FIELD VERIFICATION OF BRIDGE DECK POST-TENSIONING

by

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Research Report 389-1F

Research Project 3-5-86-389

Conducted for

Texas

State Department of Highways and Public Transportation

In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

October 1987

PREFACE

This report summarizes a field measurement program assessing the use of deck prestressing on a standard pretensioned girder and slab bridge. Potential improvements in durability of bridge decks through deck prestressing have been documented as part of Project 3-5-82-316 entitled "Application of Transverse Prestressing to Bridge Decks." The design procedures recommended in that project were utilized by Texas State Department of Highways and Public Transportation (TSDHPT) to design several spans of a new crossing of the Colorado River near LaGrange, Texas. This report contains a summary of the instrumentation of two of the slabs of that bridge to determine the level of stress attained during post-tensioning.

This work is part of Research Project 3-5-86-389, entitled "Field Verification of Bridge Deck Lateral Post-Tensioning." The study was conducted by the Phil M. Ferguson Structural Engineering Laboratory as part of the overall research program of the Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin. The work was sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration.

Liaison with the TSDHPT was maintained through the contact representative James Wall and the project liaison Mary Lou Ralls who greatly assisted the project by improving field communications. The overall study was directed by Dr. Ramon L. Carrasquillo, Associate Professor of Civil Engineering, and Dr. John E. Breen, The Nasser I. Al-Rashid Chair in Civil Engineering. The detailed work was carried out under the immediate supervision of Mr. Josef Farbiarz, Research Engineer, and Mr. Reid W. Castrodale, Assistant Research Engineer. The authors are indebted to a large number of undergraduate and graduate students who were impressed into service to help with this project and who toiled diligently through very adverse weather conditions at a distant site. Thanks are especially due to Jack Burgess, Jose Calixto, Lisa Carter, Reid Castrodale, Josef Farbiarz, David Hartmann, Dominic Kelly, and Dave Schuman. Finally, the authors are indebted to Mr. Timothy Bradberry, Assistant Research Engineer, who helped with the data reduction, and Mr. Robert B. Anderson, Assistant Research Engineer, who was responsible for analysis and plotting of the data and contours.

SUMMARY

The prestressing of bridge decks is an attractive concept with substantial potential benefits in both economy and improved durability. However, the practicality and applicability of the concept for slab and girder type bridges has not been previously documented in full size field applications. This study documents the results of measurements made during longitudinal and transverse prestressing of the decks of two spans of a full scale pretensioned girder and slab bridge crossing the Colorado River near LaGrange, Texas. It addresses such important design areas as the effective distribution of edge prestressing force across a bridge slab as affected by both diaphragm and girder restraints as well as some organizational and technical problems encountered by the constructor during prestressing. The results of the field measurements of deck strains are summarized in this report.

IMPLEMENTATION

This report summarizes a field measurement program to determine the efficiency of longitudinal and transverse prestressing of a bridge deck cast-in-situ on conventional pretensioned girders. The measurements show that a reasonable level of prestress was attained, although the distribution was far from uniform. In addition, it contains comments regarding several organizational and technical problems which occurred during the prestressing operations. The suggestions for solutions of these problems may be of assistance in future prestressed deck projects.

TABLE OF CONTENTS

LIST OF FIGURES

Figure | Page

CHAPTER 1

INTRODUCTION

1.1 Background

The possibility of making significant improvements in cast-in-situ deck durability through deck prestressing was introduced in Project 3-5-82-316 [1,2]. One of the major concerns of that project was the development of design procedures for achieving a suitable amount of longitudinal and transverse prestress of a bridge deck. The application of deck prestressing as shown in Fig. 1.1 appears to be a straightforward matter. However, the possible restraint of girders and diaphragms, as shown in Fig. 1.2, caused substantial question as to proper design techniques for the distribution of deck prestressing to overcome local restraints. A comprehensive design procedure was suggested in Ref. 2. The Texas State Department of Highways and Public Transportation (TSDHPT) decided to evaluate this design procedure by trial on a full scale prototype structure. A new crossing of the Colorado River near LaGrange, Texas, was chosen for the evaluation and the design of the bridge. Some spans were designed with lateral deck post-tensioning while other spans had both lateral and longitudinal deck post-tensioning. One span of each type was selected for instrumentation and observation during post-tensioning. The spans selected were immediately adjacent to the abutments, where access could be gained to the underside of the spans for application of strain gages.

1.2 Objective of This Research

The principal objective of this research project was to secure concrete deck strain readings before and after post-tensioning of the prototype structure for comparisons with results of the already completed laboratory studies and design recommendations.

1.3 Report Contents

This report covers the field monitoring of the two spans of the Colorado River Bridge. A description of the instrumentation and problems encountered in the field study are contained in Chapter 2. Results of the field observations in terms of observed strains and strains contours are presented in Chapter 3. Comparisons with the predicted strains based on the design procedure utilized are provided. The major conclusions are summarized in Chapter 4. The plans provided by the TSDHPT to the investigators are shown in the Appendix.

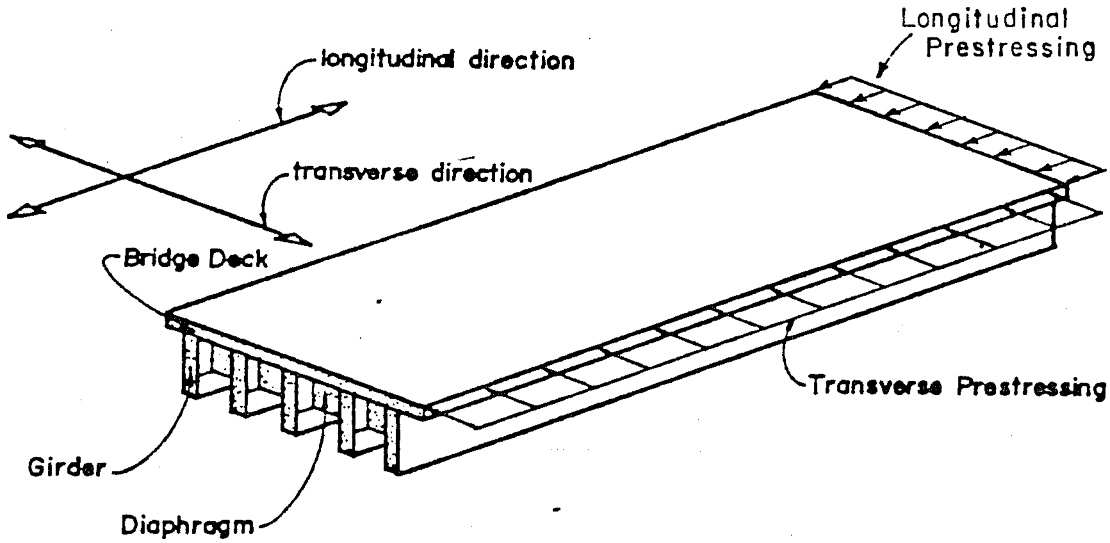


Fig. 1.1 Deck prestressing of a conventional slab-girder bridge

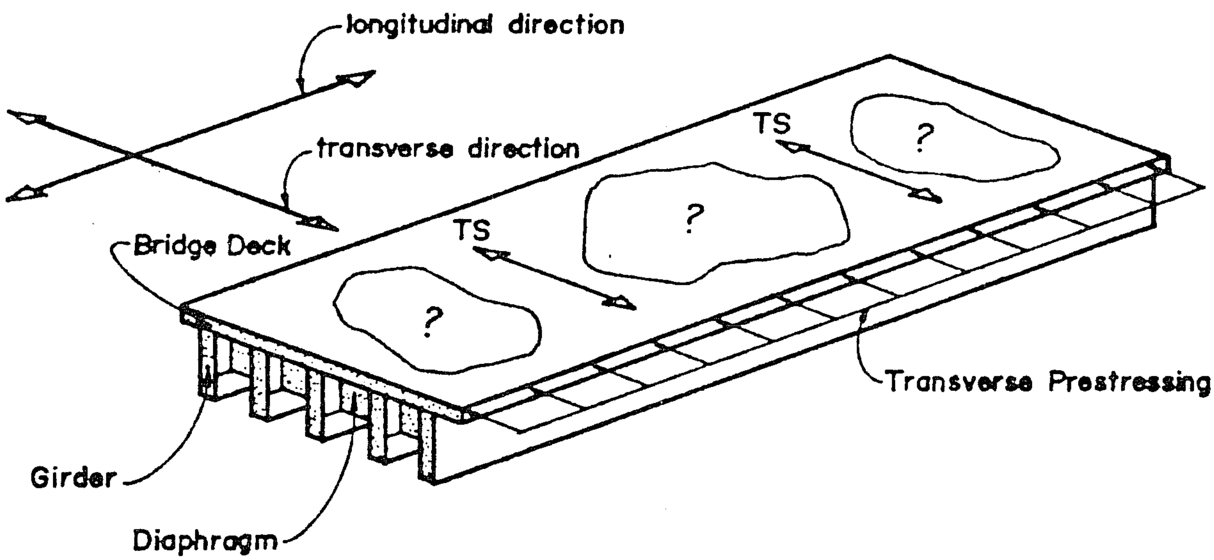


Fig. 1.2 The effects of girders, webs and diaphragms on transverse prestress distibution

CHAPTER 2

BRIDGE INSTRUMENTATION

2.1 General

The new Colorado River Bridge at LaGrange, Texas has an overall length of 1325 ft extending from Sta. 89+50 on the west to Sta. 102+75 on the east. The overall bridge is made up as follows:

Stations	Spans	Deck Post-Tensioning
89+50 - 91+00	2 @ 75'	Longitudinal & Transverse
91+00 - 96+00	4 @ 125'	None
96+00 - 99+75	5 @ 75'	None
99+75 - 102+75	4 @ 75'	Transverse

The 86-ft wide bridge spans were constructed with ten pretensioned Texas C-beams with transverse spacing of 8.89 ft and a cast-in-situ deck with 8-in. thickness. Typical details of the western spans (Sta. 89+50 to 91+00) with combined longitudinal and transverse deck post-tensioning are shown in the Appendix Figs. A.1 and A.2. Typical details of the eastern spans (Sta. 99+75 to 102+75) with transverse deck post-tensioning are shown in Figs. A.3 and A.4. The contractor elected to use the monostrand tendon option throughout. Transverse tendons were located at midheight of the slab while longitudinal tendons were located 1 in. lower to allow clearance where they intersected transverse tendons. The contractor also elected to use End Diaphragm Option No. 2 shown on Fig. A.5. The 75-ft spans had a single midspan line of transverse interior diaphragms made from [12 x 20.7] channels as shown on the typical interior detail of Fig. A.5. The interior diaphragms were installed prior to stressing. However, since the axial stiffness of these steel diaphragms is only 28% that of standard concrete diaphragms, no compensating extra tendons at interior diaphragm locations were provided.

The slab concrete was specified to be class "S" with a 3600-psi minimum 28-day f'_c. Although the plans allowed a minimum compressive strength of 2000 psi for application of post-tensioning, the post-tensioner specified 3000 psi. Tendon spacings were based on jacking to 0.75 f'_s, seating no higher than 0.70 f'_s, friction losses based on K = 0.0002, anchor set of 5/8 in., and other losses of 33 ksi. Low relaxation strands were used. The plans called for final longitudinal post-tensioning force of 11.9 k/ft of width and final transverse longitudinal post-tensioning of 40.3 k/ft of width except at diaphragms where 64.5 k/ft of width was to be provided for a 4-ft wide strip centered on interior diaphragms and from end of slab for end diaphragms.

This increased force was to compensate for exterior diaphragm restraints.

The contractor elected to use unbonded monostrand tendons. The tendon layouts are shown in Figs. 2.1 and 2.2. Final stress calculations were based on use of 1/2-in. ϕ low-lax strand 270-ksi tendons with an assumed anchor seating of 1/4 in., long term losses of 20 ksi, K = 0.0013 and a strand modulus of 29,050 ksi. On this basis the final effective post-tensioning after all losses per strand was calculated as 23.9 k/strand in the 86-ft transverse direction and 24.4 k/strand in the 150-ft longitudinal direction. The desired nominal strand spacing in the longitudinal direction was then 24.4/11.9 = 2.1 ft ≈ 26 in. The desired nominal strand spacing in the transverse direction in the non-diaphragm regions was then 23.9/40.3 = 0.59 ft ≈ 7 in. on centers. In the end diaphragm region this had to be decreased to 23.9/64.5 = 0.37 ft ≈ 4-1/2 in. on centers. The actual spacings called for were 2 ft-2 in. for longitudinal tendons and 7 in. for transverse tendons except at end diaphragms where the spacing was decreased to 5 in. over a 50-in. width. (See Figs. A.1-A.4.)

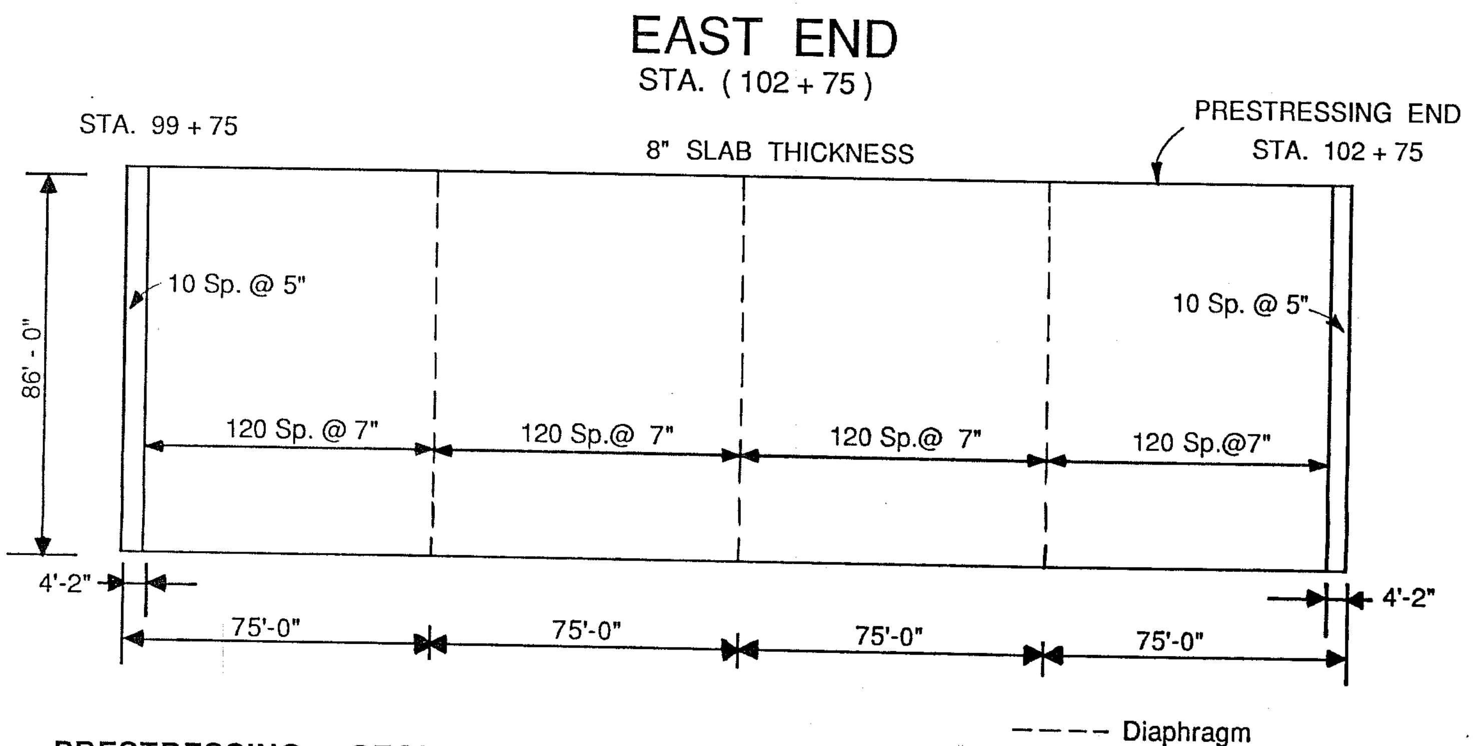
2.2 Instrumented Spans

The instrumented portions of the LaGrange Colorado River bridge under study corresponded to the half spans (37 ft-6 in. in length) immediately adjacent to the abutments at both west (Sta. 89+50) and east (Sta. 102+75) ends of the bridge. Because of the symmetry of the span and the limited budget available, the decision was made to concentrate instrumentation in a typical half span for each of the proposed post-tensioning layouts. These sections were instrumented with strain gages and thermocouples applied at locations on the top and the bottom of the slab surface as shown in Figs. 2.3 and 2.4. Because of surface preparation difficulties as outlined subsequently, slab top gages could not be properly applied to the test section on the east end (near Sta. 102+75) so that only bottom gages were effective in that span.

The four span section at the east end of the bridge, which was 300-ft long, contained transverse prestressing only. The two span section at the west end of the bridge, which was 150-ft long, was prestressed in both longitudinal and transverse directions. Figures 2.1 and 2.2 show the tendon distribution and spacing, and the prestressing sequence followed in each case.

2.3 Instrumentation

2.3.1 Strain Gage Type. The strain gages used were of the type PL-60-11, manufactured by TML Industries, having a gage length of 60 mm.

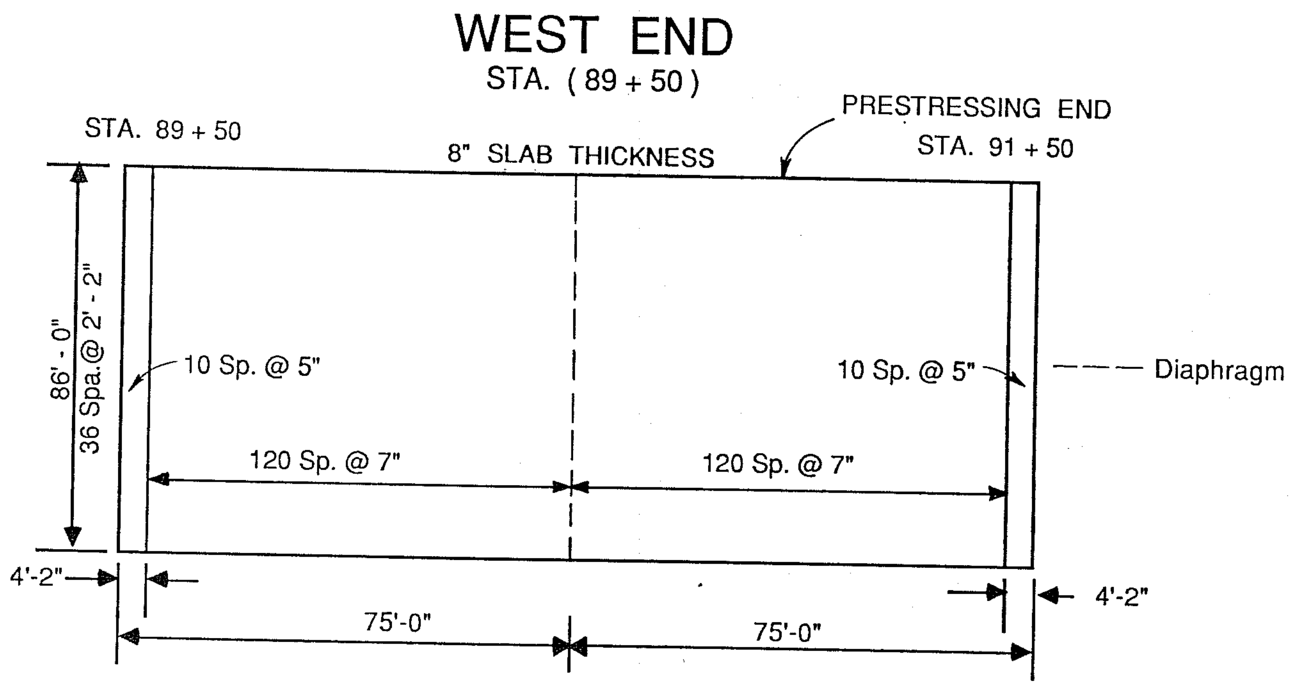


PRESTRESSING SEQUENCE: $1/2$ inch - Grade 270 Strands

(Stress Pattern : Stress every 4 th Tendon in Four Runs until all Tendons were stressed.

1 st RUN: Beginning from Tendon No.1 from Abutment at Sta. 102+ 75, Stress every 4th Tendon.
2 nd RUN: Beginning from the Third Tendon from Abutment at Sta. 99+ 75, Stress every 4th Tendon.
3 rd RUN: Beginning with Tendon No.2 from Abutment at Sta. 102+ 75, Stress every 4th Tendon.
4 th RUN: Beginning with the 4th Tendon from Abutment at Sta. 99 + 75, Stress every 4th Tendon.

Fig. 2.1 Tendon distribution and tensioning sequence for section with only transverse post-tensioning



PRESTRESSING SEQUENCE: $^1/_2$ inch - Grade 270 Strands

(Stress Pattern : Transverse tendons stressed before longitudinal tendons.
Transverse tendons stressed in four runs from end to end.)

TRANSVERSE TENDONS:

1 st RUN: Beginning from Tendon No.1 from Abutment at Sta. 89 + 50, Stress every 4th Tendon.
2 nd RUN: Beginning from the Fourth Tendon from Abutment at Sta. 91+ 50, Stress every 4th Tendon.
3 rd RUN: Beginning from Tendon No.2 from Abutment at Sta. 89 + 50, Stress every 4th Tendon.
4 th RUN: Beginning from Tendon No.3 from Abutment at Sta. 91 + 50, Stress every 4th Tendon.

LONGITUDINAL TENDONS:

1 st RUN: Beginning from Tendon No.1 on the North edge stress every other Tendon.
2 nd RUN: Beginning from Tendon No.2 on the North edge stress every other Tendon.

Fig. 2.2 Tendon distribution and tensioning sequence for section with both transverse and longitudinal post-tensioning

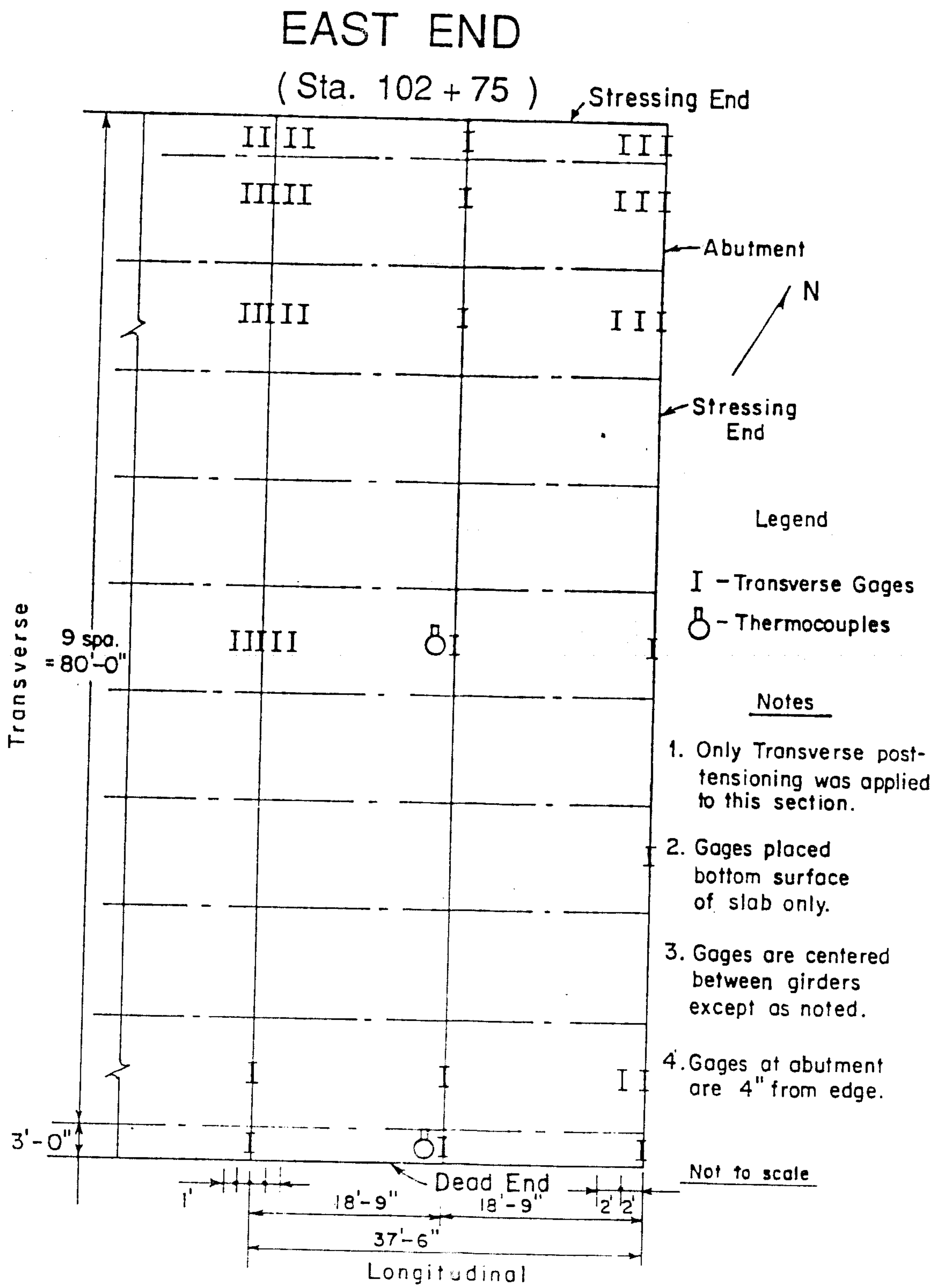


Fig. 2.3 Instrumentation locations for section with only transverse post-tensioning

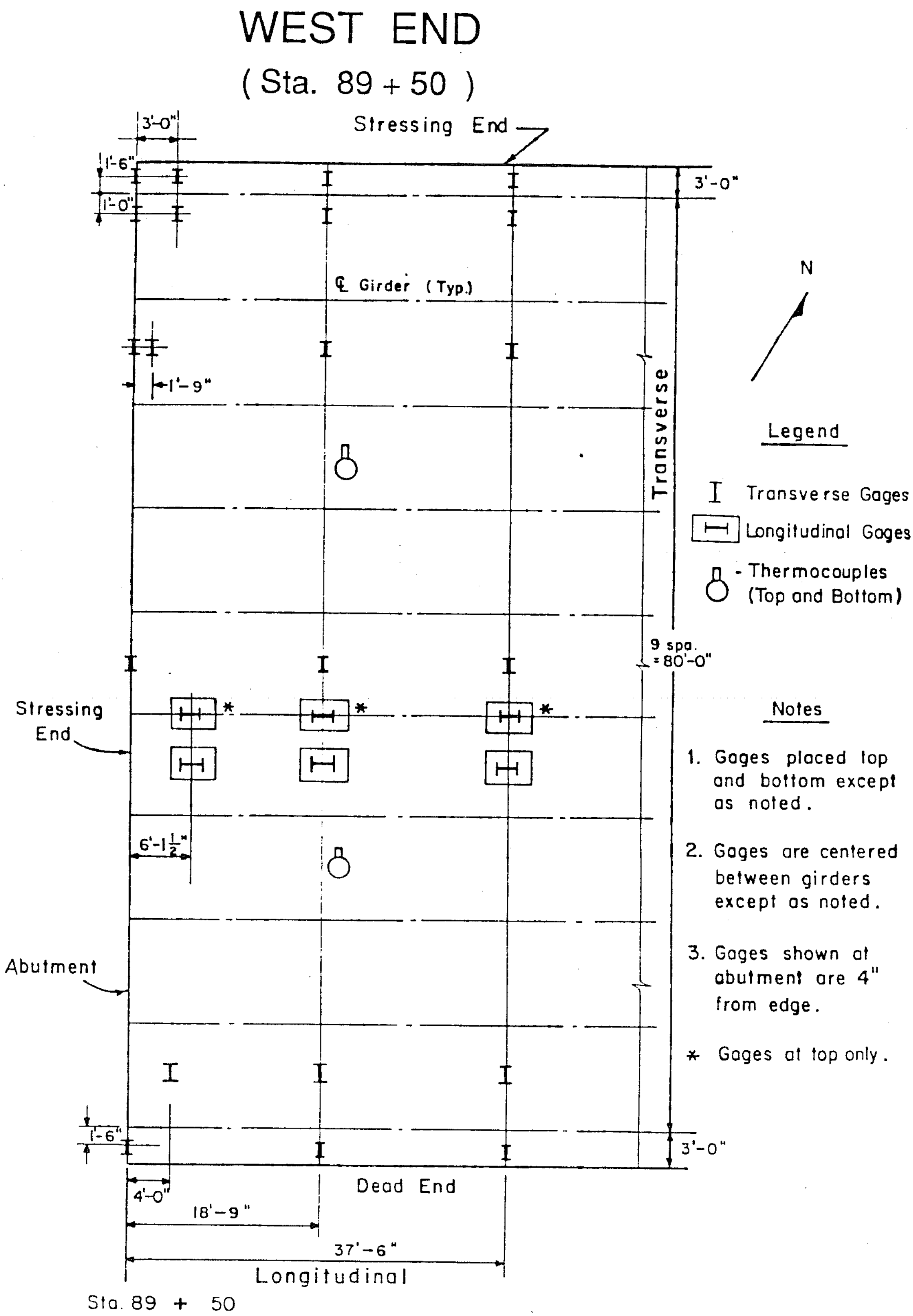


Fig. 2.4 Instrumentation locations for section with both transverse and longitudinal post-tensioning

2.3.2 Application. At the location where a strain gage was to be installed, about 1/16-in. depth of the surface of the concrete slab was removed by grinding to expose sound material free of dust and/or weak paste. The prepared surface was then cleaned using acetone. Strain gages were subsequently glued to the surface using a commercially available 5-minute epoxy as can be seen in Fig. 2.5.

2.3.3 Problems Encountered. Any experimental project conducted out of a laboratory environment poses special difficulties. These are intrinsic contradictions between a carefully controlled, specific-purpose laboratory experiment as opposed to the rush instrumentation of an ongoing construction job executed in an uncontrolled environment.

A field monitoring experiment generally involves instrumentation that requires time consuming and elaborate preparation and installation procedures, as well as lengthy and careful data collection while testing. These characteristics often call for longer planning periods and stricter plan interpretation than those expected on a regular construction job. In this case, the problem was complicated by lack of special provisions for delays in the construction contract and by a very short time available for planning.

Furthermore, the virtually impossible task of controlling the environment in an open field at a construction site makes it very difficult for the investigators to isolate the effect of the variables being monitored from those unaccounted for in the experiment's design (temperature variation with time, weather effect on instruments, etc.).

In addition, the geographical remoteness of this project demanded extra coordination. Only the patience and cooperation of the parties involved in this study at both the Austin and the LaGrange ends of the project minimized the difficulties experienced during all stages of the research. Any such future study should have more lead time and improved coordination.

A few problems that affected this study are of interest and should be considered in planning future research as listed below.

2.3.3.1 East end (Sta. 102+75).

(a) Bottom surface: The bottom surface was easily prepared for strain gage application. Because of the height above the ground, the use of ladders and platforms was required (as shown in Fig. 2.6) which caused substantial difficulty and slowed down the application procedure.

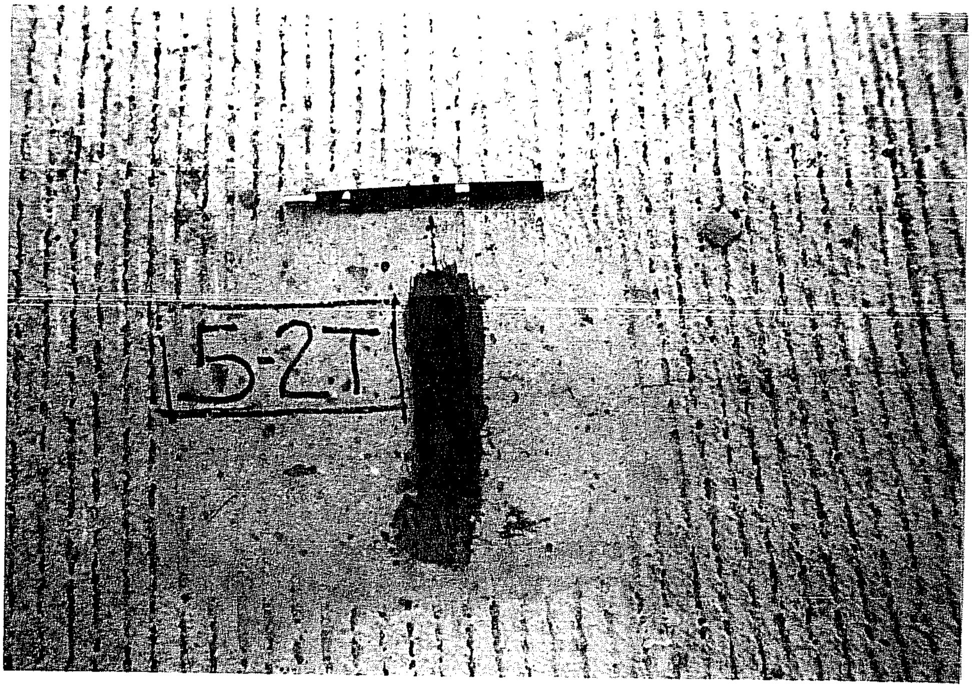


Fig. 2.5 Strain gage installed on ground surface on top of deck slab

Fig. 2.6 Platform used to reach bottom surface of slab at east end of bridge for instrumentation application

(b) Top surface: Locations for the strain gages were not discussed with the contractor ahead of casting. The top surface of the slab was specified to have a rough finishing due to pavement serviceability requirements. This finishing made the placing of the strain gages extremely difficult. Efforts were made to grind the surface to obtain a flat and smooth area of about 5 sq.-in. at six selected locations but the resultant surface was less smooth than desirable. Furthermore, top gages were damaged by rain prior to completion of their application. The investigators repaired four of them. Time constraints prevented them from replacing all of the gages. Unfortunately, the four repaired gages were damaged again by precipitation on the morning of the beginning of the prestressing process.

2.3.3.2 West end (Sta. 89+50).

(a) Bottom surface: The bottom surface was easily prepared for strain gage application. Its height above the ground level permitted easy access to the concrete surface without the need of ladders or platforms. (See Fig. 2.7.) However, there was lack of light and artificial lighting was needed.

(b) Top surface: Locations for strain gage applications were selected prior to casting the slab so a flat surface of 5 sq.-in. could be left unroughened during concrete placement. Unfortunately, misinterpretation of plans submitted by the investigators resulted in the preparation of these smooth surfaces at the wrong locations. The surface at the correct locations had to be prepared by grinding off the rough finishing of the slab. The resultant surface was smooth but not as flat as would be desired for strain gage application.

2.3.3.3 General. The major problem in gathering strain data during prestressing was the low level of the strains combined with the extensive time required for transverse post-tensioning of the spans. Assuming that f'_c = 4500 psi at time of stressing, which was reasonable for the mature concrete, E_c would be about 3.8×10^3 ksi. The effective prestress after seating and elastic losses with reasonable allowance for anchor seating and friction would be 180 to 200 ksi. For 7-in. spacing of 1/2-in. ϕ strands of Grade 270 ($A_s = 0.152$ in.^{2}) in an 8-in. thick slab, the expected average prestress would be $0.152 \times 180/(7 \times 8) = 0.489$ ksi. This corresponds to a concrete strain of $0.489/3.8 \times 10^3 = 0.000129$ in./in. or 129 microstrain. This total expected microstrain

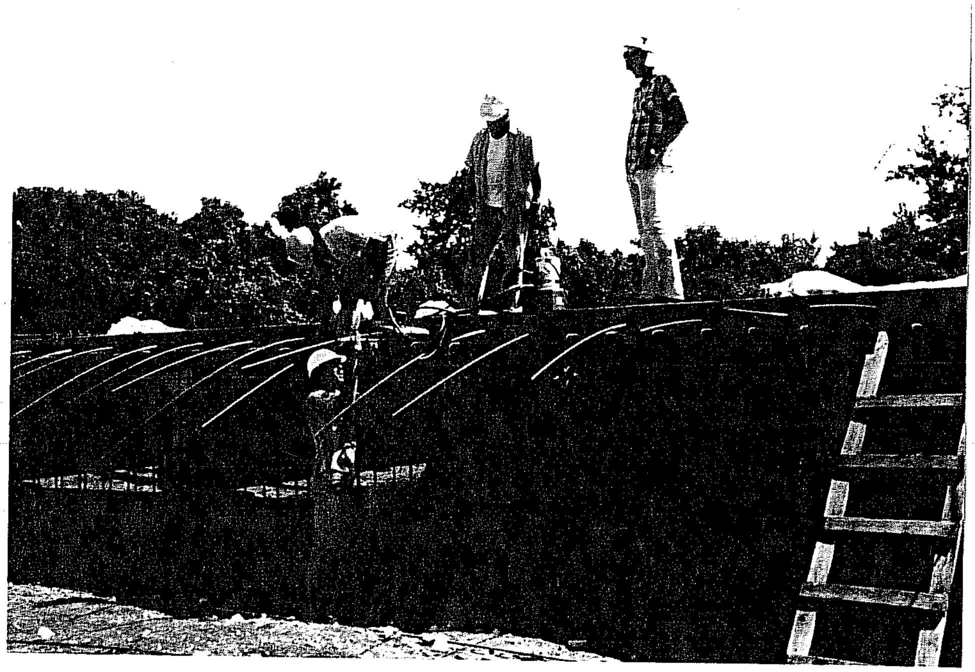


Fig. 2.7 Access to the bottom of the slab at the west end of bridge was possible without the use of platform

had to be achieved in stressing increments over four days of stressing since the contractor had to stress every fourth tendon and insisted on stressing all first tendons of all spans of that type before beginning to stress the next quartile of tendons. Thus, in one case four days of strain increments were observed and had to be added to obtain the total strain. Daily increments were as low as 15 microstrain which is near the lower sensitivity of the instrumentation.

In any future study this problem must be overcome by isolating the instrumented spans in the contract provisions so that all stressing operations for the instrumented span could be completed without delays due to stressing adjacent spans. In addition, substantial improvements in gage application and waterproofing are required to minimize gage loss and drift if strains of such low magnitude are to be measured.

2.4 Instrumentation Readings

As shown in Fig. 2.8 strain gage leads were run to switch and balance boxes with capacity for 10 channels which were then connected to strain indicators which were used to read the strain gages.

The stressing sequence called for the contractor to stress every fourth tendon. Strain gage readings were taken immediately after each two of these tendons were stressed. In addition, readings were taken before the start and upon completion of prestressing operations on any given day. Because of better weather and the shorter length of the total post-tensioned span (150 ft) it was possible to stress the west end in only two days. The combination of inclement weather causing frequent delays in stressing as well as the longer length of the total post-tensioned span (300 ft) required four days of stressing operations for the east span. Daily incremental strains were determined from the readings of each day. The daily zero was taken as the initial daily reading assuming no change during overnight periods. Any overnight change was attributed to thermal effect or instrument drift.

Thermocouples were bonded to both the top and bottom surfaces of the slab at two different locations to monitor concrete surface temperature during prestressing. (See Fig. 2.9.) Temperatures were taken on an average of every 20 minutes during prestressing of the instrumented region. In addition, readings were taken before the start and upon completion of prestressing operations on any given day.

All data were subsequently reduced using a spreadsheet program which added the total daily effective strain increments noted over the various stages read on each day into a final accumulated strain. This represented total strain referenced to the condition before any stressing operations. This procedure eliminated any effect of instrument drift overnight between stressing operations.

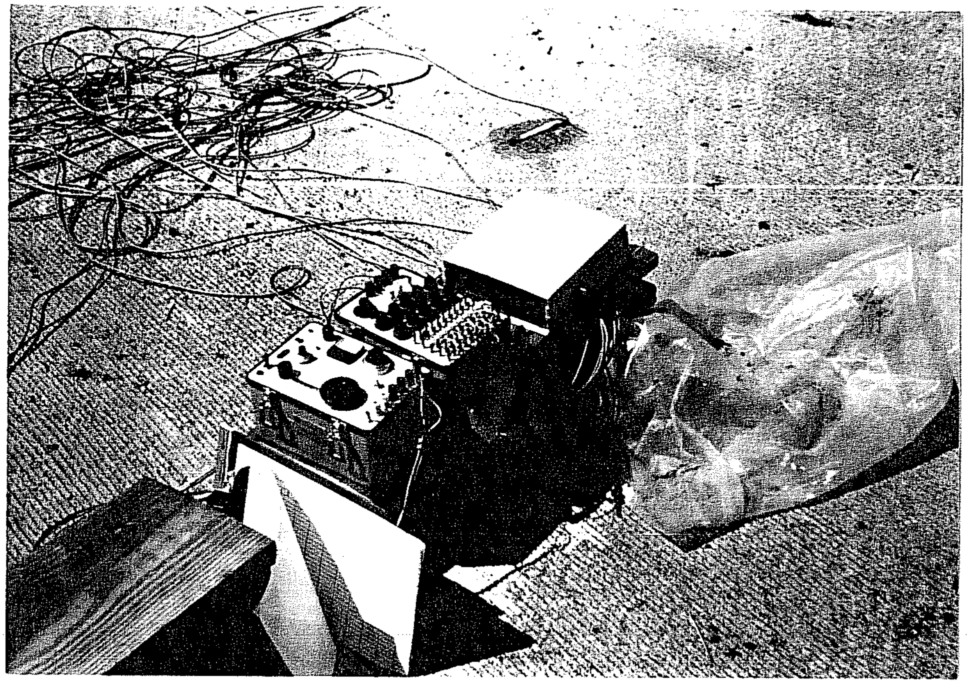


Fig. 2.8 Switch and balance box and strain indicator used to read strain gages

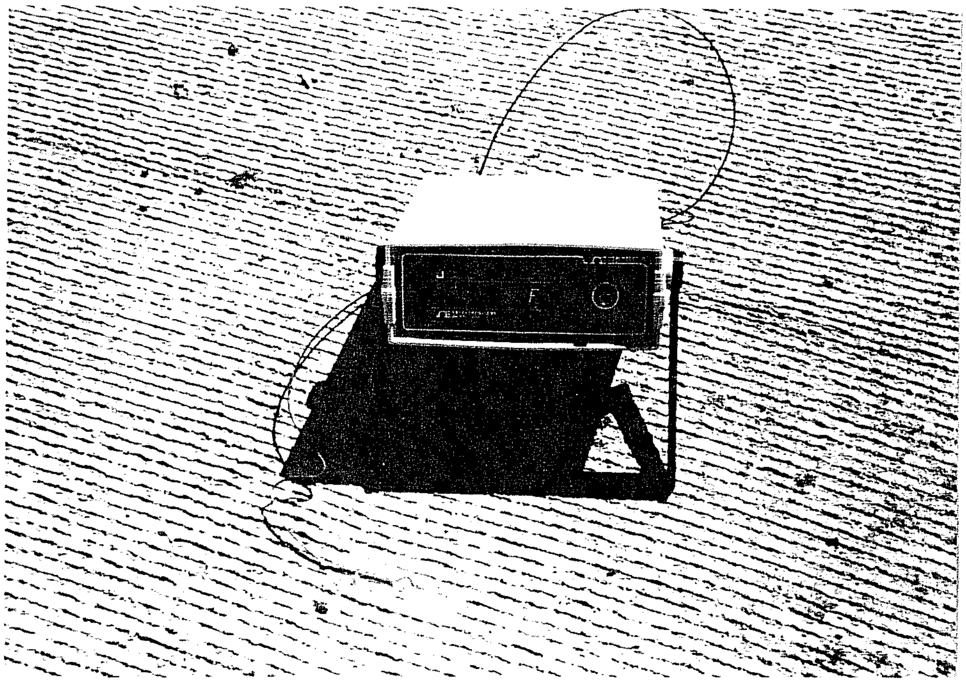


Fig. 2.9 Thermocouple and thermocouple readout

CHAPTER 3

FIELD OBSERVATIONS

3.1 General

Because of the limited scope of this relatively low-budget project, comprehensive instrumentation of the six post-tensioned spans was impossible. Since a major interest was the effectiveness of the increased transverse post-tensioning in the vicinity of the end diaphragms in counteracting the transverse restraint of those diaphragms, instrumentation on the slabs was concentrated in the half spans adjacent to the end abutments. In addition, since stressing operations required two to four days for even those small sections due to the contractors decision to stress sequentially every fourth tendon in the overall bridge being stressed, it was necessary to keep all gages continuously hooked up to the instrumentation devices. These devices had to be enclosed in a van which was parked below the span. Concerns over inaccuracy induced by long lead wires as well as budget constraints resulted in not placing gages in adjacent spans. This resulted in incomplete data, particularly in respect to longitudinal strains and verification of friction effects. Finally, the relatively small number of gage stations make it difficult to draw truly meaningful strain contours in a span. The contours presented are useful to give a very general idea of the strain distribution but should be regarded as "general indications" and not highly accurate profiles.

3.2 Longitudinal Strains

Longitudinal strains were measured only on the half span at the far west end of the bridge which was subjected to longitudinal post-tensioning as well as transverse post-tensioning. Measured longitudinal strains correspond to longitudinal post-tensioning only. Actual measurements were made before and after the longitudinal post-tensioning operations.

The location of longitudinal strain stations is shown in Fig. 2.4. Three gages were placed on the top of the slab over the fifth girder while three sets of gages were placed on both top and bottom of the slab centered between the fourth and fifth girders. The gage locations were at the one-sixth point, the quarter point, and the midpoint of the 75-ft longitudinal span. However, since the longitudinal spans were stressed with longitudinal tendons which were continuous for two spans (150 ft), these points correspond to one-twelfth, one-eighth and one-quarter of the tendon length.

The measured longitudinal strains are shown in Fig. 3.1 as a function of the distance from the stressing end. The midheight strain shown was computed as the average of the top and bottom gages at a specific location and is a reasonable indication of the average midheight strain corresponding to the average post-tensioning stress level in the slab.

In Reference 3, the proposed criterion for longitudinal deck prestressing to control shrinkage and temperature stresses and promote deck durability was that the force provided be sufficient to have a minimum average compressive stress of 100 psi in the slab after all losses. Again, assuming $f'_c = 4500$ at time of stressing, E_c would be about 3.8×10^6 psi. The recommended 100 psi stress level would correspond to a strain of $100/3.8 \times 10^6 = 26$ microstrain. Thus the midheight levels of longitudinal strains measured which varied from about 40 to 60 microstrains should be adequate even after longer term losses. The key region for prestress to prevent deck cracking is the top of the slab. It is encouraging to note that the level of short term deck top longitudinal strains achieved are about twice the recommended midslab level. This indicates that effective levels of longitudinal deck precompression are easily attainable.

The only major concern regarding longitudinal strains is the rapid drop-off of strain as a function of distance from the stressing end. Classical friction loss and wobble effect predicts that

$$F_2=F_1 e^{-\mu\alpha-KL}$$

where F_2 is the prestress force at distance L
F_1 is the prestress force at jacking end
μ is a curvature coefficient
α is the cumulative angle change of the tendon in radians
K is a wobble coefficient per ft
L is the distance from the jacking end in ft

If one assumes the baseline force F_1 as the force corresponding to the measured prestressing strain at the gages at the one-sixth point of the span, and the force at L as corresponding to the measured prestressing strain at the gages at midspan, then $F_1 = C(61)$ and $F_2 = C(41)$ where C is a constant of proportionality. Thus, $F_2/F_1 = 41/61 = 0.672$. From the previous equation $F_2/F_1 = 0.672 = e^{-\mu\alpha-KL}$. For these tendons there is no intentional angle change so $\alpha = 0$ and $0.672 = e^{-KL}$. This corresponds to $KL = 0.397$. Since $L = 37.5 - 6.125 = 31.375$, K would be 0.0127 which is extremely high when compared to the usual range for pregreased tendons of 0.0003 to 0.0020 (4). The project post-tensioners had suggested a value of 0.0013. However, the data seem quite consistent since such a high friction-wobble coefficient would predict a strain at the quarter point of $\varepsilon_2 = (61)e^{(0.0127 \times 12.62)} = 52$

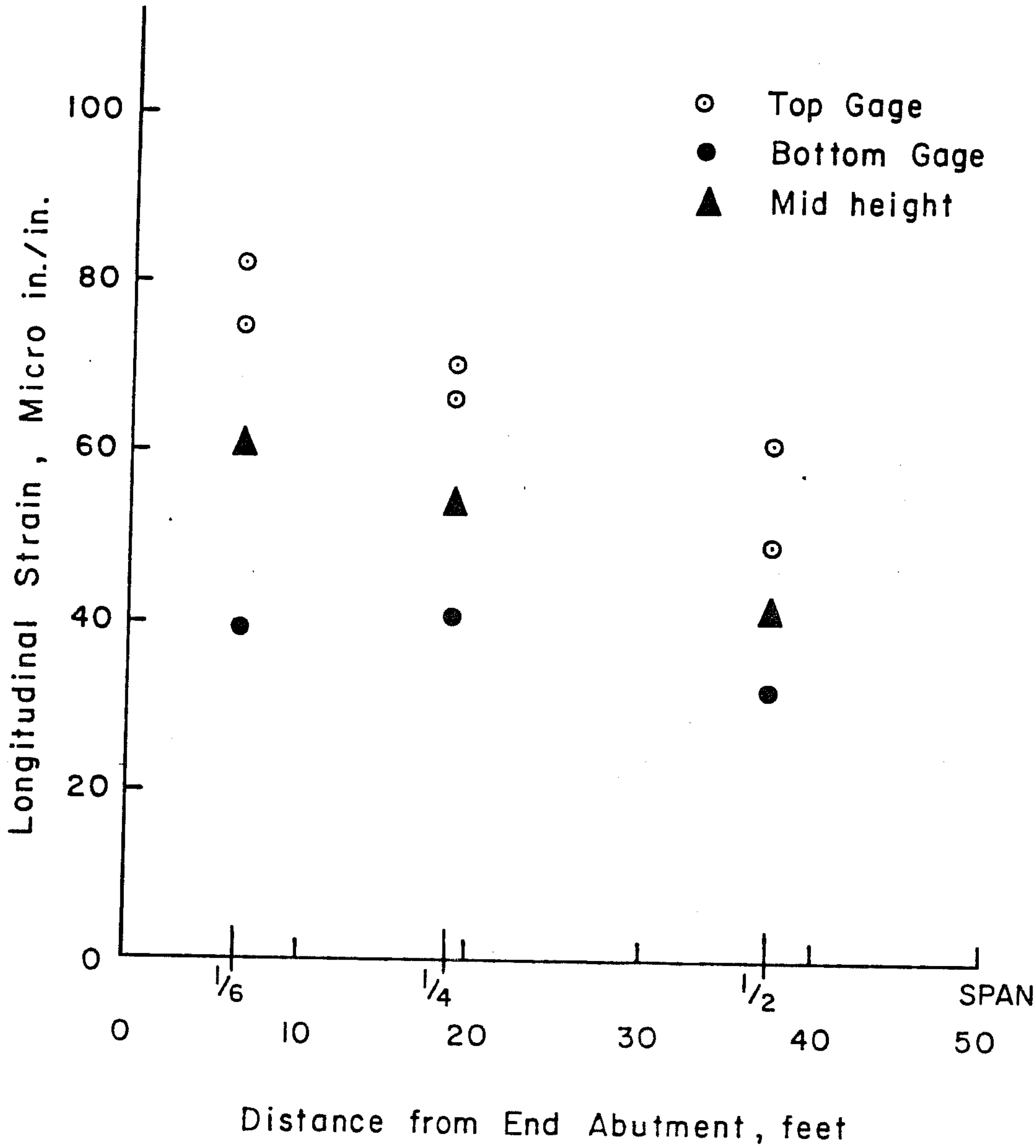


Fig. 3.1 Measured longitudinal strains on west end

microstrain which is very close to the 54 microstrain measured. No data are available to verify actual strain further along the tendons. If such losses were actually occurring, the strain at the far end of the second span would be only $\varepsilon = (61)e^{-(0.0127 \times 144)} = 10$ microstrain, which would be quite inadequate. On the other hand, if the actual wobble coefficient was the highest value given in Ref. 3 for pregreased tendons of K = 0.0020, then the far end strain would be expected to be $\varepsilon = (61)e^{-(0.002 \times 144)} = 46$ microstrain which is quite acceptable. On a future project, instrumentation should be placed to record typical strains along the full profile of the longitudinal tendons and elongation measurements should be checked to verify actual stressing losses.

3.3 Transverse Strains

Transverse strains were measured both on the half span at the far east end of the bridge which was subjected to transverse post-tensioning only and on the half span at the far west end of the bridge which had both longitudinal and transverse post-tensioning. Again, the measured strains are for the transverse post-tensioning only.

The location of transverse strain gages on the east end is shown in Fig. 2.3. Because of the poor slab surface preparation and the heavy rains at the site, all top slab gages were inoperable. Post-tensioning operations could not be delayed for gage replacement. Bottom gage readings seemed dependable but do not permit determination of average stress levels at midheight of the slab since both laboratory tests (1) and subsequent measurements on the west end show substantial local bending effects are present. Actual gage locations are shown on Fig. 3.2 as small circles. Strain contours were fitted to match the observed strain values by excercise of a good deal of judgment but are informative. The upper portion of the figure corresponds to the stressing end of the transverse tendons while the lower portion corresponds to the dead end of the tendons. The right-hand edge is the location of the heavy transverse end diaphragm which was in place at time of stressing. The left-hand edge is the vicinity of the midspan intermediate diaphragm. Tendon forces had been selected so that the average midheight strain would be about 129 microstrain as calculated in Sec. 2.3.3.3. Closer spacing of tendons had been specified in the end diaphragm region to offset the restraint of the heavy diaphragm and produce essentially uniform strain.

As can be seen from these contours, a great deal of unevenness in strain levels was noted with the regions around the end and midspan diaphragms showing less effective strain and a group of low values both along the stressing edge (due to local diffusion problems) and near the dead end (probably reflecting friction and wobble losses greater than

EAST END

(Sta. 102 + 75)

Actual Transverse Strains (Bottom of Slab)

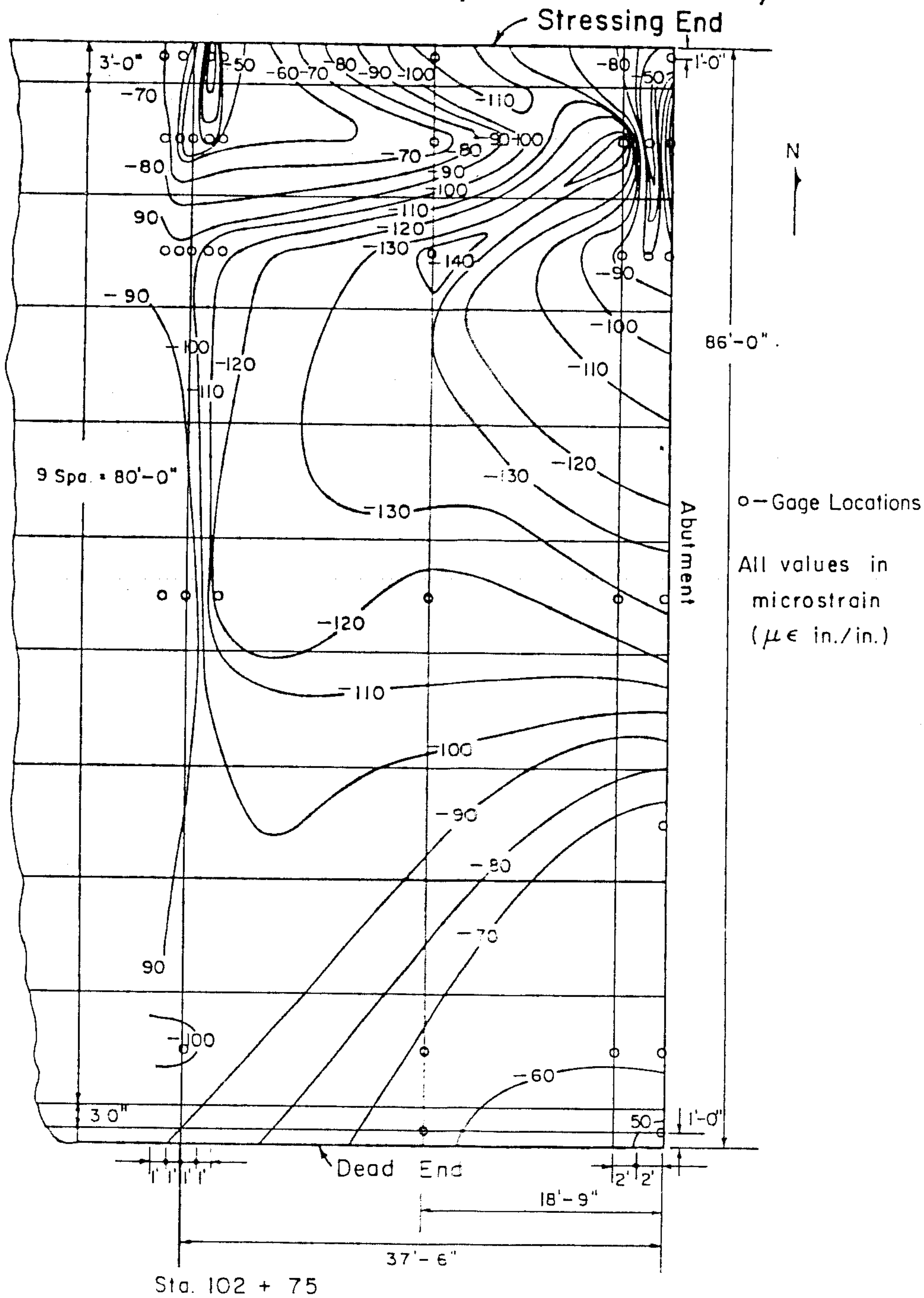


Fig. 3.2 Slab bottom transverse strain contours for east end

expected). Because of the lack of top gages, this plot is less useful than those following.

The location of transverse strain gages on the west end was shown in Fig. 2.4. Most stations had both top and bottom gages permitting averaging to cancel local bending effects. Figure 3.3 shows the slab bottom transverse strain contours for the west end. The top of the figure corresponds to the stressing end of the transverse tendons while the bottom of the figure corresponds to the dead end. The left-hand side corresponds to the abutment edge. However, the end diaphragm was not in place at time of stressing to permit access for stressing the longitudinal slab tendons. (See Fig. 2.7.) The observed strain values are substantially lower than those seen in the bottom gages of the east end in Fig. 3.2. Much of the slab seems to have a bottom strain only about one-third of the desired average value.

The same orientation is used to display the top gage strain results in Fig. 3.4 and the averaged midheight strain results in Fig. 3.5. The top strain contours indicate a substantially higher level of compression on the top of the slab even though the tendons were placed at midheight without drape. The higher value of prestress specified along the abutment edge is apparent since the restraint provided by the not yet placed end diaphragms was absent. Some effect of restraint along the interior diaphragm line is also apparent. Low values were noted along both active stressing and dead end anchorage regions indicating some diffusion problems even with these very closely spaced tendons.

The most important plot of the series is the average or midheight strain contours shown in Fig. 3.5 which should remove local bending effects. This plot indicates that most of the interior of the slab was within ±25% of the desired prestress strain levels of 129 microstrain. However, the lack of extra tendons near the midspan interior diaphragms results in less effective prestress strain in that region and the prestress diffusion problem near the stressing edge is apparent although the contour plots are probably distorted by the scarcity of gage stations along the edge. Most of the slab has a reasonable level of the desired prestressing with the highly stressed abutment edge having extra levels of stress which should help resist local impact.

Considering the difficulties in measuring low strain levels in an exterior environment with stressing taking place over six different days, the general scatter shown in Figs. 3.2 to 3.5 is not unexpected.

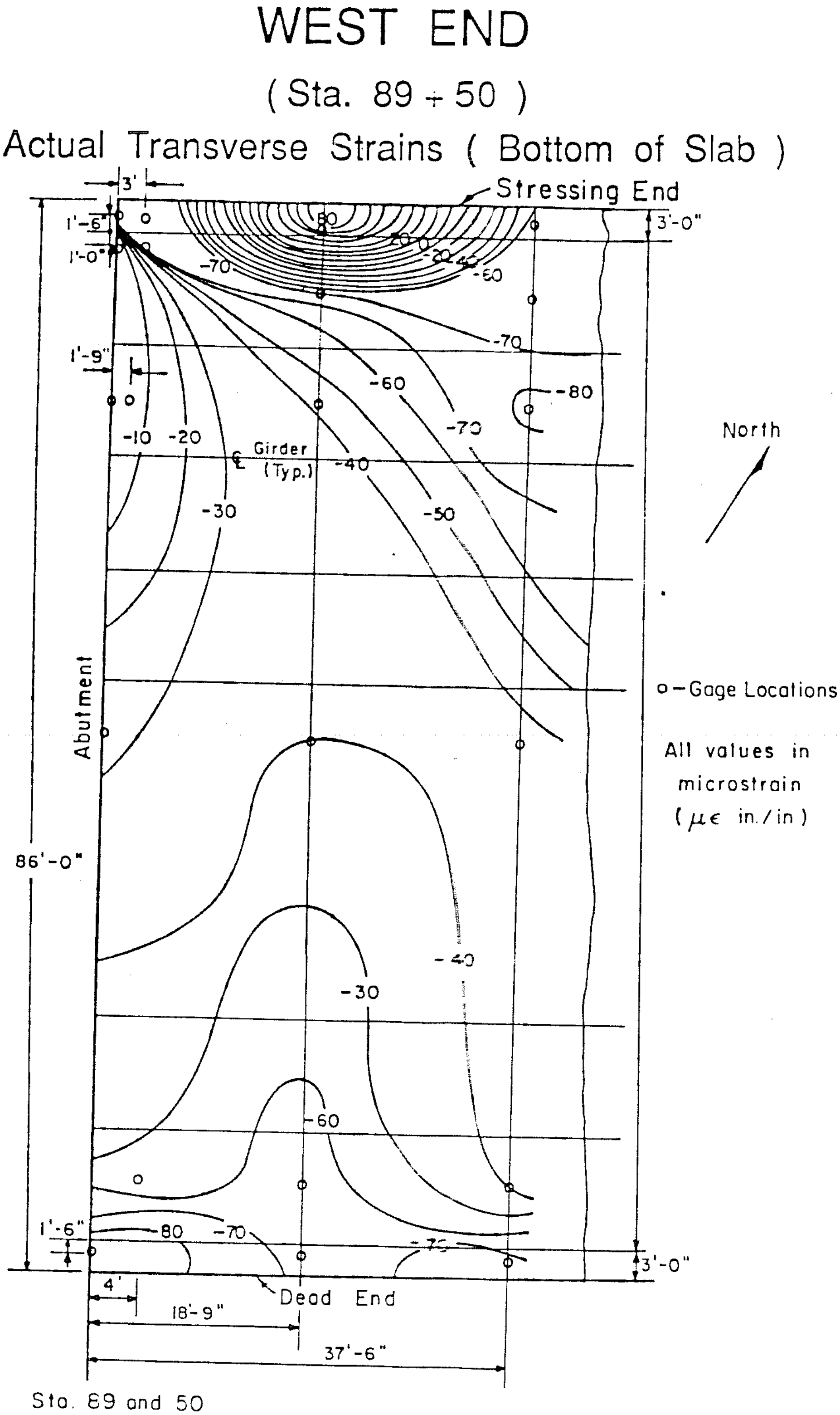


Fig. 3.3 Slab bottom transverse strain contours for west end

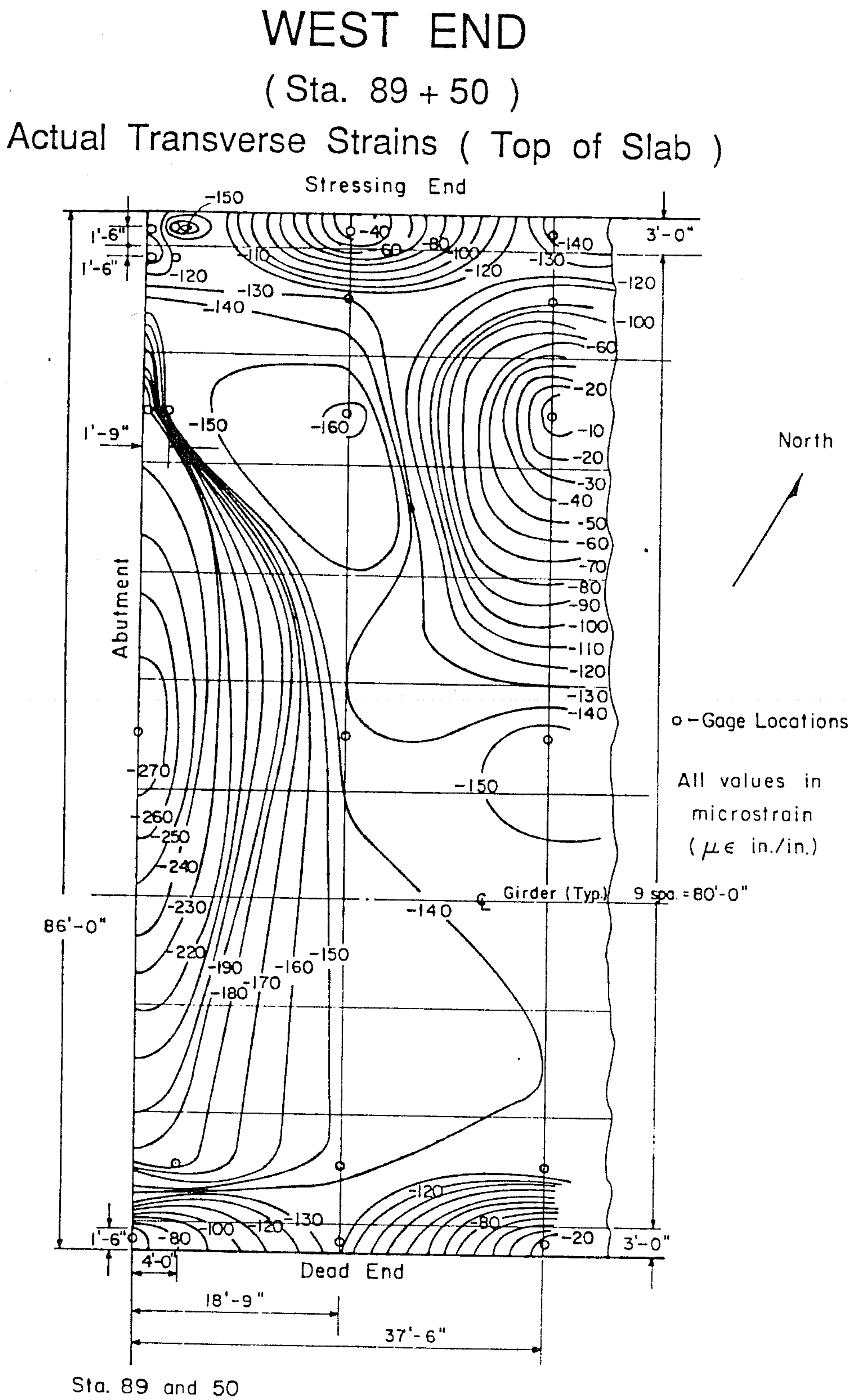


Fig. 3.4 Slab top transverse strain contours for west end

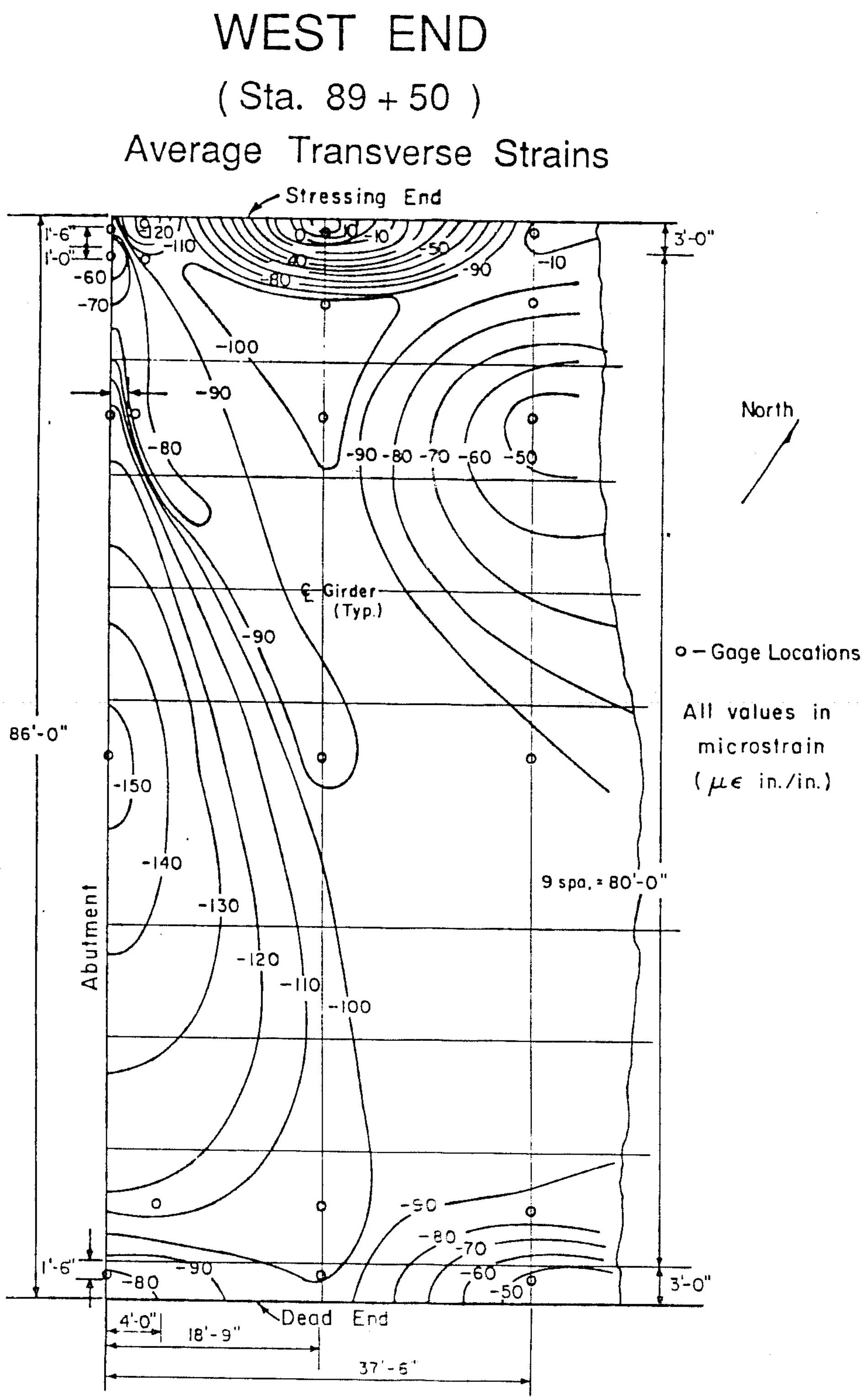


Fig. 3.5 Average or midheight slab transverse strain contours for west end

3.4 Thermocouple Readings

Typical data from thermocouples mounted on the top and bottom of the deck are shown in Fig. 3.6 and 3.7 for the east end and west end, respectively. Each data point represents the average of two thermocouples which were generally in very good agreement. Greater thermal gradients were experienced during the stressing at the west end when maximum through-deck thermal gradients of 31°F were noted. Much of the east end was stressed in wet, rainy weather and the gradients were often substantially smaller. The daily variations for the east end are different from the west end which was stressed in only two days with hot and sunny conditions.

3.5 General Field Observations

A detailed critique of all construction operations is beyond the scope of this study. However, a few items observed in the field may be useful in planning future post-tensioned decks.

In view of the fact that this was an initial experimental use of post-tensioned decks in an actual field environment, there was a decided lack of training and coordination. The intrumentation program was added late with insufficient lead time for proper staffing and was of too small a scope to utilize a full-time research staff member. Consequently, too many "borrowed" personnel were involved and decisions made proved unwise. TSDHPT field personnel were cooperative but powerless to control the contractor's schedule of items like post-tensioning operations since no specific provisions for instrumentation and possible delays were written into the contract documents.

The use of transverse monostrands was a contractor option which resulted in a great deal of congestion, many anchorage pockets which had to be filled and many moves of stressing rams. For all practical purposes the entire edge became a big anchor plate with the approximately 5-in. wide anchors on 7-in. c/c spacing. The "gut" feel of those observing the project was that use of the alternate flat multistrand tendons would be a more efficient system and the subsequent grouting of the tendons would improve general structural integrity.

There was a great deal of confusion on the site because two different anchorage devices were used and several different types of anchorage teeth were on site. Inevitably anchor bodies and teeth were mixed with some slipping of tendons. Job site control should be improved so that the post-tensioner must use matching equipment on an entire project phase to eliminate the possibility of mismatching anchorage parts.

Ram handling operations were time consuming. The contractor first tried to work from a crane-suspended basket but this proved

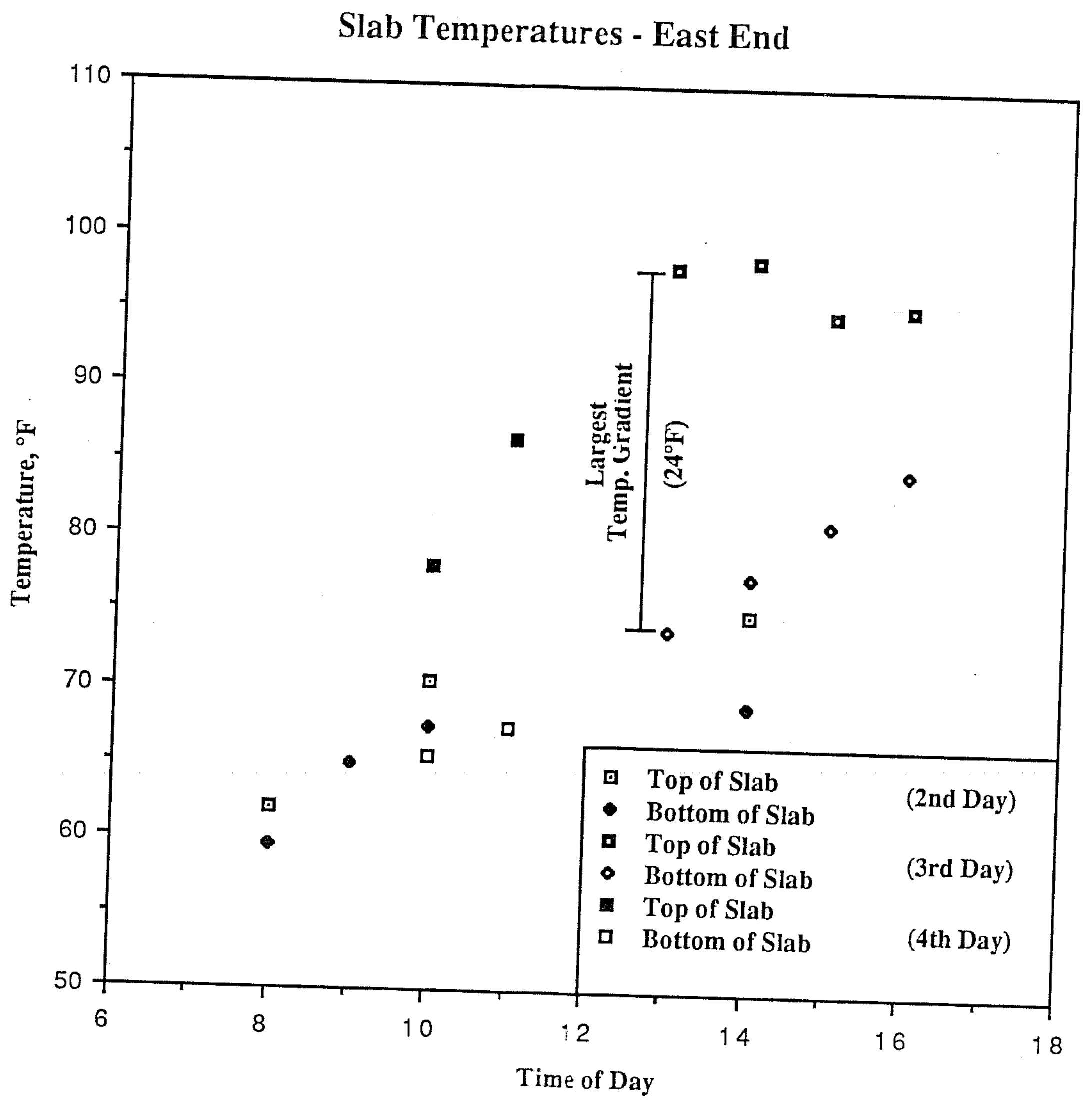


Fig. 3.6 Temperature measured on both top and bottom surface of slab at east end

Slab Temperatures - West End

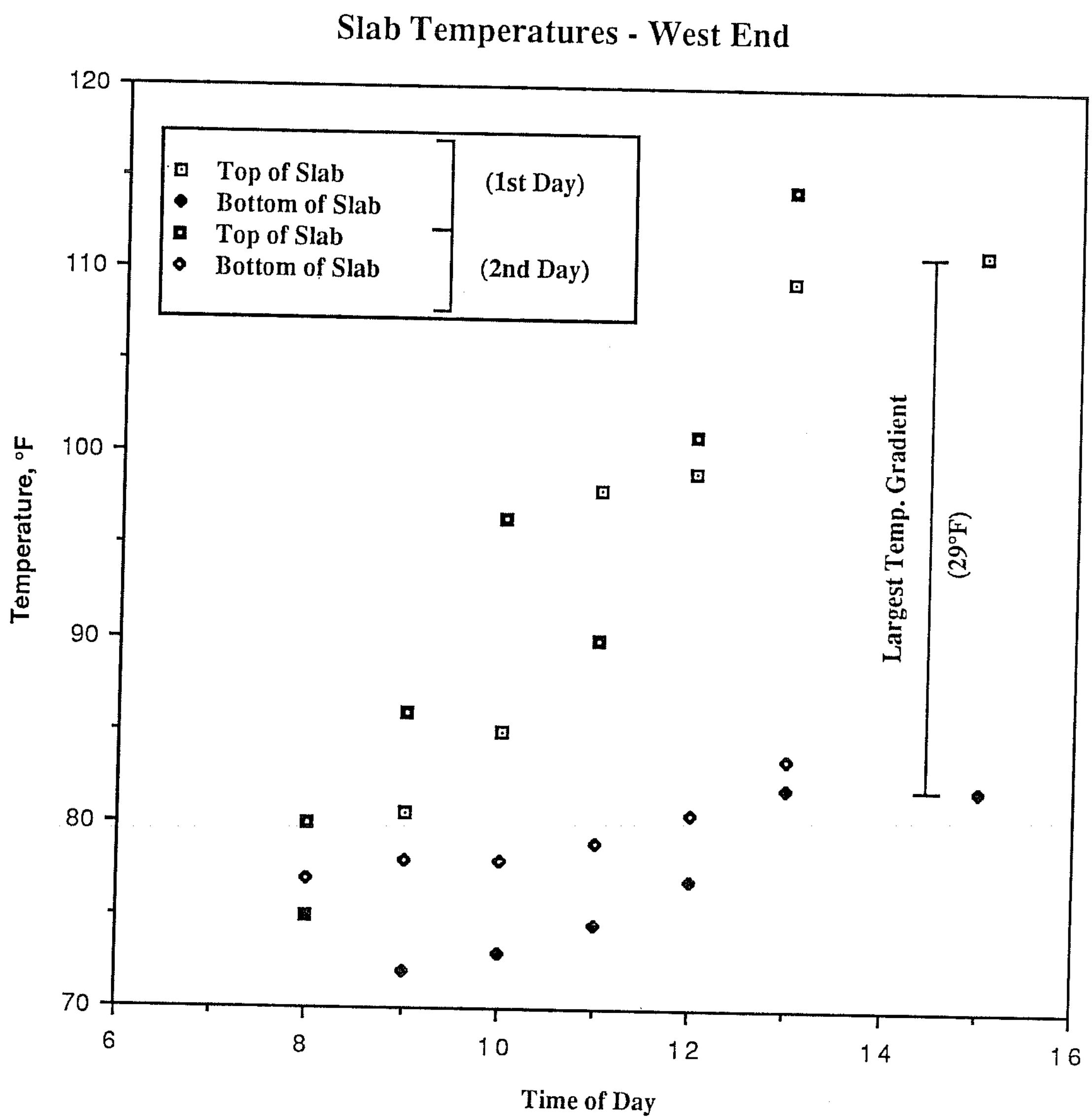


Fig. 3.7 Temperature measured on both top and bottom surfaces of slab at west end

cumbersome. He then switched to a deck-mounted device which went much faster. The use of multistrand tendons with access from the deck being stressed should greatly facilitate stressing operations.

CHAPTER 4

CONCLUSIONS

4.1 Conclusions

This project had limited scope. In addition there were substantial difficulties encountered in obtaining meaningful readings because of environmental problems, a minimal number of gage stations, and the coordination difficulties of working remotely from the Austin base. In spite of those difficulties this modest instrumentation program provided useful information regarding the deck post-tensioning of the Colorado River bridge at LaGrange, Texas.

It was concluded:

1. The longitudinal deck post-tensioning produced somewhat in excess of the needed minimum level of longitudinal compressive stress in the end span. However, strain gradients along the span due to friction and wobble losses were higher than expected.

2. The transverse deck post-tensioning produced general slab midheight transverse compressive stresses generally within ±25% of the desired levels. However, there were some regions with less effective stress than desired particularly near the interior diaphragms where no extra compensating tendons were furnished and along the slab edge where the prestress diffusion seemed limited in spite of the very closely spaced tendons.

3. The sensitivity of strain gage strain measurement required for the low value of strains expected is not consistent with exposed field operations and multiple day stressing sequences. Future studies should consider these complications in determining instrumentation needs.

4.2 Recommendations

In order to extract further benefit from this study is is recommended that:

1. Bi-annual follow-ups be made to determine relative deck condition contrasting the conditions of the unidirectional prestressed decks, the bidirectional prestressed decks and the nonstressed decks. Special attention should be paid to the condition of the sealed anchorage access holes.

2. Future field instrumentation projects should have sufficient lead time to provide for involvement of the research group in preparation of contract-document special provisions and should provide warning to the constructor that special approval of proposed post-tensioning sequences will be required. The contractor should be compensated for foreseeable delays. Special surface requirements to allow installation of instrumentation devices should be specified.

3. Instrumented spans should be so indicated in contract provisions and stressing plans developed so that all stressing operations for instrumented spans can be completed with minimum delay to minimize instrumentation drift.

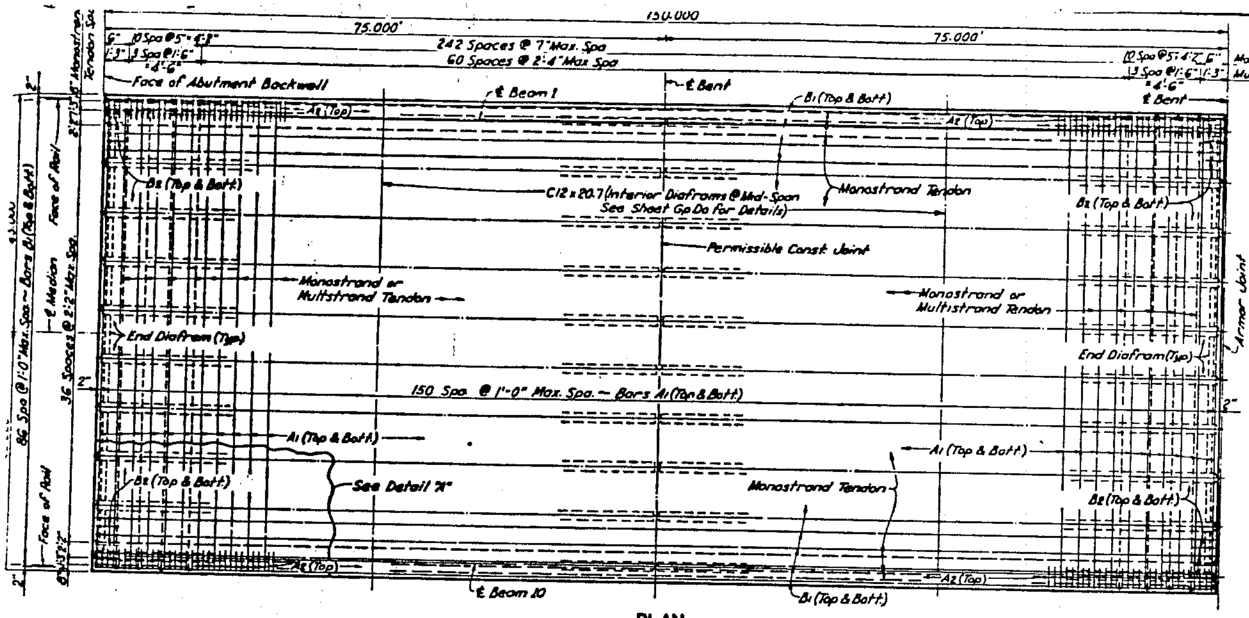
R E F E R E N C E S

1. Poston, R.W., Carrasquillo, R.L., and Breen, J.E., "Durability of Prestressed Bridge Decks," Research Report 316-1, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, July 1985.

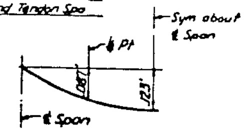
2. Phipps, A.R., Almustafa, R.A., Ralls, M.L., Poston, R.W., Breen, J.E., and Carrasquillo, R.L., "Structural Effects of Transverse Prestressing in Bridge Decks," Research Report 316-2, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, July 1985.

3. Lin, T.Y., and Burns, N.H., "Design of Prestressed Concrete Structures," Third Edition, John Wiley & Sons, New York, 1981, p. 108.

A P P E N D I X



PLAN



DEAD LOAD DEFLECTION DIAGRAM

Deflections shown are due to cast-in-place concrete only ($E = 5 \times 10^6$)

Span	Horiz. Distance c-c Bent	Beam Length ●
	All Beams	All Beams
1	75.000'	74.667'
2	75.000'	74.667'

● Beam lengths shown are bottom flange lengths with adjustments made for beam slope.

TABLE OF ESTIMATED QUANTITIES

Bar	No	Size	Length	Weight
A1	302	#5	86'-9"*	17,501
A2	600	#4	4'-6"	1,804
B1	174	#4	151'-9"◇	17,638
B2	344	#4	2'-0"	460
DK	72	#5	7'-5"	557
DS	144	#4	4'-10"	465
DN	2	#8	80'-11"	432

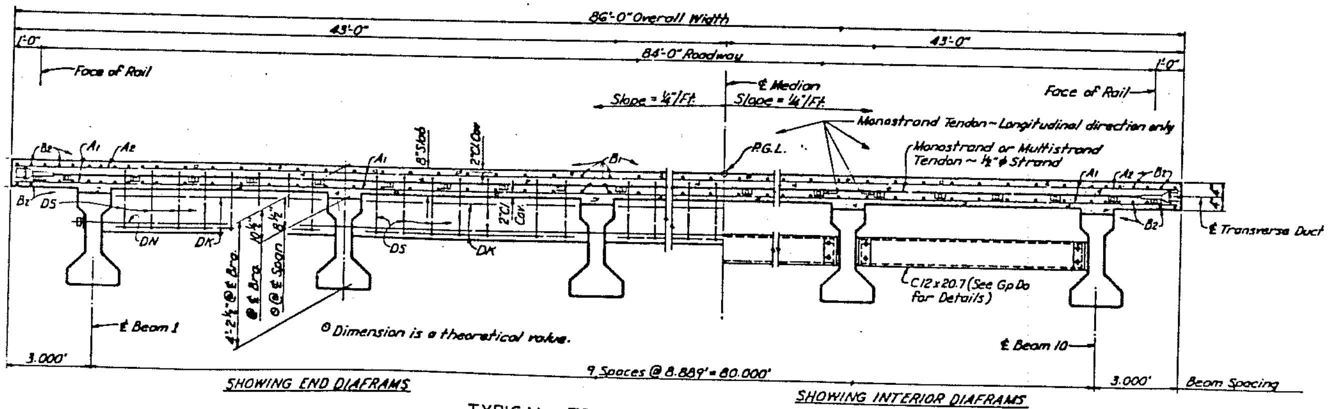
Reinforcing Steel	Lb	38,857†
Prestr. Conc. Bm ~ Type C	L.F.	1493.33
Struct. Steel ~ Armor Joint	Lb	1160 ‡
Struct. Steel ~ (HYC)	Lb	3685†

SPAN	Prestr. Conc. Slab - S.F.	Class S Conc. ~ C.Y.† Diafram	Class S Conc. ~ C.Y.† Slab
1	6450	2.8	161.5
2	6450	2.8	161.5
Total	12,900	328.6	

† For Contractor's information only.

* Includes one ~ 1'-0" Min. Lap.
◇ Includes two ~ 1'-0" Min. Laps.
‡ Wt. shown is for ½ of one complete Armor Joint.

Fig. A.1 Typical details on reinforcement and tendon-distribution and girder and diaphragm-location for west end unit of bridge



TYPICAL TRANSVERSE SECTION

HS 20 LOADING   Sheet 1 of 2

STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

150.00' PRESTR. CONC. BEAM UNIT NO. 1

(POST TENSION SLAB)

COLORADO RIVER BRIDGE

Dec. 1984

FAYETTE

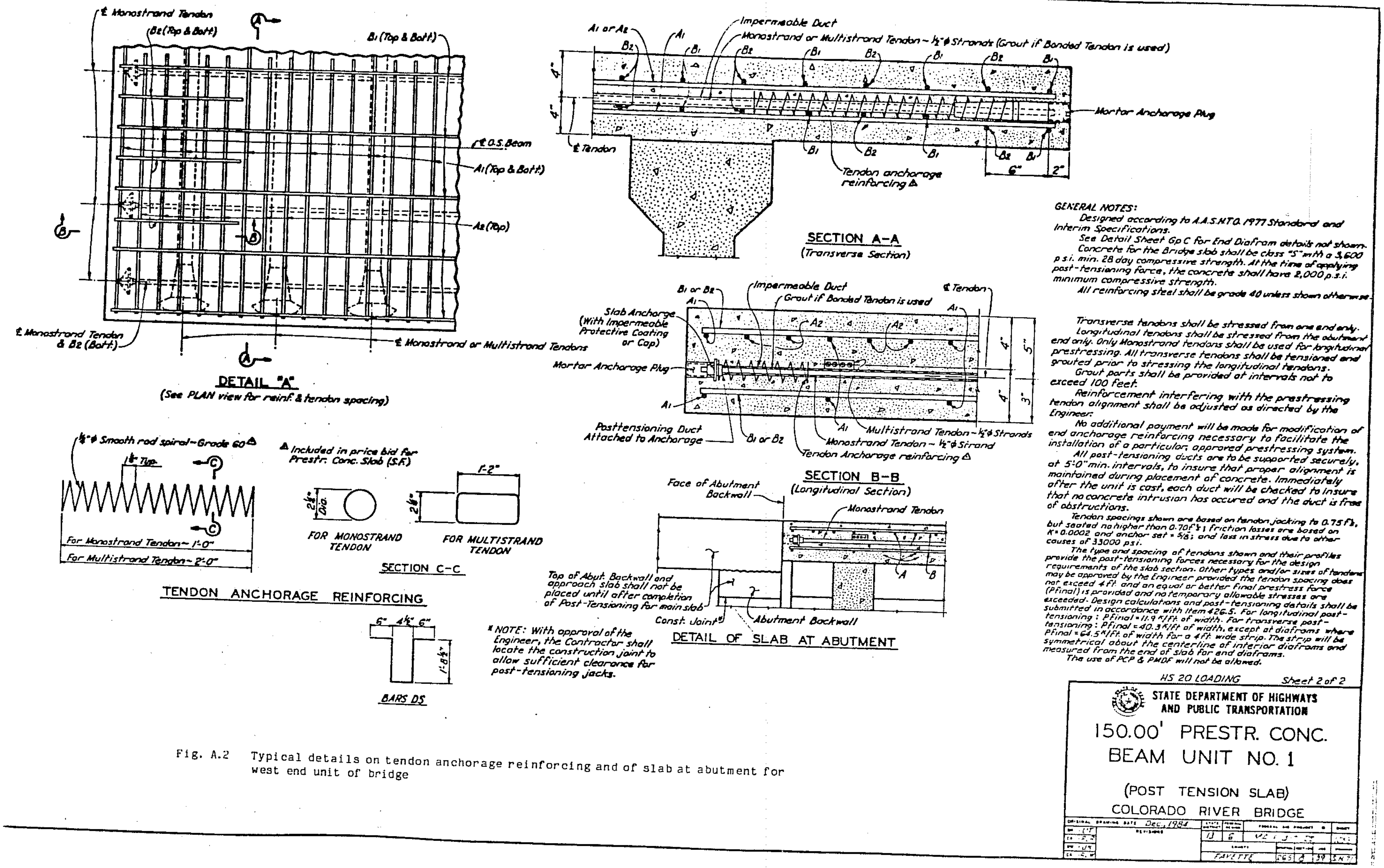
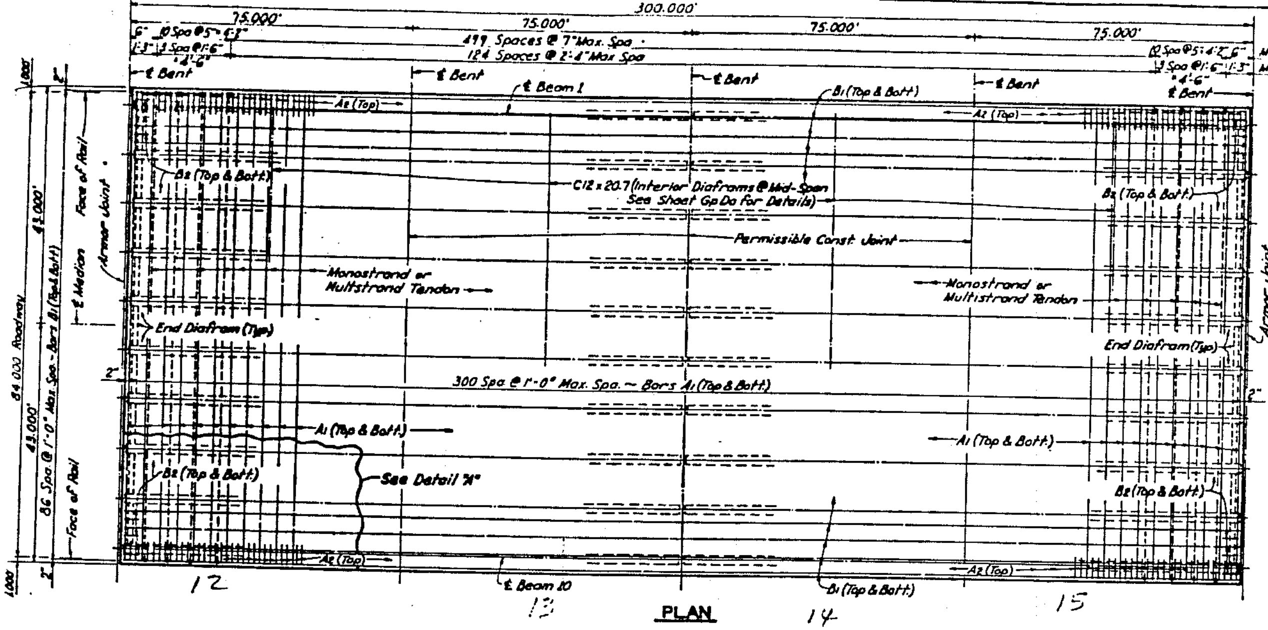
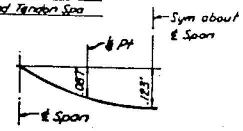


Fig. A.2 Typical details on tendon anchorage reinforcing and of slab at abutment for west end unit of bridge





DEAD LOAD DEFLECTION DIAGRAM

Deflections shown are due to cast-in-place concrete only ($E = 5 \times 10^6$)

Span	Horiz. Distance c-c Bent	Beam Length ●
	All Beams	All Beams
12	75.000'	74.668'
13	75.000'	74.669'
14	75.000'	74.669'
15	75.000'	74.669'

● Beam lengths shown are bottom flange lengths with adjustments made for beam slope.

TABLE OF ESTIMATED QUANTITIES

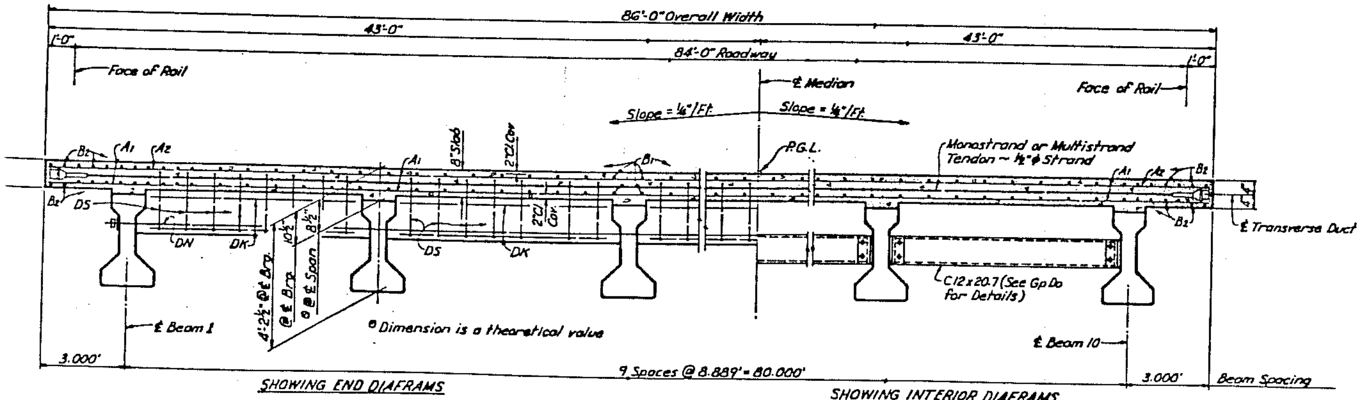
Bar	No	Size	Length	Weight
A1	602	#4	86'-9" *	34,883
A2	1200	#4	4'-6"	3,607
B1	174	#4	304'-9" ◇	35,422
B2	344	#4	2'-0"	460
DK	72	#5	7'-5"	557
DS	183	#4	4'-10"	465
DN	2	#8	80'-11"	432

Item	Unit	Quantity
Reinforcing Steel	Lb	75,828 †
Prestr. Conc Bm ~ Type C	LF	2986.75
Struct. Steel ~ Armor Joint	Lb	2320 #
Struct. Steel ~ (NYC)	Lb	7,370 †

SPAN	Prestr. Conc. Slab ~ S.F.	Class S Conc. ~ CY †	
		Diafram	Slab
12	6450	2.8	161.5
13	6450	~	161.5
14	6450	~	161.5
15	6450	2.8	161.5
Total	25,800	651.6	

† For Contractor's information only.
# Wt. shown for one complete Armor Joint.
* Includes one ~ 1'-0" Min. Lap.
◇ Includes five ~ 1'-0" Min. Laps.

Fig. A.3. Typical details on reinforcement and tendon distribution and girder and diaphragm location for east end unit of bridge.



TYPICAL TRANSVERSE SECTION

HS 20 LOADING Sheet 1 of 2

STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

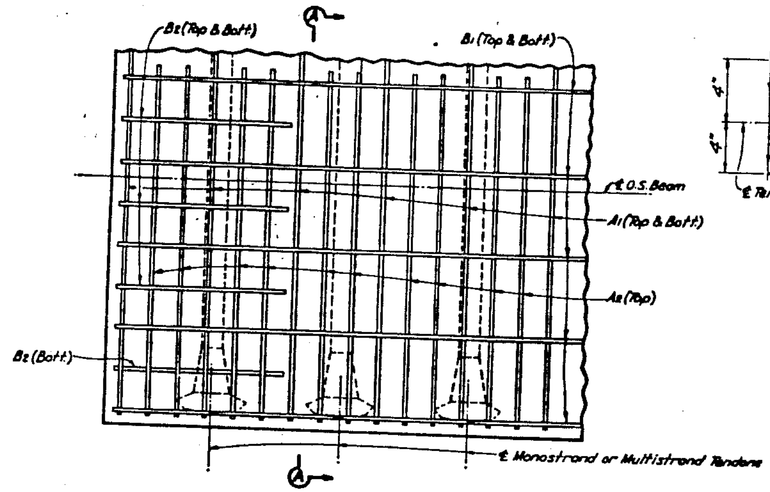
300.00' PRESTR. CONC. BEAM UNIT NO. 6

(POST TENSION SLAB)

COLORADO RIVER BRIDGE

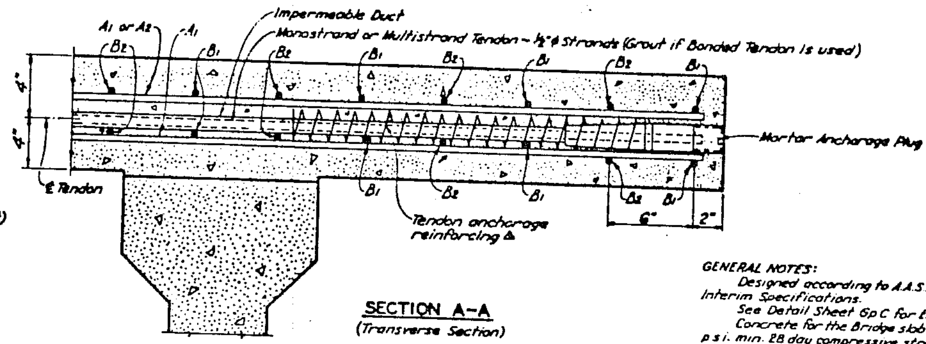
Original drawing date Dec. 1984

FAYETTE



DETAIL "A"

(See PLAN view for reinf. & tendon spacing)



SECTION A-A

(Transverse Section)

GENERAL NOTES:

Designed according to A.A.S.H.T.O. 1977 Standard and Interim Specifications.

See Detail Sheet 6pC for End Diafram details not shown.

Concrete for the Bridge slab shall be class "S" with a 3,600 p.s.i. min. 28 day compressive strength. At the time of applying post-tensioning force, the concrete shall have 2,000 p.s.i. minimum compressive strength.

All reinforcing steel shall be grade 40 unless shown otherwise.

Transverse tendons shall be stressed from one end only.

Reinforcement interfering with the prestressing tendon alignment shall be adjusted as directed by the Engineer.

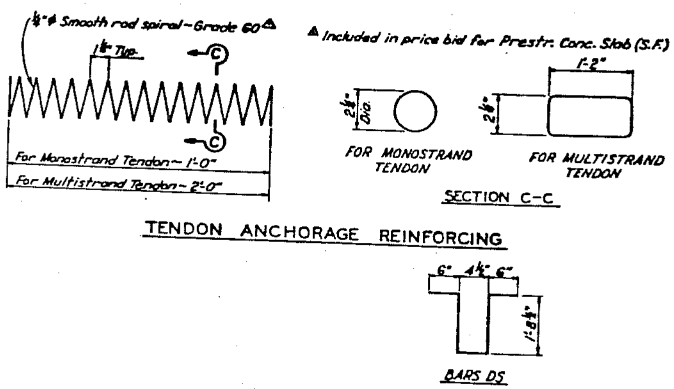
No additional payment will be made for modification of end anchorage reinforcing necessary to facilitate the installation of a particular, approved prestressing system.

All post-tensioning ducts are to be supported securely, at 5'-0" min. intervals, to insure that proper alignment is maintained during placement of concrete. Immediately after the unit is cast, each duct will be checked to insure that no concrete intrusion has occured and the duct is free of obstructions.

Tendon spacings shown are based on tendon jacking to 0.75f's, but seated no higher than 0.70f's; friction losses are based on K = 0.0002 and anchor set = 5/8"; and loss in stress due to other causes of 33000 psi.

The type and spacing of tendons shown and their profiles provide the post-tensioning forces necessary for the design requirements of the slab section. Other types and/or sizes of tendons may be approved by the Engineer provided the tendon spacing does not exceed 4 ft. and an equal or better final prestress force (Pfinal) is provided and no temporary allowable stresses are exceeded. Design calculations and post-tensioning details shall be submitted in accordance with Item 426.5. For longitudinal post-tensioning: Pfinal = 11.9 K/ft. of width. For transverse post-tensioning: Pfinal = 40.3 K/ft. of width, except at diaframs where Pfinal = 64.5 K/ft. of width for a 4 ft. wide strip. The strip will be symmetrical about the centerline of interior diaframs and measured from the end of slab for end diaframs.

The use of PCP and PMDF will not be allowed.



TENDON ANCHORAGE REINFORCING

HS 20 LOADING — Sheet 2 of 2

STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

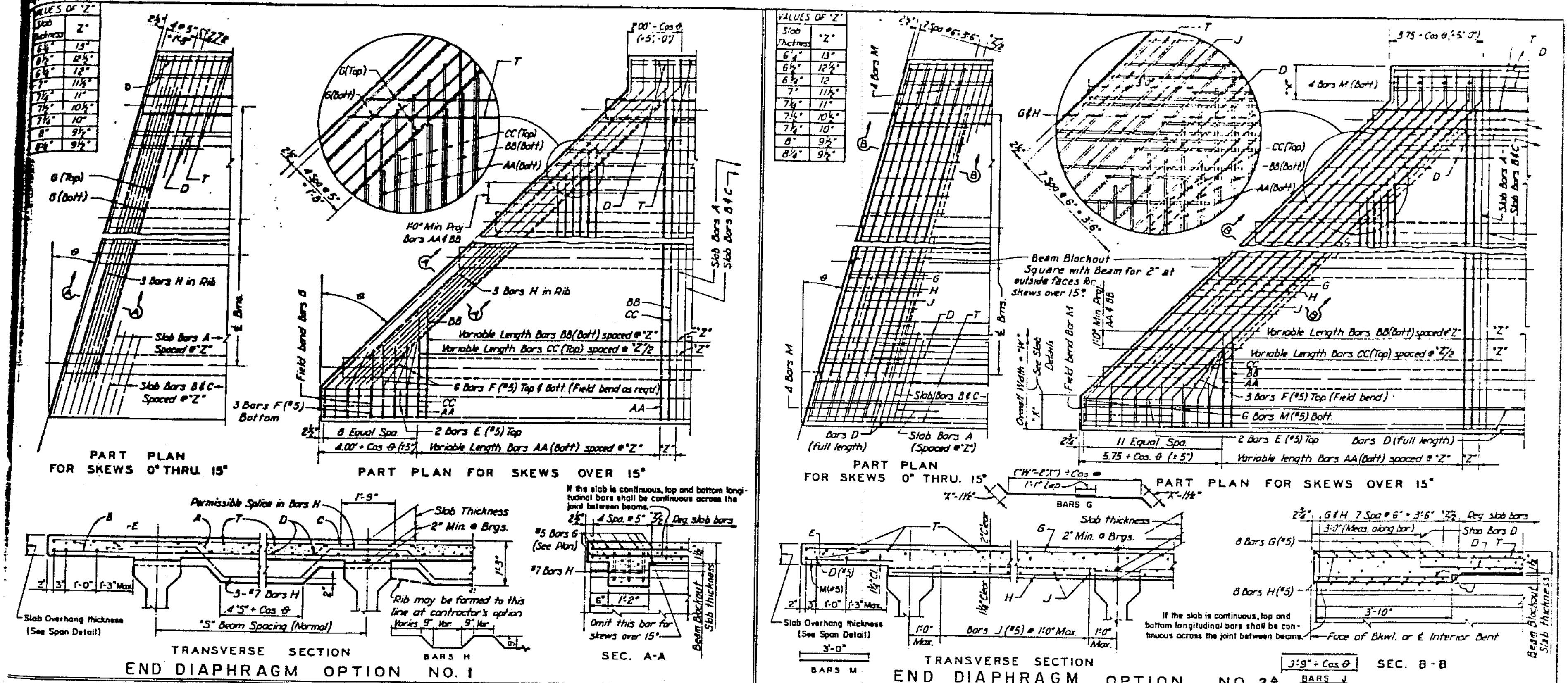
300.00' PRESTR. CONC. BEAM UNIT NO. 6

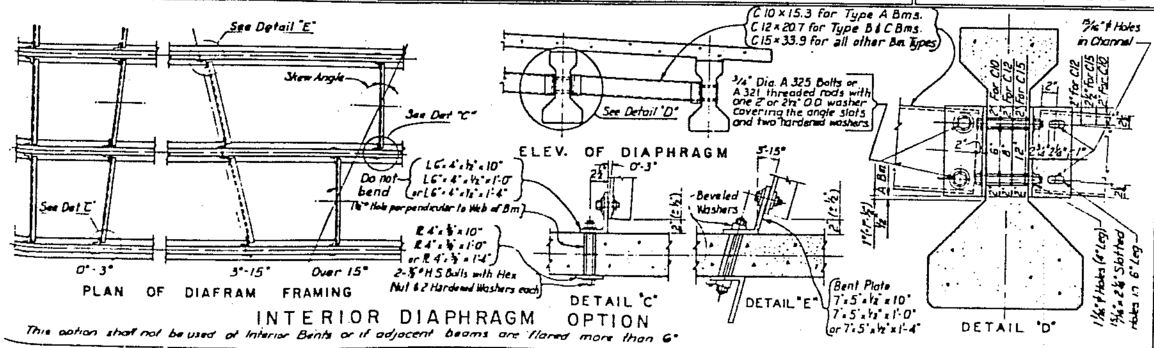
(POST TENSION SLAB)

COLORADO RIVER BRIDGE

Dec. 1984

Fig. A.4 Typical details on tendon anchorage reinforcing and of slab at abutment for east end span of bridge





GENERAL NOTES:

Diaphragm options hereon may be used in lieu of the diaphragms show on Gp C.

Unless otherwise shown or noted hereon, all span details remain unchanged.

Contractor shall notify prestressed beam and steel fabricators as to which one he intends to use. Options selected shall be incorporated in shop drawings.

All parts of steel diaphragm shall be galvanized after fabrication. After erection, all scratched or otherwise damaged galvanized parts shall be repaired in accordance with Item 450 5.

Prestressed Beam Modifications: For Interior Diaphragm Option, holes in beam shall be 1½"⌀. For End Diaphragm Options, holes in ends of beam shall be eliminated. For End Diaphrag the upper 1½" of the top flange of the beam shall be blocked out to provide the required concrete depth over the beam.

Payment for any diaphragm option used shall be included in the price bid for Reinforced Concrete Slab.

Fig. A.5 Diaphragm options for prestressed beam spans

STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

DIAPHRAGM OPTIONS FOR PRESTRESSED BEAM SPANS

Gp DO

DEC 1980