 years. These data were provided by the SDHPT.

TABLE 3.7. TRAFFIC DATA FOR THE EXPERIMENTAL SECTION IN McLENNAN COUNTY

Period	Total Number of 18-k Equivalent Single Axle Load Applications (one direction)
1952-1984	23,855,000
1985-2004	52,606,000

RR555/556-1/03



(a) Existing non-overlaid JCP sections.

(b) Existing overlaid JCP sections.

(c) Prestressed cross section.

Fig 3.11. Cross sections of existing pavement and prestressed overlay.

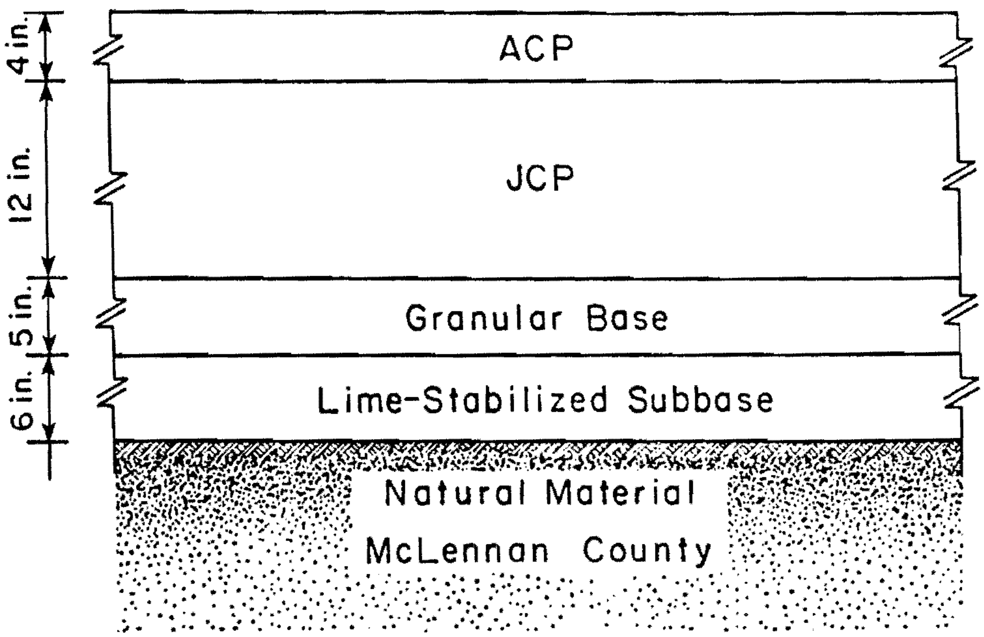


Fig 3.12. Typical section in McLennan county.

Base Information

For the McLennan section, the following initial information was obtained.

Layer Properties. Layer properties were estimated for the McLennan section through back calculation from the deflection basins measured with the Dynaflect. A set of Dynaflect deflections was provided by the SDHPT for the one-mile experimental section. A plot of the deflection profile is shown in Fig B.7, in Appendix B. Back calculation was performed on every point reported along the section and the cumulative frequency distributions of the elastic moduli of the five layers were plotted. The distributions are shown in Figs B.8 through B.12. As for the design of the Cooke County section, the design values were selected from the plots based on confidence levels. The 10 and 90 percent values where chosen, as convenient, for each one of the layers, depending on the location where the tensile stresses were desired and the structural stage of the pavement. Figure 3.13 shows the four models required to be analyzed by multi-layered elastic theory. The moduli selected are shown, as is the location where the stresses are desired for each structural stage.

Critical Stress Factors. The critical stress factors selected for the design of the McLennan County section were the same as the ones used for the design of the Cooke County section.

Design Period. The prestressed section in McLennan County was designed for a twenty-year design period.

Traffic Data. The design traffic carried by the design lane is presented in Table 3.8. A lane distribution factor of 0.9 was assumed for this two-lane facility.

ANALYSIS OF STAGES

First Stage - Remaining Life of Original JCP

The tensile stress σ_{tJ1} at the bottom of the JCP was computed by running the elastic layer program ELSYM5 (Ref 5) on the model presented in Fig 3.12(a):

$$\sigma_{tJ1} = 72 \text{ psi} \tag{3.10}$$

TABLE 3.8. TRAFFIC ON DESIGN LANE FOR THE SECTION IN McLENNAN COUNTY

Period	Total Number of 18-k Equivalent Single Axle Load Applications in Design Lane
1952-1984	21,469,500
1984-2004	47,345,400

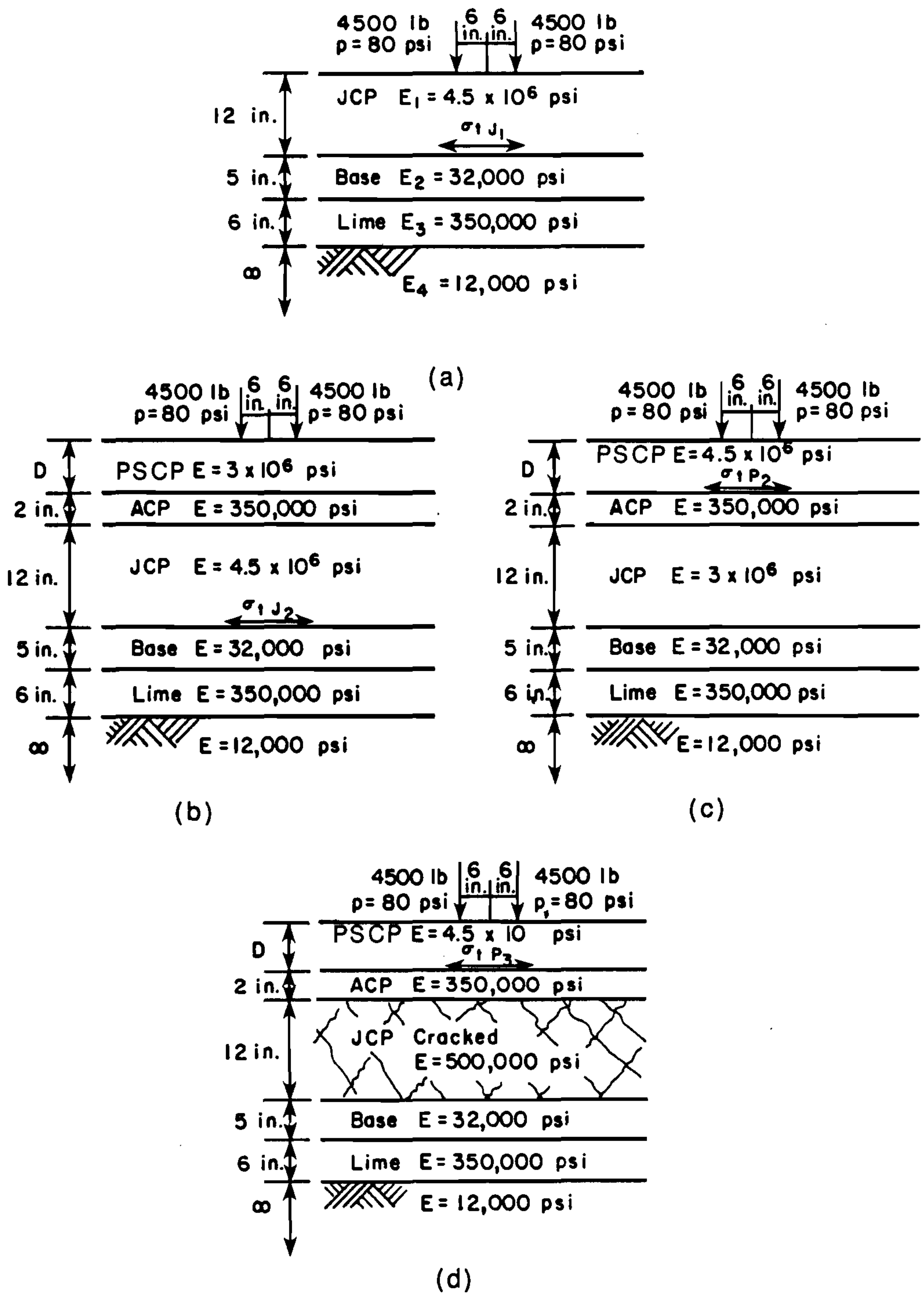


Fig 3.13. Models to be analyzed by multi-layered elastic theory to determine thickness and effective prestress level.

Then, the critical stress at the bottom of the JCP was determined by applying the stress factor CSF_J :

$$\sigma_{tJ1} = 100.8 \text{ psi} \tag{3.11}$$

The remaining life of the existing pavement was computed following the steps used for the Cooke County section. Therefore,

$$N_{5\%} = N_{18} = 9.36 \times 10^6 \text{ 18-k ESAL}$$

$$N_{90\%} = 72.50 \times 10^6 \text{ 18-k ESAL} \tag{3.12}$$

Finally, the remaining life of the JCP, R_L, was obtained:

$$R_L = \left(1 - \frac{N_{1984}}{N_{90\%}}\right) \times 100$$

$$R_L = \left(1 - \frac{21.47 \times 10^6}{72.50 \times 10^6}\right) \times 100$$

$$R_L = 70.4\% \tag{3.13}$$

It is important to indicate at this point that the existing structure in McLennan County was comprised of more layers of better quality material than the section in Cooke County. Therefore, the remaining structural life of the existing pavement in McLennan County was computed to be larger than the remaining life of the existing rigid pavement in Cooke County.

Generation of Design Alternatives

Given that the existing pavement at McLennan County showed a high structural remaining life and based on the design of the Cooke County section where it was observed that the existing JCP would never reach the broken condition during the design period of the overlay, it was decided to go directly to the second stage of the design methodology to generate the design alternatives.

Second Stage

The tensile stress σ_{tP2} at the bottom of the overlay was determined by running BISAR on the model presented in Fig 3.12(c). σ_{tP2} was computed for a range of overlay thicknesses varying from 4 to 10 inches and converted into the critical stress $\sigma_{tP2\,CR}$. The stress factor CSF_P = 1.3 was used for this purpose.

Table B.3, in Appendix B, shows the critical stresses σ_{tP2} and $\sigma_{tP2\,CR}$ for different overlay thicknesses and Fig 3.14 is the corresponding plot.

Different prestress levels were imposed on the critical stress $\sigma_{tP2\,CR}$ presented in Table B.3. Then, by applying the fatigue equation for non-overlaid PCC pavements, the plot shown in Fig. 3.15 was developed. By superimposing the projected design repetitions, the alternatives presented in Table 3.9 were obtained.

Selection of a Suitable Design Alternative

Alternative 1 was selected as the most suitable alternative since 65-psi effective prestress is an acceptable prestress level in a 6-inch PCC slab. Furthermore, as shown in Table 1.2, most PSP projects conducted in the U. S. have used thicknesses of approximately 6 inches.

Cross Section

Figures 3.16(a) and 3.16(b) show two cross sections corresponding to the existing JCP pavement, one for the non-overlaid sections and the other for the sections that eventually required an ACP overlay.

TABLE 3.9. SET OF GENERATED ALTERNATIVES

Alternative	Overlay Thickness (inch)	Prestress Level (psi)
1	6	65
2	7	55
3	8	45

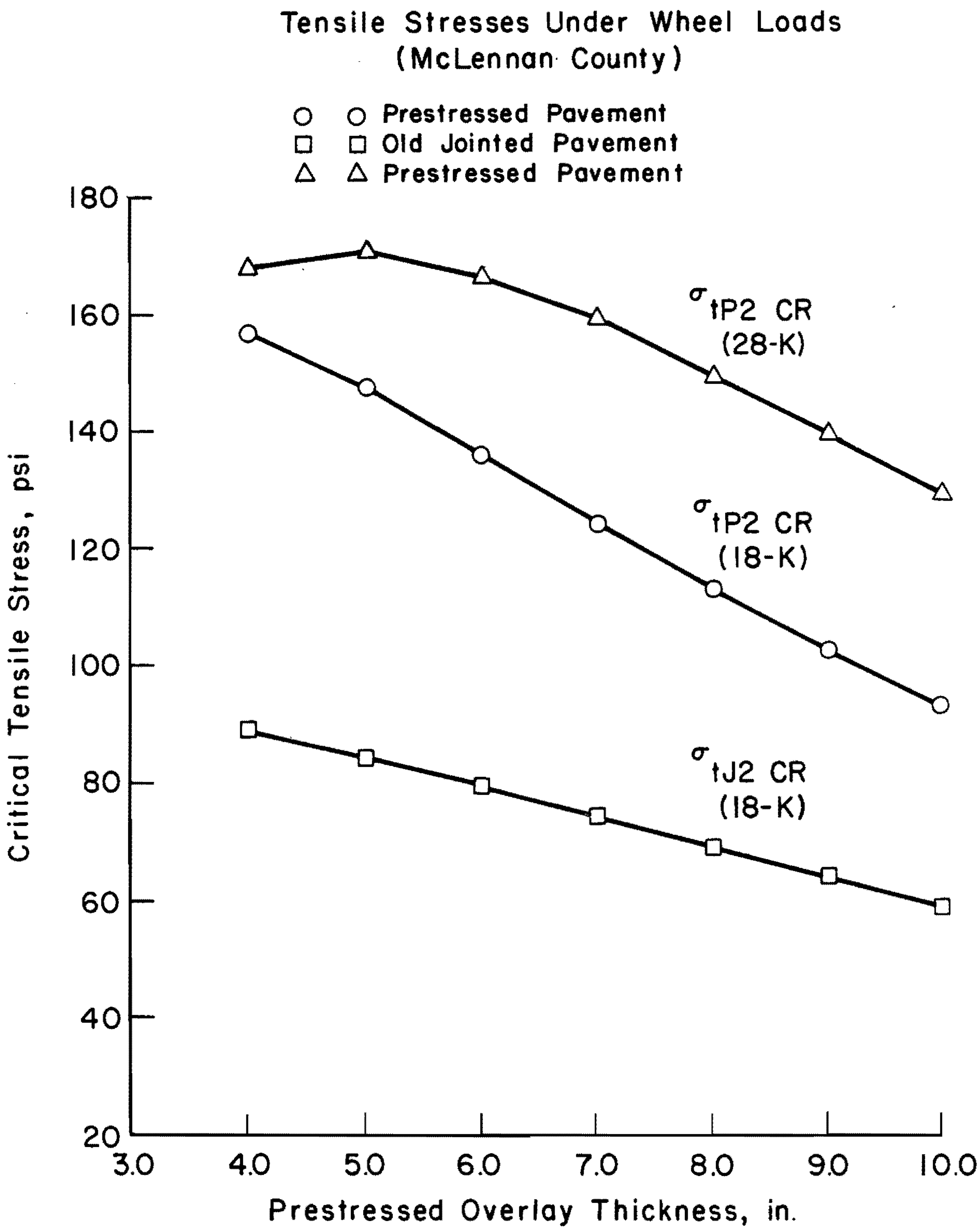


Fig 3.14. Critical stress versus prestressed overlay thickness for the non-broken JCP condition.

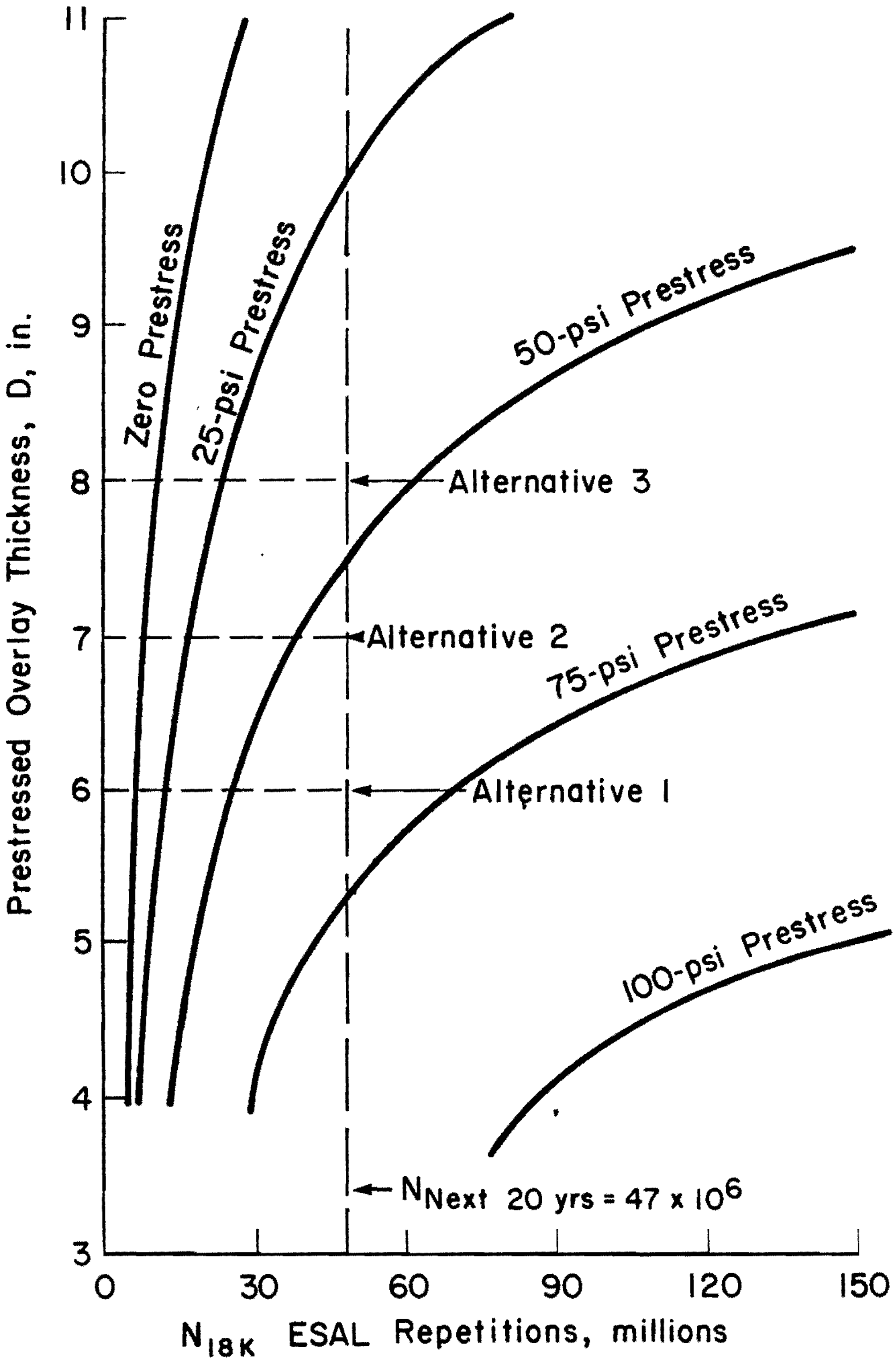
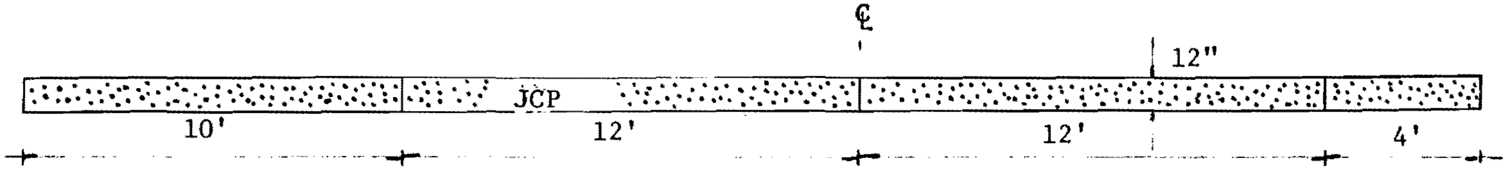
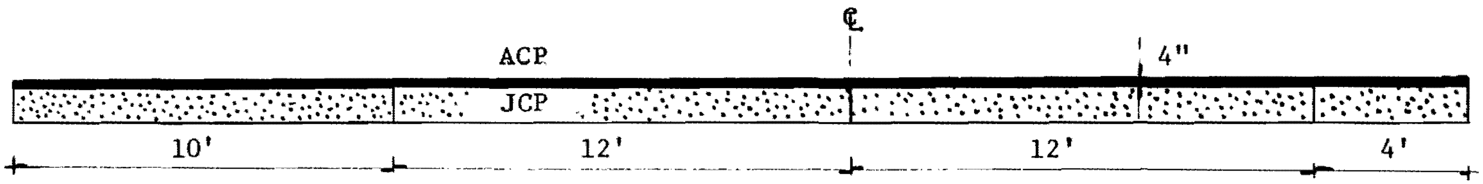


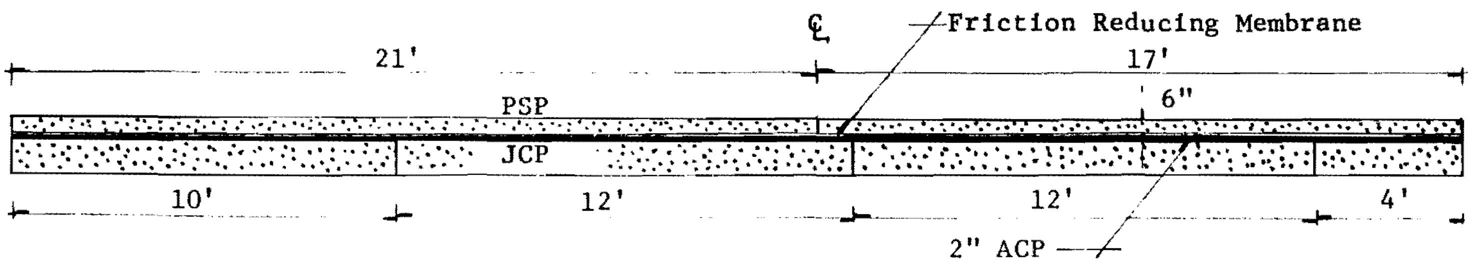
Fig 3.15. Number of 18k ESAL repetitions N_{18}versus prestressed overlay thickness.



(a) Existing non-overlaid JCP sections.



(b) Existing overlaid JCP sections.



(c) Prestressed cross section.

Fig 3.16. Cross sections of existing pavement and prestressed overlay.

Figure 3.16(c) shows the cross section of the prestressed overlay as defined from the analysis presented earlier. The old ACP was removed and 2 inches of new ACP placed on top of the existing JCP along the entire experimental section. A friction reducing membrane was placed between the new ACP and the PSCP overlay. The longitudinal joints of the PSCP and the existing JCP were displaced with respect to each other to avoid a plane of weakness in the longitudinal direction.

CHAPTER 4. SELECTION OF SLAB LENGTH

This chapter is a discussion of the suitability of the selected lengths of the prestressed overlays. The development shown herein is applicable to both projects since slab length, which is mainly dependent on expected joint movements, is practically unaffected by slab thickness. Likewise, climatological conditions, which are relevant parameters, are similar in both sites and do not cause significant differences in the magnitude of the joint openings.

Slab length is a function of several variables, including expected joint width, prestress applied at the slab ends, subgrade friction restraint, and the desired minimum prestress for the midlength location. However, the governing criterion for selecting the length of PSCP slabs is maximum joint width. The joints should not open under extreme conditions more than 4 inches unless joints containing spanner plates are used. These joints are uneconomical to construct for good performance assurance. If wider joint openings are allowed, problems may arise with the riding quality of the roadway and a hazard to traffic may result. Also, the seals may be damaged and eventually pull out of the joints.

SOME ADVANTAGES AND DISADVANTAGES OF LONG SLABS

Several advantages are derived from the use of long slabs. Some of these include:

(1) Longer slabs result in fewer joints, which are a substantial cost item in pavement construction requiring periodic maintenance.

(2) The reduction in the number of joints, which are a major cause of distress and eventual failure of rigid pavements, results in an improved pavement performance .

Some of the disadvantages of longer slabs are:

(1) The joints of longer slabs experience wider openings and generally have more potential for deteriorating under wheel load repetitions.

(2) Longer slabs require higher prestress forces at the slab ends to overcome the frictional resistance and obtain a given prestress level at the center. Higher

end forces require the use of more steel and related anchoring hardware, which may, in turn, make the design uneconomical and difficult to build. Hence, the longer slabs of prestressed pavements necessarily require the use of frictional relieving materials. For the Texas projects, the use of a single layer of 6-mil polyethylene sheeting between the slab and the asphalt base was recommended. The selection of friction relieving material was based on the results of a series of push-off tests on slabs cast over different materials. These test were conducted near Valley View, Texas, by the staff of CTR Project 401 (Ref 10).

EFFECT OF PRESTRESSING TECHNIQUE

Considering that joints are a substantial cost in pavement construction, are expensive to maintain, and cause a loss in pavement serviceability, the philosophy in prestressed pavements is to reduce the number of joints by constructing very long slabs. The limiting factor in selecting slab length is, then, maximum joint opening. In connection with maximum joint opening, the selection of prestressing technique and type of joint plays an important role. The slabs may be prestressed in short gaps left between the slabs, from 6 to 8 feet long, or in blockouts (stressing pockets) at the center, which are filled with concrete after the prestress force has been applied.

If the stressing is done in gaps, the gaps are commonly left open for some period of time to permit most of the progressive length changes to occur, including elastic shortening, shrinkage, and creep, before being filled with concrete. Therefore, the use of gap slabs has the advantage in that there is less increase in width after the gap concrete is placed and, generally, longer slabs can be obtained. Most recent projects in the United States (Refs 13, 14, 15, and 16) have had reinforced gap slabs with active joints at each gap slab end. Each joint experiences only the movement of the prestressed slab end, since, practically, the gap slab does not move. Configurations of gap slabs and a more detailed discussion on them are provided in Chapters 5 and 7.

Since poor gap slab performance (i.e., warping, curling, rocking, etc.) has been observed on some previous projects, one of the objectives of Project 401 was to develop a prestressing technique which would eliminate gap slabs. The most promising alternative was central stressing. This stressing technique is described in detail in Chapter 7.

With central stressing, the joint openings result from the movement of two prestressed slab ends rather than one prestressed slab and the gap slab. Therefore, the slabs should be shorter for central stressing than when gap slabs are used. This does not imply that central stressing results in a higher number of joints, since gap slabs, in turn, result in two consecutive joints for each long prestressed slab.

MOVEMENTS

Predicting the slab end movements is a critical step not only in the choice of slab length but also in the design of the joint detail. Uncertainty in several of the critical variables, such as friction coefficient between slab and subgrade, thermal coefficient, shrinkage, and creep coefficients, makes predictions of movements by calculation difficult. For this reason, slab end movements are predicted using measured movements from previous project reports as well as the calculated values.

Measured Movements

Slab end movement data were available for projects in Virginia, Arizona, and Mississippi (Refs 14, 15, and 16). The available measured slab movements versus temperature were plotted on graphs, which may be found in Appendix C. The mean slope of the line and the standard deviation were computed for each group of data. The design slope was taken as 85 percent of the data, or 1.04 standard deviations above the mean. Then each line was plotted on a single graph (see Fig 4.1). Since the gap slabs were typically poured about one month after pavement slabs, some data had to be adjusted for first month shrinkage, creep, and elastic shortening. Mississippi data included first month movements, but Arizona and Virginia data did not. Adjusted movements may be found in Table 4.1

The accuracy of the measured movements is uncertain. Mississippi, for instance, plotted movement as a function of air temperature although concrete temperature would have been more accurate. The Virginia report noted that some measurements were in error due to weather conditions, bumping of the instruments while straw was removed from the slab, and a

TABLE 4.1. SLAB END MOVEMENTS AND JOINT OPENING FOR $\Delta T = 100°F$

Project	Length (feet)	Air or Concrete Temperature	Slab End Movement (inch)	Joint Movement 2 Slabs (inch)	Adjusted Joint Movement* (inch)
Arizona	400	Concrete	0.76	1.52	2.40
Virginia	400	Concrete	1.18	2.36	3.24
	500		1.55	3.10	4.20
	600		1.73	3.46	4.78
	760		1.79	3.58	5.25
Mississippi	450	Air	2.27	4.54	4.54

*Adjusted for first month shrinkage, creep, and elastic shortening.

Note: Values taken as average movements.

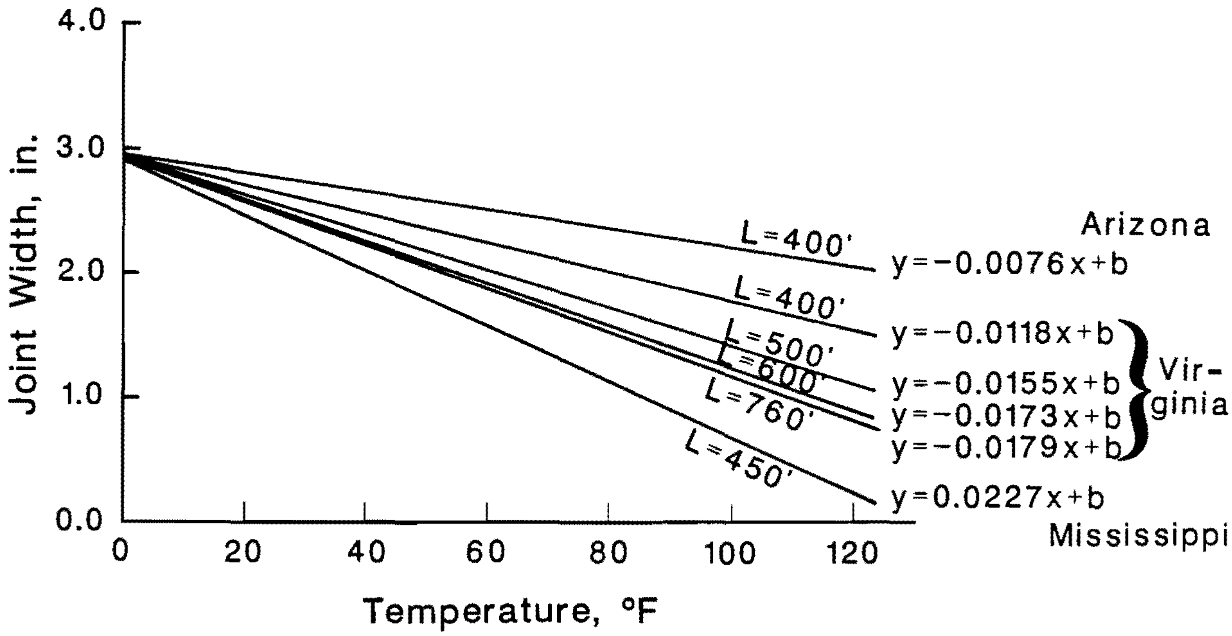


Fig 4.1. Joint width versus temperature.

series of power failures. These measured movements were combined with calculated movements to obtain the best possible prediction of joint movements.

The Mississippi project movements appeared to be quite a bit higher than the others, whereas the Arizona project reported movements lower than the normal. If Mississippi first month permanent shortenings (shrinkage, creep, and elastic shortening) were included, and measurements were expected to be high. Even though these factors may have some bearing on large movements, reports of average seasonal movements and daily movements were high for Mississippi also. Limited data for Arizona could be the reason for these low movements.

Calculated Movements

Several factors must be considered in calculating slab end movements. There is a seasonal variation in length due to the average seasonal temperature change. There is also a daily variation in length as well as permanent shortening of the slab. These three factors are illustrated in Fig 4.2. The total slab end movement may be calculated by summation of these effects, and the maximum joint opening will be a function of this total movement and the initial joint setting.

Six equations are used to calculate (1) seasonal length change, (2) summer daily change, (3) winter daily change, (4) shrinkage of concrete, (5) creep of concrete, and (6) elastic shortening of concrete (Ref 12). Each of these equations is listed below and calculations for specific lengths of slabs may be found in Appendix C.

Seasonal Movement

$$d_1 = \alpha(\Delta t)L \tag{4.1}$$

where

α = coefficient of thermal expansion,

Δt = seasonal variation in average concrete temperature, and

L = slab length.

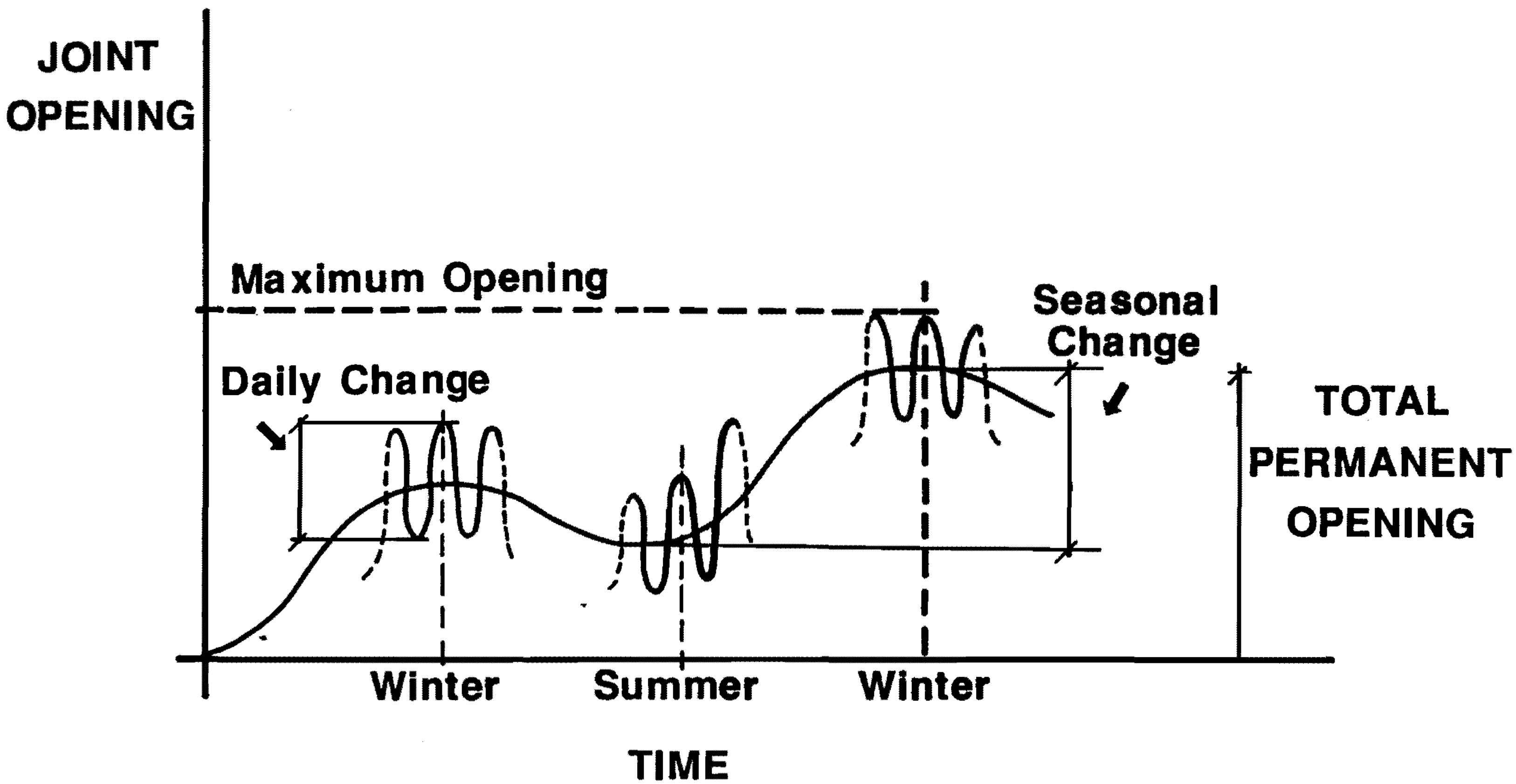


Fig 4.2. Joint openings versus time for daily, seasonal, and permanent effects.

Summer Daily Movement

$$d_2 = \left(\Delta t_s\right)L - d_f \tag{4.2}$$

where

Δt_S = summer maximum temperature less summer average temperature,

and

d_f = slab movement restrained by subbase friction.

Winter Daily Movement

$$d_3 = 0.85 \left(\Delta t_w\right)L - d_f \tag{4.3}$$

where

Δt_W = average winter temperature less minimum winter temperature.

Shrinkage

$$d_4 = \varepsilon_s L \tag{4.4}$$

where

ε_S = concrete shrinkage strain.

Creep

$$d_5 = \varepsilon_k L \qquad (4.5)$$

where

ε_k = concrete creep strain

$$= C_u \frac{f_{av}}{E_c}$$

C_u = utlimate creep coefficient
f_{av} = average prestress along the length
E_c = modulus of elasticity for concrete.

Elastic Shortening

$$d_6 = \frac{E_{ci}}{f_{av} L} \qquad (4.6)$$

where

E_{ci} = modulus of elasticity of concrete when load is applied.

SLAB LENGTHS

Two slab lengths were proposed for the overlays in Cooke and McLennan Counties: 240 and 440 feet. Two lengths were selected for comparison of performance of joints and overlays. For the proposed lengths, the predicted movements were 2.30 and 4.20 inches, respectively. An expected movement of 4.20 inches for the 440-foot-long slab made this length somewhat marginal. However, it was considered that calculated movements were slightly conservative and the actual joint openings would be less than anticipated. Joint openings were also computed, by the method outlined above, for 400, 500, and 600-foot-long slabs. Computations may be found in Appendix C. Figure 4.3 shows maximum joint movements for a 100°F change in temperature versus slab length for calculated and measured movements in Mississippi, Arizona, and Virginia. Table 4.2 shows a comparison of calculated and measured values.

Both slab lengths were verified by calculations to insure that the required prestress level would be provided at the midlengths of the slabs with reasonable strand spacings and concomitant concrete stresses. Report 3 of ACI Committee 325 (Ref 24) suggests that the prestress at the slab ends should not be higher than 650 psi, to avoid overstressing the concrete at the anchor zone. These design conditions are checked in Chapter 5.

The location of the McLennan project section is shown in Fig 4.4. The arrangement of nine 240-foot-long slabs and seven 440-foot-long slabs in the southbound lanes of IH-35 is presented. Prestressing was provided in longitudinal and transverse directions. In both directions, the prestress was given by stressing the tendons in internal pockets. The tendon layout for achieving efficiently the prestress levels is discussed in Chapter 7. The strands for the transverse prestress needed to be continuous on adjacent slabs across the longitudinal joint. This transverse prestress would keep the longitudinal joint closed.

TABLE 4.2. CALCULATED MOVEMENTS COMPARED TO MEASURED MOVEMENTS FOR EXISTING PROJECTS

Project	Length (feet)	Calculated ΔL (inch)	Measured ΔL (inch)	Percent Different
Virginia	400	3.45	3.24	6.1
	500	4.32	4.20	2.8
	600	5.18	4.78	7.7
Arizona	400	3.45	2.40	30.4
Mississippi	450	3.80	4.54	14.1

*Calculated Value Unconservative

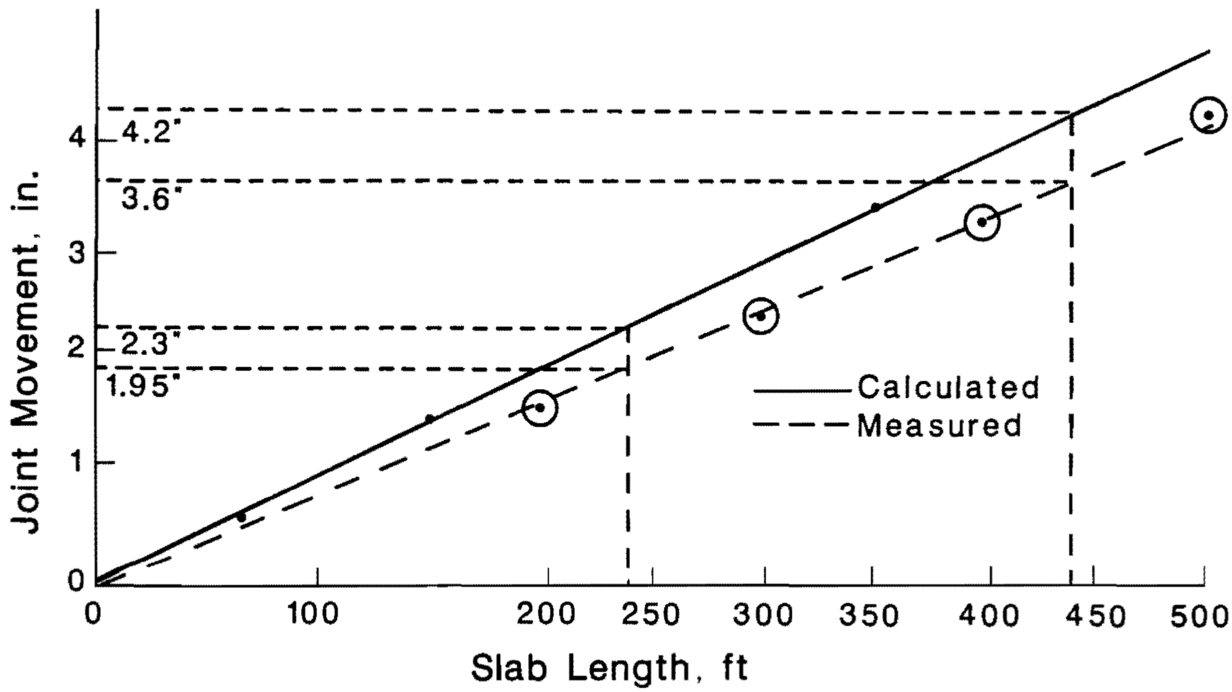
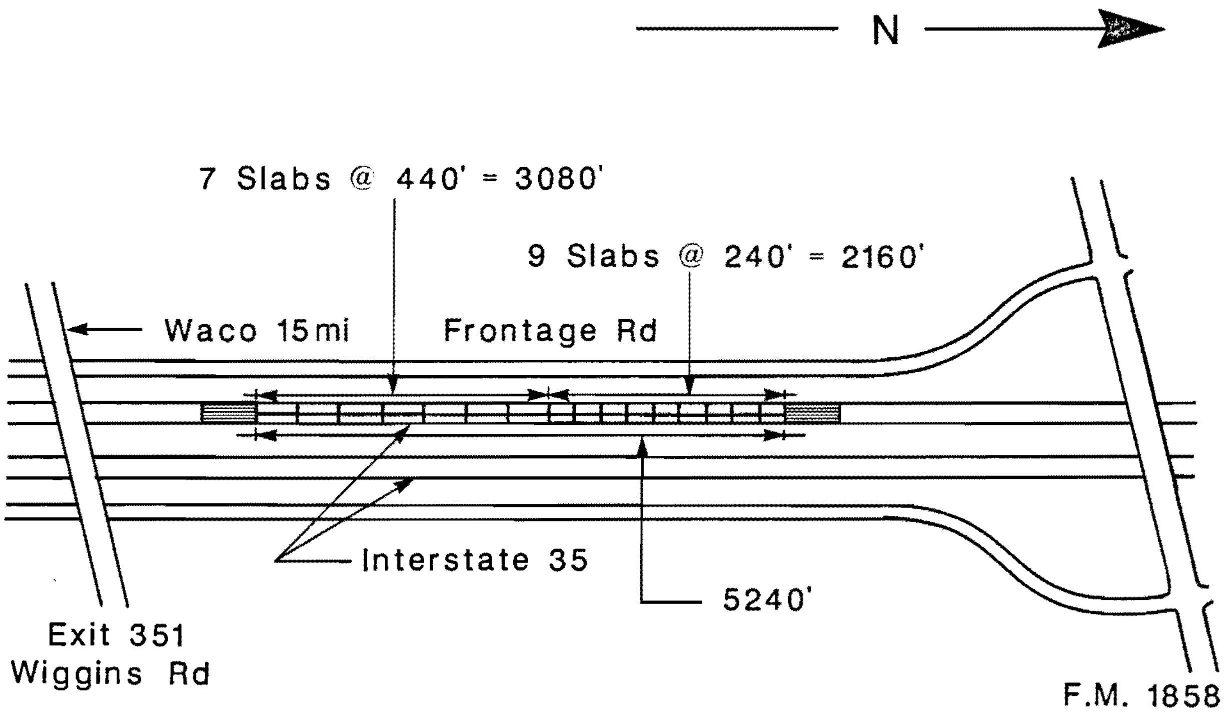


Fig 4.3. Joint movement versus slab length (shrinkage, creep, elastic shortening) ΔT = 100°F.



Total of 32 Prestressed Pavement Slabs:

9 Prestressed Slabs 240'x21'
7 Prestressed Slabs 440'x21'
9 Prestressed Slabs 240'x17'
7 Prestressed Slabs 440'x17'

Fig 4.4. Location of prestressed slabs along southbound lanes of IH-35.

CHAPTER 5. DESIGN OF TRANSVERSE AND LONGITUDINAL JOINT DETAILS

Joint design is an integral part of prestressed pavement design. This chapter includes a description of the selection process of transverse and longitudinal joint details for the prestressed overlays.

For the transverse joint, proper attention should be given to joint hardware design because a large number of items such as anchors, strands, load transfer devices, infiltration prevention devices, positioning bars, and special reinforcements should be located within a few inches of the joint. Detailing of the joint hardware is, therefore, the critical element in this case.

For the longitudinal joint, it is necessary to find a technique that permits the stressing of transverse strands with a minimum of losses. The strands extend continuously on adjacent lanes across the joint. The section lanes are constructed independently, with an average 15 to 20-day period between construction of adjacent slabs. This can create differential movements between the slabs, in the longitudinal direction along the longitudinal joints, which may require special considerations. If the strands continue across the joint, the force in the transverse tendons will keep the joint closed. In addition, the strands will tend to work like dowels at the joint, thus introducing a beneficial factor for load transfer.

TRANSVERSE JOINT DETAIL

Of the four most recent prestressed concrete pavement projects in the United States, all had 8-foot-long gap slabs and three of the four had active joints at each end of the gap slab (see Fig 5.1). In the Pennsylvania project the prestress was transferred from the PSCP slabs to the gap slabs, resulting in gap slabs with only one active joint (Ref 12). The elminimation of the gap slabs in the Texas projects meant that joint openings would result from the movement of two slabs, as in the Pennsylvania project, rather than from only one slab, as in the Virginia, Mississippi, and Arizona projects.

Estimating the total amount of joint movement due to skrinkage and creep of concrete as well as elastic shortening due to prestress and temperature effects on length changes was one of the objectives of Chapter 4. Designing a joint that can adequately perform under these movements is covered in this section.

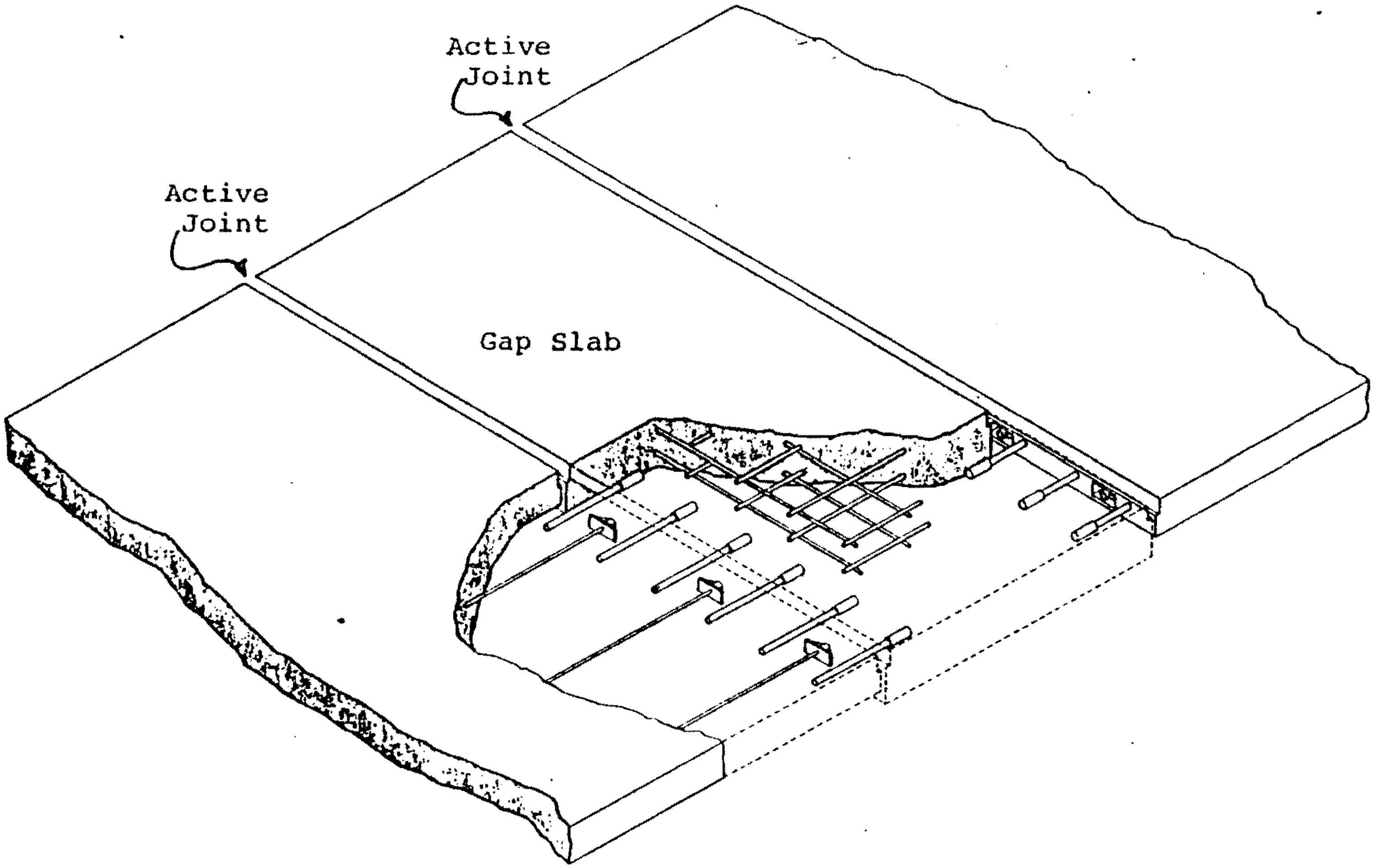


Fig 5.1. Prestressed pavement with active joints at each end of the gap slab.

Methodology

The design of the joint was carried out in three major steps. First, previous PSC pavement details were reviewed and studied. These included the four previous U.S. prestressed pavement details, European details, airport pavement details, and finally current bridge details. Secondly, the joint details that could withstand the movements predicted in Chapter 4 with adequate performance were considered. The third step was to develop recommendations with respect to initial joint opening.

Review of Previous Works

The overall project data from the four U.S. projects are summarized in Chapter 1, in Tables 1.1 and 1.2.

Virginia Joint Detail. The Virginia project used a double I-beam joint at each end of the 8-foot-long gap slab. Figure 5.2 illustrates the detail. Steel dowel bars 1-1/4 inches in diameter were used for load transfer. The I-beams were spaced at the time of construction, depending on the prevailing temperature. The opening between the I-beams was filled with foamed-in-place polyurethane.

The Virginia joint experienced some problems that made it necessary to alter it in 1975.

(1) In several of the joints,the gap concrete separated from the I-beams due to inadequacies in the design details of the reinforcing of the gap concrete.

(2) The steel dowels were corroded and frozen, and they were replaced with stainless steel dowels.

(3) The polyurethane was replaced with a neoprene seal and one end of the gap slab was tied into the prestressed slab (see Fig 5.3).

Other problems that were noted with the Virginia detail were that (1) large amounts of steel were used - 30 pounds per foot for the I-beams alone, (2) the tendon anchors wereexposed to moisture and possible corrosion (it is best to fully embed the anchor in concrete), and (3) if temperatures exceeded 100°F the joint closed and the beam flanges

TABLE 5.1. LIMITING CHANGE IN TEMPERATURE TO CAUSE CRUSHING OF CONCRETE (440-FOOT SLAB)

f_c (psi)	ΔT (°F)	Compressive Stress
100	40	98
200	55	199
300	65	293
400	75	396
500	80	474
600	90	590
700	95	674
800	100	761
900	110	893
1000	115	986

TABLE 5.2. INITIAL JOINT SETTING, MAXIMUM JOINT OPENING, AND TIME AND TEMPERATURE UNTIL 1.5-INCH OPENING FOR 240-FOOT SLAB

T_1	Initial Setting	Maximun Open (inch)	T_2	Time (d) to 1.5-inch
Above 90	0.00	4.47	70	45
			65	35
			60	25
			55	15
			50	10
80-80	0.00	4.20	60	45
			55	35
			50	25
			45	15
			40	10
70-79	0.00	3.93	50	45
			45	35
			40	25
			35	15
			30	10
60-69	0.25	3.91	50	45
			45	35
			40	25
			35	15
			30	10
50-59	0.50	3.89	50	50
			45	35
			40	25
			35	15
			30	10
40-49	0.75	3.87	40	25
			35	15
			30	10
30-39	1.00	3.85	30	15
			25	10

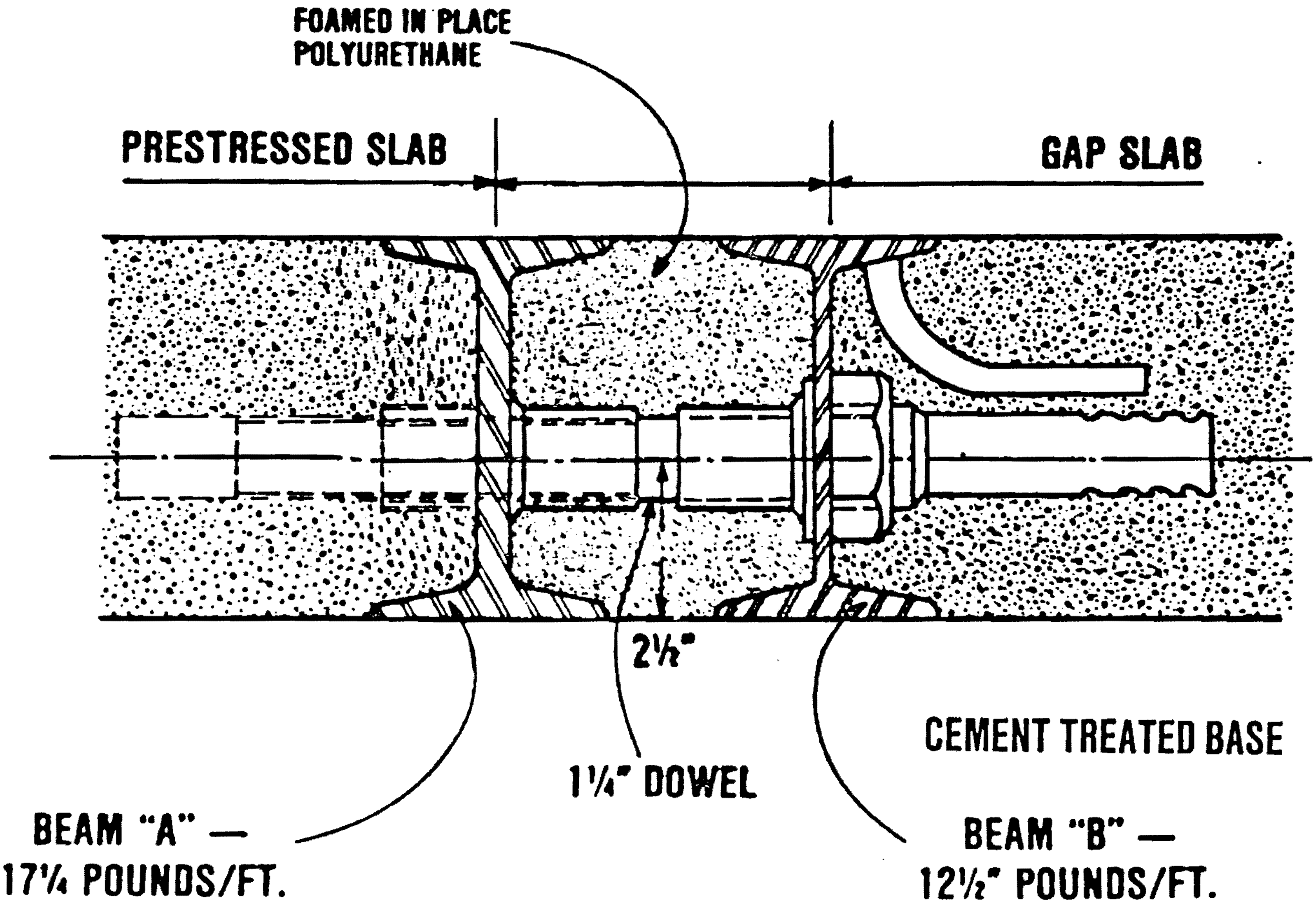
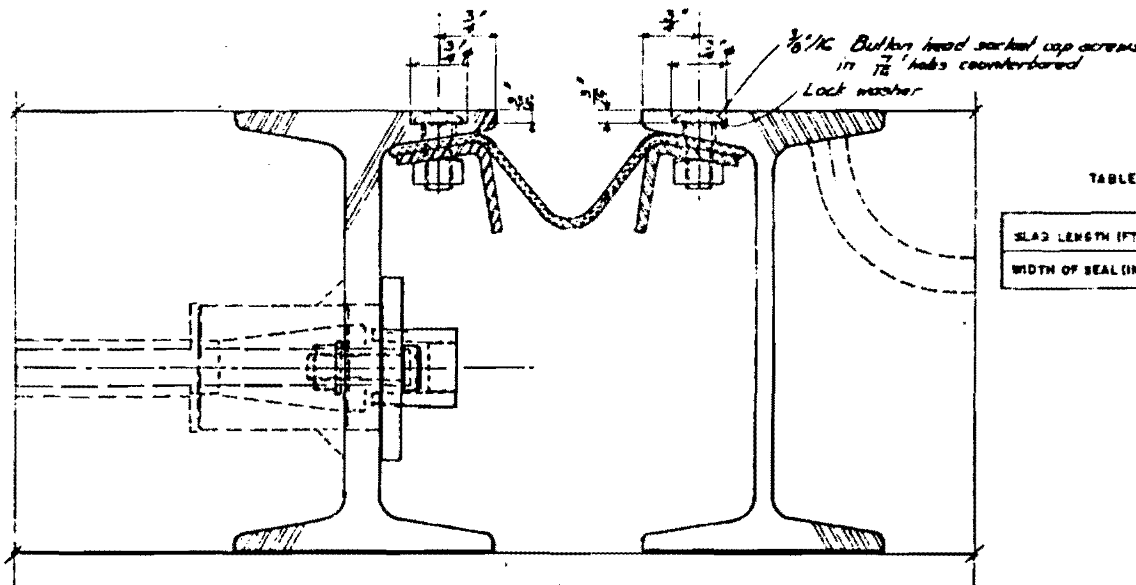


Fig 5.2. Virginia joint detail.



LONGITUDINAL SECTION OF JOINT AT NORMAL JOINT OPENING
Counter bored holes and 3/8"/16 button head screw detail

TABLE OF NEOPRENE SEAL WIDTHS

SLAB LENGTH (FT)	400	500	600	700
WIDTH OF SEAL (IN)	6	[illegible]	6 1/2	6 3/4

Fig 5.3. Revised Virginia joint detail.

would carried the compressive forces, causing the flanges to ride up on one another. A flat bearing surface avoided the last problem. The reinforced joint appeared to perform well, with seasonal movements of less than one inch and daily movements of from 0.5 to 0.75 inches over 25° to 30°F changes in temperature (Ref 13).

Mississippi Joint Detail. The Mississippi project had a joint detail which used a steel bulkhead and mandrels to form the ends of the prestressed slabs. The mandrels provided voids to receive tendon anchors and 1-1/4 inch stainless steel dowels (Ref 13). The joints were sealed with a polysulphide joint filler (Fig 5.4).

The major problems with the Mississippi detail were related to the joint seals. Almost immediately the polysulphide failed and was replaced with a 3-1/2 inch preformed neoprene seal in 100 joints and a cold poured polyurethane material in 16 joints. The polyurethane joints and several of the neoprene joints were replaced with a preformed joint material called evazote. The evazote joints had performed well since 1979, with seasonal movements of about 1.25 inch and daily movements of about 0.5 inches for 25°F change in temperature (Ref 13).

Arizona Joint Detail. The Arizona detail had a temporary steel bulkhead to form the end of the prestressed slab. The bulkhead held the stainless steel dowels and sleeves, as well as positioned forms for the prestressed anchor bearings and all extruded steel anchorage for the neoprene strip seal (see Fig 5.5).

The major problem with the Arizona detail occurred when some of the joints experienced a failure of the weld between the steel shape holding the neoprene and the anchor studs. The steel shapes that broke away were rewelded and the area was patched with epoxy.

The other joints seemed to be performing well, with seasonal movements of less than one inch.

Pennsylvania Joint Detail. The Pennsylvania project used a prefabricated steel tongue and groove joint (see Fig 5.6). The male-female assembly provided shear transfer between adjoining slabs.

The Pennsylvania joint experienced two fairly major problems. First, concrete spalling occurred at some of the gap slabs. Second, two of the 19 joints had to be repaired when the male beam interlocked with the female beam. When the temperature dropped and the slabs contracted, the female beam separated from the slab. According to a 1983 report several other joints experienced minor distress of the same type (Ref 13).

The joints experienced about a 1.5-inch seasonal movement and about a 0.25-inch daily movement for 25°F temperature change.

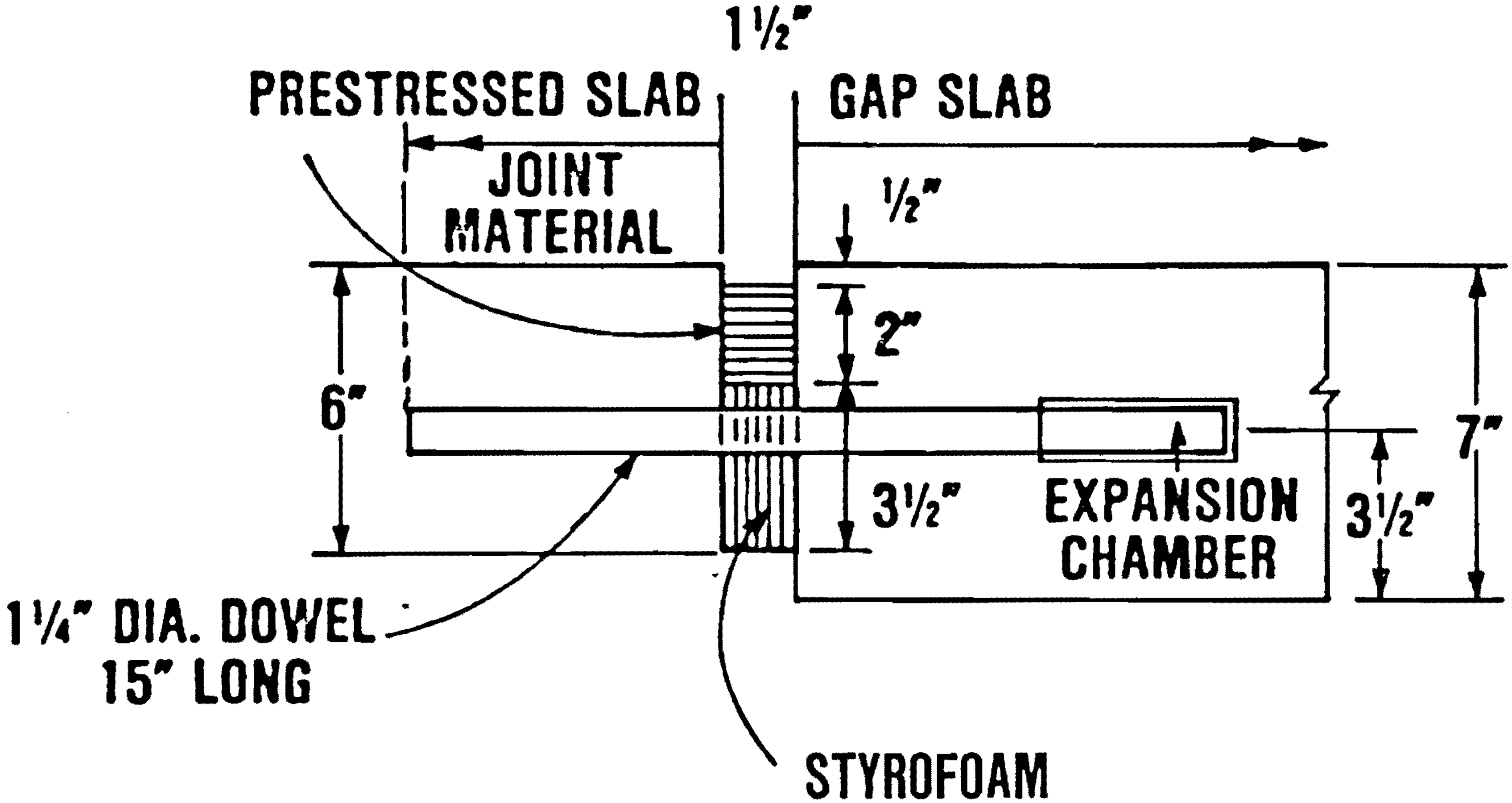


Fig 5.4. Mississippi joint detail.

ARIZONA JOINT AT STRAND ANCHOR
(1¼" x 18" DOWELS NOT SHOWN)

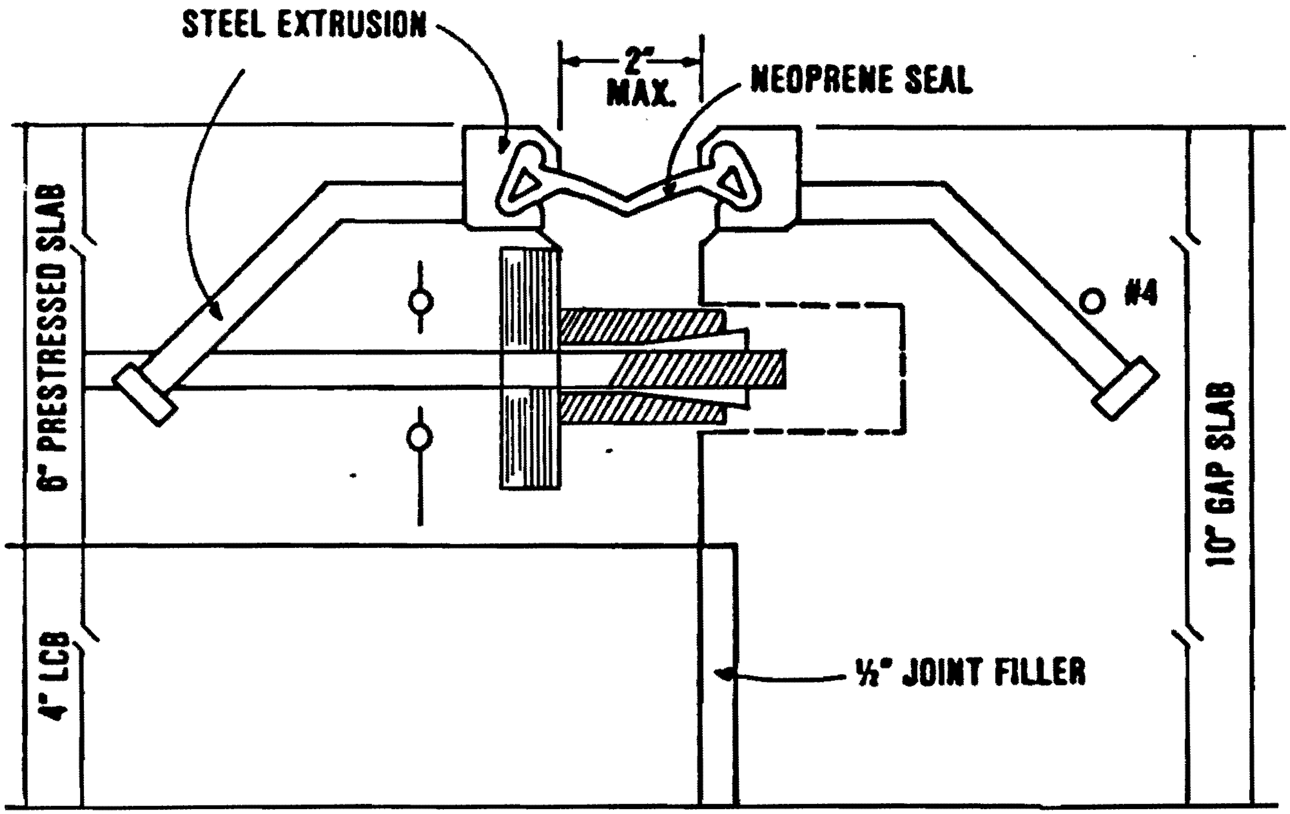
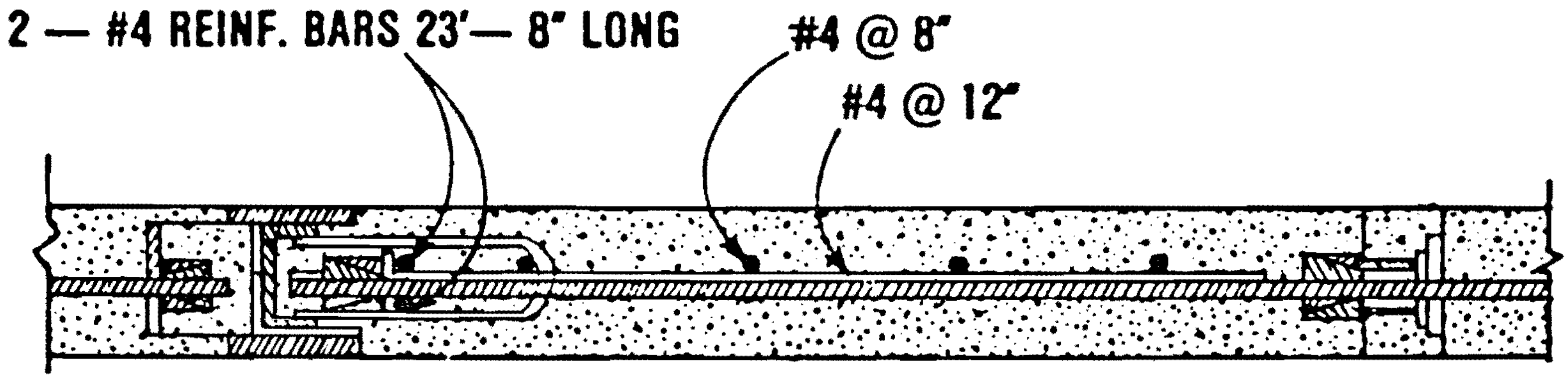
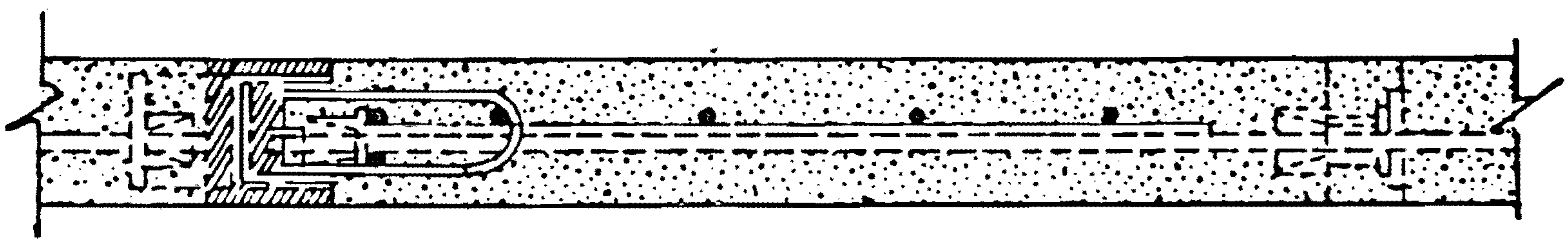


Fig 5.5. Arizona joint detail.



SECTION THROUGH TENDON CENTERLINE AT JOINT



SECTION THROUGH VERTICAL MEMBER OF THE FEMALE BEAM

Fig 5.6. Pennsylvania joint detail.

In a performance report on the four U. S. prestressed concrete pavements (Ref 13), the joint design is described as "troublesome." The report further states that "a better joint could be designed." None of the previous details are recommended.

Other Joint Details Reviewed. Various other joint details were reviewed. Several details were from airfield pavements and some bridge details were considered.

One of the details looked at was a heavy duty bridge detail made of a neoprene elastomeric belt reinforced with steel rods and placed between two steel angles. A diagram of this detail is shown in Fig 5.7. This joint was designed to accommodate between 2-1/2 and 5 inches of movement. It was not suited for Texas projects since its use is not practical on very thin slabs.

Another joint detail studied was a German airfield pavement joint. This detail, shown in Fig 5.8, was a cover plate joint which appeared to be able to accommodate the magnitude of movements that our project required and, in addition, had the advantages of being preassembled and set at an initial opening according to the prevailing temperature. Some revisions of this detail were made. The detail's description and the reason for its rejection are found in the following section.

Joint Details Considered But Rejected. The German airfield pavement joint shown in Fig 5.8 was modified to suit the situation for highway pavements. The modified joint detail may be seen in Fig 5.9. Another variant of this detail type may be seen in Fig 5.10.

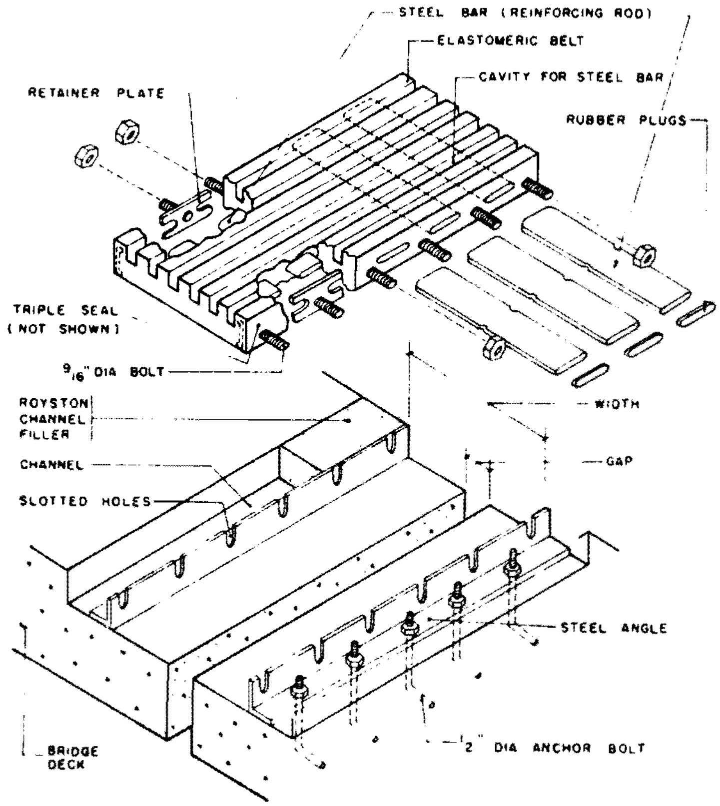
The strong points of this type of detail was that (1) it could be manufactured in the shop and placed in the field as a single unit, (2) the amount of steel anchor bar was sufficient to securely anchor the joint, (3) the weld area for the anchor bar was also sufficient, and (4) the contact was between good bearing surfaces if the joint were to completely close.

The major weakness of this type of joint, and the reason for its rejection, was revealed with further research of cover plate joints. Apparently cover plate joints had been rather unsuccessful due to the fact that with time the welds for bolts fatigue and the cover plate fell off.

Transverse Joint Proposal

After the cover plate joint was rejected, an attempt was made to incorporate the strong points of the cover plate joint into a different detail to ultimately have a satisfactory joint detail that could handle the magnitude of movements that are expected.

UNIDAM ASSEMBLY



ROYSTON TRIPLE-SEAL

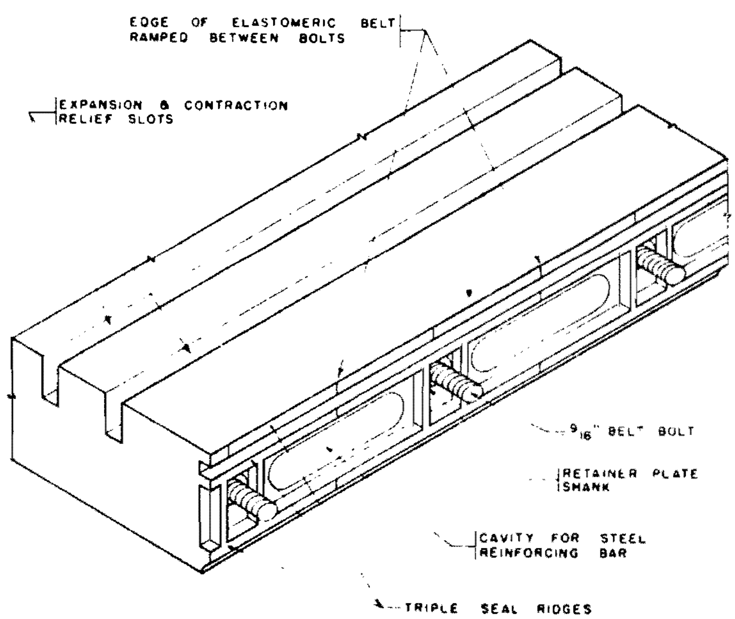


Fig 5.7. Bridge joint detail for large movements.

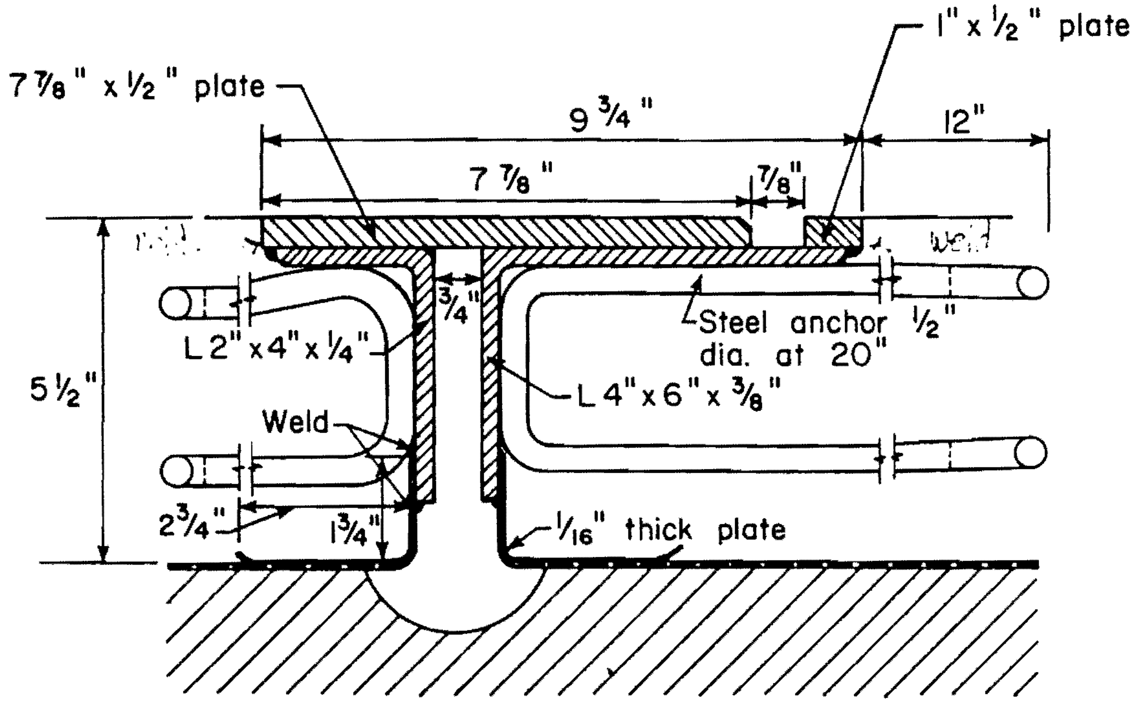


Fig 5.8. Covered joint for airfield pavements in Germany.

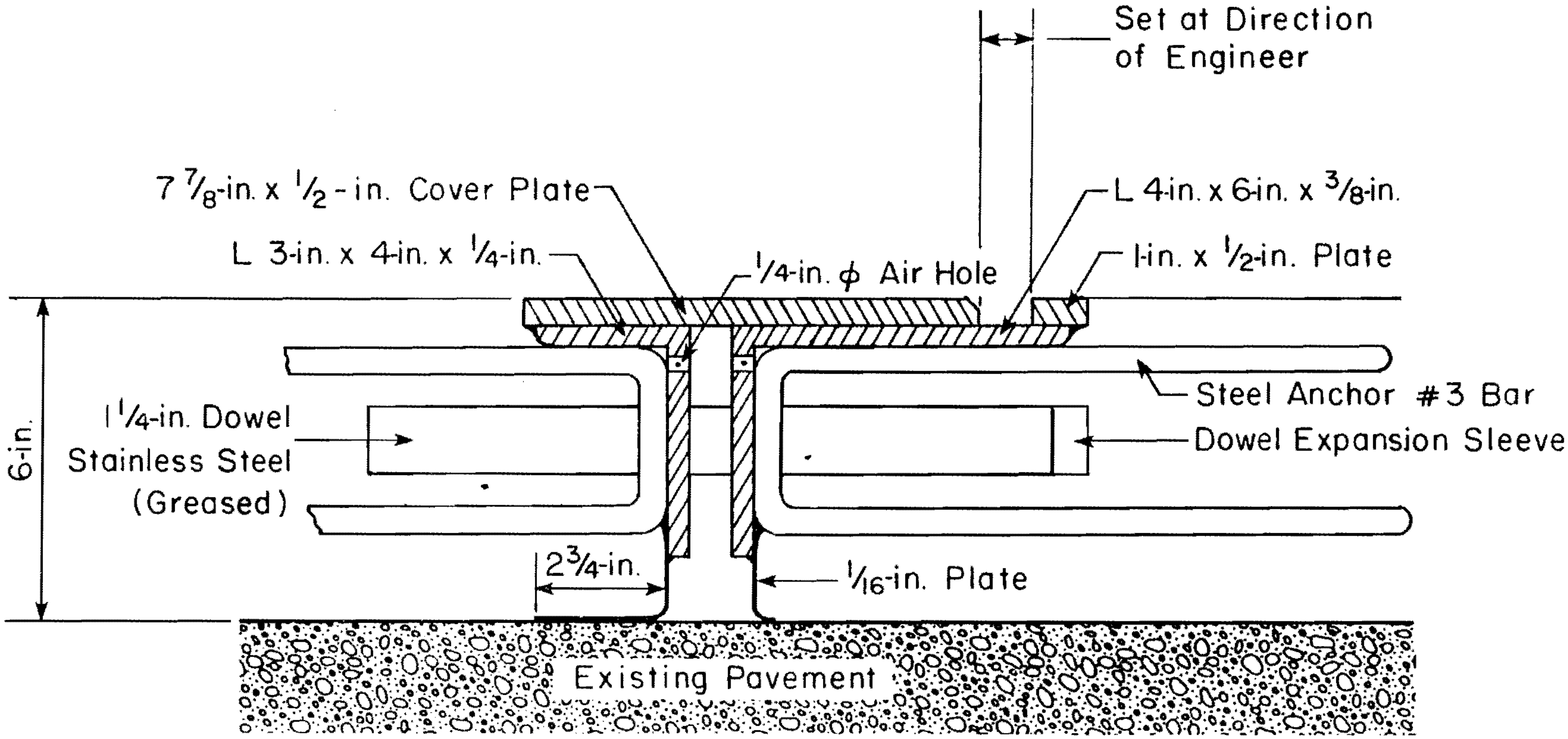


Fig 5.9. Cover plate joint detail.

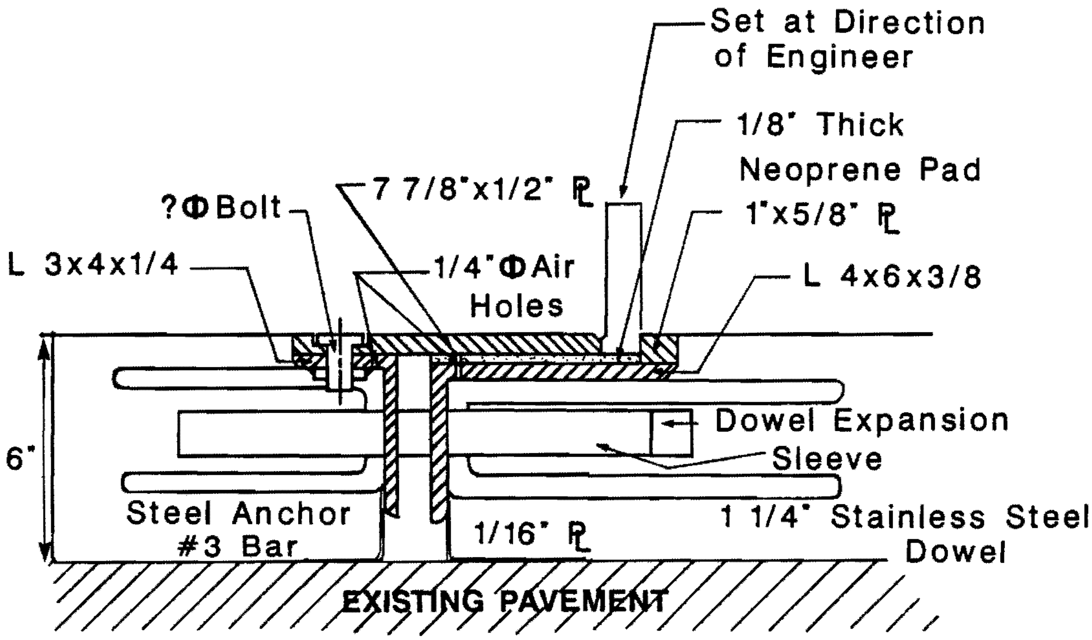


Fig 5.10. Proposed joint detail II.

One of the joints that performed well is the neoprene seal joint. Arizona used a similar detail and their only problem seemed to be fatigue of the anchor weld. Their detail had only one row of anchors, which allowed the joint to rock slightly as traffic passed over. With two rows of anchors the joint was much more stable and this rocking was eliminated.

One of the strongest traits of the cover plate joint was the fact that, if it closed completely, the angles provided a good flat bearing surface. This was incorporated into the neoprene seal detail and can be seen in Fig 5.11. More details of these seal types and how they are inserted may be found in manufacturers' catalogs. The anchor bars in the final detail were placed in two rows, alternating top and bottom every space. In other words, there was only one bar every space, not two.

The neoprene seals come in different sizes, with the largest size allowing for an opening of more than 5-1/2 inches.

Initial Joint Setting. The maximum joint opening occurs after the pavement slab shortening takes place and the slab temperature is at its lowest. This opening will be the sum of the permanent opening, the opening caused by the drop in temperature from the initial temperature at the joint setting minus the current temperature, and the initial joint setting. If the temperature at the time the joint is placed is lower than the expected high temperature and the joint is initially set closed, this maximum joint opening can be reduced substantially. The permanent slab shortening is an advantage in this case because it helps open the joint so that later, when the temperature reaches a maximum, the joint does not close.

There are a few areas of concern with respect to the initial joint setting. First, if the joint is initially set closed at temperature T_1 and then the concrete temperature rises to temperature T_2 before the slabs separate due to shrinkage, etc., the slabs will be compressed and there is the possibility of crushing the concrete. Table 5.1 shows the limiting increase in temperature to prevent crushing of the concrete. These values show that the possibility of crushing concrete is very unlikely since concrete gains strength to about 800 psi the first day (Ref 17).

The second area of concern is that the joint should provide a good bearing surface in case the joint closes. Figure 5.11 shows the diagram of the joint. The area that will bear against the opposite side of the joint is the angle which is quite capable of handling the bearing. It is also highly unlikely there will be an eccentricity great enough to damage the joint.

Tables 5.2 and 5.3 contain information with respect to two additional areas of concern. This information allowed for recommending joint setting as a function of initial temperature,

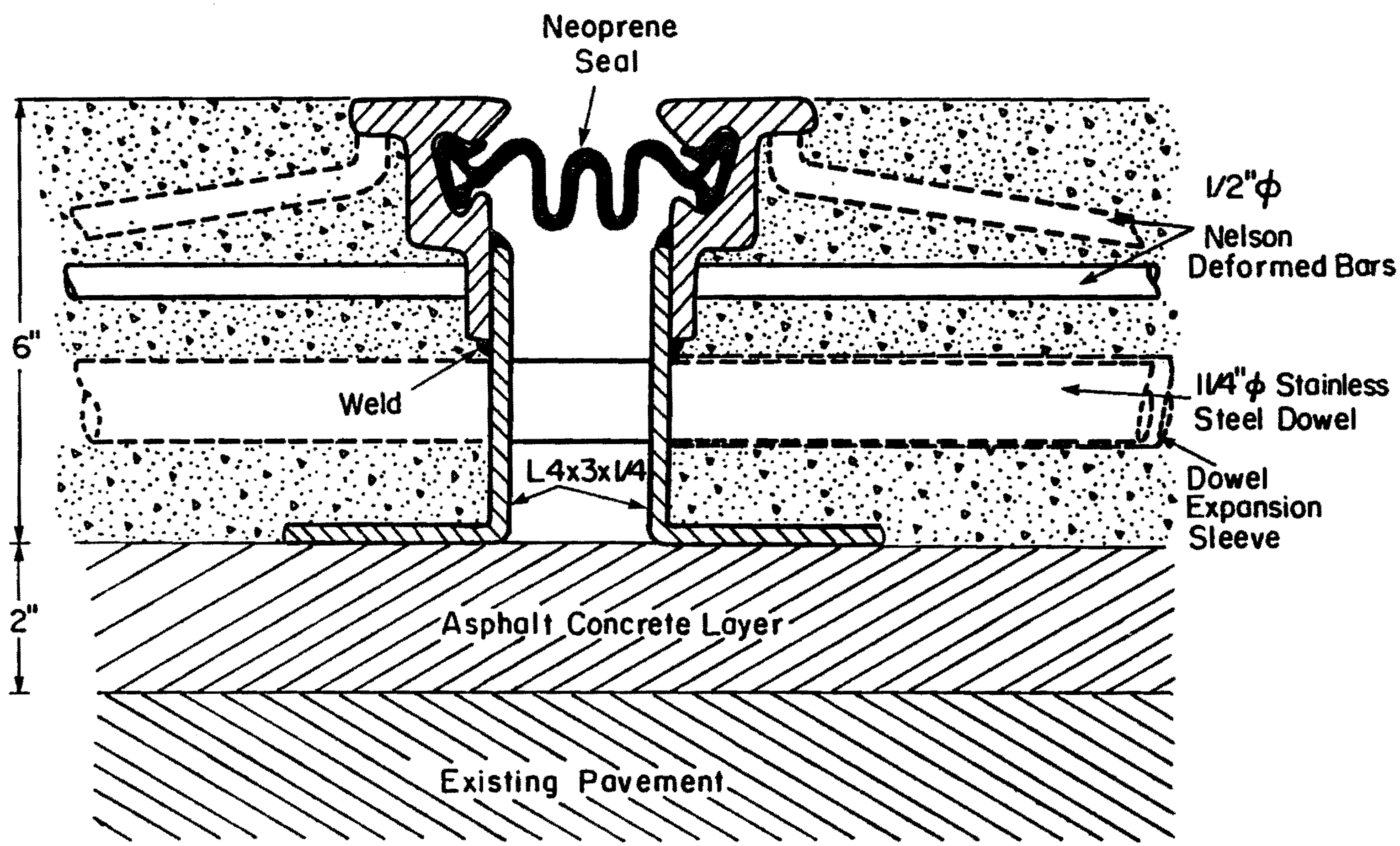


Fig 5.11. Final joint detail.

TABLE 5.3. INITIAL JOINT SETTING, MAXIMUM JOINT OPENING, AND TIME AND TEMPERATURE UNTIL 1.5-INCH OPENING FOR 240-FOOT SLAB

T_1	Initial Setting	Maximun Open (inch)	T_2	Time (d) to 1.5-inch
Above 90	1.00	3.48	90	40
			85	20
			80	15
			75	10
80-80	1.00	3.33	80	40
			75	20
			70	15
			65	10
70-79	1.00	3.18	70	40
			65	20
			60	15
			55	10
60-69	1.00	3.03	60	40
			55	20
			50	15
			45	10
50-59	1.00	2.88	50	40
			45	20
			40	15
			35	10
40-49	1.00	2.73	40	40
			35	20
			30	15
30-39	1.00	2.58	30	40
			25	20

T_1, for 440 and 240-foot-long slabs respectively. The first concern was that the joint should open up enough so that at high temperatures the joint does not completely close and damage the neoprene seal. This concern is satisfied if, for a concrete temperature of 120°F, the joint width is at a minimum of 0.75 inch.

The second concern was that the joints open enough to allow the placement of the neoprene seal in the joint. Manufacturers of the proposed joint detail recommend a joint opening of 1.5 inches for inserting the seal. Tables 5.2 and 5.3 include the anticipated maximum joint opening as well as a meter temperature, T_2, and time in days that the joints should open to the recommended 1.5 inches for insertion of the seal.

LONGITUDINAL JOINT DETAIL

Construction of the two lanes of the one-mile section in Waco was planned in the following sequence. First, the inside lane was to be built from north to south and then the outside lane was to be paved in the opposite direction. This construction sequence would avoid unnecessary movements of the paving equipment with associated costs and delays. A problem arising with the relative displacements between the slabs in the inside lane and those in the outside lane, constructed several days later, was foreseen. By the time of placement of the outside lane slabs, the inside lane slabs had already experienced a significant amount of shrinkage and creep. Therefore, the transverse strands coming out straight from the slabs of the inside lane tended to be bent, as shown in Fig 5.12, when the shrinkage and creep deformations of the outside lane slabs started occuring. This situation was especially critical between the first and last slabs constructed on the project (the inside and outside lanes of the north end of the section respectively). In order to minimize the prestress losses arising as a result of sharply bent tendons and to avoid damage of the concrete in the vicinity of the longitudinal joint, it was recommended that a one-foot-long tube of crushable material be used to wrap the tendons crossing the longitudinal joint. This detail is illustrated in Fig 5.13. Progressive slab end movements of between 0.3 and 0.5 inch were observed one month after placment of the slabs at Dulles International Airport in Virginia, for 400 to 760-foot-long slabs (Ref 14). Based on these figures, the relative displacement between ends of adjacent

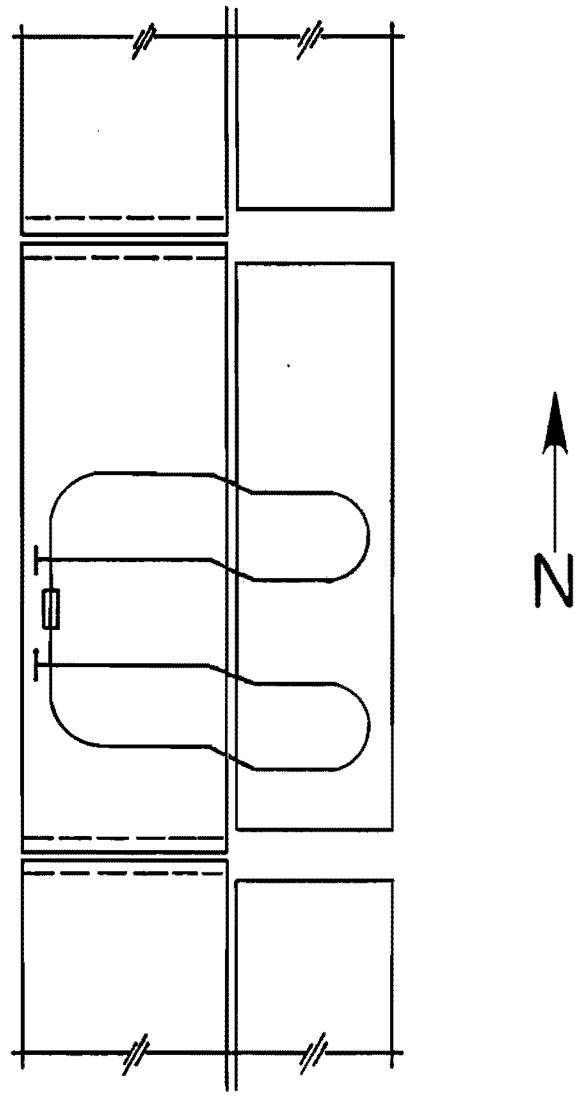


Fig 5.12. Sharp bends on transverse tendons produced by shrinkage and creep differentials between adjacent slabs.

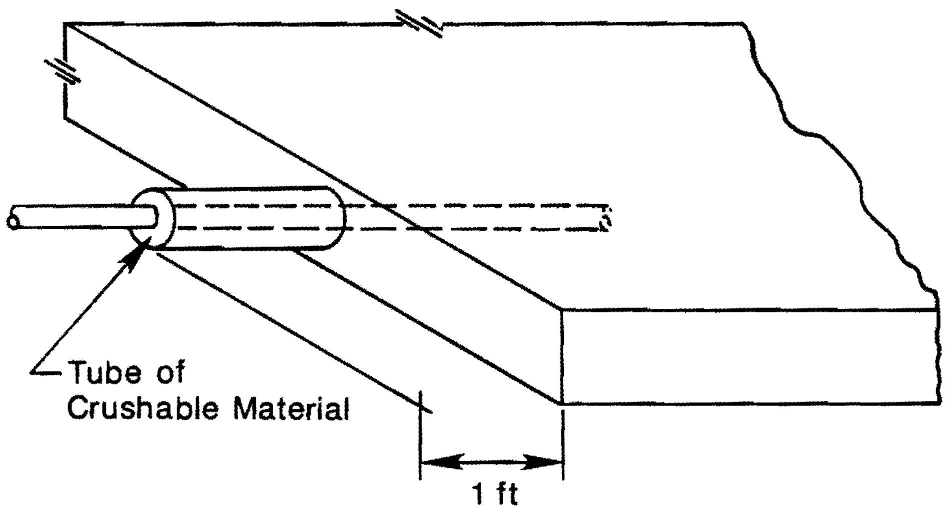


Fig 5.13. Tube of crushable material wrapping transverse tendons at longitudinal joints.

440-foot-long slabs of the Texas overlays were not to exceed 0.5 inch. The tube, therefore, was to be 0.5 inch thick in the external thirds of the slabs. The tube thickness reduced to 0.25 inch in the internal third of the slab length. This solution was implemented on short and long slabs of the sections.

CHAPTER 6. STRAND SPACING AND PROVISIONS ON APPLICATION OF THE PRESTRESSED FORCE

The design of the prestress of PSC pavements requires that two criteria be satisfied:

(1) Repeated flexural stresses caused by applied loads over the precompression at the critical location must not cause a fatigue failure of the prestressed slab.

(2) Combined load, temperature, and moisture stresses must be less at the critical location than the combined concrete strength and residual prestress to prevent cracking of the concrete.

The first criterion is known as fatigue design and was incorporated in this procedure for overlays together with the thickness design in Chapter 3. The second is referred to herein as elastic design. The prestress level useed should be the minimum meeting both design criteria.

This chapter is composed of three major parts. In the first part, the elastic design of the slabs of both sections is presented. Then, in part two, the highest prestress level at the center of the slabs between fatigue and elastic analyses is used for estimating prestress losses and defining the strand spacing. Finally, since the prestress was to be applied in stages at the earliest possible concrete age to prevent premature cracking due to shrinkage and friction with the base, a sequence for prestressing is recommended.

As background information, Table 6.1 shows the prestress data for the four past U.S. PSCP projects.

ELASTIC DESIGN OF SLABS IN LONGITUDINAL DIRECTION

The elastic design of PSCP slabs requires that the following relationship be satisfied to prevent concrete cracking:

$$f + \sigma_p \geq \sigma_t + \sigma_c + \sigma_F \tag{6.1}$$

TABLE 6.1. PRESTRESSING DATA FOR PREVIOUS U.S. PRESTRESS PROJECTS

	Virginia	Pennsylvania	Mississippi	Arizona
Pavement Width (feet)	24	24	24	31.5
Number of Strands	12	12	12	16
Strand Spacing (feet)	2	2	2	2
Diameter (inch) (7 wire - 270 psi)	0.5	0.6	0.5	0.5
Prestress Procedure (Force in Kips at Concrete Strength)				
First Stage	10 k @ 1000 psi (1st day)	14 k @ 1000 psi * (1st day)	14 k @ 1000 psi (1st day)	11 k @ 1300 psi (1st day)
Second Stage	10 k @ 2000 psi	46.9 k @ 2500 psi	33 k @ 2500 psi (5th day)	24 k @ 2400 psi (2nd day) **
Third Stage	9 k @ 3000 psi	- -	- -	31 k @ 3000 psi (3.5 days)

*Changed during project to 20 k @ 1500 psi
**Jacking forces varied with concrete srength

where

f = allowable flexural stress in the concrete,
σ_p = effective prestress at the critical location,
σ_t = stress due to applied wheel load,
σ_c = curling stress due to temperature differential through the slab, and
σ_F = friction loss with the base.

Since the overlays were to extend to the shoulder areas, the critical condition to check for elastic design was at the bottom of the slabs for an interior loading condition. In this case, the bottom of the slab was where all stresses were additive.

Allowable Flexural Stress

The allowable flexural stress is the concrete flexural strength affected by a safety factor. ACI Committee 325 (Ref 24) suggests a safety factor of two for use in the design of primary highways. Accordingly, for a concrete 28-day flexural strength of 700 psi, assumed in the fatigue analysis in Chapter 3, the allowable flexural stress, f, will be

$$f = 350 \text{ psi}$$

Wheel Load Stresses

The wheel load stress at the bottom of the slab can be calculated by using the well-known Westergaard equations (Ref 25). One of the computer codes to predict stresses from plate theory (Ref 26) and multilayered elastic theory (Ref 9) or any of the most recent finite element programs (Ref 27) can be used. In this analysis, BISAR (Ref 9) was used to determine the tensile stress at the bottom of the overlays. BISAR was run on the models presented in Figs 3.6(c) and 3.12(c) in Chapter 3 for the Cooke and McLennan sections, respectively. The design load was the maximum 20-kip single axle load allowed in interstate highways. The axle consisted of two 10-kip wheel loads spaced 72 inches from center to center. The stresses, σ_t, for 8 and 6-inch PSCP slabs respectively were

$\sigma_{t\text{-}8}$ = 258 psi

$\sigma_{t\text{-}6}$ = 362 psi

Curling Stress

This parameter should include the warping stresses due to moisture gradients too. However, since warping stresses are opposite to the most critical curling stresses, and since warping stresses cannot be predicted with any degree of reliability, usually only curling stresses are considered in the design.

Curling stresses, σ_C, can be predicted for the midlength location by Eq 6.2 (Ref 28):

$$\sigma_c = \frac{E_c \cdot \alpha \cdot \Delta T_D}{2(1-v)} \tag{6.2}$$

where

E_C = modulus of elasticity of the concrete,
α = coefficient of thermal expansion of the concrete,
ΔT_D = total temperature differential through the slab depth, and
v = Poisson's ratio of concrete, normally assumed as 0.15.

Assuming the modulus of elasticity for the PCP was the same as used in the fatigue analysis in Chapter 3 (4.5 x 10^6 psi) using recommended values for coefficient of thermal expansion of 6 x 10^{-6}/°F (Ref 24) and maximum temperature gradient in the summer of 3°F/in. (Ref 22), the curling tensions at the bottom of the slab are as follows:

$$\sigma_{c-8} = \frac{\left(4.5 \times 10^{6}\right)\left(6 \times 10^{-6}\right)(3 \times 8)}{2(1 - 0.15)}$$

$$= 381 \text{ psi}$$

$$\sigma_{c-6} = \frac{\left(4.5 \times 10^{6}\right)\left(6 \times 10^{-6}\right)(3 \times 6)}{2(1 - 0.15)}$$

$$= 286 \text{ psi}$$

Friction Stress

The loss in effective prestress due to friction is obvious during actual post-tensioning of the slab. This friction loss, σ_F, may be found from the following expression:

$$\sigma_F = \frac{\mu_{max} \cdot \gamma \cdot L}{5}$$

where

μmax = maximum coefficient of friction,
γ = concrete unit weight, and
L = slab length.

A maximum friction coefficient for a slab cast over an asphalt base with a single layer of polyethylene sheeting in between was determined to be 0.92 from the tests conducted at Gainesville (Ref 10). If the concrete is assumed to weigh 144 pounds per cubic foot, then Eq 6.3 reduces, for the 240 and 440-foot-long slabs, to the following values.

For the 240-foot slab,

$$\sigma_F = \frac{0.92 \cdot 144 \cdot 240}{2 \cdot 144}$$

$$= 110 \text{ psi}$$

For the 440-foot slab,

$$\sigma_F = \frac{0.92 \cdot 144 \cdot 440}{2 \cdot 144}$$
$$= 202 \text{ psi}$$

The computations shown above correspond to the prestress loss due to friction during actual post-tensioning of the slab and, therefore, the maximum coefficient of friction is assumed beneath the entire slab length.

Realistically, the prestress force does not have to be designed to overcome this maximum friction force - only the portion that developed with the drops of the daily temperature. These drops cause contraction movements in the slab which, if restrained by the friction, produce tensile stresses in the concrete. These tensile stresses are maximum at the center of the slabs. However, long slabs cast on polyethylene sheets over smooth bases tend to develop the maximum friction coefficient on the substantial portion of the length. Hence, friction stresses develop which are nearly those obtained assuming the entire slab is working with the maximum friction coefficient. To consider maximum coefficients of friction is the conservative approach as well as the approach recommended by most PSCP designers (Ref 29) and it was, therefore, the approach used herein.

Critical Conditions

The critical conditions to check for in the elastic design of PSC pavements in the longitudinal direction are the following:

(1) The slab midlength in the very early hours of the morning, when the tensile stresses produced by the friction are at nearly their maximum value without curling stresses. Also, because the top surface of the pavement is relatively wet at this time of the day, the warping restraint stresses, which have been found to cause permanently compressive stresses at the bottom of rigid pavements (Ref 18), provide the least compression.

(2) The slab midlength when the tensile stress at the bottom of the slab combines with the maximum curling stress without friction approximately at noon.

Table 6.2 summarizes the terms in the equations and prestress levels for both thicknesses, slab lengths, and conditions indicated in this section.

LONGITUDINAL PRESTRESS LEVEL

Table 6.3 compares the critical prestress levels from Table 6.2 with those obtained from the fatigue analysis increased by the slab-base friction losses taking place during slab post-tensioning. The prestress levels from fatigue were not affected by the curling stresses because this effect is implicit in the fatigue model used in the analysis. Table 6.3 presents the governing prestress level for each case.

There is another design criterion that had to be satisfied for the elastic design of PSC pavements in the longitudinal direction. The prestress level should exceed the maximum frictional resistance at any section of the pavement by at least 100 psi (Ref 24). This criterion is a safety factor against increases of the friction coefficient with time and seasonal climatological conditions. This criterion affects slightly the prestress of the 6-inch-thick, 440-foot-long slab. In this case, instead of 298 psi, the prestress level has to be raised to 202 + 100 = 302 psi.

Summarizing, the prestress levels of slab ends for cases considered in the analysis above were the following:

	Slab Length	
County	240 feet	440 feet
McLennan	298 psi	302 psi
Cooke	289 psi	302 psi

These values do not exceed the maximum of 650 psi allowed in the concrete at the anchor zone by ACI Committee 325 (Ref 24).

TABLE 6.2. PRESTRESS LEVELS REQUIRED IN LONGITUDINAL DIRECTION FROM ELASTIC DESIGN

Condition	Slab Length (feet)	Thickness (inch)	Stress σ_t +	σ_c +	σ_F -	f =	σ_P
1	240	6	362		110	350	122
1	240	8	258		110	350	18
1	440	6	362		202	350	214
1	440	8	258		202	350	110
2	240	6	362	286		350	298
2	240	8	258	381		350	289
2	440	6	362	286		350	298
2	440	8	258	381		350	289

TABLE 6.3. PRESTRESS LEVEL AT SLAB ENDS IN LONGITUDINAL DIRECTION FROM ELASTIC AND FATIGUE ANALYSES AND GOVERNING VALUES

Slab Length (feet)	Thickness (inch)	Analysis: Elastic	Analysis: Fatigue	Governing Values
240	6	298	175	298
240	8	289	210	289
440	6	298	267	298
440	8	289	302	302

PRESTRESS LOSSES AND FINAL PRESTRESS

Final prestress is defined as the initial prestress less the prestress losses which result from a combination of seating loss, shrinkage and creep in the concrete, and steel relaxation.

Before computations of prestress losses are shown, some characteristics of the prestressing steel specified for the Texas overlays are indicated here.

Prestressing Steel

The strands for the Texas overlays should be 7-wire grease coated plastic encased strands, 270 kip grade. The strands should have 0.6–inch nominal diameter strands and 0.216-inch^2 of nominal area. The use of low-relaxation strands is recommended. The tendons should be tensioned to an initial stress of 80 percent of the ultimate strength of the strands after the anchoring is finished. The tendons should be stressed at the stressing pockets by applying nearly equal forces on the strand segments extending at each side of the pocket. This condition forces the pockets for the longitudinal prestress to be as close to the slab center as possible.

Computation of Prestress Losses

The highest force is applied to the prestressed pavement during the prestressing operation. Later on, the strands experience reductions of the initial elongation due to the combined effect of concrete shrinkage and creep. Seating losses, strand friction developing during tensioning, and steel relaxation complete the list of factors reducing the initial prestress. Hence, after the first few years, the strand stresses may fluctuate around some average stress based on initial elongation, adjusted for steel stress relaxation and concrete length changes. Values for the losses identified above can be determined as discussed below.

Seating Loss. Central stressing is possible through the use of a steel sleeve known as a lockcoupler. The lockcoupler and the procedure for stressing the strands in internal pockets are described in Chapter 7. When the load is transferred from the jack to the anchor system in the lockcoupler, there is alway some loss in force due to seating of the wedges. However, for the long elongations required by the steel tendons of the long PSCP slabs, these seating losses are usually insignificant and can be ignored.

Strand Friction Between Conduit and the Tendon. In the central stressing scheme, the tendon forces applied at the slab center are decreased by strand friction during tensioning. Clearly, after a few years this effect will not be noticeable in prestressed pavements. This loss is usually referred to as wobble loss and can be estimated from the following ACI recommended relationships:

$$F_{end} = F_{jack} \bullet \frac{KL}{e^{2}} \tag{6.4}$$

where

F_{end} = prestressing force at anchored end,
F_{jack} = prestressing tendon force at jacking end,
K = wobble coefficient per foot of stressing tendon, and
L = slab length, feet.

Assuming a wobble coefficient of 0.001, the losses in 240 and 440-foot-long slabs for strands initially stressed at 80 percent of the ultimate strength are

$$F_{jack} = 0.8 \times 270 \times 0.216$$

$$= 46.4 \text{ kips}$$

$$\Delta FS_{1\text{-}240} = 46.4 \left(1 - 2.178^{-0.001 \times 120}\right)$$

$$= 5.2 \text{ kips}$$

$$\Delta FS_{1\text{-}440} = 46.4 \left(1 - 2.178^{-0.001 \times 220}\right)$$

$$= 9.2 \text{ kips}$$

Losses Due to Shrinkage and Creep. Shrinkage is produced by concrete drying with time. Creep is a concrete property which causes it to continue to contract under long time periods under a constant stress. Both strains occur very rapidly initially and decrease with time until a nearly constant value is approached. Final shrinkage and creep strains depend on the mix proportion as well as humidity and curing conditions. The rate of creep strain depends also on the age of concrete at loading.

The final concrete strain as defined from final shrinkage and creep strains used in Chapter 4 for computing slab movements is

$$\varepsilon_{cu} = 2.24 \times 10^{-4}$$

This corresponds to a shortening of the steel tendons by the same amount. Therefore, the loss of stress in the steel is

$$\Delta FS_2 = (Es)\left(2.24 \times 10^{-4}\right)(0.216)$$

Assuming a tendon elastic modulus of 28×10^3 ksi

$$\Delta FS_2 = 1.4 \text{ kips}$$

<u>Steel Relaxation</u>. Steel relaxation is the loss in steel stress when it is held at a constant stress level. Reference 32 provides Eq 6.5 as a reasonable estimate of relaxation after t hours of stress:

$$\Delta FS_3 = \left[\frac{\log t}{10}\left(\frac{F}{Fy} - 0.55\right)\right] F \tag{6.5}$$

where

t = time after stressing in hours,
F = prestressing tendon force,

$$
\begin{aligned}
F_y &= \text{tendon yield force} \\
&\simeq 230 \times 0.216 \\
&\simeq 49 \text{ kips}
\end{aligned}
$$

The relaxation loss after 20 years for tendons stressed with an average stress of 70 percent of their yield stress, or a force approximately equal to 35 kips, will be

$$
\begin{aligned}
\Delta FS_3 &= \left[\log \frac{175200}{10} \left(0.70 - 0.55\right)\right] (35) \\
&= 2.8 \text{ kips}
\end{aligned}
$$

Total of Losses and Final Prestress Force

In summary, the total amount of losses per strand for the cases considered herein were the following.

County	Slab Length 240 feet	Slab Length 440 feet
McLennan	9.4 kip	13.4 kip
Cooke	9.4 kip	13.4 kip

Correspondingly, the final prestress force, after deducting losses, for tendons stressed initially to 80 percent of their ultimate strength or 46.4 kips, will be

County	Slab Length 240 feet	Slab Length 440 feet
McLennan	37 kip	33 kip
Cooke	37 kip	33 kip

Strand Spacings

The strand spacings that wouldproduce the required prestress levels in the concrete with the final forces shown above, were determined as follows.

The 6-inch-thick, 240-foot-long slab of the McLennan County overlay, required 298 psi at the slab end. Accordingly, the strand spacing, SS, had to

$$SS = \frac{37000}{298 \times 6}$$
$$= 20 \text{ in}$$

Similarly, the strand spacing for the other conditions were obtained. The results are shown below.

	Slab Length	
County	240 feet	440 feet
McLennan	24 in	16 in
Cooke	18 in	12 in

PRESTRESS IN THE TRANSVERSE DIRECTION

In the transverse direction, the friction restraint stresses are insignificant and the longitudinal joint can be considered to relieve the curling stresses. The design of the prestress, then, was based upon load repetitions. Hence, the prestress levels from the fatigue analysis performed in Chapter 3 were used for defining strand spacing.

Accordingly, the strand spacings were to be

For the McLennan County overlay:	10 feet for 65 psi effective prestress
For the Cooke County overlay:	5 feet for 100 psi effective prestress

INITIAL PRESTRESSING STAGE

Concrete pavements are especially vulnerable to tensile stresses during the first hours because concrete lacks the necessary plasticity to absorb movements. After the pavement is placed, it undergoes temperature movements and drying shrinkage. The critical period when the slabs may crack transversely due to the combined effect of these movements and the base friction is commonly the first night.

In PSC pavements, it is possible to avoid premature cracking by applying an initial stage of prestress before the high tensile stresses occur. During this stage, it is not necessary to overcome 100 percent of the stress, only enough to keep the tensile stress below the tensile strength gained by the concrete at that time.

Recommendations for initial post-tensioning are needed to avoid two potential problems:

(1) transverse cracking of the pavement slab due to temperature drop and
(2 anchor zone failure.

Recommendations for initial post-tensioning should be based on two variables:

(1) time of post-tensioning and
(2) post-tensioning force.

If the pavement is post-tensioned too soon, or with too much force, anchor zone failure is likely to occur. On the other hand, if post-tensioning is not applied soon enough, or if the post-tensioning is too low, temperature cracks are likely to form.

Accordingly, the recommendations for initial post-tensioning should include four provisions:

(1) time of early post-tensioning,
(2) time of latest post-tensioning,
(3) minimum post-tensioning force, and
(4) maximum post-tensioning force.

To develop these provisions, the effect of time of placement on the stresses occurring during the first hours must be investigated.

Effect of Placement Time. Time of placement is an important variable in the occurrence of premature cracking. The slabs that are placed early in the day and whose construction is finished at high temperatures are exposed to the large day and nighttime temperature drop. The slabs whose construction is finished in the late afternoon do not undergo the fast afternoon drop. Figure 6.1 shows the maximum tensile stresses occurring during the first twenty-four hours for both cases. Clearly, early morning placement is more critical than late afternoon placement. Figure 6.1 was developed from computer program JRCP-3 (Ref 23) for average concrete properties and Waco climatological conditions (Ref 22). The friction coefficient versus displacement curve entered in the program is shown in Fig 6.2. This curve was obtained at Gainesville (Ref 10) for concrete slabs placed on a polyethylene membrane over an asphalt base.

Time of Early Post-tensioning and Minimum Amount of Post-Tensioning Force. Based on Figs 6.1 and 6.2, slabs placed early in the morning should be initially prestressed at the earliest possible concrete age. In this case, the initial post-tensioning should be done not later than midnight, thus allowing the concrete nearly 8 hours for gaining strength. Studies by O'Brien with very early post-tensioning of slabs (Ref 11) indicate that 10 kips can be applied safely at the anchor zone after 8 hours. Such precompression would practically eliminate all friction stresses in the concrete for both slab lengths.

Time of Latest Initial Post-tensioning and Maximum Amount of Force. For other cases less critical than early morning placement, the initial prestress may be applied from 8 to 12 hours but not later than 12 hours after placement. The allowable post-tensioning force, as recommended by O'Brien (Ref 11), should be determined by compressive cylinder tests at the job-site. For each pavement slab, at least three cylinders should be tested prior to post-tensioning. From average compressive strength and from Fig 6.3, the cracking load should be obtained. This cracking load should be modified by a safety factor of 2.00 to determine the

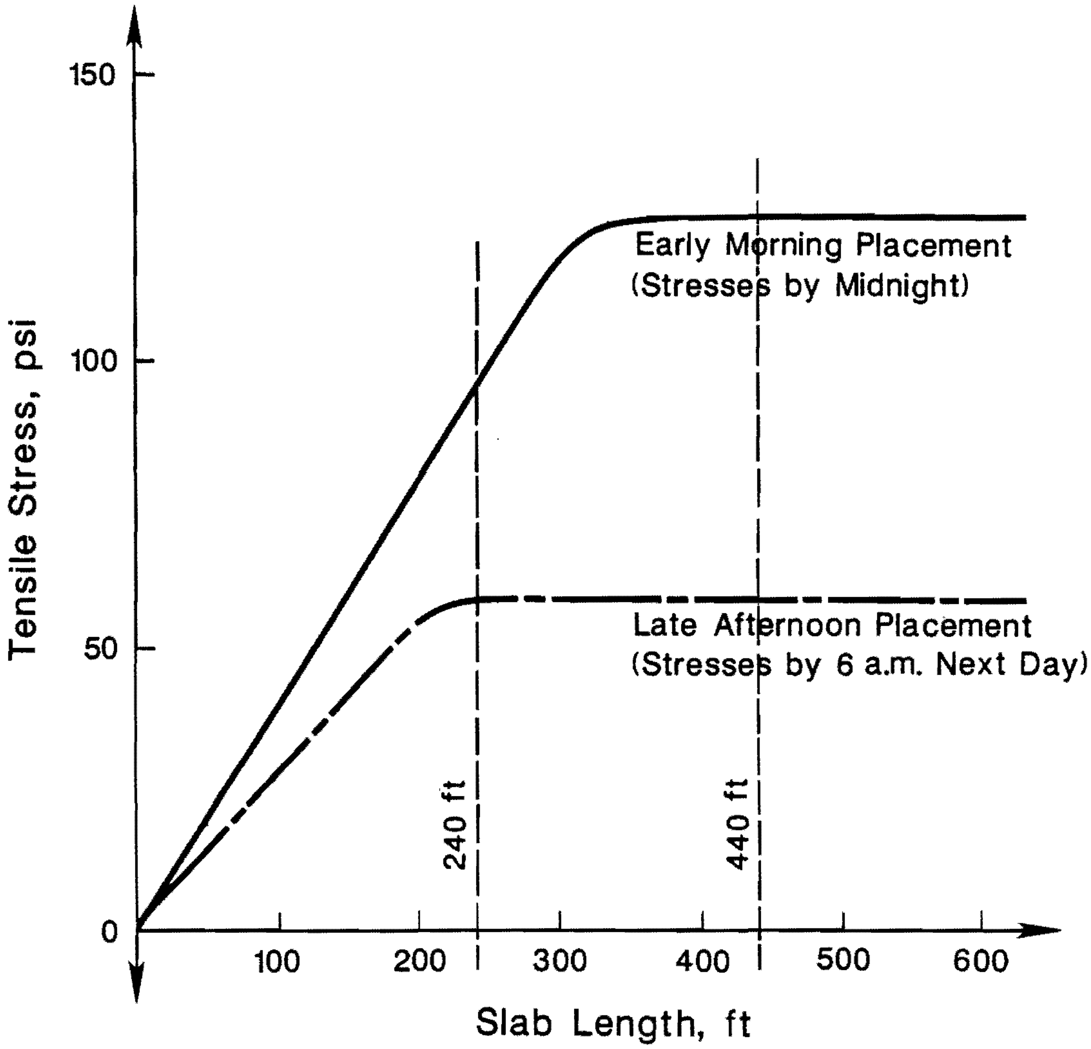


Fig 6.1. Effect of time of placement on concrete stresses occurring during the first hours.

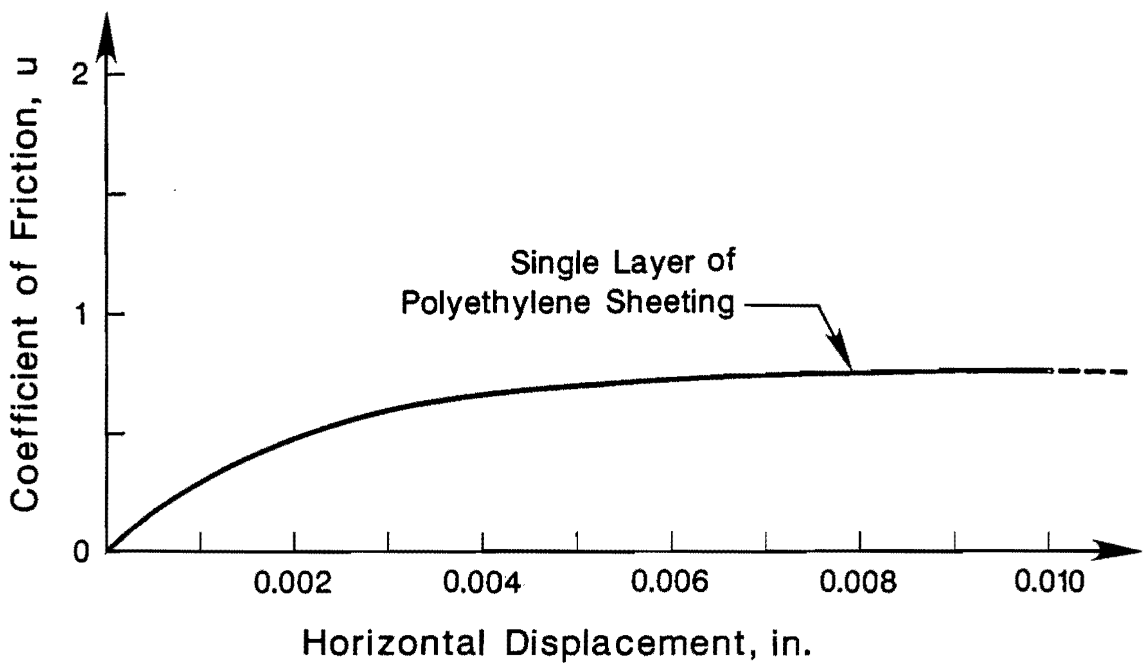


Fig 6.2. Average coefficient of friction versus horizontal displacement curve.

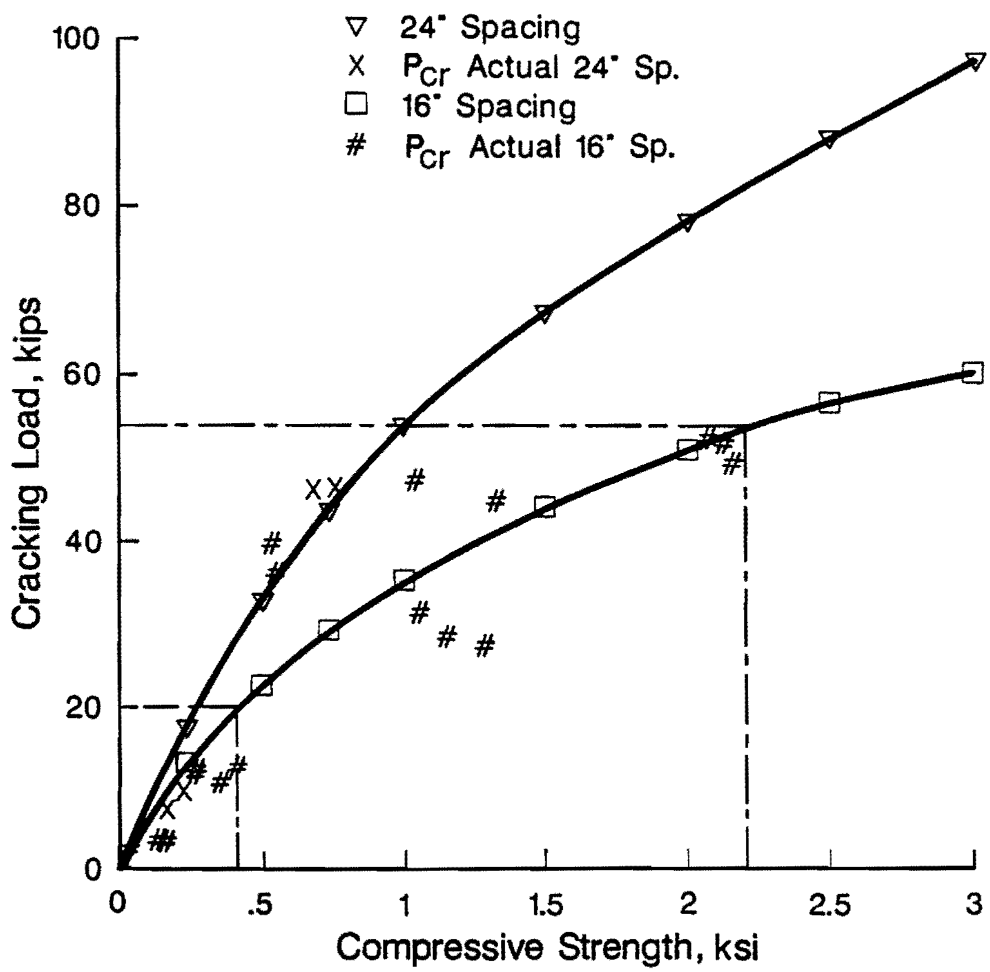


Fig 6.3. Cracking load (P_{cr}) versus compressive strength, after Ref 11.

allowable force. It is suggested additionally that this force should not exceed 15 kips to minimize the level of creep occurring after initial prestressing.

Final Recommendations. After 48 hours, the concrete has sufficient strength to be prestressed to the maximum level of 46.4 kips per strand (Ref 11).

The amount of prestress force applied in each prestressing stage should be controlled by monitoring tendon elongations as well as the stressing ram pressure.

CHAPTER 7. PRESTRESS TENDON LAYOUT AND RECOMMENDED CONSTRUCTION PROCEDURE

A plan view of the proposed pavement developed by the Project 401 staff is shown in Fig 7.1 and an enlarged plan view of a stressing pocket is shown in Fig 7.2. The advantages and disadvantages of using a looped transverse tendon configuration are discussed. This design was accepted by the Texas State Department of Highways and Public Transportation (SDHPT) for construction of the demonstration overlay projects on U.S. Interstate Highway 35. Field testing to obtain additional information related to this approach was conducted near Valley View, Texas, by the Project 401 staff.

GAP SLAB VERSUS CENTRAL STRESSING

As mentioned in Chapter 5, all of the post-tensioned PCP projects previously constructed in the United States consisted of consecutive prestressed slabs separated by short openings to permit the prestressing tendons to be conveniently post-tensioned from their ends. It was necessary to stress each tendon from both ends due to the long slab lengths and correspondingly high tendon and subgrade friction forces. Once the post-tensioning was applied and most of the progressive length changes had occurred, including all of the elastic shortening and most of the creep, this space was filled with a short reinforced concrete filler slab.

For several reasons, this type of gap slab was not entirely satisfactory. Since joints are high-first-cost items which require periodic maintenance, the initial and the long-term costs associated with a gap slab arrangement with two joints are greater than for a gap slab design requiring only a single joint. In addition, poor gap slab performance (i.e.,warping, curling, rocking, etc.) was experienced on some of the projects. In fact, replacement of several of the gap slabs on the Dulles project was necessary after only four years of service (Ref 14).

The gap slab design was modified on the Hogestown, Pennsylvania, project to allow movement at only one of the gap slab joints by post-tensioning the gap slab to one of the previously stressed pavement segments (Ref 13). This was accomplished by transferring the load from temporary anchors at one end of the previously stressed pavement segments to

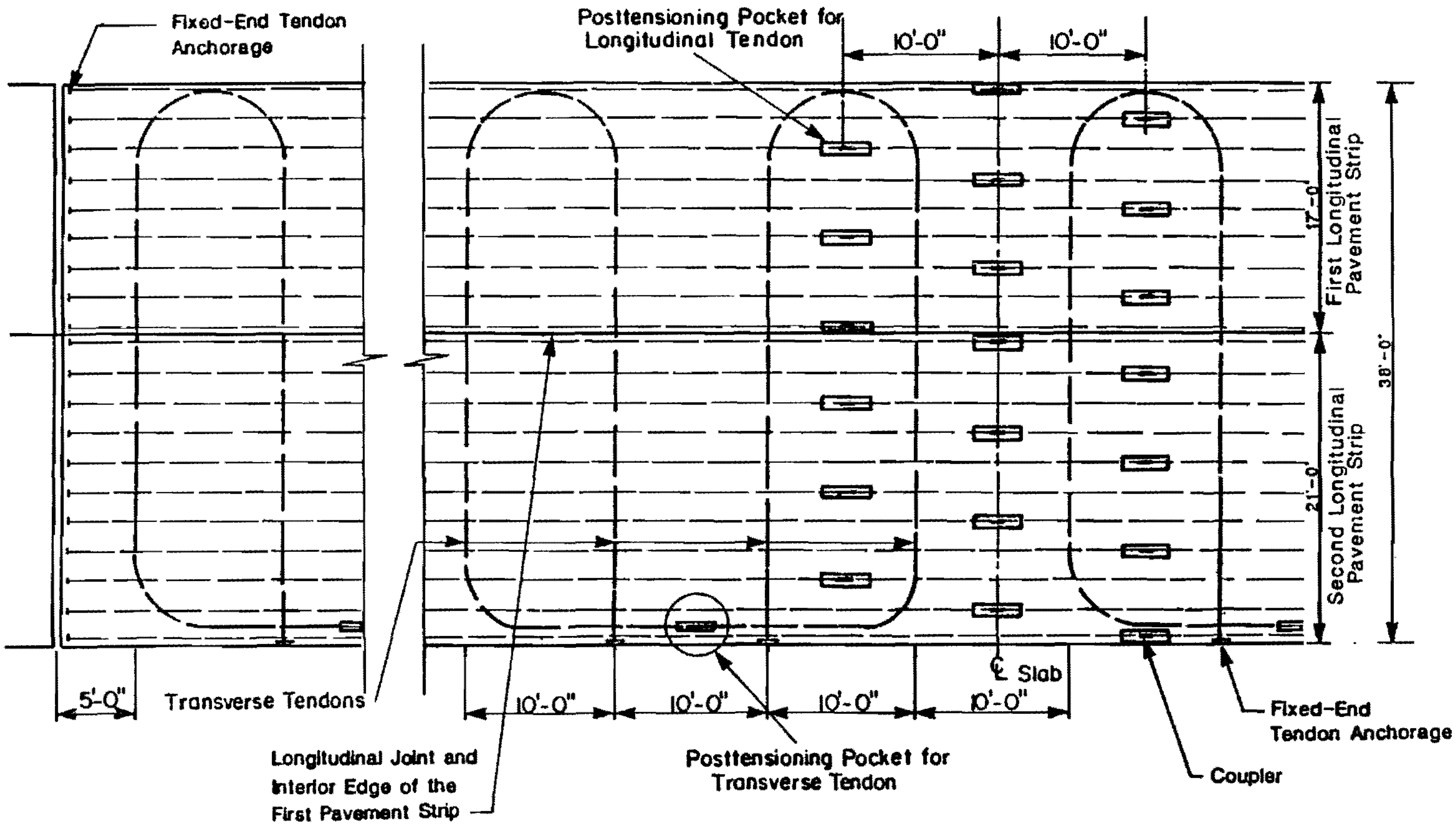


Fig 7.1. Plan view of the proposed pavement project developed by the Project 401 staff.

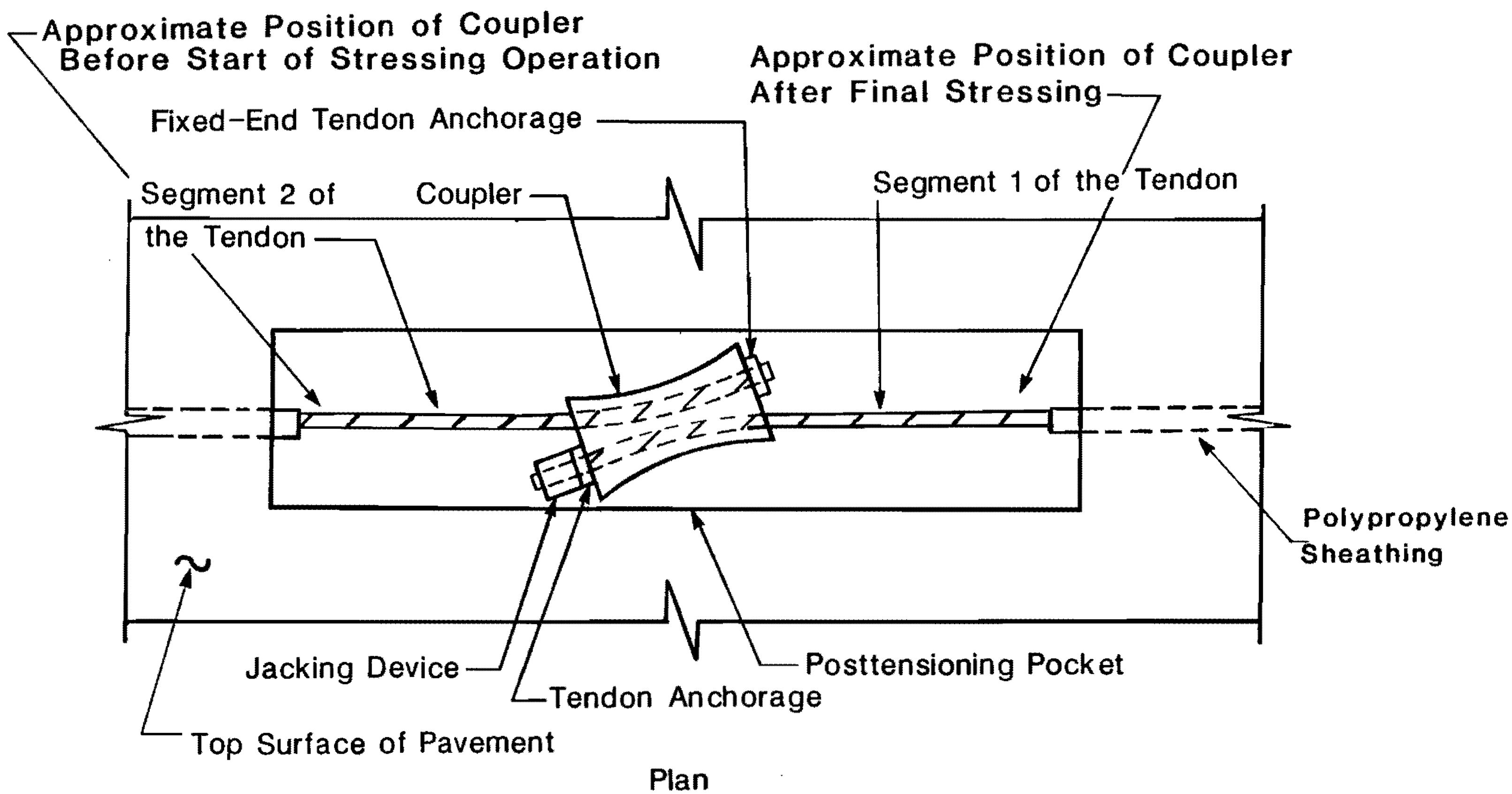


Fig 7.2. Stressing pocket detail.

permanent tendon anchors provided in the gap slab (Fig 7.3). Jacking bridges were provided behind each temporary tendon anchor to permit the release of the load on the temporary anchors. Each of these jacking bridges consisted of two steel bars, one above the strand and one below, which held the temporary tendon anchor a small distance out from the end of the pavement segment. Each tendon extended through the temporary tendon anchor and through the gap area between the previously stressed pavement segments and terminated at a permanent anchor adjacent to the active joint. A slab blockout was provided around each jacking bridge. After the concrete for the gap slab was cast and had attained sufficient strength, the jacking bridge was cut with a torch. This transferred the strand load from the temporary anchor to the permanent anchor, thus stressing the gap slab.

Several problems occurred in connection with this procedure. The first problem was that adequate consideration had not been given to the relationship between the span of the jacking bridges and the strength of the materials. Although this may seem like a fairly trivial error, it resulted in damage to the end region of the pavement slab and could have easily resulted in more serious consequences. The steel bars which comprised the jacking bridges deflected significantly under the load from final jacking of the tendons. In two cases the bars spread apart and actually slipped from behind the temporary anchors as the jacking bridges deflected. The rapid release of the tendon anchor at full load caused it to snap back against the end of the slab with great force. The first failure resulted in the anchor is smashing through the concrete and coming to rest approximately one foot into the slab. The second failure occurred adjacent to the first a few minutes later. Jacking was being done on the first tendon when failure of its jacking bridge occurred, but not on the second tendon. The second failure resulted in the anchor's coming to rest against the end of the slab. Flying steel and concrete accompanied both failures, but fortunately no injuries occurred. Although the problem with the jacking bridges was corrected by simply reducing their span length, this example does illustrate that overlooking seemingly minor details can have potentially serious consequences.

A second problem occurred during load transfer as a result of cutting through only the top bar of the jacking bridge. Although this did transfer the load, the uncut bottom bar of the bridge caused the temporary anchor to twist as it moved back, resulting in fracturing of the joint concrete around the anchor. To correct this problem, the welder was instructed to partially sever both the top and bottom bars of the jacking bridge to achieve an even release. Fewer fractures occurred as a result of this procedural change; however, the problem was never completely eliminated.

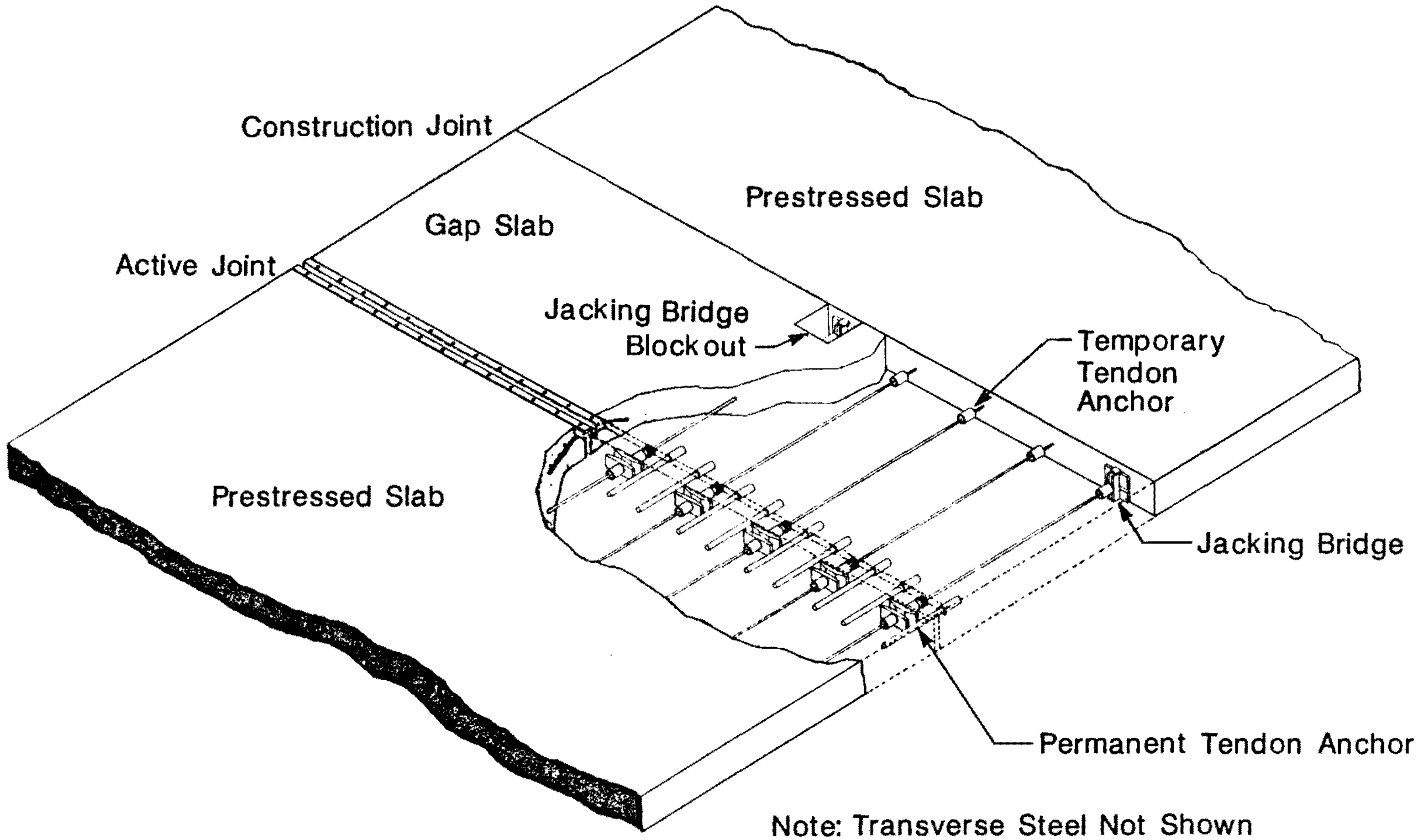


Fig 7.3. Gap slab with one active joint.

A third problem was associated with the permanent strand anchors. After the jacking bridges on several of the early joints were cut, the permanent anchors failed to hold, allowing the temporary anchors to move back against the original slab end. It was believed that the permanent anchors had become dislodged during placement and vibration of the gap slab concrete. Precautions were taken during subsequent construction operations to avoid disturbing the permanent anchors and no additional failures were observed.

One of the objectives identified in the early stages of Project 401 was to find a method of post-tensioning the strands which would eliminate gap slabs and the problems associated with their use but still permit taking advantage of the slip-form method of pavement construction. The most promising alternative was central stressing.

Central stressing is a procedure in which the strands are stressed at internal blockouts of stressing pockets. The blockouts are filled with concrete after the post-tensioning force has been applied. In addition to the fact that it eliminates the need for gap slabs, central stressing has the added advantages of centralizing all longitudinal tendon stressing operations and simultaneously applying approximately equal amounts of prestress at each end of the pavement section.

However, the use of central stressing is not without its problems. For instance, the following are foreseeable problems associated with forming the central stressing pockets for the longitudinal and transverse tendons.

(1) The proper location of the stressing pockets for each longitudinal and transverse tendon must be determined in the field. This is a time consuming, labor intensive task, especially if the prestressing tendons are not cut to the required lengths before being shipped to the job site. (A method using precut tendons is described in the following construction section and may lessen the time and labor required in the field to accomplish this task.)

(2) The stressing pocket forms have to be heldin position during concrete placement. This problem was experienced in the field by the Project 401 staff (Ref 10). (A possible method for avoiding this problem is also proposed in the following construction section.)

(3) The forms for the tendon stressing pockets may be hit by the bottom conforming screed of the passing slip-form paver.

(4) The vibrators attached to the slip-form paver have to be positioned to avoid disturbing the stressing pocket blockouts and that may result in poor concrete compaction in the vicinity of the stressing pocket blockouts. However, proper compaction of the concrete in these areas must not be neglected. Therefore, the use of hand-held vibrators is probably required in these areas.

(5) Special attention to finishing of the concrete surface in the vicinity of the stressing pocket blockouts may be required. The possible need for some hand-finishing in these areas was indicated in the field tests conducted by the Project 401 staff.

One of the disadvantages with the gap slabs used on the previous FHWA sponsored project was that they required an additional concrete placement operation and curing period after the final stress had been applied to the PSCP slabs, which slowed the entire construction process and delayed opening the pavement to traffic. Unfortunately, the use of central stressing pockets is similar to the use of gap slabs in this respect.

Concreting the central stressing pockets may be even more of a problem than for gap slabs because the steps of filling with concrete, vibrating to assure good compaction, striking the surface level, texturing to match, and cleaning the surface of the surrounding pavement of excess concrete and spillage from the concreting operation must be repeated for each stressing pocket (and there would be as many stressing pockets as there are tendons), whereas, for the gap slabs, the above steps have to be done only once for each pavement section.

Another problem associated with the use of central stressing is that it requires the use of tendon couplers (see Fig 7.2). This extra piece of hardware adds to the total cost of the pavement, especially since the coupler that appears to be best suited to PSC pavement construction is a proprietary item.

TRANSVERSE PRESTRESS

The designers of the previous FHWA sponsored projects did not believe that transverse stressing was necessary. In fact, some of the designers even felt that transverse reinforcing was unnecessary. However, after reviewing the available literature on the design and performance of previous projects, the staff of Project 401 strongly felt that transverse stressing was very important for resisting the applied wheel loads, preventing longitudinal pavement cracking, and preventing possible separation of separately placed pavement lanes or longitudinal pavement strips. Thus, transverse prestressing was used with this PSC pavement concept. Therefore, another objective identified in the early stages of Project 401 was to investigate alternate methods for transversely prestressing pavement.

A looped transverse tendon configuration, as shown in Fig 7.1, was selected. The first advantage of looped transverse tendons is that they allow transverse stressing of a greater length of the pavement with a single stressing operation than is possible with straight transverse tendons, thus reducing the amount of time required for this operation in the field. The second advantage of looped transverse tendons is that they permit the exterior edges of the pavement strips to be slip-formed without the need for formwork and without interference from protruding tendons.

Unfortunately, there are problems associated with the use of looped transverse tendons. The first problem is that it is difficult to lay out the looped transverse tendons in the field and hold them in the desired configuration. The second problem is in preventing the portions of each transverse tendon which protrude from the first pavement strip from interfering with slip-forming operations or becoming damaged while left exposed during the interim period between placement of successive pavement strips. The third problem with the use of looped transverse tendons is that the loops cause higher prestress losses, thus requiring the use of more tendons to obtain the desired level of transverse prestress.

The first two problems can be avoided by providing rigid hollow conduits in the first pavement strip along the desired paths of the transverse tendons. These conduits permit the installation of the looped transverse tendons to be delayed until immediately before the second pavement strip is to be placed, thus avoiding both problems.

RECOMMENDED CONSTRUCTION

In this section, a detailed description is given of the sequence to employ in the construction of a section of PSC pavement utilizing the concept. The construction sequence begins with the placement of the friction-reducing medium. It should be kept in mind, however, that major construction activities precede this step. These activities include preparation of the subgrade, placement of the subbase course, placement of the base course for new PSC pavement construction, preparation of an existing pavement, and placement of a levelling course (usually of asphaltic concrete) for PSC pavement overlay construction. These aspects of the construction sequence are not discussed in this report for two reasons: (a) these construction activities are essentially the same for all of the various PSC pavement concepts and do not differ appreciably from similar activities for conventional concrete pavements and (b) a worthwhile discussion of this topic is beyond the scope of this report.

For the purpose of this discussion it is assumed that two or more passes of the slip-form paver are required to construct the full pavement width. Although construction of the full pavement width in one pass eliminates the longitudinal construction joint between adjacent pavement sections and the problems associated with it, full-width construction is often not feasible. PSC pavement construction using multiple passes of the slip-form paver was not dealt with on the previous FHWA sponsored projects. In order for PSC pavement to be a truly viable option, this type of construction must be examined and problems associated with its use identified and solved.

Step 1

The first step in the anticipated construction sequence is placement of the friction-reducing medium for the first longitudinal strip of PCP on (a) the prepared subgrade for new pavement or (b) the leveling course over old pavement in the case of an overlay. Even with the problems encountered on the previous FHWA demonstration projects, a double-layer of polyethylene sheeting is desirable in order to reduce the subgrade drag and thus minimize the number of tendons required to obtain the design prestress level.

The polyethylene should be provided in double-layer rolls so that both layers are placed in a single construction operation. The width of the rolls should be at least one foot wider than the width of the pavement strip under which it is to be placed. This will help insure that the pavement is not placed in direct contact with the base course. If polyethylene sheeting is not available in this width, which was the case on the previous FHWA sponsored projects, narrower width sheeting can be used provided that it is placed in either longitudinal or transverse contiguous strips. In either case, the strips should be lapped and secured together to prevent them from shifting during concrete placement which could cause the concrete to be placed in direct contact with the base course.

There is considerable room for innovation in the techniques or specialized equipment for laying the polyethylene sheeting. Experimentation in this area by the contractor should be encouraged as long as a continuous, unrestrained double-layer of polyethylene is obtained and the quality of the pavement is in no way compromised.

Step 2

After placement, the sheeting should be anchored to the base course with tacked-down sheet metal strips placed along the outside edges to prevent it from being displaced by the wind.

Step 3

Locations of the transverse pavement joints should be determined next. In the case of an overlay, care must be exercised in this step to insure that the transverse pavement joints are not located over joints or cracks in the old pavement. After the locations have been determined, the transverse joint assemblies should be set in place. The transverse joint assembly that was proposed for use with this pavement concept is shown in Fig 5.11. This assembly would be completely fabricated and assembled in the shop, including the neoprene seal, dowels, headed studs or deformed bar anchors, and all necessary bolts for attachment of tendon anchorages. Small steel jumper plates should be provided across the top of the joint assembly and tack welded or bolted on each side of the joint opening to keep the joint assembly in a closed configuration. The joint assemblies should be shipped to the job site ready for installation in the pavement and would require no field assembly. After they are set in place, the assemblies must be temporarily secured to the subgrade to prevent them from being inadvertently displaced and so they will be able to withstand the imposed loads caused by placing a small tensile force on the longitudinal tendons to keep them straight during slip-forming of the pavement.

Step 4

Strand chairs should then be placed. Continuous chairs would be the most logical type of chair to use for this construction. Individual chairs or continuous chairs should be placed across the pavement at a predetermined interval. The interval would be dependent on the amount of tension that could be applied to the longitudinal tendons before the pavement is placed. Of course, the higher the tension, the greater the interval could be between chairs.

Step 5

The next step in the construction of the first PSC pavement strip is laying out the longitudinal tendons. Before they are shipped to the construction site, the longitudinal tendons should be prepared as follows in order to prevent confusion in the field and permit smooth construction operations:

(1) They should be cut to the proper lengths. As shown in Fig 7.1 the tendon stressing pockets were staggered. This required that the tendon be provided in several different lengths. Cutting the tendons to these required lengths is done much more efficiently and accurately in the shop.

(2) Tendon anchorage hardware should be correctly installed in the shop. On some of the previous projects the anchorages were mounted on the tendons in the field. In some cases, they were incorrectly installed and, as a result, they failed to hold. Recovery of the strand ends was often difficult and sometimes impossible.

(3) The polypropylene sheathing should be removed from the end of every tendon which is to be inserted through a coupler. This is a seemingly small task but it is tedious, time consuming, and doesn't belong in the field, where it might hold up production.

The most efficient method of longitudinal tendon placement would probably be to have the ends of the two segments of tendon which comprise one complete longitudinal tendon rolled onto a common reel, where the ends of the two tendon segments which are to be inserted through a coupler would be lapped and temporarily wired together. Each reel should be identified with a tag, corresponding to shop drawings, which clearly indicates the tendon's proper location in the pavement slab. This is necessary in order to obtain the desired stagger of the tendon stressing pockets. The anchorage on the leading end of the tendon should then be bolted to the previously placed transverse joint assembly and the reels unwound. As with many other aspects of PSC Pavement construction, there is room for innovation in the handling of the tendon reels. On the Hogestown, Pennsylvania, project, strand reels were carried on a modified flatbed truck from which they were unwound onto the pavement base course. Other types of special reel handling trailers could easily be developed. At the end of the pavement strip, the ends of each tendon could be bolted to the other transverse joint assembly. The locations of the staggered tendon stressing pockets would then be automatically located.

Step 6

Half forms (or "false" forms as they are sometimes called) should be placed along the interior longitudinal edge of the first pavement strip to form the edge of the slab below the level of the transverse tendons and to support the protruding ends of the transverse tendons.

Step 7

The next step in the construction sequence is to lay out and place the looped transverse tendons. These tendons may be laid out in the required looped configuration either at their location in the pavement or in the contractor's yard, transported to the construction site, and set in place. In either case a jig could be used to facilitate layout. In addition, it will probably be necessary to provide light-gauge wire ties in order to maintain the looped tendons in the desired shapes. During placement of the first pavement strip, the portions of each transverse tendon extending outside this strip must be either temporarily coiled or otherwise restricted to a narrow band along the edge of this strip to prevent the protruding transverse tendons from interfering with the passage of the slip-form paver.

Another option would be to provide hollow conduits in the first pavement strip along the desired paths of the transverse tendons. These conduits would terminate at the interior longitudinal pavement edge so nothing would protrude from the first pavement strip to interfere with paving operations. The ends of the conduits should be temporarily capped to prevent intrusion of concrete or other foreign matter such as water or dirt. Before placement of the second pavement strip, the transverse tendons should be threaded through these conduits.

The previously described steps of placing the polyethylene sheeting, locating and setting the transverse joint assemblies, placing the longitudinal tendons, setting the half forms along the interior longitudinal edge of the first pavement strip, and placing the transverse tendons could be integrated. Two advantages would be gained if this were done. First, the whole construction process would be made more systematic. Second, placement of the tendons on the polyethylene immediately after it is put down would help hold it in place and make it less vulnerable to the wind. This type of construction would favor the use of transverse tendons or hollow conduits laid out in the contractor's yard, transported to the construction site, and set in place.

Step 8

The next step in the construction sequence is to place the forms for the stressing pockets over the longitudinal tendons where the two segments have been lapped and wired together. The forms should have the following features:

(1) Slots should be provided to permit placement over the longitudinal tendons. Provision should be made for closing the slot below the tendons after the form is in place to prevent intrusion of concrete.

(2) They should have covers which can be closed and secured in place so that no concrete enters the forms during slip-form paving.

(3) They should be slightly shallower in depth than the pavement thickness to prevent them from being disturbed during passage of the slip-form paver.

(4) The forms should be tapered slightly inward toward the bottom. In addition, their exterior surfaces should be oiled. Both of these measures will allow the forms to be removed more easily.

(5) They should be of sturdy, reusable, possibly metal construction.

(6) They should have some provision which would allow them to be temporarily secured to the asphaltic concrete leveling course. This temporary attachment would facilitate aligning the forms and also prevent them from floating up or otherwise displacing during concrete placement.

Step 9

Next, sufficient tension should be applied to the longitudinal tendons to keep them in straight alignment during concrete placement. The magnitude of the required tension was discussed. This tension might be applied by means of a system of steel springs in the slab blockout forms.

Step 10

After the tensile stress necessary to keep the longitudinal tendons in straight alignment has been applied, the longitudinal and transverse tendons should be tied together at their points of intersection. This will help stabilize the tendon mat and keep the tendons from being displaced during slip-forming of the pavement. Care should be exercised when tying the tendons together to prevent cutting through the polypropylene tendon sheathing, which was done on some of the previous FHWA sponsored projects. To help protect against damage to the sheathing, the use of plastic strip ties or nylon reinforced tape in lieu of metal wire ties should be investigated.

Step 11

Concrete placement using a standard slip-form paver may commence after a sufficient number of pavement sections have been prepared (according to the steps described above) so that progress of the slip-form paver is not impeded. Special care must be exercised in placing, vibrating, and finishing the concrete in the vicinity of the transverse pavement joint assemblies and forms for the tendon stressing pockets.

Step 12

When the concrete has achieved sufficient compressive strength (approximately 1000 psi), the following items must be removed: stressing pocket forms, anchors used to temporarily secure the transverse joint assemblies to the asphaltic concrete leveling course, steel jumper plates, which were provided across the transverse joint assembly opening to keep it in a closed configuration, and the half forms on the interior longitudinal edge of the pavement. The longitudinal tendons would then be sufficiently stressed (in the formed pockets) to prevent shrinkage cracking of the concrete. After the concrete has gained sufficient additional strength, final tensioning of the longitudinal tendons should be done. This should be followed by coating the pocket faces with epoxy bonder and then filling the stressing pockets with concrete. High-early-strength concrete could be used for filling the pockets if it would help to expedite opening the pavement to traffic.

Step 13

Placement of the second pavement strip probably shouldn't occur until after completion of the entire length of the first pavement strip. Depending on the total project length, the intervening length of time between the construction of the two pavements strips could range from several weeks to several months. At the very least, construction of the second pavement strip must wait until traffic can be permitted on the first pavement strip.

Placement of the second pavement strip should proceed in a manner similar to the first with a few exceptions. The polyethylene friction-reducing medium, the transverse joint assemblies, the tendon support chairs, and the longitudinal tendons should be placed as described above.

Step 14

Next, the coiled portions of the transverse tendons which protrude from the first slab strip should be unrolled and made to conform to the configuration shown in the second pavement strip in Fig 7.1. If hollow conduits were provided in the first pavement strip instead of the transverse tendons, then the tendons should be threaded through the conduits.

Step 15

Light tension should be applied to the longitudinal tendons and the longitudinal and transverse tendons should then be tied together as in the first pavement strip.

Step 16

Tendon stressing pocket forms should then be placed over both the longitudinal and transverse tendons at the locations of the tendon laps.

Step 17

Either a strip of polyethylene sheeting or a spray-applied bond breaking compound should then be placed on the interior longitudinal edge of the first pavement strip. This would prevent the second pavement strip from bonding to the first strip and would allow relative movement between the two during longitudinal stressing of the second strip. No half forms would be required on the interior longitudinal edge of the second pavement strip.

Step 18

Next, the concrete of the second pavement strip should be slip-formed in place. After the longitudinal tendons of the second pavement strip have been fully stressed (in two stages as described above), the transverse tendons should be stressed.

Step 19

Construction of the second pavement strip should be completed by filling the tendon stressing pockets with concrete.

RECOMMENDED TENDON PLACEMENT

The importance of applying sufficient preliminary tension to the post-tensioning tendons to straighten and hold them in place during concreting operations was recognized even on the very early PSC Pavement projects. The tendons need to be straight and securely held in place to assure that the desired compressive stress distribution is obtained on the pavement cross section and that wobble friction losses due to unintentional misalignment of the tendons is minimized. The importance of the preliminary tension on the post-tensioning tendons was graphically demonstrated during field tests which were conducted near Valley View, Texas, by personnel from the Center for Transportation Research of The University of Texas at Austin (Ref 10). In these tests lateral displacement of unstressed post-tensioned strands occurred during concrete placement. The preliminary stress level recommended on one of the early PSC pavement projects was 10,000 psi (Ref 14). However, the actual value which should be used on a given project is a function of several variables, including tendon size, chair spacing, concrete placement techniques, etc.

Tubes or J-bars attached to the paver were used on most of the previous FHWA sponsored projects to position the longitudinal tendons in the pavement during slip-forming. One of the main advantages of this method was that it eliminated the need to support the longitudinal tendons on chairs during slip-forming. However, use of tubes or J-bars attached to the paver is much less feasible when transverse prestressing (or reinforcing for that matter) is provided. The reasons for this are as follows:

(1) Obviously, the use of this method would prevent preplacement of the transverse tendons on top of the longitudinal tendons since they would conflict with passage of the paver. The only other way to place the transverse tendons would be to depress them into the pavement after the paver has passed. This would probably be a difficult operation.

(2) The probability is great that the tubes or the J-bars attached to the paver would snag the transverse tendons if they were located below the longitudinal tendons.

(3) Regardless of whether the transverse tendons are on top of or below the longitudinal tendons, they need to be tied to the longitudinal tendons to prevent them from being displaced by the concrete mass which is pushed ahead of the slip-form paver. This in itself would prevent the tubes or J-bars from sliding along the longitudinal tendon. For these reasons, the tendons must be supported on chairs during slip-form paving.

TRANSVERSE JOINTS

Another potential problem in this kind of construction is in holding the transverse joint assembly stationary during the application of the pretension to the longitudinal tendons, which is done for two primary reasons: (1) to keep the tensions in straight alignment during concrete placement so that the tendons remain in the desired position in the pavement and (2) to minimize the amount of wobble friction loss in the tendons. The tension recommended in previous reports was 10,000 psi (Ref 14). It may be extremely difficult (if not impossible) to temporarily secure the transverse joint assembly to the subgrade to resist the force resulting from stressing the longitudinal tendons on only one side of the joint assembly to 10,000 psi. Therefore, the only feasible way of applying this level of tension to the longitudinal tendons before the concrete is placed would be to lock the two halves of the transverse joint assembly together and attempt to apply the pretension to the tendons on both sides of the joint at approximately the same time. Some type of reaction (possibly a heavy piece of machinery) must be provided at the first and last transverse joint assemblies in a run of pavement slabs to resist the unbalanced load at these locations. Temporary attachment of each transverse joint assembly to the subgrade would still be necessary to resist the inevitable discrepancies between the loads applied to each side of the assembly.

Placement and consolidation of the concrete in the vicinity of the transverse joint assembly would be another potential problem area. Due to the presence of the joint assembly, the vibrators attached to the slip-form paver must be raised out of the concrete. Thus a region of poorly compacted concrete would be created on either side of the joint assembly. To compensate for this problem, use of hand-held vibrators in this area would probably be required.

PROTECTION OF TENDON ANCHORAGES

On some of the previous FHWA sponsored projects the longitudinal tendon anchorages were exposed in the transverse pavement joints. Two potential problems are forseeable with this arrangement. The first problem is that the anchorages are more vulnerable to corrosion and hence loss of pavement prestress. The second problem is that, if the tendon anchors are left exposed in the transverse joint, they are vulnerable to damage caused by closure of the joint in hot weather. However, when central stressing is used, access to the end anchorages is not necessary in order to post-tension the tendons. Therefore, the end anchorages would be completely encased in the concrete pavement and, consequently, would not be subject to either of the previously described problems.

CHAPTER 8. INSTRUMENTATION FOR THE PRESTRESSED PAVEMENT IN MCCLENNAN COUNTY

Prestressed concrete pavements are most effectively designed using predicted values of concrete stress and slab movement. For a given pavement design, concrete stress and slab movement depend mainly on thermal effects due to daily temperature cycles. Design of prestressed pavements must also consider long-term effects of shrinkage, creep, and traffic loads. With these design considerations in mind, an instrumentation scheme was proposed for monitoring the behavior of the prestressed pavement to be built near Waco. The instrumentation plan proposed herein was for the section in McClennan County which was scheduled to be constructed first. For the section in Cooke County, a similar instrumentation plan could have been implemented.

The instrumentation for this demonstration project was intended to serve two main purposes: (1) to provide a verification of predicted values of concrete stress and slab movement and (2) to periodically monitor the condition and behavior of the pavement over the long term.

This report presents some of the background and reasoning behind the instrumentation scheme and presents specific recommendations for instrumentation at the Waco site (the McLennan County Project).

INSTRUMENTATION FOR PREVIOUS PROJECTS

Before an instrumentation scheme was proposed for the Waco site, instrumentation used on similar pavement projects was reviewed. These projects included prestressed pavement installations in Pennsylvania, Virginia, Mississippi, and Arizona, and slab tests conducted at Rolla, Missouri, and Slidell, Louisiana (Refs13, 14, 16, 15, 18, and 30). Reports of these projects provided useful information on instrumentation equipment and procedures. The instrumentation used in three of these projects is summarized in Tables 8.1, 8.2, and 8.3. In these projects, the followin items were monitored: ambient temperature, concrete temperature, concrete strain, absolute horizontal movement, curling movements, and joint width.

TABLE 8.1. PAVEMENT INSTRUMENTATION FOR THE PRESTRESSED PAVEMENT PROJECT AT DULLES, VIRGINIA

CONSTRUCTED DEC., 1971.
6 SLABS TOTAL, VARYING IN LENGTH FROM 400' TO 760'.

AMBIENT TEMPERATURE	
CONCRETE TEMPERATURE	THERMOCOUPLES AT 4 DEPTHS. HELD IN PLACE IN VERTICAL PLEXIGLASS RODS. 12-CHANNEL CONTINUOUS TEMPERATURE RECORDER. 4 TOTAL LOCATIONS IN 3 SLABS.
CONCRETE STRAIN	1) VIBRATING WIRE GAGES AT 3 DEPTHS. "HAVE NOT PRODUCED USABLE DATA." 12 LOCATIONS IN 3 SLABS. 2) 50" LENGTH CHANGE GAGE. MECHANICALLY MEASURES SURFACE LENGTH CHANGE BETWEEN TWO COUNTERSUNK POINTS. 10 TOTAL LOCATIONS IN 4 SLABS.
ABSOLUTE HORIZONTAL MOVEMENT	PIPE-WIRE-SCALE. ACCURATE TO 1/2 mm. (.02") 17 TOTAL LOCATIONS, ALL SLABS.
CURLING MOVEMENTS	1) PROFILER BEAM. PROFILE LINES 140" LONG WITH 15 PROFILE POINTS. VERTICAL DEFLECTION TO .0001". 10 LOCATIONS IN 4 SLABS. 2) 20" CLINOMETER. MEASURES SLOPE CHANGE ALONG PROFILE LINES.
JOINT WIDTH	1) 96" LENGTH CHANGE GAGE. MEASURES RELATIVE MOVEMENT OF PRESTRESSED SLABS. MEASURED ACROSS GAP SLAB. 2) DIAL EQUIPPED SLIDE CALIPER. MEASURES MOVEMENT OF ONE PRESTRESSED SLAB RELATIVE TO THE GAP SLAB. MEASURED AT ALL JOINTS.
OTHER	

TABLE 8.2. PAVEMENT INSTRUMENTATION FOR THE PRESTRESSED PAVEMENT PROJECT AT BROOKHAVEN, MISSISSIPPI

CONSTRUCTED OCT., 1976.
58 TOTAL PRESTRESSED SLABS, 450' LONG.

AMBIENT TEMPERATURE	CONTINUOUS TEMPERATURE RECORDER.
CONCRETE TEMPERATURE	TEMPERATURE SENSORS AT 4 DEPTHS. MICROMEASUREMENTS TYPE ETG-50D GAGE BONDED TO COPPER TUBING. RECORDED TO 1°F. 2 SLABS.
CONCRETE STRAIN	1) CONCRETE EMBEDMENT GAGES AT 3 DEPTHS. DID NOT PRODUCE USABLE DATA. 4 SLABS. 2) STRAIN GAGED BARS. 3' LONG, 1/2" DIA. STEEL BARS WITH STRAIN GAGES ATTACHED 12 TOTAL LOCATIONS IN 4 SLABS.
ABSOLUTE HORIZONTAL MOVEMENT	PIPE-WIRE-SCALE. MEASURED ON BOTH ENDS OF 3 SLABS.
CURLING MOVEMENTS	SURVEY EQUIPMENT. MEASURED ELEVATION CHANGE IN 3' INTERVALS ALONG SLAB ENDS AND 5' INTERVALS ALONG LENGTH. MEASURED TO .001" 2 SLABS.
JOINT WIDTH	DIAL CALIPERS AND REFERENCE PLUGS. ACCURATE TO .001". MOST SLABS.
OTHER	TENDON ELONGATION, MODULUS OF ELASTICITY OF THE CONCRETE, DYNAFLECT DEFLECTION, PCA ROADMETTER, SKID RESISTANCE.

TABLE 8.3. PAVEMENT INSTRUMENTATION FOR THE PRESTRESSED PAVEMENT PROJECT AT TEMPE, ARIZONA

CONSTRUCTED APRIL, 1977.
30 PRESTRESSED SLABS, 400' IN LENGTH.

AMBIENT TEMPERATURE	THERMISTORS. RECORDED TO 1°F.
CONCRETE TEMPERATURE	EMBEDDED THERMISTORS AT 3 DEPTHS. CONTINUOUSLY RECORDED WITH A TIMED AUTO DATA ACQUISITION DEVICE. RECORDED TO 1°F. INSTALLED IN 3 SLABS NEAR SLAB END, 1 SLAB NEAR MIDSLAB.
CONCRETE STRAIN	EMBEDMENT GAGES PLACED AT 3 DEPTHS. AILTECH CG129 GAGES. CONTINUOUSLY RECORDED. DATA IS "QUESTIONABLE." INSTALLED IN 3 SLABS NEAR SLAB END, 1 SLAB NEAR MIDSLAB.
ABSOLUTE HORIZONTAL MOVEMENT	SLAB LENGTH CHANGE. FOUND BY "DISTANCE MEASURING EQUIPMENT." RECORDED TO .001". 1 SLAB
CURLING MOVEMENTS	DIAL GAGE AT SLAB CORNER. REFERENCED TO IRON PIN DRIVEN INTO SUBGRADE. ACCURATE TO .001". 2 SLABS.
JOINT WIDTH	EXTENSOMETER BAR. ACCURATE TO .001". 5 SLABS.
OTHER	VISUAL CRACK SURVEY, ALL SLABS.

Thermocouples have proven effective in measuring both ambient temperature and concrete temperature. For concrete temperature, the thermocouples are fixed at known depths of the slab to obtain the temperature gradient through the thickness of the slab. It is best to position these gages after the concrete has been placed. In the above projects, the thermocouple leads were carried through PVC conduit out the side of the slab. This conduit was put into place before the paving operations. In the case of the Virginia pavement, the temperature gages were placed in 12–inch by 12–inch box-outs before the placement of the concrete. After the slip-form paver had passed, concrete was placed and vibrated around the arranged gages.

All three of the above projects reported trouble in recording concrete strain using embedment gages. For each of the projects, the strain gages were installed in the same manner as the temperature gages, with the gage leads carried through PVC conduit to the edge of the pavement. In explaining their failure to obtain usable data from the strain gages, the reports from the Virginia and Mississippi projects described the particular gages as being "not sufficiently rugged" for field use. Concrete strain measurements were successfully recorded in slab tests at Slidell, Louisiana, using strain gages made by T.M.L. The gages used were polyester molded gages, model PML - 60.

In Arizona, temperature and strain data were continuously recorded using a timed automatic data acquisition device. In the Virginia project, concrete temperature was continually recorded. Readings of concrete temperature and strain were taken intermittently using a digital strain indicator.

The Mississippi project also installed strain gaged bars to measure concrete strain. This apparatus was made in-house from a 1/2-inch-diameter steel bar, 3 inches long, with contact plates welded to the ends to provide bond with the concrete. Strain gages were attached to the bar to compensate for bending, temperature, and Poisson's effect. The strain taken by the steel bar was assumed equal to the strain in the concrete.

In the Virginia project, concrete strain was found by mechanically measuring the surface length change of the concrete slab. The length change was measured between countersunk profile points, 50 inches apart, mounted to the slab surface. The measuring apparatus used a Nilvar bar and a movement dial gage. Concrete strain at the interior of the slab was calculated using measurements of surface curvature and a straight line projection of surface strains.

On two previous projects, horizontal slab movement was found using a pipe-wire-scale apparatus. The reference for this measurement was a wire stretched across the pavement between two embedded vertical pipes. The wire was tightened to a specified tension to minimize variance due to wind. A scale was mounted in a pre-set position in the pavement surface so that movement of the pavement could be read according to where the wire crosses the scale. This set-up is pictured in Fig 8.1.

Slab curling movements have been measured in a variety of ways. For the Virginia project and for slab tests at Rolla, Missouri, slab curvature was found along profile lines. For the Mississippi project, survey equipment was used to measure elevation changes across sections of the pavement slab. In the Arizona project, movement of the slab corners was measured on dial gages. Iron pins driven into the subgrade were used to mount the dial gages and thus served as a reference for the movement.

For the projects in Virginia, Mississippi, and Arizona, the changes in width of the transverse joints were measured mechanically using an extensometer or similar device. All of these projects measured the joint opening between the prestressed slab and the gap slab. In Virginia, the width across the gap slab was also measured.

In the Mississippi project, tendon elongation was measured to verify the magnitude of the prestressing force. In Arizona, a visual crack survey was reported. All pavement slabs were inspected for cracks before the application of the prestress, and intermittently thereafter.

OBJECTIVES OF MEASUREMENTS

Before specifying instrumentation equipment for the Texas projects, the objectives of the measurements were defined. Instrumentation was required for three important reasons:

(1) to verify the values of concrete properties, movements, and stresses assumed in the design;
(2) to obtain a set of field data to calibrate the design procedure, and
(3) to monitor the pavement performance continuously and analyze the deterioration trend of its elements.

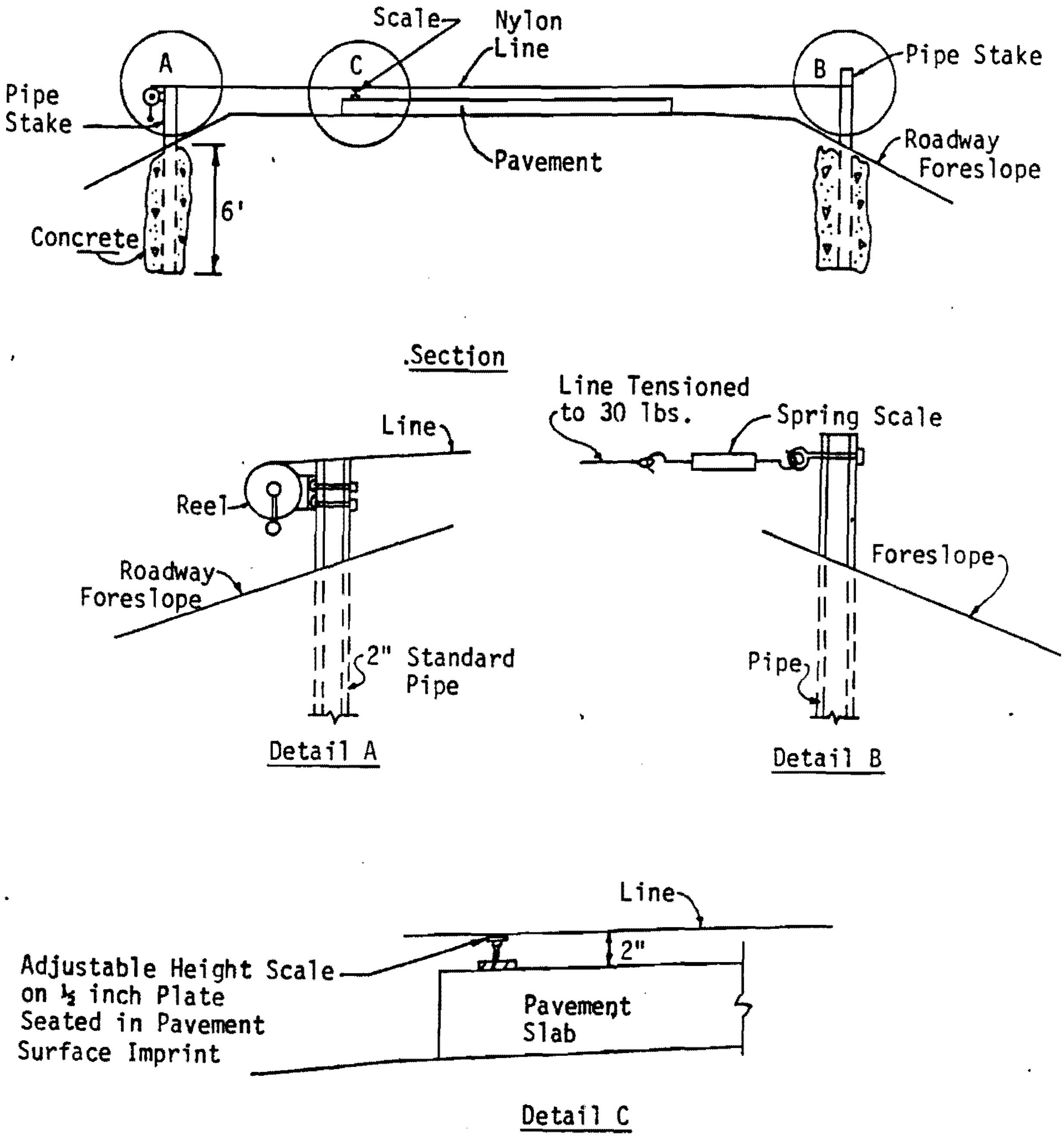


Fig 8.1. Pipe-wire-scale apparatus for measruing horizontal slab movement.

The first and second aspects coulb be accounted for by short-term measurements. Long-term instrumentation was required to cover the third aspect.

Objectives of Short-Term Measurement

Short-term measurements are needed to research the following aspects:

(1) The effect of ambient temperature on several pavement responses. Some of these responses are related to the development of temperature profiles through the slab depth. Temperatures at top, bottom, and mid-depth should be recorded continuously and correlated with slab longitudinal movements and curling deflections. Measurements of mid-depth temperatures together with longitudinal movements may allow for estimating the concrete coefficient of thermal expansion and the subbase frictional resistance. Temperature differentials from top to bottom of the slab, along with corner vertical deflections, may allow for determining the magnitude of the curling stresses in the interior portions of the slab.

(2) The effect of the longitudinal prestress on slab movements, friction forces and concrete stresses. If the longitudinal movements are recorded at the time of post-tensioning, the maximum coefficient of friction may be evaluated. Also, if the slab being prestressed is on the outside lane, the recording of the relative movement between this slab and the slab alongside might indicate whether or not bonding is occurring at the longitudinal joint.

(3) The effect of the transverse prestress on movements and width of the longitudinal joint can be detected.

(4) Measurements of tendon elongation are required to verify that the desired amount of prestress is being applied on the concrete.

(5) A continuous condition survey on the slabs during the first 48 hours after construction may allow for detection of early cracks and correlate their occurrence with the time of construction and magnitude and time of application of post-tensioning flow.

(6) Concrete compressive strength tests verify that the strength gain curve assumed in the design is correct. The elastic properties of concrete at early ages may also be defined.

Objectives of Long-Term Measurements

The purpose of long-term measurements is to determine how the pavement behaves with time. The following readings are important.

(1) Readings of horizontal slab movement and transverse joint openings may indicate whether or not the friction properties of the subgrade are changing with time, and show the extent of any bonding that may develop between adjacent slabs.

(2) Long-term monitoring of vertical slab movements may indicate whether temperature changes, increasing concrete stiffness, or differential shrinkage affects the curling behavior of the slab.

(3) Load transfer measurements at the transverse joints can show if the effect of traffic is causing progressive deterioration of the joint.

(4) Periodic condition surveys are necessary to keep a record of the development of different distress types on the pavement.

INSTRUMENTATION SCHEME

According to the objectives described above, the proposed instrumentation scheme was divided into two categories: short-term instrumentation and long-term instrumentation.

Short Term Instrumentation

The short-term instrumentation proposed for the Waco site consisted of (1) continuously recorded electronic instrumentation, (2) mechanical instrumentation to back up the continuously recorded measurements, and (3) intermittently recorded instrumentation. Short-term instrumentation was implemented on at least two pavement slabs at Waco. Since the slabs were staked at mid-length, behavior of the two halves of a slab was symmetric, and only one half of the pavement slab needed to be monitored. A minimum of one 240-foot-long slab and one 440-foot-long slab were instrumented. Instrument locations are shown in Figs 8.2, 8.3, and 8.4.

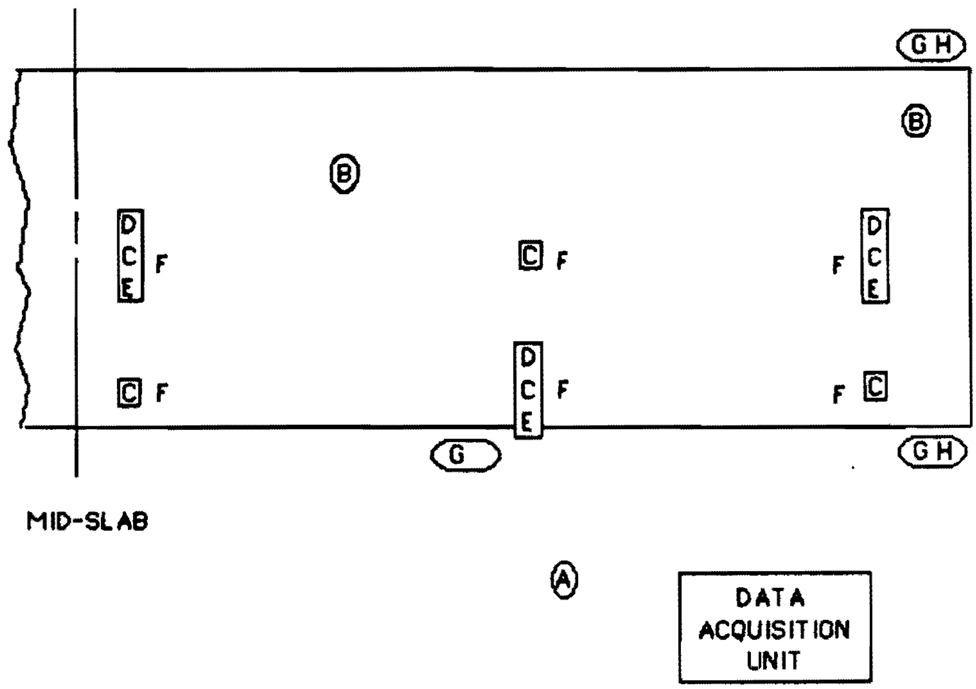
The continuously recorded electronic instrumentation is shown in Fig 8.2. This instrumentation consisted of thermocouples for measuring temperature, strain gages for measuring concrete strain, and displacement transducers for measuring horizontal and vertical movements.

Thermocouples were used to monitor both ambient temperature and concrete temperature. For concrete temperatures, the thermocouples were placed at three depths to obtain the temperature gradient through the thickness of the slab. For each slab there were two locations for the embedded thermocouples: one near the center of the pavement slab and one near the slab edge. The ambient temperature measurement consisted of an identical thermocouple placed in the shade outside of the slab.

Concrete strain was recorded using polyester molded embedment gages. For each monitored slab, these gages were placed in six locations in the concrete. In three of the locations, the gages were placed at three depths to obtain the strain gradient through the thickness of the slab. In the other three locations there was only one embedment gage, placed at mid-depth. At all six locations there were surface gages mounted above the embedment gages. Measurements of concrete strain were converted to values of stress by multiplying by the modulus of elasticity of the concrete.

Movement of the slab was monitored using displacement transducers mounted at the slab corners. At each of the two corners of the half-slab, there was a transducer to record vertical deflection, and a transducer to record horizontal deflection. In addition, there was a transducer mounted along the pavement edge, closer to mid-slab, to record horizontal movement.

The data from the instrumentation were continuously recorded over several daily temperature cycles. The automatic timed recording of several channels of input necessitated the use of a data acquisition system. Such a system is shown schematically in Fig 8.5. The voltage signals from the thermocouples, strain gages, and transducers are carried into the front end of the data acquisition device. In the case of remote gages, the voltage signal must be amplified by a pre-amp located near the gages. In the front-end of the data acquisition system, the input buffer conditions the signal by adjusting the amplitude of its voltage. Subsequently,



MEASUREMENT	NO. OF LOCATIONS	NO.OF INPUTS	APPARATUS
A-AMBIENT TEMPERATURE	1	1	THERMOCOUPLE
B-SLAB TEMPERATURE AT 3 DEPTHS	2	6	
C-MID-DEPTH CONCRETE STRAIN	6	6	EMBEDMENT GAGE
D-1/2" DEEP CONCRETE STRAIN	3	3	
E-5 1/2" DEEP CONCRETE STRAIN	3	3	
F-SURFACE CONCRETE STRAIN	6	6	SURFACE GAGE
G-ABSOLUTE HORIZONTAL MOVEMENT	3	3	DISPACEMENT TRANSDUCER
H-ABSOLUTE VERTICAL MOVEMENT	2	2	

Fig 8.2. Continuously recorded electronic instrumentation. Instrument location for a half-slab.

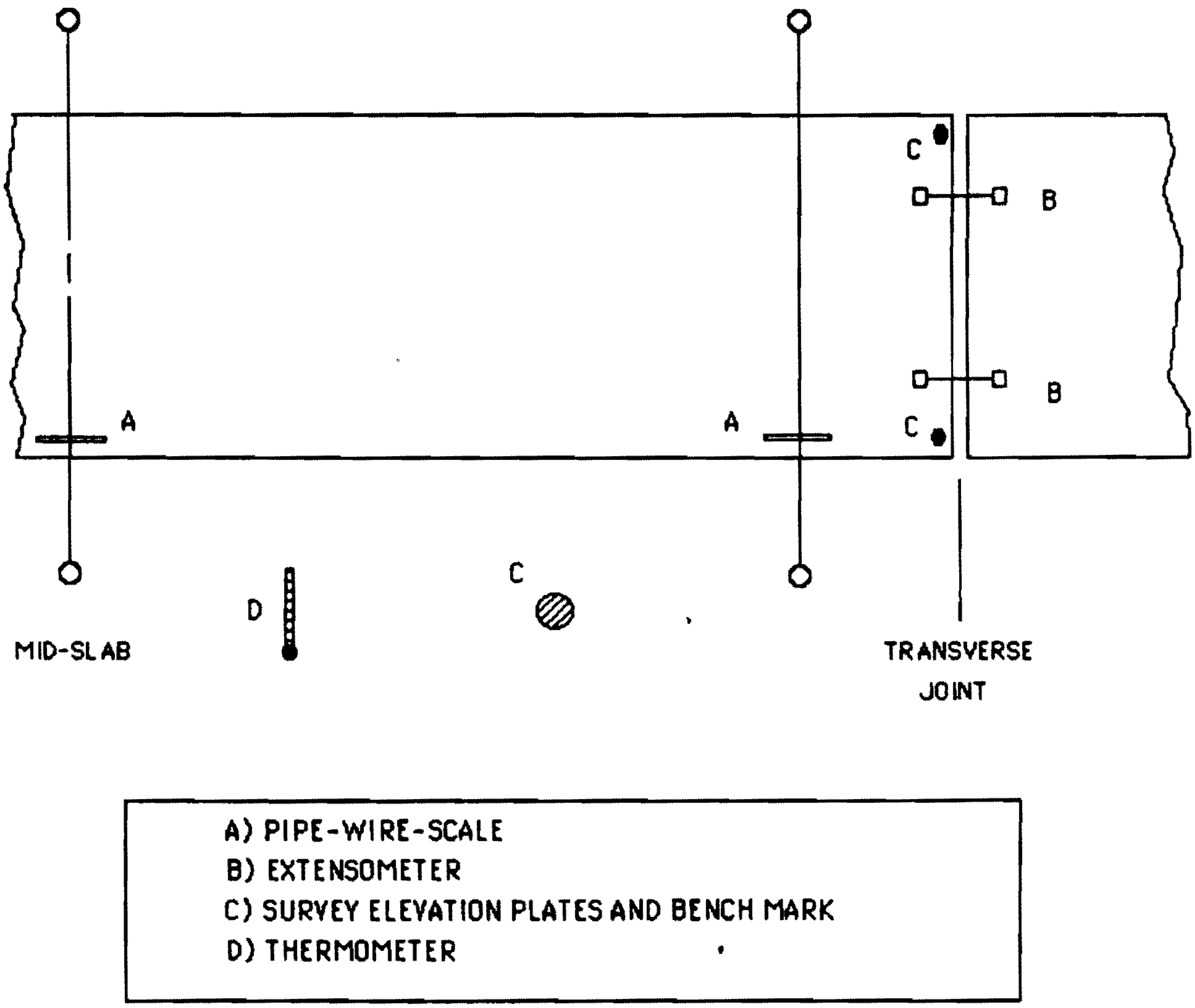
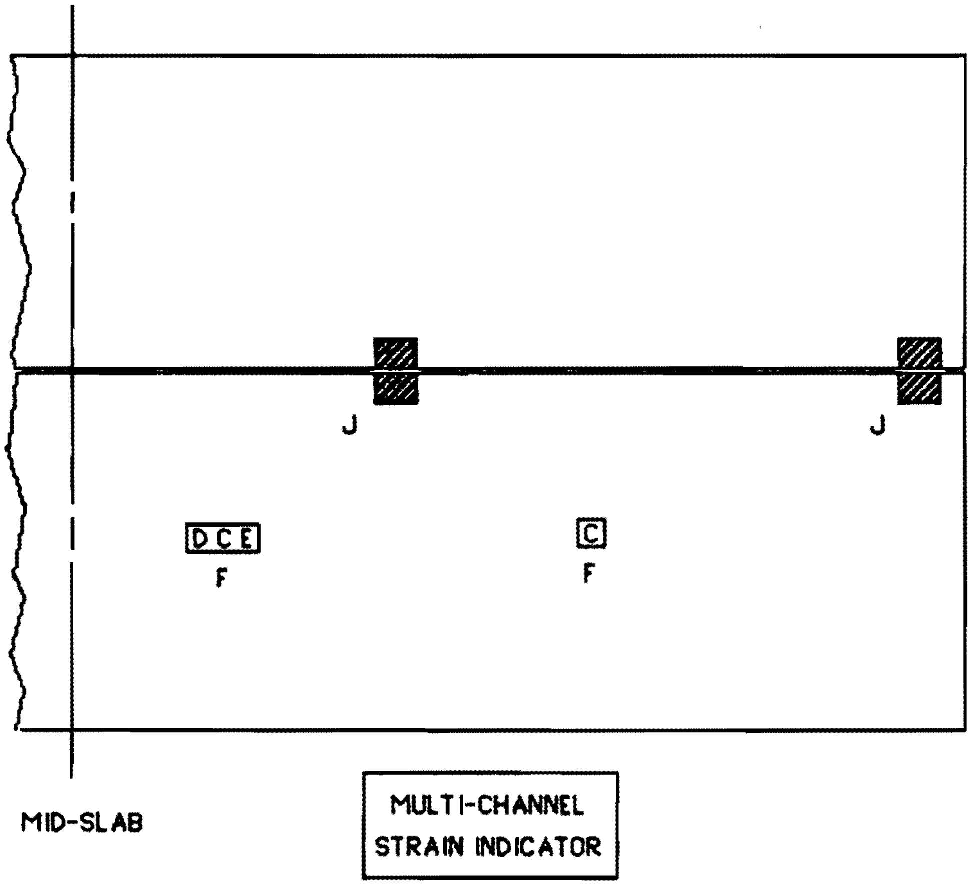


Fig 8.3. Mechanical back-up for continuously recorded instrumentation. Instrument locations for a half-slab.



MEASUREMENT	NO. OF LOCATIONS	APPARATUS
TRANSVERSE CONCRETE STRAINS:		
C-MID-DEPTH CONCRETE STRAIN	2	EMBEDMENT GAGE
D-1/2" DEEP CONCRETE STRAIN	1	
E-5 1/2" DEEP CONCRETE STRAIN	1	
F-SURFACE CONCRETE STRAIN	2	SURFACE GAGE
J-RELATIVE LONGITUDINAL MOVEMENT	2	EMBEDDED PLATE

Fig 8.4. Additional short-term instrumentation - not continuously recorded. Instrument locations for a half-slab.

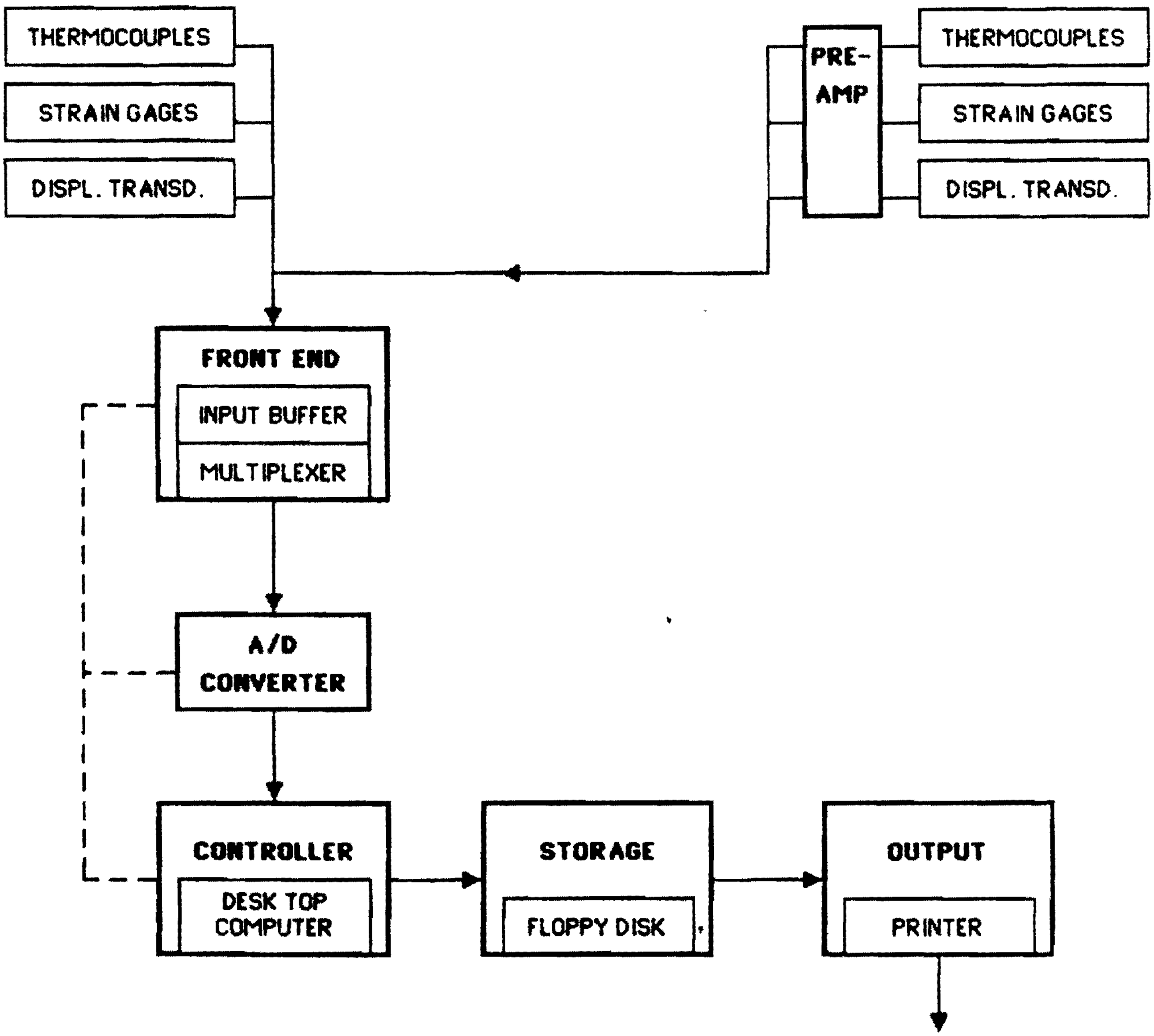


Fig 8.5. Data acquisition system.

the multiplexer allows the different inputs to be read in sequence, and the analog/digital converter records the voltage signal as a binary number. This procedure is programmed by the controller, which also directs the printing and storage of the data.

Measurements of temperature and movement were also recorded mechanically, as shown in Fig 8.3. Absolute horizontal movement was checked at midslab and near the slab end using a pipe-wire-scale set-up, as shown in Fig 8.1. Horizontal movement relative to neighboring slabs was found by measuring the width across the transverse joint, using an extensometer and reference plugs on either side of the joint. Vertical movements of the slab corners were found using survey equipment. Continuous ambient temperature readings were checked intermittently using a thermometer.

Readings of transverse concrete strain were not continuously recorded. The strain was read before and after the application of the transverse prestress, using a digital strain indicator. As with the measurement of longitudinal stresses, embedment gages at three depths and surface gages were used. Relative movement along the longitudinal joint was measured before and after the application of longitudinal prestress. The monitoring of this movement was measured between scored plates embedded in the edges of the adjacent pavement slabs.

Long-Term Measurements

Most of the measurements taken in the long-term were the same as those mechanical measurements taken in the short-term. Joint width was measured by extensometer at all transverse joints. These relative movements were related to pipe-wire-scale measurements of absolute movements on at least two slabs. Curling of the slab corners continued to be measured using survey equipment. Relative movement along the longitudinal joint of at least two sections was monitored. The load transfer capabilities of the transverse joint detail were measured using a Dynaflect apparatus, and Dynafect readings were taken on two or more pavement slabs to record the deflection characteristics at several points throughout the slabs.

CHAPTER 9. CONSTRUCTION REPORT OF THE MCLENNAN COUNTY PROJECT

This chapter describes the procedures used for placing the McLennan County overlay, the unique construction techniques employed, and a variety of construction problems encountered. The reader may compare the steps for the actual construction process with the recommended procedure described in Chapter 7. This chapter presents pertinent cost data also.

LAYING DOWN OF THE POLYETHYLENE SHEETING

Prior to the placement of the tendons, a single layer of polyethylene sheeting was placed on top of the 2-inch-thick layer of asphalt. Laboratory push-off testing conducted as part of Project 401 showed that a double layer of polyethylene was unnecessary. The required reduction in friction could be obtained with a single layer. For each of the 17-foot and 21-foot-wide pours, a single roll of polyethylene sheeting was rolled out longitudinally. As the polyethylene was rolled out, the edges were tacked down to prevent the wind from blowing the sheeting around. The part of the sheeting that was going to be underneath the overlay was held in place by temporary weights until the tendons were placed on top. In places where two layers overlapped, a sealant was used to seal the edges of the two layers together. Also, any holes in the sheeting were repaired before the overlay was poured. This operation required 4 to 5 workers and was accomplished without any problems.

PLACEMENT OF THE SIDE FORMS

Side forms, when used, were placed on top of the polyethylene sheeting and held down by spikes driven down through the polyethylene and into the underlying pavement. After a slab was poured, the side forms were removed and placed for the next slab to be poured. Side forms were initially used on both edges of the 17-foot-wide lane. However, the paving operation was slowed down because there were not enough side forms to prepare the next slab for paving. Side forms were then used on only one edge of the 17-foot-wide lane while the other edge was slip-formed. The edge of the slab with the transverse tendons extending out required the use of side forms. Slip forming on one edge speeded up the paving operation. In the paving of the 21-foot-wide lane, the edge of the 17-foot-wide lane served as a side form and the other edge was slip-formed. Not having to use side forms meant the 21-foot-wide lane could be poured faster than the 17-foot-wide lane.

TENDON PLACEMENT AND FORMING OF THE STRESSING POCKETS

The prestressing tendons were laid out on top of the polyethylene sheeting and were then placed on chairs to raise the tendons to the required height. The longitudinal tendons were laid out first, and then the transverse tendons were added on top of the longitudinal tendons. To lay out the tendons, control points were marked on the asphalt. These control points could be seen through the polyethylene and were used to position the tendons in the appropriate locations. The transverse tendons were tied to the longitudinal tendons after they were both placed on chairs. In addition to the tendons, transverse reinforcing bars were placed at this time. These bars were tied to the prestressing tendons to keep them in place and to provide support for the tendons. The longitudinal tendons were then anchored to the transverse joint, and, on the 17-foot-wide lane, the transverse tendons were guided through the side forms.

The placement of the stressing pocket forms was also done at this time. The forms consisted of steel boxes that had no bottom and a removable top and that could be slipped over the tendons after the tendons had been placed. The boxes were bolted to asphalt to hold them in place. To prevent the tendons from sagging between chairs, a light tension force was applied to the longitudinal tendons. This was achieved by prying with a lever against the stressing pocket forms and tightening a clamp around the tendon to hold the tension in the tendons.

TIE-DOWN OF THE TRANSVERSE JOINTS

The transverse joints were placed on top of the polyethylene at the same time that the tendons were placed. On all of the joints, no initial gap was provided and the two sides of each joint were held together by tack welding short metal straps across the top of both sides of the joint. The joint was anchored in place initially to prevent the joint from rotating when the tendons on one side were put in light tension. To anchor the joint, bolts with rebar welded to the end were driven into the subgrade on both sides of the joint. A large washer was then placed on each bolt, the washer overlapped the bottom flange of the joint, and a nut was tightened down on the bolt to hold the washer and the joint down. Just before the new slab was poured, these anchors were removed from the joint, leaving just the metal straps holding the joint together. At this point, the joint was kept from rotating by the previously poured slab. Once the new slab was poured, the metal straps were removed, which freed up the joint to open and close with any slab movements.

Some of the joints arrived from the manufacturer needing to have the holes for the dowels reamed out so that the holes on both sides of the joint would line up. Also, some joints arrived warped, which caused problems with tack welding the joint together with no initial opening.

CONCRETE PLACEMENT

A slip-form paver was used for the paving of both lanes of the overlay. The concrete was delivered to the site by trucks and was placed directly in front of the paver. The concrete was partially spread by hand before the paver moved over it. Hand placement of the concrete was necessary around the chairs holding up the prestressing tendons to prevent the paver from displacing the tendons as it went by. As the paver proceeded, though, the concrete was spread in front of the paver.

The concrete, as it arrived by truck, was not consistent and was not always accepted. A set retarder was required due to the distance travelled by the trucks and the hot weather during the day. However, when the weather cooled down in the evening, the set retarder was not required but was present anyway. The retarder caused very slow curing in that case and resulted in low early concrete strengths. Thus, initial post-tensioning was delayed, and, in one case, a crack occurred in the slab before initial post-tensioning was performed.

Initially, a small post-tensioning force of 7.5 kips was used in slabs with low early strength to prevent cracking. Later on in the project, the amount of set retarder was reduced by half in the loads that arrived late in the day. Some concrete was rejected because the slump was too high, causing delays in the paving operation.

In the future, the use of a small mixing plant at the construction site would provide consistent and more controllable batches of concrete. This would prevent any delays in the delivery of concrete and would enable the paving equipment to be used to its full ability. Also, the use of a mixing plant along with a spreader would eliminate problems of access to the site by concrete trucks.

CONCRETE FINISHING AND REMOVAL OF STRESSING POCKET FORMS

Once the slip-form paver had partially finished the concrete, final finishing was done by hand. During the final finishing of the concrete, the stressing pocket forms were removed. The concrete was scraped from the top of the forms and the lid of the form was removed. The bolts that held the forms down were removed and then the forms were lifted out. After all the hand-finishing of the concrete was completed, transverse grooves were placed in the concrete by hand to provide surface texture. To enhance curing, a curing compound was sprayed on with a hand-held sprayer after all finishing was complete.

When more than one slab was poured in a single run, it was necessary to lift the paver while it passed over the transverse joints to ensure that the paver would not hit and possibly damage the joint. Also, the paver was raised whenever it was passing over the stressing pocket forms, to keep from hitting and damaging the forms. This lifting of the paver made it necessary to perform additional hand finishing, which slowed the paving operation significantly.

The slip-form paver was lifted over the stressing pocket forms to prevent possible damage to the forms. One early problem was encountered when the paver was not lifted over the forms. The clearance between the top of the form and the top of the slab was not enough to permit the largest aggregate to clear the forms as the paver went over. The large aggregate was caught between the paver and the form, causing damage to the form. This problem can be solved by providing enough clearance beforehand by making the stressing pocket forms shorter so that the largest expected aggregate can clear the form. If the paver does not have to be lifted over the forms, the amount of hand-finishing required will drop significantly and the paving operation will be faster.

INITIAL POST-TENSIONING

Initial stressing of the longitudinal tendons was performed after the concrete had gained enough early strength to withstand the prestress forces. The early strength of the slab concrete was determined by the testing of cylinders cast from the same concrete. Usually, the initial stressing was performed from 8 to 10 hours after the slab had been poured. In cases where there were excessive amounts of set retarder, low early concrete strength prevented the application of the full initial prestress force. However, to prevent cracking in the slabs before the full initial prestress force could be applied, a small prestress force (5 to 7 kips) was applied about 8 to 10 hours after the slab was poured. The anchorage of the tendons to the armored joints would probably increase the anchor zone strength which would allow a larger initial prestress force to be applied or would allow an earlier application of the prestress force.

FINAL POST–TENSIONING

The final post-tensioning of the longitudinal tendons was performed approximately 48 hours after the slab was poured. The concrete strength was required to be high enough to permit final stressing and no problems were encountered in obtaining this strength within 48 hours.

Both initial and final post-tensioning were performed with a portable hydraulic ram. Early in the project, the stressing pockets were found to be too short to easily accommodate the stressing ram. The stressing pockets were then increased from 30 inches to 48 inches, which provided adequate room for the stressing ram. Minor problems were encountered throughout the whole project when the grips slipped or broke during stressing operations. Hard spots in the tendons or faulty grips were the probable causes of these grip failures.

In tensioning the longitudinal tendons in a slab, the tendons at the center of the slab were always stressed first. The tendons were then stressed by alternating out towards the tendons at the edges of the slab.

POST-TENSIONING OF THE TRANSVERSE TENDONS

Posttensioning of the transverse tendons was performed after all paving operations were complete. The stressing of each tendon was done in a single stage, instead of in the two stages used for the longitudinal tendons. The transverse tendons were stressed after allowing the 21-foot-wide slabs to move independently of the 17-foot-wide slabs, to permit the matching up of the transverse joint openings. Once the transverse tendons were stressed, very little differential movement occurred between the 17 and 21-foot-wide slabs. The initial transverse joint openings and the time allowed before stressing had to be judged so that the final joint openings were equal for the two slabs.

In tensioning the transverse tendons in a slab, the tendons at the middle of the slab were always stressed first. The tendons were then stressed by alternating out towards the two ends of the slab.

LONGITUDINAL JOINT

The longitudinal joint between the 17-foot-wide lane and the 21-foot-wide lane required preparation to prevent bonding between the two lanes. After the 17-foot lane was paved, the edges of these slabs were coated with asphalt, which served as a bond breaker between the two lanes. A compressible material was then placed around the transverse tendons at the longitudinal joint. Extra compressible material was placed on those tendons located where large differential movements between the 17-foot and 20-foot-wide slabs were expected, namely, those tendons near the ends of the slabs. The edge of the 17-foot-wide lane served as a side form for the paving of the 21-foot-wide lane. The edge of the 17-foot-wide lane used as a side form was a straight and even edge due to the fact that it was necessary to use side forms on that edge to support the transverse tendons.

PATCHING OF THE STRESSING POCKETS

The stressing pockets were filled in soon after the stressing operations were complete for each slab. The pockets were cleaned of all debris and were filled, and the concrete was finished and textured. Mats were then placed over the pockets to promote curing. The large number of pockets increased the amount of hand-finishing for the project significantly, and it was difficult to provide a good match of the finish of the slab and of the pockets due to the different concrete ages.

COST DATA

The total cost for this 1.182 miles of prestressed pavement was $1,126,840.00. Some of the larger contract items and their bid prices are shown in Table 9.1.

The bid price of $25.41 per square yard for the prestressed concrete pavement (6-inch) is slightly higher than for a reinforced concrete pavement of the same size.

According to a bid summary of all reinforced concrete pavement constructed from March 1984 to March 1985, presented by the Pavement Design Branch of the Texas SDHPT Highway Design Division, the average cost of an 8-inch reinforced concrete pavement is about $18.00 per square yard. A 10-inch reinforced concrete pavement of the same project size would cost about $22.50 per square yard.

The higher bid price of this prestressed pavement can be attributed to the following factors:

(1) This was an experimental project and the contractors who bid for this job had no prior experience in this kind of pavement.

(2) This project involved both longitudinal and transverse prestressing.

(3) This project was comparatively small. Unit prices for smaller projects tend to be higher because equipment costs cannot be spread over a longer period of time.

TABLE 9.1. MORE IMPORTANT CONTRACT ITEMS AND BID PRICES

Item Description	Unit	Quality	Bid Price (dollars)	Total
Mobilization	LS	1	150,000.00	150,000.00
Construction Detour	STA	12.00	10,000.00	120,000.00
Prestressed Concrete Pavement (6-inch)	SY	22,125.00	25.41	562,196.25
Sealed Expansion Joints (4-inch)	LF	646.00	110.00	71,060.00

CHAPTER 10. CONCLUSIONS AND RECOMMENDATIONS

The development of techniques for designing prestressed pavements has reached the point at which reliable designs can be achieved. The design presented in this report was developed from theoretical analysis supported by the experience gained on past U.S. prestressed projects. Both overlays were designed considering all possible problems. Laboratory tests were conducted for developing many of the recommendations presented herein.

RECOMMENDATIONS FOR FUTURE STUDY WITH RESPECT TO DESIGN

These are some of the factors that need further study:

(1) The development of a detailed plan for monitoring periodically the performance of the overlays should be developed. This plan should include evaluations of riding quality and structural conditions of the joints.

(2) The progressive length changes due to shrinkage and creep and the seasonal variations in pavement length related to prestress and swelling on the PSCP slabs of the McLennan County project should be monitored. Records of movements may allow criteria improvements in the current design procedure.

(3) A comprehensive design methodology including the benefits of certain environmental factors such as permanent moisture differential through the slab depth should be developed. The present technology does not allow the designer to incorporate this factor and some of its beneficial effects in the design with sufficient reliability.

(4) The friction properties of bases and interlayer materials have to be studied for developing comparative data on their performance under long rigid slabs. A study of this nature may be useful for developing guides on the merits of different materials with age and under various climatological conditions.

(5) Little is known with respect to the fatigue versus stress level relationship of prestressed concrete pavements. Therefore, criteria to protect the pavement

against fatigue due to repeated load should be developed. Extensive laboratory testing may be required in this respect.

CONCLUSIONS AND RECOMMENDATIONS FROM CONSTRUCTION

These recommendations were derived from the construction of the McLennan section:

(1) The transverse prestressing scheme used for the Waco pavement overlay did not allow the use of a trailing form on one edge when the first pavement strip was poured. An alternate design for transverse prestressing, perhaps using rigid ducts, could allow the use of a trailing form on both edges of the paving. In this way, no side forms would be needed, which would result in substantial time and labor savings. Also, alternate designs could allow for the paving of three or more pavement strips tied together transversely. In evaluating the alternate designs, variables to be considered include material costs, construction feasibility, and number of stressing operations required. (The new design would best be evaluated by its actual implementation in a prestressed pavement overlay.)

(2) In the pavement overlay at Waco the transverse joints represented a major material expense. Simplification of this joint design could make prestressed concrete pavements more feasible. Also, construction experience has indicated some problems in keeping this joint in place during concrete placement. A revised joint design should be looked for in order to solve these problems. The revised design could include a new type of expansion seal recently designed by Watson-Bowman, Inc. The new joint could also include a stronger tie-back of tendon anchors, which would eliminate zones of tensile stress in the joint region. New designs could be evaluated by setting up fatigue tests in the laboratory or by actual implementation in a prestressed pavement overlay. Also, the currently used joints should be monitored over a period of several years to record joint opening performance, load-transfer capability, and overall joint condition.

(3) The current pavement overlay used 6/10-inch strand, which required a fewer number of strands to obtain the desired level of prestress. One-half-inch strand, however, is more economical overall. The standard strand hardware is more readily available as is the stressing equipment. The cost of the 1/2-inch strand, end hardware, and equipment for 1/2-inch strand is significantly lower than that for 6/10-inch strand. A design based on 1/2-inch strand would require revisions in the number of strands, strand spacing, and strand eccentricities. Also, new data on anchor zone strength of concrete would be needed. A design using 1/2-inch prestressing strand could be implemented in an actual prestressed pavement to compare the economy of 1/2-inch strand to 6/10-inch strand.

(4) Use of multistrand anchors rather than single strand anchors would provide fewer obstructions in the joint regions and could save in material and labor costs. Also, initial posttensioning procedures could be simplified by, for example, fully stressing two tendons of each four-tendon anchor. (This would be simpler than partially stressing all tendons.) Use of the multistrand anchors would require new data on the anchor zone strength of concrete. Multistrand anchors could be tried in an actual prestressed pavement.

(5) VSL Corporation has designed a scheme for posttensioning stressing pockets that stresses two tendons at once; thus, fewer stressing pockets are required. The new stressing pocket design could be evaluated through actual implementation. As previously described by Cable (Ref 21), precast joint panels offer several advantages over casting in place around the joint detail. Design of precast joint panels could incorporate the revised design of the joint detail and the use of multistrand anchors. Actual joint panels could be fabricated and cast and then fatigue tested in the laboratory. Also, the joint panels could be evaluated through actual implementation.

(6) Precast stressing panels like those described in Ref 21 would eliminate the need for extra forming at the job site and could incorporate the revised stressing pocket design.

(7) Rather than chairing up the longitudinal tendons, these tendons could be positioned using guides attached to the slip-form paver; such a scheme was used successfully on the prestressed pavement in Arizona. The procedure would have

to be modified, however, to take into account the possible obstruction from the transverse tendons.

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APPENDIX A

PLAN VIEW OF CANDIDATE SECTIONS AND DEFLECTION PROFILES

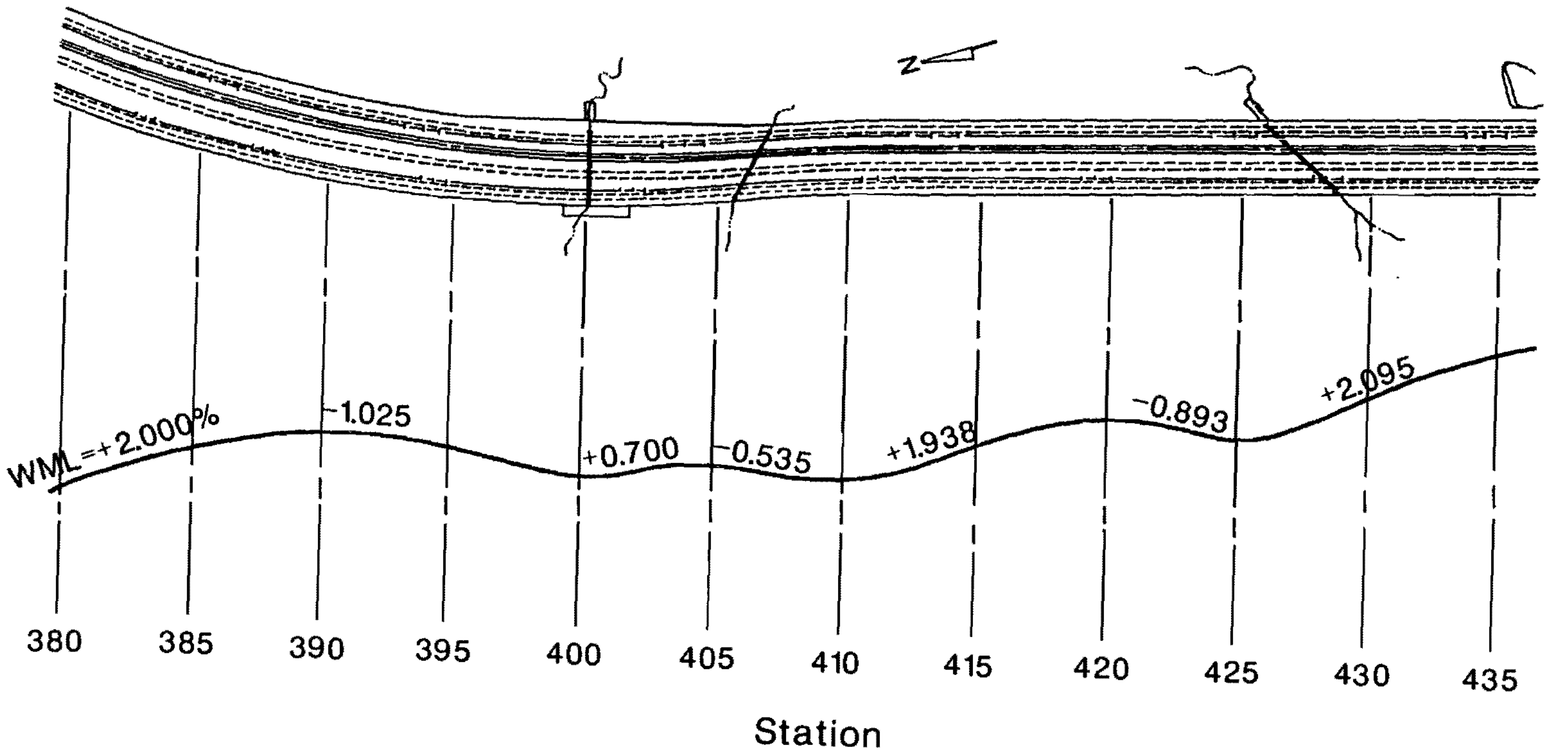

Fig A.1. Plan view of section 1, Cooke county.

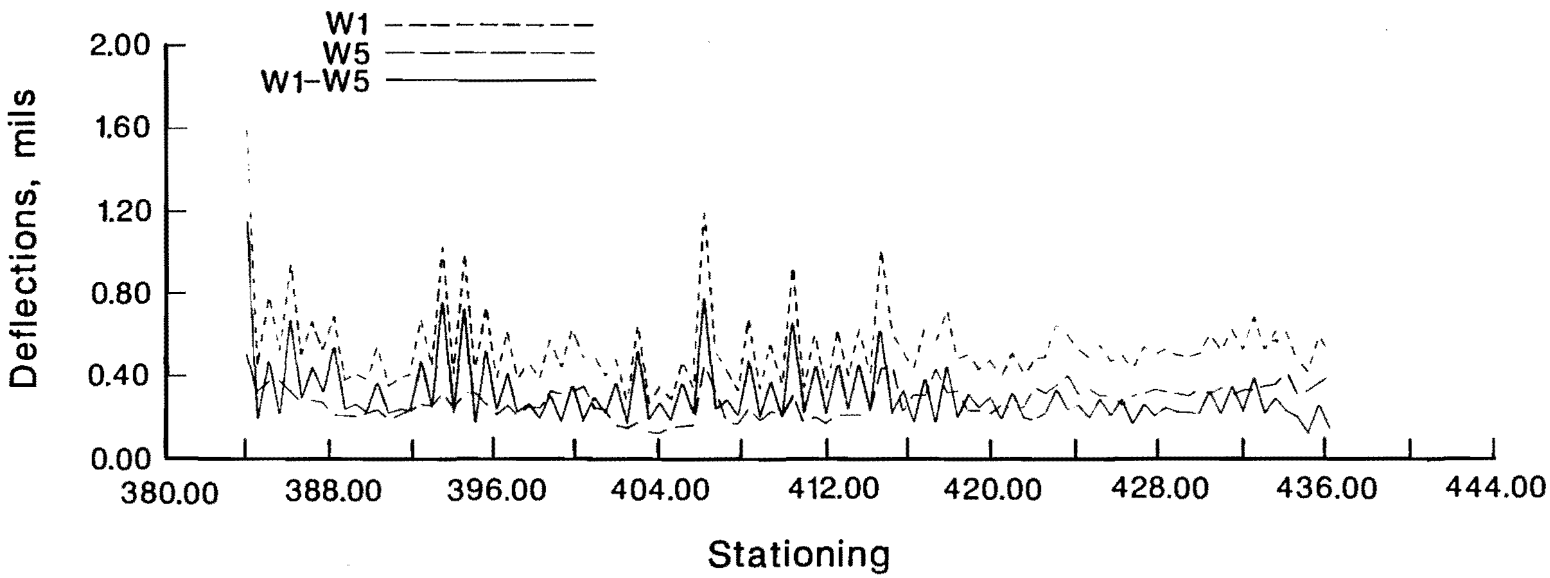

Fig A.2. Dynaflect deflections W_1, W_5, and W_1 - W_5 for section 1.

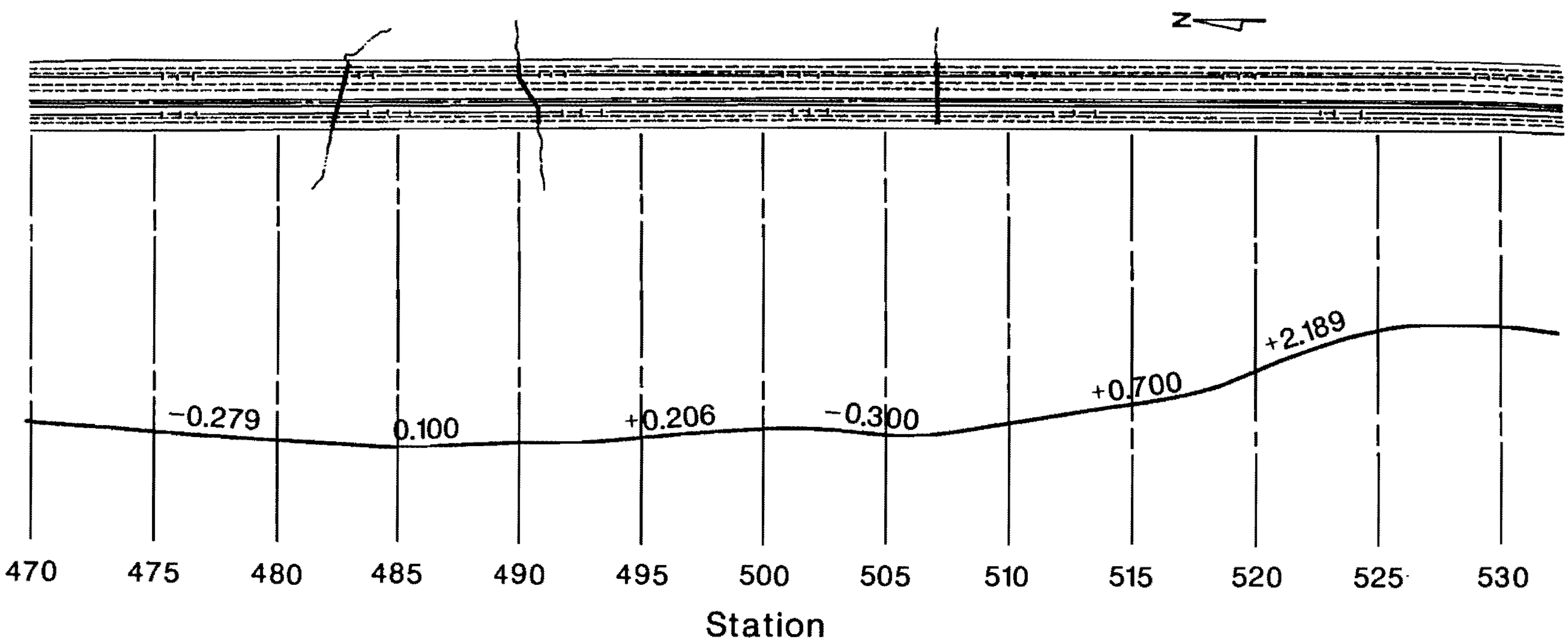

Fig A.3. Plan veiw of section 2, Cooke county.

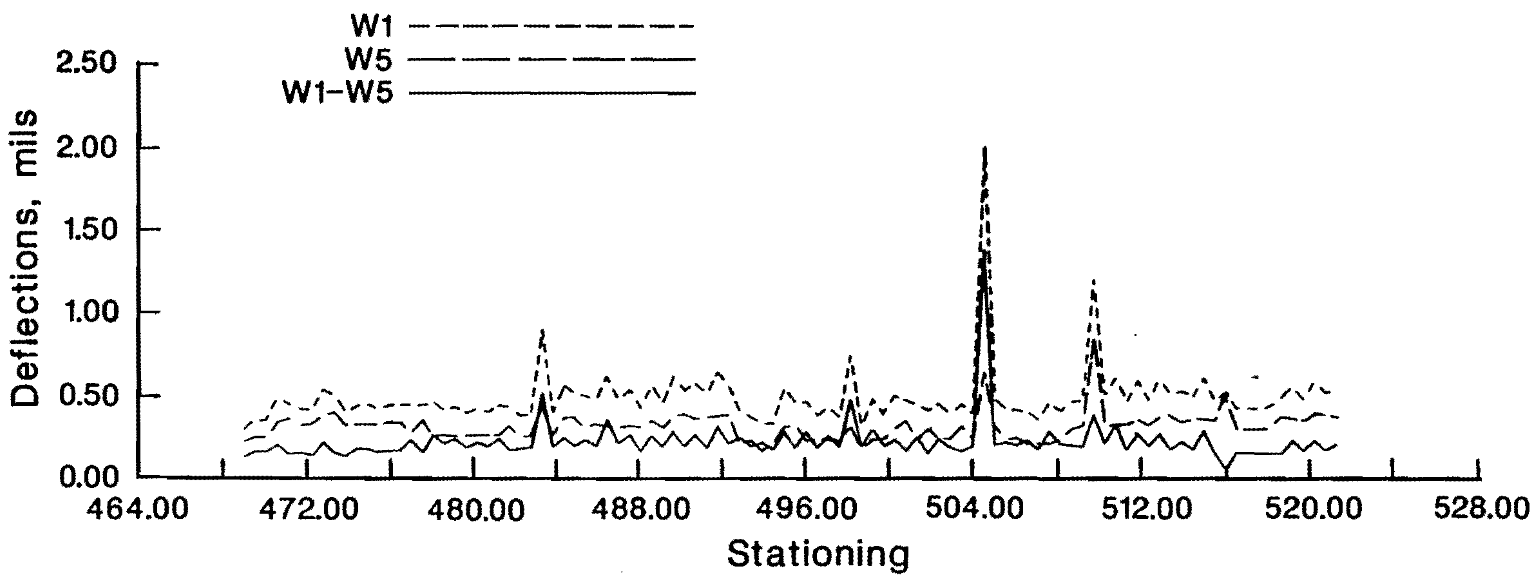

Fig A.4. Dynaflect deflections W_1, W_5, and W_1 - W_5 for section 2.

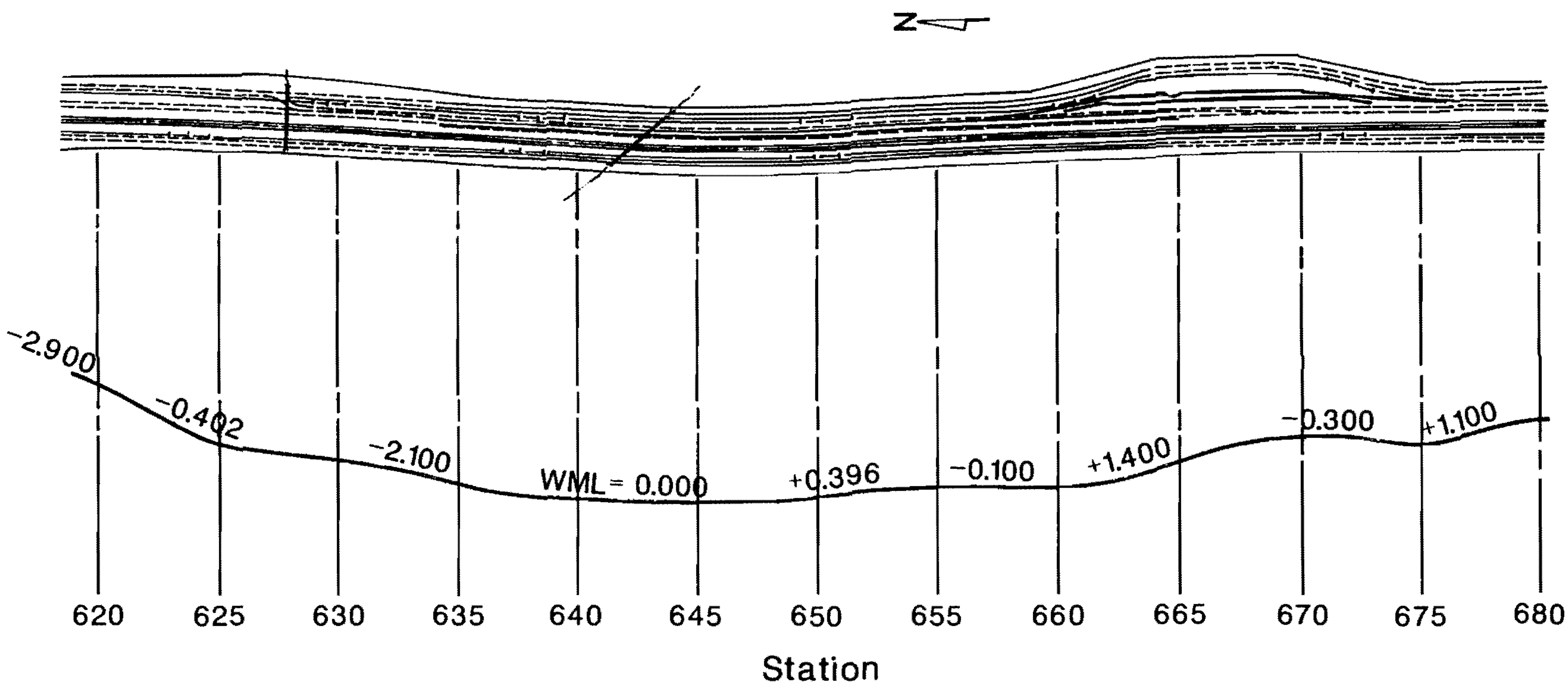

Fig A.5. Plan view of section 3, Cooke county.

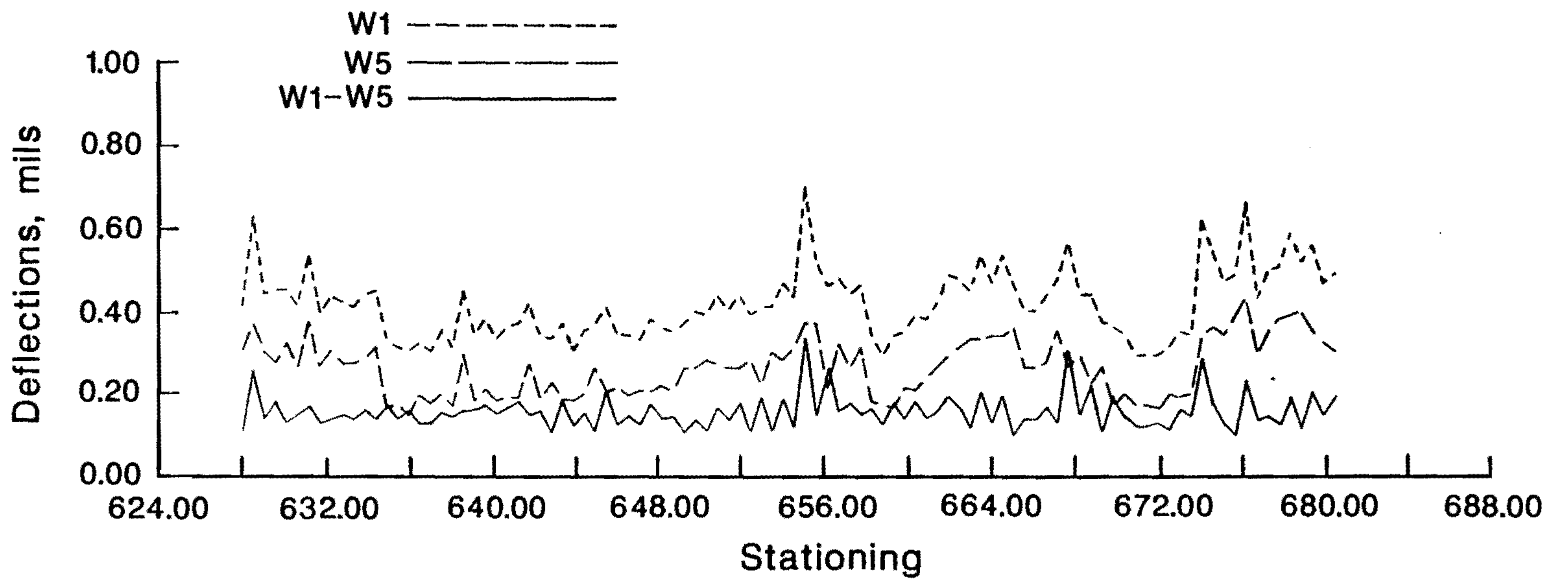

Fig A.6. Dynaflect deflections W_1, W_5, and $W_1 - W_5$ for section 3.

RR555/556-1/AA

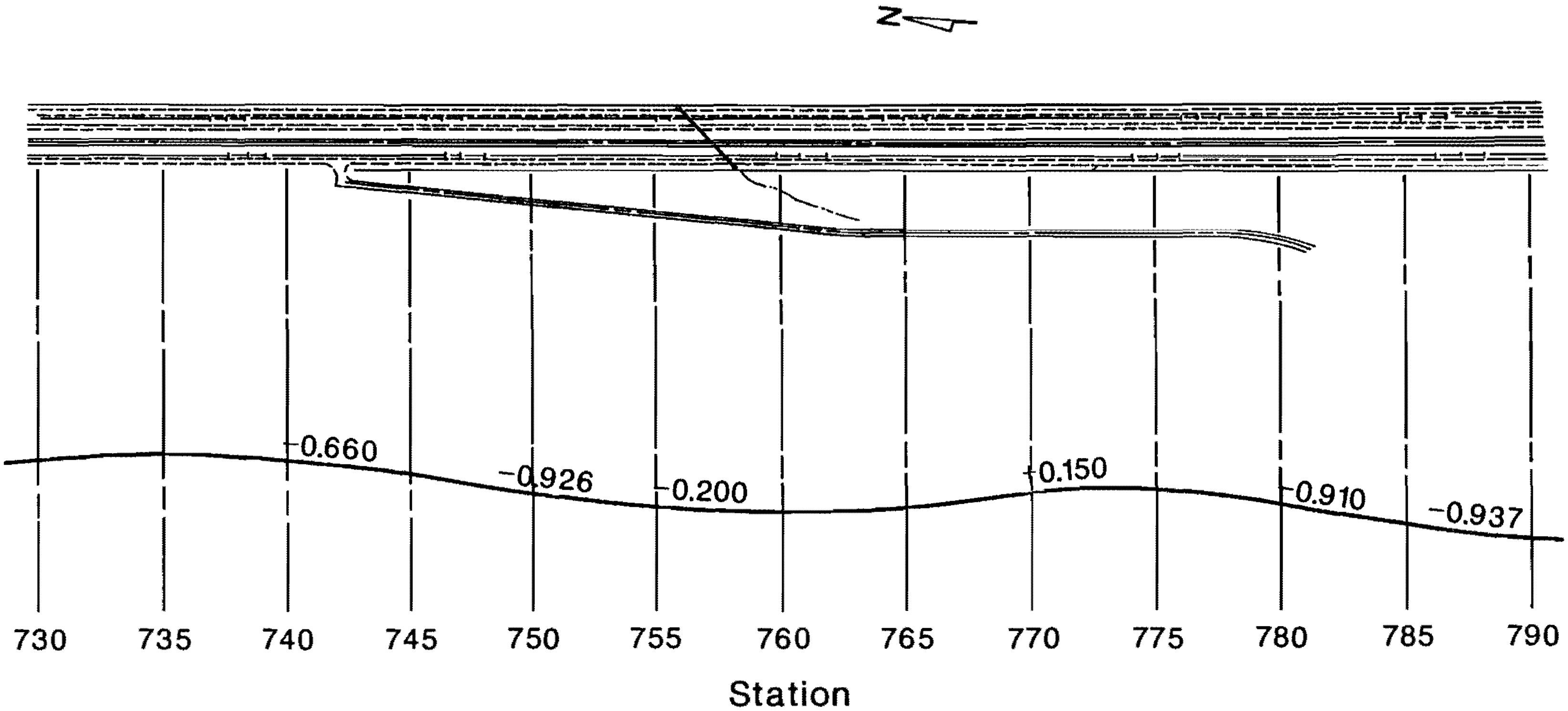

Fig A.7. Plan view of section 4, Cooke county.

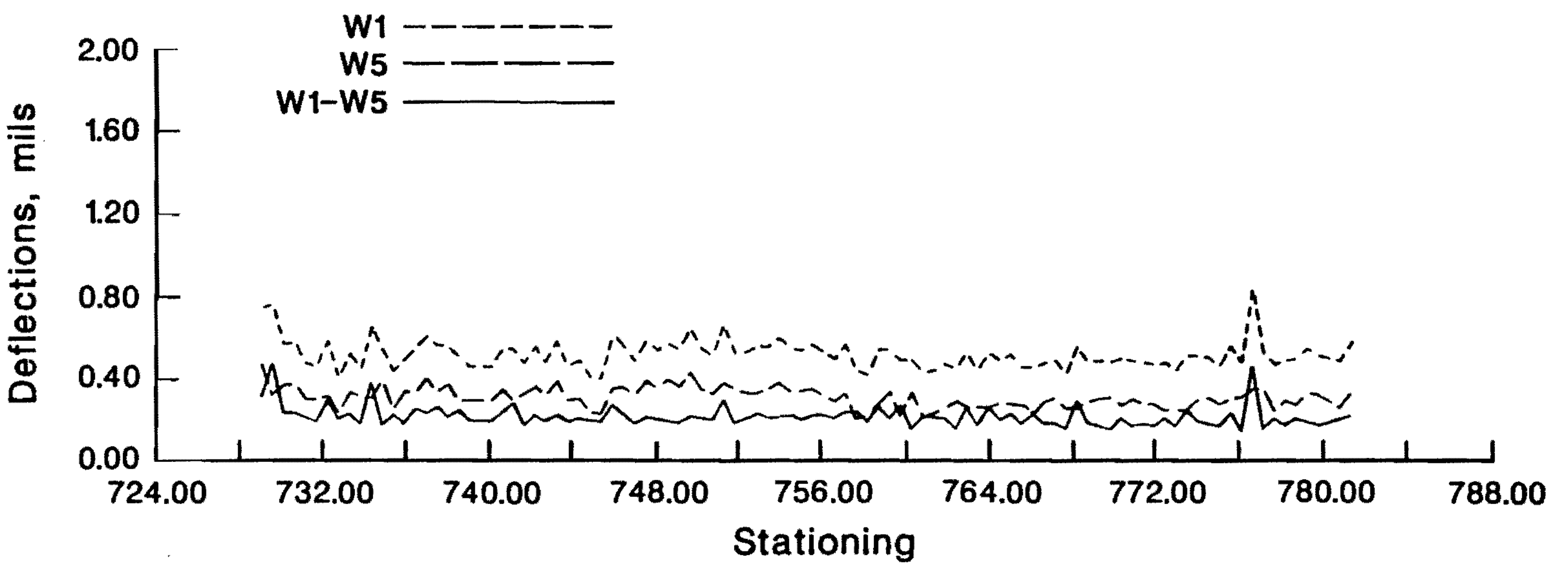

Fig A.8. Dynaflect deflections W_1, W_5, and W_1 - W_5 for section 4.

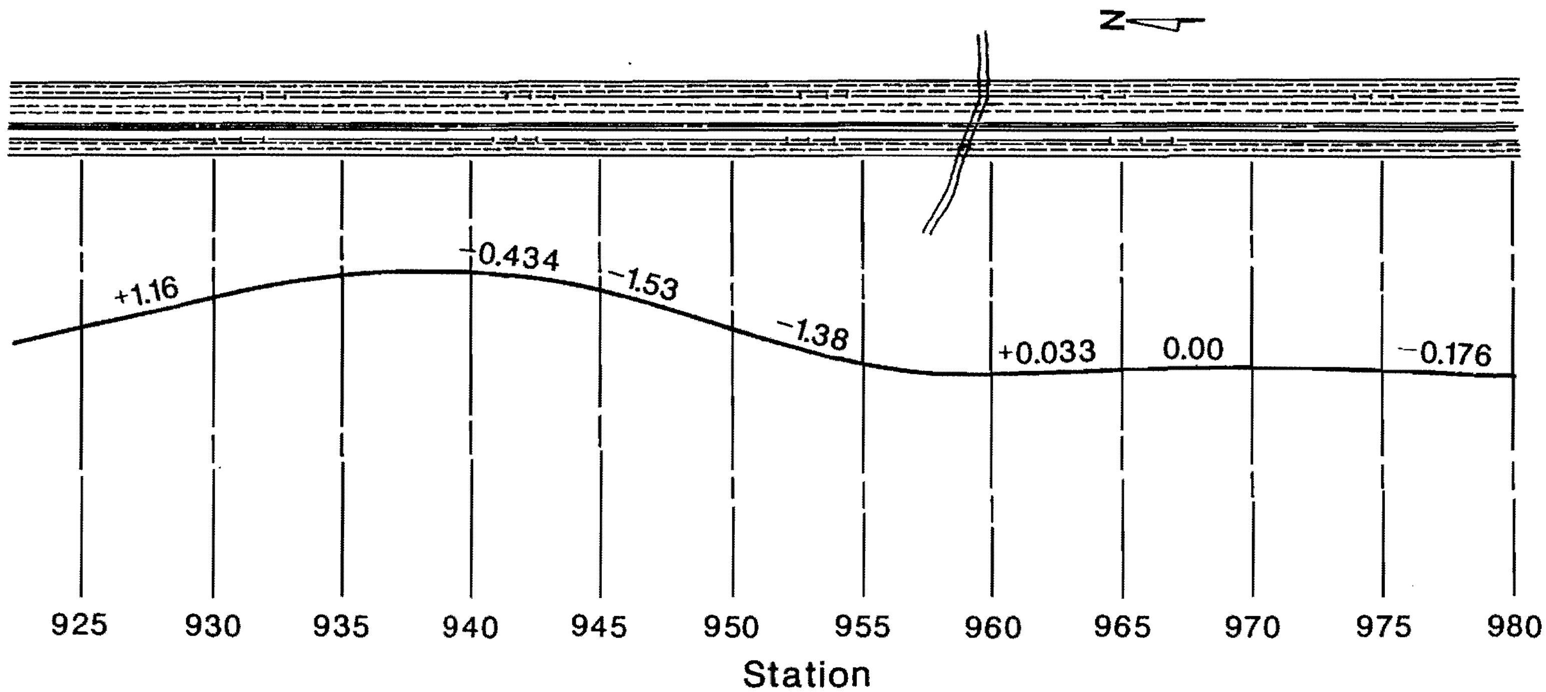

Fig A.9. Plan view of section 5, Cooke county.

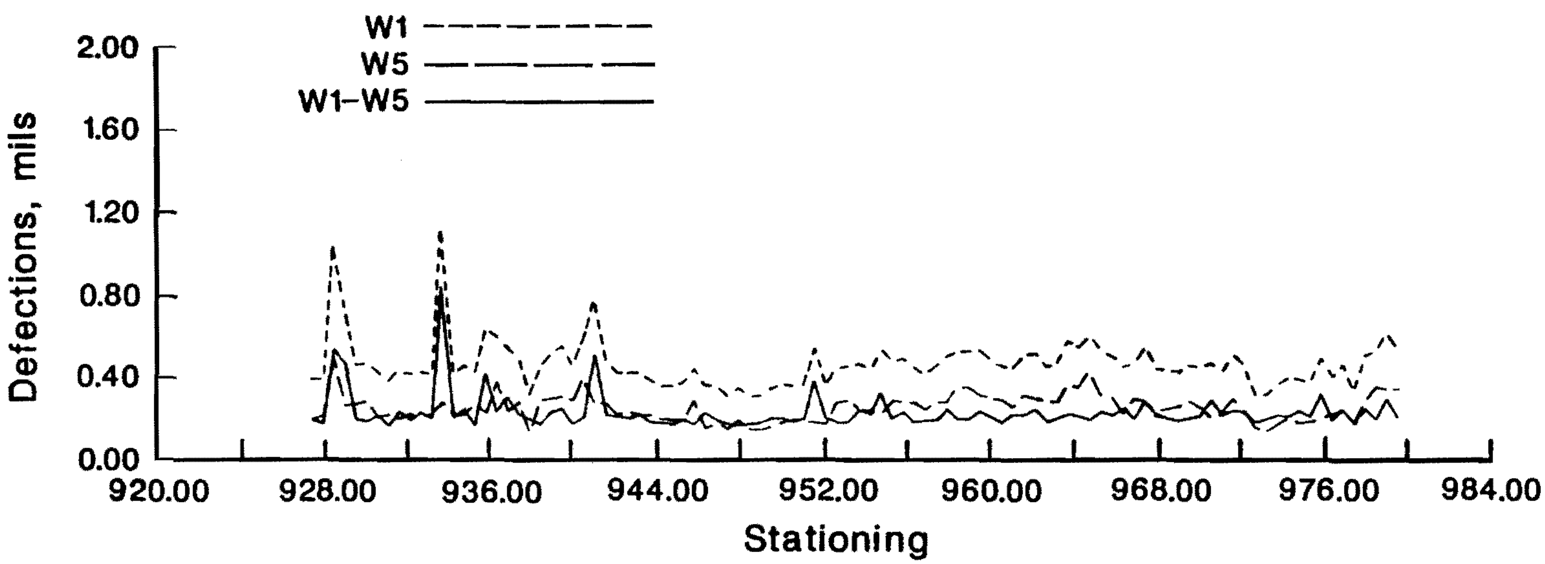

Fig A.10. Dynaflect deflections W_1, W_5, and W_1 - W_5 for section 5.

APPENDIX B

DEFLECTION PROFILES, LAYER PROPERTIES, AND COMPUTATIONS OF FATIGUE LIFE OR EXPERIMENTAL SECTIONS IN COOKE AND McLENNAN COUNTIES

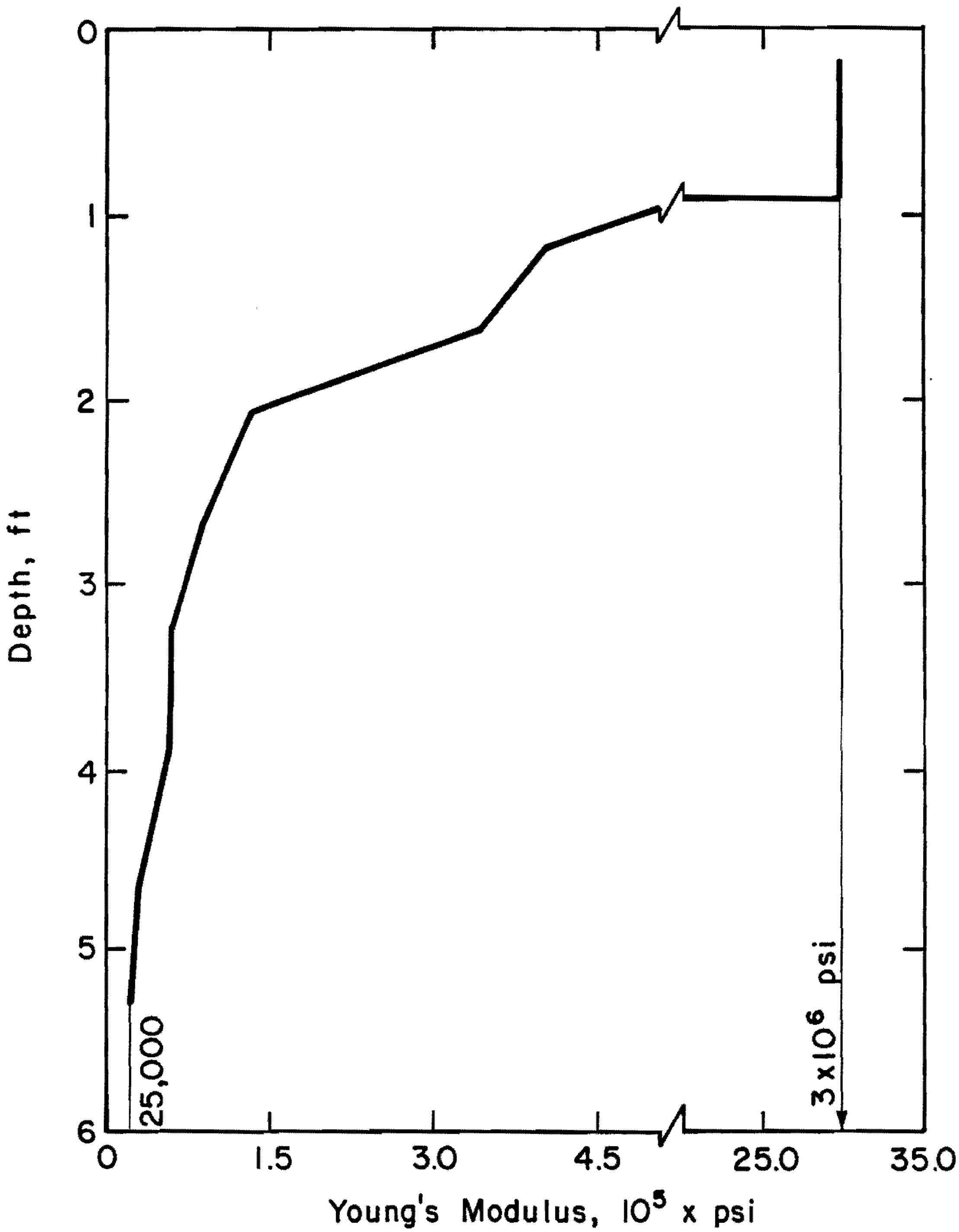

Fig B.1. Elastic modulus versus depth as determined by SASW station 742 + 23.

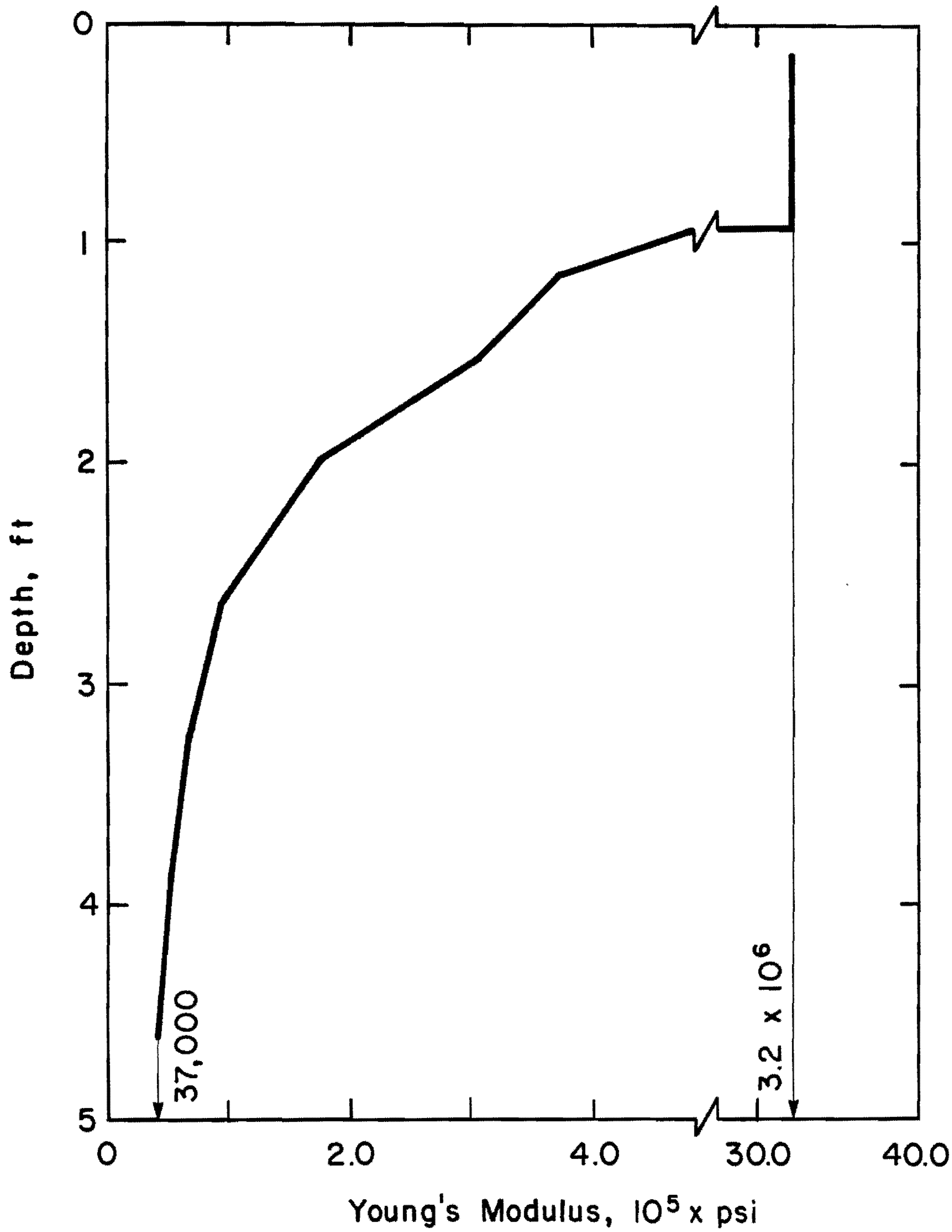

Fig B.2. Elastic modulus versus depth as determined by SASW station 760 + 02.

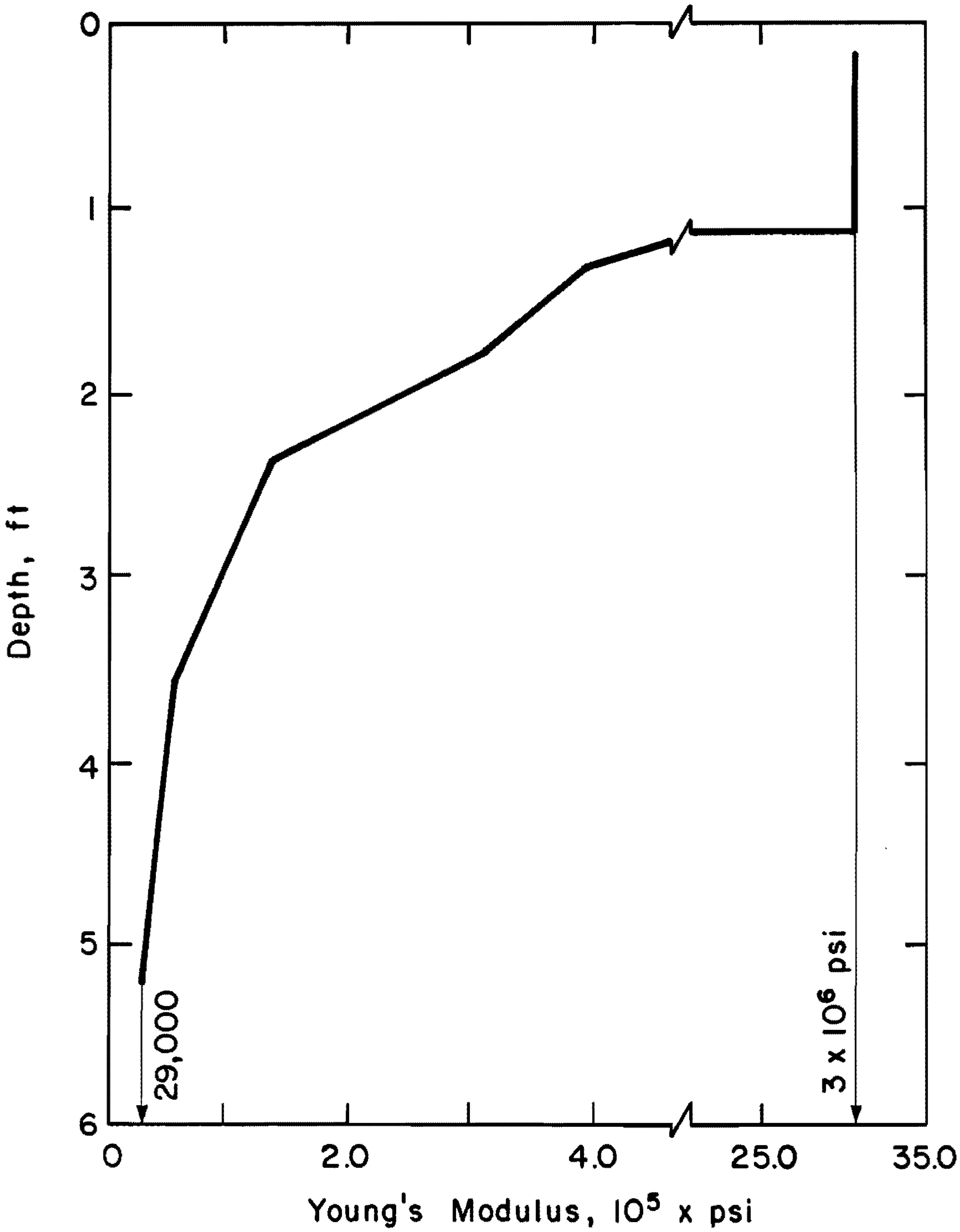

Fig B.3. Elastic modulus versus depth as determined by SASW station 776 + 00.

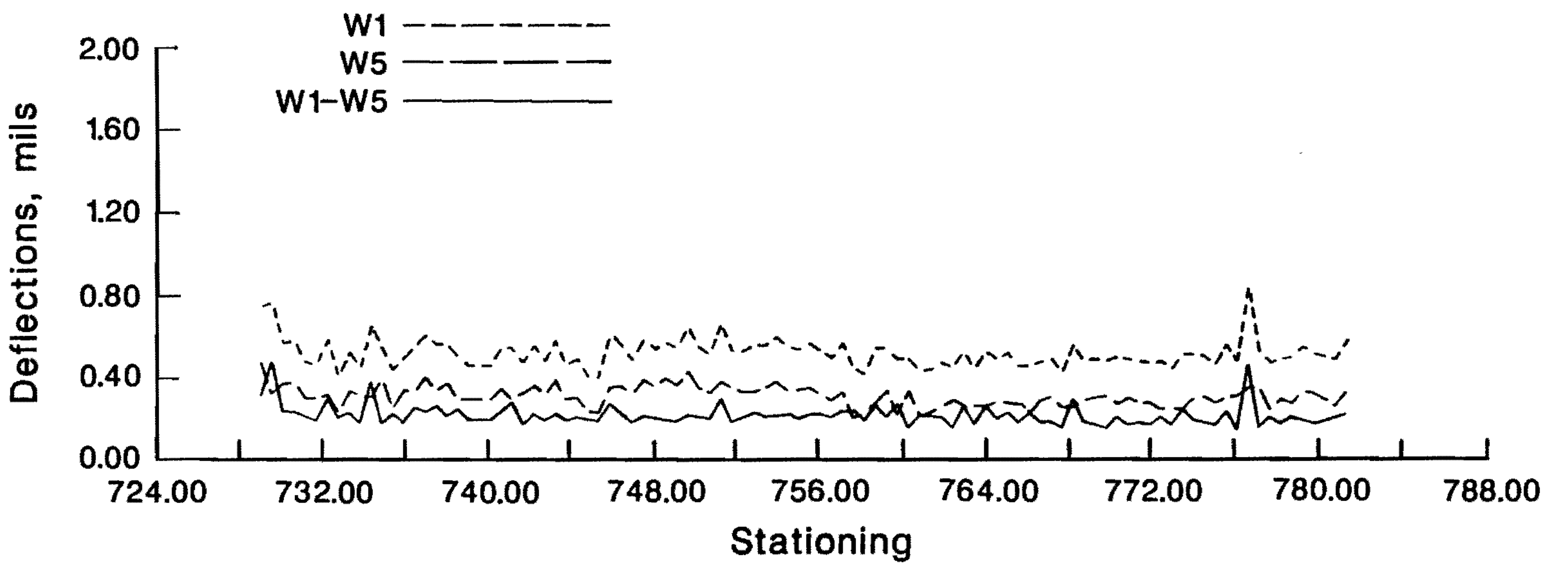

Fig B.4. Deflection profile of the Cooke sections.

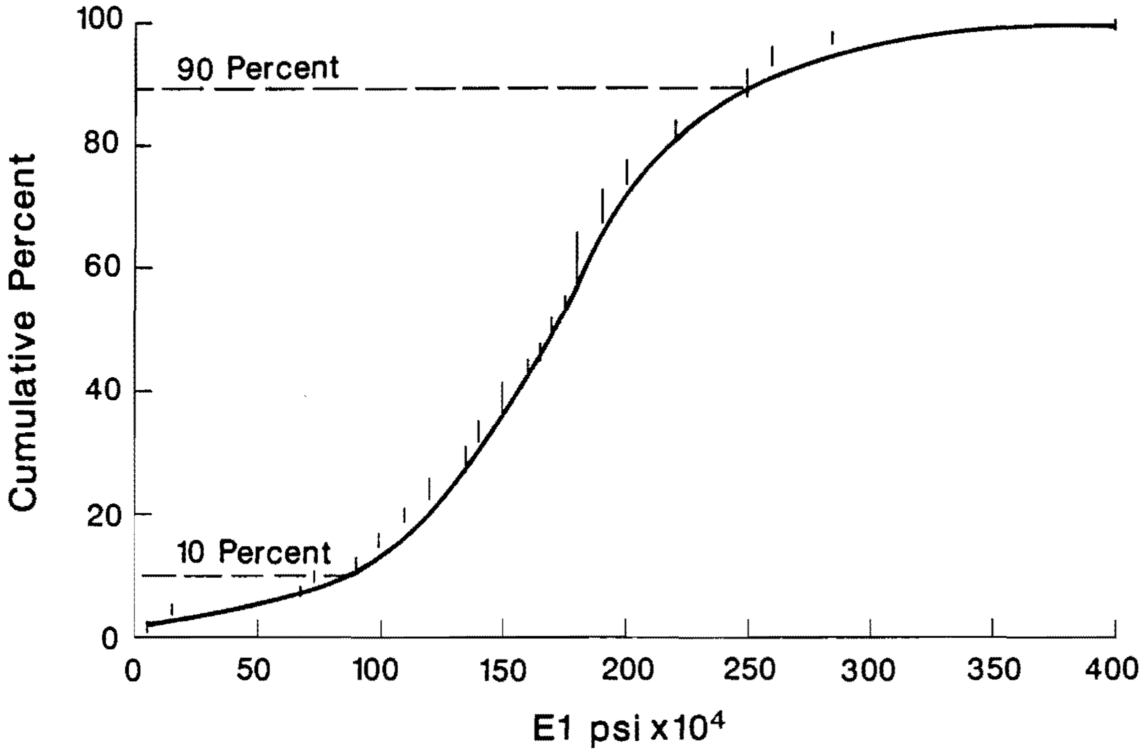

Fig B.5. Cumulative frequency distribution of Young's modulus of the first layer.

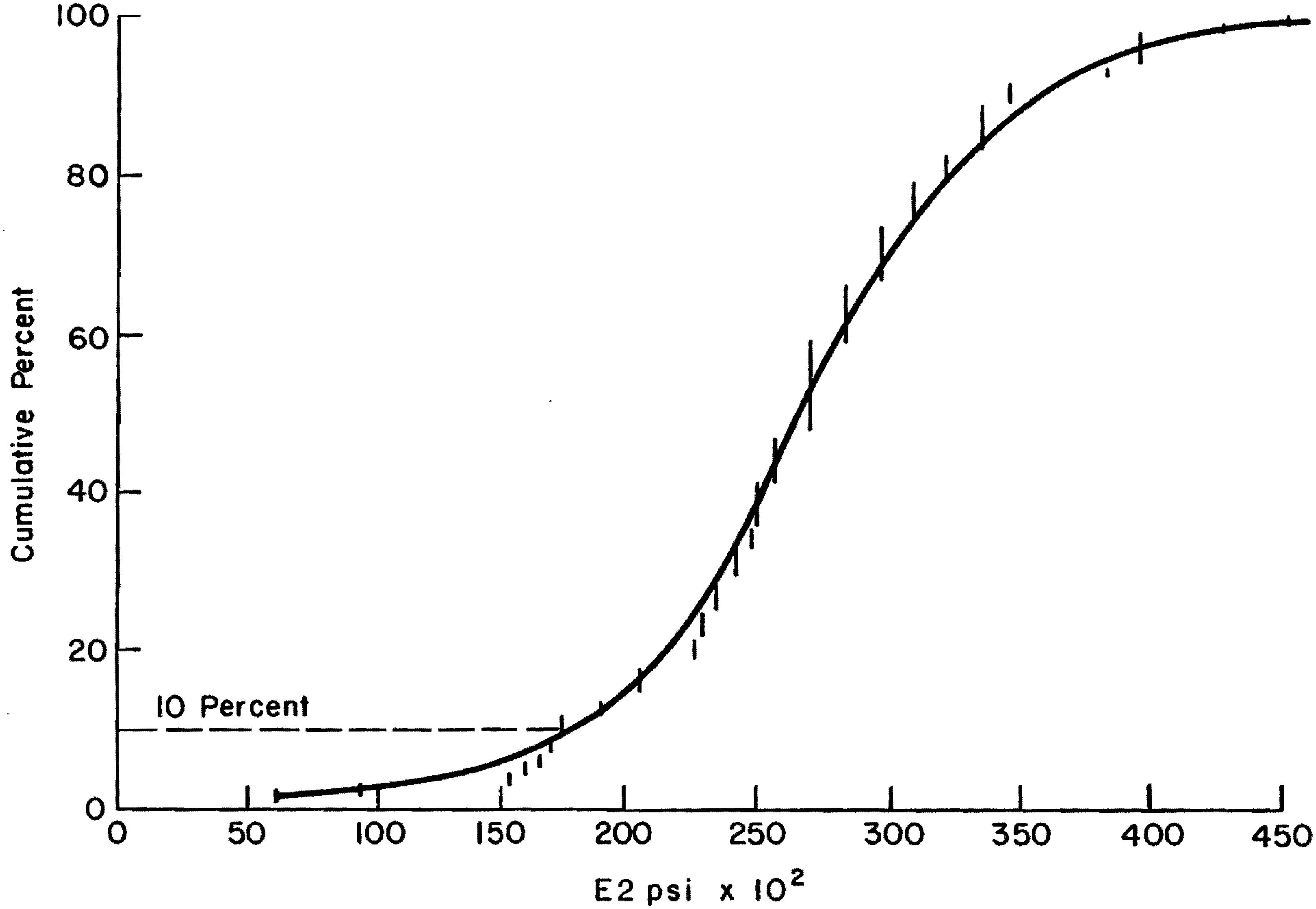

Fig B.6. Cumulative frequency distribution of Young's modulus of the second layer.

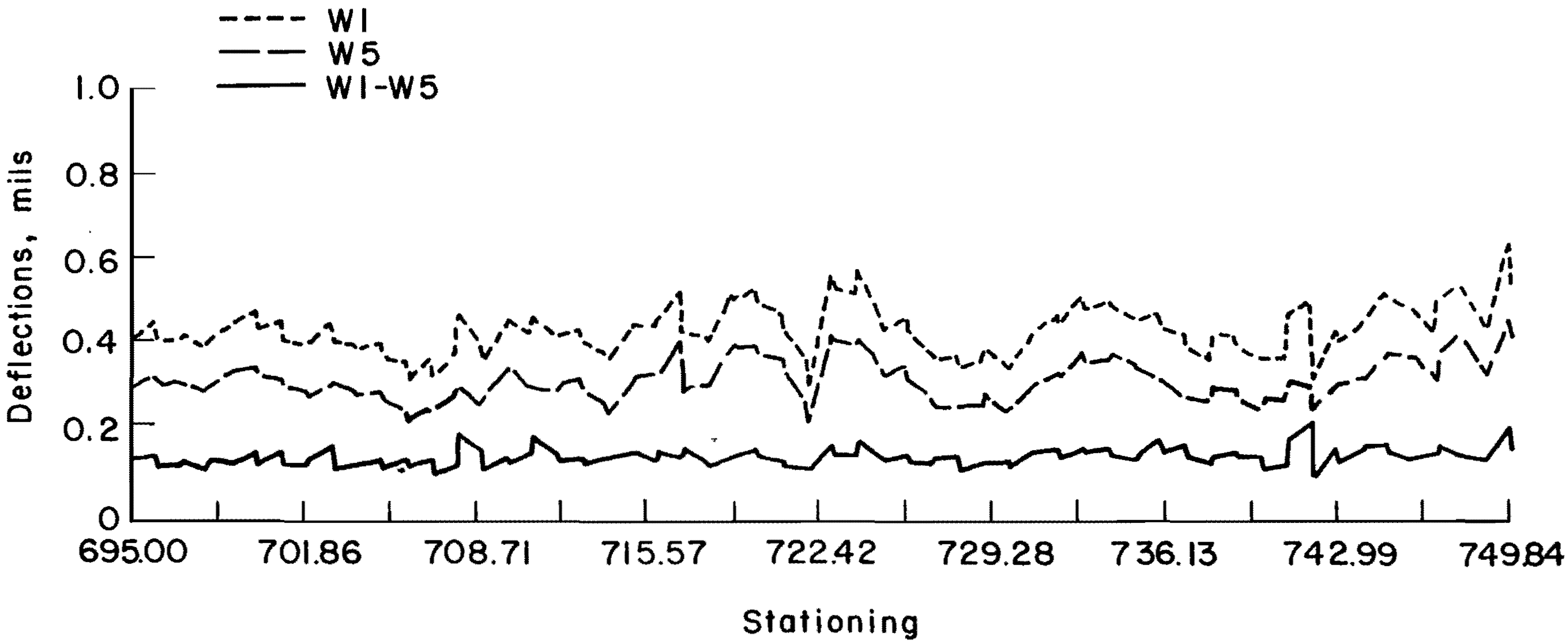

Fig B.7. Deflection profile of the McLennan section.

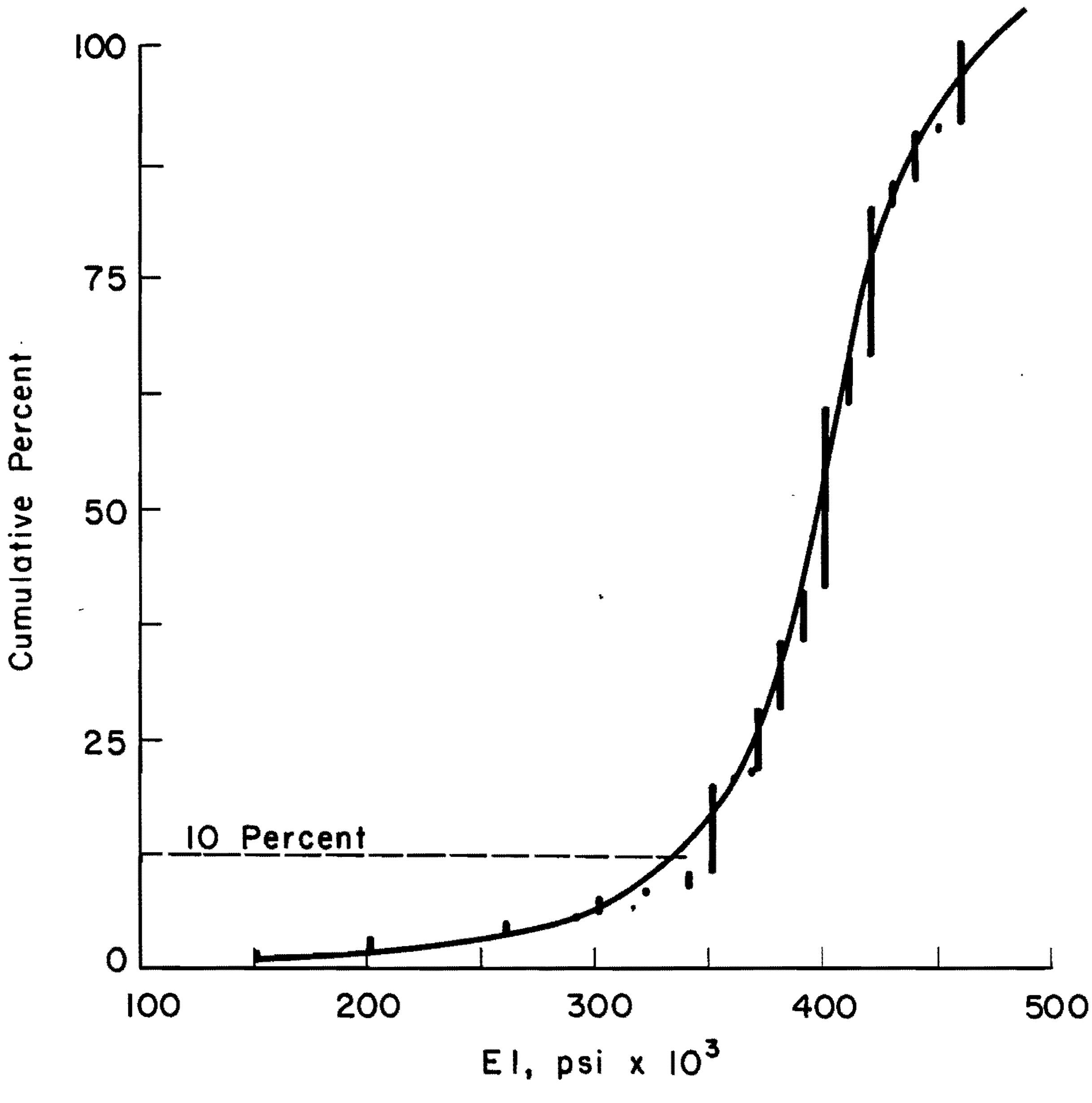

Fig B.8. Cumulative frequency distribution of Young's modulus of the first layer.

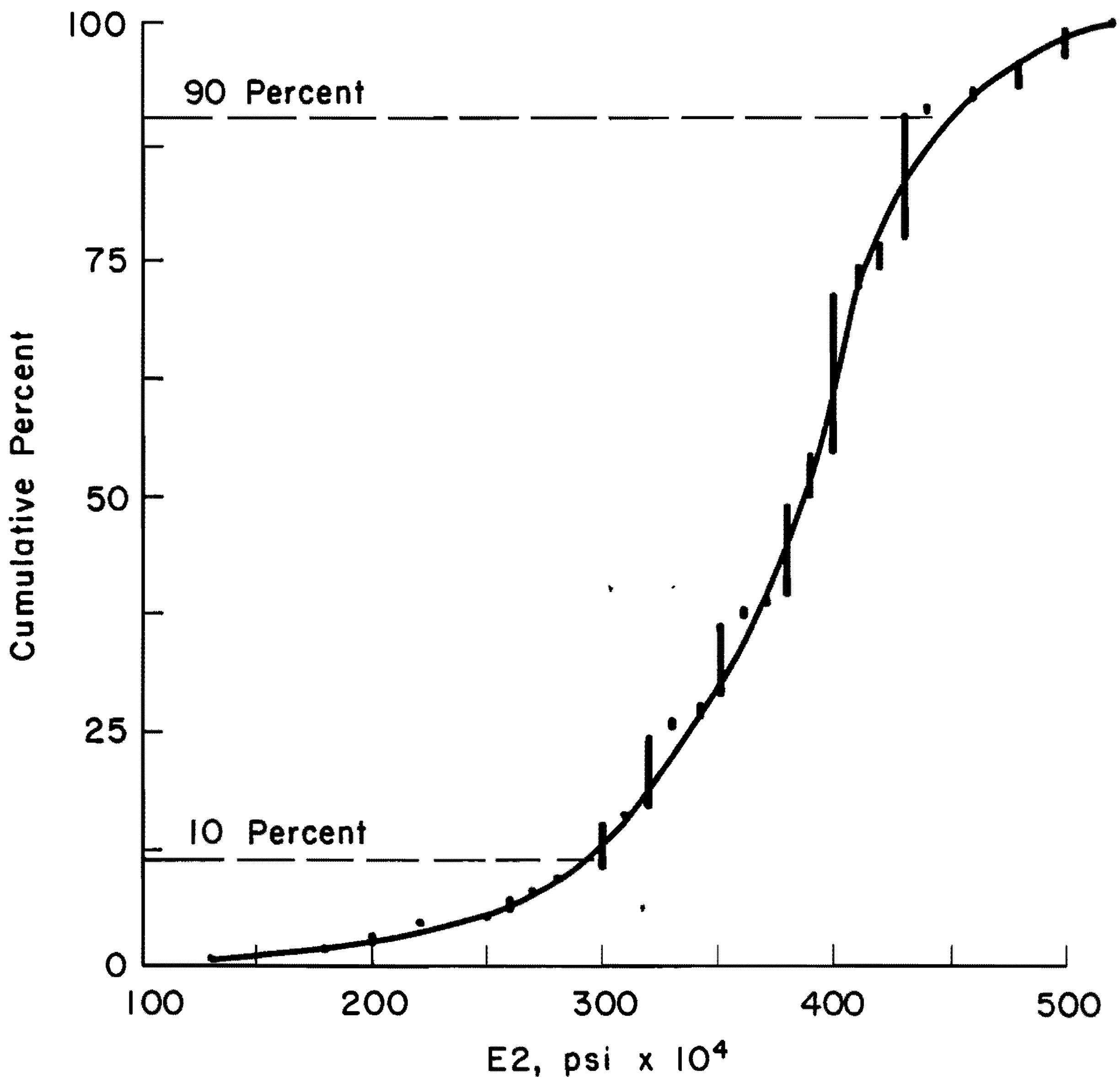

Fig B.9. Cumulative frequency distribution of Young's modulus of the second layer.

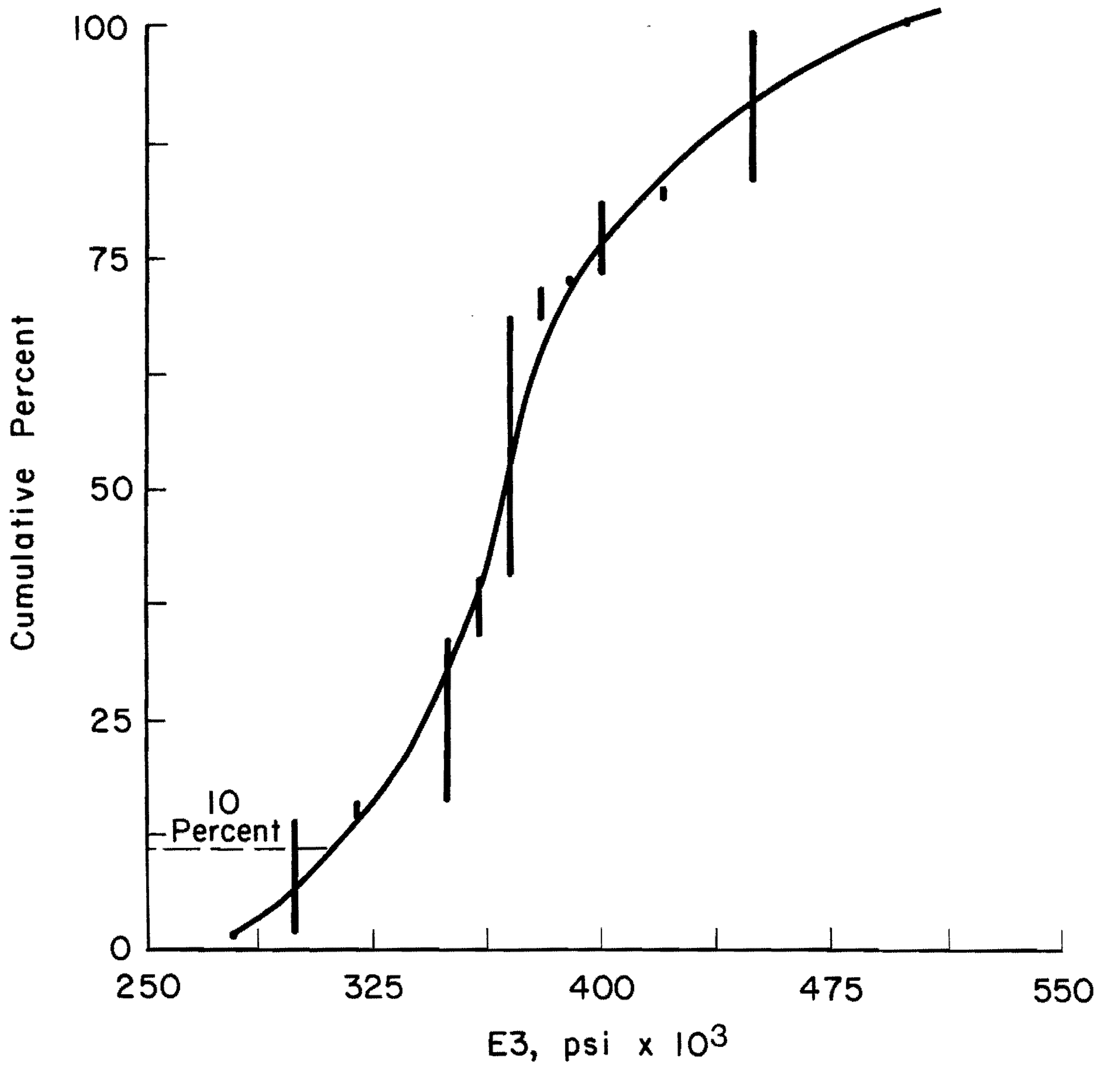

Fig B.10. Cumulative frequency distribution of Young's modulus of the third layer.

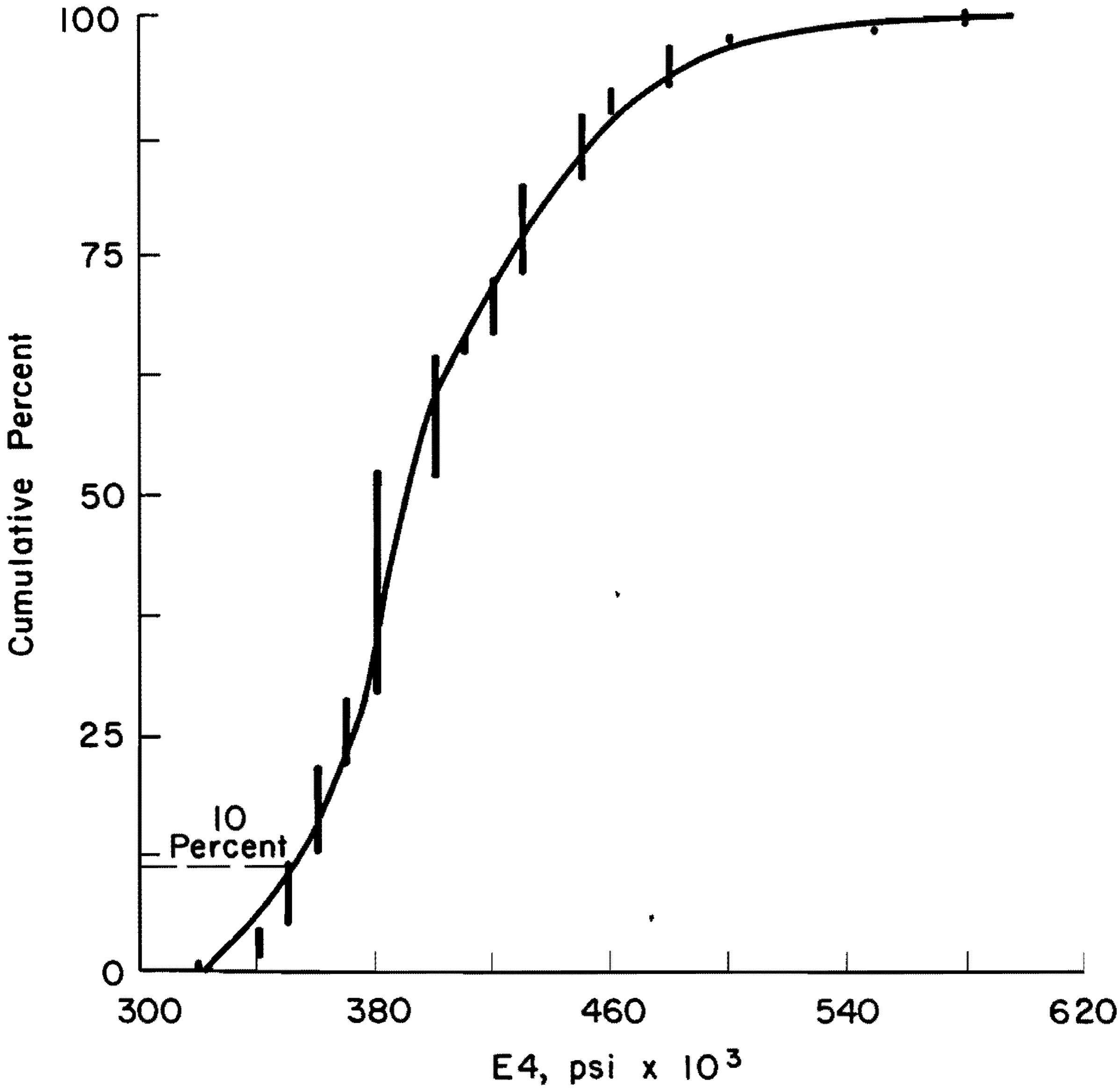

Fig B.11. Cumulative frequency distribution of Young's modulus of the fourth layer.

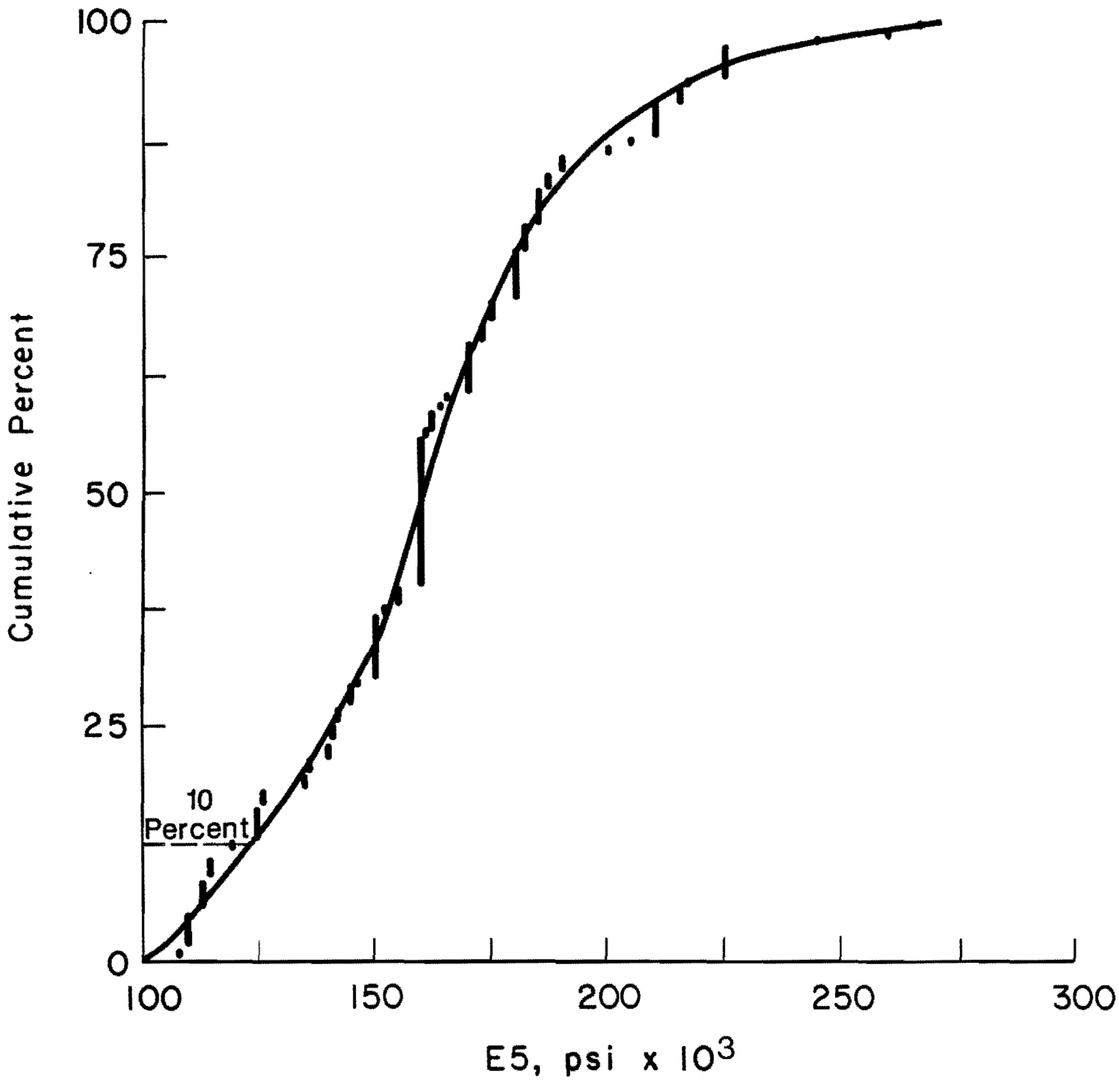

Fig B.12. Cumulative frequency distribution of Young's modulus of the subgrade, fifth layer.

TABLE B.1. CRITICAL STRESSES ON THE JCP AND THE PSP UNDER A STANDARD 18-K ESAL. NON-BROKEN JCP CONDITION

Overlay Thickness (inch)	$\sigma_{tJ2\ CR}$ (psi)	$\sigma_{tP2\ CR}$ (psi)
4	110.60	267.45
5	100.80	247.94
6	91.00	223.40
7	81.20	198.09
8	72.80	174.18
9	63.00	152.69
10	56.00	133.87

TABLE B.2. SEQUENCE OF COMPUTATIONS TO OBTAIN N_{L1}

Overlay Thickness (inch)	$N_{5\%} \times 10^6$ (18-k ESAL)	$N_{90\%} \times 10^6$ (18-k ESAL)	$N_{L1} = N_{90\%} \times \frac{RL}{100}$
4	15.77	74.30	48.29
5	21.22	100.00	65.00
6	29.43	138.68	90.14
7	42.38	199.71	129.82
8	60.11	283.26	184.12
9	95.48	450.00	292.50
10	139.18	655.86	426.10

TABLE B.3. CRITICAL STRESSES ON THE JCP AND THE PSP UNDER A STANDARD 18-K ESAL. NONBROKEN JCP CONDITION

Overlay Thickness (inch)	$\sigma_{tJ2\ CR}$ (psi)	$\sigma_{tP2\ CR}$ (psi)
4	89.24	156.93
5	84.60	147.60
6	79.65	136.14
7	74.54	124.50
8	69.40	113.36
9	64.28	102.99
10	59.37	93.53

APPENDIX C

CHARTS OF JOINT WIDTH VERSUS TEMPERATURE FROM PAST PRESTRESSED EXPERIMENTAL PROJECTS AND COMPUTATIONS OF JOINT MOVEMENTS

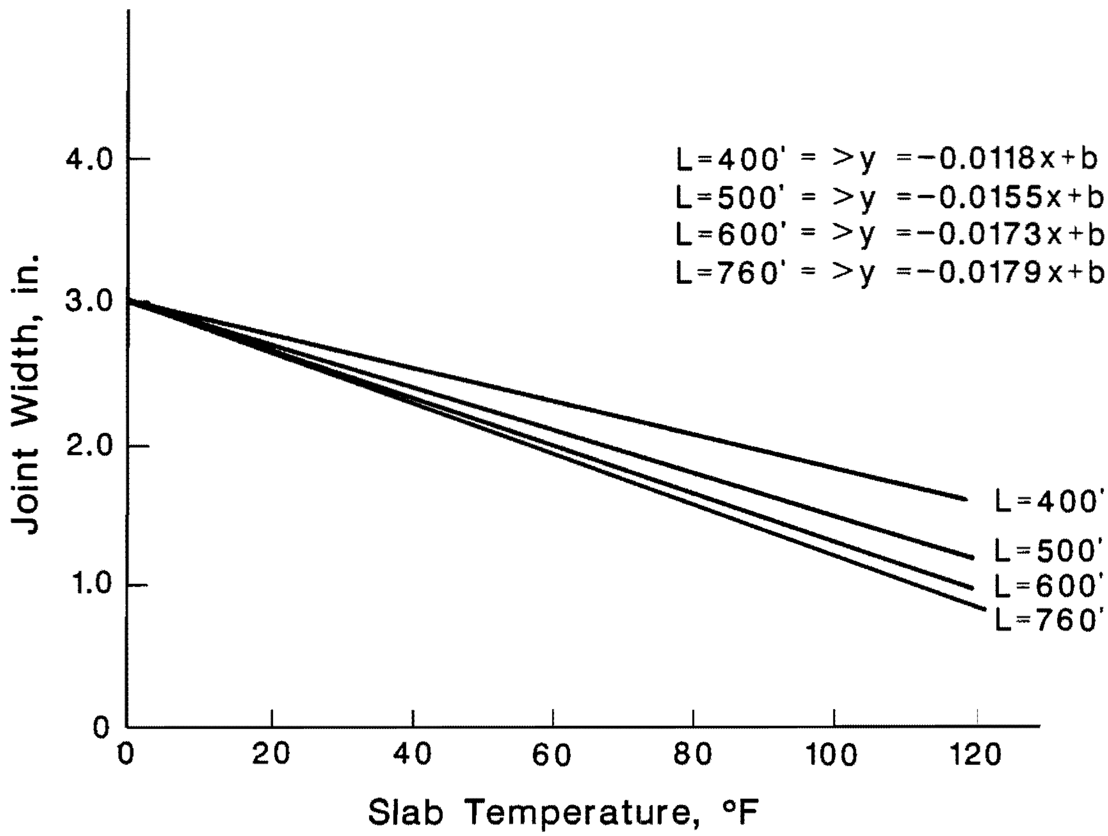

Fig C.1. Joint width versus slab temperature for different slab lengths, Virginia project (Ref 14).

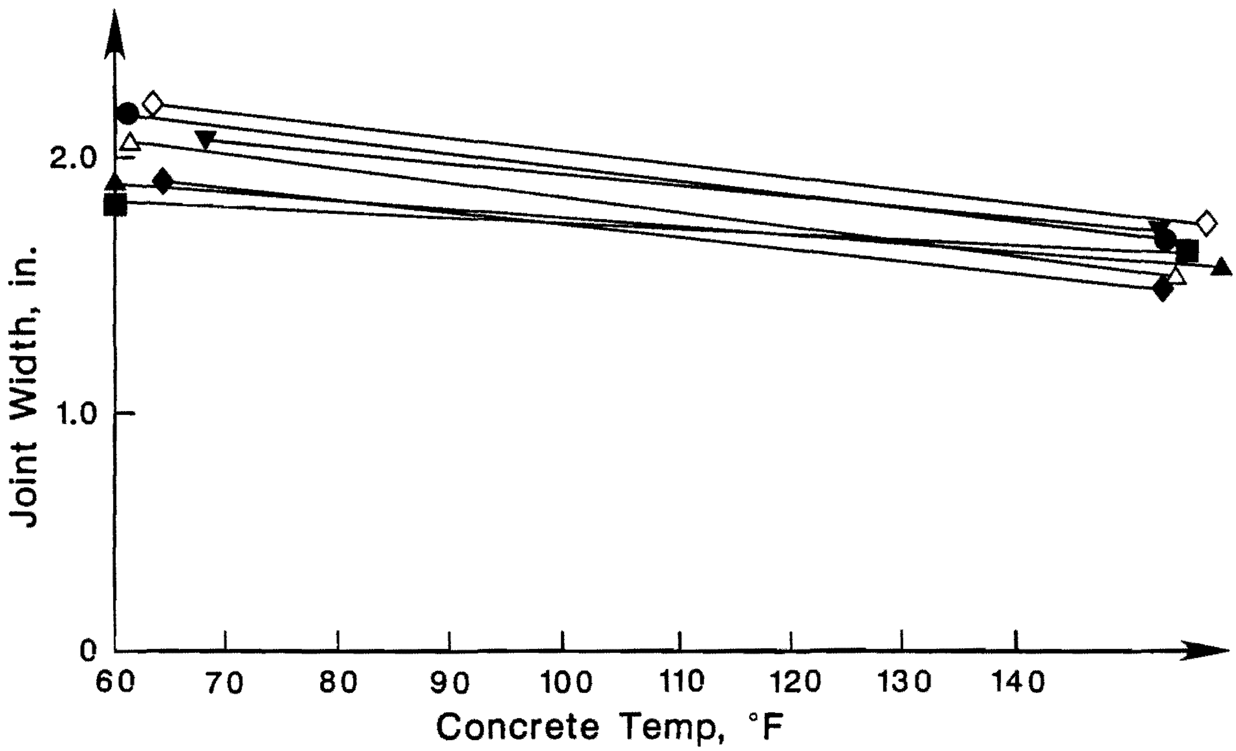

Fig C.2. Joint width versus slab temperature for slabs of the Arizona project (Ref 15).

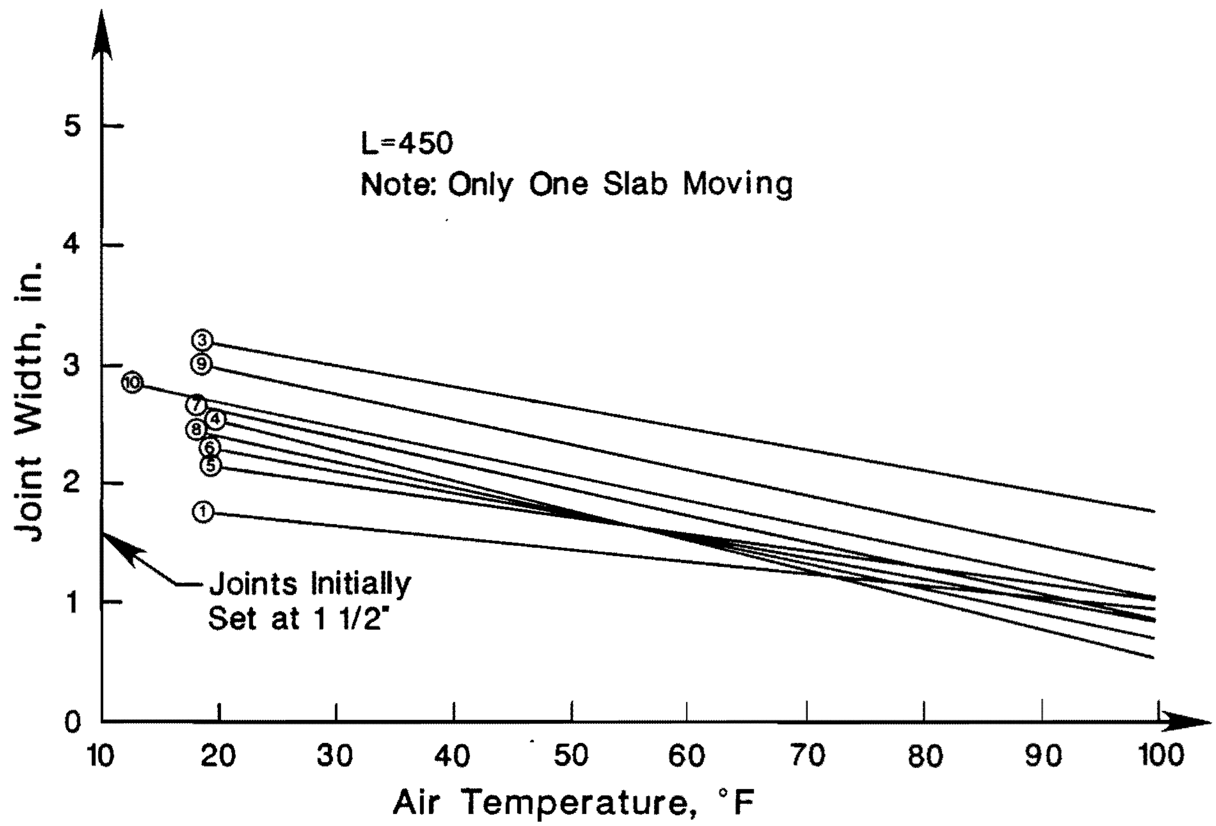

Fig C.3. Joint width versus air temperature for 450 feet slab length, Mississippi project (Ref 16).

TABLE C.1. COMPUTATIONS OF JOINT MOVEMENTS

Slab Length L, (feet)	Factors that Affect Joint Movements						Total
	Temperature						
	Seasonal d_1	Summer d_2	Winter d_3	Shrinkage d_4	Creep d_5	Elastic Shortening d_6	d_T (inch) $= \sum_1^6 di$
250	0.615	0.332	0.456	0.450	0.225	0.158	2.24
350	0.861	0.449	0.623	0.630	0.315	0.221	3.10
400	0.984	0.504	0.703	0.720	0.360	0.253	3.52
450	1.207	0.590	0.781	0.860	0.425	0.294	4.16
500	1.230	0.608	0.857	0.900	0.450	0.316	4.36
600	1.476	0.702	1.000	1.080	0.540	0.379	5.18

TABLE C.1. (CONTINUED)

Equations:

d_1	=	$\alpha \cdot \Delta T \cdot L$
d_2	=	$\alpha \cdot \Delta T_S \cdot L - d_f$
d_3	=	$0.85 \cdot \alpha \cdot \Delta T_W \cdot L - d_f$
d_4	=	$\varepsilon_s \cdot L$
d_5	=	$\varepsilon_k \cdot L$
d_6	=	$f_{av} \cdot L/Eci$

where

α	=	thermal coefficient of contraction and expansion
	=	5×10^6 in./in. - °F
ΔT	=	seasonal temperature change from summer to winter
	=	86 - 45 = 41°F
ΔT_S	=	temperature change during summer day
	=	110 - 86 = 24°F
ΔT_W	=	temperature change during winter day
	=	48 - 10 = 38°F
L	=	slab length, feet
d_f	=	amount of longitudinal movement restrained by friction
	=	$\sigma_f \cdot L\ 12 \cdot E_c = 4.5 \times 10^{-7}\ L^2$
σ_f	=	maximum friction stress = $Ms \cdot \gamma \cdot L\ 1288 = 0.3L$
us	=	average coefficient of friction = 0.6
γ	=	concrete unit weight
Ec	=	concrete modulus of elasticity = 4×10^6 psi
ε_s	=	final shrinkage strain = 0.00015 in./in.
ε_k	=	final creep strain = 0.000075 in./in.

APPENDIX D

CONSTRUCTION SPECIFICATIONS

SPECIAL SPECIFICATION

ITEM 3307

PRESTRESSED CONCRETE PAVEMENT

1. DESCRIPTION. THIS ITEM SHALL GOVERN FOR THE FURNISHING, STORING AND HANDLING OF PRESTRESSING MATERIALS AND FOR THE CONSTRUCTION AND PRESTRESSING OF CONCRETE PAVEMENT IN ACCORDANCE WITH THE PLANS AND THE REQUIREMENTS OF THIS SPECIFICATION.

2. MATERIALS.

(1) CONCRETE PAVEMENT. MATERIALS AND PROPORTIONS FOR CONCRETE USED IN CONSTRUCTION OF THIS ITEM SHALL CONFORM TO THE REQUIREMENTS AS SPECIFIED IN THE ITEM "CONCRETE PAVEMENT (WATER CEMENT RATIO)", EXCEPT COARSE AGGREGATE SHALL CONFORM TO GRADE 3 AS SHOWN IN ITEM 421, "CONCRETE FOR STRUCTURES". THE COARSE AGGREGATE WILL BE SUBJECTED TO FIVE CYCLES OF BOTH THE SODIUM SULFATE AND THE MAGNESIUM SULFATE SOUNDNESS TEST IN ACCORDANCE WITH TEST METHOD TEX-411-A. WHEN THE LOSS IS GREATER THAN 12 PERCENT WITH SODIUM SULFATE AND/OR 18 PERCENT WITH MAGNESIUM SULFATE, FURTHER TESTING WILL BE REQUIRED PRIOR TO ACCEPTANCE OR REJECTION OF THE MATERIAL.

ENTRAINED AIR WILL BE REQUIRED AND THE CONCRETE SHALL BE DESIGNED TO ENTRAIN 6 PERCENT AIR. CONCRETE AS PLACED SHALL CONTAIN THE PROPER AMOUNT AS REQUIRED WITH A TOLERANCE OF PLUS OR MINUS 1-1/2 PERCENTAGE POINTS. OCCASIONAL VARIATIONS BEYOND THIS TOLERANCE WILL NOT BE CAUSE FOR REJECTION.

THE CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3600 PSI AT THE AGE OF 28 DAYS AND SHALL CONTAIN A MINIMUM OF 6 SACKS OF CEMENT PER CUBIC YARD OF CONCRETE.

(2) REINFORCING STEEL. REINFORCING STEEL SHALL CONFORM TO THE REQUIREMENTS AS SPECIFIED IN THE ITEM, "CONCRETE PAVEMENT (WATER CEMENT RATIO)".

(3) EXPANSION JOINTS. EXPANSION JOINTS SHALL CONFORM TO THE REQUIREMENTS AS SPECIFIED IN THE SPECIAL SPECIFICATION ITEM 4006, "SEALED EXPANSION JOINT", AND AS SHOWN IN THE PLANS.

(A) ARMOR ANGLES. THE ARMOR ANGLES SHALL CONFORM TO THE REQUIREMENTS AS SPECIFIED IN THE ITEMS, "STEEL STRUCTURES" AND "METAL FOR STRUCTURES".

(B) JOINT EXTRUSION. THE JOINT EXTRUSION SHALL CONFORM TO THE REQUIREMENTS OF ASTM DESIGNATION A-588 AND THE CONFIGURATION SHOWN ON THE PLANS.

(C) DIAPHRAGM. THE NEOPRENE DIAPHRAGM USED SHALL BE AN EXTRUDED NEOPRENE MATERIAL CONFORMING TO THE REQUIREMENTS AS SPECIFIED IN THE ITEM, "ELASTOMERIC MATERIALS". GEN-STRIP CD AS MANUFACTURED BY THE GENERAL TIRE & RUBBER COMPANY, WABASH, INDIANA, OR AN APPROVED EQUAL MAY BE USED.

(D) DOWEL BARS. DOWEL BARS SHALL CONFORM TO THE REQUIREMENTS AS SPECIFIED IN THE ITEM, "CONCRETE PAVEMENT (WATER CEMENT RATIO)", AND AS SHOWN ON THE PLANS. THEY SHALL BE ENCASED WITH STAINLESS STEEL OR MONEL METAL WHICH SHALL CONFORM TO THE REQUIREMENTS OF ASTM A 167-70 OR A 176-71. THE ENCASEMENT SHALL NOT BE LESS THAN 0.01 INCH IN THICKNESS.

(E) DOWEL BAR EXPANSION SLEEVES. DOWEL BAR EXPANSION SLEEVES SHALL BE STAINLESS STEEL TUBING CONFORMING TO THE REQUIREMENTS OF ASTM A 269. THEY SHALL BE WELDED TO THE ARMOR ANGLE AT THE LOCATIONS SHOWN IN THE PLANS. THE FREE END OF THE SLEEVE SHALL BE CAPPED TO PREVENT ENTRY OF MORTAR.

(4) PRESTRESSING STRANDS. PRESTRESSING STRANDS SHALL BE SIX-TENTH (0.6) INCH GREASE COATED PLASTIC ENCASED CABLE.

(A) CABLE. THE CABLE SHALL BE UNCOATED SEVEN-WIRE STRESS-RELIEVED STRANDS CONFORMING TO THE REQUIREMENTS OF AASHTO DESIGNATION M 203-82 OR ASTM A 416-80.

(B) GREASE. THE GREASE APPLIED TO THE CABLE PRIOR TO ENCASEMENT SHALL MEET THE FOLLOWING REQUIREMENTS:

1. DROPPING POINT (MELTING POINT ASTM D 566) 300 DEGREES F MINIMUM. NO FLOWING OR LEAKAGE OFF THE WIRES AT HIGH AMBIENT TEMPERATURES.

2. FLASH POINT (CLEVELAND OPEN CUP TEST) 350 DEGREES F MINIMUM.

3. FORM A FILM THAT IS SOFT, PLIABLE AND SELF HEALING.

4. ASTM CONE PENETRATION AT 77 DEGREES F (310-335).

5. ASTM CORROSION TEST (D 1743) - PASS 1,1,1.

6. WATER SOLUBLE CHLORIDES, NITRATES AND SULFIDES SHALL EACH HAVE A PPM LESS THAN 2.

(C) ENCASING MATERIAL. THE ENCASING MATERIAL SHALL BE POLYETHYLENE CONFORMING TO THE REQUIREMENTS OF ASTM D 1248, TYPE II, CATEGORY III OR POLYPROPYLENE CONFORMING TO THE REQUIREMENTS OF ASTM D 2146, TYPE I AND SHALL HAVE A MINIMUM THICKNESS OF 0.036 INCH.

(5) PRESTRESSING STRAND ANCHORAGE DEVICES. THE ANCHORAGE DEVICES FOR POST-TENSIONING SHALL HOLD THE PRESTRESSING STEEL AT A LOAD PRODUCING STRESS OF NOT LESS THAN 95 PERCENT OF THE MINIMUM TENSILE STRENGTH OF THE PRESTRESSING STEEL.

THE LOAD FROM THE ANCHORAGE DEVICE SHALL BE DISTRIBUTED TO THE CONCRETE BY MEANS OF APPROVED DEVICES THAT WILL EFFECTIVELY DISTRIBUTE THE LOAD TO THE CONCRETE.

SUCH APPROVED DEVICES SHALL CONFORM TO THE FOLLOWING REQUIREMENTS:

(A) THE AVERAGE BEARING STRESSES ON THE UNCONFINED CONCRETE CREATED BY THE DEVICE SHALL NOT EXCEED THE VALUES ALLOWED BY THE FOLLOWING EQUATIONS:

1. AT SERVICE LOAD = 0.6F'C TIMES THE SQUARE ROOT OF THE QUANTITY A'B/AB BUT NOT GREATER THAN F'C.

2. AT TRANSFER LOAD = 0.8F'CI TIMES THE SQUARE ROOT OF THE QUANTITY (A'B/AB)-0.2 WHERE AB IS THE BEARING AREA OF THE DEVICE, A'B IS THE MAXIMUM AREA OF THE DEVICE BEARING SURFACE THAT IS SIMILAR TO AND CONCENTRIC WITH THE BEARING AREA OF THE DEVICE AND F'CI IS THE COMPRESSIVE STRENGTH OF THE CONCRETE AT THE TIME OF INITIAL PRESTRESS.

(B) BENDING STRESSES IN THE PLATES OR ASSEMBLIES INDUCED BY THE PULL OF THE PRESTRESSING SHALL NOT CAUSE VISIBLE DISTORTION WHEN 85 PERCENT OF THE ULTIMATE LOAD IS APPLIED AS DETERMINED BY THE ENGINEER. PLASTIC FLEXURAL STRENGTH OF THE PLATES OR ASSEMBLIES SHALL BE ADEQUATE FOR 125 PERCENT OF THE ULTIMATE LOAD. DESIGN SHALL NOT BE BASED ON A YIELD STRESS IN THE PLATES OR ASSEMBLIES OF GREATER THAN 50 KSI.

(6) PRESTRESSING STRAND COUPLERS. TENDON COUPLERS SHALL BE USED ONLY AT LOCATIONS SPECIFICALLY SHOWN ON THE PLANS OR APPROVED BY THE ENGINEER AND SHALL CONFORM TO THE TESTING SECTION BELOW.

(7) COATED TENDONS, END ANCHORAGES AND TENDON COUPLERS. COATED TENDONS, END ANCHORAGES AND TENDON COUPLERS SHALL DEVELOP AT LEAST 100 PERCENT OF THE REQUIRED ULTIMATE STRENGTH OF THE TENDON, WITH A MINIMUM ELONGATION OF 2 PERCENT, AND IN ADDITION SHALL WITHSTAND 500,000 CYCLES FROM 60 TO 70 PERCENT OF THE REQUIRED ULTIMATE STRENGTH OF THE TENDON WITHOUT FAILURE OR SLIPPAGE.

ALL TENDONS SHALL BE IDENTIFIED BY HEAT NUMBER, OR REEL NUMBER IN THE CASE OF SEVEN WIRE STRAND, AND TAGGED FOR IDENTIFICATION. ANCHORAGE ASSEMBLIES SHALL BE IDENTIFIED IN A LIKE MANNER THE CONTRACTOR SHALL FURNISH SPECIMENS FOR TEST PURPOSES IN ACCORDANCE WITH TEST METHOD TEX-710-I.

TESTING OF COMPLETE TENDONS FOR COMPLIANCE WITH THE REQUIREMENTS OF PARAGRAPHS (1) AND (2) ABOVE WILL BE AT THE CONTRACTOR'S EXPENSE AND THE RESULTS CERTIFIED IN WRITING TO THE ENGINEER. IN ADDITION, THE CONTRACTOR SHALL FURNISH, FOR TESTING, ONE SPECIMEN OF EACH SIZE OF PRESTRESSING TENDON, INCLUDING COUPLINGS, OF THE SELECTED TYPE, WITH END FITTINGS ATTACHED, FOR ULTIMATE STRENGTH TESTS ONLY. THESE SPECIMENS SHALL BE 5 FEET IN CLEAR LENGTH, MEASURED BETWEEN ENDS OF FITTINGS. IF THE RESULTS OF THE TEST INDICATE THE NECESSITY OF CHECK TESTS, ADDITIONAL SPECIMEN SHALL BE FURNISHED WITHOUT COST. FOR PRESTRESSING SYSTEMS PREVIOUSLY TESTED AND APPROVED ON OTHER DEPARTMENT PROJECTS, COMPLETE TENDON SAMPLES NEED NOT BE FURNISHED, PROVIDED THERE IS NO CHANGE IN THE MATERIAL, DESIGN, OR DETAILS PREVIOUSLY APPROVED. SHOP DRAWINGS OR PRESTRESSING DETAILS SHALL IDENTIFY THE PROJECT ON WHICH APPROVAL WAS OBTAINED; OTHERWISE SAMPLING WILL BE NECESSARY.

(8) STRAND STRESSING EQUIPMENT. HYDRAULIC JACKS USED TO STRESS STRANDS SHALL BE EQUIPPED WITH EITHER A PRESSURE GAUGE OR A LOAD CELL FOR DETERMINING THE JACK STRESS, AT THE OPTION OF THE CONTRACTOR. THE PRESSURE GAUGE, IF USED, SHALL HAVE AN ACCURATELY READING DIAL AT LEAST 6 INCHES IN DIAMETER WITH GRADUATIONS OF NO MORE THAN 500 POUNDS AND EACH JACK AND ITS GAUGE SHALL BE CALIBRATED AS A UNIT WITH THE CYLINDER EXTENSION IN THE APPROXIMATE POSITION THAT IT WILL BE AT FINAL JACKING FORCE, AND SHALL BE ACCOMPANIED BY A CERTIFIED CALIBRATION CHART. THE LOAD CELL, IF USED, SHALL BE CALIBRATED AND SHALL BE PROVIDED WITH AN INDICATOR BY MEANS OF WHICH THE PRESTRESSING FORCE IN THE STRAND MAY BE DETERMINED. THE RANGE OF THE LOAD CELL SHALL BE SUCH THAT THE LOWER 10 PERCENT OF THE MANUFACTURER'S RATED CAPACITY WILL NOT BE USED IN DETERMINING THE JACKING STRESS.

SAFETY MEASURES SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT ACCIDENTS DUE TO THE POSSIBLE BREAKING OF THE PRESTRESSING STEEL OR THE SLIPPING OF THE GRIPS DURING THE POST-TENSIONING PROCESS.

(9) FRICTION REDUCING MEMBRANE. THE FRICTION REDUCING MEMBRANE SHALL BE A SINGLE LAYER OF 6 MIL POLYETHYLENE SHEETING CONFORMING TO THE REQUIREMENTS OF ASTM D 2103, TYPE I. THE MEMBRANE SHALL BE PROVIDED IN SECTIONS TWO FEET WIDER AND TWO FEET LONGER THAN THE CORRESPONDING SLAB POUR.

3. CONSTRUCTION METHODS.

(1) FRICTION REDUCING MEMBRANE. THE MEMBRANE SHALL BE PLACED TRANSVERSELY OVER THE ENTIRE CROWN WIDTH WITHOUT LONGITUDINAL LAPS. EACH TRANSVERSE SECTION OF MEMBRANE SHALL LAP THE ADJACENT SECTION A MINIMUM OF 12 INCHES. THE CONTRACTOR, AT HIS OPTION, MAY FURNISH AND PLACE POLYETHYLENE MEMBRANE LONGITUDINALLY PROVIDED THAT THE MEMBRANE HAS A MINIMUM WIDTH OF THE PRESTRESSED CONCRETE PAVEMENT BEING PLACED (PLUS TWO FEET) AND THAT THE MEMBRANE HAS NO LONGITUDINAL JOINTS.

THE MEMBRANE SHALL BE IN PLACE, SECURELY ANCHORED AND WITHOUT FOLDS, BEFORE PLACING CONCRETE THEREON. THE MEMBRANE SHALL EXTEND ONE FOOT BEYOND THE SLAB POUR ON ALL SIDES. THE MEMBRANE SHALL NOT BE AFFIXED TO THE BASE COURSE, EXCEPT AT THE OUTSIDE EDGES AND WIDELY SPACED OTHER POINTS AS NECESSARY; BUT SHALL REMAIN FREE TO MOVE AS THE POST-TENSIONING FORCE IS APPLIED TO THE CONCRETE PAVEMENT OVERLAY.

(2) PLACING CONCRETE. CONCRETE PAVEMENT SHALL BE DESIGNED, FORMED, MIXED, PLACED, FINISHED AND CURED IN CONFORMANCE WITH THE REQUIREMENTS OF THE ITEM, "CONCRETE PAVEMENT (WATER CEMENT RATIO)".

(3) TRANSVERSE JOINTS. THE JOINTS SHALL BE INSTALLED IN CONFORMANCE WITH THE REQUIREMENTS OF SPECIAL SPECIFICATION ITEM 4006, "SEALED EXPANSION JOINT" AND AS SHOWN ON THE PLANS.

DOWEL BARS SHALL BE ACCURATELY INSTALLED IN JOINT ASSEMBLIES IN ACCORDANCE WITH THE PLANS, EACH PARALLEL TO THE PAVEMENT SURFACE AND TO THE CENTERLINE OF THE PAVEMENT, AND SHALL BE RIGIDLY SECURED IN THE REQUIRED POSITION BY SUCH MEANS THAT WILL PREVENT THEIR DISPLACEMENT DURING PLACING AND FINISHING OF THE CONCRETE.

PRIOR TO THE INSTALLATION OF THE NEOPRENE DIAPHRAGM, THE ARMOR EDGES SHALL BE THOROUGHLY CLEANED OF DEBRIS AND DIRT AND AN ADHESIVE LUBRICANT APPLIED LIBERALLY IN THE RECESSES OF THE JOINT EXTRUSION.

(4) PRESTRESSING STRAND PLACEMENT. LONGITUDINAL AND TRANSVERSE POST-TENSIONING STRANDS SHALL BE PLACED AT THE LOCATIONS SHOWN ON THE PLANS AND SUPPORTED BY APPROVED CHAIRS. CHAIRS SHALL NOT DAMAGE THE FRICTION REDUCING MEMBRANE OR STRAND ENCASING MATERIAL. TOLERANCE FOR THE HORIZONTAL POSITIONING OF THE STRANDS WILL BE PLUS OR MINUS ONE INCH FROM THE PLAN PLACEMENT. TOLERANCE FOR VERTICAL POSITIONING OF STRANDS WILL BE PLUS OR MINUS ONE-FOURTH INCH OR ONE PERCENT OF THE DEPTH OF PAVEMENT WHICHEVER IS GREATER. WHERE LONGITUDINAL AND TRANSVERSE STRANDS INTERSECT, THEY SHALL BE TIED TOGETHER IN SUCH A MANNER AS TO PREVENT DAMAGE TO THE STRAND ENCASING MATERIAL.

(5) POST-TENSIONING POCKET. MATERIALS USED FOR FORMING THE POST-TENSIONING POCKETS SHALL BE INERT, NON-ABSORPTIVE AND BE OF ADEQUATE STRENGTH TO MAINTAIN SUFFICIENT RIGIDITY TO WITHSTAND THE FORCES OF FLOW, VIBRATION, BUOYANCY AND WEIGHT OF THE PLASTIC CONCRETE DURING PLACING.

FORMS SHALL BE ANCHORED SECURELY TO PREVENT MOVEMENT OR MISALIGNMENT DURING THE PLACING OF CONCRETE.

THE FACING OF FORMS SHALL BE TREATED WITH FORM OIL OR OTHER BOND BREAKING COATING PRIOR TO PLACING OF CONCRETE. THE OIL OR OTHER MATERIALS USED FOR THIS PURPOSE SHALL BE OF A CONSISTENCY AND COMPOSITION TO FACILITATE FORM REMOVAL. MATERIALS WHICH APPRECIABLY STAIN OR REACT WITH THE CONCRETE WILL NOT BE PERMITTED.

ALL FORMS SHALL BE CONSTRUCTED TO FACILITATE REMOVAL WITHOUT DAMAGE TO THE CONCRETE.

UPON COMPLETION OF THE POST-TENSIONING OPERATION, THE POST-TENSIONING POCKETS SHALL BE FILLED USING CONCRETE WITH THE SAME MIX DESIGN AS THE PAVEMENT. THE SURFACE FINISH OF THE COMPLETED POCKETS SHALL CONFORM TO THE ADJACENT PAVEMENT.

(6) POST-TENSIONING. LONGITUDINAL PRESTRESSING OF EACH PAVEMENT SLAB POUR SHALL BE ACCOMPLISHED IN AT LEAST TWO (2) POST-TENSIONING OPERATIONS. THE FIRST LONGITUDINAL POST-TENSIONING OPERATION FOR EACH SLAB POUR SHALL BE COMPLETED BETWEEN 4 AND 24 HOURS AFTER PLACEMENT. THE ENGINEER WILL CONDUCT TESTS TO DETERMINE THE ACTUAL TIME OF STRESSING WITHIN THIS TIME PERIOD. THE INITIAL LOADING SHALL BE AT LEAST EIGHTEEN KIPS (18,000 POUNDS) AND SHALL BE APPLIED WITHIN THE SPECIFIED TIME REGARDLESS OF WEATHER CONDITIONS OR DAY OF THE WEEK. THE SECOND AND FINAL LOADING SHALL BE FORTY-SIX AND FOUR-TENTHS KIPS (46,400 POUNDS) PER STRAND AND SHALL BE APPLIED WITHIN TWO DAYS AFTER REPRESENTATIVE CYLINDERS HAVE FIRST REACHED A BREAKING STRENGTH OF 2,000 PSI.

EACH STRAND SHALL BE LOADED BY JACKING THE STRAND IN THE POST-TENSIONING POCKETS AS INDICATED ON THE PLANS. LOADING SHALL COMMENCE WITH THE CENTER STRANDS OF THE CORRESPONDING SLAB POUR AND SHALL PROGRESS TOWARD THE EDGES BY ALTERNATELY LOADING STRANDS ON EACH SIDE OF THE CENTER STRANDS. STANDBY LOADING EQUIPMENT SHALL BE AVAILABLE ON THE PROJECT SITE IN CASE OF MALFUNCTION OF EQUIPMENT BEING USED.

IN CASE OF EQUIPMENT FAILURE OR OTHER CONDITIONS WHICH CAUSE PAVING TO STOP, THE CONTRACTOR MAY BE REQUIRED TO SET A TEMPORARY HEADER AND TO TEMPORARILY POST-TENSION THE SEGMENT OF SLAB UNDER CONSTRUCTION. HE MAY BE REQUIRED TO PLACE NO. 6 TIEBARS 18" LONG, CENTERED BETWEEN TENDONS. BEFORE CONSTRUCTION AND THROUGHOUT CONSTRUCTION THE CONTRACTOR WILL HAVE NECESSARY EQUIPMENT AND MATERIALS AVAILABLE TO ACCOMPLISH TEMPORARY PRE-STRESSING AND TEMPORARY HEADER CONSTRUCTION.

THE TRANSVERSE STRANDS SHALL BE STRESSED IN ONE (1) POST-OPERATION AFTER THE COMPLETION OF THE SECOND AND FINAL LOADING OF THE SECOND SLAB POUR. THE TRANSVERSE STRAND LOADING SHALL BE FORTY-SIX AND FOUR-TENTHS KIPS (46,400 POUNDS) PER STRAND.

IF ANY STRAND BREAKS THE ENGINEER SHALL BE NOTIFIED AND EQUIPMENT, MATERIALS AND PROCEDURES SHALL BE CHECKED BEFORE PROCEEDING WITH ADDITIONAL TENSIONING.

THE PRE-STRESSER'S ELONGATION CALCULATIONS AND MEASUREMENTS SHALL BE CHECKED BY THE ENGINEER.

(7) INSTRUMENTATION. THE PRESTRESSED CONCRETE PAVEMENT SLABS WILL BE INSTRUMENTED FOR OBSERVATION OF CONCRETE AND SLAB BEHAVIOR FROM EARLY AGE. ALL INSTRUMENTATION, LEADS, BOXES, TEMPLATES, ETC., WILL BE FURNISHED AND PLACED BY THE ENGINEER ON THE BASE, OR ON THE SURFACE OF THE CONCRETE, OR BESIDE THE PAVEMENT. THE

CONTRACTOR SHALL USE CARE AND PRECAUTION, INCLUDING MARKING AND ENCLOSING AREAS AS REQUIRED IN ORDER NOT TO DISTURB INSTRUMENTATION, LEADS, WIRING AND BOXES.

4. CONCRETE TESTING. ALL CONCRETE TEST CYLINDERS SHALL BE MOLDED, CURED AND COMPRESSION TESTED ACCORDING TO TEST METHOD TEX-418-A WITH A MINIMUM OF 2 CYLINDERS PER LOCATION TO REPRESENT A TEST. INITIALLY, EIGHT CYLINDERS FOR CONTROLLING STRESSING OPERATIONS (4 FROM EACH END) SHALL BE CAST FROM CONCRETE PLACED AT THE TERMINAL ENDS OF EACH PAVEMENT SLAB TO BE POST-TENSIONED. THESE CYLINDERS SHALL BE CAST BY THE CONTRACTOR AND SHALL RECEIVE THE SAME CURING AS THE PAVEMENT SLAB(S) THEY REPRESENT. IN ADDITION TO CASTING AND CURING THE CYLINDERS, THE CONTRACTOR SHALL FURNISH FACILITIES FOR TESTING THE CYLINDERS, INCLUDING AN APPROVED CONCRETE TESTING MACHINE OF SUFFICIENT CAPACITY AND CALIBRATED BY AN ACCEPTABLE COMMERCIAL LABORATORY. THE MACHINE SHALL BE OPERATED BY THE CONTRACTOR'S REPRESENTATIVE IN THE PRESENCE OF A DEPARTMENT REPRESENTATIVE TO WITNESS AND RECORD STRENGTHS OBTAINED ON EACH BREAK. THE NUMBER OF CYLINDERS CAST FROM EACH SLAB MAY BE REDUCED WHEN A STRENGTH-GAINING PATTERN HAS BEEN ESTABLISHED. RECORDS OF ALL TESTS SHALL BE MAINTAINED SHOWING THE CONCRETE SLAB WHICH EACH CYLINDER REPRESENTS, THE DATES OF CASTING AS WELL AS TESTING, AND THE BREAKING STRENGTH OF THE CYLINDER.

5. MEASUREMENT. PRESTRESSED CONCRETE PAVEMENT WILL BE MEASURED BY THE SQUARE YARD COMPLETE IN PLACE BASED UPON THE DIMENSIONS SHOWN ON THE PLANS.

6. PAYMENT. THE WORK PERFORMED AND MATERIALS FURNISHED AS PRESCRIBED BY THIS ITEM AND MEASURED AS PROVIDED UNDER "MEASUREMENT" WILL BE PAID FOR AT THE UNIT PRICE BID FOR "PRESTRESSED CONCRETE PAVEMENT", OF THE DEPTH SPECIFIED, WHICH PRICE SHALL BE FULL COMPENSATION FOR FURNISHING, LOADING AND UNLOADING, STORING, HAULING AND HANDLING ALL CONCRETE INGREDIENTS, INCLUDING ALL FREIGHT INVOLVED; FOR PLACING AND ADJUSTING FORMS; FOR MIXING, PLACING, FINISHING AND CURING ALL CONCRETE; FOR FURNISHING AND INSTALLING ALL MATERIALS INCLUDING REINFORCING STEEL, PRESTRESSING STRANDS, DOWELS, END ANCHORAGES, COUPLERS AND FRICTION REDUCING MEMBRANE; FOR ALL POST-TENSIONING OPERATIONS; FOR FURNISHING AND INSTALLING ALL DEVICES FOR PLACING AND SUPPORTING THE REINFORCING STEEL, PRESTRESSING STRANDS AND DOWELS AND FOR ALL MANIPULATIONS, LABOR, EQUIPMENT, APPLIANCES, TOOLS AND INCIDENTALS NECESSARY TO COMPLETE THE WORK.

EXPANSION JOINTS WILL BE MEASURED AND PAID FOR UNDER SPECIAL SPECIFICATION ITEM 4006, "SEALED EXPANSION JOINT".