TECHNICAL REPORT STANDARD PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/LA-91/245		

4. Title and Subtitle	5. Report Date
Design Considerations for Bridge Deck Joint Sealing Systems	July 1992
	6. Performing Organization Code
7. Author(s) R. Richard Avent, Vijaya, K. A. Gopu, Herodotos A. Pentas, Keith J. Rebello	8. Performing Organization Report No. 245
9. Performing Organization Name and Address Department of Civil Engineering Louisiana State University Baton Rouge, LA 70803	10. Work Unit No. 11. Contract or Grant No. LA HPR Study No. 87-1-ST
12. Sponsoring Agency Name and Address Louisiana Transportation Research Center P.O.Box 94245 Baton Rouge, LA 70804	13. Type of Report and Period Covered Final Summary Report Oct 1986 - Jul 1992 14. Sponsoring Agency Code

15. Supplementary Notes

Conducted in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This is a report summary which summarizes a three year research effort related to the study of bridge deck expansion joint movements. Bridge deck expansion joint systems often develop serious problems requiring extensive and expensive maintenance. This has become a nuisance to users and to bridge engineers, and many states have been involved in investigations aiming to alleviate this problem. Results reported by various states regarding the behavior of specific joint sealing systems have been contradictory, indicating that the problems may not be inherent with the particular system. Rather, the problems may stem from a failure to properly assess the actual joint movements, inadequate design criteria, improper installation procedures or other factors such as differences in environmental conditions. In recognition of these problems, a comprehensive experimental and analytical investigation of bridge deck expansion joint movements was undertaken to develop rational design methodologies for joints in modern bridges.

The longitudinal across-the-expansion-joint movements of a newly constructed bridge in central Louisiana were experimentally evaluated. Since thermally induced movements comprise the bulk of the longitudinal deformations, the temperature characteristics of the bridge sections were also investigated. The movement of the supporting bents and their effects on joint movements were also studied. The bridge was instrumented to assess both short-term and long-term longitudinal movements. The recorded data were analyzed and utilized to determine whether the joints have been adequately designed to accommodate the movements. The effect of support restraints was also investigated.

(continued)

17. Key Words	18. Distribution Statement
Concrete Bridges, Expansion Joints, Creep, Shrinkage, Temperature Effects, Movements, Analysis	Unrestricted. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	52	

Form DOT F 1700.7 (8-69)

A rigorous and efficient analytical model to predict the long-term deformation behavior of bridges with multiple, precast, pretensioned, prestressed concrete girders supporting cast-in-place concrete deck slabs, was developed. The analytical procedure uses the finite element method with three-dimensional 20-node isoparametric elements to realistically model bridge geometry. Time dependent effects due to load and temperature history, creep, shrinkage and aging of concrete are included in the analysis. Creep and shrinkage strains are evaluated at different times using the well-established ACI-209, Bazant-Panula II and CEB-FIP procedures. Temperature strains are calculated from an assumed typical bridge temperature distribution based on the average ambient temperature occurring during any time period. The effect of temperature on creep is also accounted for. Prestressing tendons are modeled as being embedded in concrete and as contributing to girder stiffness. Position and slope continuity in tendon profiles are maintained. Losses in prestress due to steel relaxation and geometry changes are calculated in the analysis. The analytical model is capable of simulating typical construction schedules to predict deformations at any stage during the service life of a bridge.

A parametric study was conducted to quantify the influence of key geometric and material properties of the bridge on the long-term expansion joint movements. Bridge systems representing a wide range of the key parameters were analyzed to develop formulas to estimate creep and shrinkage movements with a certain degree of confidence. These formulas formed the basis of a rational procedure for calculating the long-term bridge deck joint movements. The recommended procedure accounts for the effects of bridge geometry and material properties on joint movements. These effects are ignored in current highway bridge deck joint design methodology. The use of the recommended procedure permits the designer to determine span lengths and the maximum number of continuous spans between expansion joints in bridge decks, if the limit of movement that can be accommodated by the chosen joint-sealing system is known.

The analytical model has been coded into a FORTRAN program which can be used to evaluate the long-term behavior of bridges with or without expansion joints, and with different support conditions.

DESIGN CONSIDERATIONS FOR BRIDGE DECK JOINT-SEALING SYSTEMS

SUMMARY REPORT

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LSU PROJECT NO. 127-15-4146

CONDUCTED FOR

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
LOUISIANA TRANSPORTATION RESEARCH CENTER
in Cooperation with
U. S. Department of Transportation
FEDERAL HIGHWAY DEPARTMENT

July, 1992

ABSTRACT

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A rigorous and efficient analytical model to predict the long-term deformation behavior of bridges with multiple, precast, pretensioned, prestressed concrete girders supporting cast-in-place concrete deck slabs, was developed. The analytical procedure uses the finite element method with three-dimensional 20-node isoparametric elements to realistically model bridge geometry. Time dependent effects due to load and temperature history, creep, shrinkage and aging of concrete are included in the analysis. Creep and shrinkage strains are evaluated at different times using the well-established ACI-209, Bazant-Panula II and CEB-FIP procedures. Temperature strains are calculated from an assumed typical bridge temperature distribution based on the average ambient temperature occuring during any time period. The effect of temperature on creep is also accounted for. Prestressing tendons are modelled as being embedded in concrete and as contributing to girder stiffness. Position and slope continuity in tendon profiles are maintained. Losses in prestress due to steel relaxation and geometry changes are calculated in the analysis. The analytical model is capable of simulating typical construction schedules to predict deformations at any stage during the service life of a bridge.

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The analytical model has been coded into a FORTRAN program which can be used to evaluate the long-term behavior of bridges with or without expansion joints, and with different support conditions.

IMPLEMENTATION STATEMENT

The results of this report provide a rational method for evaluating bridge deck joint movements. The methodology described is applicable to bridge systems consisting of multiple precast, pretensioned, prestressed concrete girders composite with cast-in-place concrete deck slabs and is a significant improvement over the current empirical procedures in use. The application of the method should greatly mitigate the problems associated with the design of joint sealing systems.

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BACKGROUND

Highway bridges generally require either expansion or contraction joints between sections of the deck or between the deck and the approach roadway. In modern bridges, it is customary to specify a sealed joint to prevent debris and water from passing through the joint and causing deterioration of the bridge. Frequently, the joint-sealing systems have not functioned as intended. The seals have either ruptured, pulled out or fallen out of their proper positions, squeezed out of position, leaked at their splice points, or the anchor bolts securing the seals have loosened or failed. Once the seals fail, incompressible solids can lodge within the joint. The foreign matter will then resist the closing of the joint causing high stresses within the slab. Cracks will form and the concrete slab will eventually spall. In short, joint seals have proved to be a continual and expensive maintenance problem for highway departments and a nuisance to the highway user. Since various states have reported contradictory behavior in specific joint-sealing systems, the problems may not be inherent with the systems. Rather, the problems may stem from improper design criteria for the bridge joint, improper installation practices, differences in the bridge types, differences in environmental conditions, or other factors.

Trends in modern highway bridge construction, such as the use of precast, prestressed concrete girders and creation of multiple continuous spans for live loads, complicate the prediction of joint movements. The current practice for the design of expansion joint-seals for Louisiana highway bridges is based on elementary formulas, and these may not accurately predict actual joint movements in modern bridges. In reality, the joint movement is a complex response. The strains which influence the joint movement are caused by a variety of factors, including thermal expansion, time dependent creep and shrinkage, and applied live loads. Systematic, detailed studies of joint movements will lead to the development of rational design methodologies for joints in modern bridges including criteria for reducing or totally eliminating joints if possible.

Highway bridges in Louisiana have been plagued by joint-sealing problems. These problems can be attributed to the performance and design of the joint sealant systems. An inspection of several recently-constructed bridges in Louisiana (the approaches to the Luling Bridge at Destrehan, Louisiana, and the I-110 interchange at Baton Rouge, Louisiana) disclosed numerous problems in all of the joint-sealing systems used for those bridges. In recognition of these problems, a conceptual research plan was developed in three phases. The goal of the first phase was to instrument a bridge and obtain experimental data on expansion joint movements, both short-term and long-term. The goal of the second phase was to develop analytical models and correlate with the experimental data. Afterward, the analytical model would be used to develop design recommendations relating to bridge deck joints. The third phase was to design a jointless bridge, instrument it during construction and evaluate its behavior.

OBJECTIVES

The specific objectives of the first two phases of the overall project were as follows:

Phase 1

1. Instrument the designated bridge for field monitoring using thermocouples, linear voltage displacement transducers (ILVDTs) and optical measurements.

2. Accumulate and synthesize design data related to performance limits, failure criteria, and mechanical properties of joint sealing systems commonly used on Louisiana highways.

3. Monitor the bridge movements over the duration of the project and evaluate the experimental data as to its relationship to expansion joints.

Phase 2

1. Develop analytical models for predicting longitudinal bridge movements.

2. Correlate experimental data with analytical models and refine the models.

3. Use models to develop design recommendations for maximum span lengths of bridges built without joints and to assess the relative affects of different spans, support stiffness, creep, shrinkage, temperature and skew.

4. Develop criteria for determining how joints can be eliminated in bridge design.

5. Develop recommendations for joint sealing systems in future bridge construction based on research data obtained in this investigation.

6. Develop an engineering methodology for assessing joint damage, appropriate repair alternatives, and specific remedial procedures for various joint types currently failing prematurely.

The various tasks related to these two phases of work are discussed in the following sections. The third phase of the conceptual research plan was not a part of this project effort.

BRIDGE MEASUREMENTS

Bridge Description

The bridge to be investigated is the east approach of the U.S. 190 highway over the Atchafalaya River at Krotz Springs, Louisiana. It consists of cast-in-place concrete slabs acting compositely with either concrete or steel girders. The concrete girders are AASHTO type IV pretensioned girders. The steel girders are built-up plate girders.

This superstructure is supported by 12 bents as shown in Figure 1. The abutment is labeled Bent 1 and the rest of the bents are numbered in ascending order from east to west. Five expansion joints are provided to allow for expansion/contraction of the bridge. These joints are numbered 1 through 5 in consecutive order from east to west as shown in Figure 1. Joints 1 through 4 are membrane (strip) seal joints while joint 5 is a toothed type joint. The bridge continues over the river as a steel through truss, however only the east approach was instrumented. The connections between the girders and the pile caps are labeled either "E" or "F" in Figure 1. Connections labeled "E" permit the girder to slide on the pile cap by the provision of slotted holes in the steel angle brackets used to attach the girder and the pile cap. At connections labeled "F" the girders are not free to slide on the pile cap. In this case, the girders are attached to the bent cap by dowels or are attached to the bent cap with steel angle brackets which do not permit movement.

The order of the bridge construction and some of the important construction dates will be now discussed. By the start of this research project, the supporting bents had been erected. The girders (both steel and concrete) were fabricated in the manufacturing plant and transported to the bridge site. They were placed on the bents in a simply supported manner. Once in place the girders were connected to each other and to the bents by the use of diaphragms and connections. It was during that period of construction when the first instruments were installed. Finally each deck was constructed as a separate continuous pour. The first slab to be poured was unit 1 followed by units 2, 3, and 4. These slab sections were poured at two week intervals. During the pouring of unit 4, a mechanical failure occurred and the construction could not be completed in one pour. A construction joint (cold joint) was therefore formed near bent 11. This slab was completed 3 weeks later. All the slab pours were completed in March 1988.

Besides the interruption which occurred in the pouring of unit 4 slab, no other major construction problems were encountered. However, a problem developed with the steel truss supporting pins of bent 12 which delayed construction considerably. The pins were found to be defective, allowing the formation of high stress concentrations at the connection because of thermal expansion of the truss. This stress build-up would, on occasion, release causing vibrations and loud noises to occur. This problem was later solved by replacing and modifying the pin assemblies. Seven months after the slab pours were completed, construction was completed and the bridge was opened to traffic in October 1988.

<u>Description Of Bridge Measurements</u>

After a careful consideration of the objectives of this research and review of related publications, it was decided that the following measurements would give a thorough evaluation of joint movements.

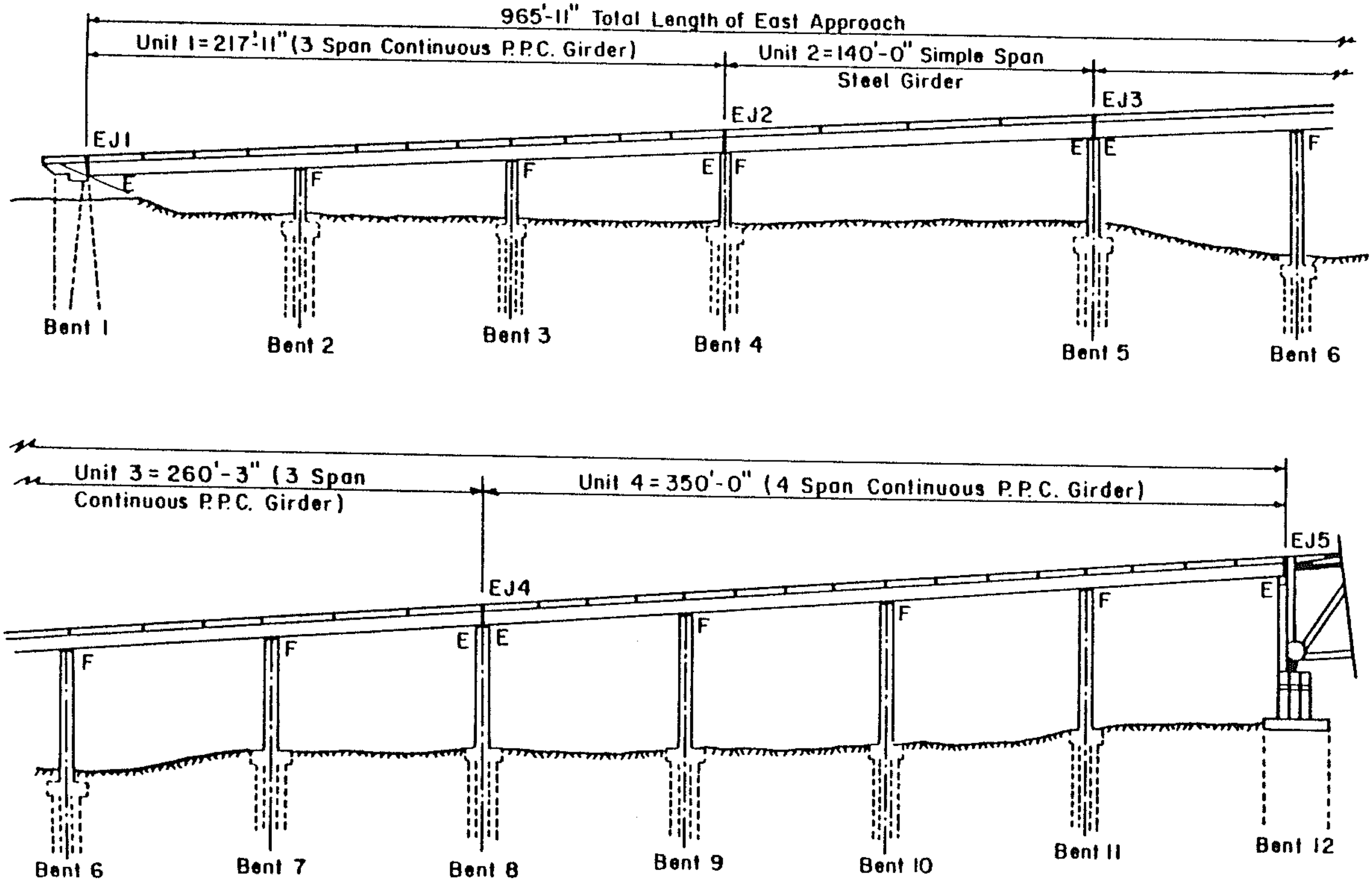

FIGURE 1 North Elevation of the East Approach of the U.S. 190 Bridge at Krotz Springs, La.

1. Measurements near the top and bottom of the girders at the expansion joints - These measurements give the relative longitudinal movements between the two adjoining composite sections. Because of restraints in the placement of the instrumentation, the relative movements at the top of the sections could not be measured directly. However they could be calculated from the two actual relative measurements taken near the top and bottom of the girders. The assumption that plane sections remain plane was made and it was later proved to be valid. It was therefore adequate to install only two instruments at each girder end and linearly extrapolate the readings to any desired point. Also of interest are the relative movements of the composite sections at their neutral axes. These movements are not measured directly but are also derived using the actual measurements taken near the top and bottom of the girders.

2. Measurements between the supporting bent cap and one of the composite sections at the expansion joints - These measurements are taken near the top and bottom of the girder. The rotation and movement of the composite section with respect to the cap can be calculated from these measurements. In addition, the rotation and movement of the other composite section at the joint can be obtained by using these relative movements in combination with the movements discussed previously in item 1.

3. Measurements of the sway of the bents supporting the girders at the expansion joints - The movements of the bent caps in the horizontal direction parallel to the roadway are directly obtained from these measurements. The horizontal movements in the direction parallel to the roadway of any point along the length of the columns are also directly obtained from these measurements.

4. Measurements of temperatures through the depth of the composite section - The temperatures are recorded at the same time as any other measurements are taken. These temperatures include top, middle, and bottom of slab; top, middle, and bottom of girder; and ambient temperature.

5. Time Measurements - Time is also recorded while taking any other measurements, thereby giving a time reference.

Instrumentation

After a careful study of many measuring devices, three types of instruments were selected to be used in this research. Linear variable differential transformers (LVDT's) were chosen to acquire the measurements of relative joint movements. A theodolite was chosen to acquire the bent sway. Finally, type T copper-constantin thermocouples were used to obtain all temperature measurements.

The LVDT's and thermocouples were wired to the monitoring station where they would be connected to a Hewlett Packard microcomputer and data acquisition system which would store the readings as well as the time for later processing. Electrical power for the system was supplied through a portable generator. The theodolite readings were taken and recorded in a field book by hand and later transcribed into the computer for processing.

LVDTs

The LVDTs were chosen to obtain measurements at the locations shown in Figure 2. The label at each location indicates the expansion joint number and the side on which it lies (North or South). Due to construction delays and the presence of equipment and construction forms, the instrumentation of expansion joint 5 was not possible. The instruments were placed at the inner sides of the exterior girders in order to protect them from the outer environment. They were mounted on specially made aluminum brackets and attached to the girders using epoxy. The use of brackets would allow for replacement of a defective LVDT and ease of removal in case of vandalism or project completion. The placement of the LVDTs and the subsequent formulations needed to obtain the movements at specific locations are addressed in the following paragraphs.

The arrangement of LVDTs at an abutment and a typical expansion joint are shown in Figures 3 and 4, and the measurements required to calculate movements at these locations are shown in Figures 5 and 6.

Total Station Theodolite

The total station theodolite can measure the horizontal angle, the vertical angle and the distance between itself and a point, as long as the point reflects light. These measuring capabilities can be combined to find the movement of points strategically marked on the bridge. First the horizontal and vertical movement of each point can be determined, then this information is combined to obtain the movement of structural components of the bridge.

For the bridge under study, the theodolite was used to obtain the sway of the supporting bents. It was also used to take measurements, which allow for calculation of relative joint movements during initial construction phases, since LVDTs could not be placed during that time. The theodolite readings began in January of 1987, therefore the references were set at that time.

One setup point, SP, was constructed for each supporting bent, therefore a total of 12 setup points were constructed. A central reference point (RP) was constructed on the levee to allow for visibility from all setup points. A schematic diagram showing the bridge and the arrangement of the setup points is shown in Figure 7. Each setup point is labeled SP and the bent number with which it is associated. The setup points and the reference point are made up of cast in place concrete columns reinforced with three #4 reinforcing bars. Each column is five feet in length with four feet in the ground and one

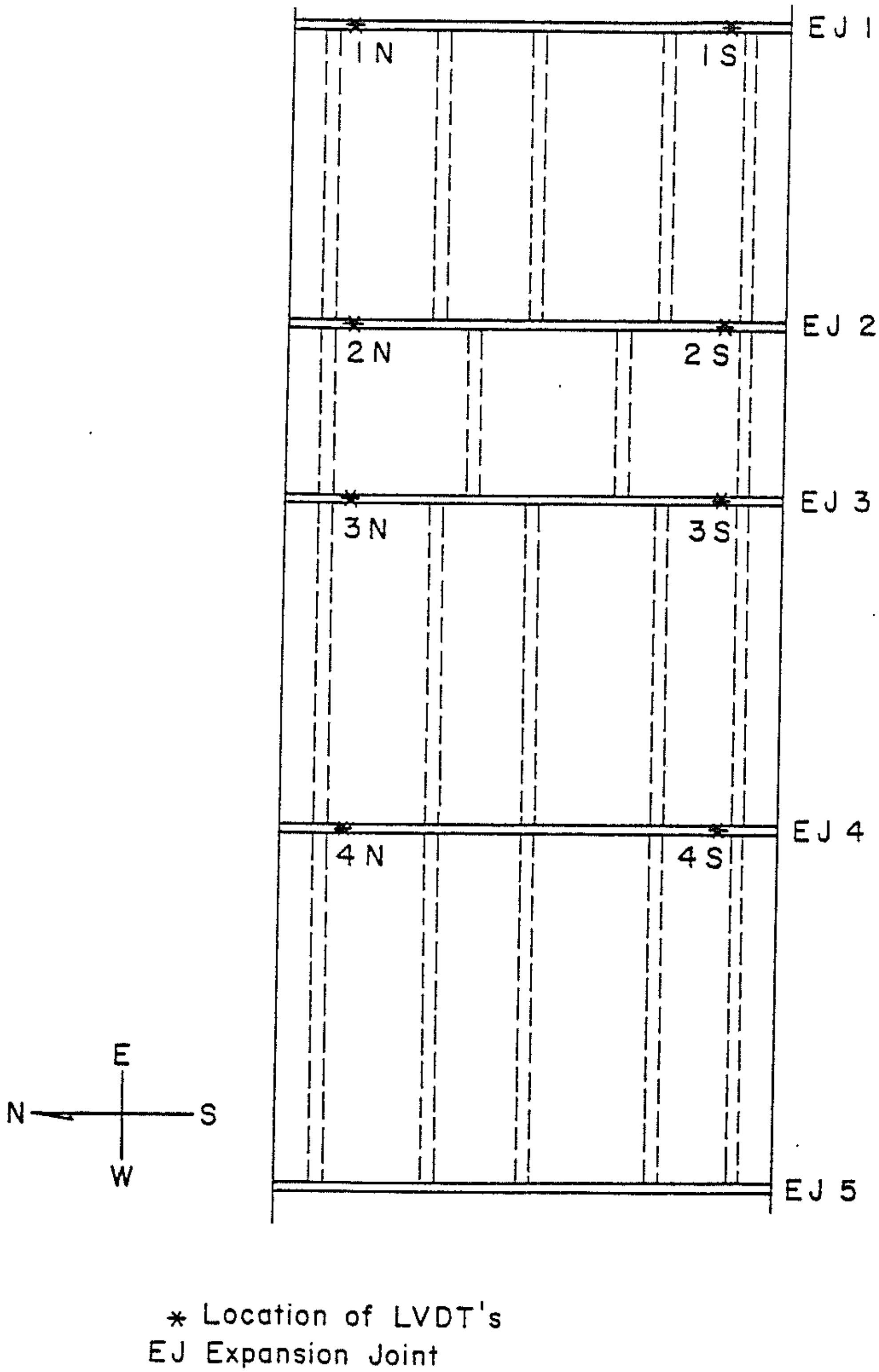

FIGURE 2 Plan View of Bridge Showing Locations of LVDT's.

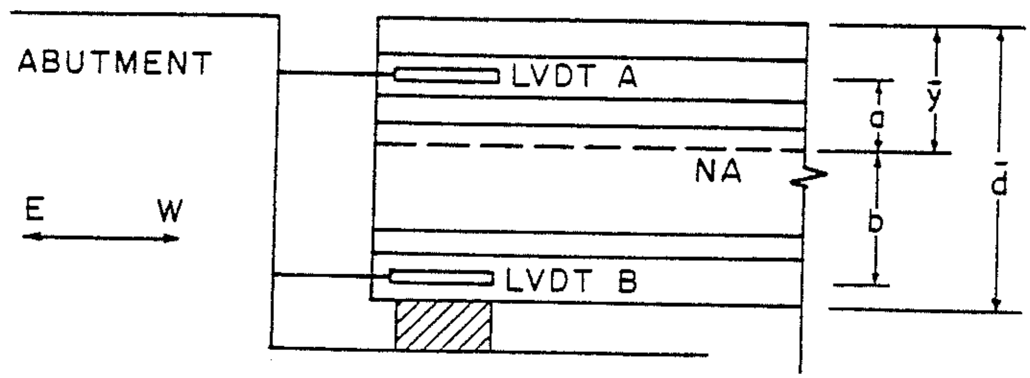

FIGURE 3 North Elevation of the Abutment Showing Measured Positions of LVDT's.

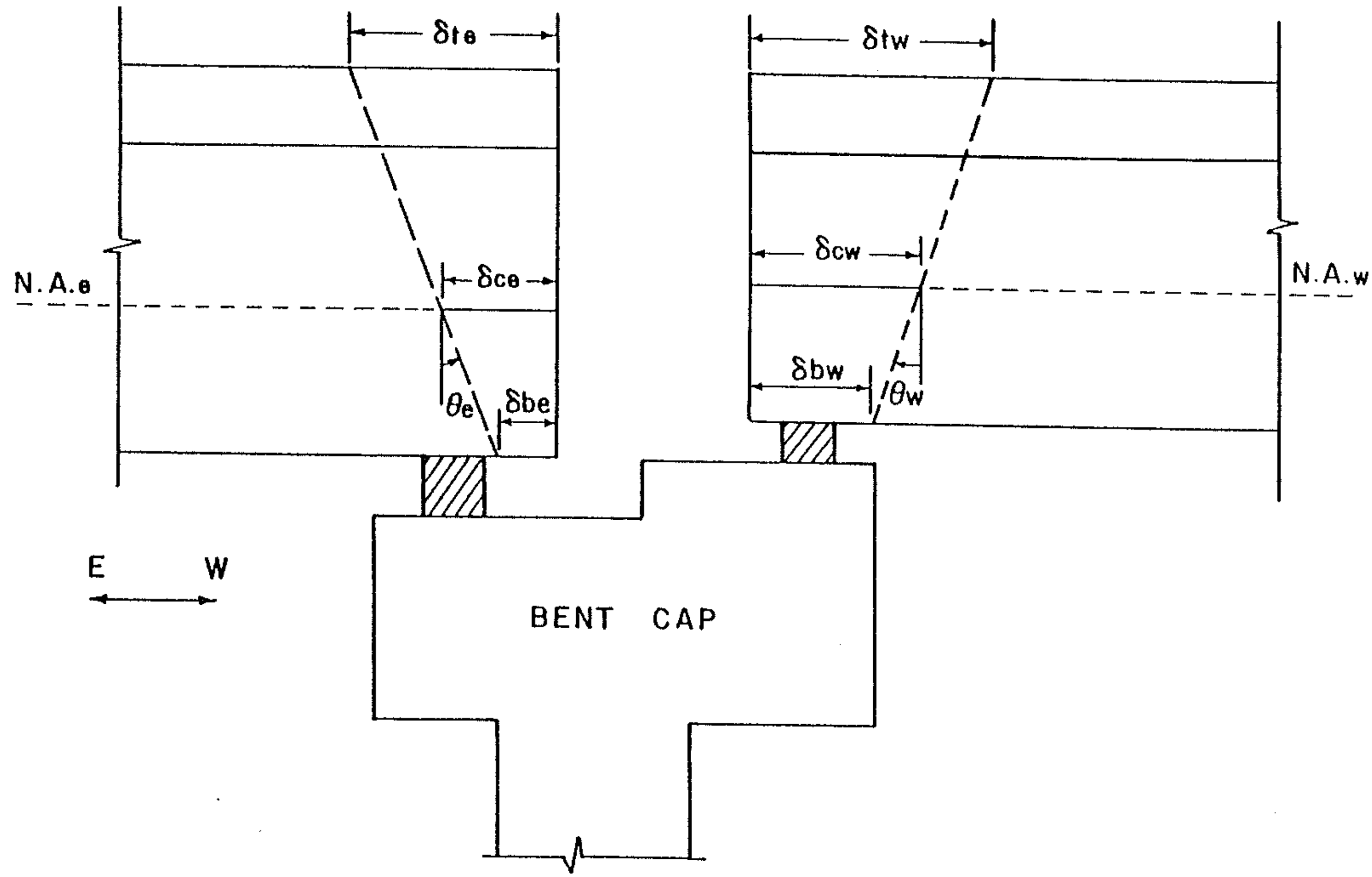

FIGURE 4 North Elevation of a Typical Expansion Joint, other than the Abutment, Before and After Longitudinal Movement.

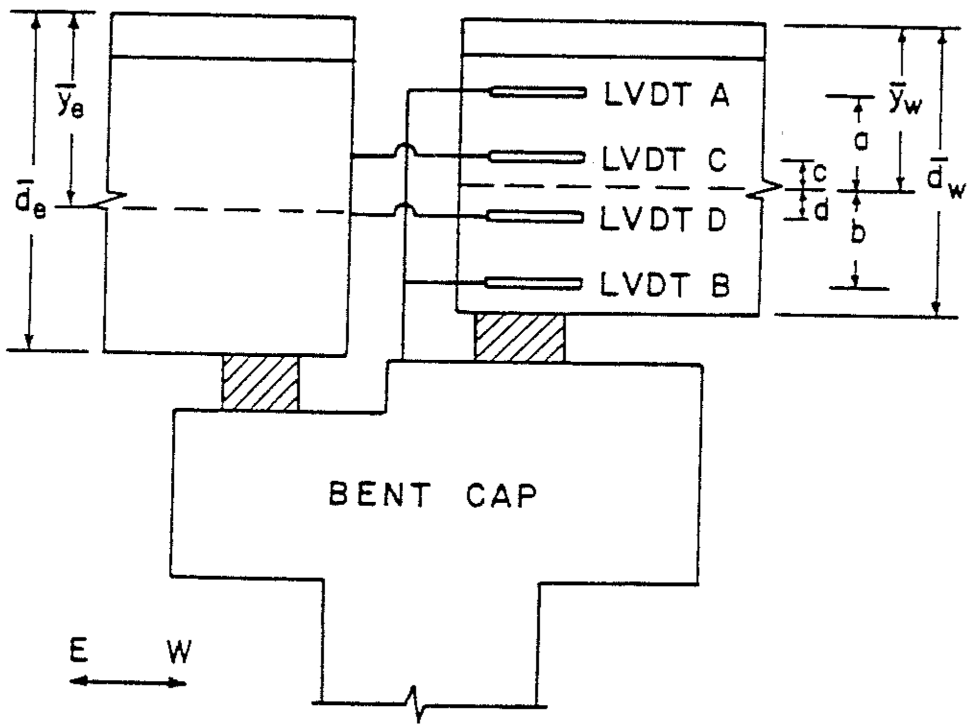

FIGURE 5 North Elevation of a Typical Expansion Joint Showing Measured Positions of LVDT's.

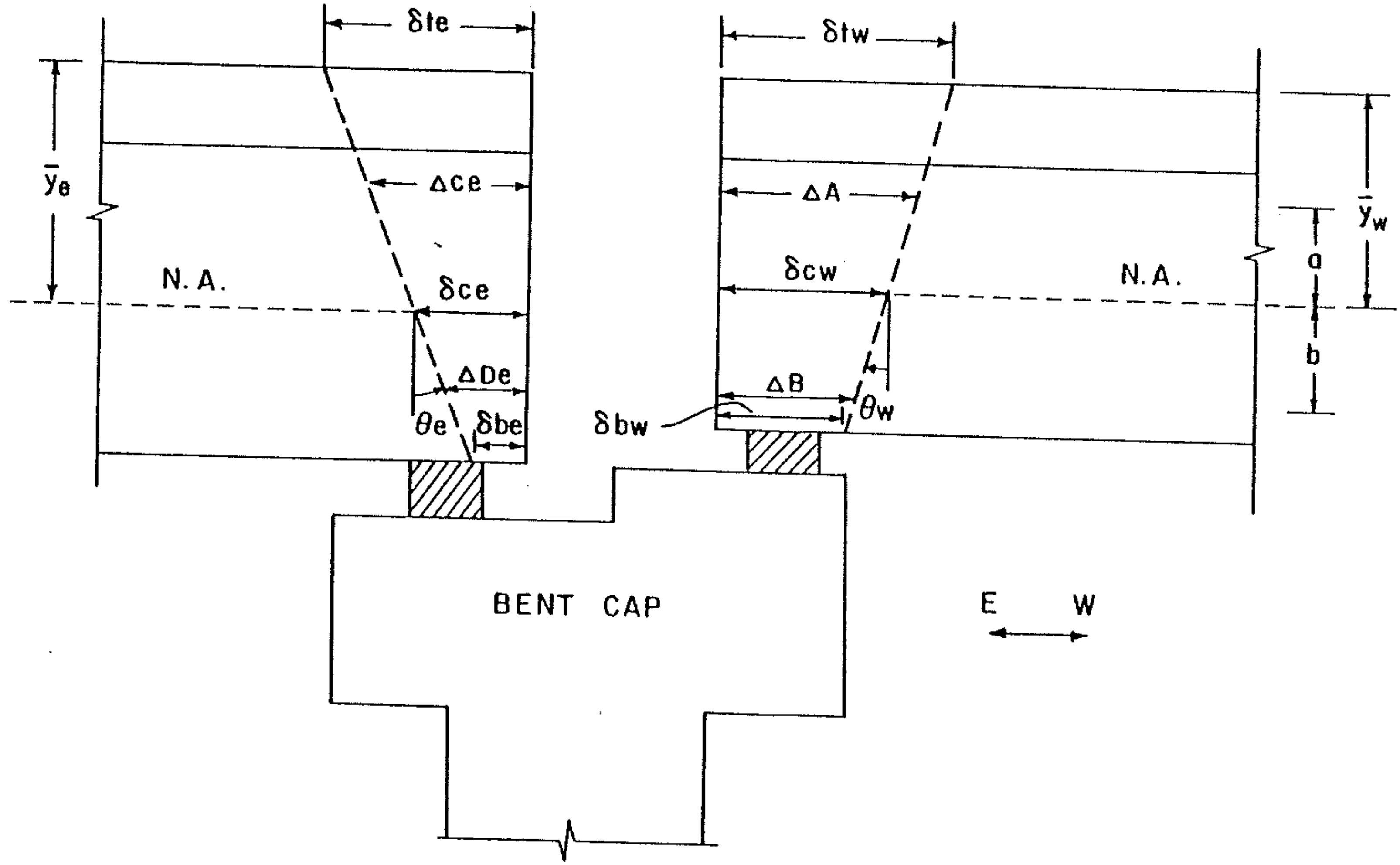

FIGURE 6 North Elevation of a Typical Expansion Joint, other than the Abutment, Showing Movements and LVDT Measurements Required to Calculate Movements.

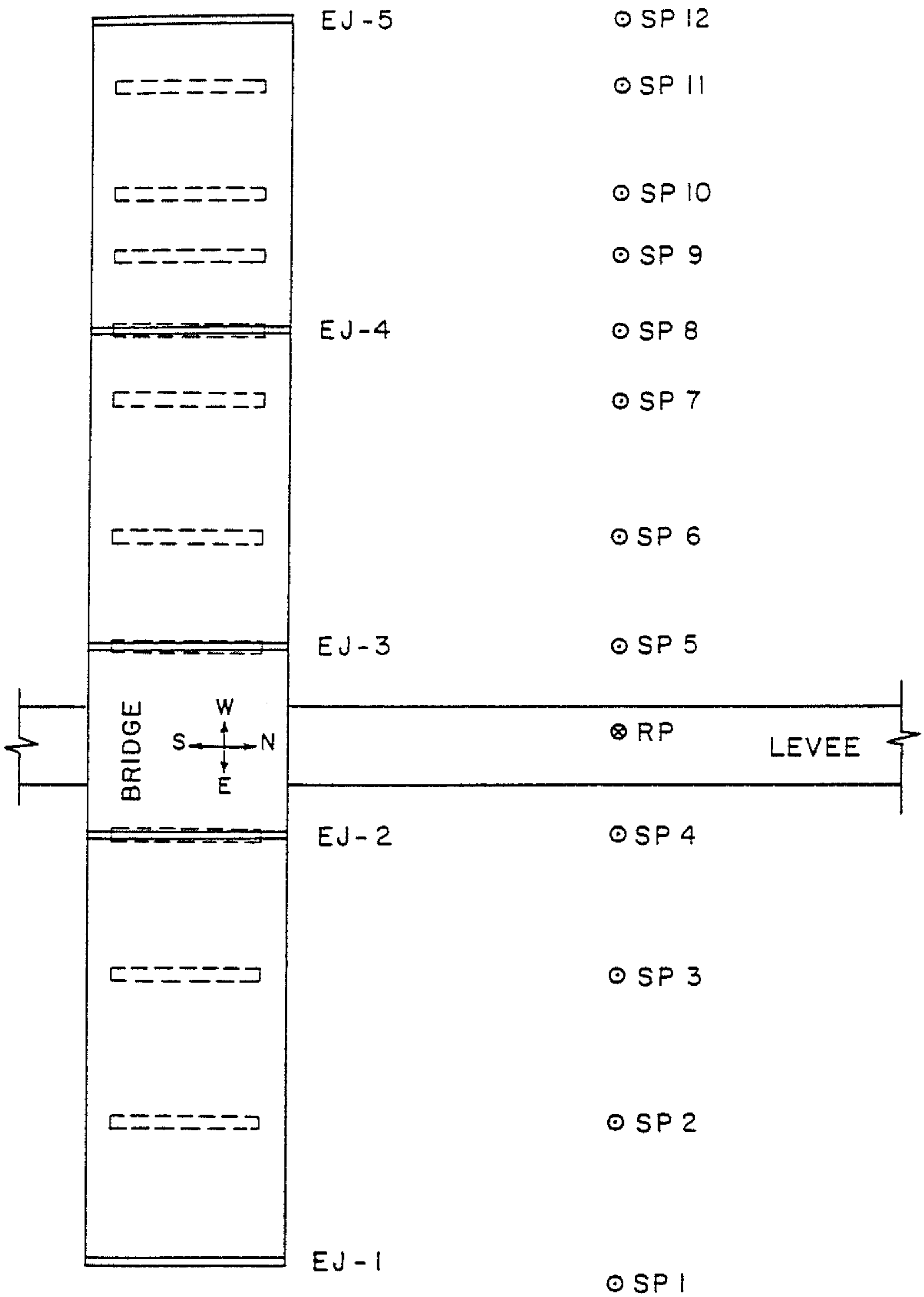

FIGURE 7 Plan View of Bridge Showing Theodolite Setup and Reference Points.

foot above the ground. The top of the setup points is marked by a brass plate embedded in the concrete. The top of the reference point is a reflector with its center well marked. The reflector allows for distances to be determined when used in conjunction with the theodolite. Bridge markers were placed on the north side of every bent so they could be monitored by the theodolite positioned over the setup points. The bridge markers were placed in two distinct configurations. The first configuration is the one used on all bents supporting continuous joints as shown in Figure 8. The exact number of markers is dependent on the height of the bent and ranges from four to eight. The markers are always numbered in the same manner from top to bottom. They are identified by a hyphen and a bent number. For example the marker 1-10 is the top marker at bent ten.

The second configuration is the one used at all bents supporting expansion joints, including the abutment, as shown in Figure 9. The exact number of markers again depends on the height of the particular bent. The markers are numbered the same for each expansion joint with the first and third markers placed on the eastward girder ends, and the second and fourth markers placed on the westward girder ends. The fifth marker is placed on the cap except at the abutment where there is no cap. The location of the markers after movement and the measurements required to calculate the overall movements are shown in Figure 10.

Thermocouple Wires

Thermocouple wires can be easily used to measure temperature. Type T copper-constantin thermocouples presented three advantages which made them the choice for this investigation. These are the following:

1. The temperature range is such that both ambient and slab temperatures could be accurately measured.

2. The thermocouples could be connected to the data acquisition system, allowing all temperatures to be measured at the same time as LVDT readings were taken.

3. Thermocouple wire is fairly inexpensive, and preparation of the wire is very simple.

Thermocouple extension wire type PP20TX was chosen for recording the temperature of the Krotz Springs bridge slab and girders as well as the ambient temperature. Since the temperature varies throughout the depth of the slab and girder, the thermocouples were placed along the depth of the section to detect this variation. Two different thermocouple arrays were used at the bridge as shown in Figure 11. Array #1 consists of six thermocouples located on both slab and girder. Array #2 consists of three thermocouples placed only in the slab. The location of these two different arrays is shown in Figure 12. A total of thirty thermocouples were placed at these locations.

The slab thermocouples were placed near the top, center and bottom of the slab

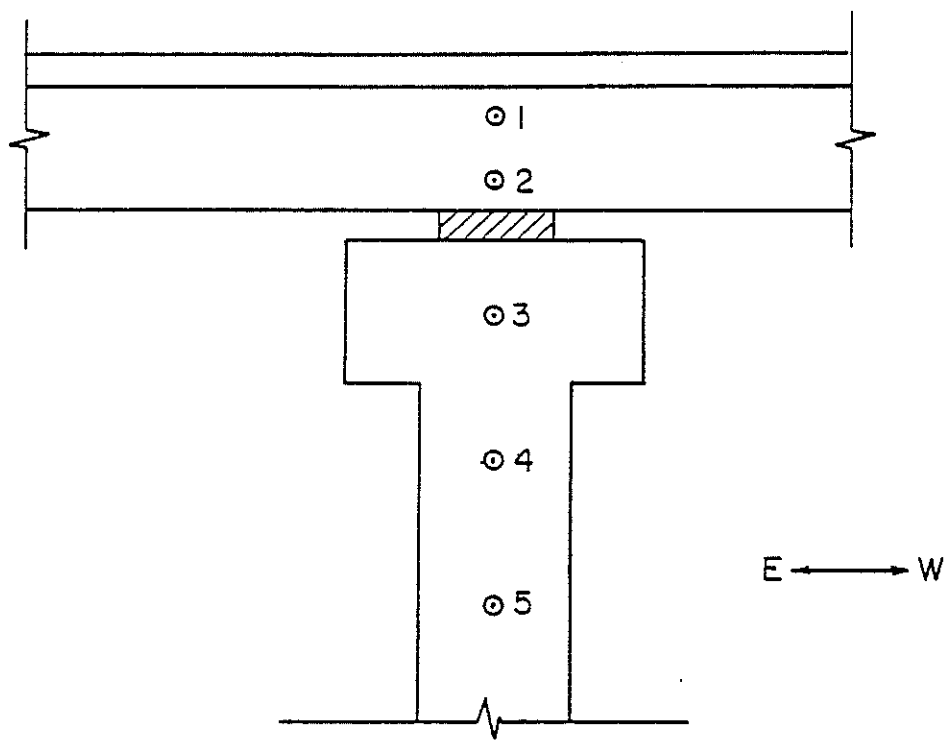

FIGURE 8 Marker Placement on a Typical Bent Supporting a Continuous Joint.

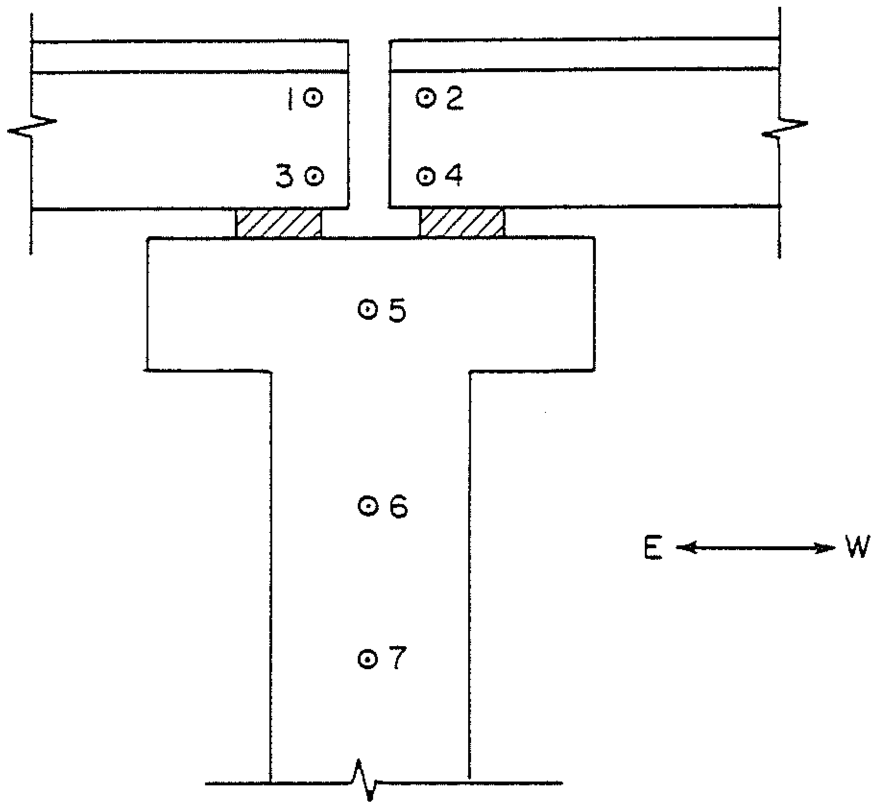

FIGURE 9 Marker Placement on a Typical Bent Supporting an Expansion Joint.

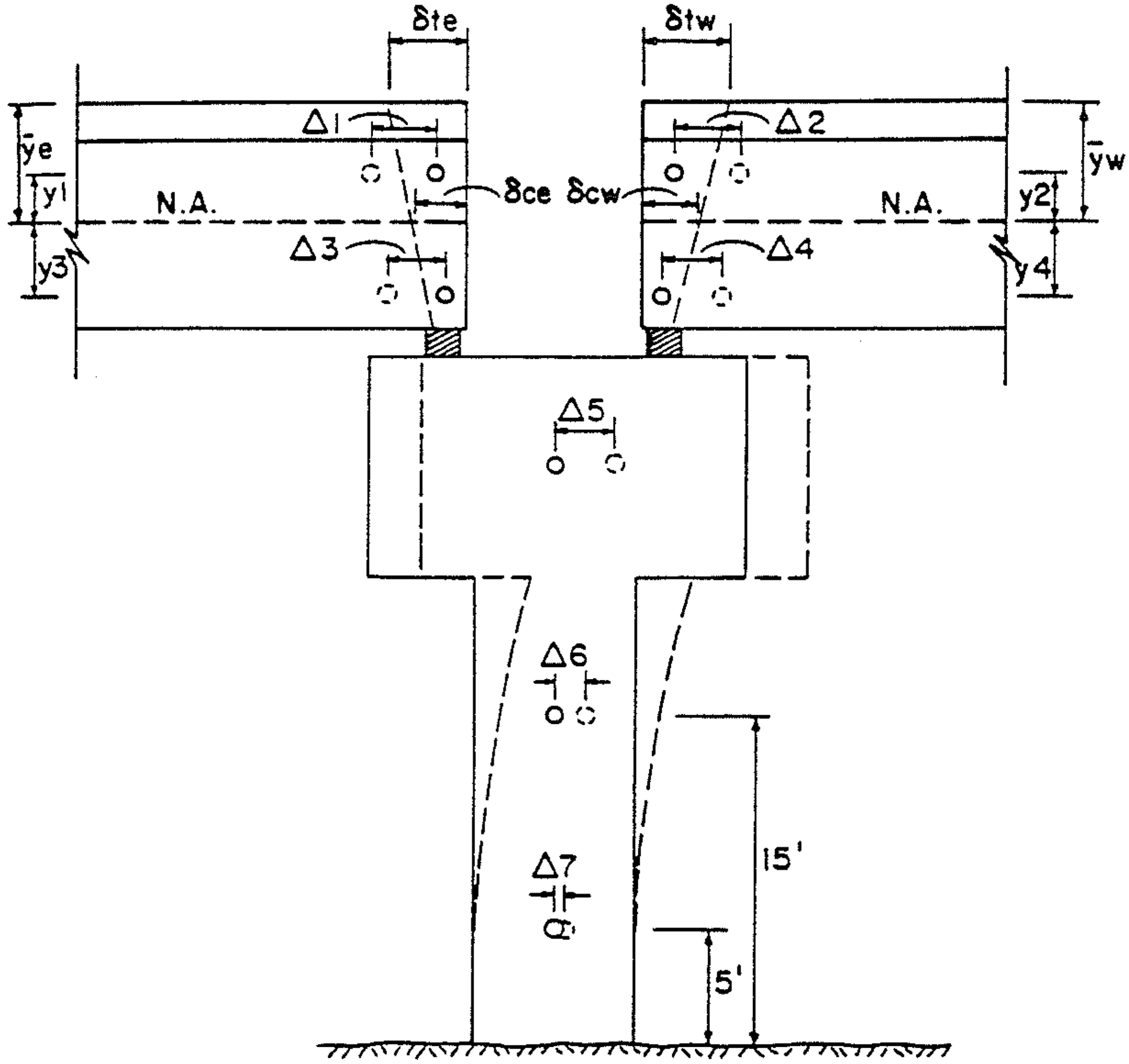

FIGURE 10 North Elevation Showing Expansion Joint and Markers Before and After Movement.

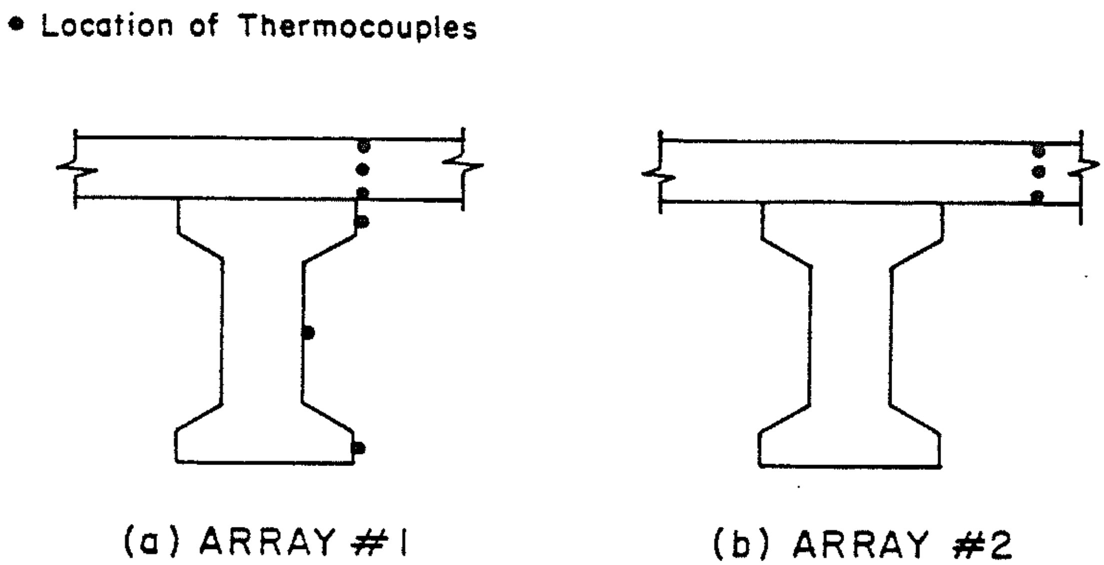

FIGURE 11 Section Through Bridge Showing Thermocouple Locations.

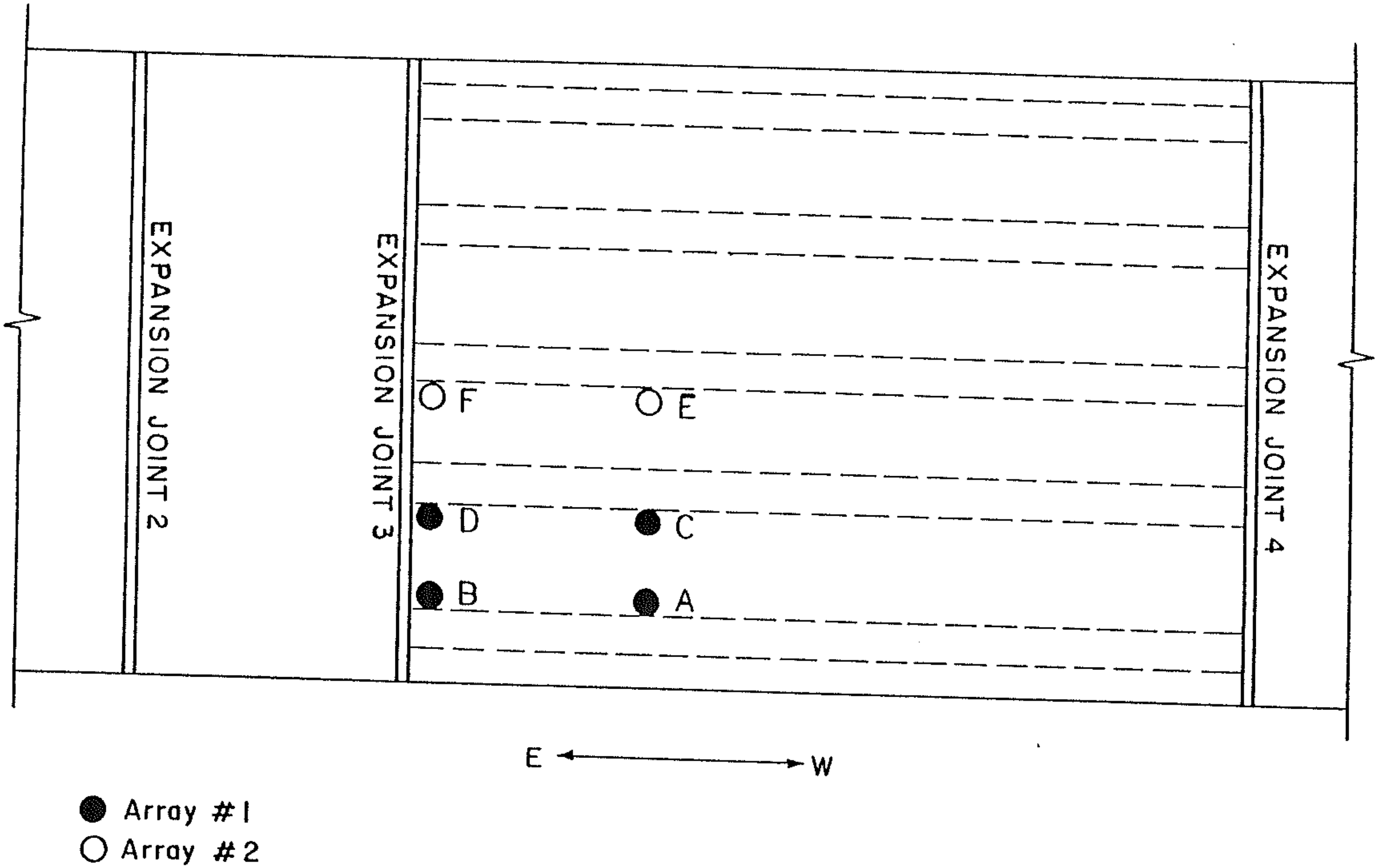

FIGURE 12 Plan View of Bridge Showing Location of Thermocouple Arrays.

at the time of pouring, thus embedding them in the concrete. After the slab forms were removed, the other ends of the thermocouple wires were connected and run under the bridge to the data acquisition system of the micro-computer at the monitoring station. The girder thermocouples were placed at a later time. These were bonded on the outer surface of the concrete girders using epoxy. A layer of hydraulic cement was applied over the epoxy after it had dried to ensure a more consistent thermal conductivity. As with the slab thermocouple wires, the girder thermocouple wires were run under the bridge to the monitoring station.

Two additional thermocouples were placed hanging under the slab to record the ambient temperature. These thermocouples were also connected to the data acquisition system.

Bridge Monitoring Schedule

The theodolite readings began in January of 1987. Each full set of readings required five to six hours to complete, therefore only one set of readings was taken on any monitoring day. In the beginning, readings were taken approximately every two weeks. As the effects of creep and shrinkage were anticipated to decrease, the frequency of data collection was gradually changed to a six week schedule. Temperatures corresponding to the days of theodolite readings were acquired from the Office of State Climatology at Louisiana State University.

The LVDTs were on line at 8 a.m. on October 22, 1987. The LVDT readings were taken approximately every month. On an alternate basis the LVDT readings were taken for 12 hours continuously, or 24 hours continuously. The thermocouple readings were recorded at the same time as the LVDT readings. In addition, during the days of data collection with the LVDTs, the theodolite was used to obtain the sway of the supporting bent cap at the expansion joints. These theodolite readings were taken every three hours in order to obtain a good pattern of movements throughout the day. Monitoring continued on schedule except for some minor interruptions. Five of the bridge markers were destroyed, either accidentally by the construction crew or by vandals. These markers were replaced and monitoring was continued. Also, one LVDT at location 4S was found to be defective and the data collected from it was discarded.

Presentation Of Results of Observed Movements

A tremendous amount of data was collected, but only the data required to evaluate the expansion joint movements was processed and presented in this executive summary.

LVDT Results

The expansion joint movements obtained using the LVDTs at Expansion Joint No. 2 North are shown in Figure 13. The movements are the result of dead loads and

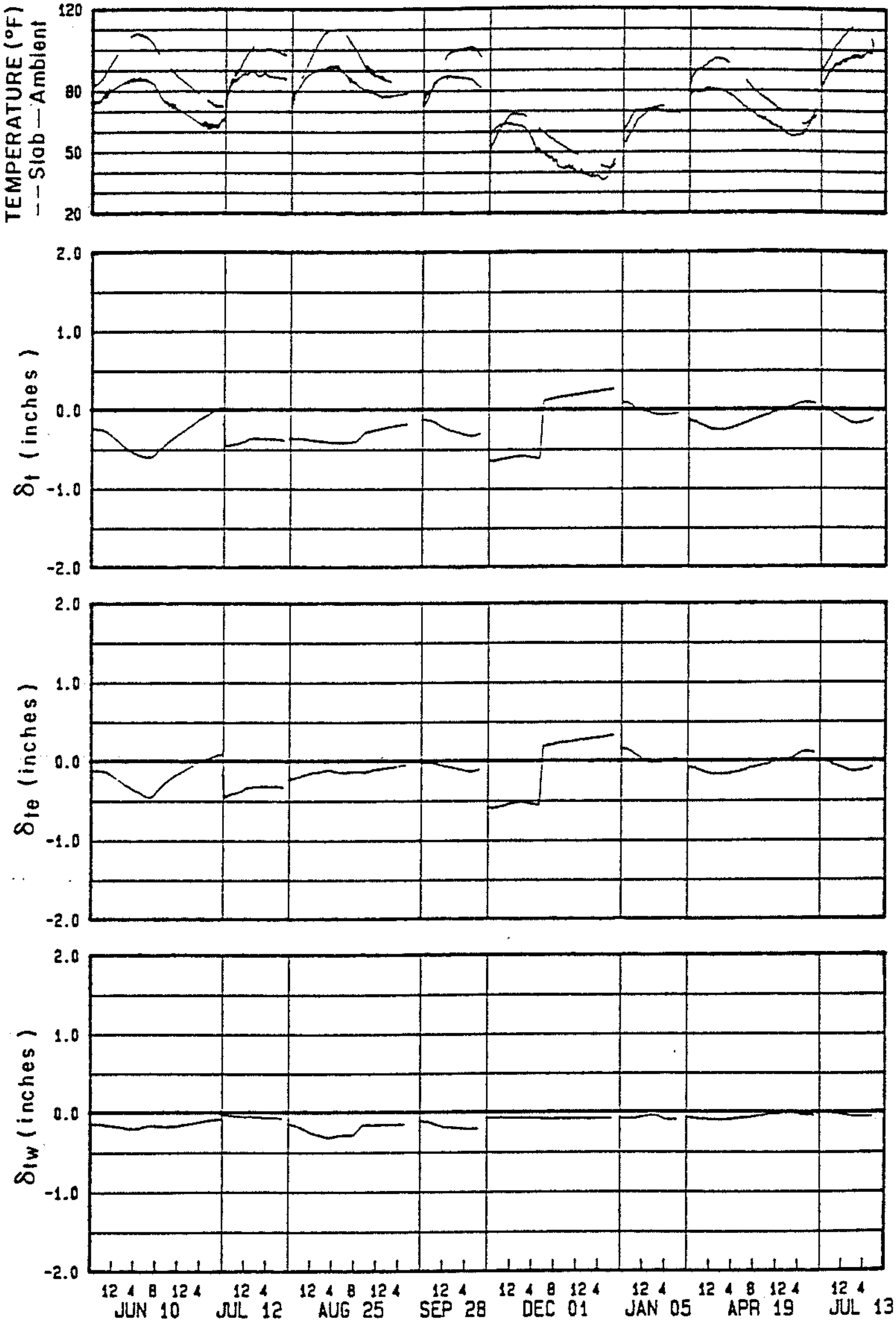

FIGURE 13 Bridge Movements Obtained from LVDT's at E.J.2 North.

thermal changes only. Since the LVDTs were not in place until nine months after the slabs were poured, creep and shrinkage effects had dissipated. Traffic had not begun on the bridge until October 27, 1988, therefore the effects of traffic loads should not be considered until after that time. The plots are with respect to time on the horizontal axis. Each day of monitoring began at approximately 8 a.m. and lasted either 12 or 24 hours as seen in the plots. The vertical axes represent the movements of the expansion joints, except for the plot at the top of each figure, which shows the ambient temperature and the middle slab temperature recorded during the days of monitoring. The movements labeled δ_p at the left of the plots were measured with the theodolite, while the other movements labeled δ_{te}, δ_{tw}, δ_t, δ_{be}, and δ_{bw} were collected using LVDTs.

The temperatures are the same for each of the figures, since monitoring of all expansion joints was done almost simultaneously. The slab temperature used in the plots was measured at the center of the slab. It often rises above the ambient temperature during the heat of the day, mainly due to solar radiation. The daily trend of the slab temperature is to begin near the ambient temperature in the morning, rise higher than the ambient temperature during the day, and cool down close to the ambient temperature over night. The ambient and slab temperature peaks generally occur between 3 pm and 4 p.m. The overall trend of the temperatures recorded during the monitoring days generally reflects the seasonal trend. The higher differential between slab and ambient temperatures occurs on the warmer days of monitoring, during the months of May through August 1988.

The general trend of the opening and closing of the expansion joints follows the temperature trend. The extreme values and ranges of movements recorded at the four expansion joints are summarized in Table 1. The table gives the maximum closing and maximum opening at the top of each joint, the day it occurred, the total range of movement, and the corresponding ambient and slab temperature differentials. Since the maximum joint movements at the north and south sides of the bridge did not consistently occur at the same time, the south side movements corresponding to the maximum north side movements and the north side movements corresponding to the maximum south side movements are also given in the table.

The following observations can be made from Table 1:

1. The maximum closing of the top of the joint occurs during the warmer days- May 16 and June 10 of 1988.

2. The maximum opening of the top of the joint occurs during the colder days- December 16, 1987, February 21, March 17, and December 1 of 1988.

3. The maximum joint movements at the north and south sides of the bridge do not necessarily occur during the same day.

4. The maximum joint movements of the north and south sides of the bridge have different magnitudes.

TABLE 1 Maximum Values of Expansion Joint Movements Obtained From LVDT's.

JOINT LOCATION	MAX. CLOSING (INCHES)	DATE	MAX. OPENING (INCHES)	DATE	TOTAL RANGE (INCHES)	AMBIENT TEMP. DIFFERENTIAL (DEGREES F.)	SLAB TEMP. DIFFERENTIAL (DEGREES F.)
1 NORTH	-0.7	May 16	+0.1	Dec 16	0.8	50	70
(1 SOUTH)	(-0.15)		(+0.05)		(0.20)		
1 SOUTH	-0.15	May 16	+0.35	Dec 01	0.50	30	50
(1 NORTH)	(-0.7)		(-0.15)		(0.60)		
2 NORTH	-0.6	Jun 10	+0.35	Mar 17	0.95	35	55
(2 SOUTH)	(-0.7)		(+0.55)		(1.25)		
2 SOUTH	-0.75	May 16	+0.55	Mar 17	1.30	40	60
(2 NORTH)	(-0.5)		(+0.35)		(0.85)		
3 NORTH	-0.8	Jun 10	+0.6	Feb 21	1.4	20	25
(3 SOUTH)	(-0.5)		(+0.1)		(0.6)		
3 SOUTH	-0.6	May 16	+0.2	Mar 17	0.8	40	60
(3 NORTH)	(-0.6)		(+0.6)		(1.2)		
4 NORTH	-0.5	Jun 10	+0.05	Feb 21	0.55	35	55

NOTE: Numbers in parenthesis represent south side movements corresponding to maximum north side movements, and north side movements corresponding to maximum south side movements.

All possible factors affecting joint movements were examined in order to explain the joint behavior observed in items 3 and 4 above. These are the following:

1. Joints reached maximum value allowed by mechanical connection - From a comparison of the maximum values and allowable movement, the physical limits were not exceeded;

2. Variations in material properties - It is possible that variations in the material properties of deck and girders have differently affected the magnitudes of joint movements in the north and south sides of the bridge. Due to the nature of this matter it was not possible to determine the magnitudes of such effects;

3. Defective steel truss pins - A problem was developed with the steel truss supporting pins of Bent 12. The sudden release of thermal stresses built-up at the pin assemblies and the resulting vibrations and shock waves could have contributed to the different joint movements observed at the north and south sides of the bridge. It should be noted, however, that the exact times of the occurrence of the shock waves were not recorded and their direct effects not possible to determine;

4. Construction crew and equipment - As mentioned earlier, bridge monitoring with LVDTs began October 22, 1987, while construction was completed in October of 1988. During this time period the bridge construction crew and equipment were on the bridge and at various locations. The presence of live loads on the bridge during construction could possibly have had an effect on the observed joint movement behavior. Again, this effect cannot be identified from the data obtained;

5. Bent movements - Measurements taken with the total station theodolite at the north side of the bridge showed that the supporting bent caps experienced longitudinal movements of up to 0.75 inches and insignificant vertical movements. Although no theodolite measurements were taken at the south side of the bridge, it is possible that the supporting bents at this side had experienced different movements. However, it is unlikely that bent movements could have affected the expansion joint movements to such an extent;

6. Connection performance - Stresses built-up at the bridge "roller" type connections could have had an effect on the observed expansion joint behavior. Visual inspection indicated that some of the slotted connections were not aligned with the cap bolts and possibly not acting ideally. Imperfections in the connections could result in stresses built-up during expansion and contraction of the girders. Depending on the level of the stress concentration at each connection, the girders may or may have not been allowed to move as expected;

7. Sun position over the bridge - It is possible that the path and position of the sun over the bridge had an effect on the different movements obtained at the north and south sides of the bridge. The bridge is located at a latitude of 30.3 degrees and oriented at an angle of 27 degrees from the east-west direction, while the sun's path can at most reach a latitude of 23.5 degrees. Consequently, the south side of the bridge is being exposed to the sun to a greater extent than the north side, and therefore expected to experience larger movements. However, since the results obtained did not indicate such behavior, and due to the relative effects of the other factors discussed previously it is extremely difficult to determine whether the sun's position over the bridge had a different effect on the north and south side movements of the bridge.

It can be deduced from the above discussion that it is difficult to identify the relative contribution of each possible cause influencing bridge joint movements. However, it is most likely that the build-up of stresses at the defective truss pins as well as at the "roller" supports had more pronounced effects on the bridge movements.

Effects Of Traffic

As indicated earlier the bridge was opened to traffic on October 27, 1988. With the exception of December 1988, when the traffic loads may have aided in releasing stresses built-up at joint supports, the movements acquired over the nine month period after the bridge was opened to traffic did not show any deviation from previous movements. The observed behavior indicates that the traffic effects on the bridge movements were insignificant when compared to the effects of thermal changes. However, to more fully evaluate these effects, monitoring over a more lengthy time period is required.

Data Discontinuities

The movements obtained from the LVDT readings showed some discontinuities. The sudden changes are not present throughout the whole set of data on the particular day, indicating that electronic malfunction was not the cause of this abnormal behavior. Since instrument error was not shown to be the cause of these changes, it is possible that sudden movements did actually occur. The exact causes of these movements have not been determined, however a possible explanation exists, which is the sudden release of stress build-up at the supporting pins of the steel. Shock waves associated with the release of stress at the truss pins act as an external force, causing the release of stress built-up at joint supports, which results in sudden movements. The sharp changes usually occur around peak temperatures, just as the shock waves caused by the truss pins were usually audible at that time. To further emphasize this point, the pins were replaced in January 1988, after which time the bridge movements did not show any sharp changes. It is important to note that the exact times of occurrence of the shock waves were not recorded and that the shock waves were not proved to be directly associated with the sudden bridge movements. However, from an examination of all 16 sets of data obtained from the LVDTs, the following observations can be made:

1. The bridge experienced sudden movements on all four monitoring days before the defective truss pins were replaced;

2. After the pins were replaced and before the bridge was opened to traffic, eight additional sets of data were obtained. Sudden changes occurred on only two of the eight sets, and as mentioned earlier, one was due to a power failure and the other due to a bad electrical connection;

3. After the bridge was opened to traffic, sudden changes occurred in only one of the four sets of data obtained, and this may be attributed to the effects of traffic.

It can be therefore concluded that the sudden release of stress built-up at the defective truss pins was the principal cause of the sudden movements occurring before the truss pins were replaced.

Analysis Of Thermal Profiles

The data obtained during the 24 hour monitoring days was used to further study the expansion joint behavior. For these days, the temperature distribution through the depth of the bridge section is plotted at four hour intervals starting at 8 a.m. as shown in Figure 14. The dashed line shown in the upper left plot of each figure represents the temperature distribution at the end of the 24 hour cycle or 8 a.m. next day. The ambient temperature is also given in each plot for relative comparison. It can be seen from Figure 14 that the thermal profiles follow a certain path over time. In particular, the slab temperatures are generally lower than or close to the girder temperatures during the morning hours, then rise higher than the girder temperatures, reaching their peak values around 4 p.m.. Finally, during the evening hours the slab and girder temperatures come close again then falling to their lowest values over night.

Thermal stress is known to cause considerable damage in bridges. Although current bridge specifications, such as those of AASHTO, recognize the existence of thermal expansion and thermal forces, they are rather vague concerning values. In particular, AASHTO recommends a range of temperature variation in bridges to account for the expansion movements, however it does not provide guidelines regarding the vertical temperature distribution through the depth of the section.

An attempt to develop a realistic and smooth temperature profile based on the experimental data is made in this research. Polynomials of different degrees were used to curve fit the temperature data obtained from the thermocouples. Based on the calculated sum of the squares of the residual, a second order polynomial is deemed to be both accurate and simple. A different curve fit is used to predict the bridge temperatures at the top of the deck T_1, the bottom of the deck T_2, and middle and bottom girder

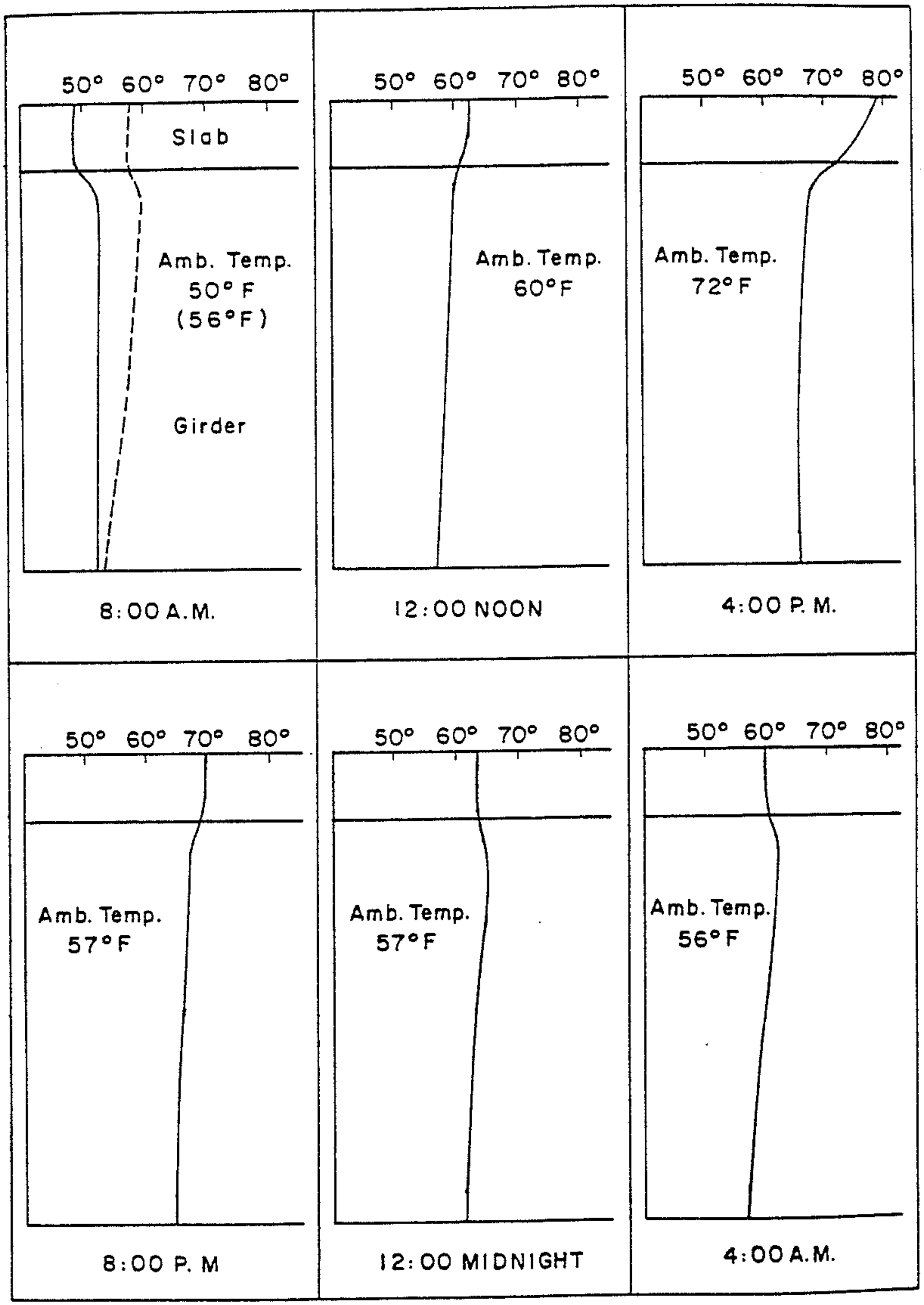

FIGURE 14 Temperature Distribution Through the Depth of the Section for October 22, 1987

temperature T_3. These curve fits are shown in Figure 15. The horizontal axis of the figure represents the ambient temperature T_a, while the vertical axis represents the bridge temperatures T_1, T_2, T_3. It should be noted that Figure 15 does not give the temperature distribution through the depth of the section but rather it gives the curves to obtain T_1, T_2 and T_3 for a known range of ambient temperatures T_a. Once T_1, T_2 and T_3 are found the thermal profile is obtained by assuming a linear temperature variation between T_1, T_2 and T_3. It should also be pointed out that the curves are valid only for the given range of ambient temperatures, which is between 32 and 92 degrees Fahrenheit. The values of T_1, T_2 and T_3 can also be calculated from the following equations:

$$T_1 = 0.095 + 0.832T_a + 0.004T_a^2$$

$$T_2 = 6.63 + 0.648\ T_a + 0.005T_a^2$$

$$T_3 = 23.88 + 0.206T_a + 0.006T_a^2$$

Analysis Of Joint Movements

It is desirable to distinguish bridge joint movements due to temperature changes from those due to other factors such as creep and shrinkage, and loss of prestress for prestressed concrete sections. In order to do so, thermally induced movements must be studied over a short period of time, essentially eliminating the effects of any other factors on bridge joint movements.

In order to identify short-term movements resulting from temperature changes from movements resulting from the other factors mentioned earlier, the behavior of the bridge was studied using the data acquired over the 24 hour monitoring days. The movements obtained from the LVDTs on a typical day are shown schematically in Figure 16. These movements are relative to the supporting bent caps and are referenced to the beginning of the particular monitoring day at 8 a.m.. It can be seen from Figure 16 that the bridge sections do not return to their initial position at the end of the 24 hour cycle, which is 8 a.m. next day, but rather stay or freeze at their deformed position. One of the reasons for such behavior may be that the initial and final temperature distributions of the bridge sections do not coincide as seen in Figures 16 but rather can differ by as much as 12 degrees. However, restraints at the "roller" connections could be the main reason the bridge units do not recover at the end of the 24 hour cycle. Since no readings were taken beyond the 24 hour period at any monitoring day it is not known when the bridge sections bounced back to their initial position. It is also observed from the figures that the behavior of units 1 and 2 is more reasonable and more consistent than the behavior of unit 3. It may be recalled that units 1 and 3 are three span continuous concrete sections while unit 2 is a simple span steel girder. From calculations performed it was found that the dead loads carried by either concrete or steel sections are approximately the same and therefore irrelevant to the different behavior of the sections. It can be also seen from the figures that the bridge sections experienced non-symmetrical movements. These movements can be attributed to the restraints associated with the joint supports.

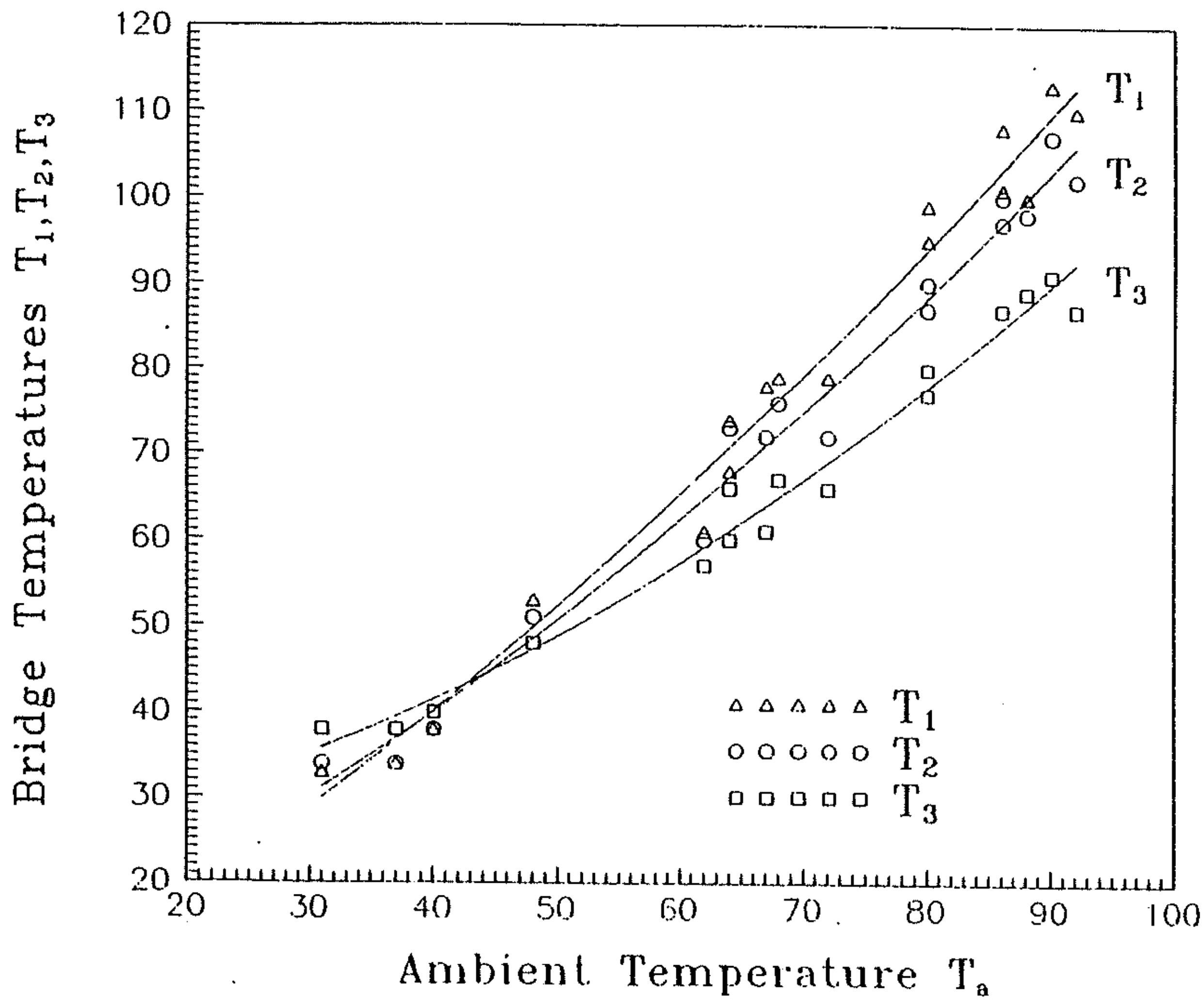

FIGURE 15 Bridge Temperatures as a Function of Ambient Temperature

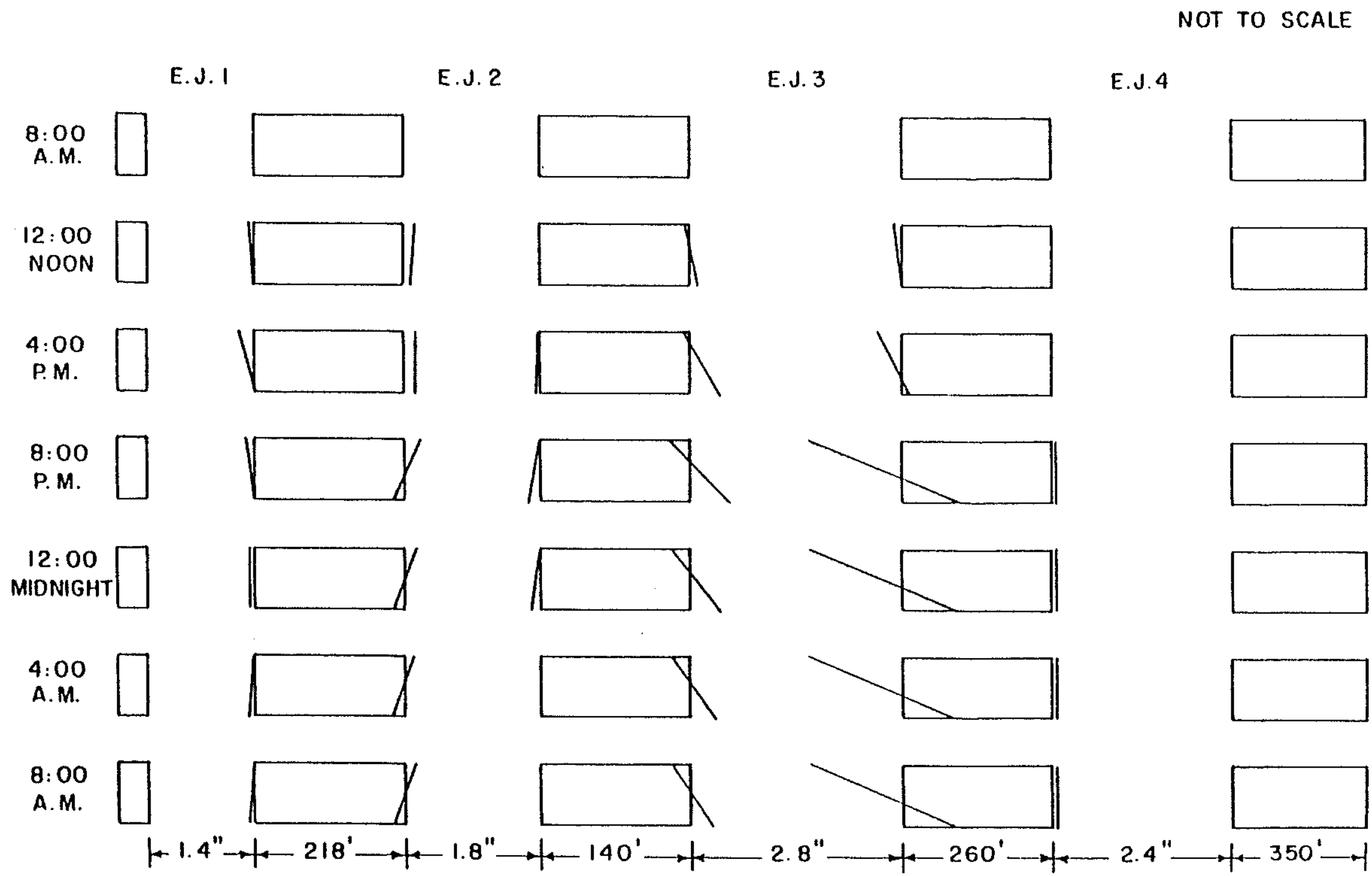

FIGURE 16 Short-Term Movements Obtained from LVDT's at North Side of Bridge for October 22, 1987

A summary of the long term movements obtained from LVDTs at the northe side of the expansion joints 1-4 are presented in Figure 17. These movements are relative to the supporting bent caps and are referenced to the first day of monitoring October 22, 1987. The movement at expansion joints obtained from the total station theodolite are given in Figure 18. Unlike the LVDT readings, the theodolite readings were only taken once during each day of monitoring. All movements shown are with respect to the position of the bridge during January 1987. The actual starting date is different for each expansion joint, but all readings began in January 1987 before the pouring of unit 2, 3, and 4 slabs. The theodolite readings were started this early in the construction phase of the bridge for three reasons. First, the LVDT instruments could not be placed on the bridge that early for reasons discussed previously; second, to detect possible effects of creep and shrinkage on the bridge sections; and third, to determine the effects of bent movements on the joint behavior. Considering the bridge section monitored as a whole, the expansion joints behaved in a manner consistent with the thermal expansion and contraction. The effects of creep and shrinkage may be evident by looking at Figure 19. When the recorded temperature in March 1988 is near the reference temperature of 60 degrees in January 1987, the top of the joint does not return to the initial zero position, but rather shows an opening of about 0.3 inches for each expansion joints 2, 3, and 4. This joint opening indicates a contraction of the composite sections and can be attributed to creep and shrinkage of the system during this period. A comparison of the LVDT results to the theodolite results indicates that both instrumentation systems detect movements of similar magnitudes. The theodolite was also used to determine the longitudinal sway along the length of the bents. The sway of a bent supporting expansion joint 4 is displayed in Figure 20. The sway of the bent is presented in chronological order, with the first day shown in each figure being the reference datum. It should be noted that the magnitudes of the movements are relatively small (maximum equal to 0.6 inches) as compared to the height of the bents. The movements shown in the figures were magnified for ease of presentation. As mentioned earlier, the theodolite readings began early in the construction phase before the LVDTs were placed on the bridge. One of the reasons the theodolite measurements were taken that early and continued throughout the course of the research, was to observe the long-term behavior of the bents and determine whether bent movements was a major factor influencing joint behavior. From the study of the bent behavior conducted the following were found: 1) the bents experienced negligible vertical movements; 2) the bent rotations were very small and considered insignificant; and 3) the maximum longitudinal movements of the bents were smaller than the maximum movements of the girders which indicates that the bents were pushed or pulled by the girders during thermal expansion or contraction. It can be therefore concluded that the bent movements had no significant effects on the joint behavior.

Comparison Of Actual Movements To Predicted Movements

The specifications of the American Association of State Highway and Transportation Officials, provide guidelines for expansion and contraction of bridge members due to temperature changes. These guidelines are generally adopted by the Louisiana Department of Transportation Bridge Design Manual, where the design of sealed expansion joints for highway bridges is based on the prediction of joint movements. The

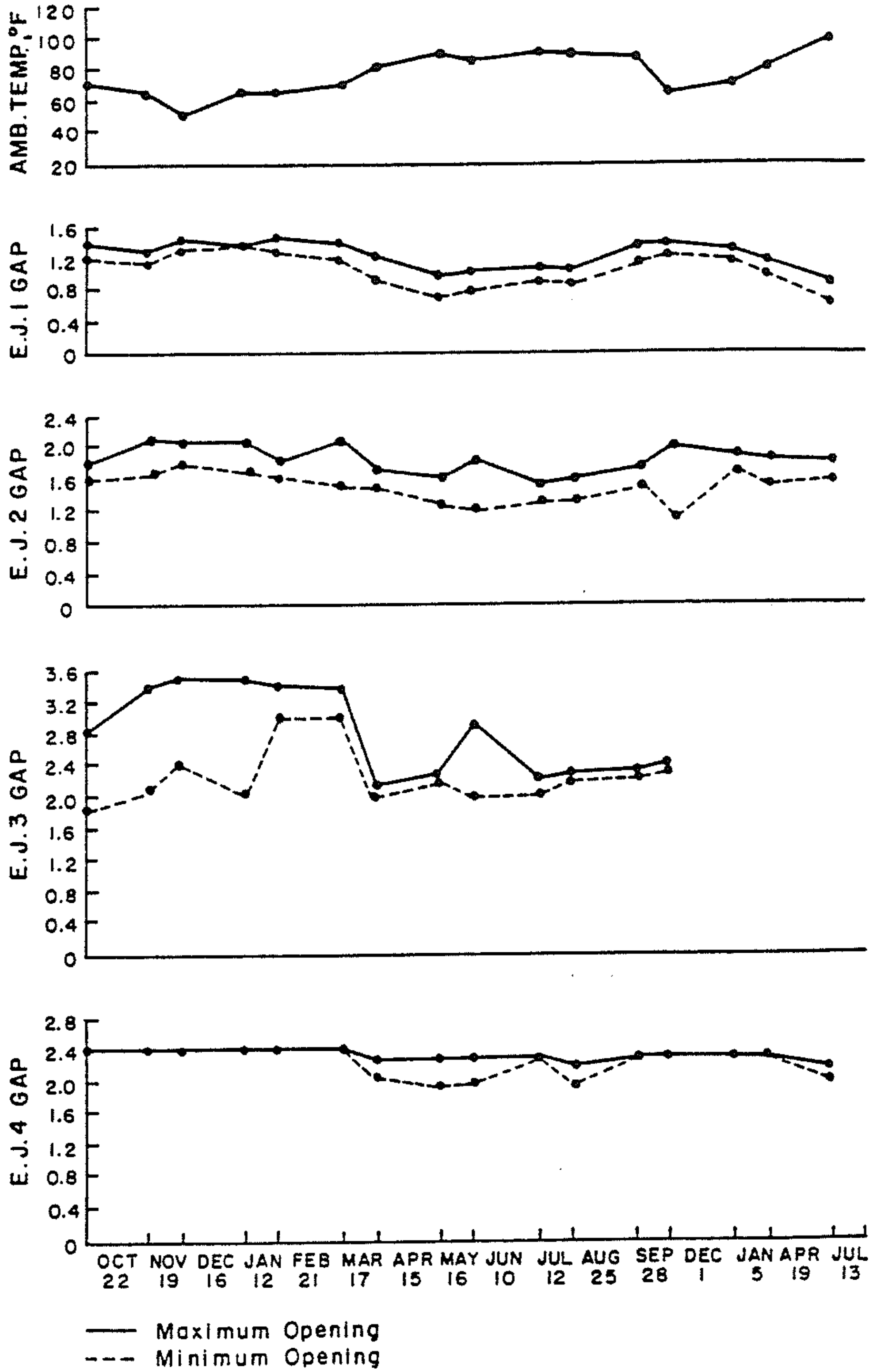

FIGURE 17 Summary of Long-Term Movements Obtained from the LVDT's at the North Side of Expansion Joints 1-4

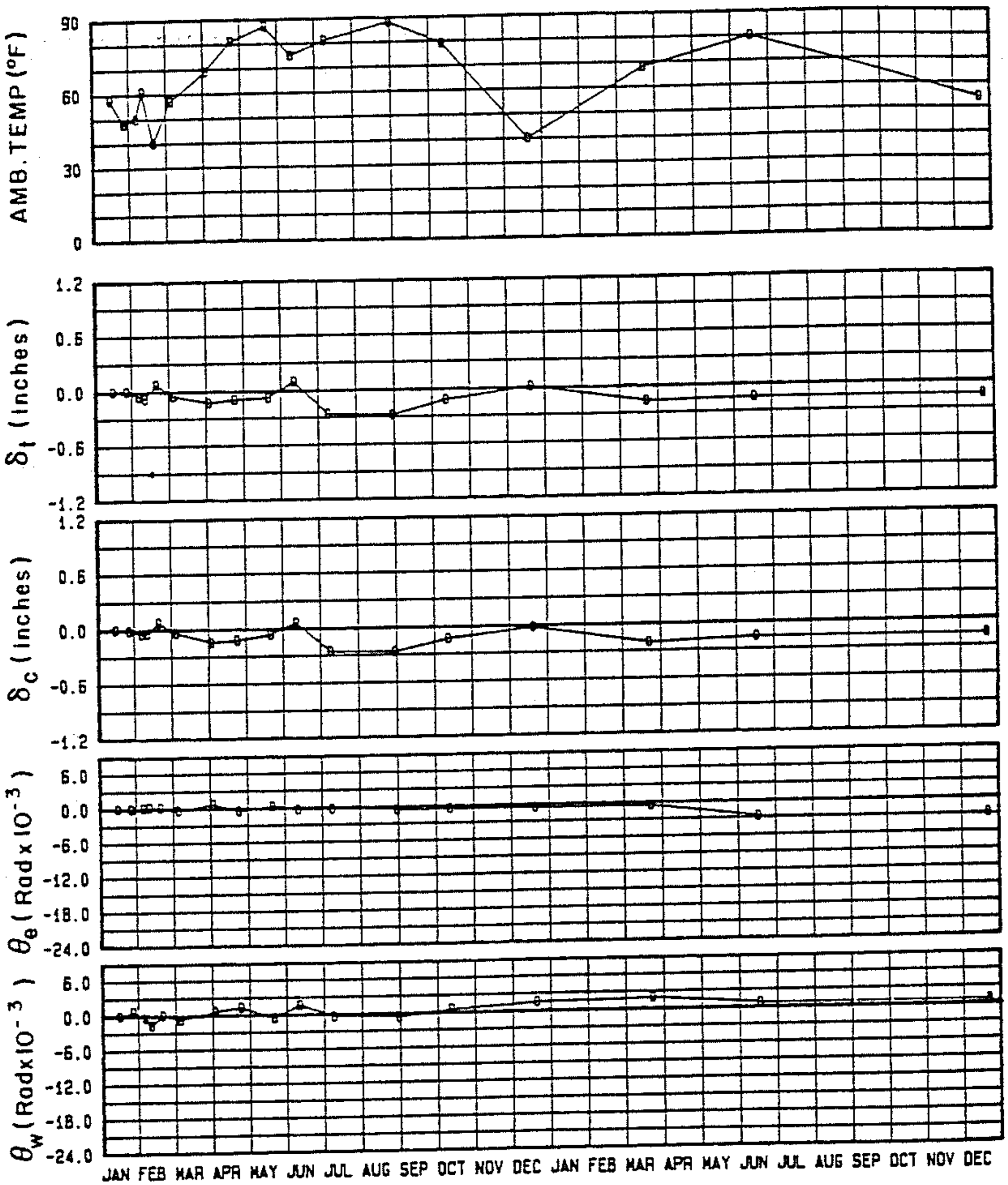

FIGURE 18 Bridge Movements Obtained from Theodolite at Expansion Joint 1.

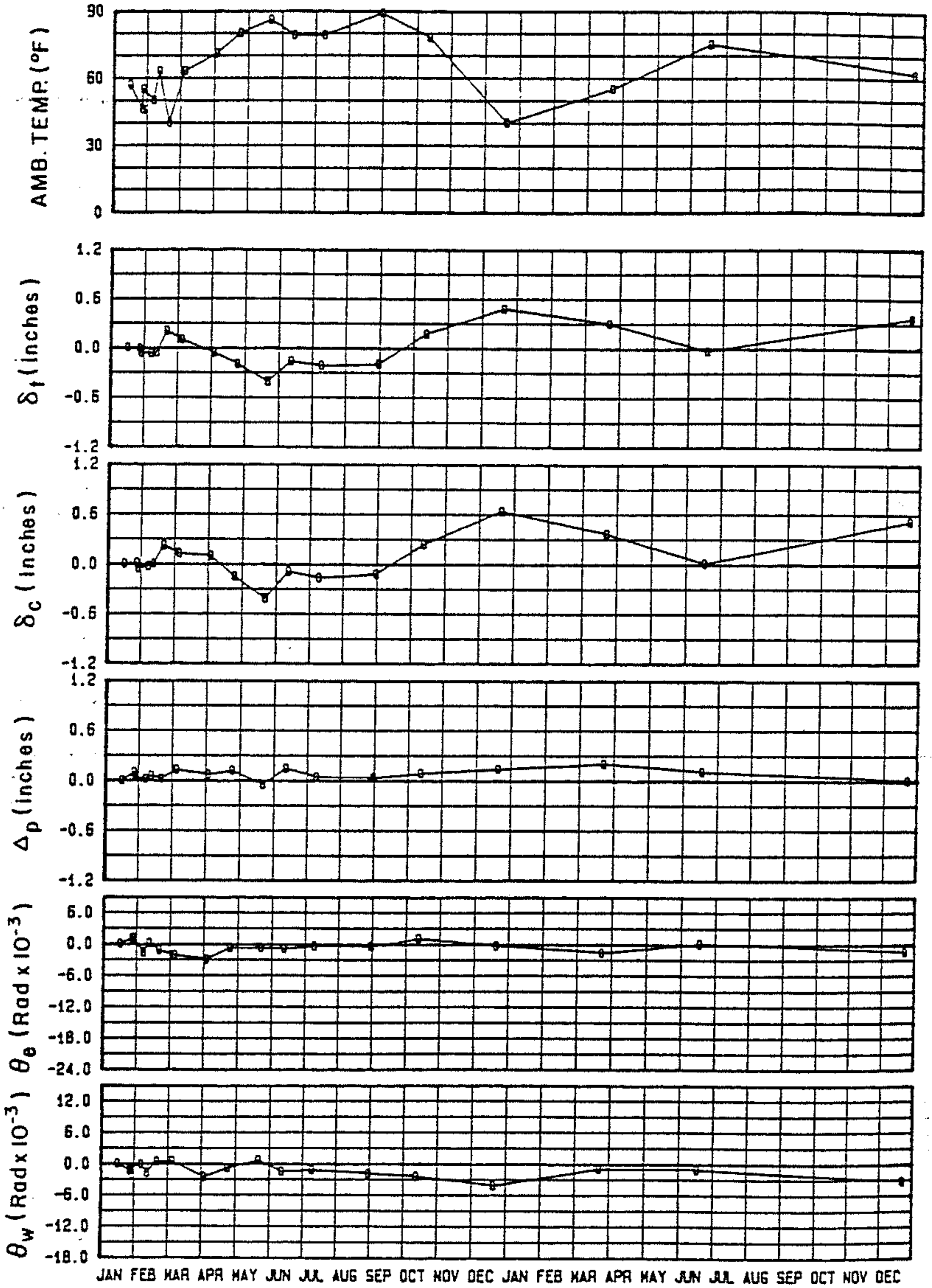

FIGURE 19 Bridge Movements Obtained from Theodolite at Expansion Joint 2.

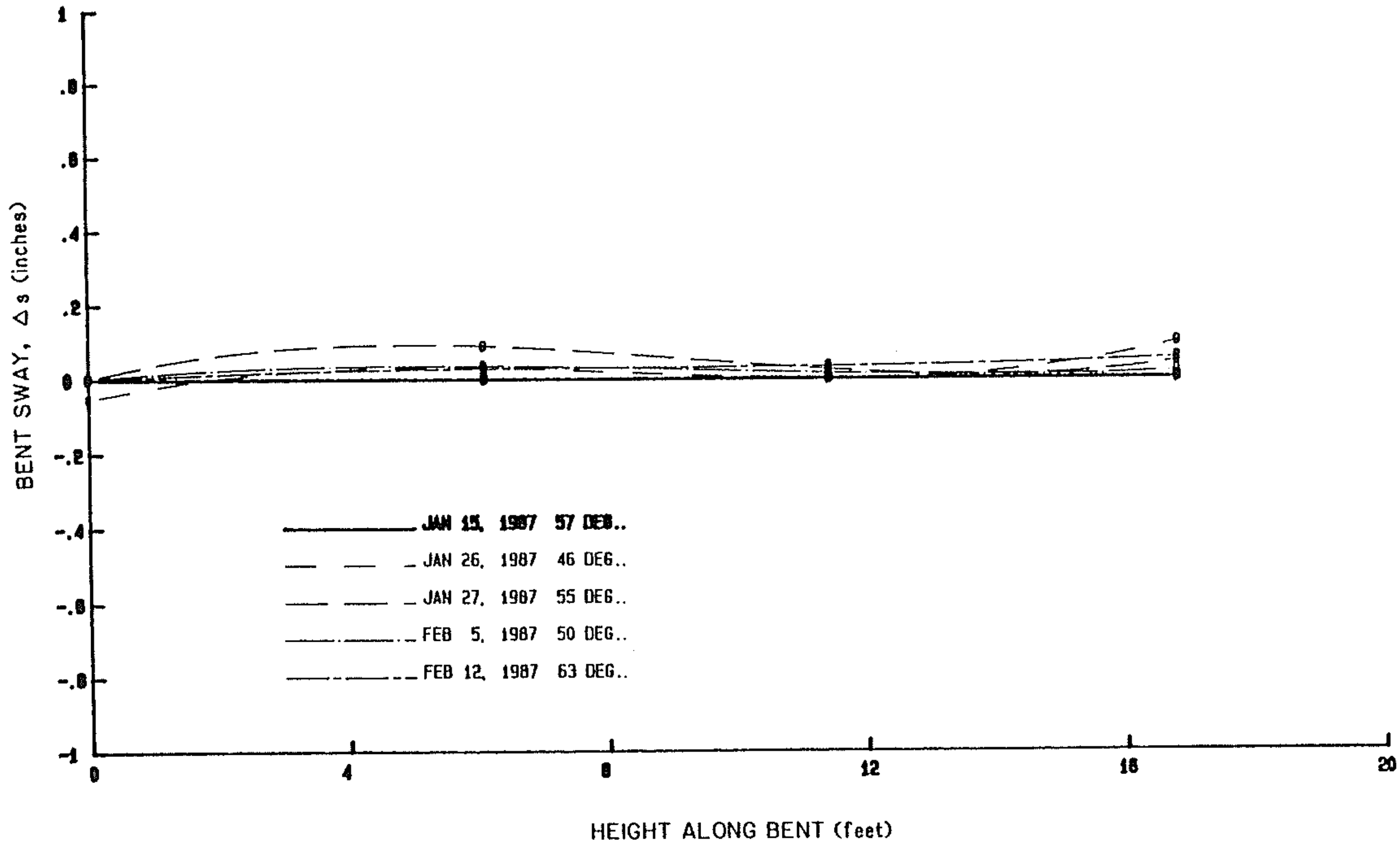

FIGURE 20 Sway of Bent 4 (Supporting E.J.2) Obtained from Theodolite

bridge design manual addresses two aspects of longitudinal bridge movement. First, the prediction of movement due to thermal effects is obtained by multiplying the coefficient of thermal expansion by the length of the member and by the range of temperature (rise and fall). The coefficient of thermal expansion is dependent on whether the girder is concrete or steel and is taken as 0.000006 per degree Fahrenheit for concrete girders and 0.0000065 per degree Fahrenheit for steel girders. The temperature range is taken as 30 degrees rise and 40 degrees fall for concrete girders and 60 degrees for either rise or fall for steel girders. The second aspect of longitudinal movement addressed is a combination of creep and shrinkage effects. The movement due to creep and shrinkage is predicted by multiplying the shrinkage coefficient by the length of the member. This coefficient is taken as 1/4 inches per 100 feet for prestressed concrete girders and 1/8 inch per 100 feet for steel girders. An installation dimension of a minimum of one inch is added to the thermal and shrinkage movements in obtaining the maximum joint opening for the design of joint seals. The above criteria was applied to predict the movements of the Krotz Springs bridge which are presented in Table 2. It can be seen that the actual movements of expansion joints 1 and 2 have either reached or exceeded their predicted values, although they were obtained at temperature ranges approximately 30 percent lower than the ones used for the predicted movements. The movements of expansion joints 3 and 4 however are well below their predicted values.

FINITE ELEMENT MODELING OF BRIDGES

The finite element method is chosen to perform the analysis of bridge structures consisting of cast-in-place concrete decks supported by multiple precast pretensioned concrete girders. These bridges pose a problem as far as their finite element representation is concerned. A true reproduction of their complex geometry can only be achieved with the use of three-dimensional elements; especially so for the case of curved superstructures. In the present analysis, the use of three-dimensional quadratic isoparametric elements is made to model both girders and the slab. The choice of such elements allows for a realistic simulation of the interaction between slab and girder and the representation of curved geometries without tedious geometric transformations. Figure 21 shows a viable configuration of these elements for a single girder and slab structure.

Time-Dependent Behavior of Structural Concrete

Concrete bridges exhibit time-dependent behavior which may significantly affect their serviceability. This behavior is due to the interaction of concrete with its environment, resulting in complex physical and chemical changes with time. It is therefore essential to investigate the long-term deformation behavior of concrete bridges and to attempt to ensure the satisfaction of serviceability criteria during the design life of the structure. Among the phenomena that affect the time-dependent behavior of concrete bridges, the three most important are creep, shrinkage and temperature. These phenomena will now be defined. Creep and shrinkage, along with the instantaneous elastic strain on loading, form the three components of deformation of concrete. These components are assumed to be independent of each other in that they are additive.

TABLE 2 Comparison Of Actual Joint Movements To Predicted Movements.

Joint Location	Actual Movements (With LVDT,s)	(With Theodolite)	Predicted Movements
E.J. 1	0.8"	0.4"	0.81"
E.J. 2	0.95"	0.85"	0.81"
E.J. 3	1.4"	1.0"	2.45"
E.J. 4	0.55"	1.05"	2.27"

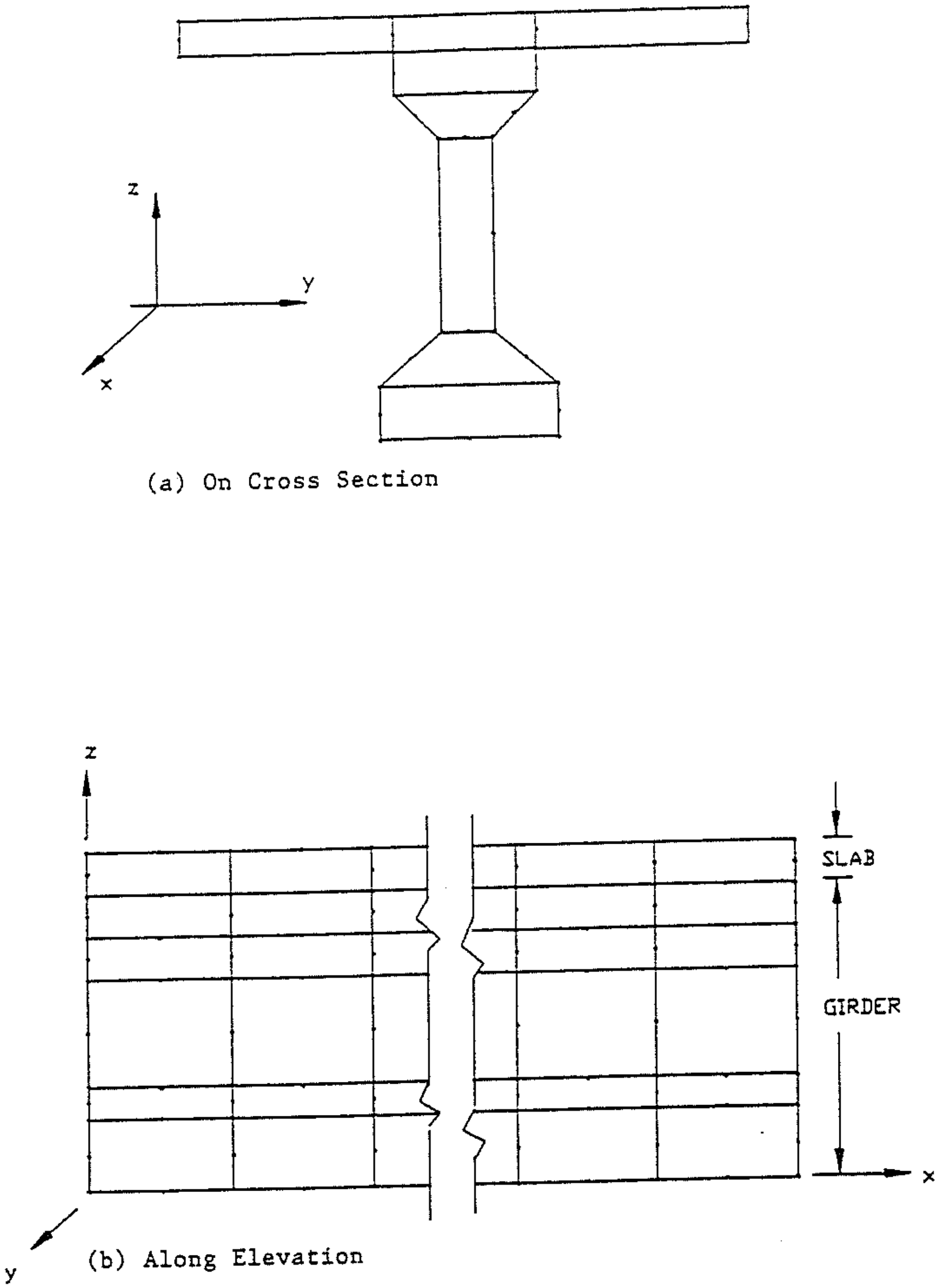

FIGURE 21 Configuration of Elements and Nodes

The three components of deformation in concrete for a specimen loaded at time t_o change with time. This is shown qualitatively in Figure 22. The instantaneous elastic strain is the strain that occurs immediately upon load application. Its value depends on the value of the modulus of elasticity E_c of concrete, which is a function of the age of concrete. The modulus of elasticity increases with time and hence the elastic strain decreases. The determination of the concrete modulus of elasticity and its change with time are generally made from the expressions proposed by the ACI Committee 209. Creep is defined as the increase in strain with time under, and induced by, a constant sustained stress. The creep strain could be broken up into two components: basic creep and drying creep. Basic creep is identified as the creep occurring when concrete is in hygral equilibrium (no moisture exchange) with its environment. Drying creep is the excess strain that occurs under conditions of drying. The creep strain in concrete is influenced by a variety of factors. Some of the important factors are: (1) age of loading; (2) stress/strength ratio; (3) type of aggregate; (4) size and shape of specimen; (5) ambient humidity; and (6) temperature.

Volume changes that occur in concrete independently of externally imposed stresses and of temperature changes are termed as shrinkage. The primary cause of shrinkage is the loss of water from the concrete during drying. The inverse process of swelling is of little significance in practice. The shrinkage process starts at the surface of a concrete specimen and gradually penetrates into the center. This results in a nonuniform distribution of shrinkage known as differential shrinkage. In concrete design and analysis, hrinkage is usually considered to be uniform. The most important factors that influence shrinkage are aggregate, water/cement ratio, volume/surface ratio and ambient humidity.

Prediction of Material Properties

The performance of a time-dependent analysis requires the knowledge of creep and shrinkage strains at any time during the lifetime of the structure.The best source of these strains are from creep and shrinkage tests performed on the concrete used. As the availability of long-term creep and shrinkage data is rarely guaranteed in bridge projects, fair estimates of the properties needed for the analysis can be made using approximate procedures. Three reliable sources of material properties are the ACI Committee 209 recommendations, the CEB-FIP recommendations and the simplified model developed by Bazant and Panula. The last model will be referred to as the BP2 model. The above mentioned procedures were utilized in this study.

Analysis of the Bridge System

The overall algorithm for use with the stiffness method in performing the time-dependent step-by-step analysis is outlined in the full report. The accuracy of the algorithm is dependent on the time intervals into which the total time frame of the analysis is divided. A rational choice of time intervals and the special considerations inherent in the analysis of bridges with cast-in-place concrete deck slabs supported by precast, pretensioned concrete girders, are presented in the report. The reader is

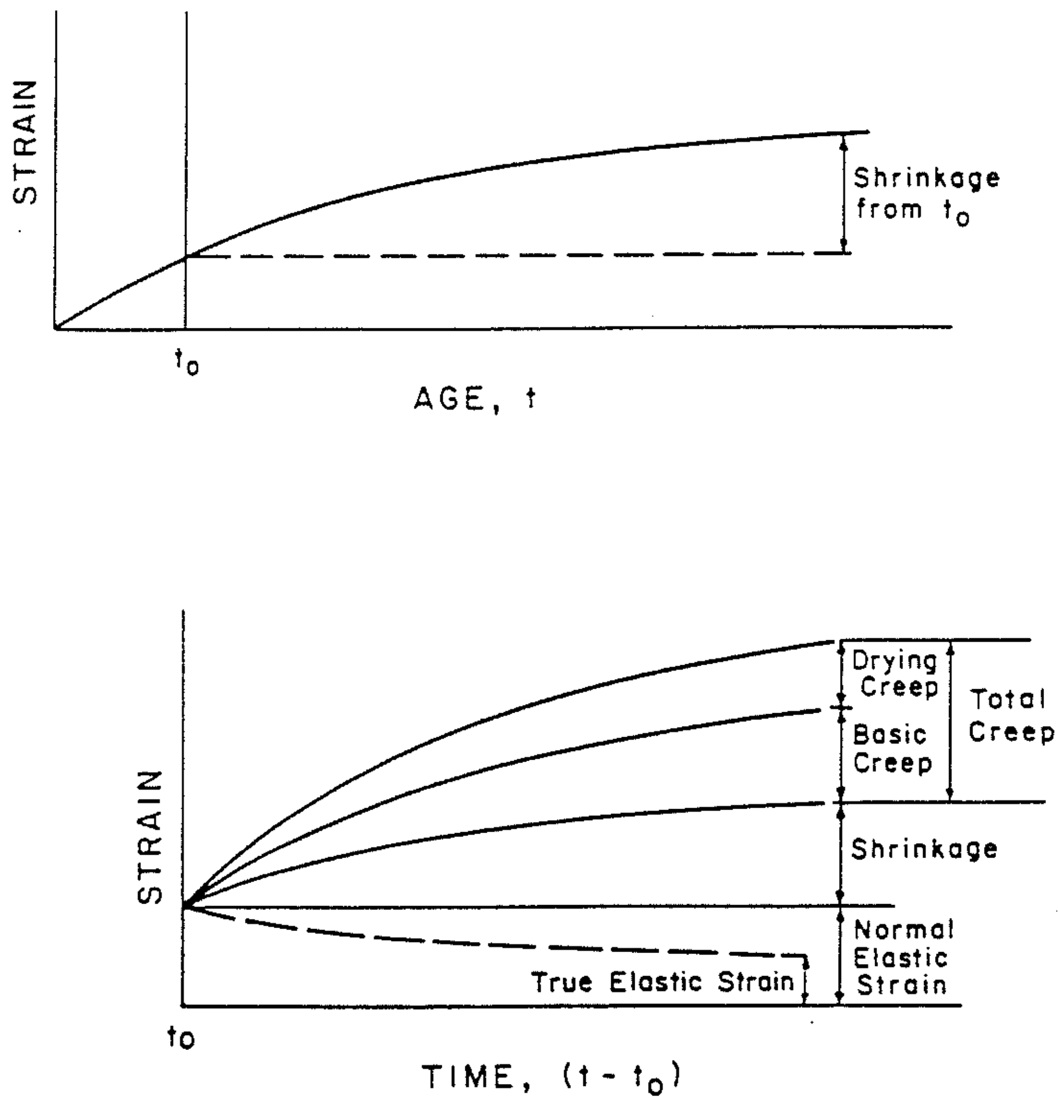

FIGURE 22 Components of Deformation in Concrete

referred to the full report for complete details relating to the analytical modeling of the bridges.

APPLICATION AND VERIFICATION OF ANALYTICAL MODEL

The analytical procedure developed for the bridges has been coded into a FORTRAN program named PCBRIDGE for use on the IBM 3090 computer. Three numerical studies using PCBRIDGE were carried out to:

1. Verify the validity and accuracy of the analytical procedure developed in this study to predict the long-term behavior of prestressed concrete bridges;

2. Gain an understanding of how the three standard creep and shrinkage models affect predicted bridge behavior; and

3. Estimate the joint movements in the Krotz Springs Bridge which was chosen for experimental evaluation.

In the first of the three numerical studies, analyses were performed on a simply supported girder to validate the applicability of the model to simple systems. Sinno and Furr tested a series of precast, pretensioned, simply supported beams and studied the deformations and the prestress loss with time after release. Live loads were not considered in the experiment. One of the beams tested by Sinno and Furr was chosen to investigate the validity of the model. The elevation and cross section views and the finite element model of the beam are shown in Figure 23. The analyses was performed using the creep and shrinkage strains predicted by (a) Sinno-Furr expressions; (b) ACI-209 model; (c) BP2 model; nd (d) CEB-FIP model. The experimental and analytical results for the midspan deflections and prestress loss as a percentage of the initial prestress are listed in Table 3. The results presented in Table 3 clearly verifies the validity of the model in simulating the response of these members.

A second numerical study was undertaken to analyze an AASHTO Type IV girder supporting a 7.5" slab. Analytical values for the deflection at midspan over a period of 700 days were obtained. The deflections predicted by the three creep and shrinkage procedures used in this study were found to differ significantly from each other. An excellent insight into the overall behavior of the system was obtained from this study.

A third numerical study was undertaken to evaluate the effectiveness of the analytical model to simulate actual bridge behavior by utilizing the experimental results for the bridge at Krotz Springs, Louisiana as a basis for comparison. Analyses were performed on the first three continuous units of the East approachway between expansion joints 1 and 2. The construction schedule was followed, and ambient temperatures were assumed based on averages for different seasons. Four days for which field monitoring data are available are October 22, 1987, February 21, 1988, April 15, 1988 and June 10, 1988. Particular attention was given to these days and analytical output was sought every four hours beginning at 8 a.m. The graphs for each day show the measured

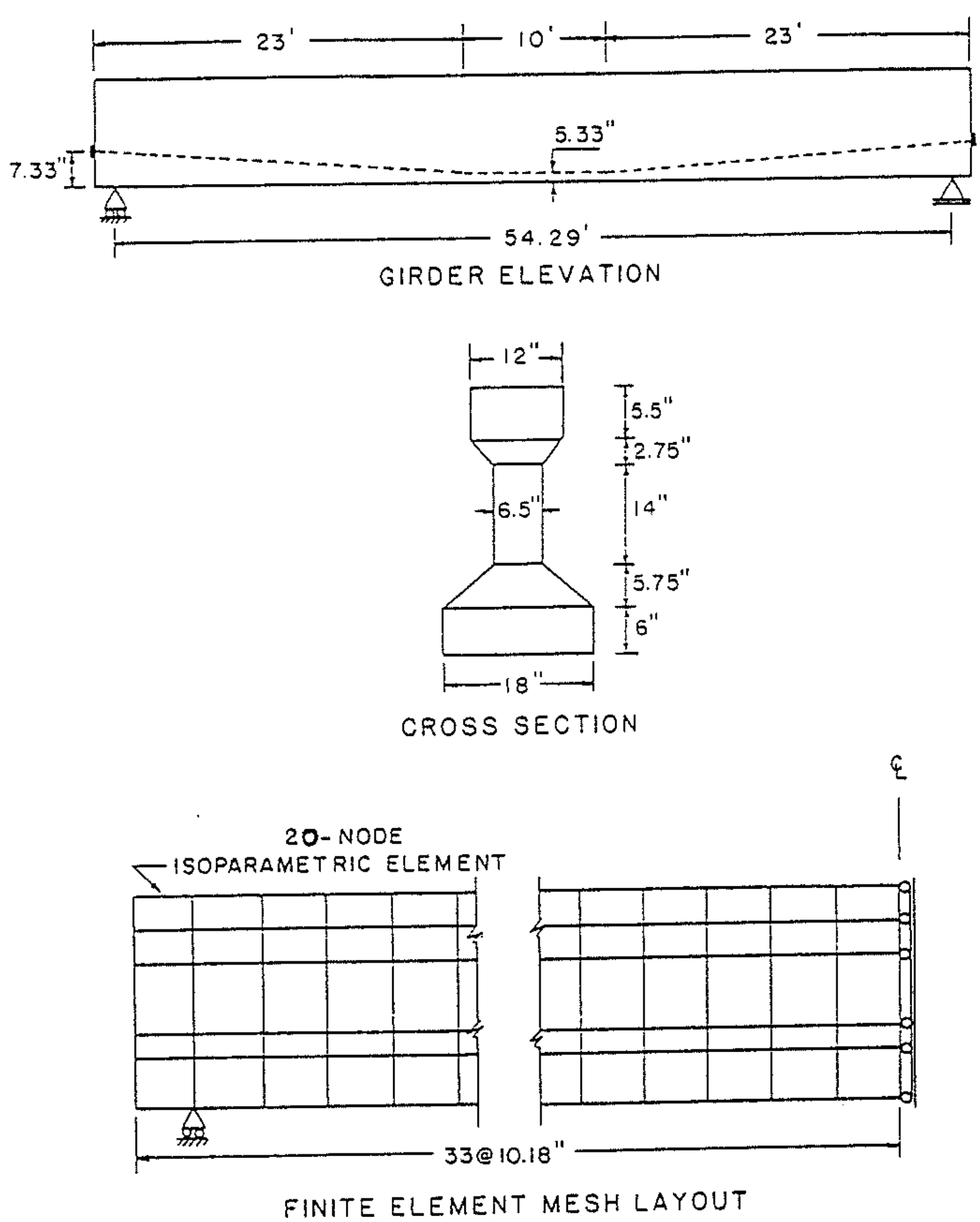

FIGURE 23 Sinno-Furr Girder: Elevation, Cross section and Mesh Configuration

TABLE 3 Comparisons with Sinno-Furr Experiment

Midspan Camber (inches)									
Days After Release	Meas-ured	ACI	% Error	BP2	% Error	CEB	% Error	Sinno -Furr Expres-sions	% Error
0	1.33	1.34	+0.75	1.33	0.0	1.31	-1.50	1.33	0.0
10	1.82	1.81	-0.55	1.71	-6.3	1.81	-0.55	1.80	-1.0
30	1.98	1.93	-2.5	1.90	-4.0	1.98	0.0	2.01	+1.5
90	2.06	2.00	-2.9	2.12	+2.9	2.01	-2.4	2.21	+7.3
300	2.11	2.09	-0.95	2.30	+9.0	2.04	-3.3	2.28	+8.0
Prestress Losses (% of initial)									
0	11.73	11.85	+1.0	11.77	+0.3	12.54	+6.9	12.50	+6.6
10	18.57	17.50	-5.8	17.11	-7.8	17.78	-4.25	18.60	+0.2
30	20.43	20.93	+2.4	22.05	+7.9	20.19	-1.2	21.25	+4.0
90	21.03	23.90	+13.6	24.83	+18.0	21.27	+1.2	22.46	+6.7
300	22.65	26.65	+17.7	28.01	+23.7	22.83	+0.8	24.15	+6.6

movement and the outcome of analyses using the three creep and shrinkage procedures. The trend of the intra-day predictions and measurements indicate that the bridge responds primarily to temperature variations. The profiles obtained from field measurements indicate that the assumed temperatures constitute an over simplification. Better theoretical responses would result if the profiles were based on the expressions for temperature variation compiled in the experimental phase of this project.

PARAMETRIC STUDIES AND RECOMMENDATIONS

The midspan deflection of the girder immediately after prestress release reflects the properties of the entire system and is an appropriate system response for use as a reference value for estimating joint movements. The determination of this midspan deflection is an easy task to perform. A parametric study concentrated on the prediction of joint movements based on initial girder deflection was conducted. Since most creep and shrinkage strains occur within the first two years of the life of the structure, the joint movement at the end of 2 years after prestress release, Δ_{JM} is expressed as follows:

$$\Delta_{JM} = 2 * C_{JM} * \Delta_i$$

where C_{JM} = coefficient of joint movement and Δ_i = initial girder deflection.

An extensive parametric study was conducted to investigate the influence of the two key bridge parameters on coefficient of the joint movement. Analyses were performed on two types of girder-slab systems representing a wide range of span lengths and numbers of continuous spans. The types chosen were systems with the AASHTO Type III and Type IV girders. The expressions for C_{JM} obtained for the girder-slab systems using the two different types of girder are given in Tables 4 and 5. The joint openings due to creep and shrinkage estimated using the LaDOTD procedure and the recommended procedure are presented in Table 6 for Type III and Type IV girders. It is clear from the table that the joint movements obtained using C_{JM} are consistently larger than those recommended by the LaDOTD. It is pertinent to make the following observations relative to the values presented in Table 6.

1. The LaDOTD specifications do not account for the influence of the girder cross sectional properties on joint movement.

2. The creep and shrinkage coefficient of 2.08×10^{-4} inches per inch of span length is empirical and valid for systems with certain girder types only.

3. The use of the coefficient C_{JM} represents a more rational procedure to estimate long-term creep and shrinkage movement at the joints.

TABLE 4 Expressions for CJM for Type III Girder-Slab Systems

No. of Spans	Creep Model	Proposed Expression	R-square
1	ACI	$C_{JM} = 1.839 - 0.04505L + 0.0002943L^2$	1.00
	BP2	$C_{JM} = 2.681 - 0.06486L + 0.0004200L^2$	1.00
	CEB	$C_{JM} = 1.149 - 0.02816L + 0.0001857L^2$	1.00
2	ACI	$C_{JM} = 2.932 - 0.07129L + 0.0004629L^2$	1.00
	BP2	$C_{JM} = 4.100 - 0.09861L + 0.0006371L^2$	1.00
	CEB	$C_{JM} = 2.074 - 0.04930L + 0.0003257L^2$	1.00
3	ACI	$C_{JM} = 4.196 - 0.10155L + 0.0006571L^2$	1.00
	BP2	$C_{JM} = 6.341 - 0.15508L + 0.0010143L^2$	1.00
	CEB	$C_{JM} = 3.057 - 0.07325L + 0.0004771L^2$	1.00
4	ACI	$C_{JM} = 5.616 - 0.13623L + 0.0008829L^2$	1.00
	BP2	$C_{JM} = 7.297 - 0.17011L + 0.0010686L^2$	0.99
	CEB	$C_{JM} = 4.076 - 0.09777L + 0.0006371L^2$	1.00

TABLE 5 Joint Openings due to Creep and Shrinkage (Span Length = 70')

Number of Continuous Spans	System using Girder Type	Initial Girder Defl-ection (inches)	Joint Opening (inches)			
			LaDOTD Proc-edure	Current Procedure using		
				ACI Model	BP2 Model	CEB Model
1	III	0.96	0.18	0.21	0.36	0.17
	IV	0.34	0.18	0.20	0.42	0.18
2	III	0.96	0.35	0.41	0.62	0.34
	IV	0.34	0.35	0.36	0.55	0.28
3	III	0.96	0.53	0.55	0.75	0.52
	IV	0.34	0.53	0.54	0.72	0.45
4	III	0.96	0.70	0.79	1.03	0.68
	IV	0.34	0.70	0.71	1.02	0.62
5	III	0.96	0.88	0.98	1.26	0.87
	IV	0.34	0.88	0.88	1.26	0.76

TABLE 6 Expressions for CJM for Type IV Girder-Slab Systems

No. of Spans	Creep Model	Proposed Expression	R-square
1	ACI BP2 CEB	$C_{JM} = 2.765 - 0.05377L + 0.0002750L^2$ $C_{JM} = 4.991 - 0.09558L + 0.0004843L^2$ $C_{JM} = 1.928 - 0.03753L + 0.0001921L^2$	1.00 1.00 1.00
2	ACI BP2 CEB	$C_{JM} = 4.151 - 0.07913L + 0.0003971L^2$ $C_{JM} = 6.025 - 0.11574L + 0.0005879L^2$ $C_{JM} = 3.331 - 0.06371L + 0.0003236L^2$	0.99 1.00 0.98
3	ACI BP2 CEB	$C_{JM} = 6.212 - 0.11911L + 0.0005986L^2$ $C_{JM} = 11.06 - 0.22029L + 0.0011450L^2$ $C_{JM} = 4.850 - 0.09231L + 0.0004671L^2$	0.98 0.99 0.98
4	ACI BP2 CEB	$C_{JM} = 7.485 - 0.14115L + 0.0007014L^2$ $C_{JM} = 12.80 - 0.25129L + 0.0021957L^2$ $C_{JM} = 6.391 - 0.12140L + 0.0006136L^2$	0.99 0.99 0.99

4. The values of joint movement, calculated using the coefficient C_{JM}, depend on the initial girder deflection, thus permitting the estimation of maximum joint openings at the design stage.

Comparison of Thermal Effects

The LaDOTD recommends that joint movements due to temperature be calculated by applying a coefficient of thermal expansion of 6 x 10^{-6} to the length of single span or continuous span bridge systems, for a temperature change of 70°F. The implicit assumption here is that the bridge experiences a constant temperature through the depth of its cross section.

In order to gain an understanding of the effects of different temperature profiles on joint movements, a simple study was conducted. A girder-slab system with two continuous spans 85 feet long with Type IV girders were chosen. Two bridge temperature profiles based on the ambient temperature were considered. The first profile (P1) is based on the recommendations of the Committee on Loads and Forces on Bridges, and the second profile (P2) is based on the results of the experimental study. The joint movements predicted by the two profiles over a 700 day period were found to differ by a small amount. The joint movements calculated using the profile (P2) based on experimental results and the LaDOTD procedure and compared in Table 7. The results presented in Table 7 shows that the joint movements predicted by the experimental temperature profile are approximately 15 percent higher than of that estimated by the LaDOTD procedure.

Recommendations for Estimating Joint Movements

A rational procedure for computing joint openings due to creep, shrinkage and temperature is recommended based on the results of the theoretical and experimental investigation. The following steps are involved in implementing the procedure.

1. Calculate the joint opening, Δ_{temp} caused by a temperature change of 70°F using the current LaDOTD procedure. This corresponds to an initial joint dimension to accommodate a temperature rise of 30°F and a joint opening because of temperature fall of 40°F. Increase the movements so obtained by 15 percent.

2. Calculate an average C_{JM} corresponding to the three models for the type of girder and number of continuous spans, using the expressions from Tables 4 and 6.

3. Calculate the initial deflection, Δ_i, of the girder.

4. Calculate the joint opening due to creep and shrinkage as

$$\Delta_{creep+shrinkage} = 2 * C_{JM} * \Delta_i$$

TABLE 7 Comparison of Movements using Profile P2 and the LaDOTD Procedure

Initial Temp. (°F)	Final Temp. (°F)	(i) Movements using profile P2[1] (inches)	(ii) Movements using LaDOTD procedure (inches)	Percentage (ii)/(i)×100
86	77	-0.28	-0.24	86
50	77	-0.54	-0.48	88
72	32	+0.40	+0.33	83
32	92	+0.86	+0.73	85

1. P2: Profile obtained from measurements on the Krotz Springs Bridge

5. Determine the joint movement due to creep, shrinkage and temperature as:

$$\Delta_{cst} = 1.15 * \Delta_{temp} + 2\ C_{JM} * \Delta_i$$

The actual maximum joint opening is then Δ_{cst} + the initial minimum joint dimension at the time of joint installation.

6. Check if the maximum joint opening, Δ_{cst} + minimum joint dimension, is less than four inches. The maximum opening that can be accommodated by strip seals used in Louisiana is four inches. If the check fails, a new design of the system is required.

CONCLUSIONS

Based on the rigorous experimental investigation of bridge deck joint movements in the approach spans of the Atchafalaya River Crossing at Krotz Springs, Louisiana, and the parallel analytical study of these movements, the following conclusions are drawn:

1. The primary causes of movements in the bridge decks were due to thermal effects. Since most instrumentation was not in place until 9 months after span construction, creep and shrinkage effects could not be monitored experimentally. The range of movements over the 21 months of monitoring with LVDTs were on the order of 0.5 to 1.4 inches depending on the joint location. Expansion joints at steel-to-concrete girder locations experienced approximately twice the movements of the concrete-to-concrete girder joints.

2. A comparison of actual joint movements with those estimated by the current LaDOTD procedures did not indicate a consistent pattern. In some cases the LaDOTD procedures over-estimated movements while in other cases the movements were under-estimated.

3. In the current LaDOTD recommendations, joint movements because of temperature changes are calculated by applying a linear coefficient of thermal expansion to the total span length. These calculations do not account for actual temperature distributions on bridge cross sections. Refined analyses using realistic bridge temperature profiles are likely to produce a method for evaluating joint movements due to temperature changes that would be a significant improvement on the LaDOTD method. A simple study showed that the LaDOTD procedure underestimates movements due to temperature by about 15 percent.

4. Measurements with LVDTs proved to be the appropriate method for investigating joint movements. Theodolite measurements had limited value and proved inefficient.

5. The analytical procedure developed in this study predicts accurately the

response of bridges with precast, pretensioned girders composite with cast-in-place concrete slabs under both short-term and long-term loads.

6. The results of the experimental study revealed the presence of restraining effects at the expansion joint bearing pads. Even with state of the art design and construction practices, proven stress-free expansion joint devices have not been developed. Stresses built up at these "roller" supports due to thermal expansion and contraction were suddenly relieved when a certain stress level was reached or when an external force was applied. An example of this behavior was demonstrated when the release of thermal stress built up at the pins of the steel truss river crossing span caused shock waves in the structure and aided in relieving stress built up at the joint supports. This behavior was also seen during one of the days of traffic usage.

7. The bridge sections experienced unsymmetrical joint movements with the north side displaying larger movements. This unsymmetrical deformation can be attributed to restraints associated with the neoprene bearing pads. Measurements showed that the bridge temperatures on the north and south sides of the bridge were similar and thus did not contribute to the unsymmetrical deformation. This pattern further supports the previous conclusion that significant restraints exist at the "roller"

8. The bridge underwent non-reversible joint movements. It was observed that in some cases the bridge sections did not bounce back to their initial positions as temperatures rose and fell to their initial values. This behavior was evident over the 24 hour monitoring cycles as well as over the long-term seasonal period. The non-reversible movements are attributed to the restraining effects present at the "roller" supports. There was no consistent pattern in this behavior further substantiating the preceding two conclusions.

9. Although non-reversible behavior was observed, a general seasonal repetitiveness of joint movement behavior occurred, which was in agreement with the seasonal temperature trends.

10. The bridge sections showed no signs of rigid body translation. There was no tendency of the bridge to move downhill over time.

11. Bents under expansion joints responded to, but did not contribute to joint movements. The bents experienced negligible vertical movements and small rotations. In addition, the maximum longitudinal movements of the bents were smaller than the movements of the girders which indicates that the bents were moving along with the girders during thermal expansion and contraction.

12. Prestress losses in pretensioned girders were predicted by incorporating the

prestressing steel as being completely embedded in the concrete. The deformations of both concrete and steel were determined at any time in one complete analysis over a time domain and under loading at different stages.

13. Typical construction procedures and schedules can be modelled in the analysis. The analysis accounts for the type of concrete used for girders and slabs, as well as the type and length of concrete curing. The analytical applications indicated that the effects of creep and shrinkage on girder deflection become insignificant after a period of three months.

14. The choice of the prediction method for creep and shrinkage strains used in an analysis affects the outcome. In general, extreme joint movements were obtained when the BP2 model was used for the creep and shrinkage analysis. Deformations in bridges, after the slab is cast, are largely affected by the differences in the rates of shrinkage in girder and slab concrete.

15. Based on the measured distribution of temperatures through the depth of the bridge sections, a model to predict this distribution was developed. The model relates the temperatures at the top and bottom of slab as well as the girder temperatures to ambient temperatures. The distribution of temperatures through the slab varied significantly but little variation was observed through the depth of the girders. The model provides an adequate description of thermal profiles through the depth of the slab and girder.

16. The data acquired over the nine month period after the bridge was opened to traffic indicated no discernible effects due to traffic loads. However, to more fully evaluate these effects, monitoring over a longer period of time is required.

17. The comparisons of theoretical joint movements with measured values for the bridge at Krotz Springs, Louisiana, indicate that differences in these values depend on the choice of creep model. The ACI-209 procedure showed the closest agreement. The analytical procedure was not utilized to predict movements caused by support restraints observed in the field and which were not part of the design. The experimental results and the theoretical analyses showed clearly that after a period of one year, temperature induced strains dominate the deformation behavior.

18. A method for estimating the maximum bridge deck joint movements has been recommended based on the results of an extensive parametric study. The method is easy to apply and takes into account the effects of bridge geometry and material properties. The use of the recommended procedure will permit the designer to determine the span lengths and the maximum number of continuous spans between expansion joints, knowing the limit of movement that can be accommodated by the joint-sealing system chosen.

19. The analytical studies on bridge systems indicate that those using medium-strength concrete girders tend to have significantly larger joint movements as compared with systems using normal-strength concrete girders. This is due to the increased differential shrinkage strains between girder and slab. If the magnitudes of joint movements are to be held within limits, higher girder concrete strength should be accompanied by a corresponding increase in slab concrete strength.

20. The analysis program provides the bridge designer with a powerful tool to evaluate the long-term behavior of bridge structures with different support conditions, and with or without joints. The analytical model is also capable of accounting for support stiffnesses and approach skew.

21. The magnitude of the tasks completed in the three year duration of the project coupled with the wide variety of joint damage reported in literature and associated with different joint sealing systems did not permit the development of appropriate repair alternatives and specific remedial procedures.