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16. Abstract

This report reviews the various techniques considered for instrumenting the Rte. I-295 cable-stayed bridge over the James River near Richmond, Virginia. From this review an instrumentation plan is developed to meet the following objectives:

1. to determine the live load stress range in several critical cable stays,
2. to obtain data that will indicate the magnitude of the torques introduced into the cable-stayed box girders as a result of service loads and stay cable forces,
3. to obtain thermal gradient data for a typical box girder section, pylon section, and stay cable,
4, to evaluate the performance of the delta frame assemblies that connect the two box segment roadways to the central, single plane, stay-cable system.

The instrumentation and data acquisition systems to be used to accomplish these objectives are presented and discussed in the report.

17. Key Words Cable-stay bridge, instrumentation, strain gages, thermocouples, strains, precast concrete, box segments, delta frame, thermal gradients, pylon, data acquisition, Environmental Protection

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mi/h	km/h	1.609 344 E+00
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Temperature: $(°F-32)^5/9 = °C$

INTERIM REPORT

FIELD MONITORING ON THE I-295 BRIDGE OVER THE JAMES RIVER -- INSTRUMENTATION INSTALLATION AND CONSTRUCTION PERIOD STUDIES --

by

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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SUMMARY

An instrumentation plan is presented for the segmentally constructed cable-stayed box girder bridge that will carry I-295 over the James River east of Richmond, Virginia. The instrumentation plan is designed to meet several objectives:

- to determine the live load stress range in several critical cable stays
- to obtain data that can be used to infer the magnitude of torques introduced into the cable-stayed box girders as a result of service loads and stay cable forces
- to obtain thermal gradient data for a typical box girder section, pylon section, and stay cable
- to evaluate the performance of the delta frame assemblies in distributing stay cable forces between the supported boxes.

Stay Cable Instrumentation

Techniques for stay cable instrumentation are reviewed in Chapter 2 of the report, and an instrumentation scheme is presented in Section 2.3. The stay cable instrumentation scheme consists of strain gages attached directly to the stay cable wires at the locations shown in Figures 2.2 and 2.3. The gages will be waterproofed using a combination of epoxy and rubberized coatings to protect the gage from the weather, the subsequent grouting process, and from any incidental shifting of the strands relative to each other. Shielded, Teflon-jacketed and Teflon-insulated lead wires will be extended from the gages out of the polyethylene sheath at the location of the expansion pipe section nearest to the deck. The gages will be independently wired to minimize loss of data as a consequence of any malfunctioning gages. Two gages will be placed on each instrumented strand on diametrically opposite wires. Four strands will be instrumented on each instrumented cable. Stays S-1, S-7, and S-13 will be instrumented on the main span.

Box Section and Pylon Strain Instrumentation

The instrumentation plan for the box girder and pylon sections is presented in Chapter 3 of the report. The strain data will be gathered from reinforcing bars instrumented with strain gages. A schematic cross section of an instrumented rebar is shown in Figure 3.2. The instrumented reinforcing bars will be located with a primary goal of obtaining data on the longitudinal strains and the shearing strains caused by

shear and torsion in the box section. Measurements of local deformations of the box girder are not a primary objective of the present study. The locations of instrumented box girder sections are shown in Figure 3.5. The locations of instrumented reinforcing bars within the box girders are shown in Figures 3.6 and 3.7. The locations of instrumentation within the pylon segments are shown in Figure 3.11. The locations of the instrumented reinforcement bars in pylon segment D-6 are shown in Figure 3.12. Instrumented reinforcement located in the top of the cast-in-place portion of the pylon is shown in Figure 3.13.

Thermocouple Instrumentation

A single section of the box girder, located as shown in Figure 4.1(b), will be instrumented with a number of type T thermocouples at the locations shown in Figure 4.1(a). The pylon section shown in Figure 4.3(b) will be instrumented with thermocouples located as shown in Figure 4.3(a).

Data Acquisition System

Chapter 5 describes the data acquisition system selected for the project. Related problems include data retrieval and analysis, electrical surge protection, and environmental protection of the data acquisition system.

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I. BACKGROUND AND OBJECTIVES OF THE STUDY

1.1 Introduction

Subsequent to World War II, the need to redevelop transportation networks damaged or destroyed during the war led European bridge designers to devise numerous innovative bridge designs and construction techniques. Two of these innovations that have recently been utilized in the United States are cable-stayed and segmentally prestressed concrete box girder bridges. Cable-stayed bridges are stiffened by a number of stay cables that are anchored to pylons, are connected directly to the stiffening girder of the superstructure, and are pretensioned to provide a stiff, load-resisting system (Podolny & Scalzi, 1982, Gimseng, 1983). In segmentally prestressed concrete construction, the traditional span restrictions for prestressed concrete bridges are avoided by constructing the span as a series of short, usually transverse segments. The individual segments may be either cast in place on an existing cantilever or precast in a bed and transported to the erection site. New segments are post-tensioned sequentially as they are added to the structure, and span continuity is assured by a cast-inplace closure pours and subsequent continuity post-tensioning (Podolny & Muller, 1982; PTI/PCI, 1978).

Cable-stayed and segmentally constructed bridges have in common the efficient use of high-strength materials. Moreover, the use of cantilever construction and cable stays in conjunction with segmental post-tensioning has been found to be competitive for spans exceeding 1000 ft. The analysis and design of cable-stayed and segmentally erected post-tensioned bridges are quite complex. Such a bridge is typically continuous over several spans. The numerous stages of erection through which the bridge passes, the inherent static indeterminacy of a multispan box girder bridge with multiple supporting stay cables, and the nonlinear behavior of the stay cables make both short-term and long-term predictions complicated. Further complexity is introduced by the presence of time-dependent deflection caused by the creep and shrinkage of the concrete and the relaxation of stay and post-tensioning cables. Numerous intermediate anchorage locations, external prestressing tendons, and points of stay cable attachment introduce concentrated loads into the box sections.

With such complex behavior, and considering the relative newness of the technology, it is not surprising that there have been some problems in previously erected segmentally prestressed structures. These problems include excessive shortening of multispan, precast, segmentally prestressed girders because of creep and shrinkage and severe cracking of the girders because of unaccounted-for stresses or construction details (Podolny, 1985). Some early concrete box girder structures have encountered considerable distress resulting from excessively thin webs, and horizontally curved concrete boxes have experienced tendon pullout (Podolny, 1985). The location of tendon profiles paralleling and occasionally inside sloping webs introduces transverse horizontal prestress forces that have sometimes led to longitudinal cracking in top and bottom flanges (Podolny, 1986). Some construction features of segmental cable-stayed bridges, such as delta frames for stay force transfer, lead to behavior that has been insufficiently verified by field measurements.

On the conservative side, present practice requires that the stay cables be designed to fatigue specifications utilizing an assumed service load stress range that may be larger than that encountered in actual structures. Considerable effort has been expended in developing suitable techniques for the analysis of segmentally prestressed and cable-stayed bridges (Brown et al., 1974; Lacey & Breen, 1975; Marshall & Gable, 1978; Van Zyl, 1978). Field studies have been conducted on several segmentally erected, prestressed bridges (Floyd & Sutton, 1982; Shiu, 1985; Shiu et al., 1983; Russell et al., 1982). Extensive data on the response of cable-stayed and segmental cable-stayed bridges are not available at this time, although several studies are underway (Sunshine Skyway Bridge, Weirton-Stubenville Bridge); hence, continued study is needed.

1.2 The James River Bridge

The purpose of this report is to describe the instrumentation plan to be implemented on the I-295 James River Bridge near Richmond, Virginia. The James River Bridge is a segmentally erected, precast, post-tensioned box girder bridge. The main (river crossing) span is 630 ft long; it is supported by 26 cable stays arranged in a single plane harp configuration, and constructed by the cantilever method with a closure pour at midspan. Typical main span segments are 10 ft long. After the first stay cable, alternating segments are cable-stayed. The main span is continuous with three adjacent 150-ft spans on either end. The side spans are constructed by the span-by-span method with closure pours at each end of the span. The two side spans closest to the river are then cable stayed together with the main span. Side spans and the main span are externally post-tensioned. An elevation drawing of the seven continuous spans is shown in Figure 1.1. The approach spans consist of twin single-cell box girders as shown in Figure 1.2.

Post-tensioning consists of two parts: temporary bars placed to assist in construction and permanent strand post-tensioning. The permanent post-tensioning profile is determined by a series of partial length tendons, which are bent at deviation blocks and anchored at anchor blocks.

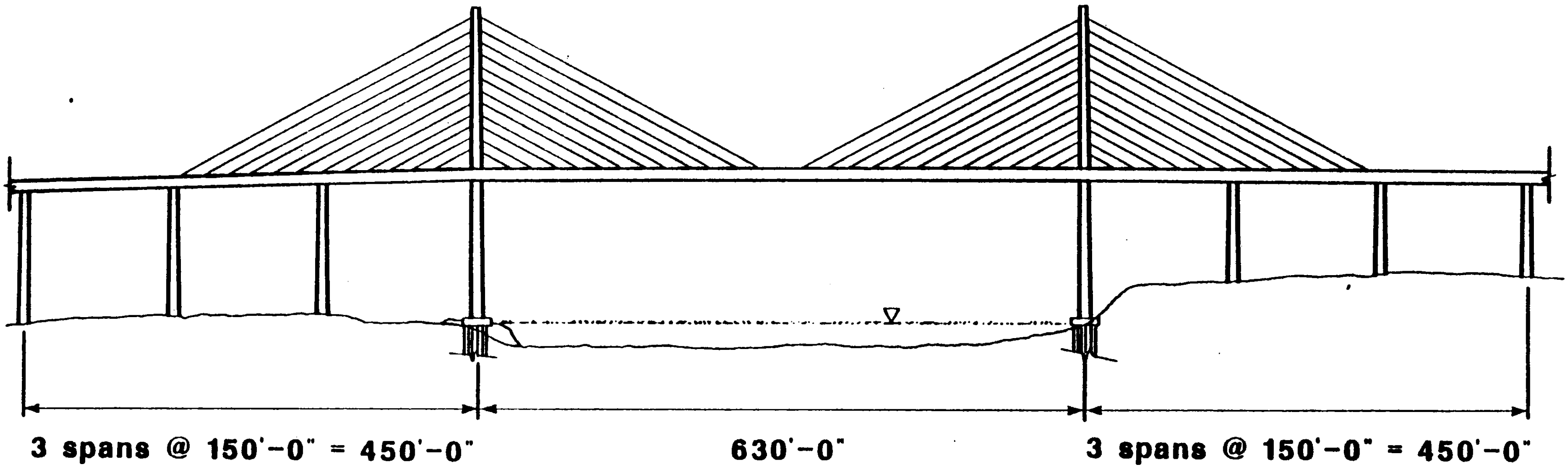


Figure 1.1 Elevation of the river span of the I-295 James River Bridge and continuous side spans.

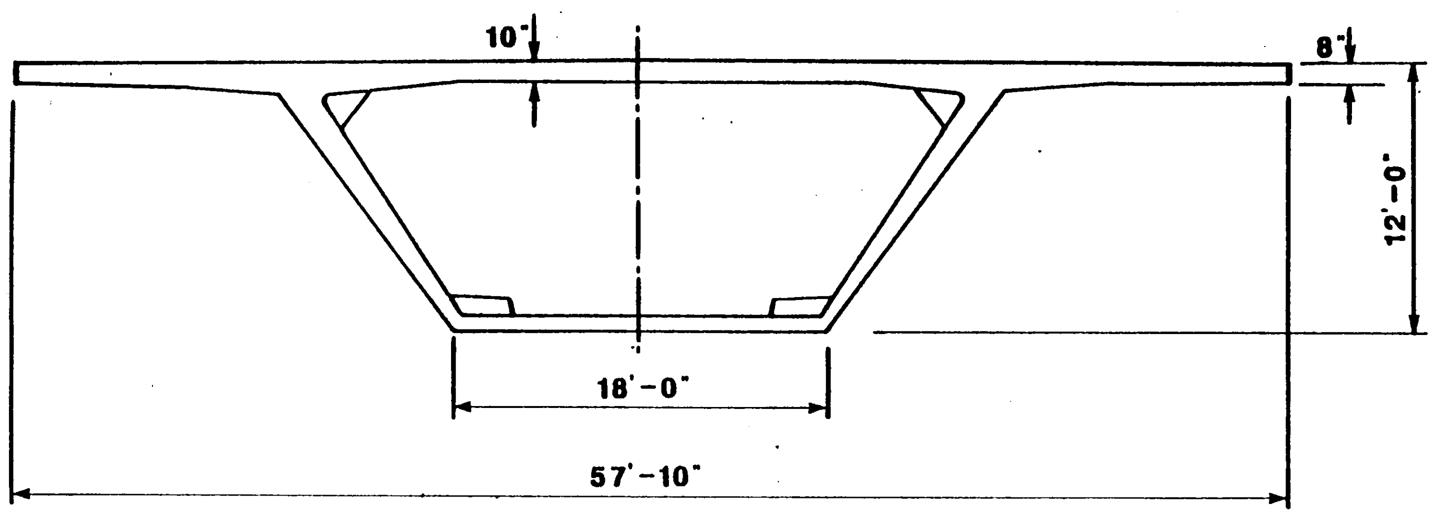


Figure 1.2 Typical approach segment dimensions.

The twin boxes are adjoined using a delta frame assembly at the locations of stay cable load transfer as shown in Figure 1.3. Figure 1.4 illustrates the section at one of the main pier/pylon locations. The pylons are cast in place below the deck level; above the deck level, they are precast and segmentally erected.

The cable stays are constructed of grade 270, seven-wire prestressing strand 0.6-in. in diameter. Individual stays consist of between 72 and 90 strands stressed to a maximum of 0.45 f_{pu}. The stay cables are continuous over the pylon saddles, enclosed in polyethelyne ducts, and anchored at the delta frames at either end. Anchorages are provided by American Stronghold Inc. Subsequent to erection, the polyethelyne ducts will be filled with concrete grout to protect the prestressing strands. Figures 1.5 and 1.6 show some details of the stay cable system.

1.3 Objectives of the Study

The objective of this study, as established in the working plan (Baber & Hilton, 1986), is to determine important stresses of the James River Bridge during construction and subsequently in service by measuring actual responses in the field. In order to fulfill this objective, it will be necessary

- to determine the live load stress range in several critical cable stays as a means of evaluating the currently recommended design practice
- to obtain data that can be used to infer the magnitude of torques introduced into the cable-stayed box girders as a result of service loads and stay cable forces eccentric to the box section centroids
- to obtain thermal gradient data for a typical box girder section, pylon section, and stay cable corresponding to the extended range of environmental conditions that occur
- to evaluate the performance of delta frame assemblies in distributing stay cable forces between the supported boxes

These specific objectives and the techniques to be utilized to obtain relevant data are discussed in greater detail in subsequent chapters of this report.

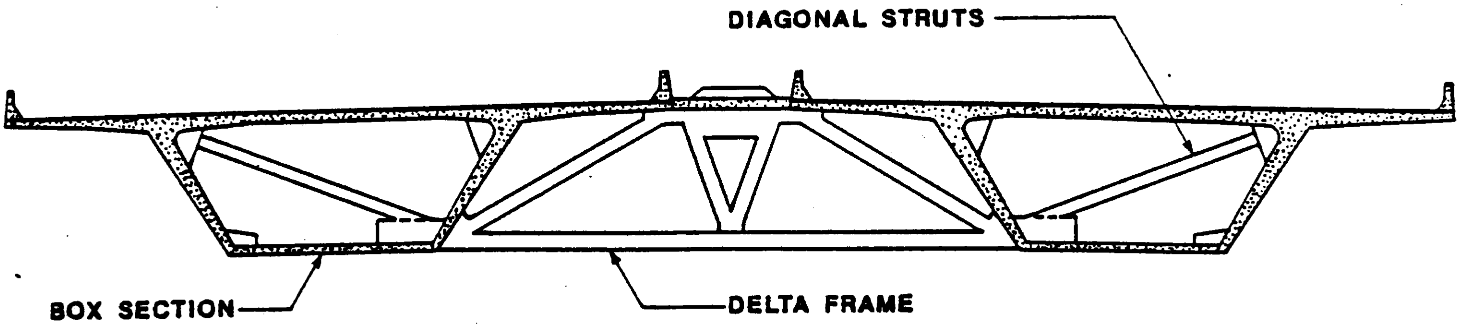


Figure 1.3 Transverse section at stay anchorage.

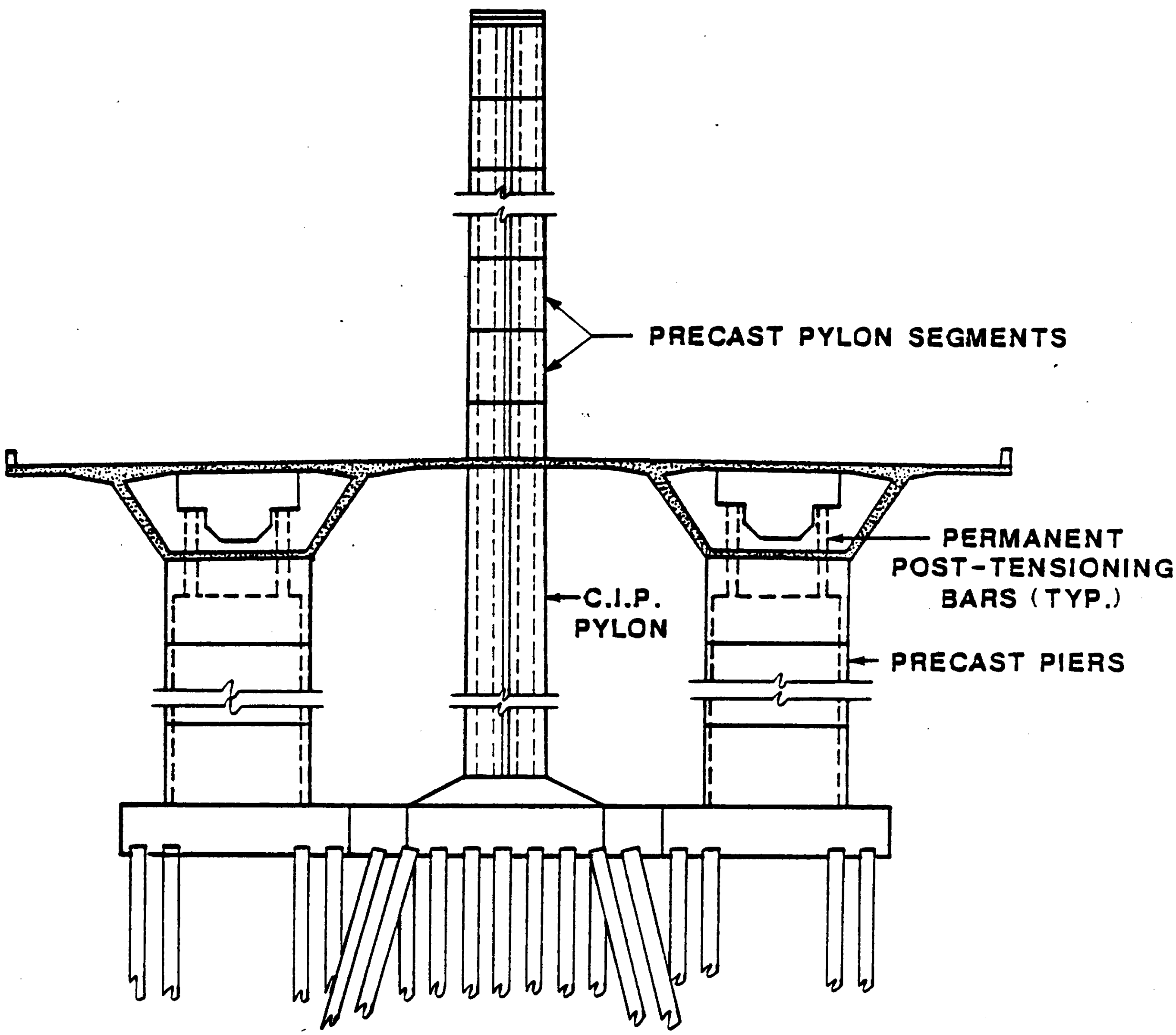


Figure 1.4 Bridge section at main pier/pylon locations.

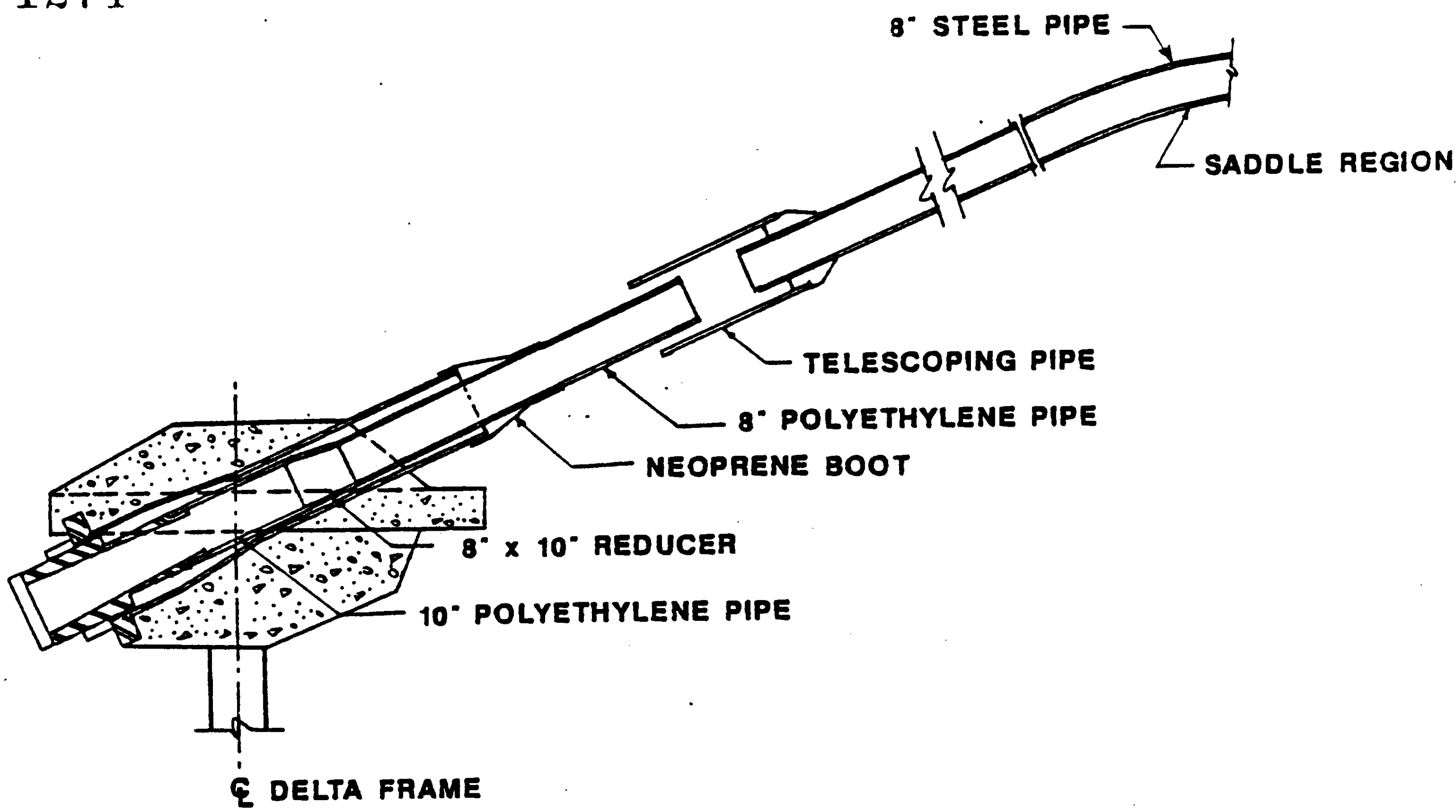


Figure 1.5 Stay cable sheating arrangement.

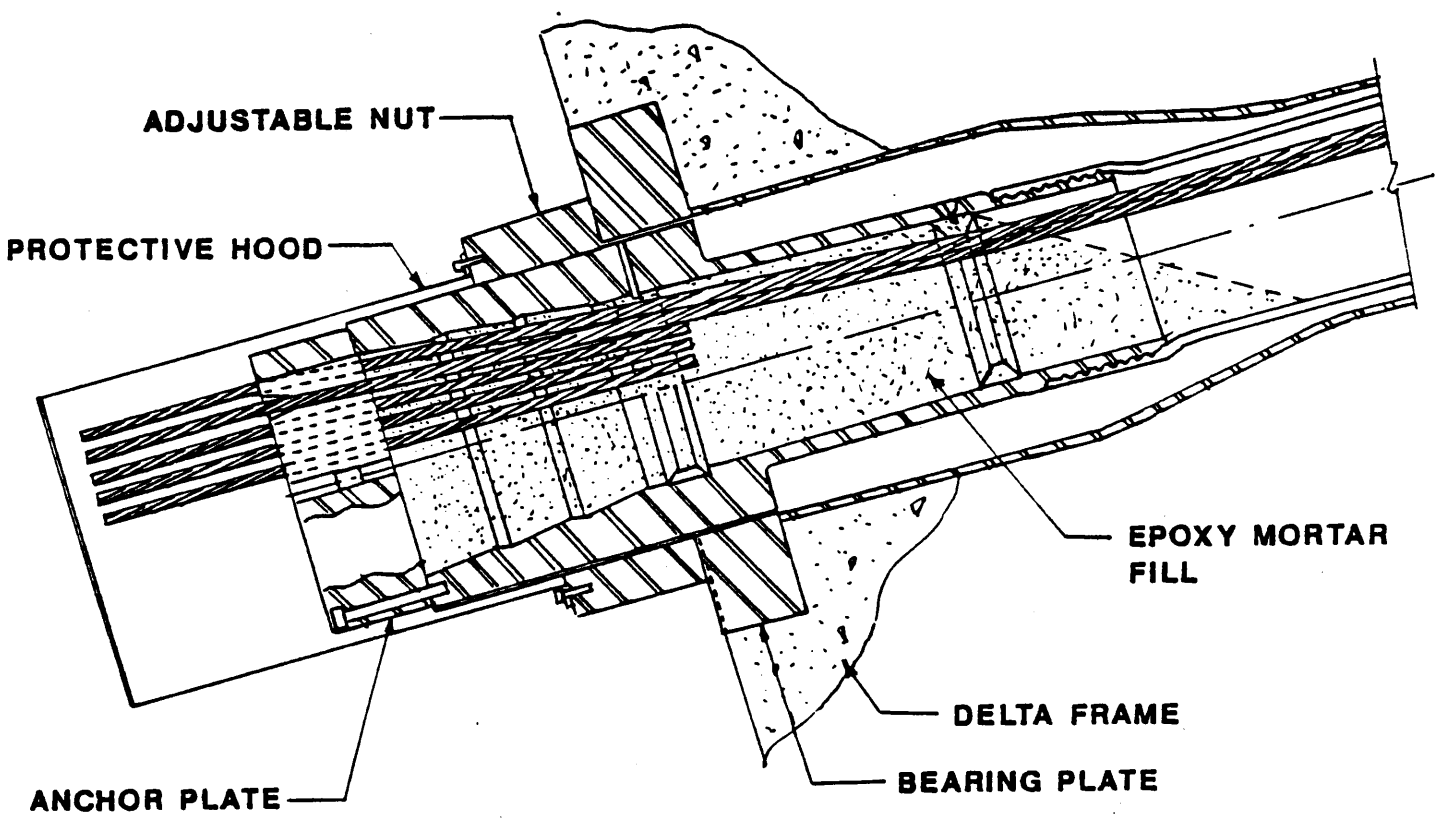


Figure 1.6 Stay cable anchorage detail.

II. STAY CABLE INSTRUMENTATION FOR LIVE LOAD STRESS RANGE MEASUREMENT

2.1 Background and Objectives

A major concern in the long-term performance of cable-stayed bridges is the satisfactory fatigue performance of the stay cables. In order to assure this, it is first necessary to obtain a sufficient set of fatigue data from controlled specimen tests to determine representative S-N curves for the materials and assemblages commonly used in the cables. Subsequently, either deterministic or probabilistic predictive models for the appropriate safety levels of stay cables under representative loadings must be developed. Finally, it is necessary to obtain representative data and develop the capability to predict stress ranges in actual cable-stayed bridges. Although considerable work has been done in the first two areas, there are at present only limited data for the response of actual bridge stay cables.

Numerous studies have been conducted to determine the fatigue resistance of stay cable components such as prestressing strand, parallel wire cable, and wire rope. Laboratory studies have been conducted by Klingenberg and Plum (1955), Lane and Ekberg (1959), Havemann (1962, 1964), Tide and VanHorn (1966), Warner and Hulsbos (1966), Baus and Brenneisen (1969), Arnold (1970), Edwards and Picard (1972), Culimore (1972), Fleming (1974), Mueller and Zellner (1975), Deubelbeiss (1979), Birkenmaier (1980), Birkenmaier and Narayanan (1982), and Castillo et al. (1985). Paulson et al. (1983) reviewed prior test results for 250 ksi and 270 ksi prestressing strand and additional results for prestressed concrete beams. They conducted a number of additional tests, developed S-N curves, and presented recommendations for the design of prestressing strand.

Current design practice requires stay cables in cable-stayed bridges to be designed for a minimum fatigue resistance determined in accordance with recommendations of the Post-Tensioning Institute Ad Hoc Committee on Cable-Stayed Bridges (1986). These recommendations provide conservative guidelines consistent with present AASHTO specifications and include provisions for acceptance testing of stay cables, anchorages, and saddles. The allowable stress and stress range are chosen based upon an assumption of two million cycles of loading during the design life and are based upon the studies discussed above. A major concern in the fatigue resistance of cable stays is performance of the complete stay cable assemblage. For instance, the anchorage and socketing system may be the weakest element in fatigue unless properly designed. Moser (1968), Andrae and Zellner (1969), Andrae and Saul (1974, 1979), Castellaw et al. (1978), Storebaeltgruppen (1979), and Grassl and Kruppe (1982), among others, have conducted studies on part or all of the anchorage/stay assembly prototypes for various cable-stayed bridge structures. Troitsky and Lazar (1971), and

Baglietto et al. (1974) have used strain gages to study the cables in small-scale bridge models.

The analytical basis for predicting useful fatigue life of structural elements, such as cable stays, given the stress range spectrum, is relatively well developed. Deterministic models utilizing standard S-N curves and Goodman diagrams are part of accepted practice for constant stress range loadings. Empirical rules, such as the Palmgren-Miner rule, while somewhat questionable from a theoretical standpoint, are widely used. Several probabilistic bases for cumulative fatigue damage analysis under a stochastically defined stress range spectrum have been developed (Bogdanoff & Kozin, 1985; Ang & Munse, 1977). Castillo et al. (1985) developed a statistical model for analyzing fatigue data for wires, strands, and cables by extrapolating results obtained on laboratory specimens to full scale cable. Similarly, the framework is available to develop a probabilistic approach for the design of anchorage and saddle regions, but does not appear to have been implemented, presumably because of a paucity of data.

A major factor that may inhibit the development of more economical design approaches for stay cables and their anchorages and assemblages is a lack of adequate data that could lead to determination of actual stress ranges to be expected in cable-stayed structures. A variety of analytical techniques have been described for determination of the static and dynamic response of cable-stayed bridges, and it is hence hypothetically possible to develop the needed data through use of analytical or computational simulation, although it is generally not considered economically feasible to do so. Moreover, very little response data is available from existing cable-stayed bridges that could be used to develop a suitable empirical data base for development of satisfactory stress-range spectra. At this time most of the available data appears to have been obtained almost entirely from European bridges. Hence, there is a real need for the acquisition of stress-range data based on North American highway loadings.

2.2 Techniques for Stay Cable Strain Measurements

2.2.1 Overview of Possible Approaches

It is apparent that several approaches to the acquisition of stress-range data for cable-stayed bridges are possible (see Figures 1.5 and 1.6). Possibilities include

- instrumentation along the stays
  - direct attachment of strain gages to the stay cable wires
  - mechanical attachment of a strain cell to the stay cable strands

- embedment of a strain cell into the grout surrounding the cables

- the use of strain-sensitive inductive devices

- instrumentation of the anchorage region

  - a "superwasher" load cell between the adjustable nut and the bearing plate

  - strain gages attached directly to the anchorage assemblage

  - strain gage instrumentation of the delta frames.

Each of these approaches has several possible advantages, together with certain disadvantages; consequently, successful instrumentation of the stay cables must take into consideration both the advantages and disadvantages of each approach, taken within the context of the intended work. Specifically, the instrumentation should satisfy the following criteria.

- Reliability: The instrumentation should be capable of functioning as intended with a high degree of reliability for an extended period of time.

- Rapid reading: A primary quantity of interest in the present study is the live load stress range. Since the live load may vary relatively rapidly, consistent with the natural frequencies of the bridge, it must be possible to take at least several readings per second on a given stay.

- Completeness of record: Although of secondary interest within the scope of the work plan, determining the long-term variation of stay cable force is of considerable importance in verifying the influence of time-dependent creep and relaxation functions in the final stay prestress levels. Consequently, the instrumentation scheme should ideally be one that is capable of obtaining as complete a record of strain history as possible. This strain record may later be used to develop predictions of the fatigue life of the cables.

- Maintenance of structural integrity: Any approach to instrumenting any part of the structure must obviously not compromise the integrity of the structure either in the short run or in the long run. In particular, instrumentation of the stay cables must not compromise the long-term fatigue resistance or corrosion protection of the stays.

- Installation considerations: Some factors that need to be considered relate to the difficulties that are encountered in trying to install a particular type of instrumentation. Because of limited funds and the time constraints of the project, the costs and delivery times of the materials need to be kept reasonably low. The ease with which the apparatus can actually be installed on the bridge and the ability to carry out the installation without significantly influencing the contractor's schedule also need to be considered.

While not all of the approaches to cable stay force determination listed above are feasible in the present study, it is felt that the investigation of these possibilities represents a significant effort that may be of use to other researchers. Hence a brief overview of each of the listed techniques is presented.

2.2.2 Instrumentation Along the Stays

Clearly, since the strains in the stay cables are the quantity of interest, instrumentation along the stays promises to be the most direct approach, provided sufficient reliability and sufficiently rapid readings can be made. A prerequisite for any of the techniques discussed in this subsection is access to the exposed stays. The most likely location for access is at the telescopic section of polyethylene sheathing pipe (see Figure 1.5). At the time of this report, it appears that this pipe will be in place but not welded to the smaller polyethylene pipe until shortly before grouting. In fact, one possible construction scheme requires the epoxy mortar fill shown in Figure 1.6 to be introduced through a pipe that is inserted at the location of the telescopic pipe. Approximately a 6-ft length of stays will probably be exposed during the construction period. Consequently, application of instrumentation along the stays appears to be possible.

Attachment of foil resistance strain gages to the wires that make up the strands of the stay cables is certainly the most direct approach to obtaining measurements of the strains in the stays. Foil resistance strain gages are capable of providing instantaneous readouts, given a suitable data acquisition system; thus they are hypothetically capable of providing a complete record of live load strain variations. Moreover, with proper attachment and correction for adhesive creep, estimation of long-term strains would appear to be feasible. Tests conducted at the Virginia Transportation Research Council indicate that direct attachment of strain gages is feasible in a laboratory setting (see the Appendix). It remains to be seen whether strain gages can be successfully attached to stay cable wires in the field. Curing of the strain gage adhesive in the field may present some difficulties because of the severe environmental conditions. Several other problems need to be overcome in order to successfully carry out such a program. The stays will be subject to twist during tensioning, which could be highly destructive to inadequately protected strain gages or the lead wires. It is not known at this time exactly how much twist will occur during the tensioning process. The grouting procedure for the anchorage regions may require that the strands be pried apart in order to insert the grouting hose. This procedure could easily damage the gages. During grouting of the cables, the gages will be subjected to hydrostatic pressures up to roughly 100 psi together with the dynamic pressure of the grout during the injection process. Consequently, adequate waterproofing and physical protection of the gages is essential. The lead wires must be able to penetrate the polyethylene sheath without compromising either the ability to withstand the grouting pressures or the integrity of the long-term corrosion protection. Some of the difficulties encountered in the grouting procedures could be minimized if the strain gauges could be installed at the upper ends of the cables near the pylon. However, this location is virtually inaccessible.

Mechanically attached strain or elongation measuring devices are capable of providing a relatively rugged means of determining the strains in the stay cables. Any of these devices would have to be attached mechanically to the stay cables to provide a direct read out. Obtaining a sufficiently rigid mechanical connection to the stays is essential to avoid slippage and to ensure that the elongations measured are consistent with the actual elongations of the stay cables and remain consistent subsequent to grouting. This is a common problem of any extensometer-like device. Several such devices immediately come to mind.

- Demountable or permanent mechanical extensometers with direct dial readouts: The primary disadvantage of such devices is that they are not capable of providing instantaneous readouts to an electronic data acquisition system and consequently will not prove suitable for strain variation measurements induced by live loads. Additional problems arise in that such gages are usually rather bulky, and the readout dials must be located outside of the polyethylene sheath. Consequently, the integrity of the long-term corrosion protection for the cable stays is likely to be seriously compromised.

- Electrical extensometer devices: Such devices could hypothetically be located between the polyethylene sheath and the stay cable strands and attached directly to the stay cables. They could be based on electrical resistance, capacitance, or inductance. Inductance devices, such as LVDTs, are readily available, but their proximity to the steel in the stay cables may lead to large systematic errors in the measurements.

Capacitance devices are not widely available. Resistance devices could be constructed but would almost certainly involve some form of foil strain gage; hence, they would be subject to all of the environmental protection requirements of strain gages attached directly to the wires plus the additional mechanical attachment problems discussed above.

A number of preliminary designs for mechanically attached strain or elongation measuring devices have been explored during the early stages of the project, and it has been concluded that any such device is more complicated to design, construct, and install than strain gages mounted directly on the stay wires and is not likely to be significantly more rugged or reliable under field conditions.

As an alternative to directly connecting a strain cell to the stay cables, it is possible, for short-term strain variations only, to embed strain cells within the grout. Clearly, this approach would not allow the early strain history or the total strains to be determined. Moreover, accurate measurement of short-term strain variations by this means depends on the validity of the assumption that the grout acts composite with the stays and undergoes essentially identical strains. Although it seems likely that this assumption is correct, it is not certain. Two such strain cells are commercially available and would probably perform satisfactorily provided there is sufficient space to fit them between the stay cables and the polyethylene sheath. The first possibility is the Carlson reinforced concrete meter (Berner & Carlson, 1986). These meters, which are about 3 ft long, are intended to simulate a roughly 1-in diameter reinforcing bar and function electrically the same as the other Carlson strain meters. Consequently, two channels of data acquisition are used to provide both strain and temperature measurements at the location of each meter. A second possibility is an encapsulated strain gage manufactured by Hottinger Baldwin Measurements. This gage was designed for embedment in concrete or ice and consists of an electrical resistance strain gage embedded in a polycarbonate matrix. The matrix is deformed at the ends to provide a mechanical bond with the surrounding material. An advantage of this gage is that it is significantly smaller than the Carlson reinforced concrete strain meter. However, because of the encapsulating polycarbonate material, it is unlikely that the long-term strain stability for which Carlson strain meters are well known will be possible. This does not appear to create a major problem in measuring live load strains, provided a periodic rezeroing is conducted.

A final technique for the measurement of tensile force in stay cables is mentioned here for completeness and because of its rather novel nature. Researchers in Belgium have developed an inductive device that uses the stay cable as the core of a transformer. It has been found that the inductive properties of the core vary directly with the strain. Under appropriate conditions, the inductive variation can be made virtually linear. A commercial version of this device called a Tensiomag is currently marketed by Freysinnet. For the present application this device would not be suitable for several reasons. Since it is an inductive device, the response time would not be sufficiently rapid to allow determination of the instantaneous strain under rapidly varying live load. Moreover, a sufficiently large device for use with the stays in the I-295 project is not presently manufactured. Being accurate within approximately 1 percent in laboratory experiments and within about 2 to 3 percent under field conditions, the inductive measurement concept incorporated in the Tensiomag cells appears to have some potential for the measurement of long-term variations of strain.

2.2.3 Instrumentation of the Anchorage Regions

Instead of attempting to attach strain measuring devices directly to the stay cables along their length, a reasonable measure of stay force could be achieved if the force transferred at the bearing plate shown in Figure 1.6 could be measured. Two possibilities appear to be feasible for such instrumentation.

All of the stay force must be transferred to the bearing plate through the adjustable nut shown in Figure 1.6. A load cell (or cells) inserted between the adjustable nut and the bearing plate and precalibrated to respond in a predictable fashion to a load of given magnitude could be used to accurately predict both short-term live load variations, and long-term variations of stay forces. Load cells of this type are currently being used for determination of long-term force variations in the stay cables on the Weirton-Stubenville bridge. These anchorage load cells were designed as part of the anchorage system, and are approximately 2 ft long to minimize uneven end bearing effects at the location of the strain gages used in the transducer design. After inquiries with the cable anchorage manufacturer, American Stronghold Corporation, it was determined that a suitable anchorage type load cell would have to be constructed of high-strength steel (95 ksi yield, 115 ksi ultimate). Unfortunately, because the load cells were not included in the original anchorage design in this project, it would be necessary to minimize the load cell length. No exact maximum length could be determined, but the probable maximum is roughly 4 in to 6 in. The necessity to use this short load cell length has the potential to significantly reduce the accuracy of the load cell due to the end effects mentioned earlier. The bearing plates and the adjustable nut are not machined smooth; consequently, prior calibration of the load cells appears to be of limited use. The most straightforward approach to calibration may be to let the bridge be the calibration load frame and to establish the calibration curve during initial tensioning. Even if this approach were followed, the load cells would need to be constructed from materials that are not readily available at most steel fabricators, and best accuracy would require that a number of strain gages located around the periphery of the load cells be utilized. Discussion of prior applications of such load cells with experienced field personnel and other researchers suggests that the resulting load cell performance would be less than satisfactory. Such cells are quite expensive to produce, ranging from $5,000 to $8,000 per cell. Such an expenditure does not seem justifiable in view of the limited accuracy likely to be achieved.

In lieu of specially designed load cells at the anchorage assembly, it is possible that a reasonable estimate of both the short-term and long-term stay cable force variations might be obtained by using the anchorage assembly itself as a load cell. Measurements would be obtained by attaching strain gages to the anchorage assembly prior to casting the delta frame. If the strain gages were then zeroed before tensioning and read periodically at known force levels during the tensioning process, a calibrated curve of stay forces versus strain gage readings could be obtained, which could then be used as a basis for determining the subsequent variation of the stay force in time. Clearly, there is some question whether the readings would produce useful data. Perhaps the greatest drawback to this approach is that the data would be suspect, regardless of how carefully it was taken.

A final technique, which suffers the same drawback as the anchorage assembly instrumentation just discussed, is the instrumentation of the delta frame. In addition to the calibration difficulties discussed in the foregoing, instrumentation installed in the delta frame would be subject to creep of the delta frame concrete. Consequently, it is unlikely that useful stay force data could be obtained in this manner.

2.2.4 Zeroing the Instrumentation

In order to obtain an accurate zero value for any type of instrumentation chosen, the devices must be calibrated at a time when there is a known force in the cables. This occurs only a few times during the cable tensioning process. During the segment installation phase of construction, the strands are tightened to about 10 percent of their strength using monostrand jacks. Each strand is jacked twice to make sure that all of the slack is out of the system and that all the strands are tensioned equally. Following this procedure, all the strands in each cable are stressed simultaneously to approximately 40 percent of strength using a multistrand jack. After the bridge is essentially complete, the multistrand jacks are used in a final retensioning operation. Soon after the final retensioning, the cables are grouted. It seems that the best time to install and calibrate the instrumentation would be immediately following the multistrand tensioning process to 40 percent of strength. At this time there will be known forces in the cables that can be used in the calibration and zeroing of the instrumentation. The final retensioning could then be used as a check on the instrumentation or possibly a point at which to rezero the measuring devices. The instrumentation needs to be fully calibrated and functional by the time the cables are grouted. In order to install the measuring devices at the correct time without interfering with the construction progress of the bridge, an exact installation schedule will have to be worked out with the contractor and updated continuously as the job progresses.

2.3 Instrumentation Plan

2.3.1 Foil Resistance Strain Gages

Because of their low cost and the fact that they are readily available, it was decided to mount foil resistance strain gages directly to wires in the strands. The gages to be used will by Micro-Measurements 062AP gages. This type of gage has been successfully attached to 0.6-in diameter seven-wire strands in laboratory tests conducted at the Virginia Transportation Research Council (see Appendix).

2.3.2 Location of Gages

The stay cables consist of 72 to 90 strands depending on their location on the bridge. Because of difficulties in accessibility, only strands in the outer layer can be readily instrumented. It was decided to instrument four strands in each cable under consideration. The instrumented strands will be at the top, bottom, left, and right sides of the cable (see Figures 1.5, 2.1, and 2.2). This arrangement of strain gages will enable not only the measurement of the overall stresses in the cable, but will also show the distribution of stress within the cable. Local bending of the cable, for instance, should show up as a change in the readings taken from the top and bottom strand gages.

Because it is not clear whether all of the wires in a strand carry exactly the same amount of load, two diametrically opposite wires in each applicable strand will be instrumented. Although the two gages will be on diametrically opposite wires, they will both be mounted on the side of the strand facing outward (see Figure 2.3). This configuration will be suitable for installation and will also help to avoid any damage that may be caused by the strands rubbing, hitting, or twisting against each other prior to grouting.

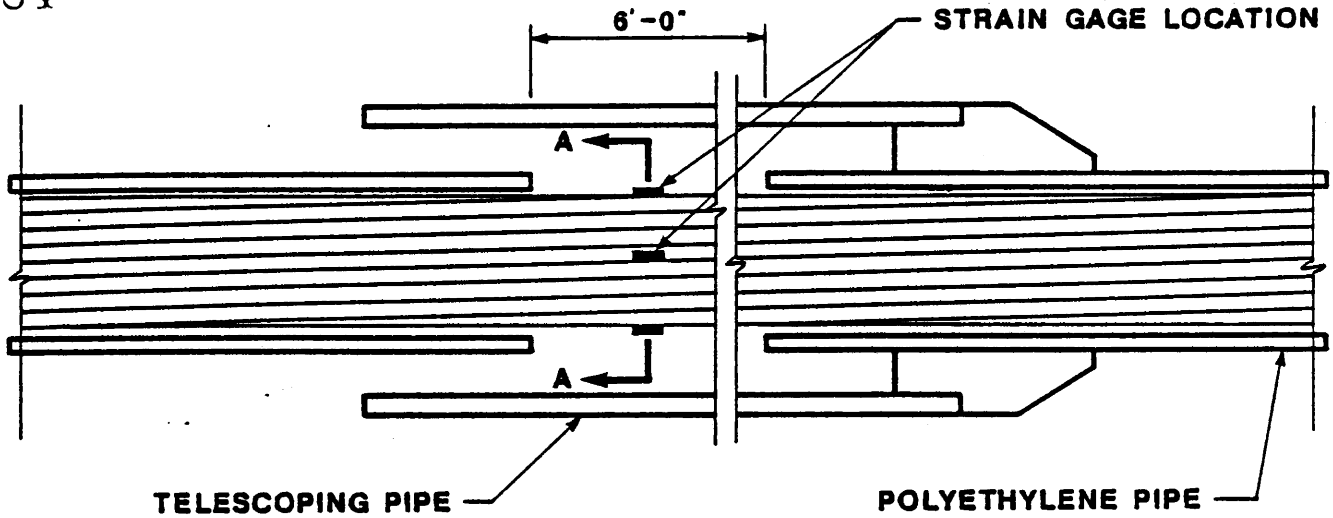


Figure 2.1 Access region in polyethylene pipe.

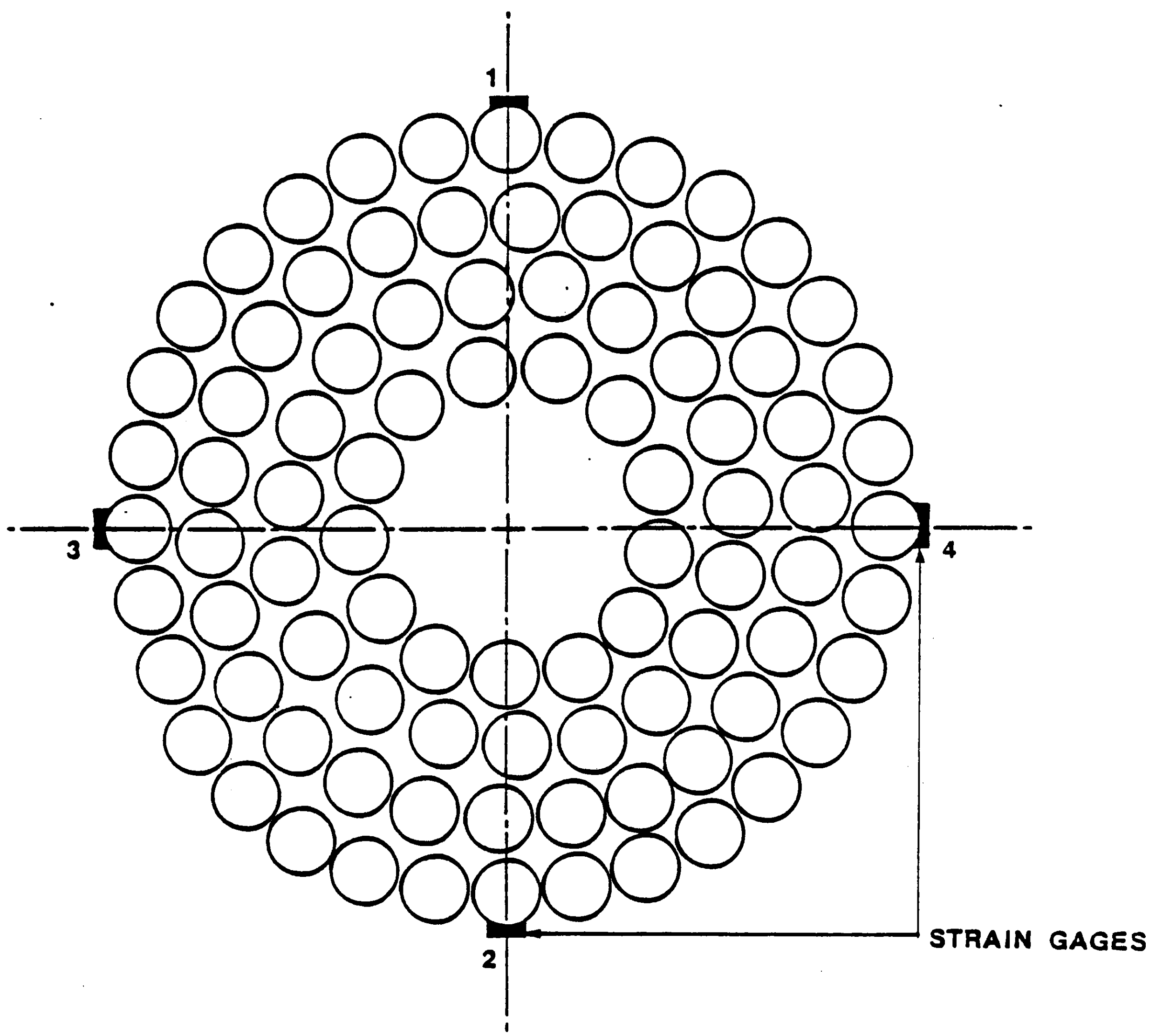


Figure 2.2 Cable cross section showing strain gage locations.

2.3.3 Installation of Gages

The installation of the gages in the field will be executed as nearly as possible like the installation under laboratory conditions. The section of the strand on which the gages are to be mounted will be cleaned with a wire brush and a degreaser such as alcohol. The wire surfaces will then be chemically treated with conditioner and then with neutralizer. The actual mounting of the strain gages will be performed in accordance with Micro-Measurements instruction bulletin B-137-11 using AE-10 epoxy adhesive. The gages will be held in place during the curing process using a pipe clamp with a Tygon tubing underlay. After the epoxy adhesive has been properly cured, the clamping mechanism will be removed and the gage cleaned.

2.3.4 Installation of Lead and Thermocouple Wires

Immediately after the epoxy has cured, the wire leads will be soldered to the gage terminals. Several loops will be introduced into the wire leads and the wire will be taped to the strand (see Figure 2.4). The loops will help to keep the leads from being torn off the strain gage if the wires are accidentally pulled. Because it is likely that some gages will be lost during grouting, all gages will be wired independently. Thermocouple wires will be attached to each strand at the strain gage location to provide temperature readings. The thermocouple wire to be used will be type T, 24 gage, teflon-insulated wire. The wires from all the gages in a cable along with the thermocouple wires will be collected and jointly run out through an opening in the polyethylene telescopic pipe. The wires will then be run to the data acquisition system located in the box girder.

2.3.5 Waterproofing and Mechanical Protection

After the lead wires are connected, the gage and terminals will be coated with a layer of AE-10 epoxy, which will be allowed to cure. For field application, waterproofing and mechanical protection will be provided by a nitrile rubber coating, a butyl rubber sealant, a sheet of neoprene rubber and aluminum foil tape. While the polysulfide coating used on the instrumented reinforcing bars (see Chapter 3) would appear to provide somewhat more durable waterproofing, application of this compound under field conditions does not appear to be feasible. The entire protective housing will then be wrapped with duct tape to keep the assembly together properly.

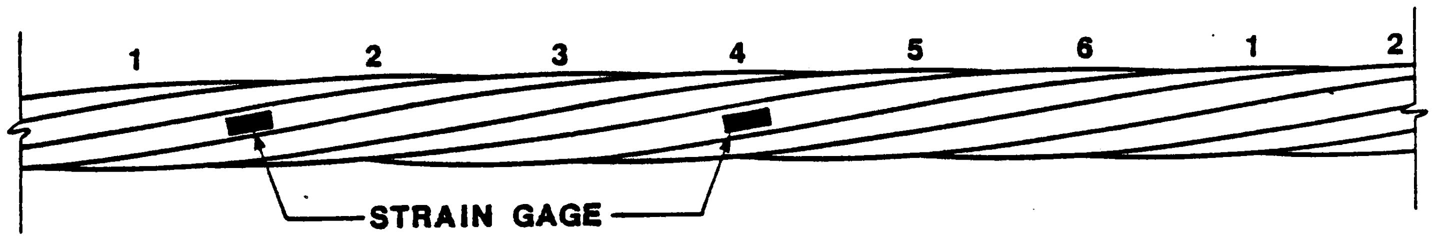


Figure 2.3 Gages on diametrically opposite wires.

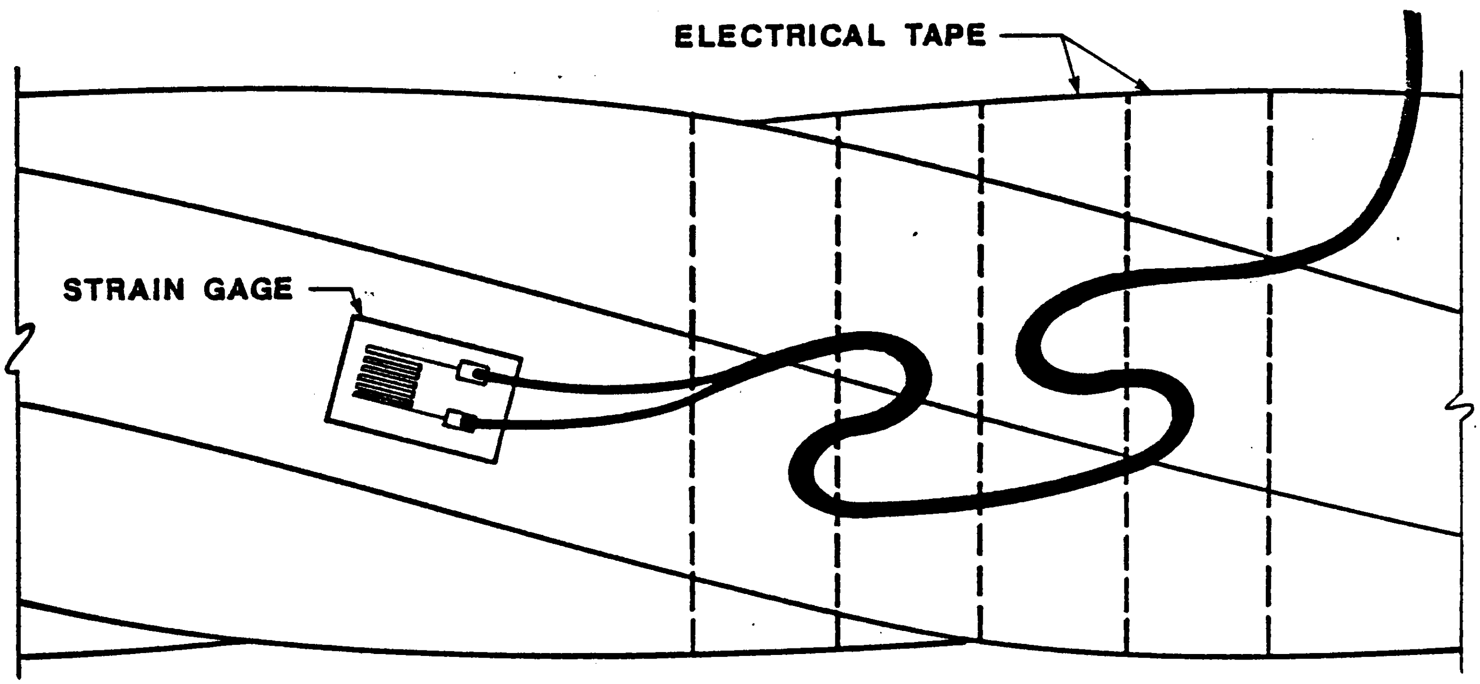


Figure 2.4 Loops in the lead wire.

2.3.6 Installation Scheduling

The precise scheduling of the installation has not yet been determined. It will have to conform to the contractor's construction schedule. The zeroing of the gages will also have to be taken into account (see Section 2.2.4). A meeting will have to be set up with the contractor to establish a schedule, which will then have to be updated periodically as the work progresses.

2.3.7 Stays to be Instrumented

Stays S-2 (near the pylon), S-13 (nearest to midspan), and S-7 (midway between S-1 and S-13) were selected for instrumentation. Instrumentation of these stays will provide data that is representative of the full range of expected behavior. Only the ends of the stay cables on the main river span will be instrumented.

III. BOX SECTION INSTRUMENTATION FOR TORSIONAL AND LONGITUDINAL STRAIN MEASUREMENTS AND DELTA FRAME TRANSFER MECHANISM

3.1 Objectives of the Measurements

The instrumentation of the concrete box segments is designed with several objectives in mind.

3.1.1 Longitudinal and Torsional Strains in the Main Span Girders

The primary responses of the main span girders of the bridge are expected to include bending within the vertical plane of the bridge and torsional twist of the sections. The twist of the sections may result from two causes: (1) a relative twist introduced by the stay cable force in the area of the discrete delta frames, and (2) a unit twist of the twin boxes as a consequence of external loads.

Relative twists of the individual sections may occur as a consequence of the eccentricity of the stay cables relative to the centerlines of the individual boxes. The delta frame assemblies are intended to distribute the stay forces to the two boxes and thus eliminate this component of twist. The intended action of the unit is clear from Figure 1.3. The delta frames, the bracing struts inside the boxes, and the top flanges of the boxes form concrete trusses that transfer the box loads laterally to the stay points. Assuming infinite rigidity of the trusses, the delta frame assembly would effectively prevent all relative rotation of the boxes at the stay location. The delta frames are not infinitely stiff; consequently, some relative twist of the boxes may occur. If present, relative twist will appear in the form of opposite sign shearing stresses in the boxes, which will accumulate from midspan toward the piers and will be seen in the deck, the bottom flanges, and in the web. Additional strains caused by the beam weight will also occur in the webs, but it should be possible to determine the relative contributions of the torsion and shearing effects. This component of twist should become apparent during the construction period and can be measured separately from any twist induced by service loads.

In addition to the twisting of the twin box sections relative to each other, which may occur as a consequence of flexibility in the delta frames, twisting may occur because of live loads. Unbalanced service loadings on the twin boxes of the single plane cable-stayed structure will cause twisting of the boxes about the stay points. While the effective functioning of the delta frame assemblies will minimize the independent twist of the twin boxes, the delta frame will not inhibit twisting of the boxes as a unit. While the sources of potential twist are well known, the magnitude of the twisting moments to be resisted and the corresponding stresses in the webs of the box section are not easily determined. Consequently, response data will be obtained that will be useful in evaluating the effects of torsion. The instrumentation needed to determine the twist of the boxes relative to each other will also be capable of determining the twist of the twin boxes as a unit.

3.1.2 Distribution of cable stay forces by delta frames

The transfer mechanism for the stay cable forces at the delta frames has not been verified experimentally. Theoretical analyses have been utilized to permit the design of several structures with delta frame assemblies, and the configuration will in all probability be utilized in future bridges as well. The James River Bridge is the first bridge constructed using delta frames. In addition to any torsional strains caused by relative twist of the sections, localized stresses will also occur where the discrete delta frame elements adjoin the box sections and where the cross-bracing elements inside the boxes adjoin the walls of the boxes. The study will monitor the strains that develop in the section in the vicinity of the delta frame assemblies. Strains will also be monitored in the diagonols of two of the delta frames.

3.2 Strain Measurements in Reinforced and Prestressed Concrete Sections

A variety of methods have been used for the measurement of strains in reinforced or prestressed concrete sections. These include mechanical demountable strain gages and electrical resistance strain gages of various types.

3.2.1 Demountable Mechanical Extensometer Type Strain Measurements

Several demountable mechanical extensometer measuring devices are available for the measurement of strains. Perhaps the most widely known such device in the United States is the Whittemore strain gage (Figure 3.1). This gage, which was formerly manufactured and marketed by the Baldwin-Lima-Hamilton corporation (now BLH), is capable of measuring relative displacements of approximately .0001 in (dial gage resolution) over a 10-in gage length. Hence, strains on the order of 10 $\mu\Sigma$ are hypothetically measurable. Some additional limitation on the resolution of the gages is likely as a consequence of extensometer mounting tolerances. The BLH Whittemore gage is no longer manufactured, but several similar gages are still marketed. One made in the United States by Soiltest has an advertised measurement precision of 0.0005 in on 6-, 8-, and 10-in gage lengths, for a resolution of approximately 50 $\mu\Sigma$. A third gage, called the Demec gage, is marketed by W. H. Mayes and Son in Windsor, United Kingdom.

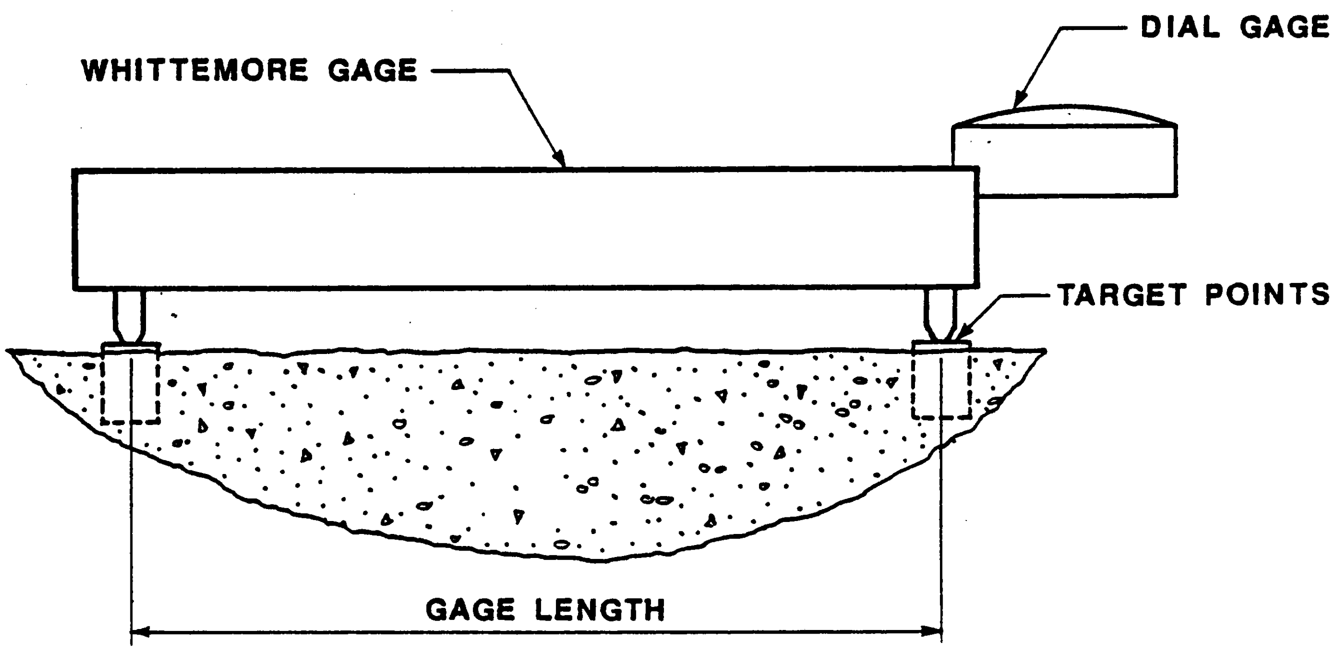


Figure 3.1 Whittemore strain gage.

The mechanical extensometer gages are used to measure surface relative displacements in reinforced concrete and prestressed concrete elements using discrete target points. The target points are mounted in the concrete either by location in the forms prior to concrete placement, by gluing to the surface with epoxy adhesive, or by insertion into drilled holes subsequent to initial concrete curing. If the target points are inserted into holes in the cured concrete, rapid curing shrinkage-compensated hydraulic cements such as Duracal allow an initial measurement to be obtained within a couple of hours of the time the concrete is stripped from the forms. Consequently, in precast sections all of the creep strain and much of the shrinkage strain can be obtained. Such a mortar will have thermal expansion properties much closer to those of the concrete than will an epoxy mortar. This will prove quite beneficial since measurements are to be taken at widely varying temperatures. Targets premounted on lucite plates and glued against the forms prior to concrete placement have been utilized successfully in casting of slab specimens for laboratory studies. For field instrumentation it appears to be preferable to mount the targets subsequent to section casting since it appears unlikely, based upon observations of the casting procedure, the targets mounted on the inside faces of the forms would survive the concrete placement with any regularity.

The mechanical gages have the advantage of providing convenient, inexpensive measurements of the long-term strain record at a given location on the surface of the concrete section, which because of their relatively long gage length, are able to minimize the local effects caused by microcracks in the concrete. It is, of course, necessary to separate the different components of strain (creep, shrinkage, elastic strains) in order to properly interpret the strain data. Unfortunately, because the mechanical gages must be manually inserted into the targets and read, they are not suitable for short-term (i.e., live load) strain measurements.

3.2.2 Discrete Electrical Resistance Strain Meters and Gages

One of the most widely used devices for the long-term measurements of strains in concrete is the Carlson strain meter (Carlson, 1935, 1979). The Carlson strain meter uses a pair of pretensioned high tensile strength wires. Under mechanical strain, the tension in one wire increases while the tension in the other decreases. Both wires have identical changes in tension under thermal strain. Consequently, Carlson meters can provide both temperature-compensated strain measurements and temperature measurements, averaged over a gage length that varies from 4 in. in the miniature strain meters to 10 in. in the largest regular strain meters. A reinforced concrete strain meter, which is designed to simulate the performance of a reinforcing bar 34.5 in long, is also available (Berner & Carlson, 1986).

Carlson strain meters have several important advantages. They have been extensively used in the instrumentation of reinforced and prestressed concrete structures for many years. They are known to display very good long-term stability under field conditions, and they are capable of providing both strain and temperature data. The disadvantage of the Carlson strain meters is that they were not designed to be read from data acquisition systems. Hence, they have nonstandard resistances, and considerable care must be taken in exciting them. More importantly, typical data acquisition systems are not designed with Carlson strain meters in mind, and many of the representatives of companies manufacturing data acquisition and control systems do not even know what a Carlson strain meter is. The useful quantities for strain and temperature measurements are the sum of the wire resistances $R_1 + R_2$ and the ratio of the resistances R_1/R_2. They have traditionally been read using a specially designed portable balance box. It is possible to read a Carlson strain meter using the resistance measuring capabilities of most industrial data acquisition and control systems. Since both R_1 and R_2 must be determined, acquiring a single piece of strain data from a data acquisition system requires two channels of a data acquisition system. If the additional temperature data is not needed at the point where the gage is located, the cost per channel of data acquired may be somewhat higher than comparable data acquired by other means.

Preconfigured discrete strain meters using bonded resistance foil strain gages provide an alternative to the Carlson strain meter. Hottinger Baldwin Measurements' electrical strain gage encapsulated in a polycarbonate sheet with deformations along the end and designed for embedment in asphalt or concrete was previously mentioned in connection with the stay cable instrumentation. These gages are single 350-ohm gages intended for wiring in a quarter bridge configuration. Temperature compensation is necessary, either by use of a dummy strain gage wired in a half bridge or by a thermocouple adjacent to the embedded gage. Similar gages may also be available from other manufacturers.

3.2.3 Strain Gaged Reinforcement

Several researchers have obtained strain data in reinforced or prestressed concrete structural elements by attaching strain gages directly to reinforcing bars that are part of the element (Wanders et al., 1979, Cleland & Baber, 1985). Other researchers have utilized dummy reinforcing bars instrumented with strain gages and inserted into the structure expressly for strain measurement. These gages can be configured in several ways.

For very simple laboratory specimens where the temperature is not a primary variable a single strain gage mounted on a surface of the reinforcing bar with the direction of strain measurement parallel to the bar's axis has been used. Alternatively, if it is suspected that some flexure as well as tension (or compression) may occur in the bar, it is more suitable to mount similar gages on opposite sides of the bar. These gages may then be located in opposite legs of the bridge to provide readings that cancel out the bending effects.

In the field, the effects of temperature cannot be neglected. Some form of temperature compensation must be utilized. If the strain gages are mounted on a flat surface, complete temperature compensation is achieved by mounting a 90° rosette strain gage on the instrumented reinforcing bar. The portion of the gage transverse to the bar then undergoes a Poisson strain together with a compensating thermal strain. If the strain gages are mounted on a curve, the temperature compensation achieved in this manner is not complete. An additional apparent strain may accumulate as a consequence of the curvature of the transversely mounted portion of the gage. The apparent strain is a function of the diameter of the curved surface, the gage backing, the gage material, and the adhesive used (Measurements Group Inc., 1983). A correction can be made for this effect provided the temperature is known within a few degrees Celsius, but mounting the gages on a flat surface alleviates the need for such corrections.

3.3 Strain-Gaged Reinforcement Design

Following previous work outlined in section 3.2.3, it has been decided to utilize strain-gaged reinforcing bars in the gross concrete section strain measurements. A schematic cross section of the instrumented reinforcing bar is shown in Figure 3.2. The instrumented reinforcing bars will be 4-ft-long no. 5 bars. Some of the bars will be located in the top flange of the precast box sections, where it is required that all reinforcing steel be epoxy coated. The instrumented bar steel is not part of the regular reinforcing, but the additional cost of epoxy coating is nominal.

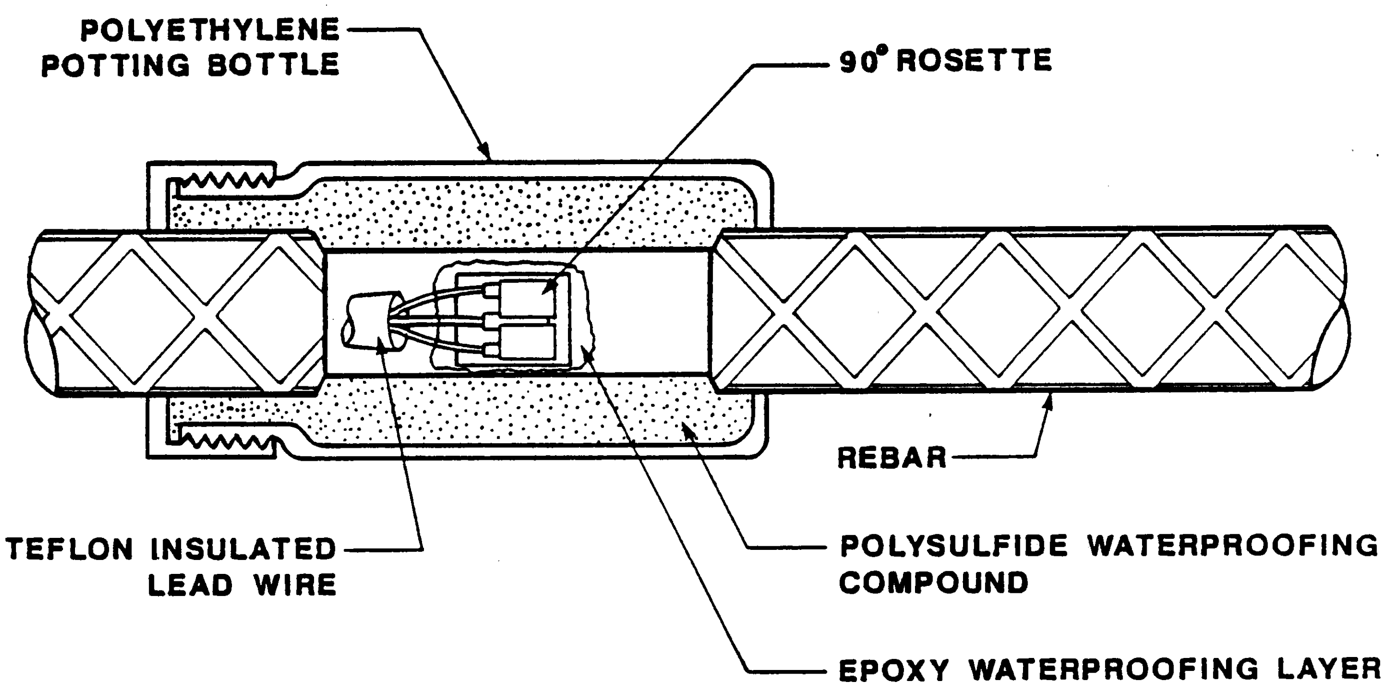


Figure 3.2 Instrumented dummy reinforcing bar for field use.

The strain gages used in the study must have excellent long-term stability against creep. For this reason, nickel-chromium alloy foil gages fully encapsulated in glass-fiber-reinforced epoxy-phenolic resin will be used. Such gages are known to display excellent long-term creep properties. The gages will have integral printed circuit terminals with polyimide encapsulation for ease of installation. This option is not ordinarily recommended for this type of gage, since they are frequently used in fatigue applications; but in the present application, the life of the gage is not likely to be compromised by the integral printed circuit terminals, and the ease of installation is considerably increased by this feature.

In the interest of obtaining optimal temperature compensation in the gages, the gage configuration used will be a 90° rosette. The transverse gage response will be proportional to Poisson's ratio times the longitudinal gage response, as shown in Figure 3.2. When wired in a half bridge, the excitation voltage E can be shown to be related to the bridge voltage by the equation

$$\frac{E_o}{E} = \frac{F(1+v)\ \Sigma \times 10^{-3}}{4 + 2(1-v)\ \Sigma \times 10^{6} + 4\ \Sigma_T \times 10^{-6}} \qquad [3.1]$$

where F is the gage factor, Σ is the strain in microstrains, v is Poisson's ratio, and Σ_T is the apparent strain caused by thermal effects. It is seen from equation [3.1] that the only temperature effect in the gage appears as a small second-order correction, which is not considered to be significant given the temperature range likely to be encountered in the studies to be conducted.

The decision whether to mount the gages on the flat or on the round was not easily resolved. Grinding a mounting surface to a flat plane large enough to accept the 90° rosettes tended to reduce the cross section at the gage significantly, which introduces considerable uncertainty in the interpretation of the measurements. From this standpoint, it appears preferable to mount the gages on a section that has a net section at the gage effectively the same as the gross cross section and has just had the deformations removed.

Mounting the gages on a round surface introduces two potential problems. First, adequate bonding of the gages on a curved surface must be assured. The fiberglass-reinforced epoxy-phenolic resin encapsulation, while providing excellent long-term measurement stability, tends to result in a somewhat stiffer gage, which does not promise to be as easily mounted on a curved reinforcing bar surface as a gage with a more flexible backing. To determine whether mounting gages on such a curved surface is possible, representative 90°-rosette gages of the size and backing to be used in the field were mounted on machined no. 4 and no. 5 reinforcing bars using an epoxy resin adhesive. Load tests subsequently conducted on the no. 4 bar indicated satisfactory bonding. The gage mounted on the no. 5 bar did not perform satisfactorily. Subsequent examination of the gage mounted on the no. 5 bar indicated that inadequate epoxy adhesive or excessive clamping pressure was used on it. Since it was possible to successfully mount and load a gage on the smaller no. 4 bar, it must be concluded that strain gages can be mounted successfully on the reinforcing bars; but some care must be taken to ensure that adequate adhesive is used and that the clamping pressure does not squeeze too much of the adhesive out of the bonding surface.

A second potential problem, already mentioned above, is the possibility of secondary thermal strain caused by multiple thermal expansion coefficient mismatches on the curving radius of the transversely oriented section of the gage. The use of a gage encapsulated in the fiberglass-reinforced epoxy-phenolic resin and of a larger diameter reinforcing bar reduce the magnitude of the problem considerably. For a given temperature change, correction for gages mounted on the round is quite possible. Since the thermally induced additional strain is not large and the required correction can be calculated from simple available formulas, reasonable correction can be made given the thermal data to be acquired at a representative section.

In view of all of these considerations, the decision was made to mount the gages on the curved surface of the reinforcing bars and to preload the gages in the laboratory before installation in the field to ensure that the gages are well bonded to the dummy reinforcing bars.

For field application, adequate waterproofing of the instrumented reinforcing bars is essential. To achieve this goal a three-layer waterproofing will be utilized. Subsequent to attachment of the lead wires, a thin layer of epoxy will be used to coat the gage and the exposed ends of the lead wires. Following the curing of the epoxy, a polyethylene potting bottle will be placed over the gage. The potting bottle will be filled with a polysulfide compound specifically designed for waterproofing sensitive electronics. This waterproofing scheme is illustrated in Figure 3.2. Subsequent to potting, a final mechanical protective layer of electrical or duct tape may be applied to reduce the probability of the lead wires being ripped loose by incidental forces occurring during the concrete placement.

For optimum long-term waterproofing of the installation, the lead wires selected for this application will be insulated with TFE Teflon, shielded to reduce the probability of transient current damage to the installation, and jacketed with TFE Teflon. The choice of TFE Teflon insulation is based on its waterproofing performance, which is superior to that for the less expensive insulation materials such as PVC. For optimal waterproofing at the gages, the ends of the Teflon lead wire insulation will be etched prior to application of the potting compound. In this manner an effective bond between the Teflon insulation and the polysulfide waterproofing compound is ensured.

3.4 Gage Locations and Orientations - Box Girder Sections

3.4.1 Gross Section Flexure and Torsion

The response of a single box section under load may be quite complex, depending on the nature of the loading, the presence or absence of diaphragms, and the relative thickness of the box flange and web walls. Maisel and Roll (1974) discussed the various modes of box section deformation and summarized a variety of simplified analytical techniques that account for different aspects of box section response.

While the geometry of the present project's twin box girders is somewhat more complex than that of a single box girder, and the influence of the concentrated cable stay forces applied at the delta frames are likely to have a significant influence on the section response, it is nevertheless expected that certain forms of section deformation will dominate the behavior. In planning the location of instrumented reinforcing bars, it was necessary to consider the kinds of section response that are to be expected and which of those responses are of interest within the scope of the proposed work.

Figure 3.3 illustrates the gross section flexural and torsional behavior of the twin boxes. Assuming that single-axis bending occurs, and that the cross section does not deform transversely, the cross-sectional response in pure flexure will produce displacements as illustrated in Figure 3.3 (a); for example, all points at a given cross section will undergo identical transverse displacements. In reality Poisson's effect will cause some cross-sectional distortion even if there is no other localized cross-sectional deformation; but for symmetrically applied loads, the responses illustrated in Figure 3.3 (a) may be expected to be the primary behavior of the gross section. It is also assumed in Figure 3.3 (b) that the boxes and connecting delta frames form a sufficiently rigid cross section that negligible distortion of the section occurs. The response mode shown in Figure 3.3 (b) is the gross section torsional response to an unbalanced load, such as an unbalanced live load. For such an unbalanced load, some transverse flexural response, such as is shown in Figure 3.3 (a), would also be expected.

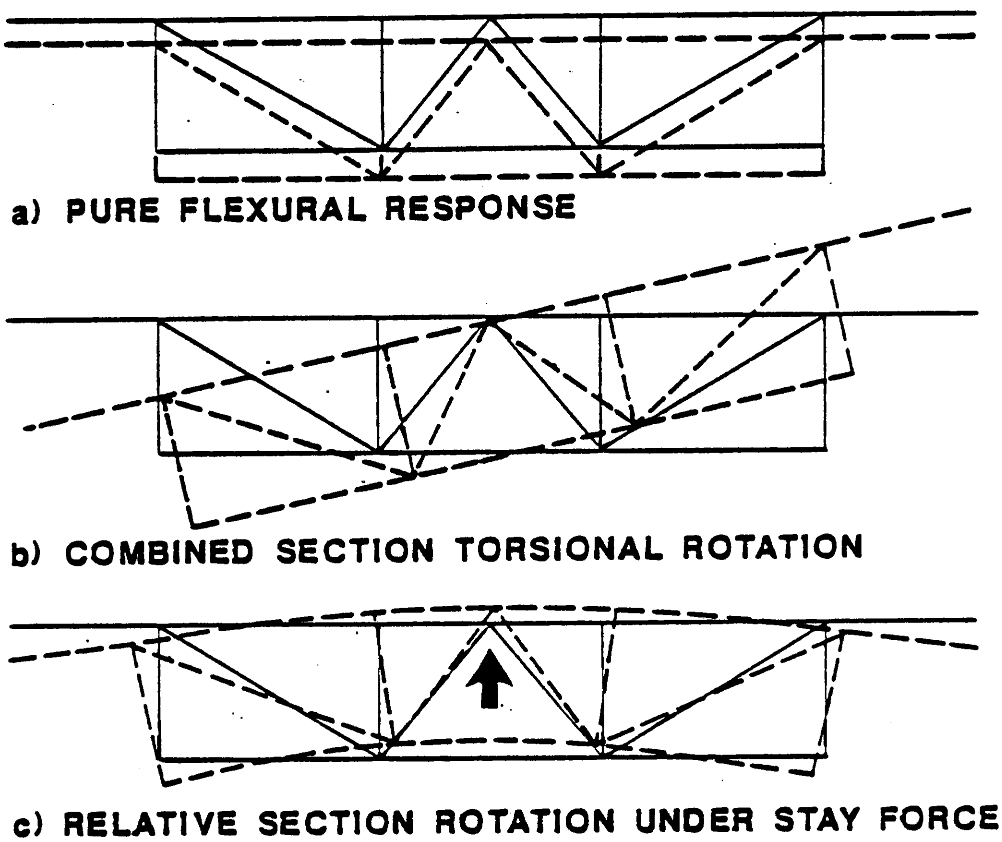


Figure 3.3 Gross cross-sectional deformation modes.

Figure 3.3 (c) illustrates the relative distortional displacement between the twin boxes that may be expected to occur in an otherwise very stiff cross section as a result of the single-plane stay cable forces at the midpoint between the twin boxes. In the absence of other unbalanced loadings, a symmetrical deformation pattern with opposite sign twist in the two boxes would be expected from the relative rotation under the stay cable force.

In addition to the gross cross-sectional deformation modes, a number of local cross-sectional distortion deformation modes may be expected to occur. Three such modes of local distortion are illustrated in Figure 3.4. Figure 3.4 (a) shows the flexural distortion of a single box section under symmetrically applied loading. Figure 3.4 (b) illustrates a shearing cross-sectional distortion. Figure 3.4 (c) illustrates a simplified form of typical thermal cross-sectional distortion if the top flange undergoes larger temperature change than the bottom flange. Each of these forms of sectional distortion are important, and transverse reinforcement must be present to maintain the integrity of the section under these forms of deformation.

One form of distortion, which Maisel and Roll (1974) have indicated is particularly important in the torsion of box girder cross sections, is the torsional distortion of the section. Such distortion has the potential to cause larger stresses than warping distortion, which tends to be somewhat limited in box sections.

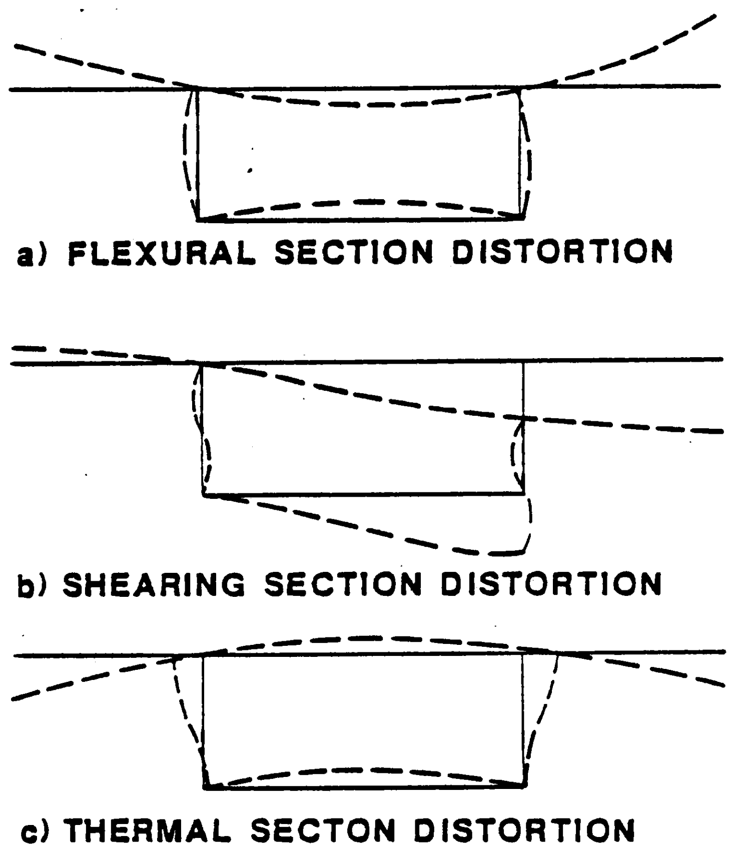


Figure 3.4 Local cross-sectional distortion patterns.

Clearly, in view of the complexity of the behavior to be anticipated in even a single box section, complete instrumentation for gross sectional deformations, and local sectional distortions would require a very large number of gages. For the present project, it was felt that the gross sectional deformation will provide the essential data needed to answer the questions of interest. With these objectives in mind, three sections on the main span will be instrumented. The locations of the sections to be instrumented are shown in Figure 3.5. The locations of the gages in the instrumented sections are illustrated in Figures 3.6 and 3.7. In Figures 3.6 and 3.7, a dot indicates the presence of a single longitudinal gage, and an "R" indicates three gages oriented in a rosette. The gage pattern shown in Figure 3.6 will allow determination of the gross cross-sectional flexure and shear acting in all four webs. The additional rosettes located only in the instrumented section nearest the pylon and shown in Figure 3.7 will allow a more complete idea of the torsional strain distribution to be obtained. Under the symmetrical loading of bridge dead load and stay cable uplift force, any variations in shear between the webs may be attributed to relative twist caused by delta frame flexibility and eccentric stay cable forces. Ideally, the response should be symmetric (see Figure 3.3 (c)). It seems unlikely that strain symmetry about the plane of the stay cables will be exactly achieved. Under a more complex deformation pattern incorporating the effect of unbalanced live load induced torsion, bending, and delta frame flexibility such symmetry will not occur. The instrumented sections are not the sections with delta frames (see Figure 3.6). This choice was made in view of the local cross-sectional distortions that are likely to occur in the delta frame sections. Such distortions, while interesting, do not represent a part of the global cross-sectional response being studied. Some of the local strains in the delta frame section are of interest, and these are considered further in section 3.4.2.

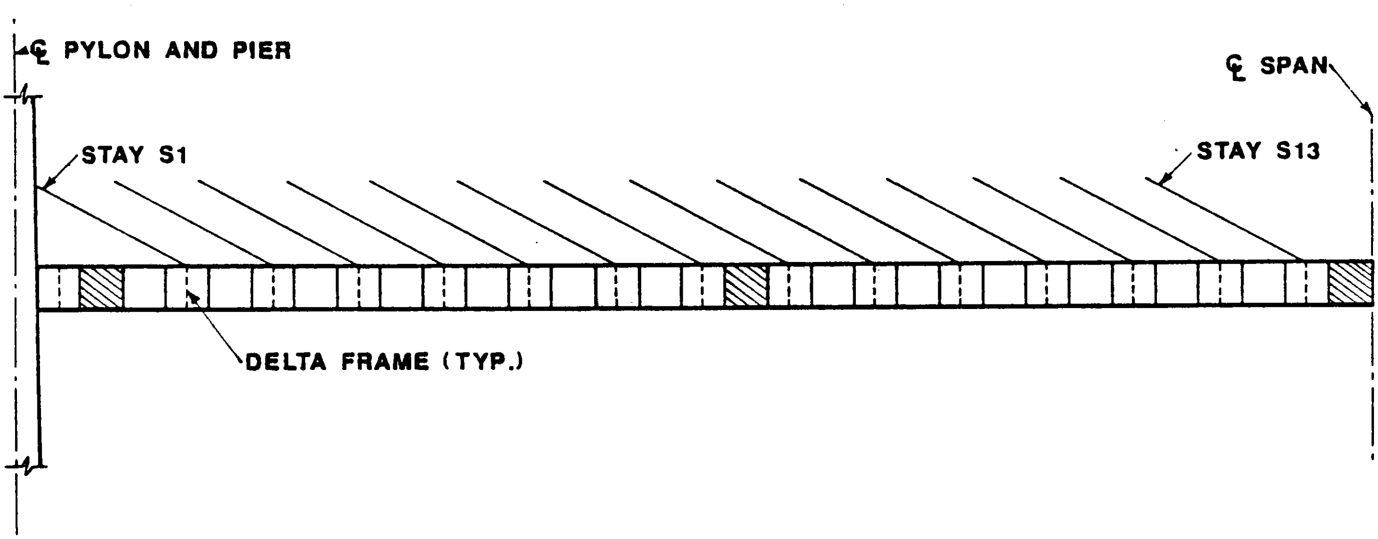


Figure 3.5 Sections for strain gaged dummy rebar instrumentation.

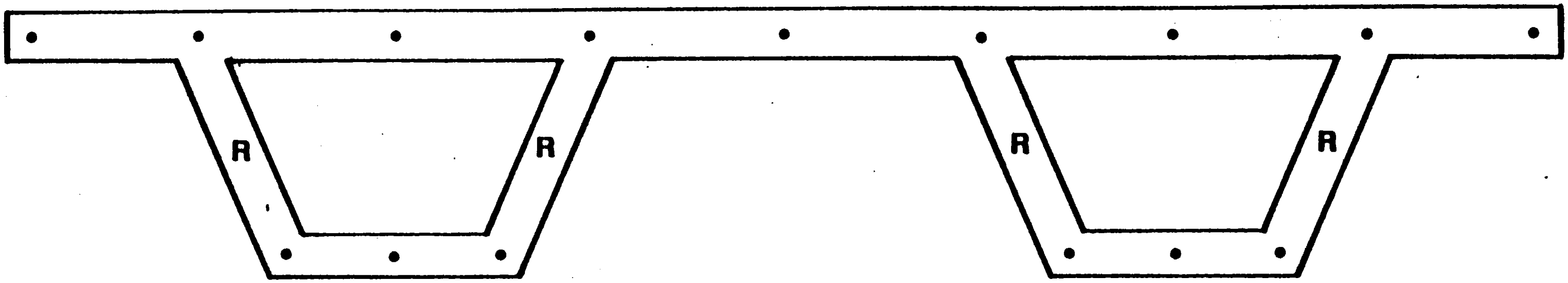


Figure 3.6 Section instrumentation -- typical for the two sections at quarter and mid-span.

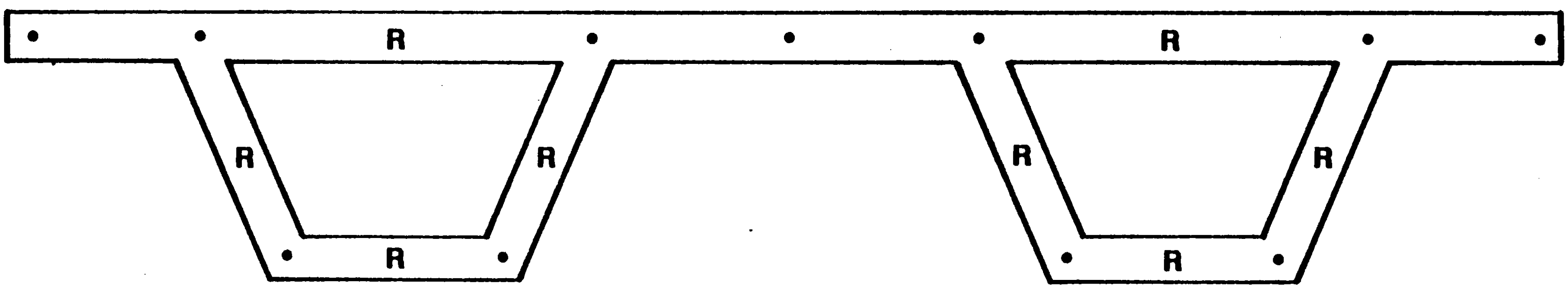
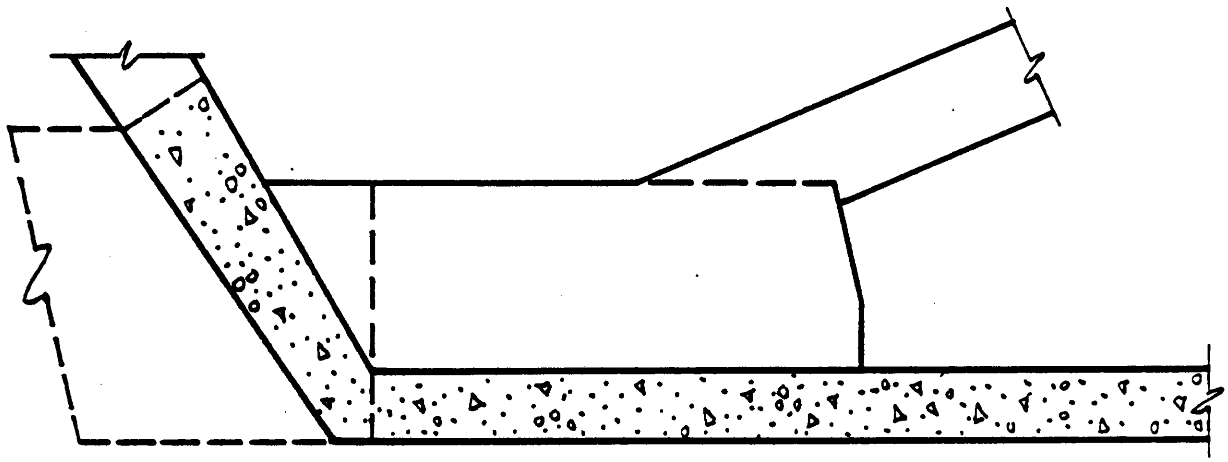


Figure 3.7 Section instrumentation near pylon.

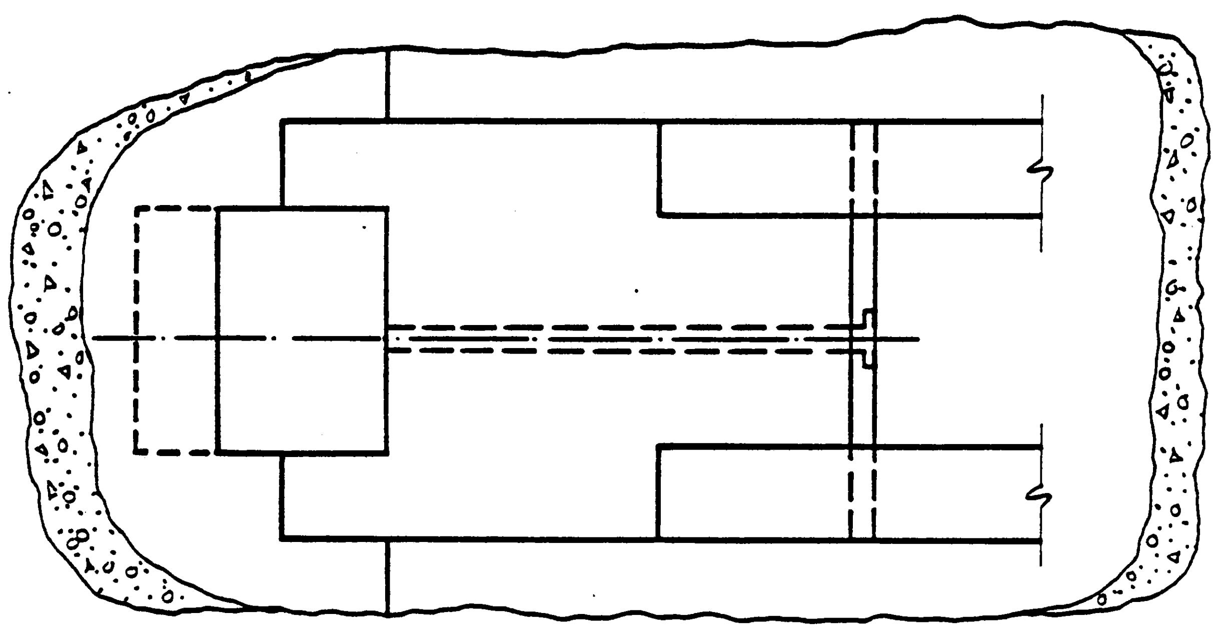
Backup data for the strain-gaged dummy reinforcing bars can be obtained relatively easily by the use of Whittemore strain gage points located on the inside surface of the box and on top of the deck flange directly adjacent to the strain-gaged rebar locations. It is not anticipated that these gages will be read as often as the foil resistance strain gages, but the Whittemore strain gages possess excellent long-term stability, which will allow periodic checks on the continued proper functioning of the foil resistance gages. Whittemore gage target points will be installed at the surface locations nearest to a number of the foil gage installation points illustrated in Figures 3.6 and 3.7.

3.4.2 Delta Frame Transfer Induced Strains

In addition to introducing bending, shearing, and possibly torsional strains into the boxes as a consequence of the stay cable forces, the delta frames will cause significant local strains at their points of connection to the box girders. The geometry of a typical connection region is shown in Figure 3.8. The magnitude of such strains in the concrete are expected to be directly proportional to the stay cable force since the geometries of all delta frame connections are essentially identical. The strains will vary rapidly with position in the vicinity of the connections between the delta frame and the box girder. However, since almost all of the force transferred by the delta frames will be the stay cable prestress force, which is virtually constant, the strains are not expected to display significant short-term variation. Clearly, it is of interest to gather some data from which the strain variation around the connection between the delta frame and the box girder can be inferred. It is particularly important to monitor such a region for the occurrence of tension cracks, which could be an indicator of localized distress. Since the primary time-varying strain to be expected is that resulting from localized creep in the region of the connection, automated data acquisition appears to be relatively unimportant for this purpose. Consequently, it is proposed to locate Whittemore gage target points at a number of locations around the transfer points illustrated in Figure 3.9. The gages will not be able to provide measurements of the highest strain gradients since the gage length is about 10 in. However, instrumentation with strain-gaged reinforcing bars or other mechanically inserted gages would do no better, and possibly not as well. The gages will provide data that can be used to estimate the average strain gradient in a region of approximately the same order of magnitude as the thickness of the box girder webs and bottom flanges. The section that appears to be critical for this is the delta frame section corresponding to the stay cables S1B, which have 90 seven-wire strands (the maximum number). The Whittemore gages will be installed, and initial readings will be taken immediately after form stripping, they will be read immediately before and immediately after initial tensioning, periodically between initial and final tensioning, and immediately before and after final tensioning. Further readings subsequent to final tensioning should show only those changes caused by creep. Two delta frames will be instrumented with gages (see Figure 3.2) that will be placed at the midpoint of each diagonal. The two delta frames to be studied will be located at cable stays 7 and 13.



a) SECTION THROUGH BOX GIRDER AT DELTA FRAME



b) PLAN VIEW OF THE SECTION AT THE DELTA FRAME

Figure 3.8 Box girder geometry at delta frame.

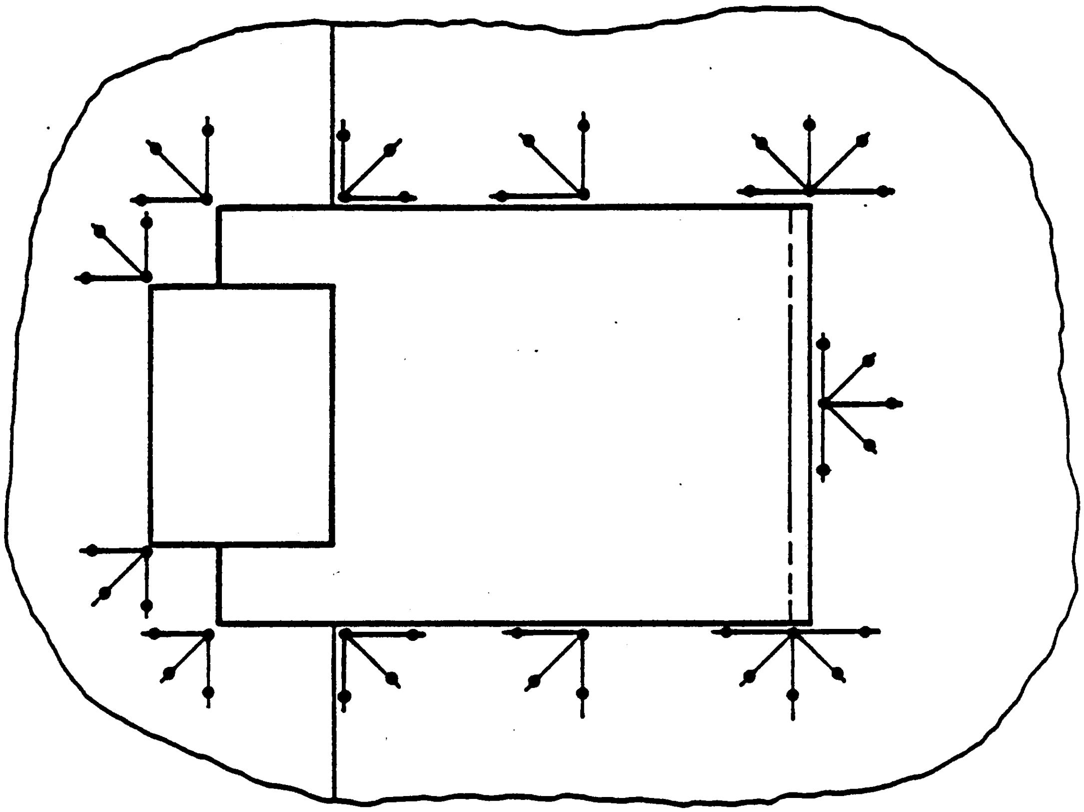


Figure 3.9 Whittemore target configuration at delta frames.

3.5 Gage Locations - Pylon Segments

The state of deformation in the pylon elements is generally expected to be somewhat simpler than that in the twin box girder sections. The cross section is relatively thick-walled, as shown in Figure 3.10, so distortion of the cross section shape will be less than in the box girders. The possible exception to this is in the near vicinity of the saddle regions, where the transfer of the stay cable force across the saddles will generate significant local strains. The objective of the present study is the gross sectional response, and only secondarily the local strains near the saddles. Accurate experimental measurement of those strains promises to be difficult in any event.

Excluding the local effects in the vicinity of the saddles, the primary responses of the pylons are expected to be uniform compression due to the cumulative stay cable load, and flexure in the plane of the stays as a consequence of unbalanced stay cable forces caused by live loads, tensioning errors, creep, shrinkage, and temperature effects. Experimental estimates of the resulting strain patterns at selected sections promises to be relatively straightforward. Two pylon sections are to be instrumented in the project: the uppermost cast-in-place section near the deck level and a section located at the segment supporting stay S7. These sections are shown in Figure 3.11. The instrumentation will consist of strain-gaged dummy reinforcing bars, similar to those used in the box girder sections.

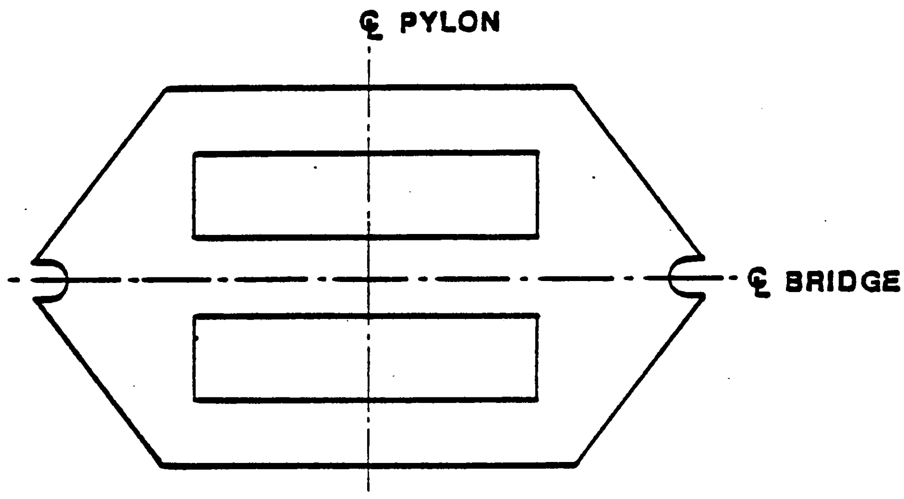


Figure 3.10 Typical pylon cross section at stay cables.

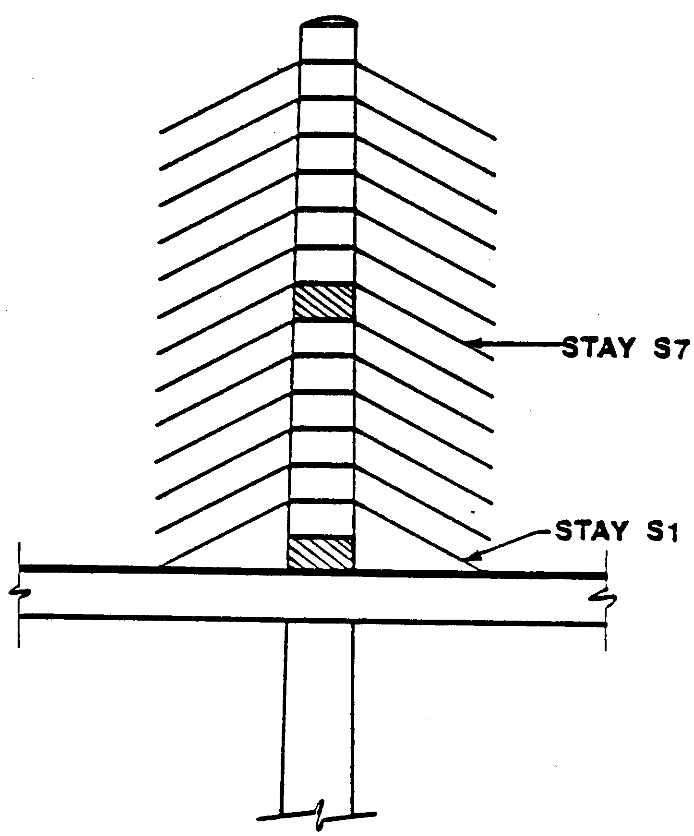


Figure 3.11 Pylon segments for strain instrumentation.

The patterns of dummy bars shown in Figure 3.12 will allow a reasonably complete picture of the compression, flexure, and time-dependent strains in the pylons to be obtained. In addition to the vertically oriented gages, an additional instrumented reinforcing bar will be located horizontally in the pylon section supporting stay S7 as shown in Figure 3.13. This bar will not allow complete determination of the response of the segment at stay S7 but will allow an estimate of the localized strains in the stay saddle to be obtained. This data will allow some interpretation of computer analyses of the near stay region, which would not otherwise be possible.

The distribution of stresses at transfer to the cast-in-place pylon is also of interest. These stresses will reflect the contributions of all stays. Consequently, the top of the pylon cast-in-place portion will be instrumented as shown in Figure 3.12.

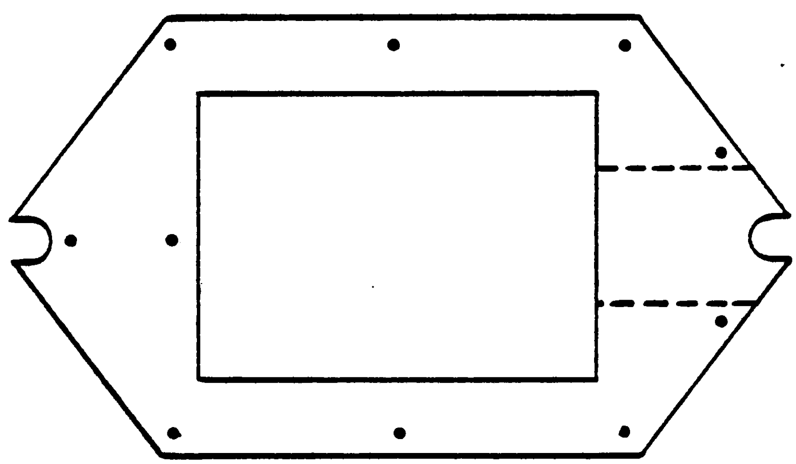


Figure 3.12 Location of strain gage rebar in top of cast-in-place pylon.

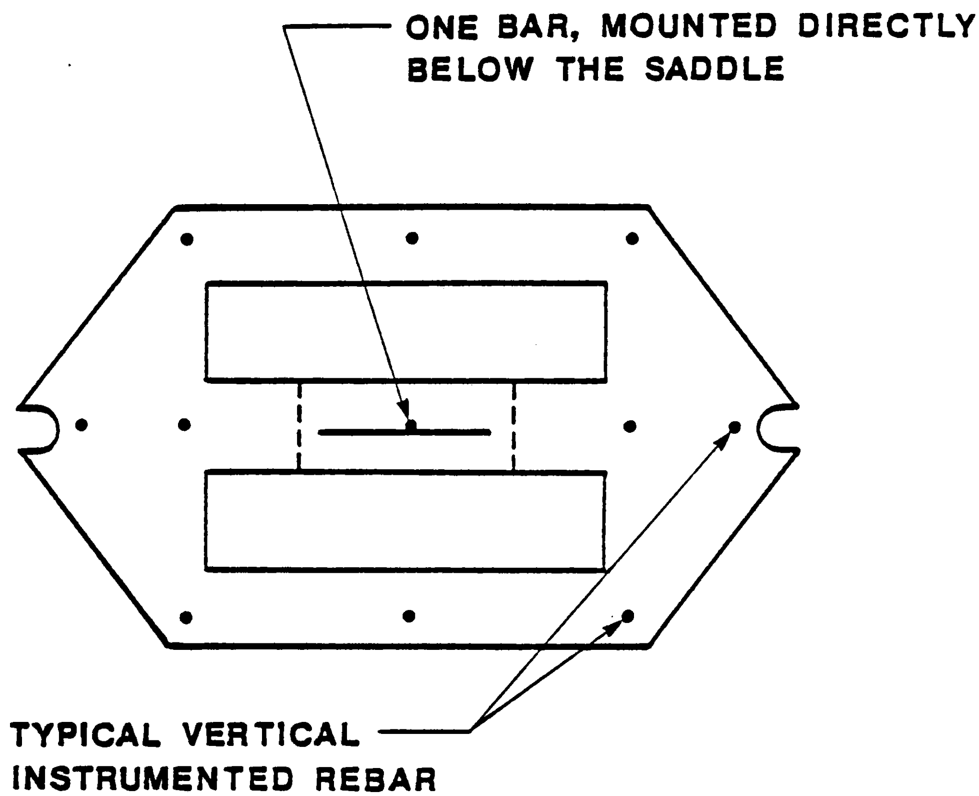


Figure 3.13 Location of additional strain gaged rebar under stay S7.

3.6 Gage Installation Procedure

The strain-gaged reinforcing bars will be located in the reinforcing cages during the period between cage construction and placement in the precasting forms. The reinforcing bars will be carefully wired into place against the reinforcing cage, and the locations of the strain-gaged reinforcement sections will be carefully noted. Recent discussions with the contractor indicate that approximately 24 hours will be available during which this activity can be carried out. At this time it is planned that the lead wires will be extended along the reinforcing cages in such a manner as to minimize the probability of damage from the concrete placement or subsequent vibration. In general this means that all runs of lead wire will be directly adjacent to the reinforcing cage bars; that the lead wires will be attached directly to the reinforcing cage bars at frequent intervals; and that the directions and locations of the wire runs will be selected to minimize the chance of damage. The lead wires will be extended to a common exit location on the inside of the section as near as practicable to the planned location of the data acquisition extender chassis or main unit, as appropriate.

Lead wires from gages located in the box girders that do not have data acquisition chasses present will be run through the deck flanges to a point where a subsequent splice with a lead wire extension into the box containing the nearest data acquisition chassis will be possible. The lead wire splices will be epoxy sealed, waterproofed with a polysulfide or other compound, and wrapped with electrical tape to minimize the probability of migration of water to the gage locations.

IV. THERMOCOUPLE INSTRUMENTATION

4.1 Background

The differential heating and cooling of the twin box girder sections and the pylon, because of environmental conditions, cause complex strains within the sections. Recent studies have indicated that because of the transient nature of the thermal loading and the relatively poor thermal conductivity of concrete, the distribution of temperatures through the box sections will be highly nonlinear. Radolli and Green (1976) considered the heat transfer processes that occur between the atmosphere and a concrete superstructure and the climatic conditions necessary for the development of temperature differentials during summer and winter seasons. Temperature-time analyses were conducted using a one-dimensional heat-flow analysis. Simple empirical design expressions were developed for thermal stresses and curvature, based on typical climatic data for summer and winter conditions. Elbadry and Ghali (1982, 1986) conducted thermal analyses of box girder sections using a finite element approach and developed design guidelines for the use of partial prestressing to reduce thermal stresses and control thermal cracking by provision of sufficient amounts of nonprestressed steel. Prakash Rao (1986) conducted heat-flow analysis in concrete bridges subjected to insulation effects and incorporated the resulting thermal strains into a finite strip program for bridge analysis.

Imbsen et al. (1985) conducted an extensive survey of available experimental data and suggested guidelines for thermal stress calculations in concrete bridge superstructures that are consistent with the analytical results discussed above. However, much of the experimental data used in developing their conclusions were obtained outside the United States. Differences in environmental conditions suggest the need for developing additional data directly applicable to American practice. Moreover, review of the literature has revealed little relevant literature on temperature variations in pylon sections and stays in cable-stayed bridges. Hence an important component of the proposed program will be to obtain data on the thermal variations within the box sections, pylons, and stay cables.

The analytical methodologies developed for the analysis of thermal gradients and resulting thermal stresses in concrete bridge deck girders, and box girders apply in the analysis of concrete pylon structures as well. Little work has been conducted toward establishing detailed thermal strain data for such elements, however. Likewise, there appears to be little data available from field studies of stay cable thermal strains. Consequently, it is quite important that the present study provide insight into the thermal strains in those structural elements.

4.2 Thermocouple Instrumentation in Box Girders

4.2.1 Location of the Thermocouples

The James River bridge has a straight roadway running approximately north-south. The approach spans have a 3 percent grade. The transition vertical curve takes place over the river span and approximately three spans on each side. The high point is located at the center of the river span. Although some variation in thermal response along the span may be expected because of slightly different angles of solar incidence and different levels of exposure to wind, it is believed that a thorough instrumentation of a single section with thermocouples will provide useful and representative data. Consequently, a single twin box section, located on the main span, near midspan to minimize shading from the pylons, will be instrumented as shown in Figure 4.1 (a). Since the thermal gradient through the thickness of the upper flange is one of the most important items to be measured, thermocouples will be located at several locations through the thickness. Thermocouples will be located across the complete twin box section to determine if significantly different thermal effects occur as a consequence of eastern versus western location. The western side box may, for example, be subjected to somewhat greater cooling due to the prevailing wind directions at the site but may also receive somewhat greater heating from afternoon sun. While the details of the thermal distributions may be expected to vary somewhat between the eastern and western box sections, the overall nature of the distribution is expected to be similar. Consequently, only one half of the box will be instrumented with multiple thermocouples through the thickness. The other half of the box will be instrumented with representative gages located on the inner and outer surfaces of the box and on the top and bottom of the overhanging flange, as indicated in Figure 4.1(a). Thermocouples do not display the precision of thermisters but are considerably less expensive to install, and the precision is sufficient for the problem at hand. Moreover, thermocouples are somewhat sturdier than thermisters. Consequently, type T thermocouples will be utilized.

4.2.2 Installation of the Thermocouples

Several thermocouples will be placed through the thickness of the top and bottom flange sections in the box girders (see Figure 4.1(b)). This approach is consistent with the knowledge that large thermal gradients occur in these locations, particularly in the top flanges. In order to obtain an accurate picture of the temperature distribution at any time, it is necessary that the thermocouples be accurately located within the section. Following similar studies conducted by others, gages will be located in the section by attachment at the appropriate elevations to polyethylene tubes that are cut to the thickness of the slab and attached to the reinforcement cages. This attachment scheme is illustrated in Figure 4.2.

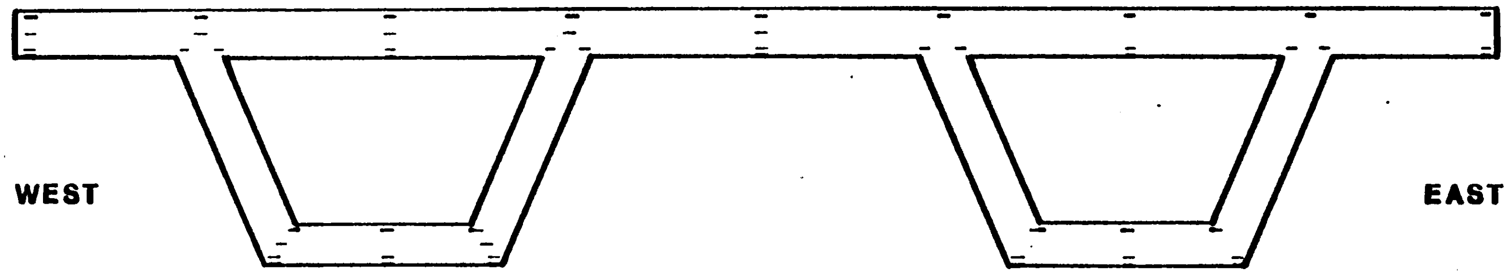


Figure 4.1(a) Type T thermocouple locations in twin box girder sections.

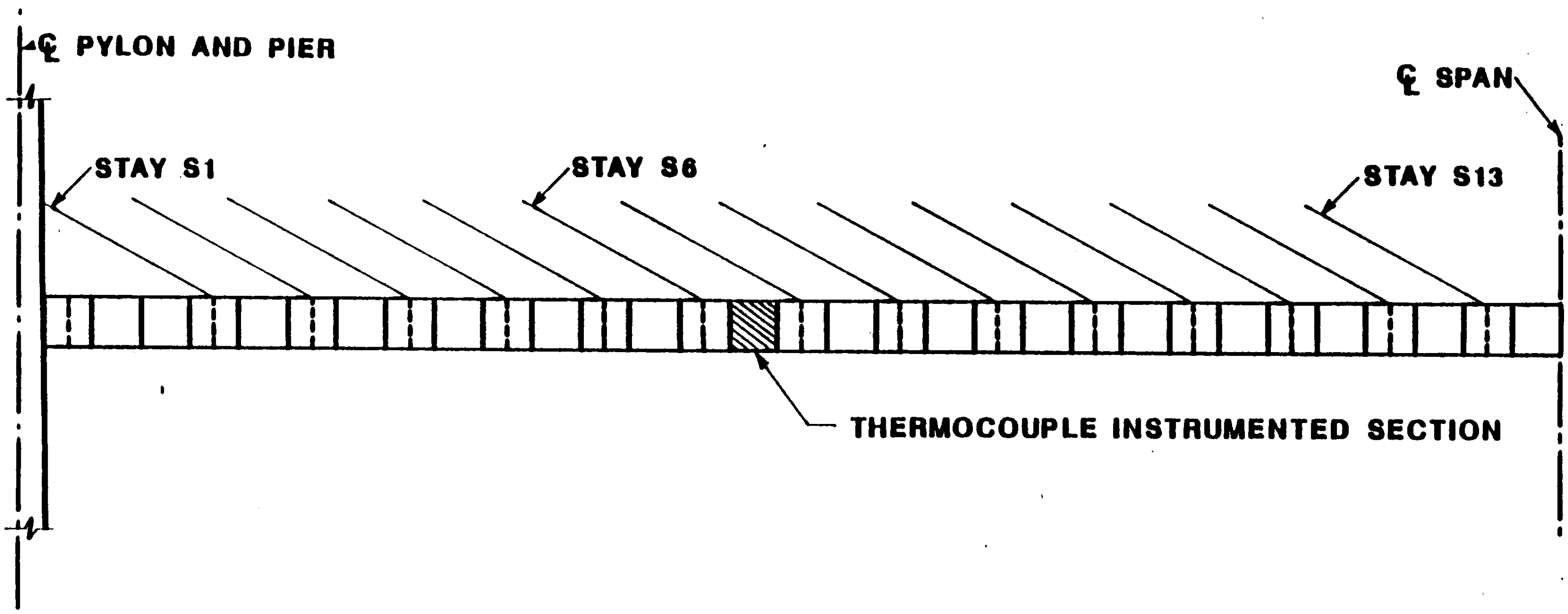


Figure 4.1(b) Girder section for thermocouple instrumentation (south cantilever).

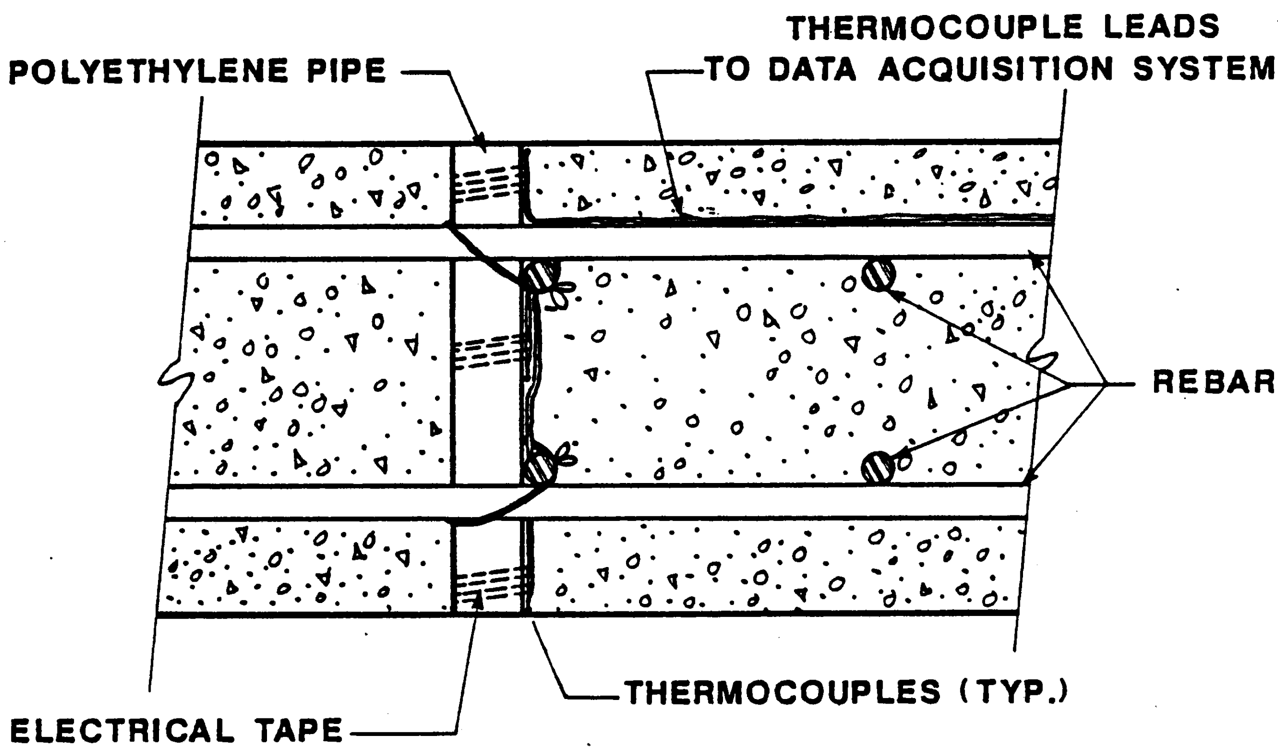


Figure 4.2 Positioning of thermocouples in section.

4.3 Thermocouple Instrumentation for Pylons

4.3.1 Locations of the Thermocouples

The pylons have a somewhat more complex configuration than the prismatic twin box girders: they taper from a maximum dimension at the roadway level to a minimum size near the top. This presents a potential complication since the details of the thermal gradient will be expected to vary somewhat depending on the dimensions of the cross section at a given elevation. In addition some vertical heat flow may be anticipated, although this effect is likely to be small. The taper in cross section is quite gradual, so it is anticipated that useful representative thermal data can be obtained from thorough instrumentation of one cross section. This section will be located above the deck level to eliminate shading by the deck and piers. Useful and different data could also be obtained for a section below the deck, but the shading is expected to reduce the thermal extremes, particularly during the hot months. The gages will be located on the cross section as shown in Figure 4.3 (a). Type T thermocouples will be used. Pylon segment D-6, located directly under stay S-7 will be instrumented (see Figure 4.3 (b)).

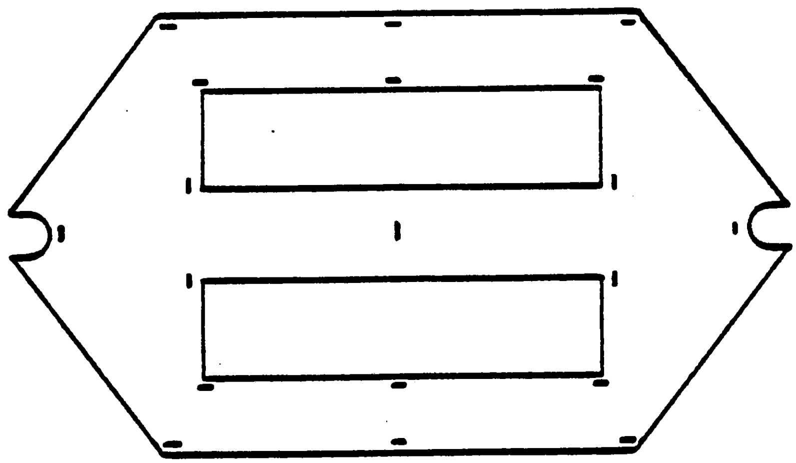


Figure 4.3(a) Thermocouple locations in pylon section.

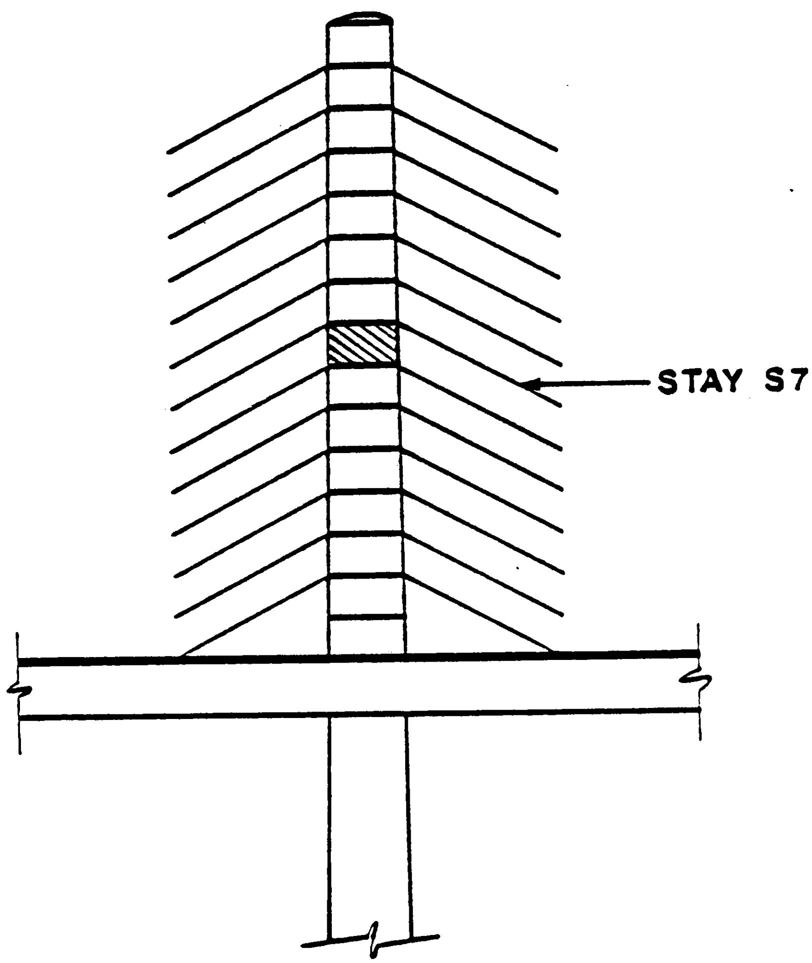


Figure 4.3(b) South Pylon Section For Thermocouple Instrumentation.

4.3.2 Installation of the Thermocouples

Because of the considerable thickness of the pylon sections, precise placement of the thermocouples in the pylon section is not anticipated to be quite as critical as in the twin box sections. Consequently, it is anticipated that thermocouples will be attached directly to either the main reinforcing cage or to dummy bars that will be wired to the existing section. Once the thermocouples have been placed, some care will be taken to determine accurately where they are located and to place the outermost thermocouples as near the outside surface of the pylon as possible.

4.4 Thermocouple Instrumentation for Stay Cables

The stay cable instrumentation can be executed in one of two ways. At locations where strain gages are fastened directly to the stay cables (see Chapter II), it is necessary to install thermocouples at the same time. Alternately, dummy sections of stay cables could be constructed, insulated at both ends, and thoroughly instrumented with thermocouples. The first alternative has the advantage of providing direct readings for the actual stay cables. Moreover, since the strain gages attached to the stay cables will not be temperature compensated, knowledge of temperatures at the strain gage locations will be necessary to allow accurate calculation of temperature corrected strains. Since access to the actual stays is necessary for strain gage installation at the stay locations, thermocouples can be placed at the same time. The use of dummy sections of cables would have the advantage that more thorough instrumentation would be possible by using several thermocouples through the thickness; but the readings must be considered indirect, and care would need to be taken that the dummy sections have an exposure similar to that of the actual cable sections.

Given the importance of the stay cable strain data and the decision to attach strain gages directly to stay cables at several points around the periphery of selected stays, the alternative to be pursued in this project will be to install thermocouples at the locations of the strain gages on the sections. The locations of the stays to be instrumented were discussed in Chapter II, and the locations of the strain gages within those stays were indicated in Figure 2.2. As a primary instrumentation option, the thermocouples will be located at these positions in the cross sections of the stays.

V. DATA ACQUISITION SYSTEM

5.1 Distributed System Concept for Data Acquisition System

In developing the data acquisition system for the I-295 bridge monitoring project, several systems were considered. Initially, the systems under consideration were laboratory systems with a single data acquisition chassis. The best of such systems are quite good, and they may have complete, integrated software for data analysis; however, they are not particularly well suited for field instrumentation. These systems are typically quite bulky and must operate under relatively restricted temperature conditions. Moreover, since all data acquisition channels are located in a single cabinet, it is necessary to run all lead wires from all gages back to a single location. The quantity of wire needed quickly becomes prohibitively expensive when a large structure such as the current cable-stayed bridge is being monitored.

As an alternative to a laboratory data acquisition system, it is possible to utilize data acquisition systems designed for industrial or field use. These units typically consist of several small scanning units, connected to a controller by data transmission lines that can transfer data over distances as large as several thousand meters. Data acquisition systems of this type will be referred to as "distributed data acquisition systems" (DDAS) because all channels need not be located in a single physical unit. A schematic figure of a DDAS is shown in Figure 5.1. While the data transmission lines used for such systems are more expensive than the lead wires used in the structural instrumentation, it is not necessary to run a different lead wire for each channel.

Industrial data acquisition and control systems are typically designed to be installed in factory environments, where temperature control such as that possible in a laboratory is not achievable. Industrial data acquisition and control systems are designed for a wide range of potential applications. Hence they tend to be quite versatile in their measurement capabilities. This versatility may be achieved in several ways. Either a large number of different scanning boards may be available for special purposes, or each type of scanning channel may be made as adaptable as possible. Several of the data acquisition systems surveyed in developing the I-295 bridge instrumentation plan were just as precise as the laboratory systems, although the software did not appear to be quite as comprehensive. Moreover, they invariably had a wider operating temperature and humidity range than the systems designed for laboratory use.

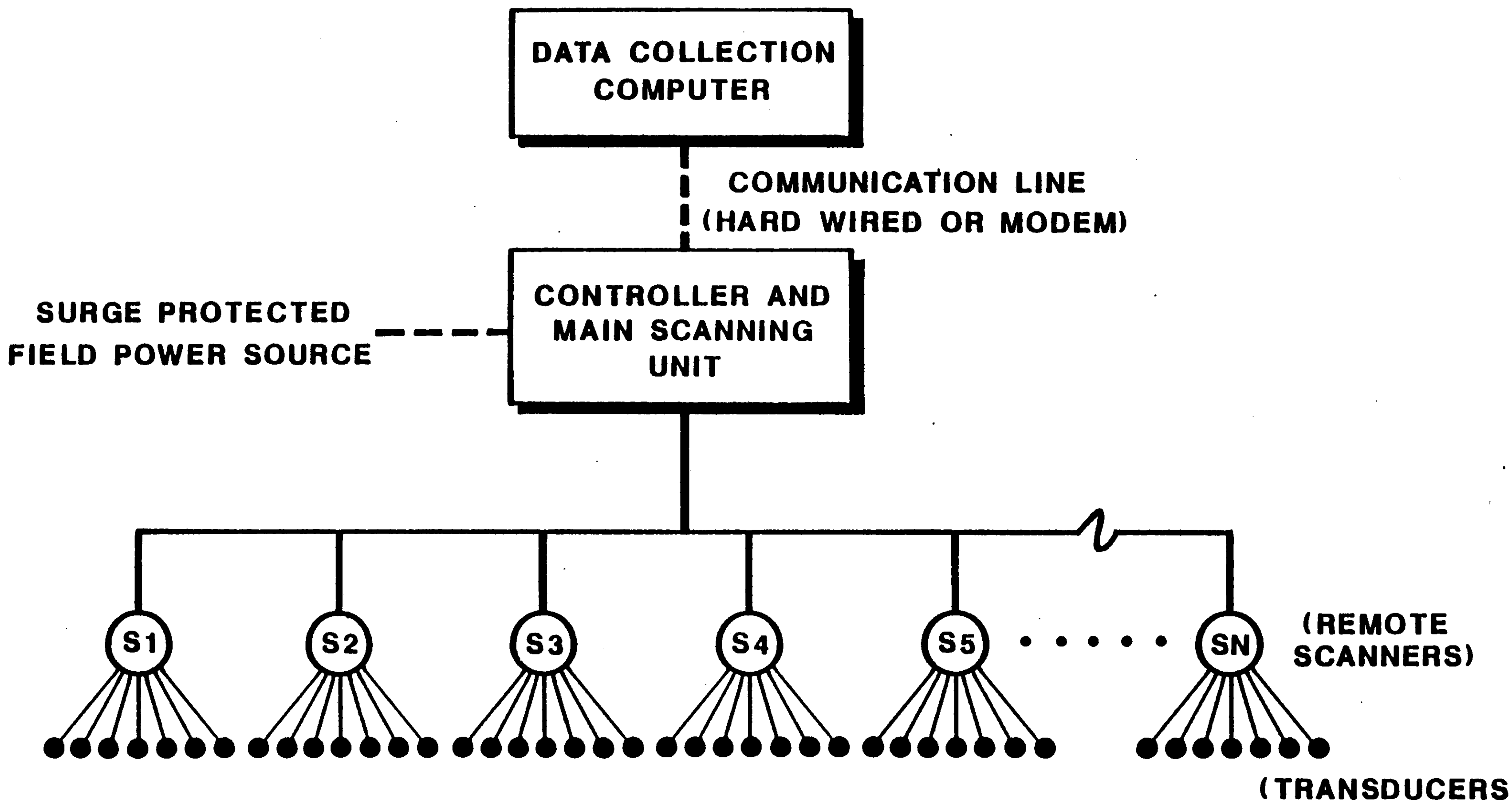


Figure 5.1 Data Acquisition System for field monitoring

Field data acquisition systems provide an alternative to industrial systems. Such systems typically also allow for distributed channels. They are usually even more rugged than industrial data acquisition systems since they are required to perform under field conditions. Several such systems have been developed with particular emphasis on long-term monitoring of large geotechnical and hydraulic projects. Often such systems must function under particularly severe environmental conditions and in locations where line power is not readily available or maintainable. Consequently, these systems often have a battery-powered or solar-powered option. Unfortunately, since the gages used for particular field applications are often not general purpose gages, field data acquisition systems sometimes tend to be somewhat less versatile than their industrial counterparts, although there does appear to be a trend toward increasing their versatility.

At the time when the particular data acquisition system used for this project was selected, the exact instrumentation scheme had not been fully planned. Consequently, it was necessary to plan for the most versatile system possible. The following criteria were used in selecting the data acquisition system.

- Extended operating-temperature range: The environment on the I-295 bridge is not expected to be quite as severe as a fully exposed outdoor environment. The thermal mass of the concrete superstructure, which is quite large, may be expected to moderate the temperature extremes somewhat, although detailed data on the bridge environment is not currently available. Consequently, either an industrial or field data acquisition system would be preferable to a unit designed for laboratory use.

- Distributed scanners: Because of the advantages outlined above, specifically lighter unit weight and lower lead wire and cabling costs, it was decided that the data acquisition system selected for the project should be a distributed system.

- Versatility: The system selected must be capable of reading a wide variety of gages including but not limited to Carlson strain meters; foil resistance strain gages in quarter bridge, half bridge, or full bridge configurations; and thermocouples. Moreover, since the system selected for the current project may be used extensively in subsequent projects, it would be ideal for the data scanning and exciting channels to be as versatile as possible; that is, each type of data scanning board used should be capable of reading the widest possible variety of gages.

- Reliable data transmission: Depending on the DDAS design, the data may be returned to the central controller in either analog or digital form. If the data is returned in analog form, it is only necessary to have a single analog to digital (A/D) converter at the central controller. If, however, the data is returned to the controller in digital form, it is necessary to have an A/D converter in every scanning unit. After numerous discussions with experienced instrumentation engineers, it was decided that the digital data transmission option provides somewhat cleaner data in a noisy environment than analog data transmission. Consequently, it was decided to specify a DDAS with digital data transmission, even though there is some additional overhead associated with the multiple A/D converters.

After considering a number of available industrial and field data acquisition systems, it was decided that a system manufactured by the John Fluke Company best fulfills the needs of this project. The system selected uses a Helios main controller to communicate with each of the remote scanning extender chasses. The Helios system is capable of reading thermocouples, variable resistances (such as Carlson strain meters), and quarter, half, and full bridge strain gage configurations with a single type of data acquisition channel and with 17-bit resolution. Reading the Carlson strain meters requires two channels of excitation and scanners per gage. Each extender chassis has its own A/D converter, with the data being shipped back to the controller in digital form. The controller chassis has additional scanning channels, its own A/D converter, the capability of storing a limited quantity of data, and the software necessary to drive the scanners. Permanent data storage is accomplished by connecting the Helios control unit to an IBM personal computer or compatible.

Communication between the personal computer and the Helios unit may be accomplished using any of several BASIC interpreters or compilers, such as GWBASIC or Quick BASIS, or by laboratory data acquisition software, such as Labtech Notebook. In addition the Fluke company manufactures two data acquisition software packages, Prologger HCL and Helios Toolbox, which may be used to communicate with the Helios system. Labtech Notebook, Prologger HCL, and Helios Toolbox may all be used to format data files for use by Lotus 1-2-3. After consideration of the Prologger HCL, the Helios Toolbox, and Labtech Notebook packages, it was decided that the Helios Toolbox would be a suitable package for the present application.

Typically, personal computers cannot operate over as large a temperature and humidity range as the data acquisition system selected here because of the presence of magnetic storage media (hard disks, floppy disks, magnetic tape drives). It is possible to purchase personal computers that utilize non-volatile bubble RAM memory and are designed expressly for industrial applications. Data could then be acquired and stored on bubble memory in an automatic mode. This would necessitate fairly frequent visits to the bridge with an external floppy disk drive. Location of the personal computer in an environmental control chamber would alleviate this problem, but the need for frequent trips to the bridge would not be eliminated. Alternately, it would be possible to connect the Helios controller to an auto-answer modem, which could then be called from a remote computer with an auto-dial modem. The computer could then be located in a suitable environment. The major expense associated with this approach to data acquisition is the installation of a telephone line on the I-295 bridge.

After considering the options outlined above, it was decided to locate the data acquisition computer in the offices of the Virginia Department of Transportation (VDOT) in Richmond, and to install a telephone line on the bridge to attach to the data acquisition system. Periodically, the data diskette will be changed by VDOT personnel and a backup copy will be made of it to preclude the loss of data; the diskette with the most recent batch of data will be mailed to the VTRC. The data will be analyzed by the VTRC research staff and assembled into the report form.

A summary of the data acquisition configuration is illustrated in Figure 5.2.

5.2 Transient Suppression in the Data Acquisition System

During the data acquisition system design, it became apparent that some electrical isolation of the system would be necessary. Several possible events could lead to loss of data, including power outages, electrical line surges, and lightning strikes. Discussions with researchers involved in previous field data acquisition applications revealed that the loss of equipment as a consequence of line surges or lightning strikes can be expected unless precautionary measures are taken. Hence, it is necessary to provide several different types of electrical isolation for the data acquisition system.

5.2.1 Protection from electrical line surges other than lighting and power outages

Even in the absence of lightning strikes, it is possible for significant transient line surges to occur, which could be damaging to the data acquisition controller or extender chasses. While these units are designed to be relatively resistant to such damage, adequate protection can be provided quite easily using surge protectors, or by

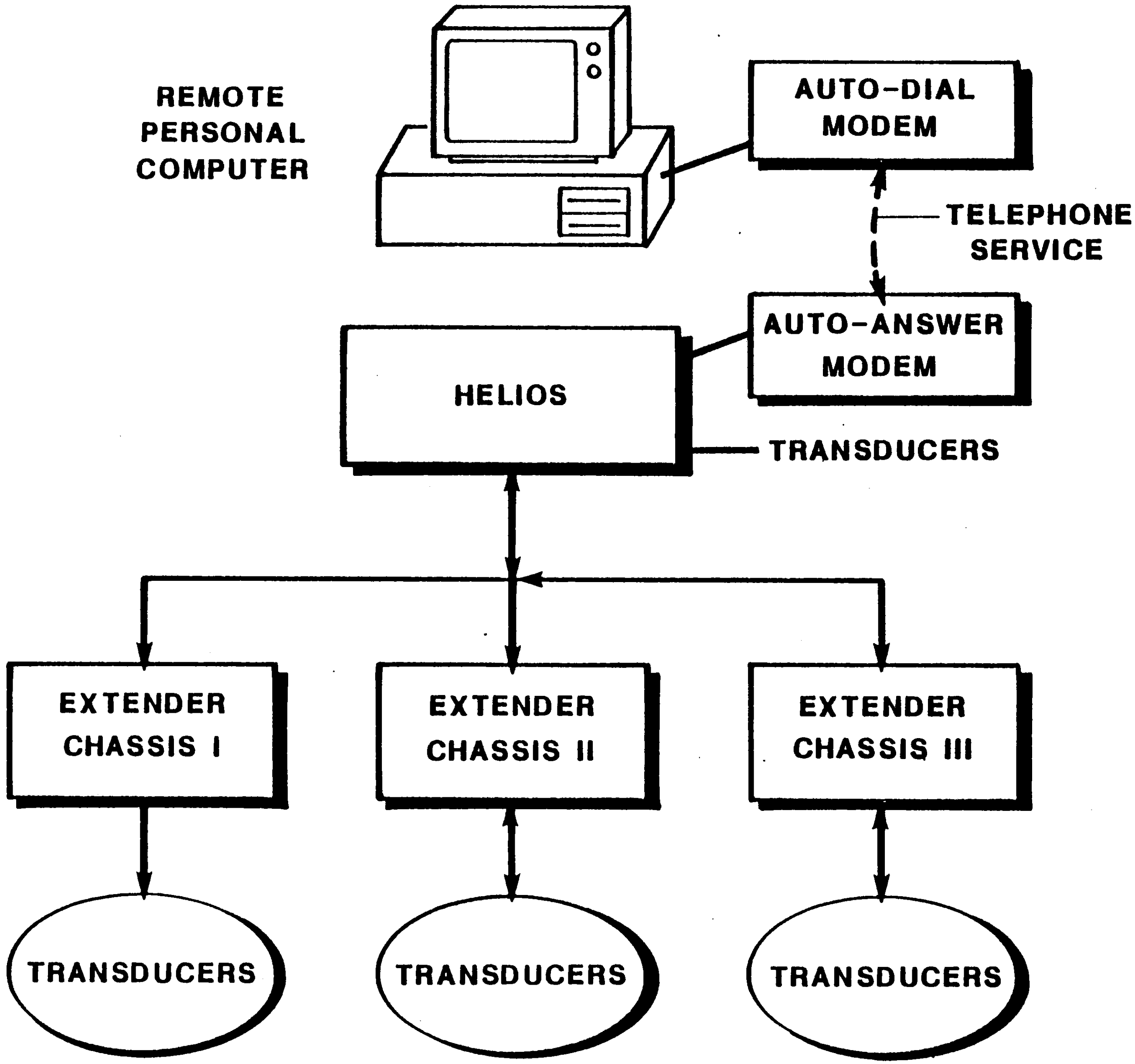


Figure 5.2 I-295 Bridge Data Acquisition System.

supplying the power via an uninterruptible power supply (UPS) with built-in surge protection. A UPS has the advantage of allowing the data acquisition system to remain functional for some time following a power failure, thus minimizing the probability of data loss, but is considerably more expensive than a simple surge protector. A UPS also needs to be protected from very large power surges, such as might be generated by a nearby lightning strike.

After considering the alternatives, it was decided to provide a 500 VA UPS for the data acquisition system. This UPS will allow the system to remain functional for roughly 1 to 2 hours following the loss of power. Additional backup power for the extender chasses will be provided by 12-volt batteries, which are recharged from the 12-volt trickle charge terminals on the extender chassis units.

5.2.2 Protection from lightning strikes

Typically, lightning strikes can generate quite large transient currents in line, data transmission, and lead wire lines. Protection of the UPS and the data acquisition system from lightning strikes requires that these currents be given an alternative path to ground. Several different levels of protection need to be considered.

5.2.2.1 Protection of 110 volt power line

These conductors will run from the line current to the UPS and from the UPS to the Helios unit. The UPS will be located quite near the Helios unit in an electrically isolated cabinet, to be described in more detail below; consequently, it is not necessary to provide surge protection between the UPS and the Helios. Schematically, 110 volt line-surge suppression will be provided as shown in Figure 5.3. The surge suppressors (SP) shown in Figure 5.3 will be heavy duty line protectors similar to those used for type 170 traffic controller cabinets. An example of this kind of surge protector is the SHP 300-10 manufactured by EDCO Inc., Ocala, Florida. These units are capable of absorbing a repetitive peak surge of up to 20,000 amps for at least 20 reoccurrences.

5.2.2.2. Cabinet Isolation

All of the units of the data acquisition system located on the I-295 bridge will be mounted in cabinets to reduce contamination by dust and provide a minimal level of environmental protection. This topic is discussed in more detail below. Additional electrical isolation will be achieved by grounding the chassis of the Helios and extender units and the cabinets in which these units will be located.

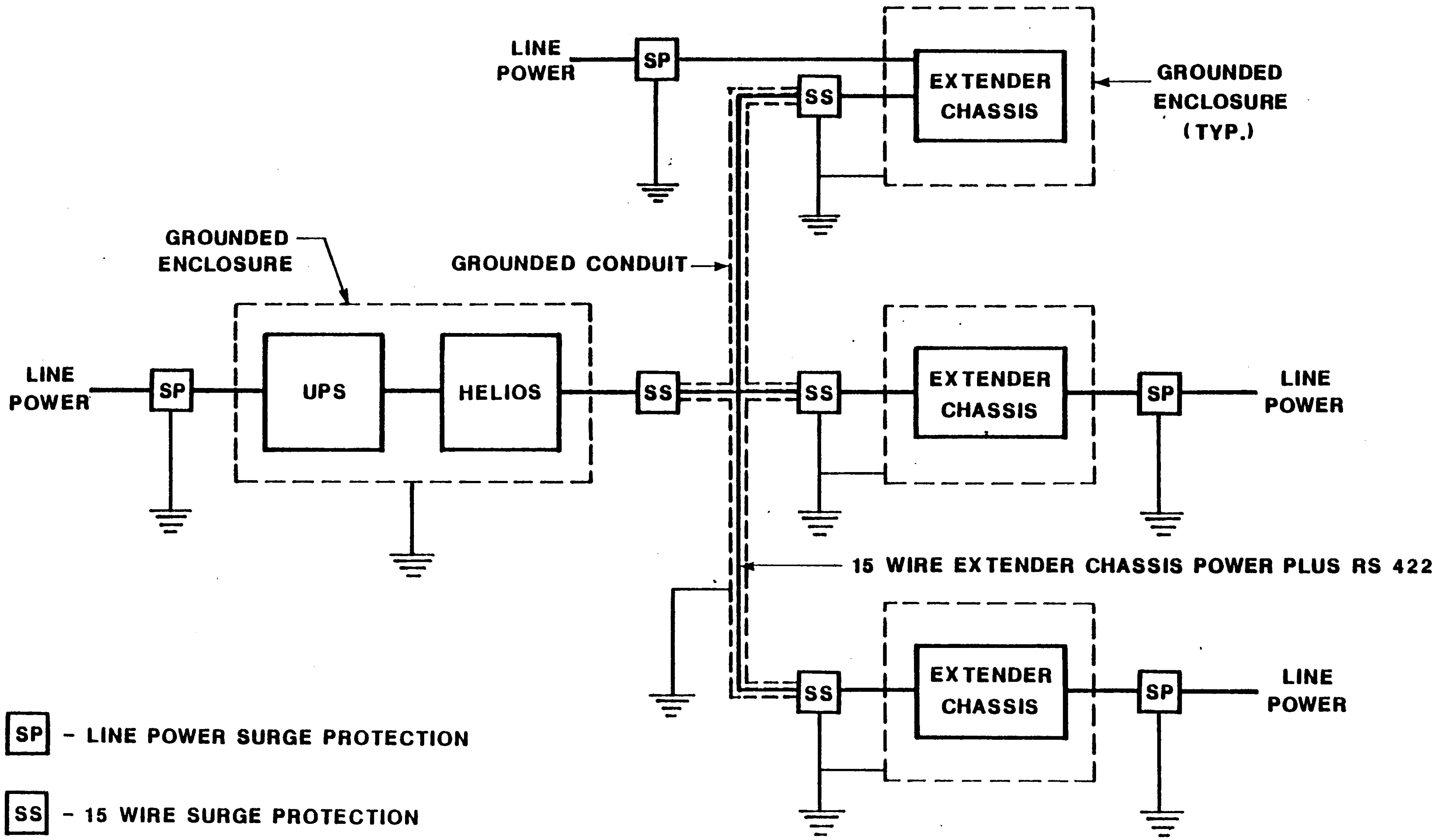


Figure 5.3 Line power and communication line surge suppression.

5.2.2.3 Data line isolation

The Helios unit communicates with the individual extender chasses via RS-422 communication channels. The data lines consist of 15-wire cables running between 15-pin connectors. The 15 pins are configured as indicated in Table 5-1.

Table 5-1

Data Pin Configurations for Helios Communication and Extender Chassis Power

Pin Numbers	Function
1-3	12 volt common power
4	-
5-6	5 volt common RS422 return
7	RX- (RS-422)
8	Rx+ (RS-422)
9-11	12 volt common return
12	-
13	Tx- (RS-422)
14	Tx+ (RS-422)
15	Shield (to chassis)

Since these lines must run for extended distances outside of the electrically isolated cabinets, there is a potential for lightning strikes along the lines to damage the data acquisition system. The protection will take two forms. First, the data and extender chassis power lines will be run in grounded conduit to reduce the probability of a direct strike on the lines. Second, the data lines will be electrically isolated using metal oxide varistors (MOVs), silicon avalanche devices, gas tubes, or other electrical isolation devices. Fluke company engineers have suggested that the maximum voltage allowed in these lines be 30 volts. Consequently, electrical isolation of the data transmission lines requires that these 15 pins be run through surge protectors with a maximum clamping voltage around 30 volts. A relatively economical means of accomplishing this is via MOVs. Since MOVs deteriorate with time, it is essential that installation involving their use be checked periodically to assure proper functioning. The MOVs will be located at terminal strips within the Helios and extender chassis cabinets and will be extended to common ground. An ideal MOV for this application does not appear to exist. Alternatives to the use of MOVs also exist. Inmac markets an RS 422 surge protector which may, however, not be adequate for lightning-induced surges. Moreover, RS422 surge protectors only provide surge protection for the RX-, RX+, TX-, and TX+ lines. Consequently, the 12-volt power lines need to be protected separately. A series MOV system which can be used for the 15-wire leads is illustrated in Figure 5.4. Edco manufactures a multiwire lightning protector designed specifically for low voltage data lines, and it appears to be quite suitable for protecting the RS 422 lines.

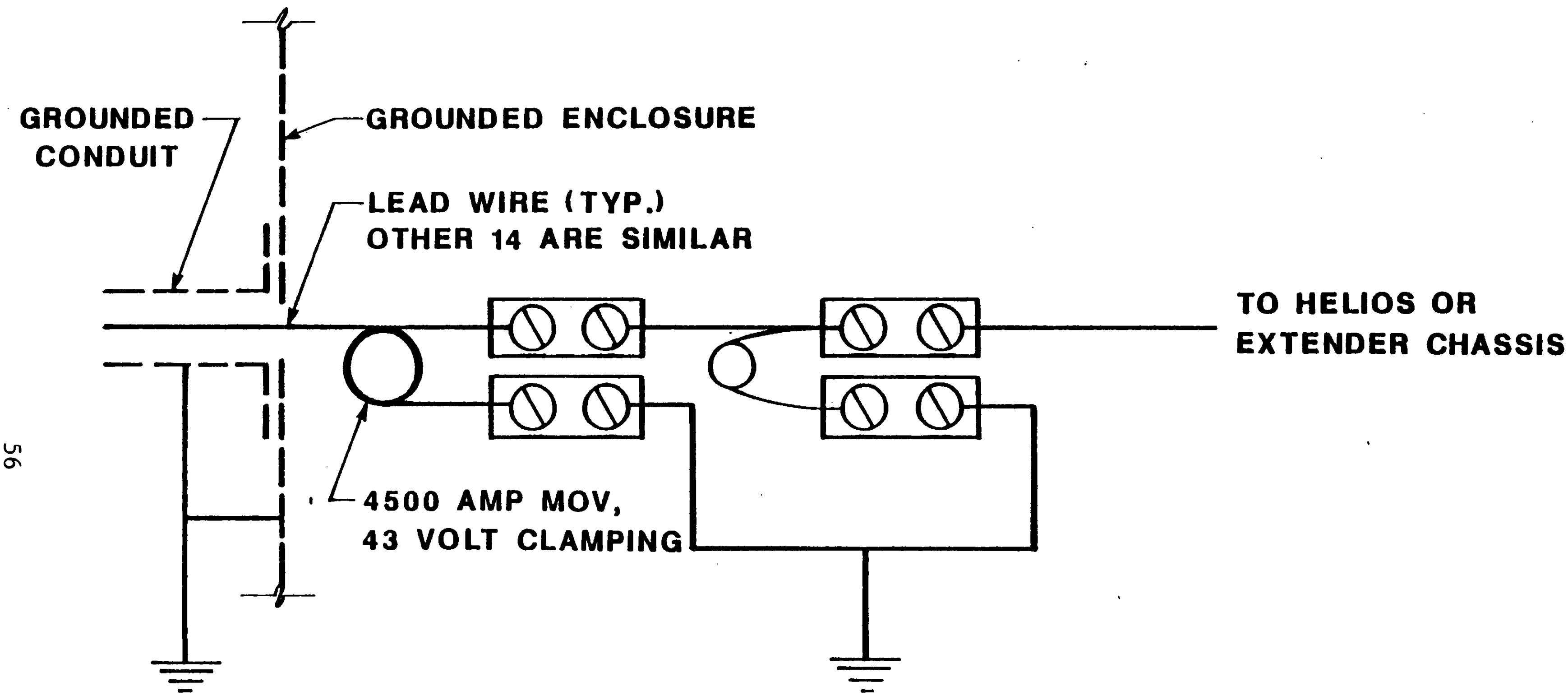


Figure 5.4 Surge protection for 15 wire communication and power lines (one wire shown).

5.2.2.4 Lead wire isolation

The individual lead wires may also be the source of damaging surges to the data acquisition system. Two primary means will be used to minimize the probability of damage from this source. First, the lead wire used will be shielded wire. The shields will be taken to common ground at the cabinets. This will serve to take much of the energy from a direct strike. Second, the lead wires will be run through terminal strips with MOVs to ground. The data acquisition channels are capable of withstanding 250 volts on a repeated basis. Since the signal voltages are roughly 4 volts, electrical isolation of these channels will be quite easy. Radial lead metal oxide varistors with a clamping voltage of 210 volts and a peak current capacity of up to 4500 amps are readily available. For example, the GE V120ZA6 is rated at 75 volts RMS or 102 volts DC continuous.

5.2.2.5 Thermocouple isolation

Thermocouple voltage isolation may be carried out in essentially the same manner as the strain gage lead wire isolation. One question that is still being addressed is the effect of the dissimilar metals and the MOVs at the terminal strips on the generated thermocouple voltages. It appears that the presence of the dissimilar metals at the terminal junctions will have no effect, provided that the lead wire from the terminal strips to the data acquisition system are made of the same alloys as the thermocouple wire. The MOVs are hypothetically electrically inactive except under surge conditions. Some controlled studies will be conducted under laboratory conditions before this scheme is installed in the field.

5.2.2.6 Gage isolation

While it is possible to protect the individual strain gages using MOVs in exactly the same manner as the data acquisition system, this step will not be used for several reasons. First, the installation of MOVs at the strain gages may be expected to considerably complicate the strain-gaged reinforcement assembly. Some loss in reliability of the individual strain gage assemblies may be expected because of the additional complication. Second, since MOVs tend to degrade over time and no replacement of these units will be possible, any protection achieved in this manner would be only temporary. It is felt that loss of some gages may be unavoidable during the course of the project.

5.3 Environmental Protection for Data Acquisition System

The data acquisition system will be located inside the instrumented pylon and box girder sections. Although the data acquisition system units will not be exposed directly to the weather, and the temperature extremes will be moderated somewhat by the considerable thermal mass of the bridge, some environmental protection of the units will be necessary.

5.3.1 Helios Unit and Uninterruptible Power Supply

The most critical units in the data acquisition system are the Helios unit and the uninterruptible power supply. The Helios unit has an operating temperature range of 0° to 50° Celsius, with an allowable humidity range of 0 to 80 percent. The uninterruptible power supply has an operating temperature range of 0° to 40° Celsius.

Three extremes appear to be of concern, although detailed data is not presently available. First, lower temperatures than 0° Celsius are likely to occur during the winter months. Enclosing units of the system in a relatively small, insulated enclosure would probably eliminate this problem since the system will generate some heat during operation. At the high end, 50° Celsius (122° Fahrenheit) is not likely to be exceeded; but as the high temperature end is approached, the maximum allowable humidity decreases so it appears quite likely that the maximum allowable operating humidity may be exceeded at the Helios unit. The uninterruptible power supply used for design of environmental protection measures does not have any limit specified for maximum relative humidity, but the upper temperature limit of 40° Celsius (104° Fahrenheit) may be reached on rare occasions.

The limiting factors controlling the design of the data acquisition housing appears to be the upper end temperature and humidity limits. These are easily controlled using environmental control cabinets, which consists of a metal enclosure cabinet, a top or side mount air conditioner designed to control temperature and humidity, and an optional heating unit. Units of this type are readily available from companies such as Hoffman Enclosures, and they run off 110 volt line power. Typically, a unit sized for a data acquisition system such as the Fluke Helios system requires a small side mount air conditioner, possibly a small heater, and costs roughly $2,000. With this added protection, it is quite feasible to place the data acquisition computer on the bridge as well, but the convenience of locating the computer in a remote site remains an overriding factor. Hence a small metal cabinet with a side mount air conditioner will be purchased and used to house the Helios and UPS units.

5.3.2 Extender Chasses

The extender chasses are somewhat more rugged than the Helios main controller, having an allowable operating temperature range of -20° to 70° Celsius and an allowable humidity range from 0 to 90 percent noncondensing, depending upon the temperature. It does not appear that this temperature and humidity range will be exceeded during service. Consequently, only some dust protection and electrical isolation will be provided for the extender chassis units. This can be achieved using NEMA Class 12 metal cabinets. Grounding these cabinets also provides additional electrical protection against lightning strikes. Consequently, each of the extender chasses will be located inside an appropriate NEMA electrical cabinet. The cabinets will be oversized sufficiently to allow the lead wire terminal strips with MOVs to be mounted inside the boxes.

5.4 Communication and Power Requirements for Data Acquisition System

After completion of the bridge, power will be supplied by a 120 volt line taken from the main bridge power. Prior to completion, it will be necessary to provide power using either a temporary electrical line or a generator. Power must be provided to the UPS and to each of the extender chasses. The UPS provides the backup power to the Helios, and the 12-volt batteries provide backup power to the extender chasses.

Communication between the data acquisition system and the remote computer will be provided via a standard voice grade telephone line on the bridge. An auto-answer modem will be located adjacent to the Helios unit, and an auto-dial model will be attached to the data acquisition computer. By locating the data acquisition computer in the VDOT offices in Richmond, Virginia, it will be possible to eliminate long distance charges, while assuring that the data is shipped to the VTRC as expediently as possible.

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APPENDIX

LABORATORY TESTS OF STRAIN GAGES MOUNTED ON SEVEN-WIRE STRAND

TESTS PERFORMED JULY 7, 1987

A.1 Purpose

The purpose of this experiment was to determine whether small strain gages could be effectively mounted on the individual wires of seven-wire strands and give results that correspond to expected strain values for strands in laboratory tensile tests.

A.2 Apparatus

The gages were mounted on 3.5-ft sections of 0.6 in. in diameter seven-wire strand, which had an anchor swedge on one end. The strands were loaded in a Baldwin Universal Testing Machine. The gages were connected with leadwires to a Budd SB-1 Switch and Balance Unit, which in turn was connected to a Budd P-350 Portable Strain Indicator.

A.3 Personnel

The laboratory tests were performed by M. Mohr, S. Hayes, and M. Burton under the direction of Dr. T. Baber.

A.4 Procedure

Sections of seven-wire strand were obtained at the jobsite and anchor swedges were installed on one end. The area on the strand where the gages were to be mounted was wire brushed and chemically cleaned with an alcohol degreaser, an acid conditioner, and a basic neutralizer. The gages were mounted in accordance with Micro-measurements Instruction Bulletin B-137-11 using an AE-10 epoxy adhesive. During the curing period, the gages were secured to the strand by using a pipe clamp with a Tygon tubing underlay. After the adhesive was cured, the lead wires were soldered to the gages' terminals. The Switch and Balance Unit and the Portable Strain Indicator were connected to the instrumented strands. The strands were loaded to 30,000 lb and gradually unloaded in the universal testing machine. Strain readings were taken at 5,000-lb load increments. This load cycle was repeated twice for each strand. The results were used to construct stress versus strain diagrams.

A.5 Specimens

Two seven-wire strand specimens were prepared for the tests. The first specimen had two Micromeasurements 062AP gages attached to wires diametrically opposite each other as described in section A-4. The second specimen had two 500GB gages attached using the same procedure. The 062AP gages are considerably shorter than the 500GB gages.

A.6 Discussion

From the results of the experiment it appears that strain gages can be adequately mounted on individual wires of seven-wire strands. No major problems were encountered during the installation of the gages or the placement of the strands in the testing machine. The stress-versus-strain diagrams that were developed showed linear relationships. Some deviation can be seen in the initial loading region. These discrepancies may have been caused by slight settlement or slippage of the strands in the loading machine. The second load cycle usually showed smaller deviations since most of the initial settlement occurred during the first cycle. Using a regression analysis, the slope of the lines were found. The values obtained represent the modulus of elasticity of the strands. The results obtained were slightly higher than expected. This was probably due to the fact that the strain gages were not oriented exactly along the axis of the strand but rather on the axis of the individual wires that wrap around the axis of the strand. This small angular deviation in the orientation of the gage will cause the strain values to be slightly smaller than they would be along the axis of the strand. The smaller strain values lead to the slightly high estimates of the modulus of elasticity. The fairly complicated interaction of the wires in the strand may also have contributed to these minor discrepancies. Overall, the use of strain gages on the wires of seven-wire strands seems to be feasible. The linearity of the results indicates that the gages were functional, and the fact that similar data was obtained from each strain gage indicates that the results are reproducible. The results obtained from the two 062AP gages were slightly closer to each other than those obtained from the 500GB gages. The greater difference observed in the 500GB gages was attributed to larger initial curvature in the strand specimen to which those gages were attached.

DATA AND RESULTS

STRAIN (microns/in.)

CYCLE 1 LOAD (lb.)	STRESS (psi)	GAGE 1 S/R/W	GAGE 2 S/G/B	GAGE 3 L/G/B	GAGE 4 L/R/W
0	0	0	0	0	0
5000	23148.14	620	565	800	455
10000	46296.29	1270	1280	1580	1175
15000	69444.44	2020	2030	2370	1910
20000	92592.59	2740	2790	3175	2660
25000	115740.7	3485	3545	3960	3440
30000	138888.8	4200	4300	4760	4195
25000	115740.7	3450	3540	3950	3420
20000	92592.59	2700	2765	3160	2675
15000	69444.44	1980	2000	2360	1905
10000	46296.29	1275	1245	1570	1175
5000	23148.14	565	500	785	440
0	0	-95	-140	-45	-135

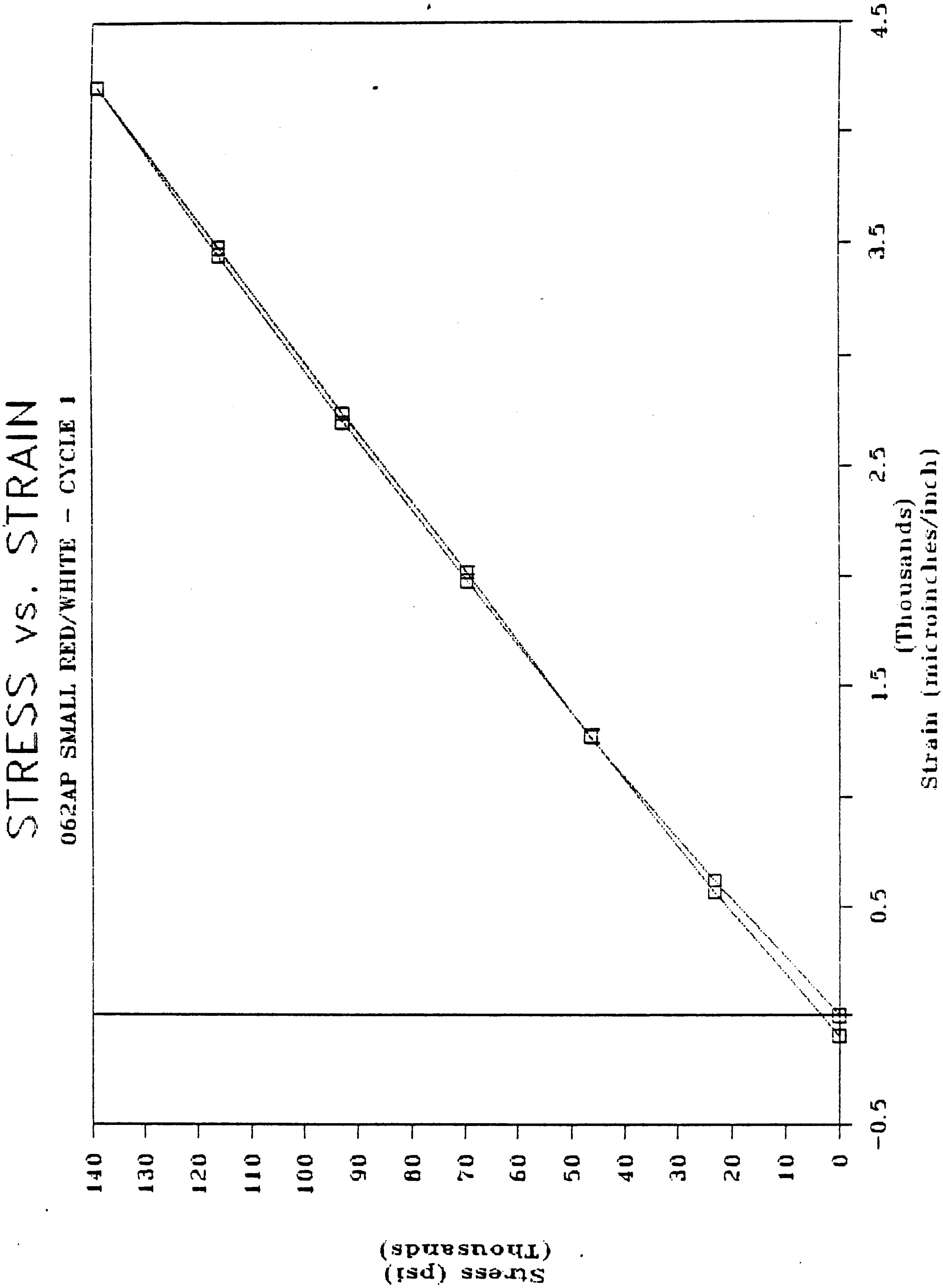
CYCLE 2 LOAD (lb.)	STRESS (psi)	GAGE 1 S/R/W	GAGE 2 S/G/B	GAGE 3 L/G/B	GAGE 4 L/R/W
0	0	0	0	0	0
5000	23148.14	590	600	810	535
10000	46296.29	1290	1345	1600	1250
15000	69444.44	2010	2100	2385	1990
20000	92592.59	2705	2860	3165	2735
25000	115740.7	3450	3625	3975	3485
30000	138888.8	4200	4395	4770	4260
25000	115740.7	3450	3615	3975	3490
20000	92592.59	2695	2835	3165	2725
15000	69444.44	1970	2090	2380	1980
10000	46296.29	1250	1310	1590	1250
5000	23148.14	565	580	795	520
0	0	0	0	0	-40

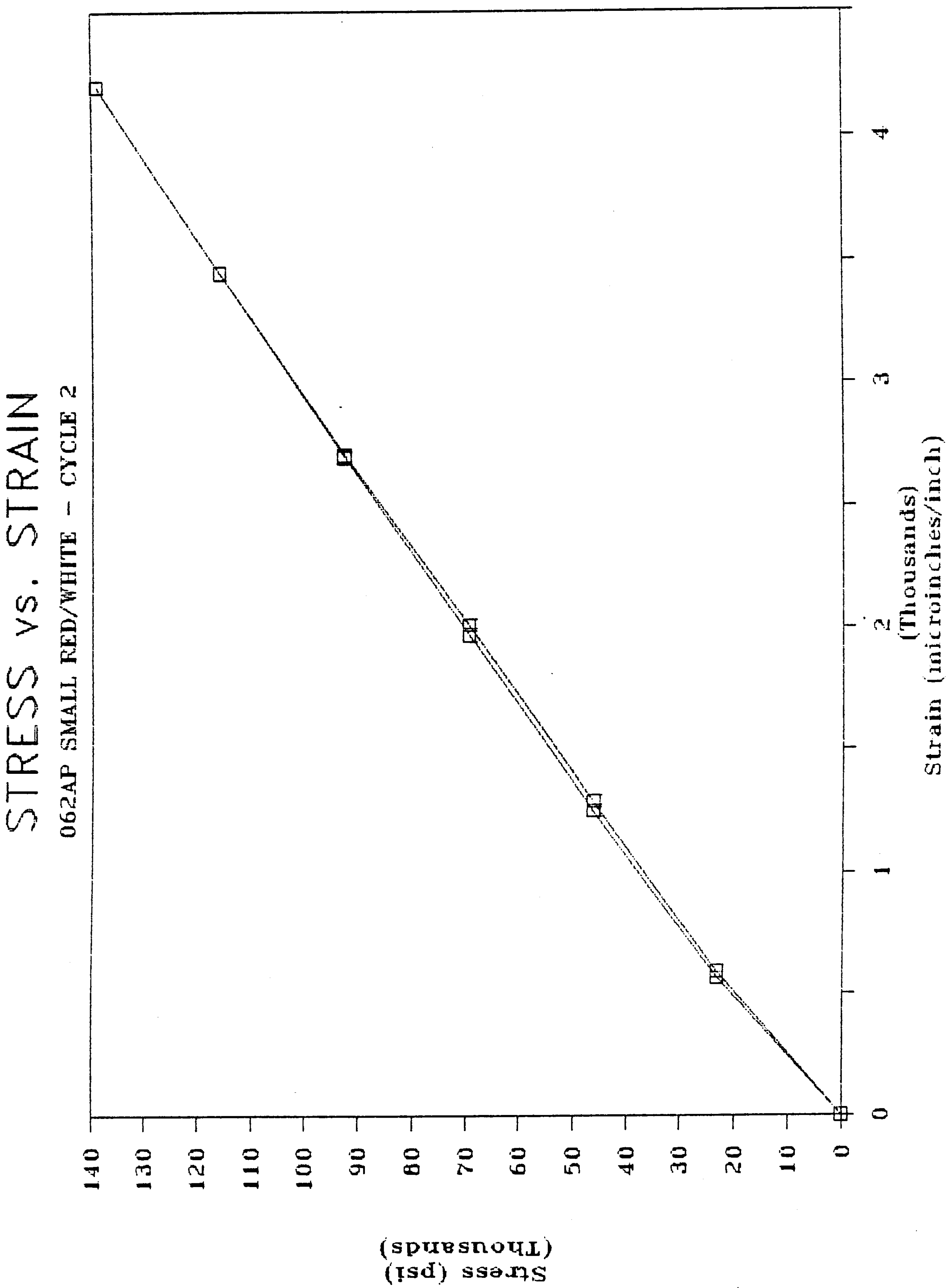
Load Rate - 5000 lb/min.
Temperature - 76 degrees F.
Strand X-Sect. Area - 0.216 sq. in.

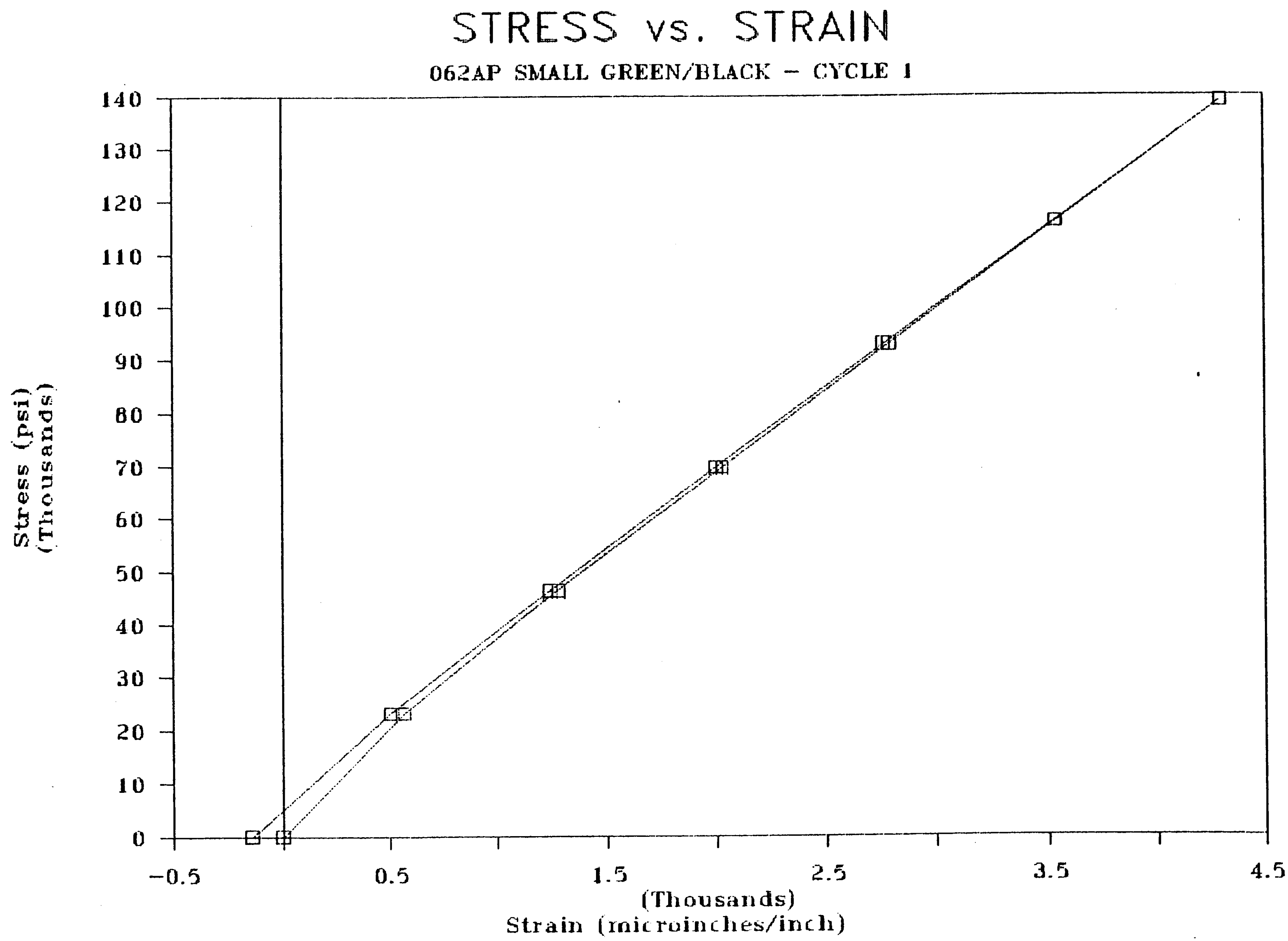
Gage 1 S/R/W - 062AP (small), red and white leads
Gage 2 S/G/B - 062AP (small), black and green leads
Gage Factor - 2.005 +/- 0.5%

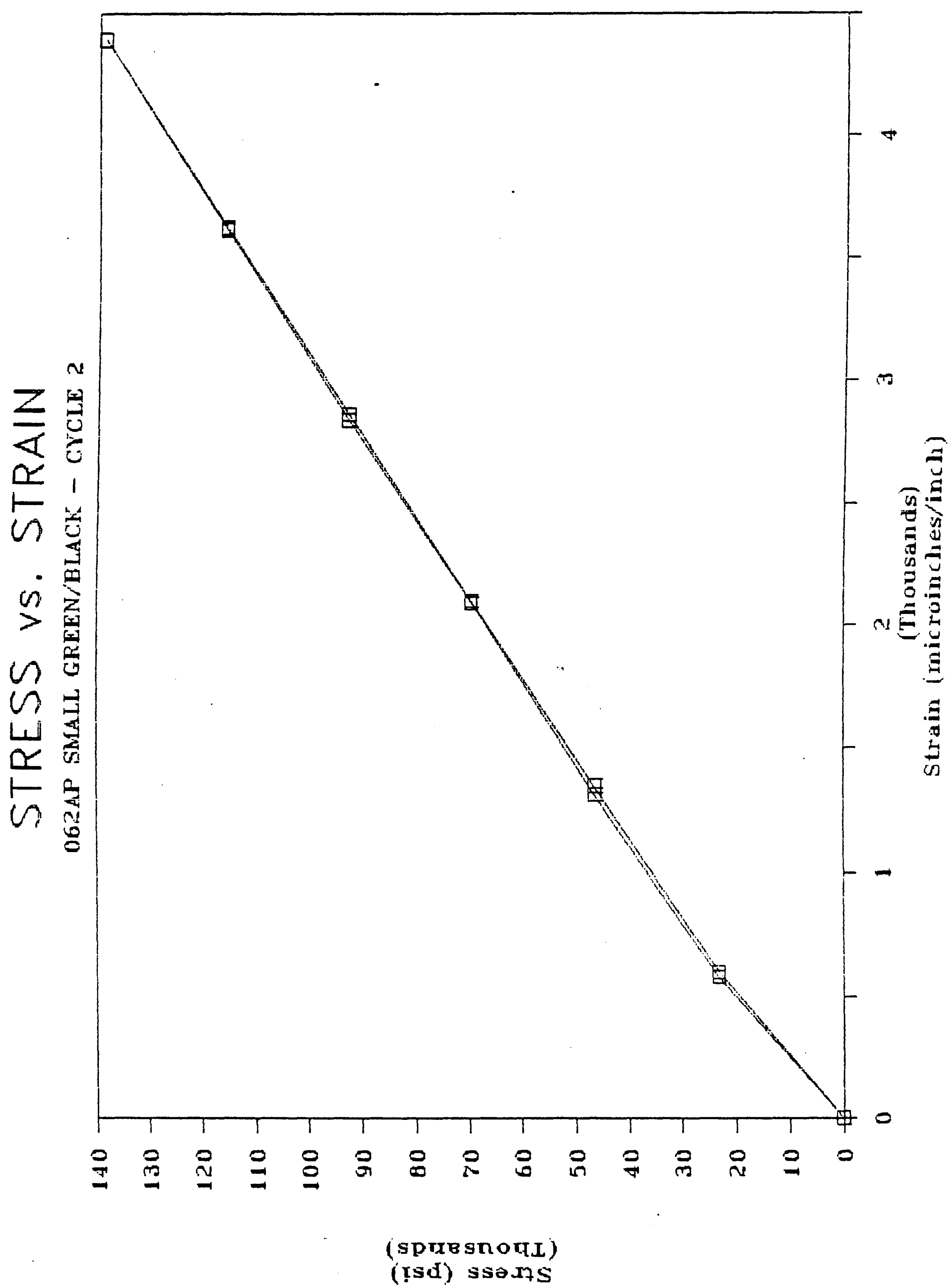
Gage 3 L/G/B - 500GB (large), black and green leads
Gage 4 L/R/W - 500GB (large), red and white leads
Gage Factor - 2.07 +/- 0.5%

Note: Before the second load cycle, the strand with Gage 4 was released and rotated in the test machine.

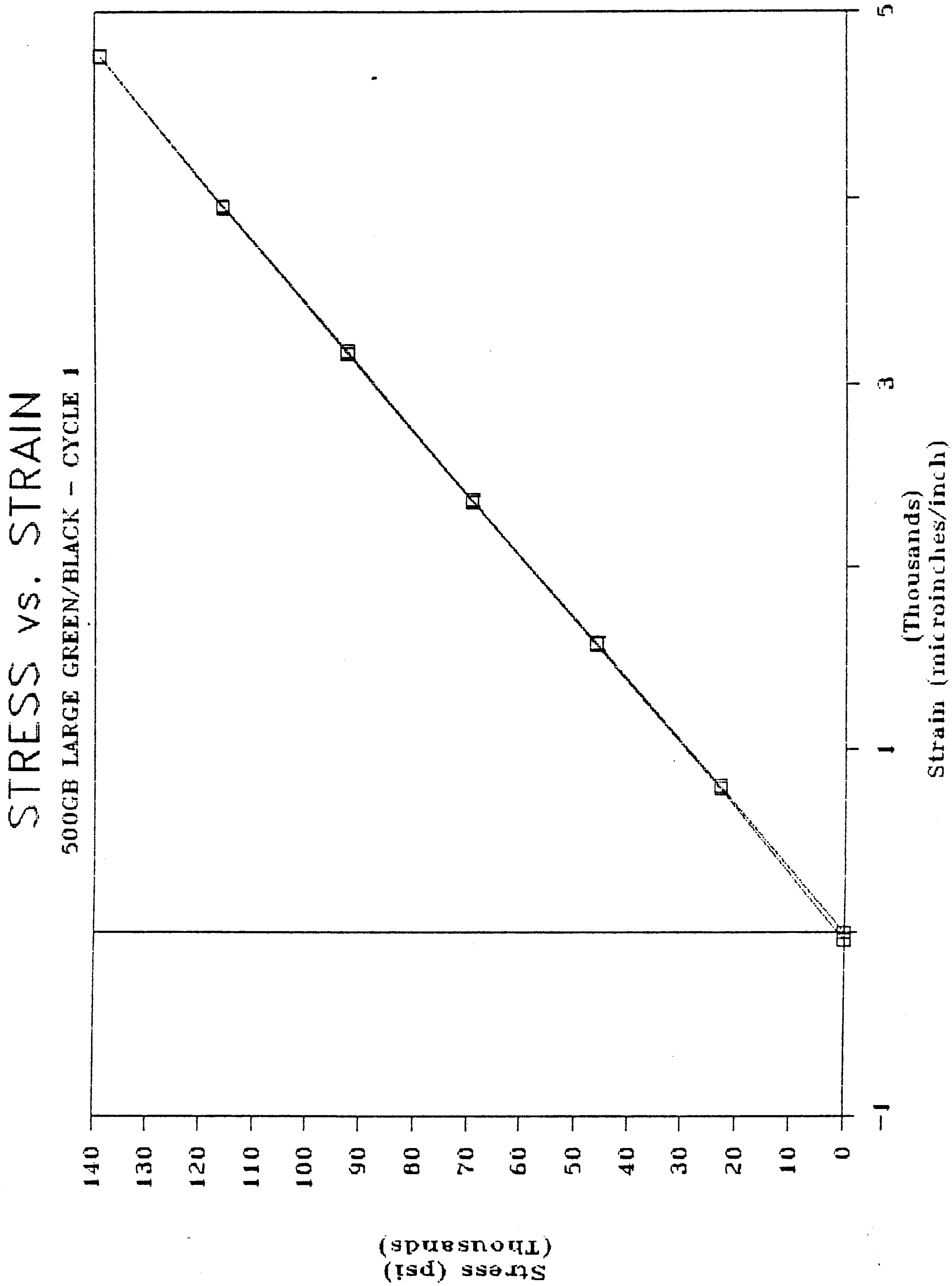

STRESS vs. STRAIN
062AP SMALL RED/WHITE - CYCLE 1
Stress (psi)
(Thousands)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-0.5
0.5
1.5
2.5
3.5
4.5
(Thousands)
Strain (microinches/inch)

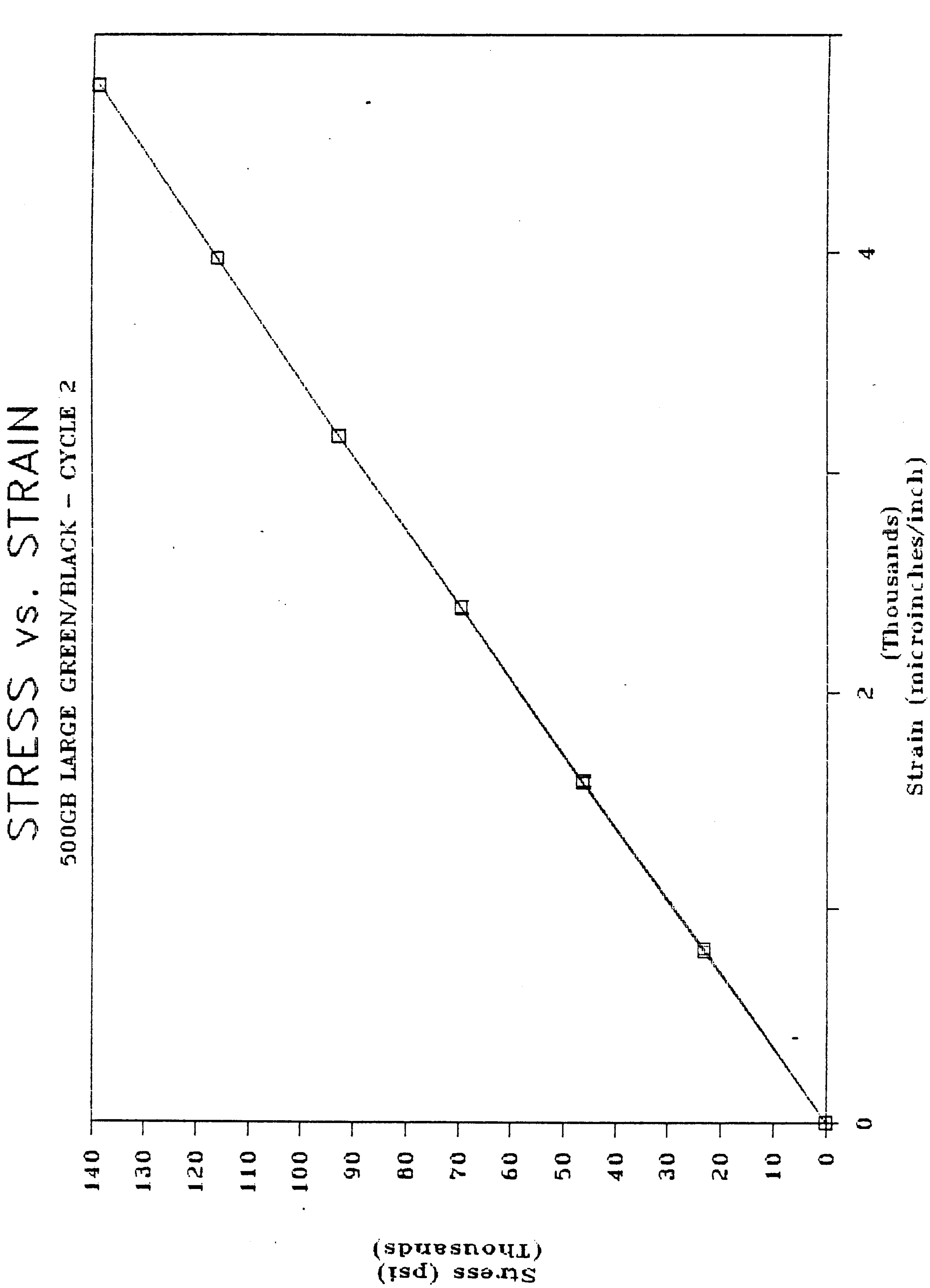

STRESS vs. STRAIN
062AP SMALL RED/WHITE - CYCLE 2
Stress (psi)
(Thousands)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
Strain (microinches/inch)
(Thousands)
0
1
2
3
4

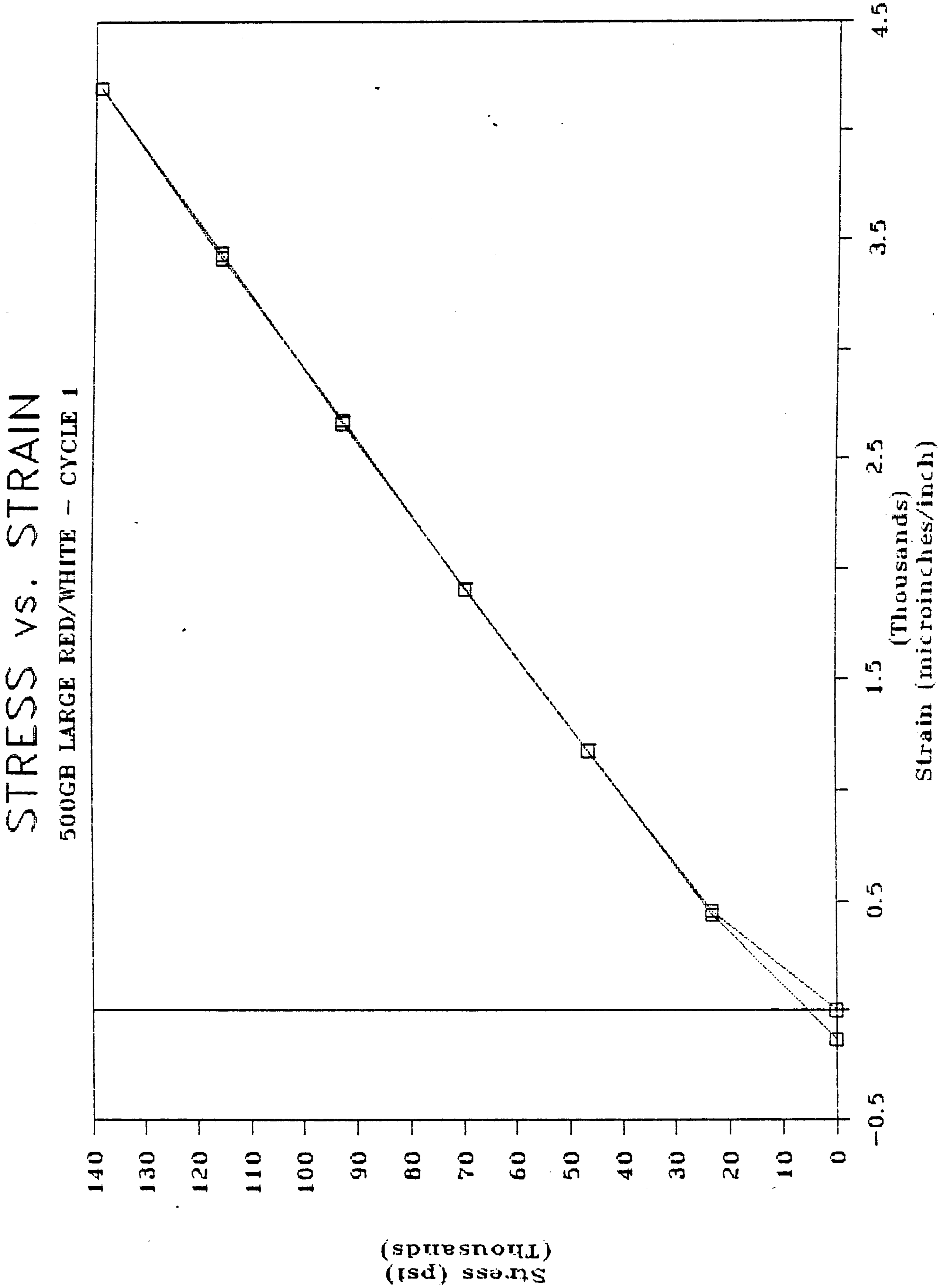

STRESS vs. STRAIN
062AP SMALL GREEN/BLACK - CYCLE 1
Stress (psi)
(Thousands)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-0.5
0.5
1.5
2.5
3.5
4.5
(Thousands)
Strain (microinches/inch)



STRESS vs. STRAIN
062AP SMALL GREEN/BLACK - CYCLE 2
Stress (psi)
(Thousands)
Strain (microinches/inch)
(Thousands)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
0
1
2
3
4

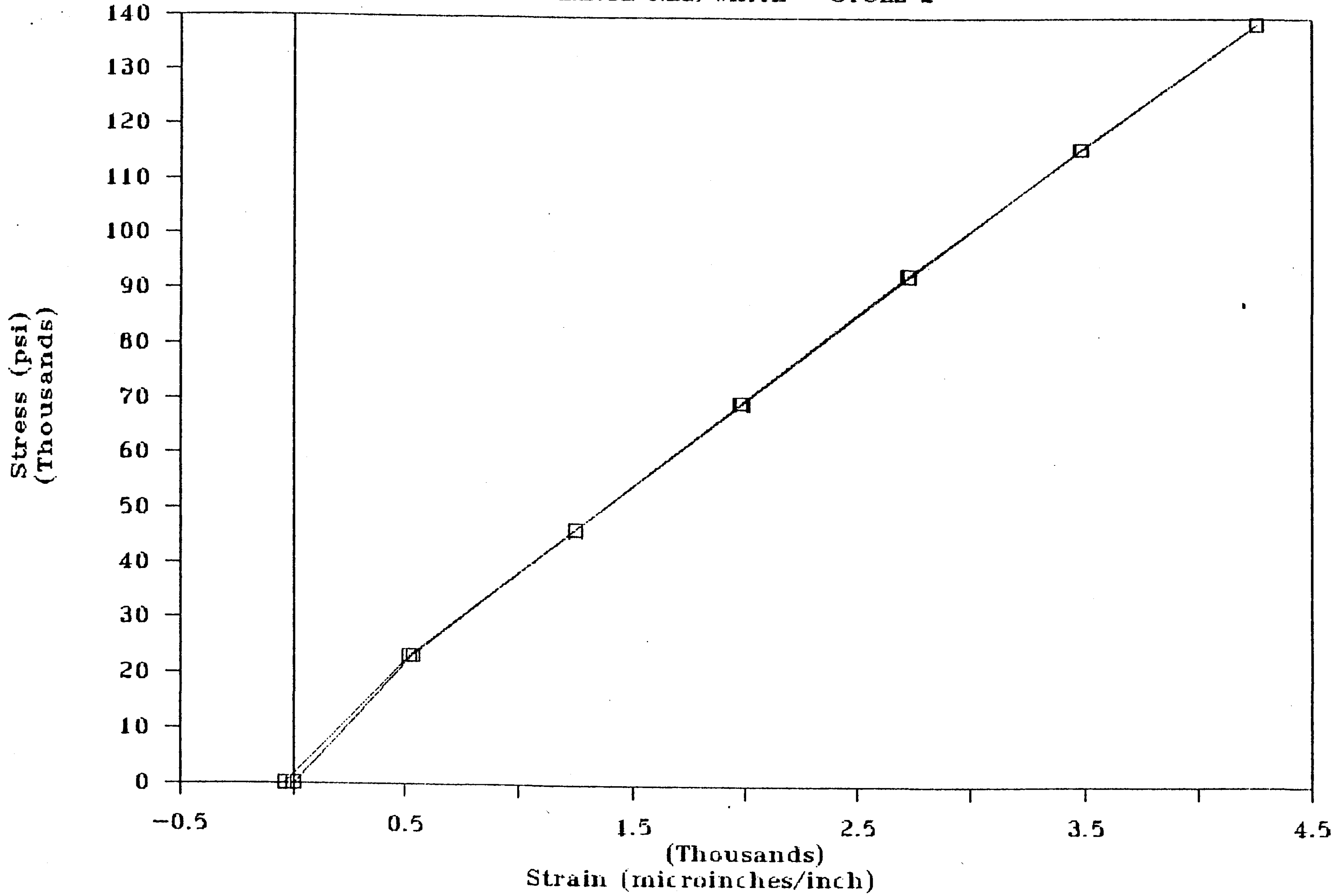

STRESS vs. STRAIN
500GB LARGE GREEN/BLACK - CYCLE 1
Stress (psi)
(Thousands)
Strain (microinches/inch)
(Thousands)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-1
1
3
5


STRESS vs. STRAIN
500GB LARGE GREEN/BLACK - CYCLE 2
Stress (psi)
(Thousands)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
Strain (microinches/inch)
(Thousands)
0
2
4


STRESS vs. STRAIN
500GB LARGE RED/WHITE - CYCLE 1
Stress (psi)
(Thousands)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-0.5
0.5
1.5
2.5
3.5
4.5
(Thousands)
Strain (microinches/inch)

STRESS vs. STRAIN

500GB LARGE RED/WHITE – CYCLE 2



Gage 1 - Cycle 1
Regression Output:

Constant	3425.521
Std Err of Y Est	1506.862
R Squared	0.998993
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	32.58164
Std Err of Coef.	0.311833

Gage 1 - Cycle 2
Regression Output:

Constant	2912.570
Std Err of Y Est	1827.478
R Squared	0.998519
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	32.90465
Std Err of Coef.	0.382022

Gage 2 - Cycle 1
Regression Output:

Constant	5025.798
Std Err of Y Est	1975.424
R Squared	0.998270
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	31.44954
Std Err of Coef.	0.394737

Gage 2 - Cycle 2
Regression Output:

Constant	3053.842
Std Err of Y Est	1816.611
R Squared	0.998537
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	31.30086
Std Err of Coef.	0.361238

Gage 3 - Cycle 1
Regression Output:

Constant	432.3516
Std Err of Y Est	387.6631
R Squared	0.999933
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	29.11918
Std Err of Coef.	0.071664

Gage 3 - Cycle 2
Regression Output:

Constant	-102.213
Std Err of Y Est	229.2793
R Squared	0.999976
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	29.17378
Std Err of Coef.	0.042463

Gage 4 - Cycle 1
Regression Output:

Constant	6529.952
Std Err of Y Est	2686.693
R Squared	0.996800
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	32.10139
Std Err of Coef.	0.548397

Gage 4 - Cycle 2
Regression Output:

Constant	4318.400
Std Err of Y Est	2176.351
R Squared	0.997900
No. of Observations	13
Degrees of Freedom	11

X Coefficient(s)	32.14202
Std Err of Coef.	0.444545