T. Wipf N. Hachicho K. Jeyapalan

Final Report

Field Measurement of Bridges for Long-term Structural Movement

Sponsored by the Iowa Department of Transportation,
Highway Division, and the Iowa Highway Research Board

November 1988

Iowa Department of Transportation

report

College of Engineering
Iowa State University

DISCLAIMER

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Iowa DOT Project HR-295
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1. INTRODUCTION

1.1. General Background

Long-term structural movement of bridges may have many causes, such as changes in temperature, movement of foundation or supports, and the application of unexpectedly large forces. The accurate monitoring of these movements can be a difficult problem primarily because of the relatively long time period over which the movements occur and the inadequacy of proper instrumentation and technique. Obtaining a stable reference point for the measurements is a significant problem. The importance of obtaining these data has long been recognized by bridge engineers. In many cases, identifying and understanding the movements are the first steps in eliminating or solving problems that may affect the service life of the bridge.

The long term data must be obtained with great care so that sufficient accuracy is maintained. A study sponsored by the Iowa Department of Transportation (Iowa DOT), Research Project HR-275, "Long-term Structural Movement" [1], henceforth referred to as Phase I, addressed the many problems associated with obtaining accurate long term field data. The study served as the initial phase of developing a data acquisition and monitoring system to detect long term movement of bridges. Two methods were studied in Phase I (tilt sensing and photogrammetric techniques) and were shown to be feasible for use in monitoring long-term structural movement. A number of pertinent and useful references were cited in this report.

Two bridges were identified by the Iowa DOT as requiring monitoring for long-term structural movement. The Black Hawk Bridge, which spans the Mississippi River in Lansing, Iowa, has been subjected to repeated barge impacts to the main span pier, Pier No. 2, over the past few years. The extent of damage is not precisely known, although some visible spalling of concrete has occurred at the waterline of the pier. The Iowa DOT has been monitoring the pier by surveying and has had the pier, including the foundation, inspected by an outside consulting firm. Concern by the DOT exists with regard to whether any significant change in pier alignment has already taken place, or will take place, should the barge impacts continue.

The second bridge identified by the Iowa DOT for monitoring is the Karl King Bridge spanning the Des Moines River in Fort Dodge, Iowa. Pier No. 4 is located on a sidehill, which is underlaid with shale. Possible movement of the shale layer, perhaps from the freeze and thaw cycle, has apparently caused the observed movement of this pier. Since the late 1970s Iowa DOT personnel have been monitoring the pier using surveying techniques and inclinometers. During this time damage has occurred to the pier footings, and movement of the pier has caused beam rocker supports to be reset on a number of occasions. Rehabilitation techniques (including the insertion of drain tiles in the sideslope) have been attempted to eliminate further movement, and DOT personnel are continuing to monitor the pier for movement. In spite of these efforts, the DOT desired additional monitoring to arrive at more conclusive results. An accurate method of obtaining data is required as the initial step in determining what measures, if any, will be required to eliminate completely further problems.

Phase II, which is presented in this report, involved the field application of the tilt sensing method that was recommended in Phase I for monitoring movement of the Black Hawk and Karl King Bridges. Data acquisition systems were designed to continuously record data from tilt sensors that were mounted on the two suspect piers at predetermined locations. Because of past problems with the Karl King Bridge, more attention was focused on developing a field instrumentation system for a thorough investigation of the bridge's behavior. Therefore, significant temperature data were recorded to study movements related to temperature variations. The thermocouple data described the temperature distribution on the cross section of the bridge near Pier No. 4, and a computer model of a segment of the bridge was developed; the model utilized this temperature information to study the pier behavior. The analysis provided information related to the temperature-related axial deformations of the superstructure and their possible effects on the movement of the pier.

The data recorded at the Black Hawk Bridge was used in a more "qualitative" form than that at the Karl King Bridge, with the objective being primarily to measure any absolute change in pier alignment rather than to study thoroughly the movement related to temperature variations. The primary task was the design and installation of a telemetry-based instrumentation system that the Iowa DOT could use for future monitoring of the pier.

1.2. Objectives

The objective of Phase I of the study was to determine the feasibility of field use for the tilt sensing system. As a result of the successful completion of Phase I, Phase II was undertaken with the overall objective of designing and installing tilt instrumentation and data acquisition systems for use in monitoring long-term structural movements of field bridges. Two bridges in Iowa were identified by the Iowa DOT bridge personnel for long-term movement monitoring: the Karl King Bridge in Fort Dodge and the Black Hawk Bridge in Lansing.

The following specific objectives were established as part of Phase II of this study:

- to design a data acquisition system for tilt sensing equipment utilizing a telephone telemetry system. This system utilizes Iowa DOT purchased equipment and will be kept in place at their discretion, for future monitoring.
- to monitor possible movement of the main span pier, Pier No. 2, at the Black Hawk Bridge in Lansing and the possible long-term movement of Pier No. 4 on the Karl King Bridge in Fort Dodge.
- to assess the feasibility, reliability, and accuracy of the instrumentation system used in this study.

To meet these objectives, laboratory tests were performed to determine the temperature sensitivity of the tilt sensors before mounting them in the field. Locations for the components of the instrumentation and data acquisition system at the bridge sites were determined. A finite-element computer model of a portion of the Karl King Bridge was developed and a detailed analysis was performed to validate the field data.

1.3. Literature Review

The Phase I report [1] included a literature review that covered the following areas related to monitoring long-term structural movement: structural engineering applications, surveying applications, and evaluation of the two applications for field use. Although the majority of these references are applicable to the work in Phase II, the material will not be repeated, but rather the reader is referred to the report in which the literature is summarized [1]. In the time since that report was completed, additional pertinent literature has been published, and this information will be briefly described in the following paragraphs. In addition, literature related to the thermal characteristics of bridge superstructures was studied to assist in the analytical investigation of thermal movement of the Karl King Bridge.

Shiu et al. [2] monitored long-term thermal and time-dependent movements of concrete box girder bridges. Measurements of longitudinal concrete strains, concrete and air temperatures, and vertical deflections were taken over a period of five years. In conjunction with the field tests, laboratory tests were conducted to determine time-dependent changes in concrete material properties such as creep and shrinkage. This information was utilized in a nonlinear analytical model that attempted to predict the long-term bridge movement. It was determined that bridges experience significant seasonal and thermal movement. Also, time-dependent movements from creep and shrinkage were significant.

In Aachen, Federal Republic of Germany, Muller-Rochholz et al. [3] studied the time history and rate of bridge movements. Two bridges were monitored for that purpose: a 400-m prestressed concrete box girder, and a 440-m steel box girder. For a period of one year, horizontal displacements were measured using high resolution transducers, while semiconductor thermocouples were used for temperature measurements. Data were recorded and interpreted on the site using a microprocessor-supported measuring system. Temperature induced movements were found to be twice as large in the steel bridge as in the concrete structure under identical environmental conditions.

In 1986 tilt sensing equipment was used to monitor movement of the Ladlow Viaduct in Cincinnati, Ohio [4]. The viaduct is a hollow barrel, concrete arch bridge 1500 ft long. The decision to monitor the bridge was made after severe rotation of two of the seven piers was detected. Sperry tilt sensors were mounted on all piers, including the two critical piers, to monitor their rotation and any possible vertical settlement.

In 1984 McClure et al. [5] studied the temperature distribution in a post-tensioned, segmental concrete box girder bridge. The investigation was part of a research program that involved obtaining field measurements on a full scale bridge to assess its behavior under various loading conditions. For a period of one year, temperature readings were recorded from thermocouples placed longitudinally and transversely on the bridge. Dial gages were used to measure vertical deflections at midspan of the bridge. Analysis of data showed no significant longitudinal temperature variation and little transverse temperature variation.

In a Louisiana study, Gopu and Avent [6] monitored the short and long term movements at selected deck joints at the Atchafalaya River Bridge. The bridge consists of nine east- and ten west-approach spans and the river crossing. All instrumentation utilized was placed on the east approach, which consisted of eight prestressed-concrete girder spans and one plate girder span. Data from linear variable differential transformers (LVDT) and thermocouples were recorded using a Hewlett Packard data acquisition system. Surveying techniques were also used to monitor movement of the bridge. It was determined that thermal movements of bridge joints can be significant, and that those movements are different for the concrete and steel joints.

Based on an extensive review of theoretical and experimental studies, Kennedy and Saliman [7] addressed the heat flow problem in composite bridge structures. They proposed a realistic and simple one-dimensional temperature distribution. This distribution is linear across the depth of the concrete deck and uniform through the depth of the steel beam or girder.

In a current study at ISU, Girton et al. [8] are investigating expansion and contraction characteristics of integral abutment bridges. Their research sought primarily to establish the effects of ambient temperature changes on the expansion and contraction of bridges of different construction materials, as well as to develop design guidelines for long integral-abutment bridges. Two integral-abutment bridges were instrumented for this study. The first was a 324-ft-long prestressed-concrete beam bridge in Webster City, Iowa, and the second a 320-ft-long welded-steel beam bridge in Woodbury County, Iowa. Instrumentation included thermocouples installed at various locations on the cross section and an LVDT for measuring change in bridge length. Also, one pile at each bridge was instrumented with strain gages. Data were recorded and stored using a data collection system that was installed at the site.

In a Virginia study, Baber et al. [9] investigated the behavior of a cable-stayed box girder bridge, both during construction and subsequently during service. The bridge consists of a seven-span , continuous, twin precast, segmentally post-tensioned concrete box girder. The middle five spans are supported by multiple cable stays arranged in a harp configuration from pylons located at both sides of the main span. Precast delta-frame assemblies were used to transfer the cable stay forces to the twin box girders. The study had several specific objectives:

- to determine live load stresses in the cable stays
- to evaluate resulting stress in the deck
- to evaluate the performance of the delta frame assemblies
- to obtain thermal gradient data for the box girders, pylons, and cable stays.

Instrumentation used on the bridge consisted of electrical-resistance strain gages, mechanical strain gages, and thermocouples. The strain gages and thermocouples were installed in different segments of the deck, pylons, and cable stays. Data were recorded and stored using a data acquisition system that was installed at the site. A personal computer was used to download the data at desirable intervals of time. Preliminary results of the study show the trends in bridge behavior to be consistent with the stages of construction.

Roeder [10] ,in a study begun in 1987 and currently in progress at the University of Washington, is investigating thermal movements of bridges and attempting to develop methods for estimating these movements. The initial phase of the study consisted of a survey of state DOTs, bridge engineers, and governmental agencies. The survey attempts to determine different methods of designing for thermal movements in bridges and to identify any unique problems associated with each. The survey also attempts to isolate specific bridges, which will be investigated analytically to establish their thermal behavior. Preliminary results of the study indicate that most bridges are designed assuming uniform thermal expansion of the bridge deck and ignoring thermal deformation of the piers and abutments. The study also concludes that movement of bridges can be related to other causes, such as traffic loading, and creep and shrinkage of concrete.

1.4. General Testing Program

Phase II of this study involved field application of the tilt sensing system for determining long-term structural movement and the development of an analytical model to verify field data. A brief description of the investigation is presented in the following sections; detailed information will be presented later in the report

1.4.1. Laboratory Testing of Instrumentation

The laboratory investigation involved the determination of the temperature sensitivity of the tilt sensors for field use. A test setup similar to the mounting procedure designed for field application was used. The tilt sensors were subjected to low temperatures by placement in a freezer, mounted on a reference monument, and continuous readings taken to develop a temperature coefficient. These values were compared with the manufacturer's recommended coefficient.

1.4.2. Field Testing Program

The field investigation consisted of designing the data acquisition systems for each of the two bridges and determining the location for placement of the instrumentation. The labor for the placement of the instrumentation was contracted for with local contractors at each bridge site with supervision provided by ISU project personnel. A temperature transducer system was designed for the Karl King Bridge so that an analytical investigation could be performed to validate measured movements. Superstructure temperatures of the concrete deck and steel stringers were monitored near Pier No. 4 to allow prediction of expansion and contraction characteristics. In addition, temperatures were taken at one location of the pier capbeam.

The data from the Karl King Bridge were downloaded manually at periodic intervals because of the close proximity of the bridge to ISU. However, the Black Hawk Bridge was located at a significant distance, thereby requiring the use of a telemetry system to download data from the system. A modem was placed at the test site and data were taken utilizing components of the existing telemetry system at the office of the Iowa DOT in Ames, Iowa. In addition, surveying measurements were made at both bridges to provide additional verification of the data obtained from the tilt sensing system.

Four tilt sensors were mounted on Pier No. 4 of the Karl King Bridge: one each on the east and west face and one each on the north and south face of the capbeam. The bridge is an eight-span continuous structure consisting of eight composite stringers that are steel-plate girders. Pier No. 4 is a four-column reinforced-concrete frame structure on spread footings. In 1970, a 90-ft, three-span concrete slab bridge was added at the west end of the existing structure.

Two tilt sensors were placed on the pier of the Black Hawk Bridge, one on each of two adjacent and perpendicular faces. The bridge consists of a three-span through truss over the main channel of the Mississippi River and six spans of a steel stringer section. The monitored pier is founded on spread footings and is near the center of the channel.

1.4.3. Analytical Program

A finite element model of a portion of the bridge superstructure and Pier No. 4 of the Karl King Bridge was developed for validation of the field test data. Superstructure temperature data from the field were input into the model to predict expansion and contraction displacements and forces. These forces were then applied to the pier model to predict displacement. Tilt, or angle change, on the pier capbeam from the model was compared with tilt data obtained in the field.

2. TESTS AND TEST PROCEDURES

2.1. Description of Bridges

The Karl King Bridge is on State Highway 7 over the Des Moines River in Fort Dodge, Iowa. The bridge was constructed in 1957 as an eight-span, continuous stringer structure. The steel plate girders are composite with the concrete deck. The total length of the bridge is 555 ft.; the general layout is illustrated in Figs. 1 through 3. In 1970, a 90 ft, three-span concrete slab bridge was added at the west end of the existing bridge. As mentioned previously, the pier of interest in this study, Pier No. 4, an expansion pier, is skewed 30° with respect to the superstructure; it is located on the west side of the Des Moines River and is underlaid with a shale layer. It is believed that the instability of this shale layer contributed to the past observed movement of the pier. Photographs show Pier No. 4, looking west (Fig. 4a) and east (Fig. 4b). The pier is approximately 27.5 ft in height above the top of the footing.

The Black Hawk Bridge is located on State Highway 9 over the Mississippi River in Lansing, Iowa. The bridge consists of six east-approach deck-truss spans and a three-span through-truss river crossing. The total length of the bridge is 1630 ft; it was completed in 1931. Figure 5 illustrates the general layout of the bridge. The bridge deck is composed of a steel grid deck on steel stringers. Five of the six east-approach spans consist of deck trusses spanning 90 ft. each. The most easterly and last of the six approach spans is a simply supported 46 ft. I-beam span (see Fig. 6). The main span pier, Pier No. 2 (a fixed pier), is also shown in the figure. Pier No. 2 was instrumented and monitored to determine if repeated barge impacts have affected the alignment of the pier and the stability of the bridge substructure. This pier is approximately 82 ft in height above the top of the footing.

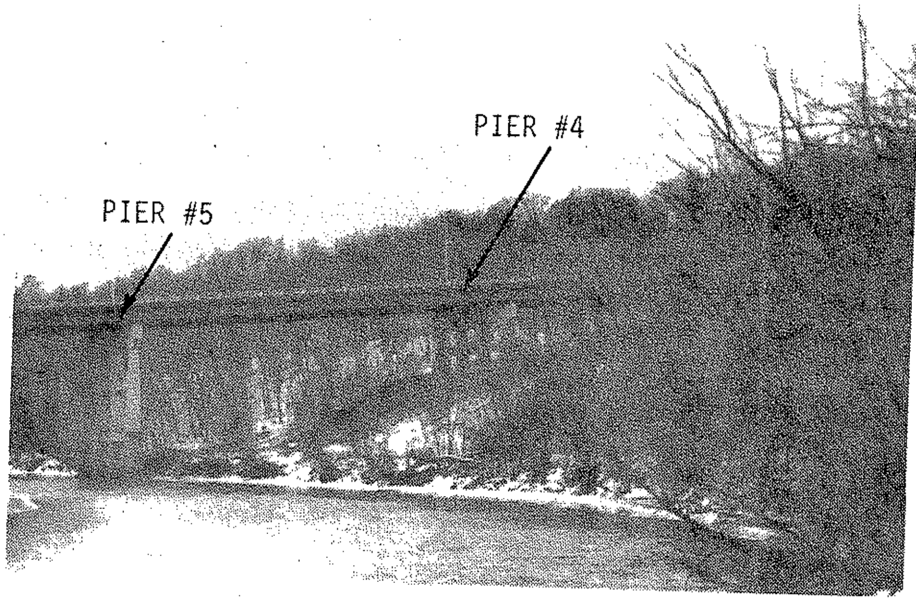
2.2. Field Instrumentation

2.2.1. General

2.2.1.1. Structural

The Karl King Bridge was monitored over a time period of approximately 15 months; the Black Hawk Bridge over an 11-month period. This time period was necessary to determine the general behavior of the bridges and to isolate significant pier movements, caused by unexpected external sources, from normal movements.

(a) Roadway (looking east).



(b) Pier No. 4 on sloping hillside (looking south, downstream).

Fig. 1. Overall view of Karl King Bridge.

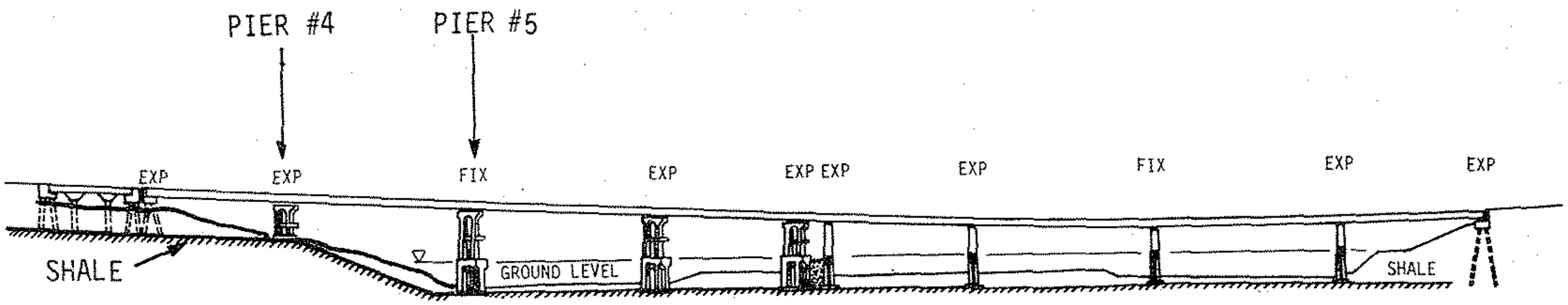


Fig. 2. Elevation of the Karl King Bridge (looking north, upstream).

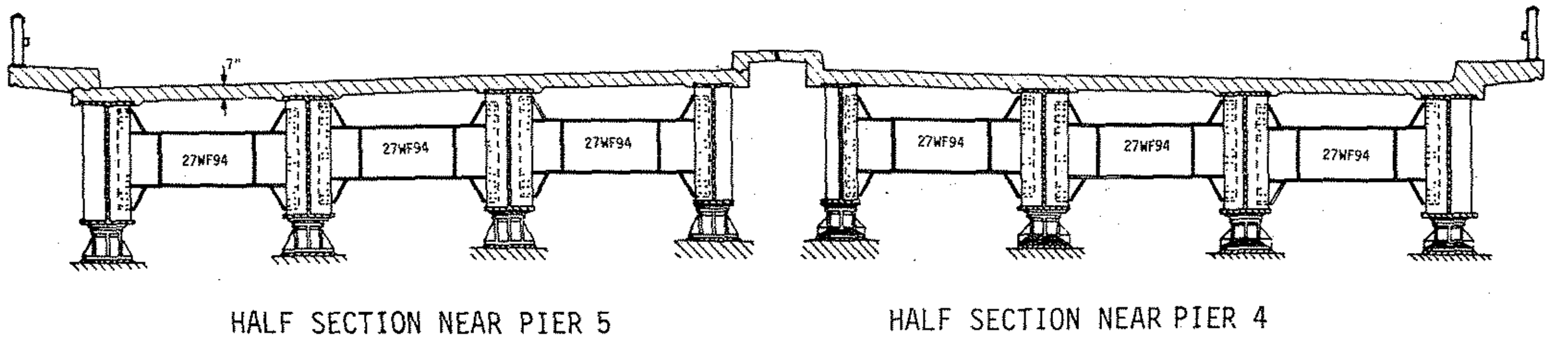
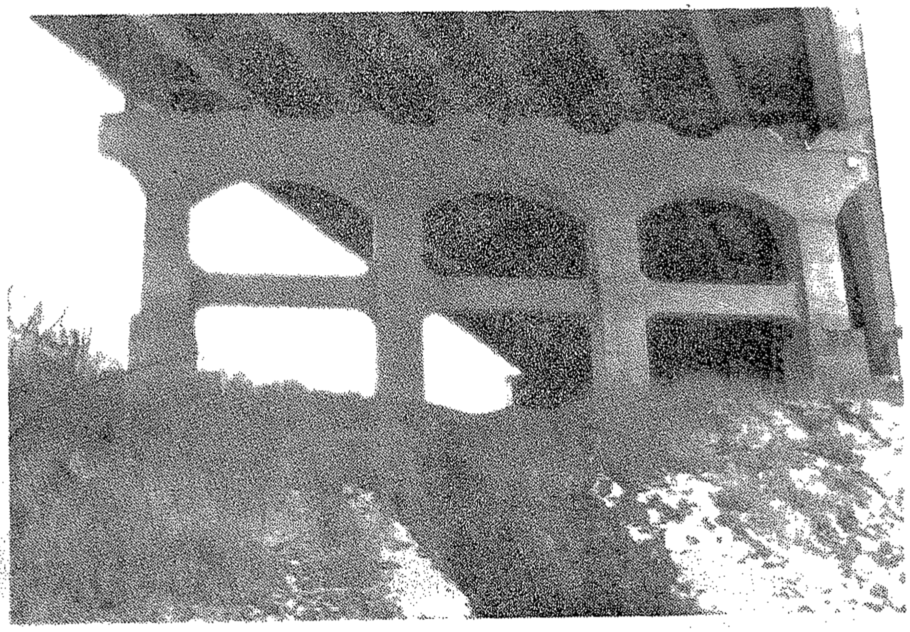
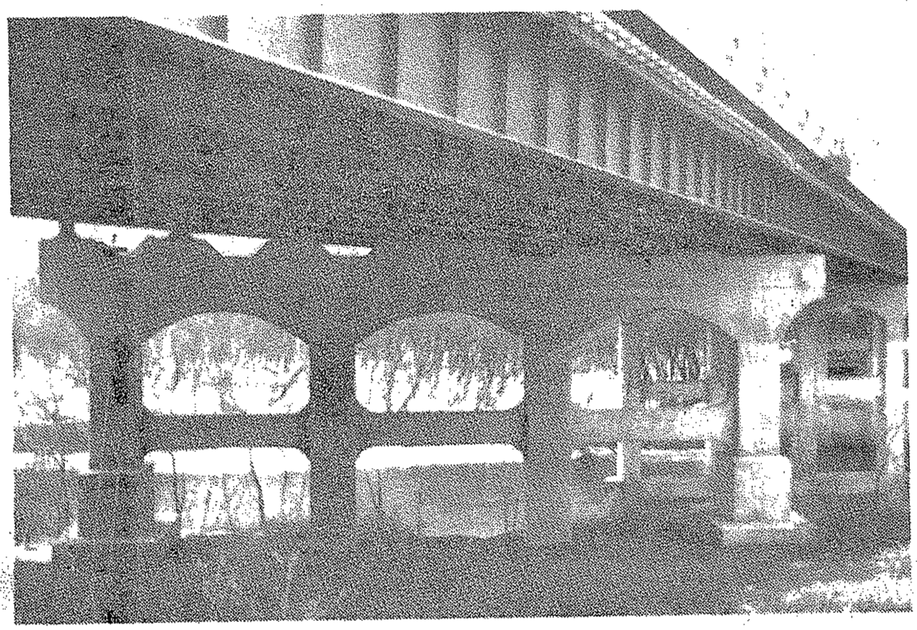


Fig. 3. Cross section of the Karl King Bridge superstructure.

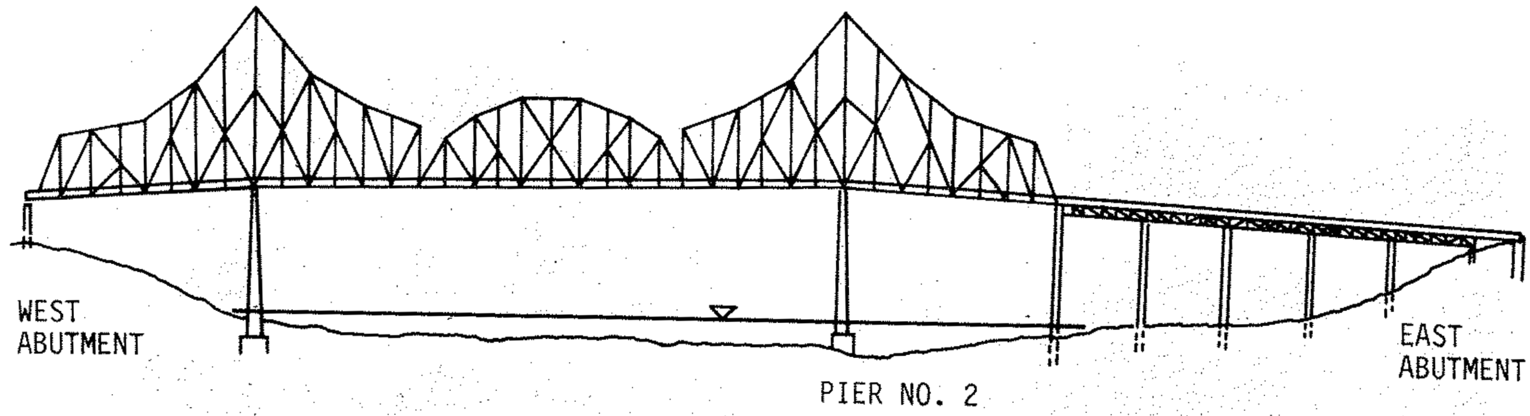


(a) Looking west.

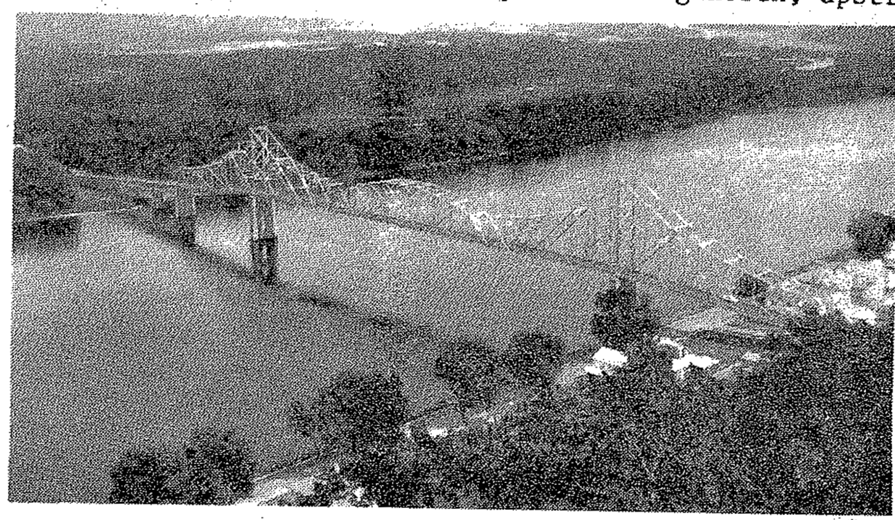


(b) Looking east.

Fig. 4. Photograph of Pier No. 4.

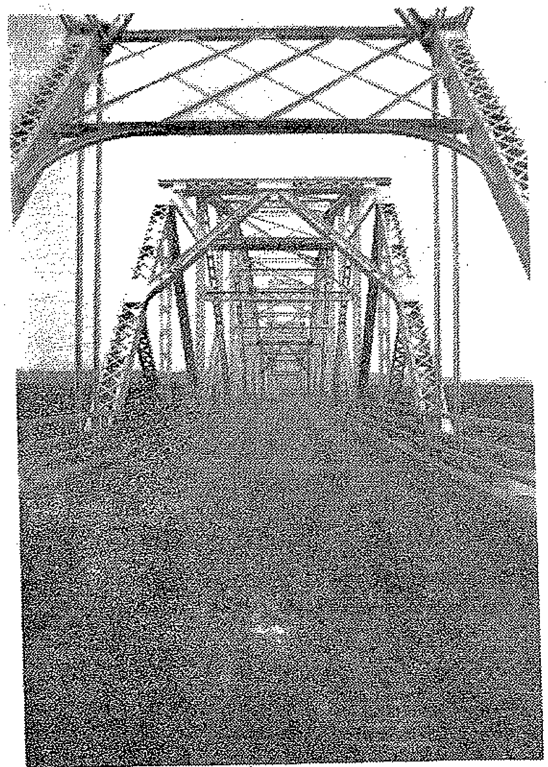


(a) Schematic layout looking north, upstream.

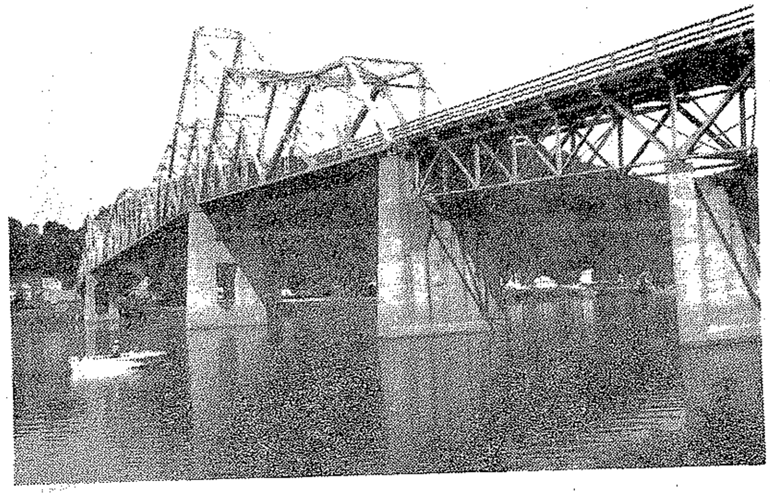


(b) View from the Iowa side looking southeast.

Fig. 5. The Black Hawk Bridge.



(a) The bridge deck (looking east).



(b) Pier No. 2 (view from the Wisconsin side looking northwest).

(c) Cross section from under deck at Pier No. 2.

Fig. 6. The Black Hawk bridge.

Once the study was authorized by the Highway Division of the Iowa Department of Transportation and the Iowa Research Board, a visit to both bridge sites was arranged to determine where to mount monitoring equipment on the piers. At both bridge sites, the equipment was installed to monitor pier movement at right angles to the pier major axes.

The monitoring system in Fort Dodge consisted of four tilt sensors, a central console unit, a data logger, an ambient temperature probe, a channel expanding device, and several thermocouples. In Lansing, two tilt sensors were utilized along with the console unit, data logger, and the ambient temperature probe. Two thermocouples were installed to determine the concrete temperature. In addition, a modem and a telephone line were used to remotely control the monitoring system.

The monitoring systems at both bridges were powered by battery systems. Data were recovered monthly and reviewed on a regular basis. A detailed description of each component of the monitoring system is provided in a later section in this chapter.

2.2.1.2. Surveying

On three different dates Pier No. 4 on the Karl King Bridge and Pier No. 2 on the Black Hawk Bridge were monitored using surveying techniques to measure movement. A method utilizing theodolites, electronic distance meters (EDM), and levels–referred to as the triple point method–was used in calculating the structural movement. A discussion on the method is provided in Ref. [11].

The surveying data–taken at the Karl King Bridge on May 18, 1987, June 20, 1987, and August 24, 1987–provided a check on data obtained with the structural instrumentation. The data obtained by surveying at the Black Hawk Bridge were taken on May 1, 1987, June 27, 1987 and August 22, 1987. A Wild NI2 level was used to establish elevations of the benchmarks, which were established on the baseline. An Iowa DOT benchmark in the area served as the reference of the Karl King site, and on the Black Hawk site a partially buried, rigid I-beam served as the reference. Angles from the baseline to targets placed on Piers No. 4 and No. 2 were measured to the nearest 0.1 second using Wild T2 and Kern DKM2 theodolites. Distances were measured between the benchmarks to the nearest 1 mm using a Leitz Red 1A EDM.

2.2.2. Instrument Description

2.2.2.1. Tilt Sensing System

A Sperry tilt sensing system was utilized for monitoring rotational movement of the piers on each bridge. The system consists of a central console unit (Fig. 7a), and a tilt sensor and mounting plate (Fig. 7b). The system was utilized in this project primarily because of its use of gravity as an absolute form of reference. From an earlier study at ISU [1], the system was found to be stable and reliable, with a minimum sensitivity to environmental effects. The mounting procedure is simple and can easily be accomplished. Figure 8 shows the tilt sensor and the vertical mounting plate, which is used to attach the sensor to a structural member. The tilt sensors have a range of ± 20 arc min with an accuracy of 0.003 arc min.

The tilt sensors are connected to the central console unit, which can monitor up to eight individual sensors. The console provides electrical power to the sensors and serves as a data source and also transmits the electrical signals from the sensors to the micrologger where the data are stored. Six-volt battery power was used to operate the console unit at both bridges.

2.2.2.2. Temperature Transducers

A Campbell Scientific Model 107 temperature probe was used to measure ambient temperature. The probe incorporates a thermistor in a water resistant tube with standard 10 ft. leads. It provides an accuracy of $\pm 0.4°$ F over the range of -26° F to 118° F.

Copper-constantan Type T thermocouples were installed at various locations to determine temperatures in the concrete slab and steel stringers. The thermocouple is a thermoelectric device with a circular cross section of approximately $\frac{1}{4}$-in. diameter that provides accurate temperature measurement by measuring the voltage difference between the points of contact of two dissimilar metals joined together. The thermocouples were connected to the micrologger to obtain temperature measurements at desirable time intervals.

2.2.2.3. Micrologger

The Campbell Scientific Model 21X data logger was used for data storage in the field (see Fig. 9). The micrologger can operate in a temperature range of -50° F to +150° F, and 0 to 90% relative humidity. Its small size and ability to operate in harsh environments made the micrologger advantageous for remote operation. The micrologger allows input through 16 analog channels. An additional 32-input channel can be added through the AM32 channel expander. The micrologger has the capability of initiating measurements, performing a wide

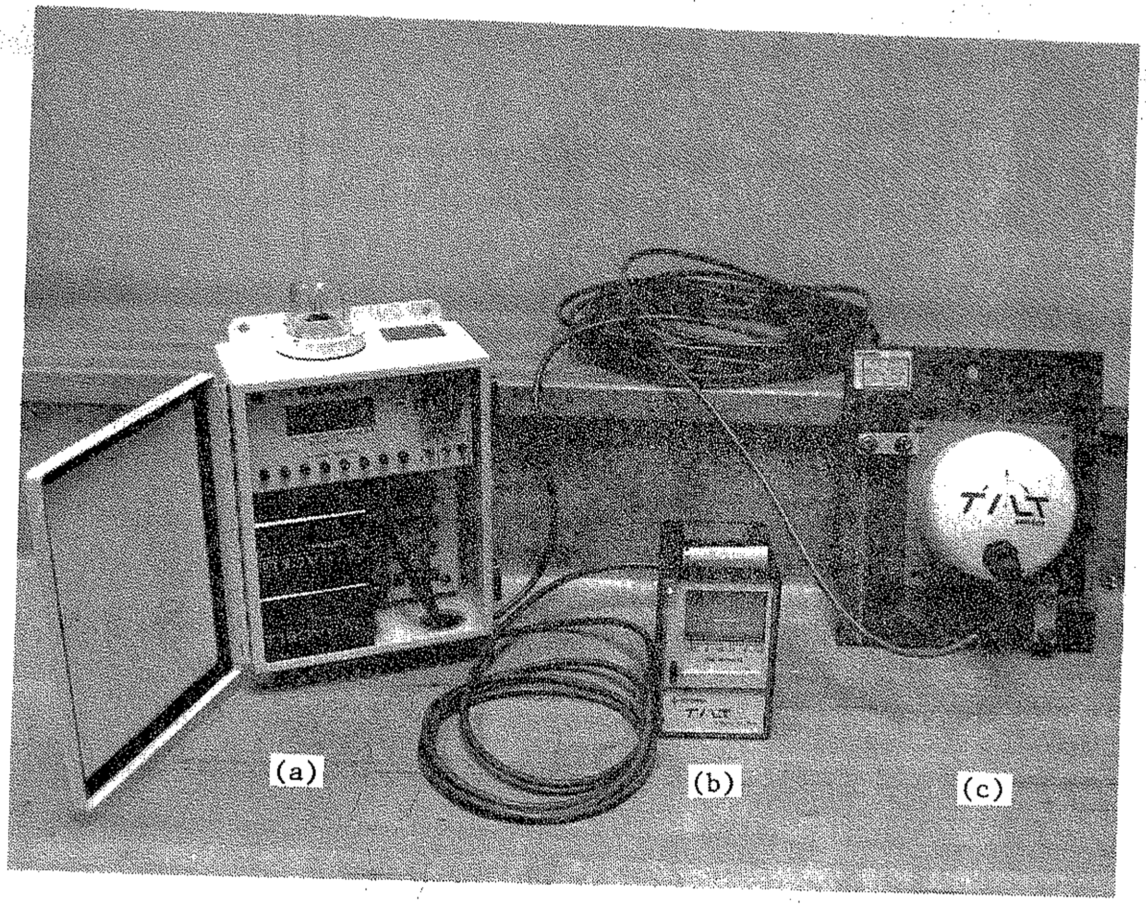


Fig. 7. Tilt sensing equipment: (a) power source, (b) recorder, and (c) tilt sensor and mounting plate

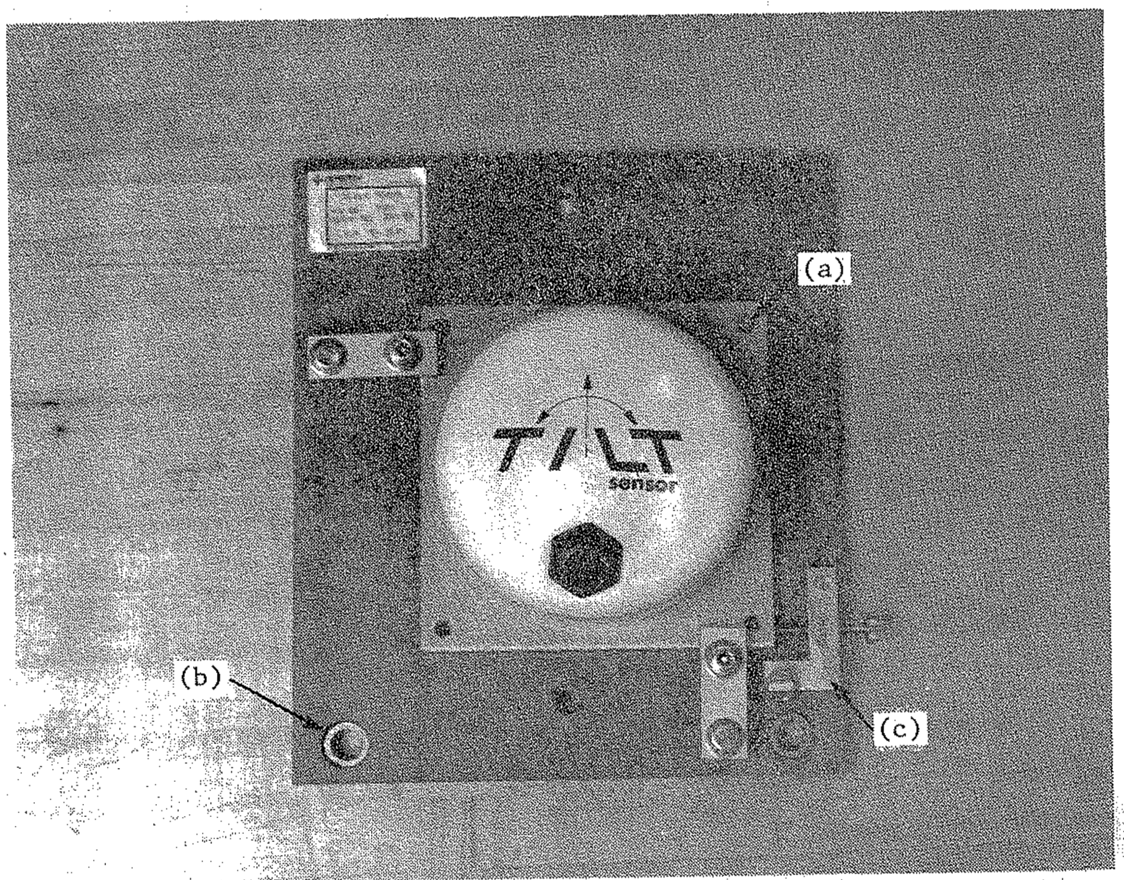


Fig. 8. Details of tilt sensor mounting to the plate: (a) pivot hole, (b) brass mounting pad, and (c) alignment mechanism

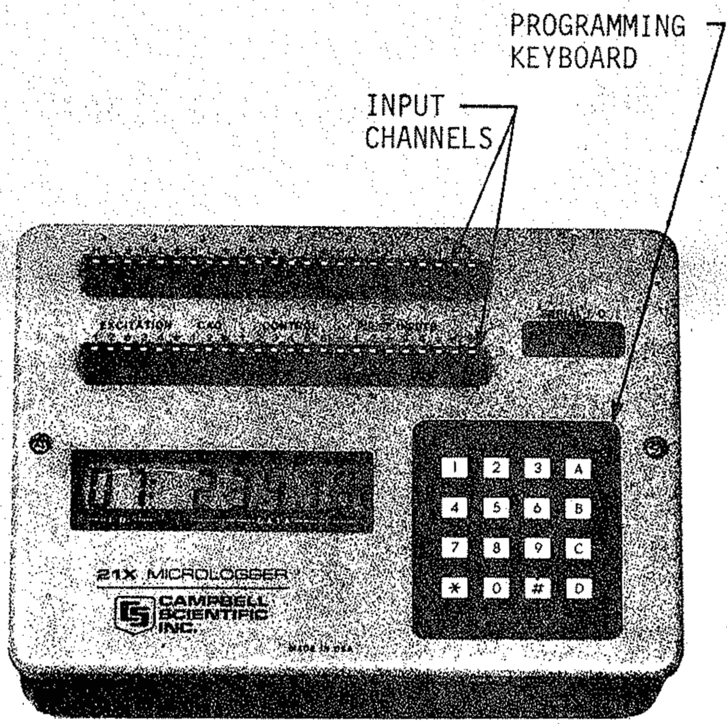


Fig. 9. Campbell Scientific micrologger.

range of processing operations, and storing 19,328 data values. Eight alkaline D cells were used to operate the micrologger.

Data stored in the micrologger can be retrieved either manually, using a Model RC35 cassette recorder and appropriate interface devices as was used in Fort Dodge, or remotely through a modem link as was used in Lansing.

2.2.2.4 Modem

Because of relatively long distance and cost of travel to the Lansing bridge, we linked a Universal Data Systems Model 212 ALP modem to a telephone line to the micrologger. The modem allowed programming, monitoring of the micrologger, and retrieval of data on a regular basis. Communication with the micrologger was accomplished using a microcomputer located at the Iowa DOT in Ames and appropriate Campbell Scientific telecommunication software packages.

2.2.3. Laboratory Setup for Tilt Sensor Tests

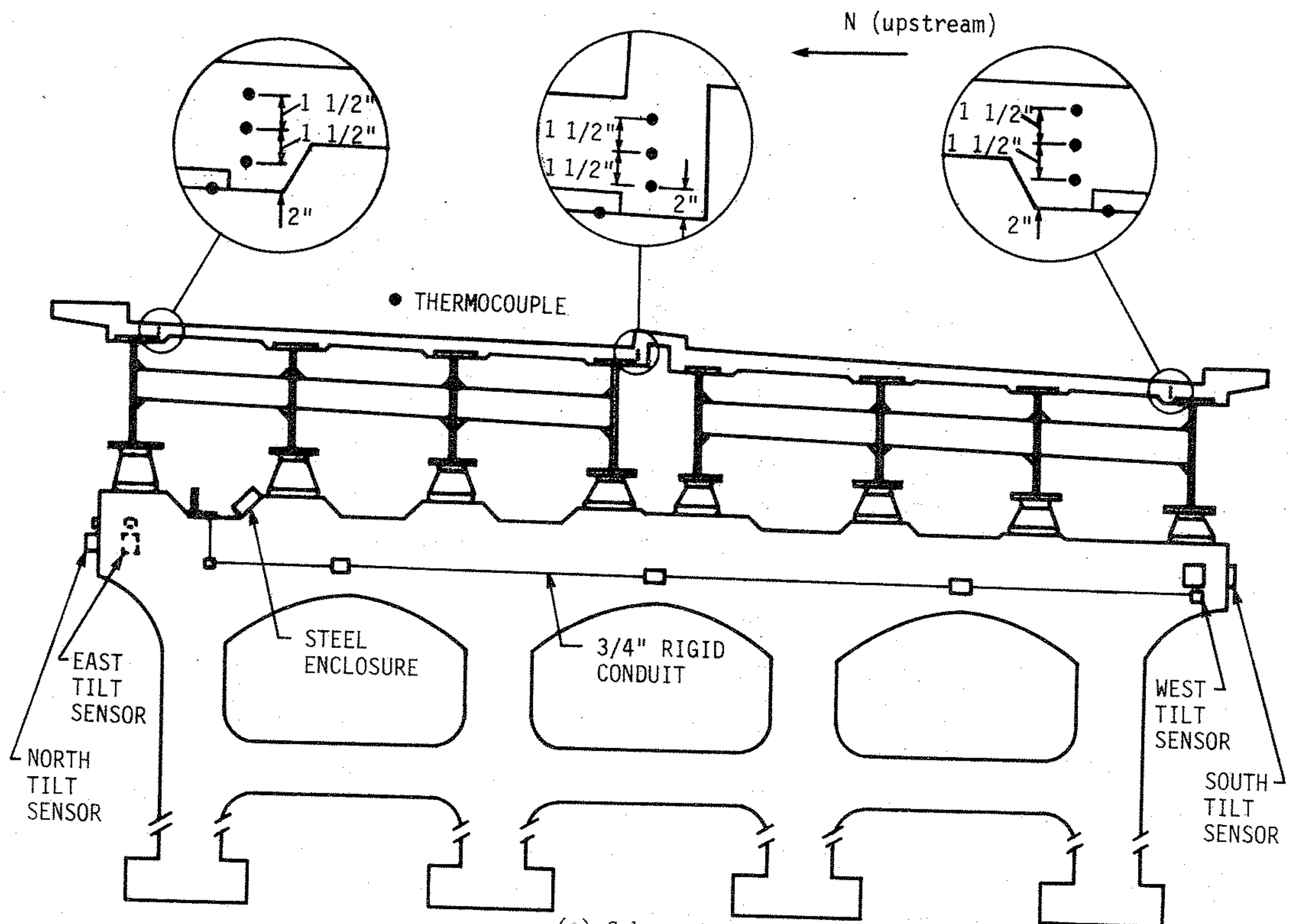
Experiments were conducted on the Sperry tilt sensors prior to their installation on the bridge pier at Fort Dodge in order to confirm the temperature coefficient stated by the manufacturer. The tests were performed in the ISU Structural Research Laboratory to simplify observation. The tilt sensor under study was mounted to a massive block of concrete located on the laboratory floor. A second tilt sensor was mounted adjacent to the "test" sensor to serve as a reference. With this setup, adjustments could be made for any unwanted movement occurring on the block. The experiment was intended to be a static test and movement was to be avoided.

The tilt sensor to be tested was mounted on a mounting plate and placed in a freezer for 24 hours preceding the test. It was then removed from the freezer, fastened to the concrete block, leveled, and allowed to return to room temperature, which was approximately 65° F. Monitoring continued for approximately 1 1/2 hours after initial placement of the sensor. The Appendix contains a summary of the test results.

2.2.4. Field Setup

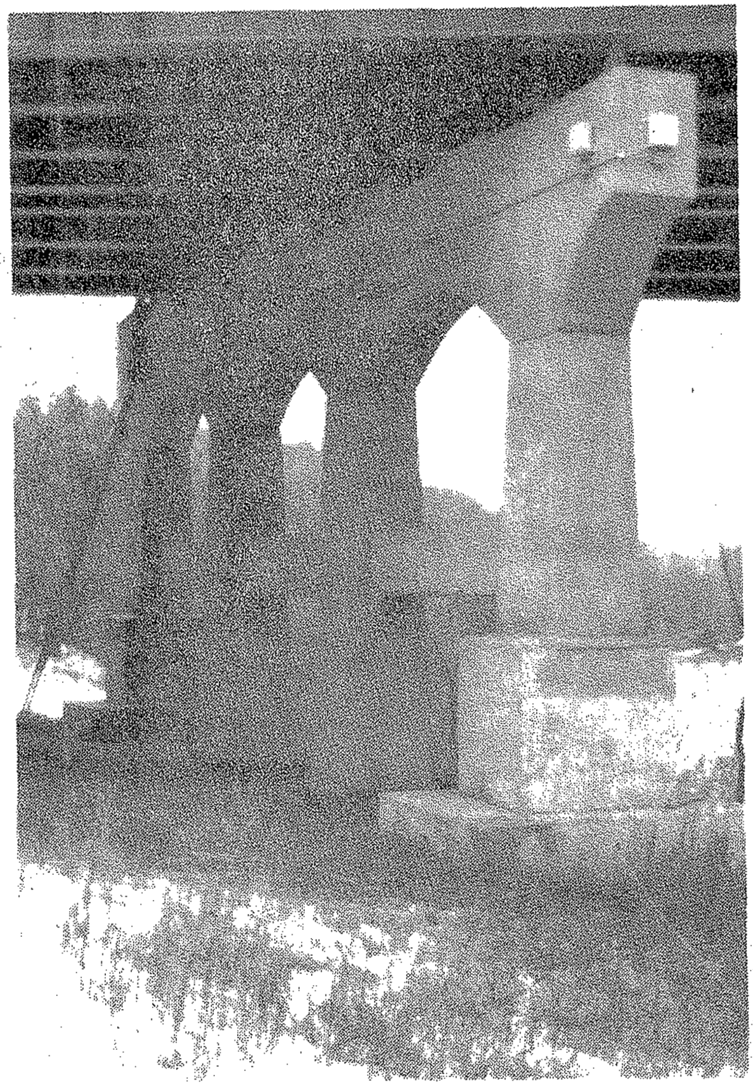
2.2.4.1. Fort Dodge

Structural–The layout of the instrumentation used on Pier No. 4 is shown in Fig. 10. The tilt sensors were mounted on both of the main axes of the pier capbeam to provide



(a) Schematic.

Fig. 10. Instrumentation layout on the Karl King bridge.



(b) Photograph.

Fig. 10. Instrumentation layout on the Karl King Bridge.

redundancy in the measurements. The four tilt sensors were placed approximately 28 ft. above ground level. Steel cover plates 10 in. x 11 1/2 in., painted white (Fig. 11), were used to protect the tilt sensors from vandals and to provide additional protection from the environment. These cover plates were attached to the pier using 1/2 in. concrete anchors. All tilt sensors were zeroed on January 3, 1987, and the monitoring process was started on January 7, 1987.

The data acquisition system, which consisted of the central console unit, micrologger, and channel expander, was placed inside a 24 in. x 24 in. x 8 in. watertight steel enclosure (Fig. 12). The enclosure was mounted on top of the pier 9 ft. from the north end of the capbeam. The tilt sensor cables were placed inside a 3/4-in. rigid conduit, which connected the four tilt sensors with the enclosure.

As mentioned previously, thermocouples were utilized for temperature measurements. Two thermocouples were embedded 3 in. into the concrete in the vicinity of the north and east tilt sensors. The superstructure was also instrumented with thermocouples to study the effects of expansion and contraction cycles on the pier. Since the literature study of the thermal characteristics of bridge superstructures showed that the temperature remains essentially constant transversely [5,7], we installed thermocouples at only three locations on the cross section. Nine thermocouples were embedded in the concrete slab, and three were mounted on the bottom side of the top flange of the steel stringers. Figure 10 shows the location of the superstructure thermocouples. The installation of the thermocouples and the rigid conduit was conducted by Paul Electric Supply Co. of Fort Dodge, Iowa.

Surveying–As previously mentioned, a triple point method of surveying was used to monitor Pier No. 4 on three different dates. Each date's observation averaged approximately 2 hours of leveling, 2 hours of EDM baseline measurement, and 3 hours of theodolite angle measurement. Measurements began at approximately 9:00 a.m. and ended at 4:00 p.m. The field setup consisted of a baseline made up of three benchmarks, which were located approximately 90 ft. upslope from Pier No. 4. Each benchmark consisted of a 3-ft-deep, 6-in.-diameter concrete cylinder with brass cap and nail tip marked to represent the station. Four targets, T_1, T_2, T_3, and T_4, were painted on the pier and a nail was driven at the center to represent the point on target, as shown in Fig. 13. The elevations of the benchmarks were established by level loops run from a nearby partially buried and rigid I-beam in the vicinity. The leveling misclosure was less than 0.01 ft. Angular observations that were taken to the pier targets were rejected if any direction difference from the mean was greater than three times the computer standard error.

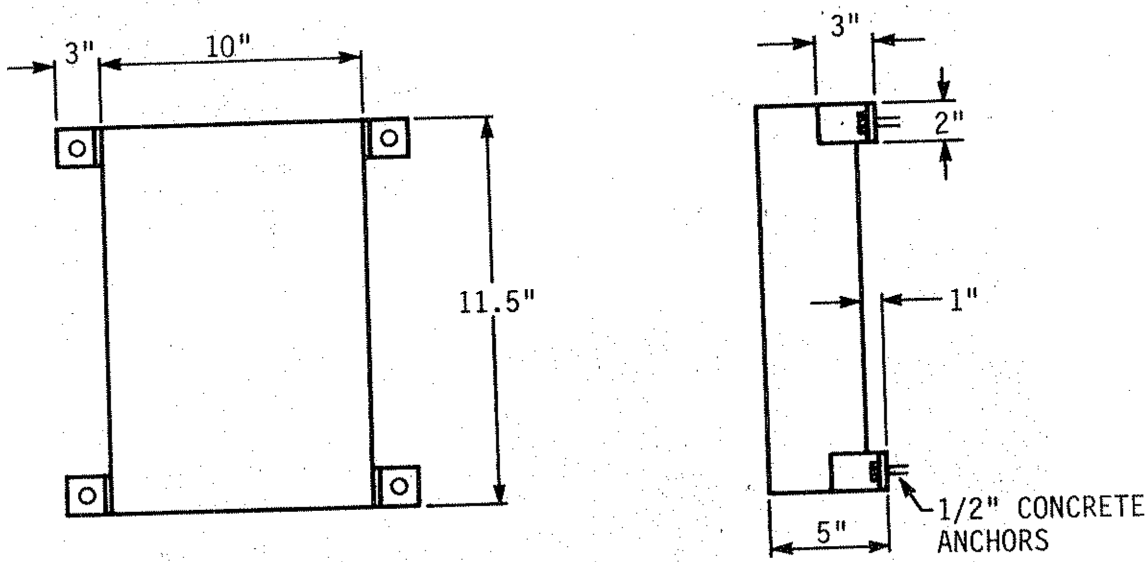


Fig. 11. Details of steel cover plates.

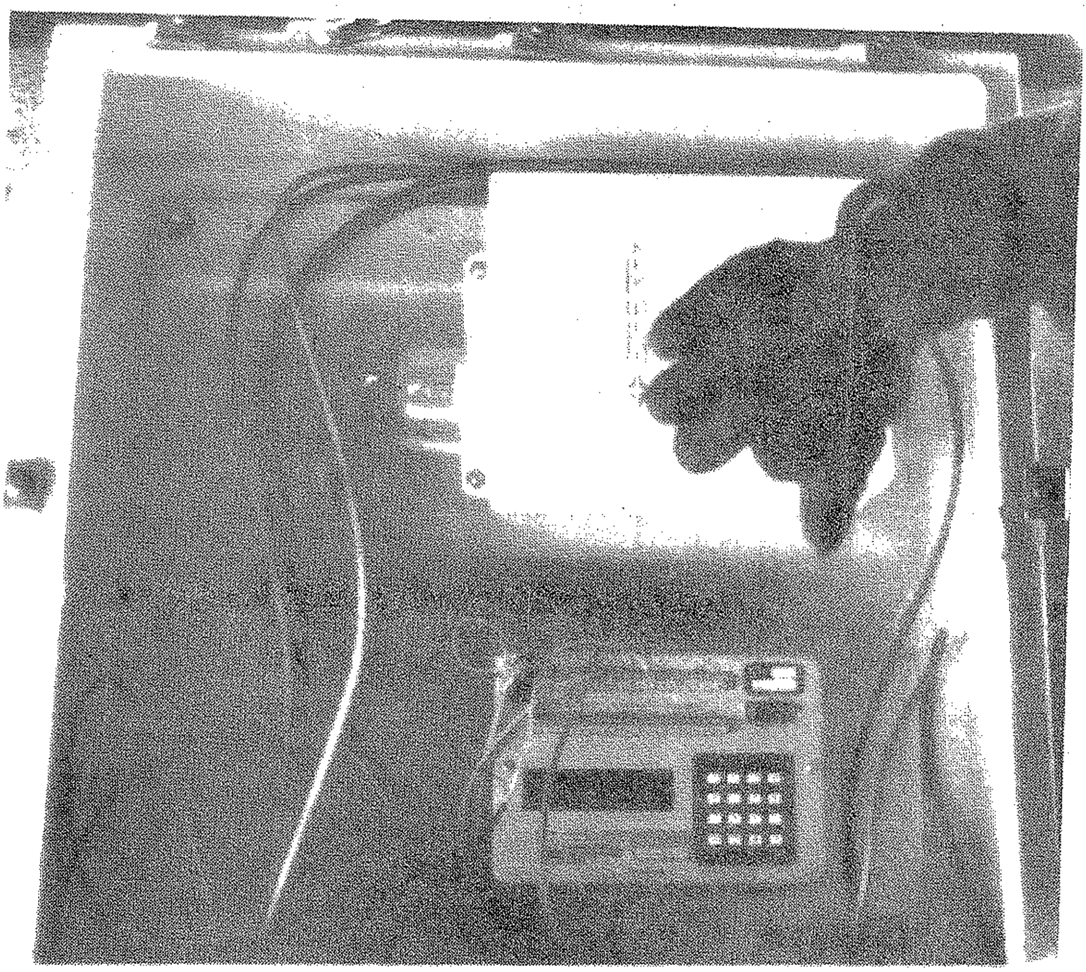


Fig. 12. Photograph of equipment inside the steel enclosure. (Channel expander not shown).

2.2.4.2. Lansing

Structural–An overall view of Pier No. 2 looking east is shown in Fig. 14. The two tilt sensors were mounted at the top of the north and west sides of the pier and can be seen along with the enclosure in the upper left of the pier (see Fig. 13). The console, micrologger, and modem were placed inside a water-tight steel enclosure similar to that used in Fort Dodge (as shown in Fig. 15). Two thermocouples were embedded 3 in. into the concrete adjacent to the north and west tilt sensors, respectively. The installation of all the instrumentation on this bridge was conducted by Cue Electric of Webster City, Iowa.

A telephone line was installed along the side of the bridge and connected to the modem inside the enclosure. Although the enclosure protected the equipment from environmental effects, the modem and telephone line connection apparently provided a pathway for lightning strikes to enter and damage the modem and micrologger. On a number of occasions, the modem and micrologger were damaged and had to be sent to the manufacturer for repair.

Surveying–The triple point method of surveying was also used in monitoring Pier No. 2 on three different dates. Each date's observations averaged 2 hours of leveling, 2 hours of EDM baseline measurement, and 5 1/2 hours of theodolite angle measurement. Operations began about 7:00 a.m. and ended about 4:30 p.m. each day. The field setup consisted of a baseline made up of three benchmarks located approximately midway between the railroad tracks (which are south of the river) and the river embankment on the Iowa side of the river. Each benchmark consisted of a concrete cylinder with brass cap and nail tip mark to represent the station. Four targets, T_1, T_2, T_3, and T_4, were painted on the pier as shown in Fig. 15. The elevations of the benchmarks were established by a level loop run from a nearby partially buried, rigid I-beam. The leveling misclosure was less than 0.01 ft. Angular observations that were taken of the pier targets were rejected if any direction difference from the mean was greater than three times the computed standard error.

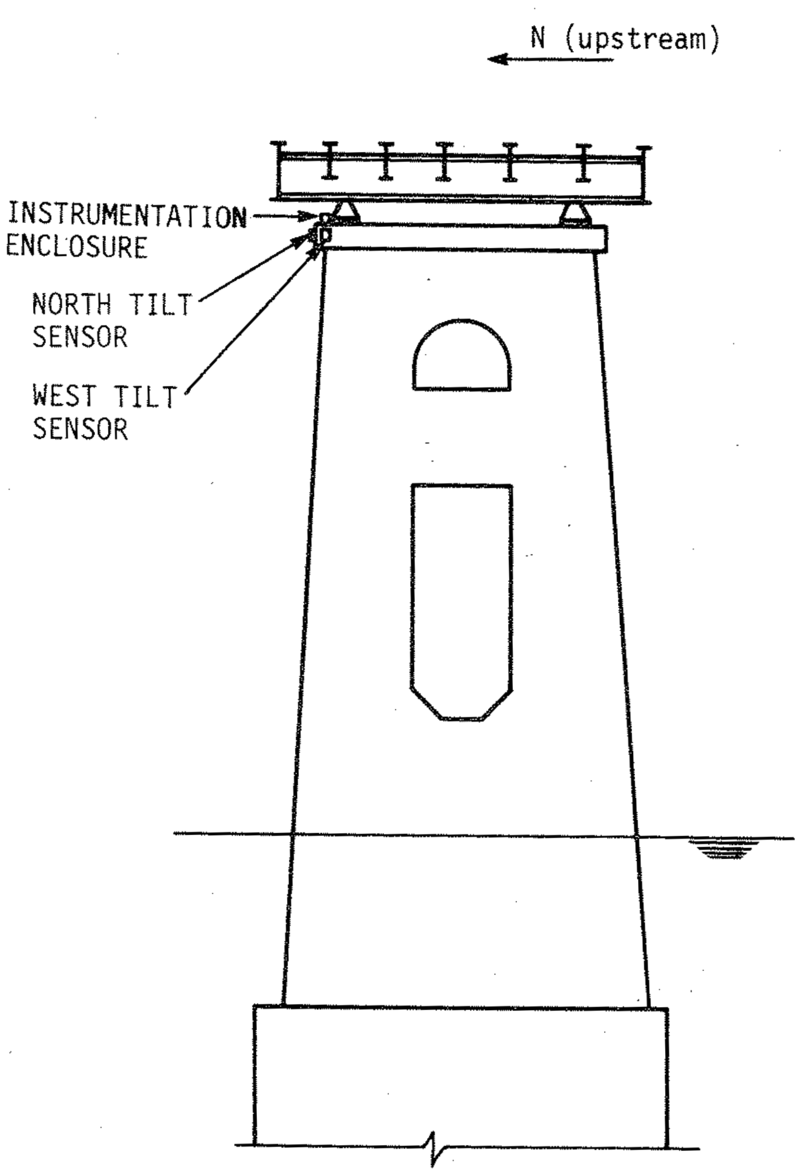


Fig. 13. Schematic layout of Pier No. 2.

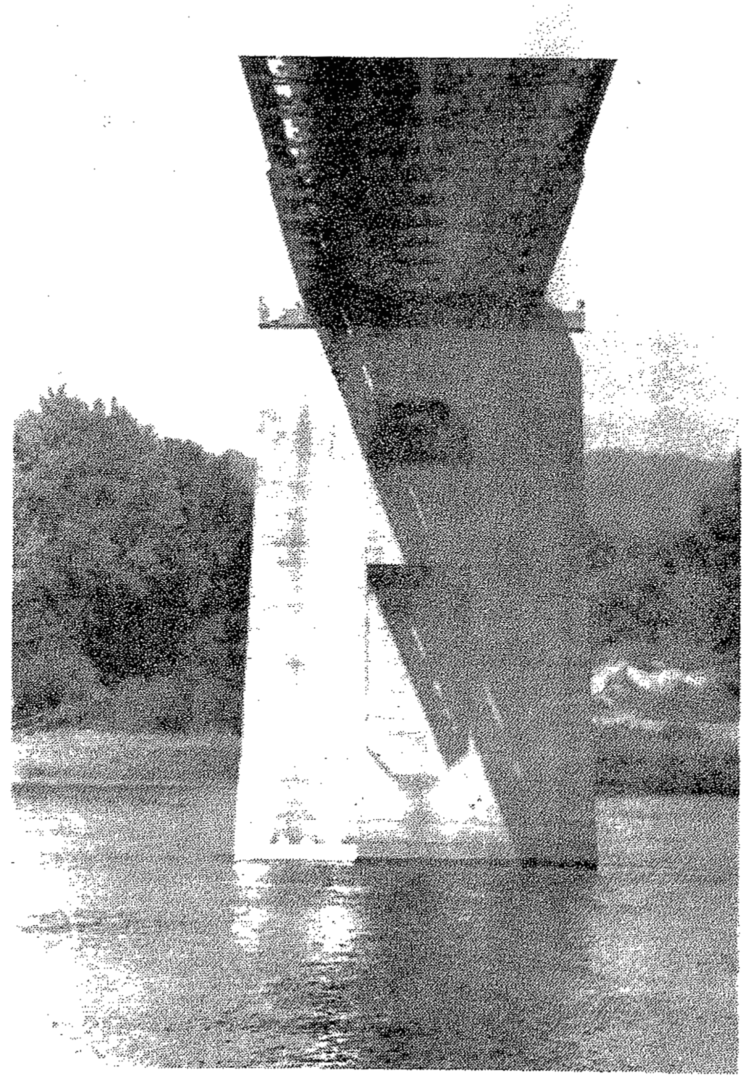


Fig. 14. Pier No. 2 (looking east).

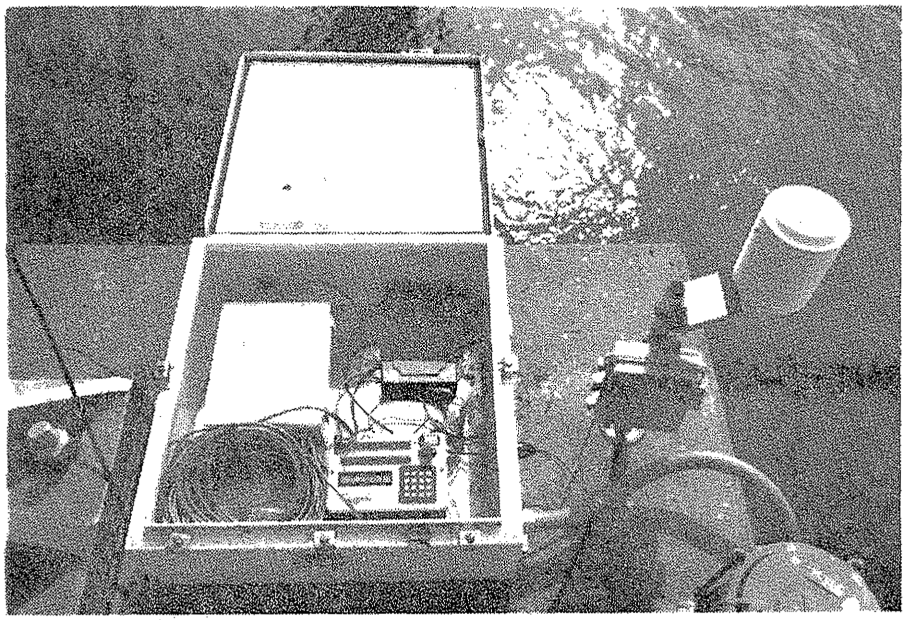


Fig. 15. Instrumentation inside steel enclosure.

3. ANALYTICAL MODELS

3.1. Introduction

Two finite element models idealizing Pier No. 4 and a portion of the superstructure of the Karl King bridge were developed utilizing Iowa State University's version of ANSYS [14]. ANSYS is a large-scale, general purpose finite-element program capable of solving several classes of engineering problems. The superstructure model idealized the span between Piers No. 4 and 5 (a total length of 153 ft) (see Fig. 16). Temperature data, obtained from the superstructure thermocouples in the field, were used in the model to determine expansion and contraction displacements and resulting longitudinal forces in the superstructure. Pier No. 5, a fixed pier regarding expansion and contraction, was assumed to be fixed against translation in the superstructure model.

The pier model idealized Pier No. 4, including the foundation, which consisted of spread footings on underlaid shale. Longitudinal forces obtained from the superstructure model were applied to the pier to predict rotations of the pier capbeam for comparison with the field tilt data.

The actual movement of the superstructure and pier is quite complex, and the simulation of their behavior must be made with a great degree of care. For this reason, a range of solutions was desired, which would represent upper and lower limits of movement. This required ranges of values to be selected for the parameters used in the computer simulation. Two important parameters in the analysis of the pier movement are the magnitude of force in the superstructure, which is dependent upon the restraint of longitudinal movement at the pier, and the foundation condition of the pier footing. Each parameter is discussed in following sections with rationale for the range of parameter values selected.

3.2. Superstructure Model

As mentioned previously, the superstructure consists of eight composite steel-plate girders. Typically, the cross-sectional properties of each girder vary throughout its length. For purposes of analysis, Pier No. 5, a fixed pier, was idealized as restrained against longitudinal translation. Pier No. 4, an expansion pier, was idealized as partially restrained against longitudinal translation, since the pier bearing devices were assumed to transfer at

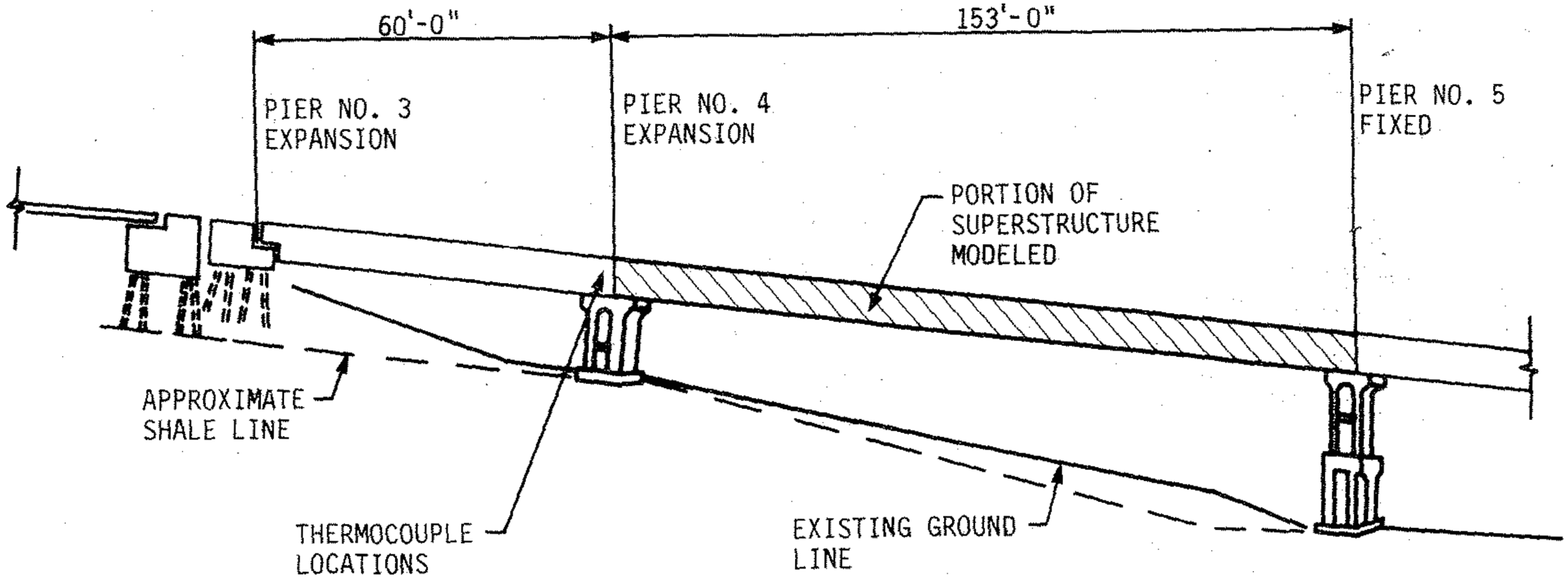


Fig. 16. Schematic of superstructure contained in computer model.

least a portion of the longitudinal forces from the superstructure to the pier capbeam. Ideally, the expansion bearing devices are assumed to allow free movement of the superstructure across the top of the pier capbeam.

Consistent with the literature contained in the literature review related to the thermal characteristics of bridge superstructures [5,7], the following assumptions were utilized in modeling the superstructure:

1. all longitudinal elements of the superstructure experience the same temperature variation
2. all cross-sectional elements experience the same temperature variation
3. the temperature is constant across the depth of the steel stringers
4. temperature varies linearly through the depth of the concrete slab

3.2.1. Description of Model Elements

The model was constructed by establishing a matrix of nodes connected by elements containing the properties of the superstructure. Each node contained six degrees of freedom, three translations and three rotations. Three types of elements were selected from the ANSYS element library to model the superstructure: three-dimensional beam elements, quadrilateral shell elements, and uniaxial tension and compression spring elements. Figure 17 shows a schematic of a portion of the superstructure model.

The three-dimensional beam elements were used to model the steel stringers and diaphragms. The modulus of elasticity, Poisson's ratio, and coefficient of thermal expansion of the stringers and diaphragms were taken as 29,000,000 psi, 0.3, and 0.0000065 in./in./°F, respectively. Quadrilateral shell elements were used to model the concrete deck. The modulus of elasticity, Poisson's ratio, and coefficient of thermal expansion of the concrete deck were assigned values of 3,372,000 psi, 0.2, and 0.0000055 in./in./°F, respectively. The thickness of the concrete deck was assumed to be 7 in. Values corresponding to the actual cross-sectional properties of the plate girders and diaphragms were used in the model. Uniaxial tension and compression spring elements were used to provide partial restraint against longitudinal translation of the superstructure at the Pier No. 4 end of the superstructure (shown in Fig. 18). The springs simulated realistic conditions of restraint that can practically occur. Bending and torsion were not considered, and only axial forces were accounted for.

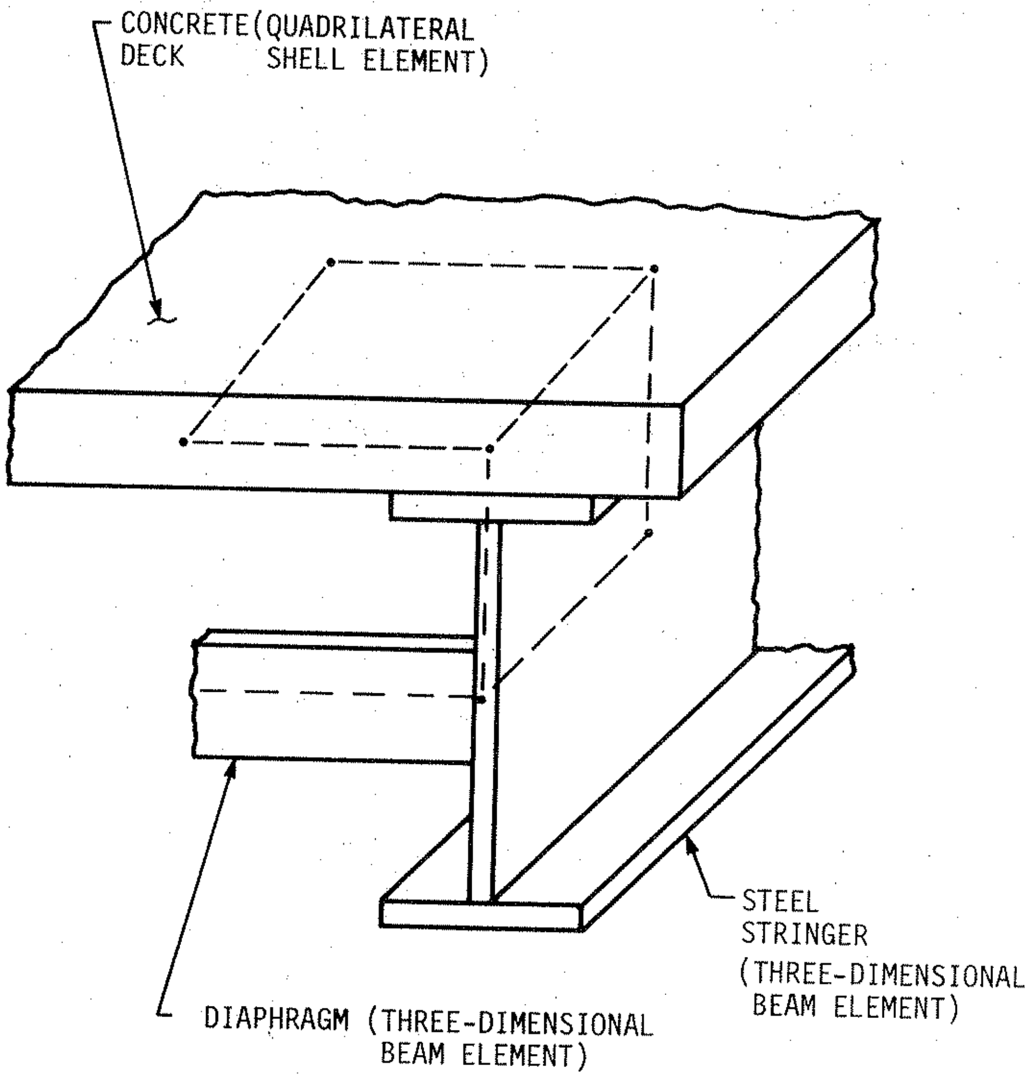


Fig. 17. Schematic view of elements in superstructure computer model.

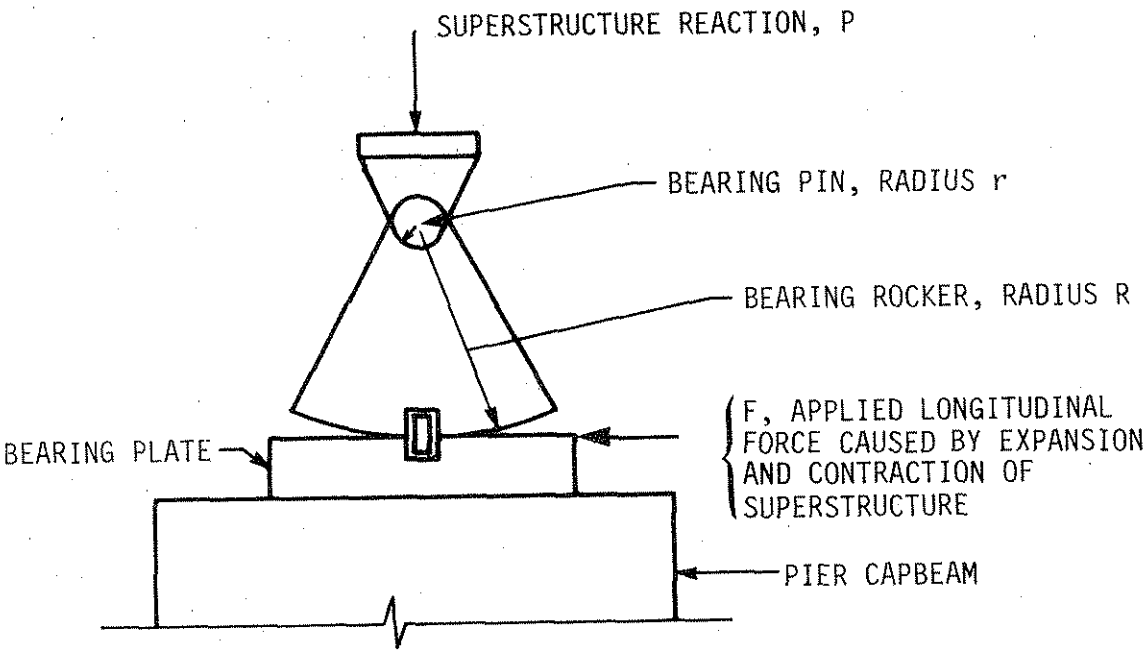


Fig. 18. Expansion bearing illustrating longitudinal forces transferred through bearing device.

An important parameter in the idealization of the superstructure model was the condition of restraint against longitudinal movement at Pier No. 4. The magnitude of the axial force developed in the superstructure is directly related to the magnitude of this restraint. Theoretically, the expansion-pier bearing devices allow only the transfer of longitudinal superstructure forces at the pier capbeam that are a function of the superstructure dead load, radius of the bearing pin and rocker, and coefficient of friction (see Fig. 18). These forces are constant over time, regardless of temperature differentials, and occur due to the superstructure expansion or contraction. Equation (1) is typically used to calculate these forces:

$$F = (P)(\mu)\left(\frac{r}{R}\right) \tag{1}$$

where

P = superstructure dead load

μ = coefficient of friction of steel = 0.25

r = radius of bearing pin

R = radius of rocker

Realistically, expansion bearing devices seldom function as described above. In many cases the devices may restrict, or at least partially restrict, rotation and subsequently cause horizontal forces to be transferred to the top of the pier from the axial forces developed in the longitudinally restrained superstructure. The magnitude of these forces is not constant over time, but rather is dependent on the change in temperature and the amount of end restraint created by the bearings. If the expansion bearing devices are assumed to be completely restrained against longitudinal movement, as in fixed bearings, the resistance against longitudinal movement of the superstructure would come from the flexural stiffness of the pier to which the longitudinal forces are transferred. In the analytical model of the superstructure this pier flexural stiffness was used to quantify the magnitude of the axial spring stiffness used to simulate longitudinal restraint. The total flexural stiffness of Pier No. 4, in the longitudinal direction of the superstructure, was calculated as 220 kips/in. However, to assign stiffness values to each of the eight steel stringers in the model, it was assumed that only a portion of the total pier stiffness was effective in resisting the longitudinal movement of the superstructure. Therefore, each stringer was assigned an axial spring stiffness value of 50 kips/in. To account for the uncertainty of the amount of restraint provided by the bearing devices, the spring stiffness was varied over a range of 30 to 90 kips/in. Little sensitivity was noted as the maximum difference in the calculated displacements over this range was 2.8%.

3.2.2. Model Assembly and Verification

The beam elements were placed at the elevation of the centroid of the steel stringers. Nodes were established along each stringer at each diaphragm connection, and at locations of cross-sectional property changes, as shown in Fig. 19.

The quadrilateral shell elements were placed at the elevation of the centroid of the concrete deck. The model assumed linear elastic behavior of the concrete deck [12]. The concrete deck was divided into eight unequal parts along the bridge length. Longitudinal divisions, established at locations where the cross-sectional properties of the steel stringers change, were symmetrical about the midspan of the model. Transverse divisions were made such that one shell element existed between adjacent stringers. Figure 20 shows the configuration of the quadrilateral shell elements. The spring elements were connected to both the beam and quadrilateral shell elements at the end of Pier No. 4 on the model. The beam elements were rigidly linked to the shell elements through master-slave node relationships (see Fig. 21).

Verification of the superstructure model was accomplished by releasing the partial restraint at Pier No. 4 against longitudinal translation and replacing the uniaxial spring elements by simulated roller supports. A temperature change of 74° F was applied to the steel stringers and the diaphragms. The concrete slab was subjected to a linear temperature gradient of 79.2° F at the top and 77.3° F at the bottom. Displacements of the steel stringers at Pier No. 4 were calculated and compared with the deformations obtained using the expansion formula (Eq. 2):

$$\delta = \alpha \, L \, \Delta T \tag{2}$$

where

α = coefficient of thermal expansion/contraction

L = length of model span

ΔT = temperature change

Both methods indicated a deformation of 0.883 inches in the 153 ft span.

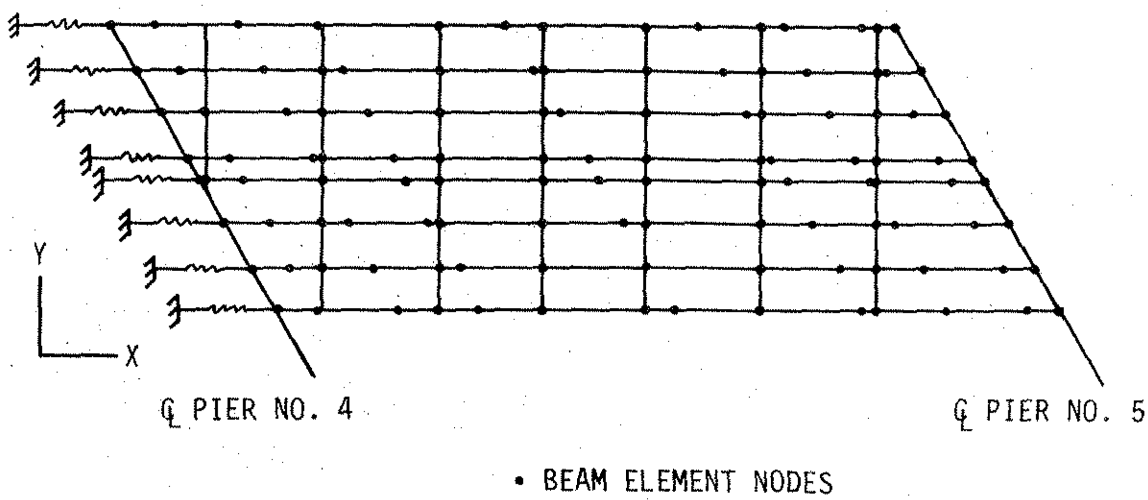


Fig. 19. Superstructure beam elements configuration.

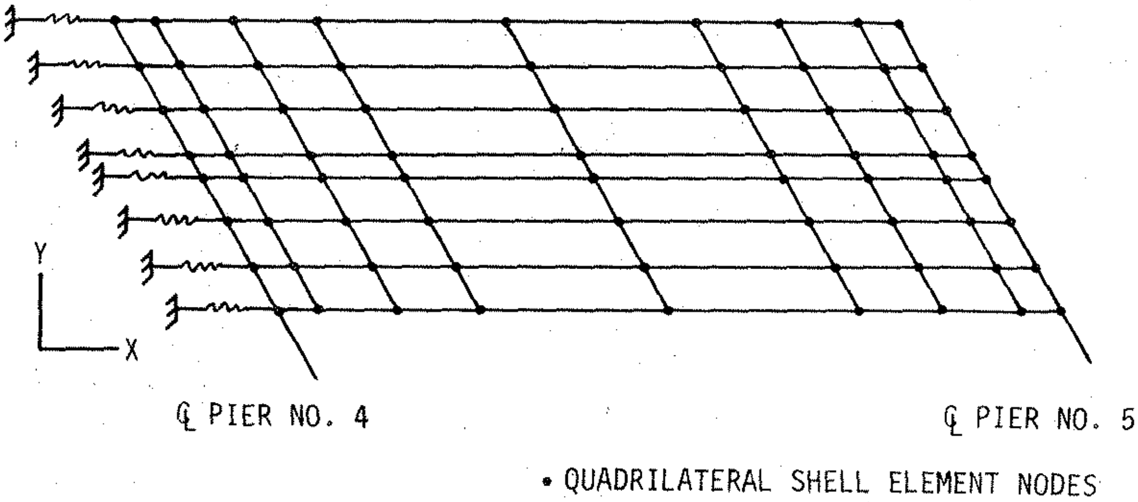


Fig. 20. Superstructure quadrilateral shell elements.

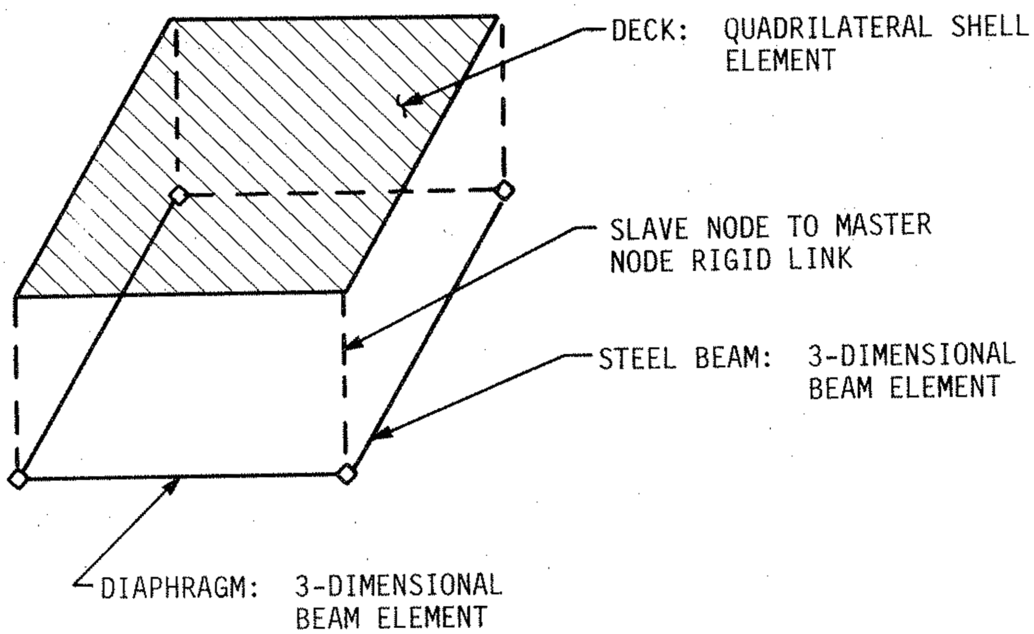


Fig. 21. Computer model detail illustrating connectivity of deck to stringers.

3.3. Pier Model

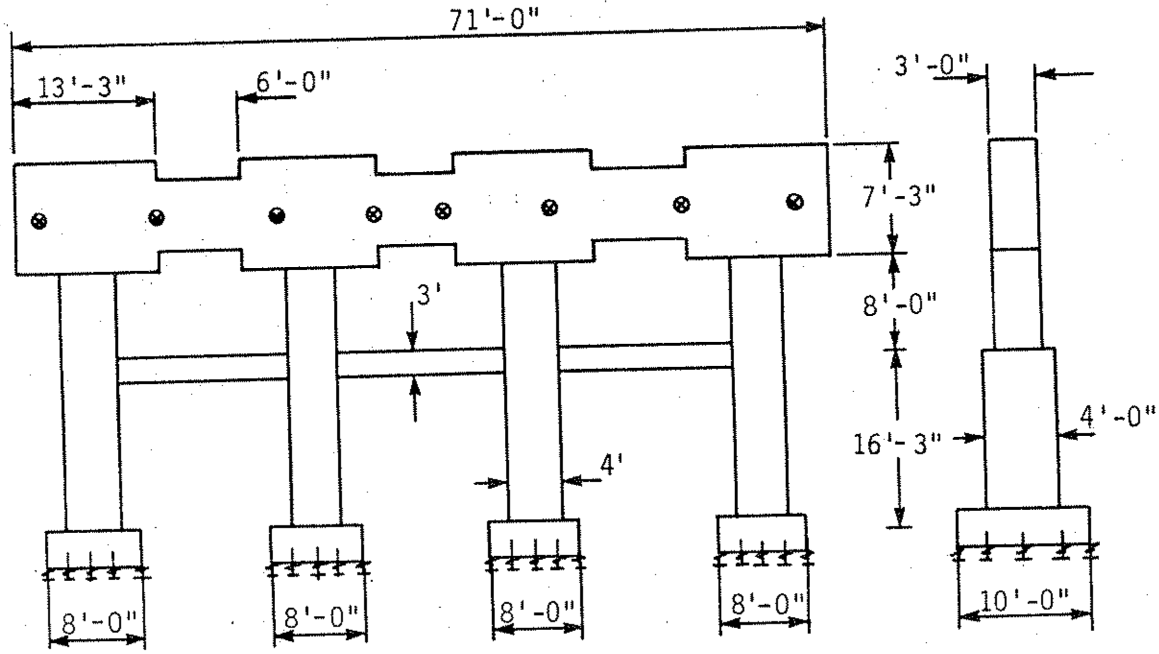
The pier model simulated Pier No. 4, including the foundation (see Fig. 22). As mentioned previously, this pier is located on a hillside underlaid with shale. Because of the geometric irregularities of the pier, and the uncertainty of the foundation stability, the following simplifications were made:

1. the concrete collars placed for maintenance purposes at the bottom of the north and south columns were ignored
2. the columns were idealized as nonprismatic concrete members
3. the notches at the top of the pier capbeam were eliminated
4. nonprismatic concrete members were used to model the pier capbeam with shallower members at the location of the arches
5. elastic springs were attached to the footings to simulate foundation support

3.3.1. Description of Model Elements

Three-dimensional beam elements were used to model the columns, footings, diaphragms, and pier capbeam. The modulus of elasticity and Poisson's ratio of these elements were assigned values of 3,372,000 psi and 0.2, respectively.

Uniaxial tension and compression elements were used to simulate an elastic foundation condition. These springs allowed only axial forces and ignored bending or torsion effects. Five springs were symmetrically spaced along the two major axes of each footing (see Fig. 23). An axial spring constant representing the stiffness of the soil was based upon the modulus of subgrade reaction. It was mentioned earlier that an important parameter in describing the pier foundation behavior was the soil condition beneath the footing. The shale, which underlaid the footings of Pier No. 4, was assumed to be uniformly distributed across the whole footing area. Soil profiles from soil borings on construction plans showed that the shale layer was significantly deep in this region. The profiles showed layers of soil that were described as soft, medium hard, and medium hard to hard shale. The upper layers in the boring nearest Pier No. 4 also showed relatively thin layers (1 to 3 ft) of stiff silty clay. Although the shale is a densely packed clay and silt material, and relatively stiff compared to other soil types, the material still exhibits properties that may be simulated as an elastic supporting material.



⊗ POINTS OF APPLICATION OF SUPERSTRUCTURE FORCES

Fig. 22. Idealized model of Pier no. 4.

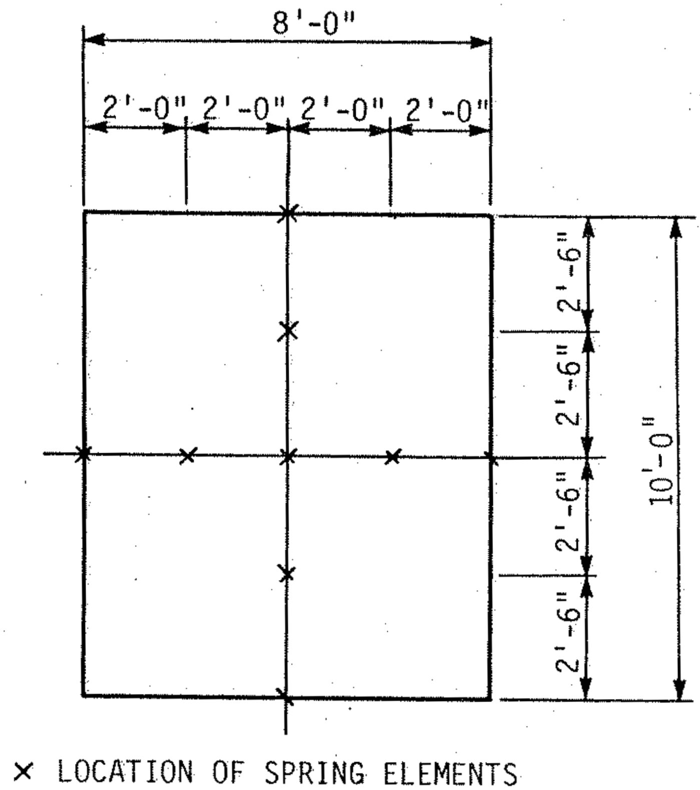


Fig. 23. Plan of footings showing location of soil springs.

The elastic springs that were used to idealize the supporting soil were assigned axial stiffnesses based on an assumed range of magnitude of the modulus of subgrade reaction for shale material. The modulus of subgrade reaction describes the ratio between the unit soil pressure and the corresponding settlement. Based on the moduli, the axial spring stiffness was calculated by multiplying the modulus of subgrade reaction by the contributary area assigned to each spring. The range of values selected for the subgrade reaction was 3000 to 6000 k/ft^3 for a lower limit and 300,000 k/ft^3 for an upper limit [13]. Based on these values, the spring stiffnesses assigned were 2000 k/in. and 200,000 k/in. In addition to these ranges of values (which represent the range of very flexible to very stiff shales), a value which represents the possibility of a pocket of soft material, such as clay, in the shale, was considered. Since no cores were taken in the actual footing areas, this was a realistic possibility. The stiffness value assigned to the footing springs to simulate this condition was 400 k/in.

4. TEST RESULTS AND ANALYSIS

4.1. Reduction of Tilt Data to Linear Displacements

In the following sections, field data are presented in units of arc min of tilt as they were recorded by the instrumentation. In order to quantify the pier tilt in terms of a linear displacement of the pier, assumptions must be made regarding the overall local movement behavior of the pier. Two conditions of movement of the pier were considered: rigid body rotation in a vertical plane and flexural deformation caused by curvature of the pier. These two conditions represented upper and lower ranges of possible linear displacement, respectively, of the pier. The rigid body assumption was applied by assuming the pier was pinned and free to rotate at the footing. The application of simple triogonometric relationships allowed calculation of the linear displacement at any point on the pier. The pier curvature assumption was applied by assuming that the pier was completely fixed against rotation at the footing and by applying principles of structural analysis for flexural curvature. Calculation of the linear displacement at the top of the pier at the tilt sensor location could then be made.

Based on the above assumptions, ranges of linear displacements of the piers at the tilt sensor location for 1 arc minute of tilt are given in Table 1. This coefficient is assumed to be linear and therefore may be applied to any other tilt reading by proportion. The upper limit values for both bridges, which are based on rigid body rotation (0.10 in. for Karl King pier and 0.27 in. for Black Hawk pier), may be most applicable in cases where the alignment of the pier would change due to a foundation settlement. The lower limit values may be most applicable for cases of direct application of forces, such as those due to expansion or contraction of the superstructure, since the pier behaves as a flexural member resisting load.

As mentioned previously, the tilt sensor readings provided information regarding the vertical alignment of the surface to which they were attached. Hence, the tilt sensor reading at any given time represented the angular position of the pier capbeam with respect to gravity. Any type of pure translational movement of the pier was not registered by the tilt sensors. The only way to record a movement of this type is by direct linear measurement from some stable reference point to a point on the pier. We believe that the stability of this reference point would be at least as questionable as the conversion of measured tilt to linear displacement. Gravity, on the other hand, serves as a very stable reference point. There is a tradeoff between accepting the advantage of a stable reference point versus a direct linear-displacement measurement. The research team believes that the assumption used to convert

Table 1. Correction coefficients for converting measured tilt readings to linear displacements.

Measured Tilt (arc min)	Pier No. 4 Karl King — Linear Displacement (in.)		Pier No. 2 Blackhawk — Linear Displacement (in.)	
	East-West	North-South	East-West	North-South
1	0.07–0.10*	0.01–0.10*	0.15–0.27*	0.03–0.27*

*Based on assumption of rigid body rotation.

Note: Linear displacements shown in table correspond to the location of the tilt sensors on the pier.

the tilt data to actual linear displacements is based on sound principles. These principles, when used in interpreting tilt data and taking into account the limitations in the conversion process, may provide accurate records of bridge movement.

4.2. Karl King Bridge

4.2.1. Structural

The installation of equipment was completed in December of 1986 and the tilt sensors were zeroed on January 3, 1987. The data recording process was started on January 7, 1987, and continued through April of 1988. As mentioned previously, data on the pier were collected from the four tilt sensors mounted on the pier capbeam (see Fig. 10), two thermocouples embedded into the concrete near the north and east tilt sensors, and the ambient temperature probe. All of the tilt data, accumulated throughout the duration of the project, were based on the initial reference established on January 3, 1987.

As shown in Fig. 10, the north and south tilt sensors are on opposite faces of the capbeam, and the east and west sensors also occupy opposite faces. The sensors were designed so that a clockwise rotation (of the sensors) represented a positive magnitude of tilt. Therefore, the orientation of the pairs of sensors (north-south and east-west) caused a magnitude of recorded tilt that was equal in magnitude, but opposite in sign (assuming, of course, that the pier moved as a unit equally in north-south and east-west directions). In order to allow a clearer comparison between the pairs of sensors, the tilt readings that are presented in the tables and graphs following have been assigned the same sign to indicate movement in a common direction. Actually, the south and west tilt-sensor readings represent recorded field magnitude, but are negative algebraic values.

Based on the location of the tilt sensors on the capbeam shown in Fig. 10, the north-south sensors provide data regarding movement of the pier in an east-west direction; the east-west sensors provide data regarding north-south movement. Discussion of movement in this section will also be referenced to the superstructure, as well as to the pier. Note that a skew between the pier and the superstructure made it necessary to convert a pier movement to a longitudinal stringer movement based on the skew geometry. In general, the superstructure longitudinal movement corresponded primarily to a movement in the east-west direction (north-south tilt readings); the transverse movement refers to a north-south direction (east-west tilt readings).

The superstructure thermocouples were installed in March 1987 and data recording started on April 15, 1987. The micrologger was programmed to record all data on an hourly basis. Due to storage limitations, the micrologger allowed retention of data in final storage for approximately 30 days before writeover occurred. Therefore, a trip to the bridge location was scheduled every four weeks to download the micrologger manually, using a computer cassette recorder and necessary interface devices, and to replace the batteries of the central console unit for the tilt sensors. On a number of occasions, weather conditions dictated delaying the trip, which resulted in losing parts of the data. In addition, component failure of the console unit and the micrologger caused the loss of data during June, part of July, and August of 1987. However, since the position of the tilt sensors was not altered during the monitoring period, the loss of data did not disturb the reference established for readings of the tilt sensors, and the continuity of the monitoring process was maintained.

4.2.2. Daily Behavior

Readings of the four tilt sensors were plotted on a daily basis. Close examination of these daily graphs and the ambient temperature records revealed a close correlation between the readings of the north and south tilt sensors and of ambient temperatures. These sensor readings correspond to a pier movement that has its major component in the longitudinal direction of the bridge superstructure, or in the longitudinal direction of the bridge stringers. A few arbitrarily selected plots are presented for discussion. Figure 24 represents the

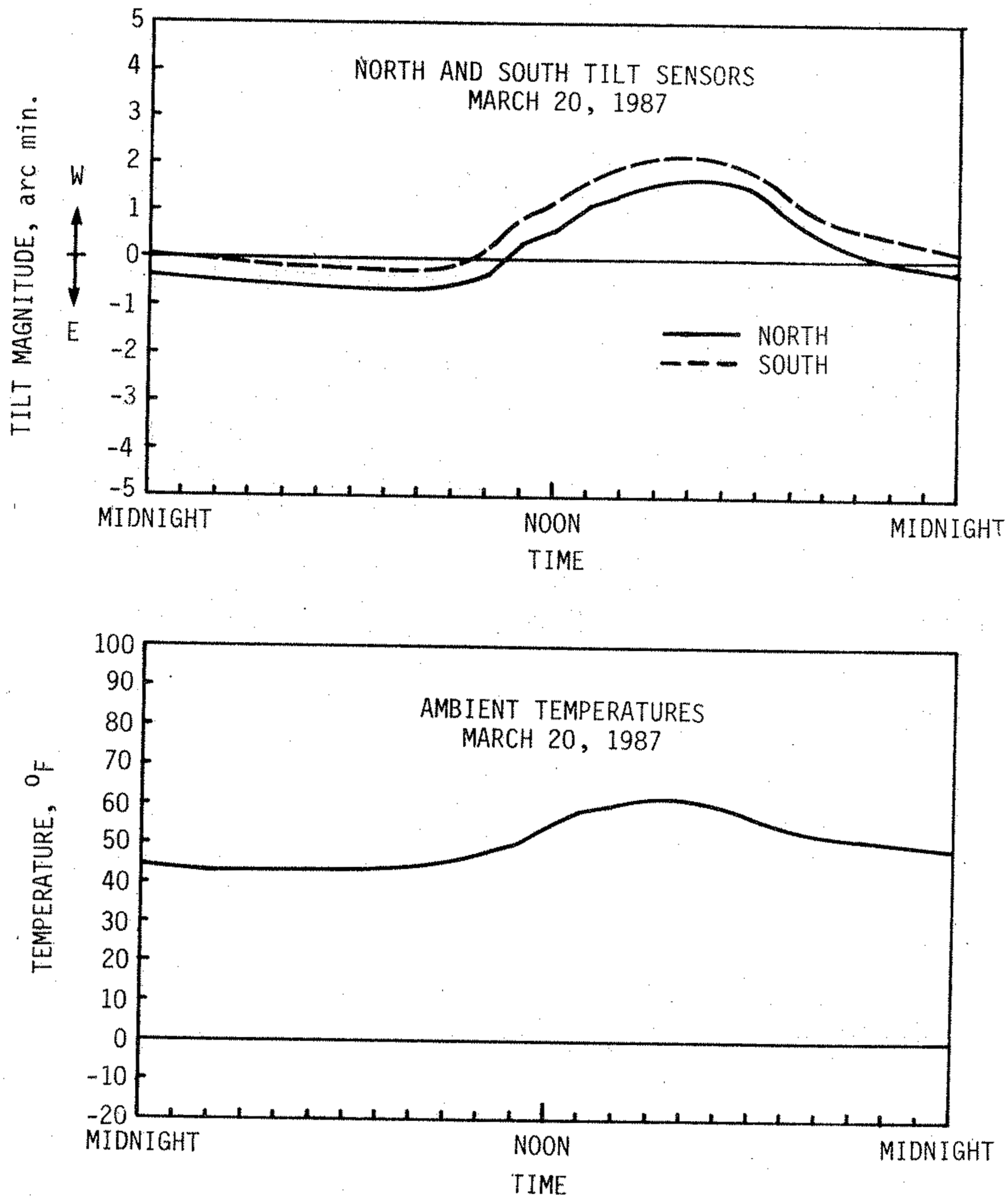


Fig. 24. Readings of north and south tilt sensors and of ambient temperatures on March 20, 1987.

readings of the north and south tilt sensors, as well as ambient temperatures, on March 20, 1987. The tilt data indicated that the pier capbeam experienced a rotation toward the east during the early morning hours. Between midmorning and midafternoon, as the ambient temperature increased, the pier again experienced a larger rotation–this time toward the west. As noted in the figure, the readings of the north and south tilt sensors were consistently different by approximately 0.50 arc min throughout the day. This difference implied that, throughout the day, the capbeam at the south end of the pier was positioned more toward the west than was the north end. Figure 25 represents the readings of the north and south tilt sensors and of the ambient temperatures on May 15, 1987. The graphs follow the same general behavior indicated in Fig. 24; however, the magnitude of the westward rotation was larger than that illustrated in Fig. 24, and the difference between the readings of the north and south tilt sensors increased to approximately 0.8 arc minute. Note that the maximum change in temperature recorded on March 20 was 20° F, while the maximum recorded on May 15 was 30° F.

Figure 26 shows a plot of the north and south tilt sensors, as well as ambient temperatures for January 21, 1987. The trend of the movement following the ambient temperatures is again illustrated. It is interesting to note that in this graph the north and south tilt readings did not differ as significantly, as shown in Figs. 24 and 25. Reasons for this will be discussed in detail in a later section. The maximum change in temperature recorded on this day was 15° F.

To contrast the above behavior, we noticed that on days where the ambient temperature remained essentially constant, the pier experienced very little change in position in the east-west direction. A typical example illustrating this is shown in Fig. 27, which represents the readings of the north and south tilt sensors and of ambient temperatures on April 14, 1987. The maximum change in temperature on that day was 2° F.

Figures 28 through 31 illustrate the corresponding readings of the east and west tilt sensors and of the ambient temperatures for the same four days just presented. These readings indicate movement with a major component corresponding primarily to the transverse direction of the superstructure. The figures indicate that the pier experienced very little rotation in the north-south direction, despite the relatively large temperature changes recorded on some of the days.

Further examination of daily graphs of tilt sensor readings for other days over the duration of the project indicated that the pier consistently followed the general behavior discussed above. That is, the pier experienced a variable magnitude of rotation in the east-west

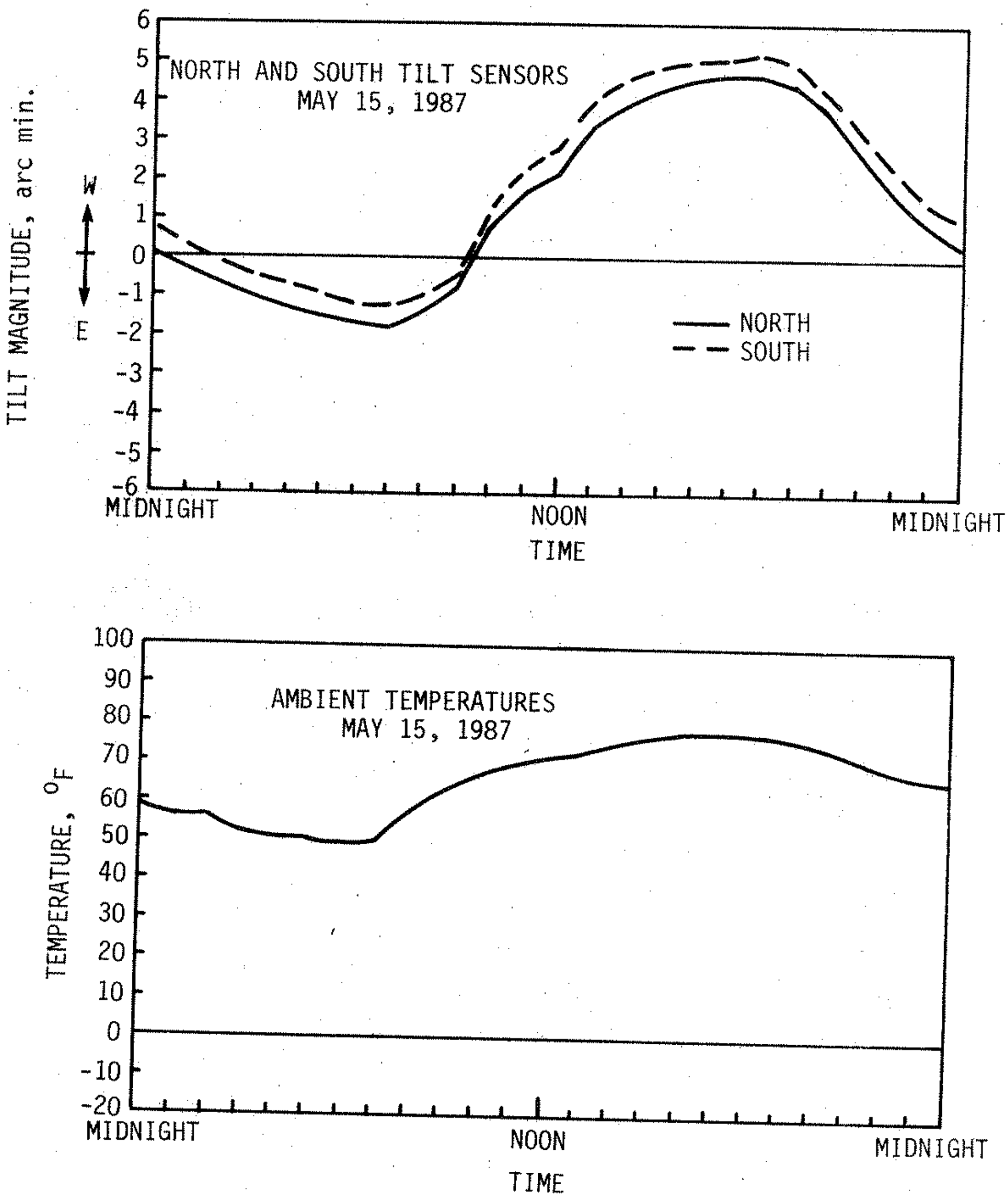


Fig. 25. Readings of north and south tilt sensors and of ambient temperatures on May 15, 1987.

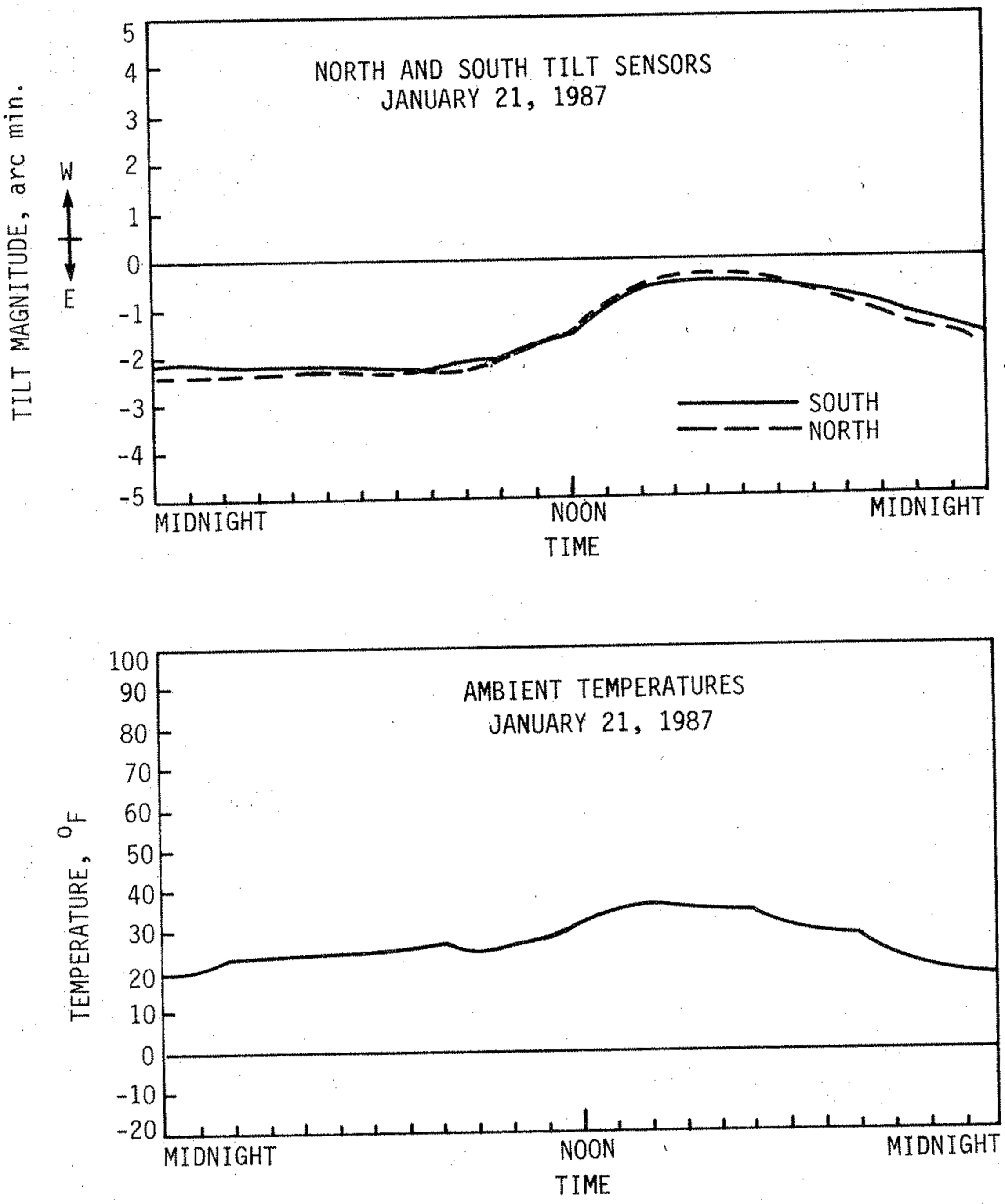


Fig. 26. Readings of north and south tilt sensors and of ambient temperatures on January 21, 1987.

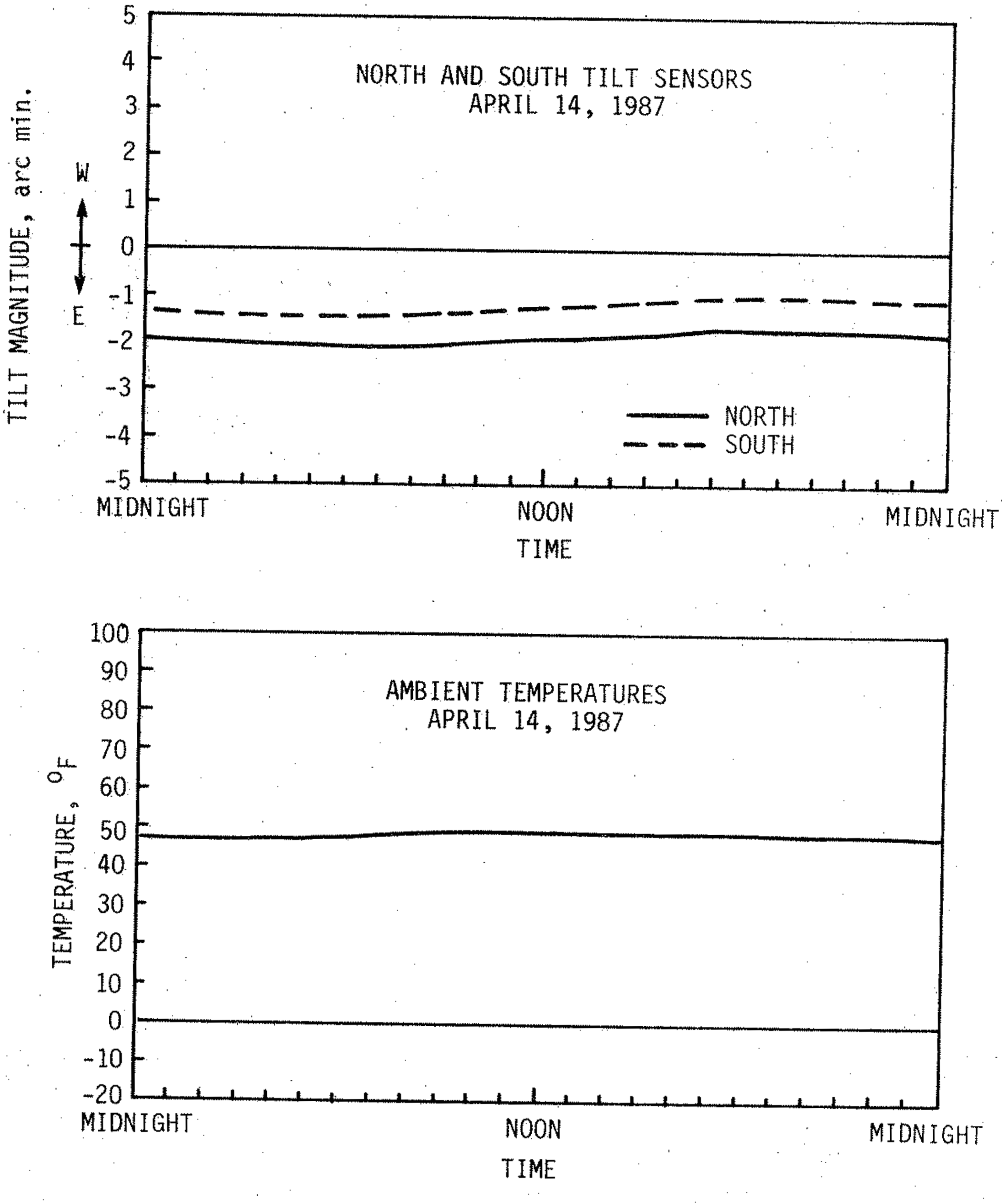
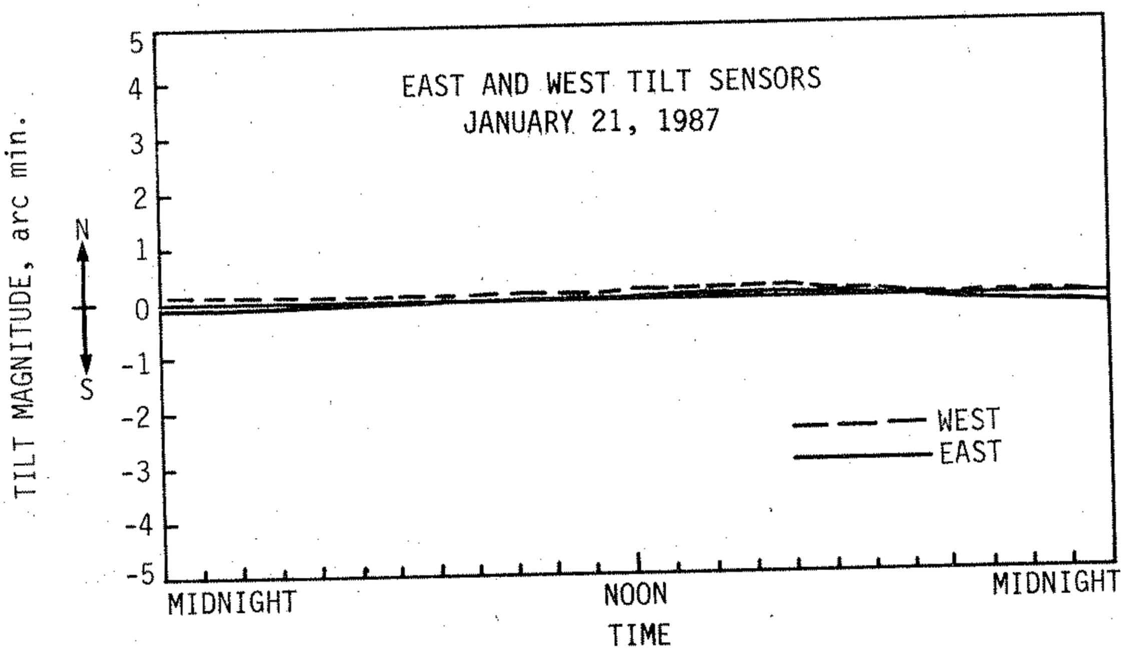


Fig. 27. Readings of north and south tilt sensors and ambient temperatures on April 14, 1987.



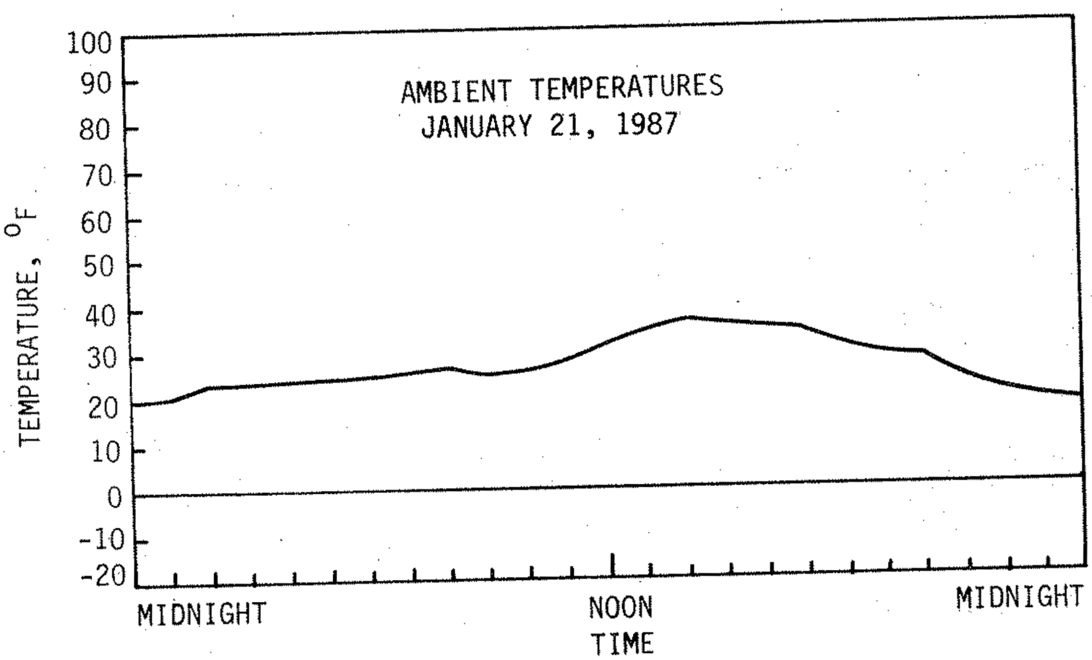


Fig.28. Readings of east and west tilt sensors on January 21, 1987.

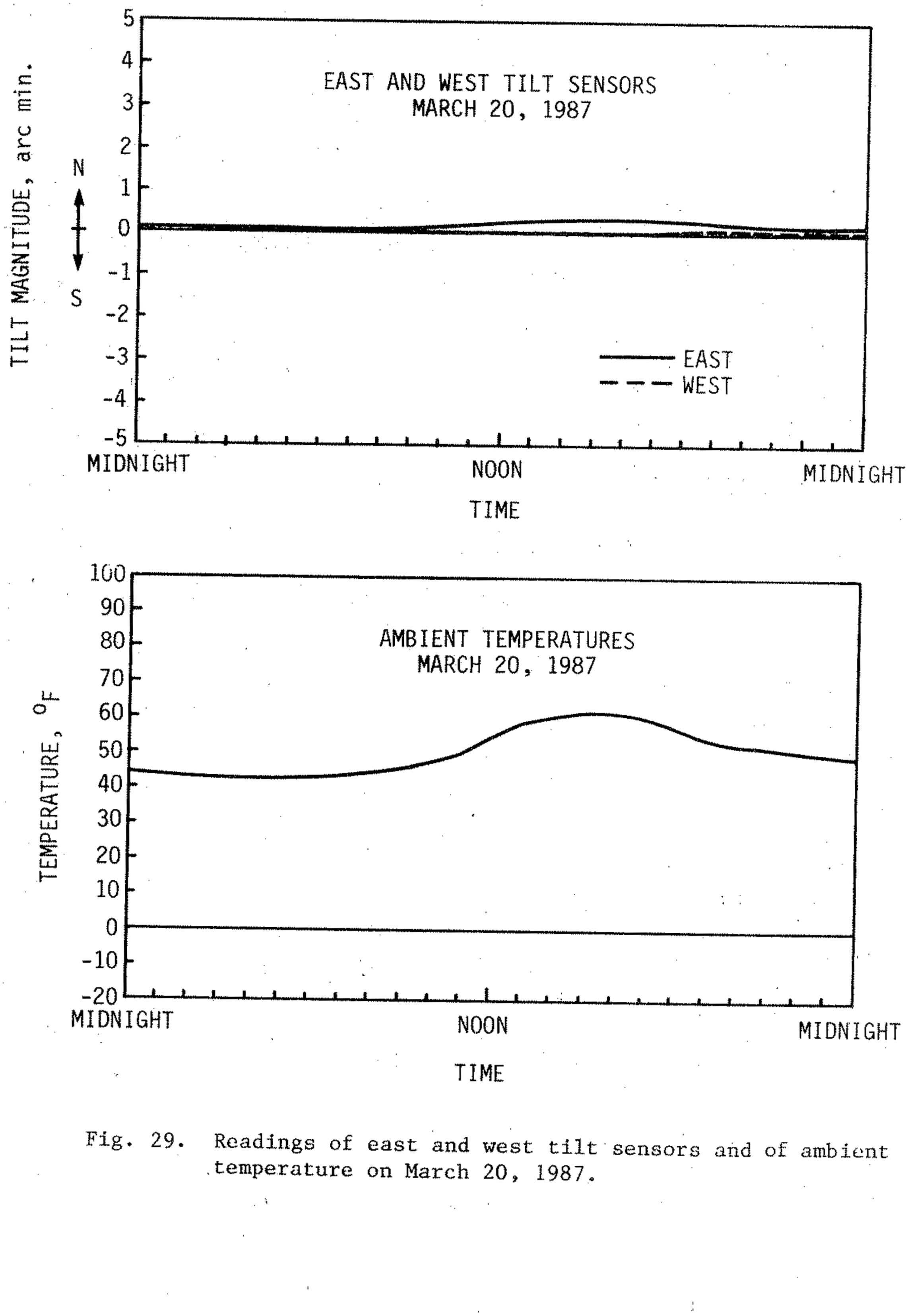
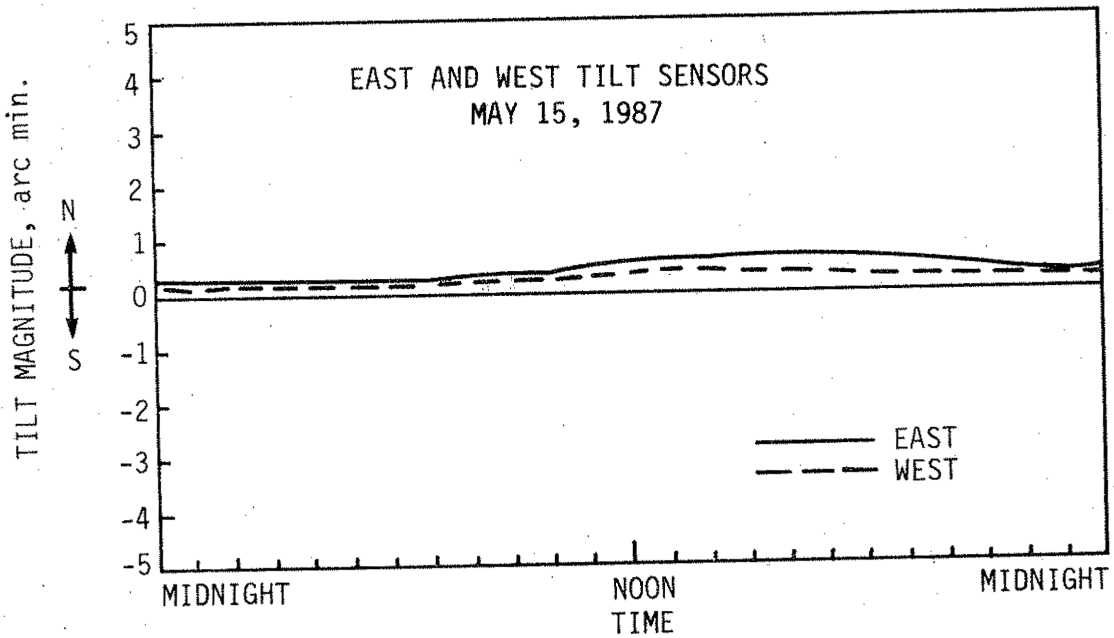


Fig. 29. Readings of east and west tilt sensors and of ambient temperature on March 20, 1987.



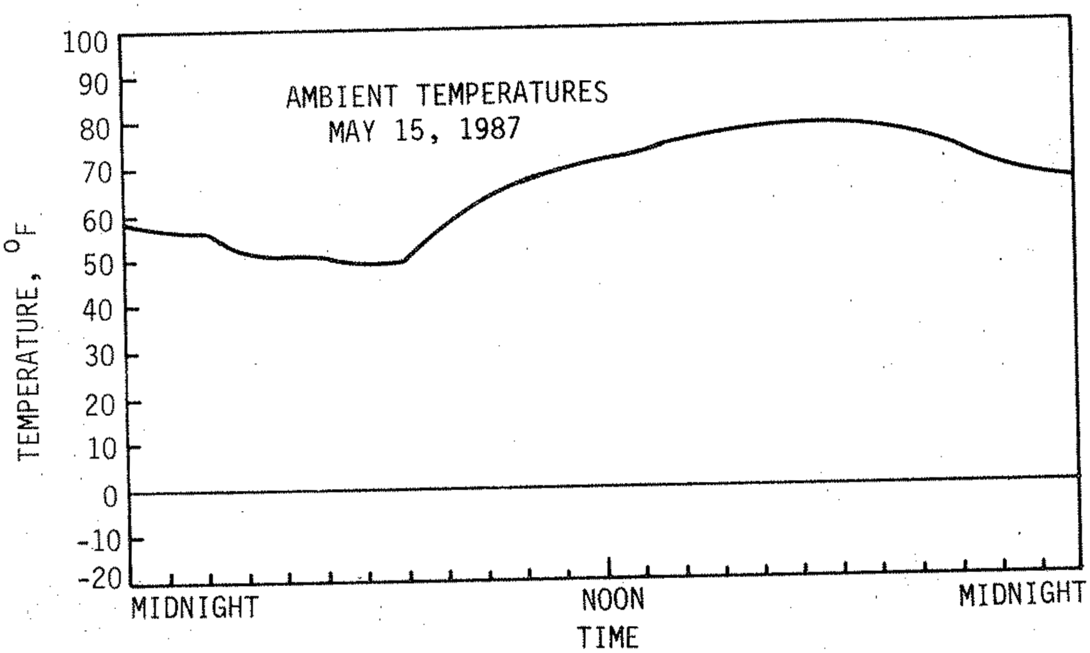


Fig.30. Readings of east and west tilt sensors on May 15, 1987

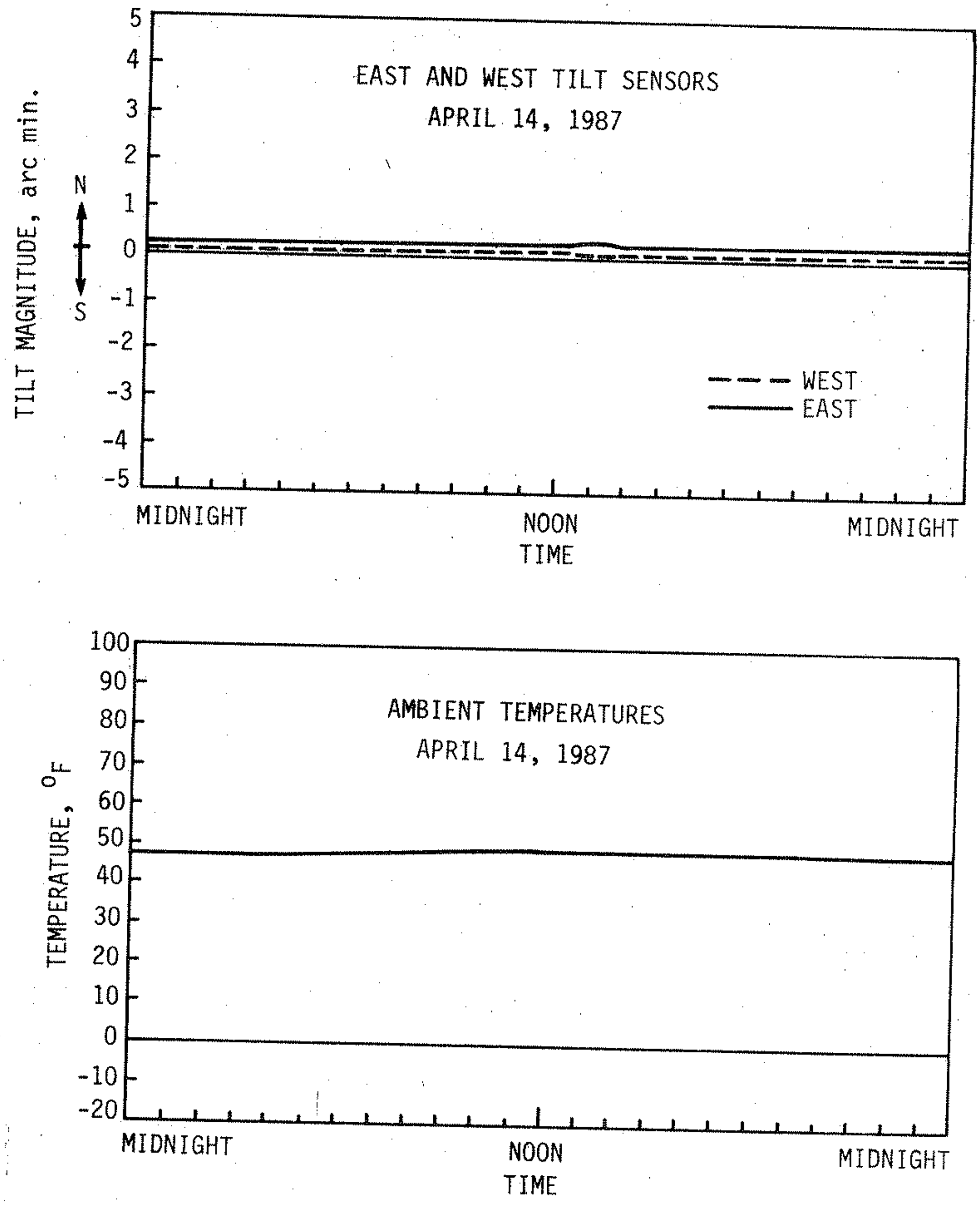


Fig. 31. Readings of east and west tilt sensors on April 14, 1987.

direction proportional to the corresponding, maximum daily temperature differentials and negligible rotations in the north-south direction. The position of the south end of the pier capbeam was consistently more westerly than the north end only beginning in mid-March. Prior to this time very small differences between north and south readings were noted. The magnitude of the differential rotation between the north and south ends on a daily basis increased gradually to a maximum of 0.80 arc min in May 1987 and remained constant thereafter. This effect is illustrated in a later section on the seasonal behavior of the bridge, where it will be discussed in greater detail.

As the plots indicate, the daily behavior of the movement of the pier in the superstructure longitudinal direction was directly related to daily temperature variations. The field temperature data were reviewed to provide a basic understanding of the relation between this movement and temperatures. As shown in Fig. 10, in addition to ambient temperature, data were obtained for both the steel and concrete near Pier No. 4. Typical temperature data are presented along with a brief discussion of the general trends noted in the data. No attempt was made to provide an in-depth study of the localized effects of temperatures on the overall movement of Pier No. 4. The data in the tables are presented to provide both an overview of temperature effects and a summary of the daily temperature variations that were found in reviewing the temperature data during the monitoring period. For the objectives in this study, the general behavior of the pier, however, might still be described as being directly dependent on temperature, regardless of whether correlation is being made with superstructure steel, concrete, or ambient temperature.

The temperature field data typically showed that changes in bridge temperature lagged behind changes in ambient temperature, and that this lag was different relative to the concrete and steel in the superstructure. Table 2 lists a set of typical daily data illustrating the differences between the superstructure steel and concrete temperatures and ambient temperatures for two arbitrarily selected days when the ranges in temperature were significant. The days represented in this table were characterized as being mostly sunny. The tabulated temperatures for superstructure steel and concrete represented the average temperatures on the cross section of the bridge. As indicated in the table, the concrete and steel temperatures were generally higher than ambient temperatures in the early morning and late evening hours, but less than ambient temperatures in the middle of the day. This implied that the extremes of ambient temperature during the course of a day are greater than those for the concrete and steel superstructure. Table 3 summarizes temperatures on a day in which the range of ambient temperatures was small. That day was mostly cloudy throughout

Table 2. Comparison of ambient temperatures and average concrete and steel temperatures for May 9 and September 7, 1987.

Date	Time (hour)	Ambient Temperature, °F	Average Concrete Temperature, °F	Average Steel Temperature, °F
5-9-1987	2	61.0	67.0	73.1
	4	60.4	65.3	70.4
	6	58.5	64.0	67.8
	8	63.9	63.3	65.6
	10	75.3	66.0	67.6
	noon	80.8	70.6	73.3
	2	85.6	74.2	78.6
	4	87.9	77.3	83.2
	6	86.5	79.5	86.1
	8	81.9	83.0	86.8
	10	76.4	79.2	85.3
	midnight	72.6	76.7	82.5
9-7-1987	2	61.8	69.0	70.3
	4	60.5	68.4	68.6
	6	59.9	67.8	67.2
	8	60.0	67.4	65.8
	10	67.0	67.8	66.7
	noon	72.9	68.7	70.0
	2	76.7	69.8	73.9
	4	67.3	70.3	75.1
	6	72.3	70.6	75.8
	8	68.1	70.3	75.8
	10	65.5	69.8	74.4
	midnight	62.1	68.9	71.5

Table 3. Comparison of ambient temperatures and average concrete and steel temperatures for May 24, 1987.

Date	Time (hour)	Ambient Temperature, °F	Average Concrete Temperature, °F	Average Steel Temperature, °F
5-24-1987	2	58.1	60.8	63.7
	4	56.7	59.9	62.5
	6	54.8	59.0	61.1
	8	53.8	58.4	58.9
	10	54.3	58.6	57.5
	noon	53.9	58.2	56.6
	2	54.6	58.4	56.4
	4	53.3	58.1	56.1
	6	54.9	57.9	56.1
	8	55.9	57.7	56.4
	10	56.1	57.4	56.5
	midnight	54.0	56.8	56.0

the day. As shown, there was a different overall behavior between the three temperatures–concrete, steel, and ambient. The temperature differences were relatively constant throughout the day, and the ambient temperatures were lower at all times.

Additional typical localized temperature data are provided for general consideration in Tables 4 and 5. Shown is a summary of temperature data across the width of the bridge near Pier No. 4 at various hours throughout the day. April 27, 1987, was arbitrarily selected as one date for the data, which are shown in Table 5. These data are typical for most of the other days that were monitored during the spring, summer, and early fall seasons. As shown in the table, from midnight to noon the concrete slab temperatures were very similar. The differences became greater through the afternoon, sometimes approaching 10°, until equalizing again toward late evening. The temperatures in the top of the slab on the south side of the bridge deck tended to be lower than those on the central and north sides. The temperatures in the bottom of the slab tended to be more similar throughout the day at the north, south, and central portions of the deck. In contrast, the temperatures of the steel superstructure were

Table 4. Comparison of temperature data for April 27, 1987.

Day	Time (hour)	Temperatures, °F								
		North Stringer	Center Stringer	South Stringer	North Slab		Center Slab		South Slab	
					Top	Bottom	Top	Bottom	Top	Bottom
4-27-1987	2	73.2	72.7	73.1	77.1	74.4	76.7	76.3	78.3	75.4
	4	68.7	68.4	69.2	73.0	69.9	72.8	72.2	75.2	72.0
	6	64.6	64.9	64.3	68.3	65.4	68.1	67.2	70.3	66.9
	8	60.9	61.7	60.8	63.8	61.7	63.7	63.4	66.3	63.5
	10	60.3	61.2	59.7	62.2	60.4	61.7	61.0	63.0	61.2
	noon	62.3	63.3	61.5	65.6	62.2	64.7	62.9	62.3	61.7
	2	65.0	66.0	64.3	61.2	65.7	70.4	67.3	64.1	63.8
	4	67.8	68.6	67.3	76.8	69.6	75.6	71.9	67.2	66.6
	6	68.8	69.2	68.3	78.9	71.6	78.2	74.5	70.1	68.5
	8	68.6	68.9	68.5	76.9	71.7	76.8	74.4	71.2	69.2
	10	66.5	67.5	67.1	72.3	69.4	72.9	72.1	70.8	68.6
	midnight	65.2	65.6	65.8	67.9	67.3	68.7	69.1	69.3	67.5

Table 5. Comparison of temperature data for January 28, 1988.

Day	Time (hour)	Temperatures, °F								
		North Stringer	Center Stringer	South Stringer	North Slab		Center Slab		South Slab	
					Top	Bottom	Top	Bottom	Top	Bottom
1-28-1988	2	15.9	15.5	16.3	17.4	17.3	15.9	16.0	16.3	16.3
	4	15.4	15.1	15.7	16.6	16.5	15.7	15.8	15.7	15.7
	6	15.3	15.1	15.9	15.6	15.9	15.1	15.4	15.9	15.9
	8	16.1	15.9	18.1	15.0	16.1	14.7	15.3	18.1	18.1
	10	19.1	19.7	21.8	16.0	18.6	15.5	16.8	21.7	21.8
	noon	23.5	24.7	28.0	19.6	22.7	18.2	20.0	27.9	28.1
	2	27.4	28.4	31.9	23.9	26.6	21.9	23.5	31.8	31.9
	4	29.0	29.5	32.7	27.2	28.7	25.4	26.3	32.7	32.8
	5	29.2	29.5	32.6	28.1	29.2	26.6	27.3	32.6	32.6
	6	29.2	29.2	31.7	28.8	29.4	27.4	27.9	31.7	31.7
	8	29.0	28.7	31.1	29.3	29.6	28.2	28.5	31.1	31.1
	10	29.4	29.0	31.4	29.1	29.8	28.4	28.8	31.3	31.4
	midnight	29.7	29.3	31.6	28.7	29.8	28.3	28.9	31.6	31.6

much more uniform from north to south on the bridge deck, varying by no more than one to two degrees throughout the day.

From these data it appears that the large differences in temperatures on top of the slab across the deck width were related to the intensity of the sunlight on the roadway. During the heat of the day, the north side of the bridge deck was subject to direct sunlight, whereas the south side, in the region of the thermocouples, was partially shielded by the bridge parapet shadow. The bottom of the slab temperature differences were smaller due to their position away from direct sunlight. This was true also for the steel superstructure, which was shielded by the roadway slab. These trends are apparently related to the retention and dissipation characteristics of steel and concrete.

Table 5, which shows data for January 28, 1988, illustrates a typical contrast in daily temperature data between days in summer and winter. As was shown in Table 5, the January data also indicate that the concrete temperatures are warmer than the steel superstructure temperatures. These differences are not as great as those typically noted in the summer. The temperature differences across the width of the roadway are not as extreme for the January data as for the April data. The tendency noted in spring, summer, and early fall of north-side temperatures being warmer was not found to occur typically during the winter. As shown in Table 5 for both the steel and concrete slab temperatures, the south side is warmer, the largest differences occurring during late afternoon. It was noted from reviewing temperature data during the winter months that this trend of the south side being warmer than the north side only occurred about 50% of the time. It was not obvious from consideration of the data why this occurred. It is also interesting to note that typically in the winter, there is very little difference between the top and bottom slab temperatures.

As mentioned previously, Figs. 24 to 27 show that the tilt readings "echo" the ambient temperature data. During the early and late parts of the day, the sensitivity between the tilt data and temperature appeared to be less than during the middle part of the day when temperatures typically increased more rapidly. From these data, the rate of change of pier movement due to change in temperature was computed. On the basis of the assumptions that the pier movement also represented the superstructure movement at the pier and that Pier No. 5, a fixed pier, was the reference point about which expansion and contraction of the superstructure occurred, a coefficient of expansion and contraction of the superstructure could be approximated. To illustrate, Fig. 24 has been divided into four time periods corresponding to apparent differences in rate of change of tilt related to change in temperature:
midnight–6 a.m., 6 a.m.–6 p.m., 6 p.m.–midnight, and over the whole 24-hr period. The

change in pier tilt was corrected to a pier linear displacement, based on discussion in Section 4.1, and a coefficient of expansion and contraction of the superstructure for each of the four time periods was determined by the following calculation.

Midnight– 6 a.m.

$$\alpha \doteq \frac{\Delta_{tilt} C_{tilt}}{L \times \Delta_{temp}} \tag{3}$$

when

Δ_{tilt}	=	change in tilt sensor reading
C_{tilt}	=	correction/1 arc minute, for linear displacement
L	=	span length between fixed Pier No. 5 and expansion Pier No. 4
	=	153 ft
Δ_{temp}	=	change in °F, in ambient temperatures

$$\alpha = \frac{(1.7\ arc\ min)\left(.07 \dfrac{in.}{arc\ min}\right)}{153\ ft \times 12 \dfrac{in.}{ft} \times 1°F} = 6.48 \times 10^{-6} \frac{\dfrac{in.}{in.}}{°F}$$

This coefficient is larger than an expected design value of approximately 6×10^{-6} in./in./°F. Table 6 provides a summary of the calculated coefficients for the other three time periods, as well as data for the other three days represented in Figs. 24 to 27. As shown, the coefficients were very similar, and in all cases slightly larger than the expected design values. A number of possible reasons exist for these differences, including incorrect assumptions in the calculation of the coefficient from the field tilt data, such as the assumption that the fixed Pier No. 5 is actually fixed against longitudinal movement. However, an important implication from the data was that the tilt readings of the pier were a result of forces transferred from superstructure expansion and contraction.

4.2.3. Monthly Behavior

When the tilt data were viewed over a longer time period than daily, it was noted that the movement was temperature dependent. The monthly data indicated that the pier

Table 6. Calculated temperature expansion and contraction coefficients based on daily ambient temperatures.

Day	Temperature Coefficient, α (10^{-6} in./in./°F)			
	Midnight–6 a.m.	6 a.m.–6 p.m.	6 p.m.–Midnight	24 hr Period
3-20-1987	6.48	5.51	4.81	5.20
5-15-1987	6.62	8.37	3.5	6.50
1-21-1987	0.485	8.3	3.19	4.80
4-14-1987	2.58	9.8	2.57	5.80

continued to follow the daily pattern of movement discussed in Section 4.2.2. In other words, the pier consistently rotated toward the east during the early morning hours and then gradually rotated toward the west in the afternoon. Typically, on a given day the pier capbeam reached the farthest eastward position around 6:00 a.m. and the farthest westward position around 6:00 p.m. The magnitude of the rotation that the pier experienced between 6:00 a.m. and 6:00 p.m. varied from day to day, depending on the corresponding change in ambient temperatures. Therefore, the readings of the north and south tilt sensors at 6:00 a.m. and 6:00 p.m. were considered as representative of the eastward and westward bounds for movement of the pier. These readings were plotted on a monthly basis to show the variation in the position of the pier. A few arbitrarily selected plots are presented for discussion. Figures 32 and 33 represent the readings of the north and south tilt sensors and of the ambient temperatures during January 1987 at 6:00 a.m. and 6:00 p.m., respectively. Note the close correlation between the pier tilt and the ambient temperature, as had been suggested by reviewing the daily data. An eastward rotation followed a decrease in temperature, while a westward rotation followed an increase in temperature with magnitudes of rotation proportional to the corresponding changes in ambient temperature. For the greater part of the month, the position of the pier was easterly with respect to its original position at the beginning of the monitoring period. However, during the last week the pier began a net rotation toward the west beyond its original position. The net rotation that the pier experienced from the beginning to the end of the month was approximately 1.40 arc min toward the west. Figures 34 and 35 represent the readings of the east and west tilt sensors during

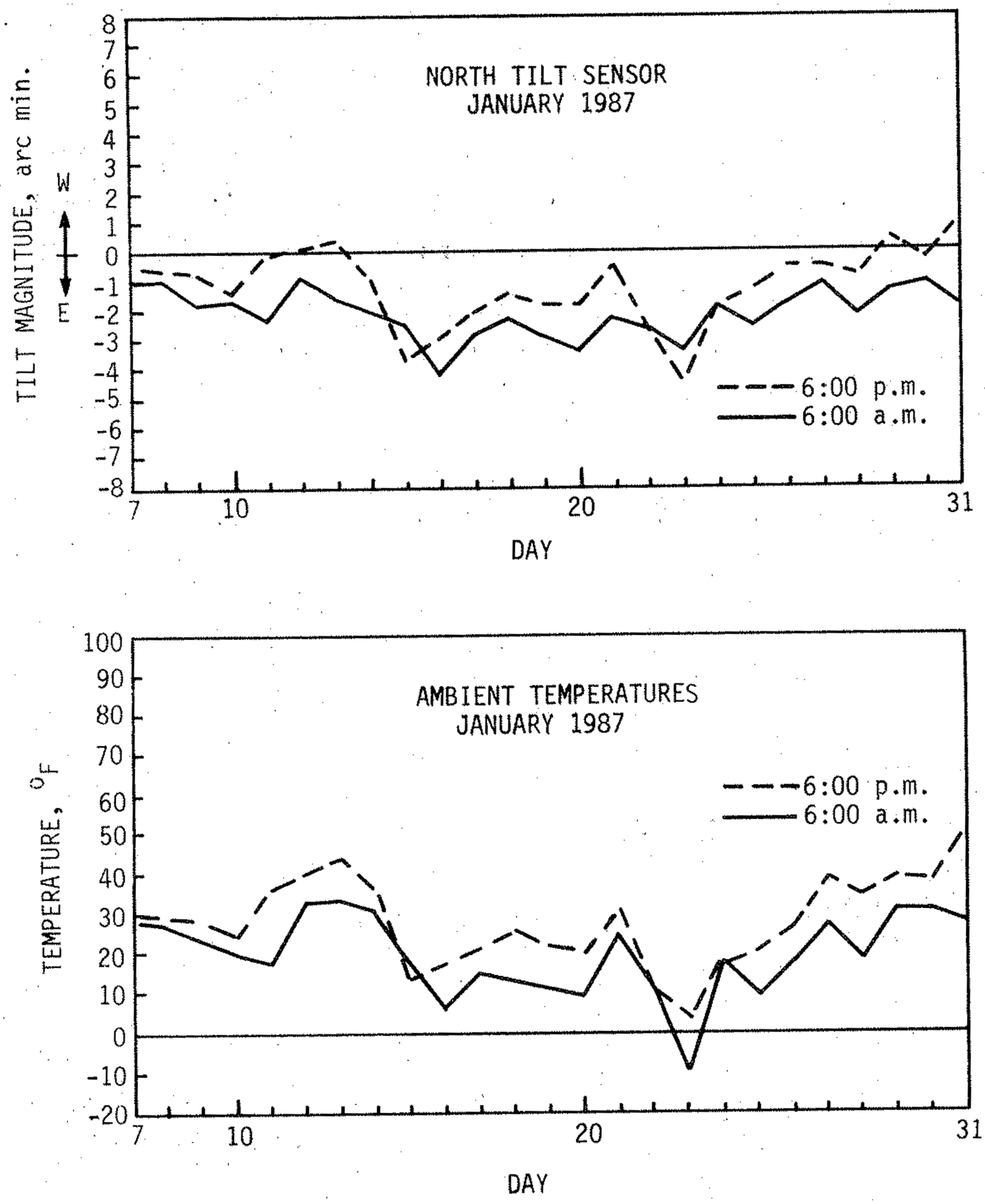


Fig. 32. Readings of north tilt sensor and of ambient temperatures at 6:00 a.m. and 6:00 p.m. during January 1987.

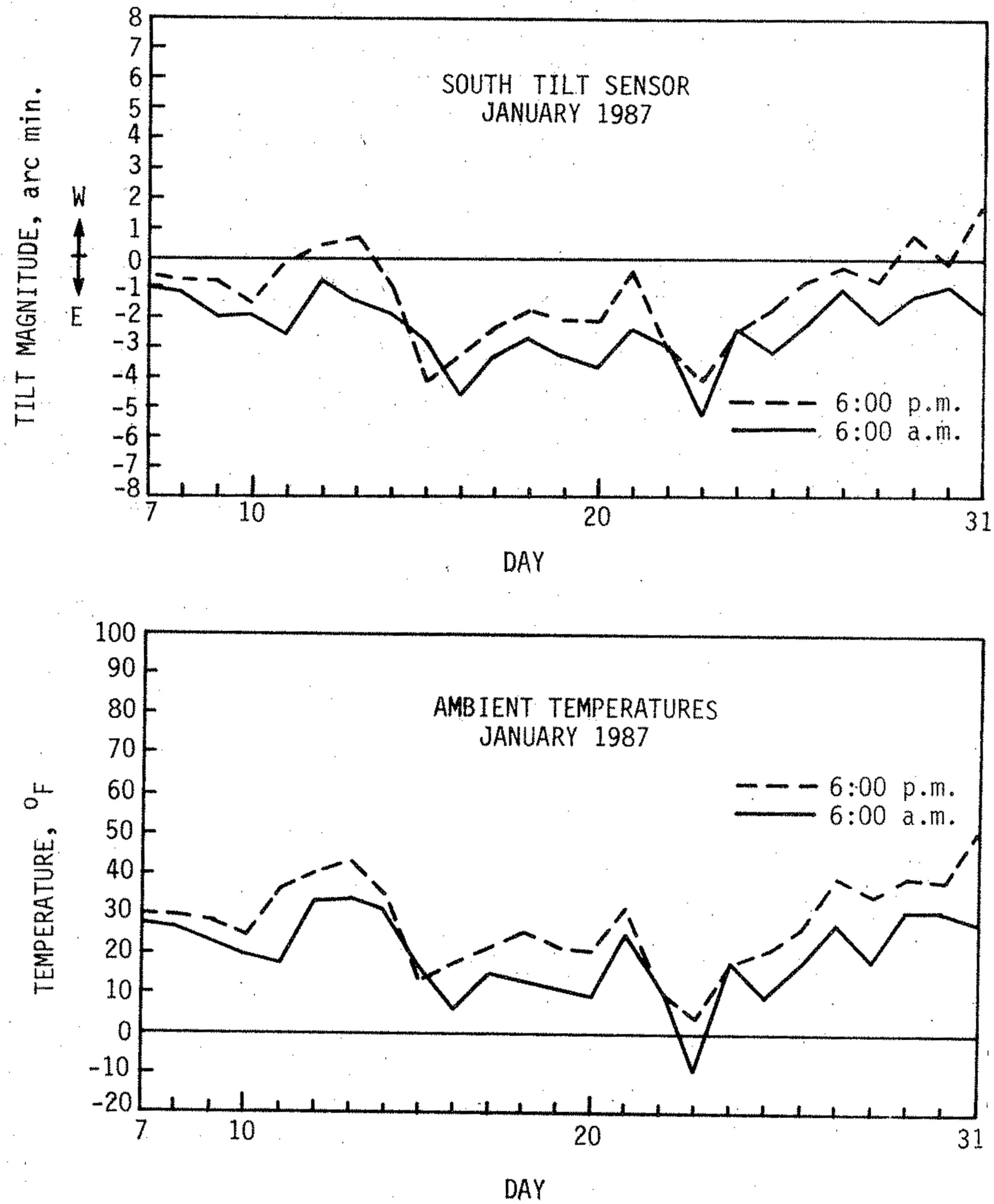


Fig. 33. Readings of south tilt sensor and of ambient temperatures at 6:00 a.m. and 6:00 p.m. during January 1987.

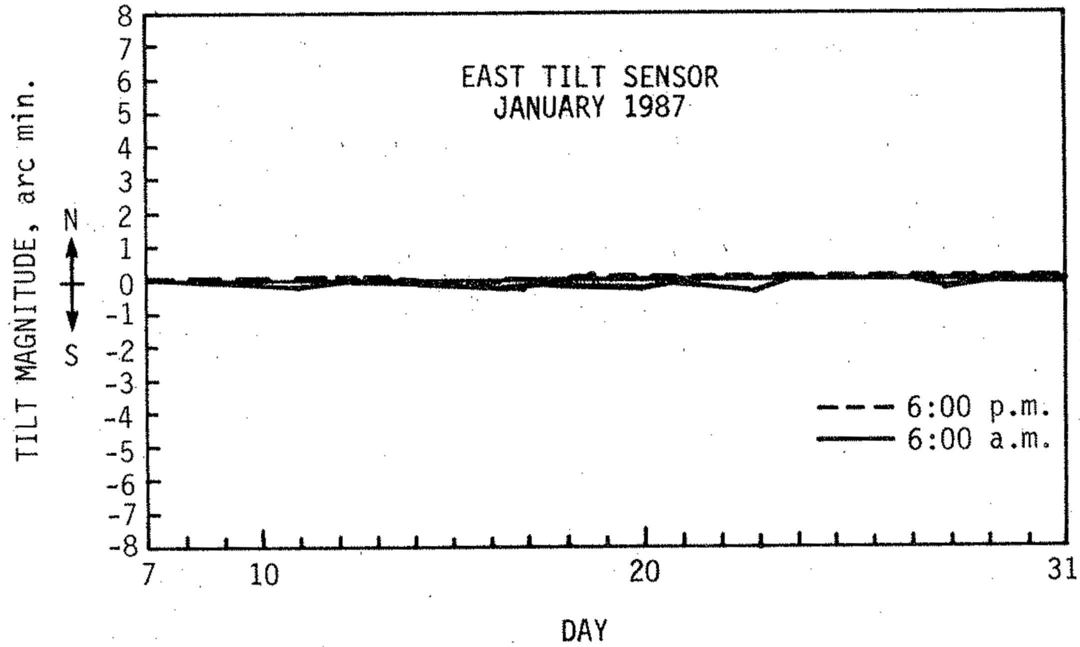


Fig. 34. Readings of east tilt sensor at 6:00 a.m. and 6:00 p.m. during January 1987.

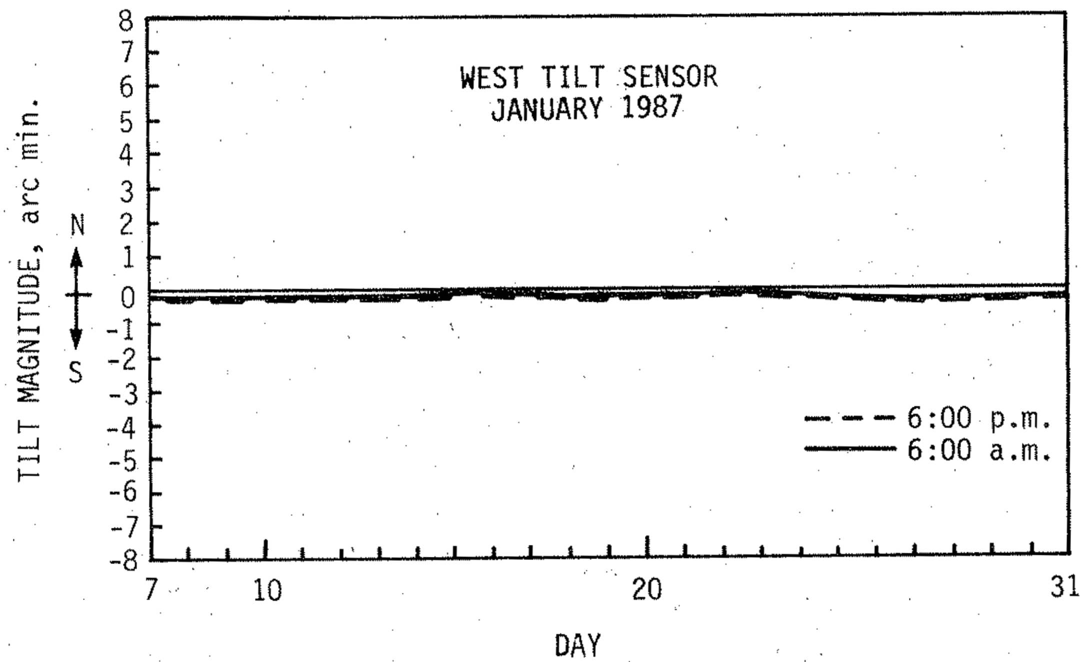


Fig. 35. Readings of west tilt sensor at 6:00 a.m. and 6:00 p.m. during January 1987.

January 1987 at 6:00 a.m. and 6:00 p.m., respectively. It is apparent that the rotation of the pier in the north–south direction was negligible.

Figures 36 and 37 represent the maximum range of readings of the north and south tilt sensors and of ambient temperatures, respectively, during May 1987. The pier moved more in its daily rotations in May than in January. This effect was directly related to the larger daily changes in ambient temperatures in May than in January. We also noted that the position of the pier was westerly with respect to its original position throughout the month. The net rotation of the pier from the beginning to the end of the month was approximately 0.43 arc min. However, the net rotation of the pier from the end of January to the end of May was 3.40 arc min toward the west, which indicated a significant westward shift in the position of the pier. This was related to the higher mean temperatures in May than in January.

Figures 38 and 39 represent the readings of the north and south tilt sensors and of ambient temperatures during October 1987. The graphs indicate that throughout the month, the position of the pier was easterly with respect to its original position. The net rotation of the pier during the month was 2.30 arc min toward the east, while the net rotation of the pier between May and October was 5.30 arc min toward the east. The net movement of the pier between January and October was 0.90 arc min toward the west. These net movements from month to month again follow the rise and fall of the seasonal ambient temperatures.

Figures 40 through 43 illustrate the readings of the east and west tilt sensors, respectively, during May 1987 and October 1987. They show that the net rotation of the pier in the north-south direction was negligible.

The monthly data clearly show that the pier movement in the east-west direction is temperature dependent. Data also indicate the general longer term expansion and contraction characteristics of the superstructure were similar to those over a shorter time frame. In other words, net movement occurred toward the east during colder weather (conduction) and toward the west during warmer weather (expansion). To provide insight into the magnitude of long term movements of the bridge superstructure, a coefficient of expansion and contraction was approximated over each month by considering the net change in movement versus the net change in temperature. Coefficients calculated for the three months illustrated in Figs. 32, 36, and 38 are shown in Table 7. As shown, coefficients ranged in magnitude from 3×10^{-6} in./in./°F to 7×10^{-6} in./in./°F. These values were very sensitive to the data used and should only be used to provide an overall trend of movement. They do provide further evidence that the movements illustrated in the graphs are principally related to temperature expansion and contraction effects and that for a general assessment of long

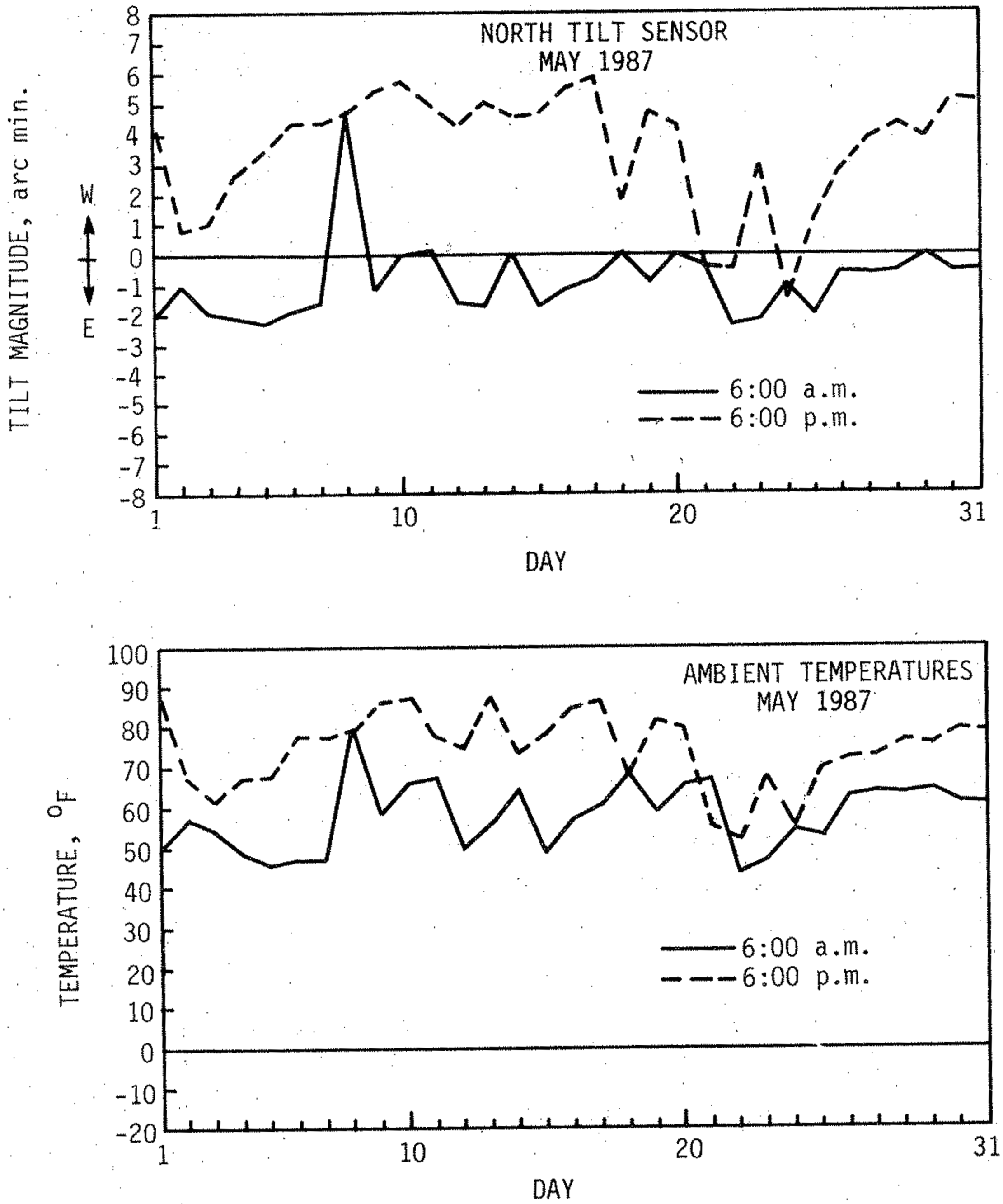


Fig. 36. Readings of north tilt sensor and ambient temperatures at 6:oo a.m. and 6:00 p.m. during May 1987.

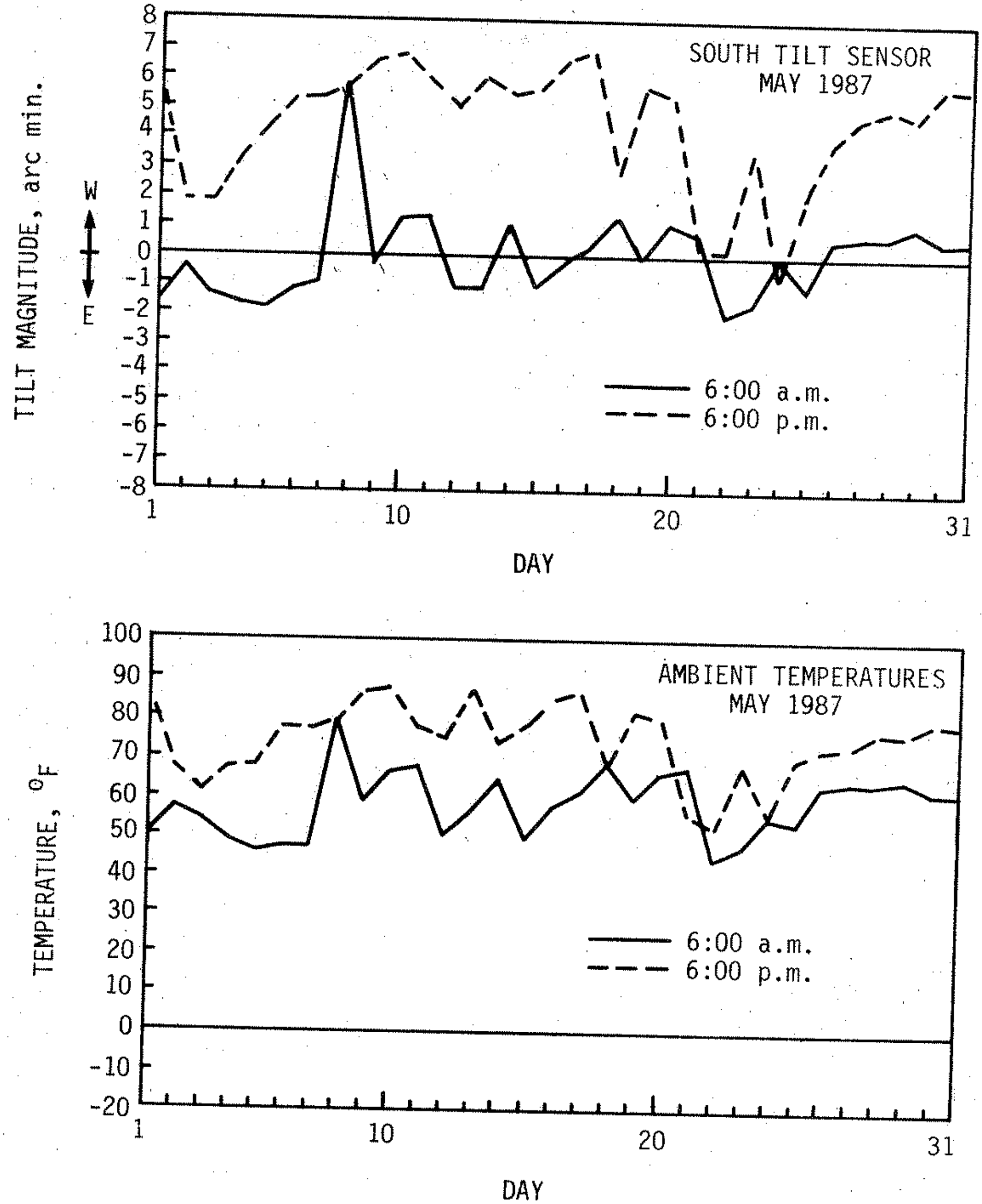


Fig. 37. Readings of south tilt sensor and of ambient temperatures at 6:00 a.m. and 6:00 p.m. during May 1987.

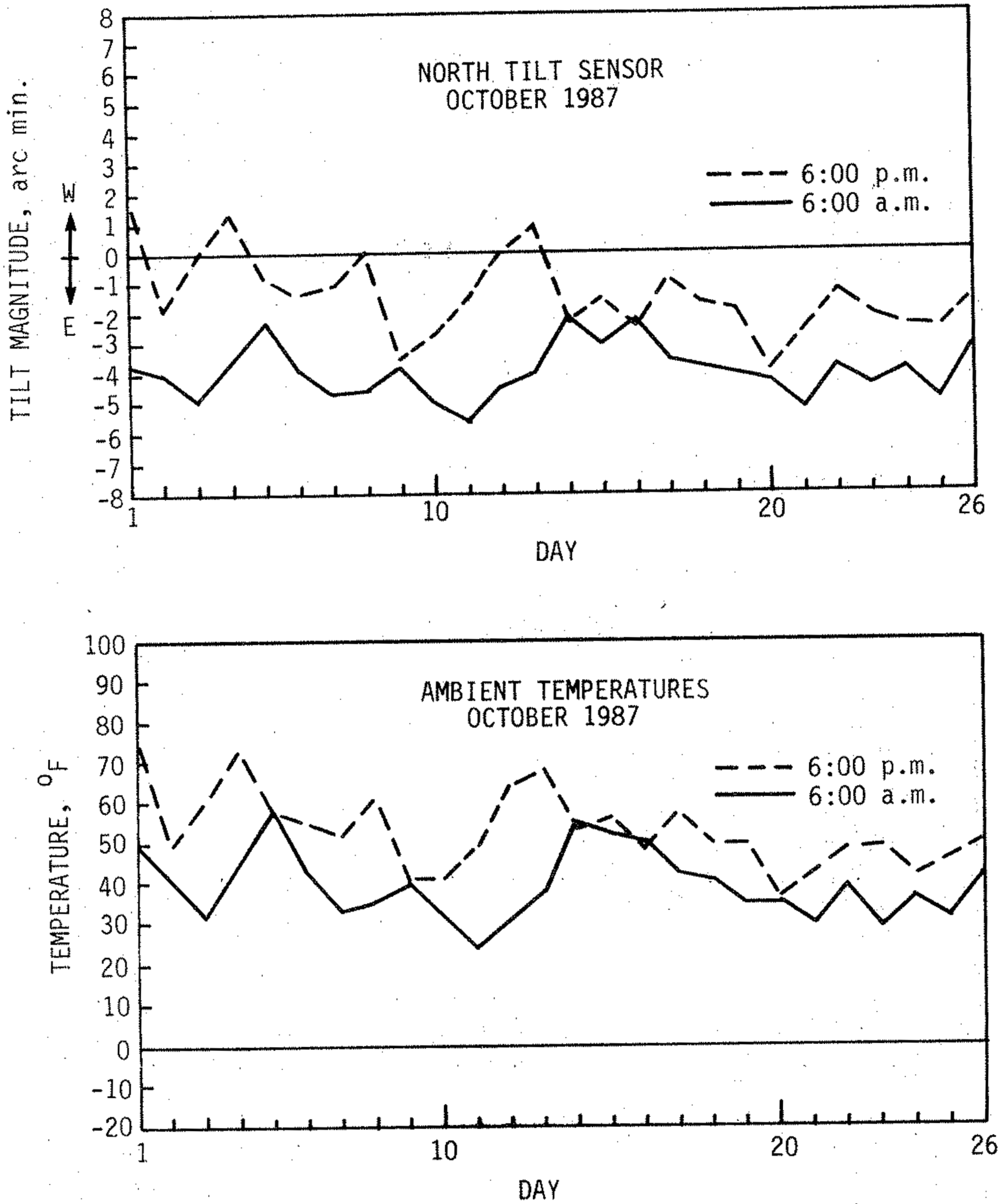


Fig. 38. Readings of the north tilt sensor and ambient temperatures at 6:00 a.m. and 6:00 p.m. during October 1987.

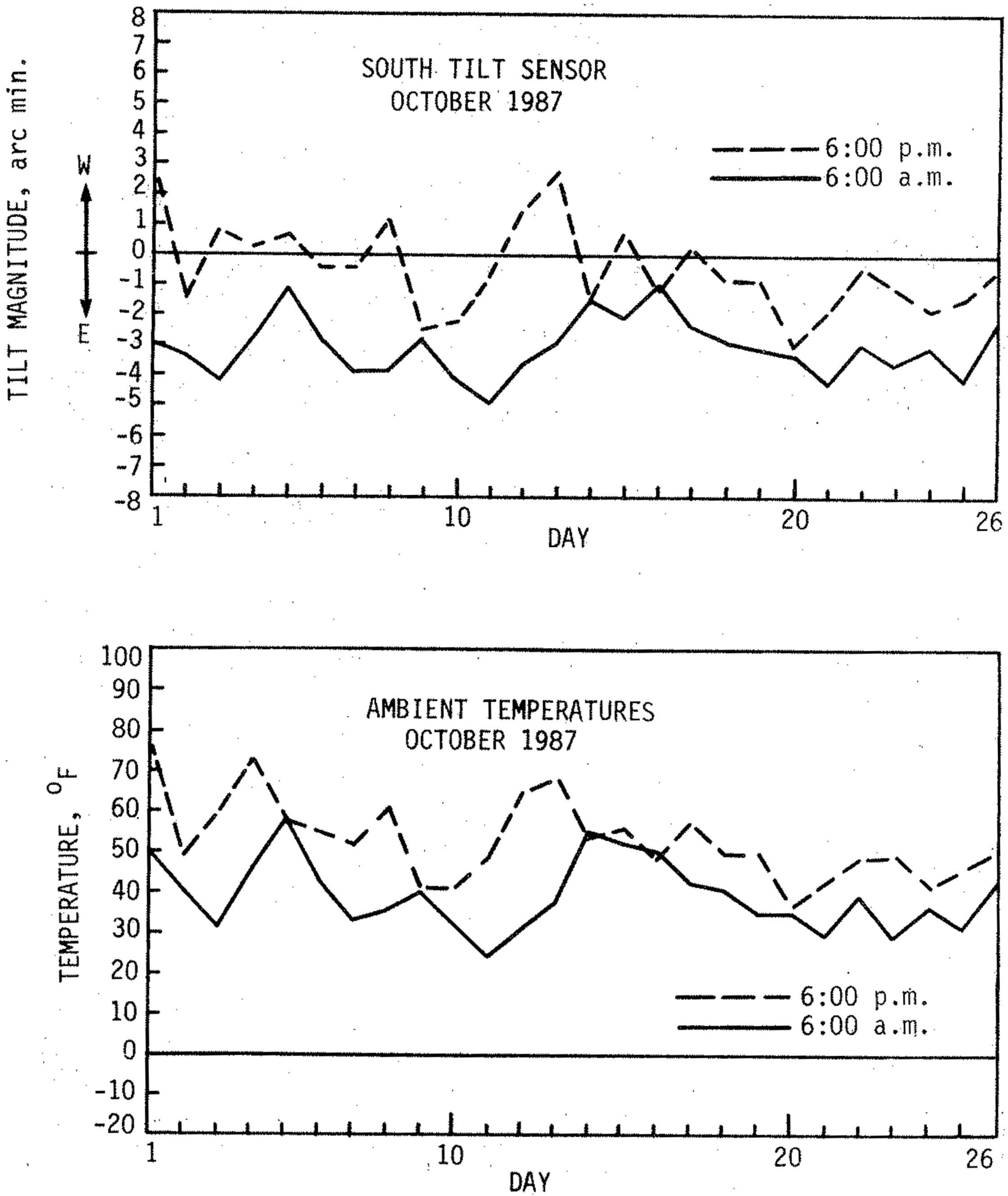


Fig. 39. Readings of south tilt sensor and ambient temperatures at 6:00 a.m. and 6:00 p.m. during October 1987.

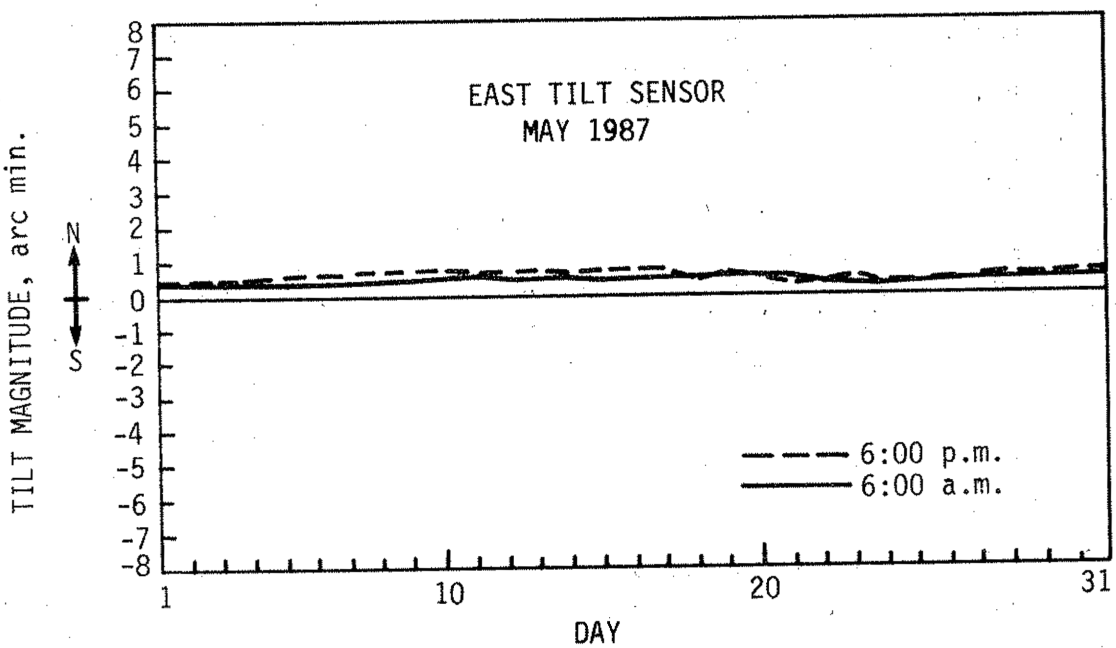


Fig.40. Readings of east tilt sensor at 6:00 a.m. and 6:00 p.m. during May 1987

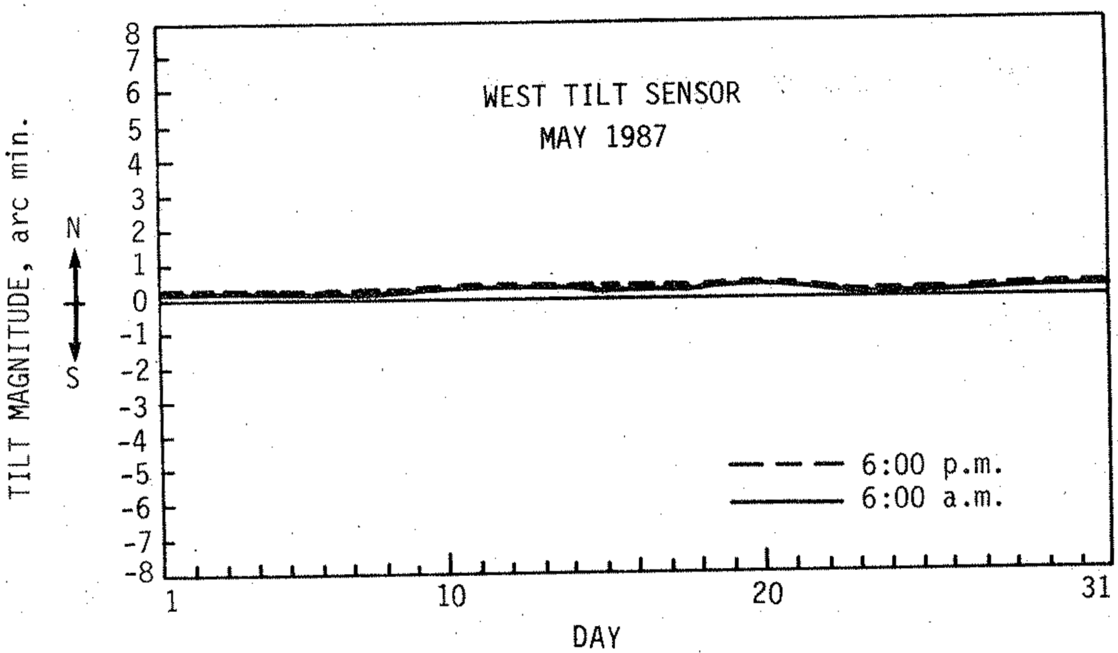


Fig.41. Readings of west tilt sensor at 6:00 a.m. and 6:00 p.m. during May 1987.

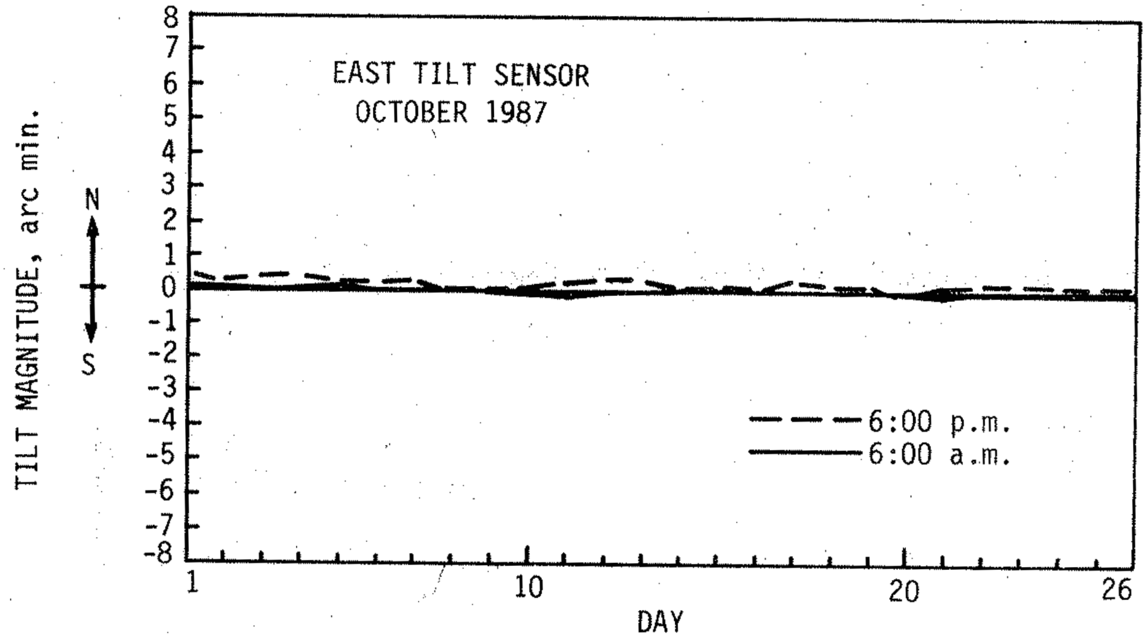


Fig. 42. Readings of east tilt sensor at 6:00 a.m. and 6:00 p.m. during October 1987.

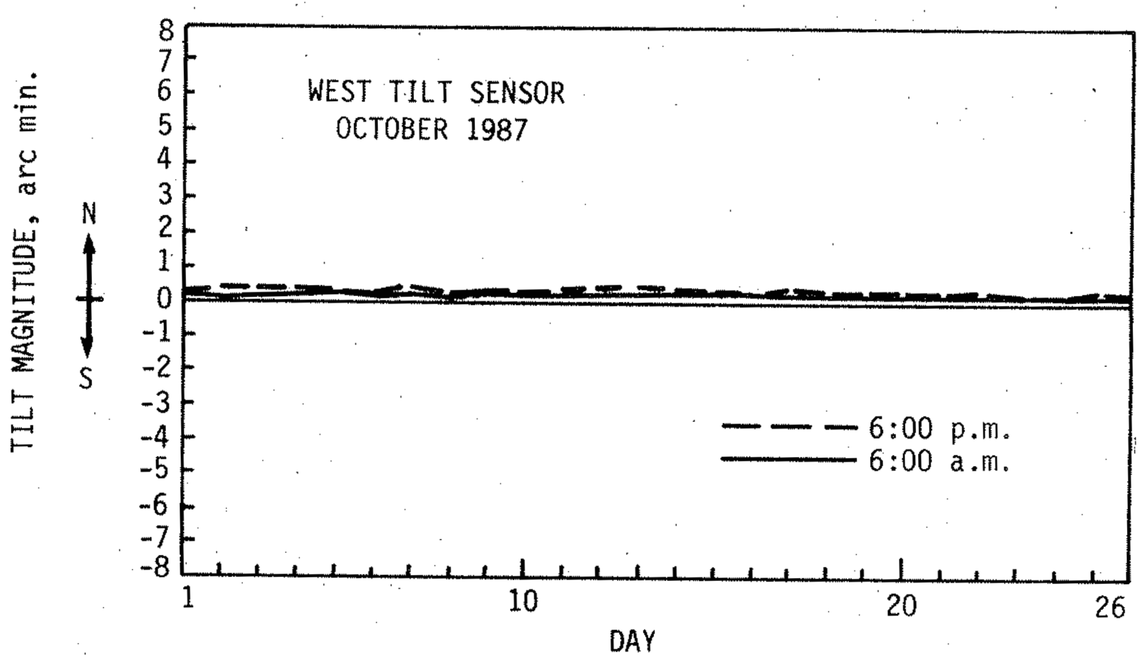


Fig. 43. Readings of west tilt sensor at 6:00 a.m. and 6:00 p.m. during October 1987

term behavior, the coefficient of expansion and contraction for materials may be considered to be essentially linear.

Table 7. Calculated temperature expansion and contraction coefficients based on monthly ambient temperatures.

Month	Temperature Coefficient, α (10^{-6} in./in./°F)	
	Based on Maximum Temperatures	Based on Minimum Temperatures
January 1987	3.2	1.1
May 1987	3.0	4.8
October 1987	3.7	2.0

4.2.4. Seasonal Behavior

The discussion of the monthly behavior of the pier, presented in Section 4.2.3, indicated that the net monthly rotation of the pier capbeam varied in magnitude and direction throughout the monitoring period. In order to study the long term behavior of the pier and identify any general trends in long term movement, the accumulated tilt and ambient temperature data were plotted over the duration of the monitoring period. Figure 44 represents the readings of the north and south tilt sensors at 6:00 a.m. and 6:00 p.m. during the period from January 1987 to the end of March 1988. The corresponding ambient temperatures are also shown. As mentioned previously, component failure of the console unit and micrologger on a number of occasions resulted in the loss of all data for most of the summer of 1987; this is represented in the figures as regions where no data are plotted.

As illustrated in an earlier presentation of data, and shown in Fig. 44, the north-south tilt data consistently showed close correlation with ambient temperatures and magnitudes of the daily rotations of the pier were proportional to the corresponding changes in ambient temperatures. The position of the pier, with respect to its original position, varied from season to season throughout the duration of the monitoring period. For the greater part of the winter of 1987 the position of the pier was easterly. Starting at mid-March and continuing

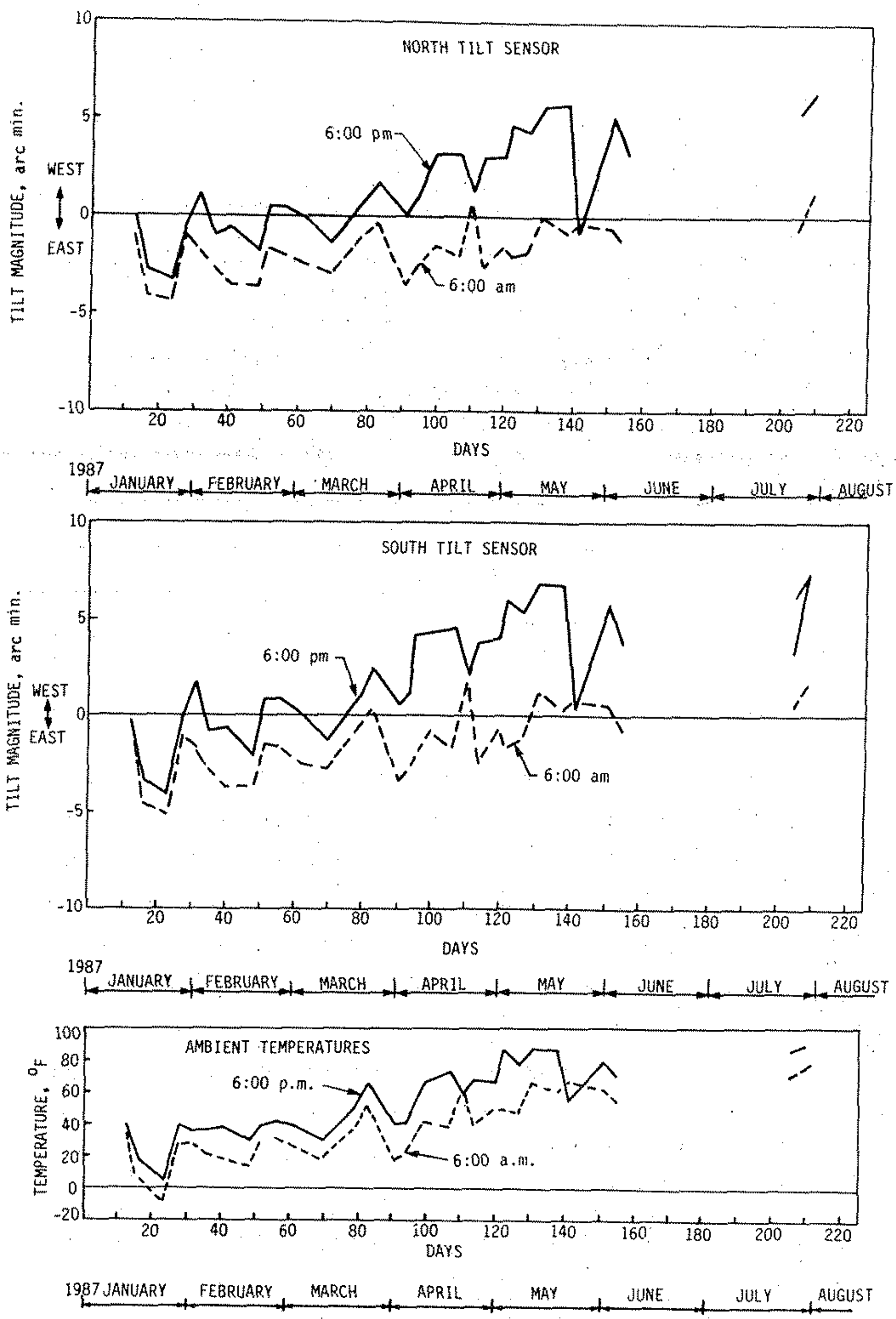


Fig. 44. Readings of north and south tilt sensors and of ambient temperatures at 6:00 a.m. and 6:00 p.m. from January 1987 to March 1988.

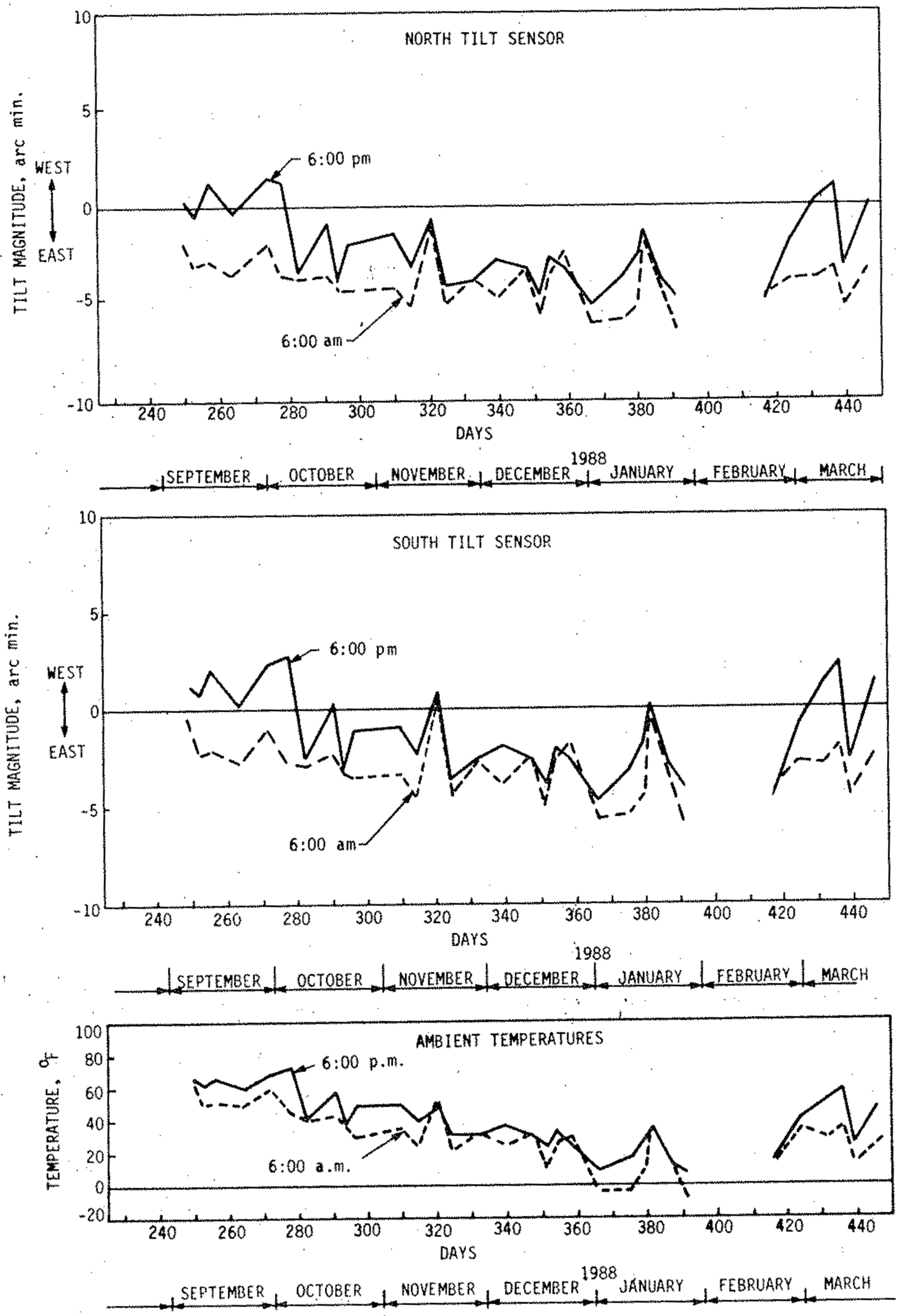

NORTH TILT SENSOR
TILT MAGNITUDE, arc min.
WEST
EAST
6:00 pm
6:00 am
DAYS
1988
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
SOUTH TILT SENSOR
TILT MAGNITUDE, arc min.
WEST
EAST
6:00 pm
6:00 am
DAYS
AMBIENT TEMPERATURES
TEMPERATURE, °F
6:00 p.m.
6:00 a.m.
DAYS

Fig. 44 (cont.)

through the end of May, the position of the pier gradually shifted toward the west, beyond its original position. During the summer of 1987, the complete behavior of the pier was not clear because of the loss of data. By the beginning of September, the pier had shifted eastward approaching its original position. During the greater part of the fall of 1987 and continuing through the winter of 1988, the position of the pier was easterly with respect to its original position. However, toward the end of March 1988, the pier again shifted westward closer to its original position. The net rotation that the pier experienced during the period from January 1987 to March 1988 was approximately 0.80 arc min. toward the west.

Although the net change in the position of the pier over the duration of the project could be described as relatively small, the net seasonal rotations, from one season to the next, that the pier experienced were of greater significance. This behavior appears to be attributable to the seasonal changes in the mean ambient temperature.

Of particular interest in studying the data illustrated in Fig. 44 are the relative differences between the readings of the north and south tilt sensors. Discussion presented in earlier sections suggested that from January to mid-March 1987 no differences existed. However, from March 1987 until the end of the monitoring period, a relatively constant difference between north and south tilt existed. This figure clearly illustrates the more westerly position of the south side of the pier from mid-March 1987 until the end of the monitoring period in March 1988. The discrepancy of approximately 0.8 arc min corresponded to a linear displacement at the top of the pier of approximately 0.05 to 0.08 in. A possible explanation related to this long term condition is presented below.

Perhaps significantly, the relative differences in tilt between the north and south end of the pier occurred from January to mid-March. The rate of change in the tilt difference eventually leveled in May and, as noted in Fig. 44, became constant in magnitude for the remainder of the monitoring period. The data suggested that the cause of the change in tilt between March and May was a permanent movement at one side of the pier. This possibility is supported by the fact that the difference between the pier tilts remained constant after the initial buildup from March to May of 1987. This effect may have been caused by a slip of the foundation on the south side of the pier in an eastward direction, or down the slope. This argument is given validity when looking at the time period over which the change occurred–from March to May. It is possible that this time period corresponded to a change of soil conditions due to seasonal changes in temperature; that is, it is possible that a freeze and thaw cycle in progress during this time period led to a foundation slip.

Although the above discussion is based upon conjuncture, the analytical models (discussed in the next section) provided reasonably accurate predictions of changes in pier tilt in the field, considering a range of realistic soil parameters for subgrade reaction. As noted in the analytical comparisons, the correlation of field and analytical data provides some justification for believing the soil foundation properties changed during changes in season.

Figure 44 also suggests an interesting trend between the pier movement and ambient temperatures. If the north-south pier tilts are superimposed with ambient temperatures for the duration of the monitoring period, two different trends are noted. From January 1987 until the end of July 1987, north and south tilt sensor readings "tracked" the temperature readings identically. From September 1987 to March 1988, however, there was a "shift" of the north-south tilt readings from temperature, so that the two plots would not identically track each other. The changes in the tilt and temperature records were nearly identical; however, in general, the north-south tilt readings during September 1987 to March 1988 suggested an eastward shift of the pier from where it would identically track temperature. Of course, the loss of some data in the summer of 1987 is regrettable, as some indication of how this shift occurred may have been obvious.

As a result of the above-mentioned occurrence, a strict interpretation of the long-term field data suggested that a permanent shift, or movement, of the pier occurred between July 1987 and September 1987. Since the tilt readings were apparently more easterly than temperature data implied, this could suggest a slip of the foundation occurred toward the west or up the slope. The apparent shift of axes of the tilt and temperature graphs corresponded to approximately 1 to 2 arc min of movement. Based on assumptions presented earlier, this corresponded to a linear displacement of approximately 0.07 to 0.2 in. of permanent movement near the top of the pier or at the foundation.

From review of the same long-term data, another, less significant, shift in axes between tilt readings and temperature was seen to occur between September 1987 and March 1988. During this time period, it was noted that the tilt readings were more westerly than temperatures would indicate. This offset is approximately 0.5 to 1 arc min, which corresponds to approximately 0.035 to 0.07 in. This possible permanent shift suggested that during this time period, the foundation would have had to move down the slope.

Figure 45 shows long term plots of tilt and ambient temperatures for the east and west sensors, respectively. Note that no noticeable movement occurred over the time span in which the movement was monitored.

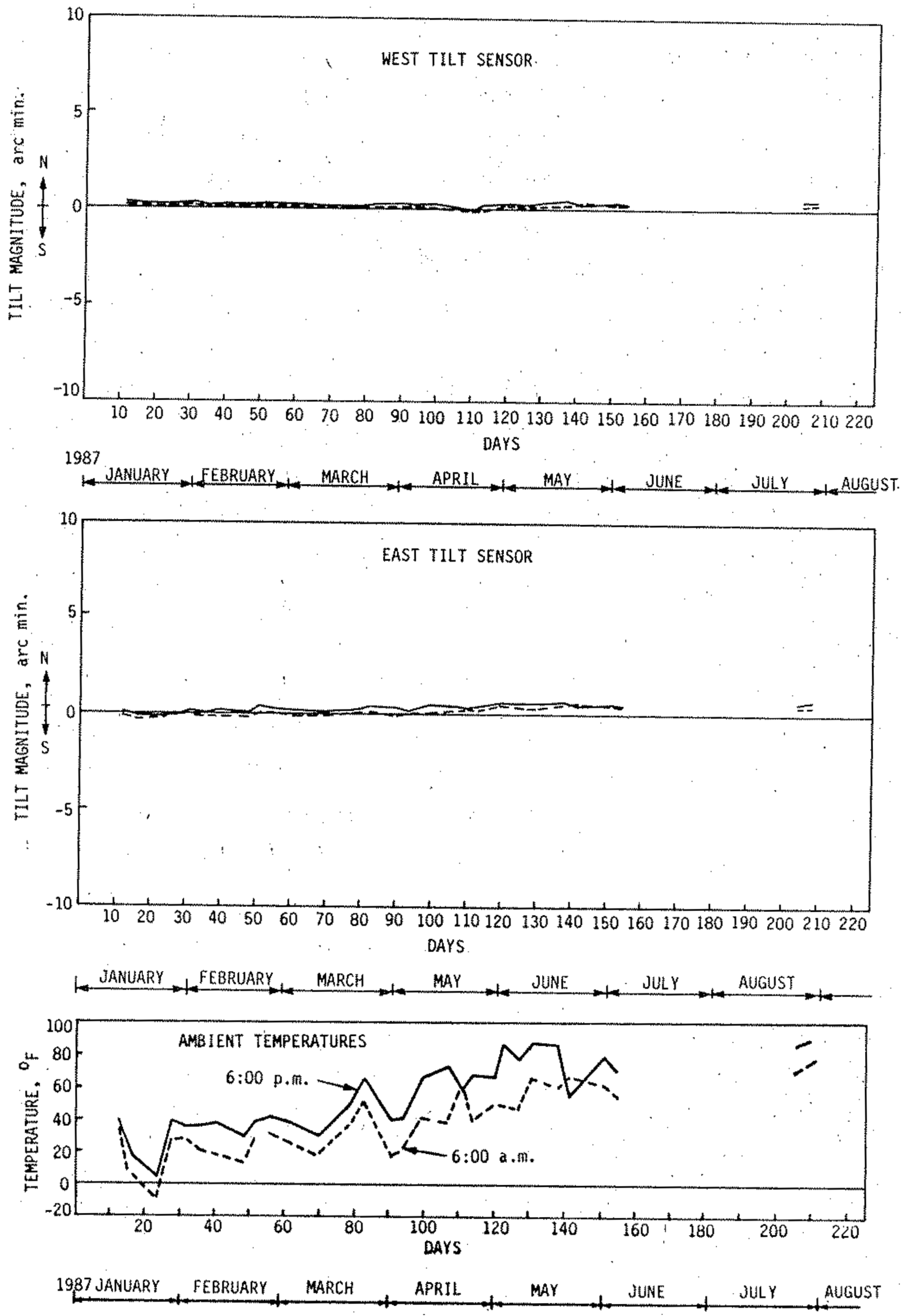


Fig. 45. Readings of west and east tilt sensors of ambient temperatures at 6:00 a.m. 6:00 p.m. from January 1987 to March 1988.

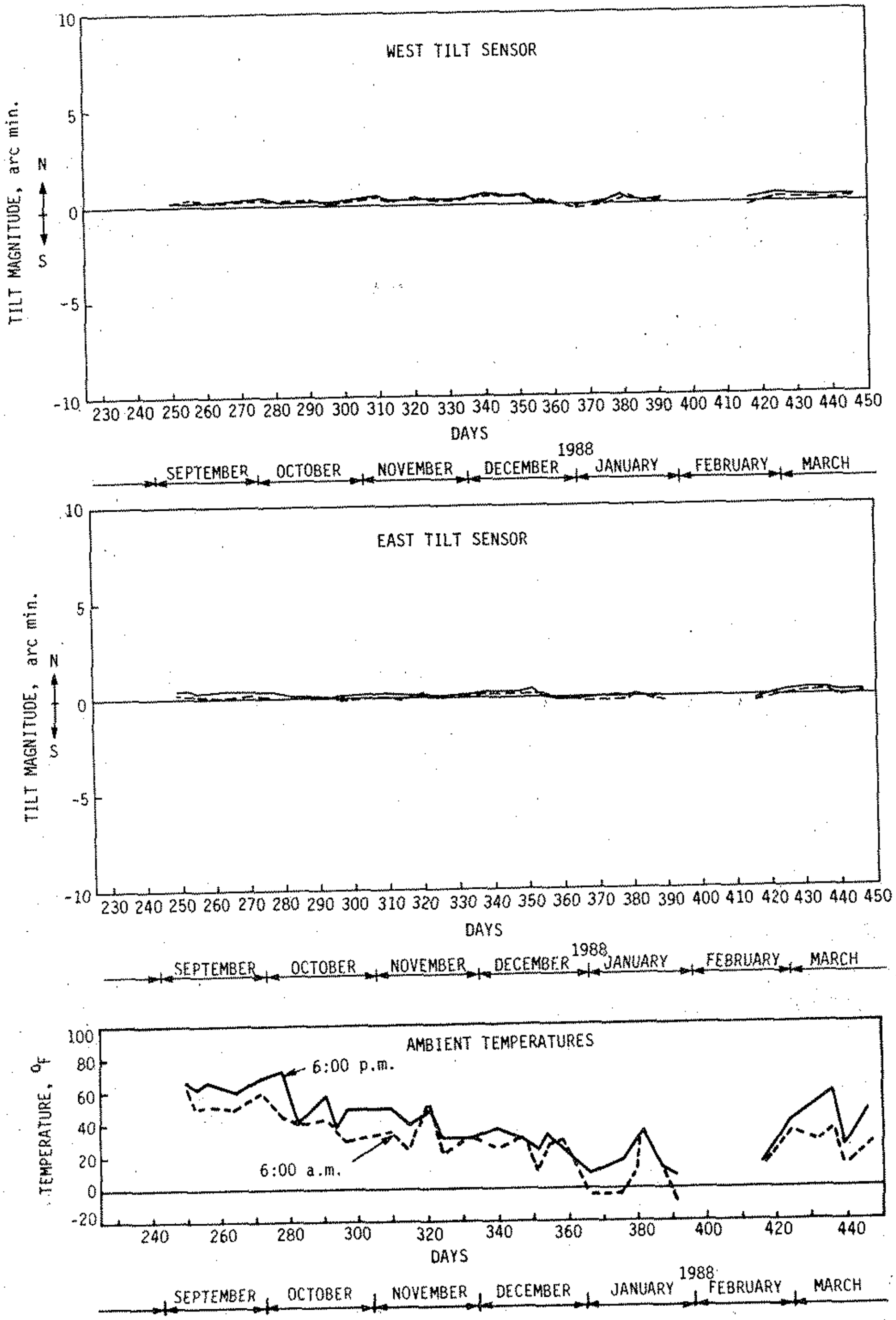

WEST TILT SENSOR
TILT MAGNITUDE, arc min.
N
S
DAYS
1988
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
EAST TILT SENSOR
AMBIENT TEMPERATURES
TEMPERATURE, °F
6:00 p.m.
6:00 a.m.

Fig. 45 (cont.)

The typical movements noted over both the short term and long term imply that the expansion bearings at Pier No. 4 were not functioning as intended. It appeared that the increase in the mean ambient temperatures between the winter and spring caused expansion of the bridge superstructure. Depending on the magnitude of restraint provided by the bearing devices, horizontal forces were developed and transmitted to the pier capbeam that caused the net westward rotation measured at the pier. In contrast, the net eastward rotation that the pier experienced between the spring and fall was caused by a corresponding decrease in the mean ambient temperature.

4.2.5. Results of the Analytical Models

As discussed in Section 3, the analytical models were developed to validate the amount of movement that occurred in the field and to provide an explanation as to the general behavior of the pier. The analytical models were used to calculate the pier rotation as a function of temperature at various intervals of time. As previously mentioned, the superstructure model utilized thermocouple, temperature field data to assess the longitudinal thermal forces induced in the composite steel stringers during various time periods. These forces were then applied to the pier model to determine the corresponding rotations and linear displacements of the pier capbeam.

Based upon the assumptions incorporated into the superstructure and substructure models (which were discussed in Section 3), comparisons were made with field data. These comparisons were made over short and long time periods. As mentioned previously, because of the uncertainty of the soil behavior, the axial springs were assigned a range of stiffness of 2000 to 200,000 k/in., simulating the possibility of very flexible to very stiff shale. Thus, rotations of the pier capbeam, calculated by using these spring stiffness values, represented possible upper and lower limits of movement of the pier on shale. In addition, rotations were calculated using a relatively small value of the spring stiffness, 400 k/in., which represented possible the possibility of a pocket of soft material in the shale. Results of both of the short and long term comparisons are presented in the following sections.

4.2.5.1. Short Term Movement

In order to verify the daily pattern of movement of the pier discussed in Section 4.2.2, we calculated changes in tilts of the pier capbeam over four short time periods. Each period consisted of three consecutive days selected arbitrarily. The data presented here are representative of comparisons for other data that were collected. Changes in tilt were calculated at 6:00 a.m. and 6:00 p.m., which represented the eastward and westward bounds for daily movement of the pier. The first comparison period presented was from May 4 to May 6, 1987. Changes in rotation were referenced to midnight on May 3, 1987. Probable changes of pier tilt and the measured changes in field tilts are presented in Table 8.

Table 8. Comparison of changes in tilt between field results and analytical model for May 4 to May 6, 1987.

Date	Change in Rotations of Pier (arc min.)					
	Time	Model			Field	
		k = 400 k/in.	k = 2000 k/in.	k = 200,000 k/in.	North Side	South Side
5-4-1987	6:00 a.m.	-2.27	-1.03	-0.70	-0.77	-0.90
	6:00 p.m.	3.22	1.45	0.99	3.97	4.10
5-5-1987	6:00 a.m.	-1.60	-0.73	-0.46	-0.97	-1.02
	6:00 p.m.	3.92	1.78	1.21	4.77	5.00
5-6-1987	6:00 a.m.	-0.54	-0.24	-0.15	-0.52	-0.39
	6:00 p.m.	8.20	3.72	2.55	5.67	6.22

Notes: Changes in rotations are referenced to midnight on 5-3-1987 (assumed tilt of 0.00).

Positive change in tilt implies westward movement, negative eastward movement.

As shown in the table, the magnitude of change in pier rotations showed good agreement with the model ranges of tilt established for the conditions of flexible shale (k = 2000 k/in.) and pockets of soft clay (k = 400 k/in.). In addition, the model results showed that the pier capbeam rotated westward on a daily basis between 6:00 a.m. and 6:00 p.m., which was consistent with the field results.

A second comparison from October 20 to October 22, 1987, with changes in pier rotation referenced to midnight on October 19, 1987, is shown in Table 9. The table implies that the net rotations of the pier capbeam showed a close correlation with the ranges of probable tilt established for flexible (k = 2000 k/in.) and very stiff (k = 200,000 k/in.) shale. The effects illustrated in Tables 8 and 9 possibly suggest that a difference in foundation soil conditions existed between May and October; that is, the soil exhibited more flexible characteristics during May, resulting in relatively a larger magnitude of rotation. The relatively small net rotations measured and predicted during October may be related to the existence of a stiffer soil condition than existed in May.

Table 9. Comparison of changes in tilt between field results and analytical model for October 20 to October 22, 1987.

Date	Time	Change in Rotations of Pier (arc min.)				
		Model			Field	
		k = 400 k/in.	k = 2000 k/in.	k = 200,000 k/in.	North Side	South Side
10-20-1987	6:00 a.m.	-2.41	-1.09	-0.74	-0.71	-0.67
	6:00 p.m.	-2.18	-0.99	-0.68	-0.30	-0.36
10-21-1987	6:00 a.m.	-6.62	-2.55	-1.74	-1.75	-1.68
	6:00 p.m.	0.62	0.29	0.20	0.85	0.75
10-22-1987	6:00 a.m.	-1.83	-0.83	-0.56	-0.23	-0.29
	6:00 p.m.	2.85	1.29	0.89	2.28	2.17

Notes: Changes in rotations are referenced to midnight on 10-19-1987 (assumed tilt of 0.00).

Positive change in tilt implies westward movement, negative eastward movement.

Table 10 shows a comparison of data between the model and field results for the period from February 20 to March 20, 1988. As shown, the changes in pier rotation of the pier capbeam compared closely with the ranges of probable tilt established for flexible (k = 2000 k/in.) and very stiff (k = 200,000 k/in.) shale. The results shown in the table are similar to those shown in Table 8 for late October and suggested relatively stiff soil conditions during February and March.

Table 10. Comparison of changes in tilt between field results and analytical model for February 1988 to March 1988.

Date	Time	Change in Rotation of Pier (arc min)				
		Model			Field	
		k = 400 (k/in.)	k = 2000 (k/in.)	k = 200,000 (k/in.)	North Side	South Side
2-20	6 a.m.	–	–	–	–	–
	6 p.m.	-0.31	-0.14	-0.09	-0.12	-0.17
2-21	6 a.m.	-0.83	-0.38	-0.20	-0.86	-0.56
	6 p.m.	7.60	3.44	2.38	2.27	-2.57
2-22	6 a.m.	8.05	3.64	2.55	2.37	2.68
	6 p.m.	8.49	3.84	2.65	2.99	3.21
3-18	6 a.m.	3.94	1.78	1.20	1.40	1.70
	6 p.m.	9.69	4.39	3.03	4.11	4.17
3-19	6 a.m.	4.66	2.11	1.46	1 44	1.75
	6 p.m.	11.83	5.36	3.69	4.46	4.89
3-20	6 a.m.	6.26	2.84	1.96	1.79	2.18
	6 p.m.	13.93	6.31	4.34	5.35	5.72

Notes: Changes in rotation are referenced to 6 a.m., Feb. 20, 1987 (assumed tilt of 0.00).

Positive change in tilt implies westward movement, negative eastward movement.

It should be noted that the model results shown do not indicate differential rotations between the north and south ends of the pier capbeam. This is due to the assumption used in the analytical model of constant temperature across the superstructure in the transverse direction.

A summary of the forces developed in the superstructure between Piers No. 4 and No. 5 is shown in Table 11. The data shown correspond to the data shown in Tables 8 and 9. These forces represent axial forces in the superstructure caused by the restraint of longitudinal movement, which is caused by the assumed nonfunctioning expansion joints at Pier No. 4. Note that the forces ranged from 2 to 10 kips and represented tension and compression values. The specific significance of these forces has not been addressed in this study, but they certainly need further consideration regarding their effect on the design adequacy of the superstructure, as well as their effect on the bearing connection details.

4.2.5.2. Long Term Movement

The analytical models were used to characterize the seasonal behavior of the pier. For this purpose, net rotations of the pier were established over two periods. The first period selected was from April 27 to October 22, 1987. Tilts were calculated at 6:00 a.m. and 6:00 p.m. and referenced to midnight on April 26, 1987. The actual measured field tilt of 1.51 arc min was used as the reference. Therefore, the numbers in the table represent actual tilt values as referenced to January 3, 1987, and not just changes in tilt as were presented in the tables in Section 4.2.5.1. The results are presented in Table 12, where a close correlation is shown between the model results and the field rotations. Both show that the pier experienced an eastward net rotation between the spring and the fall seasons. The magnitude of pier rotations in April compared well with the model results established for softer soil conditions. However, over the period from April to October, the pier net rotations compared better with the model ranges of rotations for stiffer soil conditions. This effect possibly suggests further support for the argument that the soil conditions changed from season to season and is consistent with data presented in the previous section on short term movement.

Worthy of note in comparing model results from April to October with field results is that interpretation of the field data over this period indicated that the position of the pier was more easterly than temperature data indicated. Since the analytical model results were based only on temperature data, it was expected that the model results would show a more westerly pier position than field data suggested; this is consistent with the interpretation of

Table 11. Superstructure axial forces as calculated from analytical model.

Date	Time	Axial Forces (lbs)		
		South Stringer	Center Stringers	North Stringer
5-4-1987	6:00 a.m.	-2817	-2774	-2820
	6:00 p.m.	3977	3915	3974
5-5-1987	6:00 a.m.	-1989	-1958	-1992
	6:00 p.m.	4861	4780	4872
5-6-1987	6:00 a.m.	-663	-655	-662
	6:00 p.m.	10,164	10,008	10,179
10-20-1987	6:00 a.m.	-2983	-2940	-2985
	6:00 p.m.	-2706	-2666	-2710
10-21-1987	6:00 a.m.	-6960	-6857	-6967
	6:00 p.m.	773	766	771
10-22-1987	6:00 a.m.	-2264	-2231	-2267
	6:00 p.m.	3536	3478	3543

Notes: - implies tensile forces.
+ implies compressive forces.
May data referenced to 5-3-1987 midnight.
October data referenced to 10-19-1987 midnight.

the dependency of field tilt to ambient temperature. As Table 12 shows, however, the model results were actually quite similar to the field data. Possibly these inconsistencies resulted because of incorrect characterization of modeling parameters. Of course, soil modeling appeared to have a significant effect on the pier behavior. Even stiffer soil properties would have provided more consistent long-term comparison results. Another possible reason for the discrepancy noted between the field results and expected model results could be due to nonlinear temperature effects on expansion and contraction of the superstructure. The model assumed a linear relation existed.

A second long term comparison was made for October 20, 1987, to February 22, 1988. As in the previous table, rotations were calculated at 6:00 a.m. and 6:00 p.m. and referenced to midnight on April 26, 1987. Results shown in Table 13 show that the field tilt data

Table 12. Comparison of tilt readings between field results and analytical model for April 1987 to October 1988.

Date	Time	Magnitude of Tilt Reading on Pier (arc min)				
		Model			Field	
		k = 400 k/in.	k = 2000 k/in.	k = 200,000 k/in.	North Side	South Side
4-27-1987	6:00 a.m.	-3.84	-0.91	-0.16	-0.80	-0.95
	6:00 p.m.	4.99	3.09	2.60	2.71	2.29
4-28-1987	6:00 a.m.	-7.58	-2.61	-1.33	-2.16	-2.74
	6:00 p.m.	2.18	1.81	1.71	3.78	3.95
4-29-1987	6:00 a.m.	-2.84	-0.46	0.15	-0.55	-0.43
	6:00 p.m.	3.25	2.30	1.75	4.77	4.64
10-20-1987	6:00 a.m.	-15.21	-6.07	-3.72	-4.24	-4.25
	6:00 p.m.	-14.98	-5.96	-3.64	-3.83	-3.94
10-21-1987	6:00 a.m.	-18.42	-7.52	-3.96	-5.18	-5.20
	6:00 p.m.	-13.42	-5.26	-3.13	-2.57	-2.85
10-22-1987	6:00 a.m.	-14.63	-5.80	-3.53	-3.76	-3.88
	6:00 p.m.	-9.95	-3.68	-1.65	-1.24	-1.40

Note: All tilt readings are referenced to midnight April 26, 1987 (tilt reading = 1.51 arc min).

Positive reading of tilt implies westward movement, negative eastward movement.

compared more favorably with the stiffer shale results (k = 200,000 k/in.). Note that the difference between field tilt and model tilt for the stiff shale was approximately 1.5 to 2 arc min during February. Overall, the model tilts suggested a more easterly position of the pier than did field data. This trend of a more eastward position of the model relative to the field results is consistent with the comparisons noted in Table 12 for April to October. Reasons for the same discrepancy between model and field results in Table 13 are possibly the same as given for Table 12 results. As before, stiffer soil properties would have caused better correlation between the model and field data.

Table 13. Comparison of tilt readings between field results and analytical model for October 1987 to February 1988.

Date	Time	Magnitude of Tilt Reading or Pier (arc min)			
		Model			Field
		k = 400 k/in.	k = 2000 k/in.	k = 200,000 k/in.	North Side
10-20-1987	6:00 a.m.	-15.21	-6.07	-3.72	-4.24
	6:00 p.m.	-14.98	-5.96	-3.64	-3.83
10-21-1987	6:00 a.m.	-18.42	-7.52	-3.96	-5.18
	6:00 p.m.	-13.42	-5.26	-3.13	-2.58
10-22-1987	6:00 a.m.	-14.63	-5.80	-3.53	-3.76
	6:00 p.m.	-9.95	-3.68	-1.65	-1.25
2-20-1987	6:00 a.m.	-24.60	-10.39	-6.68	-4.77
	6:00 p.m.	-24.80	-10.41	-6.70	-4.89
2-21-1987	6:00 a.m.	-25.30	-10.65	-6.81	-5.63
	6:00 p.m.	-16.90	-6.83	-4.23	-2.50
2-22-1987	6:00 a.m.	-16.45	-6.63	-4.06	-2.41
	6:00 p.m.	-16.01	-6.43	-3.96	-1.78

Note: All tilt readings are referenced to midnight April 26, 1987 (tilt reading = 1.51 arc min).

Positive reading of tilt implies westward movement, negative eastward movement.

Note from Table 13 the rate of net change in movement of the pier from October to February. Recall that from interpretation of the field results during this time period (actually from September 1987 to March 1988), which were discussed in Section 4.2.4, there was an apparent "offset" of field tilt from temperature 0.5 to 1 arc min toward the west. Note that the net change in field tilt in Table 13 suggests the pier was moving more westerly than the model predicted. Actually, the differences in net change in movement between field and model results were approximately 1.5 arc min from October to February. This effect was larger than field data indicated, but the trend in *direction of movement* was consistent.

4.2.5.3. Interpretation of Analytical Model Results

Based on the results of the short and long term comparisons of analytical and field data, we drew a number of conclusions. The analytical model studies provided further evidence that the three major components affecting the measured pier movement in the field include: supporting soil conditions, expansion-bearing translational restraint, and superstructure temperature changes. A strict interpretation of the field data showed that, over both short and long time periods, the pier response in the east-west direction was temperature dependent. In contrast, the north-south pier movement was insignificant and unrelated to temperature. For the east-west response of the pier to be temperature dependent implied that the superstructure was transmitting longitudinal forces in proportion to the superstructure temperature changes. For this to happen, the expansion bearings had to create a restraint condition against longitudinal translation of the superstructure. If the expansion bearings were functioning properly, a constant longitudinal force from the superstructure, independent of magnitudes of changes in temperature, would be acting. Calculations indicated that this force, based on (Eq. 2) shown in Section 3, was much too small to have caused the pier tilts that were monitored.

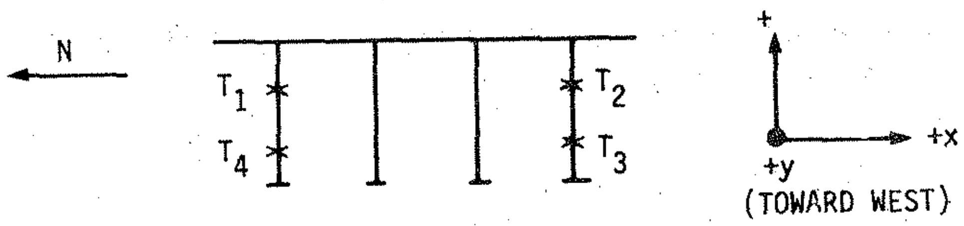
In summary, short term and long term model results, which considered ranges of realistic soil foundation characteristics, as well as the incorporation of superstructure temperature data, helped to support the hypotheses presented for the observed field data. The analytical and field data suggested that most of the movement the pier experienced during the monitoring period was a result of the forces applied to the superstructure due to temperature changes and was recoverable from season to season. A permanent movement of the whole pier appeared to have occurred during the summer of 1987, which may have been caused by a sliding of the footings up the slope. In addition, from September 1987 to March 1988, it appeared that a small permanent movement occurred that might have been caused by movement of the footings down the slope. No significant movement was recorded in the north-south direction.

4.2.6. Surveying

A summary of the reduced data from the three previously mentioned surveys is shown in Table 14.

Table 14. Summary of surveying data for Karl King Bridge.

Target	Change in Measured Movement from May 18, 1987, to June 20, 1987		
	Δx (in.)	Δy (in.)	Δz (in.)
T_1	-0.0670	-0.1176	-0.0840
T_2	-0.0228	0.0300	-0.5520
T_3	-0.0612	-0.0708	-0.1560
T_4	-0.0024	-0.0468	-0.1080
Target	**Change in Measured Movement from May 18, 1987, to August 24, 1987**		
	Δx (in.)	Δy (in.)	Δz (in.)
T_1	-0.3048	-0.9336	0.1800
T_2	0.0744	0.3696	-0.1920
T_3	0.2148	0.2076	-0.0840
T_4	-0.2184	-0.8170	-0.4080



As shown in the table, the changes in movement from May to June were relatively small, but more significant from May to August. The x, y, and z coordinates correspond to movements described, respectively, as parallel to the plane of the pier, perpendicular to the plane of the pier, and vertical. The y-coordinate movement corresponds to the movement in the east-west direction. The data show that the movement was primarily toward the east or toward the river. The magnitude of movement was very small, making it difficult to draw conclusions regarding a trend in movement. The data suggested that the magnitude of movement during this period would be approximately 0.1 in. west at the top of the pier. An approximate angle of tilt is calculated as 1.3 and 1.9 arc min, respectively, toward the east and toward the west for the north and south sides of the pier. The field tilt data (no data exist for June 20, 1987) suggested that the tilt was slightly toward the west during this time period. The survey data suggested that the south side of the pier had displaced more westerly than had the north side of the pier and that the tilt difference was approximately 0.5 arc min. It is worth noting that field tilt data had indicated that (from March 1987 until May 1987) a trend of a more westerly position of the south end of the pier had developed and the difference in tilt angle was approximately 0.8 arc min.

The trend of relative y-coordinate movement of the north and south sides of the pier continued from the period of May to August. The general displacement near the top of the pier was 0.9 in. toward the east on the north side and 0.4 in. west on the south side of the pier. The approximate tilt of the north and south sides of the pier based on the survey data was 2.3 arc min and 3.0 arc min westward, respectively. No data were available from field tilt measurements on this day for comparison. The relative difference in tilt between the north and south sides of the pier was consistent, however, with the field tilt noted during this time period.

Clearly, the magnitude of movement noted in Table 14 was greater for the east-west direction movement of the pier, consistent with tilt sensor data. Transverse movements of the pier (in the north-south direction), represented by the x-coordinates, were relatively small for the period from May to June, and it is difficult to draw conclusions. From the period of May to August, the transverse movement was more significant than the tilt sensor data indicated. In general, the tilt is toward the north at an angle of approximately 1.5 arc min.

Surveying data, which represent the vertical movement of the pier, as represented by the z-coordinates, suggested that relatively large vertical movements occurred during both of the time periods represented in Table 13. The data showed large discrepancies between pairs of targets on the same pier column, which made the data appear questionable. The axial changes in length denoted by targets T_2 and T_3 in Table 14 indicate an axial deformation of approximately 0.40 in.; no data in a similar format existed from the tilt sensor instrumentation for comparison.

A summary of the data in Table 14, when compared with tilt data over a similar time frame, indicates that the direction of movement was similar. However, the magnitude of movement for the pier was quite different, and the data suggest that the survey data were not as accurate and sensitive as the data from the tilt sensors.

4.3. Black Hawk Bridge

4.3.1. Structural

Installation of the equipment on the bridge was completed on March 15, 1987. The tilt sensors were zeroed on the same day. Data recording began on April 2, 1987, and continued through February of 1988. Data from the two tilt sensors, the ambient temperature probe, and the two thermocouples were recorded on an hourly basis. All of the tilt data, accumulated throughout the duration of the project, were based on the initial reference established on March 15, 1987.

As shown in Fig. 13, the north tilt sensor will monitor movement in an east-west direction, and the west tilt sensor movement in the north-south direction. The recorded north tilt corresponds to longitudinal movement relative to the bridge superstructure, and the west tilt readings to transverse movement relative to the superstructure.

As mentioned previously, since the bridge was located at a significant distance from ISU, a modem was placed at the test site and data were retrieved via an existing telemetry system using some equipment at the Iowa DOT in Ames. However, failure of some components of the monitoring system interrupted the data recording and retrieval process on a number of occasions. The modem link apparently provided a pathway for lightning strikes, which caused electrical damage to the modem and micrologger. In addition, the central console unit for the tilt sensors failed once due to apparent moisture effects. These effects resulted in the loss of all data during December of 1987 and January and February of 1988.

Unlike the study conducted for the Karl King Bridge, in which a more thorough understanding of the general response of the bridge was required to discuss the possible movement of Pier No. 4, the Black Hawk Bridge study was more qualitative than quantitative. The reason for investigating long term movement of this bridge was caused by concern over possible effects from accidental barge impacts of the main span, Pier No. 2. Another reason for conducting a more qualitative study was that the superstructure of the Black Hawk Bridge, being a through-truss floor-beam stringer system, is much more complex than the superstructure system of the Karl King Bridge, making interpretation of limited data a much more difficult task. The primary intent, then, of this portion of the study was to continuously monitor Pier No. 2 with tilt sensors to determine if a possible barge impact caused any significant realignment of the pier.

4.3.2. Interpretation of Test Results

The tilt data accumulated between April and the end of November 1987 were reviewed and evaluated to determine any absolute change in pier alignment. Readings of the north and west tilt sensors were plotted on a daily basis to study the daily behavior of the pier and identify any general trends in movement. A few arbitrarily selected plots are presented for discussion. Figure 46 represents the readings of the north and west tilt sensors on April 23, 1987, as well as the ambient temperatures. As shown in the figure, the pier remained essentially stationary throughout the day. The maximum change in temperature during the day was 22° F, varying from 43° F to 65° F. The north tilt-sensor reading, which indicates east-west movement (longitudinal direction of the bridge superstructure), generally followed the ambient temperature. A slight movement toward the west was indicated at the time of the day in which the ambient temperature reached a maximum. The response of the west tilt sensor, which indicates north-south movement (transverse direction of the bridge superstructure), was not affected as significantly by the change in temperature. Figure 47 represents the readings of the north and west tilt sensors on May 30, 1987, and of the ambient temperatures; they indicated that the north end of the pier rotated approximately 0.30 arc min westward during the course of the day. The rotation of the pier in the north-south direction, however, was negligible. The maximum change in temperature during the day was 20° F, varying from 61° F to 81° F. The observed increase in the westerly position of the north end of the pier was attributed to expansion of the superstructure. The change in the north tilt sensor reading again followed the ambient temperature. Figure 52 48 readings of the north and west tilt sensors on July 27, 1987, and of the ambient temperatures. As shown in the figure, the pier experienced negligible changes in rotation. Temperatures ranged between 60° F and 79° F during the day. Again, the north tilt-sensor readings generally followed the changes in ambient temperatures throughout the day. Worthy of mention is that the position of the pier was more westward than was indicated in Figs. 46 and 47. The trend of movement of the pier from April to July was an increasing westerly movement, indicating that the observed westerly shift noted in Fig. 48 was possibly due to seasonal effects of temperature on

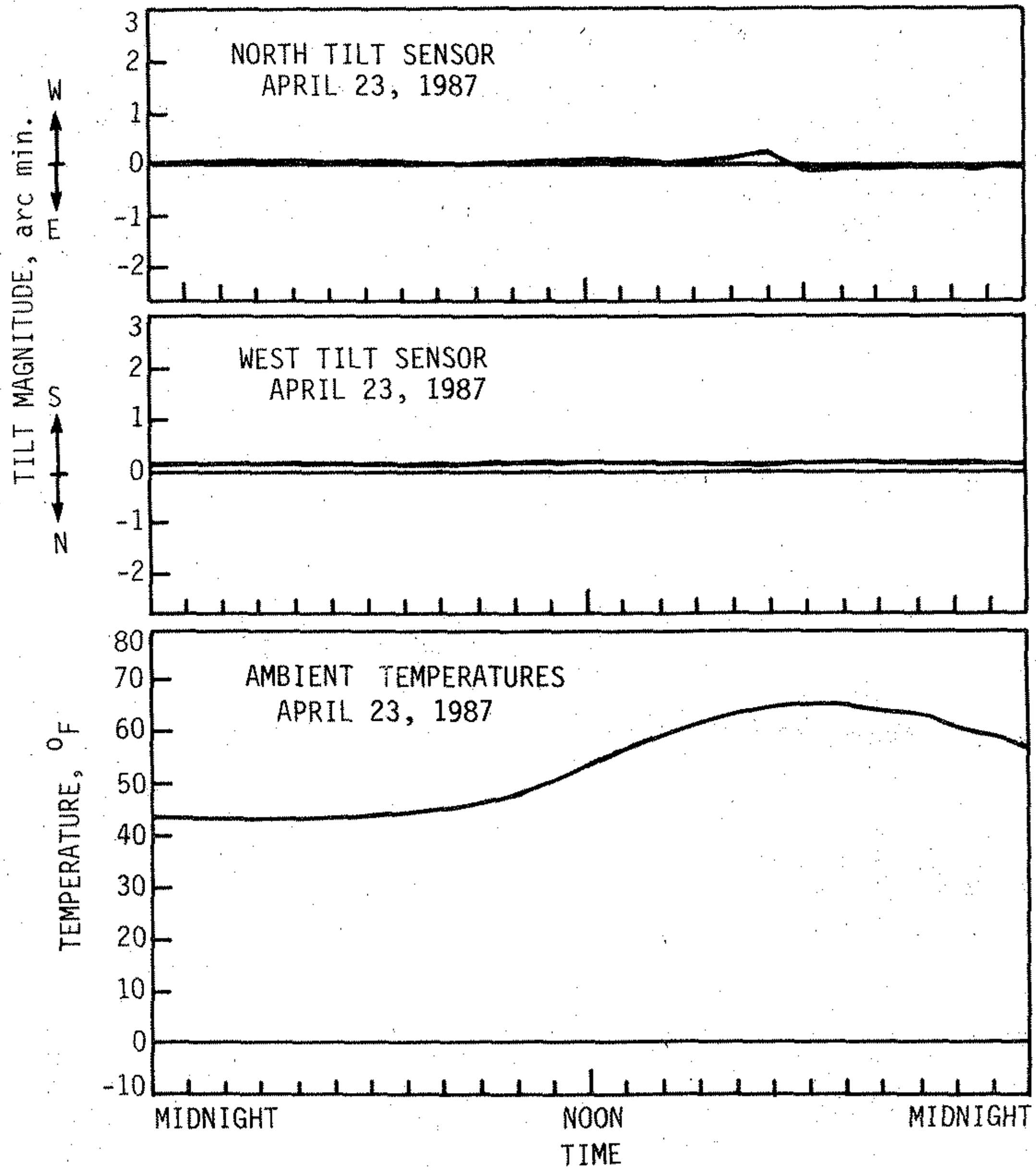


Fig. 46. Readings of north and west tilt sensors and of ambient temperatures on April 23, 1987.

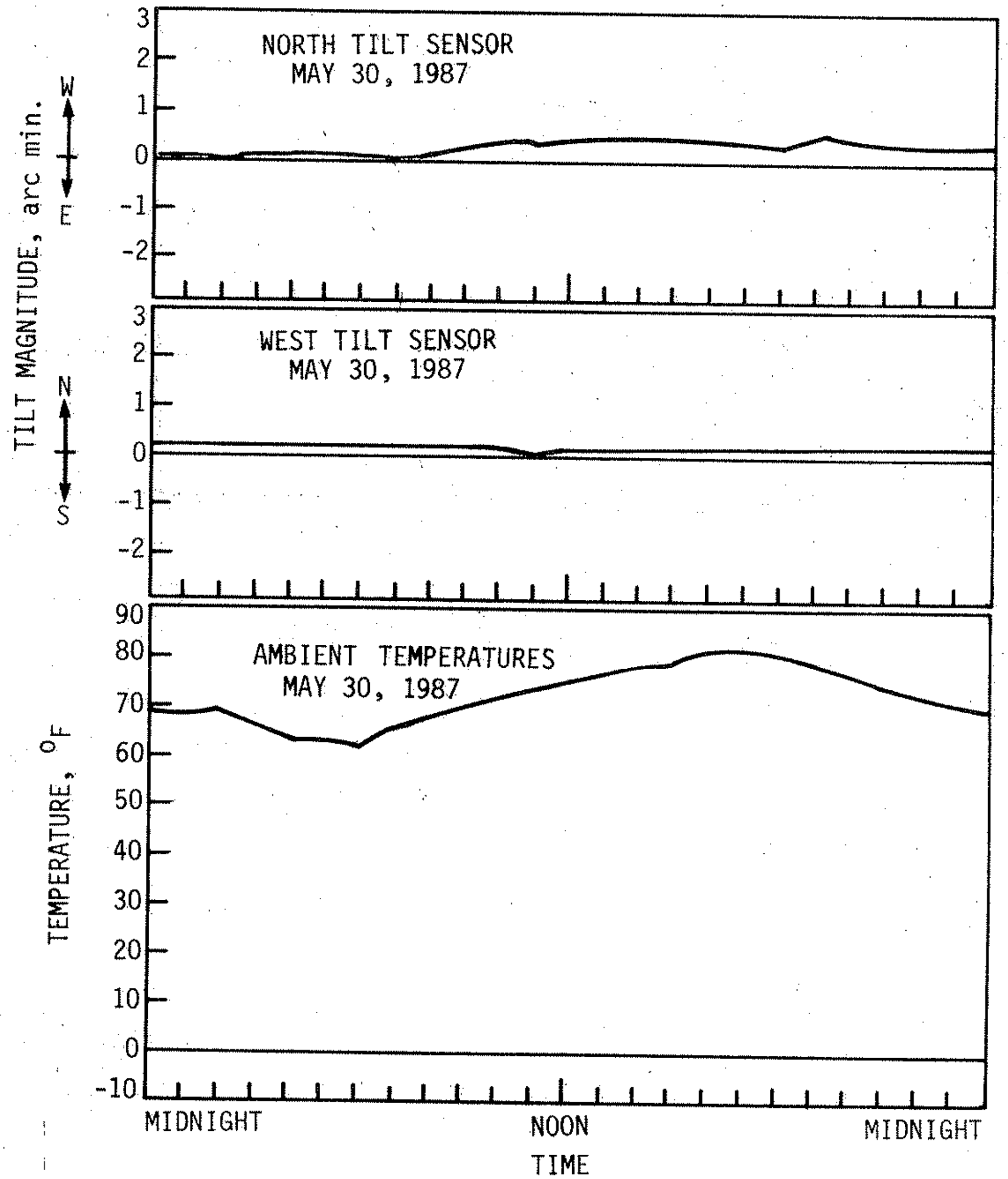


Fig. 47. Readings of north and west tilt sensors and of ambient temperatures on May 30, 1987.

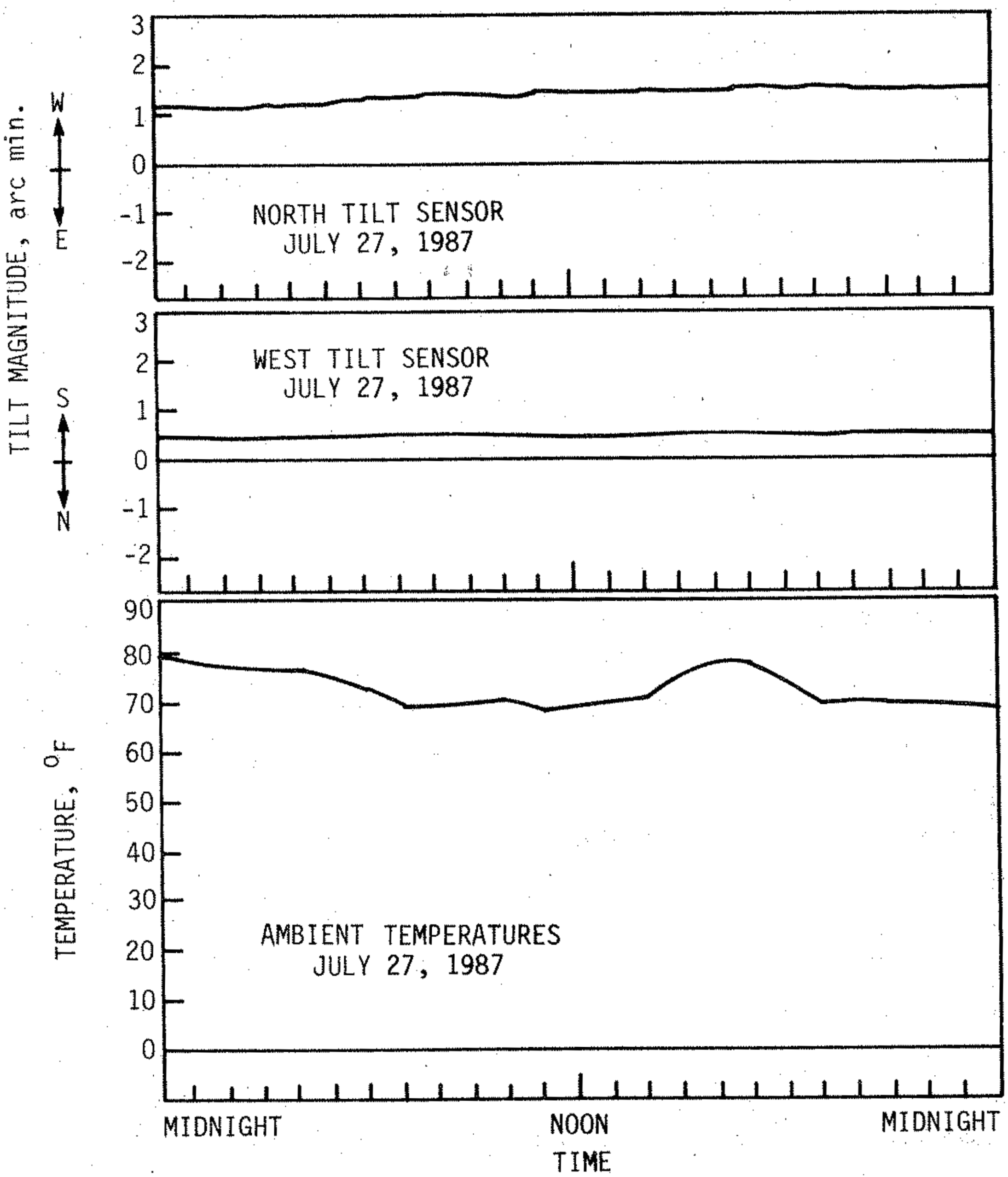


Fig. 48. Readings of north and west tilt sensors and of ambient temperatures on July 27, 1987.

the bridge superstructure. Note that during the time period from April to July, the shift in the transverse position of the pier, as defined by the west tilt sensor, was less significant than the movement in the longitudinal direction of the pier. The trend shown by the north tilt sensors in Figs. 46 to 48 is that the longitudinal movement of the pier was not very sensitive to the ambient temperatures. Although the movement generally followed the ambient temperatures, it is much less sensitive than was noted on the Karl King Bridge Pier No. 4. It is perhaps significant to again mention that Pier No. 4 was designed as an expansion pier and Pier No. 2 as a fixed pier for superstructure expansion and contraction.

Figure 49, representing data for November 4, 1987, indicates that the north end of the pier rotated approximately 1.50 arc min in the north-south direction. The temperature changed from 46° F to 59° F. The observed eastward movement during the day was not entirely unlike daily movement noted on other days. However, the relatively large change in daily rotation of the west tilt sensor was atypical. This will be discussed in greater detail later.

Further examination of the majority of the daily plots of tilt sensor readings followed similar patterns of movement of the pier noted by the previous graphs. The magnitude of daily changes in rotations of the pier in the east-west direction and north-south direction were relatively small. It is interesting to note that, unlike Pier No. 4 on the Karl King Bridge, Pier No. 2 on the Black Hawk Bridge exhibited much greater sensitivity to wind and bridge traffic loads. The research team, during periods of equipment maintenance, observed changes in tilt that the equipment was registering. It was not uncommon on the Black Hawk Bridge to see changes of tilt approaching a tenth of an arc minute during the passage of heavy vehicles, and on one particularly windy day, changes in readings of a similar magnitude were noted. The changes in tilt readings noted on the Karl King Bridge were not even discernible under similar conditions. The implication of these observations was that the daily rotations of the pier illustrated in the plots of tilt can be partly attributed to temperature changes or to applied loads such as traffic and wind. It should also be mentioned that since movement of the south and east ends of the pier were not monitored, it was not certain whether the pier rotated as a unit in the east-west and north-south directions, or experienced differential rotation of one end with respect to the other.

To determine possible long term changes in pier alignment, the tilt data were evaluated over the duration of the project. Readings of the north and west tilt sensors for arbitrarily selected days are plotted in Figs. 50 and 51, along with ambient temperatures. The tilt readings represent the maximum and minimum readings for the day for which they

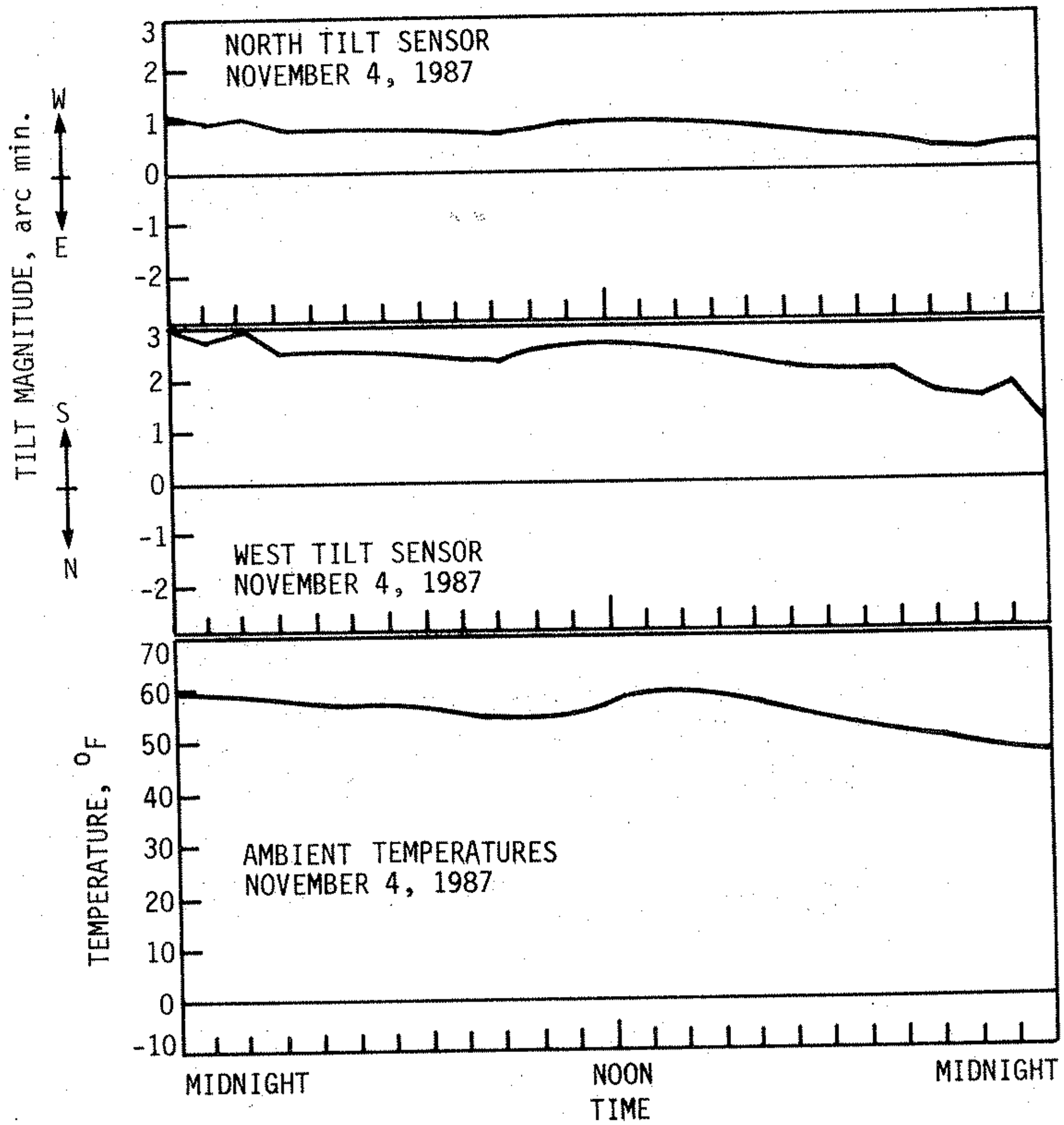


Fig. 49. Readings of north and west tilt sensors and of ambient temperatures on November 4, 1987.

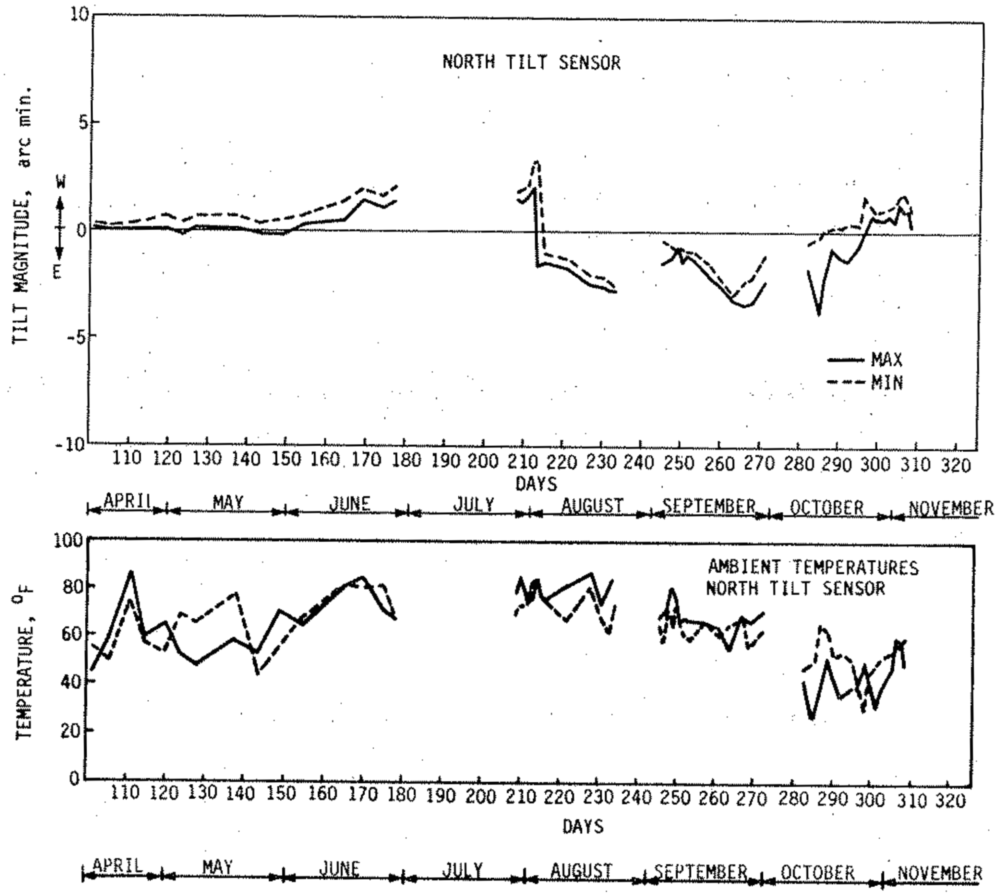


Fig. 50. Readings on north tilt sensor and of ambient temperatures from April 1987 to November 1987.

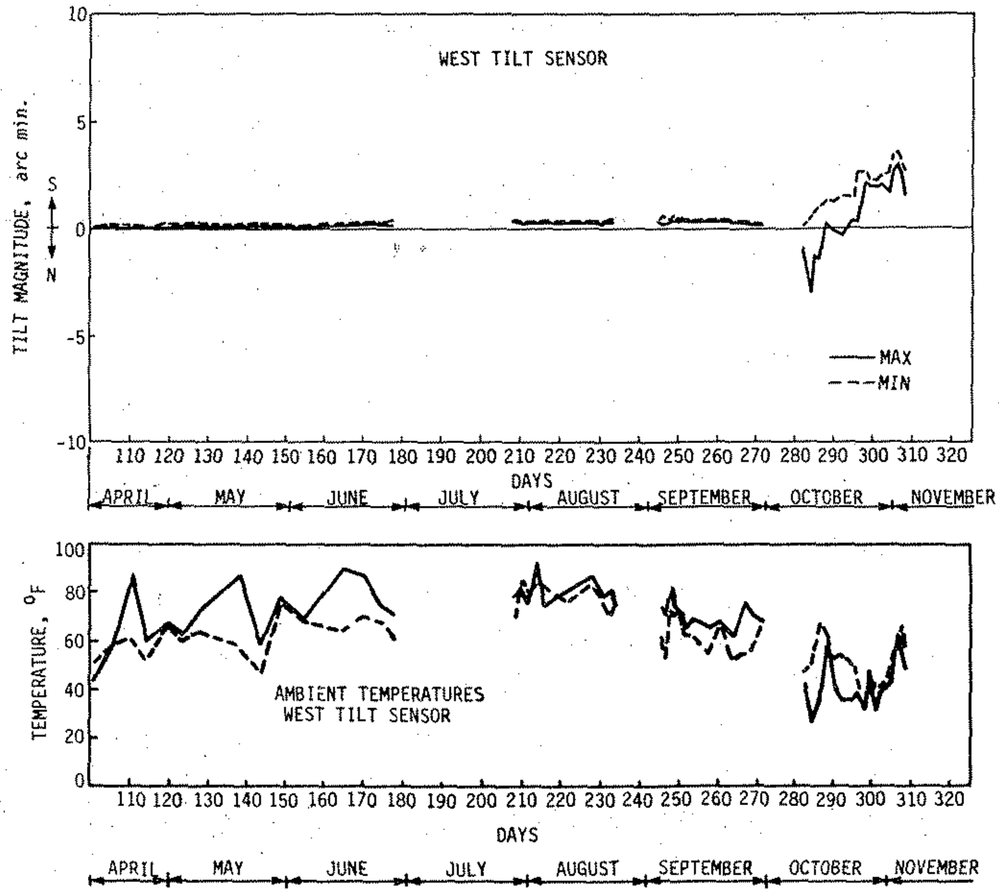


Fig. 51. Readings of west tilt sensor and of ambient temperature from April 1987 to November 1987.

are plotted. The temperature data correspond to the time at which the maximum and minimum readings of the north tilt sensors were recorded. Therefore, in some cases, these temperatures do not truly represent the absolute maximum or minimum ambient temperatures for the day. The temperature data accurately reflect the maximum temperatures, but in many cases, the actual minimum ambient temperature may be 5° to 10° F lower than shown in the figure. The data provide an approximate representation of the position of the pier in the east-west and north-south directions as a function of temperature.

Readings of the north tilt sensors showed that during April the position of the north end of the pier was westward with respect to the reference established on March 15, 1987. However, the magnitude of this westward rotation was relatively small and did not exceed 0.50 arc min. The north end of the pier continued to rotate further westward during the summer until the end of July. The net rotation of the north end of the pier between the beginning of the monitoring period and the end of July was approximately 1.50 arc min westward. As noted in Fig. 50, this observed movement corresponded fairly well with the ambient temperatures over the same time period; the net temperatures increased from April to July. Beginning July 31 and continuing into August 1 the north end of the pier started shifting position in a relatively dramatic manner. This sudden change in pier alignment was atypical and perhaps suggested that atypical external effects occurred. It is worth noting that the east tilt sensor remained stable during this time, as noted in Fig. 51. To highlight this occurrence, a plot of north and west tilt readings and ambient temperatures is shown in Figs. 52 and 53. Note that beginning at approximately midnight on July 30, the north sensor readings began increasing gradually and in a very typical manner, indicating a westward movement. The west tilt-sensor readings remained constant throughout the day, again in a manner typical of previous daily data. Early on August 1 the north tilt readings were again behaving very typically, until near 9 a.m. At this time, the north sensor readings changed suddenly over a period of approximately 4 hrs. The total change in tilt was approximately 3 arc min and indicated an easterly movement (toward the Wisconsin side of the bridge). The tilt corresponded to an approximate linear displacement at the top of the pier or at the foundation of 0.50 to 0.75 in. Note from Fig. 53 that the west tilt-sensor reading remained constant during this time.

The possibility that the significant change in tilt noted over the 4-hr period was caused by a barge impact must be considered. Based on the direction of tilt toward the east, a barge impact would have had a significant westward or eastward component. This, of course, assumes that the barge force would cause a translation or rotation of the pier footing. It

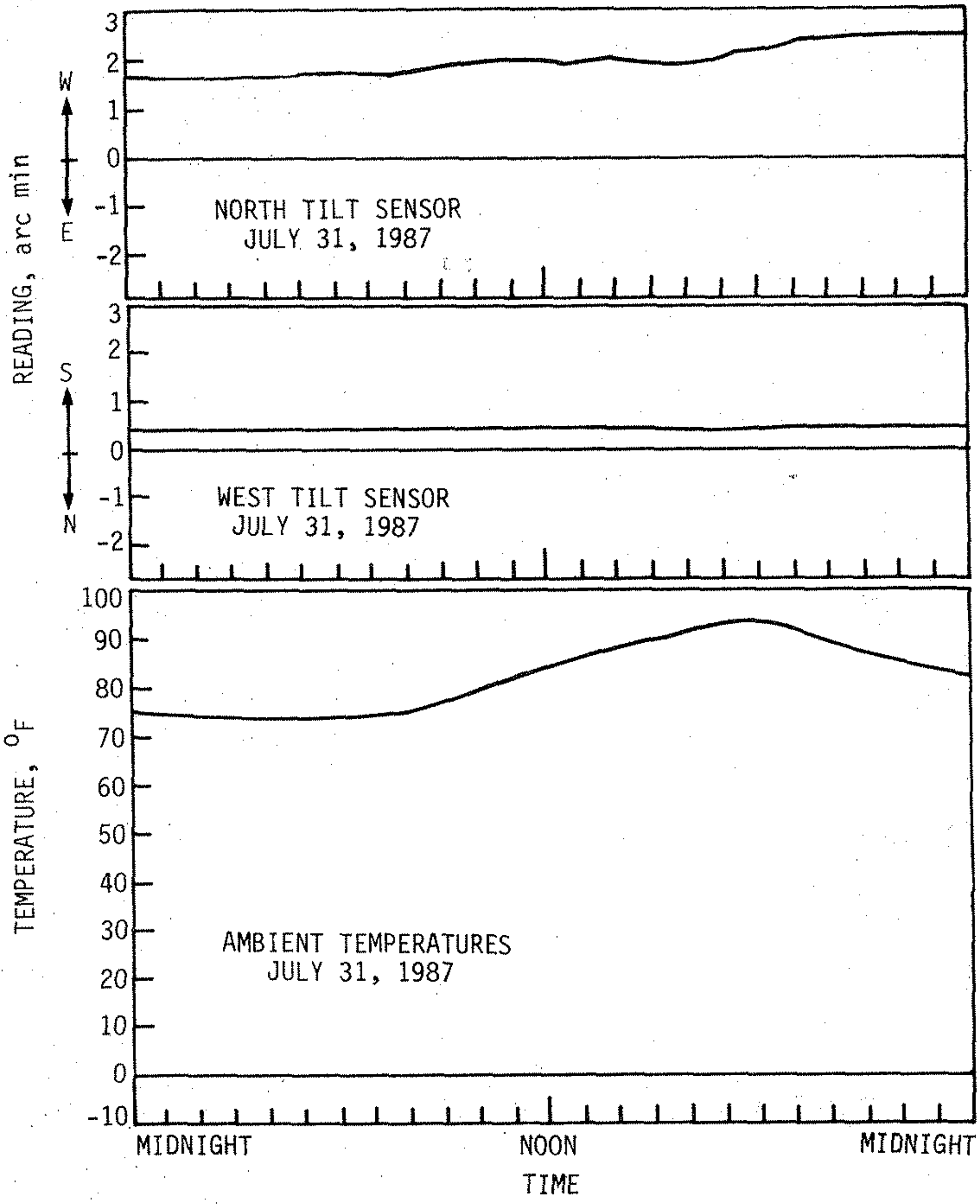


Fig. 52. Readings of north and west tilt sensors on July 31, 1987.

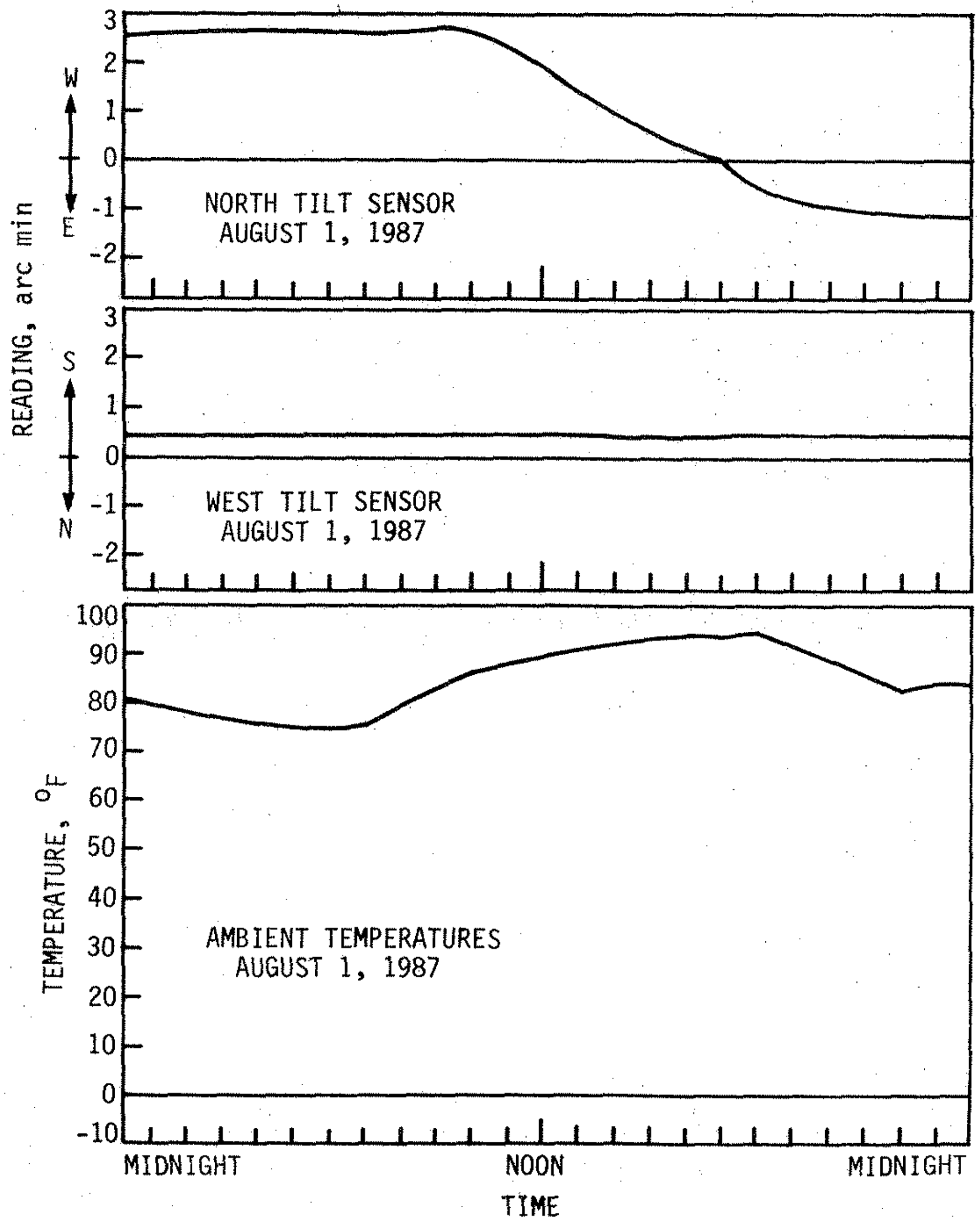


Fig. 53. Readings of north and west tilt sensors and of ambient temperatures on August 1, 1987.

is curious that the west tilt sensor recorded no change in tilt during this period. However, the pier foundation is much more stable regarding rotation in this direction than in the direction denoted by the north tilt-sensor readings. Another question to be addressed in assuming that barge impact could have caused the tilt is how the impact force and pier movement are related. The change in north tilt-sensor reading occurred over a 4-hr period, which seemed like a long time for reestablishing the stability of the pier after an impact. Of course, it seems possible that the real damage done by barge impacts is an eventual undermining of the pier footing. Any external cause that created relatively large local disturbances of the foundation material would create a less stable foundation condition. This perhaps explains the relatively long period (4 hr) before the pier tilt readings stabilized. It should be mentioned that an underwater inspection of Pier No. 2 in November 1985 by American Bridge [15] had indicated no apparent undermining of the foundation.

After the pier tilted suddenly, an eastward rotation of the pier continued for the first three weeks of August. Consistent with previous data, easterly movement followed somewhat cooler temperatures. (The first three weeks of August 1988 were unseasonably cool for the most part, with high temperatures in the 80° F range). The general movement of the pier based on north tilt-sensor readings became somewhat more erratic from early August until the end of the monitoring period than what had occurred from April to August. This was possibly due to the corresponding unusual and erratic pattern of ambient temperatures during this time period. Warm temperatures during the last week of August had the effect of causing the pier to move westerly, until temperatures began to cool at the beginning of September. At this time and through the end of September, the pier began an eastward movement. Unusually warm days during October, and an even warmer November, caused the pier to begin moving westerly again.

In contrast to the movement denoted by the north tilt-sensor readings, it is interesting to note (as shown in Fig. 51) that the west tilt-sensor readings remained essentially constant throughout the period from March to the end of September. There was a slight southward movement of approximately 0.2 arc min during late July. The position of the pier remained constant until the end of September. At this time, until the end of the monitoring period in November, the movement was very erratic. During this time period, an interesting behavior was noted by both the north and west tilt sensors. Contrary to behavior exhibited earlier in the monitoring period, both the north and west sensor readings became very dependent upon ambient temperature. The west and north readings "echoed" each other, as well as the temperature. This is noted in Figs. 54 and 55, which represent data, respectively, for

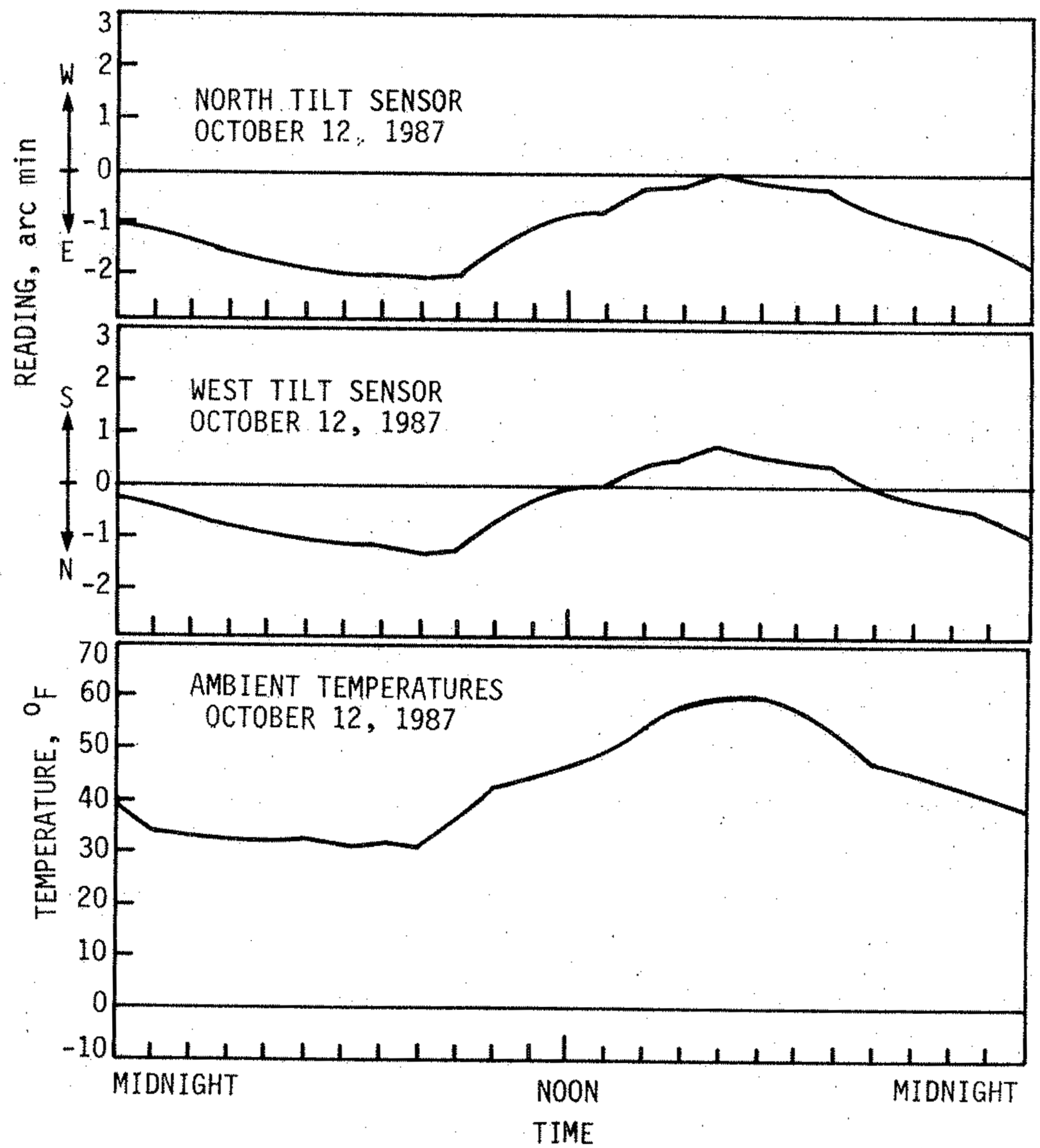


Fig. 54. Readings of north and west tilt sensors and of ambient temperatures on October 12, 1987.

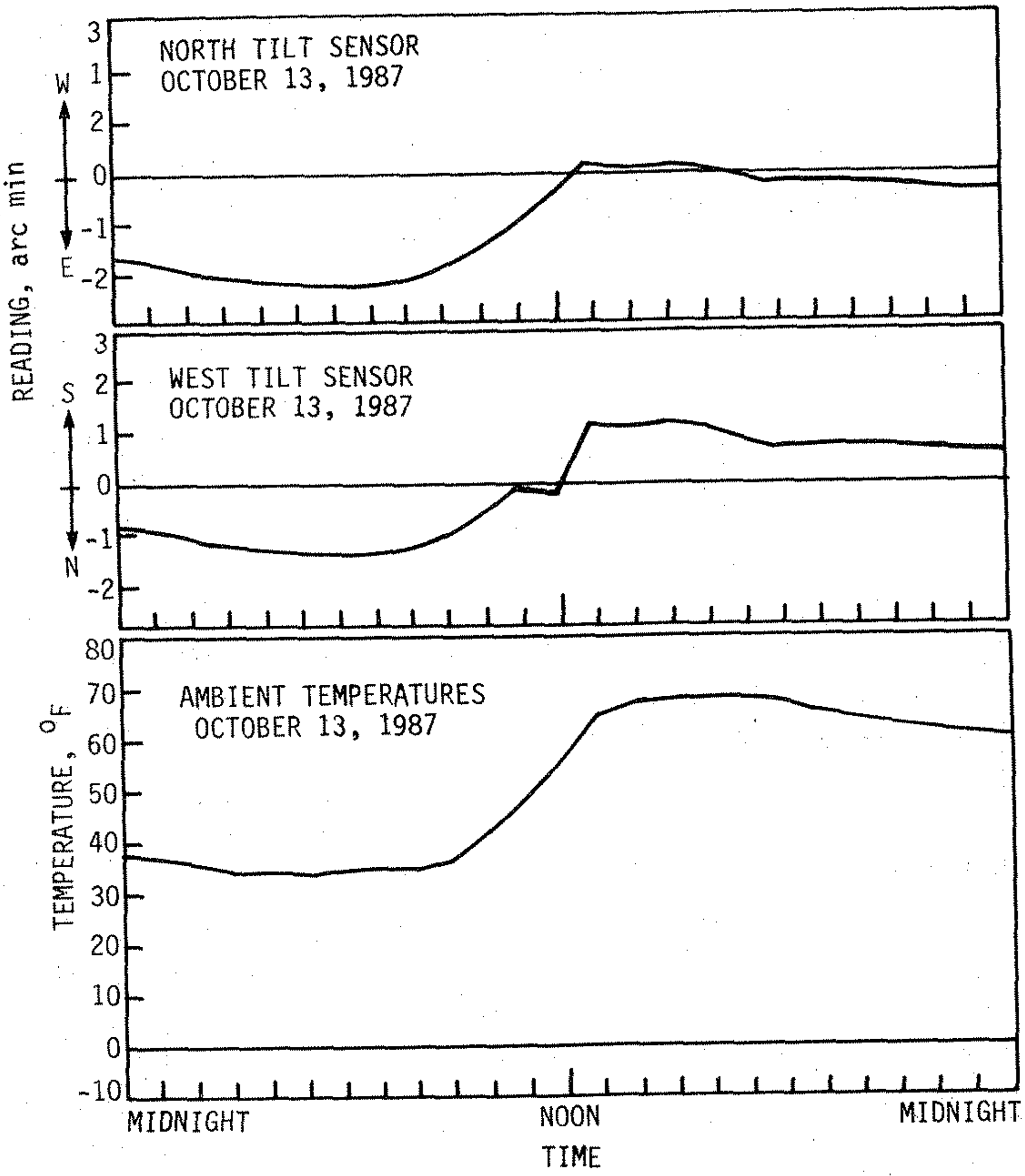


Fig. 55. Readings of north and west tilt sensors and of ambient temperatures on October 13, 1987.

October 12 and 13, 1987. The dependence of north tilt to ambient temperature was approximately two times as great on a daily basis, as was noted earlier. The west sensor dependency on temperature was considerably greater than noted in earlier data. The research team questioned the reliability of these data, in light of the bridge's previous movement tendencies. A concern existed as to whether a problem with the sensors or monitoring equipment existed. From prior experience with equipment problems, and from consultation with technical support staff from the equipment manufacturers, the research team could not conclude definitively that the sensors and equipment were functioning improperly. This left the possibility of moisture problems with the tilt sensors as the only possible equipment-related explanation for this behavior. Daily data readings during the time period were most affected on days when evening temperatures dropped below, or near freezing, and the following daytime temperatures increased significantly. The implication was that these conditions led to condensation on the tilt sensors, which affected the output signal. It would have required removal of the sensors from the pier and subsequent testing to have concluded that they were the source of the possible problem. This would have caused us to lose our initial long-term reference point for measurement. Since no other problems such as this were encountered with the other tilt sensors, after consultation with supporting technical staff, the team concluded that the probability of this being a source of error was small. Figure 56, where data are plotted for October 26, 1987, illustrates the time when the tilt readings stabilized. Still, the north and west sensors continued to "echo" each other's tilt readings.

Reasons for the erratic behavior noted from October through November are not obvious. It may be possible that the complexity of expansion and contraction movements from temperature changes for this type of structure were partly responsible. Perhaps the combination of unusually warm day time temperatures and normally cool evening temperatures were directly related to the movements. The only way to be certain of this possible explanation is to obtain significantly more data than presented here (perhaps over a few years) so that a clear pattern of long term behavior may be determined. Another possible explanation could be provided in knowing the actual condition of the pier foundation. If any undermining had occurred, it could make the pier less stable, and forces transmitted to the pier from superstructure expansion or contraction could be more significant. Still, it is puzzling that the west sensor (indicating movement of the pier transversely to the bridge span) would seemingly be affected by forces in a direction perpendicular to the direction of movement. Although the graphs in Fig. 51 show dramatic changes in west sensor tilt

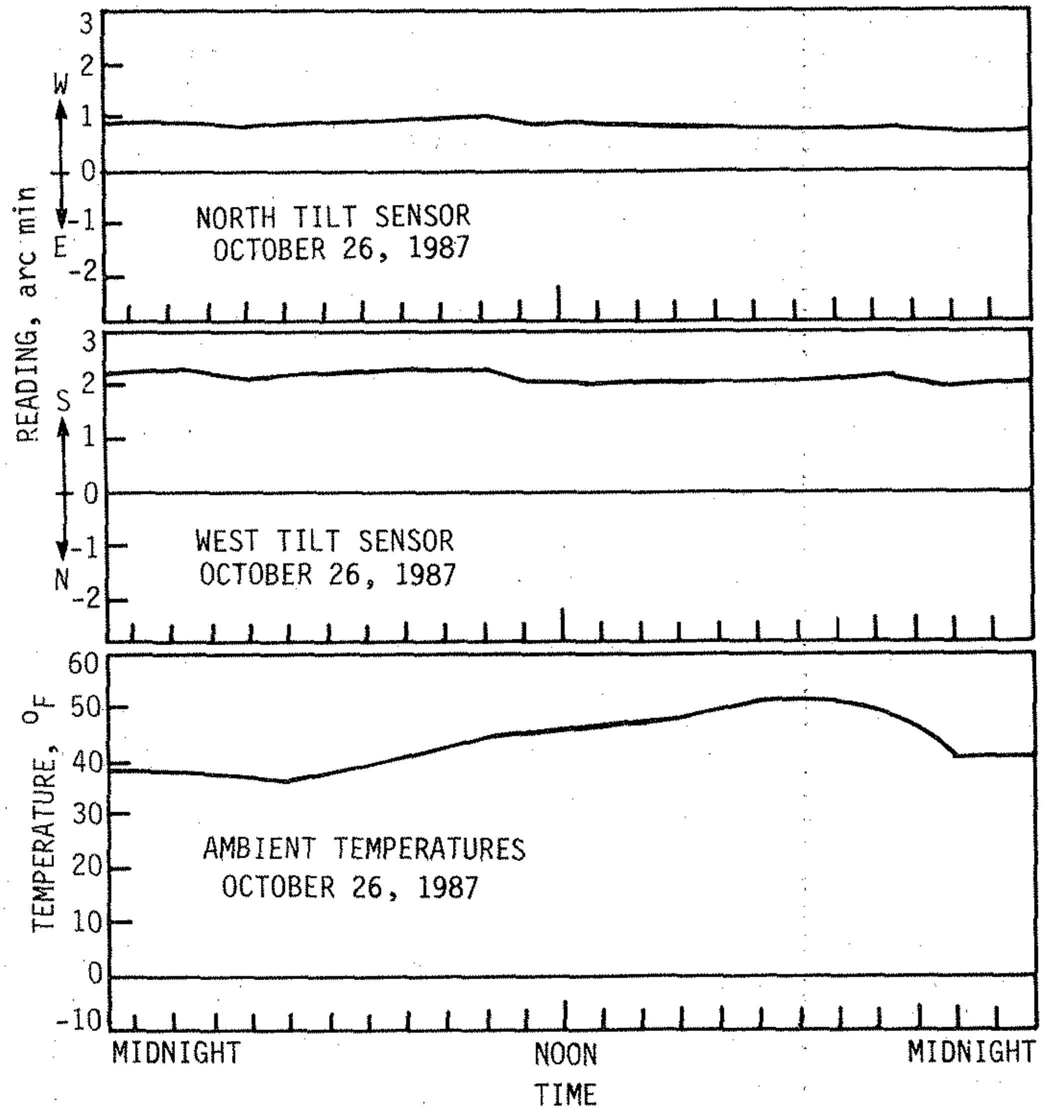


Fig. 56. Readings of north and west tilt sensors and of ambient temperatures on October 26, 1987.

readings after October 1987, the magnitude of this movement in terms of linear displacement at the top of the pier are from 0.10 to 0.15 in. over an approximately 1 mo. period. On a daily basis, these tilts indicate movements of approximately 0.05 in. or less.

In summary, the tilt data showed that the movement in the east-west direction was temperature dependent, indicating that the superstructure expansion and contraction apparently had an effect on the behavior of the pier. In contrast, the pier movement in the north-south direction was primarily independent of temperature except during the monitoring period after October 1987.

It appeared that a possible barge impact, with a major easterly or westerly component of force, occurred during the first part of August 1987. No other unusual movement in the east-west direction occurred during the monitoring period. An atypical and relatively significant movement was noted toward the end of October 1987 for movement to the south. All other movement in the north-south direction was insignificant during the monitoring period.

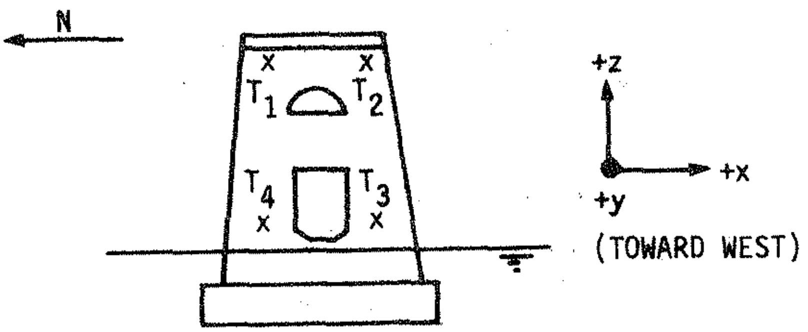
4.3.3. Surveying

Table 15 provides a summary of the surveying data taken on the three dates previously mentioned. As shown in the table, the changes in movement from May to June were relatively significant. The x, y, and z coordinates corresponded to movements described, respectively, as transverse to the bridge span, longitudinal to the bridge span, and vertical. The y-coordinate data corresponded to a direction longitudinal to the bridge superstructure. The movement from May to June was toward the west; the top of the pier was about 0.7 in. more westerly in June than in May. This trend in movement was consistent with the tilt data over the same time period. An approximate angle of tilt of the pier based on the surveying data is 2 arc min, which compared very favorably with the tilt data for this time period.

The time period from May to August 1987, also shown in Table 14, shows smaller movement than from May to June. The y-coordinate data suggested that the top of the pier was approximately 0.4 in. west of the initial position in May. Note that the position of the lower target on the south end of the pier, target T_3, moved approximately 0.2 in. toward the east. The north side of the pier, represented by target T_4, had moved very little. An approximate tilt angle of the pier based on these data suggested that the pier was tilted 1.5 arc min toward the west. This compared poorly with the measured field tilt of 2.5 arc min toward the east, as shown in Fig. 50. From June until August, the data suggested the pier top

Table 15. Summary of surveying data for Black Hawk Bridge.

Target	Change in Measured Movement from May 1, 1987 to June 27, 1987		
	Δx (in.)	Δy (in.)	Δz (in.)
T_1	0.5480	0.6864	0.4800
T_2	0.9084	0.6000	0.4800
T_3	0.1452	0.0828	0.8160
T_4	0.4404	2.3472	0.2280
Target	**Change in Measured Movement from May 18, 1987 to August 24, 1987**		
	Δx (in.)	Δy (in.)	Δz (in.)
T_1	0.1040	0.3264	0.0840
T_2	0.2472	0.3960	0.0960
T_3	-0.2280	-0.2052	0.2400
T_4	0.1044	0.0720	-0.336



moved more easterly, approximately 0.3 in. The bottom targets also suggested that from June to August the bottom part of the pier moved 0.3 in. toward the east. This net eastward movement of the pier of 0.3 in. was consistent with the prediction of probable movement based on the field tilt data.

The x-coordinate data in Table 15 showed that the pier moved approximately 0.5 in. south from May to June, rotating toward the south at an angle of approximately 0.5 to 3.5 arc min depending on which pair of targets were used for the calculation. The field tilt data indicated that only a few tenths of an arc minute of rotation toward the south occurred during the same time period. From May to August, the data implied that an even smaller rotation toward the south occurred than in the earlier period above. This calculated angle corresponded to 0 to 2 arc min, depending on the pair of targets used in the calculation. This compared to the measured field tilt of approximately 0.2 arc min from sensor data.

Relatively significant vertical displacements of the pier occurred from May to June. These displacements are denoted by the z-coordinate in Table 15. The data showed a vertical displacement of approximately 0.5 in. upward. From May to August, the data showed vertical displacements of approximately 0.1 to 0.2 in. No comparable data was obtained from field data for comparison.

In summary, comparison of the surveying and field tilt data showed that the measured trends of movement were similar. The magnitude of movement compared fairly well also. The surveying data were not continuous, nor as sensitive as the tilt data, so care must be taken in interpreting the data. It appeared that the data support the conclusions developed earlier regarding movement of the pier based on tilt data only.

5. SUMMARY AND CONCLUSIONS

5.1. Summary

This report summarizes the work performed in Phase II of a study related to measurement of long-term structural movement in bridges. The work completed in Phase I was presented in Ref. [1]. Phase II had several specific tasks; the conclusions from these follow.

An investigation into the feasibility of field use of a tilt sensing system purchased by the Iowa DOT was undertaken in Phase I of this study. The tilt sensing system has been used in bridge monitoring applications in recent years. The Phase I study verified that the system was accurate and reliable for field use. Two bridges were identified by the Iowa DOT as requiring long-term movement detection. The ISU structures team then designed and installed a tilt sensing system including data acquisition equipment as the beginning of Phase II of this study.

The bridges chosen were the Karl King Bridge in Fort Dodge, Iowa, which spans the Des Moines River, and the Black Hawk Bridge in Lansing, Iowa, which spans the Mississippi River. Pier No. 4 on the Karl King Bridge has been under observation by the Iowa DOT since late 1970, since inspections of the bridge prior to their monitoring lead to the rockers at the pier being repositioned. In addition, severe distress observed in the exterior columns of the pier near the footing led the DOT to cast large concrete collars around the distressed area.

The Iowa DOT has also monitored the Black Hawk Bridge by surveying techniques since becoming aware of accidental barge impacts occurring at Pier No. 2 in the river channel. To date, the only observed distress has been local spalling of the concrete near the waterline.

The field instrumentation systems at both bridges were installed on each of the suspect piers: Pier No. 4 on the Karl King Bridge and Pier No. 2 on the Black Hawk Bridge. The systems consisted of the tilt sensing units and power sources and a programmable data logger for storing the measured data. The Black Hawk Bridge system included a telephone telemetry system in which data was retrieved via modem hookup at the Iowa DOT office in Ames, Iowa. Additional temperature data using thermocouples were taken at the Karl King Bridge to allow a thorough study of the observed movement relative to temperature.

Analytical models of a portion of the Karl King Bridge superstructure and of Pier No. 4 were developed to study the long term behavior of the bridge. Temperature data from the field observations were used in the superstructure model to assess the magnitude of longitudinal forces developed in the superstructure. These predicted forces were then applied to the pier model for prediction of the pier movement. These data were then compared with the observed field data.

The tilt sensing equipment and data acquisition system designed for this study provided accurate continuous monitoring and recording of the bridge movements. The system was very sensitive to the bridge movement, and for most of the monitoring period it performed reliably. However, on three occasions (twice at the Black Hawk Bridge), components of the equipment failed and gaps in the data resulted. Routine measures can be taken in the future to eliminate the problems that caused this trouble, including using 110 AC power instead of battery power and providing an environment for the power console that is more moisture resistant.

The results obtained from an analytical study with finite element models of the superstructure and pier of the Karl King Bridge showed fairly good correlation with field data; the results were used to provide verification of observed movement from field data. The models showed that the soil foundation properties appeared to change from season to season. With temperature data input from field temperature sensors, an attempt was made to verify the observed changes in tilt with model results. The effect of end restraint caused by expansion bearings apparently not functioning as intended was modeled, and the model verified conclusions derived from the field data that forces larger than expected were applied to the pier.

The majority of the observed movements at the Karl King Bridge Pier No. 4 appear to be recoverable from season to season, with the exception of the relative change noted in movement between the north and south side of the pier between March and May 1987 and an apparent movement of the whole pier from September 1987 to March 1988 and during the summer of 1987.

The longitudinal movement of the Black Hawk Bridge Pier No. 2 was shown to be related to ambient temperature, although not as much as Pier No. 4 at the Karl King Bridge. However, no attempt was made to study thoroughly the bridge's temperature characteristics. Based on the data recorded, there appeared to be an event in early August 1987 where a sudden change in orientation of the pier occurred for which temperature could not be rationalized as the source of movement, thus implying a barge impact. The change in orientation was in the longitudinal direction of the bridge.

For most of the reporting period the transverse movement of the pier (transverse to the bridge span) was negligible. However, at the end of October 1987, the tilt readings became more erratic. These changes in tilt corresponded to relatively small magnitudes of deflection at the top of the pier.

5.2. Conclusions

The following conclusions were developed as a result of this study:

1. The tilt sensing system developed by ISU for this project can be advantageous for long-term movement monitoring of bridges and other types of DOT structures.
2. The tilt sensors were very stable, sensitive, and accurate.
3. Using a battery-powered control system for the micrologger had a detrimental effect on the monitoring system's reliability. Moisture problems also caused occasional problems with the Sperry console unit.
4. Typical surveying methods used to monitor structural movement are relatively ineffective and not sensitive enough to allow the movement characteristics to be accurately assessed.
5. The daily movement of Pier No. 4 of the Karl King Bridge in the bridge's longitudinal direction was cyclic in nature and was directly related to the ambient temperature.
6. The long-term longitudinal movement of Pier No. 4 was also directly related to the seasonal changes in ambient temperature and was cyclic in nature. The seasonal movement could be described as recoverable from season to season.
7. The Pier No. 4 longitudinal movement was caused by the direct application of superstructure forces developed by restraint against movement at the expansion bearings.
8. The foundation soil properties at Pier No. 4 were apparently affected by seasonal changes in the weather.
9. A relatively small, permanent longitudinal movement relative to the north side of Pier No. 4 occurred at the south side of the pier between January and March of 1987. The relative difference in movement at the top of the pier was approximately 0.05 to 0.08 in. In addition, a permanent movement of the whole pier (0.03 to 0.07 in.) was noted between September 1987 and March 1988 and a permanent movement of approximately 0.07 to 0.2 in. during the summer of 1987.

10. No long term movement was recorded in the direction transverse to the bridge span (north-south direction) of the Pier No. 4 during the monitoring period.
11. The movement of Pier No. 2 of the Black Hawk Bridge in the bridge's longitudinal direction was related to ambient temperature. This temperature dependency was not as great as that noted on the Karl King Bridge Pier No. 4.
12. An apparent and relatively significant permanent change in alignment of Pier No. 2 occurred in early August 1987, possibly as a result of a barge impact. The tilt movement was toward the east.
13. The long-term transverse movement (north-south direction) of Pier No. 2 was negligible during the monitoring period until October 1987. Movement became erratic and, in some cases, the tilt readings became much more sensitive to temperature.

6. RECOMMENDED CONTINUED STUDIES

This research study has shown that the tilt sensing system can be used successfully for long-term structural monitoring. Specifically, the results of the evaluation of the Karl King Bridge should serve notice that, although no significant long-term permanent movement was detected, the sensing system should continue to be used by the Iowa DOT for monitoring Pier No. 4. In view of the results of this study, the following is recommended:

- Continue long term study of Pier No. 4 using the tilt sensing system to get an even longer "track record" for understanding the movement due to temperature. In addition, the pier should be gaged with vibrating-wire strain gages to verify the possible strain caused by the hypothesized longitudinal forces applied by the superstructure.
- Devise further field tests that involve geotechnical studies along with structural tests to correlate both effects in isolating the pier movement. These tests should quantify the reasons for movement and suggest methods for eliminating them.
- Monitor movement by another system for obtaining redundancy in measurement on a periodic basis along with the continuous monitoring, tilt sensing system. This could be an accurate survey-based technique, but it is recommended that a second structural based technique, such as one utilizing LVDT displacement transducers, might be a better choice.
- Design an instrumentation system to monitor bearings and their behavior with regard to causing restraint against contraction and expansion of superstructure. At the same time, develop an instrumentation system to monitor superstructure forces.
- Provide electrical power at the Karl King and Black Hawk Bridge sites to replace the battery systems presently in use for the instrumentation.

7. ACKNOWLEDGMENTS

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APPENDIX: SUMMARY OF TEMPERATURE SENSITIVITY TESTS FOR TILT SENSORS

Three tests were conducted for each of the four tilt sensors and the changes in measured tilt angle were plotted versus time and temperature. From the data, temperature coefficients were calculated as the change in angular reading relative to the change in surface temperature of the sensor (for each tilt sensor) according to the following equation (Table A.1).

$$temp.\ coeff. = \frac{change\ in\ test\ angle - change\ in\ reference\ angle}{change\ in\ surface\ temperature}$$

Table A.1. Temperature coefficients for tilt sensors.

Tilt Sensor Serial Number	Temperature Coefficient (arc seconds / degree F)
21215003	-0.87
21215004	-0.04
21215005	-0.20
40215097	-2.22
Manufacturer's coefficient	0.30

The interval of time (and temperature) over which the coefficient was determined was defined by the initial readings until the change had stabilized. In most tests, the sensor angle change stabilized at approximately 10 min, and therefore, the coefficient was based upon this time frame. The test format was unable to create results that were reproducible. The temperature coefficient given in the table represents the average of the three tests. As noted, two of the sensors do not meet the manufacturer's coefficient, which was provided by Sperry for each sensor.

It should be noted that the Sperry tests are conducted under different conditions. The ISU tests were designed to simulate rather than duplicate the Sperry tests because of cost restrictions. The manufacturer's specifications are based on testing the internal components of the sensor, in place, in a controlled test chamber. The ISU tests suggested that the discrepancies were caused by external effects such as mounting the plate and sensor to the test block, localized warping of plate and/or concrete due to temperature differences, or systematic error in the test procedure. Independent tests of a similar type to ISU's tests by teh Michigan DOT [16] resulted in lack of repeatibility of results with conclusions of the same type as suggested here. Thus, the ISU team decided to use the manufacturer's coefficient for recorded field data.