

Field Study of Longitudinal Movements in Composite Bridges

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Bridge deck expansion joints 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 about the behavior of specific joint sealing systems have been contradictory, indicating that the problems may not be inherent to the particular system. Rather, the problems may stem from a failure to properly access 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 investigation was conducted to obtain thermally induced movements of a newly constructed bridge in central Louisiana. The instrumentation, field monitoring, and analysis of long-term longitudinal movements are described. The primary causes of movements obtained were thermal changes. The bridge experienced unsymmetrical and irreversible movements, and these were attributed to restraints associated with the neoprene-bearing pads at the expansion joints. The bent movements and the effects of traffic were small compared with the thermal movements.

Highway bridges generally require expansion joints between sections of the deck or at the approach roadway. The standard practice is to specify a sealed joint to prevent debris and water from passing through the joint and causing deterioration of the bridge. Frequently the joint seals have leaked, ruptured, or fallen out of position. Once the seals fail, debris can lodge within the joint and road salts, can penetrate the failed seals, causing deterioration of the structural components. In short, joint seals have proved a continual and expensive maintenance problem for highway departments. Because various states have reported contradictory performance of specific joint sealing systems, the problems may not be inherent to the systems. Rather, the problems may stem from improper design criteria, poor installation practices, differences in bridge type or environmental conditions, and failure to determine actual joint movements.

To assess the importance of these factors, an experimental study was conducted on a newly constructed bridge located in central Louisiana to determine longitudinal movements. The purpose of this paper is to describe the field monitoring procedures and report on the general behavior of the bridge and its movements.

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 joints for Louisiana highway bridges (1) is based on elementary strength

of materials formulas, which may not accurately predict the joint movements. Systematic, detailed studies are required to properly assess actual joint movements, which will lead to the development of rational design methodologies for joints in modern bridges.

RELATED STUDIES

In a broad sense, highway bridge joints can be classified as open or closed (waterproof). Common types of open joints are plate bearing, butt, or toothed. Closed joints are composed of compression seals, membrane seals, or cushion seals. Purvis and Berger (2) give a brief description of joint seals along with their applications and associated problems. Several studies have been conducted to develop the best joint sealing system that would minimize bridge joint problems (3-7). These studies focused primarily on the performance specifications and evaluation of bridge joint systems.

The most significant bridge movements and the ones that by far cause most of the joint seal problems are the longitudinal cross-the-joint thermal movements. Reynolds and Emanuel (8) have written a concise summary of prevalent research conducted in this area between 1957 and 1970. They concluded that relating environmental conditions to bridge movements is extremely complex. Dillon and Kissane (9) summarized the movements of prestressed concrete girders located throughout New York State over a 2-year period. Abdul-Ahad (10) developed a theoretical method of calculating thermally induced stresses and movements in continuous bridge structures. The experimental and analytical results were close; however, the experimental data were limited and no generalized conclusions could be drawn. Moulton and Kula (11) analyzed pier and abutment movement data obtained from 180 bridges through questionnaires. The surveys suggested that abutment movements occurred more frequently than pier movements and that horizontal movements caused much greater damage than vertical movements.

Mortlock (12) investigated various types of instruments used to obtain bridge movements. He concluded that the following should be used: (a) copper constantin thermocouples to obtain the temperature variation through the slab depth; (b) linear variable differential transformers (LVDTs) to measure the joint movements; and (c) a Kipp solarimeter to measure the solar radiation of the slab. Emerson (13) used a combination of these devices on seven bridges located in England.

OBJECTIVES AND SCOPE

The purpose of this paper is to describe the experimental procedures of instrumentation and monitoring and to discuss the general behavioral characteristics of the bridge with respect to long-term move-

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ments. Reported here are also the results of the systematic study of the bridge joint movements. The reader is referred to Pentas et al. (14) for an analysis of the bridge temperatures and thermal distributions. The study was focused on a newly constructed bridge on US-190 over the Atchafalaya River at Krotz Springs, Louisiana. Specific objectives of this research were to

1. Instrument the designated bridge for field monitoring using LVDTs, thermocouples, and optical devices;
2. Field monitor the appropriate bridge movements through a program of instrumentation and periodic measurement;
3. Analyze the experimental data obtained and evaluate the bridge joint movements;
4. Compare the experimental data with current procedures predicting longitudinal bridge movements.

BRIDGE DESCRIPTION

The bridge to be investigated is the east approach of US-190 over the Atchafalaya River at Krotz Springs, Louisiana. It consists of cast-in-place concrete slabs acting compositely with either Type IV AASHTO prestressed concrete girders or steel 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 bridge movements. Joints 1 through 4 are mem-

brane seals, whereas Joint 5 is a toothed type. The bridge continues over the river as a steel through truss.

Unit 2 is the longest single span of the approach at 42.7 m. (140 ft). It consists of a cast-in-place slab 21.6 cm (8-1/2 in.) thick slab acting compositely with four steel plate girders 183 cm (72 in.) deep. The other three sections of the approach (Units 1,3, and 4) consist of a slab 19 cm (7 1/2 in.) thick acting compositely with five Type IV AASHTO prestressed concrete girders.

The supporting Bents 2 through 5 consist of concrete caps poured at the top of precast concrete piles 194 cm² (30 in.²). Bents 2 and 3 each have four precast concrete piles supporting a level cap. Bents 4 and 5 each have five piles supporting the cap. The cap is stepped to allow the top of the steel girders to match flush at the same level as the top of the concrete girders. Bents 6 through 11 consist of level concrete caps supported by two concrete columns 137 cm (54 in.) in diameter. Bent 12 consists of two concrete columns 76 cm (30 in.) in diameter anchored to a bridge pier that also supports the end rocker bearings of the river crossing truss.

At continuous joints, the girders were connected to the bent cap by imbedding a dowel into the cap extending into the continuous joint. At some expansion joint locations, the girders were pinned to the bent cap, whereas at others the girders were allowed to slide. Pinned joint connections are denoted by the letter F, whereas joints allowed to move are denoted by the letter E, as shown in Figure 1. The ends of the girders at the expansion joints and at the continuous joints over the bents were placed on neoprene bearing pads of the standard type used in Louisiana. The reader is referred elsewhere (15) for more information and bridge design details.

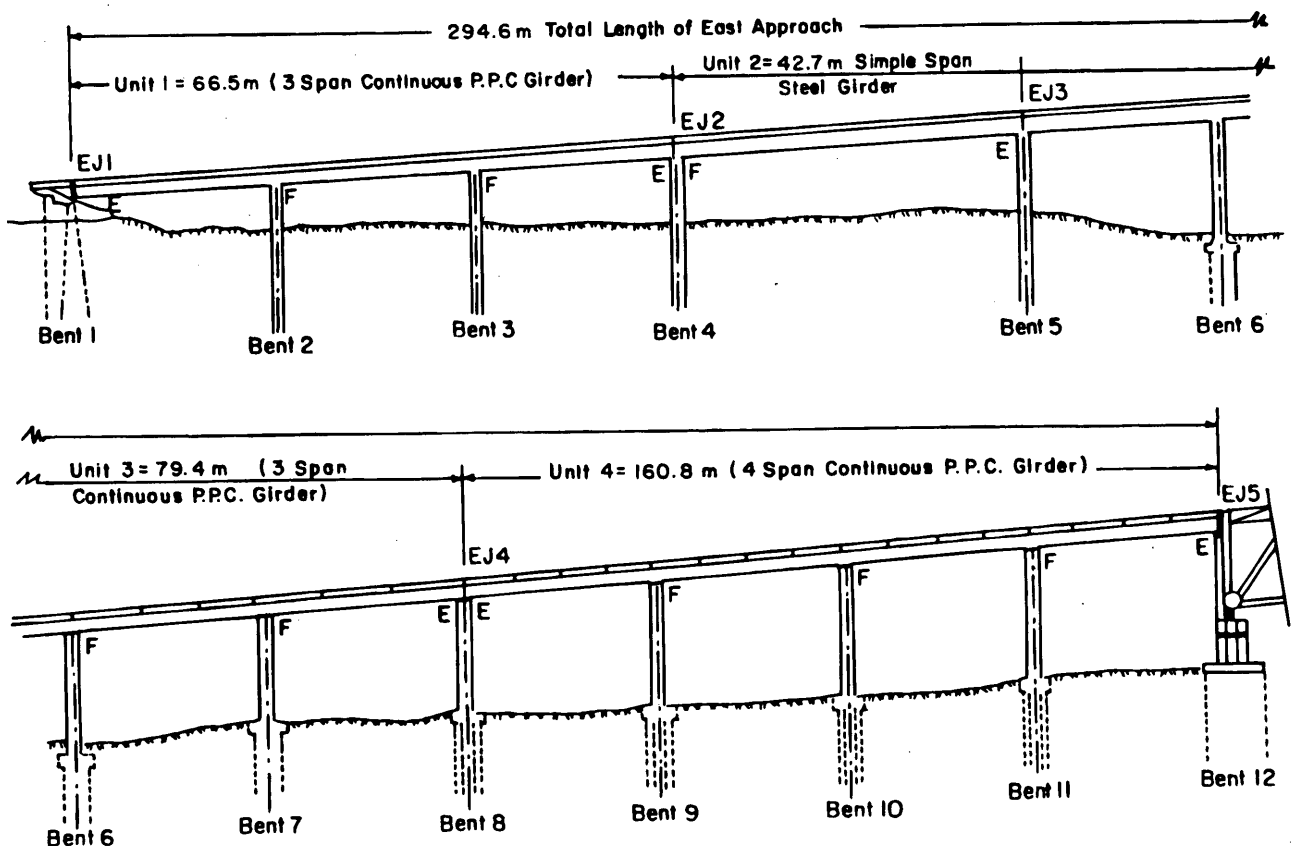


FIGURE 1 North elevation of east approach, US-90 bridge at Krotz Springs, Louisiana.

The bridge was already under construction at the beginning of this research (October 1986). The bents had been erected and the girders were already in place. It was during that period of construction when the first instrumentation was installed. At that time the decks were also constructed. On October 27, 1988, construction was completed and the bridge was opened to traffic.

BRIDGE MEASUREMENTS AND INSTRUMENTATION

Measurements were taken near the top and bottom of the girders at the expansion joints to obtain the relative longitudinal movements between the two adjoining girder sections. Measurements were also taken between the bent cap and one of the sections at the expansion joints, to obtain the movement of each section with respect to the cap. The sway of the bents at the expansion joints was also measured. The temperatures through the depth of the sections and ambient temperatures also were measured. The time was also recorded, thereby giving a time reference.

LVDTs were chosen to obtain the joint movements. A theodolite was chosen to obtain the bent sway, and thermocouples were used to measure the temperatures. 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 that would store the readings for later processing. Electrical power was supplied through a portable generator. The theodolite readings were taken and recorded in a field book and later transcribed into the computer for processing.

Combined with a data acquisition system and a microcomputer, all LVDTs placed on the bridge could be read nearly simultaneously. Furthermore, the rugged construction of the LVDTs permitted them to function properly even after exposure to substantial shock loads. The LVDTs, however, could be used only to establish local relative movements of the girders at the expansion joints. In the case of the Krotz Springs Bridge, the LVDTs were used to obtain the 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). Because of construction delays

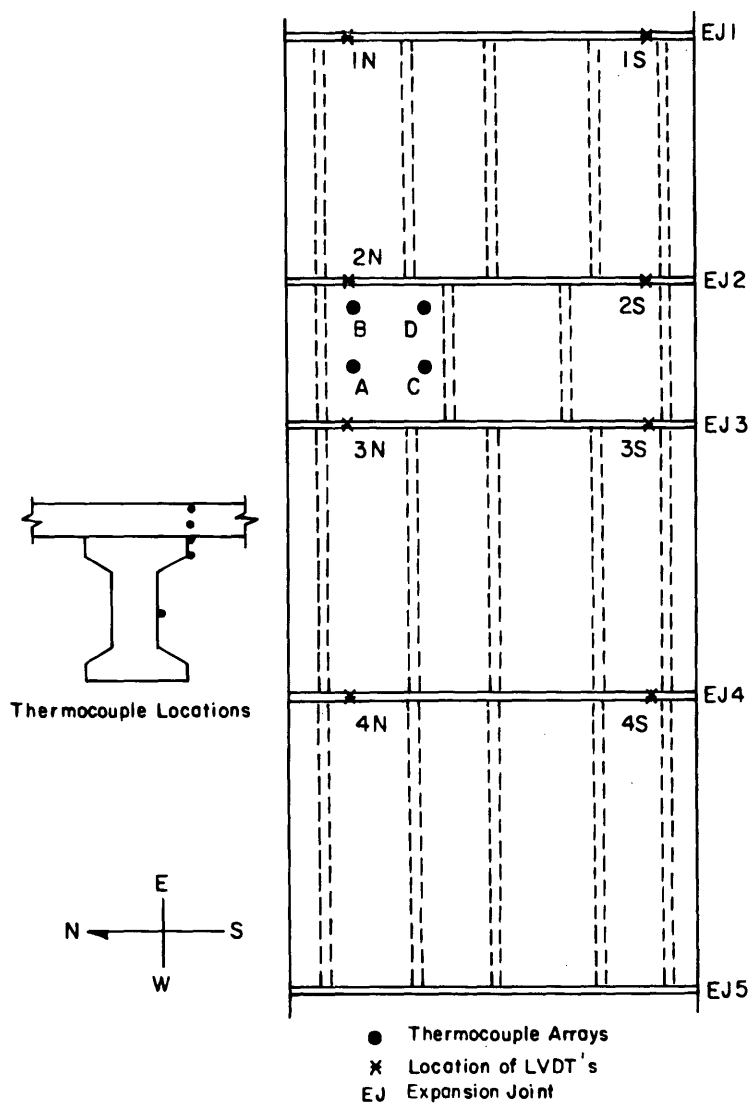


FIGURE 2 Plan view of bridge showing locations of LVDTs and thermocouples.

Joint 5 was not instrumented. The LVDTs were placed at the inner sides of the exterior girders to protect them from the outer environment. They were mounted on aluminum brackets and attached on the flanges of the girders using epoxy.

A side view of a typical section at an expansion joint is shown in Figure 3. An arbitrary positive displacement is denoted by the dashed line. LVDT A was placed near the top of the girder at a distance a from the neutral axis. The body was secured to the girder, and the core was fixed to an angle iron anchored vertically on the bent cap. LVDT B was placed near the bottom of the girder at a distance b from the neutral axis in a similar manner. LVDTs C and D had their bodies secured to the westward section and their cores secured to the eastward section. The distances labeled DA, DB, DC, and DD are the readings recorded by LVDTs A, B, C, and D, respectively. The required movements were calculated using geometric relations. The movements at the other expansion joints were obtained in a similar manner. The assumption was made that the abutment would remain stationary and was later proved to be correct by theodolite measurements. It was therefore necessary to install only two LVDTs at the abutment to calculate the joint movements.

A Pentax total station theodolite was used to obtain the bent sway of the Krotz Springs Bridge. A setup point was constructed for each of the 12 bents. A central reference point was constructed on the levee to allow for visibility from all setup points. The setup and reference points are made up of cast-in-place concrete benchmarks

reinforced with three No. 4 bars. Each benchmark is 1.5 m (5 ft) in length with 1.2 m (4 ft) in the ground. The top of the setup points is marked by a brass plate embedded in the concrete.

Thermocouple wires type PP20TX were used to measure the temperatures of the Krotz Springs Bridge. They presented the following advantages: first, the temperature range was such that both ambient and slab temperatures could be accurately measured. Second, the thermocouples could be connected to the data acquisition system, allowing all temperatures to be measured at the same time as LVDT measurements. Finally, the thermocouple wire was fairly inexpensive, and preparation of the wire was very simple. The thermocouples were placed along the depth of the sections to detect the temperature variation. Each array consists of six thermocouples located on both slab and girder, as shown in Figure 2. The slab thermocouples were placed near the top, center, and bottom of the slab at the time of pouring. The girder thermocouples bonded on the outer surface of the concrete girders using epoxy and a layer of hydraulic cement. Two additional thermocouples were placed hanging under the slab to record the ambient temperature. All thermocouples were run under the bridge to the data acquisition system at the monitoring station.

BRIDGE MONITORING SCHEDULE

The theodolite readings began on January 1987. Each full set of readings required approximately 5 hr. As the effects of creep and

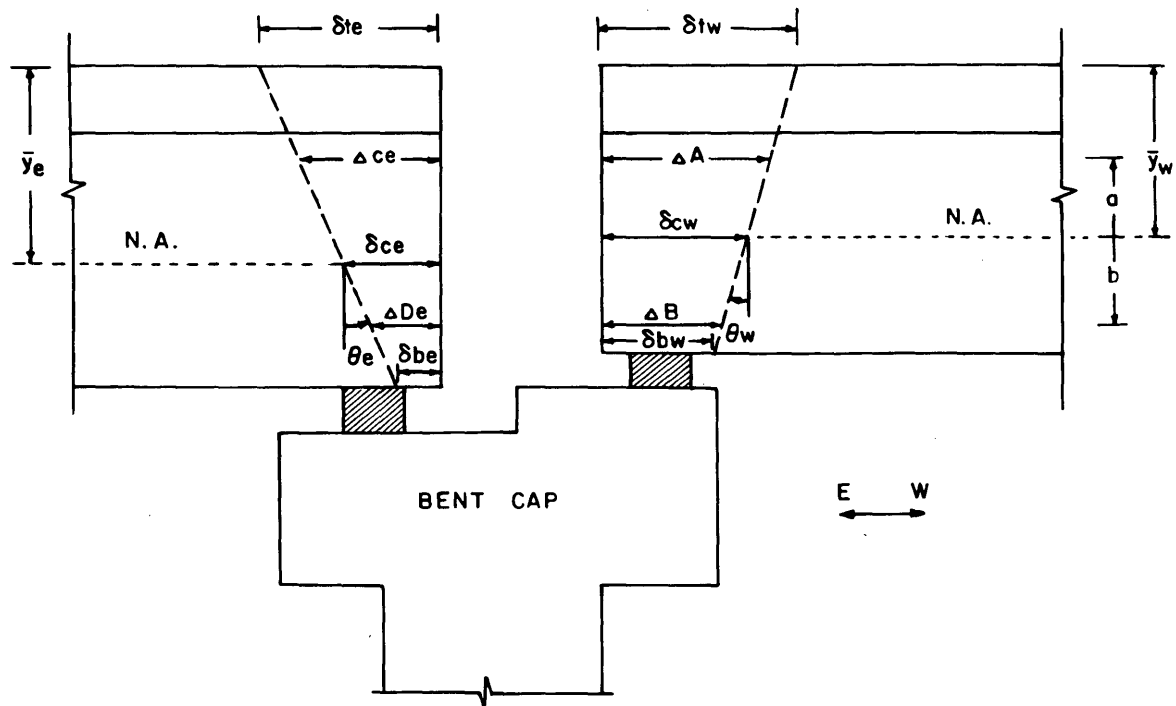


FIGURE 3 North elevation of typical expansion joint other than abutment, showing movements and LVDT measurements required to calculate movements.

shrinkage were anticipated to decrease, the frequency of data collection was gradually changed to a 6-week schedule. The LVDTs were on line at 8:00 a.m. on October 22, 1987. The LVDT readings were taken approximately every month. Alternate LVDT readings were taken for either 12 or 24 hr continuously. The thermocouple readings were recorded at the same time as the LVDT readings. During the days of data collection with the LVDTs, the theodolite was also used to obtain the sway of the bent caps at the expansion joints. 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. Also, one LVDT at location 4S was found to be defective, and the data collected at the joint were discarded.

MAXIMUM JOINT MOVEMENTS

A large amount of data was collected, but only the data required to evaluate the long-term expansion joint movements are presented in this paper. A complete analysis of the bridge temperatures is presented by Pentas et al. (14), in which a model to predict thermal distributions in bridges was developed.

The movements obtained were caused by dead loads and thermal changes only. Because the LVDTs were not in place until 9 months after the slabs were poured, creep and shrinkage effects had dissipated and could not be monitored. Traffic had not begun on the bridge until October 27, 1988; therefore the effects of traffic loads are not considered until after that time. The extreme values of movements recorded at the four expansion joints are summarized in Table 1. The following observations can be made:

1. The maximum closing of the top of the joint occurred during the warmer days of May 16 and June 10, 1988.
2. The maximum opening of the top of the joint occurred during the colder days of December 16, 1987, and February 21, March 17, and December 1, 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.

Possible factors affecting the inconsistencies of joint movement behavior include joints reaching maximum value allowed by mechanical connection; defective truss-bearing pins, construction crew and equipment; and bent movements, connection performance, and orientation of the bridge with respect to the sun path. No specific correlation related to these movements was identified. However, it is most likely that the build-up of stresses at the defective truss pins, as well as friction in the neoprene bearing pads, had the 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 obtained over the 9-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 joint movements were small compared to the effects of ther-

TABLE 1 Maximum Values of Expansion Joint Movements Obtained from LVDTs

Joint Location	Max. Closing (cm)	Date	Max. Opening (cm)	Date	Total Range (cm)	Ambient Temp. Differential (degrees C.)	Slab Temp. Differential (degrees C.)
1 North (1 South)	-1.78 (-0.38)	May 16	+0.25 (+0.13)	Dec 16	2.03 (0.51)	28	39
1 South (1 North)	-0.38 (-0.78)	May 16	+0.89 (-0.38)	Dec 01	1.27 (1.52)	17	28
2 North (2 South)	-1.52 (-1.78)	June 10	+0.89 (+1.40)	Mar 17	2.41 (3.18)	19	31
2 South (2 North)	-1.91 (-1.27)	May 16	+1.40 (+0.89)	Mar 17	3.30 (2.16)	22	33
3 North (3 South)	-2.03 (-1.27)	June 10	+1.52 (+0.25)	Feb 21	3.56 (1.52)	11	14
3 South (3 North)	-1.52 (-1.52)	May 16	+0.51 (+1.52)	Mar 17	2.03 (3.05)	22	33
4 North	-1.27	June 10	+0.127	Feb 21	1.40	19	31

2.54 cm = 1 inch

Numbers in () represent movement on opposite side of joint corresponding to the maximum value listed.

mal changes. However, to more fully evaluate these effects, monitoring over a more lengthy period is required.

DATA DISCONTINUITIES

The movements obtained from the LVDT readings showed some discontinuities. An examination of the instrumentation was conducted to ensure that these changes were not a result of a system operation error. All electronic impulses were filtered and surge protected, shielding the instruments from improper power fluctuations. The sudden changes were not present throughout the whole set of data on the particular day, indicating that electronic malfunction was not the cause of this abnormal behavior. The exact causes of these movements have not been determined; however a possible explanation might be the sudden release of stresses built up at the pins of the steel truss. Shock waves caused by release of stresses at the truss pins act as an external force causing the release of stresses built up at joint supports, which results in sudden movements. The

pins were replaced in January 1988, after which time the bridge movements did not show any discontinuities. 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.

ANALYSIS OF LONG-TERM JOINT MOVEMENTS

To identify long-term movements caused by temperature changes from other movements, the behavior of the bridge can be studied using the data obtained over the 24-hr monitoring days. Because of space limitations, only selected data are presented in this paper. The reader is referred elsewhere for more information (15). The long-term movements obtained from the LVDTs on October 22, 1987, and February 21, 1988, are shown schematically in Figures 4 through 8. Figures 5 and 7 show the movements obtained from the LVDTs located at the south side of the bridge, and Figure 8 shows the plan view movements of the deck units. Each of Figures 4 through 8 shows the movements of the bridge sections at the expan-

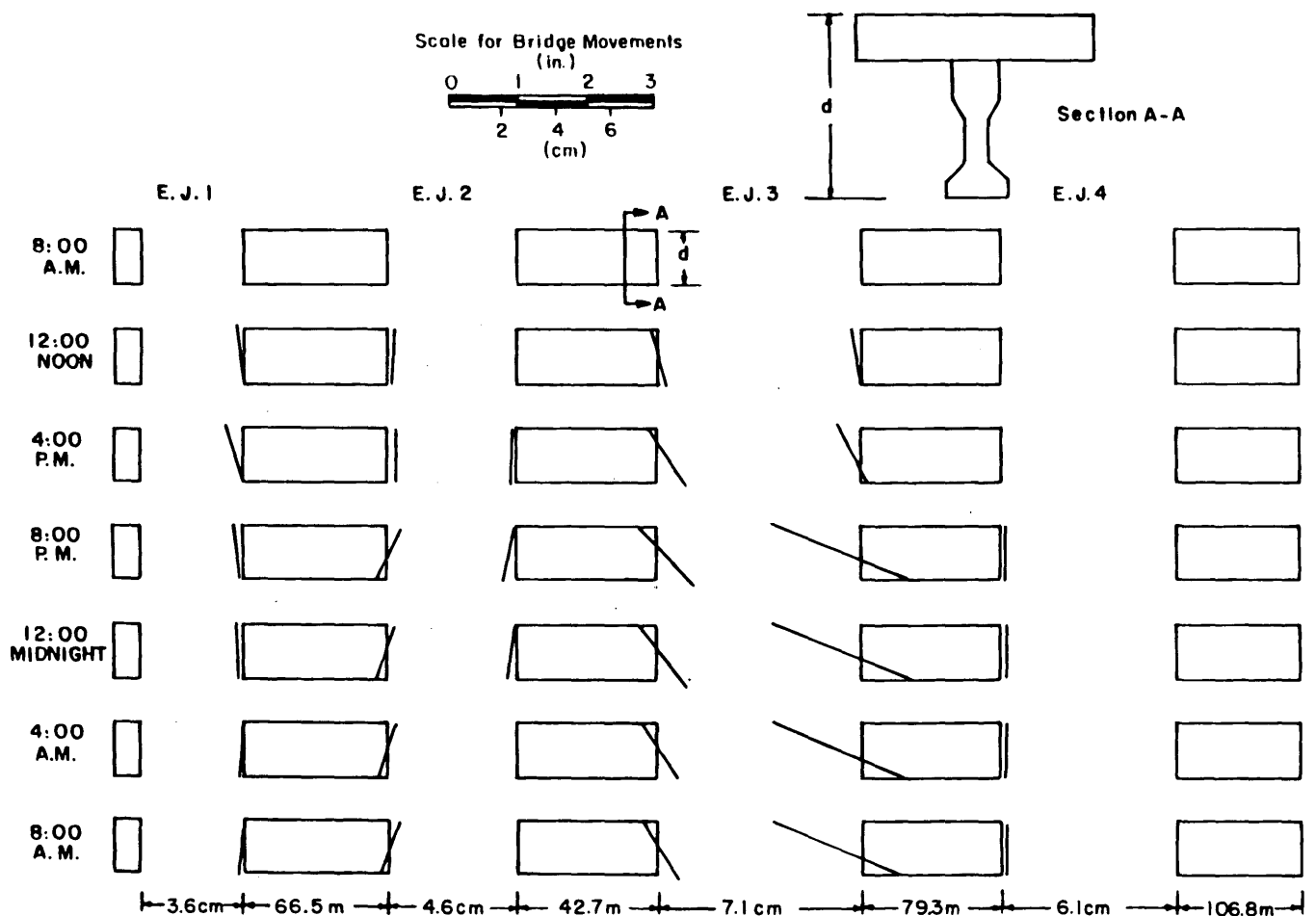


FIGURE 4 Elevation of north side girders showing 24-hr movements with respect to cap for October 22, 1987. (Note: Longitudinal dimensions of girders and joint spacings are not to scale, but actual end movements are to the scale indicated.)

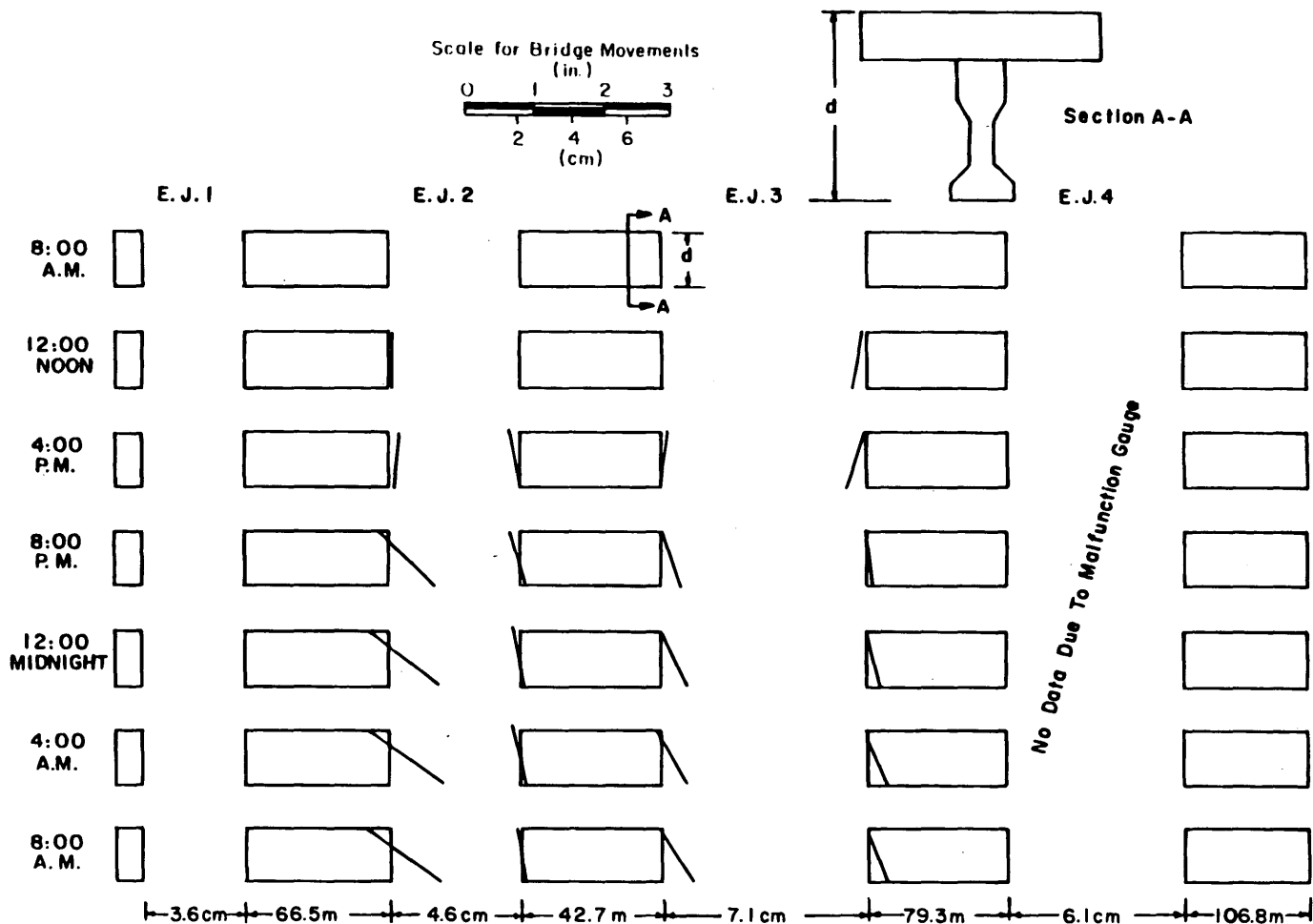


FIGURE 5 Elevation of south side girders showing 24-hr movements with respect to cap for October 22, 1987. (Note: Longitudinal dimensions of girders and joint spacings are not to scale, but actual end movements are to the scale indicated.)

sion joints for a 24-hr period. These movements are relative to the bent caps and are referenced to the first day of monitoring, which is October 22, 1987. The rectangles shown in Figures 4 through 8 represent, from left to right, the abutment and Bridge Units 1 through 4. The top row represents the initial position of the bridge units, and the subsequent rows represent the position of the bridge units at 4-hr intervals. The straight lines shown at the ends of each rectangle represent the movement of the unit for the given time. The scaled data shown in the figures can be easily used to compare with and verify the results of finite element programs or procedures predicting longitudinal bridge movements. The following observations can be made from 24-hr movements:

1. The bridge sections exhibit nonsymmetrical and nonreversible joint movements. This behavior can be attributed to restraints associated with the neoprene bearing pads;
2. There is a general seasonal repetitiveness of joint movements associated with seasonal temperature trends; and
3. There is no indication of rigid body translation.

The effects of support restraints on joint movements may be identified by comparing the joint movements obtained on two different days of similar bridge temperatures. The monitoring days of October 22, 1987, and February 21, 1988, were chosen for this comparison. The change in length at the top of the bridge units is obtained from the bridge movements shown in Figure 6 at 8:00 a.m. and adding the corresponding bent cap movements at each end of the units. Computations performed for Units 1 and 2 showed a shortening of 0.76 cm (0.3 in.) for Unit 1 and 0.51 cm (0.2 in.) for Unit 2.

The long-term movements of the expansion joints are summarized in Figure 9. The horizontal axis of the figure represents the time (day of monitoring). The vertical axis of the figure includes the ambient temperature and the movements of the expansion joints. The solid line represents the maximum opening of the joint, and the dashed line represents the minimum opening of the joint. The difference of the two lines represents the movement of the adjacent girders with respect to each other. The arrangement utilized in Figure 9 clearly shows the joint opening at each joint location relative to its position on October 22, 1987.

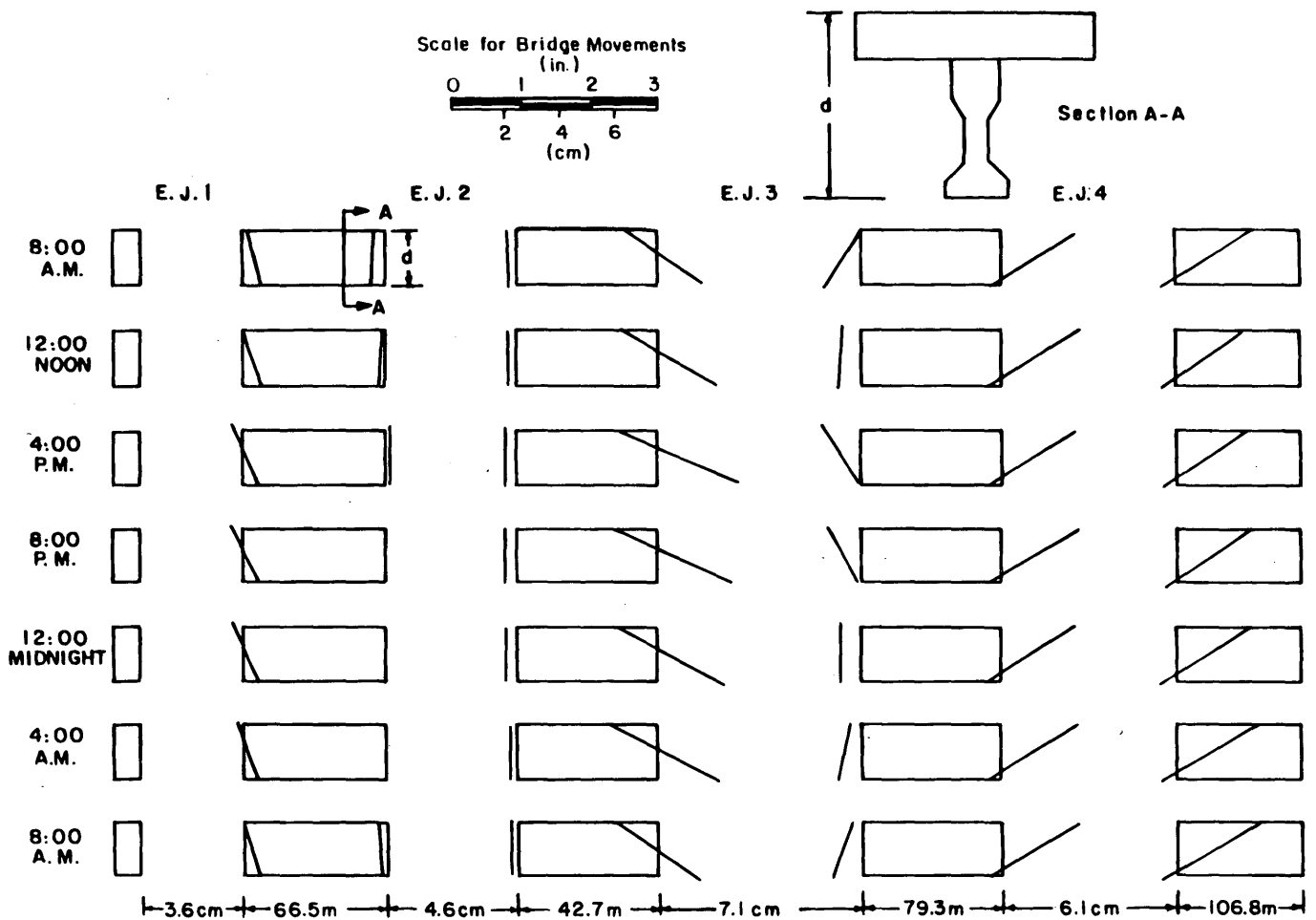


FIGURE 6 Elevation of north side girders showing 24-hr movements with respect to cap for February 21, 1988. (Note: Longitudinal dimensions of girders and joint spacings are not to scale, but actual end movements are to the scale indicated.)

The theodolite readings began early in the construction phase, before the LVDTs were placed on the bridge, to observe the long-term behavior of the bents. From the study of the bent behavior conducted the following were found: (a) The bents experienced negligible vertical movements; (b) the bent rotations were small and considered insignificant, and (c) the maximum longitudinal movements of the bents were smaller than the maximum movements of the girders.

COMPARISON OF MEASURED MOVEMENTS TO ESTIMATED MOVEMENTS

The 1983 AASHTO specifications provide guidelines for expansion of bridge members from temperature changes. These guide-

lines are generally adopted by the Louisiana Department of Transportation Bridge Design Manual, [LaDOTD (1)] where the design of expansion devices is based on bridge joint movements. The prediction of movement caused by 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 movement caused by creep and shrinkage is estimated by multiplying the shrinkage coefficient by the length of the member.

These criteria were applied to estimate the movements of the Krotz Springs Bridge. These estimated joint movements are tabulated and presented in Table 2 along with the measured joint movements given previously in Table 1. It can be seen from Table 2 that the measured movements at Expansion Joints 1 and 2 have either reached or exceeded the estimated values, although they were obtained at temperature ranges approximately 30 percent lower than the ones used for the estimated movements. The movements of

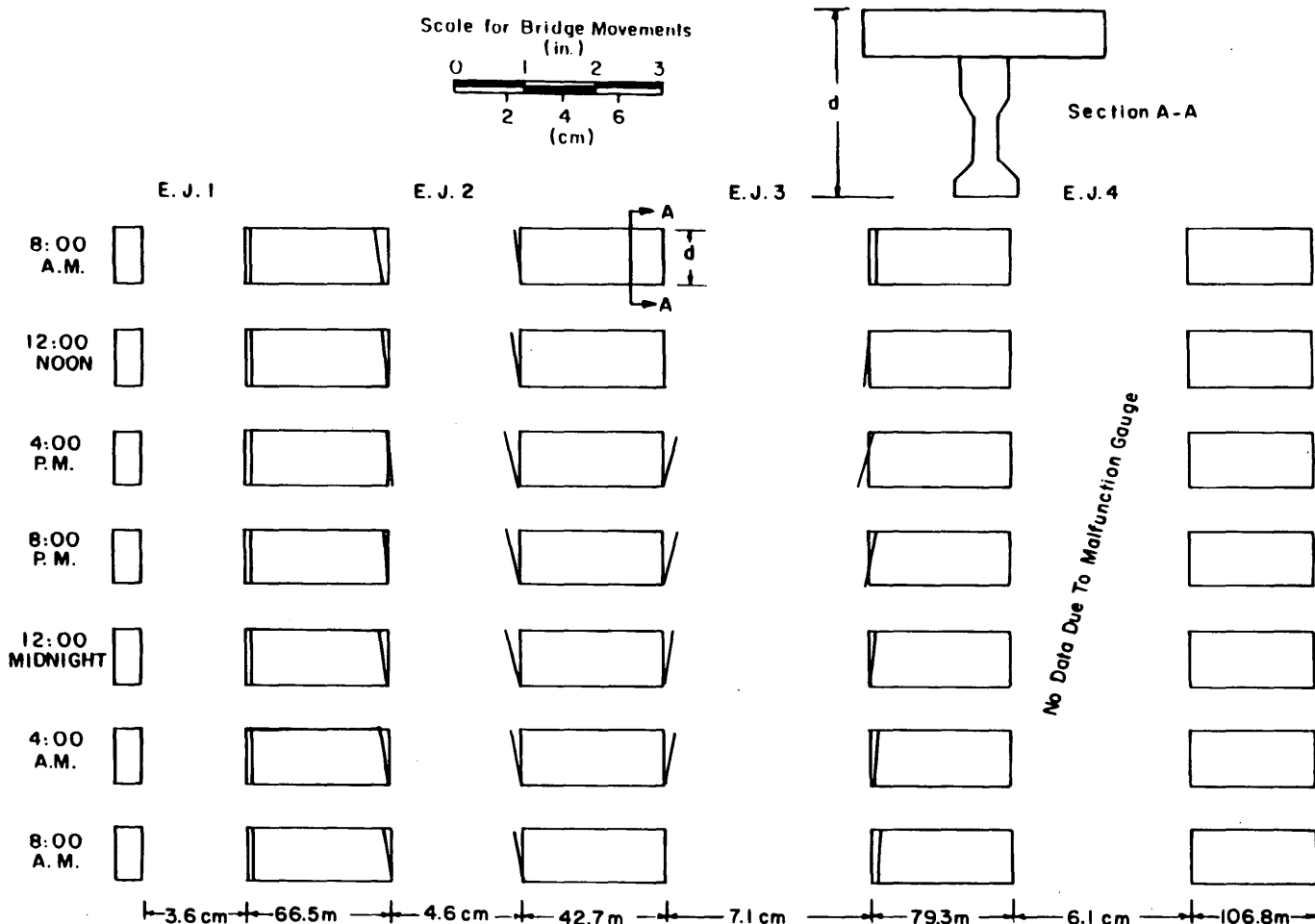


FIGURE 7 Elevation of south side girders showing 24-hr movements with respect to cap for February 21, 1988. (Note: Longitudinal dimensions of girders and joint spacings are not to scale, but actual end movements are to the scale indicated.)

Expansion Joints 3 and 4, however, are well below the estimated values.

CONCLUSIONS

A comprehensive experimental field study of longitudinal movements of bridge components was performed. Actual field monitoring extended over a 3-year period. Bridge movements were monitored from pouring of the bridge decks through 1 year of exposure to normal traffic. As a consequence bridge engineers have available to them data on the long-term longitudinal bridge behavior. These data can be utilized to test and verify computer models and procedures predicting longitudinal movements in bridges. The data gathered have significant implications on the future development of expansion joint design for bridges. In this regard the principal conclusions are as follows:

1. The primary causes of movements in the bridge decks obtained during the period of monitoring were caused by thermal effects. Since most instrumentation was not in place until 9 months after span construction, creep and shrinkage effects could not be monitored. The range of movements over the 21 months of monitoring with LVDTs was on the order of 1.3 to 3.6 cm (0.5 to 1.4 in.). Expansion joints at steel-to-concrete girder locations experienced approximately twice the movements of the concrete-to-concrete girder joints.
2. The results of the experimental study revealed the presence of restraining effects at the expansion joint supports. Stresses built up at the neoprene bearing pads as a result of thermal expansion 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 stresses built up at the pins of the steel truss river crossing section caused shock waves in the structure and aided in relieving stresses built up at the joint supports. This behavior was also seen during one of the days of traffic usage.

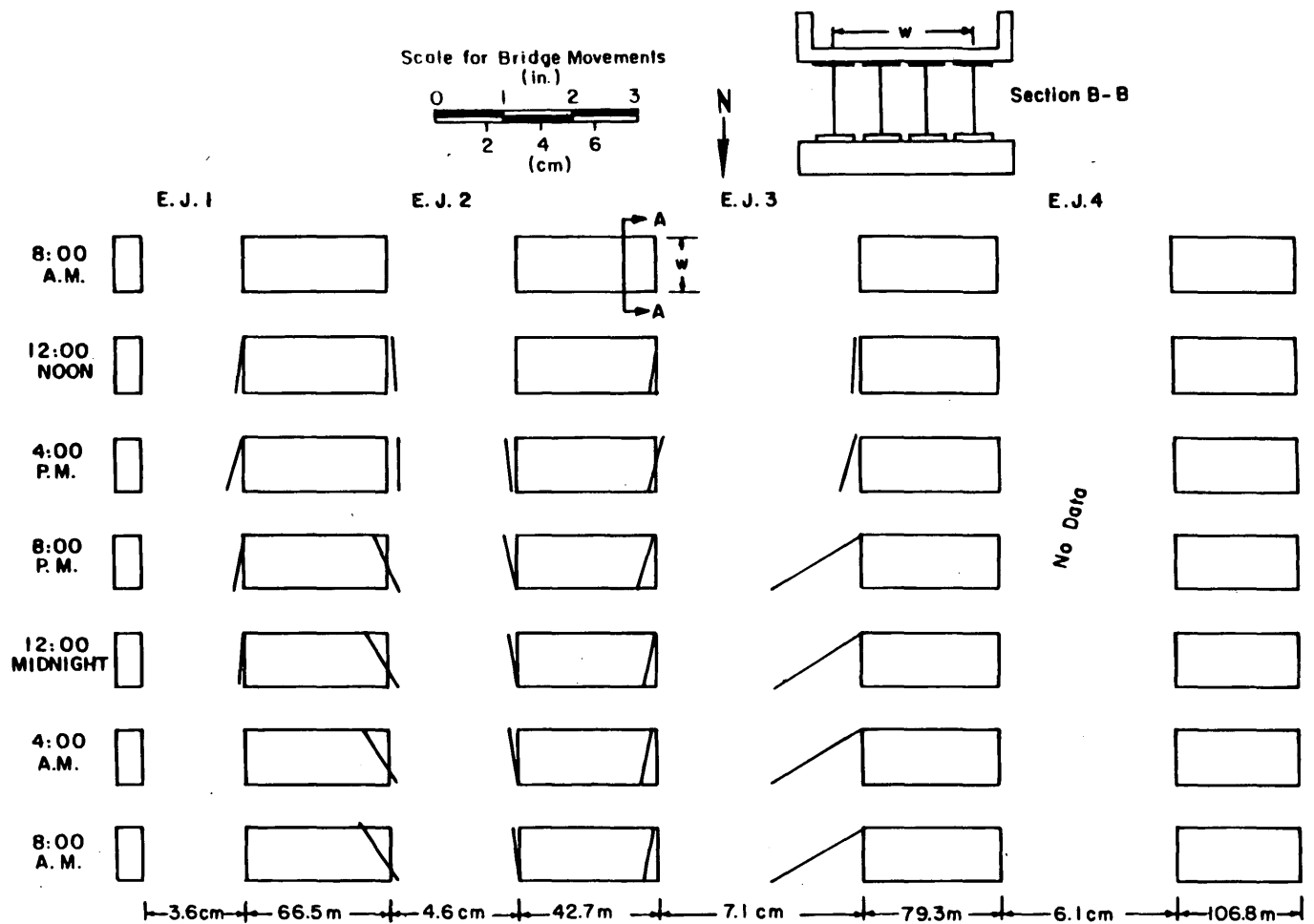


FIGURE 8 Plan view of bridge showing 24-hr movements of deck units with respect to cap for October 22, 1987. (Note: Longitudinal dimensions of girders and joints are not to scale, but actual joint displacements are to scale as shown.)

3. 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 joint supports.

4. The bridge underwent nonreversible joint movements. It was observed that in many cases the bridge sections did not return to their initial positions as temperatures rose and fell to their initial values. This behavior was evident over the 24-hr monitoring cycles as well as over the long-term seasonal period. The nonreversible movements are attributed to the restraining effects present at the joint supports. There was no consistent pattern in this behavior, further substantiating the preceding two conclusions.

5. Although nonreversible behavior was observed, a general seasonal repetitiveness of joint movement behavior occurred, which was in agreement with the seasonal temperature trends.

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

7. 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.

8. The data acquired over the 9-month period after the bridge was opened to traffic indicated no discernable effects caused by traffic loads. However, to more fully evaluate these effects, monitoring over a longer period of time is required.

9. A comparison of measured joint movements with those estimated by the current LaDOTD procedures did not indicate a consistent pattern. In some cases the LaDOTD recommendations over-estimated the movements but in other cases under-estimated them.

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

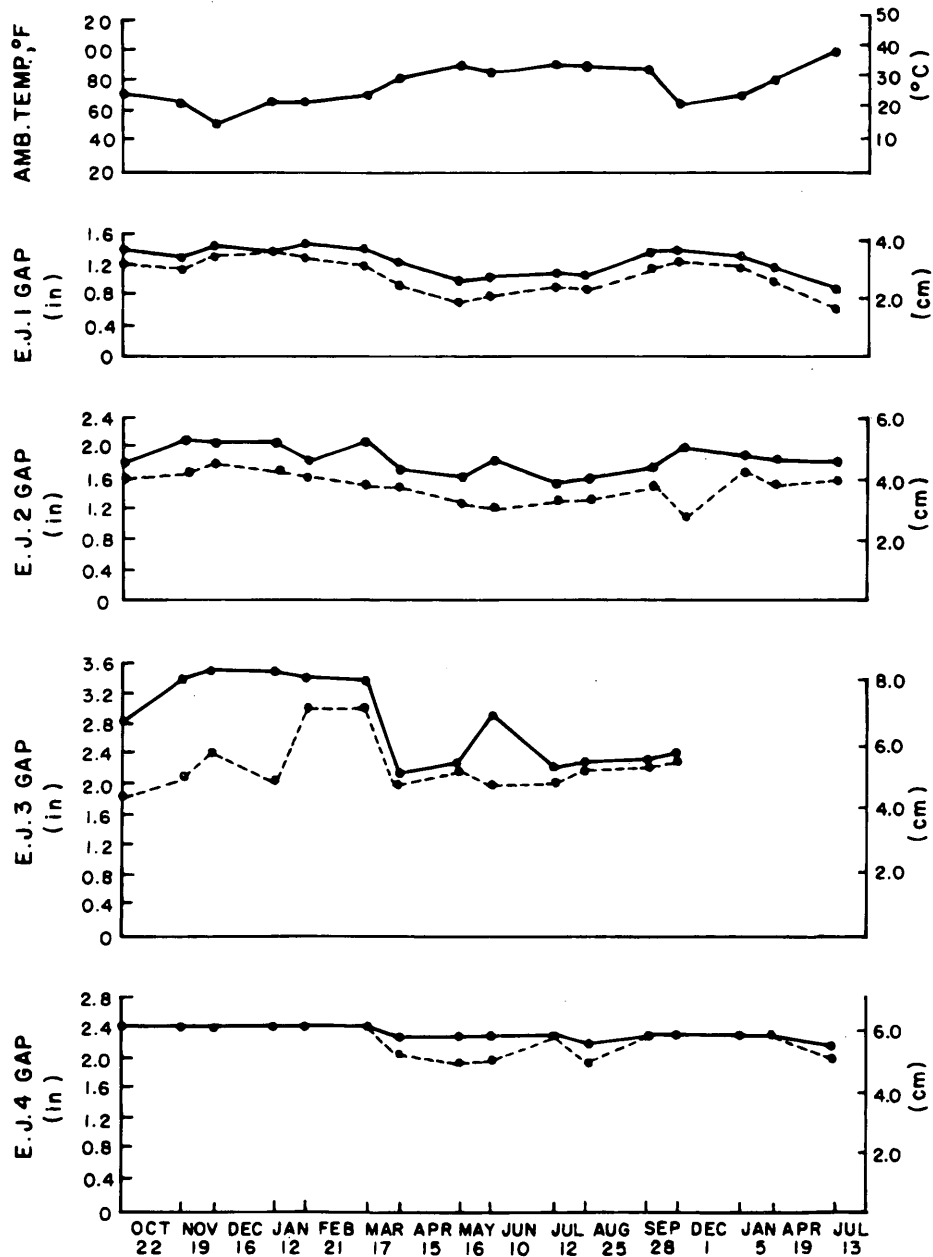


FIGURE 9 Summary of long-term movements obtained from LVDTs at north side of Joints 1-4.

TABLE 2 Comparison of Measured and Estimated Movements in Centimeters

Joint Location	Measured Movements (with LVDT's)	Estimated Movements*	
		Thermal	Total
E.J. 1	2.03	1.40	2.06
E.J. 2	2.41	1.40	2.06
E.J. 3	3.56	5.00	6.22
E.J. 4	1.40	3.91	5.77

2.54 cm = 1 inch

*Estimated movements based on LDOTD procedures which include creep & shrinkage as well as Thermal (shown separately).

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